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Development and Validation of Characterization Method Using Finite Element Numerical Modeling and Advance Laboratory Methods for Western Australia Asphalt Mixes

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DECLARATION

This is thesis is wholly my own composition and contribution, and where I have used other sources I have acknowledged their contributions. This thesis has not previously been accepted for any degree in this or another institution and has been entirely accomplished during enrolment in the degree held at Curtin University. Thesis is largely composed of nine papers which are published. The journals and international conference proceedings publishers have given permission for these papers to be included in this thesis.

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ABSTRACT

With the increased demand for new development of asphalt paving hot-mix asphalt (HMA), it has become very difficult to manage the stresses and strains of the road effectively and efficiently, which has been caused by the traffic loading and environmental factors (temperature, moisture, ageing, etc., and so forth). Over the past five decades, the versatility of asphalt has led to its increase use in other application such as HMA airport runways are funding increased acceptable in many countries in the world. The last five decades has been seen a remarkable increase in demand for moving people and goods, and the surface transportation system has been under pressure and also been very difficult to sustain and manage the substantial increase in heavy truck traffic volume levels. The annual expenditures worldwide are allocated for maintenance and rehabilitation of urban traffic road is huge. For example, over the past 10 years, the use of full depth asphalt to construct and rehabilitate heavily loaded urban roads has grown rapidly in Western Australia and it has become difficult to sustain and maintain the traffic road networks. This dissertation aims to address some of these deficiencies by exploring the true characterization of asphalt mixtures using numerical and laboratory techniques.

With the current moves adapted mechanistic-empirical concept in design of pavement structure, state-of-the-art mechanistic methodologies are needed to determine accurate flexible pavement responses such as stress, strain, and displacement. An experimental program was performed to evaluate the engineering properties of the asphalt mixtures with various types of materials based on engineering principles and pavement materials characterization of hot-mix asphalt (HMA) mixtures. The goals for these experiments are to assess the effect of aggregate type and asphalt binders in the HMA mixtures, determine an appropriate pavement thickness and develop pavement design so that an accurate and reasonable performance of asphalt pavement mixtures can be achieved. Different types of asphalt mixes were produced in the experiment; and these experiments test involved various parameters. Results demonstrated that AC20-75 and AC14-75 Blow asphalt mixes were more effective and durable in pavement performance as compared to the others asphalt mixes. In general, all the asphalt mixes that are used in this study can be strength and stable the mixture stiffness of asphalt that is notable. For the purpose of pavement design, the modification effect rank can be described as AC20-75 blow > AC14-75 blow > AC14-50 blow > AC7-50 blow >

SMA7-50 blow in this research.

To properly characterize the asphalt mixtures and improve the methodologies and engineering techniques of pavement performance, this dissertation also describes the development and implementation of the Network Optimization Systems (NOSs). Two methods: Lao Road Design Manual (LRDM) inventory data, was used to assess the current pavement road networks and Probabilistic network-Markov-Chain Process and Chapman-Kolmogorov method, were used to predict the pavement behavior in Western Australia (WA). The main purpose of this study is to assess and analysis the pavement network performance of WA and also to apply the existing pavement management system tools relevant to WA road networks.

Another, this dissertation is used seven asphalt concrete mixtures of different types of polymer modified binders (PMB) that were produced in the laboratory experiment to develop a master curve and a dynamic (complex) modulus, E^* to characterize the modulus of asphalt mixtures. The main role of this particular research is to evaluate the influence and/ or effective of these polymer modifiers on pavement of performance of asphalt mixtures with the dynamic (complex) modulus, E^* of HMA mixtures. The complex mixture moduli was related to temperature and time rate of loading, which has been an integral part of several mechanistic-empirical (M-E) design procedures and throughout of the world. In this study, the influence of temperature, loading frequency, and confining pressure on the dynamic characteristics of asphalt mixtures were analyzed and master curve of dynamic modulus of HMA were developed and data's were interpreted, and relationship between viscosity of binders and temperature were established.

To reduce a tremendous amount of time and money that has been speeding each year on maintenance and rehabilitation of existing asphalt pavement distress, this dissertation conducted a field survey and collected a pavement distress data on a significant components for effective long-time pavement performance. The main objective of this study is to identify and quantify of surface distress in a given segment of pavement, to perform details distress ratings, to predict pavement temperature and cost analysis of individual pavement distress on heavily urban roads in WA. Field survey were conducted from three regions, and two approached were used to evaluate and analysis the pavement distress. First, the Probabilistic network Markov-Chain Process method (nonlinear algorithms methods) was used to predict the cost analysis for individual asphalt concrete surfaced pavement distress. Second, Statistical Downscaling Model (SDSM) was used to predict pavement temperature, and results were performed and analyzed.

This dissertation also describes the content understanding of shakedown concept and simulation of shakedown behavior for flexible pavements of unbound granular layer (UGL). The objective of this study is to develop a simulation model for shakedown behavior of granular layer in flexible pavement using finite element method in ABAQUS. This method is integrated with Mohr-Coulomb criterion, which used and applied layer to dynamic loading in numerical analysis. A new constitutive model is developed based on Mohr-Coulomb criterion and Drucker-Plager method for flexible pavement unbound granular material (UGM). Results showed that the new developed constitutive model is capable of considering effect in base UGL and predicting the various type of observed flexible pavement failure and the effect of various design parameters. The implementation of this new developed constitutive model is verified against published results of laboratory test data measured shakedown for UGM and was also coded in UMAT under ABAQUS simulation. Results has also shown 50% reduction of vertical plastic strain of the base (UGL) layer with shakedown model as compared to Mohr-Coulomb after 7 s repetion of cyclic loading.

Falling weight deflectometer (FWD) is currently used by most highway agencies to assess the structural condition of the highway network based on engineering. This dissertation also validates the backcalculated moduli theoretically and through instrumental response on top of asphalt pavement mixtures. The main objective of this study is to analyses the FWD test result so that the resilient moduli of unbound granular material on the Western Australia roads network can be determined and deflections basin among pavement sections can be analysis. FWD measurement was taken from seven locations of traffic roads including core data, roadway, and pavement distress information. BISDEF and ELSYM5 computer software program was used for backcalculations analysis. The subgrade was modeled as a finite-thickness, homogenous, linear-elastic layer placed on top of a bedrock and dynamic deflection basin were obtained by computed the deflection at the sixteen geophone locations. Results were compared to WESLEA (WES), Finite Element Method (FEM), and Method of Equivalent Thickness (MET). The results clearly demonstrated that the flexible pavement layer moduli and algorithms for interpretation of the deflection have improved. The variation of the moduli of all layer along the length of sections for majority of the roads were accurate and consistence with measured and computed predicting. The influence lines for vertical strain at top of subgrade as measured by gauges and predicted by different types of response (i.e., WES, FEM, and MET) also showed that the MET is seen the best predictor as compared to WES and FEM.

Finally, this dissertation developed a new numerical simulation to obtain a various structural parameters: stresses, strain, and displacement of five-layers of flexible pavement using three-dimensional nonlinear finite element (FE) method in ABAQUS. The main purpose of this study is to develop a new simulation that can perform an accurate and effective solution for stresses and strains characteristics problems in flexible pavement. New constitutive model is developed based on theory of Hooke's law (three-dimensional) for stress and strain. Also, 40 kN wheel load to represent a set of dual tires was assumed to be uniformly distributed over the contact area between tire/pavement surface and simulated using linear viscoelastic and nonlinear viscoelastic material. In addition, four different tire inflation pressures were used (350, 490, 630 and 700 kPa). Results showed that the new developed constitutive model for stress-strain behavior and/or characteristics is capable of considering the effects of stress and strain in pavement layers and predicting of various type of observed in flexible pavement failure and the effects of various pavement design parameters. The implementation of this new developed constitute model is verified against published results of laboratory test data. Results has also shown 40% reduction of vertical plastic strain of each flexible pavement layers with nonlinear viscoelastic materials as compared to the linear viscoelastic materials after 7,500 s repeated time of 30, 000 cyclic loading.

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PUBLICATION FROM THE THESIS

The publication generated as a part of this PhD thesis, like the remainder of the thesis, were authored by myself, though valuable contributions came from the co-authors of the various papers: my supervisor Professor H. Nikraz and Professor I.L. Al-Qadi, and colleagues, Mr. C. Leek, Mr. B. Ghadimi and Mr S. Herath.

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"Praised be to the name of the LORD God Almighty who has given me knowledge, skills in all literature, and wisdom and understanding. The Lord has opened many doors for me to travel to the entire world to present my PhD research works in many big international conferences. I am very grateful for my supervisor Professor Hamid Nikraz, and the Department of Civil Engineering for their financial supports for all my international conferences. Dr Ivan Haigh, Lim Poh Hong, Heather and George Thompson for their great concern and prayer are appreciated." It is the Lord (John 21:7).

DEDICATION

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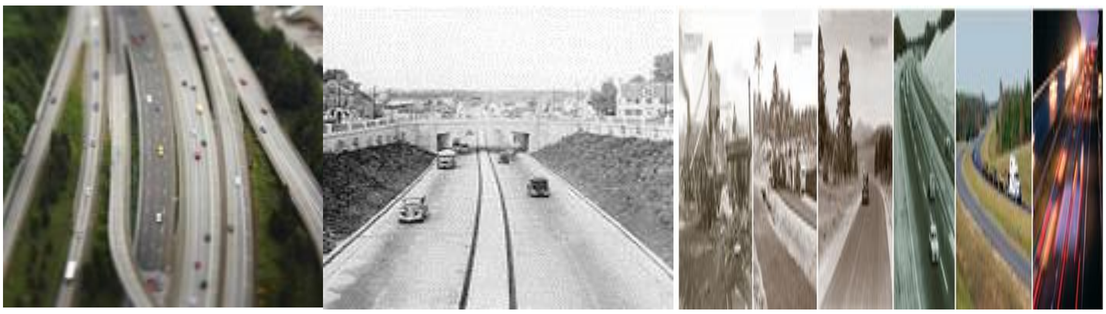
CONTENTS

CHAPTER 1: INTRODUCTION

1.1. BACKGROUND

Historically, early asphalt works were implemented to meet human needs. In the absence of large population, industries, recreations, heavily loaded urban town roads, traffic road congestions and transportation, there was not much need for asphalt and asphalt paving materials usage for road construction. However, with the increase population, industries, recreation, heavily loaded urban road, traffic congestions and transportation, the need for asphalt and asphalt paving materials usage for various road constructions sector has begun.

The story of asphalt begins thousands years before the founding of the United States. Asphalt occurs naturally in both asphalt lakes in rock asphalt (a mixture of sand, limestone, and asphalt). The first recorded use of asphalt as a road building material was in Ur in modern-day Iraq around 625 B.C., in the reign of King Naboppolassar (Gillespie 1992; McNichol 2005) and timber roads preserved in a swamp in Glastonbury in England (Larrañaga 1838). Gillespie (1992) noted that the streets and wall of Ur (i.e. modern-day Iraq) was paved with asphalt, stone and burned brick. Figure 1 shows the development of asphalt road and asphalt paving materials in a history.



(a) Ur in modern-day Iraq (b) Glastonbury, England (c) New development of asphalt

Figure 1.1 Development of asphalt (HMA) road and asphalt paving material in a history (McNichol 2005; Gillespie 1992)

The ancient Greeks were familiar with asphalt and its properties. The word asphalt comes from the Greek "asphaltos", meaning "secure" as discussed on literature by (Gillespie 1992; Larrañaga 1838; McNichol 2005). The Romans changed the word to "asphaltus", and used the substance to seal their baths, reservoirs and aqueducts (McNichol 2005). Many centuries later, Europeans exploring the New World discovered natural deposits of asphalt. Sir Walter Raleigh described a "plain" (or lake) of asphalt on the island of Trinidad, near Venezuela. He used it for re-caulking his ships (McNichol 2005).

The road builder of the late 1800s depended solely on stone, gravel and sand for construction, and water would be used as a binder to give some unity to the road surface (McNichol 2005). *John Metcalfe*, a Scot born in 1717, built about 180 miles (i.e. about 290 km) of roads in Yorkshire, England (even though he was blind), and his well-drained roads were built with three layers: large stone; excavated road materials; and a layer of gravel as described on literature by (Gillespie 1992; Larrañaga 1838; McNichol 2005).

The first road use of asphalt occurred in 1824, when asphalt blocks were placed on the Champs-Élysées in Paris (Abraham 1920). Modern road asphalt in 1870 was the work of Belgian immigrant Edward de Smedt at Columbia University in New York City as discussed on literature by (Abraham 1920; De Smedt 1870; Finn, Nair & Monismith 1972; Gillespie 1992). De Smedt invented modern road asphalt and patented it (U.S. Nos. 103,581) and called it "sheet asphalt pavement" but it became known as French asphalt pavement (Gillespie 1992). By 1872, De Smedt had engineered modern, "well-graded," maximum-density asphalt. The first uses of this road asphalt were in Battery Park and on Fifth Avenue in New York City in 1872 and on Pennsylvania Avenue, Washington D.C., in 1877 (Abraham 1920; De Smedt 1870). From this beginning, the production of asphalt in the United States has dramatically increased with more than 25 million tons of asphalt and road oil produced in 1964 (Walter 1975).

According to De Smedt (1870), Modern tarred roads were the result of the work of two Scottish engineers, *Thomas Teford and John Loudon Mcdam*. Telford designed the system of raising the foundation of the road in the center to act as a drain for water. Thomas Telford (born 1757) improved the method of building roads with broken stones by analysing stone thickness, road traffic, road alignment and gradient slopes as discussed by (Gillespie 1992; Gillespie et al. 1992). Eventually his design became the norm for all roads everywhere. John Loudon McAdam (born 1756) designed roads using broken stones laid in symmetrical, tight patterns and covered with small stones to create a hard surface (De Smedt 1870; Gillespie 1992). McAdam's design, called "macadam roads," provided the greatest advancement in road construction.

Alsing (1868) invented a new and improved asphalt pavement using distilled coal tar or English asphaltum and crude coal-tar at temperature varying from fifty to hundred degrees so that the pavement would be protected from being damage as result of moisture and change of temperature. To furnish an improved pavement, ten parts of Portland or hydraulic cement, five parts of granite, five parts of glass and coarse gravel were also crushed to the finesses of coarse sand (Epps & Monismith 1972). The results show that the mixture of the aggregates were homogenous and also improved the strength of asphalt pavement. Similarly, Dotch (1969) had also invented an improved concrete for paving, roofing and other technical purposes. The nature of inventions were consists in mixing bitumen, pine-tar, pitch and hydrocarbon with vegetable fibrous such as sawdust, tan-bark, straw, wool waste, cloth flacks, cotton seed and petroleum residue or other boiled oil (Terrell & Krukar 1970). The results indicate that the mixing materials were at the best when the fibrous materials are mixed with one or more of hydrocarbon to the exclusion of sand or any other minerals or cement.

The last 50 years have seen remarkable increase in the demand for moving people and goods. The surface transportation system has been under pressure to sustain the substantial increase in heavy truck traffic volume levels. Considerable annual expenditures worldwide are allocated for developing the transportation infrastructure including urban roads and highways. For example, In Canada, $ 8 billion dollars is allocated annually for the rehabilitation of urban roads (Ali 2005; Statistic Canada Agency 2003), and In the United States, $ 51 billion dollars is allocated to the states for road construction (McNichol 2005) and In Australia, $18 billion dollars is annually used for urban roads maintenance and rehabilitation (Australian Transport Council 2008; Australian Transport Safety Bureau 2000), with overlays being one of the most commonly used techniques. Durability of the pavement structure determines the initial cost of the road and its future maintenance requirements.

With the increased demand for new development of asphalt paving (HMA), it is becoming difficult to manage the stresses and strain of the road effectively and efficiently, which has been caused by the traffic loading and environmental factors such as temperature, healing, ageing, etc. and so forth. Although over the last 30 years, the versatility of asphalt had led to its increased use in other application. HMA airport runways are finding increased acceptance in the United States and the rest of the world (Gillespie 1992; Gillespie et al. 1992), since they provide passengers more comfortable takeoffs and landings, dramatically out back on runway maintenance, and allow for much faster construction time (Gillespie 1992; Goumans, Sloot & Aalbers 1991; McNichol 2005; Pindzola & Collins 1975; The Asphalt Institute 1963).

Pavements constructed of high-durability HMA mixes used increasingly for freight yards, where they stand up to heavy static loads. HMA is also used worldwide as a solution to water storage, flood control, erosion and conservation problems (McNichol 2005). Asphalt has been approved by the Environmental Protection Agency and used successfully as a primary liner for both sanitary and hazardous-waste landfills. It is also used to line drinking water reservoir and fish hatcheries in California and Washington (Goumans, Sloot & Aalbers 1991; McNichol 2005). However, there are still many things to take into consideration in regarding the asphalt (HMA) pavement design and planning such as variable of asphalt mixtures (i.e. difference), faulty design, unbalance aggregate compaction, human error (i.e. data analysis and test methods) and lack of skilled (i.e. judgment and insufficient idea to apply) and the difference in between laboratory and in-service or field test measurements (Baburamani 1999; Epps & Monismith 1972; Finn, Nair & Monismith 1972; Goumans, Sloot & Aalbers 1991; Marshall & Evans 1985; Mundt, Adams & Marano 2009; Mundt et al. 2009), are some of the few to mentions. Despite of all these things, the demand for hot mix asphalt is project to increase whether in Western Australia or the rest of the world.

1.1.1. HISTORY OF ASPHALT ROADS IN WESTERN AUSTRALIA

It is unlikely to find any scientific or laboratory testing of materials used in road construction in the first 10 decades of Western Australia (Hamory 1996). According to (Edmonds 1997; Hamory 1996) literature, it is possible that the Royal Engineers who were stationed at Western Australia (WA) during the middle of the 19th century had recommended with respect of standard size or even other attributes of asphalt pavement materials. There is evidence that in some cases calls for tenders by the State and Local Government Authorities to supply asphalt pavement materials specified maximum particle size (Edmonds 1997; 2008; Hamory 1996). However, it is not known how whether compliance with these specifications was established.

After arrival of the motor vehicles early in the 20th century, there was a need for dust free bituminous surfacing and pavement which were able to carry the gradually increasing roads and speeds of the traffic. The establishment of the University of Western Australia (UWA) contributed to the introductions of scientific methods to assess the quality of the material used in road construction (Edmonds 1997; Hamory 1996).

Edmonds (1997) described the two most important aspect of the history of Main Roads Western Australia that has been on: the roads, the facilities, and services associated with them, and on the other hand, the people who provides those roads and services. The author also discussed that the two areas on his literature: the Western Australia road system and the Main Roads Board (i.e. the Main Road Western Australia was created as organization for the first time in 1926).

While the Western Australian road network will continue to come under increasing pressure from growth in traffic volumes, workforce management is likely to be a key issue in delivering services to meet the increasing needs of the community. Main Roads, like many organizations, is faced with an ageing workforce at a time when labour supply is expected to tighten (MRWA2007). Main Roads was awarded a Bronze Award as part of the Australian Business Excellence Awards. The Australian Business Excellence Awards is an internationally recognized award that enables Main Roads to benchmark against leading international business practices. This success demonstrates that the deployment of current business practices is well targeted and contemporary.

Over 2006-07, almost $429 million was expended on Road Network Maintenance which made up 38% of the total road program. The following are some of the works undertaken during the year. Eight regionally based 10-year Term Network Contracts (TNCs) were established to provide road maintenance and rehabilitation services on the State road system and for regulatory signs and road lines on local roads. The contracts provide a range of maintenance services to help ensure that road users are provided with a safe and efficient road system and that the value of the road asset is preserved. During the year $131 million was spent on direct contract payments (MRWA2007).

Asphalt Mixtures Hot-mix asphalt (HMA) mixtures are a heterogeneous complex composite material of air, binder and aggregate which are used in pavement construction. HMA mixes are used all over the world. Despite, this widespread usage, the fatigue characterization of HMA mixtures to ensure adequate field fatigue performance is not very well established, and fundamental fatigue predictive models are still remain to be developed (Kim et al. 1997).

Under the heavily urban traffic loading and changing of environmental condition, HMA mixtures exhibit non-linear, visco-elastic and plastic anisotropic behavior. The mechanical properties and performance of the asphalt mixtures are dependent on loading rate, temperature, and direction of loading (Kim, Lee & Little 1997; Lytton 2000; Lytton et al. 1993). With time, HMA can be affected with fatigue and ages, although it has a potential to heal (means closure of fracture pavement surfaces) during traffic loading rest period (Kim et al. 1997). This complex nature of HMA response behavior under changing traffic loading and environmental conditions makes it difficult to adequately model HMA mixture properties, in particular with respect to pavement distress such as fatigue cracking…..

Complexity can contribute to attribute for inadequate prediction of HMA mixtures and reduced a resistance to fatigue due to effects of binder oxidation aging. These effects can increase both binder viscosity and elastic moduli, and also reduced the hot mix asphalt mixture ductility and increased its susceptibility to fatigue cracking (Glover et al. 2005). However, little is understood nor documented about the effect of binder oxidative aging on both HMA concrete mixtures properties and fatigue resistance (Walubita 2006).

The comprehensive HMA mixture fatigue analysis approach that take into account the complex nature of HMA are the desired to ensure adequate field fatigue performance is considered. Analysis of models associated with such approaches should have the potential to utilize fundamental asphalt mixture properties that are critical to HMA pavement fatigue performance and pavement fatigue life $\left(N_f\right)$. In addition, fatigue failure criteria should be based on a simulation of direct relationship damage accumulation in the field.

Various fatigue analysis approaches have been developed and some are in use in current world of asphalt mixture. However, some are inadequate in proceeding fatigue resistance HMA asphalt mixture or flexible pavement structures that are structurally adequate in fatigue throughout the pavement analysis and design life (Walubita 2006). Consequently, fatigue cracking continues to be prevalent in today HMA pavement concrete. Asphalt mixture resistance to fatigue cracking is directly tied to its mechanical response under repeated traffic loading that depends on the entire asphalt pavement structure. Fatigue response behavior of the top HMA layer under traffic loading is also dependent on the material properties and pavement structural capacity of underlying layers. This unique characteristics inevitability calls for fatigue analysis approached that adequately interface both HMA mixture fatigue characterization and pavement structural design.

Asphalt Mix Design The SuperPave asphalt mix design method has increasingly been accepted although the system has not yet implemented in Western Australia. The current SuperPave mix design approach is based on meeting certain asphalt binder, aggregate and volumetric properties such as the asphalt binder performance grading (PG) specification, aggregate gradation control limits, gradation restricted zone, asphalt mix air voids, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), and so forth. The SuperPave mix design system has continuously been evaluated to search for further improvement.

Gradation is perhaps the most property of aggregate. It affects almost all the important properties of hot mix asphalt (HMA) mixtures including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture damage (Xiao 2009). Gradation is a primary consideration in asphalt mix design, and the SuperPave specification place limits on aggregate gradation so that it can be used in HMA mixtures. The gradation of an aggregate is important to ensure that the maximum aggregate size is not either too large or small, VMA requirements are met and a satisfactory aggregate skeleton is obtained. According to the SuperPave, the aggregate gradation must be within the control limits to meet the SuperPave requirement.

For example, if a 19 mm (3/4-inch) maximum aggregate size is specified, then 100 percent of the aggregate must pass the 25- mm (1-inch) sieve size. At least 90-100 percent of the aggregate must be finer than the normal maximum size 19 mm. Less than 90 percent of the aggregate must pass the 12.5 mm (1/2-inch) sieve. In order to meet the SuperPave requirements, a coarse graded aggregate will have to be “gab-graded” to be within the normal size control limits. However, a “smoother” coarse gradation passing below the lower control limit of the normal maximum sieve size may exist to provide as similar result to the “gab-graded” curve. Research is need to evaluate the normal size control limit for SuperPave mix design and also study its effect on mechanical properties of HMA mixes.

Recently, a new AASHTO WARE Mechanistic-Empirical (M-E) Design Guide research team advanced the use of the dynamic complex modulus (E^*) as the primary test protocol to characterize the modulus response of HMA mixtures (NCHRP 1-37A2004). The research team supported the role, selection, and utilization of the dynamic complex modulus tests for asphalt concrete mixtures over the indirect tensile resilient modulus (M_R) in the National Cooperative Highway Research Program (NCHRP) 1-37A Project concerning the AASHTO M-E Design Guide for Pavement Structures (NCHRP 1-37A2002; 2004; Schwartz & Carvalho 2007), which is currently aiming to introduce more rigorous measures of performance into HMA mixture and pavement analysis and design procedures. The use of the indirect tensile test was also encouraged as a means of deforming the relative moduli response of field cores taken from rehabilitation design (NCHRP 1-37A2002; Schwartz 2007). However, the use of the test to characterization modulus at high temperature was not recommended (NCHRP 1-37A2004).

The difference between a dynamic complex modulus test and a resilient modulus test for HMA mixtures is an application of a sinusoidal or haversine loading with no rest period or with a given rest period, respectively. The dynamic complex modulus is also one of the several methods for describing the stress-strain relationship of viscoelastic materials. Modulus has a complex quantity because the real part represents the elastic stiffness while the imaginary part characterizes the internal dimpling of the material. The absolute value of the complex modulus is commonly referred to as the “dynamic modulus”.

A detailed comparison of the difference between the dynamic complex test and the indirect diametric resilient modulus test for asphalt concrete mixtures was summarized by the NCHRP 1-37A 2002 Project research team (NCHRP 1-37A2002; Witczak 1999). The transition from the resilient modulus to the use of the dynamic complex modulus test for design of flexible pavement structures has a tremendous significant for a transportation agencies. In response to the need, an effort is desired to characterize the empirical relation between dynamic complex modulus and indirect tensile resilient modulus of HMA mixture for Western Australia asphalt mixtures.

Flexible Pavement Structure (FEM) Hot-mix asphalt (HMA) with asphalt concrete (AC) are used all around in the entire world. The various layers of the flexible pavement structure have different strength and deformation characteristic which makes the layered system difficult to analysis in pavement engineering. Asphalt concrete in the pavement surface layers is a viscous material with its behavior depending on time and temperature. On the other hand, pavement foundation geomaterials such as coarse-grained unbound granular material in untreated base and subbase course, and fine-grainer in soils in the compacted and natural subgrade, exhibit stress-dependent nonlinear behavior. Most of the current used flexible pavement structural analysis models in all around the world assume linear elastic behavior. As the demand for applied wheel loads and number of load applications increase, it becomes very important to properly characterize the behavior of asphalt concrete mixes as being the top layered pavement surface structure and unbound granular material and compacted subgrade and natural subgrade soil layers as the foundation of the layered pavement structure.

Previous laboratory studies have shown that elastic or resilient response of granular materials in base, subbase and subgrade natural soils follow a nonlinear and stress-dependent behavior under repeated traffic loading (Brown & Pappin 1981; Kim 2007; Thompson & Elliott 1985). Unbound granular material exhibits stress-hardening, whereas fine-grained natural soils show stress-softening type behavior (Kim 2007). Finite element method (FEM) model in ABAQUS computer software program that analyze flexible pavement structure needs to solve this kind of nonlinear resilient characterization to more realistic predict the flexible pavement. Although specific pavement structural analysis program such as the ILLI-PAVE (Raad & Figueroa 1980) and GT-PAVE (Tutumluer 1995), take into account stress-dependent moduli, the general-purpose of finite element method program do not properly account for such nonlinear behavior of the flexible pavement geomaterials (Kim 2007). Recent work by Taciroglu (1998) and Schwartz (2002) clearly indicated the need to develop proper flexible pavement geomaterials constitutive behavior models to use in general purpose of finite element method modeling programs.

E* Predictive Models for HMA Mixtures HMA mixtures in a pavement structure are subjected to a wide range of traffic loading and environmental conditions. The response to such kind of condition is very complex and involves the elastic viscoelastic and plastic characteristics of material used in the pavement. The stiffness of a HMA mix is a specific material behavior response parameter that determines the strain and displacement of pavement structure when it is under loaded or unloaded condition. In the early 1950's Van der Poel of the shell oil company introduce term which is so called "stiffness or stiffness modulus" (Van del Poel 1954). The stiffness of a HMA mix is a modulus that is dependent upon the traffic loading time and temperature of the asphalt mix.

Due to the immense important of stiffness in the analysis, design, and performance evaluation of HMA mixture and flexible pavement structures, several researchers have been trying to develop accurate and durable stiffness modulus in laboratory test protocol as well as in developing accurate predictive models and equations for flexible pavement structure. Over the past five decades, several numerous models and regression equations have been developed to predict the stiffness (or modulus) of a HMA mix. Historically, the stiffness predictive models and equations were developed based on the conventional multivariate linear regression or non-linear regression analysis of laboratory test data to established or anticipated basic engineering behavior and material properties of the HMA mixtures (Bari 2005).

Mechanistic-Empirical (M-E) Design Efforts In the recent worldwide pavement community, potential research efforts are currently being focused on the Mechanistic-Empirical (M-E) design of pavements. Since potential M-E designs are mechanics based and they can be adapt to varied and changing distress modes, load limits and load configurations. They are also allowing for rational material tests and characterization for direct interaction between pavement structure and material design.

At present time, there are several computer software programs available to mechanistic-empirically evaluate stresses and strains in a pavement for a given set of loading, environmental and pavement cross section condition. For example, the basic material properties required for multi-layer linear elastic analysis are the material modulus (E) and poisson's ratio (ν). Unfortunately most, if not all, pavement materials are not purely elastic. For instance, for asphalt concrete (AC) layer, the modulus varies considerably with temperature and rate or time of loading (i.e. true viscoelastic responses). That is way, in recent years; pavement materials have concentrated their research efforts on the time-temperature dependent stiffness of hot mix asphalt (HMA) pavement mixtures.

Recently, Dr. Matthew W. Witczak, leader of asphalt team for the National Cooperative Highway Research Program (NCHRP) 1-37A Project and Professor at Arizona State University (ASU), completed a multi-year compressive research under NCHRP 1-37A Project (NCHRP 1-37A2004). The primary goal of this research effort was to develop the flexible pavement analysis and design part of the draft AASHTO Pavement Design Guide titled "Guide for Mechanistic-Empirical of New and Rehabilitated Pavement Structures" (NCHRP 1-37A 2002). While this project was initially known as "2002 Design Guide", it is now referred to as the M-EPDG (Mechanistic-Empirical Pavement Design Guide) (NCHRP 1-37A2002).

