MECHANICAL PERFORMANCE OF WARM MIX ASPHALT-TREATED BASES INCORPORATING RECYCLED ASPHALT PAVEMENT

By

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I declare that this dissertation, submitted for the degree of Magister: Engineering: Civil, at the Central University of Technology Free State, is my original work and has not been submitted to any other institution of higher education. I further declare that all sources cited or quoted in this dissertation are indicated and acknowledged utilizing a comprehensive list of references.

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R.F. POUOKAM KAMDEM Date

Dedication

I dedicate this dissertation to GOD the Father almighty, my late mother Helen Clodine Kamdem, late father Japhet Henry Kamdem, later brother Guy Jean Kamdem and late sister Dyane Kamdem, the Wilson family and loved ones.

Preface and Acknowledgements

I give praise and honour to GOD the ALMIGHTY FATHER for His unceasing blessing, grace, care and protection granted to help me complete this dissertation and other achievements. I thank Him for guiding my steps throughout this tremendous academic journey. I glorify His name for His unfailing promises to be by my side during my trials and tribulations. The completion of this research would never have been achieved without His permanent blessings and provisions.

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Abstract

The depletion of natural resources, the elevated cost related to the construction and the rehabilitation of pavements, the progressive change of the climate, the high heat and harmful gas emission into the atmosphere are the concerns engineers, academics, scientists and politicians have been addressing. They are co-operating toward finding efficient solutions to mitigate these global issues. Thus, the advent of Warm Mix Asphalt (WMA) incorporating Recycled Asphalt Pavement (RAP) as a long-term solution to partially or totally participating in remediating the problem of global warming, climate change and the preservation of environmental resources has gained prominent interest in certain European countries and Asia, North America and most recently in South Africa.

This thesis, therefore, aims to investigate the performances of the control Hot Mix Asphalt (HMA) technology and the WMA incorporating RAP at 15% and 30% through laboratory experiment and numerical modelling.

Consequently, the laboratory studies that involve the mix designs, the production and the testing of asphalt specimens were achieved following both the South African and the international standards. The RAP used at 15% and 30% in the WMA contains 0.8 % of 50/70 grade bitumen. The virgin aggregate called dolerite and the fillers used in the asphalt mixture were obtained at the Lafarge Olivehill Crushers site in Bloemfontein. The Sasobit, as well as the 50/70 grade bitumen binder, were collected in Sasolburg.

The numerical simulation of the WMA – 15%RAP, the WMA – 30%RAP and the control HMA was achieved through the Finite Element Method (FEM) in the Abaqus computer program and the Layered Elastic Analysis (LEA) in mePADS. The Linear Elastic Analysis (LEA) was adopted not only to validate the results found in FEM but most of all, to justify the preference of FEM over the LEA. The numerical simulation WMA – 15%RAP pavement structures, the WMA – 30%RAP pavement structures and the control HMA pavement structures was to analyse their mechanical responses under repeated loading.

The results of the laboratory experiment show that:

- the control HMA exhibits lower rutting performance than the WMA – 15%RAP and WMA – 30%RAP;
- the control HMA exhibits lower fatigue cracking performance than the WMA – 15%RAP and WMA – 30%RAP;
- the control HMA exhibits lower ITS (stiffness) performance when compared to the WMA – 15%RAP the WMA – 30%RAP;
- the control HMA exhibits close Marshall Stability and Flow performance to the WMA – 15%RAP the WMA – 30%RAP.

As far as numerical modelling is concerned, the results show that the control HMA pavement structures exhibit lower rutting and fatigue cracking performance when compared to the WMA – 15%RAP pavement structures the WMA – 30%RAP pavement structures. Overall, the WMA can successfully incorporate RAP at up to 30% and can be utilized for the new construction and the rehabilitation of low to medium-traffic volume roads.

List of Acronyms

Nr	Number of loads to rutting failure
N_f	Number of loads to fatigue failure
Ɛt	Horizontal tensile strain
Ɛc	Vertical compressive strain
Ɛ	Strain
σ	Stress
2D	Two-dimensional
3D	Three-dimensional
FEM	Finite element method
LEA	Layered elastic analysis
µƐ	Micro strain
S	Second
Min	Minute
Hz	Hertz
E^*	Dynamic modulus
mm	Millimetre
m	Meter
mm^2	Square millimetre
m^3	Cubic meter
%	Percentage
AC	Asphalt concrete
C3	Cemented natural gravel
WMA	Warm mix asphalt
HMA	Hot mix asphalt

RAP	Recycled asphalt pavement
KPa	KiloPascal
MPa	MegaPascal
KN	KiloNewton
Kg	Kilogram
Psi	Pound per square inch
g	Gram
°C	Degree Celsius
°	Angle
FACT	Fine aggregate crushing test value
PSV	Polish stone value
ACV	Aggregate crushing value
FAA	Fine aggregates angularity
AASTHO	American association of state highway and transportation officials
ITS	Indirect tensile strength
SANS	South African national standard
BC	Binder content
$\mathbf{BD_A}$	Bulk density of aggregate
$\mathbf{V_v}$	Air void percentage
$\mathbf{V_B}$	Volume of bitumen
$\mathbf{V_A}$	Volume of aggregate
VMA	Void in the mineral aggregate
VBF	Void filled with bitumen
VIM	Void in the mix
TRH	Technical recommendation for highway
TMH	Technical methods for highways

ASTM	American Society for Testing and Materials
COLTO	Committee of Land Transportation Officials
SABITA	South African Bitumen Association
LCPC	Laboratoire Central des Ponds et Chaussees
GLWT	Georgia Loaded Wheel- tracking Tester
AMPT	Asphalt mixture performance test
DOT	Department of Transportation
MEDM	Mechanistic-Empirical design method
HWTD	Hamburg Wheel-tracking Device
Abaqus	Finite element computer code
mePADS	Pavement Analysis and Design software

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CHAPTER 1 – INTRODUCTION

1.1. Background

Environmental threats such as the excessive exploitation of natural resources and the greenhouse gas emission have resulted in irreversible and permanent damage to the planet Earth . Solomon, Qin, Manning, Averyt and Marquis (2007) asserted that, though sectors, which include manufacturing, transportation and agricultural, are regarded essential to human's well-being, they contribute to endangering the environment.

Kgmanyane (2015) found that African countries depend more on road transport for trips, goods transit and freight. The importance or the preference attached to this mode of transportation (road transport) in Africa is because alternative modes of transportation are financially constraining. The author also claimed South Africa to have been classified the country with the eighteenth longest paved road in the world. The replacement of these roads would cost the country's financial budget close to two trillion Rands. Also, the majority of flexible paved road networks in South Africa and other countries are traditionally made with hot mix asphalt mixed with virgin materials and conventional asphalt binder that are highly aggressive to the environment.

Although paved road infrastructures are highly beneficial to the society, their construction, maintenance and rehabilitaton are considerered as one of the main cause of environmental pollution. Rubio, Martinez, Baena and Moreno (2012) reported that the pollution of pavement constructions lies in the processing, spreading and the conservation of asphalt mixes.

These issues have obliged public agencies, private and multinational contractors to embrace the utilization of non-conventional materials, environmentally sustainable products and innovative technologies in the design, the construction, the maintenance and the rehabilitation of pavements, railways, building constructions and several other engineering realizations (Vaitkus, Cygas, Laurinavicius & Perveneckas, 2009).

The choice of WMA technology and Recycled Asphalt Pavement (RAP) aggregates as a solution to lower greenhouse gas emission and reduced dependency of virgin aggregates is being gradually adopted by highway agencies all over the world (Tao & Mallick, 2009).

This study, therefore, focuses firstly on designing and producing the WMA – 15% RAP specimens, the WMA – 30% RAP specimens and the HMA specimens for control purposes. Secondly, the focus is on determining the performances of the produced asphalt specimens in terms of rutting, fatigue cracking, stiffness modulus, elastic modulus, flow and stability through laboratory performance tests. Finally, it focuses on designing, modelling, simulating and analyzing the control HMA pavements structure, the WMA – 15% RAP pavements structure and the WMA – 30% RAP pavements structure using Finite Element Method (FEM) in theAbaqus computer program and Layered Elastic Analysis (LEA) method in the mePADS computer program.

The knowledge emanating from this study will contribute to maintaining, rehabilitating and constructing new roads without compromising the existing natural resources, the safety of the environment and the road construction workers. It will also contributes to reducing the high cost related to the construction and the rehabilitation of roads in South Africa and other African countries where flexible paved roads are either non-existent or in poor condition.

1.2. Problem Statement

The elevated cost of construction materials, the waste generated during pavement resurfacing or widening, the high dependency of quality virgin aggregates, the scarcity or the unavailability of good quality virgin materials, the high dependency on petroleum-based products and the high heat and harmful gas emissions into the atmosphere are some of the major concerns that engineers, academics, scientists and politicians have been addressing. Efforts are being made to find efficient solutions to mitigate these global issues. The issues listed above are, in most cases, related to the use of the traditional HMA incorporating 100% virgin aggregates.

The increasing need to use construction materials and asphalt technologies that are sustainable, less expensive, environmentally friendly, safe to workers and that possess good performance against fatigue cracking failure, rutting failure, and moisture damages are pre-eminent in the pavement engineering industry.

Achieving the goal of optimizing a high amount of RAP in WMA while achieving excellent resistance against rutting failure, fatigue cracking failure and moisture damage has been unsuccessful so far. The reasons may be identified as follows:

- At high RAP percentages, the effects of mixture characterization, aggregate properties and gradation as well as the binder properties of the RAP have shown inconsistent results.
- The high amount of RAP used in the WMA mix design and which results in its inability to achieve the overall mechanical performances effectively, is mainly caused by the inability to characterize the properties of the aged RAP binder accurately.
- Although research has demonstrated a successful blending of the aged RAP binder and virgin binder, it has not been capable of determining beforehand the exact amount of aged RAP binder that could be combined with a virgin binder.
- The stiffness nature of the WMA incorporating RAP is impacted by the RAP aggregates and most importantly, by the stiffness of the aged RAP binder.
- The failure to find an adequate WMA additive or a combination of WMA additives that can improve the performance of the WMA is significant.

Hence, this thesis focuses on investigating the performances of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP through laboratory experiment and numerical modelling. The laboratory experiment involves the materials mix designs, the production of asphalt specimens, the Hamburg Wheel-Tracking Test, the Four-Point Beam Bending Test, the Indirect Tensile Strength Test, the Marshall Stability and Flow Test, and the Dynamic Modulus Test. The numerical modelling, on the other hand, involves the modelling, the simulation and the analysis of the WMA – RAP pavement structures and the control HMA pavement structures using Finite Element Method (FEM) and Layered Elastic Analysis (LEA).
The successful outcome of studies on the WMA – 15% RAP and the WMA – 30% RAP would contribute to developing safe, durable, reliable, sustainable, environment-friendly and lower-cost flexible pavements.

1.3. Research Aim

This study aims to investigate the rutting performance, the fatigue cracking performance, the stiffness modulus, the elastic modulus as well as the Marshall flow and stability of the WMA – 15% RAP and the WMA – 30% RAP against the performances of the traditionally used HMA incorporating 100% virgin aggregates. These will be achieved through both laboratory performance tests and numerical modelling. The laboratory performance tests considered in this study are the Marshal Stability and Flow test, the Indirect Tensile Strength tests (ITS), the Four-Point Beam Bending test, the Hamburg Wheel-Tracking test as well as the Dynamic Modulus test. Furthermore, the numerical modelling considered in this study includes the Finite Element Method (FEM) in the Abaqus computer program and the Layered Elastic Analysis (LEA) using mePADS.

1.4. Research Objectives

The objectives of this study are to:

- Design and produce the control HMA specimens, the WMA + 15% RAP specimens and the WMA + 30% RAP specimens following the South African National Standard Method (SANS), the Technical Method for Highways (TMH 1), the American Society for Testing and Materials (ASTM), the European Standard (EN), and the American Association of State Highway and Transport Officials (AASHTO);
- Test and evaluate the fatigue cracking, the rutting failure, the elastic modulus, the stiffness modulus as well as the Marshall Stability and Flow of the control HMA specimens, the WMA – 15% RAP specimens and the WMA – 30% RAP specimens through the following laboratory performance tests:
  - The Hamburg Wheel-Tracking Test;
  - The Four-Point Beam-Bending Test;
  - The Indirect Tensile Strength (ITS) Test;
  - The Dynamic Modulus Test;
  - The Marshal Stability and Flow Test.

- Evaluate the effect of the control HMA course layer on the multi-layers pavement structure through numerical modelling using Finite Element Method (FEM) and Layered Elastic Analysis (LEA), and thus validate the FEM.
- Evaluate the effect of the WMA – 15% RAP course layer on the multi-layers pavement structure through numerical modelling using Finite Element Method (FEM)
- Evaluate the effect of the WMA – 30% RAP course layer on the multi-layers pavement structure through numerical modelling using Finite Element Method (FEM)
- Predict the rutting performance and the fatigue cracking performance of the control HMA pavement structure, the WMA – 15% RAP pavement structure and the WMA – 30% RAP pavement structure using the empirical distress models also known as transfer functions. Thereafter, to conduct a comparative study of the performance of the WMA – 15% RAP pavement structures and the WMA – 30%RAP pavement structures against the control HMA pavement structures.

1.5. Significance of the Study

This study provides an alternative to the development of sustainable materials and new asphalt technology used in the construction of new roads and the rehabilitation of existing roads. Furthermore, using the WMA incorporating RAP as an ideal technology in the construction of flexible pavement will help to:

- Lessen the excessive waste in dump sites and dump hills;
- Markedly lower the high level of fuel emission and consumption in road construction;
- Reduce the cost of hazardous gas emission into the environment;
- Substantially reduce the excessive exploitation of natural resources.

1.6.Outline of the Dissertaion

The dissertation is organised following the procedures described below:

Chapter 1: This chapter includes an introduction to the study, the problem statement, the aims, the objectives and the significance of the study.

Chapter 2: This chapter provides a complete literature review that revolves around the WMA, WMA additives, the WMA mix design methods, the RAP, the WMA – RAP and a brief literature review on pavement design methods.

Chapter 3: This chapter provides a detailed, comprehensive and well-structured methodology used to achieve this dissertation

Chapter 4: This chapter discusses the selection and the preparation of aggregates, RAP, and bitumen needed to achieve the mix designs and the fabrication of the control HMA specimens, the WMA – 15% RAP specimens and the WMA – 30% RAP specimens. It also discusses the tests performed on the selected aggregates, the RAP and the bitumen as well as their properties as found.

Chapter 5: This chapter discusses the preparation of the control HMA specimens, the WMA – 15% RAP specimens and the WMA – 30% RAP specimens before they can be subjected to laboratory performance tests. It also discusses the procedures and the steps used to achieve the laboratory performance tests on the fabricated asphalt specimens. The laboratory performance tests include the Marshal Stability and Flow, the Indirect Tensile Strength, the Hamburg Wheel Tracking, the Four-Point Beam Bending and the Dynamic Modulus. Furthermore, this chapter analyses the results found on asphalt specimens subjected to laboratory performance tests.

Chapter 6: This chapter covers the numerical modelling done on the control HMA pavement structures, the WMA – 15% RAP pavement structures and the WMA – 30% RAP pavement structures using the Finite Element Method (FEM) in the Abaqus computer program and the Layered Elastic Analysis (LEA) in the mePADS computer program. This chapter is closely related to Chapter 4 as it uses data collected in Chapter 4 to make the simulation and the analysis possible.

Chapter 7: This chapter summarizes and concludes the laboratory experiments performed, the numerical modelling done, the findings made, resulting in possible recommendations and future studies to be done.

CHAPTER 2 - LITERATURE REVIEW

2.1. Introduction

The structure of a flexible paved road is generally composed of granular layers and asphalt concrete layer(s). The top surface of the pavement structure is ideally made out of asphalt concrete. The base layer, on the other hand, can be constructed either with natural granular materials or asphalt concrete. The lower layer of the structure of the road is constructed using either untreated natural gravels or modified gravels, which aim to improve its poor quality or functionality.

Various types of asphalt mix are available for the construction of flexible roads. The Sabita Manual 35 / TRH8 (2019), provides a few of them:

- The conventional hot mix asphalt;
- The special asphalt mixes. These are mixes produced at low temperature, such as. warm mix asphalt, the *enrobé à module elevé*, stone mastic asphalt, porous asphalt;
- The cold mix asphalt. This type of asphalt mix is intended for the patching and the repair of potholes;
- The mixes for light traffic;
- The mixes incorporating reclaimed asphalt pavement and the mixes incorporating waste materials. For example slags.

The Empirical method and the Mechanistic–Empirical (ME) design method are the two major methods used in the deign of pavement, be it flexible pavement or rigid pavement.

Thus, this chapter covers a literature review of asphalt mixes technology that are of interest in this study, such as the conventional hot mix asphalt HMA, and the warm mix asphalt incorporating recycled asphalt pavement (WMA – RAP). In addition, it covers the literature review on the the WMA organic additives and the pavement design methods.

2.2. Warm Mix Asphalt (WMA)

WMA technologies are asphalt mixes that are produced and spread at temperatures 20°C to 40°C lower than the HMA. A broad literature about the WMA additives, the mix design methods that govern the WMA technology, its mechanical performances, its various benefits and its drawbacks are reviewd and discussed.

2.2.1. WMA Additives

The addition of organic additives, chemical additives and foaming additives in the WMA play the role of lowering the mixing temperature,the compaction temperature of the WMA while improving its workability and its properties.

a) Organic Additives

Organic additives can be claclassified as Fischer-Tropsch wax, Montan wax and Fatty Acid Amid wax. The maximum amount of organic additive into the asphalt mix does not generally exceed 4% of the total mass of the bitumen (European Asphalt Pavement Association, 2012). According to manufacturers of additive technologies, the dosage prescribed should be respected to avoid undesirable effect on the properties of the asphalt mixture.

There are four types of organic additives that exist in the market, which include the Evotherm 3G, Rediset LQ, Advera, Asphaltan B, and Sasobit. Their addition into the WMA reduces its compaction and mixing temperature at up to 30°C while improving its performance against rutting failure and shearing failure (Dinis-Almeida, Castro-Gomes, Sangiorgi, Zoorob, & Afonso, 2016). RAP is known to be stiff by nature and its stiffness nature can reduce the rutting performance of an WMA. Nevertheless, the positive influence the organic additive has on the WMA in terms of improved rutting and shearing performance results in the WMA being the most qualified asphalt mix technology to accommodate RAP.

b) Sasobit Organic Additive

Sasobit is a Fischer-Tropsch (FT) wax in the form of white grain or white powder that plays a particular role when added into WMA. Besides reducing the mixing and compaction temperature, Sasobit, like any other type of organic additives, has the capability to reduce the optimum asphalt bitumen content, increase the resistance of WMA against rutting, to reduce the creep compliance as well as to increase the viscosity of the asphalt binder. Jamshidi, Hamza and You (2013) substantiate the positive effects of Sasobit when they found that Sasobit could effectively reduce the fatigue cracking and the ageing of the asphalt bitumen as long as the Sasobit content and the type of bitumen are selected objectively and with care.

Overall, the Sasobit organic additives possess similar benefits with chemical additives when added into the asphalt mix. Those benefits include the good workability, the improved compaction at low temperature (up to 100°C), the reduction in air voids in the asphalt mix resulting in the improved strength of the WMA – RAP. Furthermore, their easy availability and affordability on the market justify their choice over other WMA additive technologies.

c) Chemical additives

Evotherm, Cecabase RT (Reference Technology), Rediset, Revix, Iterlo T (Technology) are the type of chemical additive technologies using the combination of emulsifying agents, surfactants and polymers to improve the properties of the WMA (European Asphalt Pavement Association, 2010).

Pereira, Almeida-Costa, Duarte and Benta (2018) stated in a study that the WMA chemical additives contribute to reducinge the mixing and the compaction temperature of the asphalt mix at up to 40°C, which is 10°C lower than the temperature reduction offered by the Sasobit organic additives. The low mixing and compaction temperature of the WMA may result in poor workability of the WMA, inadequate coating of aggregates and poor compaction. The addition of chemical additives in WMA may, therefore, eliminate those negative effects and improve the strength and durability of the WMA. Since the chemical additives possess the capability to promote the adhesion between the bitumen and the aggregate and to improve the workability of WMA, it implies that RAP can be compatible with the WMA.

d) Foaming Additives

In a study, Larsen (2001) defines the foaming of asphalt bitumen as a procedure which involves the addition of small amounts of water into the hot binder or directly into the mixing machine. In other words, the water added into the hot bitumen evaporates, and the steam generated is trapped. The trapped steam forms a large volume of foam, which leads to a temporary increase in volume of the initial hot bitumen and reduction of its viscosity.

Butz, Rahimian and Hildebrand (2001) stated that the foaming technology improves the coating and workability of the mix. However, the duration for effective workability is limited. The limited period of workability entails that the WMA mixed with foaming additives must be spread and compacted immediately after it has been produced. This implies that the WMA mixed with foaming additives can only be produced on the road construction site or at an asphalt plant very close to the road construction site.

Zaumanis (2012) reported that the reduction of production and compaction temperature of the WMA through foaming technology can go down to 125°C. This is far lower than the reduction of temperature through chemical additives and organic additives. Although foam additives are more beneficial to WMA in terms of lower mixing and compaction temperaturature of the WMA, they become disadvantageous because of their shorter period of workability. This implies that preference would be given to chemical or organic additives as they keep the WMA workable for a longer period of time than the foam additives.

2.2.2. Mix Design Methods for WMA

Countries all over the world have their mix design methods. Europeans, for instance, use the Marshall Design method and Americans use the SuperPave mix design method to design hot asphalt mixture. The European Standards (EN13108-1to7) for “Bituminous mixture”, implemented in March 2008, is applicable for hot asphalt mix design. However, research has extended the use of the European standards to WMA mix design as well, but with the conditions that *in-situ* density requirement needs to be satisfactory. Rubio et al. (2012) stated in a study that the mix design methods which are used for the HMA mix design can also be used for the mix design of the WMA technologies. This implies that they are no restriction whatsoever to transferring the HMA mix design methods onto the mix design of WMA technologies or WMA – RAP, though slight modification may be applied.

2.2.3. Mechanical Performance of WMA

A study conducted by Ramon (2011) inidicates that WMA mixture possesses fatigue cracking resistance superior to the HMA mixture, but lower rutting resistance compared to the HMA. The better fatigue cracking performance of the WMA is justified through the reduced ageing that occurs during the mixing process at the lower temperature.

The increased performance of WMA against rutting failure could be made possible if certain types of organic additives are added into the WMA during the mixture stage. As such, Sengoz, Topal, Oner, Yilmaz, Dokandari and Kok (2017) found that WMA prepared with Sasobit organic additive, natural Zeolite additive, synthetic Zeolite additive, Rediset and Advera chemical additive may improve both its rutting and fatigue cracking performance to performances close or higher than the HMA.

The presence of an excessive amount of moisture in the asphalt mix can result in its loss of strength and durability. However, Ramon (2011) reported that the WMA mixture has an improved moisture resistance compared to the HMA mixture. The better moisture resistance of the WMA over the HMA is due to the anti-strip additive added to the WMA during the mixing stage.

In their study, Hurley and Prowell (2005) compared the structural capacity of the WMA against the HMA and found that WMA offers a structural strength value equivalent to the HMA. Based on that finding made Hurley and Prowell, one can infer that WMA, which has achieved good results through required laboratory performance tests, would surely achieve good resistance against rutting and fatigue cracking failures when it is laid within or on the flexible pavement system.

In conclusion, the good performance of the WMA in term of fatigue cracking resistance, moisture resistance, high structural capacity, the low mixing and compaction temperature gives solid reasons for the WMA to be preferred over other type asphalt mix technologies, more specially the HMA.

2.2.4. Benefits of WMA

It is challenging to group all the WMA benefits in one category to illustrate their inferiority or superiority to HMA. However, the general benefits of the WMA were made as follows:

a) Health and Environmental Benefits

D'Angelo, Harm, Bartoszek, Baumgardner, Corrigan, Cowsert, Harman, Jamshidi, Jones, and Newcomb (2008) found that there is no emission of bitumen at a temperature lower than 150°C. The authors added that significant emission of bitumen starts occurring at 150°C and above. Based on that fact, the WMA technology produces very low heat emissions and fumes due to its greenhouse gases that are much lower than the HMA.

Eventually, the low heat emission of the WMA would contribute to the safety of the environment. In addition, its reduction of emissions and fumes would undoubtly offer a comfortable and safe working environment to road workers, whose health is permanently at risk through constant exposure to fumes produced during the asphalt paving process, mostly aggravated in a non-open air situation such as a tunnel.

b) Economic Benefits

The economic benefits of asphalt mixes depend on the type of energy used in production, the cost of production, as well as the possible pollution at production. It is well known that the WMA technologies are generally produced at temperatures ranging between 80°C and 120°C, which is much lower than the HMA that are produced at a temperature ranging between 140°C and 180°C. Mokhtari and Nedjad (2013) stated that, based on the temperature factors, the low temperature production and the low fumes emission of the WMA would eventually be more economically beneficial than the HMA in regions where the cost of pollution and heat emission are very high. Howerver, Capitao, Picado-Santos and Martinho (2012) reported that the cost saving behind the low temperature production and low fumes emission of the WMA is jeopardised by the production of WMA binder and the WMA additives. This implies that the initial production cost of the WMA may be more or less equal to the production cost of the HMA.

c) Paving Benefits

The prime objective of using WMA additives is to modify the properties of the asphalt bitumen that would eventually improve the properties and the performances of the WMA. The Sasobit organic additives, for instance, modify the viscosity of the bitumen binder. The modified viscosity of the bitumen binder would, therefore, result in enhancing the workability of the WMA, facilitate its placing and reduce its compaction effort during its placement on the pavement structure.

The European Asphalt Pavement Association (2010) adds another benefit, which is paving roads in cold weather condition. The WMA is produced at a temperatures more or less closer to ambient temperature. The low production temperature of the WMA therefore presents the possibility of paving roads in cold weather condition as the drop in heat in the WMA is less dramatic. The proximity between the temperature of the WMA and the ambient temperature make the laying and the compacton time longer.

You, Goh and Van Dam (2007) stated that the WMA can maintain its workability for a longer time. This implies the possibility of making the paving possible in areas that are difficult to access. Also, it is possible to haul warm mix asphalt for longer distances. In other words, properties of the WMA cannot be affected in cases where asphalt plant sites are located at remote distances of the construction sites. The WMA can also be used in the context that necessitates rapid opening of roads such such as rehabilitations of airports, cities with high volume of traffic, and much more.

d) Production Benefits

Bonaquist (2011) indicated that the high workability of WMA mixes could accommodate a higher percentage of RAP, which makes it particularly advantageous over the HMA. The advantage of the WMA to accommodate a high percentage of RAP is possible because of the high workability of the WMA produced at a lower temperature with less ageing of the bitumen. Therefore, that low production temperature of the WMA and the lesser ageing of its bitumen does eventually counter the stiffness effect of the aged RAP binder.

2.2.5. Drawbacks of WMA

The WMA technology possesses remarkable advantages over the HMA technology, yet the WMA raises concerns that emanate from its low temperatures feature as well. Below are the reviews on the issues the WMA technology may exhibit.

a) Rutting and Moisture Susceptibility

Rutting potential and moisture susceptibility in the WMA are the result of the low production temperature, the lesser ageing of its bitumen and the presence of anti-stripping agents (ASAs) that may be added to improve the cohesion between the aggregates and the bitumen and prevent the loss of aggregates in the asphalt mix during the in-service of the pavement. As such, Hurley and Prowell (2006) proposed in a study that the reduction of moisture susceptibility in the WMA mixture is possible through proper and adequate mix design.

Hill (2011) discovered that the addition of RAP into the WMA mixtures could also be a solution to improve moisture sensitivity of the WMA. The reduced moisture sensitivity in the WMA – RAP would result in reducing premature rutting failure of the flexible pavement structure.

The addition of the RAP into the WMA once again substantiates the additional benefits of the RAP and its usability in the new construction, the rehabilitation and the maintenance of flexible pavement.

b) Cost-effectiveness

WMA technology is known for its lower energy consumption over HMA technology. This advantage of WMA technology constitutes an economical benefit to the asphalt producer, particularly located in countries where strict emission regulations are enforced. However, Kristjansdottir(2006) stated that the initial cost and the recurrent cost, also called the royalty cost, are the main reasons contractors are reluctant to use WMA. On the other hand, Diefenderfer and Hearon (2008) pointed out that the recurrent cost, that is the cost related to various WMA additives, could be significantly reduced if proven long-term performance of the WMA is achieved.

c) Long Term Performance

Vaitkus, Cygas, Laurinavicius and Perveneckas (2009a) reported in a study that it is not yet possible to prove the long term mechanical performance of the WMA as it is the case with the HMA. This is because the WMA is still known to be a relatively new technology.

Though the long term mechanical performance of the WMA has not yet been proven, previous experimental laboratory evidences have shown that some properties of the WMA are comparable to or higher than the HMA.

2.3. Recycled Asphalt Pavement (RAP)

Recycled Asphalt Pavement (RAP) is an aggregate obtained from reusing deteriorated flexible pavement. The majority of traditional flexible pavements are constructed or rehabilitated with asphalt mix that uses virgin aggregates. However, the scarcity of quality virgin aggregates due to its depletion has led to the introduction of RAP as an alternative solution to reduce the dependency of virgin aggregates and to preserve the natural resources.

The RAP incorporated in the asphalt mix can serve as a surface layer, base or sub-base course in the flexible pavement structure. In a study, Ramzi, Ali, Khalid and Muamer (2002) obtained satisfactory performances when they produced a mix of 100% RAP stabilized with 3% - 7% of cement. The authors also found that the use of 100% RAP stabilized with cement to serve as a base or a sub-base in the pavement structure could reduce the thickness of the surface layer by up to 50mm. The reduced thickness of the surface due to partial or 100% use of RAP in the asphalt mix can, therefore result in reducing the cost of pavement construction and the amount of waste generated during the rehabilitation or the upgrading of the old deteriorated pavement.

Kendhal and Malick (1997) found in one of their studies that the use of 20% to 50% RAP in pavement construction or rehabilitation can save up to 34% of the total cost from the design stage to the maintenance stage. Mohammad Nasir and Mohamad Yusri (2016) added that the reduction in the total cost due to the reduction of virgin material in the asphalt, the transportation cost of the virgin material, and the reduced amount of asphalt binder used in the asphalt mix may be possible through the adding of RAP in the asphalt mix.

Ramzi et al. (2002) conducted some laboratory tests in line with the AASTHO guideline to compare the physical properties of RAP with virgin aggregates. The following were found:

- Sieve analysis performed as per AASHTO T27 showed that RAP is a well-graded (GW) aggregate;
- RAP has a maximum dry density that is comparable to other virgin aggregates;
- RAP presents a much lower optimum moisture content compared to virgin aggregates;
- The Atterberg (liquid) limit of RAP performed following AASHTO T89 was found to be eight and was non-plastic as it is the case with virgin aggregates;
- The sand equivalent of RAP (97%) was found to be higher than the virgin aggregates (67%)
- The Los Angeles abrasion of RAP (33.6%) was higher than the virgin aggregates (18.8%).

Overall, the mulitiple benefits which include structural, environmental, and economical benefits, which the RAP offers, make it safe, durable and reliable for the design and the construction of sustainable roads.

2.4. Warm Mix Asphalt Incorporating Recycled Asphalt Pavement

The incorporation of RAP into the WMA at up 100% undoubtedly guarantees environmental and health benefits. However, researches are still progressing to guarantee its satisfactory distress performances at maximum percentage of RAP. Vaitkus, Vorobjovas and Ziliut (2009b) conducted studies on WMA mixed with RAP with percentages above 50% and came up with good results.

Warm Mix Asphalt (WMA) incorporating RAP technology has merged into the engineering of pavement as an exceptional pavement material that is environmentally friendly and cost-efficient. A further advantage is that it guarantees a safer working condition for workers. The WMA incorporating RAP material pavement technology has long been adopted and used in a few states in America and some of the European countries. Naidoo, Marais, Nordje, Rocher and Lewis (2011) reported that South Africa, through the South African Bitumen Association (SABITA), has recently started producing WMA incorporating RAP aggregates with the first trial done in 2008. Therefore the recent coming of WMA – RAP in South Africa is an indicator that researches need to be done on WMA – RAP using local materials bitumen binder and additeves.

2.4.1. Mechanical Performance of Warm Mix Asphalt Incorporating Recycled Asphalt Pavement

The main advantage of WMA technology compared to HMA resides in its capacity to accommodate the high amount of RAP owed to the increased workability in WMA. Experimental laboratory studies have shown that the WMA incorporating RAP can exhibit good fatigue and rutting performance and moisture resistance if properly mixed with adequate WMA additives. This is substantiated by a study conducted by Oliveira, Silva, Peralta, and Zoorob (2012), when they found that the WMA incorporating RAP possesses better fatigue resistance compared to WMA mixed with 100% virgin materials.

Sengoz et al. (2017) conducted experimental studies, the results of which showed that WMA incorporating RAP aggregate has a high stability value and a low deformation value which leads to a high Marshal Quotient value (MQ). The high MQ of the WMA incorporating RAP aggregates signifies that it is highly capable of spreading the applied load uniformly across the WMA – RAP pavement structure. The advantage of a good load distribution is that the pavement structure would stand a better chance to creep deformation.

2.4.2. Benefits of WMA Incorporating RAP

The WMA incorporating RAP aggregates is an asphalt mix technology of which its progressive preference over the traditional HMA is based on the numerous benefits it possesses. Kusam (2014) found that the WMA could offer satisfactory working conditions in terms of better workability, through the addition of PTI foaming mixture which reduces viscosity better than Evotherm 3G additive. Although WMA chemical additive was found to improve certain properties of WMA incorporating RAP aggregates, it appears that the presence of RAP aggregates mixed with WMA chemical additives reduces the thermal cracking resistance of WMA incorporating RAP aggregates (Hill, Behnia, Buttlar & Reis 2013). Guo, You, Zhao, Tan and Diab (2014) argued that WMA chemical additives do not affect the low thermal cracking resistance of WMA incorporating RAP. These findings imply that WMA – RAP constructed in regions that experience extreme hot and cold weather conditions may develop cracking failure. In other words, cracking failure on the WMA – RAP may be both by the extreme weather condition and loads repetition.

Chiu, Hsu, and Yang (2008) reported that the Polycyclic Aromatic Carbon (PAH) used in the production of HMA is known to be environmentally aggressive. This PAH compound is significantly reduced by up to 50% during the production of WMA mixed with RAP. This has, therefore, resulted in the benefit of workers being less exposed to fumes and PAH (WAM-foam) and energy being saved by up to 23%. Further studies have also indicated that the reduced fuel consumption during the production of WMA incorporating RAP has lowered the fumes and the greenhouses gas emission, therefore making WMA incorporating RAP aggregate more environmentally friendly than other types of asphalt mixes.

WMA incorporating RAP material as a highly sustainable product due to their cost-effectiveness and environmental friendliness has greatly contributed to lower ing the environmental impact caused by the construction or rehabilitation of pavement. Mohammad Nasir and Mohamad Yusri (2016) in the *ARPN Journal Of Engineering And Applied Sciences*, agreed when they stated that the addition of RAP material in the WMA reduces the environmental impact by up to 23%.

In summary, the advent of WMA incorporating RAP has made significant contributions to saving the environment, cutting down cost on road construction in term of materials as well as reducing the dependency of virgin aggregates through the use of sustainable materials to develop and to produce a better quality of flexible pavements.

2.4.3. Drawbacks of WMA Incorporating RAP

There are other factors besides the extreme low and high weather conditions and the high loads repetition which may cause the failure of the WMA – RAP pavement through fatigue cracking. Zhoa, Huang, Shu and Woods (2013) reported that the additional factor at the origin of pavement fatigue cracking is the high percentage of RAP in the asphalt mixture. In other words, the fatigue cracking resistance of WMA incorporating RAP tends to decrease when the RAP in the asphalt mix is increased.

Xhoa et al. (2013) further suggested that the reduced proportion of the RAP in the WMA would result in its improved resistance against fatigue cracking. Although solving the problem of fatigue cracking in the WMA – RAP pavement resides in the the reduction of RAP in the WMA, yet they will eventually contribute to the elevated cost of the production of the WMA – RAP and promote the dependency of virgin aggregates. More effective solutions may be found in this study.

2.5. Pavement Design Methods

The design of a flexible pavement structure or a rigid pavement structure is the art of developing adequate layer thicknesses and layer materials capable of carrying forecasted traffic load during a determined service life.

Conventional flexible pavement generally consists of a surface asphalt layer, an unbound aggregates base layer and a stabilized or treated sub-grade layer. Thus, the purpose of designing a pavement structure is to ensure that its layers system is not prematurely overstressed as a result of external factors (traffic loading and environmental condition) and internal factors (layers materials). Therefore, the empirical design method and the mechanistic–empirical design method are the two major approaches that are used in the design of pavement structure.

Over the years, the empirical design method has been used in the design of roads. The empirical design method relies solely on the field experiments (road already contructed) or the engineers' experience to justify the relationship between the input variables (traffic, materials layer, climatic condition) and the outcome (pavement distress). Gadhimi (2015) stated that pavement design based on empirical approach is doubtfull and cannot be reliable as it does not provide any comprehensive scientific knowledge to explain pavement mechanical responses. Wei Tu (2007) added the loading conditions, the environmental condition and the materials variation as the limits rendering the empirical design method inefficient.

The Mechanistic-Empirical (ME) design method was introduced in the early 2000s to overcome the limitations of the primary and traditional empirical design. These are the ability to accommodate loads changing, the better characterization of materials, the provision of more reliable performance prediction, the better definition of the role of construction, the ability to accommodate environmental factors, heavier loads factor and the ageing effects on materials, the usability in both rehabilitation of the existing pavements and the development of new roads are the various advantages of mechanistic-empirical design method over the purely empirical design method (Pavement Interactive, 2010). Nonlinear, anisotropic, inhomogeneous, viscoelastic, elastic and particulate are the characteristics pertaining to materials constituting the layers of flexible pavement. de Holanda, Junior, de Araújo, de Melo, Junior and Soares (2006) stated that the complex constitutive behaviour of flexible pavement could make solutions to stress, strain and displacement problems very complicated or even impossible to solve in the functional design condition unless the mechanistic-empirical design approach is considered.

Overall, a pavement structure or any type of engineering structure develops stress, strain and deflection phenomena when subjected to external physical forces. As such, the mechanistic design method on one hand seeks to explain through mathematical models the various phenomena that occur within a pavement structure as a result of loading. On the other hand, the empirical design methods are developed to define the amount of stress, strain and deflection that would cause pavement distresses, which include fatigue cracking failure and rutting failure.

Besides the long used traditional empirical design method and the recently introduced ME design method, other design methods such as the limiting shear failure method, the the limiting deflection method and the regression method were developed and used between the years 1940 and 1960. Those design methods, however, became obsolete and were abandoned due to their various limitations and lack of fundamental requirements in the ideal design of pavement (Huang 2004).

Overall, the advantages of the mechanical–empirical design method over the traditional empirical design method in the ideal design of pavement structure would be more beneficial to achieving the analysis and the design of the asphalt pavements considered in this dissertation.

In conclusion, many researches have been conducted since the advent of WMA incorporating RAP aggregates. Those researches have found several benefits and drawbacks of the WMA – RAP at various RAP percentages, stabilized with different type of binders and additive technologies. However, there is a need to conduct experimental studies to investigate the Marshall performances and the distress performances of the WMA – 15% RAP and the WMA – 30% RAP mixed with Sabit additive. In addition, there is a neccessity to explore the mechanical performances and to predict the distress performances of the WMA – 15% RAP pavement structures and WMA – 30% RAP pavement structures through numerical modelling and simulation.

