SIMULATION OF FLEXIBLE PAVEMENT UTILIZING FLY ASH AS ALTERNATIVE STABILIZER

By

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Dedication

This dissertation is dedicated to the ALMIGHTY GOD, my family and loved ones.

Acknowledgements

Firstly, with a grateful heart, I give thanks to ALMIGHTY GOD for HIS love, grace, strength, wisdom, knowledge and understanding. I thank HIM for always being by my side. Without HIM, the completion of this research study would not have been possible.

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Abstract

The stabilization process in pavement construction is not a new process, but hitherto this process has not been fully implemented in the design methods for pavement structure. Its partial implementation in design has contributed to the failures experienced in pavement structure, which result in such pavement needing excessive maintenance and rehabilitation, thereby increasing the operational cost of the roads. Additionally, the use of an empirical design method for pavement structure has led to the over-design of pavement, resulting in wasteful design and construction of pavement structure. Nevertheless, Mechanistic-Empirical seems to be the way out. Consecutively, with the advent of powerful design software based on different methods such as the Finite Element (FE), Discrete Element, Finite Difference, Boundary Element Methods, the possibility of design and construction of quality pavement structures are enhanced. Therefore, the main focus of this study is to provide a modelling tool for using fly ash as alternative stabilizer for base layers of flexible pavement. To achieve the aim of the study, various objectives were set in place based on literature reviews which are documented in this study.

Considering the fact that FE is the method most adopted in pavement analysis and with the ability to obtain stresses and strains at the bottom of the surface layer, and compressive stress/strain within the base layer and at the top of sub-grade, it was considered in this study. Validations of a 3D FE model over 2D were conducted for fly ash stabilized base layer. Thereafter, the importance of an asphalt layer on a stabilized base layer was checked, and the efficiency of non-linear model for material characterization was also checked. Overall, a comparative analysis of FE modelling and an empirical method of pavement design was conducted. The results show that the use of 3D FE models is more efficient than 2D axisymmetric models; use of a non-linear material characterization model is more efficient than linear material characterization, and the use of empirical design methods results in the over-designing of pavement structure. Thus, the overall results suggest the use of 3D FE models, coupled with a non-linear material characterization model are suitable for the design of flexible pavement with a stabilized base layer.

Abbreviations

®	Registered trademark symbol
2D	Two dimensional
3D	Three dimensional
AASHTO	American Association of State Highway and Transportation Officials
ACAA	American Coal Ash Association
Al_2O_3	Aluminium oxide
ASTM	American Society for Testing and Materials Classification
C3	Cemented natural gravel materials used as base or sub-base according to COLTO 1998 classification
C3D8R	8-node solid continuum elements with reduction integration
C4	Cemented natural gravel materials used as base or sub-base according to COLTO 1998 classification
Ca	Calcium
CaO	Calcium oxide
CAX4R	4-node bilinear axisymmetric quadrilateral elements with reduction integration
CBR	California Bearing Ratio
COLTO	Committee of Land Transport Officials
CSIR	Council for Scientific and Industrial Research
CSLHA	Coconut shell, leaf and husk ash
D-P	Drucker-Prager plasticity model
FE	Finite Element
Fe_2O_3	Iron (III) oxide or ferric oxide
FEM	Finite Element Method
FM5-410	Field Manual 5-410
G5	Unbound granular used as sub-base material according to COLTO 1998 classification
GBS	Granular blastfurnace slag
K	Flow-stress ratio
kN	kilo Newton

ksi	kilo-pound per square inch
m	metre
M-C	Mohr-Coulomb plasticity model
ME	Mechanistic-Empirical method
MEDG	Mechanistic-Empirical design guide
MgO	Magnesium oxide
mm	millimetre
MPa	Mega Pascal
M_R	Resilient Modulus
Psi	pound per square inch
SAMDM	South African Mechanistic-Empirical Design Method
SANRAL	South African National Road Agency Ltd
SAPDM	South African Pavement Design Method
SiO_2	Silicon dioxide
SN	Structural Number
UCS	Unconfined Compressive Strength Test
WAC	West Africa Compaction
WASHO	Western Association of States Highway Officials
ψ	Dilation angle

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CHAPTER 1: INTRODUCTION

1.1. Background

Road transportation among transportation modes has expanded the most over the past 50 years, both for passengers and freight transportation (Rodrigue, Slack and Comtois, 2013). In South Africa, there are 750 000 kilometres of road network and 9.7 million vehicles, which make all sectors of the economy depend on roads to transport goods. The majority of goods, estimated at 83 percent, are transported by road, and in addition, forecasts reveal that freight transport demand will grow by 200 percent to 250 percent over the next 20 years (Ndebele, 2012). Considering its significant role in the economic and communication activities of the modern societies, researchers have been searching to attain the most suitable road pavement behaviour (Shafabakhsh, Motamedi and Family, 2013a), and consequently design and construct safe, stable, cost-effective and environment-friendly roads. With all the attention from researchers, pavement structures experience failure before the desirable design life resulting from the low bearing capacity of soil (Kordi, Endut and Baharom, 2010), overloading of the pavements, inadequacy in designs and unsuitable design methods used (Kordi et al. 2010; Shafabakhsh et al., 2013a). Its construction becomes uneconomical most often because of the cost incurred on materials used. With an appropriate method of soil stabilization, the soil's stability may be improved; resulting in stable pavements as well as the cost of construction may be reduced. However, the challenges with respect to the design of pavements remain. With the advent of powerful design software based on different methods such as the Finite Element, Discrete Element, Finite Difference, and Boundary Element Methods, the possibility of design and construction of quality pavement structures is enhanced. Therefore, in this study an attempt is made to simulate the behaviours of the flexible road pavements having fly ash as an alternative soil stabilizer, by using Finite Element Method (FEM).

1.1.1. Materials

Selection of materials for road pavement design is based on a combination of suitable materials, environmental consideration, construction methods, economics and previous experience (Bureau for Industrial Cooperation, 2012). Previously, road construction had depended mainly on the virgin materials from the nearest borrow pit, but in situations where the available soil lacks some geotechnical properties such soil needs to be stabilized. Soil stabilization, refers to the method aimed at increasing or maintaining the stability of soil mass and the chemical alteration of soils to enhance their engineering properties via different techniques, such as mechanical compaction, dewatering and addition of materials which are more advantageous (Gyanen, Savitha and Gudi, 2013; Yadu and Tripathi, 2013). According to Aminaton, Nima and Houman (2013), stabilizing soil using lime, cement, chemicals, plastics, rice husk ash, millet husk ash, corn cob ash, coconut shell ash, foundry sand, cement kiln dust, granular blastfurnace slag (GBS), or fly ash increases the soil's resistance, strength and permeability. Furthermore, results and experience show that lime as a stabilizer yields better results than others, but its use will make pavement structure uneconomical, which in turn makes fly ash an alternative stabilizer.

Fly ash, a finely divided residue that results from the combustion of pulverized coal, an amorphous ferro-alumino silicate with a matrix very similar to soil and its elemental composition varies with types and source of coal (Comberato, Vance and Someshwar, 1997). These ash particles are transported from the combustion chamber by exhaust gases as a result of their light weight and collected in control devices such as filter bags and electrostatic precipitators. They are spherical in shape and range in size from 0.5 micron to 100 micron (Heyns and Mostafa Hassan, 2013). From the point of view of the American Coal Ash Association (ACAA) (1995), fly ash particles are composed of glass with crystalline matter, carbon, and varying quantities of lime. Its chemical and physical properties depend greatly on several factors such as production type, raw feed and the handling method. This in turn gives the two classes of fly ash based on the chemical composition. Class C ashes are from sub-bituminous and lignite coals and may contain more than 20 percent CaO with 1 percent to 3 percent free lime, while Class F ashes are generally obtained from bituminous and anthracite coal and contain less than 20 percent CaO with no free lime ASTM C618 (ASTM-C618 2011).This industrial by-product is considered in this research because it is readily available and various measures of success have been achieved when used as stabilizer in pavement structures.

1.1.2. Design and Analysis

Pavement structural design is a daunting task with the basic geometry being quite simple, while everything else is not. Its traffic loading is a heterogeneous mix of vehicles, axle types, and axle loads with distributions that vary with time throughout the day, from season to season, and over the pavement design life (Schwartz and Carvalho, 2007). Also, pavement material characteristics such as viscoelasticity, non-linearity and linearity, respond to these loads in complex ways coupled with stress state and magnitude, temperature, moisture, time, loading rate, and other factors. Previously, design was done by the empirical method, then by layered elastic method, but as a result of the assumptions of the aforementioned, design sometimes results in errors (Huang, 2004). Thus, to model pavements correctly, it is necessary to use numerical methods, such as the finite difference method, the boundary element method and FEM (Áurea, Evandro and Lucas, 2006). However, FEM is the most adopted in pavement analysis and will be considered.

FEM is a numerical technique for finding approximate solution to boundary value problems for differential equations, also with the ability of handling changes of material properties such as Resilient Modulus and Poisson's Ratio in both vertical and horizontal directions and having successfully been used not only for designing pavement structures, but also for optimizing the design by stimulation (Brooks, Hutapea, Obeid, Bai, and Takkalapelli, 2008; Shafabakhsh et al., 2013a). Additionally, it is suitable for eliminating tensile stresses in granular layers by stress transfer method and also enables pavement designers to predict with some amount of certainty the life of the pavement (Brooks et al. 2008). FEM includes two-dimensional (2D) and three-dimensional (3D) methods, both of which can be employed to capture the structural response of flexible pavements.

Lastly, the failures experienced in pavement structures has been a long-standing global challenge to roads engineers, but with the success recorded through soil stabilization (fly ash as stabilizer), it would be ideal to feature this in the FEM to cut costs and time spent on laboratory work. In addition, the structural characteristics and mechanical behaviours of stabilized soil bases have not been investigated so extensively (Peng and He, 2009). Hence, this brought about this research work.

1.2. Problem Statement

The poor performance of flexible pavements results from the use of poor-quality materials, inappropriate stabilization (Paige-Green, 2008) and/or inadequacy in designs (Kordi et al., 2010; Shafabakhsh et al., 2013a). These later result in higher expenditure on maintenance and rehabilitation. Consequently, this influences the national annual budget, which is observed to be increasing at a higher rate every year (Ndebele, 2012). Furthermore, it is observed that flexible pavements in most of the South African roads experience permanent deformation (usually referred to as rutting), cracking of surface course and creation of potholes (Council of Scientific and industrial Research (CSIR) 2010), which are generally caused by various factors, such as soil expansion (Kordi et al., 2010), inadequate soil stabilization, inappropriate use of materials in the base courses and provision of inadequate thickness of pavement layers. As a result, the roads need regular maintenance and rehabilitation, which increases the operational cost of the roads. However, with the help of FEM, the behaviour of the flexible pavements can be simulated and the adequacy of pavement design can be examined by considering soil stabilizers as one of the major influential parameters for flexible road pavements. Based on the developed simulated scenarios, appropriate design and construction interventions can be taken to design and construct the pavements adequately using fly ash as a base course stabilizer, consequent upon which the cost of the maintenance and rehabilitation of flexible pavement roads will be reduced. Hence, this study pertains to the use of FEM to simulate the behaviour of flexible pavements of South African roads in which fly ash will be considered as a soil stabilizer in the base course.

1.3. Research Aim

This research aims at providing a modelling tool for the use of fly ash as alternative stabilizer for base layers of road. Moreover, this tool can be extended to other non-traditional materials as well.

1.4. Specific Objectives

To achieve the aim of this research, the following specific objectives need to be considered:

1. To evaluate the efficiency of using 3D FE model for design of flexible pavement.
2. To determine the structural response of stabilized base layers in flexible pavement system due to traffic loads using the 3D FE model.
3. To validate the use of fly ash as stabilizer through a 3D FE model.
4. To compare laboratory test empirical results already available against the 3D FEM results.

1.5. Delimitation

In this dissertation the development/formulation of a new mathematical model for the characterization of the material (fly ash stabilized base) is not considered. However, appropriate selection is made from the existing material characterization model. Lastly, this selection is based on the proper findings from literature reviews and the ability of the model to represent the behaviour of the material under loading.

1.6. Significance of the Study

This research is worth doing because of the important information it renders to future road engineers and researchers. Overall, the design of new road projects located in areas short of high-quality materials would result in very long material hauls. Thus, it may require pavement structure alternatives other than the conventional granular base. Understanding such material behaviour under loading is of great importance for effective pavement design. Recent studies undertaken on the use of waste and by-product materials as soil stabilizers have left a gap, between fly ash as stabilizer (empirical design approach) and its computer-aided design for pavement structures. Simulation of pavement structures has been carried out for different purposes, but not in the use of fly ash as alternative stabilizing material. As a result, this study will save time, human error and cost of laboratory experiments in carrying out projects and address the problems relating to road construction industry.

1.7. Outline of Dissertation

The remainder of this dissertation is organized as follows:

Chapter 2: This chapter presents the literature review on flexible pavement, pavement composition and behaviour, secondary materials in road construction and a brief background to pavement design.

Chapter 3: This chapter continues with the literature review on numerical simulation of flexible pavement with reviews on stresses, strains and deflections in flexible pavement, approach of mechanistic empirical design, layered elastic and finite element simulation of flexible pavement.

Chapter 4: This chapter presents the detail simulation design used for fly ash stabilized base layer flexible pavement via Abaqus® 3D FEM analysis; four models' analysis were used in line with the set objective and comparative analysis of laboratory test empirical results, and 3D FEM was carried out.

Chapter 5: Presentations and discussions of results obtained from the models in Chapter 4 were undertaken. These presentations were presented graphically and through contour plots.

Chapter 6: This chapter gives the conclusions, general recommendations and further studies for this dissertation.

CHAPTER 2: LITERATURE REVIEW

2.1. Flexible Pavement

Before any design of pavement structure, an appropriate pavement type selection is of importance as it is usually based on some critical factors such as soil composition, climate, traffic volume, life cycle, constructability and cost. In addition, there are secondary factors that need to be also considered, including: tire-pavement noise generation, surface smoothness and environmental sustainability. Flexible pavements have suitably met all the requirements, which made it to be used most frequently (Asphalt Pavement Alliance, 2010).

Flexible pavements with asphalt on the surface are used all around the world. The various layers of this pavement structure have different strength and deformation characteristics which make the layered system difficult to analyse in pavement engineering. At the surface there is a viscous material with its behaviour depending on time and temperature, and pavement foundation geomaterials; coarse-grained unbound granular materials in base/sub-base course; and fine-grained soils in the sub-grade, exhibiting stress-dependent non-linear behaviour (Kim, 2007). Furthermore, with the introduction of soil stabilization which brought about the use of new materials with different characteristics such as cementitious and polymeric, the design of flexible pavement has become more complex. However, the analysis of pavement via empirical methods, as previously mentioned, sometimes result in errors, but if material characterization is properly understood, finite element analysis can be successfully used in the design of flexible pavement, which in turn makes design adequate.

This chapter covers the literature review on pavement composition and behaviour, secondary materials, soil stabilization concept, fly ash as stabilizing agent, and lastly, details on pavement design background.

2.2. Pavement Composition and Behaviour

Pavement structure a composite system, consisting of superimposed layers of processed materials above the natural soil sub-grade, with the primary function of distributing the applied vehicle loads to the sub-grade. This structure’s ultimate aim is to ensure that the transmitted stresses due to the loading are sufficiently reduced, so that they will not exceed sub-grade bearing capacity (Adu-Osei, 2001). In other words, the tensile and compressive stresses induced on the pavement by heavy wheel loads decreases with increasing depth (Figure 2.1). In order to take maximum advantage, pavement layers are usually arranged in order of descending load bearing capacity, with the highest load-bearing capacity material on the top and the lowest load-bearing capacity material at the bottom, as seen in flexible pavement (Figure 2.1). However, in flexible pavements the unbound granular layers serve as a major structural component of the structure (Adu-Osei, 2001). Further, in developing countries like South Africa, the main structural element is formed by the unbounded granular layer as thick base and sub-base layers placed over the sub-grade; and for economic reasons, the asphalt layer is very thin, with a limited structural function, which mainly provides protection against water ingress (Araya, 2011).

Overall, the material properties and their influence on pavement behaviour must be thoroughly understood. According to The South African National Road Agency Ltd. (SANRAL) (2013a), there are a number of fundamental properties that influence the behaviour of a material regardless of its situation. These are: inter-particle friction, particle distribution, cohesion, elasticity, particle hardness, durability and porosity. In addition to fundamental properties, there are the situational properties that influence the behaviour, such as density, moisture content and temperature. The majority of these properties are considered in the design of pavement, but the most essential of these are the engineering properties which are actually the basic results in the design. Some of the engineering properties are: ultimate strength, elastic modulus, resistance to deformation and crack propagation and fatigue, all obtained from various laboratory tests.

Furthermore, various factors that have significant effects on the soil behaviour can be loading condition, stress state, soil composition, compaction and soil physical states (Kim, 2007). As a result of these factors the material characteristics of the entire pavement change continuously over time with environmental changes which later result in pavement failure. To avert pavement failure and reduce the cost of hauling natural materials, researchers introduced the use of secondary materials.

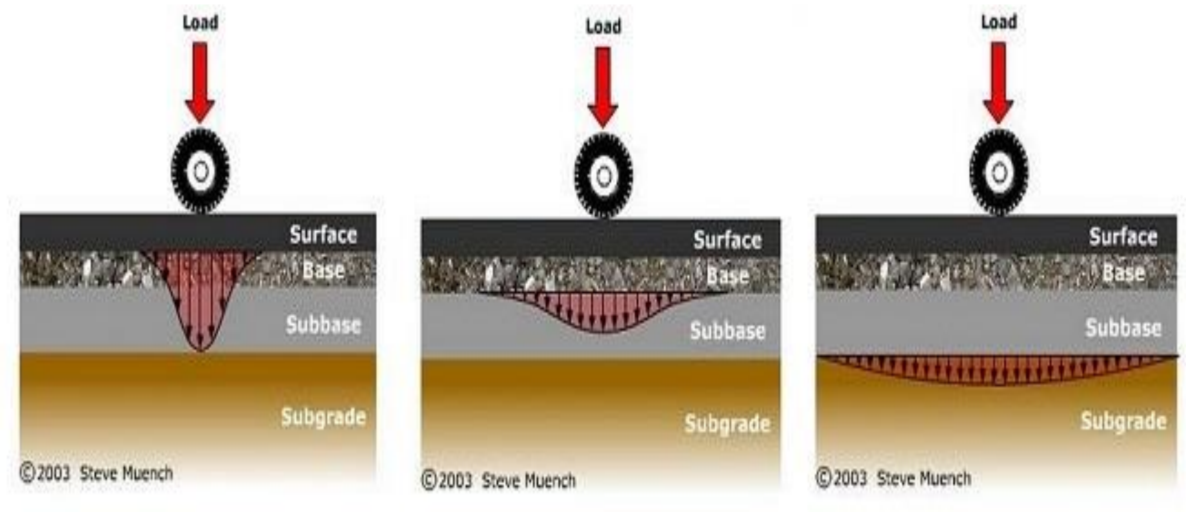


Figure 2.1. Typical Flexible Pavement and Load Distributions (Steve Muench, 2003)

2.3. Secondary Materials

Using by-products, recycled and waste materials as alternatives to naturally occurring aggregates in the construction of roads helps to conserve the supplies of good-quality aggregates, leads to less energy and environmental cost associated with the extraction and transportation of conventional aggregates, and assists in problems arising from the disposal of unwanted materials (Sherwood, 1974). Such materials are referred to as secondary materials or aggregates. This practice results from the current and projected high demand for conventional aggregates and the increasing difficulty of obtaining planning consent for their extraction. Moreover, this is combined with a greater awareness of the considerable quantities of ‘waste aggregates’ that are stockpiled and currently arising from the mineral extraction industries, the construction/demolition industry and industrial processes. All these have stimulated greater interest in the use of secondary materials in road construction (Nunes, Bridges and Dawson, 1996; Brennan and O’Flaherty, 2002). Some of the secondary materials considered for road works are blast furnace and steel slag, spent oil shale, china clay waste, slate waste, rice husk ash, millet husk ash, corn cob ash, coconut shell ash, waste foundry sand, cement kiln dust, fly ash, bottom ash and demolition and construction waste (Sherwood, 1974; Mostafa Hassan and Khalid, 2010; Amin, 2012; Bindu and Vysakh, 2012; Yadu and Tripathi, 2013). These materials are subjected to various laboratory tests before considering their use for road construction work. Such laboratory tests may include grain size analysis, specific gravity, compaction, Triaxial and leaching tests, etc., depending on the material type. Overall, the use of any of these materials depends on its availability at a particular location.

All in all, secondary materials are inferior to the natural materials used in construction, but the lower cost of these inferior materials makes it an alternative if adequate performance can be achieved (Heyns and Mostafa Hassan, 2013). In the quest to certify the use of secondary materials, researchers discovered that the one or a mixture of these materials with unstable natural materials yields an increase in its engineering properties. Hence, this relates to the process called soil stabilization.

2.3.1. Soil Stabilization Concept

In South Africa, the bearing capacity of the pavement is provided by the unbound base and sub-base or by the unbound base and stabilized sub-base (Araya, 2011). The asphalt layer provides a smooth riding surface and provides skid resistance. These structures have been successfully used in South Africa for moderately and heavily loaded roads. However, the minimum California Bearing Ratio (CBR) required for the sub-grade is 15 percent; when this is not reached, improvement of the sub-grade should take place (Molenaar, 2009).

Yet the concept of soil stabilization is not new, as it can be dated back to 5000 years ago. McDowell (1959) mentioned that stabilized earth roads were used in ancient Mesopotamia and Egypt, and that the Greeks and Romans once used soil-lime mixtures. Over the years, research has focused on improving the durability, safety and efficiency of pavement materials and structures within both economic and environmental constraints. This brought about the various means of stabilizing soil which are practical and economical.

Soil stabilization mainly aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, waterproofing the particles or a combination of the two (Sherwood, 1993). It is used to treat a wide range of materials including expansive clays to granular materials (Openshaw, 1992). The stabilization process can be accomplished by several methods. All these methods fall into two broad categories (FM5-410 2012), namely:

- ***Mechanical stabilization***

  Stabilization is achieved via a physical process by altering the physical nature of natural soil particles by either induced vibration or compaction and also by introducing coarse or fine materials and geosynthetic materials. Recently, mechanical stabilization has been used for pavement structure through geotextiles materials (Hejazi, Sheikhzadeh, Abtahi, and Zadhoush, 2012) which yielded a great increase in the property strength of the structures. Further, using a geogrid, Al-Azzawi, (2012) noted that placing this reinforcement at the base-asphalt interface leads to the highest reduction of the fatigue strain.

- ***Chemical stabilization***

  Stabilization depends mainly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect, including lime, cement, secondary materials and chemicals. Recently, with the increase in the problem posed by secondary materials and its availability locally, researchers considered their use as an alternative stabilizer. Some of these are: Mgangira (2006) Waste Foundry Sand on clayey soils, Bindu and Vysakh (2012) Coconut Shell, Leaf and Husk ash (CSLHA) on lateritic soils, Yadu and Tripathi (2013) GBS and Fly ash on soft soils, and Amin (2012) reviewed on soil stabilization using low-cost methods. Based on these studies, GBS, foundry sand, CSLHA, fly ash and scrap tyres are low-cost and effective as stabilizer. Further review will be done on fly ash as stabilizer as it is a centre to this dissertation.

Nevertheless, among these stabilization methods, results have shown that chemical stabilization is more advantageous (Makusa, 2012; Gyanen et al., 2013; Yadu and Tripathi, 2013). Overall, researchers noted that the presence of organic matters, sulphate, sulphide and carbon dioxide in the stabilized soils may inhibit the stabilization process (Makusa 2012). Likewise, compaction, moisture content, temperature and freeze-thaw further contribute (Sherwood, 1993; Makusa, 2012). Additionally, Paige-Green (2008) noted that failure in stabilization process may further result from lack of suitable skill and experience, inadequate specification, change in construction equipment and construction techniques.

2.3.2. Fly Ash as Stabilizer

South Africa being the fourth largest producer of fly ash at 30 mega ton per year after China, USA and India, results from the fact that coal plays an important role in its economy and is the primary energy source for electricity generation (Furter, 2011). Fly ash is a heterogeneous material with SiO_2, Al_2O_3, Fe_2O_3 and occasionally CaO as its main chemical components. This ash also contains Ca-bearing minerals such as Anorthite, Gehlenite, Akermanite and various Calcium Silicates and Calcium Aluminates identical to those found in Portland Cement (Snellings, Mertens and Elsen, 2012). Considering its production per year in South Africa, the government is at the stage where it is strategically finding ways to reduce fly ash through treatment, re-use and beneficiation (Heyns and Mostafa Hassan, 2013).

All over the world, fly ash is being used for various purposes such as cement production, concrete production (Torii, Hashimoto, Kubo and Sannoh, 2013), soil stabilization, asphalt (Lin Li, benson and Edil, 2007), embankment, flow-able fill and waste stabilization owing to its cement-like property, yet in South Africa only 6 percent of the annual production is utilized. Further, in pavement structure, fly ash has a wide application which is incorporated in sub-grade, granular base/sub-base, asphalt base/surface and structural fill (United States Environmental Protection Agency, 2009). Also, it has been combined with other products or by-products to improve pavement materials and its light weight and ability to be handled easily on construction site with little safety precaution (Kim, Prezzi and Salgado, 2005; Mathur, 2011; Heyns and Mostafa Hassan, 2013), contributes to it's usage.

Various studies have been conducted on its utilization as stabilizer and as an alternative to the use of virgin materials. Senol, Bin-Shafique, Edil, and Benson, (2002) carried out a study on the use of self-cementing class C fly ash for the stabilization of soft sub-grade. In this study, the optimum mix design and stabilized layer thickness were estimated by strength and modulus-based approaches. The results obtained showed that the engineering properties such as unconfined compressive strength (UCS), CBR and resilient modulus increase substantially after fly ash utilization. Also, in 2002, Pandian and Krishna conducted laboratory CBR tests on the stabilized fly ash-soil mixtures and observed that fly ash is an effective admixture for improving the soil quality. In addition, Brooks (2009) reported the soil stabilization with rice husk ash and fly ash mixed together with natural soil, the study showed improvement in CBR values and UCS. Also, researchers have proven that mixtures of fly ash with inert materials reach 50 percent to 70 percent of the strength of the corresponding cement-inactive materials (Eskioglou and Oikonomou, 2008).

Further, an innovative research was undertaken by Heyns and Mostafa Hassan (2013), utilizing three different types of fly ash (Kendal Dump Ash, Durapozz and Pozzfill) at 16, 18, 20 and 22 percent enhanced with cement on G5 sub-base material classified according to the Committee of Land Transport (COLTO) (1998) officials, and results show that G5 sub-base material is stabilized to meet up with the C3 and C4 stabilized standard according to COLTO. Fly ash controls the shrink-swell by cementing the soil grains together and also have the tendency to increase the maximum dry density (Ban and Park, 2014) and improve the CBR of soil by 100 percent (Umar, Alhassan, Abdulfatah, and Idris, 2013). Consequently, any fly ash that has at least some self-cementitious properties can be engineered to perform in transportation projects.

Furthermore, studies have been concluded that, if fly ash is used properly, it is not hazardous to the environment when used for soil stabilization. This was done with a combination of batch-leaching tests to determine potential impact on the environment of fly ash as trace element mobility in soil stabilization (Heebink and Hasselt, 2001). Similarly, Tanosaki, Yu and Nagasaki (2011) studied an image of fly ash being an ‘environmentally friendly’ product. The measurements were carried out only on Hunter brightness or reflectivity. Three hundred lots of coal ash samples were analysed, whereby it was determined that coal ash possesses a wide range of colour hues. Due to strong correlations between hue and spherical rate, Chroma(C) and $CaO^{+}MgO$ content of coal ash, it could be used as a base for quality control standards. Overall, the use of fly ash is accepted worldwide due to saving in cement, consuming industrial waste and making durable materials, especially due to improvement in the quality when used as stabilizer (Heyns and Mostafa Hassan, 2013).