Dynamic Modulus As HMA Stiffness Historically, various types of material properties and parameters have been used for presenting the stiffness (or modulus) characteristic of pavement asphalt mix that include flexural stiffness, creep compliance, relaxation modulus, resilient modulus, dynamic modulus and so forth (Bari 2005). At present, one of the most universally used methodologies to characterize the modulus of asphalt mixtures is the dynamic (complex) modulus (E^*). Dr. Matthew W. Witczak of Arizona State University (Previous of the University of Maryland) was the principal investigator for NCHRP Project 9-19; Mr. Harold Von Quintus of Pargo-BRE, Inc., Dr. Charles W. Schwartz of University of Maryland were the co-principal investigators, and they demonstrated that the complex (dynamic) modulus (E^*) can be used as a good performance indicator for the HMA design stage (Witczak et al. 2002).

Witczak and other colleagues working on the NCHRP Project 9-19 have summarized several advantages of the use of E^* in HMA pavement analysis and design over the stiffness parameters such as the Resilient Modulus as: (1) E^* data allows a hierarchical HMA mixture characterization approach to be used; (2) aging can be taken into account; (3) vehicle speed (time of load) can be taken into account; (4) E^* can be linked to the SHRP Performance Graded binder specification; and (5) E^* is more fundamentally and theoretically compared to the Falling Weight Deflectometer (FWD) back-calculated modulus of HMA mixture (Witczak et al. 2002).

Features of Dynamic Modulus $(\boldsymbol{E^*})$ NCHRP Project 1-37A is producing the new 2002 Design Guide for New & Rehabilitated Pavement. The guide is based on mechanistic principle and requires a modulus, analogous to E for steel, to complete stress and strain in hot-mix asphalt (HMA) pavement (NCHRP 1-37A2002). In 1999, the NCHRP Panel for Project 1-37A selected E^* for this purpose. The selection was based on a Paper authored by Matthew W. Witczak (Witczak 1999).which compared E^* to an Indirect Diametral Test (M_R) (Dougan et al. 2003). Both of these test procedures have been in use by the research community for over 40 years.

Briefly, E^* is the modulus of a visco-elastic material. The dynamic (complex) modulus of a visco-elastic test is a response developed under sinusoidal loading conditions. It is a true complex number as it contain both real and imaginary component of the modulus and is normally identified by E^* $(or\ G^*)$ (Dougan et al. 2003). In the visco-elastic theory, the absolute values of the complex modulus $|E^*|$, by definition is the dynamic modulus. In the general literature, however, the term, “Dynamic Modulus”, is often used to denote any type of modulus that has been determined under “non-static’ load condition.

The dynamic modulus protocol defines a linear visco-elastic test for asphaltic material that was originally developed by Coffman and Pagen at Ohio State University in the 1960’s (Dougan et al. 2003). The test can be applied in a uniaxial (triaxial) condition in either compression or tension. Most of the test results obtained over the past 40-45 years been in compression and are generally denoted as E^*. The test has also recently been used in shear for both mixtures and binder during the SHRP and SuperPave research project (Witczak et al. 2002). These results are generally denoted as G^* (or G^*b). The E^*

test was adopted as the "Modulus Test of Choice" by the Asphalt Institute in the late 1960's by Kallas, Shook and Witczak (Shook & Kallas 1969). It subsequently became an ASTM test in the early 1970's; its designation is ASTM D3496. Most recently the E^* protocol has been refined by Witczak and others at Arizona State University (Dougan et al. 2003).

Dynamic modulus - For linear viscoelastic material such as HMA mixes and asphalt binders, the stress-strain relationship under a continuous sinusoidal loading in frequency domain is defined by the complex modulus (E^*). This is a complex number that related stress to strain for linear visco-elastic material subjected to continuously applied sinusoidal loading in frequency domain. The complex is defined as the ratio of the amplitude of the sinusoidal stress at any given time, t, and the angular loading frequency, ω. The complex modulus can be obtained from a standard laboratory testing. The laboratory testing can be done using either a normal or shear stress mode. When the applied stress is normal, the complex modulus is denoted by E^*; whereas when a shear stress is applied, the complex modulus is denoted by G^*. The sinusoidal stress (δ) can be represented as:

$$\delta = \delta_0 e^{i\omega t} = \delta_0 \cos(\omega t) + i\delta_0 \sin(\omega t) \tag{1.1}$$

Where, δ is the sinusoidal stress manganite at time, t, δ_0 is the maximum stress amplitude, ω is the angular velocity radians/s, t is loading time, second. The angular velocity (ω) is related to the loading frequency (f) as:

$$\omega = 2\pi f \tag{1.2}$$

As the result of the sinusoidal stress (δ), the viscoelastic material experience a sinusoidal strain(ε), this generally lags the stress by a phase angle. In the case when the applied stress is normal, the phase angle is denoted by ϕ; whereas when a shear stress is applied, the phase angle is denoted by δ. For a normal applied sinusoidal stress, the lagging-sinusoidal strain is expressed as:

$$\varepsilon = \varepsilon_0 e^{i(\omega t - \phi)} = \varepsilon_0 \cos(\omega t - \phi) + i\varepsilon_0 \sin(\omega t - \phi) \tag{1.3}$$

Where, ε_0 is the maximum strain amplitude. The ratio of the sinusoidal stress to the sinusoidal stress defines the complex modulus, denoted by either E^* or G^* depending on the stress type (Dougan et al. 2003). In a steady state response (Figure 2), the dynamic (complex) modulus can be expressed as:

$$E^* = \delta/\varepsilon = \delta_0 e^{i\omega t}/\varepsilon_0 e^{i(\omega t-\phi)} = \delta_0 \sin(\omega t)/\varepsilon_0 \cos(\omega t - \phi) \quad (1.4)$$

Where, δ_0 is the peak (maximum) stress, ε_0 is peak (maximum) strain, ϕ is phase angle degrees, ω is angular velocity, t is time, seconds, i is imaginary component of the complex modulus. Mathematically, the dynamic modulus is defined as absolute value of the complex modulus as:

$$|E^*| = \delta_0/\varepsilon_0 \quad (1.5)$$

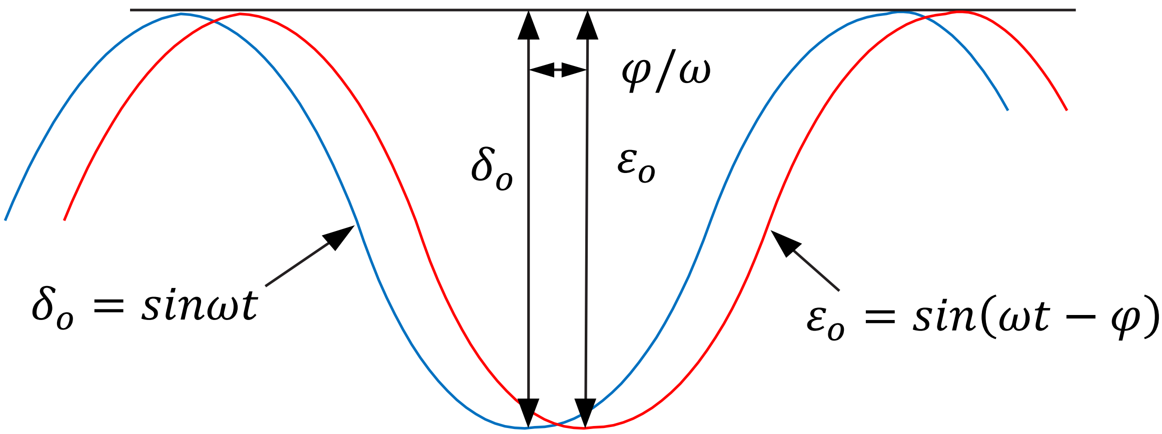


Figure 1.2 Dynamic (Complex) Modulus

The primary of output of the test is dynamic modulus $|E^*|$and the phage angle(ϕ), which is a direct indicator of elastic-viscous properties of asphalt mix or binder material. The dynamic modulus in compression $|E^*|$ of the mix, is similar in principle to the G^*, complex shear modulus binder, developed in SHRP and SuperPave programs at the University of California, Berkley, and Penn State University (Dougan et al. 2003). The two moduli, E^* and G^* are theoretically related through engineering mechanics by relationship:

$$E^* = 2(1+\mu)G^* \quad (1.6)$$

Where, μ is the Poisson's ratio.

Elastic-Plastic Problem and Shakedown Theory A structural formation of the elastic-plastic problem and/or shakedown theory has been adopted to account for this complex response in the form given by Martin et al. (1987). This framework is essentially identical to that used by (Cohn, Maier & Grierson 1979). The formation was presented in discrete terms, in a manner which can readily be identified with finite element approximations (Martin et al. 1987). Chazallon, Hornych and Mouhoubi (2006) developed a new elastoplastic model for the long-time behaviour of unbound granular material in flexible pavement taken into account in both isotopic and kinematic harding. In this model, modified Boyce model (Boyce 1980) was implemented in order to account for sand response. And then, Allou, Chazallon and Hornych (2007) implemented the constrictive mathematical material models that was presented by Habiballah and Chazallon (2005) into a finite elements modeling (FEM) simulation of a low volume traffic road.

Based on repeated load triaxial tests, a further general procedure has been developed by Chazallon et al. (2009) for the determination of the material parameters of constitutive model in order to integrate the previous studies (Allou, Chazallon & Hornych 2007) in a FEM modelling by taking into account the Boyce model (Boyce 1980) and shakedown for UGM layers. Another, 2D-FE simulation is conducted by Ling and Lin (2003) where the response of reinforced asphalt pavement concrete under plane strain model condition subjected to monotonic loading is investigated. The simulation is run through PLAXIS program and UGM elastoplastic behaviour is considered through Mohr-Coulomb Criterion (Ghadimi, Nega & Nikraz 2015) in order to obey the associated and non-associated flow rules.

The shakedown concept has been used to describe the behaviour of conventional engineering structure under repeated cyclic loading (Werkmeister, Dawson & Wellner 2001). The interesting question is whether a critical stress level exists between stable and unstable condition in pavement structure. The concept maintains four categories of material response are under repeated loading: purely elastic, elastic shakedown, plastic shakedown, and incremental collapse (Werkmeister, Dawson & Wellner 2001).

Stresses And Strains Problems Pavements are deceptively very complex structure system that involves the interaction of numerous variables. The pavement performance has been influenced by several factors such as traffic loading (stresses and strains as various changes with wheel and axle load traveling at different speeds), the environmental (temperature and moisture), material properties and construction practice. Pavement analysis and design procedures depend heavily on empirical relationships based on long-term experience and field tests such as the American Association of State Highway Official (AASHO) Road Test (AASHO1962; HRB1961).

The AASHO Road Test was performed in the late 1950s, is the basis for most pavement design procedures that use the American Association of State Highway and Transportation Officials (AASHTO) methods of equivalent factors (AASHTO1972; Helwany, Dyer & Leidy 1998). The relationships between traffic loading and pavement performance obtained from the AASHO Road Test are recognized to apply only to the condition under which they were developed. The relative damage to pavement caused by new vehicle characteristics and configurations may be different from that caused by the axle loads used at the AASHO Road Test (AASHTO1986a; Helwany, Dyer & Leidy 1998).

The mechanistic (primary) response parameters of pavement can be evaluated analytical using damage prediction models. These parameters include the vertical strain on the top the subgrades, the tensile strain at the bottom of the pavement, the surface vertical deflection, and the tensile stress in a concrete pavement (Helwany, Dyer & Leidy 1998). There are several multi-layer computer programs such as KENLAYER, ELSYM5, CHEVRON, EVERSTRS, WESLEM, ILLI-PAVE, DAMA, MnPAVE, BISAR, CIRCLY5, MICHPAVE, and ABAQUS that are available for solution of the stresses and strains or “boundary values” problems of a multi-layered pavement system (AASHTO2008; Elshaer 2009; Harichandran, Baladi & Yeh 1989; Harichandran, Baladi & Yeh 1990; Huang 1993; Kopperman, Tiller & Tseng 1986; Michelow 1963; NCHRP2002; Nega, Nikraz & Al-Qadi 2015b; Nega, Nikraz & Al-Qadi 2015a; Raad & Figueroa 1980; Van Cauwelaert et al. 1989).

In this literature, on top of laboratory techniques, three alternative models that overcome the difficulties associated with dynamic (complex) modulus of hot-mix asphalt mixtures, elastic-plastic (i.e., shakedown) and stress and strain modeling have been presented. Both of these models can be used to simulate the significant use of pavement material characterization, performance response to flexible pavement design and stresses and strains characteristic behavior in flexible pavement. Similarly, the laboratory experiment assesses the characterization of hot-mix asphalt mixtures of Western Australia asphalt mixes and it's significant of development and validation to full depth asphalt pavement. Alternatively, new constitutive models equations can develop for the determination of the material parameters and asphalt pavement performance that have been influenced by several factors such as growth of traffic loading and environmental conditions. To fully understand the pavement distress in the field and also manage the pavement system, another numerical algorithm can be used (multi-layer computer software: BISDEF, WESLEA, ELSYM5, BISAR and MET).

The suitability of these alternative models has not fully assessed and/or fully develop to validation to whether these models can provide insight to the characterization of asphalt mixtures, pavement material and structure of flexible pavement where most modeling and/or laboratory tests fails. Most of the assumption on which the theoretical models and/or laboratory tests are based, are incorrect for pavement material structure (Mahoney 2000; Mehta & Roque 2003; Sebaaly, Mamlouk & Davis 1986; Ullidtz, Zhang & Baltzer 1999). Most of the assumption on which the theoretical model and/ or laboratory tests are based, are incorrect for pavement material and structure. Most pavement materials are not solid but particulate, pavement deformations are not elastic but also viscous, visco-elastic and plastic, and they may be highly dependent upon the stress condition. The purpose of structural evaluation of pavement is normally to estimate the residual life or to calculate the overlay thickness needed to achieve certain residual life. To do this the critical stresses and/or strains are calculated and used in empirical relationship for pavement fatigue, rutting or roughness.

If the theoretical method can predict these stresses and strains, with reasonable degree of accuracy, the method may be quite useful for structural evaluation of pavement. But it is not simple to measure stresses and strains in pavement because of its complexity (i.e., various changes in traffic loadings varies with time and various changes in environmental conditions varies with time; and it makes critical to measure). Whether it is the laboratory experiment and/or models are not accurate regarding these issues. Note that the work presented in this thesis focuses extensively upon laboratory experiments to assess and analyses the important of (i.e. characterization of asphalt mixtures, pavement materials characterization and evaluation and validation of characterization methods for fatigue performance of asphalt mixes) and the use of two-dimensional (vertical) and three-dimensional (axi-symmetric) models (i.e., flexible pavement design and stress and strain behavior). In addition, this thesis focuses on developing new constitutive models equations for shakedown and stress and strain characteristics behavior and others numerical algorithm techniques to evaluate pavement distress on the field and pavement management tools to assess the performance of pavement structure.

Several researchers in Asphalt pavement field have highlighted the important of characterization of asphalt mixes, material characterization, pavement management system, stresses and strains behavior in flexible pavement and pavement analysis and design procedures over the past several decades (Al-Khateeb et al. 2006; AASHTO1986b; Goetz 1989; Haas & Hudson 1978; Harvey et al. 1995; Hicks & Monismith 1981; Hoiberg 1979; Huang 1993; Huang 2004; Hudson & Hudson 1994; Martin et al. 1987; Said 1988; Santucci 1977; Smith et al. 1984; Thompston 1989). Despite several multi-layer computer programs are currently available for solution of stress and strain including the highly advance laboratory techniques, the lack of understanding to estimate the pavement residual life is still remain and/or unchanged due to critical stress and strain characteristics behavior and incorrect pavement characterization of asphalt mixes. For example, growth of traffic loadings in heavily urban area (i.e. stresses and strains various change with wheel and axle load along with the traveling at different speeds) and the environmental condition (i.e., temperature and moisture change at different time rate) is still unpredictable and unsolved. This thesis presents a largely development of various numerical modeling strategies and advance laboratory techniques to simulate issues that has risen in details in this thesis to Western Australia and beyond. The models development and advanced laboratory techniques are used to investigate several specific research objectives.

1.2. RESEARCH QUESTIONS

This research project was broadly upon providing an answer to whether the characterization of hot-mix asphalt mixtures and Western Australia asphalt mixes can be understood using numerical and laboratory techniques. Also, the research addresses the question of assessing the significant of development and validation of characterization of asphalt mixes at the full depth asphalt pavements. In addition, the research addresses the significant use of pavement material characterization, pavement performance, pavement management systems, flexible pavement design and stresses and strains in flexible pavements. The research focuses upon providing insight into these broader questions by focusing upon the following detailed questions:

- Can hot-mix asphalt (HMA) mixtures in Western Australia (WA) be characterized on the engineering properties of asphalt mixes? Is it possible to produce these engineering properties of HMA mixtures in a laboratory test so that stress-strain dependent modulus characterization of WA asphalt mixes will be evaluated and modify in terms of pavement performance of one asphalt mix over another mixes?

- Can pavement materials characterization of HMA mixtures be able to use in order to construct and restore the heavily urban roads where damage has been induced due to traffic rapidly grown in WA? Can each asphalt mixes be analyses using tensile strength test, resilient test, wheel tracking test, binder contents test, Marshall compaction test and air void contest in laboratory test so that stiffness of asphalt pavement will be strengthen and stable? Can the laboratory test provide improved (i.e. effective and efficient) method of characterizing asphalt and asphalt paving material in service? To what extent will ensure the functionality of pavement material characterization in terms of pavement design life?

- Should Australia adopt the United States procedures for characterization of asphalt mixes, as a whole or in part? If it so, Will it be effective or efficient in evaluation and validation of characterization method for fatigue performance of asphalt mixes for WA? Is it possible to determine an appropriate pavement thickness in the laboratory test that is suitable and applicable to asphalt pavement concrete surface in the field? Can the effects of coarse aggregate gradation limits and aggregate type will be able to analyses and modify on the engineering properties of HMA mixtures?

- Can the development of long-term performance of HMA mixtures be projected using laboratory test and at the same time using field investigation for Western Australia? Will the development of long-term performance assessment of the roads improve the asphalt pavement design and analysis? If it is so, would the pavement network management tools be a solution to the problem so that the cost for pavement maintenance and rehabilitation of asphalt roads, asphalt mixtures variable, faulty design, unbalance aggregate combustions, human error and judgment and so forth can be reduces?

- Can adoption of mechanistic-empirical pavement design guide (MEPDG) will be used for new asphalt pavement design that is required a dynamic (complex) modulus of hot-mix asphalt? Should dynamic modulus that is measured in a laboratory need a verification of falling weight deflectometer (FWD) in the laboratory test? Will the effect of modifiers affect the performance of dynamic (complex) modulus of HMA mixtures? Since the dynamic (complex) modulus is an important input parameter in asphalt pavement design, Will the effect of loading frequency and temperature affect the features of dynamic (complex) modulus? Is it possible to develop a master curves and predict a dynamic (complex) modulus from the polymer-modified asphalt mixtures and also its shift factor? From the analysis, can it be project the relationship between binder viscosity and temperature? Can these polymer-modifiers also enhance the binding behavior of the asphalt mixes both at low and high temperature?

- Can the physical distress in asphalt pavement as a result of stress, strain, or displacement induced by traffic load and environment condition (such as temperature and moisture) be observed using distress identification and flexural stiffness criteria? Can different types of distress be identify and quantify of pavement surface distress in a given segment of asphalt concrete in the field survey? Is it possible to perform details distress ratings, predict pavement temperature and cost analysis of individual pavement distress of heavily urban roads in WA?

- Can the relative performance of numerical techniques used to solve the two-dimensional (vertical) variably shakedown behavior of flexible pavement of unbound granular layer be assessed? Can this assessment provide an unbiased comparison of the standard numerical techniques used to solve the linear governing equations for unbound granular layer? Can this assessment provide and/or develop a new constitutive model equations for the Shakedown behavior and method will be integrated with Mohr-Coulomb criterion, and Drucker and Plager method? Can this new simulation model and/or analyses provide an unbiased comparison of vertical strain in Shakedown model and vertical strain in Mohr-Coulomb model? Is it possible to verify the new developed simulation model with a laboratory experiment?

- Can a numerical algorithm for solving inertial force problem is consider in pavement structure analysis using the falling weight deflectometer (FWDs) test? Is it possible to evaluate structural condition based on engineering principles and develop the layer moduli of flexible pavement? Can analysis be extended to consider the dynamic analysis of backcalculation moduli so that the numerical algorithm can also be used for interpretation of the FWDs data and also develop the deflection basin of flexible pavement layer moduli? In addition, Is it possible for the numerical non-linear model to assess whether the information and/or FWD deflection data can improve the consistency and assurance of existing pavement structure?
- Is the use of three-dimensional (ax-symmetric) a numerical analysis technique using finite element (FE) model in ABAQUS be able to obtain a various structural parameters (stress, strain, displacement or deflection) of flexible pavement layers? Is it possible to develop a new simulation constitutive model equation for stresses and strains characteristics problem of flexible pavement layered moduli (asphalt concrete layer, base and subbase layer, compacted and natural subgrade layer)? Can the new develop constitutive model of new simulation be an accurate and effective solution for stresses and strains characteristics problems? Can the numerical analysis technique further extend to analyses the impact of the wheel load (i.e., represent a set of dual tires) and tire inflation pressure of dual tires over the discretize five-layer systems of flexible pavement during cyclic loading? Can the three-dimensional numerical analysis technique using FE model be able to use in order to investigate the effectiveness and accuracy of two methods (i.e., the comparison of experiment test and finite element model in ABAQUS) in reducing vertical surface deflection and critical tensile strain in the asphalt concrete layer?

1.3. THESIS STRUCTURE

This thesis is, in accordance with postgraduate and research scholarship regulation (4c) (ii) of Curtin University, presented as a series of scientific papers that resulted from the study. The ten main chapters of thesis consist of an introductory account of the research, followed by nine chapters, which contain expanded version of nine scientific papers. Therefore, these nine chapters can be read either as a part of the whole thesis, or as separate entities. Each of these chapters contains an independent introduction, literature reviews, methods, results and discussion section, conclusion and therefore some overlap, especially in the presentation of laboratory testing of different types of asphalt mixes used in Chapter 2, 3 and 4 is unavoidable since each chapter concerned with a similar or related research question. In addition, each chapter is independently referenced to acknowledge the contribution of previous related works. A general discussion and conclusion chapter closed the thesis.

In the current chapter, the aims, background, scope and purpose of the research are presented. This chapter provides the basic stimulus for the study and maps out the fundamental research question to be addressed in the thesis.

In the second chapter (Chapter 2, Engineering characterization of hot-mix asphalt in Western Australia), a general discussion and comparison of the fundamental different types of asphalt mixes that were produced in the laboratory according to the Australia Standard methods of Sampling and Testing asphalt used to modified pavement performance is presented. The analysis presents an unbiased comparison of the hot-mix asphalt (HMA) mixtures in terms of functionality of pavement performance to Western Australia. In addition, analyses of different types of asphalt mixes terms evaluate the relative advantage of one asphalt mix over another to determine the best mixes. Laboratory test for tensile strength, resilient modulus, wheel tracking, asphalt binder contents, and Marshall Compaction test were also taken to each asphalt mixtures to assess the entire asphalt mixes performance based on engineering principles and ethics.

Chapter 3 presents a pavement materials characterization of hot-mix asphalt mixes in Western Australia, a comparison of five different types of asphalt mixes that were produced in the laboratory test used to construct and restore the heavily urban roads, where damage has been induced due to traffic growth in Western Australia is presented. Analyses of the different types of the asphalt mixes in term of advantages of one asphalt mix over another mixes were compared by taking into consideration the characterization of pavement material properties. All the asphalt mixes that are used in this study can strength and stable the stiffness of pavement structure that is notable. The modification effort rank can be varied from one asphalt mix to another mixes.

Chapter 4 present an evaluation and validation of characterization methods for fatigue performance of asphalt mixes for Western Australia, a general discussion and comparison of different types of asphalt mixes that were produce in the laboratory test used to determine an appropriate pavement thickness and evaluate the material characterization methods for fatigue performance of asphalt mixes to Western Australia is presented. In this study, indirect tensile modulus, dynamic creep, wheel tracking and aggregate gradation laboratory test were taken and analyses to each asphalt mixtures for design traffic road. The discussion and analysis showed that asphalt mixes with thickness of 14 and 20 mm with 75 asphalt mix blow of each (or AC14-75 and AC20-75 asphalt mixes blow) are the most effective and efficient in pavement performance as compared to other asphalt mixes. Modification of the asphalt mixes depends on the development and validation of characterization methods that are used for pavement fatigue design life.

Chapter 5 presents a comparison of pavement network management tools and it's probabilistic of pavement engineering for Western Australia, a general discussion and comparison of the fundamental numerical techniques used to solve the linear model for pavement maintenance and rehabilitation (Pavement M&R) relevant to Western Australia roads networks. The analyses present an unbiased comparison of the relative advantages of the linear model base on the Probabilistic Network Markov-Chains Process and Chapman-Kolmogorov optimization pavement management tool with the past 30 years LRDM data inventory. The linear model techniques, and utilizes the comparison results to present a new and improved modelling strategy for simulating Pavement M&R shows promise as a research tool for future use.

The dynamic (complex) moduli relationship related mixture moduli to temperature and time rate of loading has been an integral part of several mechanistic-empirical (M-E) design procedures against a laboratory-scale model is presented in the following. Chapter 6 (Developing master curves, binder viscosity, and predicting dynamic modulus of polymer-modified asphalt mixtures) presents for the first time. The primary objective of this study was to evaluate the influence of polymer modifiers on a pavement performance of asphalt mixtures with the dynamic (complex) modulus $|E^*|$ of hot-mix asphalt (HMA) mixtures. To achieve this objective, seven asphalt concrete mixtures of different types of polymer modified binders (PMB) were produced in a laboratory to modify performance of asphalt mixture. The governing equation and analytical model are used as a part of the analysis of the laboratory data; and the influence of temperature, loading frequency, and confining pressure on the dynamic characteristic of asphalt mixture were analysis, master curves of dynamic (complex) modulus of HMA mixtures were developed and data's were interpreted.

Collecting and analysis of pavement distress data is a significant component for effective long-term pavement design life. Accurate, consistent, and repeatable pavement distress type's evaluation can reduce a tremendous amount of time and money that has been spending each year on maintenance and rehabilitation of existing pavement distress. Collecting and analysis of pavement distress is presented in the following chapter. Chapter 7 (Distress identification, cost analysis, and pavement temperature prediction for long-term pavement performance for Western Australia) presents for the first time. The objective of this study was to identify and quantify of surface distresses in a given segment of pavement, perform details distresses rating, predict pavement temperature and cost analysis on heavily urban roads in Western Australia. To achieve these objectives, thirty six roads survey were used to identify and characterized the types of pavement distresses. Governing equation and non-linear model: Probabilistic Network Markov-Chain Process method and the Statistical Downscale Model (SDSM) model were used to predict the cost analysis, asphalt surface pavement distress and pavement temperature for individual asphalt concrete surface pavement distresses The model used to describe a new procedure in order to assess the worthiness of test problems to be used as a benchmark for pavement management system (PMSs) for Mainroads Western Australia.

The eighth chapter focuses upon the limitation problem of full depth asphalt concrete pavements that are generally design to control fatigue cracking and reduce potential rutting when subjected to repeat heavily traffic roads. A new constitutive model equation that was developed for shakedown effect, pavement distress, impact of temperature and loading frequencies in pavement (analytical and the non-linear numerical model problem) is presented in the following chapter. Chapter 8 (Simulation of shakedown behavior for flexible pavement's unbound granular layer) present for the first time. The purpose of this study is to develop a simplified simulation model for the shakedown behavior of granular layer in flexible pavement. To achieve this purpose, a new constitutive model based on Mohr-Coulomb criterion and Drucker Plager method for flexible pavement of unbound granular material (UGM) was developed.

This method (Chapter 8) is integrated with Mohr-Coulomb Criterion, which is used and applied to simulate the response of unbound granular layer to dynamic loading in a numerical analysis. Analyses showed that the new developed constitutive model is capable of considering shakedown effect in base UGL and predicting the various flexible pavement failures (such as pavement distress problem in Chapter 7) and effect of various design parameters (such as impact of temperature and loading frequencies in pavement in Chapter 6). The implementation of this new developed model is verified against published results of laboratory test data measured shakedown for UGM. This model also compared with other's simplified model studies of pavement unbound granular layers based on the shakedown and Mohr-Coulomb criterion for verification.

Falling weight deflectometer (FWD) testing has extensively been done in the past to evaluate structural condition based on engineering and also determine the layer moduli of flexible pavement. FWD testing data of traffic roads for consistency and assurance of existing pavement structure is presented in the following chapter. Chapter 9 (Dynamic analysis of falling weigh deflectometer test results for strength of character of pavement layer moduli) presented for first time, a benchmark of both FWDs test and implementation of modulus for pavement lasers that obtain from backcalculation. The main objective of this study is to analyses the FWD test results for strength of character of flexible pavement layer moduli in Western Australia so that allowable loads for existing pavement structure can be determine. To achieve the main objective, FWD measurements were taken from 7 locations of traffic roads including core data, roadway data and pavement distress information. Governing equation and BISDEF and ELSYM5 computer software were used to analysis backcalculation moduli based on FWDs data. The general discussion and analysis has drawn, and it shows that the strength of character of flexible pavement layer moduli has actived and algorithms for interpretation of the deflection basin were improved; and FWD test of these traffic roads are compared for consistency and assurance of existing pavement structures of Western Australia. In addition, the variation of moduli of sections of all layers along the length sections for majority of the roads were accurate and consistence with measured and computed predicting.

The tenth chapter focuses on multilayer solutions that calculate stresses, strains, and displacement in flexible pavement structures that have been caused by traffic surface loading. Most elastic layer computer software applications have been developed for mainframe computers. A numerical analysis technique to obtain a various structural parameters such as stresses, strains and deflections or displacements of flexible pavement layers using three-dimensional finite element analysis in ABAQUS is presented in the following chapter. Chapter 10 (New numerical simulation for stresses and strains characteristics in flexible pavement using 3D finite element analysis) presented for the first time. The main objective of this study is to develop a new simulation for stresses and strains characteristics problem in flexible pavement so that an accurate and effective solution can be found; to increase asphalt modulus, base modulus, subbase modulus, and subgrade modulus so that the deflection can be reduced using interchanging and/ or varying the pavement design configuration. To achieve these main objectives, a new constitutive model equation was developed and five flexible pavement layers: asphalt concrete, asphalt treated base, unbound subbase, compacted subgrade, and natural subgrade layers were developed using three-dimension finite element method in ABAQUS.