2.6. Summary

Chapter 2 provides an extensive literature review of the benefits and drawbacks of using WMA technology, the RAP, and the WMA incorporating RAP technology on the economic aspect, the environemental aspect, the health aspect and the health aspect. It also reviews the mix designs methods that are adopted in the production of WMA and are applicable to the production of WMA – RAP. Furthermore, it gives a comprehensive literature review of the performances of the WMA and WMA – RAP in term of fatigue cracking, rutting failure and moisture susceptibility. Finally, it reviews the various pavement design methods that are used in the sound design of flexible pavement structures. The next chapter provides a detailed methodology adopted for the achievement of this dissertation.

CHAPTER 3 – METHODOLOGY

3.1. Introduction

Evaluating the mechanical performance of the WMA – 15% RAP and the WMA – 30% RAP and comparing them against the control HMA were achieved through experimental studies. The data needed were purely quantitative and were collected through various laboratory tests and numerical modelling. The methodology that governs the achievement of this study is described through described phases.

3.2. Phase I – Asphalt Mix Design and Fabrication of Asphalt Specimens

The mix design of the WMA incorporating RAP and HMA were achieved using the Superpave mix design method.

The Superpave method which stands for Superior Performing Asphalt Pavement was implemented by state agencies in the United States to substitute HVeem and Marshal Design methods (Westruck Forensic Team, 2001). The Superpave mix design method has two key components, which are the performance-based asphalt binder specification and the volumetric mix design and analysis. Furthermore, the Superpave-graded is centred on providing binder capable of resisting fatigue cracking and rutting failure (Westruck Forensic Team, 2001). The fabrication of the mix specimens using Superpave mix design method was conducted as follow:

- **Materials Selection**
  - **Virgin Materials**

Aggregates are the largest constituents of asphalt mixtures, making approximately 85% of the mixture by volume and 95% of the asphaltic mixture by weight. The characteristic and quality of aggregate plays an important role in the performance of asphalt mix design.

In this study, dolerite rock was selected as the most appropriate type of virgin aggregates to serve for the fabrication of the WMA – 15% RAP, the WMA – 15 RAP and the control HMA.

The dolerites rock used in the mix design includes coarse aggregates (28mm, 20mm, 14mm, and 10mm), fine aggregates (crusher dust) and fillers (lime).

Numerous tests such as the grading analysis, atterberg limit, flakiness index, aggregate crushing value (ACV), fine aggregate crushing value test (10% FACT), deleterious materials, fractured faces, durability and water absorption were performed on the dolerites virgin aggregates in order to ensure their suitability in the asphalt mixture as per the design requirement and specification. Furthermore, G4 quality materials were used as it responds better to medium trafficked road and dry climate conditions (South African Pavement Engineering Manual, 2014).

- **Recycled Asphalt Pavement aggregates**

Before proceeding with the addition of RAP into the WMA, the properties of both the aged asphalt bitumen and the RAP aggregates were determined. The separation of aged bitumen binder from the RAP aggregates was to determine the amount of virgin bitumen binder that should be added into the WMA – RAP.

The extraction and the measurement of the old asphalt bitumen were done through the solvent extraction technique, which consists of using solvent such as trichloroethylene or alternatively an ignition oven and solvent combination (Devecseri, 2010).

The determination of the bitumen contained in RAP (P_b) consisted of preparing ten (10) batches of RAP each weighing one (1) kilogram (Kg). An extraction test was then performed on each of the batches with a centrifuge extractor, called a Rota Test.

After the above extraction test was completed, the properties of the aged bitumen extracted were characterized through conventional tests. A sieve analysis test was finally performed on the extracted aggregates and its bulk specific gravity (G_{sb}) was obtained.

- **Selection of Virgin Bitumen Binder**

The Superpave design method generally requires that the selection of asphalt binder performance grade (PG) should be based on the traffic loads and the estimated pavement temperature prevailing in the region of interest. In this study, the asphalt binder PG 50/70 was selected as it was appropriate for the temperature prevailing in the Free State province.

The optimum binder content of 4.5% was selected based on the maximum size of aggregates in the asphalt mix, this with an allowable target air void (Va) varying between 4% to 6%.

- **Bitumen additives**

In order to reduce the viscosity of the bitumen binder and improve the workability of the asphalt mixes, 2% Sasobit organic additive was added into the bitumen binder pre-heated at the temperature of 110°C before mixing.

- **Mix Design and Production of Asphalt Premixes**

The sieve analysis was performed on aggregates following procedures and requirements of the South African National Standard (SANS). A mix proportion of aggregates was then determined to ensure that various aggregates sizes are proportionally distributed in the asphalt mix.

The production of asphalt premixes was achieved by blending the virgin aggregates, the RAP, the bitumen binder into an asphalt mixer at 150 ºC for the HMA and at 120ºC for the WMA – 15% RAP and the WMA – 30% RAP. The asphalt premixes were then portioned to serve for properties measurement and the fabrication of asphalt specimens.

- **Fabrication of Asphalt Specimens**

Slabs, briquettes and cylindrical specimens were made to perform the Marshall Stability and Flow test, the Indirect Tensile Strength (ITS) test, the Hamburg Wheel Tracking test, the Four-Point Beam Bending test and the Dynamic Modulus test.

The HMA were mixed at 150ºC and compacted at 140ºC whereas the WMA – 15% RAP and the WMA – 30% RAP specimens were mixed at the temperature of 120ºC and compacted at 110ºC.

The cylindrical specimens aimed for the ITS and Marshal Stability and Flow Test were compacted using the Marshall Drop Hammer. The cylindrical specimens aimed for the Hamburg Wheel-Tracking Test and the Dynamic Modulus Test were compacted using a gyratory compactor. Finally, the slabs aimed to perform the Four-Point Beam Bending Test were compacted using a slab compactor machine.

The gyratory compactor was operated as per the AS/NZS 2891.2.2:2014 standards specification. The number of gyrations was adjusted at 120, the ram pressure of 240 kPa was applied to account for heavy traffic loads, and the gyration angle was maintained at an angle of 3°.

The volumetric properties, which include the void in mineral aggregate (VMA), the voids filled with bitumen (VFB) and the air void content (Va) of the asphalt premixes and the compacted asphalt specimens were determined. The volumetric properties are generally controlled to ensure that the asphalt mix performs well on the field as expected.

Further properties of the asphalt mix, which include the binder content (BC), the bulk density (BD) as well as the void in the mix (VIM) popularly known as the rice test, were also determined.

3.3. Phase II - Laboratory Experiment

- **Hamburg Wheel-Tracking Test**

The Hamburg Wheel-Tracking Test was performed in order to evaluate the susceptibility of the asphalt mixes to deformation under applied repetitive wheel load.

The Hamburg Wheel-Tracking Test was performed by first preparing two cylindrical briquettes per asphalt mix. The cylindrical briquettes measuring 150mm in diameter and 60mm in depth were prepared and subjected to repetitive wheel loads of 700N and at a fixed temperature of 50°C. The characteristics, which include the thickness, the void in the mix, the bulk density, the maximum theoretical density of the cylindrical asphalt mix specimen were determined and used as input data needed to perform the Hamburg Wheel-Tracking Test.

- **Four–Point Beam-Bending Test**

The WMA – 15% RAP, WMA – 30% RAP and the control HMA – 100% beam specimens were subjected to the Four-Point Beam-Bending Test in order to evaluate their fatigue cracking failure at an intermediate temperature. The intermediate temperature is assumed to be the temperature to which the pavement operates.

The Four-Point Beam-Bending Test was performed by first preparing two beams per asphalt mix. The beams measuring 62 mm in width, 48 mm in depth and 380 mm in length, were subjected to a selected load of 2.5 KN at the frequency of 400µƐ and 1.4 KN at the frequency of 200µƐ. Furthermore, the input data such as the bulk density of the beam and the void in the beam were determined prior performing the Four-Point Beam Bending Test.

- **The Dynamic Modulus Test**

The Dynamic Modulus Test was performed on the WMA – 15% RAP, the WMA – 30% RAP and the the control HMA – 100% cylindrical specimens to determine their elastic modulus properties. The elastic modulus of the asphalt concrete is considered to be an essential input data in the Abaqus and mePADS computer programs using FEM and LEA respectively.

The cylindrical asphalt specimens used for the Dynamic Modulus measure 147 mm in height and 100 mm in diameter. The asphalt specimens were conditioned at 20°C in an oven for several hours before subjecting them to loads at various rates in the axial compression test machine.

- **The Marshall Stability and Flow Test**

The Marshall Stability and Flow Test was performed to measure the ability of the WMA – 15% RAP at WMA – 30% RAP and the HMA – 100% to resist distortion, displacement, rutting and shearing through maximum load applied. Two cylindrical briquettes measuring 66mm average in thickness and 101.5mm in diameter were first conditioned into a hot water bath at the temperature of 60°C for 30 minutes before subjecting them to Stability and Flow Test.

- **The Indirect Tensile Strength Test (ITS)**

The ITS tests were performed to evaluate the stiffness modulus of the WMA – 15% RAP, the WMA – 30% RAP and the HMA – 100% specimens before performing the flexural fatigue test, the rutting test and the dynamic modulus test.

Four cylindrical briquettes measuring 66mm average in thickness and 101.5mm in diameter were conditioned at 25°C in the oven before subjecting them into an ITS testing machine.

The entire first and second phase can be summarized in the form of Figure 3.1 below presented:

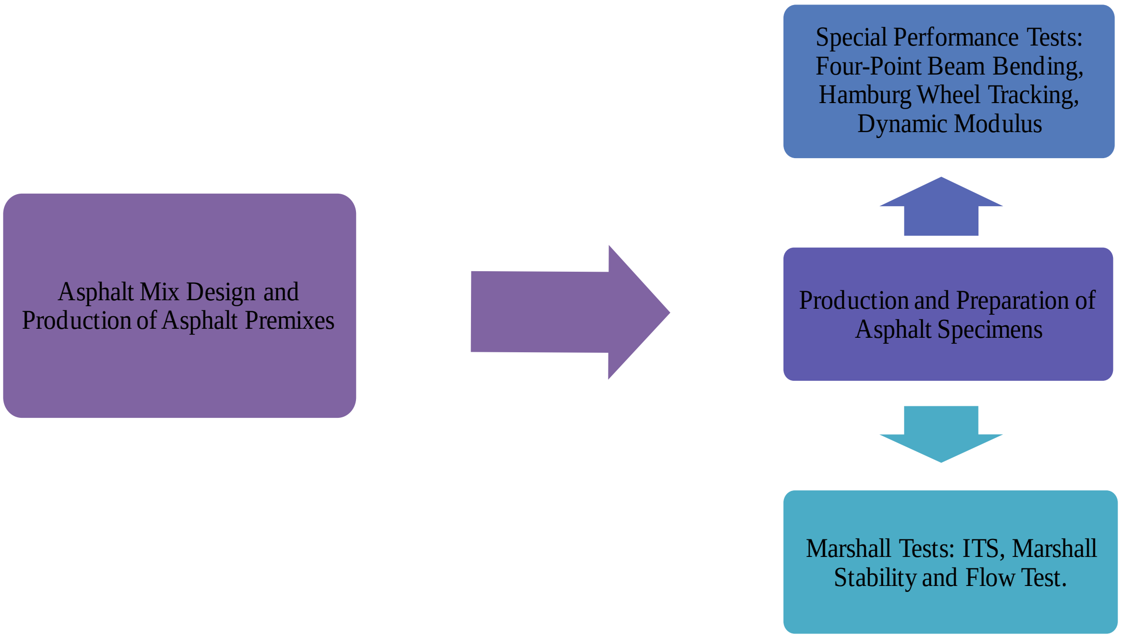

Figure 3.1. Schematic Summary of Phase I and II

3.4. Phase III – Modelling

The stress, the strain and the displacement phenomena within the flexible pavement generally occur as a result of static loads, dynamic loads, and climatic effect. Those phenomena would eventually lead into pavement failures known as rutting failure and fatigue cracking failure when their allowable value is exceeded.

The modelling, the simulation and the analysis of the asphalt pavements, considered in this study were achieved using the Mechanistic–Empirical method (ME). Besides the ME method there is another method called the empirical method for pavements design. However, the choice of ME method over the empirical one is because the ME method has the capability to accommodate heavier traffic loads, new materials properties, various climatic conditions as well as its ability to handle long-life pavement designs.

The ME method uses mathematical models such as the Finite Element Model (FEM) to determine the stress, the strain and displacement phenomena within the flexible pavement. There are other approaches, such as the layered elastic analysis method, the generalized finite element and the discrete finite element method, that exist for pavement numerical modelling (Seracino & Rhaman, 2013). However, the choice of Finite Element Model (FEM) over other numerical methods resides in its capability to analyse distresses of complex pavement structure (dynamic interactive pavement) with complex material properties and so obtain approximate solutions to particular problems (Khennane, 2013). Layered Elastic Analysis (LEA), on the other hand, was also used to compare and validate the results obtained in FEM. The LEA model assumed that each layer in pavement structure is homogeneous, isotropic and linearly elastic. This signifies that it is the same everywhere and will return to its original position once the load is removed.

The FEM built-in Abaqus computer program and LEA built-in mePADS computer program were used in the numerical modelling and analysis of the flexible pavements structure considered in this dissertation. These two computer program were selected based on their availability at the time of the study.

There are two types of pavement analysis models in the Abaqus program which are a static and a dynamic model. However, the static model was chosen in this study based on the fact that the asphalt pavements were analysed in 2D axisymmetric. On the other hand, mePADS, which is a South African modelling program, oprates only in the static model and in 2D.

The results obtained from the calculated stress and strain were used as input data to predict the fatigue cracking and the rutting failure, using empirical equations (also called transfer function) as shown below (Bruce, 2001). It should be noted that these empirical fonctions are unique:

- The fatigue cracking was calculated based on the following equation:

  $$\mathrm{Nf} = 18.4\,(0.00432\mathcal{E}_t - 3.291\mathrm{E} - 0.854)$$

  where:

  N_f = the number of load repetition causing fatigue failure

  $\mathcal{E}_t$ = the horizontal strain at the bottom of the asphalt layer

  E = the modulus of the asphalt layer in Psi

  Another way to determine the fatigue failure was from laboratory testing as described above.

➢ The rutting failure was calculated as follow:

$$N_R = 5.5\times10^{15}\left(\frac{1}{\epsilon\upsilon\times10^{6}}\right)^{3.949}$$

where:

N_R = number of loads required to cause base rutting failure;

$\mathcal{E}\upsilon$ = the vertical strain at the top of the sub – grade.

The design model of the flexible pavements with various structural scenarios can be summarized as presented in Figure 3.2 below

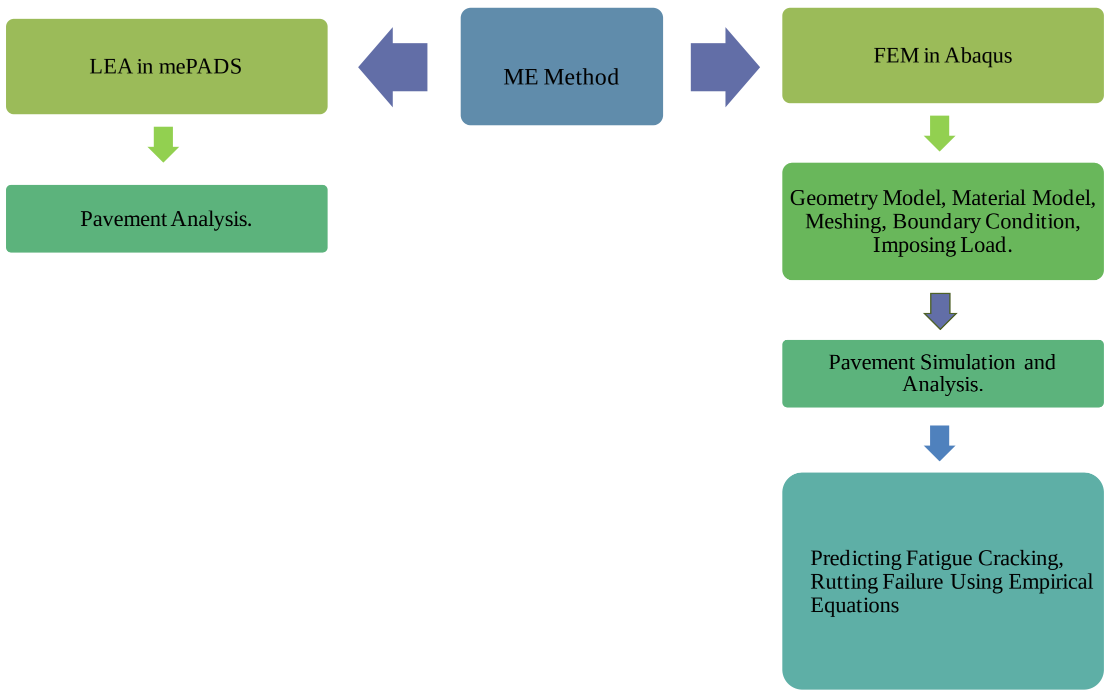

Figure 3.2. Schematic Summary of Phase III

CHAPTER 4 – MIX DESIGN AND PRODUCTION OF ASPHALT PREMIXES

4.1. Introduction

The mix design and the production of WMA – RAP premixes and the HMA premixes constitute one of the core parts of this study. It is, therefore, of utmost importance that, to obtain reliable performance test results on the asphalt compacted specimens, successful selection of materials, appropriate materials mix designs and quality production of asphalt premixes be achieved.

The Marshall method, the Hveem method and the Superpave method are the mix design methods used for the mix design of asphalt mix. The Superpave is the new design process aimed to provide Superior Performing Asphalt Pavement (Superpave), and it has been implemented by many agencies in several countries to replace the traditional and obsolete Marshall and the Hveem method (Westrack Forensic Team Consensus Report, 2001). Hence, the choice of the Superpave mix design method in this study over the Hveem and the Marshall Mix design methods. The Superpave mix design procedure involves the materials selection and their characterisation as well as the production and the characterisation of asphalt premixes.

The materials used in the production of HMA and the WMA – RAP premixes were first tested to verify their suitability in the asphalt premix. Therefore, the tests carried out on the materials include the aggregate sieve analysis, the bulk density, the water absorption, the sand equivalency, the flakiness index, the percentage of bitumen content in the RAP, the rate of moisture content, the fine aggregate, the polished stone value, the angularity and the 10% fine aggregate crushing test. The tests carried out on the bitumen binder include the penetration test, the softening point test and the Brookfield viscosity test. Moreover, the volumetric properties of the asphalt premix, which include the theoretical density of the mix, the bulk density of the aggregates (BD_A), the air voids percentage (V_v), the volume percentage of the bitumen (V_B), the volume of the aggregates (V_A), the voids in mineral aggregates (VMA) and the percentage of voids filled with bitumen (VFB), were finally found.

Thus, the asphalt production guidelines and the specification manuals based on Superpave method for the quality production of asphalt mixes were utilized to aid in the appropriate selection of materials, the production of asphalt premixes and their characterization.

The results obtained were compared and validated based on the design criteria set by agencies' standard specifications such as the American Society for Testing and Materials (ASTM) and the American Association of State Highway and Transportation Officials (AASHTO).

4.2. Asphalt Mix Design

The objective of asphalt mix design is to determine the combination of binder and aggregates that will offer excellent workability, durability, cost-effectiveness and fatigue performance (Sabita Asphalt Manual 35, 2019). Furthermore, the design of the asphalt mix must possess suitable aggregates configuration in terms of structure and space between particles to accommodate the bitumen binder, thus preventing bleeding and permanent deformation. Abedali (2014) associates the poor performance of asphalt pavement to either poor and inappropriate mix design, or to the in-situ production of asphalt mixture different from what was initially designed in the laboratory. As such, it is of paramount importance to achieve a good mix design that will eventually lead to accurate results of the performance tests

The asphalt specimens subjected to the Marshall performance tests, the Four-Point Beam-Bending test, the Hamburg Wheel-Tracking test and the dynamic modulus, were fabricated in accordance to the level II mix design that satisfies a category B pavement structure (heavy traffic volume of up to 30 million ESALs) forecasted to serve for 20 years.

4.3. Preparation and Selection of Materials

The materials used in this study for the fabrication of HMA and WMA incorporating RAP technology include the coarse virgin aggregates (28mm, 20mm, 14mm and 10mm), crusher dust, fillers, lime, fine RAP, virgin binder (50/70), and Sasobit organic additives. Various tests were performed on the aggregates to evaluate their properties and their suitability in the asphalt mixtures.

4.3.1. Bitumen

In this study, the selection of penetration grade bitumen was made based on the climatic conditions that prevail in the Free State province in South Africa, the mode of damages, the availability of aggregates locally available, and the level of traffic the asphalt mixes would withstand.

As far as the location is concerned, it is assumed that Bloemfontein would be the region of interest. The Sabita Manual 35/ TRH 8 (2019) presents four levels of mix design, which are level IA, level IB, level II and level III. Each level of mix design has a performance related to a maximum volume of road. In other words, the level IA has performances that apply to light and low-volume road (0.3 million E80s), the level IB has performances that apply to low and medium-volume road (0.3 million to 3 million E80s), the level II has performances that apply to medium and heavy-volume road (0.3 million to 30 million E80s), and the level III has performances that apply to heavy and extreme heavy-volume road (30 million E80s and above). As far as this study is concerned, level II mix design was selected to serve in the design of asphalt mixes. The asphalt mixes were designed to suit in the pavement as treated base course or asphalt surface. According to the Sabita Manual 35/ TRH 8 (2019) regarding the level II mix, the design is expected to undergo moderate to severe rutting failure and fatigue cracking failure,.hence, the manual further recommends a good stiffness performance of the asphalt layer to overcome premature failures.

As far as the binder grade selection is concerned, the Sabita Manual 35 recommends that the selection of PG asphalt binder be based on the 7-day average maximum asphalt temperature at 20mm depth (Sabita Manual 35, 2019).

The South African maps depicting the 7-day average maximum asphalt temperatures at 20 mm depth and the 1-day minimum asphalt temperatures at the surface were utilized to aid in the determination of temperature prevailing in the zone of interest.

As such, according to the latest CSIR map, it was found that the temperature prevailing on the Free State paved roads can reach up to 60°C in summer and drop down to -6.5°C in winter (Sabita Manual 35, 2019). Based on the regional temperature provided by the maps in Figures 4.1 and 4.2, the asphalt binder PG 50/70 was selected as it best suits the high and the low temperatures that prevail in the city of Bloemfontein. Furthermore, the asphalt bitumen PG50/70 was added at 4.5% in the total asphalt mix mass.

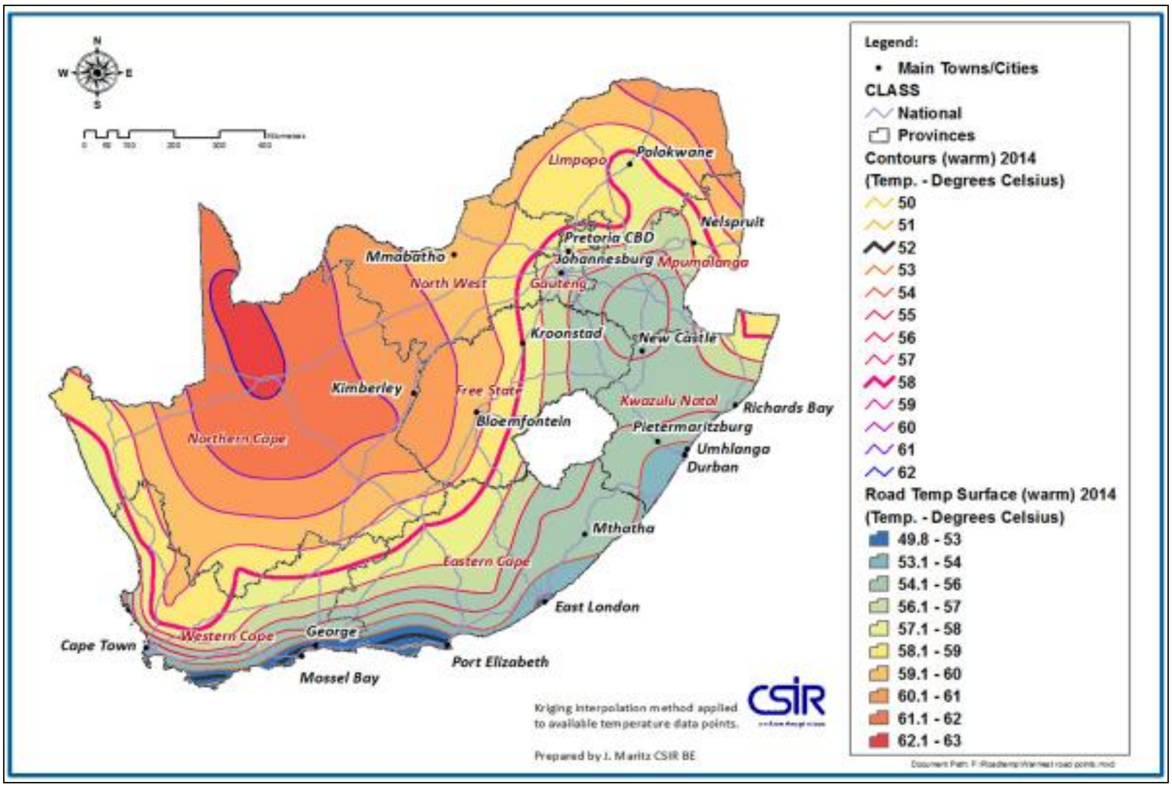

Figure 4.1. Day Average Maximum Asphalt Temperatures (Sabita Manual 35, 2014)

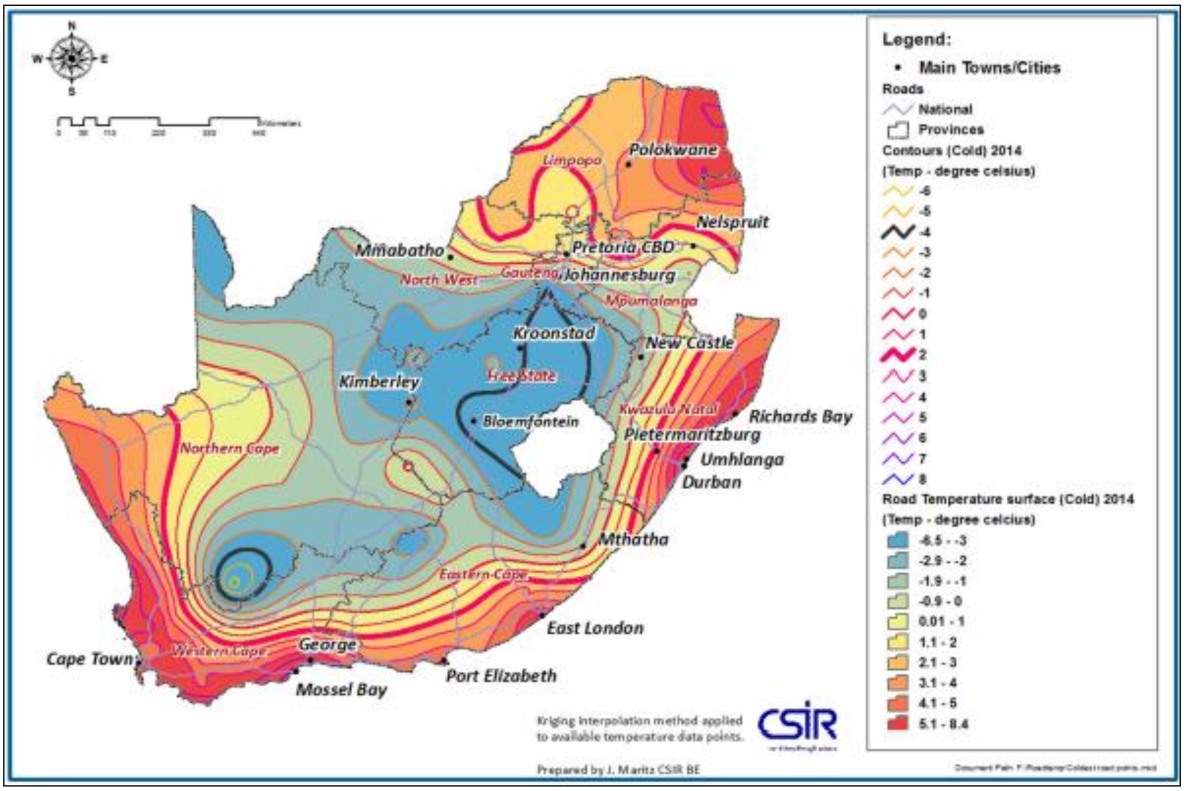

Figure 4.2. Minimum Asphalt Temperatures (Sabita Manual 35, 2014)

To ensure that the performance of bitumen on the road is satisfactory, the properties, which include the rheology, the adhesion, the cohesion and the durability, should be controlled (Read, Whiteoak, & Hunter, 2003).

However, this study focuses uniquely on the rheology property of the bitumen binder to assess the durability of the bitumen. The rheology property of bitumen is essentially characterized by the value of penetration, softening point and the viscosity.

Read et al. (2003) further state that the durability of bitumen is the ability to maintain satisfactory rheology, cohesion, and adhesion properties of bitumen in the long-term service. The authors also add that the oxidative hardening, the evaporative hardening and the oxidative hardening are identified to be the durability factors of bitumen. In other words, hardening is the principal cause of bitumen ageing.

The penetration, the softening point values as well as the viscosity of the bitumen PG 50/70 were determined using the Penetrometer apparatus, the Ring-And-Ball apparatus and the Rheometer respectively. The test procedures were observed using the standard test methods EN 1426 to test for the penetration, the ASTM D36M to test for the softening point and the ASTM 4402 to test for the viscosity.

The results of the bitumen binder 50/70 revealed the penetration to be 61.5mm, the softening point to be 48.3°C and the Brookfield viscosity to be 166.7 Pa.s at 60°C and 285mPa.s at 135°C. Thus, those results fall within the range of standard specification which recommends the penetration value to lag between 50 - 70, the softening point to be between 46 - 56 and the Brookfield viscosity to be 120 Pa.s min and 300 Pa.s at 60°C and to range between 220 and 500 mPa.s at 135°C.

To put it bluntly, the analysis of the results found shows good performance of the bitumen binder 50/70 against ageing as it falls within the standard specification (SANS 4001-BT1; 2016).

4.3.2. Warm Mix Asphalt Additive

The Sasobit added into the virgin bitumen binder PG 50/70 at 2% of the total mass of the binder was aimed to reduce the viscosity of the asphalt bitumen and therefore to enhance the workability and the compaction of the recycled asphalt mix. The RAP is enveloped by a very aged hardened binder hence its stiffness nature. According to the Sabita Manual 32 (2011), the high proportion of RAP (15% and above) in the recycled asphalt mix must be accompanied by the use of softer bitumen grade and rejuvenator. Thus, the increased amount of the Sasobit into the binder was based on the increased amount of RAP in the WMA.

Furthermore, the increase in Sasobit relative to the increase in RAP aggregates in the recycled mix was to examine the effect of Sasobit organic additive in terms of mechanical performance of WMA with a high percentage of RAP.

The adding of Sasobit into the bitumen binder was preceded by first heating up the bitumen binder PG 50/70 at 110°C. Once the desired temperature of the PG 50/70 was reached, the Sasobit was added into the bitumen and stirred up until total dissolution was noticed. The modified PG 50/70 binder was then poured into the mixed aggregates as per the manufacturer's recommendation. The manufacturer indicates the Sasobit melting point to be 100°C and full miscibility to be 115°C. In addition, Jamshidi et al. (2013) found that beyond 100°C melting point, the Sasobit liquefies and greatly reduces the viscosity of the bitumen, resulting in reducing its mixing temperature by up to 30°C. The author added that at any temperature lower than the melting point mentioned above, the Sasobit would form a lattice structure in the bitumen binder, leading to better stability.

4.3.3. Aggregates

The virgin aggregates used in the mix design were dolerites rock, which includes coarse aggregates (28mm, 20mm, 14mm, and 10mm), fine aggregates (crusher dust) and fillers (lime).

In South Africa, dolerite rock may be used for the construction of road layers such as the surface layer, the base layer as well as the sub-base layer. Furthermore, the preference of dolerite rock over other types of rock available in South Africa was based on their local availability, which in this case is the Free State province (SAPEM, 2014). Dolerite can also be found in other provinces of South Africa, which include Gauteng, KwaZulu- Natal, Mpumalanga, the Northern Cape, and the Eastern Cape (SAPEM, 2014).

The coarse and fine dolerite aggregates used for the WMA – RAP and HMA were cleaned and freed of decomposed materials, vegetable matter and other harmful substances. The virgin materials were oven-dried for approximately 17 hours before proceeding with the grading test.

Furthermore, the grading tests carried out on the coarse and the fine aggregates, as well as the fillers, were done using the standard test method SANS 3001-AG1. The results recorded are presented below in Table 4.1 and Figure 4.3.

Table 4. 1. Virgin Materials Gradation Test Results

Agg size	Sieve analysis and % passing									
	Aggregates						Fillers	Combined Grading	Specification	
	28	20	14	10	7.1	C/DUST	Lime			
% In Mix	**12.0**	**12.5**	**12.0**	**8.5**	**9.5**	**44.0**	**1.0**	**100**	**Min**	**Max**
37.5	100	100	100	100	100	100	100	100	100	100
28.0	90	100	100	100	100	100	100	95	86	95
20.0	8	91	100	100	100	100	100	86	73	86
14.0	2	24	98	100	100	100	100	76	61	76
10.0	1	2	30	83	100	100	100	66	52	68
7.1	1	1	3	11	93	100	100	55	57	72
5.00	1	1	1.1	3	22	98	100	47	37	54
2.00	1	1	1.1	2	4	64	100	30	23	40
1.00	0.5	1.2	1.1	2.2	3.6	41.7	100	20	17	32
0.600	0.5	1.2	1.1	2.2	3.4	31.6	100	16	13	27
0.300	0.5	1.2	1.1	2.2	3.2	22.7	100	12	9	21
0.150	0.5	1.2	1.1	2.1	3.0	17.1	99	9	6	17
0.075	0.5	1.1	1.1	2.0	2.7	13.4	90	7.6	4	12

(Source: Author, 2019)

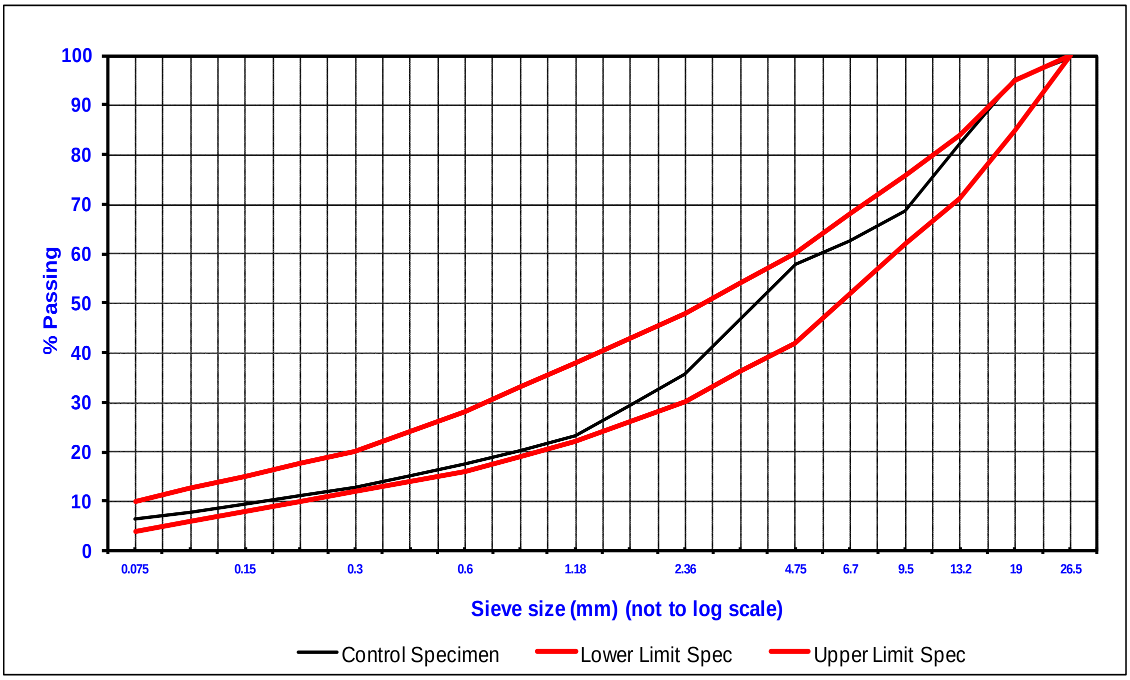

Figure 4. 3. Grading Envelope Graph of the Virgin Materials (Source: Author, 2019)

The results in Table 4.1 above show a good aggregate blending proportion as the combined aggregates fall within the guideline specification.

Furthermore, the results of the grading test were used to plot the grading envelop graph. The upper line and the lower line of the grading envelope represent the boundary of the guideline specification; the black line inbetween represents the combined gradation result of the coarse aggregates, the fine aggregates and the fillers.

The conclusion drawn from the grading test table appears to be similar with the grading envelop above plotted as the combined aggregates results lay pretty well between the upper and the lower boundary lines specification. Adjustment of the proportion of the aggregates was suitably made when the initial blended aggregates failed to meet the criteria.

4.3.4. Recycled Asphalt Pavement

RAP aggregates were collected from a completely deteriorated road in the city of Bloemfontein and transported to Lafarge plant to be crushed. The crushing of the RAP at Lafarge crushing plant was to obtain the production of fine RAP through the process of crushing, screening and fractioning. The RAP used in this study was free of foreign materials such as unbound granular base, broken concrete and remnants of geosynthetic grid or clothes.

The fine RAP was mixed at various proportions with the virgin aggregate as well as the un-aged bitumen and Sasobit additive to produce the WMA incorporating RAP at 15% and 30% to fit the purpose of bitumen-treated base (BTB) or asphalt surface within the pavement structure. The virgin aggregates of different sizes and the RAP were sieved over the standard nest of sieves.

In addition, the grading test carried out on the coarse and the fine aggregates, the fillers and the RAP were done using the standard test method SANS 3001-AG1. The results were recorded and are presented below in Table 4.2 and Figure 4.4.