Generally, the compressive strength of fly ash-stabilized soils is dependent on in-place soil properties, delay time, moisture content at time of compaction and fly ash addition ratio as discussed by ACAA (1995). Summarily, the performance of pavement structure depends on the satisfactory performance of each material used, thus proper evaluation is required in respect to the properties of each material separately. Overall, the proper understanding of the behaviour of natural materials, secondary materials and the soil stabilization process are important to the successful implementation of any design method.

2.4. Pavement Design Background

Pavement design is the process of developing the most economical combination of pavement layers (in relation to both thickness and materials type) to suit the soil foundation and the traffic to be carried during design life. From the SANRAL (2013b) point of view, pavement design is to ensure that materials within the pavement layers are not overstressed at any time during the course of these changes in the pavement's life. Over the years, in the pursuit of accurate simulation of pavement structure behaviour under loading, various design methods have been developed. These design methods, on individual capacity, have been used to simulate pavement behaviour based on some assumptions. Figure 2.2 gives a background on the design of flexible pavement.

At the outset, pavement designs were based on empirical methods which are back-dated to the development of the Public Roads soil classification system in the 1920s (Huang, 2004; Schwartz and Carvalho, 2007). Empirical methods are derived from experience in terms of field observation performance of in-service pavement or laboratory test sections. The purpose of laboratory methods is to subject a representative pavement material sample to an environment (consisting of simulated traffic loading and environmental conditioning) that closely simulates field conditions (Adu-Osei, 2001). These methods also define the interaction between pavement performance, traffic loads and pavement thickness for a given set of paving materials, soil, location and environmental conditions (Schwartz and Carvalho, 2007). Although the design of flexible pavements is still largely empirically based, these methods remain accurate only for the exact conditions for which they were developed, and perhaps invalid outside the range of variables used in its development.

This brought about the various examples of empirical design methods developed with different location such as the American Association of State Highway and Transportation Officials (AASHTO) in the USA (1993), Road Note in the UK (Road Research Laboratory, 1970), Western Association of States Highway Officials (WASHO) in Malad and West Africa Compaction (WAC) in West Africa Countries (Fall, Ba, S., Sarr, Ba, M. and Ndiaye, 2011), to mention but a few.

Even though empirical methods tend to be simple and easy to use, these methods are associated with various limitations such as one climate condition, limited traffic, material type, and new construction only (i.e. cannot be used for rehabilitation). If these conditions change, the design is no longer valid (Wang, 2001; Huang, 2004). To further buttress this point, Huber, Andrewski, and Gallivan (2009) found that the AASHTO 1993 pavement design guide typically over-designed pavements in Indiana by 1.5 to 4.5 inches, amounting to approximately 600 to 800 tons of materials per lane-mile beyond what is needed.

Before the final introduction of mechanistic-empirical design guide (MEDG) in the 21st century, other design methods aside, empirical methods were developed between 1940 and the 1960s which are: limiting shear failure method, limiting deflection method, and regression method, all based on pavement performance and/or road test. However, these methods have various limitations and likewise do not satisfy all necessary requirements for an ideal design which makes them obsolete (Huang, 2004). Ultimately, pavement design methods differ from one to another yet, they are affected by the same factors which are: traffic and loading, structural models, materials, environment, and failure criteria. Nevertheless, a better approach to the design of perpetual pavements is the mechanistic-empirical method.

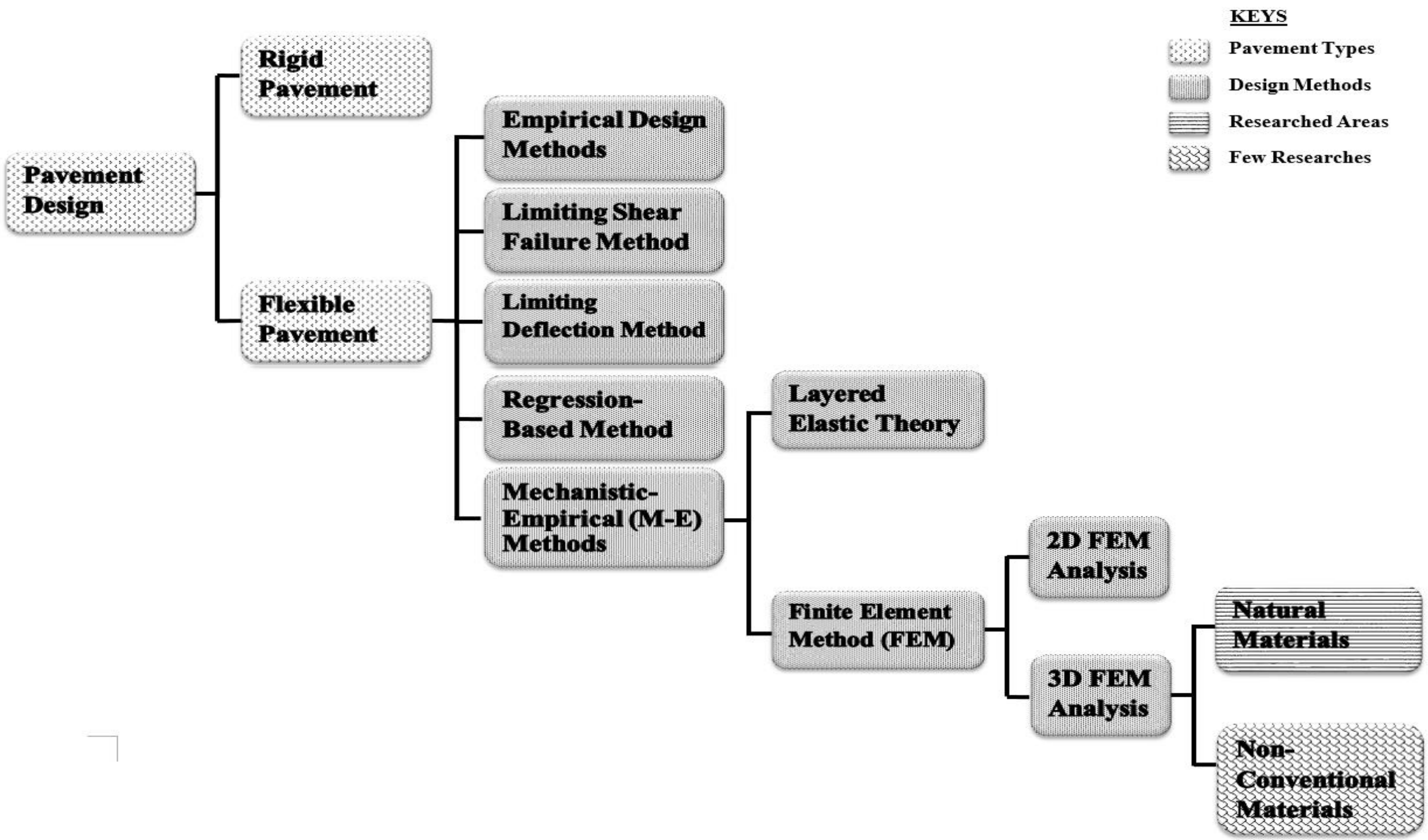


Figure 2.2. Flexible pavement design background

2.5. Summary

Chapter Two of this dissertation dealt with the literature review on flexible pavement, pavement composition and behaviour, secondary materials, the soil stabilization concept, and fly ash as stabilizer and pavement design background. Research has shown that the use of fly ash and other industrial by-products as stabilizer in pavement structure has recorded great success and also, fly ash is proven to be environment-friendly if proper precaution is taken into consideration when used. Further, on the aspect of design, the inadequacy of empirical, limiting shear failure, limiting deflection and regression based method for pavement design as contributed to pavement failures, but the FEM is seen as a way out. With the foundation made in this chapter, the next chapter will be building upon it by reviewing in detail on numerical simulation of flexible pavement.

CHAPTER 3: NUMERICAL SIMULATION OF FLEXIBLE PAVEMENT

3.1. Stresses, Strains and Deflections in Flexible Pavement

Pavement analysis has been transitioning from empirical methods to numerical approaches (Kim, 2007). However, beforehand - due to the limitations of computational capabilities - pavement designs were dominated first by empirical methods which are limited to a certain set of environmental and material conditions, but with the advent of powerful computer storage capabilities, this design can now be done on personal computers (Adu-Osei, 2001; Huang, 2004; Shafabakhsh et al., 2013a). In numerical approach (also known as mechanistic), the pavement is treated as a layered structure with the proper understanding of its components in respect to the constituent materials (Kim, 2007).

Mechanistic Analysis exploits mathematical capability to calculate the stress, strain, or deflection in a multi-layered system such as pavement, when subjected to external loads (Hafeez, 2010). The stresses, strains and deflections generated in flexible pavements result from the material properties and thickness of each layer and loading condition (Al-Khateeb, Saoud and Al-Msouti, 2011). Further, with the use of computer programs one can evaluate the theoretical stresses, strains, and deformations anywhere in the structure. However, there are a few critical locations which are of interest and are often used in pavement analysis (NCHRP, 2004; Pavement Interactive, 2008; Darwish, 2012; SANRAL, 2013b) (Figure 3.1) such as;

- Surface deflection
- Tensile horizontal strain at the bottom of the surface course (for surface course fatigue cracking)

- Compressive vertical stresses/strains within the base/sub-base layers (for rutting of unbound layers)
- Compressive vertical stresses/strains at the top of the sub-grade (for sub-grade rutting)

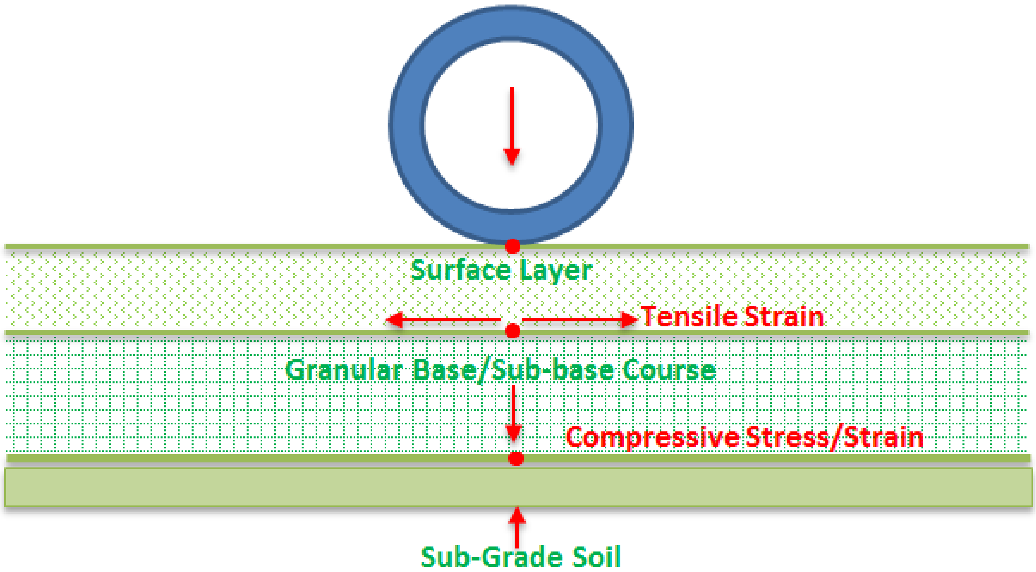


Figure 3.1. Critical Pavement Responses and Locations (Pavement Interactive, 2008)

Furthermore, in mechanistic analysis, the material's resistant behaviour is characterized using mathematical models. Thus, this method translates the analytical calculations of pavement response to performance. Nevertheless, the design of pavement structure is not totally mechanistic, as dependence on observed performance is necessary because theory alone has not proved sufficient to realistic pavement design (Huang, 2004), and also laboratory testing is often required to provide a relationship between loadings and failure which enhance the development of a proper mathematical model. Hence, this brought about the concept of the mechanistic-empirical (M-E) method of pavement design.

This chapter gives detailed background on pavement design through layered elastic simulation and finite element simulation with more attention on critical factors, such as the geometry selection, material characterization, and boundary and loading condition. Further, in review of FEM types, the software ABAQUS® will be introduced and thereafter the concept of failure analysis was discussed.

3.2. Approach of Mechanistic Empirical (M-E) Design

An M-E design approach uses empirical relationships between cumulative damage and pavement distress to determine the adequacy of a pavement structure to carry the expected traffic load (Nicholas and Lester, 2010). This approach combines theory and physical testing with the observed performance in pavement design. This design is also historically aimed at developing more accurate pavement models with a lot of emphasis on developing the mechanistic parts of the model (Theyse and Muthen, 2000). As a result, this gives M-E analysis advantage over empirical methods. Some of such advantages are: accommodation of new materials and changing load types, better utilization of available materials, capability of being used for the design of both existing pavement rehabilitation, and new pavement design in which empirical methods are limited.

Despite the advantages of M-E analysis, many developing countries still rely on empirical methods, realizing that more sophisticated mechanistic design procedures often require too many assumptions regarding material behaviour and too complicated material testing techniques to be of direct practical use (Araya, 2011). Nevertheless, the end results outweigh its complexity. Via M-E analysis, two major approaches are employed to compute the stresses and strains in pavement structures which include layered elastic theory and FEM, which are further discussed. In addition, the effectiveness of any M-E method relies on the accuracy of the predicted stresses and strains. Hence, this gives FEM an edge over the layered elastic theory. Further, the success of this method hinges on some critical variables, which are material properties, traffic, environmental conditions, and pavement geometry. Nonetheless, for accuracy in pavement response prediction through M-E methods, more focus should be placed on constituent materials' behaviour and their accurate characterization (Johnson, Sukumaran, Mehta, and Willis, 2007; Araya, 2011).

3.3. Layered Elastic Simulation

Layered elastic simulation is the most common and easily understood procedure of the M-E design methods. In this simulation the pavement structure is divided into an arbitrary number of horizontal layers with the thickness of each individual layer and materials assumed to be homogeneous and linearly elastic (Wang, 2001). Firstly, Burmister (1943) obtained the primary equations for a two-layer, three-layer and later multilayer system. These equations were derived from the original elastic theory by Boussinesq (1885). The original elastic theory was used to compute stress and deflection in a half-space soil composed of homogeneous, isotropic and linearly elastic material which is still widely used in soil mechanics and foundation design (Wang, 2001; Huang, 2004). Yet, Burmister's equations led to the significant development in pavement analysis using mechanistic method; these equations are included in the earliest software CHEV 1963. In addition, other software was developed based on these equations but with different modifications. A number of these are: BISAR 1973, developed to incorporate rate independence; VESYS 1974, to incorporate the serviceability and reliability concept; ELSYM5 1986, to incorporate multilayers; DAMA 1979, to incorporate nonlinear elastic granular materials; and also KENLAYER for nonlinearity in granular materials, which is still commonly used (Wang, 2001; Huang, 2004; NCHRP, 2004).

In South Africa, a great contribution has been made through the development of the South African Mechanistic-Empirical Design Method (SAMDM), which is now known as the South African Pavement Design Method (SAPDM) (Van Vuuren, Otte and Paterson, 1974; Theyse, de Beer, Maina, and Kannemeyer, 1996; SANRAL, 2013b). The SAMDM analysis for flexible pavement is based on linear elastic multilayer theory and here the structural pavement layers are assumed to be isotropic (Steyn, Maina and Repsold, 2013). Although SAMDM is sound in principle and has been applied successfully to the design of pavement, this method is faced with the intense challenge of its inability to cater for the cross-anisotropic behaviour of materials (Steyn et al., 2013) and its over-sensitivity to the changes in the input variables, which lead to inadmissible and counter-intuitive results and provide unrealistic pavement design (Theyse et al., 2011). These in turn contribute to the increases in its scrutiny and criticism in the recent past (Jooste, 2004). However, for SAMDM to achieve more realistic values of predicted life for pavement section, it must include cross-anisotropic analysis (Steyn et al., 2013); as a result, SAMDM is being revised (SANRAL, 2013b).

Overall, considerable efforts have been reported regarding linear elastic simulation of pavement structures, yet the assumptions, on which this approach works, make it inappropriate for the real pavement properties and actual scenario on-site. Such assumptions are (Tutumluer and Thompson, 1997; Wang, 2001; Huang, 2004);

- Each layer is homogeneous, isotropic and linearly elastic with a finite thickness.
- Material is weightless.
- Circular uniform pressure is applied on the surface.
- Continuity and frictionless interface condition.

However, Mansurkhaki, Hesami, Khajehhassani, and Khordehbinan (2014) maintain that there are no significant differences between the mean values of the parameters obtained from layered elastic analysis and FEM, which is similar to the opinion of Ameri, Salehabadi, Nejad, and Rostami, (2012), but on the other hand Ameri et al. state that results from FEM are most appropriate compared with that of multi-layer system. Also, Gupta and Kumar (2014) reported discrepancies in results from KENLAYER; compared with those of FEM it shows that maximum vertical deflections are lower in KENLAYER. In addition, various studies have shown that using linear elastic simulation for pavement vertical stress and strain prediction results in error, especially in low-thickness layers of asphalt pavement (Theyse et al., 1996; Abed and Al-Azzawi, 2012; Al-Azzawi, 2012; Shafabakhsh et al., 2013a). Yet the simplicity and speed of multi-layer analysis has been used as justification for relative results obtained (Zaghloul, 1993). Overall, since stress, strain and relative conditions of different layers in pavement structure are used in predicting pavement failures, the need for considering materials’ behaviour in nonlinear form increased significantly. This substantiates the fact that many researchers have found that the nonlinear elastic behaviour of base and sub-grade materials is important in accurately estimating stresses and strains in pavements (De Beer, Fisher, and Jooste, 1997; Mun, 2003; Tiliouine and Sandjak, 2014). In view of the aforementioned limitations, FEM is more preferred because it provides a more realistic analysis for predicting pavement response (Zaghloul, 1993) and its capability to accommodate nonlinearity of pavement materials (Tiliouine and Sandjak, 2014).

3.4. Finite Element Modelling (FEM)

FEM has wide application in lamella mechanics, hydrodynamics, soil mechanics, and structural mechanics because of its great capability for finding approximate solution to boundary value problems (Peng and He, 2009). In FEM, the whole problem is divided into small and simpler parts through mesh generation which are called finite elements and solved by calculus of variation in order to minimize associated error function (Reddy, 2005; Dixit, 2007; Yagawa, 2011). Over the years, FEM has been applied extensively in road engineering (Peng and He, 2009) and so far, it is the most versatile of all analysis techniques, with capabilities for 2D and 3D geometric modelling, able to analyse stable (static), time-dependent problems, non-linear material characterization, large strains/deformations, dynamics analysis and other sophisticated features (NCHRP, 2004). Furthermore, FEM can deal with complicated loading (static, dynamic and spatially distributed form) conditions and more accurate than the multilayer elastic method. The application of FEM to solve any problem consists of three separate stages, as shown in Figure 3.2.

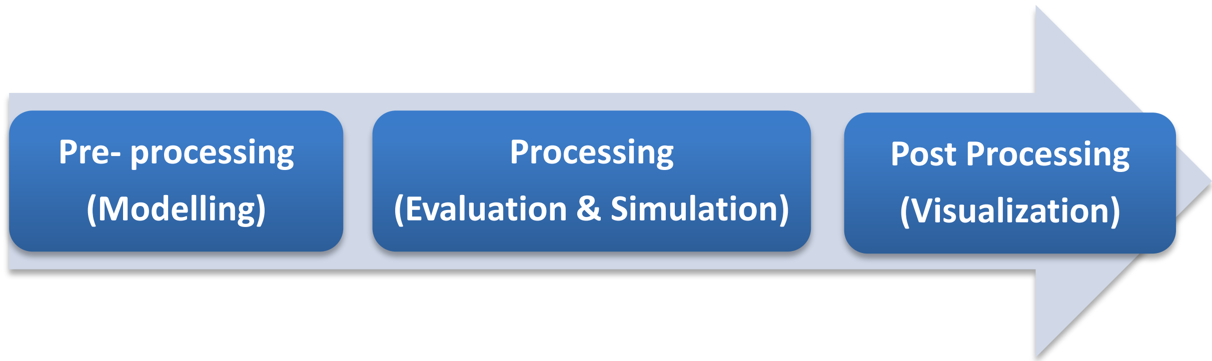


Figure 3.2. FEM application stages (Abaqus, 2013)

- ***Pre-processing (Modelling)***
  This is the first stage in any FEM analysis, and here can be referred to as the input files stage, which is the most critical for the accurate prediction of the result in terms of stress, strain and deflection. At this stage the following selection/input are made: the geometry of pavement (in terms of dimensions), material characterization, relationship between parts (assembling and interactions), loading and boundary conditions, and analysis type. Further discussion will be introduced on the input files in this thesis.

- ***Processing (Evaluation and Simulation)***

  In this stage, the job step is the main step and the input files are processed to produce the results (output file). Basically, at this stage the analysis process is only monitored in case an error is detected.

- ***Post-processing (Visualization)***

  This stage is a graphic rendering phase of the output file from the processing stage. Results are well represented in the realistic format and the maximum and critical area of interest can easily be accessed. Further, results in graph format can be obtained as well.

3.4.1. Critical Factors in FEM Simulation of Pavement

Generally, creating a FE model for flexible pavement analysis involves the consideration of all the steps in the pre-processing, with a critical look at some factors. Any FEM generated must capture important features of the physical situation without irrelevant details (Abaqus Inc., 2003). Overall, the success of any FEM simulation depends greatly on these factors as it can lead to error in the design of pavement if not properly put into consideration. Some of these factors are discussed below, with their effects on pavement design.

3.4.1.1. Geometry

In pavement simulation via FEM, geometry in terms of dimension and sharpness is of importance as it affects the overall analysis, time efficiency and accuracy of the results. Regarding geometry, there are some major areas of concern such as: dimension size, element type and mesh. Generally, the larger the dimension size and complexity of model, the more analysis time is required, as a result, Duncan, Monismith, and Wilson (1968) reported that a reasonable pavement response can be obtained by using 50 times and 12 times the circular loading area in vertical and horizontal direction respectively.

FEM achieves its aim by dividing the problem domain into a number of simpler subdomains - the finite elements. Various element types exist in the use of FEM for pavement simulation, such as: linear or first-order, quadratic or second-order, modified second-order, continuum and infinite element, to mention but a few (Figure 3.3). These element types further utilize the reduction integration techniques for reducing analysis running time (Abaqus, 2013; Zaghloul, 1993). Thus, careful consideration must be given to the element aspect ratios; however, the use of infinite elements was discouraged as it is not necessary in achieving accurate result (Sukumaran, 2004). In addition to 3D FEM analysis, Zaghloul (1993), Rahman, Mahmud and Ahsan, (2011) and Ibrahim, Gandhi and Zaman (2014) recommend the use of the solid continuum element with reduction integration as it has the capability of representing large-scale deformation and material nonlinearity.

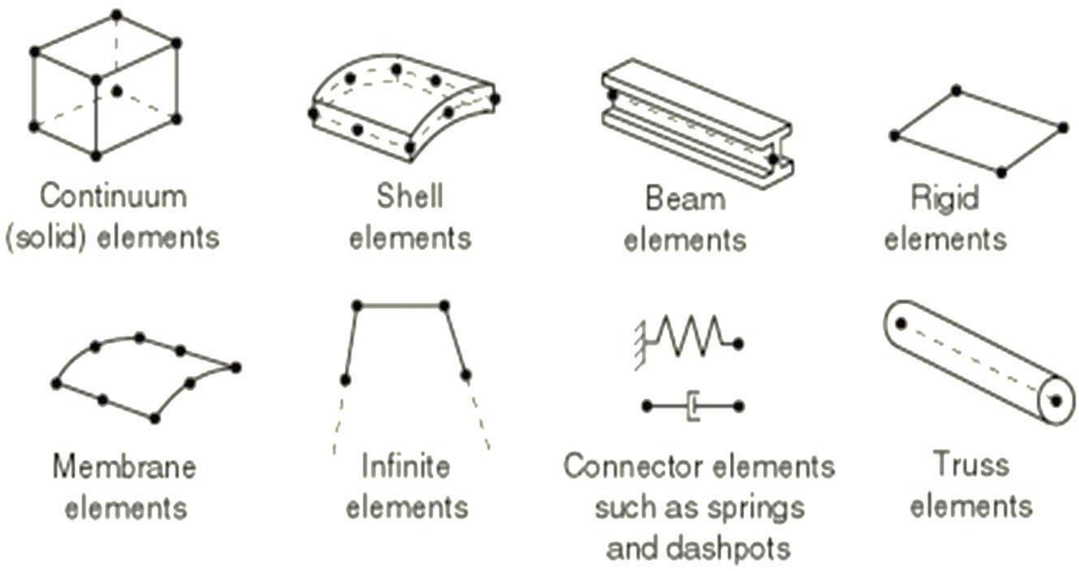


Figure 3. 3 Some element families in Abaqus (Psarras et al. 2002)

On mesh, FEM employs mesh generation technique for dividing a complex problem into small elements (Lo, 2002; Yagawa, 2011). It is known that the finer the mesh, the more accurate the results obtained (Al-Jhayyish, 2014) and the more analysis time required. On this note, Hjelmstad, Kim and Zuo (1996) investigated issues on mesh construction aspects of modelling pavement structures with 3D finite element analyses such as mesh refinement, domain extent, computational memory, and element size transitions, result shows that there is a great relationship between mesh refinement and accuracy of results obtained and good aspect ratio resulted in accurate results and reduce computation time. More so, on the aspect of mesh construction and refinement reports as shown that the more fine the meshing is, the more the accuracy in the result generated, consequently, researchers concluded that, the mesh should be fine near loading area and coarse at distances away from applied load for efficient model (Hjelmstad, Kim and Zuo, 1996; Sukumaran, 2004; Peng and He, 2009; Tiliouine and Sandjak, 2014).

3.4.1.2. Material Characterization

Proper material characterization is another major aspect of FEM-based design of pavement for accurate response prediction as the reliability of pavement design depends on it (Çöleri, 2007). However, an accurate material characterization is the selection or formulation of proper constitutive equations to represent the behaviour of the materials under loading (Kim, 2007). Qualitative choice is needed in material characterization and it is important that the model captures the major features of material behaviour while minor features may be ignored in the model (Abaqus Inc., 2003). Furthermore, resilient modulus (M_R) is one of the important inputs alongside with Poisson's Ratio and it is a primary material property for characterizing all unbounded layers and soils in any FEM model for flexible pavement design (Kim and Siddiki, 2006; Harold and Von Quintus, 2007; Ji, Siddiki, Nantung, and Kim, 2014). M_R values may be estimated directly from laboratory testing such as: Triaxial, Oedometer and Shear test (level 1 input), indirectly through correlation with other laboratory/field tests (CBR, Isotropic compression test, Uniaxial strain test, Indirect tensile strength and UCS) (level 2 input) or back-calculated from deflection measurements (level 3 input) (Mallela, Harold, Von Quintus, Smith, and Consultants, 2004; Harold and Von Quintus, 2007; Eluozo, 2013; Ji et al., 2014). Yet, correlation (level 2) is commonly used, based on the fact that level 1 depends on difficult laboratory testing. However, further discussions are presented in section 3.4.2 and 3.4.3 for level 2 and level 1 respectively, based on the available results for this study.