The flexible pavement model discretization of a five-layer system having a 50 mm-thick AC layer, a 150 mm-thick unbound base layer, a 250 mm-thick unbound subbase layer, a 75 mm-thick compacted subgrade layer and an infinite natural subgrade is considered in this study in Chapter 10. Also, The 40 kN wheel load to represent a set of dual tires is assumed to be uniformly distributed over the contact area between tire and flexible pavement surface, and the tire inflation pressure of dual tires was 700 kPa according to Australian standard. Within each cycle, the loading is applied with duration of 0.008 s to simulate the vehicle speed 95 km/h and then, the load is removed for 1.0 s. The cyclic time is a total of 10 second with a loading of 0.1 s and resting 0.9 s. Falling weight deflection (FWD) experimental data from seven main roads and creep test from which the model verification was derived with permission to make sure the implemented to assure integrity of the in situ experimental data used in the verification of the finite element. Finally, investigated the effectiveness and accuracy of two methods in reducing vertical surface deflection w_0 and the critical tensile strains in the asphalt layer ε_t the matter of the fact, it is indeed desirable to reduce the vertical surface deflection as much as possible.

The final chapter presents a closing discussion and concluding mark. This section provides an overall discussion of the issue raised in the thesis and points to new questions open for further research a as results of the thesis.

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CHAPTER 2: ENGINEERING CHARACTERIZATION OF HOT-MIX ASPHALT IN WESTERN AUSTRALIA

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ABSTRACT

The use of full depth asphalt pavement construct and rehabilitate heavily loaded urban roads has rapidly grown in Western Australia (WA) over the past 3 years. Five different types of asphalt mixes were produced in the laboratory according to the Australian Standard methods of sampling and testing asphalt to modify pavement performance mixture. The main role of this research is to evaluate and assess the hot-mix asphalt pavement performance characteristic for Western Australia roads. In this study, laboratory test for tensile strength, resilient modulus, wheel tracking, asphalt binder content and Marshall Compaction test were taken and analyzed to each asphalt mixtures. Results showed that AC20-75 and AC14-75 Blow asphalt mixes were more efficient and effective in pavement performance as compared to the other mixes. In general, all the asphalt mixes that are used in this study can be used in order to strength and stable the mixture stiffness of asphalt that is notable. The modification effect rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research.

Author keywords: Characterization; asphalt mixture; hot-mix asphalt; tensile strength; resilient modulus; wheel tracking; asphalt binder content; Marshall compaction; Western Australia

2.1. INTRODUCTION

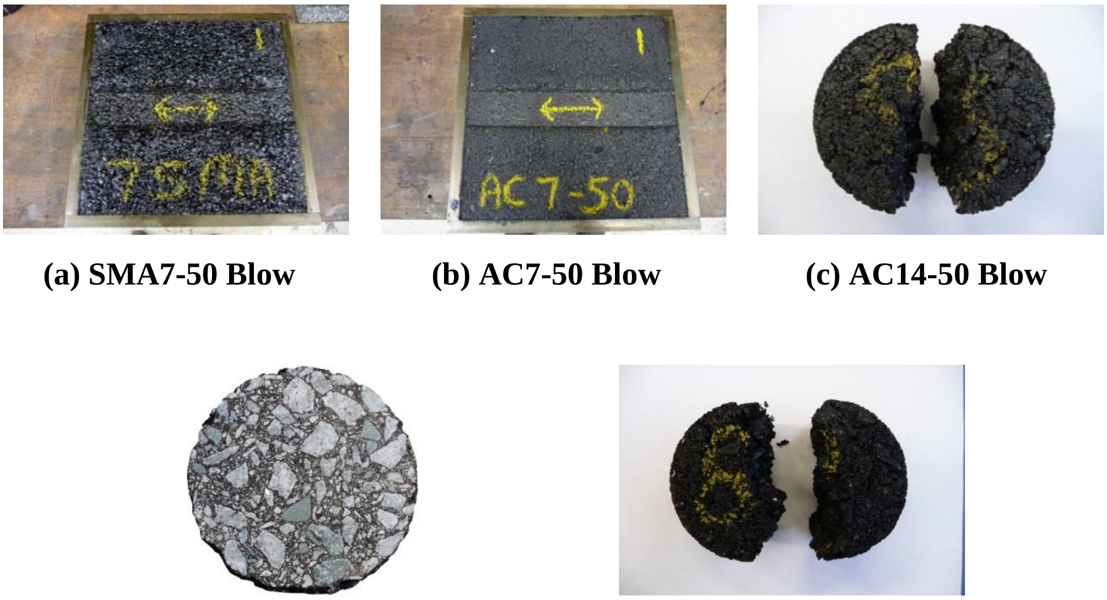
The asphalt concrete or hot mix asphalt (HMA) is the one the most widely used infrastructure materials for road construction. Hot mix asphalts can be described as a multiphase heterogeneous material composed of a viscoelastic asphalt binder, irregular rigid aggregate particle in high volume fraction, and small percentage of air voids (Gopalakrishnan & Kim 2011). These component materials exhibiting various properties contribute to complex mechanical behavior of HMA, which can be characterized as viscoelasticity, and plastic under different condition such as temperature, load application and aging (Dibike et al. 2001; Gopalakrishnan & Kim 2011). Therefore, the mechanical behavior of hot mix asphalt should be understood by not only the individual properties of HMA components, but also by considering asphalt binder and aggregate acting together.

Hot-mix asphalt is known by many different names: HMA, asphaltic concrete, plant mix, bituminous mix, bituminous concrete, and many others (Gillespie 1992; Gillespie et al. 1992). It is a combination of two primary ingredients – aggregates and asphalt binder. The aggregates total approximately 95% of the total mixture by weight. They are mixed with approximately 5% asphalt binder to produce HMA (Gillespie 1992; Gillespie et al. 1992). Hot and cold asphalt mixes are comprised of two major materials: aggregates (i.e. mixture of sand, gravel, crushed stone, slag and mineral filler) and asphalt cement (crude oil, hydrated lime and dust) as discussed on literature by (ACIC2007; CCAA2009; Mrawira & Luca 2006; Yang et al. 1995; Yang, Lin & Huang 1996). Bitumen had been defined by various sources as crude oil with a dynamic viscosity at reservoir conditions more than 10,000 centipoise (AASHTO1986b; ACIC2007; Hoiberg 1964; Hoiberg 1979; Uzan 2003; Yildirim 2007).

The last 50 years have seen remarkable increase in the demand for moving people and goods. The surface transportation system has been under pressure to sustain the substantial increase in heavy truck traffic volume levels. Considerable annual expenditures worldwide are allocated for developing the transportation infrastructure including urban roads and highways. For example, In Canada, $ 8 billion dollars is allocated annually for the rehabilitation of urban roads (Ali 2005; Statistic Canada Agency 2003), and In the United States, $ 51 billion dollars is allocated to the states for road construction (McNichol 2005) and In Australia, $18 billion dollars is annually used for urban roads maintenance and rehabilitation (Australian Transport Council 2008; Australian Transport Safety Bureau 2000), with overlays being one of the most commonly used techniques. Durability of the pavement structure determines the initial cost of the road and its future maintenance requirements.

The use of full depth asphalt pavements to construct and rehabilitate heavily loaded urban roads has rapidly grown in Western Australia (WA) over the past 3 years. There is limited data available from testing carried out by the Mainroads about the characteristics and variability of WA asphalt mixes. Although some data is available from testing carried out by others on Mainroads contracts, but it also would be necessary to determine whether Mainroads WA owns this data and has the right to publish the data so that it can be used to predict the likely performance of WA full depth asphalt pavements.

The goal of this study is to evaluate and assess the engineering chacteristics of hot-mix asphalt using a laboratory tests so that data can be used to predict the likely performance of Western Australia full depth asphalt pavement. Figure 2.1 shows the different types of asphalt mixes in this study.



(a) SMA7-50 Blow **(b) AC7-50 Blow** **(c) AC14-50 Blow**

(d) AC14-75 Blow **(e) AC20-75 Blow**

Figure 2.1 Types of Asphalt Mixes during the Laboratory Tests

2.2. MATERIALS AND METHODS

2.2.1. MATERIALS

Types of hot mixed asphalt used on the Mainroads WA network are dense graded asphalt (DGA), open graded asphalt (OGA) and stone mastic asphalt (SMA). DGA, the most common type of asphalt, provides optimal structure strength and generally good resistance to deformation. OGA is designed to drain water through the asphalt to remove excess water from the tyre/road surface. SMA is similar to OGA but has a high proportion of dust and high binder contents to achieve an improved fatigue life. SMA has a texture surface but does not drain water through its layer as does OGA (ACIC2007; Brown, Kandhal & Zhang 2004; MRWA2007). All Materials selected for this project were from local sources and are indigenous of Western Australian pavement materials used in the industry.

In order to assess the engineering characterization of hot-mix asphalt mixes performance, it was necessary to obtain laboratory data on tensile strength, resilient modulus, wheel tracking, asphalt binders and compaction tests. During an individual asphalt mixes run, specimen was taken and assessed in different categories of asphalt mixes. Laboratory asphalt mix specimens were subjected to the following laboratory characterization tests:

- Bulk specific density
- Voids content determination,
- Tensile strength ration (TSR) test
- Resilient modulus test
- Wheel tracking test
- Asphalt binder content test
- Marshall Stability test.

2.2.2. METHODS

The design method specified by Mainroads Western Australia is the Marshall method of mix design. The aim of the method is to satisfy specified design criteria. The descriptions of asphalt mixes design are as following:

- SMA7-50 blow: thickness of 7 mm granite stone mastic asphalt (SMA)
- AC7-50 blow: thickness of 7 mm open graded granite
- AC14-50 blow: thickness of 14 mm dense graded granite (intersection mix)
- AC14-75 blow: thickness of 14 mm dense graded granite (intermediate mix)
- AC20-75 blow: thickness of 20 mm dense graded granite (intermediate mix)

The bulk specific gravity test was performed after samples had cooled to room temperature according to the Australian Standard Testing Method AS 2891. The maximum specific gravity test was performed using the method of sampling and testing asphalt in Australia Standard Test Method on the sampled material from local sources and indigenous of Western Australia roads. Air voids were calculated using bulk specific gravity and maximum theoretical specific gravity data. Three each specimens for tensile strength (i.e. dry and moisture condition), resilient modulus, wheel tracking, binder content and Marshall Stability for each asphalt mix were tested as per AS 2891 and AG: PT/231and 232 (Austroads 1992; 2006; 2008). Specimens were placed in a water bath at 60^{o}C for a period of 30 min and were tested for Marshall Stability and flow. The details methods of sampling and testing of hot-mix asphalt in Australian Standard Testing Method are shown in Table 2.1 and 2.2.

Table 2.1 Methods of Sampling and Testing Asphalt in Australian Standard Test Method (Austroads 1992; 2006; 2008)

TSR Property	**Resilient Property**	**Wheel Tracking Property**	**Binder Property**	**Marshall Compaction Property**
Air Void Content 7-9%	Test Temperature 25±5 ^{o} C	Test Temperature 60±1 ^{o} C	DGA Temperature 180 ^{o} C	DGA Temperature 150±5 ^{o} C
Mix Size 20 mm	Rise Time (10% to 90%) 0.04±0.01 s	Sample Thickness 10 & 14 mm: 50±5 mm	OGA Temperature 160 ^{o} C	OGA Temperature 130±5 ^{o} C
Diameter 100 ±2 mm	Pulse Repetition Period (10% to 10%) 3.0±0.01s	20 mm Normal Size: 75±5	SMA Temperature 170 ^{o} C	Diameter 150 mm
Saturation 55-80%	Recovered Strain 50±20 με	Air Void Content 5±0.5 %	Balance 0±0.5	Over All Depth 200 mm
Testing Temperature 25±1 ^{o} C	Sample Thickness 35-90 mm	Vertical load 700±20 N,	Control Oven Temperature 105-110 ^{o} C	Test Temperature 25±1 ^{o} C
Water Bath 120±5 min	Sample Diameter 150 mm	Tracking Depth 15 mm, Number Cycles 10000	Sieve Opening 0.600 mm	Marshall Load 4.5± 0.045 kN,22±0.11kN
Freezing Saturated Samples for 18 hours	Peak Load 0.4-3.9 kN, Poisson Ratio 0.4, 10 Cycle for Each Specimen 1000 cycle	Test Specimen 300 mm x 300 mm in view, diameter 200 mm	Retained Binder 0.3%, Binder Mass 5 kg, Cool in air 23±3 ^{o} C	Time Rate 60±10 rpm

Table 2.2 Methods of Sampling and Testing Asphalt in Australian Standard Test Method (Austroads 1992; 2006; 2008)

Type of Mix	Material Test	Test Method
SMA7-50 Blow	Tensile Strength Ratio	AGPT/T232
AC7-50 Blow	Resilient Modulus	AS 2891.13.1
AC14-50 Blow	Wheel Tracking	AGPT/T231
AC14-75 Blow	Asphalt Binder Content	AS 2891.1.1
AC20-75 Blow	Marshall Compaction	AS 2891.9.3

2.2.2.1. PURPOSE OF ASPHALT MIXES IN LABORATORY

To achieve an optimal mix design work needs to be undertaken in a materials laboratory to determine the best proportions of the available aggregates and bitumen to give a product (asphalt) that is durable, workable, has resistance to deformation and premature fatigue and provides an adequate surface texture for its intended purpose. There has to be sufficient bitumen in the asphalt to completely coat the aggregate particles and bond them together. If the design of an asphalt mix is not optimized it is likely to have performance issues such as early fatigue, rutting or shoving, raveling of the aggregate particles, have a low surface texture creating a skid resistance problem or stripping of bitumen from the aggregate particles in service, any of which may require premature replacement of the asphalt.

2.3. ENGINEERING CHARATERIZATION OF ASPHALT MIXES

2.3.1. TENSILE STRENGTH RATIO

The conflict between bitumen, water and aggregate affinities has been an issue since the inception of asphalt as a paving material. In many situations the issue is minor, but when it does manifest as a stripping failure, the results can be catastrophic (Austroads 2008). Moisture sensitivity relate to the potential for loss adhesion between the binders and aggregate in the presence of asphalt moistures. This kind of adhesion is commonly referred as stripping potential of asphalt- tensile strength ratio (TSR) (Austroads 2006; 2008). Stripping in asphalt is a complex mechanism. Where stripping occurs, it is often a combination of more than one: such as climate and traffic, asphalt mix permeability, class of binder, poor coats of aggregate due to presence of clay or dust contamination, affinity of bitumen and asphalt mix design including type of tiller and use of other additives.

A variety of test have been used that attempt to identify the moisture sensitivity and binder stripping potential of asphalt mixes. Tensile strength ratio of asphalt mixes is an indicator of their resistance to moisture suitability. The test was carried out according to AASHTO T283 (commonly known as the Modified Lottman Test) specification by loading a Marshall specimen with compressive loading acting parallel to and along the vertical diametric-loading plane (AASHTO1986b). In Australia, the development and implication of practical fundamental and simulative tests for characterization potential of asphalt mixes- tensile strength ratio test has implemented on AG:PT/T232, adapted from ASTM D 4867-92 (Austroads 2006; 2008).

Tensile strength ratio (TSR) of asphalt mixes is an indicator of their resistance to moisture suitability. The test was carried out according to AASHTO T283 specifications by loading a Marshall specimen with compressive load acting parallel to and along the vertical diametric-loading plane. The test was conducted at 25°C temperature and the load at which the specimen fails is taken as the dry tensile strength of the asphalt mix. The specimens were then placed in a water bath maintained at 60°C for 24 hours and then immediately placed in an environmental chamber maintained at 25°C for two hours. These conditioned specimens were then tested for their tensile strength. The ratio of the tensile strength of the water-conditioned specimens to that of dry specimens is the tensile strength ratio.

2.3.2. RESILIENT MODULUS

The resilient modulus is defined as a ratio of the deviator stress to the recoverable strain. It is known that the bituminous material is not elastic, but it experiences some permanent deformation after each load application (Jahromi & Khodaii 2009). However, if the load is small compared to the strength of material and is reported for number of times, the deformation under each load repetition is nearly completely recoverable and proportional to the load can be considered being elastic (Huang 1993).
Resilient modulus of pavement material is an important material property in any mechanistically based design/analysis procedure for flexible pavements (FHWA2002). The resilient modulus (M_R) is the material property required for the 1993 American

Association of State Highway and Transportation Officials (AASHTO) Design Guide, which is an empirically based design procedure, and is the primary material input parameter for the 2002 Design Guide (AASHTO1993).

Several types of moduli have been used to represent the stiffness of asphalt concrete mixture. Three of these are dynamic, resilient, and complex. The Modulus of Resilient is most commonly used for asphalt concrete mixture evaluation (Khan et al. 1998). Resilient modulus is a measure of material's deflection behavior where a pavement life and surface deflection are strongly related (Elliott & Thornton 1988). It is also a fundamental and rational material property that needs to be includes in pavement design.

2.3.3. WHEEL TRACKING

Australia initially adopted the dynamic creep test as the preferred method of determining the rut resistance of asphalt mixtures. The test was simple to operate and test specimens were simple to compact in the laboratory or obtain by coring from existing pavements (Alderson 1998; Austroads 2008). Wheel tracking was selected as the most suitable test method for measuring the rut resistance of asphalt mixtures. Olive and Alderson (1995a) indicated that the test had been shown to correlate well with rutting of roads in-service. The present standard in use in Australia is an Austroads test method (AG: PT/T231) that permit to determine the characteristic of wheel tracking test for both laboratory and field asphalt mixes (Austroads 2008).

The wheel tracking test consists of a loaded wheel assembly and a confined mould in which a 300×300×200 mm specimen of asphalt mix is rigidly restrained on its four sides. A motor and a reciprocating device provide the forward and backward motion to the wheel at the rate of 24 passes/minute along the length of the slab. The temperature during the test is maintained by a water bath over and around the mould.

2.3.4. ASPHALT BINDERS CONTENT

The addition of polymers, chains of repeated small molecules to asphalt has been shown to improve performance. Pavement with polymer modification exhibits greater resistance to rutting and thermal cracking, and decreased fatigue damage, stripping and temperature susceptibility. Polymer modified asphalt binders have been used with success at location of high stress such as intersection of busy streets, airports, vehicle weigh station and race tracks (King et al. 1999; Ma et al. 2010; Yildirim 2007). Similarly, Hesp and Woodhams (1992) discussed that the polymer-in-asphalt increased rutting resistance, low temperature cracking resistance, elevated tensile strength and others.

Zhao, Lei and Yao (2009) evaluated the characteristic of polymer-modifies asphalt binders and mixes using the conventional and Superpave tests. The polymers investigated included styrene–butadiene–styrene, styrene–butadiene–rubber, and complex polymers or C-polymer. It is shown that the characteristics of polymer-modified asphalt binders and mixes may vary with the polymer type, polymer dosage, asphalt source, aggregate types , and mix type as discussed by (Zhao, Lei & Yao 2009). The use of a polymer can enhance the dynamic stability of asphalt mix and the use of rut depth can better measure the rut resistance. The presence of a polymer can also enhance the binding behavior of the asphalt mix at low temperature. It is concluded that AC-13 mix may generate better pavement performance than AC-16 mix. The C-polymer has shown promising result in improving the characteristics of hot-mix asphalt.

A research that has been conducted on polymer modified binders over the three decades (Ma et al. 2010; Yildirim 2007) discussed that polymer modification of asphalt has been increasingly become the norm in designing optimally performing pavements, particularly in the United State, Canada, Europe and Australia. Specific polymers that have used include SBS (styrene-butadiene-styrene), SBR (styrene-butadiene-rubber), EVA (ethylene vinyl acetate), elvaloy, rubber, polyethylene and other (Yildirim 2007). Specifications have been designed and pre-existing ones modified to capture the rheological properties of polymer modified binders. Desirable characteristics of polymer modified binders include greater elastic recovery, a higher softening point, greater viscosity, and greater cohesive strength. King et al. (1999), Ma et al. (2010) and Yildirim (2007) recommended that the elastic recovery is good at determining the present of polymers in an asphalt binder but is less successful at predicting field performance of the pavement.

Blankenship et al. (1998) conducted field and laboratory tests in Kentucky in US and found that PG 70-22 made using different methods of modification gave different results for laboratory test. They used the Dynamic Shear Rheometer (DSR) and Bending

Beam Rheometer (BBR) tests to identify five different PG 70-22 binders, two SBS modified, one SBR modified, one chemical modified, and one neat and compared their behavior in various tests. These binders were found to differ as far as rutting, moisture damage and modulus testing was concerned. However, the rutting difference was no more than 10 mm in between binders.

2.3.5. MARSHALL COMPACTION OF HOT-MIX ASPHALT PAVEMENT

The compaction process has a great effect on the strength and durability of hot-mix asphalt pavements. The main objective of pavement compaction is to achieve an optimum density. This helps to ensure that pavement will have the necessary bearing capacity to support the expected traffic loads and durability to withstand weathering (Chadbourn et al. 1996).

Recent laboratory studies have shown that the compaction can highly affect the performance of the hot-mix asphalt (HMA) and stone mastic asphalt (SMA) mixtures (Khan et al. 1998; Linden & Van Der Heiden 1987; Linden & Van Der Heiden 1989; Linden, Mahoney & Jackson 1992; McLeod 1967; Pourtahmasb & Karim 2010). The goal of compaction is to achieve the optimum air void content and compressing the coated stones together by increasing the density of the mix to the considered level of compaction with a minimum change in the gradation and structure (Airey, Hunter & Collop 2008; Hunter, Airey & Collop 2004; Pourtahmasb & Karim 2010).

Inappropriate compaction may draw the binder to the surface of HMA and SMA causing flushing of the surface and loss of texture or aggregate segregation (Pourtahmasb & Karim 2010; Thyagarajan et al. 2009). California kneading compactor, Gyratory compactor and Marshall Hammer are being used as SMA compactors due to mix design method (Khan et al. 1998; Linden & Van Der Heiden 1987). However according to (Thyagarajan et al. 2009), the performance of the HMA and SMA compacted specimens could not simulated the field compaction upto 100%.

Research has shown a 1000-fold increase in asphalt binder viscosity as temperature dropped from 135^{o}c to 57^{o}c and a ten-fold increase in resistance to compaction as temperature dropped from 135^{o}c to 63^{o}c (McLeod 1967). Attention to compaction is especially crucial in cold weather condition when air voids after compaction can be as high as 10 percent. Pavements with this level air void have shown signs of deterioration after two years (McLeod 1967).

2.4. RESULTS AND ANALYSIS

A summary of average tensile strength ratio of dry and moisture condition is shown in Table 2.3. From the data presented, it can be seen that the AC7-50 blow asphalt mix has generally had high TSR of 112.9% as compared to other asphalt mixes. AC20-75 blow was the second best to have nearly reached a TSR of AC7-50 blow. This showed that the asphalt mixes are non-moisture susceptible. SMA7-50, AC14-50 and AC14-75 blow asphalt mixes had also relatively low as compared to AC7-50 and AC20-75 blow asphalt mixes but both of them are not susceptible to moisture. According to AASHTO T283,"Resistance of Compacted Bituminous Mixture to Moisture Induced Damage", the design asphalt mixture is judged to be non-moisture susceptible if it has a TSR greater than 80 percent (Airey et al. 2008; AASHTO2000; Apeagyei, Buttlar & Dempsey 2006).

Table 2.3 Tensile Strength (Dry and Moisture) for Different Types of Asphalt Mixes

Moisture Sensitivity						
Rank	**Asphalt Mixes**	**Tensile Strength (kPa)**		**TSR (%)**	**Air Void Ratio (%)**	
		Dry	**Moisture**		**Dry**	**Moisture**
3	SMA7 - 50 Blow	686.9	626.6	91.2	8.0	7.9
1	AC7 - 50 Blow	831.5	938.7	112.9	8.2	8.3
4	AC14 - 50 Blow	990.4	894.8	90.3	8.4	8.2
5	AC14 - 75 Blow	1225.5	995.7	81.2	8.3	8.5
2	AC20 - 75 Blow	995.4	1024.8	103.0	7.8	7.4

An average resilient modulus for different types of asphalt mixes is given in Table 2.4 and Figure 2.2. As it can be seen from the results, AC20-75 blow asphalt mix had high resilient modulus of 6824 MPa. However, SMA7-50 and AC7-50 blow asphalt mix had poor resilient modulus of 3713 and 4618 MPa, respectively. It is clear that asphalt mix that has low resilient moduli do tend to contribute to low plasticity, low group index, high silt content, low clay content, low specific gravity, and high organic contents. Hicks and Monismith (1981) stated that the resilient modulus of partially crushed aggregate decreased with an increase in fine contents, while the modulus increased for crushed aggregate with increasing in fine content. The total recovered strain (Figure 2.3), rise time and temperature (Table 2.4) of the resilient modulus test for each asphalt mixes are similar. This showed that the resilient modulus is based up within the determination of Australian Standard.

Table 2.4 Resilient Modulus for Different Types of Asphalt Mixes

Resilient Modulus Results							
Mix Type	**Specimen No.**	**Force (N)**	**Total Recovered (µƐ)**	**Rise Time (ms)**	**Load Time (ms)**	**Resilient Modulus (MPa)**	**Temp (°C)**
SMA7 – 50 Blow	1	717.8	48.5	40.0	138.8	1525.0	24.8
	2	1846.8	52.7	36.0	121.2	3600.0	24.6
	3	1942.1	52.1	36.8	119.8	3825.0	24.5
AC7 – 50 Blow	1	2627.1	54.2	38.0	121.4	5019.0	25.1
	2	2120.3	51.9	36.4	118.0	4223.0	25.3
	3	2338.0	52.3	36.6	117.6	4615.0	25.1
AC14 – 50 Blow	1	2694.6	52.1	39.8	122.8	5337.0	25.1
	2	2577.8	54.0	40.2	123.6	4921.0	24.8
	3	1315.1	53.0	35.2	127.6	2584.0	24.9
AC14 – 75 Blow	1	2902.7	52.1	39.0	122.0	5630.0	24.8
	2	2831.8	48.3	37.2	119.8	5959.0	25.2
	3	2865.5	52.1	36.8	119.8	5576.0	24.8
AC20 – 75 Blow	1	3244.2	48.8	45.6	133.6	6991.0	24.9
	2	3301.9	51.2	39.0	119.8	6794.0	24.6
	3	3361.7	52.0	42.2	123.6	6688.0	24.5

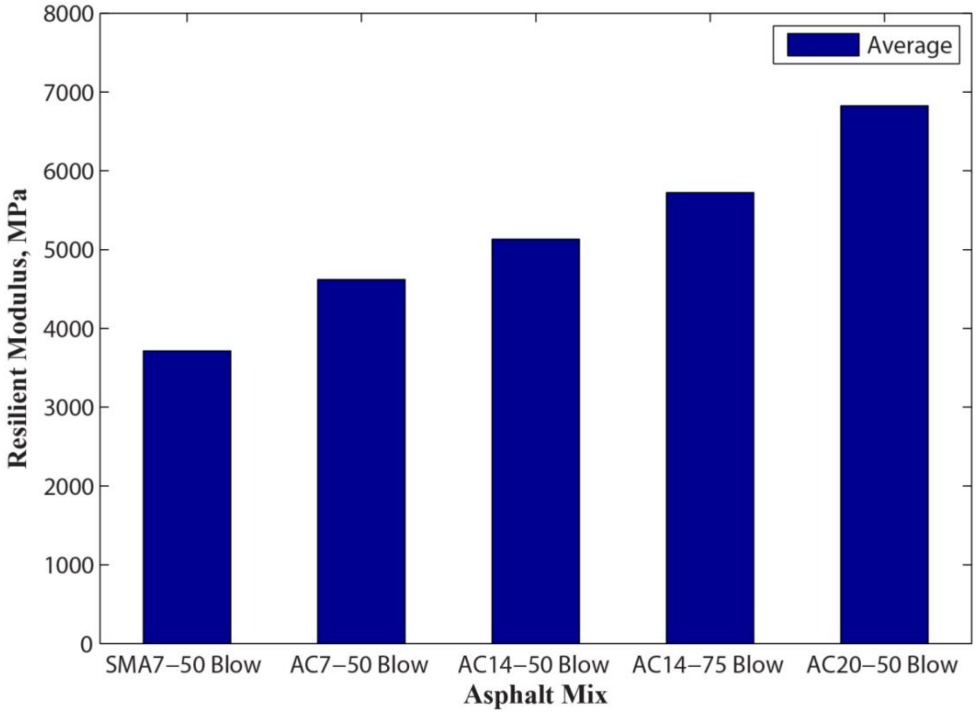


Figure 2.2 Average Resilient Modulus for Different Types of Asphalt Mixes

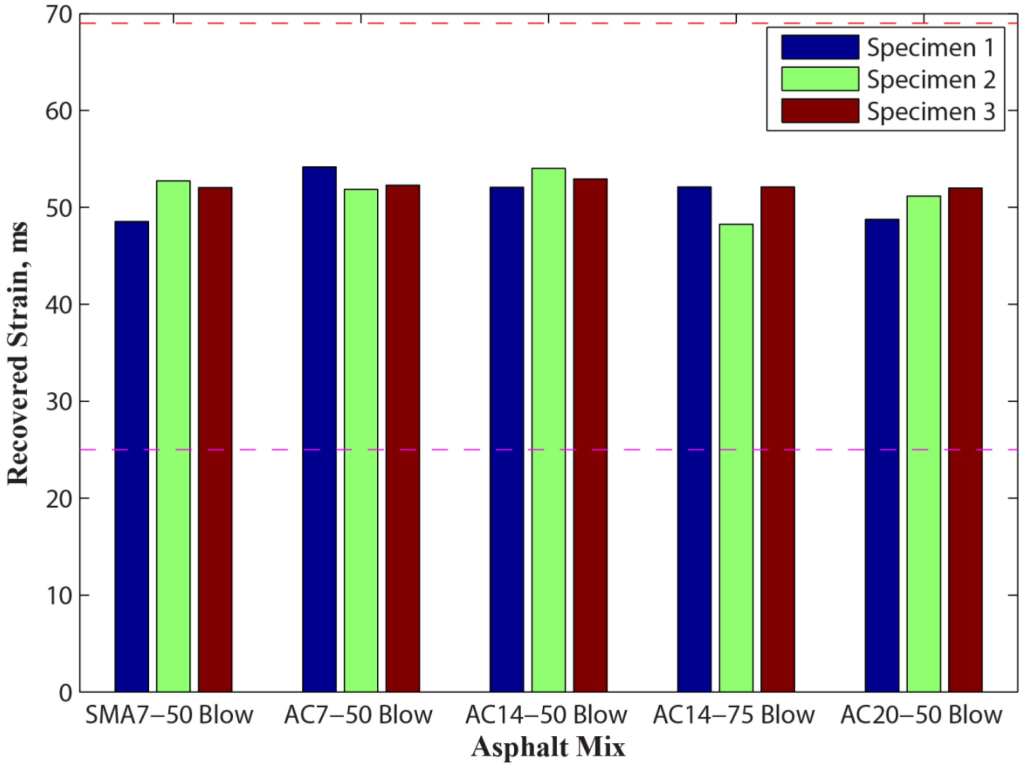


Figure 2.3 Resilient Modulus (Recovered Strain) for Different Types of Asphalt Mixes

The average wheel tracking test for different types of asphalt mixes is shown in Figure 2.4. The analyses indicated that AC20-75 blow asphalt mix had low rut depth of 1.9 mm as compared to other asphalt mixes. AC14-75 blow was the second in rank with a rut depth of 2.4 mm. This showed that these asphalt mixes are high rut resistance of asphalt mixture and less to pavement distress and asphalt fatigue cracking. However, the rut depth observed for SMA7-50 blow was 15 mm after 8, 452 cycles while 5 mm and 4.2 mm for AC7-50 blow and AC14-50 blow after 10,000 cycles, respectively. This indicated that SMA7-50 blow asphalt mix has high pavement distress and low rut resistance of asphalt mixture. There was also a sudden steep change in slope after 8,000 cycles for SMA7-50 blow. This may be attributed to the stripping of aggregate. No stripping was however, observed after 10, 000 cycles to other asphalt mixes.