Table 4.2. Gradation Test Results of Virgin Materials Combined with 15% RAP

Sieve analysis and % passing										
Aggr size	**Aggregates**					**RAP**	**Filler**	**Combined Grading**	**Specifica-tion**	
	28	**20**	**14**	**10**	**C/DUST**	**F/RA**	**Lime**			
% In Mix	**5.0**	**16.0**	**10.0**	**10.0**	**43.0**	**15.0**	**1.0**	**100**	**Min**	**Max**
37.5	100.	100.0	100.0	100.0	100.0	100.0	100.0	100	100	100
28.0	90.0	100.0	100.0	100.0	100.0	100.0	100.0	100	100	100
20.0	8.0	91.0	100.0	100.0	100.0	100.0	100.0	94	85	95
14.0	2.0	24.0	98.0	100.0	100.0	100.0	100.0	83	71	84
10.0	1.0	2.0	30.0	83.0	100.0	99.0	100.0	71	62	76
7.1	1.0	1.0	3.0	10.6	100.0	98.0	100.0	60	52	68
5.00	1.0	1.0	1.0	2.9	96.0	93.0	100.0	57	42	60
2.00	1.0	1.0	1.0	2.2	61.0	62.0	100.0	37	30	48
1.00	0.5	1.2	1.1	2.2	42.1	41.5	100.0	26	22	38
0.600	0.5	1.3	1.1	2.2	32.7	31.7	100.0	20	16	28
0.300	0.5	1.2	1.1	2.2	24.2	22.8	100.0	15	12	20
0.150	0.5	1.2	1.1	2.1	18.7	15.0	99.0	12	8	15
0.075	0.5	1.1	1.1	2.0	15.2	11.0	90.0	9.6	4	10

(Source: Author, 2019)

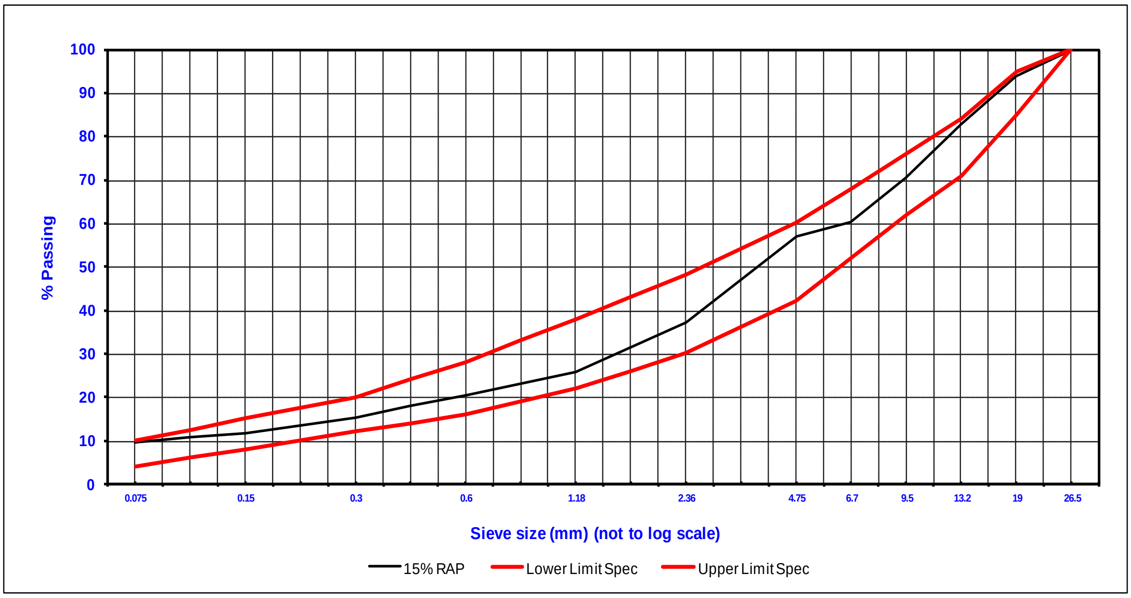

Figure 4.4. Grading Envelope Graph of Virgin Aggregates Combined with 15% RAP (Source: Author, 2019)

The gradation test results of the virgin aggregates mixed with 15% RAP presented in Table 4.2 above show that the grading of the combined aggregate obeys to the South

African National Standard specification. Furthermore, the results plotted in Figure 4.4 lie within the grading envelope.

Table 4.3. Gradation Test Results of Virgin Materials Combined with 30% RAP

Sieve analysis and % passing										
Aggr size	**Aggregates**					**RAP**	**Filler**	**Combined grading**	**Specification**	
	28	**20**	**14**	**10**	**C/DUST**	**F/RA**	**Lime**			
% In Mix	**9.0**	**16.0**	**7.5**	**7.5**	**28.0**	**30.0**	**1.0**	**100**	**Min**	**Max**
37.5	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100	100	100
28.0	90.0	100.0	100.0	100.0	100.0	100.0	100.0	98	100	100
20.0	8.0	91.0	100.0	100.0	100.0	100.0	100.0	89	85	95
14.0	2.0	24.0	98.0	100.0	100.0	100.0	100.0	78	71	84
10.0	1.0	2.0	30.0	83.0	100.0	99.0	100.0	68	62	76
7.1	1.0	1.0	3.0	10.6	100.0	98.0	100.0	60	52	68
5.00	1.0	1.0	1.0	2.9	96.0	93.0	100.0	56	42	60
2.00	1.0	1.0	1.0	2.2	61.0	62.0	100.0	37	30	48
1.00	0.5	1.2	1.1	2.2	42.1	41.5	100.0	26	22	38
0.600	0.5	1.3	1.1	2.2	32.7	31.7	100.0	20	16	28
0.300	0.5	1.2	1.1	2.2	24.2	22.8	100.0	15	12	20
0.150	0.5	1.2	1.1	2.1	18.7	15.0	99.0	11	8	15
0.075	0.5	1.1	1.1	2.0	15.2	11.0	90.0	8.9	4	10

(Source: Author, 2019)

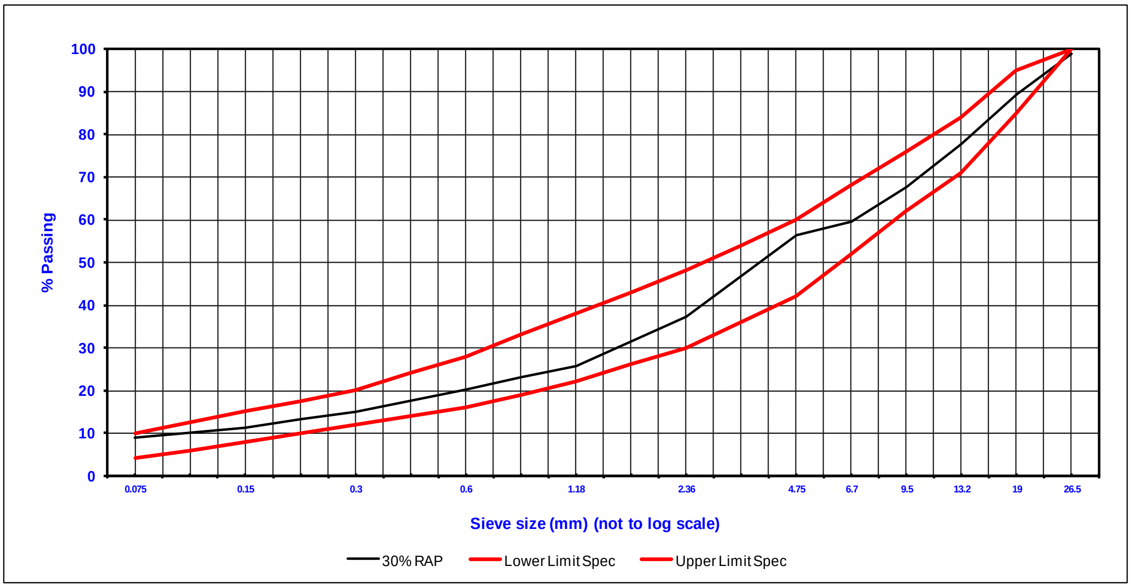

Figure 4.5. Grading Envelope Graph of Virgin Aggregates Combined with 30% RAP (Source: Author, 2019)

The gradation test results of the virgin aggregates mixed with 30% RAP presented in Table 4.3 above show that the grading of the combined aggregates adheres to the South African National Standard specification. Furthermore, the results plotted in Figure 4.5 lie within the grading envelope.

The gradation test results of the virgin aggregates mixed with 15% and 30% RAP presented respectively in Table 4.2 and 4.3 show that the grading results of the combined aggregates obeys to the South African guideline specification. To obey the boundary limit dictated by the guideline specification, the percentages of coarse and fine virgin aggregates as well as the fillers were altered at various percentage increase of reclaimed asphalt. In other words, the percentage adjustment of the virgin aggregates was made whenever the initial percentage of aggregates failed to meet the criteria.

The results of the grading test were used to plot the grading envelop graph. The upper line and the lower line of the grading envelope represent the boundary limit of the guideline specification; the black line lying within the red upper and lower lines represents the combined aggregates gradation results of the coarse aggregates, the fine aggregates, the fillers and the RAP.

The analysis and conclusion drawn from the grading envelopes plotted above are judged to be satisfactory as the results of the combined materials lie quite well between the upper and the lower boundary lines specification.

4.4. Properties of Virgin Aggregates and RAP

Besides the grading tests, the properties of the RAP and the virgin aggregates were determined to assess their suitability in the asphalt mixes. Properties of the RAP determined are the moisture content, the bulk density and the binder content. The characteristics of the virgin aggregates determined are the bulk densities, the water absorption, the sand equivalency, the flakiness index, the percentage of moisture content, the fine aggregate angularity, the polished stone value and the 10% fine aggregate crushing. The overall results obtained are presented in Table 4.4.

Table 4.4. Material Properties

Aggregates										
	Coarse Aggregate			Fine Aggregate		RAP		Fillers	Combined Grading 100%	Specification
Properties	22mm	14mm	10mm	Crusher Dust	Crusher Sand	Coarse	Fine	Filler		Min - Max
Bulk Density	2940.0	2945.0	2950.0	2950.0	2910.0	2943.0	2944.0	2852.0	2935.4	-
ACV	18.60	18.60	18.60	-	-	-	-	-	-	- 25
10% FACT (dry)	319	319	319	-	-	-	-	-	-	160 -
10% FACT (wet)	305	305	305	-	-	-	-	-	-	160 -
PSV	53	53	53	-	-	-	-	-	15.9	49 -
Water Absorption	0.30	0.40	0.60	0.8	1.0	-	-	-	0.51	1.0 - 1.5
Sand Equivalent	-	-	-	52.0	72.0	-	-	-	64.9	50
FAA Method C	-	-	-	37.8	38.4	-	-	-	-	38 - 52
Flakiness Index	11.3	13.1	19.9	-	-	-	-	-	14.8	- 30
Bitumen Content (%)	-	-	-	-	-	2.4	3.5	-	-	-
Moisture Content (%)	0.2	0.2	0.8	1.8	5.3	2.86	5.01	-	-	-

(Source: Author, 2019)

The 10% FACT and the ACV test were performed according to the SANS 3001-AG10. The specification requires that for the aggregates to have good strength and durability, the wet/dry ratio of the 10% Fine Aggregates Crushing Test value (FACT) should be greater than 75%. The results obtained in this study show a wet/dry ratio of 96%, which is an indicator of great strength and durability needed in both the recycled asphalt mix and the virgin asphalt mix. The same conclusion also applies to the Aggregate Crushing Value (ACV) result since both the ACV test and the 10% FACT are used to assess the strength property of aggregates (SANRAL, 2014).

The moisture content in the stockpile of RAP was initially found to be 5.01%, as it can be seen from the results in Table 4.4. Sabita Manual 32 (2011) states that moisture content exceeding 4% in the RAP has the potential to increase the rate of moisture in the finished recycled asphalt mix. Therefore, the excesseive amount of moisture into the asphalt mix may lead to adhesion problems of its compound. The Sabita Manual 32 (2011) adds that the high moisture content in the RAP may result in the decrease rate of production and the increase of fuel energy costs. The reduction of the rate of production and the increase in fuel energy costs will, in turn, increase the gas emissions. Thus, to avoid such problems to arise in the final product of recycled asphalt mix, the RAP was oven-dried to lower the amount of moisture content to about 2%.

The flakiness index and the polished stone value (PSV) were performed to assess the suitability of the coarse fraction aggregates and the fine fraction aggregates into the asphalt mixes in terms of particles shapes and texture. The flakiness index test and the polished stone value (PSV) test conducted following the test methods SANS 3001-AG4 and SANS 3001-AG11respectively, produced results that satisfy the standard specifications (Sabita Manual 35, 2019)

The amount of water susceptible to be absorbed by the coarse aggregates and the fine aggregates was determined while following the prescribed standard methods SANS 3001-AG20 on the coarse aggregates and SANS 3001-AG21 on the fine aggregates. The results obtained from the tests carried out, confirm the suitability of the aggregates into the asphalt mixes as they satisfy the standard specification (Sabita Manual 35, 2019).

A sand equivalency test was performed on fine aggregates to determine the relative proportion of fine dust fraction it contains, while adhering to the SANS 3001-AG5. The results obtained from the test are above the minimum fine fraction prescribed by the standard specification. Consequently, the aggregates are well suited to be utilized in the asphalt mix. Thus, it is expected that the good bonding between the aggregates and the asphalt binder would ensure the durability of the asphalt mixture (Sabita Manual 35, 2019).

The fine aggregates angularity (FAA) test was also conducted on the fine aggregates while following the AASHTO T34 standard test method. The results obtained are satisfactory as it meets the standard specification, which is an indication of less rounded fine particles present in the mix.

4.5. Production of Asphalt Premixes

The production of the control HMA and WMA incorporating RAP premixes aimed to fabricate asphalt specimens for the Marshall Stability and Flow test, ITS test, the Wheel-Tracking test, the Dynamic Modulus tests and the Four-Point Beam-Bending tests were done following SANS 3001-AS1 standard procedure.

4.5.1. Production of WMA – RAP Premixes

To prepare the WMA – RAP specimens, the bitumen and the aggregates were heated at 120°C in the oven overnight. The Sasobit was added into the bitumen, heated at 110°C and manually stirred up until complete dissolution of the Sasobit was achieved.

The aggregates and the modified asphalt bitumen were then blended into a mixer machine set at 120°C and continually mixed until the perfect coating of the bitumen binder over the aggregates was observed.

Once the blending phase was completed, the recycled asphalt mix was poured on a designated surface to allow the premix to cool down overnight, followed by the riffling of the premix via a riffling splitter and a weight machine. The purpose of riffling the asphalt premixes is to allow uniform distribution of different aggregate sizes which will guaranty accurate determination of asphalt premixes' volumetric properties and quality compaction of asphalt specimens.

4.5.2. Production of HMA Premix

The production of HMA premix was performed using standard procedures as of the WMA – RAP described above. However, the blending of the HMA was done at 150°C.

4.6. Volumetric Property of the Asphalt Mixes

HMA and the WMA-RAP premix of 1500g and 2440g were measured to test and determine the binder content (BC), the maximum void-less density, the percentage of void in the asphalt premix and the bulk density respectively. Thus, the tests were performed in respect with the South African National Standard (SANS 3001) procedures.

Further volumetric properties, which include the theoretical density of the mix, the bulk density of the aggregates (BD_A), air voids percentage (V_v), volume percentage of the bitumen (V_B), the volume of the aggregates (V_A), voids in mineral aggregates (VMA) and the percentage of voids filled with bitumen (VFB), were determined and are reported in Table 4.5.

Table 4.5. Characteristics of Asphalt Premixes

Premix Properties											
Test Methods	**SANS 3001 AS20**	**SANS 3001 AS11**	**SANS 3001 AG1**	**SANS 3001 AS11**	**SANS 3001 AS11**	**SANS 3001 AS11**	**SANS 3001 AS11**	**SANS 3001 AS11**	**SAS 3001 AS11**	**SANS 3001 AS11**	**SANS 3001 AS10**
Mixes	**%BC**	**Rice**	**BD_A**	**%VIM**	**%VMA**	**V_A**	**V_B**	**V_{DA}**	**V_{BEF}**	**%VFB**	**BD**
HMA	4.6	2751.6	2938	4.0	17.2	342.9	44.09	382.6	11.5	66.9	2543
WMA-15% RAP	4.6	2732.8	2938	4.1	14.3	328.9	42.3	369.5	6.4	44.8	2631.2
WMA-30% RAP	4.7	2723	2938	5.3	16.1	319.6	42.1	360.7	10.6	65.8	2579.3

(Source: Author, 2019)

At 4.5% bitumen, the void in the asphalt mix was found to be 4%, 4.1% and 5.3% for HMA. WMA – 15% RAP and WMA – 30% RAP, respectively. Sabita Manual 32 / TRH 8 (2011) recommend that if a percentage of 4.5% bitumen binder in the asphalt mix is used, the void in asphalt mix determined should not be lower than 4% and higher than 6%. As such, the HMA and WMA – RAP mixes fall within the asphalt design guideline specification.

Furthermore, the voids in the mineral aggregates, as well as the voids filled with mineral aggregates, are reported to meet the requirements specified in the SANS 3001 AS11 as the voids in the mineral aggregates are superior to 13% and the voids filled with mineral aggregates fall between the minimum 65% and the maximum 75%.

4.7. Summary of the Chapter

This chapter discussed the selection of materials and the bitumen binder used in the production of asphalt premixes. It discussed the tests recommended for the characterization of virgin aggregates, the RAP and the bitumen binder. It explained the procedures used in the production of HMA premixes and the WMA – RAP premixes. Finally, this chapter discussed the volume properties of the HMA premixes and the WMA – RAP premixes.

The next chapter discusses the steps used in the production and the prepartaion of the HMA, the WMA – 30% RAP and the WMA – 30% RAP specimens. It further discusses the tests performed on the asphalt specimens.

CHAPTER 5 – PERFORMANCE TESTS OF ASPHALT MIXES

5.1. Introduction

This chapter focuses on evaluating the rutting performance, the fatigue cracking performance, the Marshall performances (the stiffness, stability and flow) and the dynamic modulus of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP. It also focuses on comparing the performance results of the WMA – 15% RAP and the WMA – 30% RAP against the performance results of the control HMA.
The evaluation of the distress performances, the Marshall performance and the dynamic modulus were achieved through the Marshall Stability and Flow Test, the Indirect Tensile Strength Test (ITS), the Hamburg Wheel-Tracking Test, the Four-Point Beam-Bending Test and the Dynamic Modulus Test. The procedures followed to prepare the specimens before testing and to perform the tests are gorverned by standard and specification set by national agencies such as the technical recommendation for highway (TRH), the South African bitumen association (SABITA), the American society for testing and materials (ASTDM) and the technical methods for highways (TMH).

5.2. Marshall Stability and Flow Test

The Marshall Stability and Flow test were performed on the control HMA, the WMA – 15% RAP as well as the WMA – 30% RAP to assess its asphalt content and its capability to carry and to spread the loads across the pavement structure.

5.2.1. Production, Preparation of Specimens and Test Procedure

The sample moulds were heated to about 100°C as per SANS recommendation before filling them with the control HMA, WMA – 15% RAP as well as the WMA – 30% RAP pre-mixes. The SANS recommendation to pre-heat the mould at an approximate temperature to the asphalt mix before pouring any hot or warm asphalt mixture into, is to eliminate any adverse effects on the mixing temperature of the asphalt mixture.

The compaction of specimens at 140°C for the control HMA and 110°C for the WMA – 15% RAP and for WMA – 30% RAP were achieved using a Marshall Drop Hammer weighting 4536g, dropped at the constant height of 457.2mm on each face of the specimens. The specimens were subjected to a total number of 75 blows to account for the designed traffic load.

A solid steel device was then used to extract the compacted asphalt specimens from the mould. The extracted briquettes were left to cool off for 17 hours, then submerged into a water bath heated at a constant temperature of 60°C during 30 minutes and immediately subjected to stability and flow tests in the compression testing machine.

The briquettes inserted into the breaking head assembly of the compression testing machine were loaded at a steady rate of 50mm/min until the maximum force causing failure of the specimen was reached. The stability data were recorded to the nearest 50N as well as the flow deformation. Figure 5.1 below represents the control HMA specimen, WMA – 15% RAP and WMA – 30% RAP specimens prepared and used to perform the Marshal Stability and Flow test as well as the ITS test.

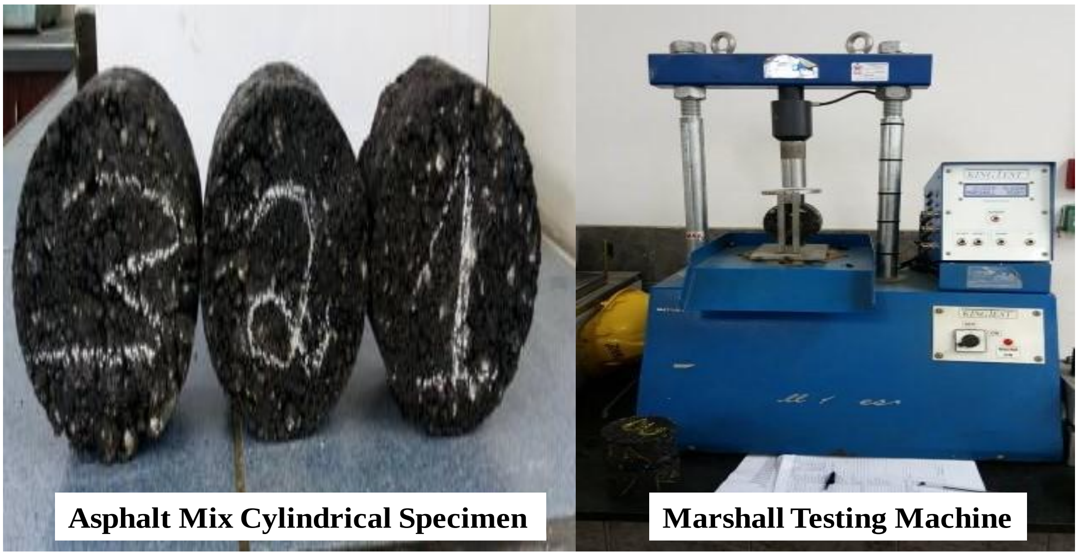

Figure 5. 1. Asphalt Cylindrical Specimen Mounted on the Marshall Stability and Flow Testing Machine (Source: Author, 2019)

5.2.2. Data Collection

The data collected before subjecting the asphalt briquette specimen to the Marshal Stability and Flow tests are presented in Table 5.1. The bulk densities and the dimensions were determined in accordance with SANS 3001-AS10 to ensure that the asphalt specimens fabricated meet the requirements for the Marshall Stability and Flow test.

Table 5.1. Data Collection on Asphalt Specimens

Description	HMA		WMA – 15% RAP		WMA – 30% RAP	
No of Briquettes	1	2	1	2	1	2
Thickness of Briquettes (mm)	66.4	66.5	66.3	65	65	65.6
Diameter of Briquettes (mm)	101.5	101.5	101.5	101.5	101.5	101.5
Bulk Densities of Briquettes (Kg / m^3)	2458	2464	2448	2443	2427	2456
Testing Temperature (°C)	60	60	60	60	60	60
Compaction Temperature (°C)	140	140	110	110	110	110
Stability Reading (KN)	9.62	10.4	10.2	10.3	11.5	11.4
Flow Reading (mm)	3.9	4.0	4.4	4.3	4.5	4.4

(Source: Author, 2019)

The stability values and the flow values obtained were used to calculate the average values of the asphalt mixes and further the Marshall Quotient (MQ). The Marshall Quotient of the asphalt mix was determined through the division of the stability values by the flow values. Those average values and the Marshall Quotient determined are reported in Table 5.2.

5.2.3. Results and Discussion

The results in Table 5.2 represent the results of the stability and flow tests and the calculated Marshall Quotient. Figure 5.2, on the other hand, represents the plot of Marshal Quotient for the control HMA, the WMA – 15% RAP and WMA – 30% RAP.
The guideline from the Sabita Manual 32 / TRH 8 (2011) indicates that the stability of asphalt mixes should not be lower than 8 KN and higher than 18 KN. Thus, results of the stability test obtained and presented in Table 5.2 below show that the control HMA, WMA – 15% RAP as well as the WMA – 30% RAP are satisfactory as they all fall within the interval standard specification.
The guideline Sabita Manual further recommends that the flow of asphalt mixtures should lag between 2mm and 6mm. Thus, the results of the flow test obtained and

presented Table 5.2 reveals that the controls HMA, WMA – 15% RAP as well as the WMA – 30% RAP conform to the standard specification. In other words, this signifies that the binder in the control HMA, WMA – 15% RAP as well as the WMA – 30% RAP is at the desired content.

The Marshall Quotient (MQ) of the asphalt was determined through the division of the stability by the flow, and then plotted, as shown in Figure 5.2. The Marshall Quotient (MQ) values of the control HMA were found to be 8% lower than that the WMA – 15% RAP's MQ and 4% lower than the WMA – 30% RAP's MQ. These percentage differences are noticed to be less than 10%, which infers that the WMA – RAP possess close capability to carry and to spread loads across the pavement layer to that of the control HMA.

Table 5.2. Marshall Stability & Flow Test Results

Description	HMA		WMA – 15%RAP		WMA – 30%RAP	
Specimen No	1	2	1	2	1	2
Stability (KN)	9.62	10.4	10.2	10.3	11.5	11.4
Ave. Stability (KN)	10.0		10.3		11	
Flow (mm)	3.9	4.0	4.4	4.3	4.5	4.4
Ave. Flow (mm)	4.0		4.4		4.5	
Marshall Quotient (KN/mm)	2.5		2.3		2.4	

(Source: Author, 2019)

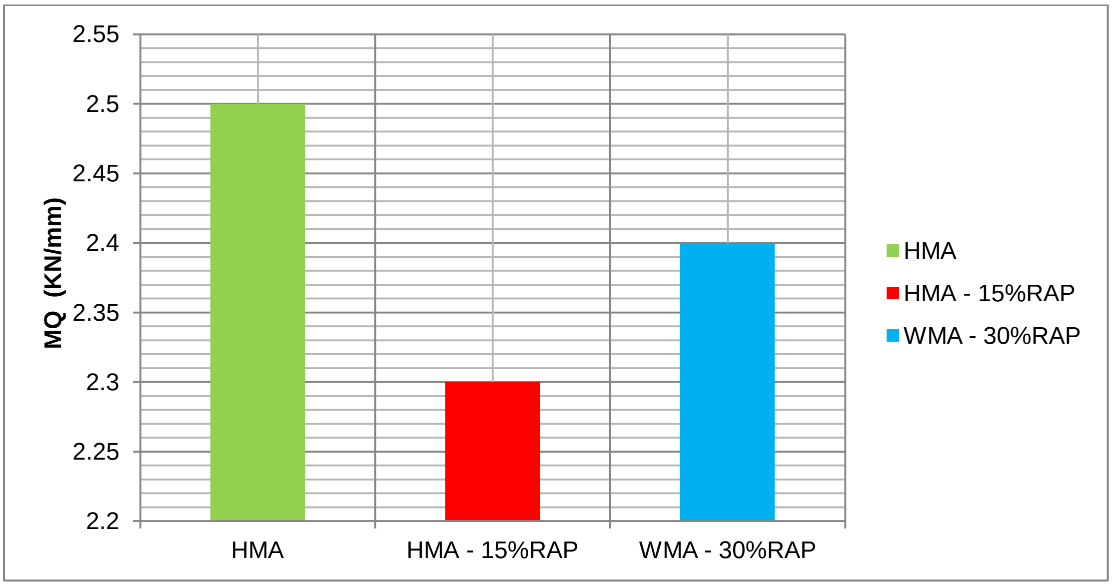

Figure 5.2. Marshall Quotient of Mixtures (Source: Author, 2019)

5.3. Indirect Tensile Strength Test

The Indirect Tensile Strength **(**ITS) is generally known to be one of the most popular asphalt performance tests, which offers a reliable indication of cracking potential on asphalt mixtures. Furthermore, asphalt mixes with elevated stiffness values simply indicate its ability to carry and spread loads to the underlying layers that compose an asphalt pavement structure.

The ITS test was performed on the control HMA, WMA – 15% RAP and WMA – 30% RAP to determine their stiffness values. It should be recalled that asphalt mixes with good stiffness values are generally required for asphalt layers, hence the necessity to carry out ITS tests on asphalt specimens considered in this study.

5.3.1. Production, Preparation of Specimens and Test Procedure

The preparation of specimens to perform the ITS test was done following the same procedure as of the Marshall Flow and Stability. However, in the case of the ITS test, four briquettes per asphalt mix were kept in the oven at 25°C during two hours before subjecting them to ITS testing in the Marshall Compression testing machine, as shown in Figure 5.3. The ITS test on asphalt specimens was carried out in accordance with the

American Standard Test Method ASTM D6931 – 07. Tables 5.3, 5.4 and 5.5 show the data recorded before and after completing the ITS tests.

Figure 5.3. Asphalt Mix Specimen Configured in the ITS Test Machine

5.3.2. Data Collection

The data collected before and after subjecting the asphalt briquette specimens to the ITS tests are presented in Tables 5.3, 5.4 and 5.5. The bulk densities and the dimensions were determined in accordance with SANS 3001-AS10 to ensure that the asphalt specimens fabricated meet the requirement for the ITS test.

Table 5.3. HMA Data Collection

Description	HMA			
No of Briquettes	1	2	3	4
Thickness of Briquettes (mm)	63.5	63.0	62.4	63.1
Diameter of Briquettes (mm)	101.5	101.5	101.5	101.5
Bulk Densities of Briquettes (kg/m^3)	2455	2451	2453	2454
Testing Temperature (°C)	25	25	25	25
Reading (KN)	14.3	13.3	13.4	12.7
Δ Thickness (mm)	2.8	2.0	2.0	2.14

(Source: Author, 2019)

Table 5.4. WMA – 15% RAP Data Collection

Description	WMA – 15% RAP			
No of Briquettes	1	2	3	4
Thickness of Briquettes (mm)	63.1	62.9	63.0	63.2
Diameter of Briquettes (mm)	101.5	101.5	101.5	101.5
Bulk Densities of Briquettes (kg/m^3)	2398	2432	2450	2420
Testing Temperature (°C)	25	25	25	25
Reading (KN)	16.5	15.4	17.3	16.4
Δ Thickness (mm)	1.5	1.8	1.7	1.8

(Source: Author, 2019)

Table 5.5. WMA – 30%RAP Data Collection

Description	WMA – 30% RAP			
No of Briquettes	1	2	3	4
Thickness of Briquettes (mm)	63.2	63.0	64.7	62.8
Diameter of Briquettes (mm)	101.5	101.5	101.5	101.5
Bulk Densities of Briquettes (kg/m^3)	2417	2428	2420	2427
Testing Temperature (°C)	25	25	25	25
Reading (KN)	16.7	17.55	13.59	15.91
Δ Thickness (mm)	2.74	1.98	1.68	1.98

(Source: Author, 2019)

The data collected from Tables 5.3, 5.4 and 5.5 in term of dimensions and loads causing failure of the asphalt specimens were used in Equation 5.1 to determine the ITS values of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP presented in Table 5.6. The ITS results of the control HMA, WMA – 15% RAP, WMA – 30% RAP shown in Table 5.6 were calculated based on the following equation:

$$ITS = \frac{2P}{\pi.L.\emptyset}$$ **Equation 5.1**

ITS = Indirect Tensile Strength (KPa)

P = Load causing failure (KN)

L = Thickness of the specimen (mm)

Ø = Diameter of the specimen (mm)

5.3.3. Results and Discussion

Sabita Manual 32 / TRH 8 (2011) specifies that the asphalt mix should be above 1000 KPa to meet the requirement in term of indirect tensile strength. As such, the average ITS results determined in Table 5.6 below were found to be satisfactory as per the design guideline specification.

Figure 5.4 shows the ITS values for the WMA – 15 RAP and the WMA – 30 RAP higher than the control HMA. However, the ITS of WMA – RAP is reduced with an increased amount of RAP in the WMA. Thus, the higher ITS value of WMA – RAP compared to the control HMA signifies a better tensile strength of that WMA – RAP that eventually results in its acceptable capability to resist cracking failure. The finding of this study agrees with the one made by Oner and Sengoz (2015), who found the ITS of the WMA – RAP relatively higher than the control HMA. The authors further emphasized that the high ITS performance of WMA – RAP is due both to the presence of RAP and to the WMA organic additive.

Table 5.6. Indirect Tensile Strength Results

Type of Mix	ITS (KPa)				Ave. ITS (KPa)
HMA	1412	1324.11	1346.89	1262.37	1336.34
WMA – 15% RAP	1642.7	1535.62	1722.34	1627.57	1632.05
WMA – 30% RAP	1657.35	1747.23	1317.44	1589.0	1577.75

(Source: Author, 2019)

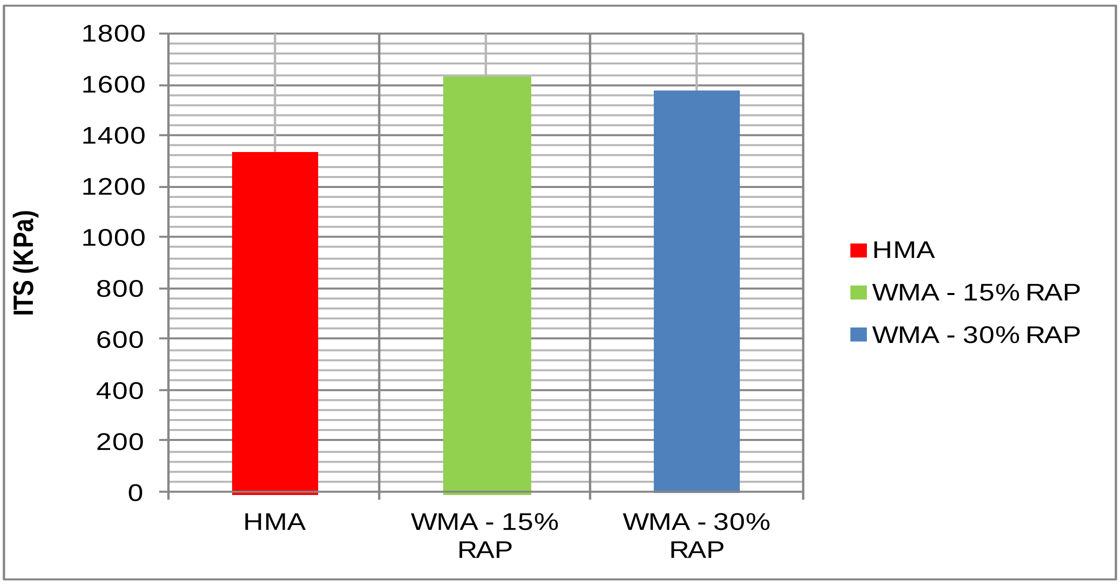

Figure 5.4. ITS Ranking of HMA and WMA-RAP (Source: Author, 2019)

5.4. Hamburg Wheel-Tracking Test

The Hamburg Wheel-Tracking test was performed on the control HMA, the WMA – 15% RAP and the WMA – 30% RAP to evaluate their resistance against premature rutting failure and aggregates stripping. It should be recalled that these defects may originate from an unwanted aggregate structure in the mix, inadequate binder adhesion between the asphalt binder and the aggregates as a result of moisture damage.

The French LCPC (Laboratoire Central des Ponts et Chaussees) Rutting Tester, the GLWT (Georgia Loaded-Wheel Tester), commonly known as the Asphalt Pavement Analyser, and the HWTD (Hamburg Wheel-Tracking Device) are the three types of Wheel-Track Tester machines that are currently available in the civil engineering market. Although these rutting and stripping asphalt performance testers are conceived to perform the same functions, they differ in terms of design, load configuration and test conditions. Furthermore, the passing or fail criterion of asphalt mixture is different for each of these machines (Westrack Forensic Team Consensus Report, 2001).

The investigation led by the Westrack Forensic Team Consensus Report (2001) found that the LCPC and GLWT have different wheel load configurations and dimensions. The design of the LCPC machine does not accommodate asphalt specimens that are fabricated with coarse aggregates larger than 20mm (this is because weak asphalt mixture tends to have better rutting performance than expected, therefore leading to difficulties in discriminating the performance between the good and poor asphalt mixture). The GLWT, on the other hand, can accommodate asphalt specimens with aggregates larger than 20mm, hence its preference in this study.

The ability of the GLWT to perform tests on asphalt mix specimens in dry and wet conditions makes its testing ability better than the LCPC and the HWTD. Furthermore, the HWTD uses steel instead of pneumatic wheels. The steel wheel is known to increase the severity of the rutting and the stripping of the asphalt mix specimen. The load applied by the steel wheel upon the asphalt mix specimen is much higher than the load the asphalt pavement in service would be subjected to during the field loading. As such, the asphalt mixture that survives the test severity of the HWTD has a higher chance to exhibit good rutting and stripping performance during its service in the field. Westrack Forensic Team Consensus Report (2001) further adds that the asphalt specimen that fails the test through the HWTD may result in the rejection of an acceptable asphalt mixture.

5.4.1. Production, Preparation of the Specimens and Test Procedure

The preparation of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP specimens and their testings were done in respect of the standard EN 12697 – 22. The asphalt premixes were loaded in two high-density polyethylene moulds and compacted at 140°C for the control HMA and 110°C for the WMA – 15% RAP and for the WMA – 30% RAP to achieve an air void target that falls between 4% and 6%. The premixes were compacted in the Superpave gyratory compactor machine to 50 gyrations.

The cylindrical specimen measuring 150 mm in diameter and 60 mm in thickness were extracted out the mould and left to stand at room temperature for 24 hours. Thereafter, the prepared cylindrical specimens were placed in the cutting template mould, then inserted into a sawing machine and cut along the flat edge of the mould.

The high-density polyethylene moulds containing the sawed specimens were mounted into the trays and fitted adequately into each one. The mounting trays were fastened into the water bath heated at a monitored temperature of 50°C. Figure 5.5 shown below is the representation of the asphalt mixture specimen mounted in the Hamburg Wheel-Tracking machine.

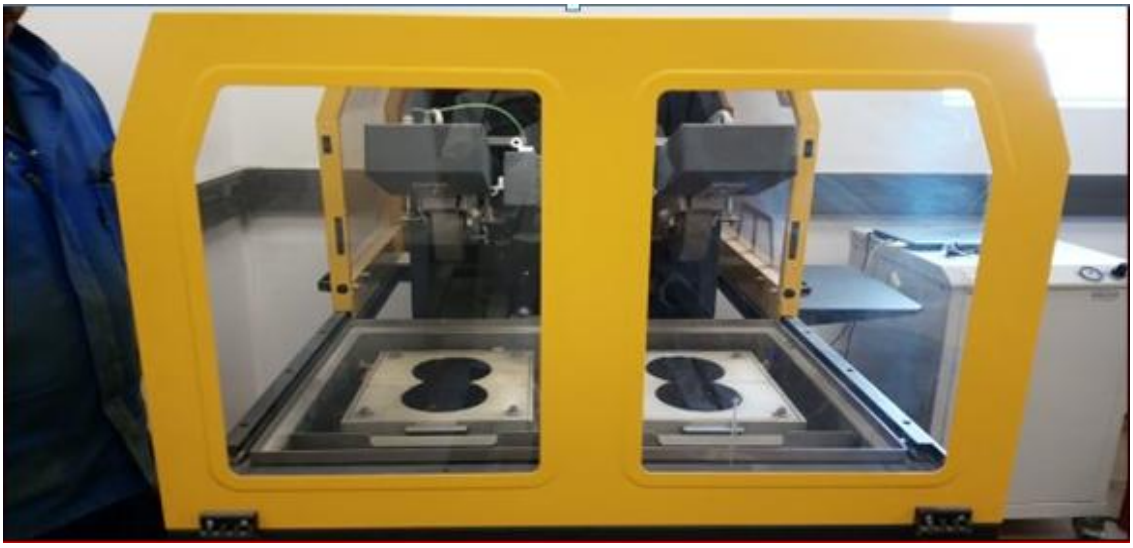

Figure 5.5. Hamburg Wheel-Tracking Machine Containing Asphalt Specimens (Source: Author, 2019)

5.4.2. Data Collection

Table 5.7 presented below contains the collected asphalt mix information used as input data on the software supplied with the Hamburg Wheel-Tracking machine before running the test. The data collected and presented in Table 5.7 includes the diameter, the thickness, the type of compaction, the bulk density and the void in the mix.

The information entered was verified, and the test was allowed to start and automatically stopped once the machine has reached the desired maximum number of cycle, which in this study was set to be 1000 load cycles.

Table 5.7. Hamburg Wheel Tracking Test Input Data

Mixture	HMA	WMA – 15% RAP	WMA – 30%RAP
Diameter (mm)	150	150	150
Thickness (mm)	60	60	60
Max Density (kg/m³)	2512	2512	2490
Void in the Mix (%)	4.0	4.1	4.8
Compaction Type	Superpave Gyratory	Superpave Gyratory	Superpave Gyratory
Water Temperature (°C)	50	50	50
Max Load Cycle	1000	1000	1000
Wheel Speed (Cycle/min)	26	26	26

(Source: Author, 2019)

5.4.3. Results and Discussion

Table 5.8 summarizes the results of the Hamburg Wheel-Tracking test performed on the control HMA, the WMA – 15% RAP and the WMA – 30% RAP until complete failure has occurred. Figure 5.6 shows typical rut depths versus loads cycle plot when the load reaches the maximum of 1000 load passes.