Primarily, two material constitutive models are used in pavement structures, which are Elasticity; Elastic and Viscoelastic, and Plasticity; Viscoelastic, Drucker-Prager (D-P), Mohr-Coulomb (M-C), Modified Cam-Clay model, and Modified Cap model to mention but a few (Abaqus Inc., 2003; Ti, Huat, Noorzaei, Jaafar, and Sew, 2009; Desai, 2012). Ultimately, a realistic constitutive model should better understand the mechanical behaviour of the represented material and must be capable of representing material behaviour in any relevant spatial situation (i.e. 1-dimensional, 2D and full 3D analysis). If the models are not properly selected it may lead to under- or over-design of the pavement structure.

3.4.1.3. Analysis Type, Boundaries and Loading Conditions

In FEM analysis, there are two major types of analysis procedure (also called STEP in Abaqus®) depending on the modelling nature; these analyses are: general and linear perturbation. In these procedures, there are forms such as; Geostatic, Mass diffusion, Heat transfer, Static, dynamic analysis, etc. However, these two analyses' procedure can be used in pavement analysis. The linear perturbation analysis procedure is usually employed for linear analysis work, while the general analysis procedure goes with the non-linear analysis work (Abaqus, 2013). As a result, the use of linear perturbation for non-linear analysis will only consider the linear effects, thus resulting in error.

Furthermore, boundaries conditioning is of importance and has a significant influence on the predicted response (Zaghloul, 1993). Boundary conditions are the degrees of freedom at each node in an element. A model can either be restrained in vertical, horizontal direction or set of nodes; on the other hand, if boundary conditions are not properly selected it may lead to generation of excess stresses and strains, both in vertical and horizontal direction. In view of these, researchers have suggested the use of fixed constraints at the bottom of the element (sub-grade) and roller constraints on the vertical boundaries (Peng and He, 2009; Al-Khateeb et al., 2011; Rahman, Mahud and Ahsan, 2011; Abed and Al-Azzawi, 2012; Sinha, Chandra and Kumar, 2014). Furthermore, various forms of contact interaction (mechanical and thermal) occur between the pavement layers. This interaction usage was encouraged (Peng and He, 2009) as it improves the results. Additionally, most researchers (Peng and He, 2009; Shafabakhsh et al., 2013a; Shafabakhsh et al., 2013b) prefer the use of perfect bond between the layer so allow uninterrupted distribution of stresses, strains and deflections through the layers, yet this is not the real scenario in reality as full bounding is not always achieved (Sutanto, 2009).

Regarding loading, tyre load representation is another critical factor to be considered in pavement simulation. Representing the tyre contact wrongly will affect the overall results. Over the years, various methods have been suggested in representing the loading in pavement design. Initially, a circular representation is used (Al-Khateeb et al., 2011; Sinha et al., 2014; Tiliouine and Sandjak, 2014), but at present, various representations have also been made in different studies (Peng and He, 2009; Rahman et al., 2011) with positive results. Furthermore, in reality, pavement is subjected to a moving loading, yet several researchers (Rahman et al., 2011; Shafabakhsh, Talebsafa, Motamedi, and Badroodi, 2013b; Sinha et al., 2014) have used static load for analysis rather than dynamic load because of the theoretical and practical difficulties involved in the analysis when using a dynamic load (Kim, 2002).

In a nutshell, with the great aptitude of FEM to analyse stable problems, time-dependent problems and those problems with non-linear properties of materials (Salehabadi, 2012), a careful balance is required in all the above-mentioned factors to meet the demand for solution and memory without sacrificing accuracy (Sukumaran, 2004). FEM has been successfully used in the analysis of the major forms of failure in pavement structure such as rutting and fatigue cracking at different layers (Walubita and van de Ven, 2000; Al-Khateeb et al., 2011; Abed and Al-Azzawi, 2012), and also used to determine the accurate positioning of geogrid materials (Al-Azzawi, 2012), thickness of each layer (Shafabakhsh et al., 2013a; Sinha et al., 2014) and the interaction between pavement and its instrumentation (Zafar, Nassar and Elbella, 2005; Yin, 2013).

3.4.2. Correlation Equations in FEM Simulation

M_R is the measure of material stiffness (i.e. stress divided by strain for rapidly applied loads). This can be mathematically expressed as the ratio of applied deviator stress to recoverable strain (George, 2004; Pavement Interactive, 2007; Ji et al., 20014). Determining M_R is of vital importance for any mechanistically based design/analysis procedure for pavements because it represents the structural strength of pavement layer on through which the thickness design is based (Eluozo, 2013; Ji et al., 2014). However, AASHTO recommends that M_R be obtained from repeated Triaxial testing, but due to the complexity of the test and time required, its results are not readily available. In view of this, researchers improvise through the use of correlation equations for readily available test results, for example CBR (Heukelom and Klomp, 1962; Sas, Głuchowski, and Szymański, 2012) and UCS (Little, Snead, Godiwalla, Oshiro, and Tang, 2002; Kim and Siddiki, 2006; Rao, Titus-Glover, Bhattacharya, and Darter, 2012; Al-Jhayyish, 2014). This material input method is referred to as level 2 inputs. Correlation equations help to convert readily available results to corresponding M_R values. However, results from UCS testing is more common and popularly used as its data are readily available (Rao et al., 2012) and a better property to predict design M_R (George, 2004). As a result, it is a necessity to evaluate design M_R of stabilized base layer based on the available UCS data. Table 3.1 suggests few of several equations to estimate M_R with results from UCS test.

Of all the promising equations suggested in the above table, according to Little et al. (2002) and Al-Jhayyish (2014), the correlation equations proposed by Barenberg (1977) for cement-stabilized soils are in good agreement with the laboratory results. However, a cement-fly ash-base layer is considered in the research, yet there is no direct correlation equation for it. Considering and validating the two equations by Barenberg, it was found the equation for ‘cement-stabilized coarse-grained sandy soils’ gives a closer result when compared with the recommended M_R for cemented materials used in SAMDM 1996 (SANRAL, 2013b). Since the material used in the previous research is a G5 material (usually gravel with coarse-grained properties) which is stabilized to C3 and C4 by using cement-fly ash as stabilizer (Heyns and Mostafa Hassan, 2013). Thus, the correlation equation by Barenberg (1977) for ‘cement-stabilized coarse-grained sandy soils’ is suitable and will be used to estimate the design M_R which will serve as the input for material property in the software. Largely, careful consideration should be given to the unit of parameters in the equation and their conversion to avoid error.

Table 3.1. Summary of correlations between the unconfined compressive strength and modulus

Correlation	*Source of the Correlation*	*Application Area*
M_R (ksi) = 500 + UCS (psi)	American Coal Ash Pavement Manual (1990)	Lime-cement-fly ash stabilized soils
M_R (psi) = 1200 UCS (psi)	Barenberg (1977)	Cement-stabilized coarse-grained sandy soils
M_R (psi) = 440 UCS (psi) + 0.28 UCS^2 (psi)	Barenberg (1977)	Cement-stabilized fine-grained soils
M_R (ksi) = 0.124 UCS (psi) + 9.98	Thompson (1966)	Lime-stabilized soils
M_R (psi) = 0.25 UCS^2 (psi)	McClelland Engineers (unpublished)	Lime-cement-fly ash mixtures
M_R (MPa) = 2240 $UCS^{0.88}$ (MPa) + 110	Australian Road Research Laboratory (1998)	Cemented natural gravel

3.4.3. FEM Material Characterization via Direct Testing Results

Although, the use of Triaxial, Oedometer and Shear test results as material characterization are level 1 input, considered more accurate (Mallela et al., 2004), but as a result of these tests' unavailability, it is considered second in the research studies. The use of direct testing results (level 1 inputs) in material characterization gives a more realistic constitutive model, which consequently gives a better understanding of the mechanical behaviour of the material (in terms of material non-linearity) (Abaqus Inc., 2003; Mallela et al., 2004). While the level 2 (correlation input methods) only gives room for obtaining limited parameters (such as M_R, Poisson ratio) which therefore, results in the use of linear material characterization and are basically considered for preliminary study. Using any of the direct test results requires at least one to two laboratory tests for calibration in the FE model. Additionally, these test results are used in obtaining the M_R and further inputted into various constitutive models in the FE model for the characterization of the material in question.

Over the years, various models have been developed for obtaining M_R through Triaxial laboratory results. Table 3.2 suggests a few of the several models available. Overall, amongst the listed models in Table 3.2, the LTTP model, – a modification of the Universal model – is adopted in the NCHRP 1-37A Design Guide (United States Department of Transportation – Federal Highway Administration (USDT-FHA) 2014), thus will be considered in this study based on its general acceptance. Further study can be found on these various models for M_R calculation in a report by George (2004). However, these models are affected by important parameters such as Atterberg limits, grain size distribution, moisture content and density, which are used in the calculation of coefficients (k) to form regression analysis (George, 2004; Dione, Fall, Berthaud, and Makhaly, 2013; Ji et al., 2014). Also, the result in terms of M_R obtained is inputted in constitutive material models in the FE Model.

As mentioned earlier (section 3.4.1.2), the two constitutive material models are Elasticity and Plasticity, but the Plasticity model has got various models which can be used as a close representative of non-linearity of geotechnical materials such as gravel and soil (Abaqus Inc., 2003; Shafabakhsh et al., 2013a). However, out of the various Plasticity models (Viscoelastic, D-P, M-C, Modified Cam-Clay, Modified Cap model, etc.), the D-P and M-C Plasticity model had been considered to be a better representation for base, sub-base and sub-grade layer materials in pavement. Yet, more consideration has been given to D-P because of its capability to model material behaviour in high stresses, volumetric shear and strain (Peng and He, 2009; Ti et al., 2009; Al-Azzawi, 2012; Shafabakhsh et al., 2013a; Maharaj and Gill, 2014) and simplicity (Al-Khateeb et al., 2012), therefore, it is considered in this study.

D-P model is a Plasticity model and a modified version of Mises criteria which is approximate to M-C criterion for simulating frictional materials (Abaqus Inc., 2003; Peng and He, 2009). In this model, there is a period of purely elastic response, after which some material deformation is not recoverable (plastic), thus it should be used along with Elasticity models, which makes this model elasto-plastic in nature (Abaqus Inc., 2003; Abaqus, 2013; Shafabakhsh et al., 2013a). The D-P model has a choice of three different yield criteria, such as: linear, hyperbolic and a general exponent form (Abaqus Inc., 2003; Abaqus, 2013). Nevertheless, the most common of the three yield criteria is the exponent form, which provides the most flexibility in matching Triaxial test data, such that Abaqus® determines the material parameters required for this model directly from the Triaxial test data, thus minimizing relative error (Abaqus, 2013). However, D-P is not non-linear, yet according to Rodriguez-Roa (2003), there is no much difference between non-linear elastic and elasto-plastic behaviour, thus, the elasto-plastic model such as D-P can be used as a close representation of non-linearity in pavement materials.

Furthermore, the yield criteria for the general exponent form provide the most general yield criteria available which is expressed in equation 3.1. Overall, other parameters used in the D-P model – such as: Dilation angle (ψ), Flow-stress ratio (Κ) – can be determined by the M-C model.

$$F = aq^b - p - p_t = 0 \text{..............................Equation 3.1}$$

Where: F = Yield surface

ab = Constant with respect to stress

p = Mean normal stress

p_t = Hardening parameter that represents the hydrostatic tension strength of material

q = Mises equivalent stress

Essentially, various research studies have been done on the layers in flexible pavement via FEM. Yet granular materials do not feature strongly, as more focus is given to designing the asphalt layer and sub-grade condition (Araya, 2011). Similarly, only limited work has been done on stabilized base and sub-base layers (Peng and He, 2009). Hence, stabilized granular material as a base layer will be considered. As earlier mentioned in introduction, FEM can be applied in two ways: 2D and 3D. However, the use of 3D appears to be the best approach (Wang, 2001; Sukumaran, 2004; Rahman et al., 2011; Shafabakhsh et al., 2013a). Nevertheless, there are various sources of error in pavement performance predictions and most are more difficult to control than the response model (NCHRP 2004). Therefore, a reality check through validation of results with field testing or available results is of importance.

Table 3. 2. Summary of resilient modulus constitutive models (George, 2004).

S/N	Model	Source of Model	Comments
1	$M_R = k_1 \left(\frac{\theta}{P_a}\right)^{k_2}$	Seed et al. (1967) (k–θ model)	This model does not incorporate the realistic responses of confining and deviator stress in M_R properties.
2	$M_R = k_1 \left(\frac{\sigma_3}{P_a}\right)^{k_2}$	Dunlap (1963)	It does not consider the effect of deviator stress on the M_R.
3	$M_R = k_1 \left(\frac{\sigma_d}{P_a}\right)^{k_2}$	Moossazadeh and Witczah (1981) (k-σ_d model)	Adequate for cohesive soils but does not consider the effect of confining stress on M_R for clay soils.
4	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3}$	May and Witczah (1981)	It describe the nonlinear behaviour in Triaxial test by considering the effect of shear stress, confining stress and deviator stress in terms of bulk and deviator stress.
5	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}\right)^{k_3}$	Uzan (1992) (Universal model)	Universal cause of the ability to conceptually represent all types of soil from pure cohesive to non-cohesive.
6	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left[\left(\frac{\tau_{oct}}{P_a}\right) + 1\right]^{k_3}$	Yau and Quintus (2002) (LTTP model)	It combines both the stiffening effect of the confinement or bulk stress.

Where; M_R= Resilient modulus; θ = Bulk stress ($\sigma_1 + \sigma_2 + \sigma_3$); P_a = Atmospheric Pressure; σ_d = Deviator stress; σ_3 = Confining stress; τ_{oct} = Octahedral stress ($\sqrt{\frac{2}{3}}\sigma_d$); k_i= Regression coefficient

3.5. 2D versus 3D Analysis

The 2D FEM analysis generally assumes plan strain or axis-symmetric condition in the development of the model (pavement structure) utilizing the horizontal and vertical (X and Y) dimensions. Comparing this against the layered elastic method, it is more practical as it considers material anisotropy, non-linearity and variety of boundary conditions, yet the method is challenged by some limitations as it cannot accurately capture non-uniform tyre contact and multiple wheel loads (Stoffels, Solaimanian, Morian, and Soltani, 2006; Rahman et al., 2010). In contrast, 3D FEM analysis is more of the real-life representation of the pavement structure utilizing the horizontal, vertical and depth (X, Y and Z) dimensions – not only that, it has the ability of accounting for multiple wheel loads as well as moving wheels (Wang, 2001).

Effectively, through the use of 2D FEM analysis programs such as DSC2D, JULEA, MICHPAVE, ILLIPAVE and ABAQUS® (NCHRP 2004), investigation of flexible pavements' responses has been done and also utilized for single-wheel load analysis (Harichandran, Yeh and Baladi, 1989; Sukumaran, 2004). Al-Khateeb et al. (2011) predicted rutting in flexible pavement using 2D FEM: results show that the use of linear-elastic models to predict stresses and strains in pavement structures can lead to significant errors and rut depth increases with decreasing sub-grade strength. Further, Tiliouine and Sandjak (2014) used 2D axis-symmetric in simulation of granular materials behaviour on the basis that it can adequately represent the granular material non-linearity under various stress conditions. On a comparison note, Cho, McCullough, and Weissmann, (1996) present 2D axis-symmetric as a good alternative over 2D plan strain and 3D, when traffic load is away from the edges, considering the failure of 2D plan strain in calculating the appropriate stress distribution and high computation resources of 3D, but it must be noted that axis-symmetric cannot model moving traffic load only static loading. Likewise, Hua (2000), using both 2D and 3D model to predict surface profile under 5000 wheel passes, shows that there is no significant difference (< 2 percent) between the two models.

Although, the 2D FEM analysis has been adequate for the study of nonlinear analysis, the 3D FEM is believed to be used for more accurate pavement responses

(Kim, 2007; Al-Khateeb et al., 2011). Considering this fact, 3D via Abaqus® has been used in the study of flexible pavement under spatially varying tyre/contact pressure by which 2D is limited (Wang, 2001; Rahman et al., 2011). The 3D-based EverStressFE1.0 Software for analysis of flexible pavement was developed by Davids (2009), addressing the shortcomings of traditional analysis software packages such as EverStress, mePADS, BISAR, or KENLAYER. Further, through Abaqus® 3D, Zaman, Pirabarooban, and Tarefder (2003) developed a FEM to simulate the laboratory testing of asphalt mixes in asphalt pavement analysis for rutting; results show that the speed of moving load has a significant effect on predicted rutting. Shafabakhsh et al. (2013a) via Abaqus® presented the influence of asphalt thickness on settlement of flexible pavement: an increase in the pavement thickness and a decrease displacement value. Also, Shafabakhsh et al., (2013b) reported the consistency of results for a 3D model with moving load impact on the tensile strain at the bottom of the asphalt layer when compared with that of field measured pavement responds, and similarly, Abaza (2007) discovered that cyclic and non-linear materials give results close to field measurement results. In 2011, Rahman et al., using Abaqus® 3D FEM, study a preliminary research of traffic-related factors in the design of flexible pavement under specific material properties, model geometries, etc.

Conclusively on 3D, Peng and He (2009) simulated the design and construction process of flexible pavement with cement-stabilized base layer using ADINA FEM software. However, the construction process has little effect on the outcome of results, yet the use of 3D is encouraged, based on its ability for layer-contact modelling. As mentioned earlier, the use of 3D is a drawback because it’s difficult, high demand in data preparation and computation time (Wang, 2001; Zafar et al., 2005; Al-Khateeb et al., 2011). In view of this, through Abaqus®, Sukumaran (2004) tried to discover a less computationally intensive 3D model that would still maintain accuracy; as a result, the use of 3D symmetric model was presented as a suggestion on mesh construction, mesh refinement and element aspect ratio. Besides, the newer versions of the 3D software have been improved by making it user-friendly and interactive and overall increased speed in the analysis time.

3.5.1. Motivation for 3D FEM

Among the three models of representation in FEM analysis (2D plan strain, 2D axis-symmetric and 3D), the 3D FEM model is used to achieve the aim of this research work. According to the above review, the following are the tangible reasons why 3D FEM is used:

- Its ability to capture the effect of non-linear materials or the effect of combination of loads (Abaza, 2007; Shafabakhsh et al., 2013b).
- Its capability to account for multiple wheel load as well as moving wheel load (Wang, 2001; Zaman et al., 2003).

3.5.2. Comparative Study on FEM Software

Quite a number of FEM software programs are available and many have been found useful for pavement design purposes. Basically, in pavement design, there are two major categories of FEM software, the general purpose and the specific purpose software (NCHRP 2004). The general purposes are those with a wide range of applications aside from pavement design, in areas such as medicine, lamella mechanics, hydrodynamics, soil mechanics, structural mechanics; examples are Abaqus®, ADINA, ANSYS and DYNA3D, while the specific purposes are developed particularly for analysis of pavement design. Examples are EverStressFE, ILLI-SLAB, ILLI-PAVE and MICH-PAVE. Various successes have been recorded through the use of the aforementioned software, yet the general purpose is more powerful and capable of conducting 3D non-linear dynamic analysis perfectly. Additionally, it provides optimum flexibility to manipulate a variety of FE models with sophisticated geometry and boundary conditions (Wu, Chan and Young, 2011). Further, NCHRP (2004) did a comparative study on the software regarding issues of efficiency issues and operational issues; in that study, Abaqus® is considered as a potential candidate based on its technical capabilities and its extensive past usage in research oriented pavement analysis, but it was disregarded because of its high licensing costs and restrictive licensing terms. Yet, Abaqus® has wide applications in the aspect of pavement design; this software is introduced and used in this study.

3.5.3. *Abaqus® as FEM Software*

Abaqus®, a general purpose and commercial FEM modelling software, has widely been applied for pavement analysis. As mentioned above, in Finite element simulation (section 3.4), it contains three major process stages: pre-processing, processing and post-processing. In 1990, Chen Marshek, and Saraf, comprehensively studied various pavement analysis programs and showed that the results from the Abaqus® program were comparable to those from other programs. Also, from the above review (section 3.4.3 and 3.4.5) on FEM it can be seen that Abaqus® has been preferred above others.

Further, Abaqus®/CAE is a complete Abaqus® environment that provides a simple, consistent interface for creating, submitting, monitoring and evaluating results from Abaqus®/Standard, Abaqus®/Explicit simulation and others (Abaqus, 2013). It is divided into modules and in each module logical representation of the modelling process can be defined such as defining the geometry, generating a mesh and assigning material properties, etc. The model built in Abaqus®/CAE generates an input file to submit to the Abaqus®/Standard or Abaqus®/Explicit analysis product. The analysis product performs the analysis, sends information to Abaqus®/CAE to allow the progress of the job to be monitored, and generates an output database. Finally, the visualization module of Abaqus®/CAE is used to read the output database and view the results of the analysis (Abaqus, 2013).

Furthermore, Abaqus® is a modular code consisting of a library of over 300 different element types and a comprehensive material model library with materials ranging from linear to nonlinear and isotropic to anisotropic behaviour, which are useful for pavement analysis and also allows for the introduction of new materials through its user-defined sub-route (Rahman et al., 2011; Abaqus, 2013). Also, it is capable of analysing a variety of problems (linear, nonlinear, static, dynamic, structural and thermal) (Britto, 2010). Overall, ABAQUS® usage is enhanced by its friendly and interactive user-guide which is available in PDF and HTML version.

Motivation for Abaqus®

ABAQUS® will be used in this research because of its capabilities in solving pavement engineering problems:

- Material modelling as linear and nonlinear elastic, viscoelastic, and elasto-plastic in 2D and 3D analysis
- Static, harmonic dynamic and transient dynamic loading simulation
- Contact/Interface modelling with friction
- Cracking propagation modelling (Abaqus, 2013)
- Analysis which involve temperature gradient (Sukumaran et al., 2004; Britto, 2010; Shafabakhsh et al., 2013a).

3.6. Failure Criteria in Numerical Simulation

Failure criteria based on fracture mechanics have been developed for pavement layers, with the aim of enhancing design to provide sufficient resistance to pavement failure (Mamlouk and Mobasher, 2004). This analysis requires models which relate the output from FEM or elastic-layered analysis (stress, strain, or deflection) to pavement behaviour in terms of performance, cracking, rutting, roughness and life span (Ekwulo and Eme, 2009). It is one of the empirical portions of M-E design and also known as damage models (SANRAL, 2013b). Equations used for these models are derived from observation and performance of pavement with relation to observed failure and initial strain under various loads, thereby computing the number of loading cycles to failure (Pavement Interaction, 2008). Various types of failure criteria exist depending on the type of pavement layer in question, such as: Asphalt surface – Fatigue cracking; Unbound granular base and sub-base layer − Permanent deformation; Cemented base and sub-base layers − Crushing failure, Effective fatigue and Permanent deformation; Sub-grade – Permanent deformation or rutting. Nonetheless, two are widely recognized: fatigue cracking in asphalt and deformation in the sub-grade (Pavement Interaction, 2008; Ekwulo and Eme, 2009; SANRAL, 2013b).

In South Africa, failure analysis has been checked through damage models suggested by SAMDM (1996), but according to SANRAL (2013b), (1996) SAMDM fatigue transfer functions for asphalt are not that reliable and permanent deformation transfer functions for granular materials are on the conservative side. As a result, the SAMDM damage model is out-dated (SANRAL, 2013b), therefore, it is appropriate to consider other damage models, such as: Shell (Huang, 2004), Transport and Road

Research Laboratory, Asphalt Institute (Asphalt Institute, 1982), etc. However, the fatigue criterion in the M-E approach is centred on limiting the horizontal tensile strain at the bottom of the asphalt layer due to repetitive loads on the pavement surface. If this strain is excessive, it will result in cracking (fatigue) of the layer (Ekwulo and Eme, 2009), and the relationship is given in the equation 3.2 by Asphalt Institute (Asphalt Institute, 1982), which is commonly accepted. Permanent deformation can initiate in any layer of the structure, making it more difficult to predict than fatigue cracking (Pavement Interaction, 2008). However, critical rutting can be attributed mostly to a weak pavement layer (sub-grade). This is typically expressed in terms of the vertical compressive strain at the top of the sub-grade layer and is given by Asphalt Institute by equation 3.3.

$$N_f = 0.0796(\varepsilon_t)^{-3.291}(E)^{-0.854}$$ ……………………… Equation 3.2

Where: N_f = Number of repetitions for fatigue cracking

ε_t = Tensile strain at the bottom of the asphalt surface in *microstrain*

E = resilient modulus of asphalt in *psi*

$$N_r = 1.365x10^{-9}(E_c)^{-4.477}$$ ……..……………………… Equation 3.3

Where: N_r = Number of repetitions for sub-grade rutting failure

E_c = Compressive strain on top of the sub-grade.

Overall, the failure analysis models are used to define the point at which failure occurs in a pavement by determining the incremental damage.

3.7. Summary

Chapter Three of this dissertation presented reviews on numerical simulation of flexible pavement. Away from the empirical method of design, numerical simulation uses the level of stress, strain and deflection in the design of pavement structure. However, there are two major approaches in numerical design of pavement, but the effectiveness of FEM in predicting the stress and strain gives it an edge over the layered elastic method. 3D FEM has its own challenges of input parameters and computational time, but reviews have shown that it is the more preferred of the two FEMs because of its ability to design more complex problems relating to the actual conditions of pavement structures.

Furthermore, various material characterization inputs (level1 and 2 inputs) were explored and thereafter, failure criteria in numerical simulation were considered and the Asphalt Institute model for both fatigue and rutting in terms of number of repetitions before failure would be used in this study. Abaqus® provides user flexibility and as a result; it has wide attention in pavement design. Therefore, based on the useful information from this chapter, the following study methods will be presented in the next chapter:

- Development of the geometry model for representation/description of a fly-ash- stabilized base layer in a typical South African road through Abaqus®;
- Appropriate selection of a material model for the accurate characterization of the stabilized base layer; and
- Appropriate selection of boundary conditions and loading analysis that suitably represent the already available laboratory results.

CHAPTER 4: DESIGN AND SIMULATION

4.1. Introduction

This chapter establishes detail on the steps for development of FEM and its analysis. Likewise, it presents a description of two FE models in line with the set objectives and a comparative analysis of the laboratory and FEM results. The first model was used to validate the efficiency of using 3D FEM over axisymmetric in the design of pavement structure; it was also used to examine the structural response of a cement-fly ash-stabilized base layer in terms of the stresses and strains on the top of the sub-grade, while the second model was developed to evaluate the protective importance of surface layer over stabilized base layer by estimating the tensile strain at the bottom of the surface layer and the surface of the sub-grade. Thirdly, a comparative analysis of the non-linear and linear material characterization will be undertaken. Lastly, the results obtained from FEM analysis will be compared with the already available laboratory empirical results for validation. Overall, all the models were developed using Abaqus® 6.13 software.

4.2. Development of Flexible Pavement Model

Abaqus® analysis modules starts with a batch program, with the objective of assembling an input file which describes a problem so that Abaqus® can provide an analysis (Liang, 2000). The input file for Abaqus® contains model data and history data. Model data defines a FEM in terms of geometry, element properties, material definitions and any data that specifies the model itself (Liang, 2000; Britto, 2010; Abaqus, 2013). Further, the history data define what happens to the model and the sequence of loading for which the model's response is sought, including the procedure type, control parameters for time integration or non-linear solution procedures, loading and output request (Liang, 2000; Britto, 2010; Abaqus, 2013). Data can be defined by the user with relevant option blocks provided in the modules (Abaqus, 2013).

Applying the file, Abaqus® automatically controls the time step and increments of the load and records the message and data in all the analysis procedures according to data defined in the file; afterwards the results are obtained by using Abaqus®/post. Overall, there are two basic methods of inputting data into Abaqus® software (Abaqus, 2013) which are

- Input file usage
- Abaqus®/CAE usage

However, of these two input methods, Abaqus®/CAE usage is preferred because of simplicity in terms of not including code writing, so it is used in this study. The general steps for the development of flexible pavement through Abaqus®/CAE usage input method are summarized in Figure 4.1 and briefly explained thereafter.