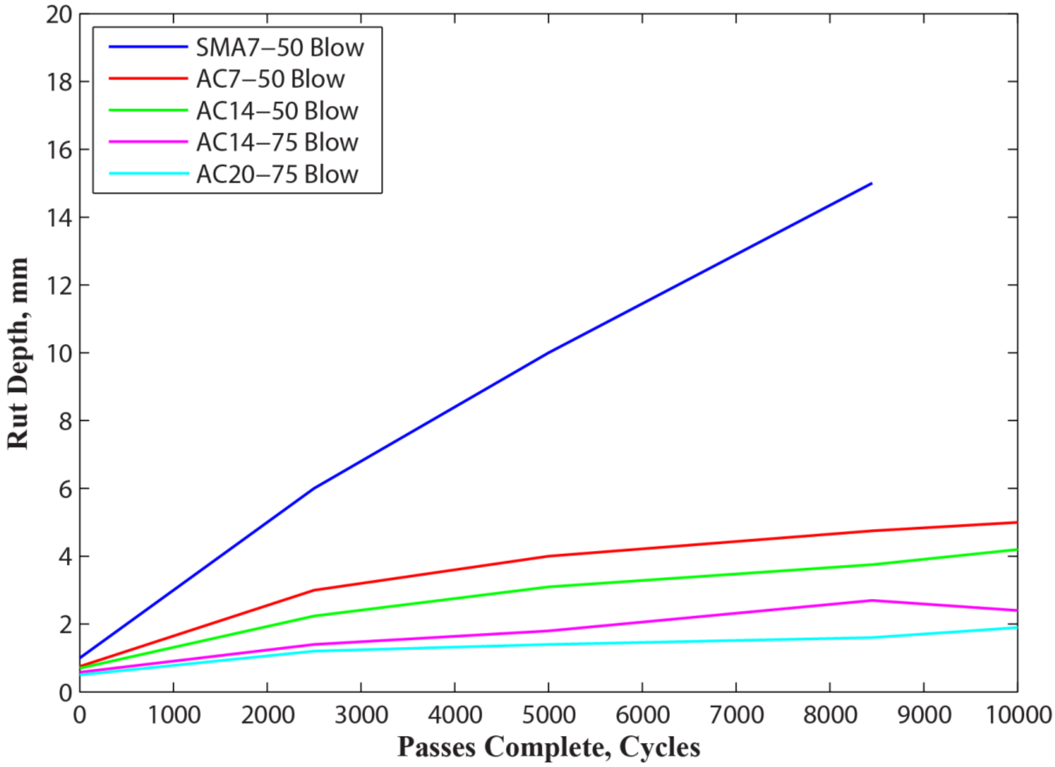


Figure 2.4 Average Wheel Tracking Tests for Different Types of Asphalt Mixes

A summary of asphalt binder content for different types of asphalt mixes is shown in Figure 2.5. The results indicated that AC14-75 and A20-75 blow property of asphalt mix had low percentage of binder content of 4.7% and 4.3% in the given order. This showed that AC14-75 and AC20-75 blow asphalt mixes might have increased the frictional contact between aggregate particle and the overall stiffness and stability of the asphalt mixes as compared to other asphalt mixes. However, SMA7-50, AC7.50 and AC14-50 blow asphalt mixes had high percentage of binder content. This showed that increase in binder content reduce the frictional contract between aggregate particles. Beyond a certain value, further increases in binder content reduce the frictional contact between aggregate particles and the overall stiffness and stability of the asphalt (Austroads 2006). At low percentages, added binder content increases the mix cohesion and strength (Anderson, Walker & Turner 1999)..

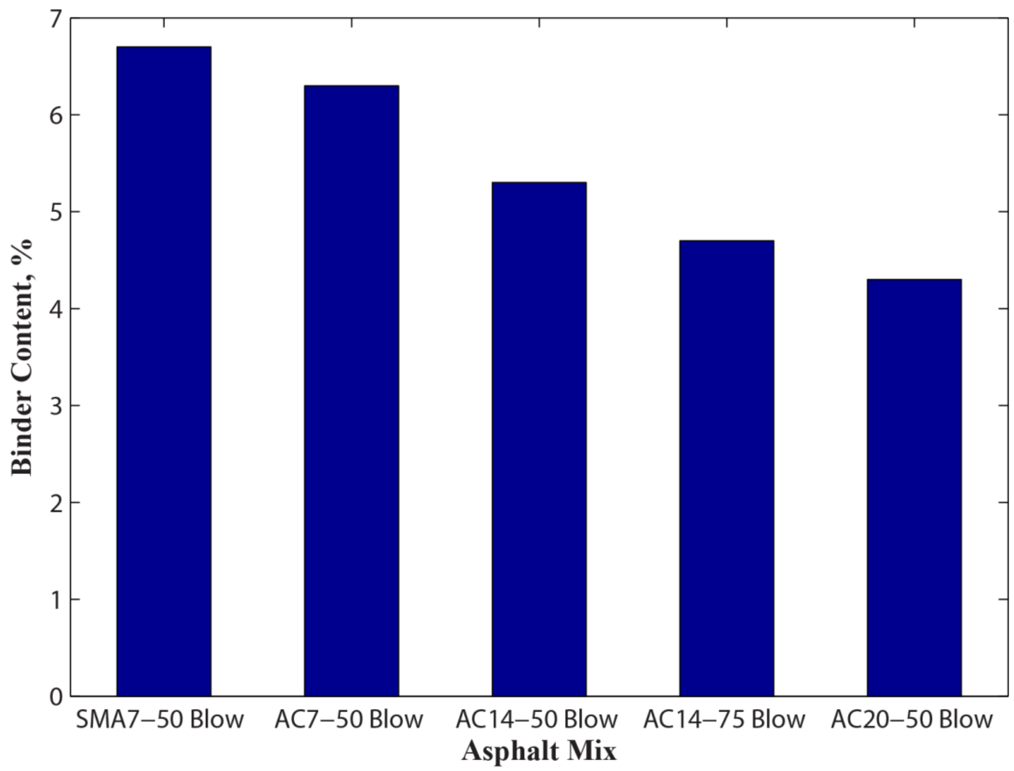


Figure 2.5 Asphalt Binder Content for Different Types of Asphalt Mixes

Average Marshall Compaction stability and Marshall Compaction flow for different types of asphalt mixes are shown in Figure 2.6 and 2.7. As it can be seen from the analyses, AC20-70 blow asphalt mix had high stability of 16 kN (Figure 2.6) as compared to other asphalt mixes. AC7-50, AC14-50 and AC14-75 blow asphalt mix had a similar strength and stability of 15.1 kN. This indicated that the asphalt mixes pavement will have the necessary bearing capacity to support the expected traffic loads and durability to withstand weathering. Marshall Compaction flow rate for AC20-75 blow was less as compared to other asphalt mixes (Figure 2.7). However, SMA7-50 blow had poor strength and stability and high Marshall Compaction flow of 5.3 mm, and can highly affect the pavement performance. Recent laboratory studies have shown that the compaction can highly affect the performance of the hot-mix asphalt mixtures (Linden, Mahoney & Jackson 1992; Pourtahmasb & Karim 2010).

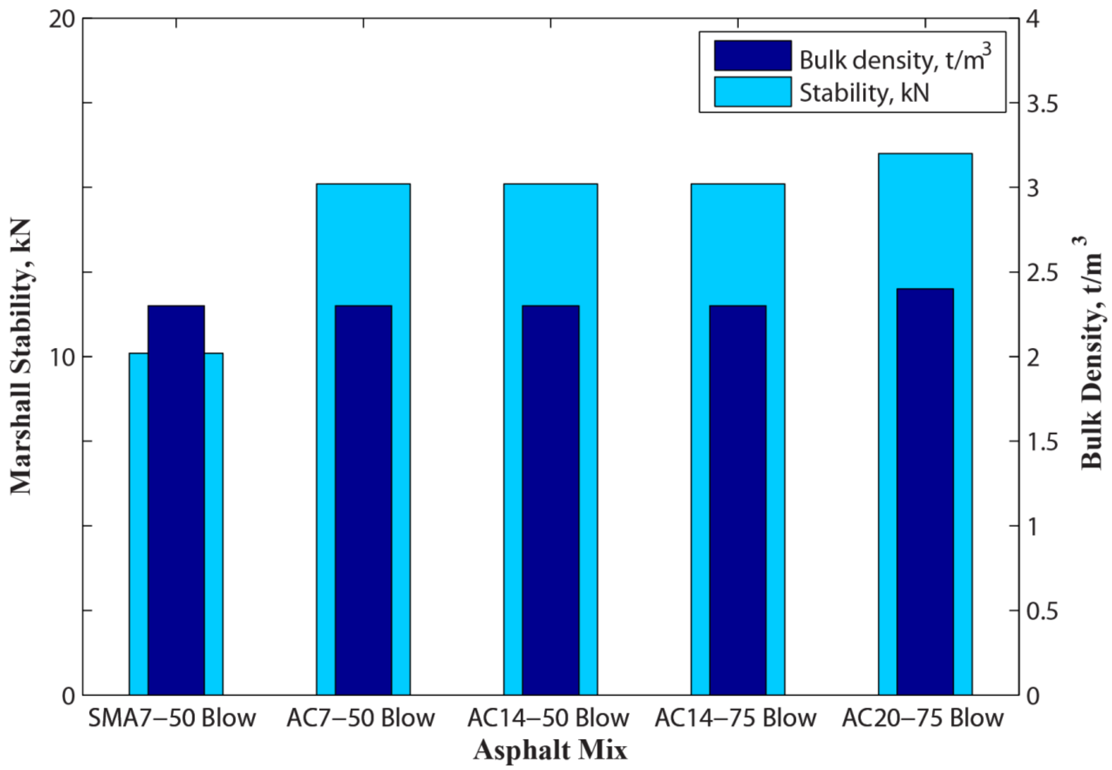


Figure 2.6 Average Marshall Compaction Stability and Bulk Density of the Different Types of Asphalt Mixes

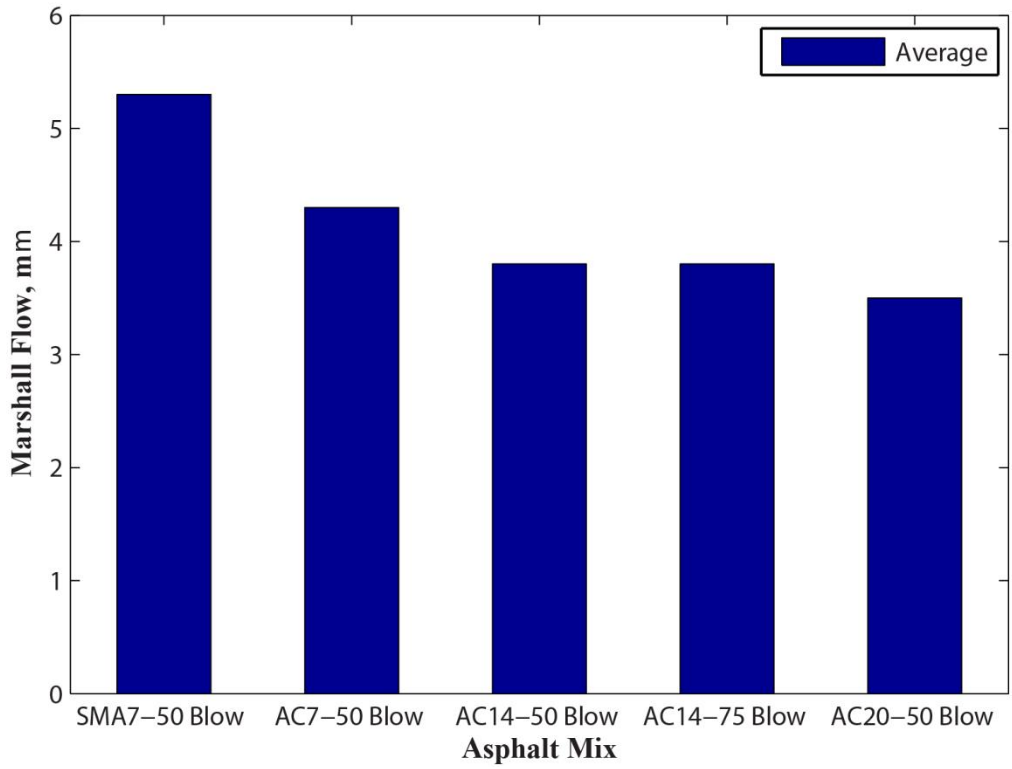


Figure 2.7 Comparison of Marshall Compaction Flow in Different Types of Asphalt Mixes

2.5. CONCLUSIONS

The pavement materials performance for strength and durability of hot mix asphalt mixtures was assessed and analyzed using the engineering characteristics and variability of flexible pavement for Western Australia roads in laboratory tests.

The comparison of the different types of asphalt mixes using a standard laboratory tests methods and techniques revealed that an AC20-75 blow and AC14-75 blow asphalt mix are the most efficient and effective in all categories of engineering characterization and variability of asphalt performance measures for strength and durability as compared to other asphalt mix.

In general, all the asphalt mixes that are used in this research study can be used in order to strength and stable the mixture stiffness of asphalt that is notable. The modification effect rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research.

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CHAPER 3: PAVEMENT MATERIALS CHARACTERIZATION OF HOT-MIX ASPHALT MIXES IN WESTERN AUSTRALIA

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ABSTRACT

The use of deep strength asphalt materials characterization to construct and restore the heavily urban roads where damage has been induced is rapidly grown in Western Australia. Five different types of asphalt mixes that were produced in laboratory are used to assess pavement performance of asphalt mixtures. The main role of this research is to evaluate the pavement materials characterization for Western Australia road. In this study: tensile strength, resilient modulus, wheel tracking, binder contents, Marshall Compaction, and air voids contents laboratory test for were taken to in order analyze each types of asphalt mixtures. The results indicated that AC20-75 and AC14-75 blow asphalt mixes were in a good pavement performance as compared to other asphalt mixes. For a mix design purposed, all the asphalt mixes in this study can be used to strength and stable the stiffness of pavement that is notable. These the modification effect ranks can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research.

Author keywords: Asphalt mixture; air voids contents; binders contents; Marshall compaction; materials characterization; resilient modulus; tensile strength; wheel tracking; Western Australia

3.1. INTRODUCTION

The use of full depth asphalt pavements to construct and rehabilitate heavily loaded urban roads has rapidly grown in Western Australia (WA) over the past 4 years. There is limited data available from testing carried out by the Mainroads about the characteristics and variability of WA asphalt mixes. Although some data is available from testing carried out by others on Mainroads contracts, but it also would be necessary to determine whether Mainroads WA owns this data and has the right to publish the data so that it can be used to predict the likely performance of WA full depth asphalt pavements.

High demand for new asphalt pavement often requires that paving be done unfavourable construction conditions. For example, Low air temperatures, high winds, and night construction create adverse conditions for hot-mix asphalt paving (Chadbourn et al. 1996). This presents a risk for road owners and contractors. To achieve optimum load-bearing and weathering characteristics, an asphalt mix must be compacted to a specific range of density, and the time required for hot-mix asphalt to reach the proper compaction temperature to achieve this density decreases with an increased rate of cooling (Chadbourn et al. 1996).

Hot-mix asphalt (HMA) is known by many different names: asphaltic concrete, plant mix, bituminous mix, bituminous concrete, and many others (Gillespie 1992; Gillespie et al. 1992). It is a combination of two primary ingredients – aggregates and asphalt binder. About 95% of the total mixtures by weight are aggregates, and these are mixed with an approximately 5% of asphalt binder to produce HMA (Gillespie 1992; Gillespie et al. 1992). Hot and cold asphalt mixes are comprised of two major materials: aggregates (mixture of sand, gravel, crushed stone, slag and mineral filler) and asphalt cement (crude oil, hydrated lime and dust) as discussed on literature by (American Concrete Institute Committee 2007; Cement Concrete & Aggregates Australia 2009; Mrawira & Luca 2006; Yang et al. 1995; Yang, Lin & Huang 1996). Bitumen had been defined by various sources as crude oil with a dynamic viscosity at reservoir conditions more than 10,000 centipoise (American Association of State Highway and Transportation Officials 1986b; American Concrete Institute Committee 2007; Hoiberg 1964; Hoiberg 1979; Uzan 2003; Yildirim 2007).

The asphalt concrete or hot mix asphalt (HMA) is the most widely used infrastructure materials for road construction. Hot mix asphalts can be described as a multiphase heterogeneous material composed of a viscoelastic asphalt binder, irregular rigid aggregate particle in high volume of fraction, and small percentage of air voids (Gopalakrishnan & Kim 2011). These various properties of materials component contribute to the complex mechanical behavior of HMA, which can be characterized as viscoelasticity, and plastic under different condition such as temperature, load application and aging (Dibike et al. 2001; Gopalakrishnan & Kim 2011). Thus the mechanical behavior of hot mix asphalt should be understood by not only the individual properties of HMA components, but also by considering asphalt binder and aggregate acting together.

The goal of this study is to evaluate the pavement materials characteristics of hot-mix asphalt for Western Australia roads using a laboratory tests so that data can be used to predict the likely performance of Western Australia flexible pavement.

3.2. MATERIALS AND METHODS

3.2.1. MATERIALS

Types of hot mixed asphalts that used on the Mainroads Western Australia network are dense graded asphalt (DGA), open graded asphalt (OGA) and stone mastic asphalt (SMA). DGA, the most common type of asphalt, provides optimal structure strength and generally good resistance to deformation. OGA is designed to drain water through the asphalt to remove excess water from the tyre/road surface. SMA is similar to OGA but has a high proportion of dust and high binder contents to achieve an improved fatigue life. SMA has a texture surface but does not drain water through its layer as does OGA (American Concrete Institute Committee 2007; Brown, Kandhal & Zhang 2004; Main Roads Western Australia 2007). All Materials selected for this project were from local sources and are indigenous of Western Australian pavement materials used in the industry.

3.2.2. METHODS

The mix design method that is specified by Mainroads Western Australia is the Marshall Mix design method. The aim of this method is to satisfy specified design criteria. The descriptions of asphalt mixes design are as following:

- SMA7-50 blow: thickness of 7 mm granite stone mastic asphalt (SMA)
- AC7-50 blow: thickness of 7 mm open graded granite
- AC14-50 blow: thickness of 14 mm dense graded granite
- AC14-75 blow: thickness of 14 mm dense graded granite
- AC20-75 blow: thickness of 20 mm dense graded granite

In order to assess the pavement material characterization of hot-mix asphalt mixes, it was necessary to obtain laboratory data. During an individual asphalt mixes run, specimen was taken and assessed in different categories of asphalt mixes. Specimens were subjected to the following laboratory characterization tests:

- Tensile strength ratio (TSR) test
- Resilient modulus test

- Wheel tracking test
- Asphalt binder content test
- Marshall Stability test
- Air voids content test

After samples had cooled to room temperature, bulk density and maximum specific gravity were performed according to the Australian Standard Test Method, AS 2891. Three specimens to each asphalt mixes were tested as per AS 2891, AG PT/231 and AG PT/23 (Austroads 1992; 2006; 2008). Air voids were calculated using bulk specific gravity and maximum theoretical specific gravity data. Specimens were placed in a water bath at 60^{o}C for a period of 30 min and were tested for Marshall Stability and flow. The details methods of sampling and testing of hot-mix asphalt in Australian Standard Testing Method are shown in Table 3.1.

Table 3.1 Methods of Sampling and Testing Asphalt in Australian Standard Test Method (Austroads 1992; 2006; 2008)

Material Test	**Test Method**
Tensile Strength Ratio	AGPT/T232
Resilient Modulus	AS 2891.13.1
Wheel Tracking	AGPT/T231
Asphalt Binder Content	AS 2891.1.1
Marshall Compaction	AS 2891.9.3
Air Voids Content	AS 2891.9.2

3.3. PAVEMENT MATERIALS CHARACTERIZATION OF ASPHALT MIXES

3.3.1. TENSILE STRENGTH RATIO

The conflict between bitumen, water and aggregate affinities has been an issue since the inception of asphalt as a paving material. In many situations the issue is minor, but when it does manifest as a stripping failure, the results can be catastrophic (Austroads 2008). Moisture sensitivity relate to the potential for loss adhesion between the binders and aggregate in the presence of moisture. This of adhesion is commonly referred to as stripping potential of asphalt- tensile strength ratio (TSR) (Austroads 2006; 2008). Stripping in asphalt is a complex mechanism. Where stripping occurs, it is often a combination of more than one such as climate and traffic, asphalt mix permeability, class of binder, poor coats of aggregate due to presence of clay or dust contamination, affinity of bitumen and asphalt mix design including type of tiller and use of other additives.

3.3.2. RESILIENT MODULUS

The resilient modulus is defined as a ratio of the deviator stress to the recoverable strain. It is known that the bituminous material is not elastic, but it experiences some permanent deformation after each load application (Jahromi & Khodaii 2009). However, if the load is small compared to the strength of material and is reported for number of times, the deformation under each load repetition is nearly completely recoverable and proportional to the load can be considered being elastic (Huang 1993). Resilient modulus is a measure of material's deflection behavior where a pavement life and surface deflection are strongly related (Elliott & Thornton 1988). It is also a fundamental and rational material property that needs to be includes in pavement design.

3.3.3. WHEEL TRACKING

Australia initially adopted the dynamic creep test as the preferred method of determining the rut resistance of asphalt mixtures (Austroads 2008). Currently, wheel tracking is selected as the most suitable test method for measuring the rut resistance of asphalt mixtures (Austroads 2008; Olive & Alderson 1995a). The wheel tracking test consists of a loaded wheel assembly and a confined mould in which a 300×300×200 mm specimen of asphalt mix is rigidly restrained on its four sides. A motor and a reciprocating device provide the forward and backward motion to the wheel at the rate of 24 passes per minute along the length of the slab. The temperature during the test is maintained by a water bath over and around the mould.

3.3.4. BINDER CONTENTS

The combined effect of binder content and air voids in a mix in the form of percent voids filled binder (VFB) has also been considered as an useful parameter in the fatigue life predication models (Harvey et al. 1995; Said 1997; SHRP1994b). Santucci (1977) applied a correction factor to estimate the fatigue lives of mixes with binder and air void contents other than the mixes with V_b = 11% and V_a = 5%, evaluated controlled stress fatigue tests.

3.3.5. MARSHALL COMPACTION

Recent laboratory studies have shown that the compaction can highly affect the performance of the hot-mix asphalt (HMA) and stone mastic asphalt (SMA) mixtures (Khan et al. 1998; Linden, Mahoney & Jackson 1992). Inappropriate compaction may draw the binder to the surface of HMA and SMA causing flushing of the surface and loss of texture or aggregate segregation (Pourtahmasb & Karim 2010). California kneading compactor, Gyratory compactor and Marshall Hammer are being used as SMA compactors due to mix design method (Khan et al. 1998). The California kneading compactor and Gyratory compactor are the most current preferred that are being used as mix design method.

3.3.6. AIR VOIDS CONTENTS

The air voids content in a mix is a function of void in mineral aggregate (VMA), binder content and level of compaction. The air voids (AV) content of a mix can affect the stability of durability of asphalt pavement. Asphalt mixes should be designed to have the lowest practical air voids value so that it can be reduce the aging of the binder, water penetration and stripping of binder from the aggregate. If the air void content asphalt in service is too less (less than 2 or 3%), plastic flow may occur resulting in flushing, bleeding, shoving and rutting of the pavement (Austroads 2006).

Choubane, Page and Musselman (1998) described that the air void content of an asphalt mixture is an important factor that affect the performance of the pavement throughout its service life. High air voids in a finished pavement, particularly if the voids are connected, will adversely affect its stability and durability of asphalt pavement performance in various ways such as air filtration into a permeability pavement can accelerates the aging and lead to potential pavement distresses. The permeability of a pavement is generally assumed to be proportional to its air void content. However, the lack of void interconnection and size dimension of the individual void may result in a watertight pavement of relatively increase AV content.

3.4. RESULTS AND ANALYSIS

A summary of average tensile strength ratio of dry and moisture condition is shown in Table 3.2. From the data presented, it can be seen that the AC7-50 blow asphalt mix has generally had high TSR of 112.9% as compared to other asphalt mixes. AC20-75 blow was the second best to have nearly reached a TSR of AC7-50 blow. This showed that the asphalt mixes are non-moisture susceptible. SMA7-50, AC14-50 and AC14-75 blow asphalt mixes had also relatively low as compared to AC7-50 and AC20-75 blow asphalt mixes but both of them are not susceptible to moisture. According to AASHTO T283,"Resistance of Compacted Bituminous Mixture to Moisture Induced Damage", the design asphalt mixture is judged to be non-moisture susceptible if it has a TSR greater than 80 percent (Airey, Hunter & Collop 2008; American Association of State Highway and Transportation Officials 2000; Apeagyei, Buttlar & Dempsey 2006).

Table 3.2 Tensile Strength (Dry and Moisture) for Different Types of Asphalt Mixes

Moisture Sensitivity				
Rank	**Mix Type**	**Tensile Strength (kPa)**		**TSR (%)**
		Dry	**Moisture**	
3	SMA7 - 50 Blow	686.9	626.6	91.2
1	AC7 - 50 Blow	831.5	938.7	112.9
4	AC14 - 50 Blow	990.4	894.8	90.3
5	AC14 - 75 Blow	1225.5	995.7	81.2
2	AC20 - 75 Blow	995.4	1024.8	103

The average resilient modulus for different types of asphalt mixes is given in Figure 3.1. As it can be seen from the results, AC20-75 blow asphalt mix had high resilient modulus of 6824 MPa as compared to the other asphalt mixes. AC14-75 blow was the second in rank with 5722 MPa. This shows that the asphalt mixes are more stable and durable than the other asphalt mixes in pavement performance. However, SMA7-50, AC7-50 and AC14-50 blow asphalt mix had poor resilient modulus of 2983, 4619, and 4282 MPa, respectively. And none of these asphalt mixes tested exceeded the Australian standard limit of 5500 MPa. Mainroads Western Australia (2010) and Austroad (2008) stated that the indirect tensile test asphalt modulus used must exceed 5500 MPa. Hicks and Monismith (1981) stated that the resilient modulus of partially crushed aggregate decreased with an increase in fine contents, while the modulus increased for crushed aggregate with increasing in fine content. Test temperature (Figure 3.1), the total recovered strain and 10% to 90% rise time (Figure 3.2) are similar for each asphalt mixes. This showed that the resilient modulus test is based on the determination of Australian Standard. Austroads (2008) described a standard that requires a total horizontal strain of 30 to 70 micro strain (ms) has to achieve in sample results if test is used to evaluate the resilient modulus of asphalt mixes.