Highways agencies have different recommendation about the ideal asphalt mix rutting performance depth. On the one hand, the Colorado DOT recommends a maximum rut depth of 4mm at 10,000 load cycles and 10mm at 20,000 load cycles. In the other hand, the German recommends a maximum rut depth of 4 mm at 19,200 load cycles (FHWA, 2003). These recommendations are concerned with rutting performance of asphalt mix typically designed with coarse aggregates not larger than 20mm. They, therefore, cannot be used as a benchmark to assess the good performance of the control HMA, WMA – 15% RAP and WMA – 30% RAP which are designed with 28mm coarse aggregates. Evaluating the rutting performance of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP in this research would, therefore, not consider the standard recommendations. Instead, the results obtained would be compared with previous findings made by researchers who conducted close or similar studies. Figure 5.6 presents the typical rut depth of the WMA – RAP and the control HMA when the load cycles reach 1000 passes.

The observation made from Figure 5.6 shows the rut depths of the control HMA higher than the WMA – 15 %RAP and WMA – 30 %RAP. This signifies a better rutting resistance of the WMA – RAP against the control HMA when the load cycles gradually increase. Past experimental studies conducted on the WMA have shown that the WMA usually has lower rutting performance than the traditional HMA. However, the finding made in this study is an experimental evidence which proves that the rutting performance of the WMA can be improved and may even perform better than the HMA if the RAP is added into the WMA and the organic Sasobit additive is used.

The findings made in this studies as far as the rutting performance is concerned substantiate the solution proposed by Hill (2011) and Oner et al. (2017). These authors suggested the use of certain WMA additives and the RAP can contribute on improving the rutting performance of WMA.

Table 5.8. Hamburg Wheel-Tracking Test Results

Specimen Type	HMA	WMA – 15%RAP	WMA – 30%RAP
Rut Depth at 1000 Passes (mm)	4.317	2.704	2.386
Rut Depth at 2000 Passes (mm)	5.232	3.886	3.257
Rut Depth at 3000 Passes (mm)	6.601	4.902	3.987
Rut Depth at 4000 Passes (mm)	7.501	5.637	4.639
Rut Depth at 5000 Passes (mm)	8.880	6.908	5.728
Rut Depth at Failure (mm)	8.880	7.757	6.344
Number of Passes at Failure	6000	5406	5406
Temperature	50°C	50°C	50°C

(Source: Author, 2019)

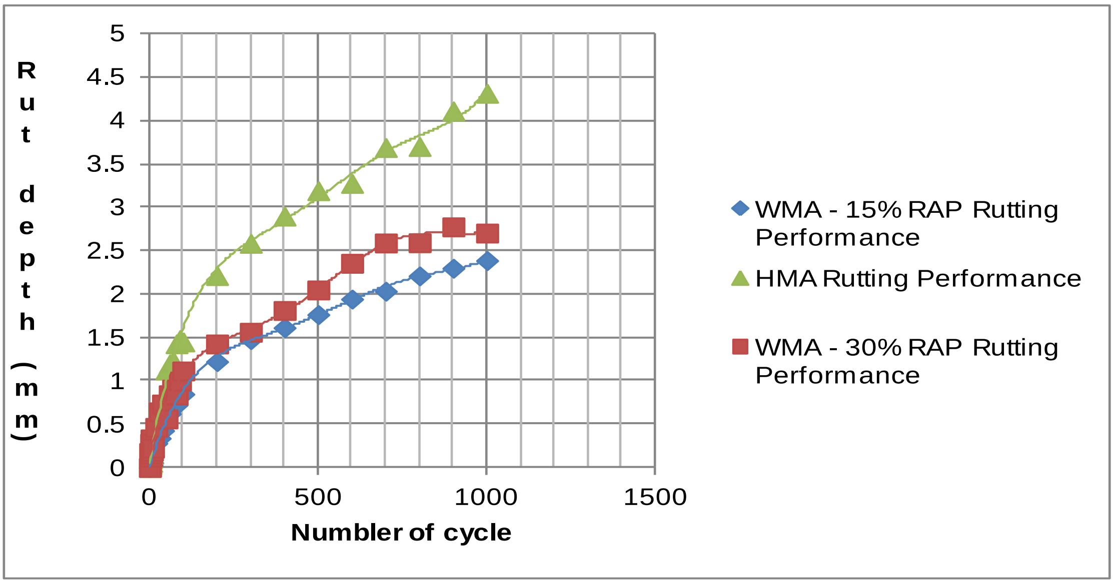

Figure 5.6. Typical Rutting Performance of HMA and WMA – RAP (Source: Author, 2019)

Figure 5.7, shown below, represents the specimen of the asphalt mixture after completion of the Hamburg Wheel-Tracking test.

Figure 5.7. Asphalt Mixture Specimens Subjected to Hamburg Wheel-Tracking Test (Source: Author, 2019)

5.5. Dynamic Modulus Test

The Dynamic Modulus test was performed to measure the stiffness of the WMA – 15% RAP, the WMA – 30% RAP and the control HMA. The results of this Dynamic Modulus test were used as an essential input data in the numerical modelling software program studied in the following chapter.

5.5.1. Production, Preparation of Specimens and Test Procedure

The preparation and the test of the WMA – 15% RAP, the WMA – 30% RAP and the control HMA aimed to measure their amount of stiffness modulus were done in respect of the AASHTO TP 79 standard (AASHTO, 2013). Each cylindrical asphalt mix specimen measuring 150mm diameter and 170 mm high was compacted using the Superpave gyratory compactor machine to achieve a target air void between 4% and 6% with an allowable deviation of ±1%. The cylindrical asphalt specimen was compacted at 140°C for the control HMA and 110°C for the WMA – 15% RAP and for the WMA – 30% RAP.

The compacted cylindrical asphalt mix specimens measuring 150mm in diameter and 170 mm in height were cut and cored with a wet sawing machine. The cutting and sawing of the cylindrical asphalt mix were to obtain final specimens measuring 100mm in diameter and 150mm in height and to achieve in-place voids. A unique specimen per asphalt mixture was fabricated and left to dry at 20°C in the oven for 24 hours. Thereafter, the cylindrical asphalt mix specimens were mounted in the Asphalt Mixture Performance Tester (AMPT) to evaluate its dynamic modulus

Figure 5.8, shown below, represents the cut and sawn asphalt mix specimen with gauge points inserted into it. The steel gauge serves to measure the dynamic modulus of the specimen. Figure 5.9, on the other hand, represents the asphalt cylindrical specimen mounted in the Asphalt Mixture Performance Tester (AMPT).

Figure 5.8. Asphalt Cylindrical Specimens (Source: Author, 2019)

Figure 5.9. AMPT Machine Containing a Cylindrical Asphalt Specimen (Source: Author, 2019)

The dynamic modulus |E| of an asphalt mix can be defined as the ratio of the stress amplitude (σ) to the strain amplitude (ε) that results in a steady-state response at a given frequency (You, Adhikari & Kutay, 2009). As such, the dynamic modulus |E| of the WMA – 15% RAP, the WMA – 30% RAP and the control HMA specimens were measured by subjecting them to compressive stress at the temperature of 20°C and at varied range of loading frequencies, which include 0.1 Hz, 0.5 Hz, 1 Hz, 5 Hz, 10 Hz, and 25 Hz.

5.5.2. Data Collection

Table 5.9 presented below contains the data collected on the asphalt mix specimens. Those data were used as input data on the software supplied with the AMPT. The input data used to run the test includes the the bulk density of the cylindrical specimen as well as its air void. The standard specification recommends the void in the asphalt mix to be between 4% and 6% with an allowable deviation of ±1.5% (Sabita Manual, 2014). Thus, the air void in the asphalt mixes was found to meet the requirements imposed for the dynamic modulus test.

Table 5.9. Dynamic Modulus Test Input Data

Mixture	HMA	WMA – 15%RAP	WMA – 30%RAP
Number of Specimens	1	1	1
Thickness (mm)	170	170	169
Type of compaction	Gyratory	Gyratory	Gyratory
Dry Mass in Air (g)	6809.3	7057.1	6802.3
Surface Dry Mass in Air (g)	6884.5	7103.7	6909.9
Mass in Water (g)	4009.7	4202.1	4020.3
Bulk Density of Mix Specimen (kg/m³)	2361.7	2425.1	2347.2
Max Theoritical Density (Kg/m³)	2543.6	2543.6	2543.6
Void in Mix Specimen	7.1	4.7	7.7

(Source: Author, 2019)

5.5.3. Results and Discussion

Figures 5.10, 5.11 and 5.12 typically show the plot of the applied axial stress and strain response versus the phase angle at 34.13° for the WMA – 15% RAP, the WMA – 30%

RAP and the control HMA respectively. Table 5.10, 5.11 and 5.12, on the other hand, represent the summary of the results released by the AMPT computer software machine when the test is completed.

From each table, it is observed that the dynamic modulus of each asphalt technology increases as the loading frequency increases. Furthermore, the dynamic modulus of the control HMA appears to be lower than the dynamic modulus of the WMA – 15% RAP and higher than the dynamic modulus of the WMA – 30 % RAP.

As such, one can imply from the test results that the increase of RAP in the WMA negatively affects the WMA as it reduces its dynamic modulus property. Overall, the substantial drop in dynamic modulus of the control WMA – 30% RAP compared to the WMA – 15% RAP is owned to the increased RAP percentage in the WMA. This is substantiated through the finding made by Carvajal-Muñoz et al. (2015). Carvajal - Muñoz and his co–authors found in their study that the type of asphalt mixture, the temperature, the loading history, the type of loading, the loading frequency and the ageing could potentially affect the dynamic modulus of the asphalt mix.

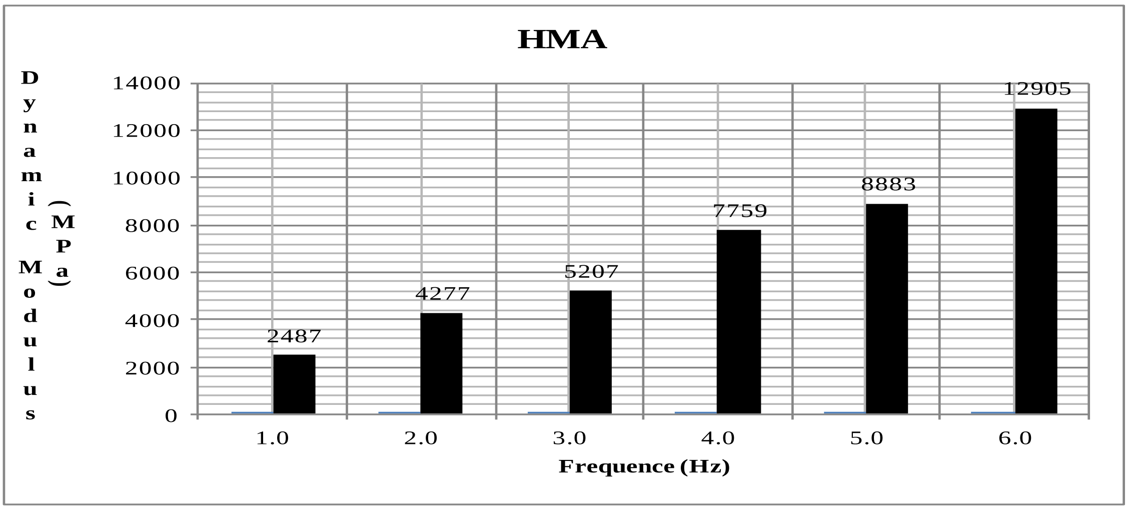

Figure 5.10. Dynamic Modulus of the HMA – 100% virgin aggregates (Source: Author, 2019)

Table 5.10. Tabulated Results Summary for the Control HMA

Frequency (Hz)	25	10	5	1	0.5	0.1
Dynamic Modulus (MPa)	12905	8883	7759	5207	4277	2487
Phase Angle (°)	26.75	19.47	22.04	28.33	30.49	35.79
Average Temperature (°C)	20	20	20	20	20	20

(Source: Author, 2019)

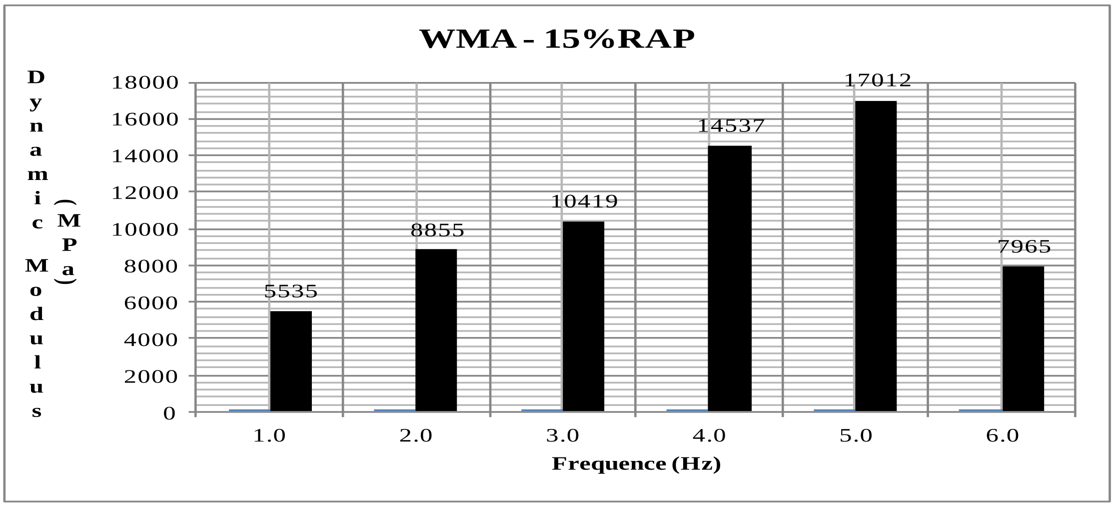

Figure 5.11. Dynamic Modulus of the WMA – 15% RAP (Source: Author, 2019)

Table 5.11. Tabulated Results Summary for the WMA – 15% RAP

Frequency (Hz)	25	10	5	1	0.5	0.1
Dynamic Modulus (MPa)	7965	17012	14537	10419	8855	5535
Phase Angle (°)	8.26	15.68	11.58	33.43	36.05	34.13
Average Temperature (°C)	20.0	20.0	20.0	20.0	20.0	20.0

(Source: Author, 2019)

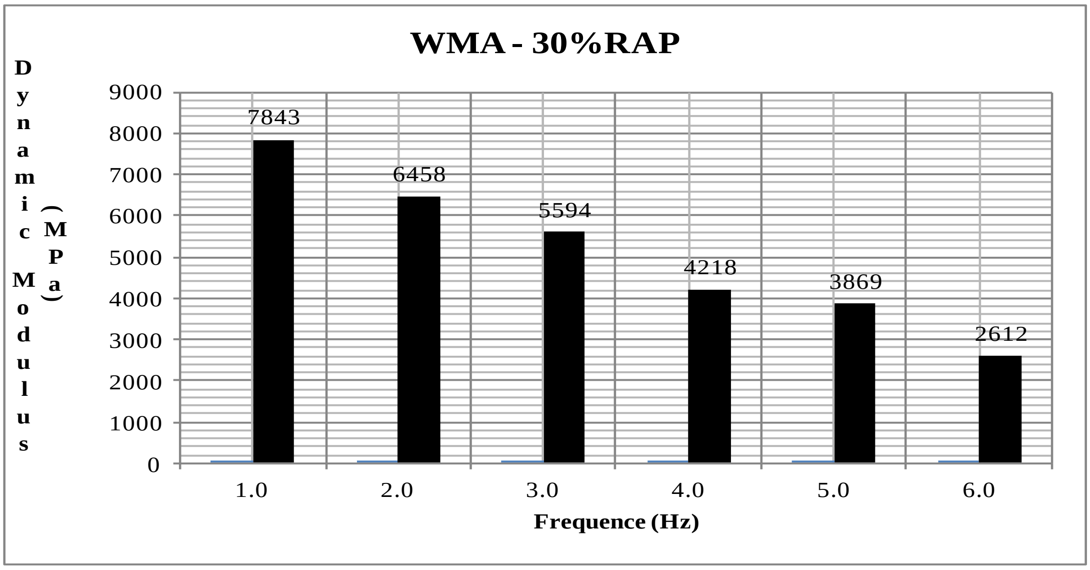

Figure 5.12. Dynamic Modulus of the WMA – 30% RAP (Source: Author, 2019)

Table 5.12. Tabulated Results Summary for the WMA – 30% RAP

Frequency (Hz)	25	10	5	1	0.5	0.1
Dynamic Modulus (MPa)	7843	6458	5594	4218	3869	2612
Phase Angle (°)	14.61	19.02	22.25	26.36	27.72	32.33
Average Temperature (°C)	20.0	20.0	20.0	20.0	20.0	20.0

(Source: Author, 2019)

5.6. Four-Point Beam Bending Test

The Four-Point Beam Bending Test was performed to evaluate through laboratory simulation the ability of the WMA – 15% RAP and the WMA – 30% RAP to resist repeated tensile strains without fracture or cracking and further compared their fatigue cracking performance against one of the control HMAs.

It should be recalled that the fatigue cracking failure of pavement in service occurs because of repeated numbers of applied loads exceeding the ability of the asphalt mix to withstand its associated tensile strains (Sabita Manual, 2014). As such, the Four-Point Beam Bending Test has been utilized all over the world to establish the relations of fatigue design life for various types of asphalt pavements.

The Four-Point Beam Bending Test uses different ranges of temperatures which are 0°C, 5°C, 10°C, 15°C, 20°C, 25°C, and 30°C, with 20°C being the most common temperature used. Furthermore, the frequency applied would be either 10Hz or 30Hz. However, the most commonly utilized testing condition is 30Hz (Herkens & Kalf, 2000). In the case of this study, the temperature of 10°C and an applied frequency of 10Hz were used as testing conditions instead of 20°C and an applied frequency of 30Hz known to be the typical test temperature and applied load frequency.

The Four-Point Beam Bending Test was conducted following the standard methods of testing and determining fatigue cracking of the compacted asphalt mix subjected to repeated flexural bending. Two beams per asphalt mixture were prepared to achieve a target air void between 4% and 6% with an allowable deviation of ±1.5%.

5.6.1. Production, Preparation of Specimens and Test Procedure

The preparation and testing of the WMA – 15% RAP, the WMA – 30% RAP and the control HMA aimed to measure their strength against repeated flexural bending force was done by compacting the asphalt pre-mixes through a slab compactor machine.The slabs was compacted at 140°C for the control HMA and 110°C for the WMA – 15% RAP and for WMA – 30% RAP.

The compacted slabs specimen measuring 300mm wide, 400mm long and 70mm thick, as presented in Figure 5.13, was left to stand at room temperature for 24 hours. The slabs were cut and sawn with a wet sawing machine to obtain two slabs measuring 65mm wide, 400mm long and 50mm thick as presented in Figure 5.14.

Figure 5.13. Compacted Asphalt Mix Slab (Source: Author, 2019)

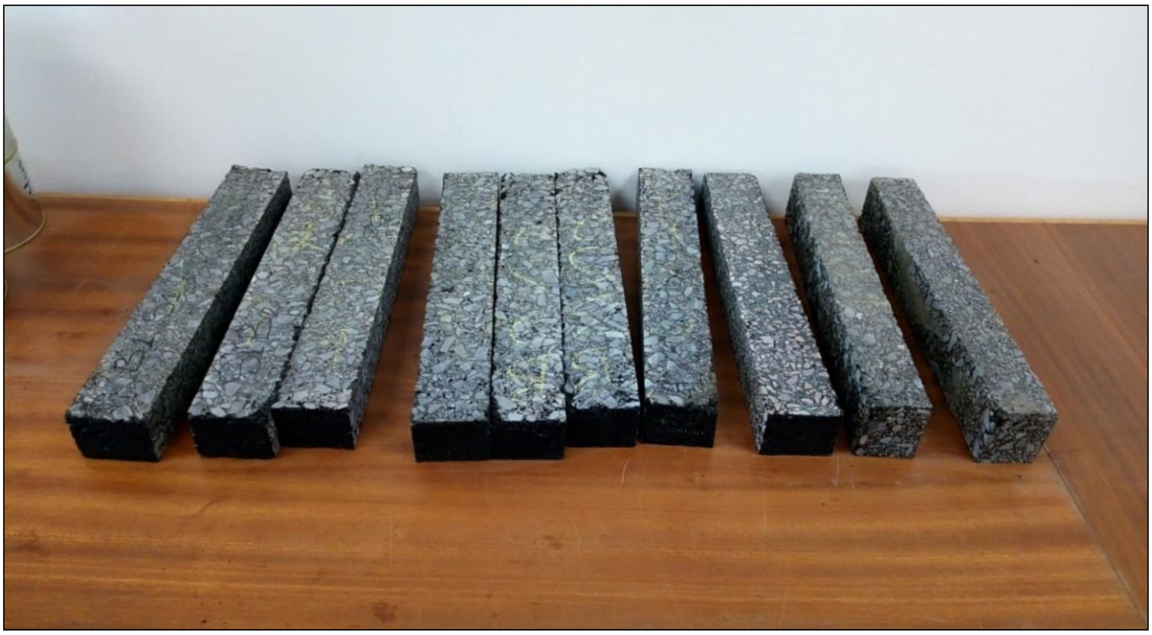

Figure 5.14. Cut and Sawed Asphalt Mix Beams Aim to Perform Four-Point Beam Bending Test (Source: Author, 2019)**.**

Various characteristics serving as input data in the computer software were collected before leaving the beam specimens to dry at room temperature for 24 hours. The data collected include the dimensions of the beam, the void in the mix and the density of the beam.

The beams prepared at level II design were tested at a unique test temperature of 10°C. The beam specimens were subjected to 10Hz repeated sinusoidal deflection wave, and the resulting applied loads were monitored during the test period.

Two different strain levels of 200 $\mu\varepsilon$ and 400 $\mu\varepsilon$ were used on the beams. In other words, at the first test round, 200 $\mu\varepsilon$ was applied to the beam number one, and at the second test round, 400 $\mu\varepsilon$ was applied to the beam number two.

The fatigue life of the control HMA beams specimens, the WMA – 15% RAP beam specimens and WMA – 30% RAP beam specimens were determined by allowing the repeated loading cycle to continue until the stiffness of the beam was reduced to 50% of the initial stiffness and the first 50th load cycle was reached.

AASTHO T 321 recommends that the failure criterion of the beam should be limited at 40% reduction of the initial stiffness measured at the first 50th cycle of loading, as this criterion ensures that the set strain of the test will capture the real failure of the asphalt beam specimens. However, the AASTHO's recommendation to obtain the real failure was not considered in this study. Instead, the failure criterion of the asphalt beam specimen limited at 50% reduction of the initial flexural stiffness measured at the first 50th cycle of loading was considered as it is known to be the most acceptable failure criterion (Kim et al., 2018).

The prepared asphalt specimen was inserted and well kept in place by four servomotor-driven clamps at four equally spaced points. In other words, the distance between the clamps is equal, therefore dividing the beam specimen into three equal spans, as seen in Figure 5.15 shown below.

The LVDT (linear variable differential transformer) which measured the deflection of the beam at the centre of the beam was positioned over the asphalt beam specimen, so that it initially reads zero (0) value. The measurement of the deflection at the centre of the beam is relative to the deflection at the middle of the two outer spans.

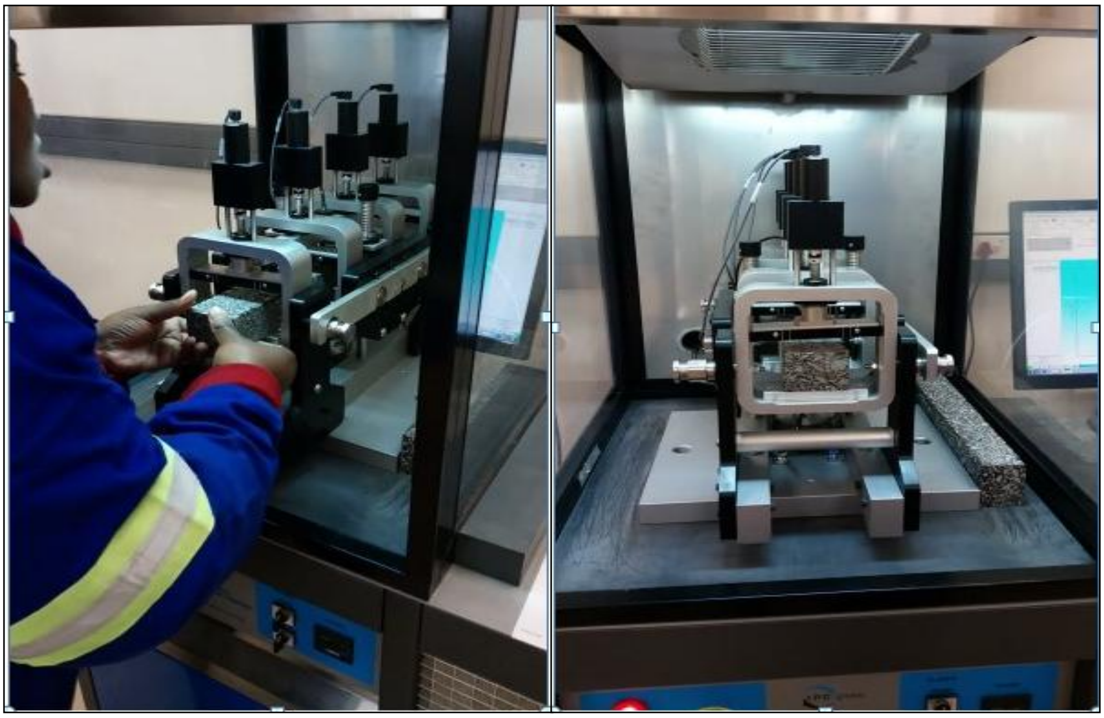

Figure 5. 15. Asphalt Beam Specimen Positioned in the Four-Point Beam- Bending Machine (Source: Author, 2019)

5.6.2. Data Collection

Tables 5.13 below shows the dimensions, densities and void percentages of the control HMA, the WMA – 15%RAP and the WMA – 30%RAP beam specimens which would be used as input data in the software supplied with the Four-Point Beam Bending machine to start and to run the test.

The standard specification recommends the void in the asphalt mix to be between 4% and 6% with an allowable deviation of ±1.5% (Sabita Manual, 2014). Thus, the air void in the asphalt mixes was found to meet the requirements imposed for the Four- Point Beam Bending test.

Table 5.-13. Four-Point Beam Bending Test Input Data

Type of mix	HMA		WMA-15%RAP		WMA-30%RAP	
Description	**Initial**	**Current**	**Initial**	**Current**	**Initial**	**Current**
No of Beam	1	2	1	2	1	2
Beam Width (mm)	62	62	62	62	62	62
Beam Height (mm)	48	48	48	48	48	48
Beam Length (mm)	380	380	380	380	380	380
Compaction Temperature (°C)	140	140	110	110	110	110
Density (kg/m³)	2463.8	2462.8	2456	2421	2430	2438
Void in the Mix (%)	3.6	3.6	4.0	5.2	5.0	4.6

(Source: Author, 2019)

The following data were also entered into the computer installed with the Four- Point Beam Bending testing machine:

- Test temperature: 10°C
- Test frequency: 10Hz
- Rest period: 10s
- Mode of loading: Displacement
- Loading wave: Sine
- Terminal pulse count: 4.000.000

Test loading strain level: 200 µƐ and 400 µƐ.

5.6.3. Results and Discussion

The output data listed below were produced and displayed on the computer screen when the tests were completed:

- Test loading time (hours, minutes and seconds)
- Applied load
- Beam deflection
- Tensile stress and strain
- Core and skin temperature of a dummy specimen
- Phase angle

- Loading cycle count
- Flexural stiffness
- Cumulative dissipated energy.

Figures 5.16 and 5.17 show the flexural stiffness reduction applied at 200µℇ and 400µℇ level, respectively, when the load cycle gradually increases. The figures illustrate by excellence the conventional definition of fatigue failure as being 50% reduction reached of the initial flexural stiffness when measured at 50^{th} loading cycle.

Tables 5.14 and 5.15 presented below show the test conditions and the results of the control HMA, WMA – 15% RAP and WMA – 30% RAP determined after the test was completed. Figures 5.16 and 5.17 on the other hand graphically show the fatigue life ranking of the control HMA, the WMA – 15% RAP and WMA – 30% RAP at 200µℇ and 400µℇ level respectively.

Observations were made as far as the duration of the test is concerned. The tests set up at 200µℇ were completed relatively faster than the tests set up at 400µℇ. The faster completion time is explained by the fact the time interval of load repetitions is higher at 200µℇ and smaller at 400µℇ.

The results presented in Tables 5.14 and 5.15 indicate that the maximum number of loading cycles reached by the asphalt specimens is influenced by the stiffness characteristic of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP.

From Figure 5.16, the test made at 200 peak-to-peak strain level indicates that the flexural stiffness of the control HMA is lower than the flexural stiffness of the WMA 15% RAP and the WMA – 30% RAP. Similarly, the amount of stress the HMA can withstand was found to be lower than to the WMA – 15% RAP and the WMA – 30% RAP.

The trend of results observed at 200 peak-to-peak micro strain does not appear to be identical with the trend of results observed in 400 peak to peak micro strains. In Figure 5.17, the HMA presents a lower fatigue performance against the WMA – 15% RAP in one hand, but on the other hand, it presents a higher but close fatigue cracking performance to the WMA – 30% RAP. This close but low fatigue cracking performance of the WMA – 30% RAP against the control HMA may be caused by the fact that the WMA – 30% RAP beam specimen had endured minor damages through the process of cutting and sawing whereby little loss of aggregates was noticed. The loss of aggregates on the WMA – 30% RAP beam specimen eventually creates air voids that may contribute to weakening its resistance against fatigue cracking failure.

Overall, the lower fatigue cracking performance of the WMA – 30% RAP against the WMA – 15% RAP at 200 peak to peak micro strains and 400 peak to peak micro strains as observed in this study, is explained by the increased amount of RAP in the WMA. Zhoa et al. (2013) and Kim et al. (2018) substantiate this finding in their studies. The authors stated that the adding of RAP in the asphalt mixture generally appeared to reduce the fatigue life performance of the asphalt mixture and the asphalt pavement in service.

To conclude, the results of this study have shown that the WMA incorporating RAP has the capability to exhibit higher fatigue resistance than the traditional HMA. Nevertheless, that resistance of the WMA – RAP against fatigue failure can be jeopardised if careful attention is not given to the amount of RAP being added in the WMA.

Table 5.14. Four-point beam bending test results at 200 µƐ

Type of mix	HMA		WMA – 15%RAP		WMA – 30%RAP	
Description	Initial	Current	Initial	Current	Initial	Current
Test loading time (hrs: min: sec)	N/A	2:34:17	N/A	23:26:46	N/A	11:06:40
Load Applied (KN)	0.5630	0.2811	1.531	0.762	1.354	0.676
Beam Deflection (mm)	0.1108	0.1104	0.112	0.112	0.111	0.111
Peak to Peak Strain Level (µƐ)	200	200	200	200	200	200
Core Temperature of Specimen (°C)	9.5	9.8	9.7	9.7	9.1	9.8
Skin Temperature of Specimen (°C)	8.7	8.9	8.9	8.9	8.6	8.9
Phase Angle (°)	10.9	15.6	8.5	17.0	9.0	9.1
Loading Cycle Count	$4x10^6$	92570	$4x10^6$	844060	$4x10^6$	400000
Flexural Stiffness (MPa)	12009	6021	18645	9318	16955	8461
Peak to Peak Stress (KPa)	2407	1203	3749	1865	3391	1693
Cumulative Dissipated Energy (KJ/m³)	N/A	5.7	N/A	57.1	N/A	27.4

(Source: Author, 2019)

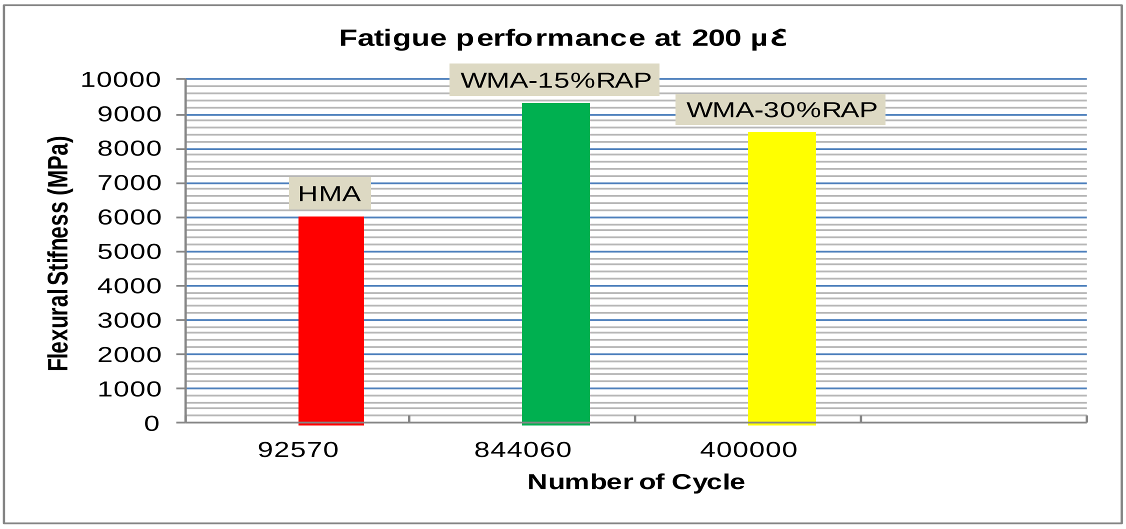

Figure 5.16. Fatigue Life Ranking of Mixes at 200µƐ (Source: Author, 2019)

Table 5.15. Four-Point Beam Bending test results at 400 µƐ

Type of mix	HMA		WMA –15%RAP		WMA –30%RAP	
Description	Initial	Current	Initial	Current	Initial	Current
Test loading time (hrs: min: sec)	N/A	00:03:07	N/A	00:30:50	N/A	00 : 20 :16
Load Applied (KN)	1.8584	0.9311	2.482	1.235	1.772	0.86
Beam Deflection (mm)	0.2220	0.2210	0.220	0.219	0.222	0.222
Peak to Peak Strain Level (µƐ)	400	400	400	400	400	400
Core Temperature of Specimen (°C)	9.9	9.9	8.8	9.0	9.4	9.5
Skin Temperature of Specimen (°C)	9.2	9.3	8.5	8..6	8.7	8.9
Phase Angle (°)	11.4	22.6	11.2	15.2	11.1	15.1
Loading Cycle Count	$4x10^6$	1870	$4x10^6$	18500	$4x10^6$	12160
Flexural Stiffness (MPa)	11199	5631	14709	7354	10417	5213
Peak to Peak Stress (KPa)	4498	2252	5930	2942	4174	2082
Cumulative Dissipated Energy (KJ/m³)	N/A	0.5	N/A	4.3	N/A	2.0

(Source: Author, 2019)

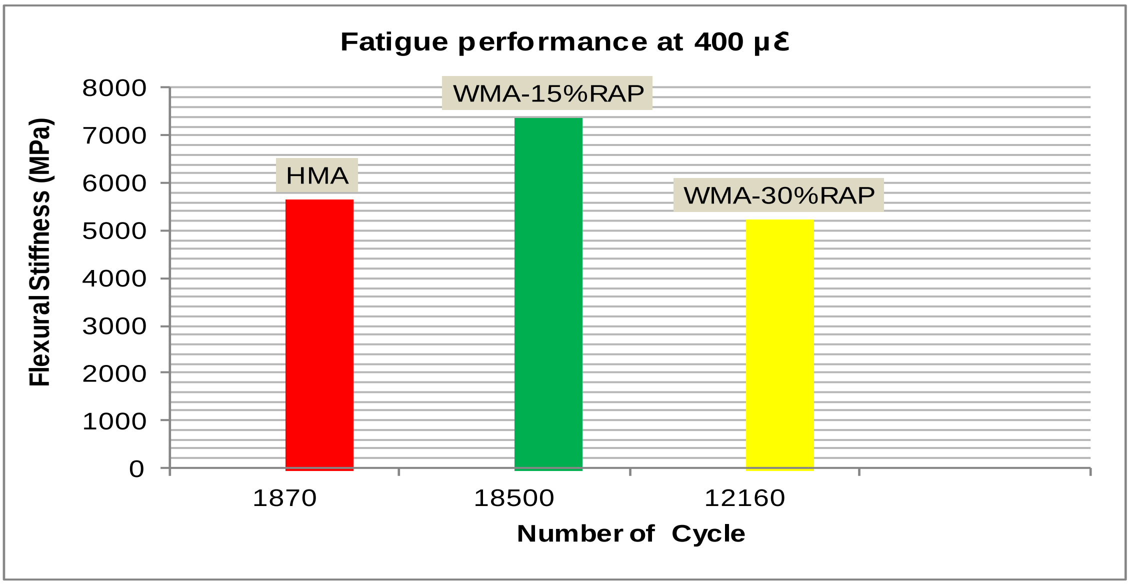

Figure 5.17. Fatigue Life Ranking of Mixes at 400µƐ (Source: Author, 2019)

5.7. Conclusion

This chapter describes the Marshall Stability and Flow Test, the Indirect Tensile Strength Test, the Dynamic Modulus Test, the Hamburg Wheel-Tracking Test and the Four -Point Beam Bending Test performed on the control HMA, the WMA – 15% RAP and the WMA – 30% RAP. It also compares the distress performances of the control HMA specimens against the distress performances of WMA – 15% RAP and WMA – 30% RAP specimens.

The next chapter provides a comparative study between the Abaqus computer programme and the mePADS computer program. Furthermore, it discusses the modelling, the computer simulation and the numerical analysis done on the control HMA pavement structures, the WMA – 15% RAP pavement structure as well as the WMA – 30% RAP pavement structures. It also predicts the distress performances of the WMA – 15% RAP pavement structure and the WMA – 30% RAP pavement structures and compares them against the control HMA pavement structures.

CHAPTER 6 – MODELLING OF MULTI-LAYERED FLEXIBLE PAVEMENT

6.1. Introduction

Flexible pavements are generally constituted of three sometimes four layers which include an asphalt surface, a stabilized base course, a granular sub-base layer and a treated in-situ materials sub-grade layer. Under the effect of traffic load and environmental conditions, the pavement structure would develop distresses (i.e. rutting failure and/or fatigue cracking failure) which originate from mechanistic responses (i.e. strain, stress, and deflection) increasing beyond the allowable limit (Gadhimi, 2015). The design and the analysis of multi-layers flexible pavement can be very complex and tedious. Its complexity may be related to factors such as the difference in material behaviour of pavement layers, the interaction of layers within each other, the imposition of heavier load and the climatic conditions. Methods such as the Bousineq's one-layer model, the multi-layer linear elastic theory, the finite element model, the layered elastic analysis and the discrete finite element (Wie Tu, 2007 and Gadhimi, 2015) are used by the Mechanistic-Empirical Design Methods to predict the mechanistic responses developed within a pavement system. As part of this study, a 2D axisymmetric model and linear elastic materials characterization in FEM were used to determine the mechanistic responses developed within the flexible pavement structures subjected to static wheel loading.

6.2. Modelling and Simulation of Pavement Structures

The FEM is known to be the most accepted and accurate method of simulation. Furthermore, some previous researches conducted have shown that the FEM can predict more accurately the stress, the strain as well the deflection developed within the layered pavement system as a result of the loading and the extreme environment conditions (Wie Tu, 2007). Few computer programs based on the FEM were developed to simulate loads imposed on asphalt pavement. Those software packages include CIRCLY, KENLAYER, NOAH, SYSTUS, BISAR, CAPA3D, VEROAD, ASTRAN, STRAND, ANSYS and ABAQUS (Gadhimi, 2015).