4.3. Abaqus®/CAE Usage for Flexible Pavement Model

In Abaqus®/CAE usage, there is no particular order for modelling of a member in Abaqus®, but in any FEM analysis the input (in terms of geometry) are basically considered first. Since a conventional flexible pavement which contains surface-, base- and sub-grade layer is used in this study; the use of this 3-layer pavement system is to properly understand the behaviour of the stabilized base layer without the interference of other layers such as a sub-base. The part module is used to input the pavement layers' geometries by creating a 2D sketch which is extruded in 3D, with other features such as partitioning, generating meshes for parts, creating sets and assigning of names to all members. On the material properties module, the characterization of each part, such as surface, base and sub-grade layer, are inputted; thereafter, via the assembling and steps module, the parts are put together to form a composite conventional pavement structure. The step module is used to capture changes in the loading and boundary conditions with respect to the parts' interaction with each other.

Various forms of contact interaction (mechanical and thermal) occur in pavement, thus the interactions and load module are used to define the interface in line with the already created steps in the step module; the load defines the transferred load (traffic load) and boundary condition in an attempt to represent the on-site condition of the pavement. Lastly, job module in Abaqus is used to submit, analyse and monitor the created pavement model and afterwards the results are viewed in the visualization module. Conclusively, in using Abaqus® software, careful attention should be given to all the modules to avoid warnings and prevent errors.

4.4. 3D versus Axisymmetric

4.4.1. Description

This FEM is a scenario of unpaved pavement structures which are developed for a two-layered system (base and sub-grade layer) with the aim of achieving the set objectives which are;

1. To evaluate the efficiency of using 3D FE model for design of flexible pavement.
2. To determine the structural response of stabilized base layers in flexible pavement system due to traffic loads using 3D FE model.

The scenario consists of 16, 18, 20 and 22 percent fly ash with 1 percent cement stabilized base layer over a sub-grade and would be modelled in axisymmetric and 3D FEM. In this model, the thickness of the sub-grade layer is kept constant at a specific depth (2000 mm), while the base layer thickness changes over a range (100 mm – 500 mm). Comparative analyses of the results obtained from axisymmetric and 3D FEM would be undertaken for the structural response of the base and sub-grade layer in terms of:

1. Compressive vertical stresses/strains within the base layer
2. Compressive vertical stresses/strains at the top of the sub-grade layer.

4.4.2. Model Geometry and Material Properties

The axisymmetric model is basically 3000 mm radius with a total depth that varies based on the thickness of the base layer, which changes over a range of 100 mm – 500 mm (Figure 4.2). This geometry, particularly the radius (breadth), is similar to that used by Al-Jhayyish (2014). However, sub-grade depth is infinite, but for the purpose of boundary conditioning it is assumed to 2000 mm (Rahman, 2011), since there is no deformation after a certain depth. On material properties, level 2 input methods (section 3.4.1.2) would be used and material properties of the stabilized base layers were obtained from laboratory testing (UCS) conducted by Heyns and Mostafa Hassan (2013).

The UCS results used were those of 16, 18, 20, 22 percent fly ash with 1 percent cement, where AFRISAM was the cement and Pozzfill as the fly ash (Table 4. 1) (Heyns and Mostafa Hassan, 2013). These material properties are obtained using correlation formula by Barenberg (1999) (section 3.4.2) and other material properties are selected from SANRAL (2013b), as presented in Table 4. 1 and 4.2. All material properties are assumed to be linearly elastic for simplicity as non-linear properties require many input parameters which are not readily available (Al-Jhayyish, 2014).

The 3D FE model utilizes 3000 mm length by 3000 mm breadth with the total depth varying based on the thickness of the base layer as in the axisymmetric model (Figure 4.3). This geometry is similar to that used by Ahmed (2006), with the aim of avoiding edge error when loaded. Materials properties are all assumed to be linearly elastic, thus a static linear perturbation analysis procedure type will be used. These material properties are presented in Table 4. 1 and Table 4.2; these data were utilized to define the material properties of the model layers in ABAQUS®.

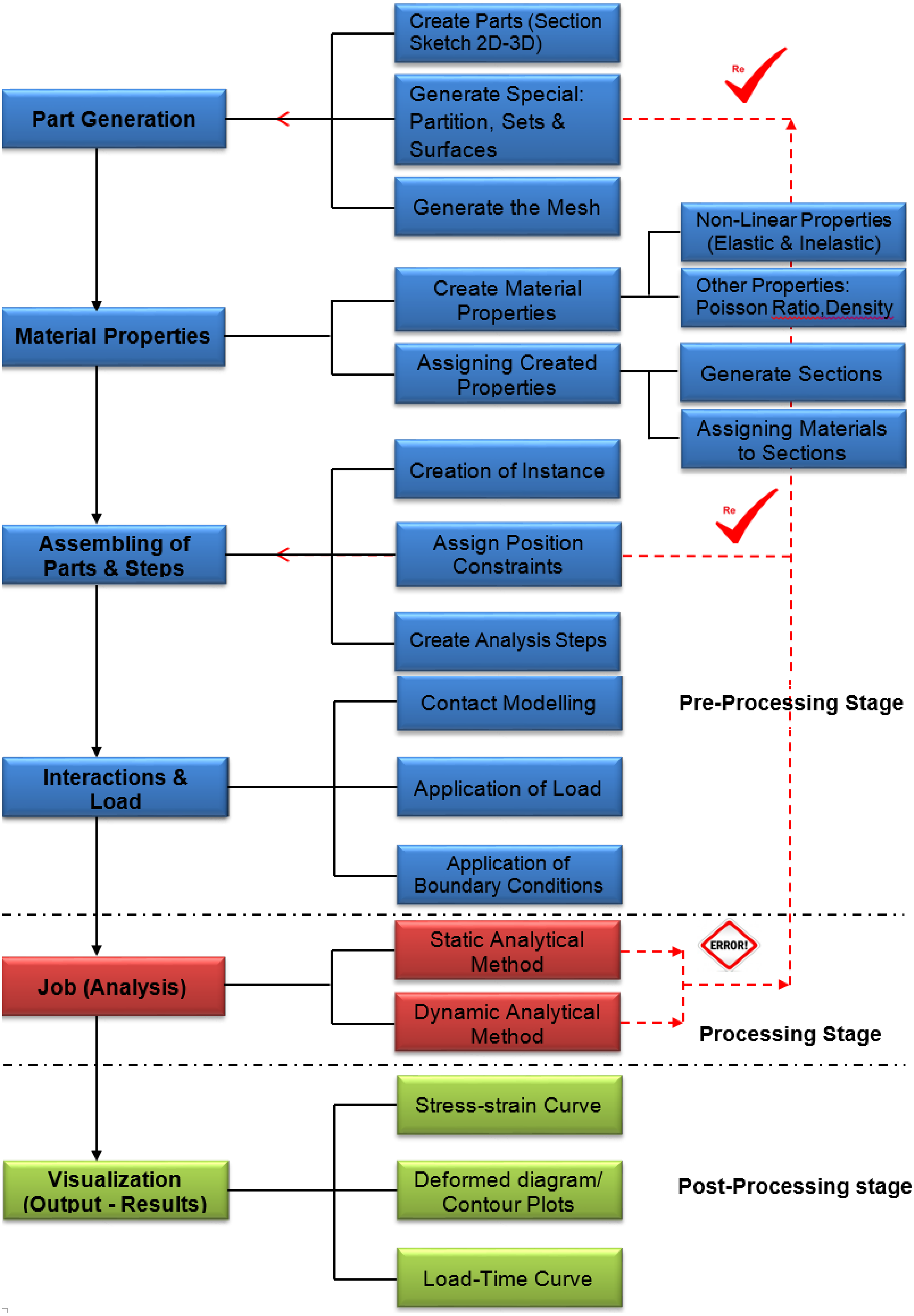


Figure 4.1. General steps for the development of flexible pavement (Abaqus®/CAE usage)

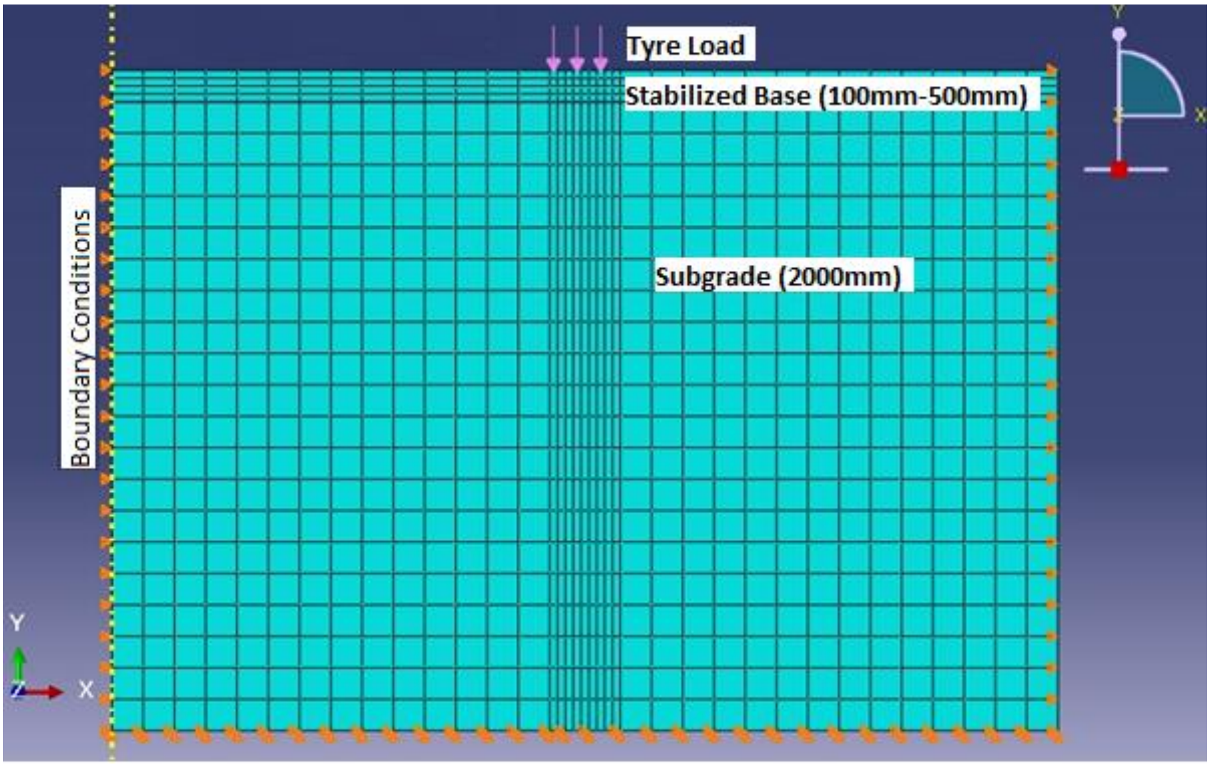


Figure 4.2. Axisymmetric model geometry of the stabilized base and sub-grade layer with meshing, load and boundary conditions

4.4.3. Model Mesh and Element types

In the axisymmetric model, 4-node bilinear axisymmetric quadrilateral elements (CAX4R) with reduction integration were used. The stabilized base and sub-grade layer were seeded at 0.025 m at the loading area because displacement gradients are higher in this region, while other areas were seeded at 0.1 m; as a result, meshes are fine in/near loading area and coarse at distances away from applied load for efficient modelling, as suggested by Peng and He (2009) and Tiliouine and Sandjak (2014) (Figure 4.2). The total number of elements range from 888 – 1480. For the 3D FEM model, 8-node solid continuum elements (C3D8R) with reduction integration were used; similarly, the stabilized base and surface layer were seeded as in axisymmetric model, overall the total number of elements range from 19443 – 36075 (Figure 4.3).

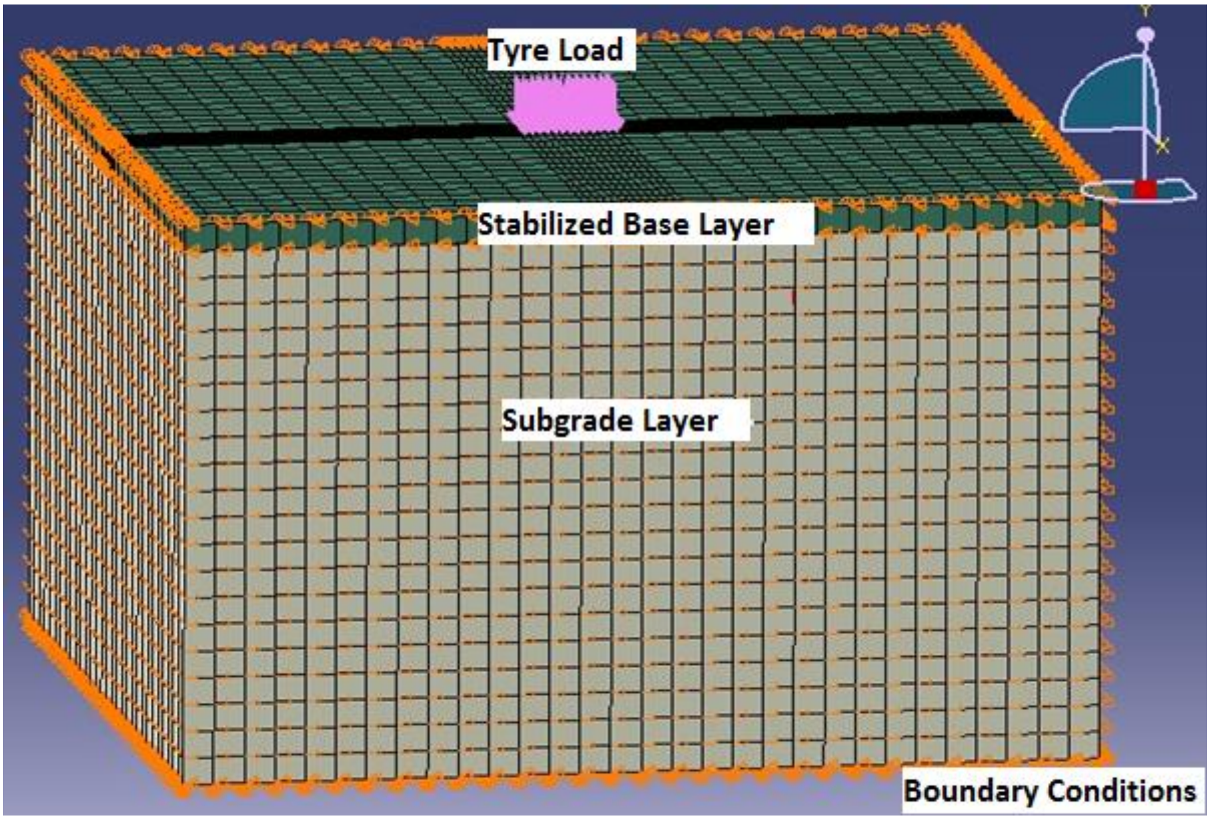


Figure 4.3. 3D model geometry of the stabilized base and sub-grade layer with meshing, load and boundary conditions

4.4.4. Boundary Conditions and Loading

The pavement layers were assumed to bond together perfectly; although in reality full bond is not always achieved (Sutanto, 2009) but proper distribution of stresses, strains and deflections between the layers, it is assumed to be perfectly bonded. Also, the models are fixed at the bottom of the sub-grade and roller constraints on the vertical boundaries (i.e. the model can move only in y-direction) (Figure 4.2 and 4.3). On loading (section 3.4.1.3), a static standard equivalent single-axle load with dual tyres was used in these models, since Wu et al*.* (2011) specified that the maximum stress at a specific point in the pavement occurs when the wheel load is directly above it, while the stress can be assumed at zero when the load is quite far from that point. In an axisymmetric model, the breadth of tyre load (224 mm) proposed by Huang (2004) (Figure 4.2) was used while, in the 3D model contact area of 72557 mm^2 (Figure 4.3) with a rectangular area of contact was placed above the stabilized layer (Huang, 2004; Al-Jhayyish, 2014). These loads were standard equivalent single-axle load (80 kN) with dual tyres and applied uniformly with a pressure of 0.65 MPa in accordance with South African standard (TRH 4, 1996; Theyse et al., 2011). Conclusively, this analysis will be run as a static linear perturbation analysis procedure type.

Table 4. 1. Material properties of the stabilized base layer (obtained from Heyns and Mostafa Hassan, 2013)

Stabilized Base (percent Flyash+1 percent Cement)	Material code (Colto, 2008)	USC (Kpa)	Modulus of Elasticity (MPa)	Poisson's Ratio
16	C3	3310	3972	0.35
18	C3	2133	2560	0.35
20	C3	3830	4596	0.35
22	C3	2298	2758	0.35

Table 4.2. Material properties of other pavement layers

Layer	Material code (Colto, 2008)	Modulus of Elasticity (MPa)	Poisson's Ratio
Surface	AG	3000	0.44
Granular Base	G5	200	0.35
Sub-grade	G10	45	0.35

4.5. Paved Stabilized Base Layer

4.5.1. Description

A scenario of paved flexible pavement is developed for a three-layered system of the pavement structure, which are: asphalt surface, 18 percent fly ash with 1 percent cement stabilized base, and sub-grade layer. The 3D FEM was used in the development of these models. The thicknesses of the stabilized base and sub-grade layer were kept constant at a specific depth in accordance with results from 3D vs axisymmetric case (300 mm and 2000 mm respectively). Here, the protective importance of surface layer over stabilized base layer will be evaluated in terms of:

1. Surface layer deflection;
2. Tensile strain at the bottom of the surface layer;

3. Compressive vertical stresses/strains within the base layer; and
4. Compressive vertical stresses/strains at the top of the sub-grade layer.

4.5.2. Model Geometry and Material Properties

A 3D model with 3000 mm length by 3000 mm breadth and the total depth varying based on the thickness of the surface layer over a range of 25 mm–100 mm was developed. This geometry is also similar to that used by Ahmed (2006), with the aim of avoiding edge error when loaded. The material properties and analysis procedure type are similar to the above, with properties for 18 percent fly ash with 1 percent cement stabilized as the material for the base layer (Table 4. 1).

4.5.3. Model Mesh and Element type

In order to keep the size of the problem manageable in terms of analysis time and storage capacity (Saad, Mitri and Poorooshasb, 2006), the meshing is fine in/near loading area and coarse at distances away from applied load; this is similar to those in the 3D vs axisymmetric case. Additionally, 8-node solid continuum elements (C3D8R) with reduction integration were used; as they have the capability of representing large deformation and material nonlinearity.

4.5.4. Boundary Conditions and Loading

Similarly, pavement layers were also assumed to bond together perfectly and the models are fixed at the bottom of the sub-grade and roller constraints on the vertical boundaries (Figure 4.3) (section 4.4.4). Here, a rectangular contact area of 72 557 mm^2 was placed on the asphalt surface layer and was applied uniformly with a pressure of 0.65 MPa (Theyse et al., 2011).

4.6. Non-Linear versus Linear Material Characterization

A comparative analysis of the two material characterizations input in the FE model is undertaken here. Although, the linear material characterization method has been used in the first two analyses, which is justifiable by the fact that results are easily available. In this analysis, a scenario (Non-Linear Material model) of paved flexible pavement is developed for a three-layered system of the pavement structure: asphalt surface, 18 percent fly ash with 1 percent cement stabilized base, and sub-grade layer. The model geometry, model mesh and element type, and boundary conditions and loading are the same as in the paved stabilized base layer (section 4.5), with the introduction of non-linear material characterization for the stabilized base layer in a static-general analysis procedure, so as to consider the non-linear effect.

As mentioned in the review (section 3.4.3), LTTP model (Yau and Quintus, 2002) which was adopted in the NCHRP 1-37A Design Guide (USDT-FHA 2014) was used in obtaining the M_R (1301 MPa) for 18 percent fly ash with 1 percent cement-stabilized base layer, parameters such as (bulk stress = 1854kPa) are obtained from Heyns and Mostafa Hassan (2013) and regression coefficients (k_1 = 3000psi and k_2 = 0.5) suggested by AASHTO (as cited in USDT-FHA 2014). Coupled with the elastic model, D-P plasticity model in Abaqus was used for the material characterization to be non-linear. In the D-P model, the shear criterion is selected to be 'exponent form' so as to allow for the use of sub-option (Triaxial test data) (Appendices A1 – C1) and the dilation angle is assumed to be 15^{o}. Furthermore, to validate the results obtained from D-P model, a quick M-C model will be run in the model as well. Thereafter, the results obtained will be compared with those obtained for linear material characterization.

4.7. Comparative Analysis

The fourth objective of this dissertation is to compare laboratory-test empirical results already available against that of 3D FEM. A comparative analysis of the empirical, multilayer linear elastic software (mePADS) (Theyse and Muthen, 2000) and 3D FEM (Non-Linear and Linear Material) results for a paved three-layered system with 18 percent fly ash plus 1 percent cement stabilized base layer is carried out. However, estimating the structural capacity of flexible pavement through empirical methods can be undertaken by the following: Pavement Structural Number, dynamic cone penetrometer, 1993 AASHTO Structural Number (SN), etc. (SANRAL 2013b). Nevertheless, the use of 1993 AASHTO SN is widely accepted, yet it has its own disadvantages based on its assumptions (Pavement Interactive, 2008; SANRAL, 2013b). This method is based on the results of the AASHTO road test executed in

Ottawa, Illinois during the late 1950s to early 1960s and can be used for new and rehabilitation pavement design. AASHTO SN empirical method is presented by equation 4.1:

$$\log_{10}(SC) = Z_R \, X \, S_o + 9.36 \, X \, \log_{10}(SN+1) - 0.2 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40+\frac{1094}{(SN+1)^{5.19}}} +$$

$$2.32 \, X \, \log_{10}(M_R) - 8.07 \quad \text{.. Equation 4.1}$$

Where: SC = Predicted number of 80 kN ESALs

Z_R = Standard normal deviate

S_o = Combined standard error of the traffic and performance predictions

SN = Structural number of the total pavement thickness

ΔPSI= Difference between the initial (PSI_0) and terminal (PSI_t) serviceability indices

M_R = Sub-grade resilient modulus (in psi)

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \; .... \quad \text{.............................Equation 4.2}$$

Where: SN = Structural number of the total pavement thickness

a = i^th layer coefficient (per inch) (Table 4.3)

D = i^th layer thickness (inches)

m = i^th layer drainage coefficient; assumed = 1.0

Table 4.3. Layer coefficients (SANRAL, 2013b)

Materials	Ranges for South African Materials
Asphalt concrete	0.20 – 0.44
Crushed stone	0.06 – 0.14
Cemented-treated material	0.10 – 0.28
Bituminous-treated material	0.10 – 0.30

Using the above equation, the structural capacity (in terms of ESALs) of flexible pavement is calculated. To use this equation, the following input assumptions were extracted from AASHTO design procedure (1993), Pavement Interactive (2008), and

SANRAL (2013b); the pavement was assumed to be a category B with the following characteristic: reliability = 90 percent ($Z_R = 1.282$), S_o = 0.45, total equivalent traffic loading = 0.3 – 10 X 10^6, PSI_0 = 4.5 and PSI_t = 2.0 and M_R = 45 MPa (Table 4.2). Careful consideration should be given when using this equation as it is in imperial units. Results obtained for empirical method (AASHTO SN) were compared with those obtained for the 3D model in paved stabilized base layer. Furthermore, the use of mePADS (see Appendices A2 – F2 for inputs data) serves as a check for the performance of the 3D models. mePADS is mechanistic pavement design software, which combines a stress-strain computational engine with pavement material models and it's capable of analysing pavement for bearing capacity. mePADS generates outputs inform of pavement layer lives and contour plots of stresses and strains (CSIR Built Environment, 2009). Although, there are various multilayer linear elastic software but mePADS was selected based on its availability and suitability for South Africa pavement design. Further, table 4.4 presents brief comparison between Abaqus® and mePADS. Yet, recent report states that mePADS is currently been updated since it works with the SAPDM principle which is currently under review (CSIR Built Environment, 2009; SANRAL, 2013b) (see Section 3.3 and 3.6).

Table 4. 4 Comparison between Abaqus® and mePADS

Comparison Criterial	Abaqus	mePADS
Analysis Method	3D and 2D	2D-Axisymmetric
Developer	Abaqus Inc.	CSIR
Development Status	Actively Developed	Under Review
Loading Type Capacity	Static and Dynamic	Static
Operation Techniques	FEM	Multi-layer Elastic Method
Pavement Layer	Non-limited	5 layers maximum
Problem Type Capacity	Linear, Non-linear, etc.	Linear only
Required Disk Space	Very Small Required	A lot of Space Required
Time of Analysis	Seconds to Hours (analysis dependent)	Seconds
Year of Originally Released	1978	2000

4.8. Summary

In this chapter, four different basic scenarios were developed to achieve the set objectives of this study. These scenarios are: axisymmetric versus 3D, paved stabilized base layer, non-linear versus linear material characterization, and a comparative analysis of empirical and 3D FEM results, and a check by mePADS software. Essentially with these scenarios the following will be achieved:

1. The efficiency of using 3D FEM for design of flexible pavement over axisymmetric and the structural response of stabilized base layer in flexible pavement;
2. The structural response effect of asphalt layer over stabilized base layer;
3. Efficiency of non-linear material characterization over linear; and
4. Benefits of 3D FEM design for flexible pavement over empirical methods.

CHAPTER 5: RESULTS AND DISCUSSION

5.1. 3D versus Axisymmetric Results

This model was employed to study the efficiency of using the 3D model for design of flexible pavement and the effect of unpaved stabilized base layer thickness on the vertical compressive strain and stress at the top of the sub-grade. Firstly, from Figure 5.1 and 5.2, it was observed that the addition of stabilizer to natural G5 material decreases the vertical compressive strain and stress at the top of the sub-grade layer. On the other hand, the vertical compressive strain and stress at the top of the sub-grade layer decrease with increase in the thickness of the stabilized layer for axisymmetric model (Figure 5.1 and 5.2). It was observed that the increase in the fly ash percentage was added, which resulted in an increase of the modulus of elasticity of the base contributing to the reduction of vertical strain and stress at the top of the sub-grade layer. Thus, increase in the modulus of elasticity of a layer reduces the vertical strain and stress in the underneath layer. Similarly, the vertical compressive strain and stress at the top of sub-grade layer decrease with the stabilized layer thickness increase in 3D model. However, the results obtained from 20 percent fly ash-stabilized base layer shows better results when compared with others; this results from the high modulus of elasticity of the 20 percent stabilization. However, considering the economical aspect and the fact that beyond 20 percent fly ash strength starts to decrease, thus, 18 percent fly ash is considered best and economical (Figure 5.1 and 5.2). However, it is recommended that a lower stabilization percentage (10 percent–15 percent) should be experimented. Secondly, comparing the results obtained from axisymmetric and 3D model for 18 percent fly ash-stabilized base (Figure 5.3 and 5.4); results show that the 3D model is more efficient, as vertical strain is centralized in the model against that of the axisymmetric which tends to diverge toward a side of the model (Figure 5.5), which is far from reality and overall vertical strains at the sub-grade are smaller. This implies that numbers of load repetitions will be very small for axisymmetric (4.92×10^{3}) when compared with that of 3D (1.30×10^{6}) and ASSHTO SN (11.54×10^{6}) results, taking 300mm stabilized base layer as an example. Thus, the axisymmetric model tends to under-design, which is not economically wise. In both models the stabilized base of

100 mm generated excess strain and stress; this shows that the use of thinning stabilized base layer would quickly result in pavement failure.