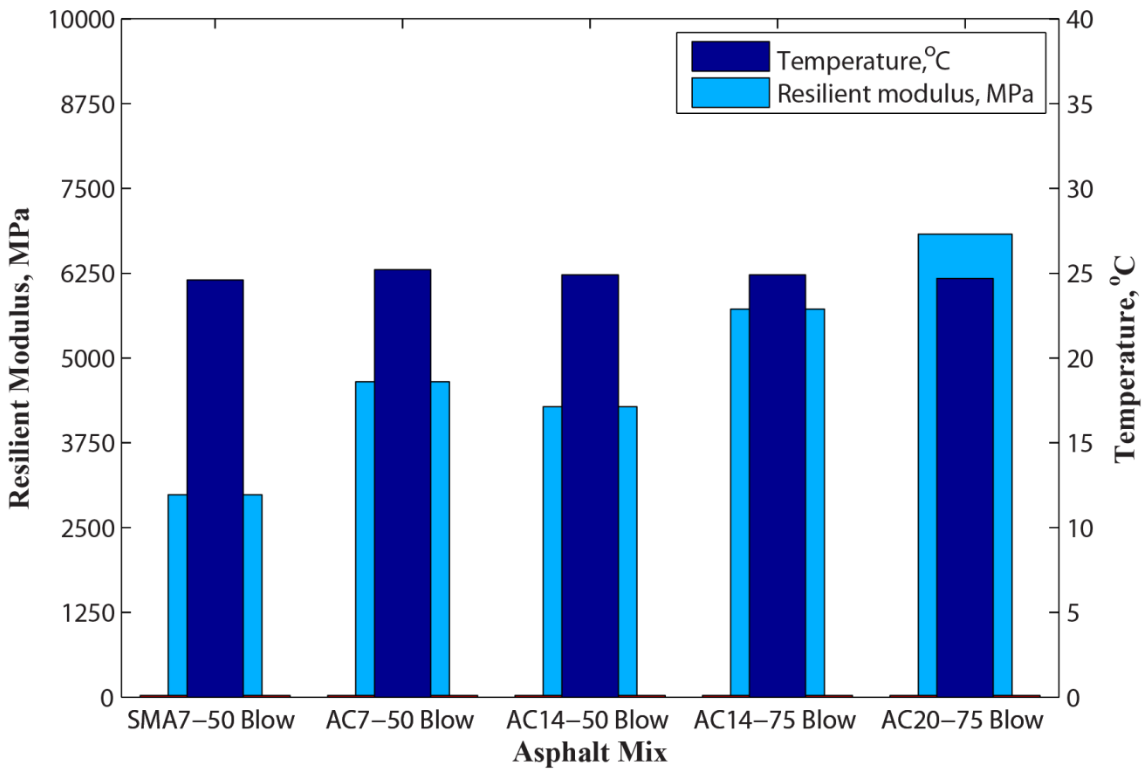

Figure 3.1 Resilient Modulus (MPa) and Temperature (°C) of Different Types of Asphalt Mixes

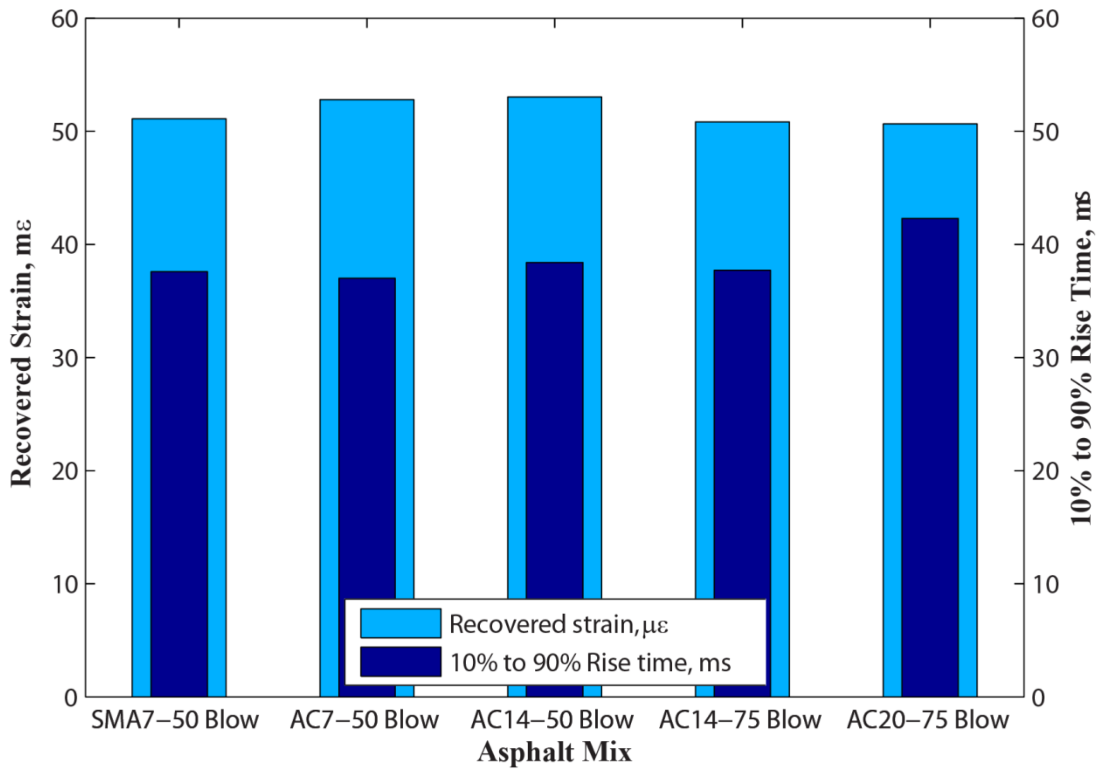

Figure 3.2 Recovered Strain (με) and 10% to 90% Rise time (ms) for Different Types of Asphalt Mixes

The average wheel tracking test for different types of asphalt mixes is shown in Figure 3.3. The analyses indicated that AC20-75 blow asphalt mix had low rut depth of 1.9 mm as compared to other asphalt mixes. AC14-75 blow was the second in rank with a rut depth of 2.4 mm. This showed that these asphalt mixes are high rut resistance of asphalt mixture and less to pavement distress and asphalt fatigue cracking in asphalt pavement mixture. However, the rut depth observed for SMA7-50 blow was 15 mm after 8, 452 cycles while 5 mm and 4.2 mm for AC7-50 blow and AC14-50 blow after 10,000 cycles, respectively. This indicated that SMA7-50 blow asphalt mix has high pavement distress and low rut resistance of asphalt mixture. There was also a sudden steep change in slope after 8,000 cycles for SMA7-50 blow. This may be attributed to the stripping of aggregate. No stripping was however, observed after 10, 000 cycles to other asphalt mixes.

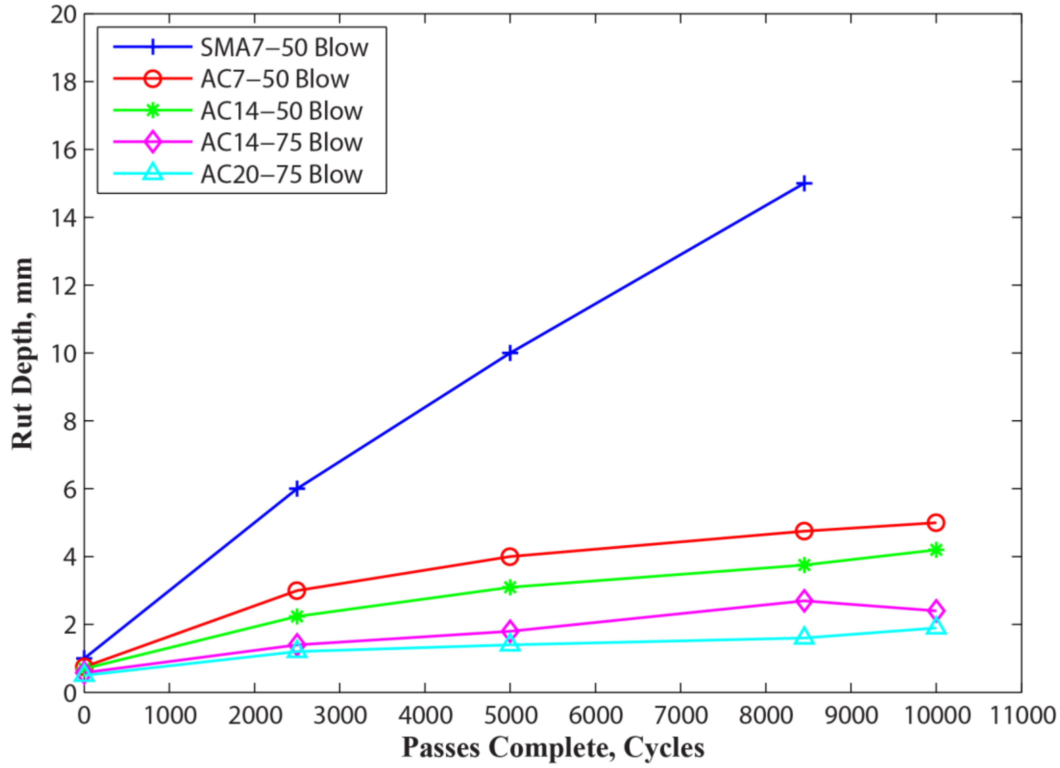

Figure 3.3 Average wheel tracking tests for different types of asphalt mixes

A summary of asphalt binder content for different types of asphalt mixes is shown in Table 3.3. The results indicated that AC14-75 and A20-75 blow property of asphalt mix had low percentage of binder content of 4.7% and 4.3% in the given order. This showed that AC14-75 and AC20-75 blow asphalt mixes might have increased the frictional contact between aggregate particle and the overall stiffness and stability of the asphalt mixes as compared to other asphalt mixes. However, SMA7-50, AC7.50 and AC14-50 blow asphalt mixes had high percentage of binder content. This showed that increase in binder content reduce the frictional contract between aggregate particles. Beyond a certain value, further increases in binder content reduce the frictional contact between aggregate particles and the overall stiffness and stability of the asphalt (Austroads 2006). At low percentages, added binder content increases the mix cohesion and strength (Anderson, Walker & Turner 1999).

Table 3.3 Asphalt Binder Content for Different Types of Asphalt Mixes

Asphalt Binder Contents		
Rank	**Mix Type**	**Binder Content (%)**
5	SMA7-50 Blow	6.7
4	AC7-50 Blow	6.3
1	AC14-50 Blow	5.3
3	AC14-75 Blow	4.7
2	AC20-75 Blow	4.3

Average Marshall Compaction stability and Marshall Compaction flow for different types of asphalt mixes are shown in Table 3.4. As it can be seen from the analyses, AC20-70 blow asphalt mix had high stability of 16 kN as compared to other asphalt mixes. AC7-50, AC14-50 and AC14-75 blow asphalt mix had a similar strength and stability of 15.1 kN. This indicated that the asphalt mixes pavement will have the necessary bearing capacity to support the expected traffic loads and durability to withstand weathering. Marshall Compaction flow rate for AC20-75 blow was less as compared to other asphalt mixes. However, SMA7-50 blow had poor strength and stability and high Marshall Compaction flow of 5.3 mm, and can highly affect the pavement performance. Recent laboratory studies have shown that the compaction can highly affect the performance of the hot-mix asphalt mixtures (Linden, Mahoney & Jackson 1992; Pourtahmasb & Karim 2010).

Table 3.4 Average Marshall Stability and Flow of Different Types of Asphalt Mixes

Marshall Stability and Flow			
Rank	**Mix Type**	**Stability (kN)**	**Flow (mm)**
5	SMA7-50 Blow	10.1	5.3
4	AC7-50 Blow	15.1	4.3
1	AC14-50 Blow	18.0	3.8
3	AC14-75 Blow	15.1	3.8
2	AC20-75 Blow	16.0	3.5

Average air voids contents for different types of asphalt mixes, and mass that are needed for the specimens in order to get 5% and 8% air voids to each mixture are shown in Figure 3.4 through 3.8. From the data it has been demonstrated AC20-75 asphalt mix (Figure 3.8) had superior in pavement performance. This shows that the content of a mix is more stable and durable of asphalt pavement. AC7-50 (Figure 3.5) and AC14-75 (Figure 3.7) blow asphalt mixes had shown increased in air void content above the limit. Linden and Van Der Heiden (1989) described that every 1% increase in voids above

7%, there would be a reduction in pavement life of about 10% or that the pavement life would be reduced by about 1 year. However, SMA7-50 blow (Figure 3.4) and AC14-50 blow (Figure 3.6) has poor performance with less than 2% air voids. This shows that the asphalt mixes might be exposed to flushing and rutting of asphalt pavement. Austroad (2006) stated that air voids should not be too low (less than 2 or 3%) because plastic flow may occur resulting in flushing, bleeding, shaving or rutting of pavement. Similarly, Aggregate particles that are into close contact are able to resist load with lower strain and hence are stiffer.

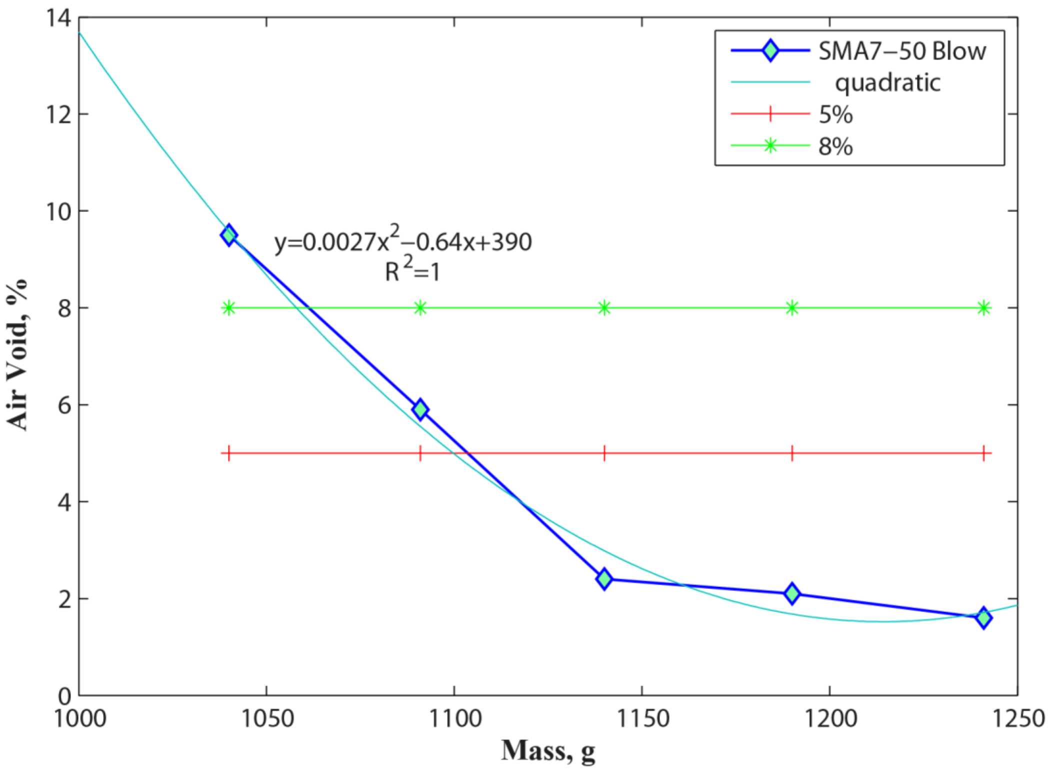

Figure 3.4 Mass versus Air Voids Weight for SMA7-50 Blow Asphalt Mixes

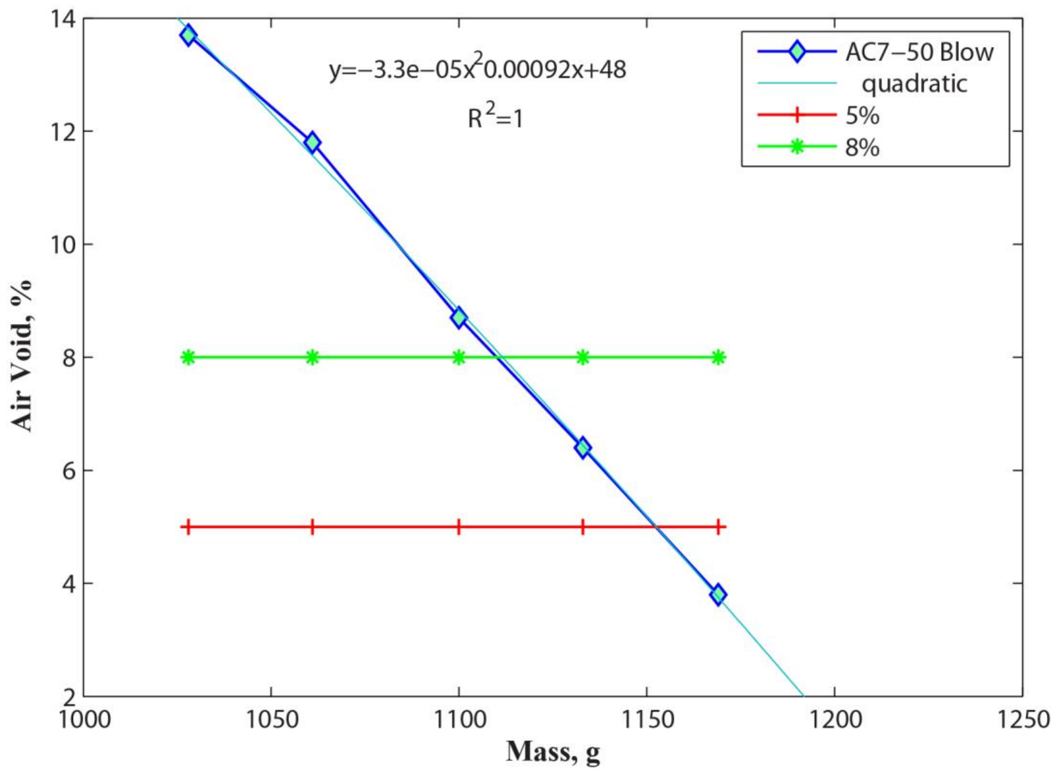

Figure 3.5 Mass versus Air Voids Weights for AC7-50 Blow Asphalt Mixes

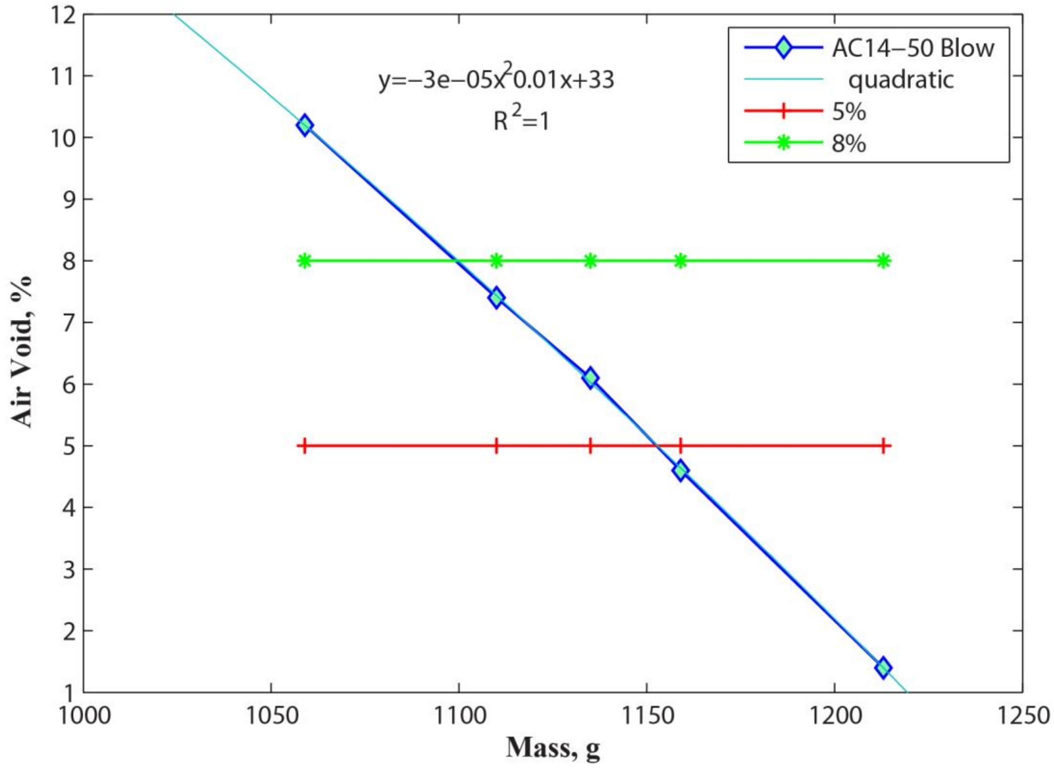

Figure 3.6 Mass versus Air Voids Weights for AC14-50 Blow Asphalt Mixes

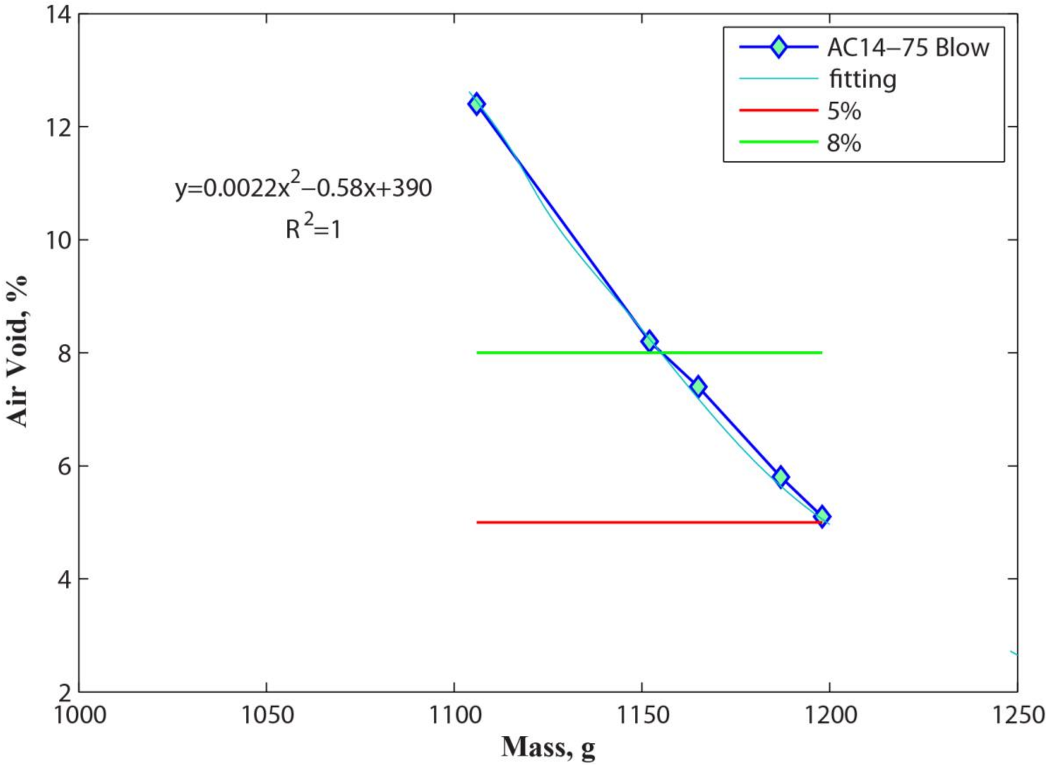

Figure 3.7 Mass versus Air Voids Weight for AC14-75 Blow Asphalt Mixes

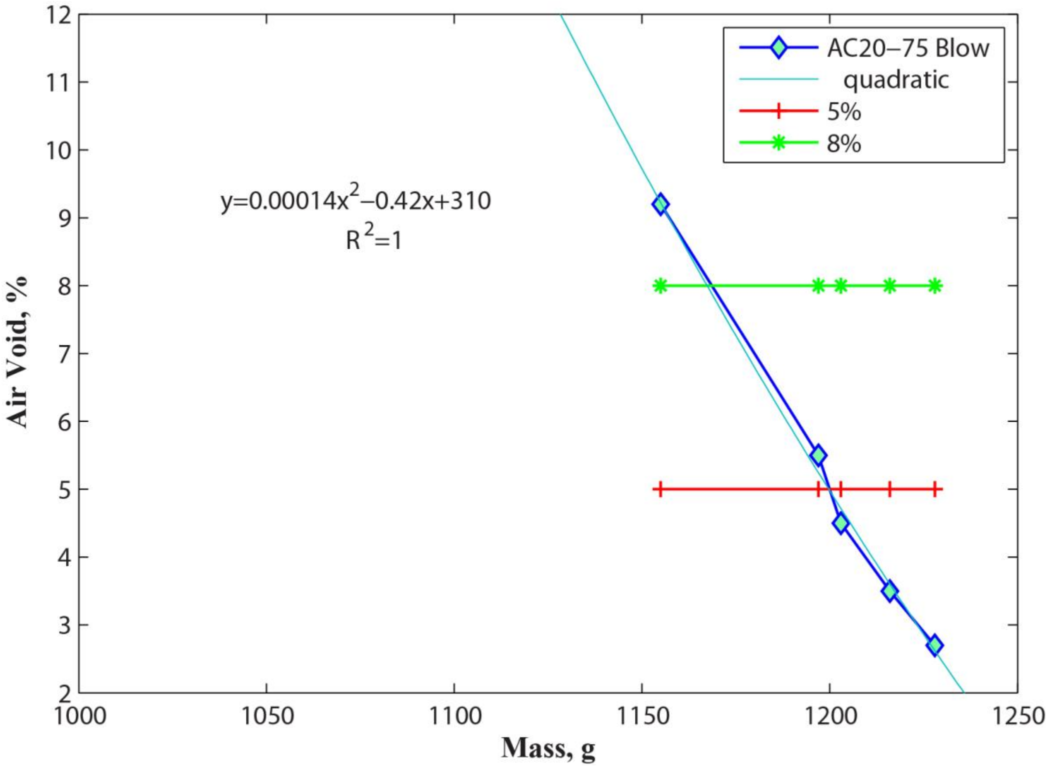

Figure 3.8 Mass versus Air Voids Weight for AC20-75 Blow Asphalt Mixes

3.5. CONCLUSIONS

The pavement materials performance for strength and durability of hot mix asphalt mixes using the engineering characterization and variability for asphalt mixes parameters such tensile strength ratio, resilient modulus, wheel tracking, asphalt binder content and Marshall compaction tests in a laboratory experiment for different types of asphalt mixes was assessed.

The comparison of the different types of asphalt mixes using a standard laboratory tests methods and techniques revealed that an AC20-75 Blow asphalt mix method is the most efficient and effective in all categories of engineering characterization and variability of asphalt performance measures for strength and durability of HMA than SMA7-50, AC7-50, AC14-50, and AC14-75 blow asphalt mix. AC14-75 blow asphalt mix was the second best that increases the frictional contact between aggregate particles and overall stiffness and stability of the asphalt mix.

For a mix design purposed, all the asphalt mixes that are used in this study can strength and stable the stiffness of pavement that is notable, and the modification effect rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research.

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CHAPTER 4: EVALUATION AND VALIDATION OF CHARACTERIZATION METHODS FOR FATIGUE PERFORMANCE OF ASHPHALT MIXES FOR WESTERN AUSTRALIA

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ABSTRACT

The determination of appropriate pavement thickness using laboratory determined parameters is one of the key issues facing the road manager. Five different types of asphalt mixes that were produced in laboratory are used to modify pavement performance mixture. The main objective of this study is to evaluate the characterization methods for fatigue performance of asphalt mixes to Western Australia road. In this study, laboratory test for indirect tensile modulus, dynamic creep, wheel tracking and aggregate gradation tests were taken to analyze each asphalt mixtures for a design traffic road. The results and analysis showed that AC20-75 asphalt mix blow is the most effective and efficient in pavement performance than the other asphalt mixes. AC14-75 was the second in rank to strengthen and durability of asphalt pavement. All asphalt mixes in this study can be used to strength and stable the overall stiffness of pavement, and modification rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research.

Author keywords: Pavement performance; asphalt mixture; characterization; indirect tensile modulus; dynamic creep; wheel tracking; aggregate gradation; Western Australia

4.1. INTRODUCTION

Around the world, demand for new asphalt pavement is increasing and lack of quantitative information on the long term field performance of asphalt mixes with conventional and other binder types, lack of quantitative information in the deformation of the appropriate pavement thickness and development of long term performance assessment of hot-mix asphalt thermal properties compaction using laboratory and field investigation is one of the key issues facing the main road and asphalt plant manager (Baburamani 1999).

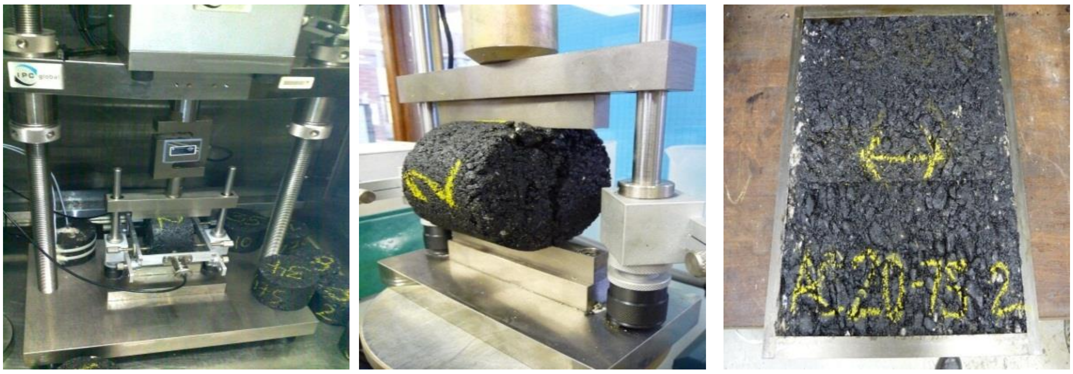
High demand for new asphalt pavement often requires that paving be done unfavourable construction conditions. For example, Low air temperatures, high winds, and night construction create adverse conditions for hot-mix asphalt paving (Chadbourn et al. 1996). This presents a risk for road owners and contractors. To achieve optimum load-bearing and weathering characteristics, an asphalt mix must be compacted to a specific range of density, and the time required for hot-mix asphalt to reach the proper compaction temperature to achieve this density decreases with an increased rate of cooling (Chadbourn et al. 1996).

The difference between laboratory and in-service pavement fatigue lives is attributed a difference into loading condition, vehicle types and axle configuration, rest between vehicle load (effects of residual stresses and healing), traffic distribution (mixed traffic effect), vehicle wander, difference in mix compaction levels achieved and environmental factors such as seasonal temperature variation and temperature gradients that occur in the pavement.

To account for these differences, transfer functions or 'shift factor' are applied to derive predication of in-serve performance. As mentioned by several researchers and universities scholars, the magnitude of the shift factor is generally between 10 and 20 (Baburamani 1999; Chadbourn et al. 1996; Rowe 1993; Said 1988; Strategic Highway Research Program 1994a; The Asphalt Institute 1998) depending on the perceived level of cracking in the pavement that could be tolerated. However, there is a strong need for further investigation, initially through a detailed laboratory characterization of indigenous asphalt mixes with conventional, multigrade and modified binders of the effect of volumetric properties, rest period, healing, and ageing, and multi-axle loading effects on fatigue performance (Baburamani 1999; Strategic Highway Research Program 1994a).

There is a lack of quantitative information on the long – term field performance of asphalt mixes with conventional and other binder types. This information would assist in the estimate of shift factor and in the deformation of the appropriate pavement thickness.

The aim of this research is to evaluate the performance characteristics of the main mixes supplied to Mainroads Western Australia in terms of fatigue and deformation resistance, so that these data can be used to predicting thickness requirements for heavy duty pavements. Figure 4.1 shows a specimen in different types of testing equipment.



(a) Dynamic creep test **(b) Indirect tensile test** **(c) Wheel tracking test**

Figure 4.1 Specimens in Different Types of Testing Equipment

4.2. MATERIALS AND METHODS

4.2.1. MATERIALS

Types of hot mixed asphalt used on the Mainroads Western Australia (WA) network are dense graded asphalt (DGA), open graded asphalt (OGA) and stone mastic asphalt (SMA). DGA, the most common type of asphalt, provides optimal structure strength and generally good resistance to deformation. OGA is designed to drain water through the asphalt to remove excess water from the tyre/road surface. SMA is similar to OGA but has a high proportion of dust and high binder contents to achieve an improved fatigue life. SMA has a texture surface but does not drain water through its layer as does OGA (American Concrete Institute Committee 2007; Brown, Kandhal & Zhang 2004; Main Roads Western Australia 2007). All Materials selected for this project were from local sources and are indigenous of Western Australian pavement materials used in the industry.

4.2.2. METHODS

The design method specified by Mainroads Western Australia is the Marshall method of mix design. The aim of the method is to satisfy specified design criteria. The descriptions of asphalt mixes design are as following:

- SMA7-50 blow: thickness of 7 mm granite stone mastic asphalt (SMA)
- AC7-50 blow: thickness of 7 mm open graded granite
- AC14-50 blow: thickness of 14 mm dense graded granite

- AC14-75 blow: thickness of 14 mm dense graded granite
- AC20-75 blow: thickness of 20 mm dense graded granite

In order to assess the evaluation and validation of characterization methods for fatigue performance of different types of asphalt mixture, it was necessary to obtain laboratory data on individual asphalt mixtures. During an individual asphalt mixes run, specimen was taken and assessed in different categories of asphalt mixes. Laboratory asphalt mix specimens were subjected to the following characterization tests: resilient modulus, dynamic creep, wheel tracking, and aggregate gradations test.