Alongside the FEM software packages, there are also multi-layer elastic analysis (LEA) computer programs, which include CHEV, MECDE1, ELSYM5, MichPave, ELMOD, MODCOMP, MODULUS and mePADS (Maina et al., 2008). mePADS is the latest version of the Mechanistic–Empirical analysis and design software developed by the Council for Scientific and Industrial Research (CSIR). However, only mePADS and Abaqus were used in this study FEM to simulate the pavement structures in two dimensions model. More FEM software with three-dimensional characteristic were not readily available at the time of this study due to the high cost to obtain valid licences and the high hardware demands.

The Empirical Design Methods and the Mechanistic-Empirical Design Methods (MEDM) are the two approaches engineers and researchers used for the design of flexible pavement. The Empirical Design Method, known to be the traditional design method, has long been used for the design of flexible pavement. In fact, the Empirical Design Method was developed based on the performance observation of roads constructed in the field and subjected to a real-life test scenario. In the Empirical Design Method, the material properties, the structure of the pavement and the environmental condition of the pavement system are known before the performance test (Gadhimi, 2015)

The Mechanistic-Empirical Design Method (MEDM), on the other hand, is a new design technique created to overcome the deficiency found in the Empirical design method. The substitution of the Mechanistic-Empirical Design Method over the traditional Empirical Design Method is owed to its capability to accommodate heavier traffic loads, new materials properties, various climatic conditions as well as its ability to handle long-life pavement designs. The advantages of the Mechanical-Empirical Design Method over the traditional Empirical Design Method justify their preference in this study to achieve the analysis and the design of the asphalt pavements considered.

The MEDM is developed based on the principle of physics models aimed to analyse and to predict the mechanistic responses (stress, strain and deflection) which occur within a layered pavement system and the strength of materials. The results of the stress, strain and deflection obtained through mechanistic models are then transferred to empirical formula called transferred functions or transferred models which correlates the mechanistic responses to actual pavement performances (Gadhimi, 2015).

Equations 6.1 and 6.2 mentioned below are the transferred functions given by Bruce (2001):

$\text{Nf} = 18.4\,(0.00432\mathcal{E}_t - 3.291\text{E} - 0.854)$ **Equation 6.1**

where:

N_f = the number of load repetition causing fatigue failure;

$\mathcal{E}t$ = the horizontal tensile strain at the bottom of the asphalt layer;

E = the modulus of the asphalt layer in Psi.

$\text{Nr} = 5.5 \times 10^{15} \left(\frac{1}{\epsilon\upsilon \times 10^6}\right)^{3.949}$ **Equation 6.2**

where:

Nr = the number of loads required to cause base rutting failure;

$\mathcal{E}\upsilon$ = the vertical compressive strain on top of the sub-grade.

Overall, the repetitive loads on top of the pavement structure causes horizontal tensile strain at the bottom of the surface layer and vertical compressive strain on top of the sub-grade layer to develop. The occurrence of those mechanical phenomena does not cause the failure of the pavement structure. However, the failure of the pavement structure occurs when the phenomena of horizontal tensile strain and vertical compressive strain go beyond the permissible. Therefore, empirical models as provided in Equation 6.1 and Equation 6.2 are failure models which define the point at which the actual rutting and fatigue cracking failure would occur as a result of the excessive amount of horizontal tensile strain and vertical compressive strain in the lower layers of the pavement structure.

The first pavement scenario would comprise a surface layer composed of WMA – 15% RAP, the second scenario a surface layer composed of WMA – 30% RAP and the third scenario a surface layer composed of the control HMA. All of the three flexible pavement scenarios comprise a sub-base constituted of C3 granular material, and a sub-grade constituted of Soil

In Chapter 5, laboratory experiments were conducted on the WMA – 30% RAP, WMA – 15% to evaluate their resistance against the rutting failure and the fatigue cracking failure and thus to compare them with the control HMA made with 100% virgin materials. However, Chapter 6 aims to model, simulate and analyse WMA – 15% RAP pavement structure, WMA – 30% RAP pavement structure and the control HMA pavement structure using FEM in the Abaqus computer program. Overall, Chapter 6 would help to evaluate, understand and compare the rutting and fatigue cracking performance of the WMA – RAP against the control HMA when laid as an asphalt layer within the respective pavement structures. Therefore, the aims of this chapter were achieved by:

- Modelling, simulating and analysing the control HMA pavement structures at various asphaltic layer thicknesses using both FEM in Abaqus Axisymmetric model and LEA in mePADS;
- Conducting a comparative study between the FEM in Abaqus Axisymmetric model and LEA in mePADS in order to select the methods which offer the most reliable results;
- Modelling, simulating and analysing the WMA – 15% RAP and the WMA – 30% RAP pavement structure at various asphaltic layer thicknesses using the prefered method of numerical modelling (either FEM in Abaqus Axisymmetric model or LEA in mePADS);
- Predict the rutting performances and the fatigue cracking performances of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP pavement structures;
- Conducting a comparative study between the control HMA, the WMA – 15% RAP and the WMA – 30% RAP pavement structures in terms of distress performances.

6.2.1. The Geometry of the Models

The modelling of the layered pavement system requires the selection of a convenient geometrical model (i.e. 2D plain, 2D Axisymmetric and 3D). The pavement structures modelled in 3D is known to possess the most probable inclusive simulation of the actual problem based on its highly accurate results. However, 3D model was not considered in this study, as it demands expensive hardware and software licences and requires relatively high computational time (Myers, Roque, & Birgisson, 2001). As part of this study, the 2D Axisymmetric model in Abaqus software was selected to model the asphalt pavement structures.

In real life, the actual road pavement has a longitudinal dimension and width in which the unit of measure is in several kilometres and/or metres. Hence, modelling a limited pavement area very close to the tyre contact in 2D model is necessary. Thus, the flexible pavement structures measuring 3000mm in the transversal direction was modelled. The depth of the sub-grade layer is generally assumed infinite. So, instead of having an infinite depth at the sub-grade layer, a 2000mm depth was selected. Ahmed (2006) adopted a similar geometry to avoid edge error during the sequence of analysis. Furthermore, in a study, Raman (2011) recommended the depth of 2000mm , as there was no deformation occurring after a certain amount of layer depth and for the sake of boundary conditioning.

Each layer of the pavement structure idealized as a beam had a stabilized base (C3) measuring 150mm in thickness and an in-situ treated materials subgrade measuring (Soil) 2000mm in depth. Only the asphalt layer made of WMA – 15% RAP (AC20), WMA – 30%RAP (AC20) and the control HMA (AC20) technology had its depth varying from 50mm to 150mm and this at an increment of 50mm. The changing of the depth of the asphalt top layer was to analyse and determine the depth that will sustain the pre-determined imposed load without exhibiting considerable distress failures. The interfaces between the asphalt layers and the lower layers were fully bounded.

6.2.2. Material Characterization Assigning

The pavement structure is generally composed of asphalt layers, granular treated or untreated layers and a subgrade in-situ layers. Each of these layers behaves differently because of the different material properties they are made of. Therefore, the asphalt layer may possess one of the three types of model characteristic: the elastic model, the viscoelastic model, or the viscoelastoplastic model. The granular and the sub-grade layers, on the other hand, are usually modelled as a linear elastic model, non- linear elastic model or linear elastoplastic model. However, the modelling done in this study would consider only the linear elastic model accounting both for the asphalt top layer and for the lower layers; this is because the 2D axisymmetric model considered in this study accommodates linear elastic materials properties.

The material properties of the asphalt course used in this study were obtained through the Dynamic Modulus and Four-Point Beam Bending Test discussed in Chapter 5.

Heyns and Mostafa Hassan (2013) conducted a study whereby the properties of the sub-base layer were determined. Thus, the properties obtained by Heyns and Mostafa Hassan (2013) were used in this study to account for the material properties of the sub-base. The South African design manual, on the other hand, recommended the properties of the sub-grade. Table 6.1 presented below shows the materials code, the Poisson ratio, the elastic modulus and the layer thicknesses adopted in this study.

Table 6.1. Mechanical Characterizations of Granular Layers

Pavement Layer	Layer Thickness (mm)	Density (kg/m^3)	Elastic Modulus (MPa)	Poisson Ratio	Material Code (Colto2008)
Stabilised Base	300	2133	2560	0.35	C3
Sub-grade	2000	1680	60	0.35	Soil

(Source: Heyns and Mostafa Hassan, 2013)

Table 6.2 presents the mechanical characteristic of the asphalt layer. Those asphalt characteristics were used as input data in mePADS and Abaqus programs to model and analyse the control HMA, the WMA – 15% RAP and the WMA – 30% RAP pavement structures upon wheel loading.

The densities and the elastic modulus of the control HMA, the WMA – 15% RAP and the WMA – 30% RAP asphalt layers were obtained through the Dynamic Modulus and the Four-Point Beam Bending Test results.

Table 6.2. Mechanical Characterizations of Asphalt Base Course

Pavement Layer	Layer Thickness (mm)	Density (kg/m^3)	Elastic Modulus (MPa)	Poisson Ratio	Material Code (Colto2008)
Asphalt (HMA)	50 – 150	2512	2487	0.44	AC20
Asphalt (WMA – 15% RAP)	50 – 150	2512	5535	0.44	AC20
Asphalt (WMA–- 30% RAP)	50 – 150	2490	2612	0.44	AC20

(Source: Author, 2019)

6.2.3. Meshing and Boundary Condition

The boundary condition was selected in a way that it resembles the real-life boundary condition. Thus, a roller was used to restrict the degree of freedom on individual layers. The asphalt surface, the gravel base layer, as well as the treated sub-grade layer, were restrained from moving in the transversal and the longitudinal directions (i.e. the degree of freedom 1 and 3). The sub-grade layer, on the other hands, was restrained from moving not only in the transversal and horizontal direction but also in the vertical direction (i.e. the degree of freedom 1, 2 and 3). The fixation at the bottom of the sub-grade layer is intended to prevent any displacement or rotation that may be caused by velocity and acceleration.

Saad, Mitri and Poorooshasb (2006) recommended the meshing to be fine near the loading area and coarse at the remote distance where the tyre load is being applied. The meshing of the models in this study was done by selecting axisymmetry with reduced integration as elements type, the selection of this element type was based on the fact that they have the potential and the capability of showing large deformation and material linearity. Figure 6.1 illustrates the typical 2D axisymmetry FEM mesh used for the pavement simulation.

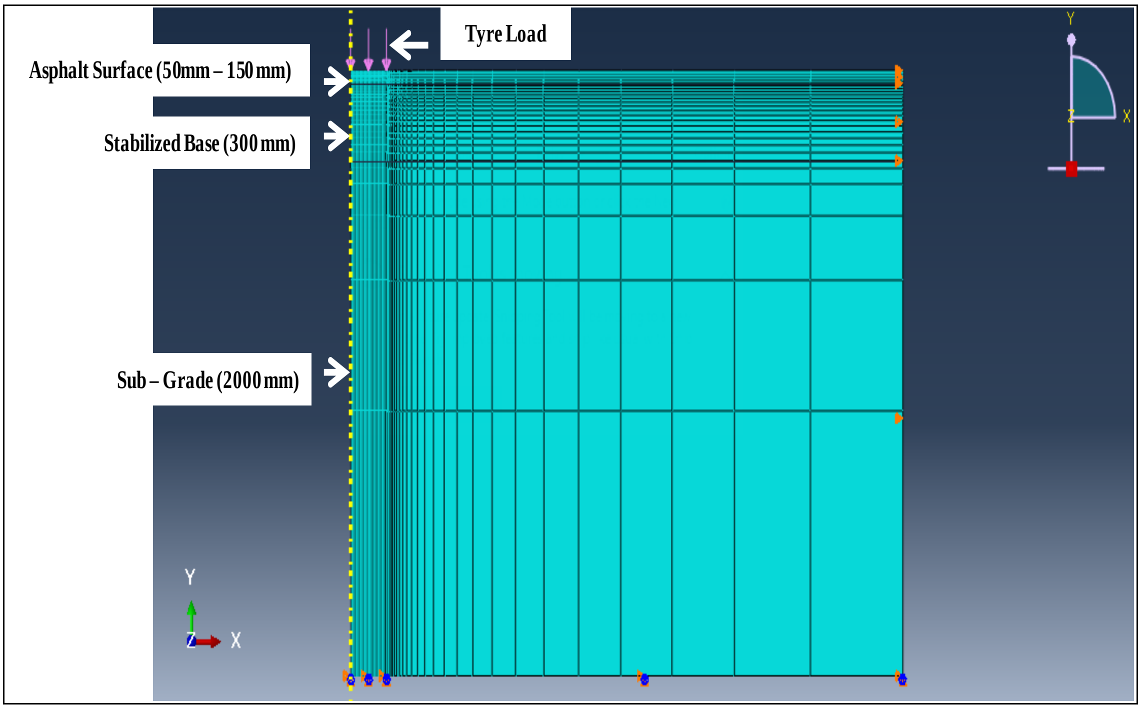

Figure 6.1. Typical Axisymmetry Model of a Pavement Structure with Mesh, Load and Boundary Condition (Source: Author, 2019)

6.2.4. Loading

A standard equivalent wheel load of 80 KN was uniformly applied on the pavement structure. The contact area between the applied single tyre load and the pavement surface was assumed to have a circular shape having a radius of 194.954mm and a total area of 29850.677mm^2. Furthermore, the tyre pressure again the surface of the pavement structure was assumed to be 670KPa or 0.67MPa. Theyse, de Beer, Maina and Kannemeyer (2011) successfully applied the same approach in a study.

It is essential to further emphasize that the loading assumed in this study did not take into account the effect of the moving tyre. However, the focus was instead on the impact caused by repeating the load cycle on the same spot. The same approach was previously adopted by Bodhinayake (2008) and Al-Qadi, Wang and Tutumluer (2010) in their researches. Based on the previous study led by the Australian researcher Bodhinayake (2008), the load moving at a speed of 100 km/hr was simulated in a way that it would remain on the area of interest for 0.1s followed by 0.9s period of rest.

6.3. mePADS versus Abaqus Axisymmetric

The control HMA pavement structure at 50, 100, and 150 mm layer thicknesses were modelled, simulated and analysed using mePADS and Abaqus. Table 6.3 presented below shows the results obtained in terms of horizontal tensile strain located at the bottom of the asphalt top layer and the vertical compressive strain located at the top of the subgrade. The 2D Axisymmetric model in Abaqus gives very close results to that of the multi-layered elastic model in mePADS as the percentage differences between them varies between 0.5% and 55%. Dwelling on the fact that the majority of the differences between Abaqus and mePADS are less than 3%, it implies that modelling and analysing of pavement structures using the multi-layered elastic model in mePADS or FEM Axisymmetric in Abaqus will not make much difference. However, the FEM Axisymmetric in Abaqus would be of more advantage than the mePADS as the stress, the strain and the deflection can be accurately determined at any dissected single element of the entire pavement structure, be it in horizontal or in the vertical axis. Appendix A1 – A7 and A8 – A9 represent the numerical model and analysis of the control HMA structures using mePADS and Abaqus respectively.

Table 6.3. mePADS versus Abaqus Axisymmetric

Pavement Type	Pavement Responses	mePADS	Abaqus Axisymmetric Model	Percentage Difference (%) btw mePADS & Abaqus Axisymmetric Model
Asphalt (HMA) 50 mm	Ɛt (10^{-6}) Bottom of Surface	65.49	65.79	0.5
	Ɛc (10^{-6}) Top of Sub-grade	560	548.2	2
Asphalt (HMA) 100 mm	Ɛt (10^{-6}) Bottom of Surface	4.09	9.11	55
	Ɛc (10^{-6}) Top of Sub-grade	440	433.3	2
Asphalt (HMA) 150mm	Ɛt (10^{-6}) Bottom of Surface	32.87	32.24	2
	Ɛc (10^{-6}) Top of Sub-grade	355	351.8	1

(Source: Author, 2019)

6.4. Effect of Control HMA Thicknesses Variation on Pavement Responses

Table 6.4 shown below presents the results of the mechanical responses determined using the 2D Axisymmetric model in Abaqus as well as the HMA pavement distresses determined using the empirical equations 6.1 and 6.2.

It can be seen from Table 6.4 that the horizontal tensile strains produced at the bottom of the HMA layer change inconsistanly when the thickness of the HMA layer is increased at 50mm increments. However, the 100mm HMA pavement structure exhibits the lowest horizontal tensile strain. This implies that, in comparison with the 100mm and 150mmm HMA pavement structures, the 100mm HMA pavement structure possesses the highest strength against horizontal tensile strain.

The predicted fatigue failures of pavement structure is influenced by the horizontal tensile strain produced at the bottom of the HMA layer. However, the maximum number of loads causing fatigue failure on the HMA pavement structure remains unchanged regardless of the variation of the horizontal tensile strains.

Furthermore, it is observed from Table 6.4 that the vertical compressive strain produced at the top of the sub-grade layer decreases as the thickness of the HMA layer increases. This signifies that HMA pavement structure improves in strength against compressive strain when the thickness of the HMA layer is big.

The prediction of the number of loads causing rutting failure on the HMA pavement structure is influenced by the vertical compressive strain at the top of the sub-grade layer. Thus, it can be seen from Table 6.4 that the gradual decrease of the vertical compressive strain does result in the gradual increase of the maximum number of loads casuing rutting damage on the HMA pavement structures.

Table 6.4. HMA Pavements Responses

HMA				
Asphalt Layer Thickness (mm)	**εt (10^{-6}) Bottom of Surface**	**No of Load Repetition to failure N_f (10^{6})**	**εc (10^{-6}) Top of Sub-grade**	**No of Load Repetition to failure N_r (10^{3})**
50	65.79	21.8	548.2	84.00
100	9.11	21.8	433.3	212.66
150	32.24	21.8	351.8	484.22

(Source: Author, 2019)

6.5. Effect of Control WMA – RAP Thicknesses Variation on Pavement Responses

Tables 6.5 and 6.6 shown below present the mechanical response and the distress results of the WMA – 15% RAP and WMA – 30% RAP pavement structures, respectively. It should be recalled that the distress results presented in Tables 6.5 and 6.6 were determined using the empirical equations 6.1 and 6.2. Appendix A1 – A16 represents the numerical model and analysis results of the control HMA pavement structures using FEM 2D Axisymmetric model in Abaqus programme and LEA in mePADS program. Appendix B1 – B13 represent the numerical model and analysis results of the WMA – 15% RAP pavement structures using FEM 2D Axisymmetric model in Abaqus program. Appendix C1 – C13 represents the numerical model and analysis results of the WMA – 30% RAP pavement structures using FEM 2D Axisymmetric model in Abaqus program.

It can be seen from Tables 6.5 and 6.6 that the horizontal tensile strains produced at the bottom of the WMA – 15% RAP layer and of the WMA – 30% RAP layer change inconsistanly when the thicknesses are being increased at 50mm increments. However, the 100mm WMA – 15% RAP pavement structure exhibits the lowest horizontal tensile strain. This signifies that the 100mm WMA – 15% RAP pavement structure possesses the highest strength against horizontal tensile strain in comparison with the 50mm and 150mm WMA – 15% RAP pavement structures. The same observation and implication is made with the WMA – 30% RAP pavement structure.

The predicted maximum number of load repetitions causing fatigue failure on the WMA – RAP pavement structures is influenced by the horizon tensile strain produced at the bottom of the WMA – RAP layer. Thus, the increased horizon tensile strains at the bottom of the WMA – RAP layers have no effect whatsoever on the predicted maximum number of load repetition causing fatigue failure on both the WMA – 15% RAP pavements system and the WMA – 30% RAP pavements system. In other words, the resistance of the WMA – RAP pavement structures against fatigue cracking failure remains unchanged regardless of the variation of the horizontal tensile strains. Furthermore, it is observed from Tables 6.5 and 6.6 that, the WMA – 15% RAP pavement structures have higher fatigue cracking performances than the WMA – 30% RAP pavement structures.

Unlike the horizontal tensile strain, the vertical compressive strain shows consistent and gradual decrease along with the 50mm incremental increase of the WMA – RAP layers thicknesses. Therefore, one can assume that both the WMA – 15%RAP pavement and the WMA – 30%RAP pavement improves in strength against vertical compressive strain when the thickness of the WMA – RAP layers is thicker.

It should be recalled that the predicted maximum number of loads causing rutting failure on the pavement structure is influenced by the vertical compressive strain located at the top of the sub-grade. Thus, Tables 6.5 and 6.6 show that both the WMA – 15%RAP and the WMA – 30%RAP pavement structures exhibit gradual increase in the number of loads causing rutting failure when the vertical compressive strains decrease. This implies that the WMA – 15%RAP and WMA – 30%RAP pavement structures have better resistance against rutting failure when the WMA – RAP layer thickness is increased.

It is further observed from Table 6.5 that the WMA – 15%RAP pavement structures have the highest number of load repetition causing rutting failure. This signifies that pavements constructed with WMA – 15% RAP asphalt layer would stand higher ability to resist rutting failure compared to the WMA – 30%RAP ones.

Table 6.5. WMA – 15%RAP Pavements Responses

WMA – 15%RAP				
Asphalt Layer Thickness (mm)	**Ɛt (10^{-6}) Bottom of Surface**	**No of Load Repetition to failure N_f (10^6)**	**Ɛc (10^{-6}) Top of Sub-grade**	**No of Load Repetition to failure N_r (10^3)**
50	39.65	48.61	494.2	126.51
100	28.80	48.61	376.7	369.62
150	44.92	48.61	296.5	951.34

(Source: Author, 2019)

Table 6.6. WMA – 30%RAP Pavements Responses

WMA – 30%RAP				
Asphalt Layer Thickness (mm)	**Ɛt (10^{-6}) Bottom of Surface**	**No of Load Repetition to failure N_f (10^6)**	**Ɛc (10^{-6}) Top of Sub-grade**	**No of Load Repetition to failure N_r (10^3)**
50	64.53	22.94	545	85.97
100	9.742	22.94	429.6	219.98
150	33.39	22.94	348.1	504.86

(Source: Author, 2019)

6.6. Comparative Study of Control HMA versus WMA – RAP

Figure 6.2 and 6.3 represent the plot distress responses of the control HMA pavement structure, the WMA – 15% RAP and the WMA – 30%RAP pavement structures. Thus, from Figure 6.2, it observed that the control HMA pavement structures have lower resistance against fatigue cracking than both the WMA – 15% RAP and WMA – 30% RAP pavement structures.

The same observation was made on Figure 6.3, where the 50 mm, 100mm and 150 mm HMA pavement structures exhibit the lowest fatigue cracking performance in comparison with WMA – 15% RAP pavement structures and the WMA – 30% RAP pavement structures.

Overall, the WMA – 15%RAP and the WMA – 30%RAP pavement structures present better rutting and fatigue cracking performance than the control HMA pavement structures. This with the WMA – 15% pavement structures exhibiting the highest rutting and fatigure cracking performance.

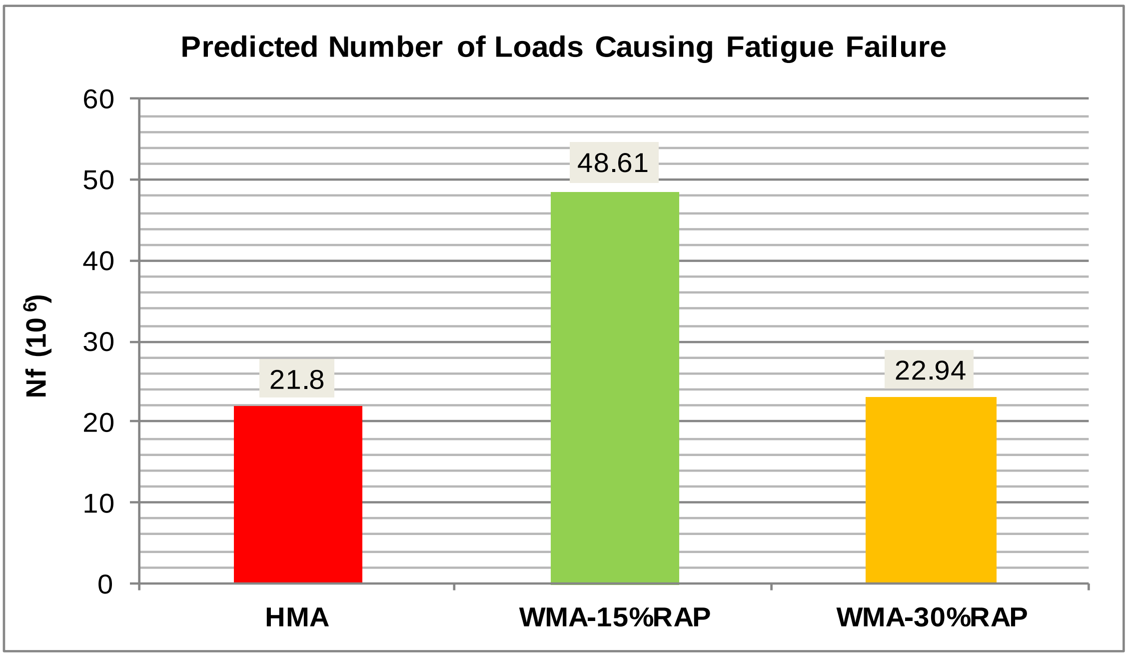

Figure 6.2. Pavement Structures Fatigue Cracking Performances (Source: Author, 2019)

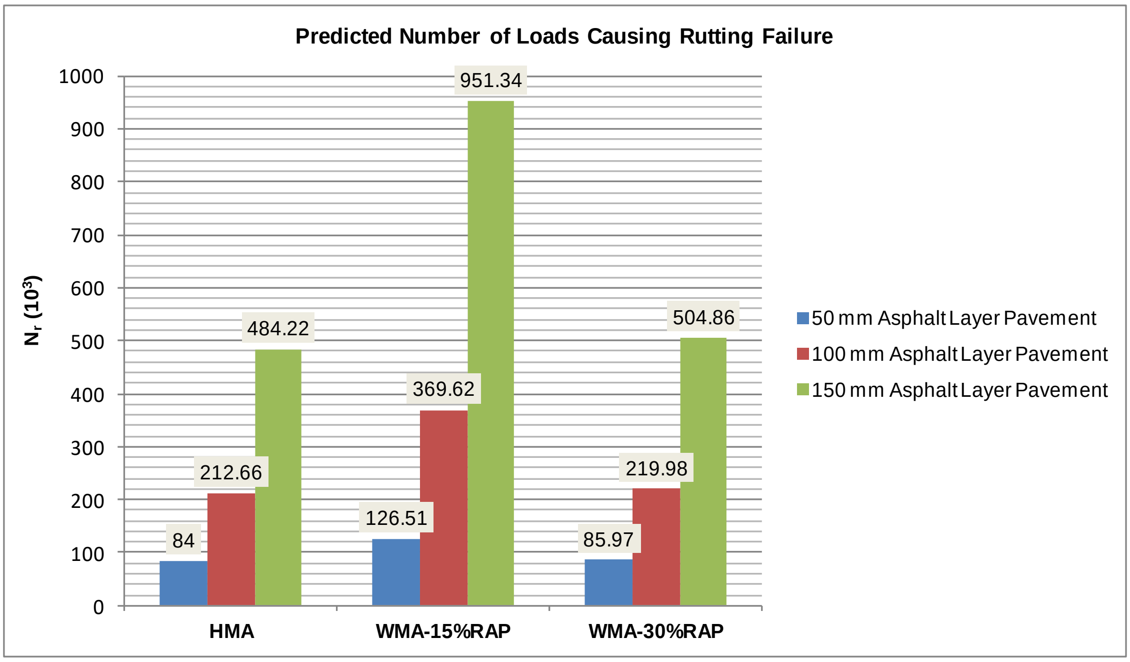

Figure 6.3. Pavement Structures Rutting Performance Prediction (Source: Author, 2019)

6.7. Summary

This chapter provides a comparison between FEM in Abaqus and LEA in mePADS. It discusses the modelling, the simulation and the analysis of the HMA, the WMA – 15% RAP and tbe WMA – 30% RAP pavement structures using Abqus computer programs based on FEM. Furthermore, it presents the procedures used to predict the rutting and the fatigue cracking failures of the HMA, WMA – 15% RAP and WMA – 30% RAP pavement structures. Finally, it compares the rutting and fatigue cracking performance of the control HMA pavement structures against the WMA – 15% RAP and WMA – 30% RAP pavement structures.

The next chapter provides the summary of the study, the major findings and the conclusion. It also proposes few recommendations to carry on future studies.

CHAPTER 7 - CONCLUSION AND RECOMMENDATION

7.1. Introduction

The greenhouse gas emission, the excessive abuse and exploitation of natural resources and the heat emission are regarded amongst serious threats planet Earth is currently facing. Conscious of the short- and long-term consequences of those threats, sectors which include manufacturing, agricultural, construction, transportation and other key development sectors have taken and adopted measures that would contribute to curbing the devastating rising effects of pollution and the depletion of natural resources. Kheradmand et al. (2013) reported the instance of the asphalt industry that is constantly making efforts to reduce its emission by decreasing the mixing and compaction temperature of asphalt mixture. Furthermore, Tao and Mallick (2009) presented the WMA technology and the addition of the RAP aggregates as solutions to lower greenhouse gas emission and reduced the dependency on virgin aggregates. The WMA incorporating RAP was introduced to replace the traditionally used HMA, known to be aggressive to the environment. Studies conducted in the past have revealed that, though the WMA technology may offer environmental, cost and health benefits, those benefits can be extended to mechanical performance if the WMA is mixed with appropriate additives and RAP. Hence, the aim of this dissertation was to investigate the mechanical and Marshall performances of the WMA – 15% RAP, the WMA – 30% RAP and the traditional HMA, this through laboratory experiment and numerical modelling. Thus, the laboratory experiments involve the selection of materials, the mix designs, the production of asphalt premixes, the determination of volumetric properties, the fabrication of asphalt mix specimens, the Marshal Stability and Flow test, the Indirect Tensile Strength test (ITS), the Dynamic Modulus test, the Four- Point Beam Bending test and the Hamburg Wheel-Tracking test. On the other hand, the numerical modelling, computer simulation and the pavement analysis were achieved using the LEA in mePADS and FEM in Abaqus.

7.2. Critical Findings

The laboratory experiments conducted on the HMA, the WMA – 15% RAP and the WMA – 30% RAP specimens have led to the following findings:

- The Marshall Quotient (MQ) values of the control HMA were found to be 8% lower than the WMA – 15% RAP MQ and 4% lower than the WMA – 30% RAP MQ. These differences are less than 10% which implies that the WMA – RAP possess close capability to that of the control HMA to carry and spread loads across the pavement layer.
- The Indirect Tensile Strength (ITS) of the control HMA was found to be lower than the WMA – 15% RAP and the WMA – 30% RAP ones. This infers that the WMA – 15% RAP and WMA – 30% RAP possess higher tensile strength than the control HMA. It is an indication that the WMA – 15% RAP and the WMA – 30% have better resistance against fatigue cracking than the control HMA.
- The rut depths of the control HMA were found to be higher than the WMA – 15% RAP and WMA – 30% RAP. This signifies a better rutting resistance of the WMA – 15% RAP and WMA – 30% RAP when compared to the control HMA. In addition, it is worth noting that the WMA – 15% RAP performs better when compared to the WMA – 30% RAP. This indicates that the increased RAP in the WMA could affect its capability to resist rutting failure.
- At 200μƐ test, the control HMA exhibits the lowest fatigue performance compared to the WMA – RAP. However, at the 400μƐ test, the control HMA presents a lower fatigue performance against the WMA – 15% RAP and a close but higher fatigue performance to the WMA – 30% RAP. This indicates that the increase of RAP in the WMA could affect its capability to resist fatigue cracking.

As far as the numerical modelling of the pavement structures is concerned, the following findings were made:

- The close difference of results between the FEM 2D axisymmetric model in Abaqus and the LEA in mePADS implies that both models can confidently be used for pavement modelling and designs of flexible pavement.

- Pavement structures designed with the HMA surface layer increase in resistance against rutting failure when the vertical compressive strain at the top of the sub-grade layer decreases. This implies that the thicker the HMA layer, the higher the rutting performance of the HMA pavement structure. As far as the resistance against fatigue cracking is concerned, it remains constant regardless of the change in horizontal tensile strain located at the bottom of the HMA layer. This signifies that the change in HMA layer thickness does not affect the fatigue cracking performance of the HMA pavement structure.
- Pavement structures designed with the WMA – 15% RAP surface layer and WMA – 30% of RAP surface layer increase in resistance against rutting failure when the vertical compressive strain at the top of the sub-grade layer decreases. This implies that the thicker the WMA – RAP layers, the higher the rutting performance of the WMA – RAP pavement structures. As far as the resistance against fatigue cracking is concerned, it remains constant regardless of the change in horizontal tensile strain located at the bottom of the HMA layer. This signifies that the change in WMA – 15% layer thickness or WMA – 30% RAP layer thickness does not affect the fatigue cracking performance of the WMA – RAP pavement structures.
- The control HMA pavement structures exhibit lower rutting and fatigue cracking performance than the WMA – 15% RAP and the WMA – 30% RAP pavement structures. This, with the WMA – 15% RAP pavement structures exhibiting the highest rutting and fatigue cracking performance of them all.
- The finding made on evaluating the rutting and fatigue cracking of the control HMA pavements structures, the WMA – 15% RAP pavement structures and the WMA – 30% RAP pavement structures through numerical simulation are in agreement with the findings made on evaluating the rutting and the fatigue cracking performance of the control HMA specimens, the WMA – 15% RAP specimens and the the WMA – 30% RAP specimens through laboratory experiments.

7.3. Conclusion

The objective of this study was to evaluate the distress performances of the WMA – 15% RAP and the WMA – 30% RAP and compare them against the distress performances of the traditionally used HMA.

The evaluation was achieved through laboratory experiments and numerical modelling of pavement layers. The laboratory experiment involves the selection of materials, the materials mix designs, the production of asphalt premixes, the production of asphalt specimens, the preparation of asphalt specimens, the subjection of the asphalt mix specimens to Hamburg Wheel-Tracking test, Four-Point Beam Bending test, Indirect Tensile Strength test, Marshall Flow and Stability test and finally Dynamic Modulus test. The procedures revolving around the laboratory experiments were governed by standards and specifications set by national highway agencies around the world.

The numerical modelling. on the other hand, involves materials modelling, geometry modelling, the load simulation and the numerical analysis of the control HMA pavement structures, the WMA – 15% RAP pavement structures and the WMA – 15% RAP pavement structures. The design was based on the Mechanistic – Empirical Design Method. The modelling of the pavement structures and their analysis were achieved using finite element method (FEM) in the Abaqus computer program and layered elastic analysis (LEA) in mePADS.

Major findings reveal that the WMA – 15% RAP and the WMA – 30% RAP pavement structures possess better fatigue cracking performance and rutting performance than the traditionally used HMA pavement structures. In addition, the WMA – 15% RAP and the WMA – 30% RAP pavement structures have higher capacity to carry and spread loads across the pavement structure than the WMA pavement structures.

In conclusion, the WMA – 15% RAP and the WMA – 30% RAP can contribute to the new construction and the rehabilitation of flexible pavements structure that are strong, safe, sustainable and environmentally friendly. These conclusions are eventually based on the evidence that the WMA – 15% RAP and the WMA – 30% RAP possess better mechanical and distress performances than the traditional HMA. In addion, the technology behind the WMA – 15% RAP and the WMA – 30% RAP uses reusable materials and production techniques that are less harmful to the environment and to workers.

7.4. Recommendation for Further Studies

The laboratory experiments showed a reduced strength performance of the WMA when the percentage of RAP added into the WMA is being increased. This phenomenon may be caused by the fact that the recommended dosage of the Sasobit organic additives in the WMA – 15% RAP was the same in the WMA – 30% RAP. Therefore, future study which aims to equalise or surpass the mechanical and the distress performances of the WMA – 30% RAP against the WMA – 15% RAP is recommended.

In addition, the numerical modelling performed in this study considers the pavement structures to be linearly elastic throughout, and it uses the FEM 2D axisymmetric model in Abaqus. The performance results obtained through the usage of the linear elastic materials and 2D axisymmetric model in Abaqus were considered satisfactory as they were similar to the results found through laboratory experiments. However, for better accuracy purposes, advanced research is recommended to model and analyse the WMA – 15% RAP and the WMA – 30% RAP pavement structures using viscoelastic material model and FEM 3D geometry model in Abaqus.

Overall, pursuing further studies as recommended above, may result in discovering and exploring further unknown benefits of WMA technology incorporating RAP and therefore expand its usability on the African continent level and throughout the world.