Furthermore, sub-grade rutting failure criteria analysis (section 3.5) using Asphalt Institute model equation 2 (section 3.5) (Asphalt Institute 1982) for 18 percent fly ash-stabilized base layer are presented in Table 5.1. The 3D model shows better results, as it is obvious that using a 100-mm stabilized base layer would initiate permanent deformation in the sub-grade layer under some loadings, such as 2.57 load repetitions against the result from axisymmetric model (62.6 number of load repetitions). Thus using thinning stabilized base layer should not be encouraged and overall, proper curing of stabilized base is necessary. Additionally, the 3D model shows an increase in the number of load repetitions to failure for 200 mm–500 mm thickness of base layer over that of axisymmetric model, implying that axisymmetric tends to under-design for deep thickness and over-design for thin thickness. However, using 300-mm stabilized base layer thickness against 100-mm increases the capacity of the structure by approximately 100 percent.

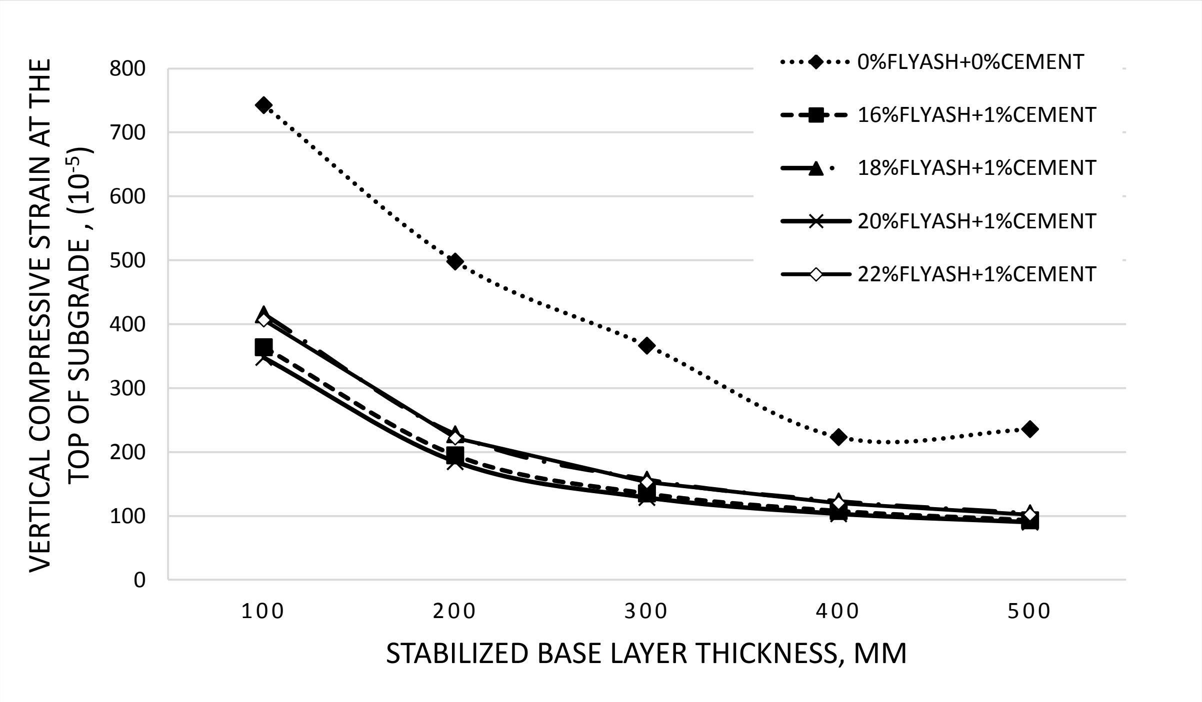


Figure 5.1. Effect of stabilized base layer thickness on vertical compressive strains on the top of sub-grade (Axisymmetric model)

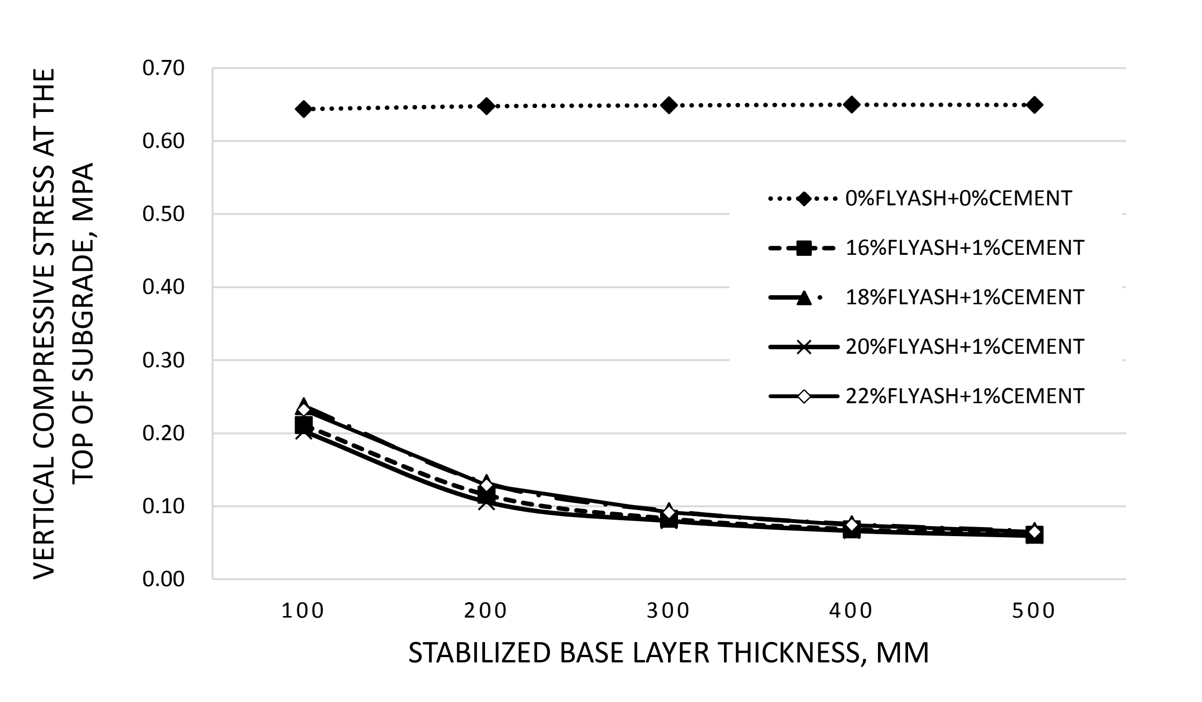


Figure 5.2. Effect of stabilized base layer thickness on vertical compressive stress on the top of sub-grade (Axisymmetric model)

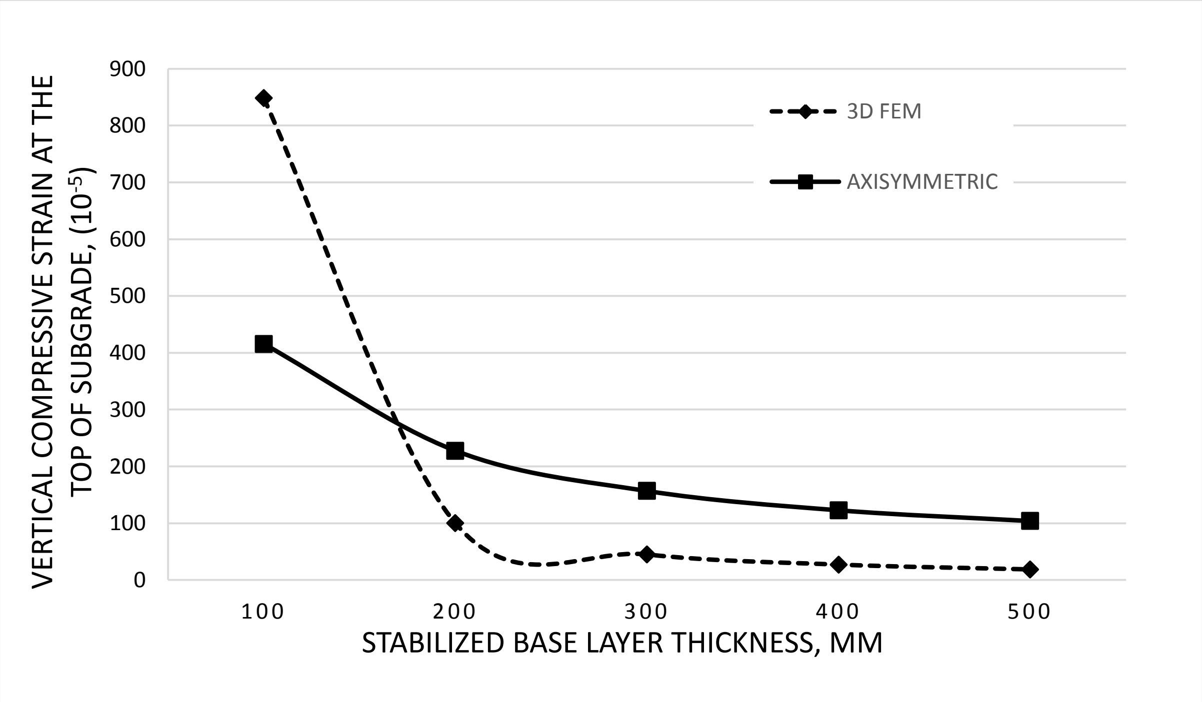


Figure 5.3. Effect of stabilized (18 percent fly ash + 1 percent cement) base layer thickness on vertical compressive strains on top of sub-grade (Axisymmetric and 3D model)

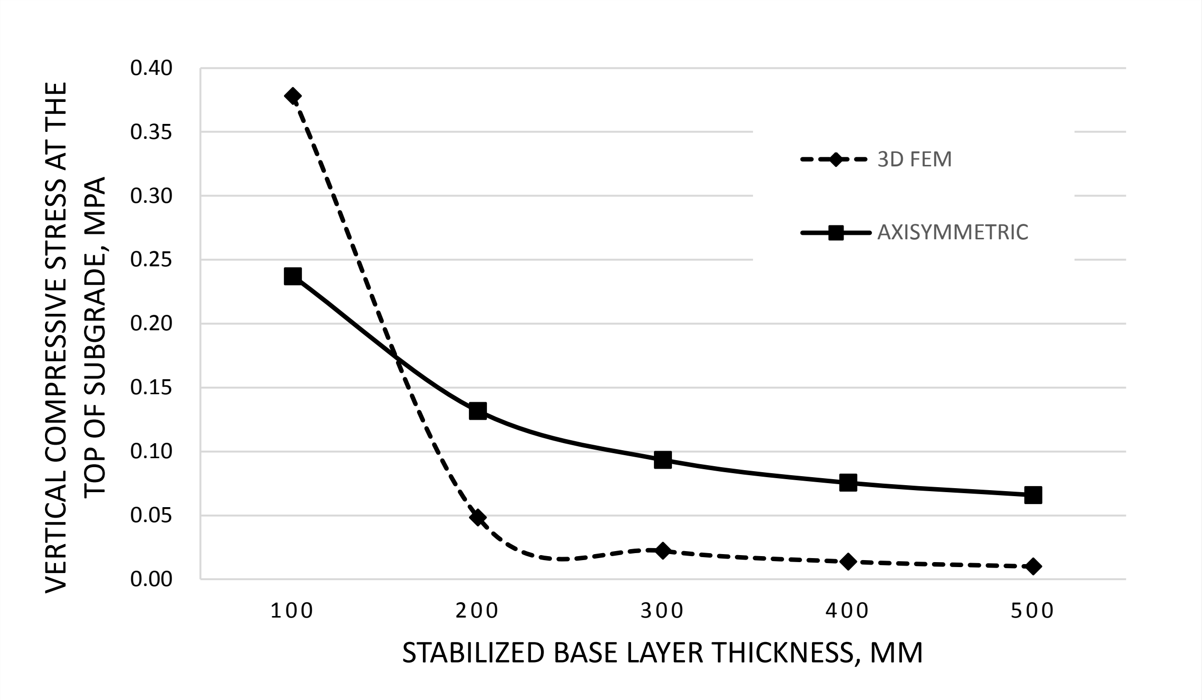


Figure 5.4. Effect of stabilized (18 percent fly ash + 1 percent cement) base layer thickness on vertical compressive stress on top of sub-grade (Axisymmetric and 3D model)

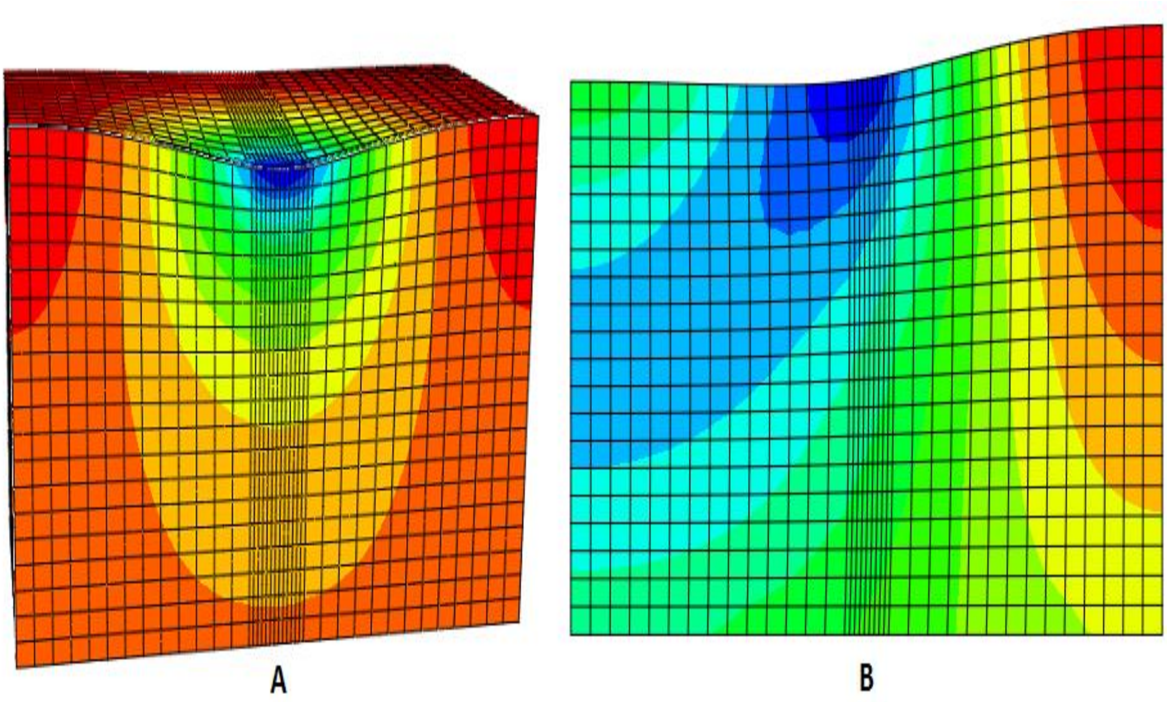


Figure 5. 5 Contour plots showing deformation of vertical compressive strain at the top of sub-grade (A- 3D Model and B- 2D Model)

Table 5.1. Rutting failure analysis based on Asphalt Institute response model (1982) for Axisymmetric and 3D model

Rutting Criterion				
Base Layer Thickness (mm)	**Vertical Strain ε_c (10^{-6}) in Sub-grade (3D)**	**No. of Repetitions to Failure N_r (3D)**	**Vertical Strain ε_c (10^{-6}) in Sub-grade (2D)**	**No. of Repetitions to Failure N_r (2D)**
100	8481	2.57	4155	62.6
200	999.9	36.8×10^3	2276	0.9×10^3
300	451.0	13.0×10^5	1568	4.9×10^3
400	272.7	12.4×10^6	1228	14.7×10^3
500	187.9	65.6×10^6	1039	31.0×10^3

5.2. Paved Stabilized Base Layer Results

The protective importance of surface layer over stabilized base layer is evaluated by the paved stabilized base layer model. On surface deflection, it can be seen from Figure 5.6*;* that asphalt layer-deflection decreases with an increase in thickness, which is related to the conclusion in a study by Shafabakhsh et al. (2013a). The conclusion was that increase in asphalt layer thickness reduces the surface deflection and the other layer. Similarly, vertical compressive stress/strain at the top of the sub-grade layer compared with the unpaved also decreases in the same manner (Figure 5.7 and 5.8). However, the use of 50 mm thickness of asphalt is recommended in developing countries for economic reasons (Araya, 2011).

Similarly, the tensile horizontal strain at the bottom of the surface layer (Figure 5.9) shows a decrease: quite the reverse for 100 mm thickness, as there was an increase in strain; this implies high probabilities for bottom-up fatigue cracking to occur with increase of asphalt surface thickness over a stabilized base layer. Additionally, the stabilized base layer-vertical strain increases initially and tends to decrease on 100 mm thickness of asphalt (Figure 5.10); this implies that an increase in asphalt layer has a significant effect on stresses and strains generated in all layers of flexible pavement.

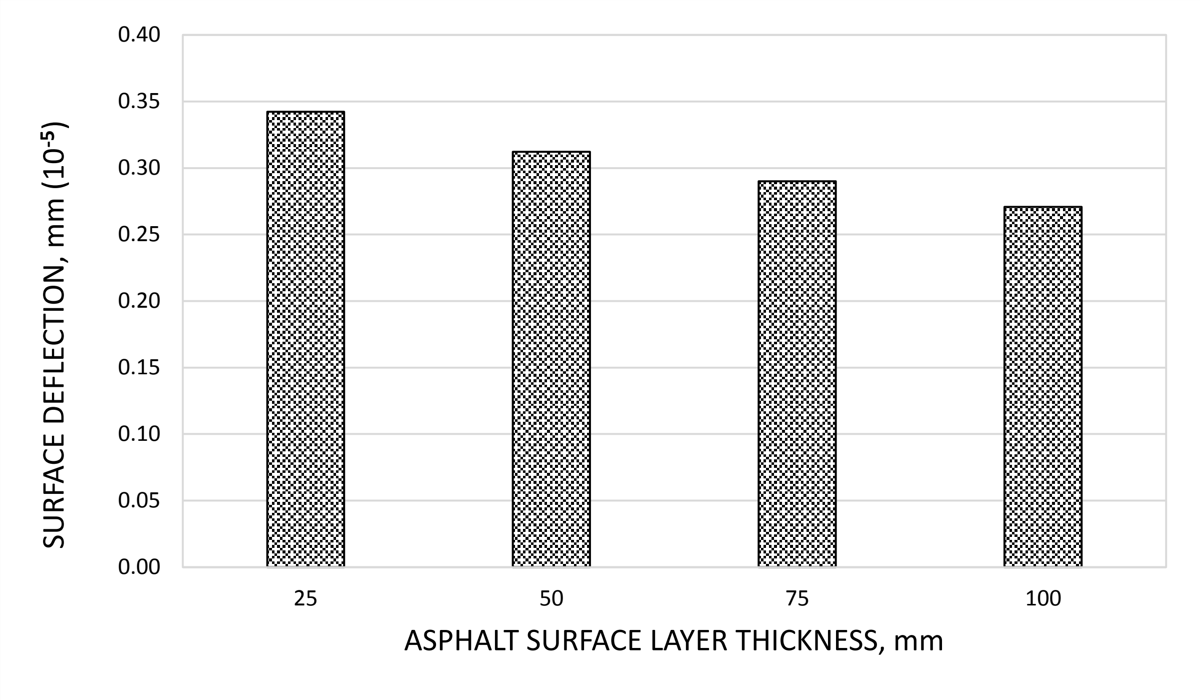


Figure 5.6. Effect of asphalt layer thickness on surface deflection over a stabilized base layer (3D Model)

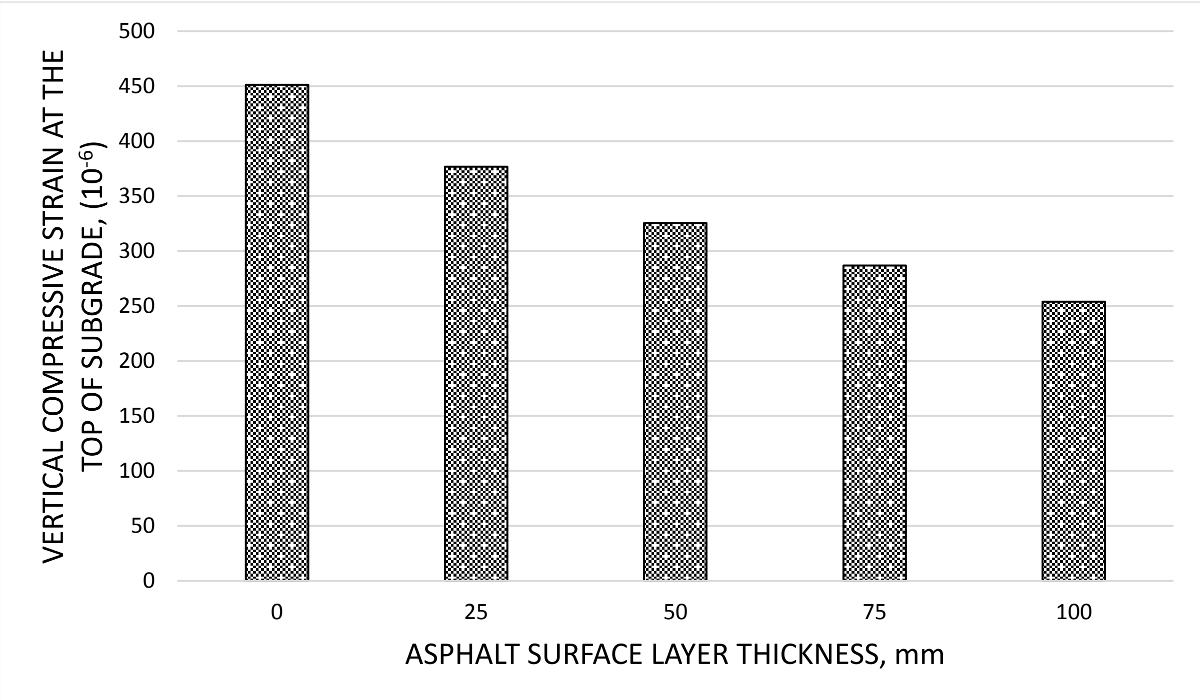


Figure 5.7. Effect of asphalt layer thickness on vertical compressive strains on the top of sub-grade (3D Model)

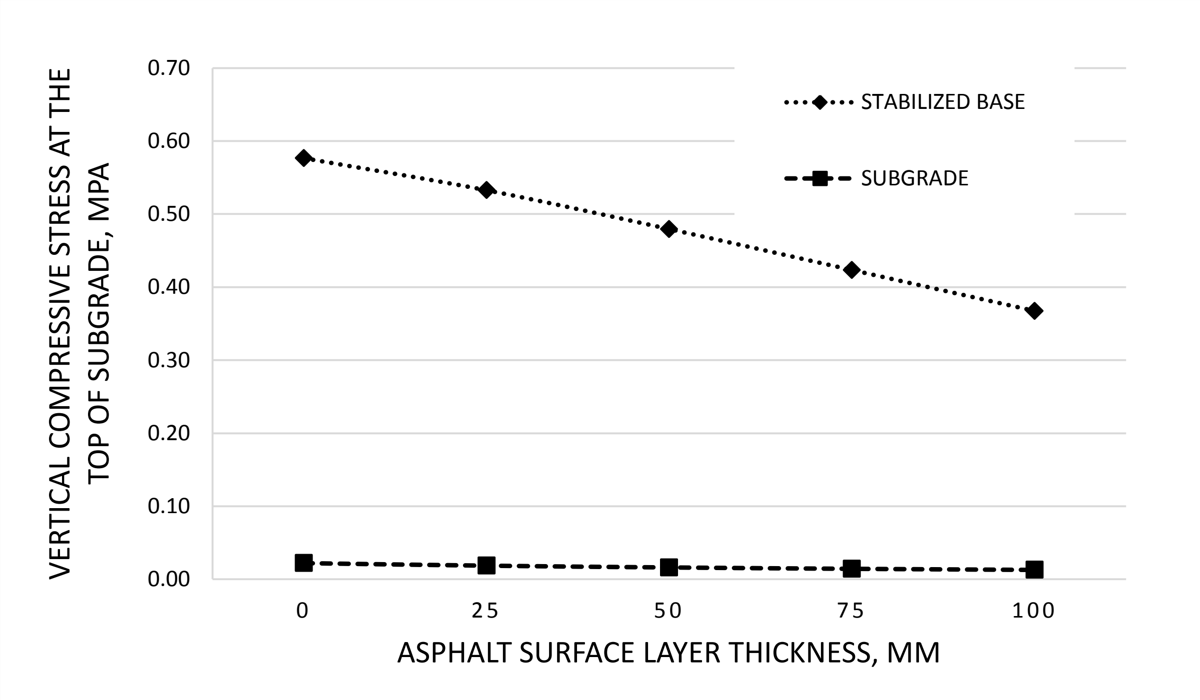


Figure 5.8. Effect of asphalt layer thickness on vertical compressive stress within stabilized base/on the top of sub-grade (3D Model)

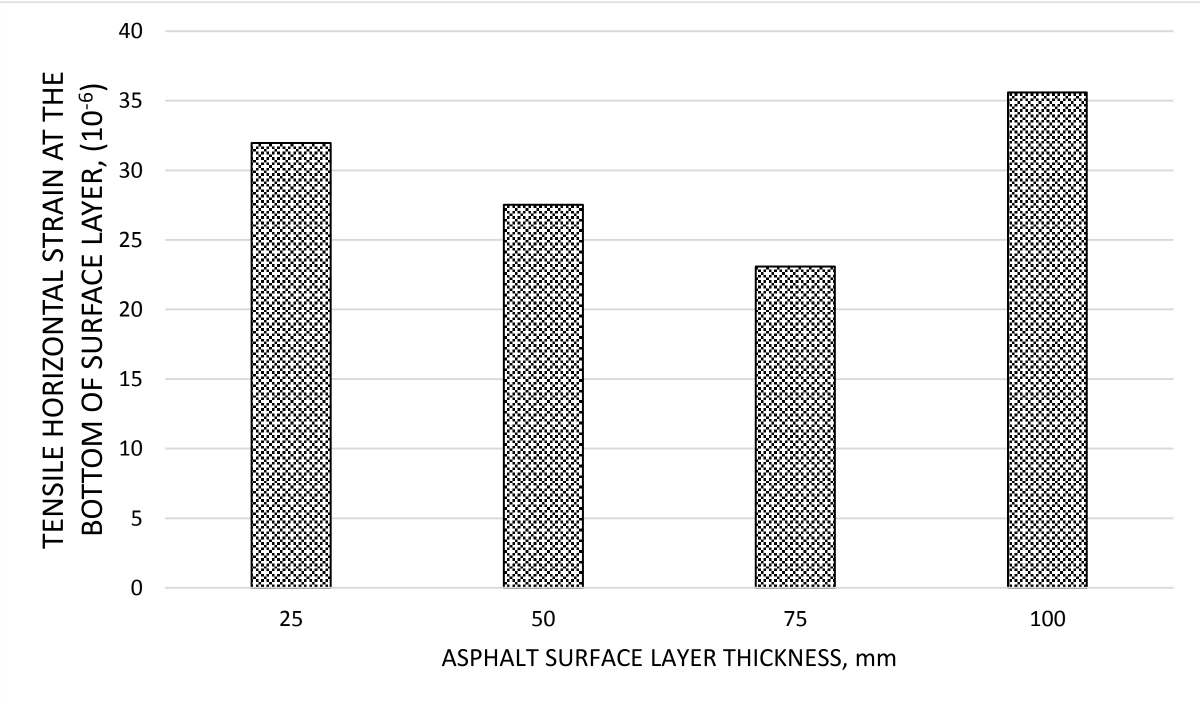


Figure 5.9. Effect of asphalt layer thickness on tensile horizontal strain at the bottom of asphalt layer (3D Model)