After samples had cooled to room temperature, bulk density and maximum specific gravity were performed according to the Australian Standard Test Method, AS 2891 from the local sources and indigenous of Western Australia roads. Air voids were calculated using bulk specific gravity and maximum theoretical specific gravity data. Five specimens to each asphalt mixes were tested as per AS 2891 and AG: PT/T231 (Austroads 1992; 2006; 2008). Absorption values have been determined from presaturation testing using five masses and applied to the other four tests. The details methods of sampling and testing of hot-mix asphalt in Australian Standard Testing Method are shown in Table 4-1.

Table 4.1 Methods of Sampling and Testing Asphalt in Australian Standard Test Method (Austroads 1992; 2006; 2008)

Material Test	Test Method
Resilient Modulus (Indirect Tensile Test Method)	AS 2891.13.1
Dynamic Creep	AS 2891.12.1
Wheel Tracking	AGPT/T231
Aggregate Gradations	AS 2891.3

4.3. CHARACTERIZATION METHODS FOR FATIGUE PERFORMANCE OF ASPHALT MIXES

4.3.1. INDIRECT TENSILE MODULUS / RESILIENT MODULUS

Resilient modulus of pavement material is an important material property in any mechanistically based design/analysis procedure for flexible pavements (Federal

Highway Administration 2002). The resilient modulus (M_R) is the material property required for the 1993 American Association of State Highway and Transportation Officials (AASHTO) Design Guide, which is an empirically based design procedure, and is the primary material input parameter for the 2002 Design Guide (American Association of State Highway and Transportation Officials 1993). Several types of moduli have been used to represent the stiffness of asphalt concrete mixture. Three of these are dynamic, resilient, and complex. The Modulus of Resilient is most commonly used for asphalt concrete mixture evaluation (Khan et al. 1998).

4.3.2. DYNAMIC CREEP

Rutting is one of the predominant types of distress observed in hot-mix asphalt. Mallick, Ahlrich and Brown (1995) used a dynamic creep to predict rutting. To evaluate the potential of dynamic creep, test were conducted on mixes of different aggregates and aggregate graduations to identify mixes with rutting potential. From the results that were obtained, there were a good correlation between permanent creep stain and rutting rates of pavements. The dynamic creep test was able to quantify the effect of aggregate type and graduation on rutting potentials of the mixes (Mallick, Ahlrich & Brown 1995). Mixes with crushed aggregate performed better in the creep test than the mixes with uncrushed aggregate.

4.3.3. WHEEL TRACKING

Australia initially adopted the dynamic creep test as the preferred method of determining the rut resistance of asphalt mixtures. The test was simple to operate and test specimens were simple to compact in the laboratory or obtain by coring from existing pavements (Alderson 1998; Austroads 2008). Wheel tracking was selected as the most suitable test method for measuring the rut resistance of asphalt mixtures. Olive and Alderson (1995a) indicated that the test had been shown to correlate well with rutting of roads in-service. The present standard in use in Australia is an Austroads test method (AG: PT/T231) that permit to determine the characteristic of wheel tracking test for both laboratory and field asphalt mixes (Austroads 2008).

4.3.4. AGGREGATE GRADATION

The fatigue life of asphalt concrete mixture is influenced by several factors such as bitumen type, binder content, and air voids content and other factors such as temperature, frequency and resting period of applied load. Although the influence of binder type and voids contents has been extensively studied, the effect of some aggregate properties such as the aggregate gradation has not been widely presented. Goetz (1989) noted that the important role of aggregate gradation was recognized from the earliest days of asphalt mixture design.

Aggregate type and graduation are routinely considered in hot-mix asphalt (HMA) design. The line between aggregate gradation and asphalt mixture performance was recognized early in the development of the mix design methods (Huber & Shuler 1992). Various method of analyzing gradation were evaluated for an aggregate gradation "law" which could be used in selection of gradation for mixture design and one of them is 0.45 power chart.

Aggregate graduation is an important factor that influences the permanent deformation of HMA. One common way of characterizing aggregate gradation is by making a gradation plot on a 0.45 power chart (Kandhal & Mallick 2001), which also continue the maximum density line. The use of 0.45 power chart is to estimate available voids in the mineral aggregate (VMA) of the compacted mixture (Huber & Shuler 1992). Increase VMA is obtained moving further from the maximum density line. Superpave has introduced a restriction zone superimposed on the maximum density between a line 2.36 mm and 0.3 mm sieve sizes, through which graduation are not recommended to pass (Strategic Highway Research Program 1994a). It is believed that graduation passing through this zone can have low stability or resistance to rutting.

4.4. RESULTS AND ANALYSIS

A summary of average resilient modulus (indirect tensile modulus test) obtained for SMA7-50, AC7-50, AC14-50, AC14-75, and AC20-75 Blow asphalt mixes are given in Table 4.2 through 2.6. From the data presented, it can be seen that AC20-75 Blow had high resilient modulus of 6794 MPa (Table 4.6) as compared to other asphalt mixes. AC14-75 blow was the second in rank of 5959 MPa (Table 4.5). This shows that the asphalt mixes are more stable and durable in pavement performance than others. However, SMA7-50, AC7-50, and AC14-50 Blow had a poor resilient modulus of 3600 MPa (Table 4.2), 4223 MPa (Table 4.3), and 4921 MPa (Table 2-4) in the given order, and none of these asphalt mixes tested exceeded the Australian standard limit. Mainroads Western Australia (2010) and Austroad (2008) stated that the indirect tensile test asphalt modulus used must exceed 5500 MPa.

The recovered horizontal strain and the resilient displacement deformation are given in Figures 4.2 through 4-6. From the data demonstrated, it can be seen that the results are more or less similar for each asphalt mixes, and also within similar ranges of displacement of each pulse of each asphalt mixture. This shows that each one of them is within the required displacement and recovered strain standard. Austroads (2008) described the standard requires that a total horizontal strain of between 30 to 70 micro strain (ms) has to achieve in a sample test results if the indirect tensile modulus test is used to evaluate the resilient modulus of asphalt mixes. This ensures that there is sufficient deformation for the linear variable displacement. It is clear that asphalt mix properties that tend to contribute low resilient moduli are low plasticity, low group index, high silt content, low specific gravity, and low group contents. Hicks and Monismith (1981) discussed that resilient modulus of partially crushed aggregate decreased with an increase in fine contents, while the modulus increased for crushed aggregate with increase in fine content. Similarly, Thompson (1989) mentioned that for given gradations, crushed material provide an increase in resilient modulus.

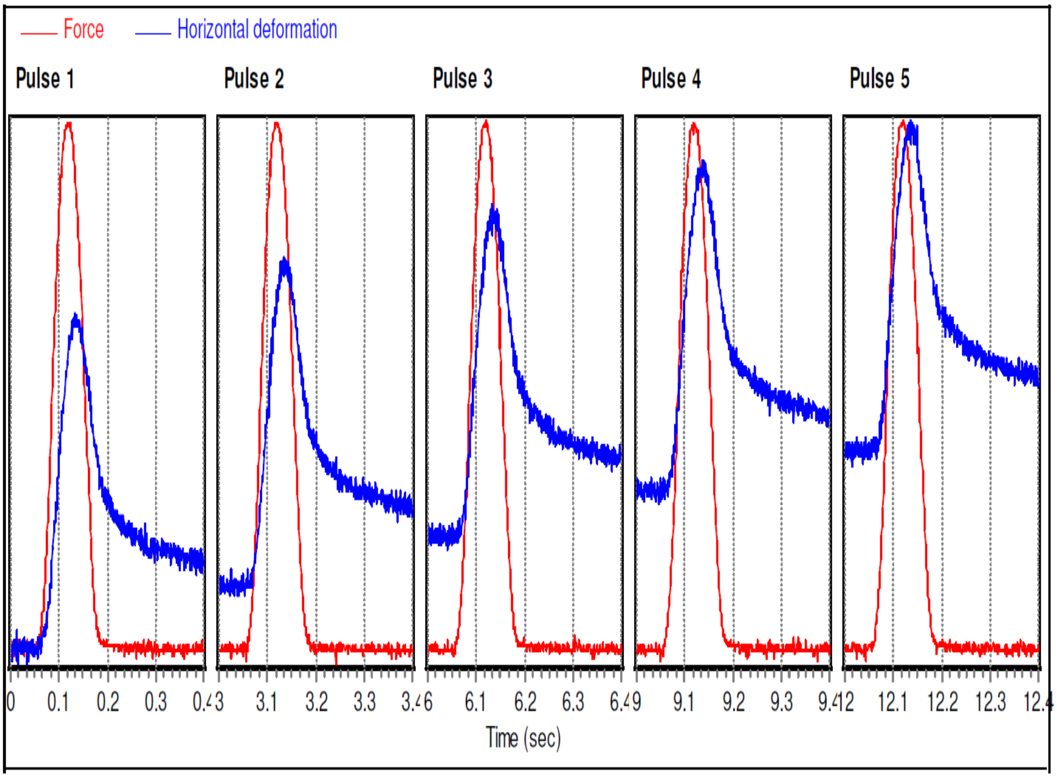


Figure 4.2 Indirect Tensile Modulus for SMA7-50 Blow Asphalt Mixes

Table 4.2 Indirect Tensile Modulus Test for SMA7-50 Blow Asphalt Mixes

SMA7-50 Blow	**Pulse 1**	**Pulse 2**	**Pulse 3**	**Pulse 4**	**Pulse 5**	**Mean**	**Std. Dev**	**CV (%)**
Resilient modulus (MPa)	3711.0	3651.0	3590.0	3509.0	3537.0	3600.0	73.9	2.1
Recovered horizontal strain (με)	51.0	51.8	53.0	54.1	53.8	52.7	1.2	2.2
Peak load (N)	1841.3	1842.7	1852.5	1845.7	1852.0	1846.8	4.7	0.3
10% to 90% rise time (ms)	36.0	35.0	36.0	36.0	37.0	36.0	0.6	1.8
Load time (ms)	121.0	121.0	119.0	122.0	123.0	121.2	1.3	1.1
Phase delay at 90% (ms)	18.0	16.0	13.0	13.0	11.0	14.2	2.5	17.5
Resilient horizontal Deformation (μm)	2.6	2.6	2.7	2.7	2.7	2.6	0.0	2.3

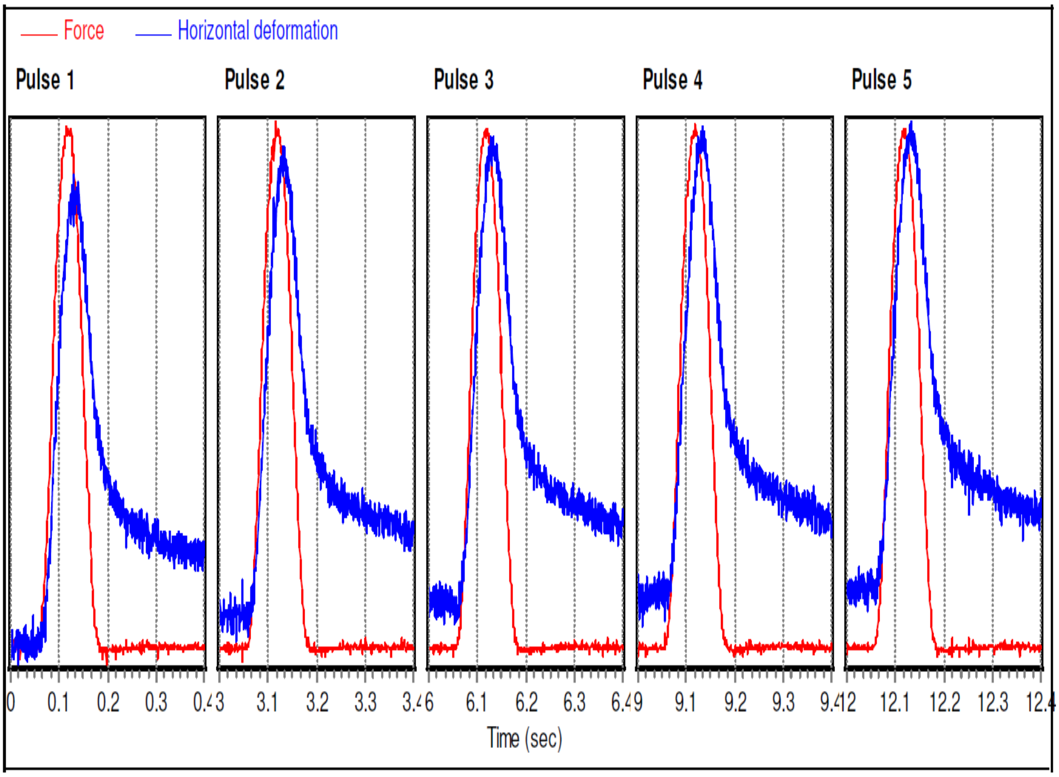


Figure 4.3 Indirect Tensile Modulus for AC7-50 Blow Asphalt Mixes

Table 4.3 Indirect Tensile Modulus Test for AC7-50 Blow Asphalt Mixes

AC7-50 Blow	**Pulse 1**	**Pulse 2**	**Pulse 3**	**Pulse 4**	**Pulse 5**	**Mean**	**Std. Dev**	**CV (%)**
Resilient modulus (MPa)	4377.0	4283.0	4152.0	4172.0	4133.0	4223.0	92.9	2.2
Recovered horizontal strain (με)	50.0	51.5	52.5	52.7	52.7	51.9	1.0	2.0
Peak load (N)	2181.2	2137.2	2109.9	2129.4	2107.0	2120.3	11.5	0.5
10% to 90%rise time (ms)	36.0	36.0	36.0	38.0	36.0	36.4	0.8	2.2
Load time (ms)	120.0	118.0	116.0	118.0	118.0	118.0	1.3	1.1
Phase delay at 90% (ms)	26.0	18.0	15.0	13.0	16.0	17.6	4.5	25.6
Resilient horizontal Deformation (µm)	2.5	2.4	2.6	2.6	2.6	2.6	0.1	2.3

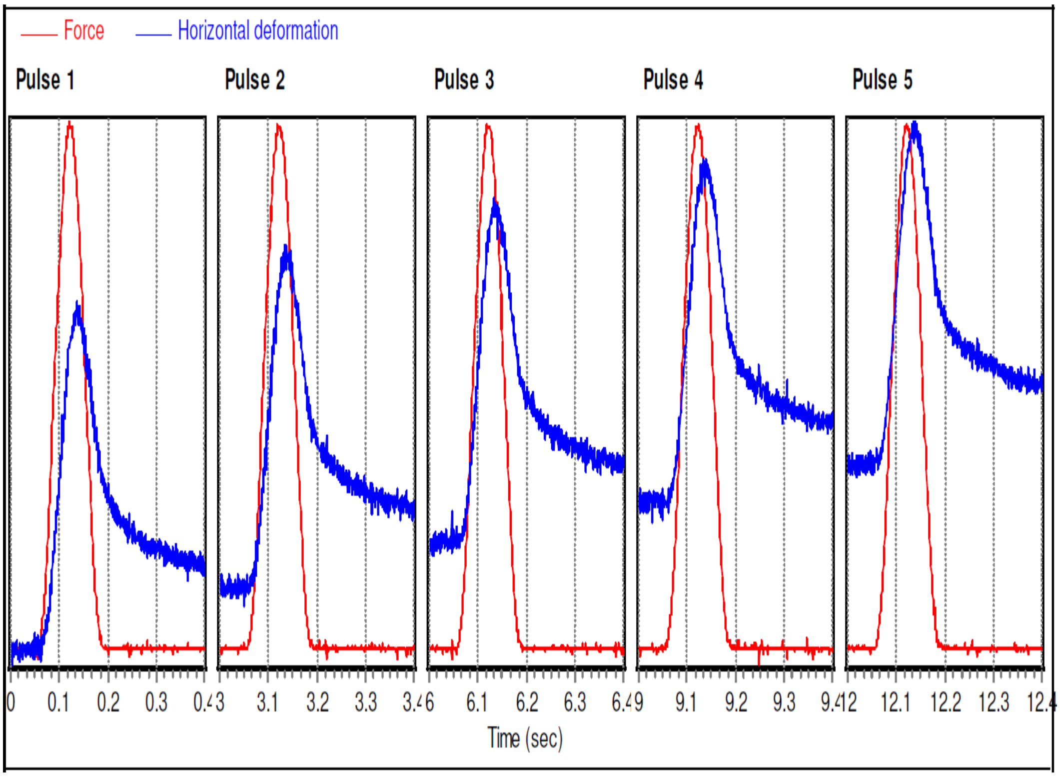


Figure 4.4 Indirect Tensile Modulus for AC14-50 Blow Asphalt Mixes

Table 4.4 Indirect Tensile Modulus Test for AC14-50 Blow Asphalt Mixes

AC14-50 Blow	**Pulse 1**	**Pulse 2**	**Pulse 3**	**Pulse 4**	**Pulse 5**	**Mean**	**Std. Dev**	**CV (%)**
Resilient modulus (MPa)	5158.0	4962.0	4842.0	4845.0	4745.0	4921.0	130.5	2.7
Recovered horizontal strain (με)	51.7	53.5	54.8	54.8	55.3	54.0	1.3	2.5
Peak load (N)	2584.6	2576.3	2575.6	2577.8	2573.4	2577.8	3.7	0.1
10% to 90% rise time (ms)	41.0	40.0	41.0	39.0	40.0	40.2	0.8	1.9
Load time (ms)	127.0	123.0	123.0	122.0	123.0	123.6	1.7	1.4
Phase delay at 90% (ms)	17.0	12.0	12.0	9.0	11.0	12.2	2.6	21.6
Resilient horizontal Deformation (μm)	2.6	2.7	2.8	2.8	2.7	2.7	0.1	2.7

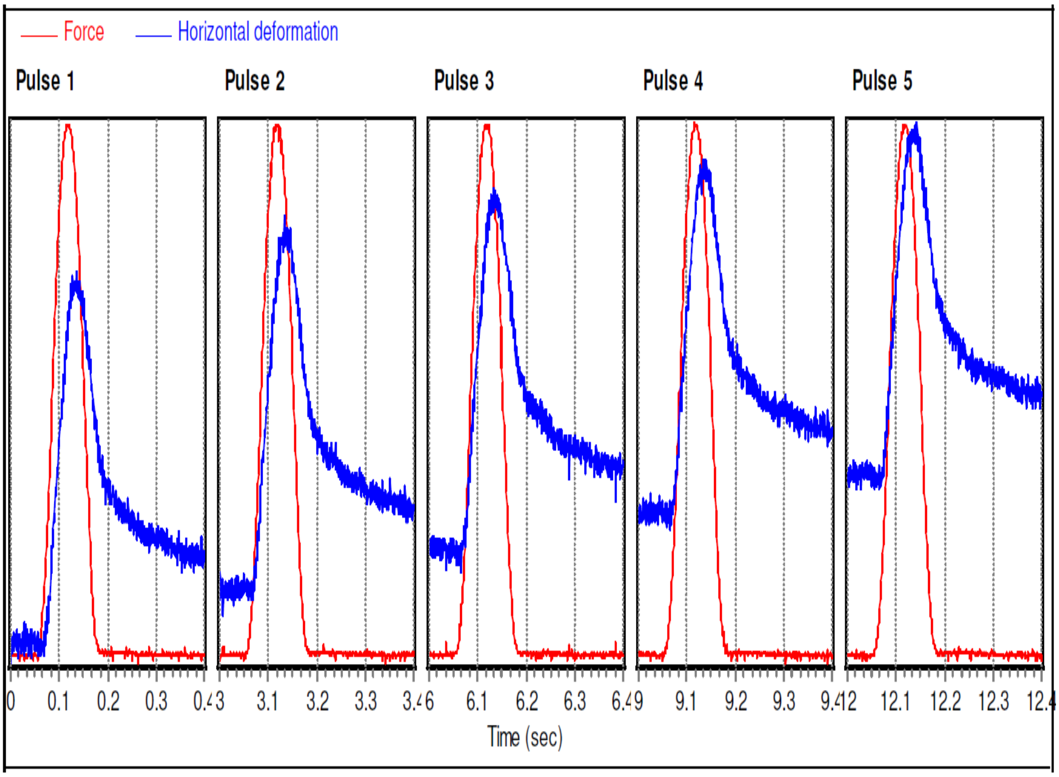


Figure 4.5 Indirect Tensile Modulus for AC14-75 Blow Asphalt Mixes

Table 4.5 Indirect Tensile Modulus Test for AC14-75 Blow Asphalt Mixes

AC14-75 Blow	**Pulse 1**	**Pulse 2**	**Pulse 3**	**Pulse 4**	**Pulse 5**	**Mean**	**Std. Dev**	**CV (%)**
Resilient modulus (MPa)	6016.0	5776.0	5912.0	6107.0	5983.0	5959.0	110.8	1.9
Recovered horizontal strain (με)	47.7	49.7	48.6	47.2	48.0	48.3	0.9	1.8
Peak load (N)	2926.9	2830.3	2832.7	2838.1	2831.3	2831.8	3.7	0.1
10% to 90% rise time (ms)	37.0	37.0	37.0	37.0	38.0	37.2	0.4	1.1
Load time (ms)	119.0	121.0	119.0	119.0	121.0	119.9	1.0	0.8
Phase delay at 90% (ms)	16.0	21.0	17.0	13.0	15.0	16.4	2.7	16.2
Resilient horizontal Deformation (μm)	2.4	2.5	2.4	2.4	2.4	2.4	0.1	12.8

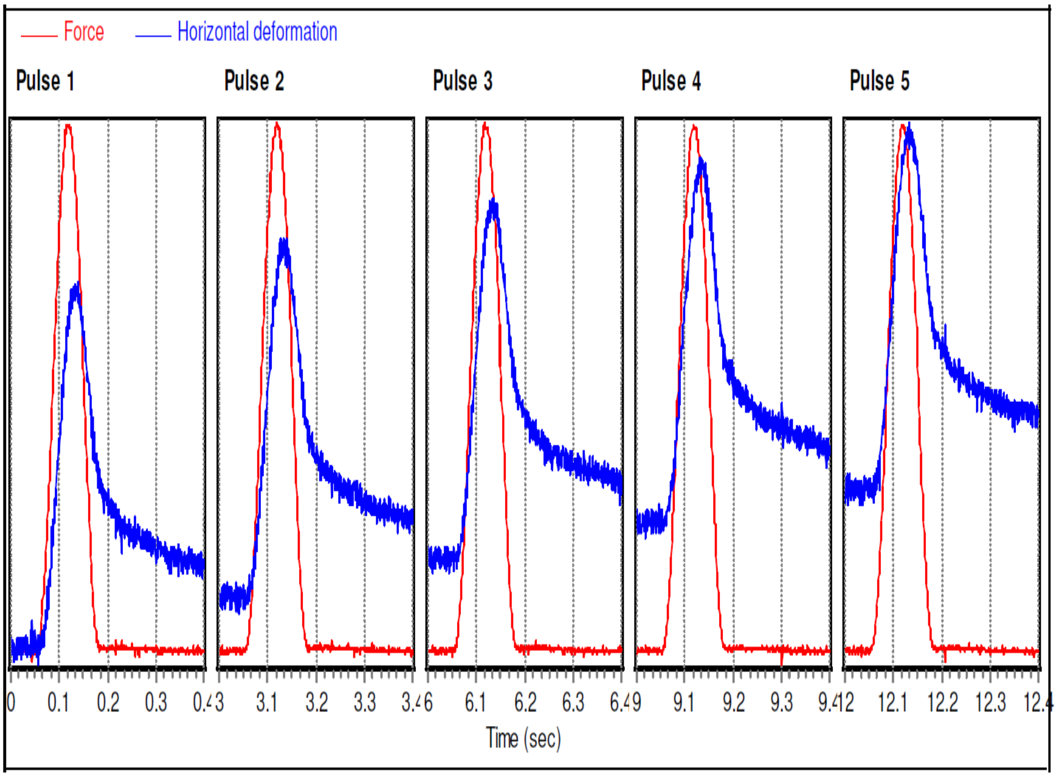


Figure 4.6 Indirect Tensile Modulus for AC20-75 Blow Asphalt Mixes

Table 4.6 Indirect Tensile Modulus for AC20-75 Blow Asphalt Mixes

AC20-75 Blow	**Pulse 1**	**Pulse 2**	**Pulse 3**	**Pulse 4**	**Pulse 5**	**Mean**	**Std. Dev**	**CV (%)**
Resilient modulus (MPa)	7047.0	6931.0	6786.0	6638.0	6567.0	6794.0	177.8	2.6
Recovered horizontal strain (με)	49.3	50.3	51.3	52.1	52.8	51.2	1.3	2.5
Peak load (N)	3300.1	3314.3	3309.4	3288.4	3297.2	3301.9	9.1	0.3
10% to 90% rise time (ms)	39.0	39.0	39.0	39.0	39.0	39.0	0.0	0.0
Load time (ms)	120.0	120.0	121.0	118.0	120.0	119.8	1.0	0.8
Phase delay at 90% (ms)	15.0	12.0	11.0	11.0	10.0	11.8	1.7	14.6
Resilient horizontal Deformation (μm)	2.5	2.3	2.4	2.5	2.5	2.4	0.1	3.0

Average dynamic creep for different types of asphalt mixes are shown in Figure 4.7. From the data presented, it can be seen that the asphalt mixes that exhibited high resilient modulus generally had high dynamic creep (compressive stress) and low minimum slope of strain. The average compressive stress for AC20-75 Blow asphalt mix was 200.2 kPa while minimum slope of strain was 2.9 um/m/c. AC14-75 Blow asphalt mix had an average compressive stress of 199.7 kPa and minimum slope of strain of 3.6 um/m/c, and made the asphalt mix in a second rank. The lower minimum slope at the accumulated strain at end of 5271 cycles indicated that high asphalt resistance to permanent deformation of asphalt mix. As regard the dynamic creep (compressive stress), asphalt mix that showed high value than other mixes is high resistance to rutting. However, SMA7-50, AC7-50, and AC14-50 Blow asphalt mix had high minimum slope of strain of 11.5, 5.9, and 10.3 um/m/c, respectively. These indicated that low asphalt resistant to permanent deformation of asphalt pavement.

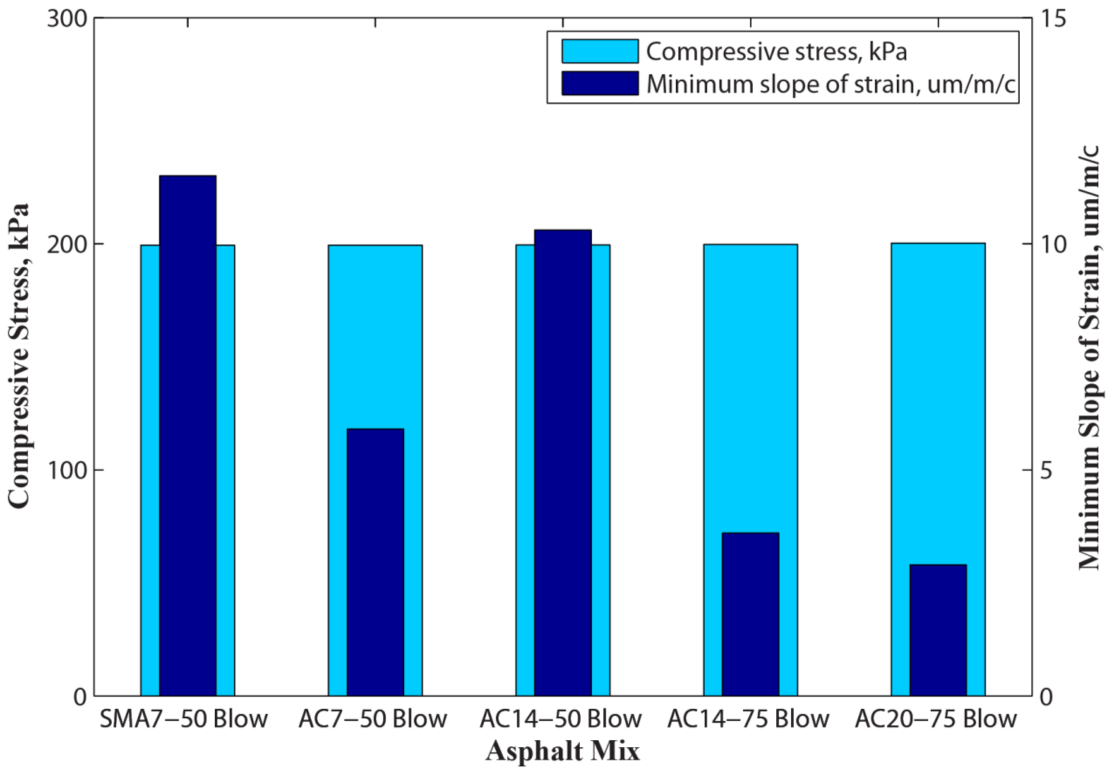


Figure 4.7 Dynamic Creep (Average Compressive Stress and Minimum of Slope of Strain) of Different Types of Asphalt Mixes

Average wheel tracking test for different types of asphalt mixes is shown in Figure 4.8. The analyses indicated that AC20-75 blow asphalt mix had low rut depth of 1.9 mm as compared to other asphalt mixes. AC14-75 blow was the second in rank with a rut depth of 2.4 mm. This showed that these asphalt mixes are high rut resistance of asphalt mixture and less to pavement distress and asphalt fatigue cracking. However, the rut depth observed for SMA7-50 blow was 15 mm after 8, 452 cycles while 5 mm and 4.2

mm for AC7-50 blow and AC14-50 blow after 10,000 cycles, respectively. This indicated that SMA7-50 blow asphalt mix has high pavement distress and low rut resistance of asphalt mixture. There was also a sudden steep change in slope after 8,000 cycles for SMA7-50 blow. This may be attributed to the stripping of aggregate. No stripping was however, observed after 10, 000 cycles to other asphalt mixes.