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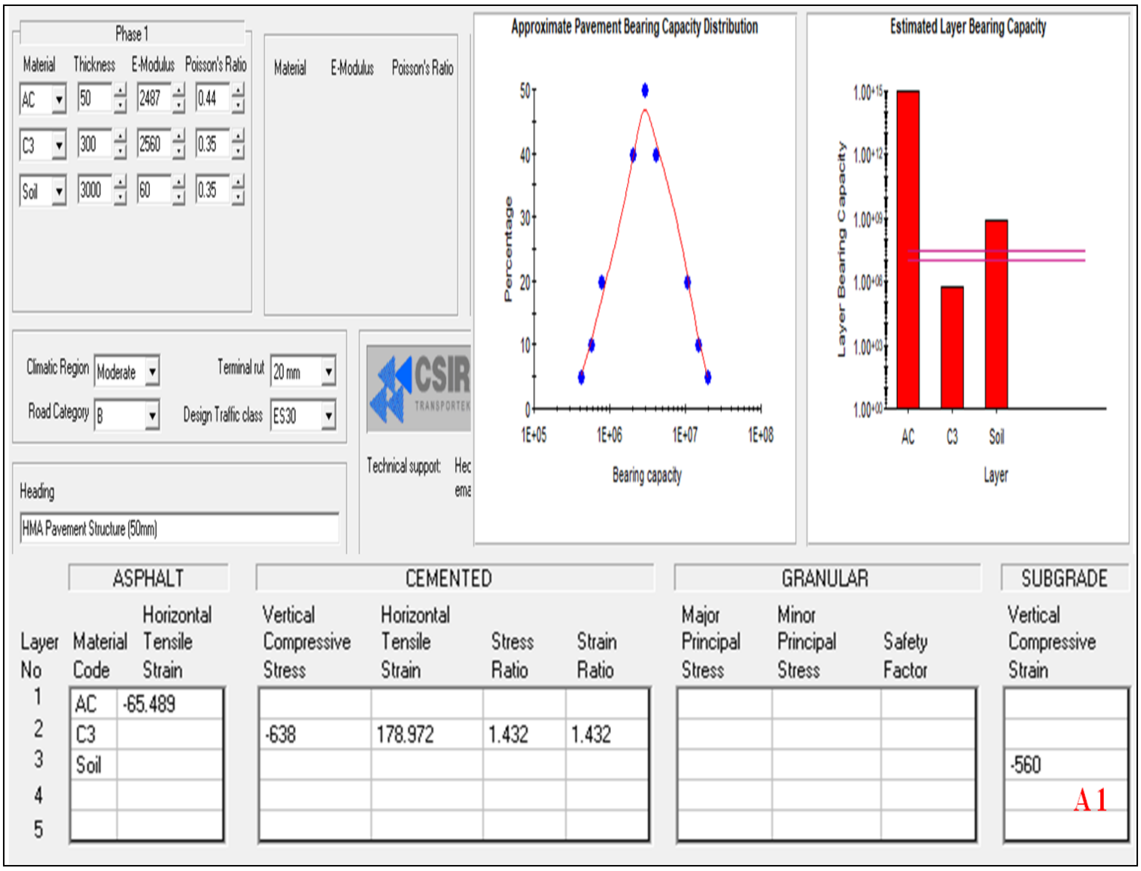
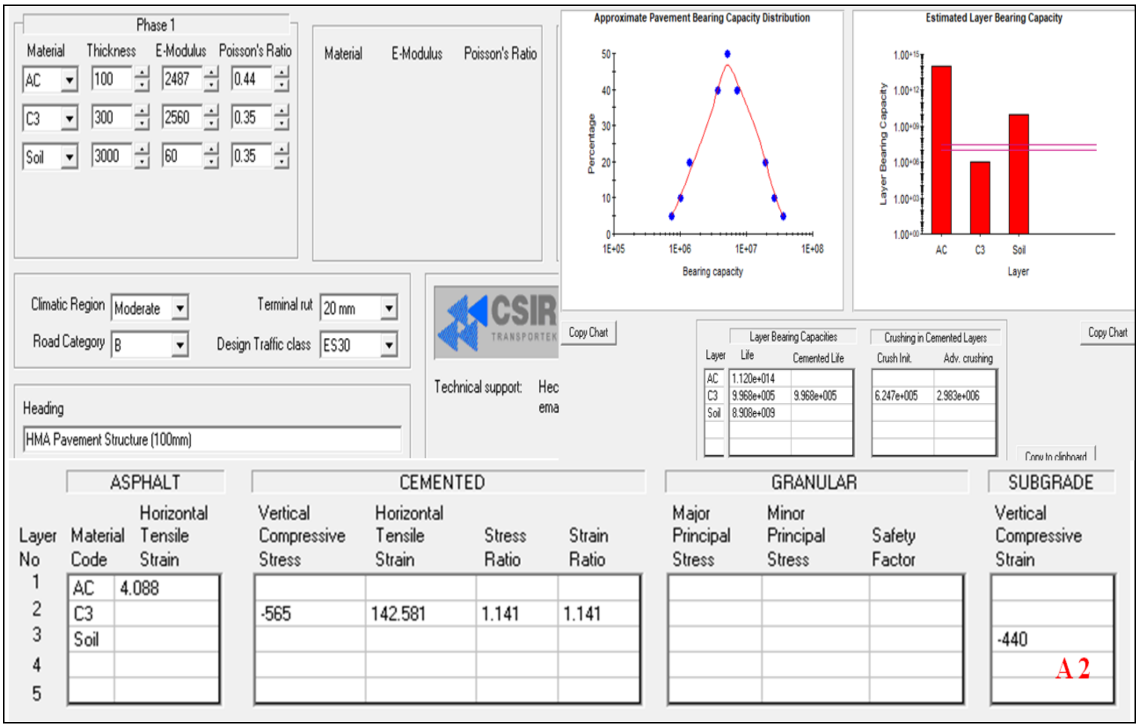
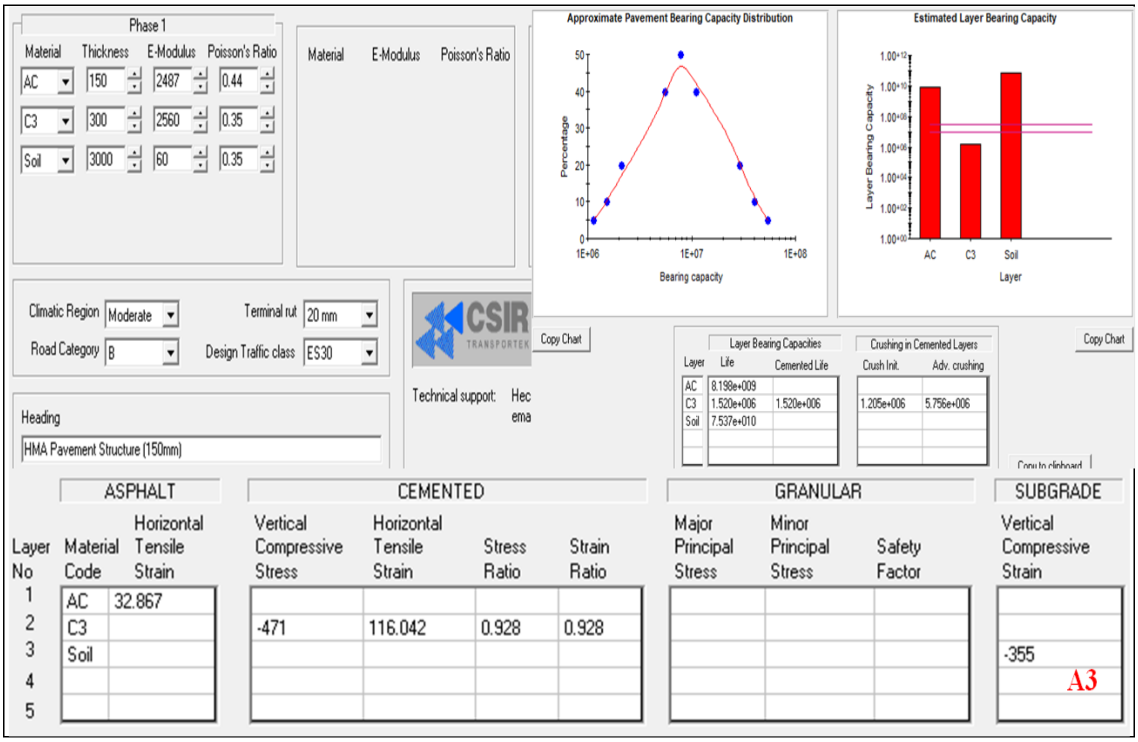
APPENDICES



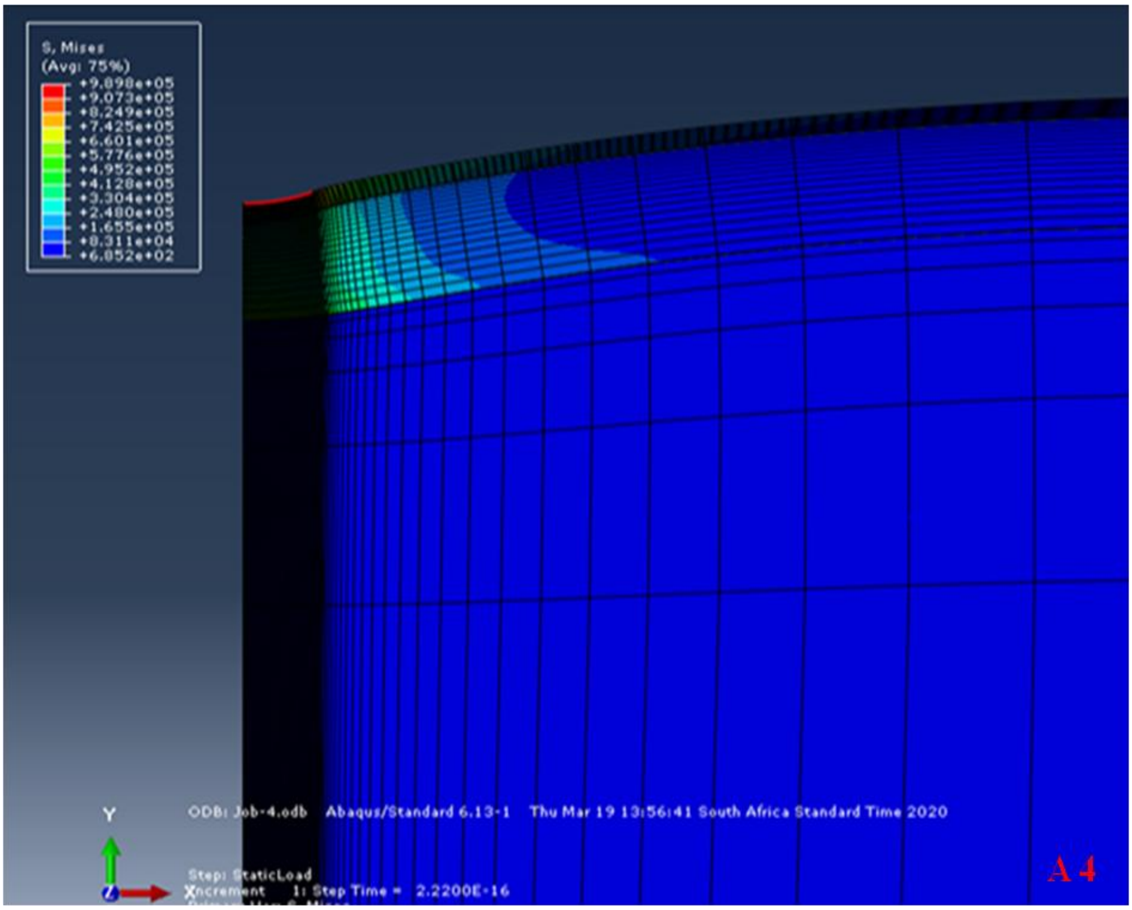
Appendix A1. Numerical Modeling of 50 mm HMA Pavement Structure Using mePADS software



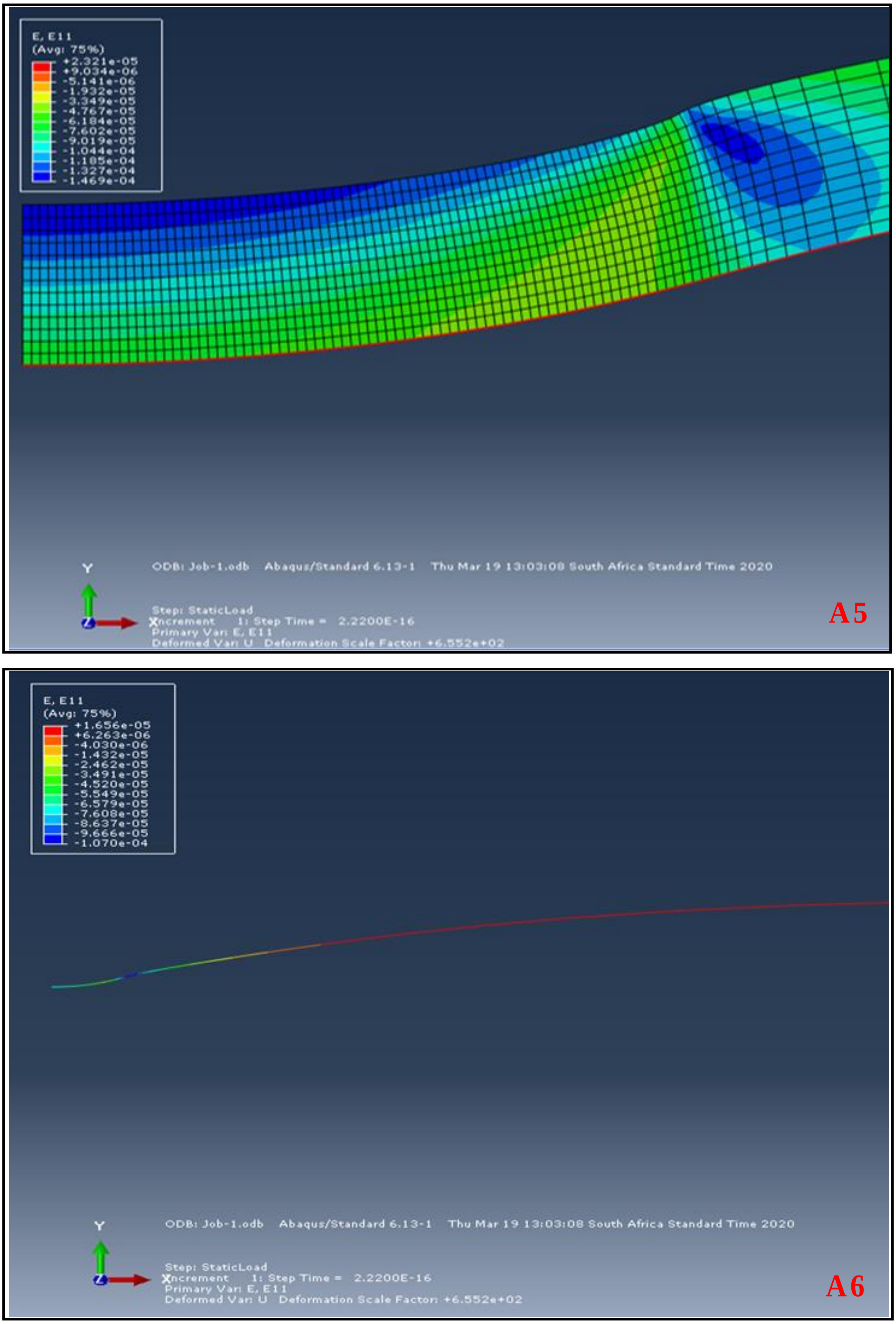
Appendix A2. Numerical Modeling of 100 mm HMA Pavement Structure Using mePADS software



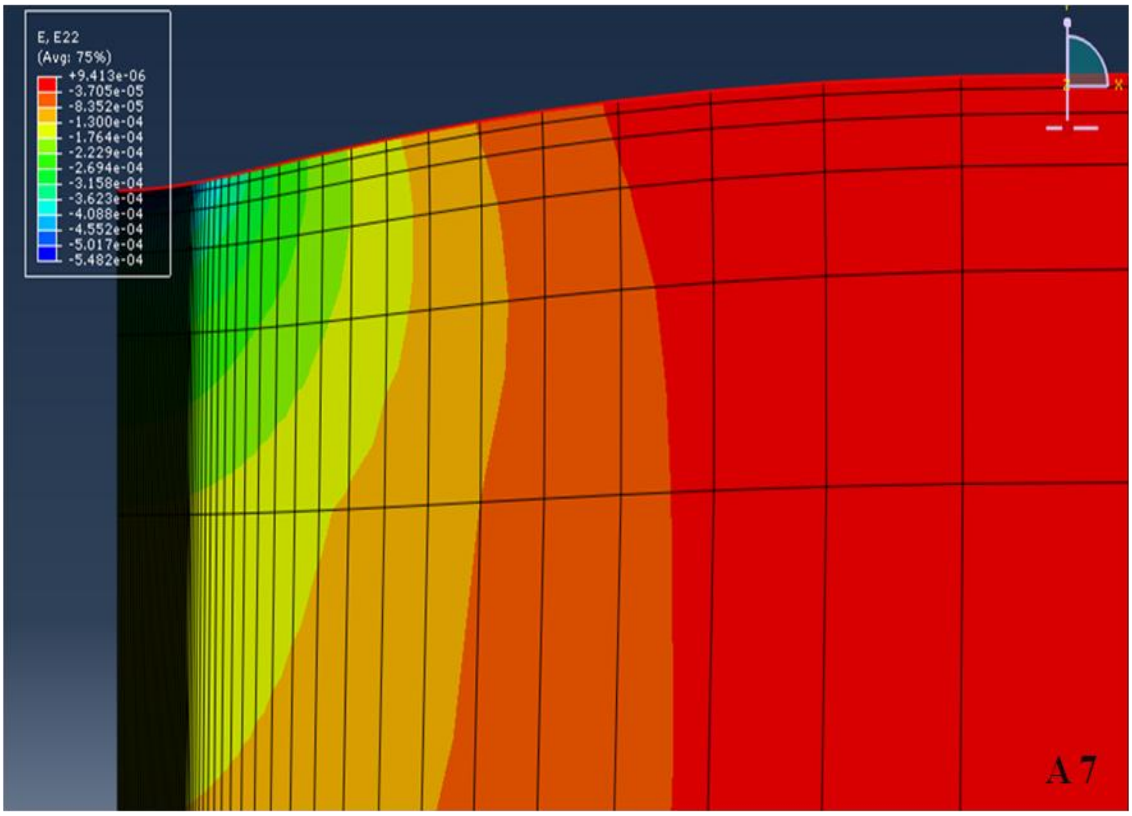
Appendix A3. Numerical Modeling of 150 mm HMA Pavement Structure Using mePADS software

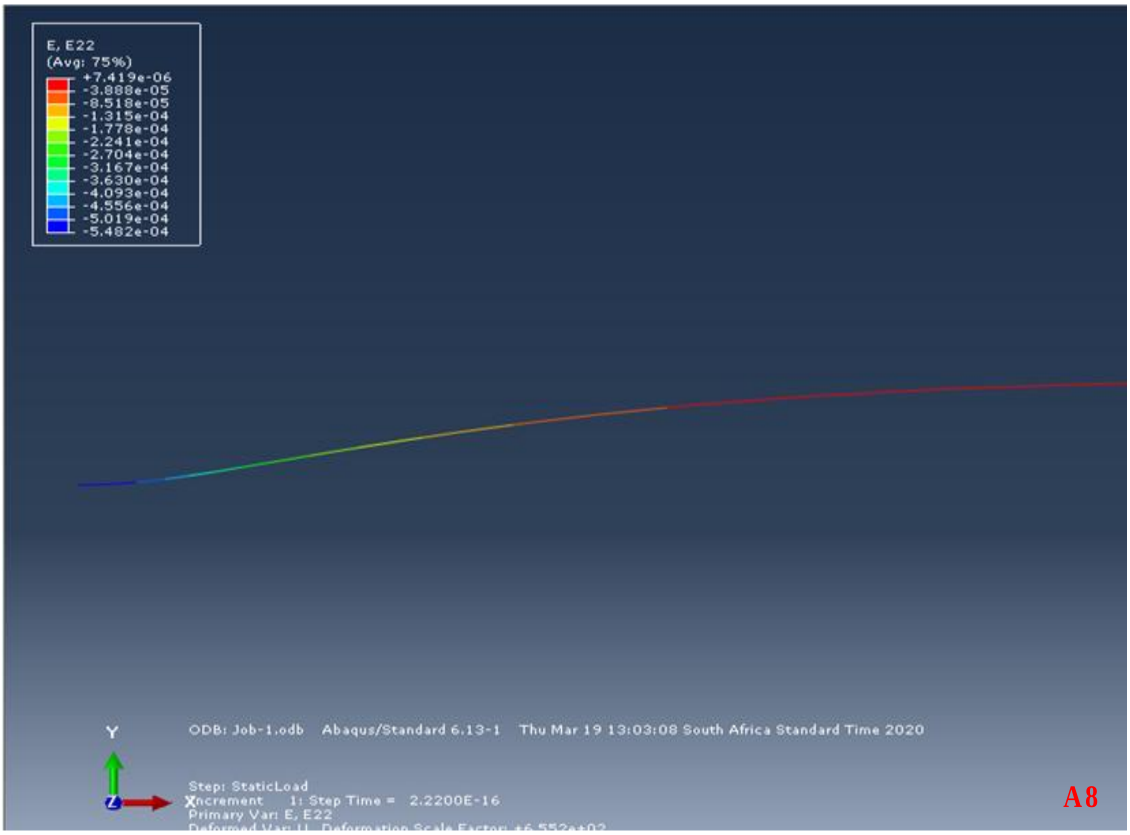


Appendix A4. Tyre Path on Control HMA Pavement

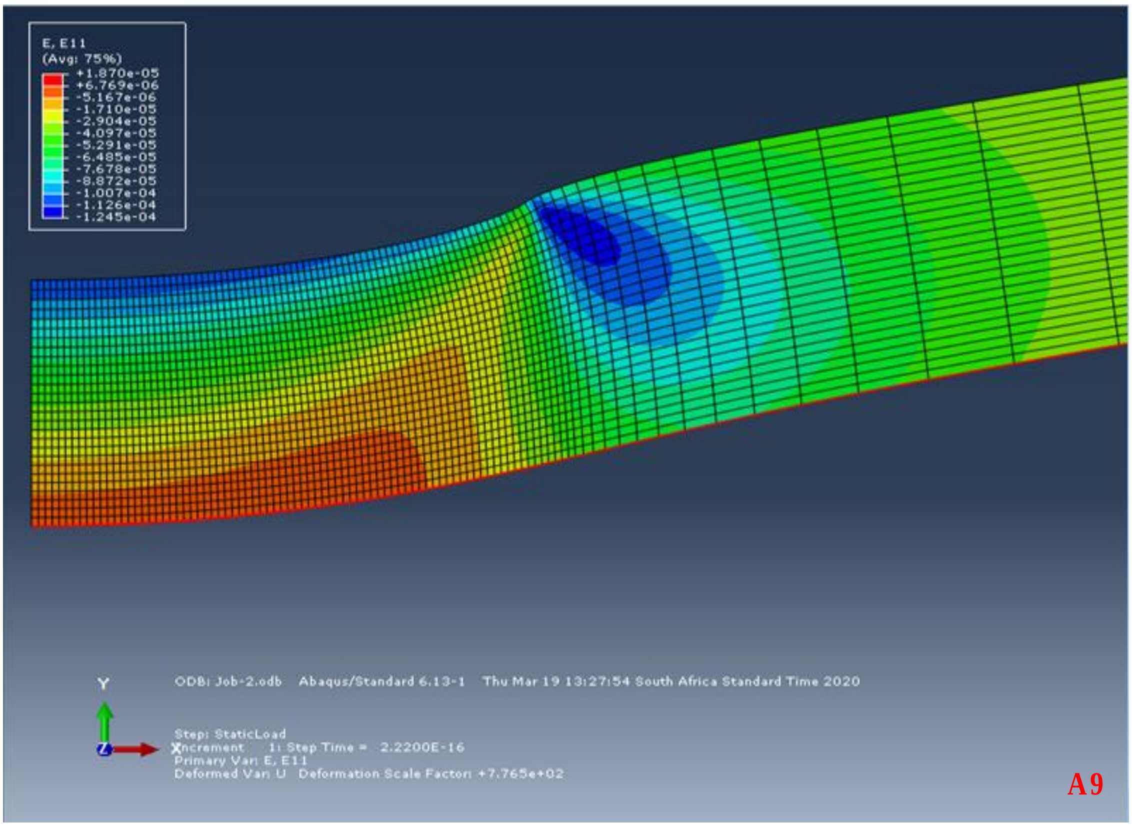


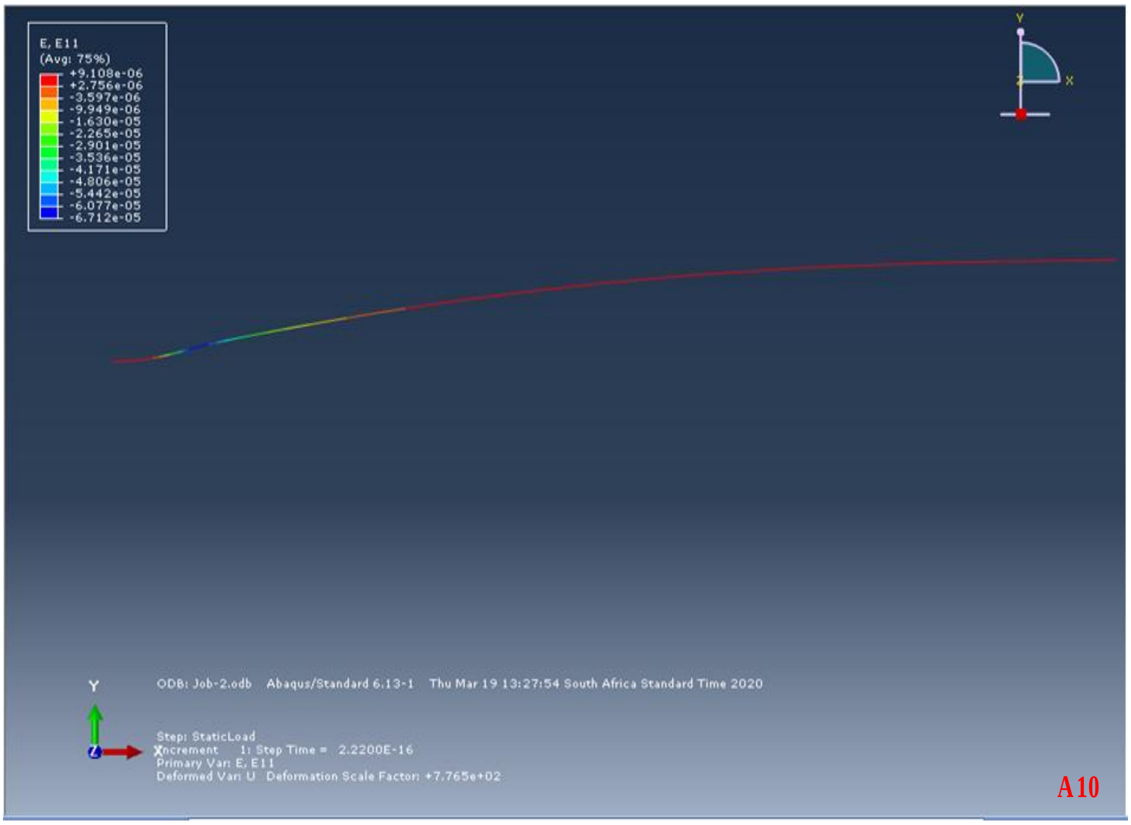
Appendix A5 – A6. Horizontal Tensile Strain at the Bottom of 50 mm HMA Layer



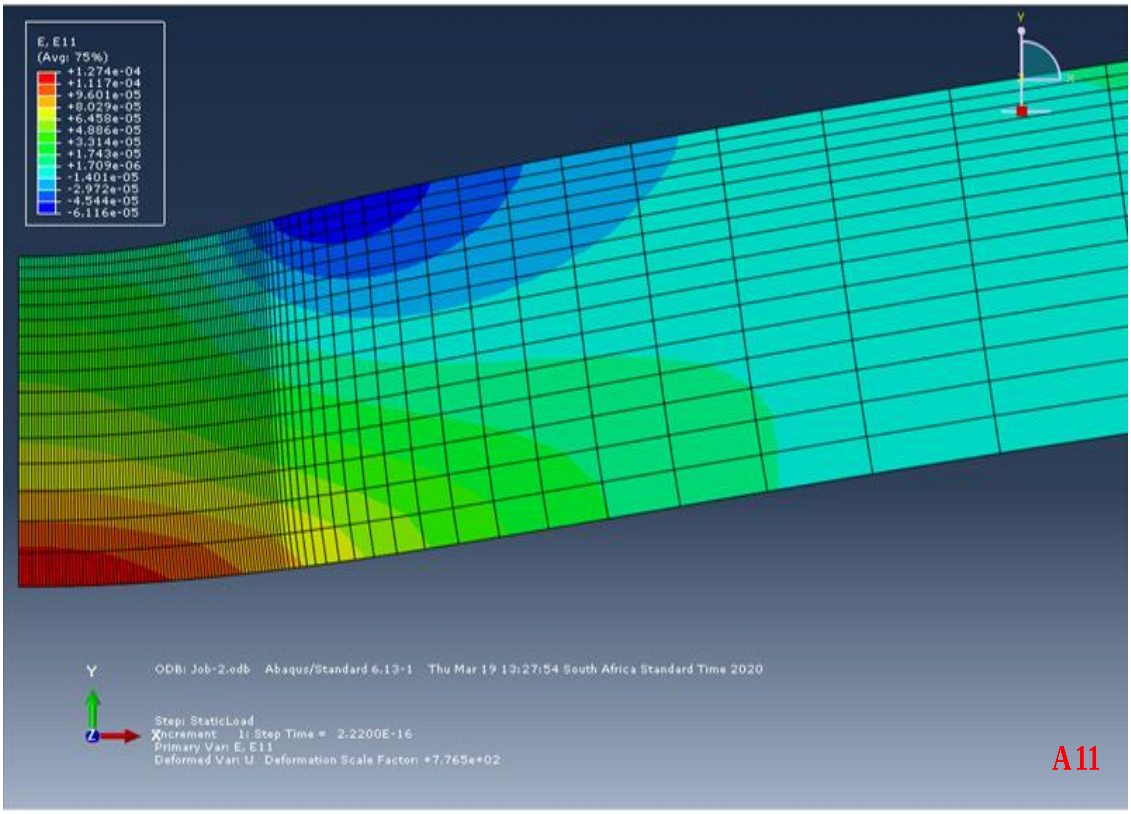


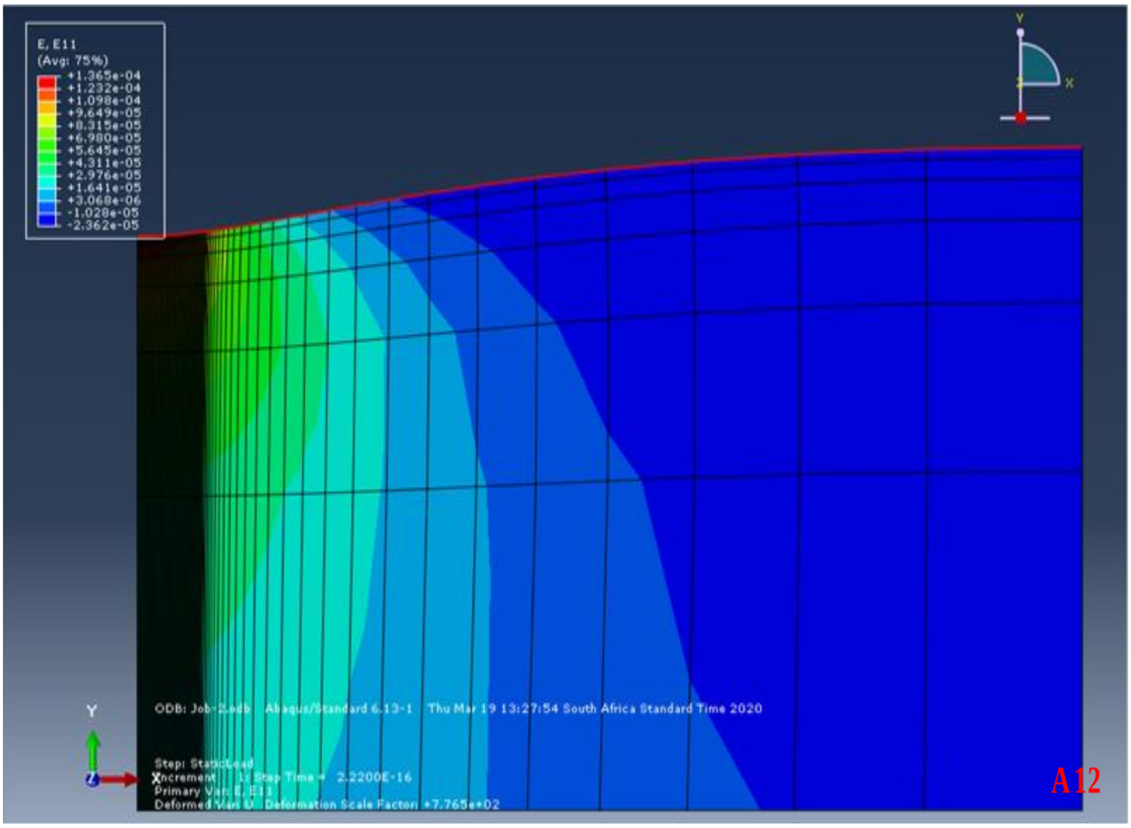
Appendix A7 – A8. Vertical Compressive Strain at the Top of Sub- Grade of 50 mm HMA Pavement Structure



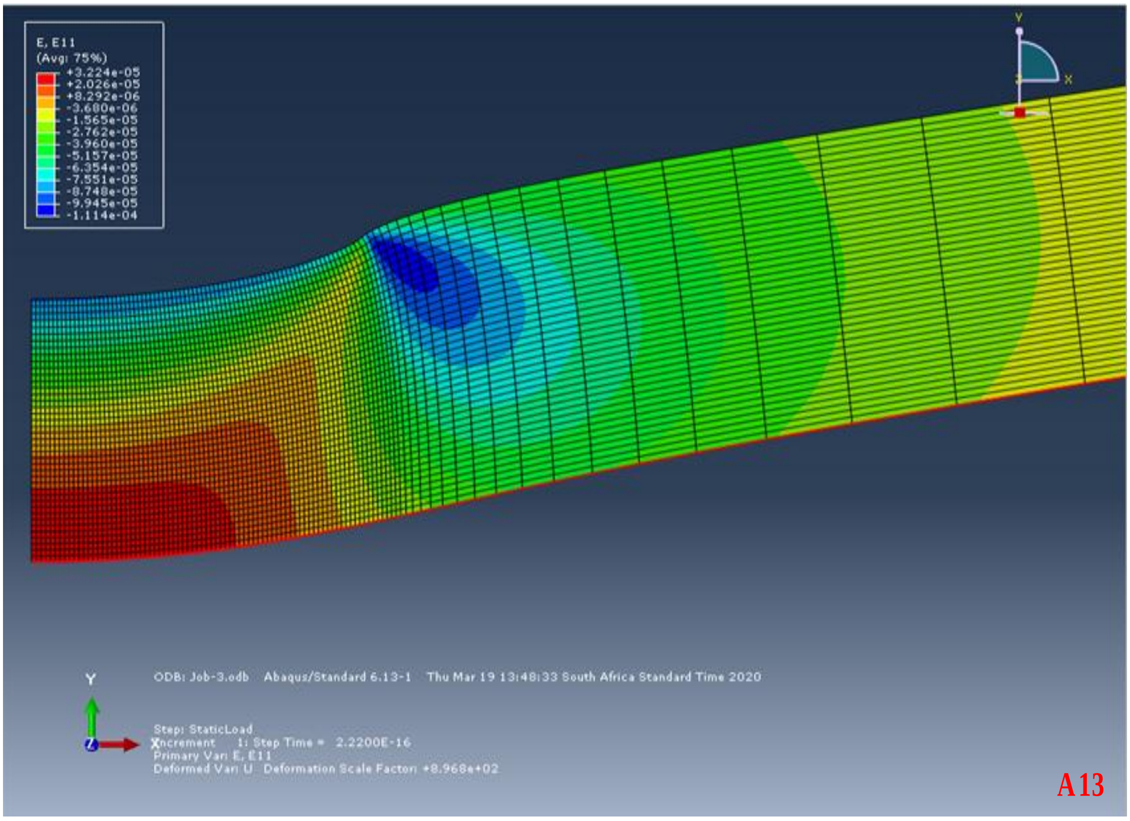


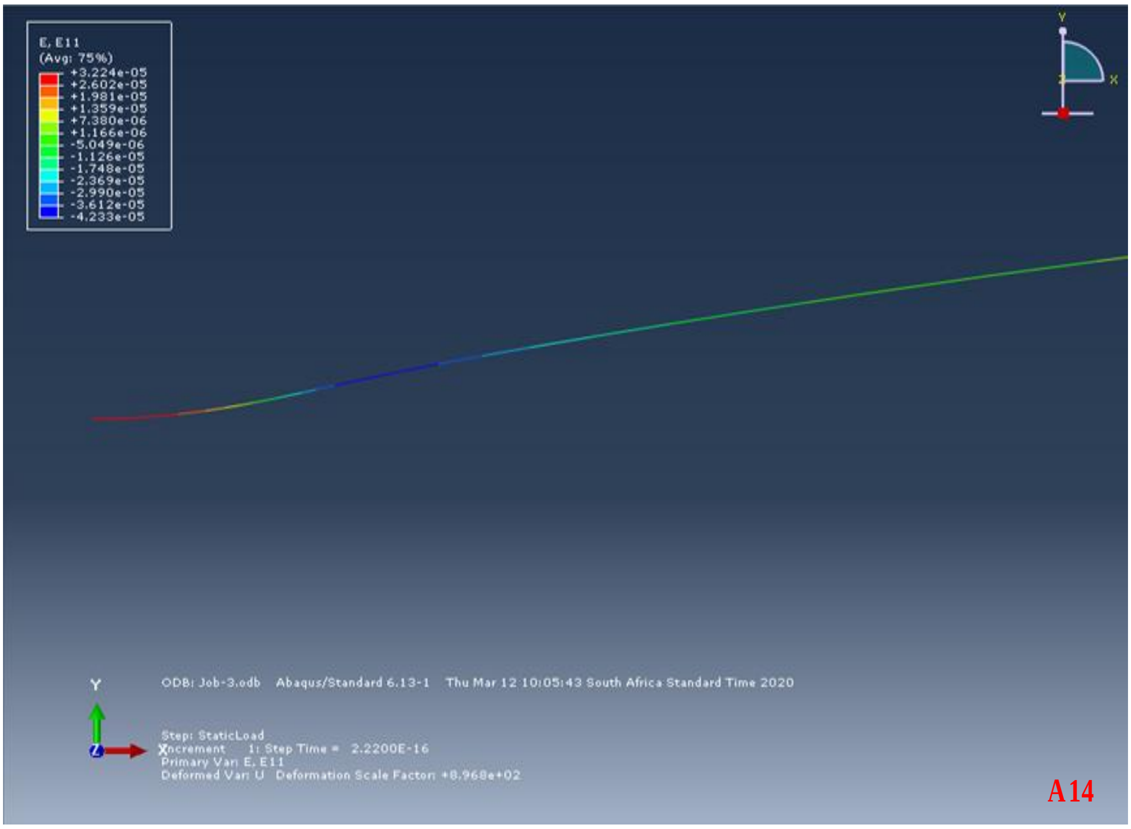
Appendix A9 – A10. Horizontal Tensile Strain at the Bottom of 100 mm HMA Layer



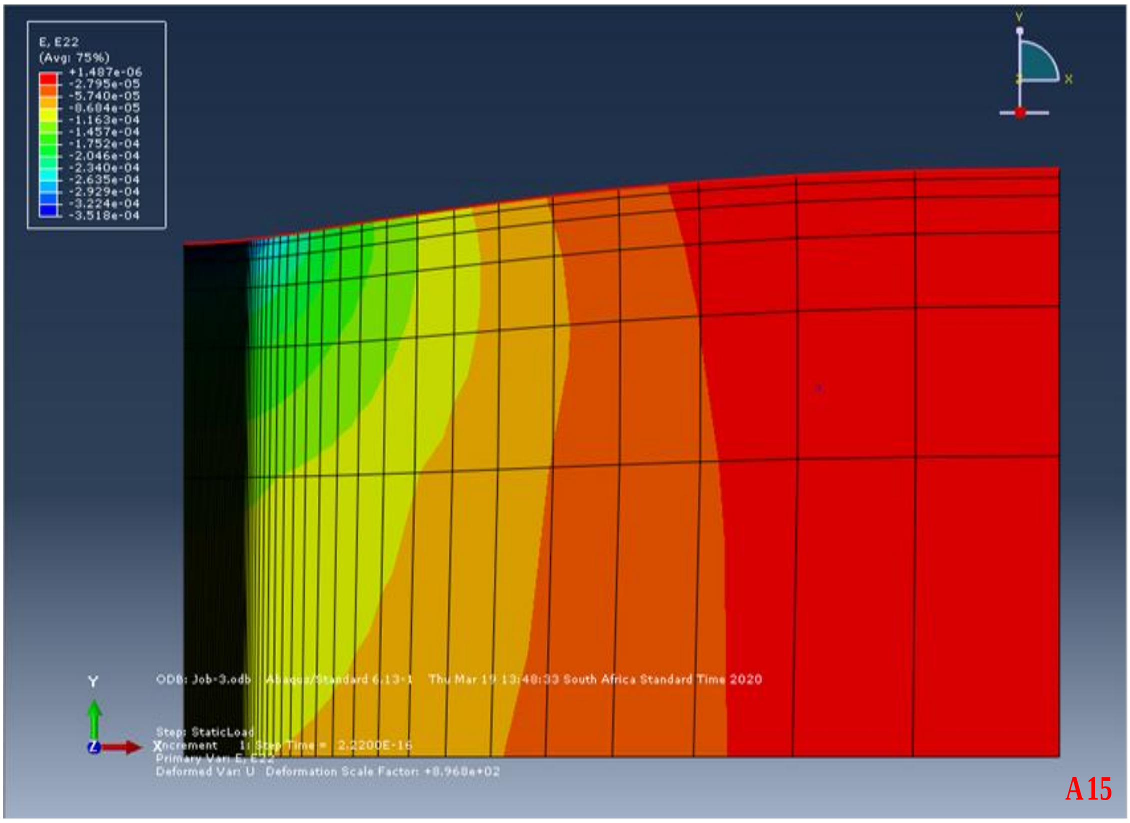


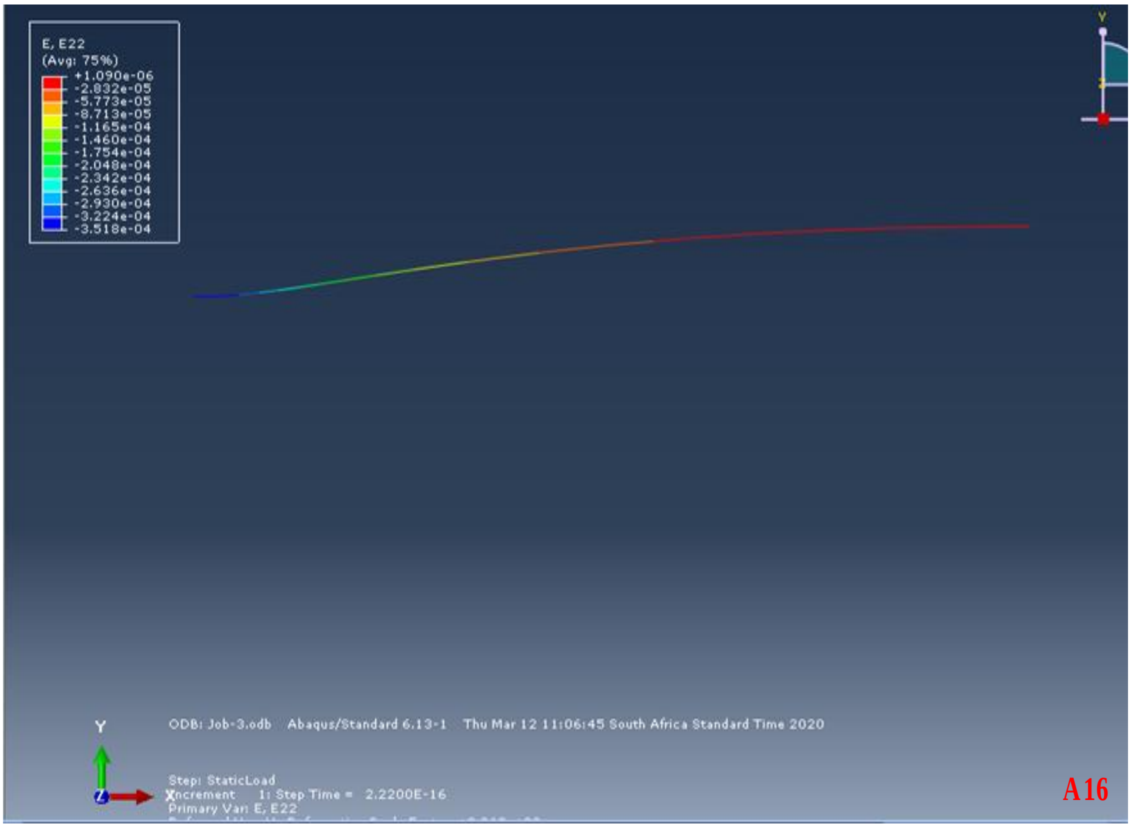
Appendix A11 – A12. Vertical Compressive Strain at the Top of Sub- Grade of 100 mm HMA Pavement Structure