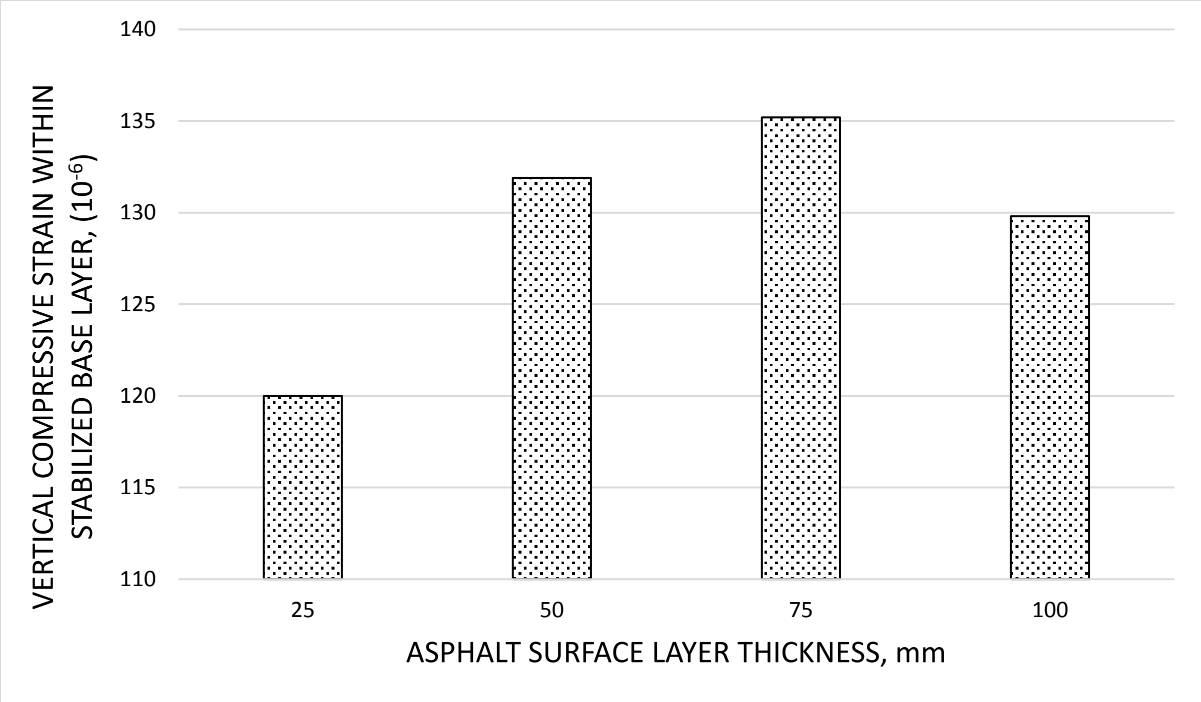


Figure 5.10. Effect of asphalt layer on vertical compressive strain within stabilized base layer (3D Model)

Table 5.2. Fatigue and rutting failure analysis based on Asphalt Institute Response Model (1982) for 3D model

Asphalt Layer Thickness (mm)	Fatigue Criterion		Rutting Criterion	
	Tensile Strain Ɛt (10^{-6}) bottom of Asphalt Layer	**No. of Load Repetitions to Failure N_f**	**Vertical Strain Ɛc (10^{-6}) in Sub-grade**	**No. of Load Repetitions to Failure N_r**
25	31.94	7.6 x 10^8	376.60	2.92 x10^6
50	27.53	12.39 x 10^8	325.50	5.60 x 10^6
75	23.08	22.14 x 10^8	286.70	9.89 x 10^6
100	35.60	5.32 x10^8	253.90	17.04 x10^6

According to the model suggested by Asphalt Institute, the capacity in terms of number of load repetition before failure is calculated for the paved stabilized base layer pavement. From Table 5.2, it was observed for both asphalt fatigue and sub-grade rutting; that increase in thickness of asphalt surface layer increase the number of load repetition before failure. Although, for 100 mm thickness there was an increase load repetition (17.04 x10^6) before rutting failure, yet the fatigue failure in terms of load repetition (5.32 x10^8) decreases at this thickness. Thus, increase in the asphalt surface layer does not necessary increase the bearing capacity of the pavement structure as other pavement layers are contributing factors to flexible pavement bearing capacity.

5.3. Non-Linear versus Linear Material Characterization Results

According to Abaza (2007), non-linear material characterization over linear gives a close field measurement, thus here a comparative analysis of non-linear and linear material characterization was undertaken. Figures 5.11 – 5.13 show the contour plots for displacements, strains and stresses in 25 mm asphalt thickness layer respectively. From Figure 5.11, it was observed that the maximum magnitude of deflection (rutting - 4.544 x 10^{-4} m) was higher in Figure 5.11B, which is for non-linear model, implying that material acts like an elasto-plastic thus did not totally return to the original state. Similarly, from Figure 5.12, the maximum strain (1.838 x 10^{-4} m) was higher in the non-linear model but also worth to noting that the minimum strain (-5.076 x 10^{-6} m) was higher in the linear model, thus implying that strain in the linear model extended to the lower part of the sub-grade which will overall result in failure.

In Figure 5.12, the maximum stress transfer (tyre load) through the linear model was high, thus implying that more stress is transferred to the rest of the layers. Overall, there are not many differences in the results obtained, despite the M_R (1301 MPa) used in non-linear model is smaller when compared with that of linear model (2560 MPa).

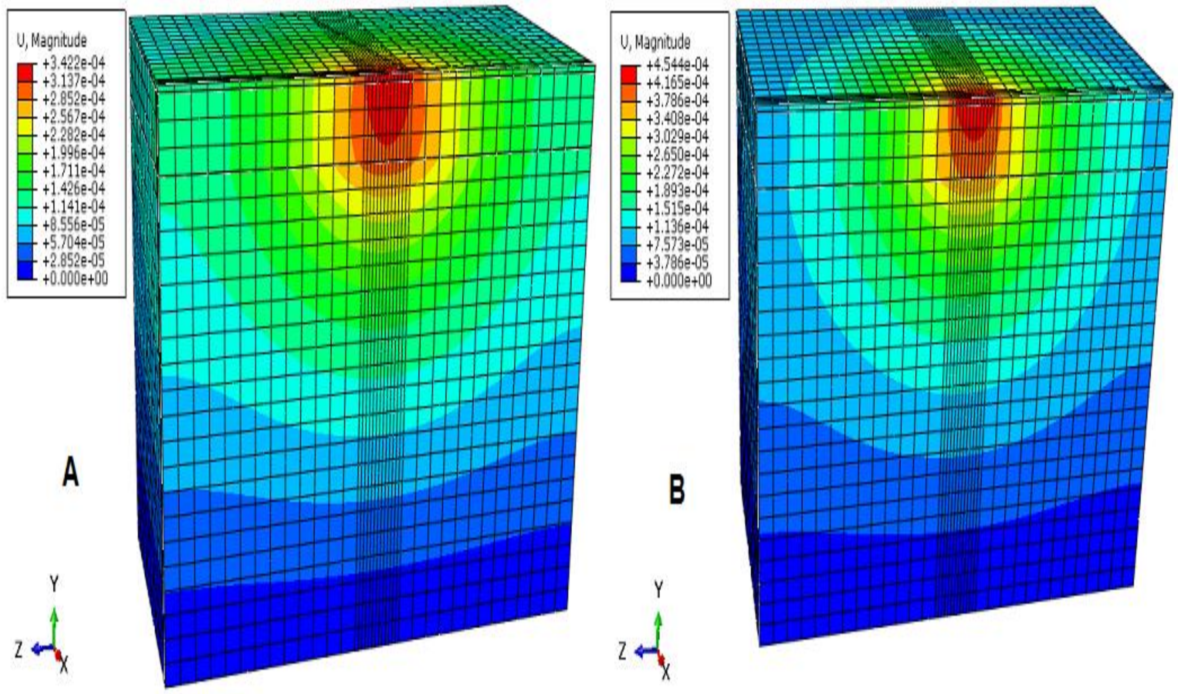


Figure 5.11. Displacement contour plot for 25 mm thickness asphalt layer (A- Linear model and B- Non-linear model)

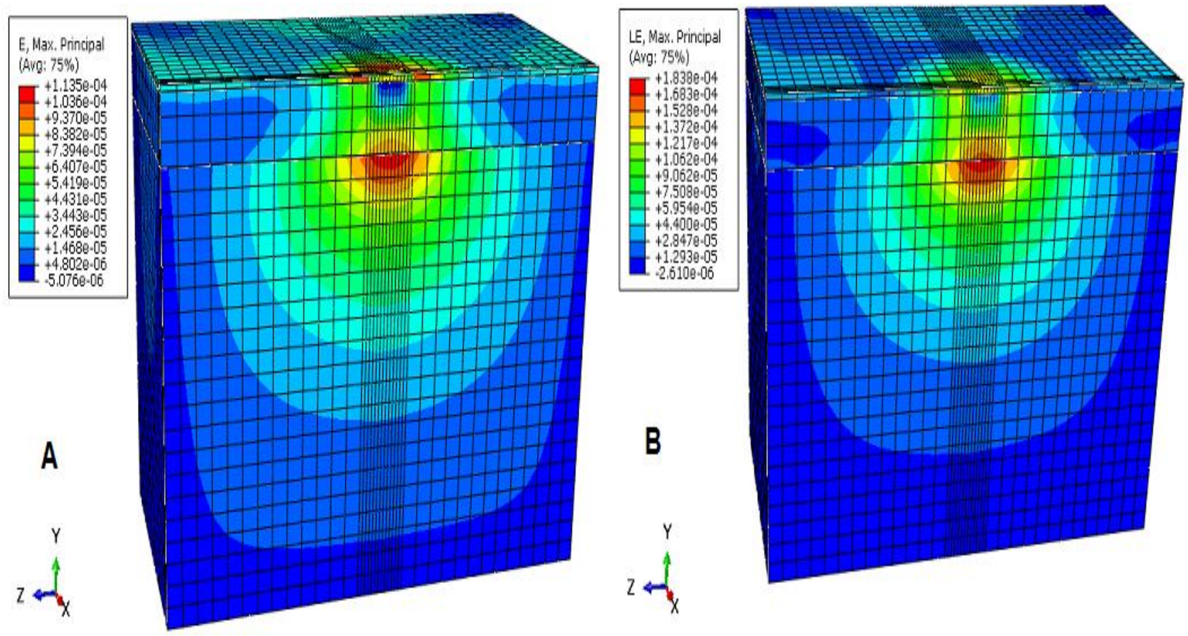


Figure 5.12. Strain contour plot for 25 mm thickness asphalt layer (A- Linear model and B- Non-linear model)

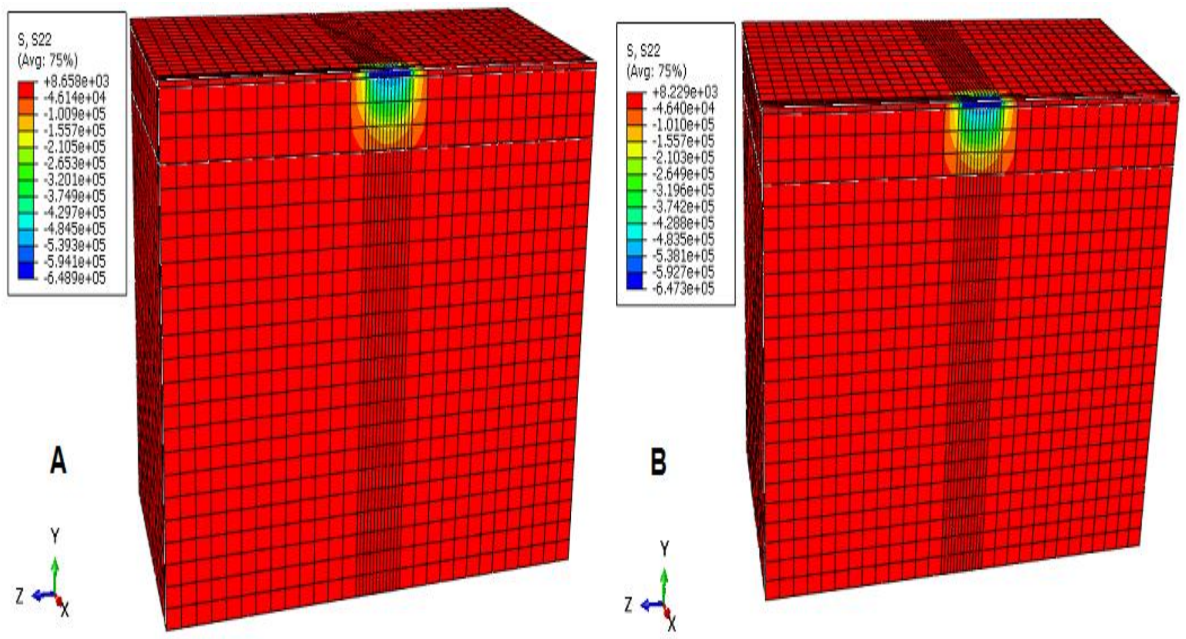


Figure 5.13. Stress contour plot for 25 mm thickness asphalt layer (A- Linear model and B- Non-linear model)

Table 5.3. Effect of asphalt layer thickness for non-linear material model

Asphalt Layer Thickness (mm)	Vertical Strain Ɛc (10^{-6}) in Stabilized base Layer	Tensile Strain Ɛt (10^{-6}) bottom of Asphalt Layer
25	259.1	38.57
50	285.7	30.92
75	273.9	41.46
100	247.5	61.55

Furthermore, from Table 5.3 above it is of a great interest to note that it is against the trend in the linear model for increase in thickness of asphalt layer which was reported in Figures 5.9 and 5.10 for vertical compressive strain in stabilized base and tensile horizontal strain in asphalt layer. The results for the non-linear model experienced an increase in the compressive strain for stabilized base in 50 mm thickness asphalt layer and thereafter a decrease. Conversely, the horizontal decreases in the 50 mm thickness and thereafter increases for subsequent thickness, thus, implying that the thickness of asphalt layer beyond 50 mm may result in bottom-up fatigue cracking. On a comparative note, the results obtained from D-P model compared with those of the M-C model; at first (i.e. in 25 mm Asphalt thickness layer) experienced a difference of about 0.2 percent in the results obtained for displacements, strains and stresses. However, the subsequent results were comparable. This implies that the D-P model or M-C model is a good non-linear material representation for stabilized base layers in pavement design. Overall, it is worth noting that the use of 50 mm thickness of asphalt layer over the stabilized base layer by developing countries, is not only justifiable by economic reasons, but also on its effectiveness to prevent failure such as bottom-up fatigue cracking which can be experienced in thicknesses beyond 50 mm.

5.4. Comparative Analysis Results

Comparing the results obtained from Abaqus (Linear model) and that of mePADS in terms of horizontal strain at the bottom of asphalt layer and the vertical strain in the subgrade (Table 5.4). Results show that the strains generated in the mePADS are generally low when compared to that of Abaqus. On like in Abaqus, results at 100 mm asphalt layer thickness did not follow the regular pattern but that of mePADS was consistent. Thus, the results from Abaqus can be said to be dynamic in nature.

Table 5. 4 Effect of asphalt layer thickness for Abaqus and mePADS

Asphalt Layer Thickness (mm)	Abaqus 3D (Linear Model)		mePADS	
	Tensile Strain Ɛt (10^{-6}) bottom of Asphalt Layer	Vertical Strain Ɛc (10^{-6}) in Sub-grade	Tensile Strain Ɛt (10^{-6}) bottom of Asphalt Layer	Vertical Strain Ɛc (10^{-6}) in Sub-grade
25	31.94	376.60	14.13	195
50	27.53	325.50	26.26	169
75	23.08	286.70	40.58	149
100	35.60	253.90	41.81	134

Furthermore, Table 5.5 presents the pavement structural capacity results obtained from the use of 1993 AASHTO SN empirical method, mePADS and those obtained using 3D FEM (Non-Linear and Linear Material (Table 5.2)) with the Asphalt Institute model. Results from the mePADS (see Appendices A2 – F2); which serves a check for the performance of 3D FEM models, although within a close range yet, tends to be higher than those of AASHTO SN and those of 3D FEM models. This is so because the SAPDM damage model used in software in question is outdated and currently under review (SANRAL, 2013b).

However, there are not many differences in the results obtained from AASHTO SN and those of 3D FEM-linear materials, yet those of 1993 AAHSTO were higher. In the report by Huber, Andrewski and Gallivan (2009), the AASHTO 1993 pavement design guide was found to have typically over-designed pavements in Indiana by 1.5 to 4.5 inches beyond what was needed. Thus, it can be concluded that the 1993 AASHTO SN tends to over-design, which makes its use uneconomical. Additionally, from Table 5.5, results from linear models are higher than those of non-linear, which also show that the linear model tends to over-design as a result of the M_R of the stabilized base layer used. This M_R is obtained using level 2 inputs (lower reliability when compared with level 1), thus it can be concluded that M_R has a significant effect on the design of pavement through FEM. Overall, the 3D FE non-linear model tends not to be partial in its design as there are few assumptions to be made in using it for the design of pavement structure and the fact that M_R was obtained through Triaxial testing, which gives the true strength of materials used in pavement structure.

Table 5.5. Structural capacity results for 1993 AASHTO, mePADS and 3D FEM models

Asphalt Layer Thickness (mm)	Predicted No. of 80 kN ESALs (1993 ASSHTO SN)	Sub-grade Bearing Capacity (mePADS Results)	No. of Load Repetitions to Failure Nr (Linear Model)	No. of Load Repetitions to Failure Nr (Non-Linear Model)
25	10.59×10^{7}	30.70×10^{12}	2.92×10^{6}	5.41×10^{5}
50	31.00×10^{7}	12.70×10^{14}	5.60×10^{6}	1.13×10^{6}
75	79.00×10^{7}	43.11×10^{14}	9.89×10^{6}	2.13×10^{6}
100	185.20×10^{7}	10.00×10^{15}	17.04×10^{6}	3.91×10^{6}

5.5. Summary

Results of this study were presented in this chapter. As expected, based on the literature reviews, the following results were observed:

1. 3D FE model results for design of flexible pavement were more efficient when compared with those of axisymmetric;
2. The structural response of stabilized base and asphalt layer were discovered and are of great importance in flexible pavement;
3. The current update for mePADS software is quite necessary, especially in terms of the damage models;
4. Non-linear material characterization model is efficient over linear model; and
5. Overall 3D FEM design for flexible pavement is efficient over empirical methods.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1. Introduction

The stabilization process in pavement construction is not a new process, but hitherto this process has not been fully implemented in the design methods for pavement structure. Although, in recent decades, researchers have tried to implement it in the existing empirical methods (Al-Jhayyish, 2014), but these methods are already inaccurate in their design and are limited in their capacity (Huang, 2004), thus, bringing about the use of FEMs. Considering the success recorded using FEMs, it is a necessity to incorporate the stabilization process such as fly ash-stabilized base layer into it, which was the essence of this study. As a result, an attempt was made to simulate the behaviour of the flexible road pavements having fly ash as an alternative soil stabilizer using FEM. This simulation study was undertaken by creating FEMs using Abaqus® to study the structural responses of the stabilized base layer and the responses of flexible pavement when constructed with fly ash-stabilized base layer. Therefore, in this final chapter the main conclusions of this thesis are summarized and some recommendations are given.

6.2. Conclusions

As a result of the modelling and analysis which were performed in this study the following conclusions were obtained;

- 3D FE models are more efficient than 2D axisymmetric models.
- Increase in the M_R of any material in pavement structure, increases the overall pavement resistivity to failure.
- Increase in the thickness of fly ash-stabilized base layer increases the resistance of pavement to failure in terms of surface deflection, vertical compressive stresses/strains on top of the sub-grade layer; however, increase beyond 300 mm results in strength decrease.
- In the same manner, increase in the thickness of asphalt layer increases pavement resistivity to failure; however, increase in thickness beyond certain

thickness, especially over a stabilized base may result in bottom-up fatigue cracking.

- The use of non-linear material characterization model is more efficient than linear material characterization. However, as a result the unavailability of Triaxial test results, the linear material characterization model can be used as a preliminary study.
- The results obtained from D-P and M-C models are comparable, thus either can be used in a material characterization model in pavement design.

Overall, the uses of empirical deign methods result in over-designing of pavement structure, consequently resulting in uneconomical pavement design and construction. However, the use of 3D FE models and most especially, the non-linear material characterization model provides better results and gives some amount of certainty on the design life of the pavement.

6.3. Recommendations

Since the structural element in the pavement is formed by the thickness and strength base and sub-base layers placed over the sub-grade, there is a need for further study on the materials used in these layers. Furthermore, it is recommended that the fly ash as a stabilizer should be experimented with a lower percentage (10 percent – 15 percent), as percentages beyond 20 percent result in strength reduction and economical unwise.

6.4. Further Studies

Firstly, it was discovered that the fly ash stabilizer for pavement materials lacks correlation equations for deriving M_R using UCS test data; secondly, there is a need to develop resilient modulus constitutive material models for South Africa granular material, especially for stabilized materials as it is commonly used as a base and sub-base layer in flexible pavement. Lastly, for further study there is a need to put into consideration the effect of climate conditions in terms of temperature, rainfall, etc., on the material characterization model in FEM design of pavement structures.

REFERENCES

Abaqus Inc. (2003). *Analysis of geotechnical problems with ABAQUS. 2003*. ABAQUS, Inc.

Abaqus, (2013). Analysis User's Guide (6.13). [Online] Available at: http://129.97.46.200:2080/v6.13/books/usb/default.htm [Accessed: 2 Apr. 2014].

Abaza, O. (2007). Simplified ME Approach for the Design of Flexible Pavement Structures. *An-Najah University Journal for Research*, 21, pp.129-150.

Abed, A.H. and Al-Azzawi, A.A (2012), Evaluation of Rutting Depth in Flexible Pavements by Using Finite Element Analysis and Local Empirical Model. *American Journal of Engineering and Applied Sciences*, 5(2), pp.163-169.

Adu-Osei, A. (2001). *Characterization of Unbound Granular Layers in Flexible Pavements*. Technical Reported, Texas A&M University Texas Transportation Institute College Station, Texas.

Al-Azzawi, A. A. (2012). Finite element analysis of flexible pavements strengthened with geogrid. *ARPN Journal of Engineering and Applied Sciences*, 7(10), pp.1295-1299.

Al-Jhayyish, A. K. (2014). *Incorporating Chemical Stabilization of the Subgrade in Pavement Design and Construction Practices*. PhD Thesis, Ohio University.

Al-Khateeb, L.A., Saoud, A. and Al-Msouti, M.F. (2011). Rutting prediction of flexible pavements using finite element modelling. *Jordan Journal of Civil Engineering*, 5(2), pp.173-189.

Ameri, M., Salehabadi, E.G., Nejad, F.M. and Rostami, T. (2012). Assessment of Analytical Techniques of Flexible Pavements by Final Element Method and Theory of Multi-Layer System. *Journal Basic Applied Science Research*, 2(11), pp.11743-11748.

American Association of State Highway and Transportation Officials (AASHTO). (1993), *AASHTO Guide for design of pavement structures*. Washington, DC.

American Coal Ash Association. (1990). *Flexible Pavement manual*. Alexandria, Virginia.

American Coal Ash Association. (1995). *Fly Ash Facts for Highway Engineers*. American Coal Ash Association, Inc., Alexandria, Virginia FHWASA-94-081.

Amin, E. R. (2012). A Review on the Soil Stabilization Using Low-Cost Methods. *Journal of Applied Sciences Research*, 8 (4), pp.2193-2196.

Aminaton, M., Nima, L. and Houman, S. (2013). Stabilization of Laterite Soil using GKS Soil Stabilizer. *Electronic Journal of Geotechnical Engineering*, 18, pp.521-532.

Araya A. A. (2011). *Characterization of Unbound Granular Materials for Pavement.*, Master's Thesis, Road and Railway Engineering Section, Faculty of Civil Engineering and Geosciences, Delft University of Technology.

Asphalt Institute. (1982). *Research and Development of Asphalt Institute's Thickness Design Manual*. 9th Ed., Research Report 82-2, the Asphalt Institute.

Asphalt Pavement Alliance (APA). (2010). *Perpetual asphalt pavements: A synthesis*. APA 101, Lanham, Maryland.

ASTM C618. (2011). *Standard Specification for Fly Ash and Raw or Claimed Natural Pozzolans for Use as a Mineral Admixture in Portland Cement Concrete*. American Society for Testing and Materials, Annual Book of ASTM Standards, Volume 04.02, West Conshohocken, Pennsylvania.

Áurea, S. H., Evandro, P.J. and Lucas, T. B. (2006). *Finite Element Modelling of Flexible Pavement*.

Australian Road Research Laboratory (1998). *Correlations between compressive strength and modulus*. Sydney, New South Wales, Australia.

Ban, H. and Park, S. (2014). Characteristics of Modified Soil-Aggregate System and Their Application in Pavements. *KSCE Journal of Civil Engineering*, (0), pp.1-7.

Barenberg, E. J. (1977). *Evaluating stabilized materials*. National cooperative highway research program, contract no. HR 63-4-1.

Bindu J. and Vysakh P. (2012). Stabilisation of Lateritic soil using coconut shell, leaf and husk ash. *Institution of Electrical and Electronics Engineers*, 978-1-4673-2636-0/12, pp.247-279.

Boussinesq, J. (1885). *Application of the potential for the study of equilibrium and motion of elastic solids*. Paris. Lille. pp. 57 – 75.

Brennan, M. J. and O'Flaherty, C. A. (2002). *Materials used in road pavements*, In O'Flaherty, C.A., *Highways: the location, design, construction & maintenance of road pavement*. Butterworth-Heinemann, Oxford, pp.118-162.

Britto A. *Running ABAQUS 6.9* [Online]. University of Cambridge. 2010. Available at: http://www-h.eng.cam.ac.uk/help/programs/fe/abaqus/doc/aba69fdouble.pdf. [Accessed: 6 December 2014].

Brooks, R. M. (2009). Soil stabilization with fly ash and rice husk Ash. *International Journal of Research and Reviews in Applied Sciences*, 1(3), pp.209-217.

Brooks, R. M., Hutapea, P., Obeid, I., Bai, L. and Takkalapelli, K.V. (2008). *Finite Element Method - a Tool for Learning Runway Design*. ASEE Mid-Atlantic (Zone I) Conference, West Point, NY.

Bureau for Industrial Cooperation. (2012). *FINAL REPORT - Preparation of a Transport Facilitation Strategy for the East African Community*', [Online] Available at: www.eac.int/infrastructure/index.php. [Accessed: 20 September 2014].

Burmister, D. M. (1943). *The Theory of Stresses and Displacements in Layered Systems and Application to the Design of Airport Runways*. Proceedings, Highway Research Board, 23.

Chen, H. H., Marshek, K. M. and Saraf, C. L. (1990). *Effects of Truck Tire Contact Pressure Distribution on the Design of Flexible Pavements: A Three Dimensional Finite Element Approach*. Transportation Research Record, No. 1095, pp.7278.

Cho, Y. H., McCullough, B. F., Weissmann, J. (1996). *Considerations on Finite-element method application in pavement structural analysis*. Transportation Research Board, 1539, pp. 96-101.

Çöleri, E. (2007). *Relationship between Resilient modulus and Soil index properties of unbound materials*. Master's Thesis, Middle East Technical University.