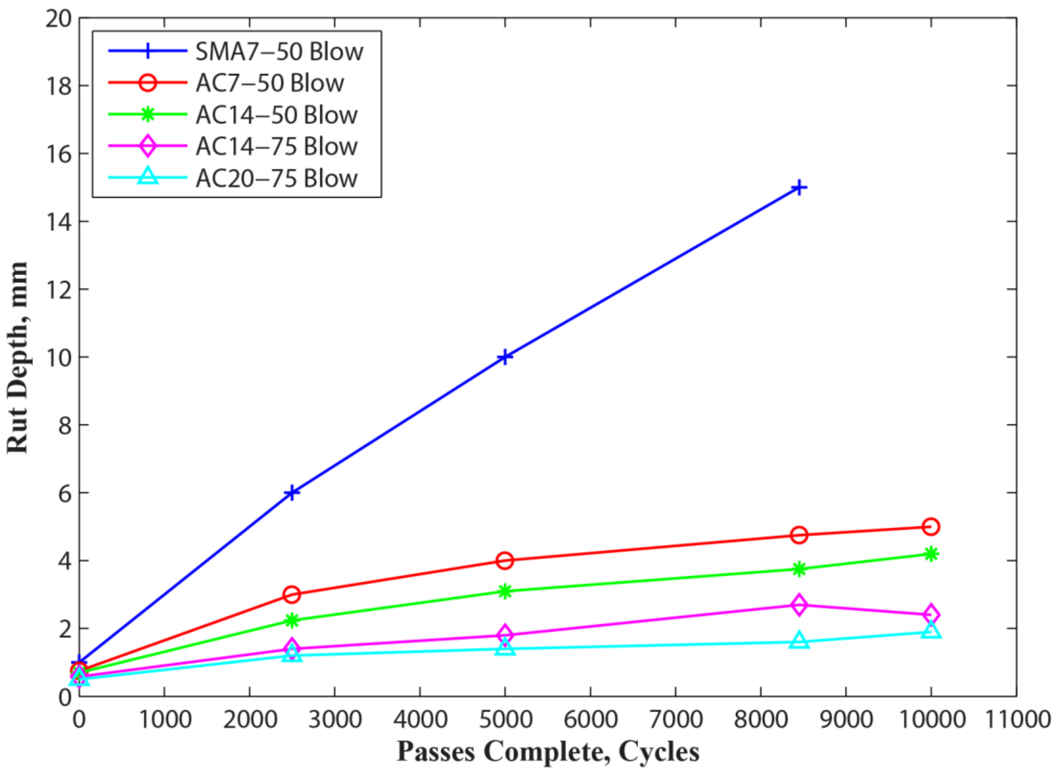


Figure 4.8 Average wheel tracking tests for different types of asphalt mixes

A summary of aggregate gradations along with specific limit achieved for different types of asphalt mixes are shown in Table 4-7 and Figure 4-9. From the date presented, it can be seen that SMA7-50 and AC20-75 Blow asphalt mixes had high rank in aggregate gradation as compared to other asphalt mixes. This shows that the mixes are high stable in pavement performance, and also high resistance to rutting. However, AC7- 50, AC14-50, and AC14-75 have passed a restricted zone superimposed on the maximum density between a line 2.36 mm and 0.3 mm sieve size through gradation are not recommended to pass. This indicated that the mixes have low stability and low resistance to rutting. Kandhal and Mallick (Kandhal & Mallick 2001) noted that Superpave has introduced a restricted zone superimposed on the maximum density between a line 2.36 mm and 0.3 mm sieve sizes, through which gradation are not recommended to pass. It is believe that gradation passing through this zone can have low stability or resistance to rutting.

Table 4.7 Aggregate Gradation and Their Proportions Along With Specific Limits of Different Types of Asphalt Mixes

Sieve Size (mm)	Percentage Passing by Weight					Austroad Spec. Limits
	SMA7-50 Blow	AC7-50 Blow	AC14-50 Blow	AC14-75 Blow	AC20-75 Blow	
26.5	100	100	100	100	100	100
19	100	100	100	100	98.2	100
13.2	100	100	98.1	96.9	78.3	100
9.5	100	100	80.1	80.8	61.2	100
6.7	97.6	98.3	63.5	71.1	50.9	85-100
4.75	35	83.1	52.8	57.1	45	30-63
2.36	22.5	57.6	39	38.8	35.8	20-35
1.18	18.5	41.89	28.3	27.01	25.54	16-28
0.6	15.75	31.4	20.07	18.56	17.95	14-24
0.3	13.06	18.37	12.96	11.58	11.95	12-20
0.15	10.83	8.59	7.74	6.57	6.34	10-16
0.075	9.01	4.76	4.57	3.65	3.91	8-12

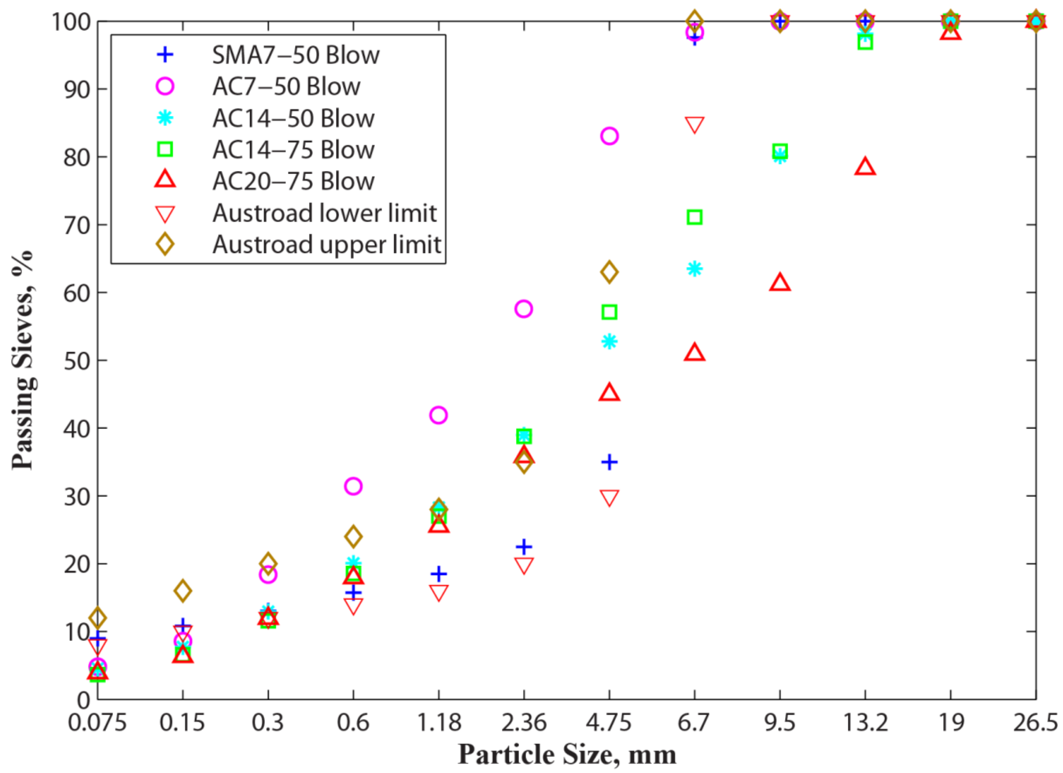


Figure 4.9 Aggregate Grading Curve for Different Types of Asphalt Mixes

4.5. CONCLUSIONS

The pavement materials performance for strength and durability of flexible pavement was assessed and analyzed using the evaluation and validation of characterization methods for fatigue performance of different types of asphalt mixes for Western Australia road.

The comparison of the different types of asphalt mixes using a standard laboratory tests methods and techniques indicated that AC20-75 blow asphalt mix method is the most efficient and effective in all categories of engineering characterization and variability of asphalt pavement as compared to other asphalt mixes. AC14-75 blow asphalt mix was evaluated as the second in rank that increases the frictional contact between aggregate particle and overall stiffness and stability of the asphalt mixture.

In general, all the asphalt mixes that are used in this research study can be used to strength and stable the mixture stiffness of asphalt that is notable. The modification effect rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow in this research. Aggregate particles that are in closed contact and interlocked with each other are able to resist traffic load and environmental factor with lower strain, and hence are stable, durable and stiffer.

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CHAPTER 5: COMPARISON OF PAVEMENT NETWORK MANAGEMENT TOOLS AND ITS PROBABILISTIC OF PAVEMENT ENGINEERING FOR WESTERN AUSTRALIA

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ABSTRACT

Since the Association of American State Highway Officials (AASHO) road test of 1956-62 at Ottawa in Illinois, enormous efforts have been devoted to improve the methodologies and engineering techniques of pavement performance predication. For instance, the successful implementation of the Network Optimization Systems (NOSs) in the Arizona Department of Transportation (ADOT) in the 1980-82 was one of a tremendous effort that represented advancement in predication methodology and engineering technique by using Markov Chain-Process based to define the transition process of pavement network condition. The main role of this paper is to evaluate and analysis the pavement network performance of Western Australia (WA) and also applied the existing pavement management tools relevant to WA road networks. Two approaches were used to evaluate and analysis the pavement network of WA. First, the current pavement performance data was used to assess the State road networks and then, predict the future from the past and current pavement network data. Second, the Probabilistic network – Markov-Chain Process and Chapman-Kolmogorov method was used to predict the pavement behavior in Western Australia. The results showed that the pavement performance of the predicting model using probabilistic network process (i.e. Linear) perform well in all categories as compared to the past 30 years LRDM data inventory. This study will draw into appropriate and effective pavement engineering management system to account for proper pavement design, preliminary planning, future pavement M & R networks, service life and functionality.

Author keyword: Pavement engineering; pavement management; Markov chain-process; pavement network-probabilistic behavior; Western Australia

5.1. INTRODUCTION

Pavements are an important part of highway transportation infrastructure that constitutes an enormous investment of public funds. A tremendous amount of time and money is spent each year on construction of new pavements as well as on maintenance and rehabilitation (M&R) of existing pavement. To maximize benefits and minimize overall costs, a systematic and scientific approach is needed to manage pavements (Lee, Park & Mission 2013). Pavement management systems (PMSs) provide consistent, objective, and systematic procedures to determine priorities, schedule allocating resources and budgeting for pavement M&R (Federal Aviation Administration 2006). The management of pavement is a decision making action with uncertainties. Though we had unlimited resources, there is not always agreement on the best approach and time of pavement maintenance and rehabilitation. Since resources are invariably severely constrained, the difficulty of the decision making is almost overwhelming. However, over the last six decades hard and soft technology has emerged which provide us with improved decision making decision making ability such as microcomputers and decision support system (DSS), soft technology, operation research methodologies, artificial intelligence technologies, modeling aids including spreadsheet program and data base systems for the management of information and so forth.

Pavement management was first conceived in the mid-1960s as a result of major work done in the United State and Canada (Hudson & Hudson 1994). The early pavement management concepts focused on the project level, design, rehabilitation, maintenance, and pavement performance modeling. By the mid-1970s pavement management had expanded to primary use at the networks varying-sizes. By the mid-1980s the application of these same system concepts to bridge became evident to engineering community, and a major research project was funded by the National Cooperative Highway Research Program (NCHRP) to develop bridge management systems (Hudson & Hudson 1994). Since then bridge management has become relatively commonplace. Work has been performed by other on building management system, sewage management system and other systems for managing the world's infrastructure. However, pavement management systems based on engineering knowledge are limited.

Pavement engineering management systems uses the systems approach to provide a unified treatment of pavement design, testing, construction, maintenance, evaluation, and restoration (Haas & Hudson 1978). Improving road safety through proper pavement engineering and maintenance should be one of the major objective of pavement management systems (Tighe et al. 2000). When pavement are evaluated in terms of safety, a number of factor related to pavement engineering properties are raised, such as pavement geometric design, pavement materials and mix design, pavement surface properties, shoulders type and pavement color and visibility (Tighe et al. 2000). A good pavement engineering management system requires an accurate and efficient pavement performance (Butt et al. 1987) so that prediction models based on the Pavement Condition Index and the age of the pavement can be developed.

The objective of this study is to evaluate and analysis the pavement network of Western Australia roads using a pavement management data and also predict the likely pavement maintenance and rehabilitation (M&R) performance of the State using probabilistic model.

5.1.1. BACKGROUND

Most highway agencies are now engaged in the development implementation, and operation of pavement management systems. The Guideline for Pavement Management Systems was published by American Association of State Highway and Transportation Officials (AASHTO) in July 1990 (AASHTO1990), contained information needed for establishing frame work for pavement management systems. However, this document does not address the day-to-day issue uncounted by pavement engineers or the issue associated with new and emerging technologies. Transportation is essential for a nation’s development and growth. In both the public and private sector, opportunities for engineering careers in transportation are exciting and rewarding. Elements are constantly being address to the world’s highway, rail, airport, and mass transit system safety and economically (Garber & Hoel 2009). Many organization agencies exist to plan, design, build, operate, and maintain the nation’s transportation system.

A pavement management system was developed for the state of Arizona to produce optimal maintenance policies for each mile of 7, 400 mile network of highway (Golabi, Kulkarni & Way 1982). The heart of the system is a mathematical model which captures the dynamic and probabilistic aspect of pavement maintenance. The model integrates management policy decision, budgetary policies, environmental factors, and engineering decisions. During the first year of implementation fiscal year 1980-81, the system saved 14 million dollars (Golabi, Kulkarni & Way 1982). According to Abaza, Ashur and Al-khatib (2004), integrated pavement management has been designed to provide the pavement engineers with an effective decision-making tool for planning and scheduling of pavement maintenance and rehabilitation (M&R) work. The developed system applies a discrete-time Markovian model to predict pavement deterioration with the inclusion of pavement improvement resulting from M&R actions.

Long-term collection of pavement M&R data is essential so that the deterioration and performance trends can be established (Simpson & Thompson 2006). Pavement predication performance models are used to analysis the condition expressed in the form of a quality or quality index (Michail & Patrick 1998) and determine M&R requirements as mentioned by (Ismail, Ismail & Atiq 2009). When the pavement is only somewhat degraded, it is then the least expensive approach. If too much deterioration is permitted before repair is initiated, the result is greater cost over the entire planning horizon and the maintenance of deteriorating can be a great challenge.

Pavement represents gradually deteriorating structures for which observations of advance signs of impending failure are possible. Most agencies collect pavement condition data on a regular basis to identify the sign. However, neither of this time of occurrences of these sign not timing of actual failure following the signs can be predicted with certainty. As the result of these issues, a dynamic decision model is much more appropriate for pavement management decision as the selection of cost-effective pavement preservation action and forecasting of future performance of a highway network (Kulkami 1984).

Modeling of payment performance is absolutely essential to pavement management on all levels: project level to national network level. Performance, it its broadest sense, is predicted by deterministic and probabilistic models (Lytton 1987). The deterministic models include those for predicting primary response, structural, functional and damage performance of pavements, whereas the probabilistic model include such as survivor curves, Markov and Semi-Markov transition process (Lytton 1987). Damage models are particularly important because they are impact load equivalence, cost allocation, and a variety of the other tax related subjects. A cumulative damage model based upon a Markov process has been proposed by Bogdanoff and Kozin (1985) and has been successfully used to investigate crack propagation and fatigue. The Markov process seems superior to the curve-fitting approaches mentioned above because it introduce a rational structure for interpreting road condition data. It can also be used to predict future pavement condition in a probabilistic manner (Carnahan et al. 1987).

Lytton (1987) underlined the concepts of pavement performance prediction and modeling including the limitation and the use of the models. Project level models are different from and more detailed than network level models for they are used in the analysis and design of pavements of life-cycle cost analysis of alternative designs and other related purpose. While Network level models are necessary less detailed but used in the selection of optimal maintenance and rehabilitation strategies, size and weight and cost allocation studies, and network level trade-off analysis between pavement damage, maintenance, and other user costs. Well-developed performance models, resting on the twin pillars of statistics (i.e. experimental design) and mechanics can satisfy both technical and economical requirement for managing pavement based on engineering (Lytton 1987). The development of better performance models should be a continuing tasks and much remain to clone.

The damage equation used at the AASHO Road Test is of the form (Lytton 1987):

$$g = \frac{p_i - p}{p_i - p_t} = \left(\frac{W}{\rho}\right)^{\beta} \tag{5.1}$$

where, g = the "damage" index after the passage W standard loads or equivalent standard loads; p_i = the initial serviceability index; p_t = the terminal or unacceptable level of serviceability; p = the serviceability index after the passage of W standard load; W = the number of standard loads or equivalent standard load and ρ, β = constants which depend upon the structural design of the pavement, the stiffness of the subgrade, management of the load, and the climate.

$$g = \frac{a}{a_t} \tag{5.2}$$

$$g = \frac{s}{s_t} \tag{5-3}$$

where, a = percent of the total area of a pavement that is distressed; a_t = 'terminal' or maximum acceptable percent of total area of distressed pavement; s = the severity level and s_t= 'terminal' or maximum acceptable severity of distress. A performance equation (structural or functional) can be converted to equation by dividing it by the range of acceptable values of distress or serviceability index. The number of load application of this load, N_{18}, which cause a level of damage, g is given by

$$N_{18} = \rho_{18}(g)^{1/\beta_{18}} \tag{5.4}$$

5.1.1.1 EQUIVALENT FACTORS

A load equivalent factor is a ratio of the number of load applications of a standard load to cause a defined level of damage to the number of load applications of another load to cause the same level of damage. In the AASHO Road Test, the standard load was established to be 18-kip single axle load. Similarly, the number of load applications of another, j, to cause the same level of damage is:

$$N_j = \rho_j (g)^{1/\beta_{18}} \tag{5.5}$$

The load equivalent factor for second load is defined as

$$(L.E)_j = \frac{N_{18}}{N_j} \tag{5.6}$$

5.1.1.2 MARGINAL LOAD EQUIVALENT FACTORS

Markow (1986) suggested that load equivalent factors should be calculated based upon a marginal damage concepts, marking the rang from 0 to 1 be between any two pre-determined levels of distress or serviceability index (Lytton 1987). The load equivalent factors for the same vehicle will change as the pavement become more distressed, and will also depend upon the presence of other types of distress and upon timely maintenance action. The objection may be made that this will make the calculations of load equivalence factors more difficult and pavement design for mixed traffic more complicated but the reply is that it will also made it more realistic.

5.1.1.3 SURVIVOR CURVES

Survivor curves are used for planning maintenance and rehabilitation alternative on pavement networks (Lytton 1987). The construction, maintenance, and rehabilitation histories that are recorded from which to develop survivor curve. Lytton (1987) defined a survivor curve as a graph of probability versus time. The probability drops off with time (or traffic) from a value of 1.0 down to zero and it expresses the percentage of pavement that remain in service after a number of years (or passes of standard load) without requiring major maintenance or rehabilitation. The probability density curve for survival may be constructed from historical data by deforming the percentage of pavements that must be maintained or rehabilitated each year after its resent major repair or new construction. The survivor curve and probability density function for survival is shown in Figure 5.1.

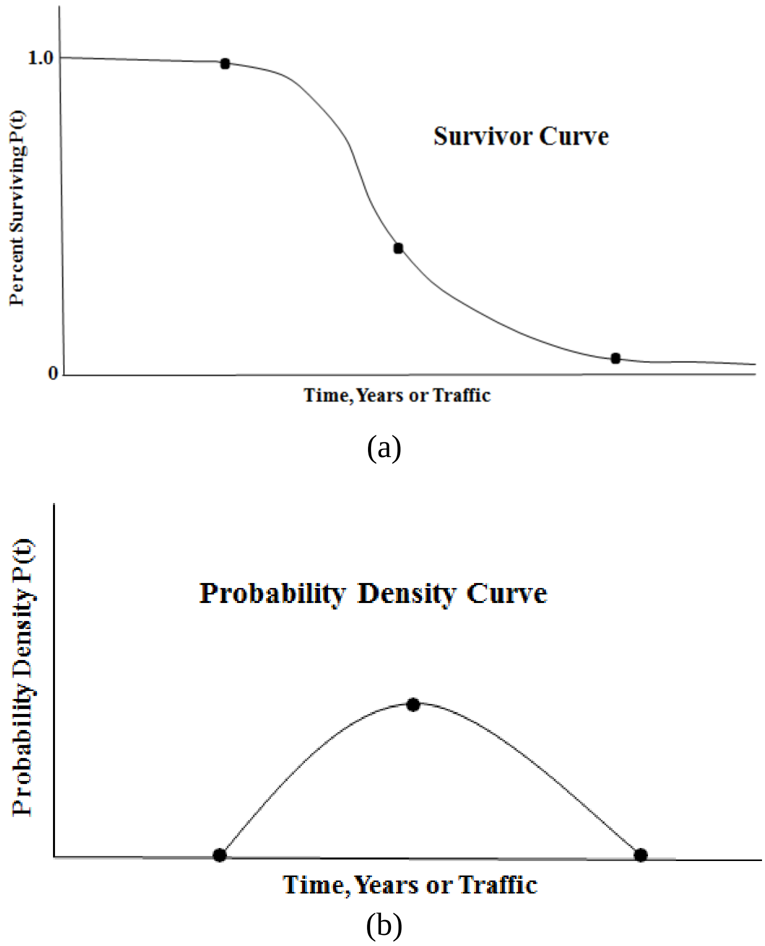


Figure 5.1 Survivor curve and probability density function for survival

5.2. METHODS AND TECHNIQUES

5.2.1. LRDM INVENTORY

Pavement management data's are collected from Lao Road Design Manual (LRDM) inventory from seven roads categories on sixty seven locations of Western Australia roads networks. Pavement data are for the past thirty years on traffic roads survey, and it includes: surface type, construction and maintenance and rehabilitation (M&R) year, average daily traffic (ADT), heavy vehicle in road (HV, %), Thornwaite Moisture Index (TMI), and pavement cracking types.

5.2.2. PAVEMENT NETWORK MANAGEMENT TOOLS

Linear and non-linear programming models are the two main types of algorithms utilized by researchers in developing pavement management optimization models (Gao, Chou & Wang 2010). In linear programming models, key assumptions of all functions that includes objective and constrain function are consider as linear. However, in non-linear programming, this assumption does not accumulate at all (Hillier & Lieberman 2010). Pavement condition prediction models are significant component of pavement optimization models. These are two types of prediction models: deterministic models and probabilistic models. According to Butt et al. (1994), the pavement deterioration rates are often "uncertain", frequently used the probabilistic model based on the Markov process approach to evaluate and analysis the pavement condition (Chen et al. 1996).

5.2.2.1. LINEAR MODEL ALGORITHM

The linear model for pavement maintenance and rehabilitation optimization is formulated as follows (Gao, Chou & Wang 2010):

$$\sum_{t=1}^{T} \sum_{k`=1}^{K} \sum_{i=1}^{5} \sum_{k=0}^{K} Y_{tk`ik} \cdot C_{tk`ik} \tag{5.7}$$

$$\sum_{k=0}^{K} Y_{tk`jk} = \sum_{i=1}^{5} \sum_{k=1}^{K} Y_{(t-1)kik`} \cdot P_{k`ij} + \sum_{i=1}^{5} Y_{(t-1)k`io} \cdot DN_{k`ij},$$

for all t = 2, . . T: k' = 1, . . K; j = 1, . . I and $Y_{tk'k} \geq 0$ for all t = 1, . .T; k' = 1, . . K; I = 1, . I; k = 0, K (5.8)

$$\sum_{t=1}^{K} \sum_{k`=1}^{5} \sum_{k=0}^{K} Y_{tk`jk} = 1 \; for \; all \; t = 1, \dots, T \; ; \; S_{Tj} \leq \varepsilon_{Tj} \; for \; selected \; j \leq \varepsilon_{Tj \; for \; selected \; j} \tag{5.9}$$

$$\sum_{t=1}^{T} \sum_{k`=1}^{K} \sum_{i=1}^{5} \sum_{k=0}^{K} Y_{tk`ik} \cdot C_{tk`ik} \cdot L \leq \beta_t \; for \; all \; t = 1, \dots, T \tag{5.10}$$

where, $Y_{tk`ik}$ = proportion of pavement in state I with lost treatment k` receiving new treatment k in year t; $C_{tk`ik}$ = unit cost of applying treatment k in year t to pavement in state I with lost treatment k`; P_{kij} = probability that pavement receiving new treatment k transit from state i to state j; $DN_{k`ij}$ = probability that pavement with new treatment k` receiving no new treatment (don nothing moves from state i to j; S_{Tj} = proportion of pavement in state j at year at final T; ε_{it} = upper limit of proportion of pavement in condition I in year t; L = total length of entire pavement network; β_t = maximum available budget in year t; T = number of analysis years; and K = number of repair treatment types.

5.3. COMPARISON OF PAVEMENT MANAGEMENT OPTIMIZATION MODELS

Linear and non-linear programming models are the two main types of algorithms utilized by researchers in developing pavement management optimization models (Gao, Chou & Wang 2010). In linear programming models, key assumptions of all functions that includes objective and constrain function are consider as linear. However, in non-linear programming, this assumption does not accumulate at all (Hillier & Lieberman 2010). Pavement condition prediction models are significant component of pavement optimization models. These are two types of prediction models: deterministic models and probabilistic models. According to Butt et al. (1994), the pavement deterioration rates are often “uncertain”, frequently used the probabilistic model based on the Markov process approach to evaluate and analysis the pavement condition (Abaza & Ashur 1999; Chen et al. 1996; Golabi, Kulkarni & Way 1982).

5.3.1. LINEAR MODEL ALGORITHM

The linear model for pavement maintenance and rehabilitation optimization is formulated as follows (Gao, Chou & Wang 2010):

$$\sum_{t=1}^{T} \sum_{k`=1}^{K} \sum_{i=1}^{5} \sum_{k=0}^{K} Y_{tk`ik} \cdot C_{tk`ik} \tag{5.11}$$

$$\sum_{k=0}^{K} Y_{tk`jk} = \sum_{i=1}^{5} \sum_{k=1}^{K} Y_{(t-1)kik`} \cdot P_{k`ij} + \sum_{i=1}^{5} Y_{(t-1)k`io} \cdot DN_{k`ij}$$

for all t = 2, . , T; k` = 1, . , K; j = 1, . . ., I (5.12)

$Y_{tk`k} \geq 0$ for all t = 1, . . , T; k` = 1, . . , K; I = 1, . . . , I; k = 0, . . , K (5.13)

$$\sum_{t=1}^{K}\sum_{k`=1}^{5}\sum_{k=0}^{K} Y_{tk`jk} = 1 \; for\; all\; t = 1, \ldots, T \tag{5.14}$$

$$S_{Tj} \leq \varepsilon_{Tj} \; for\; selected\; j \tag{5.15}$$

$$\sum_{t=1}^{T}\sum_{k`=1}^{K}\sum_{i=1}^{5}\sum_{k=0}^{K} Y_{tk`ik} \cdot C_{tk`ik} \cdot L \leq \beta_t \;\; for\; all\; t = 1, \ldots, T \tag{5.16}$$

where, $Y_{tk`ik}$ = proportion of pavement in state I with lost treatment k` receiving new treatment k in year t; $C_{tk`ik}$ = unit cost of applying treatment k in year t to pavement in state I with lost treatment k`; P_{kij} = probability that pavement receiving new treatment k transit from state i to state j; $DN_{k`ij}$ = probability that pavement with new treatment k` receiving no new treatment (don nothing moves from state i to j; S_{Tj} = proportion of pavement in state j at year at final T; ε_{it} = upper limit of proportion of pavement in condition I in year t; L = total length of entire pavement network; β_t = maximum available budget in year t; T = number of analysis years; and K = number of repair treatment types.

5.3.2. NON-LINEAR MODEL ALGORITHM

The non-linear programming model for pavement maintenance and rehabilitation optimization can be formulated as follows (Gao, Chou & Wang 2010):

$$\sum_{t=1}^{T}\sum_{j=1}^{5} S_{tj} X_j L C_J \tag{5.17}$$

$$S_{tj} = \sum_{i=1}^{5} S_{t-1i}\left\{(1 - X_i) DN_{ij} + X_i P_{ij}\right\} \tag{5.18}$$

$$X_i \geq 0 \text{ for all I} = 1, \ldots, 5 \tag{5.19}$$

$$\sum_{k=9}^{4} X_{ik} = 1 \; for\; all\; i = 1, \ldots, 5 \tag{5.20}$$

$$STj = \leq \varepsilon_{Tj} \text{ for selected j} \tag{5.21}$$

$$\sum_{j=1}^{5} S_{tj} X_j L C_j \leq \beta_t \; for\; t = 1, \ldots, t \tag{5.22}$$

where, S_{tj} = proportion of pavement in j at year t; Xi = proportion of pavement in state i receiving treatment; T = number of analysis year; L = total length of entire pavement network; C_j = unit cost of applying treatment to pavement in state j; DN_{ij} = probability that pavement receiving no treatment (do nothing) moves from state i to state j; P_{ij} = probability that pavement receiving new treatment transit from that i to state j; ε_{Tj} = upper limit of proportion of pavement in a condition j in final year T; and β_t = maximum available budget in year t.

5.3.3. PROBABILISTIC BEHAVIOR OF PAVEMENTS

Since the AASHO road test of 1962, tremendous efforts have been devoted to improve the methodologies of pavement performance prediction. The successful implementation of Network Optimization System (NOS) in the Arizona Department of Transportation (ADOT) in the 1980s represented on advancement in the prediction methodology by using Markov-process based on Transition Probability matrices (TPMs) to define the transition process of pavements condition.

Pavement serviceability and its prediction based on engineering data, were initiated at the AASHO (1962) road test. Since the road test, extensive research has been conducted in predicting pavement performance in order to quantitatively establish the relationship among pavement serviceability, traffic road, age, and rehabilitation action (Wang, Zaniewski & Way 1994). The development of the award winning Network Optimization System (NOS) by Woodward-Clyde Consultant (WCC) in 1990 for Arizona Department of Transportation was a major pioneering effort to optimize the highway preservation program by combining the pavement performance prediction technique of Markov process with linear programming (Wang, Zaniewski & Way 1994). It was the first successful implementation of sophisticated mathematical models based on large scale optimization and stochastic prediction techniques for pavement management system. During the first year of implementation of NOS, the system saved 14 million dollars for the State of Arizona (Golabi, Kulkarni & Way 1982; Wang, Zaniewski & Way 1994). It has been used since then to develop the annual preservation budget and prepare for the schedule of projects.

5.3.3.1. THEORETICAL BACKGROUND

The Markov Process is a time- independent stochastic description of event development. ADOT`s pavement management system modeled pavement behavior with the Markov Process in 1980 (Kulkarni et al. 1980; Wang, Zaniewski & Way 1994). Subsequently a number of pavement management agencies have implemented Markov-Process based prediction models such as Feighan et al. (1988) for PAVER; Davis and Dine (1988) for Connecticut DOT, Alaska DOT's NOS, and Kansas DOT`s NOS based on An Advanced (1991); Thompson et al. (1987) for Finland; and Harper and Majidzaeh (1991) for Saudi Arabia (Wang, Zaniewski & Way 1994).