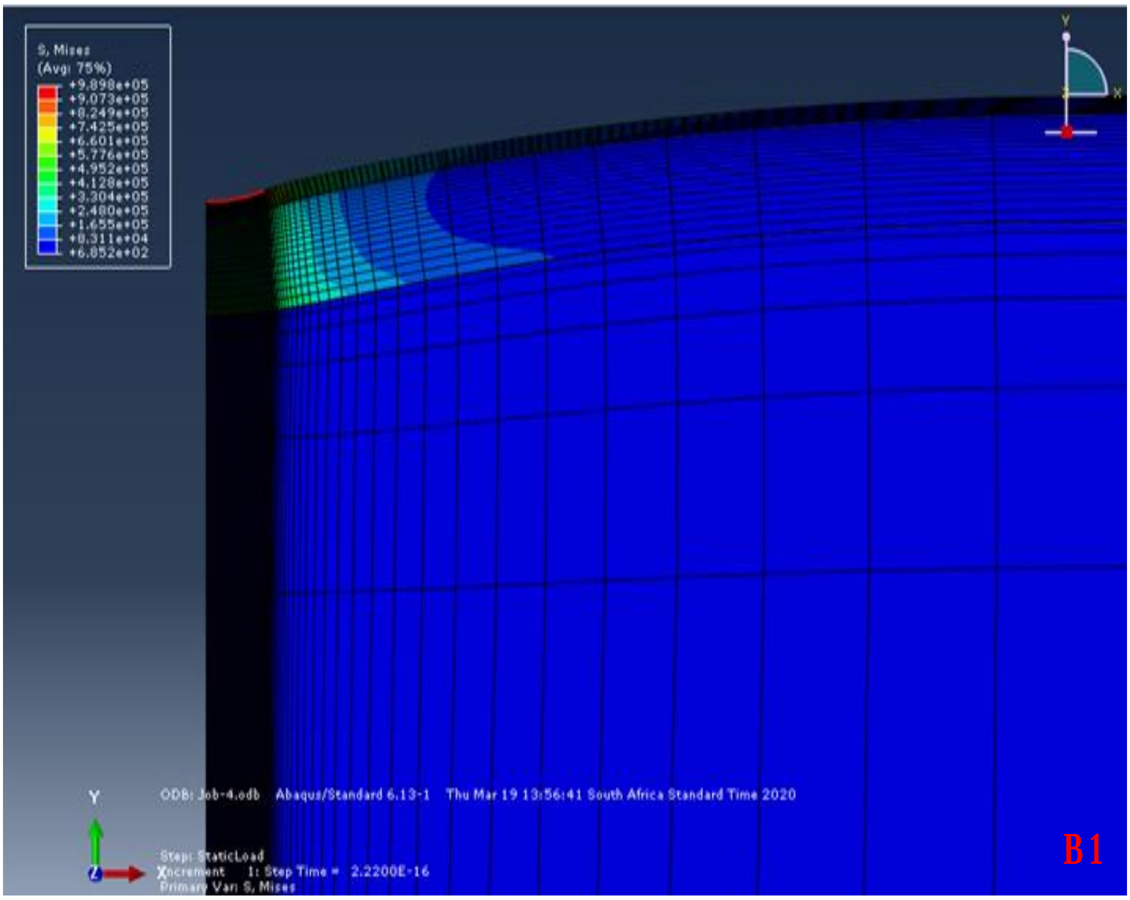


Appendix A13 – A14. Horizontal Tensile Strain at the Bottom of 150 mm HMA Layer

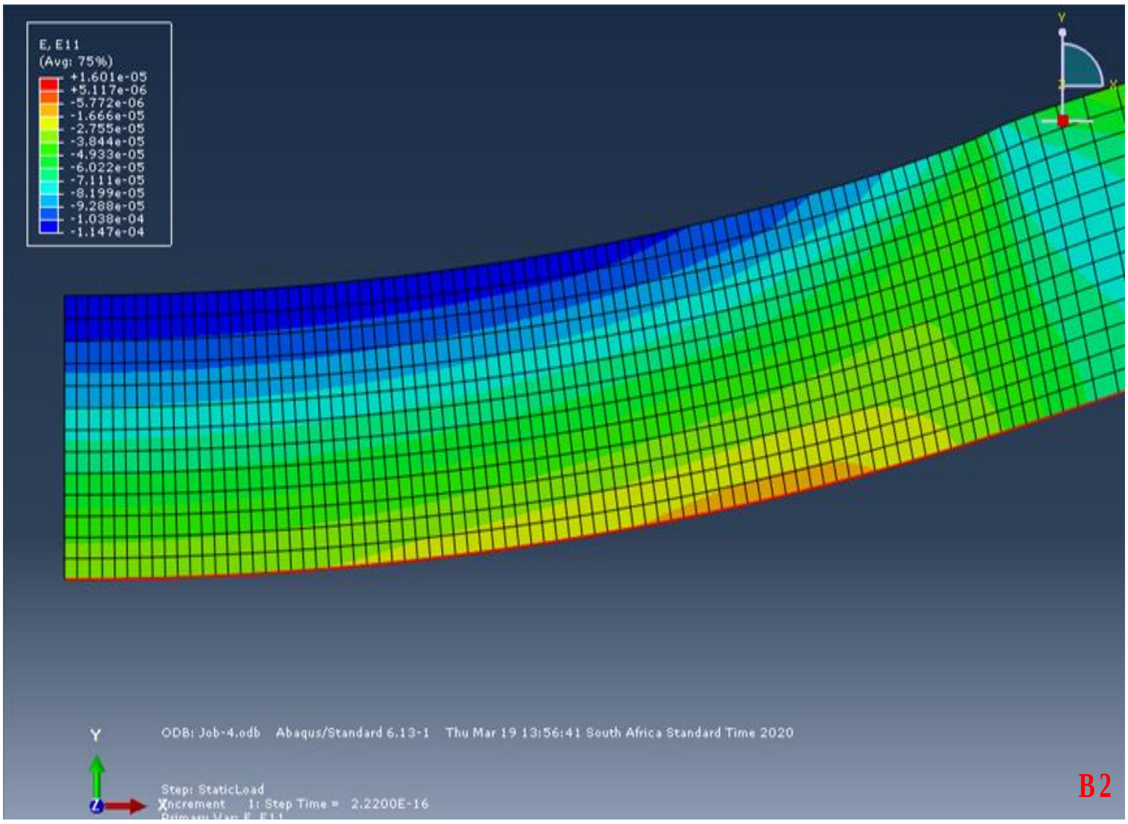


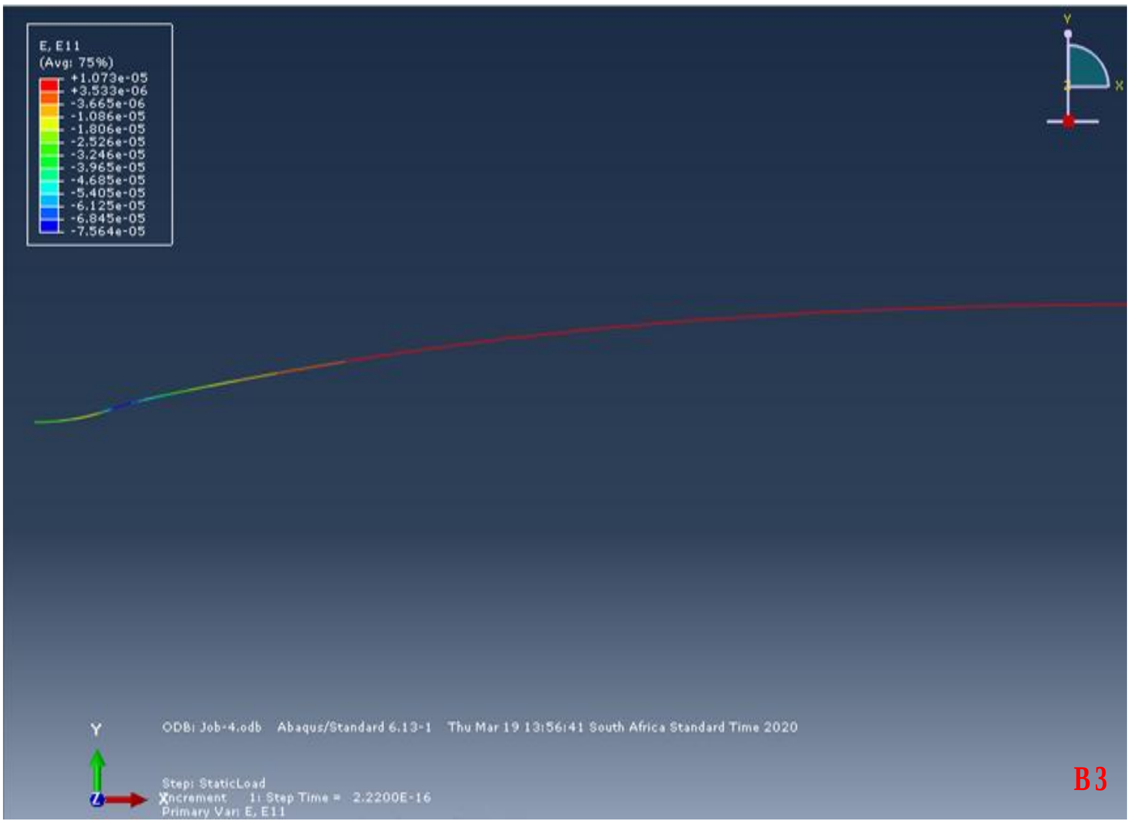


Appendix A15 – A16. Vertical Compressive Strain at the Top of Sub- Grade of 150 mm HMA Pavement Structure

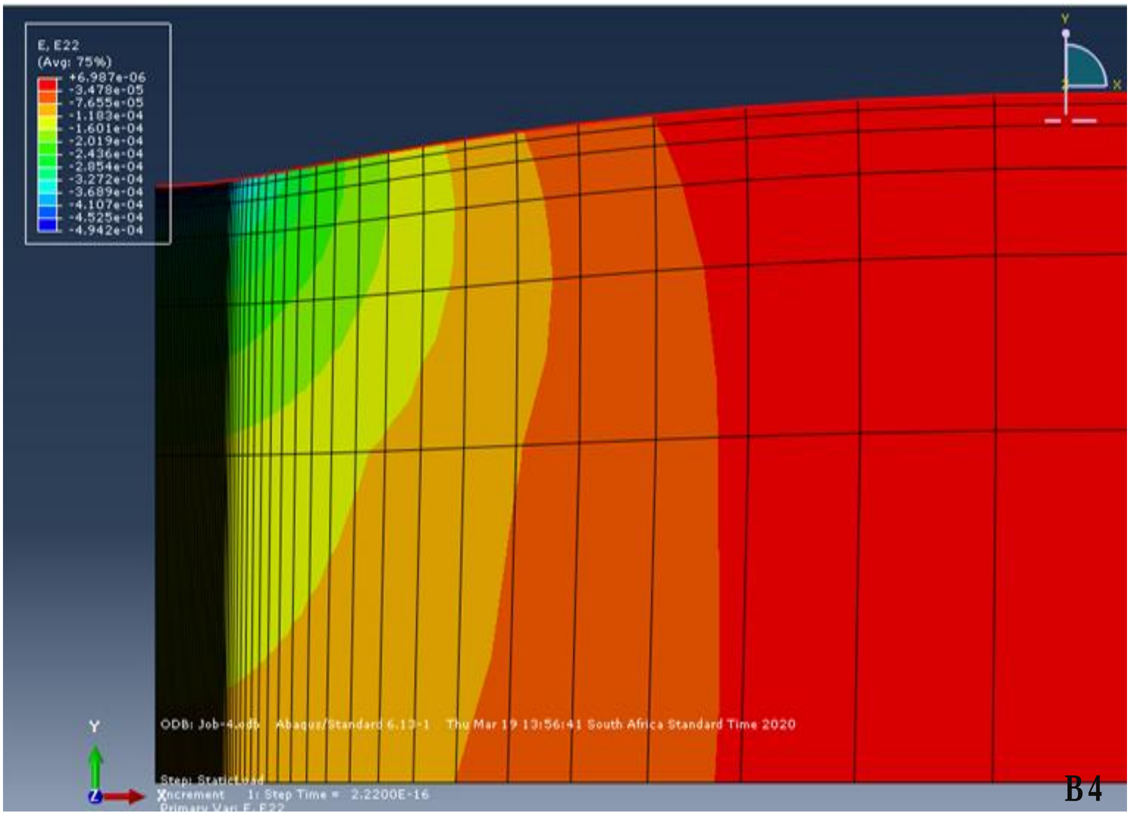


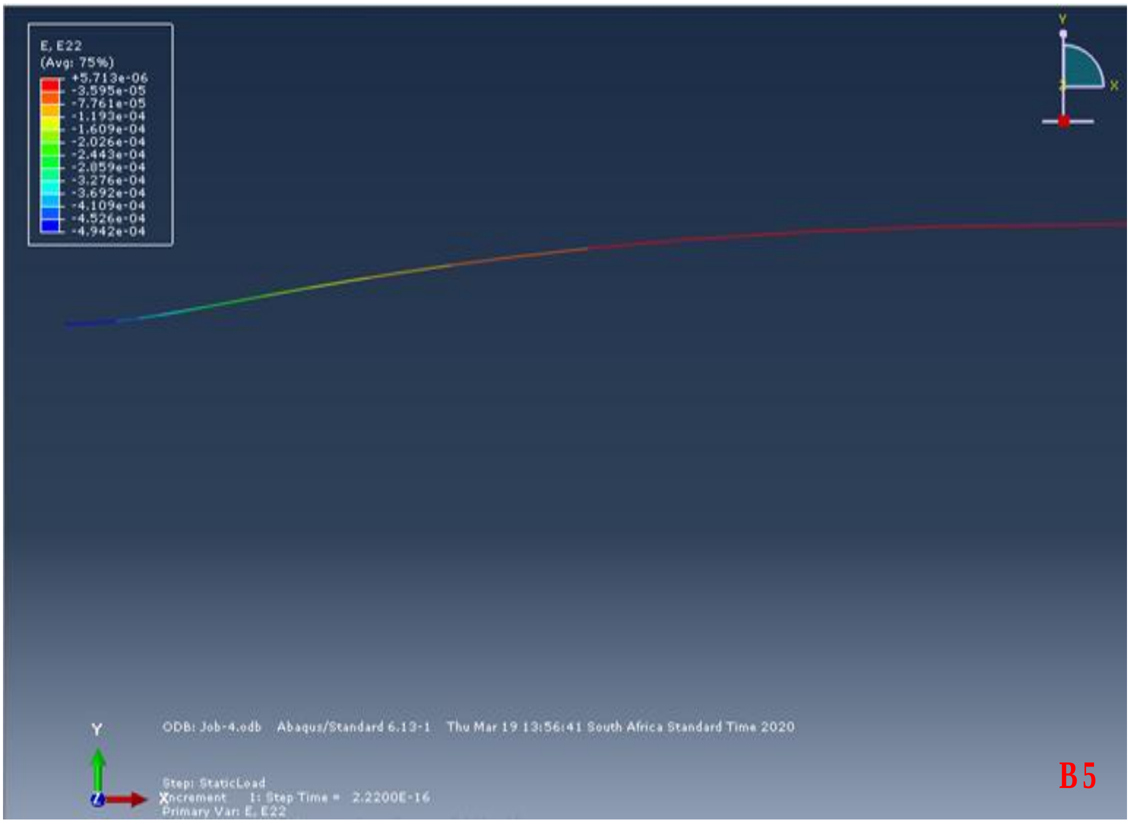
Appendix B1. Tyre Path on WMA – 15% RAP Pavement



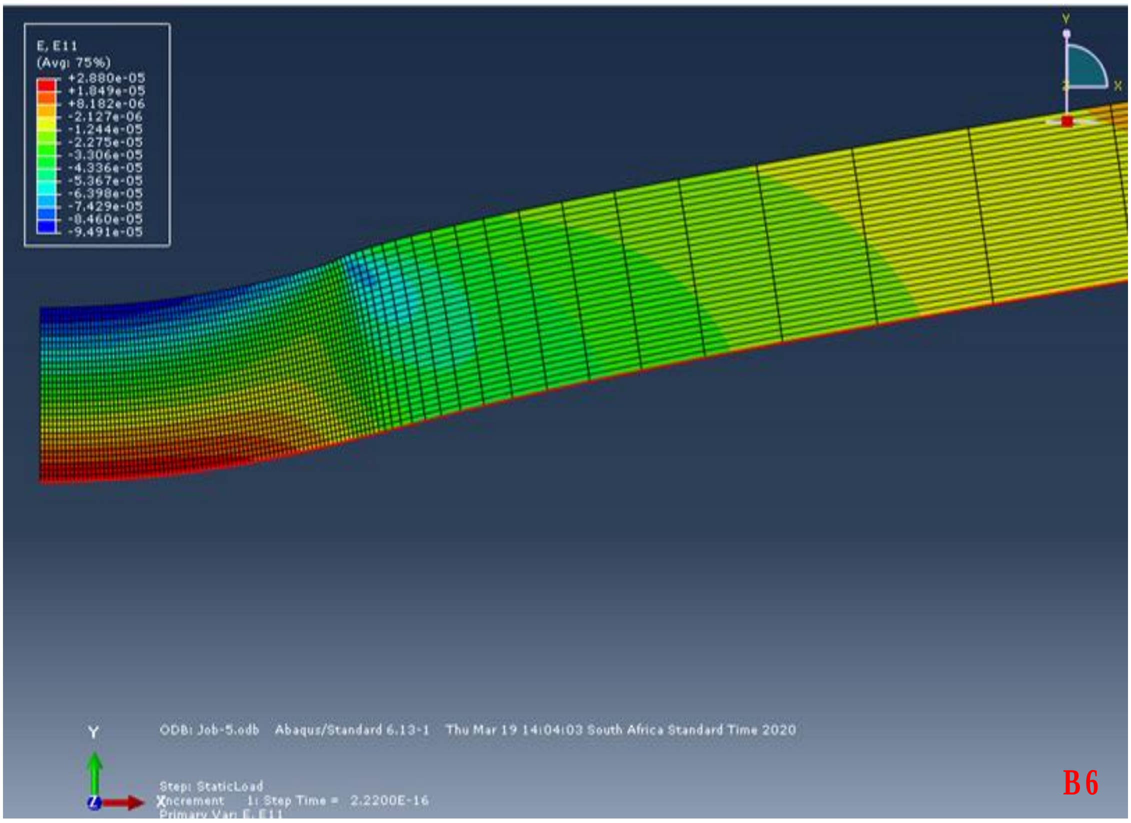


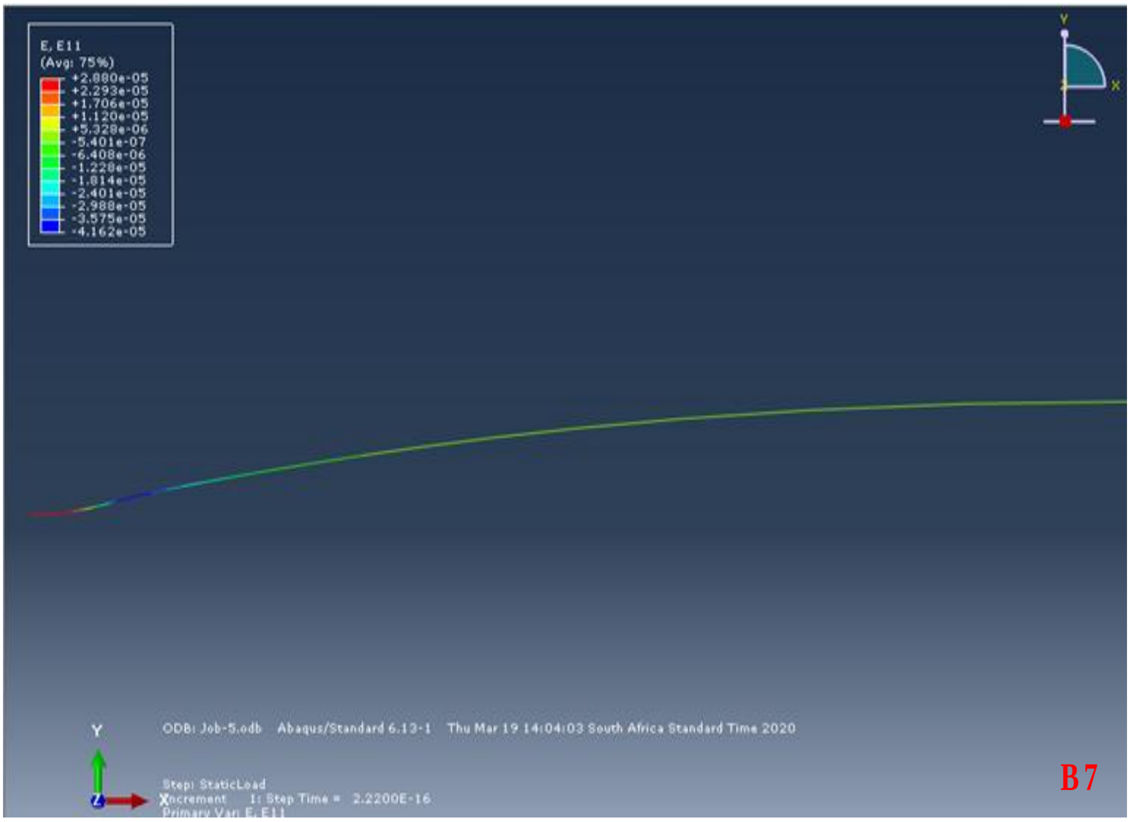
Appendix B2 – B3. Horizontal Tensile Strain at the Bottom of 50 mm WMA – 15% RAP Layer



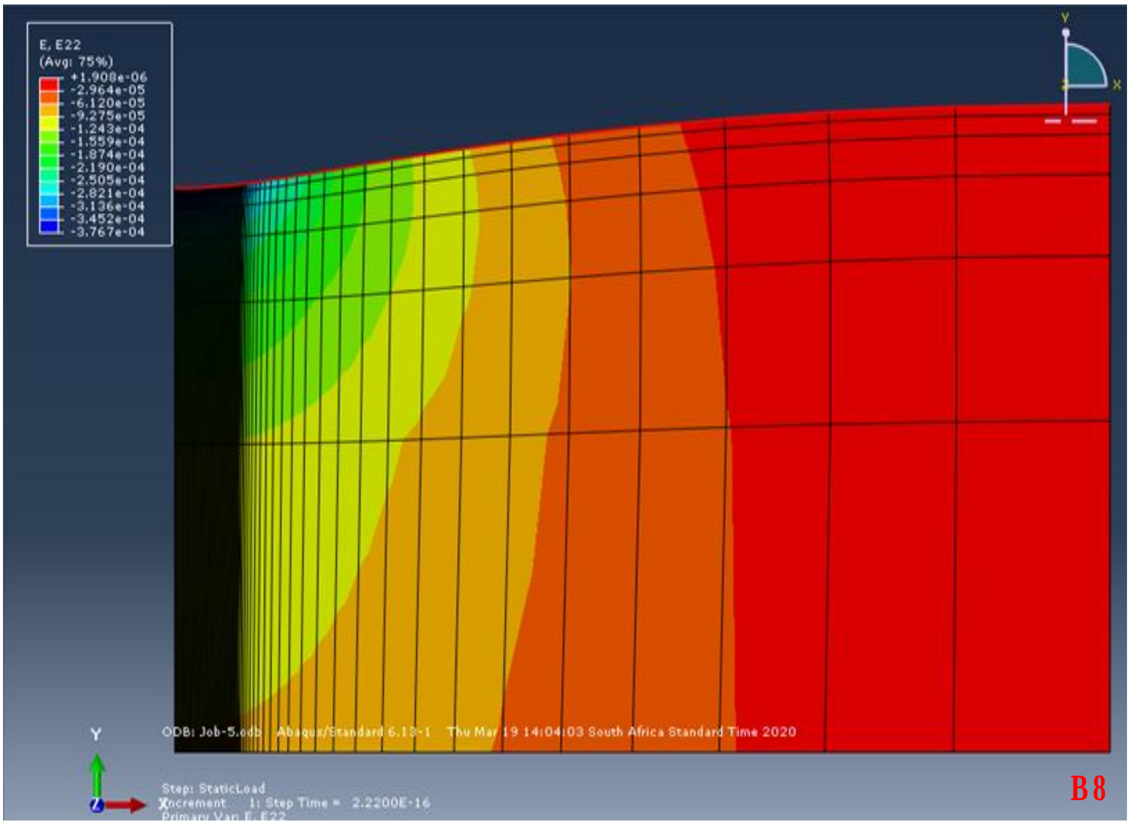


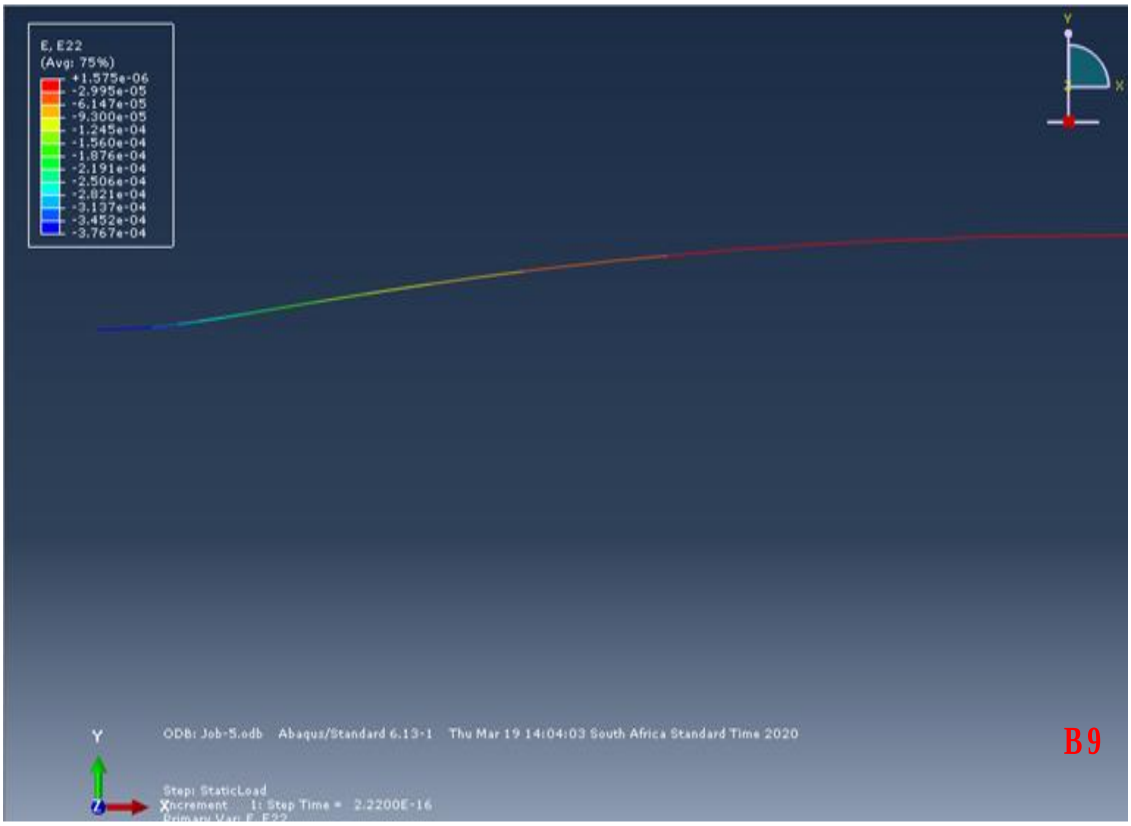
Appendix B4 – B5. Vertical Compressive Strain at the Top of Sub- Grade of 50 mm WMA – 15% RAP Pavement Structure



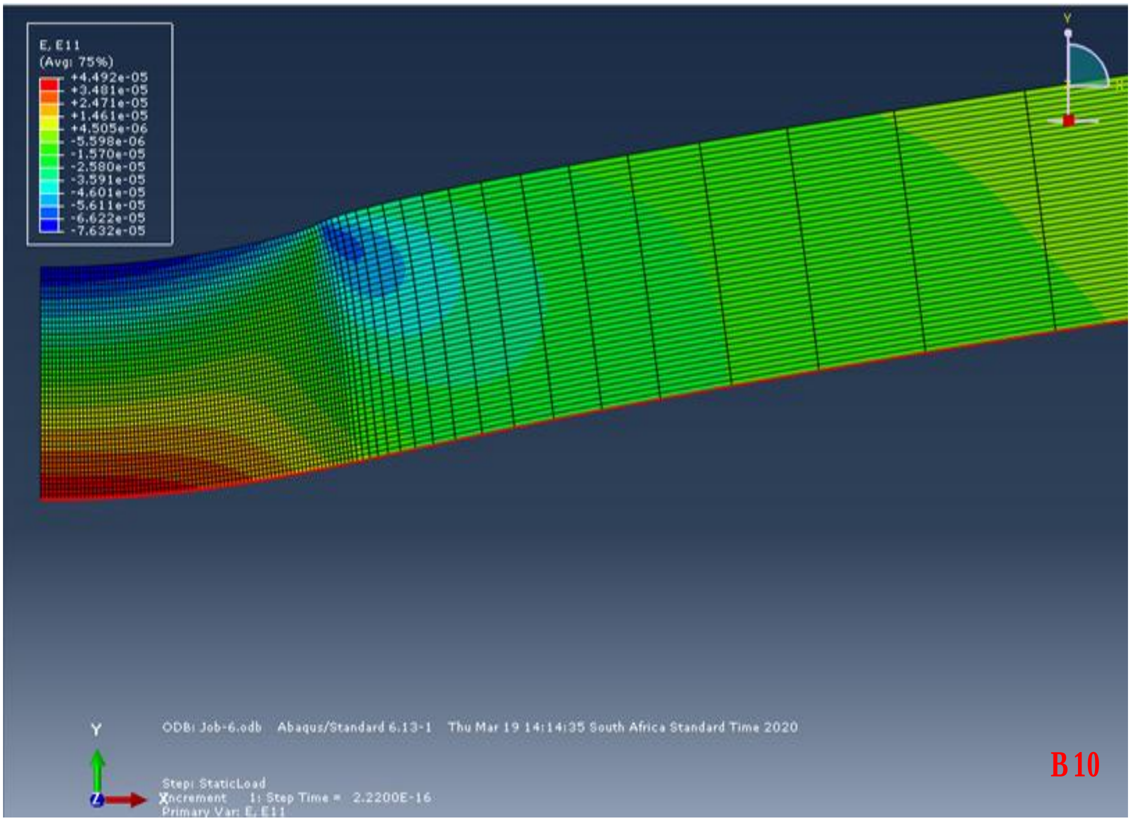


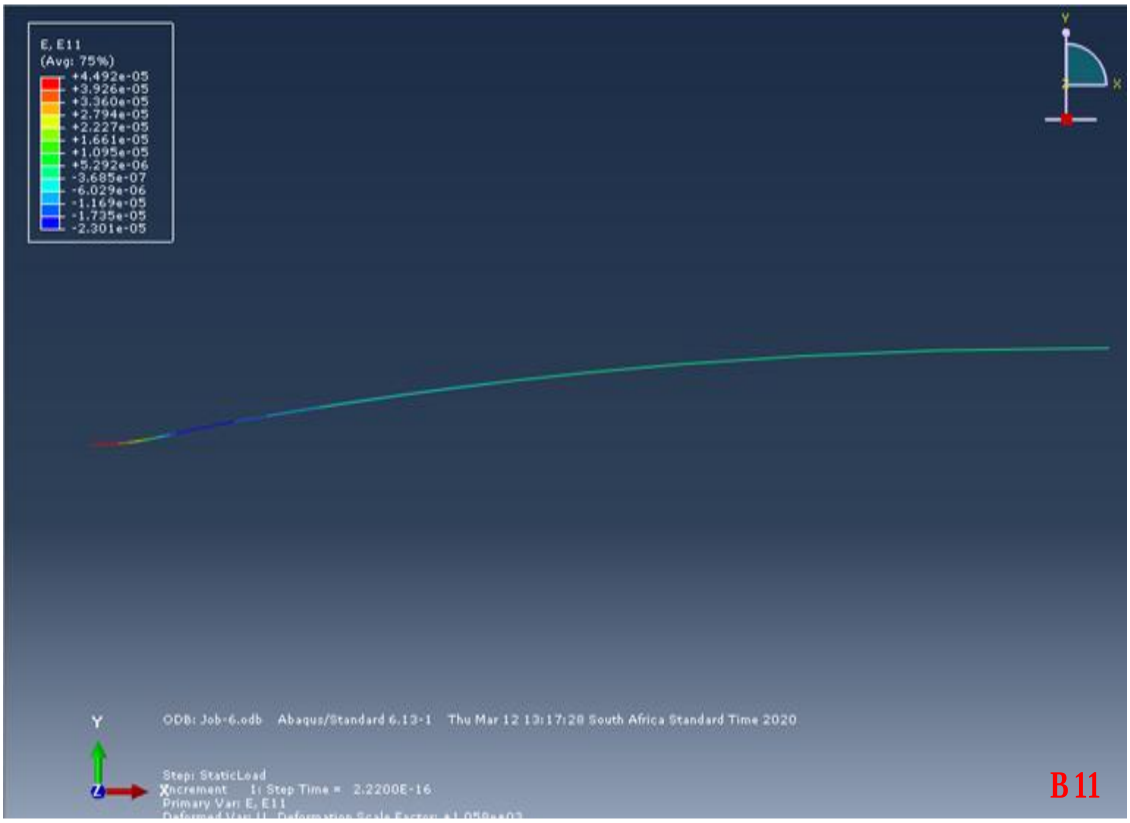
Appendix B6 – B7. Horizontal Tensile Strain at the Bottom of 100 mm WMA – 15% RAP Layer



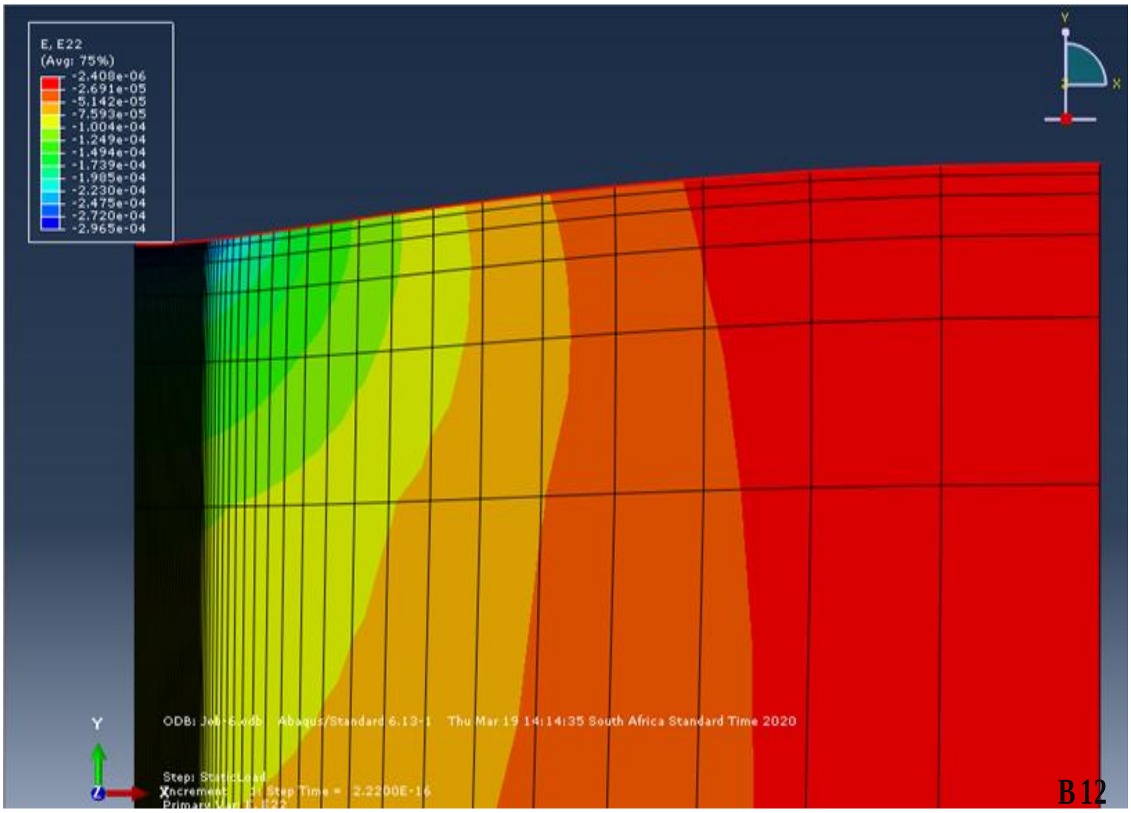


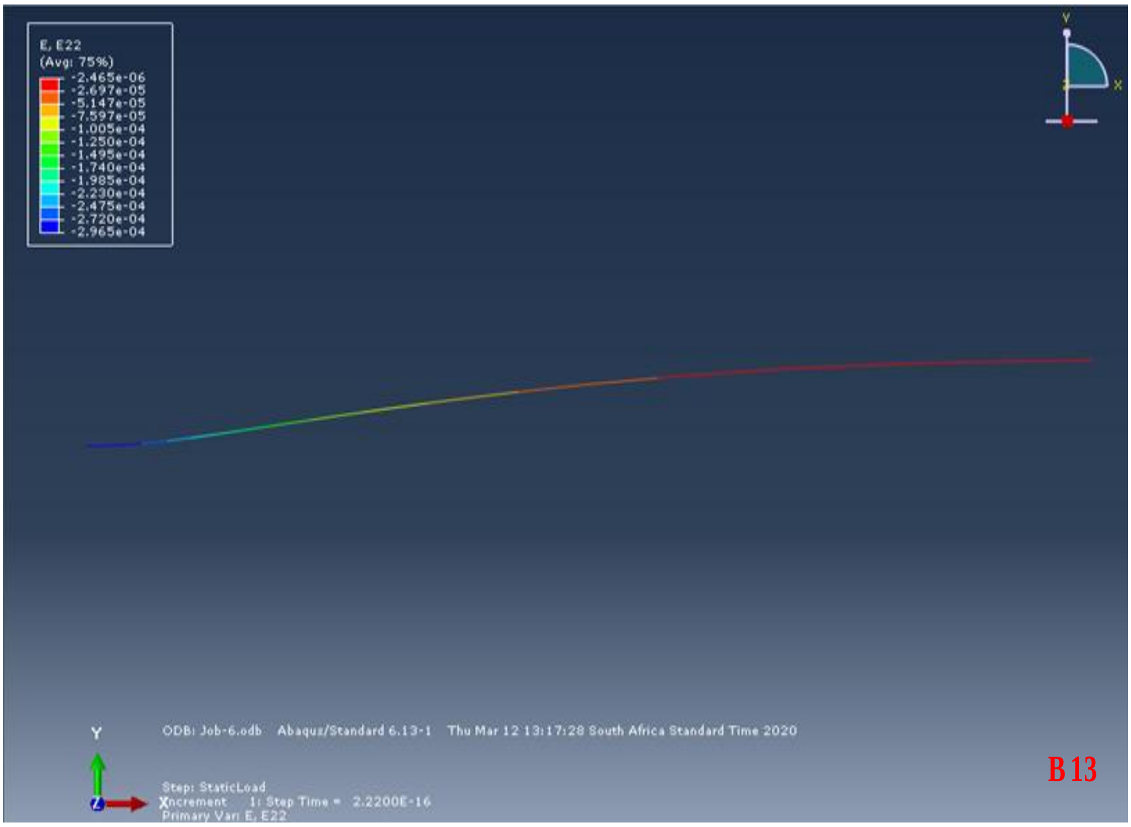
Appendix B8 – B9. Vertical Compressive Strain at the Top of Sub- Grade of 100 mm WMA – 15% RAP Pavement Structure