Comberato, J. J., Vance, E. D. and Someshwar, A. V. (1997). *Composition and land application of paper manufacturing residuals*. In: Rechcigl, J., Mackinnon, H. (Eds.), *Agricultural Uses of By-products and Wastes*. ACS, Washington, DC, 185–203.

Committee of Land Transport Officials (COLTO), 1998 Edition.

Council of Scientific and Industrial Research (CSIR). (2010). *Potholes: Technical guide to their causes, Identification and Repair*. [Online] Available at: www.csir.co.za.pothole_guides. [Accessed: 21 September 2014].

CSIR Built Environment (2009). *Pavement Analysis and Design Software*. [Online] Available at: http://asphalt.csir.co.za/samdm/. [Accessed: 29 July 2015].

Darwish S. G., (2012). *Stress and Strain in Flexible Pavement*. [Online] Available at: http://www.unimasr.net/ [Accessed: 27 March 2014].

Davids, W. G. (2009). *EverStressFE1.0 Software for 3D Finite-Element Analysis of Flexible Pavement Structures*. The Washington State Department of Transportation.

De Beer, M., Fisher, C., and Jooste, F. J. (1997). Determination of pneumatic tire/pavement interface contact stresses under moving loads and some effects on pavements with thin asphalt surfacing layers. In: *Proceedings 8th International Conference on Asphalt Pavements*, University of Washington, Seattle, pp.179–227.

Desai, C. S. (2012). *Application of Finite Element and Constitutive Models*.

Dione, A., Fall, M., Berthaud, Y., and Makhaly, B. (2013). Estimation of Resilient Modulus of Unbound Granular Materials from Senegal (West Africa). *Geomaterials*, 2013(3), pp.172-178.

Dixit, U. (2007). *Finite Element methods in Engineering*.

Duncan, J. M., Monismith, C. L. and Wilson, E. L. (1968). *Finite Element Analysis of Pavements.* HRR-228, Highway Research Board, National Research Council, Washington, D.C.

Dunlap, W. S. (1963). *A report on a mathematical model describing the deformation characteristics of granular materials*. Technical Report 1, Project 2-8-62-27, TTI, Texas A & M University.

Ekwulo, E. O., and Eme, D. B. (2009). Fatigue and rutting strain analysis of flexible pavements designed using CBR methods. *African Journal of Environmental Science and Technology*, 3(12), pp.412-421.

Eluozo, S. N. (2013). Predictive Model to Determine Resilient Modulus on Bituminous Mixtures in Load Repetition on Pavement Failure. *International Journal of Materials, Methods and Technologies*, 1(5), pp.82-89.

Eskioglou, P. and Oikonomou, N. (2008). *Protection of Environment by the use of Fly Ash in Road Construction*.

Fall, M., Ba, S., Sarr, D., Ba, M. and Ndiaye M. (2011). An Alternative Method to the West African Compaction (WAC) Test Procedure. *Geomaterials*, 1(2), pp.25-27.

FM5-410, (2012). *Soil Stabilization for Road and Airfield*. [Online] Available at: www.itc.nl/~rossiter/Docs/FM5-410. [Accessed: 25 March 2014].

Furter, E. (2011). *Coal Fly ash holds African opportunities.* [Online] Available at: www.sheqafrica.com/fly-ash-conference/ [Accessed: 25 March 2014].

George, K. P. (2004). *Prediction of Resilient Modulus from Soil Index Properties – Final Report*. The University of Mississippi.

Gupta, A. and Kumar, A. (2014). Comparative Structural Analysis of Flexible Pavements Using Finite Element Method. *The International Journal of Pavement Engineering and Asphalt Technology* (PEAT), 15(1), pp.11-19.

Gyanen, T., Savitha, A.L. and Gudi, K. (2013). Laboratory study on soil stabilization using fly ash mixtures. *International Journal of Engineering Science and Innovative Technology*, 2(1), pp.477-482.

Hafeez, I. (2010). *Empirical and mechanistic flexible pavement designed*. [Online] Available at: http://pec.org.pk/sCourse-files/Lec1.ppt. [Accessed: 25 March 2014].

Harichandran, R. S., Yeh, M. S., and Baladi, G. Y. (1989). MICH-PAVE: *A Nonlinear Finite Element Program for Analysis of Pavements*. Transportation Research Record 1286, TRB, National Research Council, Washington, D.C., pp.123-131.

Harold, L. and Von Quintus, P. (2007). *Evaluation of Procedure to Estimate Subgrade Resilient Modulus for use in Pavement Structural Design*. Kansas: Kansas Department of Transportation Bureau of Materials and Research.

Heebink, L. V. and Hassett, D. J. (2001). *Coal Fly Ash Trace Element Mobility in Soil Stabilization*. www.flyash.info, International Ash Utilization Symposium, Centre for Applied Energy Research, University of Kentucky, Paper #64.

Hejazi, S. M., Sheikhzadeh, M., Abtahi, S.M. and Zadhoush, A. (2012). A simple review of soil reinforcement by using natural and synthetic fibres. *Construction and Building Materials*, 30, pp.100–116.

Heukelom, W., and Klomp, A. G. (1962). Dynamic Testing as a Means of Controlling Pavements during and after Construction. In: *Proceedings of the International Conference on the Structural Design of Asphalt Pavement*, Ann Arbor, Michigan, U.S.A.

Heyns, M. W. and Mostafa Hassan, M. (2013). South Africa Class F Fly Ash for roads: physical and chemical analysis. *Interim: Interdisciplinary Journal*, 12(3), pp.28-41.

Hjelmstad, K. D., Kim, J. and Zuo, Q. H. (1996). *Finite Element Procedures for Three Dimensional Pavement Analysis*. Transportation Research Record 1539, TRB, National Research Council, Washington, D.C, pp.66-71.

Hua, J. (2000). Finite Element Modelling and Analysis of Accelerated Pavement Testing Devices and Rutting Phenomenon', Ph.D. Thesis, Purdue University, West Lafayette. Indiana, USA.

Huang, Y. H. (2004). *Pavement Analysis and Design*. 2nd Edition, Pearson Education Inc, NJ.

Huber, G., Andrewski, D. and Gallivan, V. (2009). Design and Construction of Highways for Very Heavy Trucks. In: *Proceedings International Conference of Perpetual Pavements*. Ohio University. Columbus. CD-ROM.

Ibrahim, S. F., Gandhi, G. S. and Zaman, T.T. (2014). An Approach in Evaluating of Flexible Pavement in Permanent Deformation OF Paved and Unpaved Roads over Sand Dunes Subgrade under Repeated Loads. *Journal of Environment and Earth Science*, 4(14), pp.78-89.

Ji, R., Siddiki, N., Nantung, T. and Kim, D. (2014). Evaluation of Resilient Modulus of Subgrade and Base Materials in Indiana and Its Implementation in MEPDG. *The Scientific World Journal*, pp.1-14.

Johnson, D., Sukumaran, B., Mehta, Y. and Willis, M. (2007). *Three Dimensional Finite Element Analysis of Flexible Pavements to Assess the Effects of Wander and Wheel Configuration*. Presented for the 2007 FAA worldwide airport technology transfer conference, Atlantic City, New Jersey, USA.

Jooste F. (2004). Re-Evaluation of Some Aspects of the Mechanistic-Empirical Design Approach. In: *Proceedings of the 8th Conference on Asphalt Pavements for Southern Africa*. Sun City, South Africa. [CD].

Kim B., Prezzi M. and Salgado R. (2005). Geotechnical properties of fly and bottom ash mixtures for use in highway embankments. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(7), pp.914–924.

Kim, D. (2002). *Effects of Super-Single Tire Loading on Subgrade*. Ph.D. Dissertation, Purdue University, West Lafayette, Indiana.

Kim, D. and Siddiki N. Z. (2006). *Simplification of Resilient Modulus Testing for Subgrades*. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana.

Kim, M. (2007). *Three-Dimensional Finite Element Analysis of Flexible Pavements Considering Nonlinear Pavement Foundation Behaviour*. Department of Civil Engineering, Graduate College of the University of Illinois at Urbana-Champaign.

Kordi, N. E., Endut, I. R. and Baharom, B. (2010). Types of Damages on Flexible Pavement for Malaysian Federal Road. In: *Proceeding of Malaysian Universities Transportation Research Forum and Conferences*.

Liang, A. (2000). *Finite Element Analysis of Distortional bucking of Cold-formed Stainless Steel Columns*. Master's Thesis, University of Johannesburg.

Lin Li, C. H., Benson, T. B. and Edil, B. H. (2007). *Sustainable Construction Case History: Fly Ash Stabilization of Recycled Asphalt Pavement Material*. Transportation Research Board, 86th Annual Meeting.

Little, D., Snead, B., Godiwalla, A., Oshiro, P., and Tang, P. (2002). Characterization of Design Properties (Compressive Strength and Resilient Modulus) Of Lime, Cement, Fly Ash Stabilized Structural Recycled Concrete Base As A Function Of Curing Time. *In Presentada en The 2002 Federal Aviation Administration Airport Technology Transfer Conference*, pp. 5-17.

Lo, S. H. (2002). Finite element mesh generation and adaptive meshing. *Progress in Structural Engineering and Materials*, 4(4), pp.381-399.

Maharaj, D. K. and Gill, S. (2014). The behaviour of flexible pavement by non-linear finite element method. *International journal of latest research in science and technology*, 3(1), pp.53-57.

Makusa, G. P. (2012). *Soil Stabilization Methods and Materials*. PhD Thesis, Department of Civil, Environmental and Natural resources, Luleå University of Technology, Luleå, Sweden.

Mallela, J., Harold Von Quintus, P. E., Smith, K. L. and Consultants, E. R. E. S. (2004). *Consideration of lime-stabilized layers in mechanistic-empirical pavement design*. The National Lime Association.

Mamlouk, M. and Mobasher, B. (2004). Cracking Resistance of Asphalt Rubber Mix versus Hot-Mix Asphalt. *International Journal of Road Materials and Pavement Design*, 5(4), pp.435-452.

Mansurkhaki, A., Hesami, S., Khajehhassani, I. and Khordehbinan, M.W. (2014). Evaluation of Role of ABAQUS and KENLAYER Software in Linear Elastic Analysis of Flexible Pavement. *Nationalpark-Forschung in Der Schweiz (Switzerland Research Park Journal)*, 103(1), pp.546-562.

Mathur, S.S. (2011). *Utilization of Fly Ash in Roads and Embankment Works*.

May, R. W. and Witczak, M. W (1981). *Effective Granular Modulus to Model Pavement Response*. Transportation Research Record 810, TRB, National Research Council, Washington, D.C.

McClelland Engineering (1986), unpublished. *Correlations between compressive strength and modulus for LCF material on Runway 9-27*. Intercontinental Airport, Houston, Texas.

McDowell, C. (1959). *Stabilization of soils with lime, lime-fly ash and other lime reactive Materials*. Highway Research Board Bulletin, 231, Washington, DC, pp.60-66.

Mgangira, M. B. (2006). Laboratory assessment of the influence of the proportion of waste foundry sand on the geotechnical engineering properties of clayey soils. *Journal of the South African Institution of Civil Engineering*, 48(1), pp.2–7.

Molenaar, A. A. A, (2009). *Part III: Design of flexible pavements*. [Online] Available at: http://www.citg.tudelft.nl/. [Accessed: 28 March 2014].

Moossazadeh, J. and Witczak, M. W. (1981). *Prediction of Subgrade Moduli for Soil that Exhibits Nonlinear Behaviour*. Transportation Research Record 810, TRB, National Research Council, Washington, D.C., pp.9-17.

Mostafa Hassan, M., & Khalid, H. (2010). Mechanical and environmental characteristics of bituminous mixtures with incinerator bottom ash aggregates. *International Journal of Pavement Engineering*, 11(2), pp.83-94.

Mun, S. H. (2003). *Nonlinear Finite Element Analysis of Pavements and Its Application to Performance Evaluation*. Dissertation, Department of Civil and Environmental Engineering, North Carolina State University.

NCHRP (2004). *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures: Appendix RR-Finite Element Procedures for Flexible Pavement Analysis*, Transportation Research Board, National Research Council, Washington D.C.

Ndebele, S. J. (2012). *Keynote address by the minister of transport*. Presented at the occasion of department's budget vote, 25 April.

Nicholas J Garber and Lester A. H. (2010). *Traffic and Highway Engineering, SI version*. 4th edition, Cengage Learning, Stamford, USA.

Nunes, N. C., Bridges, M. G. and Dawson, A.R. (1996). Assessment of secondary materials for pavement construction: Technical and Environment aspects. *Waste management*, 16(1-3), pp.87-96.

Openshaw, S. C. (1992). *Utilization of Coal Fly Ash*. Master's thesis, Department of Environmental and Civil Engineering, University of Florida.

Paige-Green P. (2008), 'A reassessment of some road material stabilization problems', Proceedings of the 27th Southern African Transport conference, July 7-11, pp. 125-134.

Pandian, N. S. and Krishna, K. C. (2002). The Pozzolanic Effect of Fly Ash on the CBR Behaviour of Black Cotton Soil. *Journal of Testing and Evaluation*, ASTM, 31(6), pp.479-485.

Pavement Interactive. (2008). *Flexible pavement mechanistic models.* [Online] Available at: www.pavement-interactive.org/article/flexible-pavement-mechanistic-models/ [Accessed: 27 March 2014].

Peng, Y. and He, Y. (2009). Structural characteristic of cement-stabilized soil bases with 3D finite element method, *Front Architect Civil Engineering China,* 3(4), pp.428-434.

Psarras, S., Falzon, B. G. and Pinho, S. T. (2012). *Damage-Tolerant Design of Stiffener Run-Outs: A Finite Element Approach*. INTECH Open Access Publisher.

Rahman, M.T., Mahmud, K. and Ahsan, S. (2011). Stress-Strain characteristics of flexible pavement using Finite Element Analysis. *International Journal of Civil and Structural Engineering*, 2(1), pp.233-240.

Rao, C., L. Titus-Glover, B. Bhattacharya, and M. I. Darter. (2012). *User's Guide: Estimation of Key PCC, Base, Subbase, and Pavement Engineering*

Properties from Routine Tests and Physical Characteristics (Technical Report; FHWA-HRT-12-031). Federal Highway Administration, Washington, DC.

Reddy, J. N. (1993). *An introduction to the finite element method (Vol. 2, No. 2.2)*. New York: McGraw-Hill.

Road Research Laboratory. (1970). *A Guide to structural design of pavements for New roads, Road Note 29*, Department of the Environment, H.M.S.O., London, 3rd edition.

Rodrigue J., Slack B. and Comtois C. (2013). *Transportation modes: An overview*. In Rodrigue J., *The geography of transportation systems*, 3rd ed., New York: Routledge.

Rodriguez-Roa, F. (2003). Observed And Predicted Behaviour of Maipo River Sand. *Soils and Foundations*, 43(5), pp.1-11.

Saad, B., Mitri, H., and Poorooshasb, H. (2006). 3D FE analysis of flexible pavement with geosynthetic reinforcement. *Journal of transportation Engineering*, 132(5), pp.402-415.

Salehabadi, E. G. (2012). The Linear Elastic Analysis of Flexible Pavement by the Finite Element Method and Theory of Multiple-Layers System. *National park-Forschung in Der Schweiz*, 101(9), pp.363-37.

Sas, W., Głuchowski, A., and Szymański, A. (2012). *Determination of the resilient modulus MR for the lime stabilized clay obtained from the repeated loading CBR tests*. Annals of Warsaw University of Life Sciences-SGGW. Land Reclamation, 44(2), 143-153.

Schwartz, C. W. and Carvalho, R.L. (2007). *Implementation of the NCHRP 1-37A Design Guide Final Report, Volume 2: Evaluation of Mechanistic-Empirical Design Procedure*. MDSHA Project No. SP0077B41

Seed, H. B., Mitry, F. G., Monismith, C. L. and Chan, C. K. (1967). *Prediction of pavement deflection from laboratory repeated load tests*, NCHRP Report 35, USA.

Senol, A., Bin-Shafique, M., Edil, T. B. and Benson, C. H. (2002). Use of Class C fly ash for the stabilization of soft soil as sub-base. *ARI Bull Istanbul Tech University,* 53, pp.89–95.

Shafabakhsh, G. A., Motamedi, M. and Family, A. (2013a). Influence of Asphalt Concrete Thickness on Settlement of Flexible Pavements. *Electronic Journal of Geotechnical Engineering*, 18, pp.473-483.

Shafabakhsh, G. A., Talebsafa M., Motamedi, M. and Badroodi, S. K. (2013b). Validation of Load Movement on Flexible Pavement Using Pennsylvania Field Test. *Technical Journal of Engineering and Applied Sciences*, 3(22), pp.3125-3130.

Sherwood, P. (1974). The use of waste and low-grade materials in road construction: guide to materials available. *TRB publication*.

Sherwood, P. (1993). *Soil stabilization with cement and lime. State of the Art Review*. London: Transport Research Laboratory, HMSO.

Sinha, A. K., Chandra, S. and Kumar, P. (2014). Finite Element Analysis of Flexible Pavement with Different Subbase Materials. *Technical Paper, Indian Highway*. 42(2), pp.53-63.

Snellings, R., Mertens G. and Elsen J. (2012). Supplementary cementitious materials. *Reviews in Mineralogy and Geochemistry*, 74, pp.211–278.

South African National Road Agency Ltd (SANRAL). (2013a). *Pavement Composition and Behaviour: South African Pavement Engineering Manual*. An initiative of the South African national roads agency ltd, South Africa.

South African National Road Agency Ltd (SANRAL). (2013b). *Pavement Design: South African Pavement Engineering Manual*. An initiative of the South African national roads agency ltd, South Africa

Steyn, W., Maina, J. and Repsold, A. (2013). *Comparison of isotropic and cross-anisotropic analysis of pavement structures*. Transportation Research Board 92nd Annual Meeting, 25(3), 26.

Stoffels, S., Solaimanian, M., Morian, D., and Soltani, A. (2006). Field Instrumentation and Testing Data from Pennsylvania's Superpave In-Situ Stress/Strain Investigation. *Airfield and Highway Pavement*, 191(10), pp. 107-118.

Sukumaran, B. (2004). *Three Dimensional Finite Element Modeling of Flexible Pavement.* Presented for the 2004 FAA worldwide airport technology transfer conference, Atlantic City, New Jersey, USA.

Sutanto, M. H. (2009). *Assessment of Bond between Asphalt Layers*. Ph.D. Dissertation, Department of Civil Engineering, Nottingham Transportation Engineering Centre.

Tanosaki, T., Yu, Q. and Nagataki, S. (2011). Are Fly Ashes Green? *2011 World of Coal Ash (WOCA) Conference*, May 9-12, 2011, [Online] Available: www.flyash.info. Accessed: 20 March 2014.

Theyse H. L., de Beer M. and Rust F. C. (1996). *Overview of the South African Mechanistic Pavement Design Method.* Transportation Research Record 1539. Transportation Research Board, Washington D.C.

Theyse, H. L., and Muthen, M. (2000, July). Pavement analysis and design software (PADS) based on the South African mechanistic-empirical design method. In: *South African Transport Conference (SATC)*. Action in Transport for the new Millennium, pp.17-20.

Theyse, H.L., de Beer, M., Maina, J.W. and Kannemeyer, L. (2011). *Interim Revision of the South African Mechanistic-Empirical Pavement Design Method for*

Flexible Pavements. Presented at 10th Conference on Asphalt Pavements for Southern Africa.

Thompson, M. R. (1966). *Shear strength and elastic properties of lime soil mixtures*. Highway Research Board, University of Illinois, Champaign, IL.

Ti, K. S., Huat, B. B., Noorzaei, J., Jaafar, M. S., and Sew, G. S. (2009). A review of basic soil constitutive models for geotechnical application. *Electronic Journal of Geotechnical Engineering*, 16, pp.1-18.

Tiliouine, B. and Sandjak, K. (2014). Numerical simulation of granular materials behaviour for unbound base layers used in Algerian pavement structures. *International Journal of Civil and Structural Engineering*, 4(3), pp.419-429.

Torii, K., Hashimoto, T., Kubo, T. and Sannoh, C. (2013). The effective utilization of classified fine fly ashes for production of highly-durable concrete mixtures. In: *3rd Int. Conf. on Sustainable Construction Materials & Technologies*, SCMT3, pp.109-118.

TRH 4. (1996). Structural Design of flexible pavements for interurban and rural roads. Pretoria: Committee of Land Transport Officials (COLTO), Department of Transport (DoT). (DoT Technical Recommendations for Highways; Draft TRH 4 (1996)).

Tutumluer, E. and Thompson, M. R. (1997). *Anisotropic modelling of Granular Bases in Flexible Pavements*. Transportation Research Record, No. 1577. Transportation Research Board, National Research Council, Washington, DC.

Umar, M., Alhassan, H., M., Abdulfatah, A.Y. and Idris, A. (2013). Beneficial use of Class-C Fly Ash in Improving Marginal Lateritic Soils for Road Construction. *Electronic Journal of Geotechnical Engineering*, 18, pp.2815-2822.

United State Environmental Protection Agency. Using recycled industrial materials in roadways [Online]. 2009 Available at: http://www.epa.gov/epawaste/conserve/imr/pdfs/roadways.pdf [Accessed 5 December 2014].

United States Department of Transportation - Federal Highway Administration (USDT-FHA). Geotechnical Aspects of Pavements Reference Manual [Online]. 2014 Available at: http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/05b.cfm. [Accessed 3 January 2015].

Uzan, J. (1992). *Characterization of Granular Material*. Transportation Research Record 1022, TRB, National Research Council, Washington, D.C., pp.52-59.

Van Vuuren D. J., Otte E. and Paterson W. D. O. (1974). The Structural Design of Flexible Pavements in South Africa. In: *Proceedings of the 2nd Conference on Asphalt Pavements in South Africa*. Durban, South Africa.

Walubita, L.F. & van de Ven, M.F. (2000). Stresses and Strains in Asphalt-Surfacing Pavements. South African Transport Conference Organised by: Conference Planners, 'Action in Transport for the New Millennium'.

Wang, J. (2001). *Three-Dimensional Finite Element Analysis of Flexible Pavements*. Department of Civil Engineering, University of Maine.

Wu, Z., Chen, X., and Yang, X. (2011). *Finite Element Simulation of Structural Performance on Flexible Pavements with Stabilized Base/Treated Subbase Materials under Accelerated Loading* (No. FHWA/LA. 10/452). Louisiana Transportation Research Centre.

Yadu, L.K. and Tripathi, R.K. (2013). Stabilization of Soft Soil with Granulated Blast Furnace Slag and Fly Ash. *International Journal of Research in Engineering and Technology*, 2(2), pp.115-119.

Yagawa, G. (2011). Free Mesh Method: fundamental conception, algorithms and accuracy study. In: *Proceedings of the Japan Academy. Series B, Physical and biological sciences*, 87(4), pp.115.

Yau, A. and Quintus, H. L. V. (2002). *Study of LTTP laboratory resilient modulus test data and response characteristics*. Final Report (Publication No. FHWA-RD-02-051), Maclean VA: Federal Highway Administration.

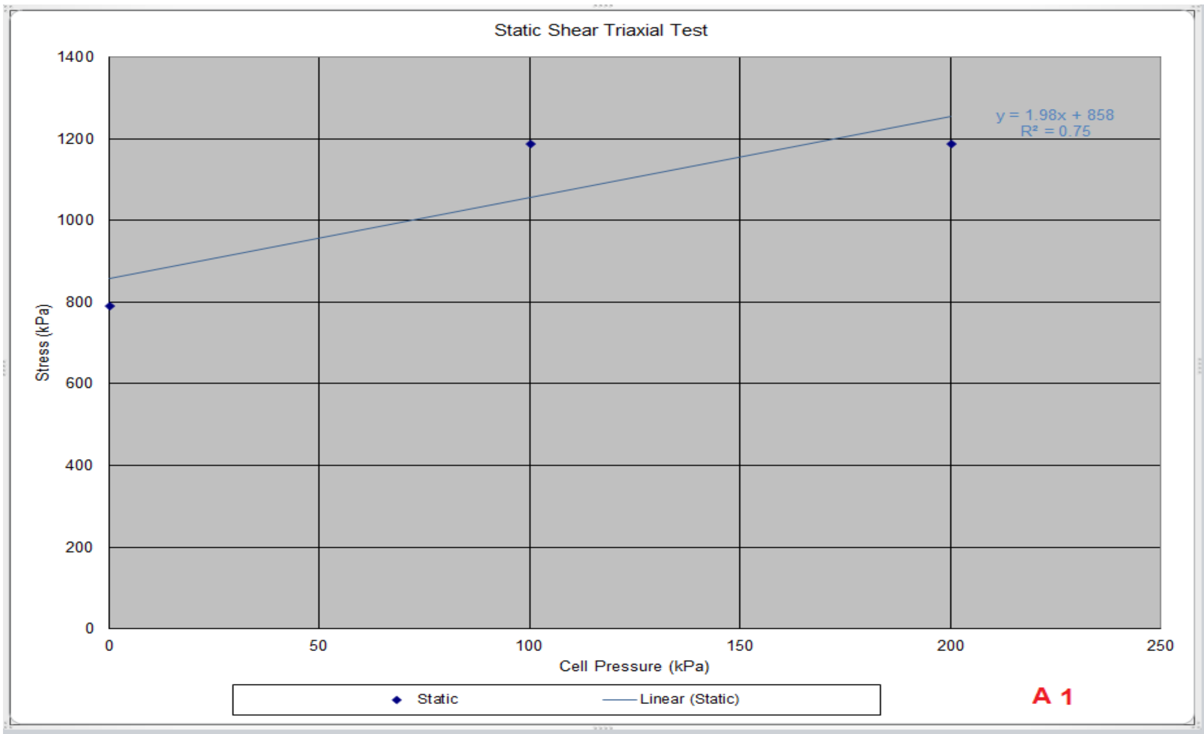
Yin, H. (2013). The Impact of Strain Gage Instrumentation on Localized Strain Responses in Asphalt Concrete Pavements. *International Journal of Pavement Research and Technology*, 6(3), pp.225-234.

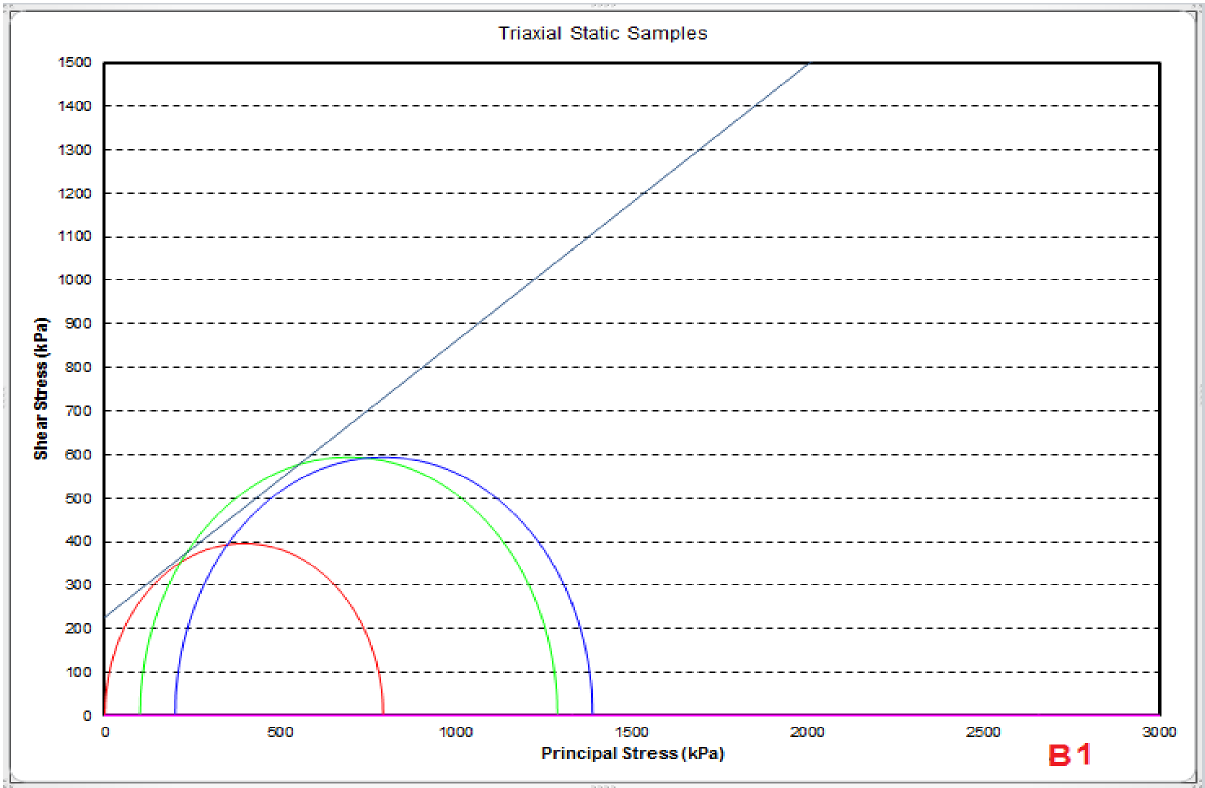
Zafar R., Nassar, W. and Elbella, A. (2005). Interaction between Pavement Instrumentation and Hot-Mix-Asphalt in Flexible Pavements. *Emirates Journal for Engineering Research*, 10(1), pp.49-55.