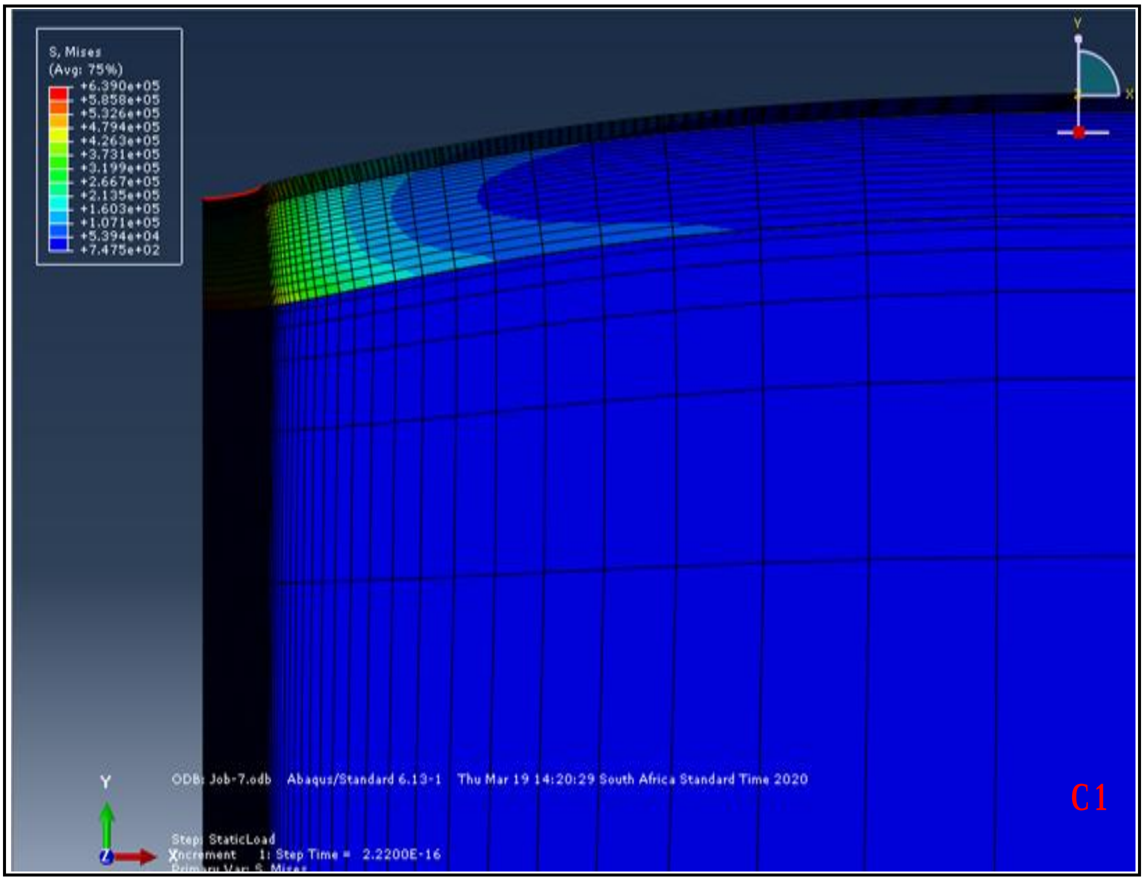


Appendix B10 – B11. Horizontal Tensile Strain at the Bottom of 150 mm WMA – 15% RAP Layer

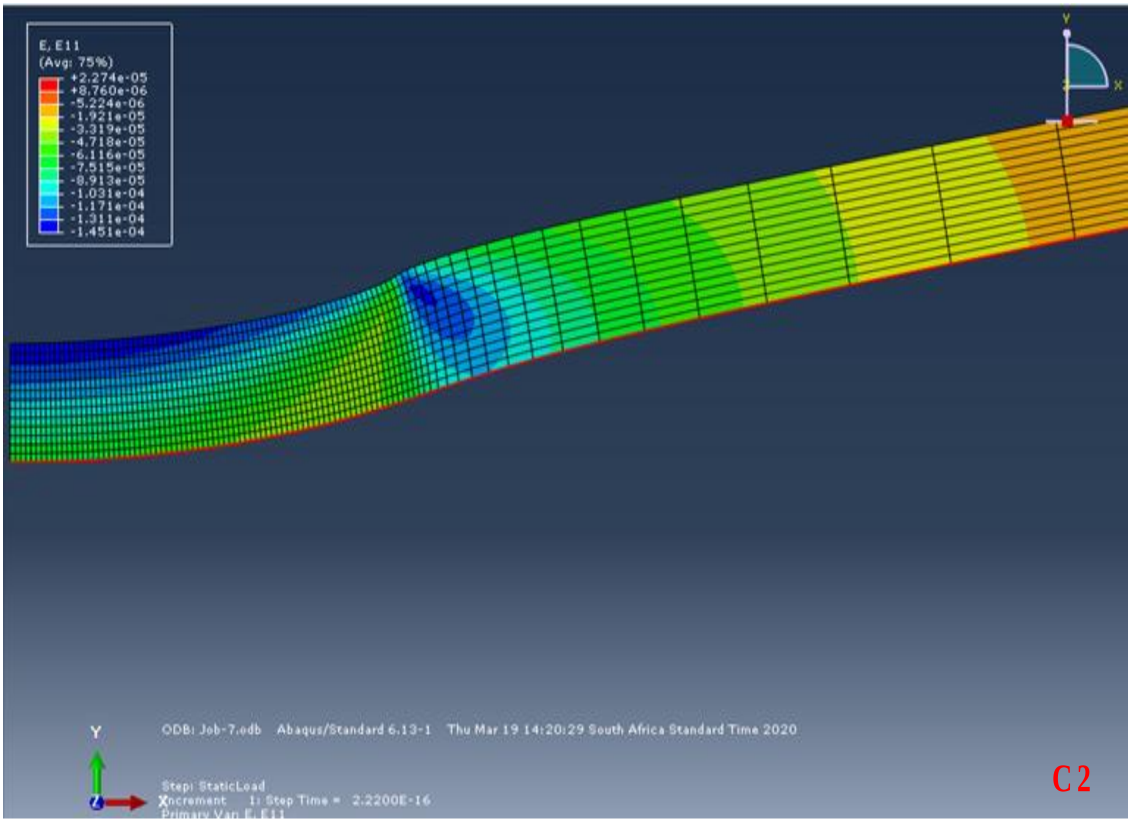


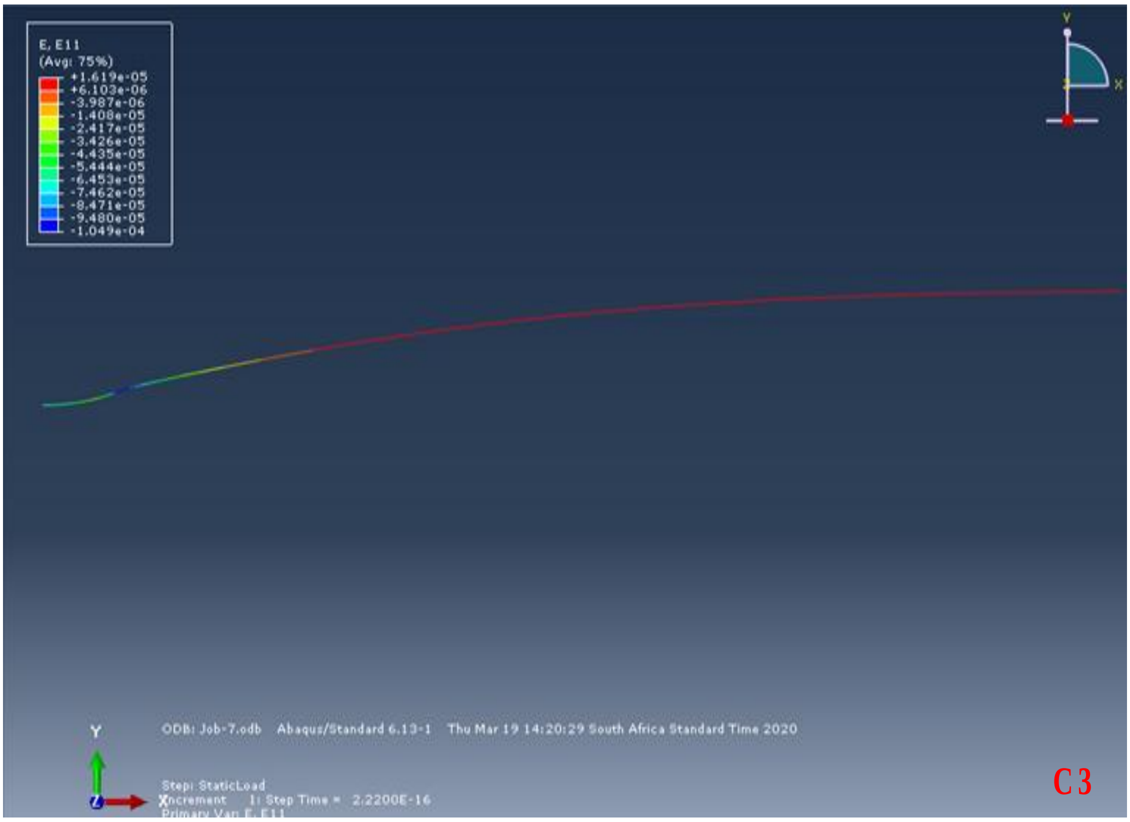


Appendix B12 – B13. Vertical Compressive Strain at the Top of Sub- Grade of 150 mm WMA – 15% RAP Pavement Structure



Appendix C1. Tyre Path on WMA – 30% RAP Pavement





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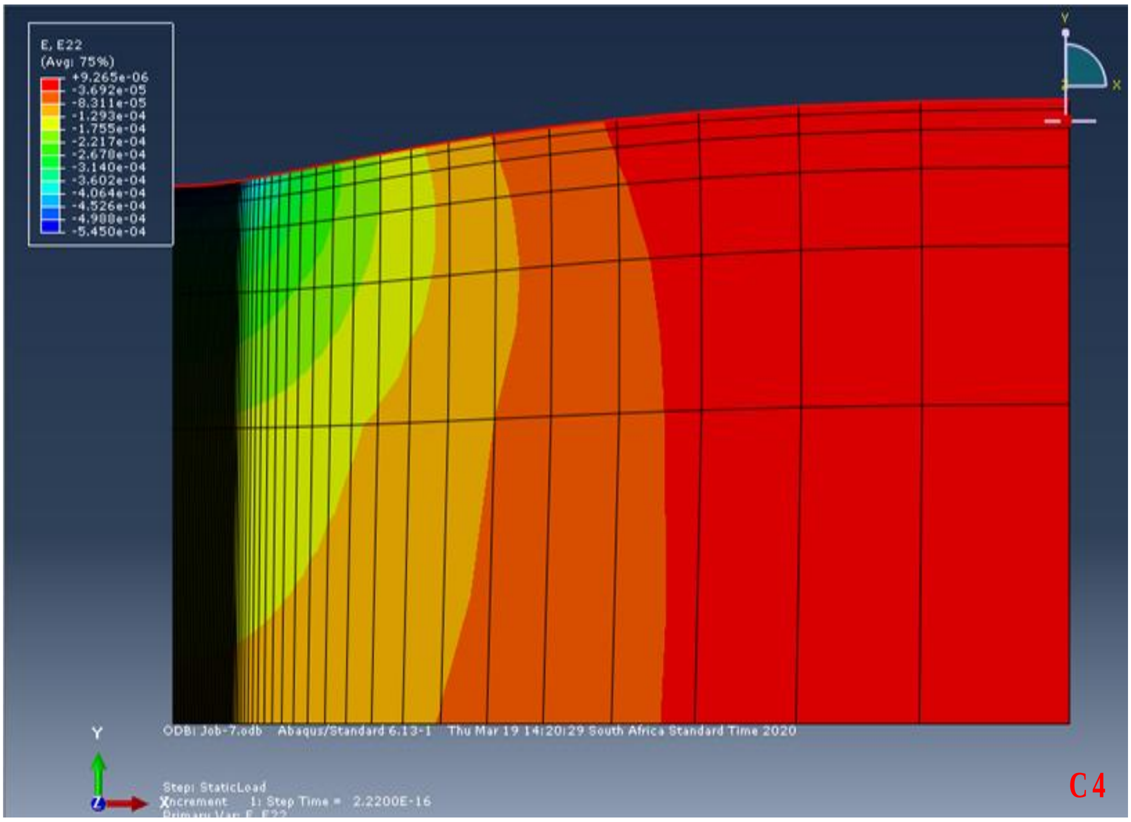

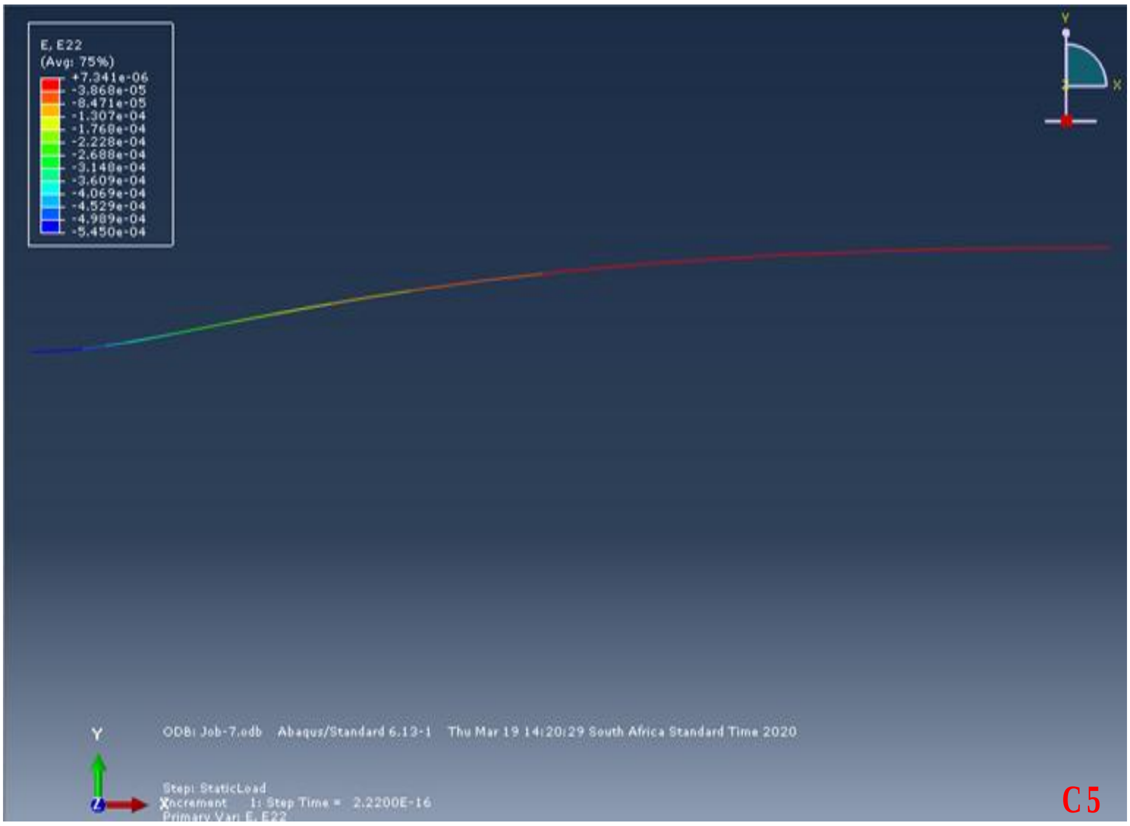

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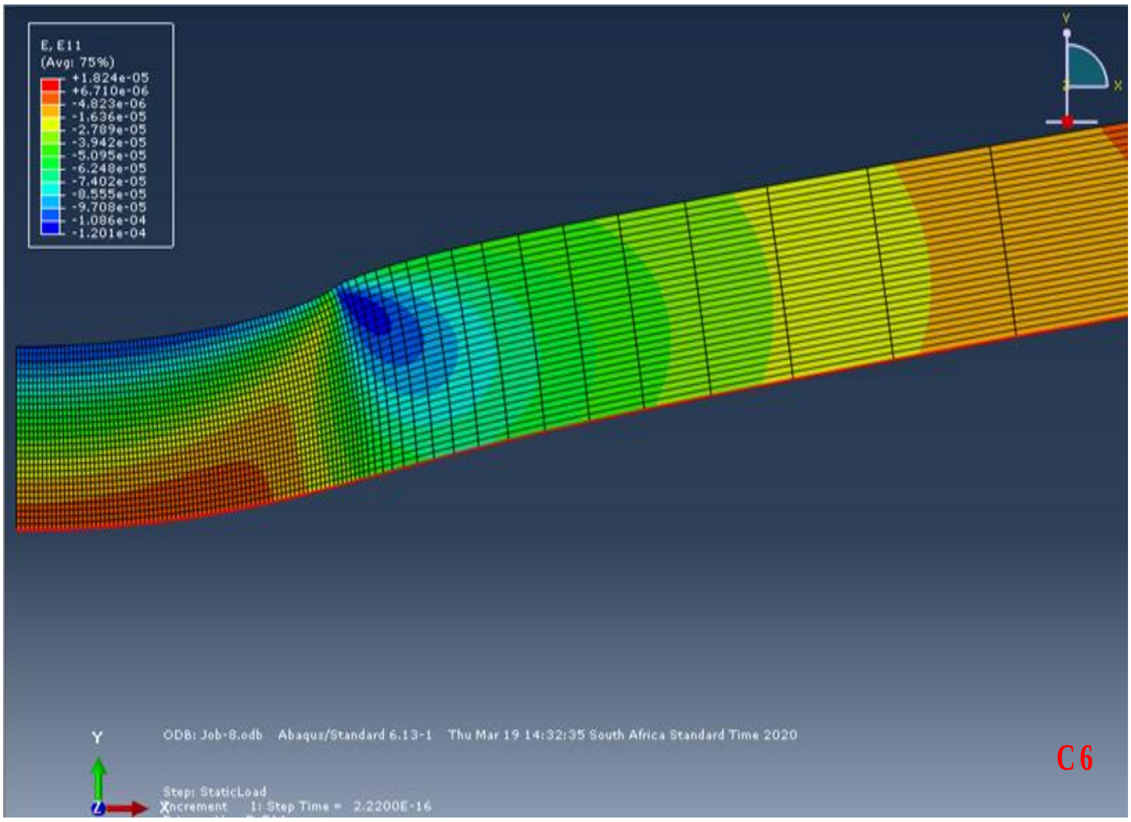

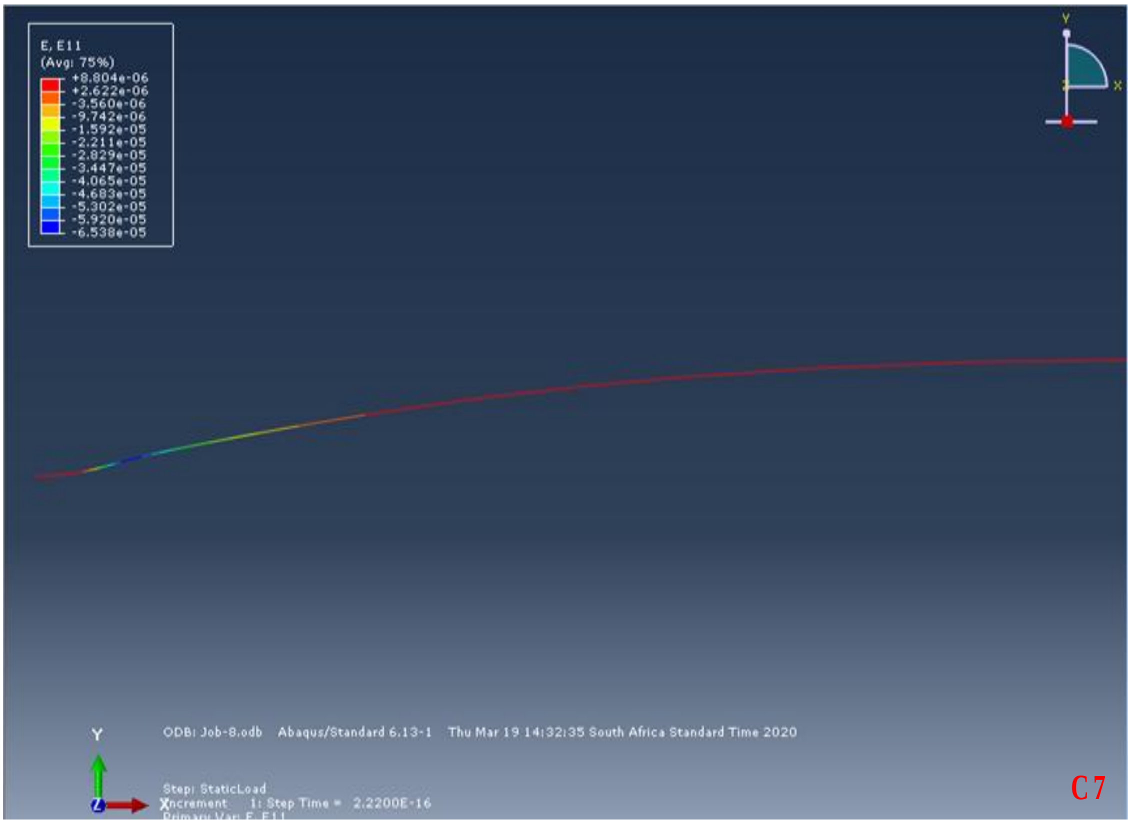

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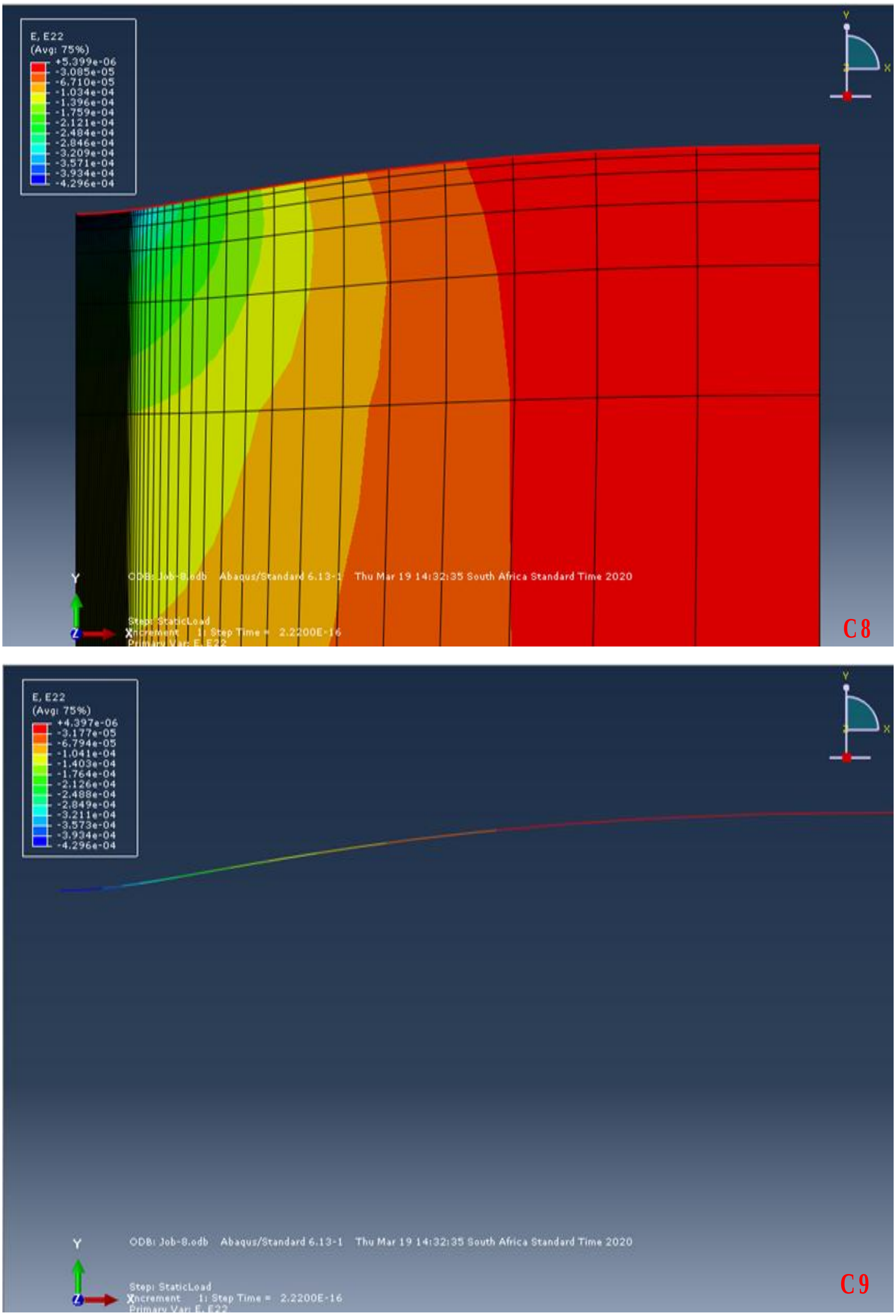

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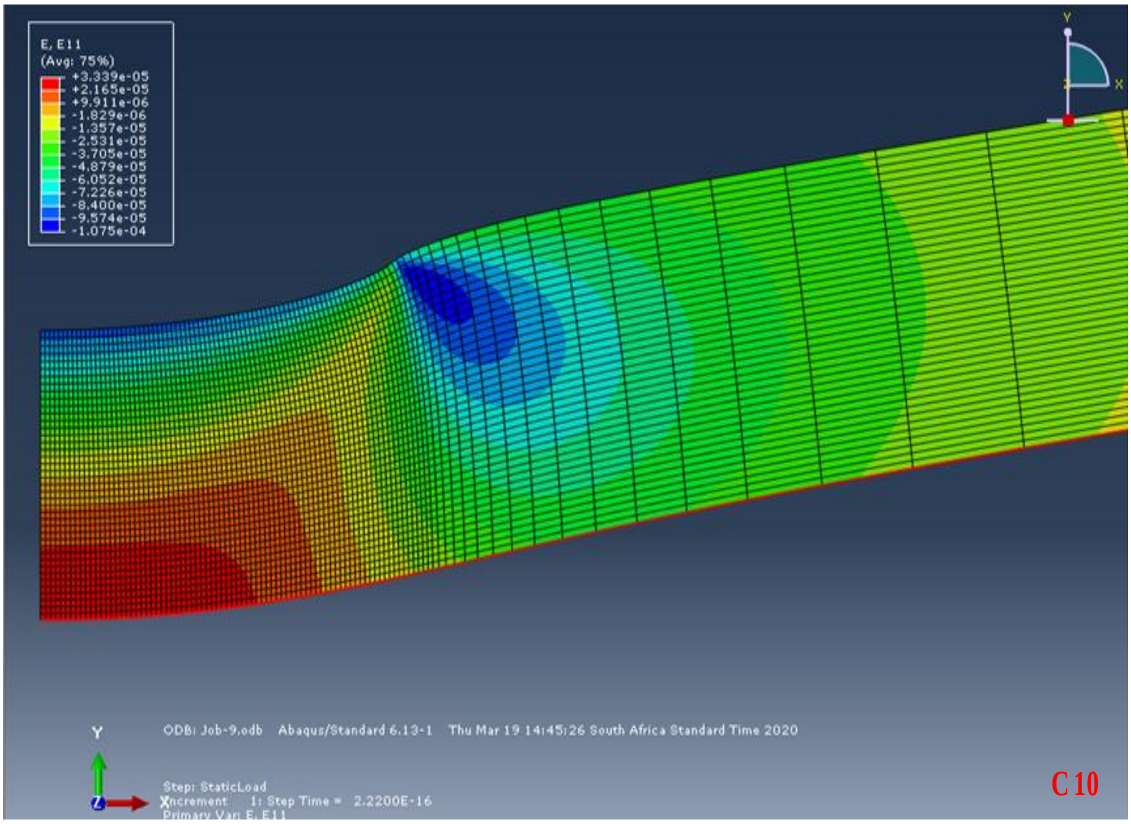
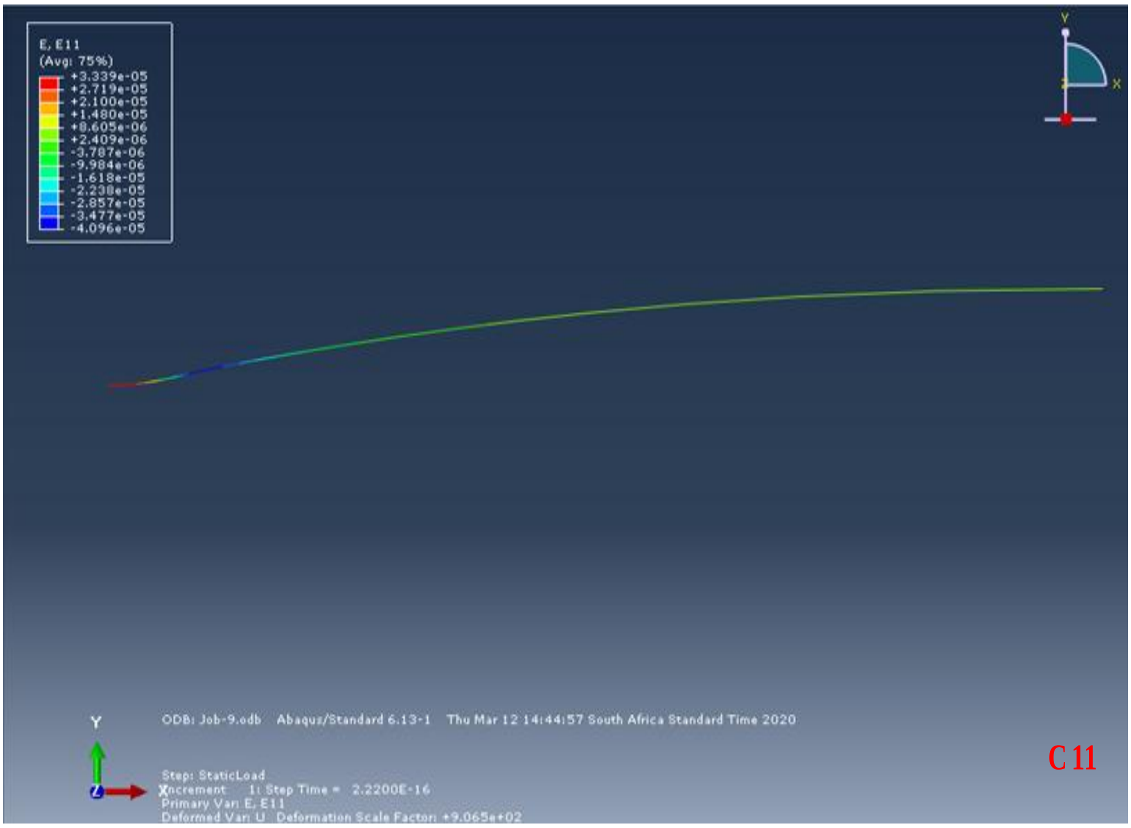
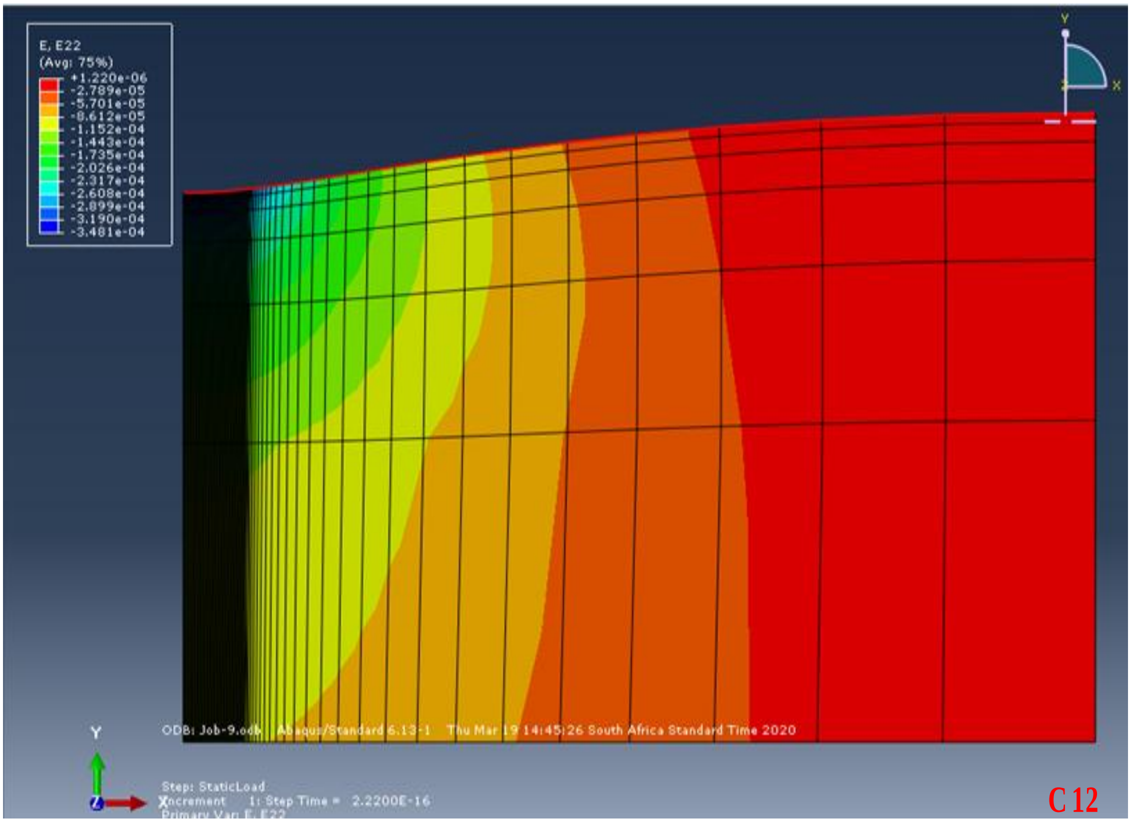
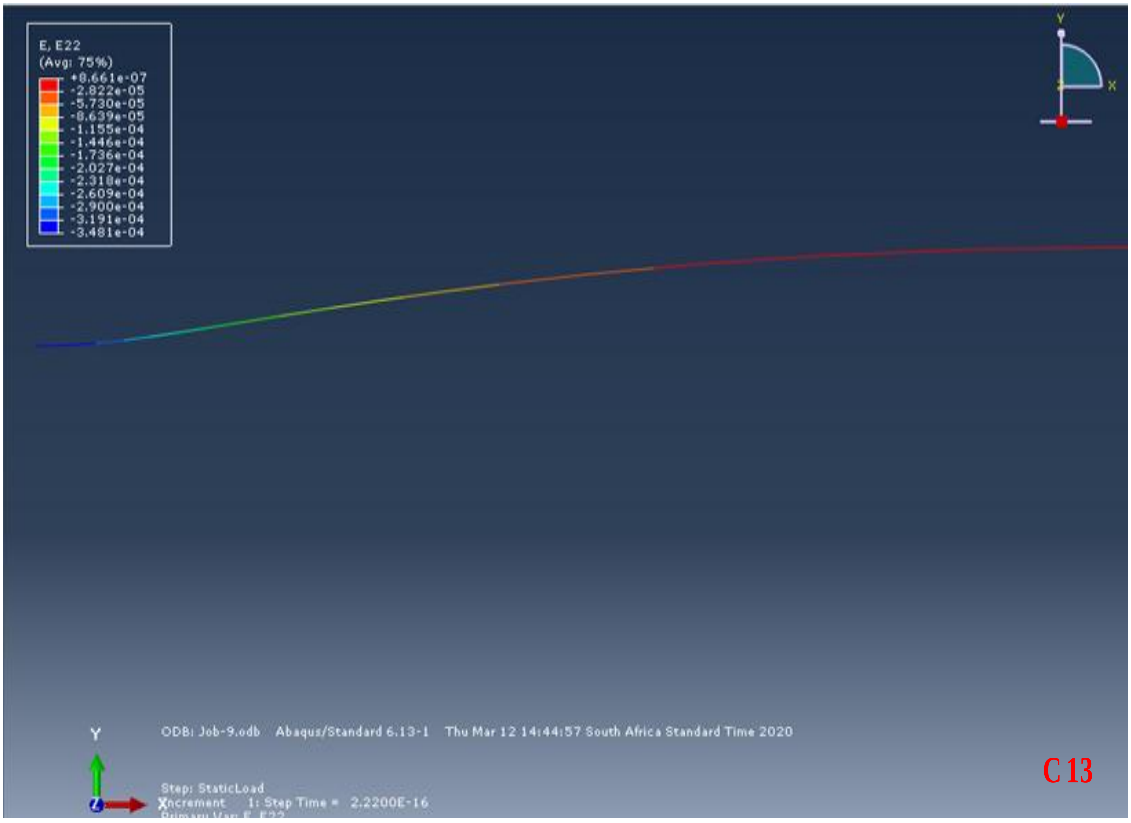




Appendix C10 – C11. Horizontal Tensile Strain at the Bottom of 150 mm WMA – 30% RAP Layer





Appendix C12 – C13. Vertical Compressive Strain at the Top of 150 mm WMA – 30% RAP Pavement Structure