Zaghloul, S.M. (1993). *Non-linear dynamic analysis of flexible and rigid pavements*. Ph.D. Thesis, Purdue University, West Lafayette. Indiana, USA.

Zaman, M., Pirabarooban, S. and Tarefder, R.A. (2003). Evaluation of Rutting Potential in Asphalt Mixes Using Finite Element Modelling. *Annual Conference of the Transportation Association of Canada, St. John's, Newfoundland and Labrador*.

APPENDICES


Static Shear Triaxial Test
1400
1200
1000
800
600
400
200
0
Stress (kPa)
0
50
100
150
200
250
Cell Pressure (kPa)
y = 1.98x + 858
R² = 0.75
Static
Linear (Static)
A 1


Triaxial Static Samples
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
Shear Stress (kPa)
0
500
1000
1500
2000
2500
3000
Principal Stress (kPa)
B 1

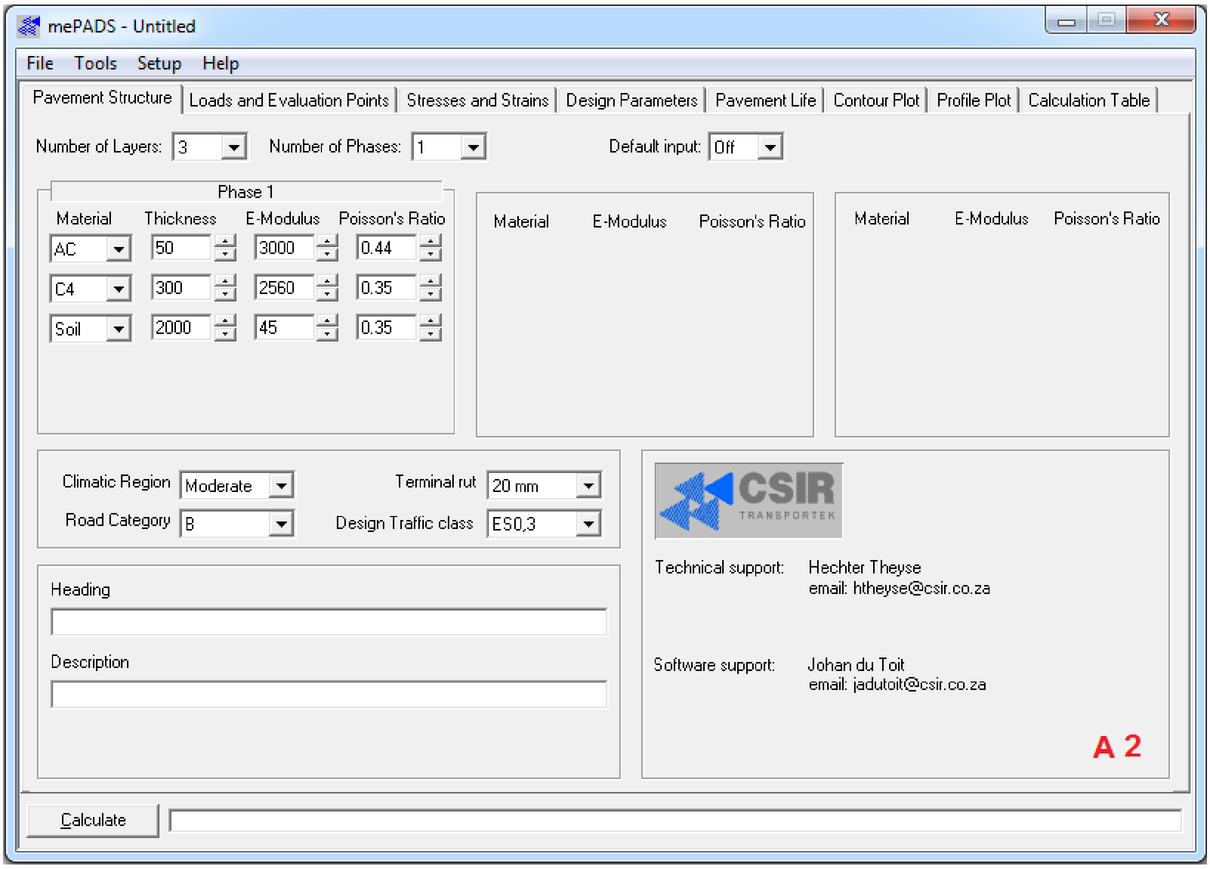
18%-Poz	Static			Sigma 1 (kPa)	p (kPa)	q (kPa)	C, ϕ R^2
			Stress				
		0	792	792	396	396	226.8
		100	1188	1288	694	594	32.4
		200	1188	1388	794	594	0.942
	Slope	1.98		Slope			
	Offset	858		Offset			
		200	1254				
		150	1155				
		100	1056				
		50	957				
		20	898				

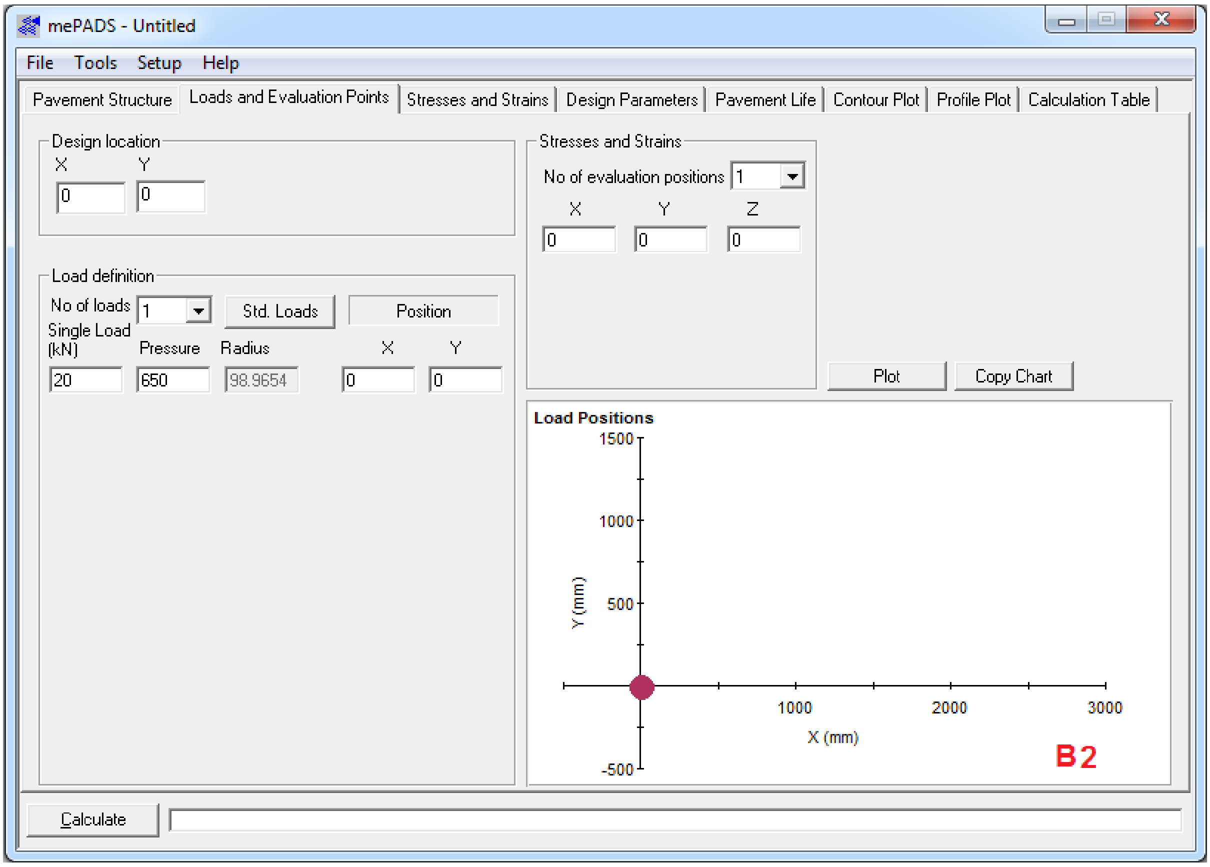
Static

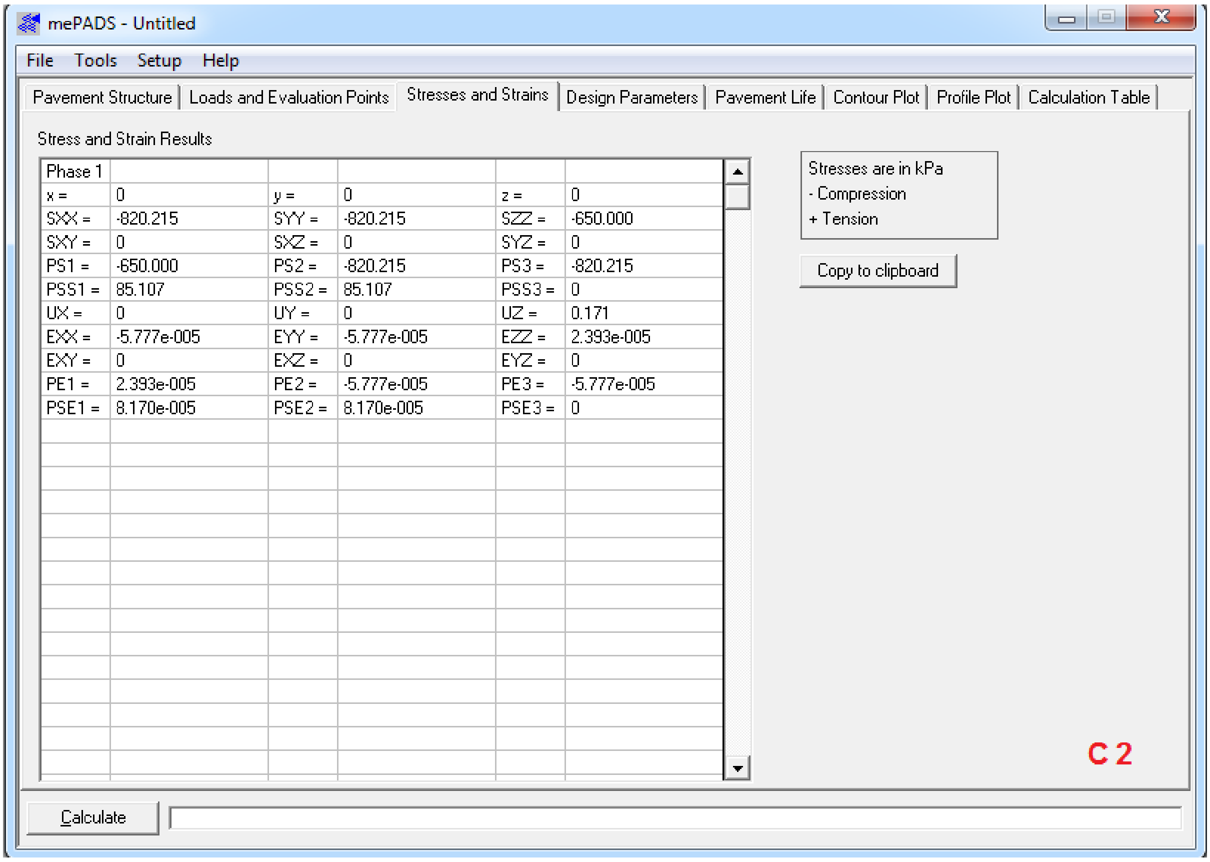
	Circle1	Circle2	Circle3	Circle4		
0	0	#NUM!	#NUM!	0	226.8329	0
2	39.74921	#NUM!	#NUM!	#NUM!	228.102	2
4	56.14268	#NUM!	#NUM!	#NUM!	229.3711	4
6	68.67314	#NUM!	#NUM!	#NUM!	230.6402	6
8	79.19596	#NUM!	#NUM!	#NUM!	231.9093	8
10	88.43076	#NUM!	#NUM!	#NUM!	233.1784	10
12	96.74709	#NUM!	#NUM!	#NUM!	234.4475	12
14	104.3647	#NUM!	#NUM!	#NUM!	235.7166	14
16	111.4271	#NUM!	#NUM!	#NUM!	236.9857	16
18	118.0339	#NUM!	#NUM!	#NUM!	238.2548	18
20	124.2578	#NUM!	#NUM!	#NUM!	239.5239	20
22	130.1538	#NUM!	#NUM!	#NUM!	240.793	22
24	135.7645	#NUM!	#NUM!	#NUM!	242.0621	24
26	141.1241	#NUM!	#NUM!	#NUM!	243.3312	26
28	146.26	#NUM!	#NUM!	#NUM!	244.6003	28
30	151.1952	#NUM!	#NUM!	#NUM!	245.8694	30
32	155.9487	#NUM!	#NUM!	#NUM!	247.1385	32
34	160.5366	#NUM!	#NUM!	#NUM!	248.4076	34
36	164.9727	#NUM!	#NUM!	#NUM!	249.6767	36
38	169.269	#NUM!	#NUM!	#NUM!	250.9458	38
40	173.4359	#NUM!	#NUM!	#NUM!	252.2149	40

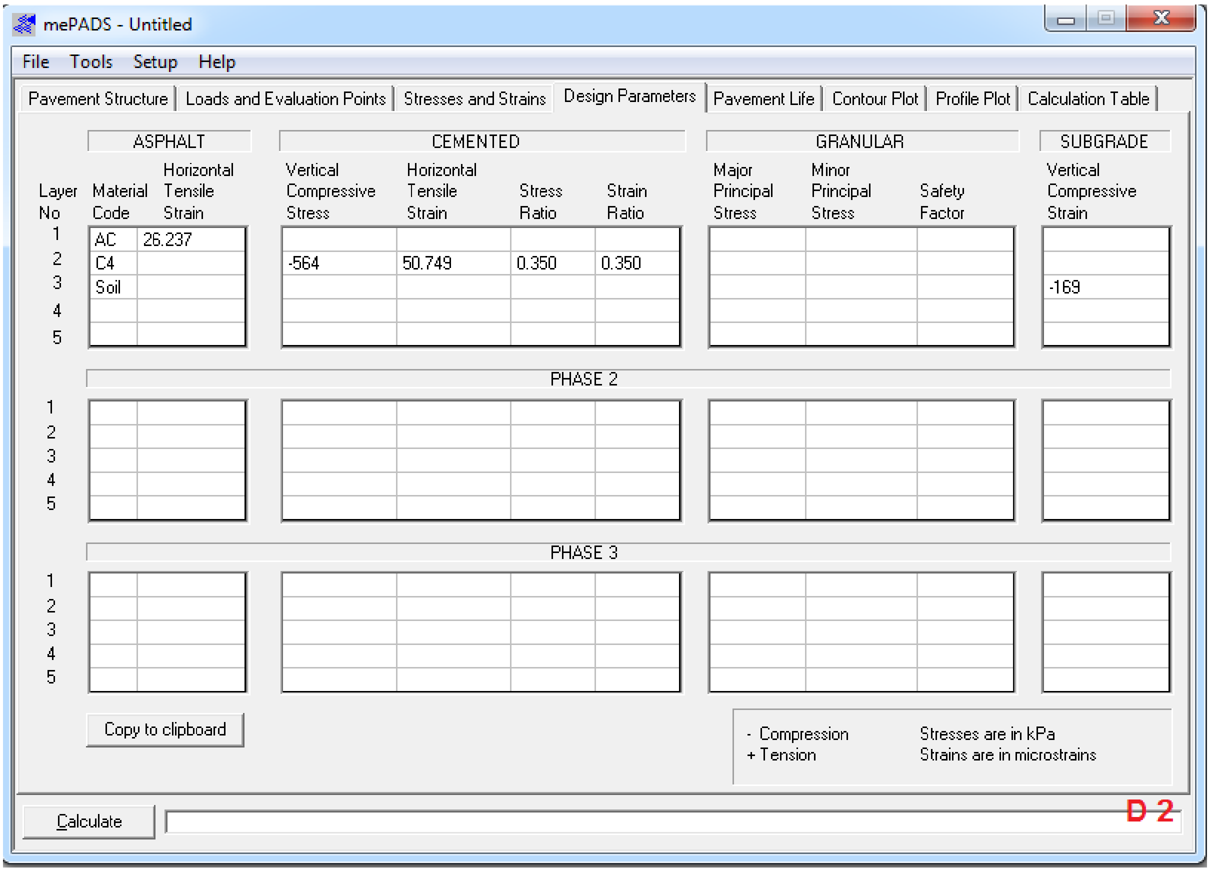
C 1

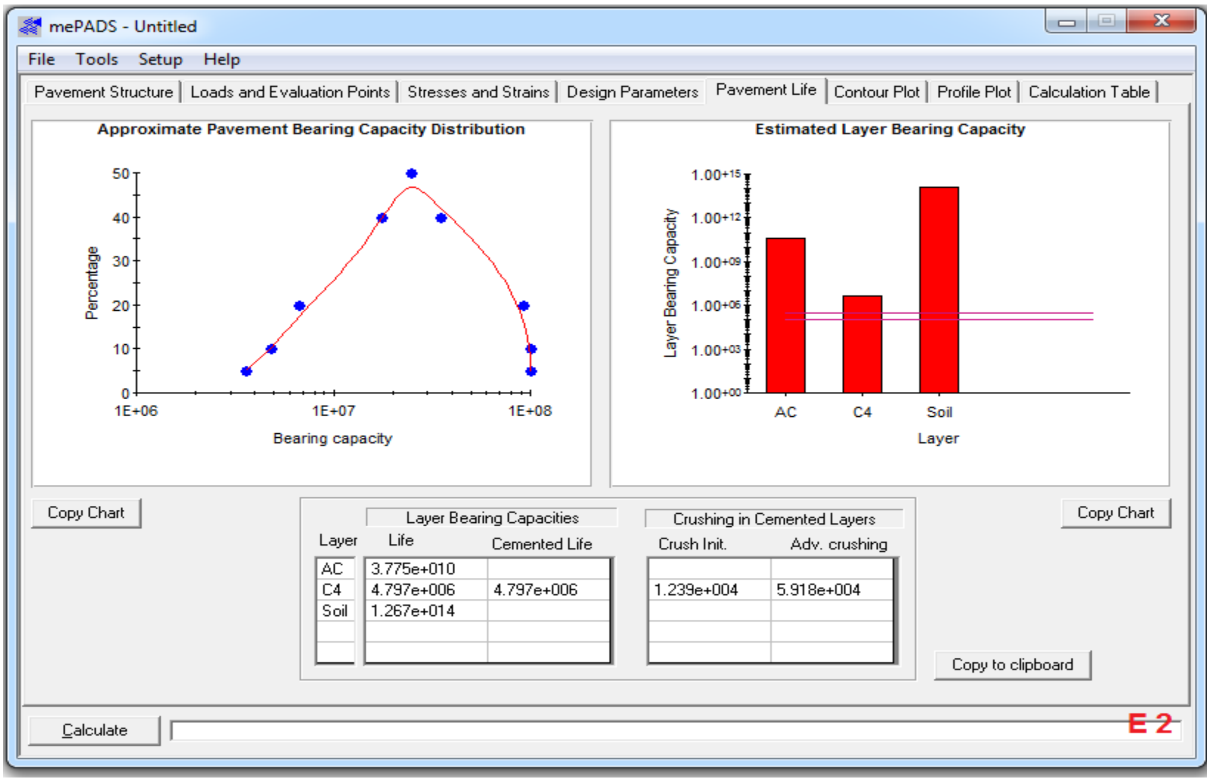
Appendix A1, B1 and C1 are extract form the Triaxial test results (Heyns and Mostafa Hassan, 2013).

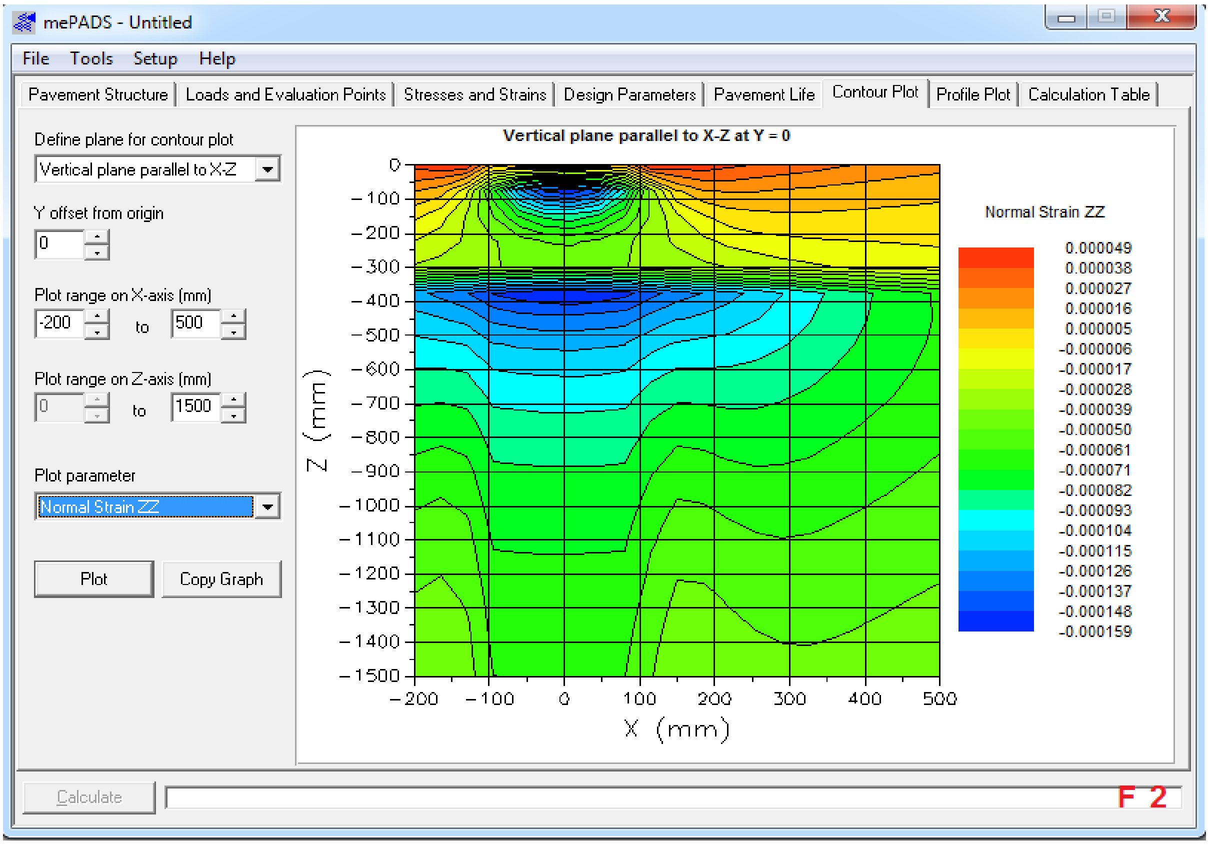

mePADS - Untitled
File Tools Setup Help
Pavement Structure | Loads and Evaluation Points | Stresses and Strains | Design Parameters | Pavement Life | Contour Plot | Profile Plot | Calculation Table
Number of Layers: 3
Number of Phases: 1
Default input: Off
Phase 1
Material Thickness E-Modulus Poisson's Ratio
AC 50 3000 0.44
C4 300 2560 0.35
Soil 2000 45 0.35
Material E-Modulus Poisson's Ratio
Material E-Modulus Poisson's Ratio
Climatic Region Moderate
Road Category B
Terminal rut 20 mm
Design Traffic class ES0,3
CSIR TRANSPORTEK
Technical support: Hechter Theyse
email: htheyse@csir.co.za
Software support: Johan du Toit
email: jadutoit@csir.co.za
Heading
Description
A 2
Calculate


mePADS - Untitled
File Tools Setup Help
Pavement Structure | Loads and Evaluation Points | Stresses and Strains | Design Parameters | Pavement Life | Contour Plot | Profile Plot | Calculation Table
Design location
X Y
0 0
Load definition
No of loads 1
Std. Loads
Position
Single Load (kN) Pressure Radius X Y
20 650 98.9654 0 0
Stresses and Strains
No of evaluation positions 1
X Y Z
0 0 0
Plot
Copy Chart
Load Positions
1500
1000
500
-500
Y (mm)
1000
2000
3000
X (mm)
B2
Calculate


mePADS - Untitled
File Tools Setup Help
Pavement Structure | Loads and Evaluation Points | Stresses and Strains | Design Parameters | Pavement Life | Contour Plot | Profile Plot | Calculation Table
Stress and Strain Results
Phase 1
x = 0 y = 0 z = 0
SXX = -820.215 SYY = -820.215 SZZ = -650.000
SXY = 0 SXZ = 0 SYZ = 0
PS1 = -650.000 PS2 = -820.215 PS3 = -820.215
PSS1 = 85.107 PSS2 = 85.107 PSS3 = 0
UX = 0 UY = 0 UZ = 0.171
EXX = -5.777e-005 EYY = -5.777e-005 EZZ = 2.393e-005
EXY = 0 EXZ = 0 EYZ = 0
PE1 = 2.393e-005 PE2 = -5.777e-005 PE3 = -5.777e-005
PSE1 = 8.170e-005 PSE2 = 8.170e-005 PSE3 = 0
Stresses are in kPa
- Compression
+ Tension
Copy to clipboard
C 2
Calculate


mePADS - Untitled
File Tools Setup Help
Pavement Structure | Loads and Evaluation Points | Stresses and Strains | Design Parameters | Pavement Life | Contour Plot | Profile Plot | Calculation Table
ASPHALT CEMENTED GRANULAR SUBGRADE
Layer No | Material Code | Horizontal Tensile Strain | Vertical Compressive Stress | Horizontal Tensile Strain | Stress Ratio | Strain Ratio | Major Principal Stress | Minor Principal Stress | Safety Factor | Vertical Compressive Strain
1 AC 26.237
2 C4 -564 50.749 0.350 0.350
3 Soil -169
4
5
PHASE 2
1 2 3 4 5
PHASE 3
1 2 3 4 5
Copy to clipboard
- Compression
+ Tension
Stresses are in kPa
Strains are in microstrains
D 2
Calculate





Appendix A2 – F2 are extract from the mePADS software results showing the various steps and results.