The Markovian property is equivalent to stating that conditional probability of the future event, given any past event and the present state, is independent of the past event, given any past even and the present state, is independent of the past even and depends upon only the present state of the process (Hillier & Lieberman 1990). The conditional probability for the process to transition from one state to another is called transition probability. The transitions is also called steps (Wang, Zaniewski & Way 1994). Thus the n-step transition probability $P_{ij}^{(n)}$ (Equation 5.23) is defined as the conditional probability that the random variable X, starting is state I, will be in the state j after exactly n steps (time units) and $P_{ij}^{(n)}$ has to satisfy the following equation by (Wang, Zaniewski & Way 1994):

$$P_{ij}^{(n)} \geq 0, for\ all\ i\ and\ j, n = 0, 1, 2, \ldots \quad (5.23)$$

$$\sum_{j=0}^{M} P_{ij}^{(n)} = 1\ \ for\ all\ i, and\ n = 0, 1, 2, \ldots \quad (5.24)$$

where, i and j both defined within the same (m+1) – state space. From equation (5.23) and (5.24), a convenient notation for representing the transition probabilities is the matrix form as:

$$P^{(n)} = \begin{bmatrix} P_{00}^{(n)} \ldots P_{0M}^{(n)} \\ \vdots \qquad \vdots \\ P_{M0}^{(n)} \ldots P_{MM}^{(n)} \end{bmatrix} \quad (5.25)$$

where, $P^{(n)}$ = transition probability matrix (TPM). As applied in ADOT NOS and others countries the transition process of the pavement condition states conforms that to the finite state Markov-chain process. Future pavement condition is dependent only on the current condition. The performance model used in the NOS is based on the transition-probability matrices. A transition probability $P_{ij}(a_k)$ is assumed to be the same as the proportion of roads in the state I that move state j in one year if the k^{th} rehabilitation action is applied. It defines the probability of transition from one condition state to another in one year under one of the rehabilitation actions, including routine maintenance. Separate TPM, are used for each of the---- road categories in Western Australia. Chapman-Kolmogorov equation (Hillier & Lieberman 1990) provide a method for computing the n-step transition probability matrix a single-step transition-probability matrix as used in NOS (Wang, Zaniewski & Way 1994):

$$P_{ij}^{(n)} = \sum_{k=0}^{M} P_{ik}^{(v)} . P_{kj}^{(n-v)} \; for\ all\ i, j, n, and\ 0 \leq v \leq n \qquad (5.26)$$

The matrix of n-step transition probabilities can be obtained by multiplying matrices of one-step transition probabilities as discussed by Wang et al. (1994). It is then becomes clear by looking at special cases when v = 1, that the n-step transition probabilities can be obtained from the one-step transition probabilities recursively.

$$P^{(n)} = P.P...P = P^n \qquad (5.27)$$

The transition probabilities of pavement condition for a period of n years can obtained based on the existing one-step transition probabilities of pavement condition.

5.3.3.2. MODELING OF PAVEMENT PROBABILISTIC BEHAVIOR

A traditional performance model is illustrated by the performance curve shown in Figure 5.2. The deterioration of pavement start from the best condition state, when it is new, until the serviceability of the pavement is not acceptable and rehabilitation is required (Wang, Zaniewski & Way 1994). Condition is improved with a rehabilitation action, and then deterioration resumes. According to Haas and Hudson (1978), a two performance predication model is one that can calculate the expected serviceability-age (or traffic) relationship over the entire design period. Pavement behavior modeling based on the Markov process and the Chapman-Kolmogorov equation provides an alternative to demonstrate pavement long-term behavior. The transition probabilities for n number of periods or years can be calculated using equation (5.27) by multiplying the one-steps or the original TPM n times (Wang, Zaniewski & Way 1994). Whereas, pavement behavior can be determined for entire design period even if rehabilitation action (s) are applied during this period of time by using equation (5.26). Assume that the design period for the pavement is from 0 to N. At time v, one rehabilitation action is applied. The pavement probabilistic behavior is as follows:

$$P_{ij}^{(n)} = \sum_{k=0}^{M} P_{ik}^{1} \cdot P_{kj}^{(n-1)} \ \ when\, n \leq v \tag{5.28}$$

$$P_{ij}^{(n)} = \sum_{i=0}^{M} \left(\sum_{k=0}^{M} P_{ik}^{v} \cdot P_{kl}^{(1)a} \right) \cdot P_{lj}^{(n-v-1)}\, , when\, n > v$$

for all i, j, l, n, and $0 \leq v \leq N$, $0 \leq n \leq N$. (5.29)

Where $P_{ij}^{(n)} =$ n-step transition probability from condition state i to j when n = N, it is the transition probability for entire design period; M+1 = total number of pavement condition states; N = design period, v = period when the rehabilitation is applied P_{ik}^{v} = v-step transition probability from condition state l to k under routine maintenance. A rehabilitation action is applied after the v steps or periods; $P_{kl}^{(1)a}$ = one- step transition probability from condition k to l at period v. It is the probability concerning the effectiveness of rehabilitation action to improve the serviceability of the pavement immediately after the application; and $P_{lj}^{(n-v-1)}$ = (n-v-1) step transition probability from condition l to j under routine maintenance. It is the transition probability after rehabilitation.

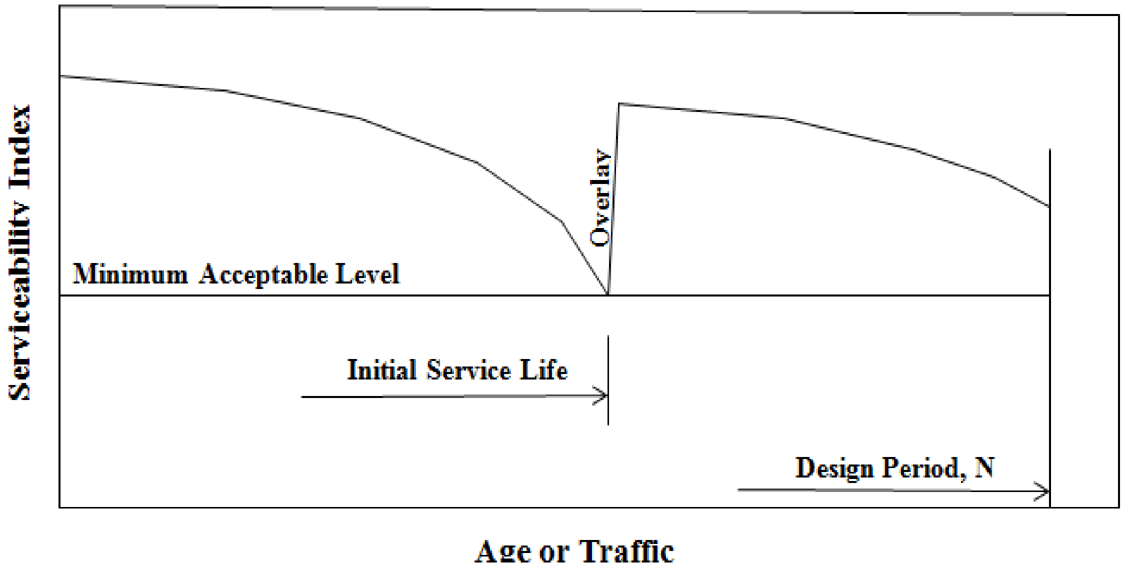


Figure 5.2 Pavement Performance and Prediction

It should be noted that Equation (5.28) and (5.29) can be easily expanded to analyses pavement probabilistic behavior where more than one rehabilitation action are applied. The following pavement probabilistic behavior equation for one rehabilitation action is vector form can be established (Wang, Zaniewski & Way 1994):

$$P^{(n)} = \begin{cases} P_{routine}^{(n)}; & n \leq v \\ P_{routine}^{(v)} \cdot P_{rehab}^{(1)} \cdot P_{after\,rehab}^{(n-v-1)}; & n > v \end{cases} \tag{5.30}$$

where, $P^{(n)}$ = n-step TPM; $P_{routine}^{(n)}$ = n-step TPM before the rehabilitation when n < v; $P_{routine}^{(v)}$ = v –step TPM when the rehabilitation is applied; $P_{rehab}^{(1)}$ = one-step TPM based on the effectiveness of the rehabilitation at the period of v immediately after the application; and $P_{after\,rehab}^{(n-v-1)}$ = (n-v-1) step TPM after rehabilitation. As shown from equation (5.28) through (5.30), three TPMs are needed to conduct the analysis of long-term probabilistic behavior for entire design period until one rehabilitation is applied (Wang, Zaniewski & Way 1994). The data generated based on Equation (5.30) can be used to plot pavement probabilistic behavior curve (PPBC). The pavement probabilistic behavior curve (PPBC) is defined as the probability of being in a given condition state over time. Thus each condition state can have its own set of PPBC. Figure 5.3 illustrate typical long-term pavement probabilistic behavior curve of design period N for interstate pavement condition state. The vertical axis represents the probability of pavement remaining in the best condition.

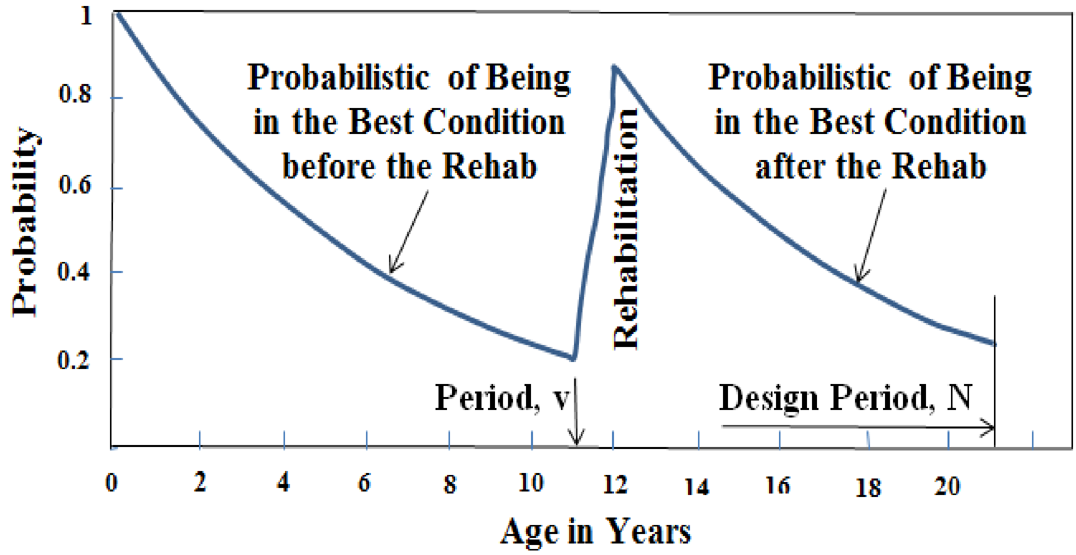


Figure 5.3 Typical Pavement Probabilistic Behavior Curves for Design Period

Karan and Haas (1976) presented a Markov model for predicting pavement performance

$$v(n) = v(0) \cdot M^n \tag{5.31}$$

where, $v(n)$ = predicted condition state matrix at year n; $v(0)$ = initial condition state matrix at year 0; M = one-step TPM. Feighan et al. (1988) presented a similar relationship between initial condition state and the prediction condition state (Wang, Zaniewski & Way 1994). Both studies by Karan and Feighan use the technique of matric multiplication to derive n-step TPM, However, the prediction presentation and analysis are not mentioned in detailed in either reference, The relationship displayed in Equation (5.31) can be obtained from the NOSs multi-period transition equation (Kulkarni et al. 1980; Wang, Zaniewski & Way 1994).

$$\sum_k w_{j,k}^l = \sum_{i,k} w_{i,k}^{l-1} \cdot p_{ij}(a_k); \;\; for\ 1 < l \leq T \tag{5.32}$$

where, $w_{j,k}^l$ = proportions of roads of a given road category that condition state j at the beginning of lth time period of horizon T, and to which the k preservation action is applied; $w_{i,k}^{l-1}$ is similarly defined; $p_{ij}(a_k)$ = pavement transition probability from condition from condition state i to j due to the rehabilitation action k. Assume no rehabilitation action is applied after period 1, k = 1 (routine maintenance) for the analysis period T. Then we can have the equation as follow:

$$\sum_k w_{i,1}^l = \sum_i w_{i,1}^{l-1} \cdot p_{ij}(1) \quad (5.33)$$

where, $p_{ij}(1)$ = one-step transition probability from condition state i to j under routine maintenance. Equation (5.33) can be converted to vector form:

$$w^l = w^{l-1} \cdot P(1) \quad (5.34)$$

where, w^l, w^{l-1} = vectors of proportions of pavement in each condition state at period l and l-1; and $P(1)$ = one-step TPM under routine maintenance. Equation (5.33) and (5.34)) are recursive. Therefore, the equation can be written as:

$$w^l = w^0 \cdot P^l(1) \quad (5.35)$$

where, w^0 = vector of pavement proportion in their condition states. Prediction model based on the Markov is process in NOS is an aggregate prediction method based on statistical assumption.

5.4. PAVEMENT MAINTENANCE AND REHABILITATION

The most important, and priorities decisions that can be made is to rehabilitate the existing road rather than having selection to be build new roads. Pavement management includes all planning, design, construction, maintenance, and rehabilitation of pavement part of a public work program. Whereas, a pavement management system is a set of methods which support the decision makers in arriving at the optimal strategies for the construction and maintenance of traffic roads in serviceable condition for a given period of time (Hudson, Haas & Pedigo 1979). A pavement management is nothing more than a decision support system in the terminology of operation research (Hugo et al. 1989).

Several pavement management systems have been described in the literature. For example, some use integer goal programming (Cook 1984), linear goal programming (Benjamin 1985), linear programming (Karan & Haas 1976), linear integer programming (Garcia-Diaz & Liebman 1980; Lytton, Phillips & Shanmughan 1982) and others. However, all these approaches fail when the special circumstances surrounding the problem and discussion are taken into account. Integral and integer goal programming problems cannot be solved on a microcomputer (i.e. problems of practical size), while the linear and goal programming problems cannot handle the constraint on the duration such as of any project, because no project may stretch over more than two years but only two consecutive years (Hugo et al. 1989).

5.4.1. EXISTING PAVEMENT MANAGEMENT SYSTEMS

The existing pavement management systems are: the Arizona Department of Transportation (Kulkarni et al. 1982) ; the Washington State Department of Transportation (LeClerc & Nelson 1982); the U.S.A. Army Engineering Cooperatives (Shanin & Kohn 1982); Urban Pavement Improvement (Karan & Haas 1976); and the Texas Transportation Institute (Lytton, Phillips & Shanmughan 1982).

5.4.2. CURRENT AND THE SHORTCOMING OF THE PREVIOUS SYSTEM

The current pavement management system is computerized and has procedures for gathering data, a data base, procedures for the validation of data, procedures for analysis and reporting of data, and procedures for the identification and prioritization of rehabilitation. Previous pavement management system can be compared with the current condition of pavement (systematically and objectively). However, it does not take the future behavior of pavement into account, and it is significant to do so because each pavement deteriorates has a unique action, and deformation is also dependent among others. For instance, the structure materials in the pavement and traffic loading is expected to carry environmental condition such as temperature and moisture (Hugo et al. 1989). It is necessary to consider the characteristic of different types of roads into account in order to obtained a better schedule for maintenance and rehabilitation in particular, models for forecasting the behavior of pavement such as model for ride comfort, cracking forming, user costs and routine maintenance as described by (Hugo et al. 1989; Tarboton 1985).

5.4.3. OVERVIEW OF PAVEMENT MANAGEMENT SYSTEMS

A pavement management system (PMS) is a decision support system which is designed to be used by pavement personnel to help make cost-effective decisions concerning the maintenance and rehabilitation of the pavements for which they are responsible pavement management provides a means to organize the massive amount of data that develops with a pavement network. Pavement management is generally described and developed at two levels, network and project level. The difference between network-level and project level include the level for which the decisions are being made and the amount and type of data required (Smith 1991).

The purpose of the network-level system is normally related to the budget process of identifying pavement maintenance and rehabilitation fund needs and determining the impact of various funding scenario on the healing the pavement. At the project level, the purpose is to provide the best maintenance or rehabilitation strategy possible for that defined section of pavement for the funds area available (Smith 1991). The primary result of project-level PMS includes an assessment of the cause of deterioration

The basic elements of a network-pavement management system include: an inventory, condition assessment, identification of project when fund are constrained and a method to determine the impact of funding decisions on future condition. Project-level pavement management is basically the engineering analysis and design required to develop the most cost-effective maintenance or rehabilitation treatment for a specific section of pavement that was hopefully selected for repair by the network- level system. Pavement analysis and design are complex engineering problems requiring a systematic approach to quantify and analysis the many variables which influence identification and selection of appropriate maintenance and rehabilitation techniques.

In new design, many design parameters are assumed or developed from laboratory (Smith 1991). However, many of the materials to rest traffic and environmental induced damage are in place when maintenance and rehabilitation are being planned, and the existing material properties can be determined along with condition, traffic, and other constraints. Project-level analysis can be approached as series of steps to determine the cause of deterioration and identify relevant constraints (Gole 1985). However, it is essential that the process determine that cause and extent of deterioration to insure that the solution developed addresses the cause rather than just a symptom (Smith 1991).

5.4.4. PAVEMENT CONDITION ASSESSMENT

Pavement condition assessment begin with collecting data to determine type, amount, and severity of surface distress, structural integrity, ride quality, and skid resistance of the pavement. Pavement condition data is necessary for pavement evaluation and determination of maintenance and rehabilitation needs. There are used to project pavement performance, establish maintenance and rehabilitation funds. Pavement condition measured using the following factor (Smith 1991):

- Surface distress
- Structural capacity
- Roughness (ride quality)
- Skid resistance

These four pavement condition factors can be used to determine the overall pavement condition and identify the most-effective and optimum maintenance and rehabilitation treatment. The pavement condition factor can vary in their response of important in term of pavement performance and maintenance and rehabilitation needs. It is obvious that any treatment recommended to correcting the structural load-carrying capacity of the pavement will take care of all other deficiencies that might be present, including roughness. Smith et al. (1984) and Smith (1991) described any treatment selected to correct pavement roughness will in turn improve the surface skid resistance as well as surface distress.

There are an number of “optimization tools” available which could be used to determine the “ optional allocation of funds”(Haas & Hudson 1982; Haas et al. 1985; Liebman 1985; Lytton 1985; Way 1985). These include linear programming, integer programming, Markov decision analysis, and dynamic programming (Smith 1991). Several factors have inhibited the use of true optimization tools in PMS. Many agency personnel oppose their use because these technologies are complex and provide answers that they may not readily understand or explain (Smith 1991).

5.4.4.1. PRIORITY PROGRAMMING OF PAVEMENT REHABILITION

Priority programming of rehabilitation for paved road networks has existed among highway agencies for many years. Prior the 1970s such priority programs were almost invariably arrived at through ranking procedures (Haas et al. 1985). These ranged from simple subjected ranking to ranking by various weighted traffic and distress parameters, including sufficiency rating methods. Each priority programming for pavement rehabilitation involves simple subjective ranking or sophisticated mathematical programming, has specific features in terms of the pavement rating parameters and types of economic analysis applied. However, every priority programing method developed to date for network level pavement management is basically composed of four major steps: information, identification of needs, priority analysis and output reports (Haas et al. 1985). A basic step in priority programming for pavement rehabilitation is shown in Figure 5.4.

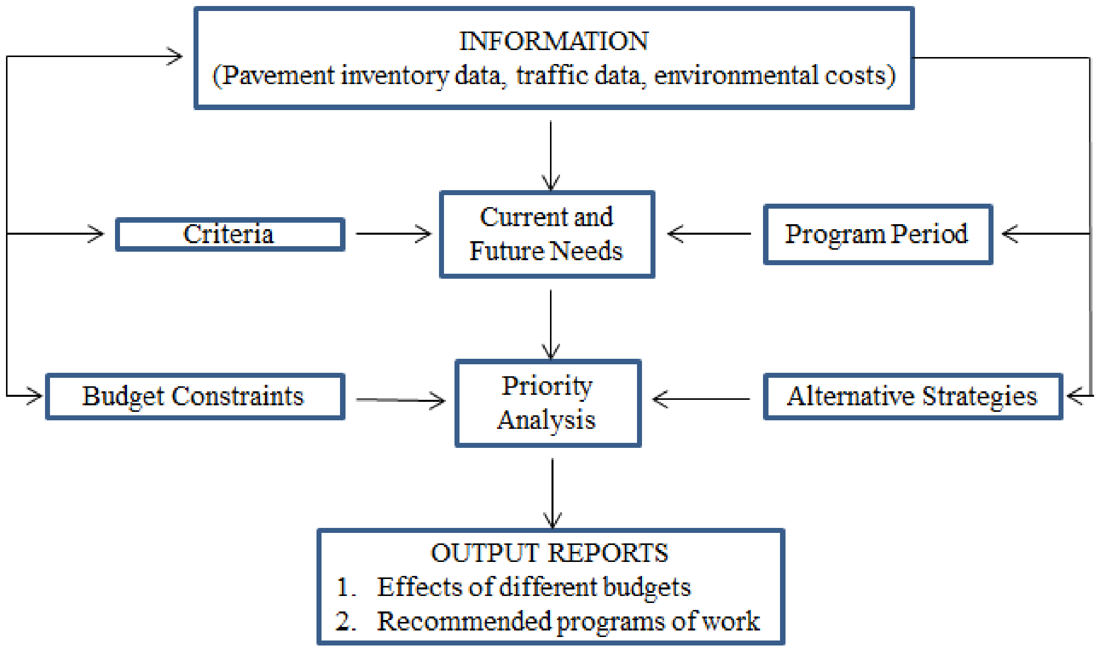


Figure 5.4 Basic Steps in Priority Programming for Pavement Rehabilitation

5.5. PAVEMENT MAINTENANCE AND REHABILITATION COST ANALYSIS

Estimation and scheduling of future pavement maintenance and rehabilitation (M&R) works has been one of the primary concerns of transportation and highway agencies due the limited allocations in budget and resource. Several researchers have estimated the future M & R using different methods in the past 6 decades. However, the estimation of pavement maintenance and rehabilitation somehow lucks the engineering principles. Two approximation methods for estimation of pavement rehabilitation costs are presented and compared for Western Australia roads networks based on highway present condition index (HPCI) and rehabilitation history. The former is based on pavement condition, while the other is based on historical and statistical trends. Database from pavement management systems (PMSs) of various highway section survey data in Western Australia were used in this study to establish a relationship between HPCI and pavement service life to analyze the trend of rehabilitation periods. The two methods provide a useful information and probability range of the several highway and transportation agencies, and the MainRoads Western Australia to guide them in the preliminary engineering planning, budgeting, estimating and scheduling of their future pavement M & R works.

Pavements are an important part of the transportation highway infrastructure that constitutes an enormous investment of public funds. A tremendous amount of time and money has been spending each year on construction of new pavement as well as on maintenance and rehabilitation (M&R) of existing pavements. To maximize benefits and minimize over all costs, a systematic scientific approach based on engineering principles are needed to manage pavement. Pavement management systems (PMSs) provide consistent, objective and systematic procedures to determine priorities, schedule, allocating resources and budgeting for pavement M & R (Federal Aviation Administration 2006).

The goal of most PMSs is to maximum benefits of the available fund. The process of PMS consists of four main components: network inventory; evaluation; performance prediction models and planning methods as well as described by (Lee, Park & Mission 2013; Simpson & Thompson 2006). Pavement prediction performance models are used to analyze the condition in the form of al quality or quantity index (Ismail, Ismail & Atiq 2009). The best maintenance programs are usually attended with the help of decision support tools in a PMSs as recommended by (Wang, Zhang & Machemehl 2003). The setting of M & R priorities is to make the maximal use of available resource (Lee, Park & Mission 2013).

The objective of this study is to evaluate and analyze approximate estimation and scheduling of future pavement M & R based on engineering pavement works. Highways present condition index (HPCI) and rehabilitation were compared. To estimate number of maintenance and rehabilitation that was needed, pavement management data in Western Australia were analyzed to determine the relation between HPCI and pavement service life as well as trends of rehabilitation period.

5.5.1. PAVEMENT MANAGEMENT DATA

Twenty percent of the highway pavement section in PMSs database were surveyed and gathered from LRDM in this study. The database contains pavement distress data such as traffic volume, number of lanes, M & R history, fatigue cracking, rut depth, pavement roughness, construction and pavement service life year, and other pavement structural information.

5.5.2. ESTIMATION OF REHABILITATION COST

Many technique are available for developing pavement deterioration models bases on engineering principles such as extrapolation, regression equation, mechanistic-empirical, polynomial constrained least square, S-shaped curve probability distribution, Markovian (Shanin & Kohn 1982).

Pavement distress and roughness data for each of the highway, pavement section were analyzed to determine the HPCI using an empirical equation and deterioration model developed by (Park et al. 2009)

$$HPCI = 4.564 - 0.348IRI - 0.36RUT - 0.01(TC + AREA)^{0.5} \quad (5.36)$$

where, $HPCI$ = highway present condition index; IRI = international roughness index; RUT = rut depth; TC = thermal cracking; and $AREA$ = area of alligator cracking and patching. Based on a linear regression analysis, another equations can be established from Equation (5.36) as follows (Park et al. 2009):

$$HPCI = 3.8247 - 0.0479x, \quad for\ two\ new\ pavement \quad (5.37)$$

$$HPCI = 3.7703 - 0.0586x, \quad for\ two - line\ rehabiliated\ \ pavement \quad (5.38)$$

$$HPCI = 3.9219 - 0.0877x, \quad for\ two\ new\ pavement \quad (5.39)$$

$$HPCI = 3.7756 - 0.0851x, \quad for\ four -$$

$$line\ rehabiliated\ pavement \tag{5.40}$$

Equations (5.37) through (5.40) are straight line equations of the form $y = b + mx$, where abscissa x is the pavement service life. The negative slope m indicated that the rate of deterioration and degradation of the pavement condition with time as the service year increased from its initial condition. The y – intercept b at pavement service life when x =0 indicated the average initial HPCI value at the service of the pavement. It can be seen from Equation (5.37) through (5.40) that the new pavements have higher HPCI values at the start of their service compared as to rehabilitation pavements that have already under gone with pervious maintenance work in the past.

5.6. RESULTS AND ANALYSIS

A summary of average daily traffic (ADT) and a pavement rehabilitation year over the past 30 years (1970 – 2002) are shown in Figure 5.5. From the data presented, it can be seen that the ADT has been increasing by about 53.8% since 1994 in Western Australia (WA). This showed that the use of full depth asphalt pavement to construct and rehabilitate heavily loaded urban roads has rapidly grown in WA over the past nearly 10 years and the costs for Pavement M & R have also been increasing from year to year. For example, the average daily traffic in 1984, 1990 and 1994 was 5971, 6860 and 7850, repectively. However, very good correlation (R^2=1) between construction and rehabilitation years as function of traffic flow on the sixty seven road locations. Guyer (2009) evaluated the pavement thickness that must be design to withstand the anticipated traffic roads, categorized by type and weight of vehicles, and measured by average daily volume (ADV) of each type for the design life of pavement. Author stated increasing the gross weight by as little 10 percent can be equivalent to increasing the volume of traffic by as much as 300 to 400 percent and imposed largely a fatigue effect on the flexible pavement as rapidly increased number of loads repetition per vehicle operation.

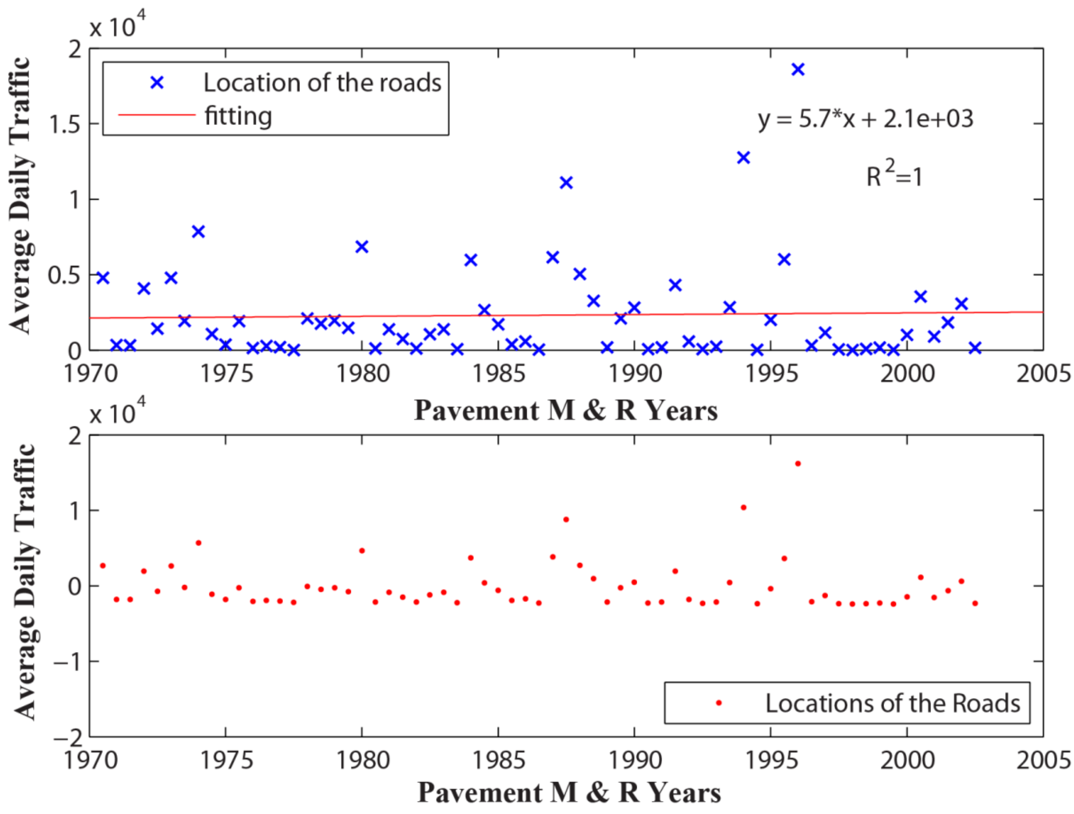


Figure 5.5 Average Daily Traffic versus Pavement Maintenance and Rehabilitation Years of Seven Roads Categories on Sixty Seven Locations

A summary of pavement crack distributions (i.e. crack 1, crack 2 and crack 3) is shown in Figure 5.6. As it can be seen from the results, the crack distribution for type 1 and 3 in 1984 – 1986 were above 85% all over the area. In addition, the crack types 3 were above 50% even in 1999. This showed that the stress and strain of the flexible asphalt pavement as the result of traffic loading and environmental factors (i.e. Temperature, ages and healing) might have deteriorate and damage the asphalt pavement on these particular years. Besides, the pavement construction and rehabilitation performance might not have also been as such effective and efficient. Although, similar pattern and good correlation (R^2=1) between crack type 1, crack type 2 and crack type 3 in terms of the size distribution. The Federal Highway Administration (FHWA) of the U.S. Department of Transportation (Stubstad et al. 2012) developed an approach for incorporating techniques used to interpret and evaluate deflection data for network-level pavement management system (PMS). According the FHWA guide fatigue cracking should not exceeding 25 percent of the total area within the first 15 years' service (Stubstad et al. 2012).

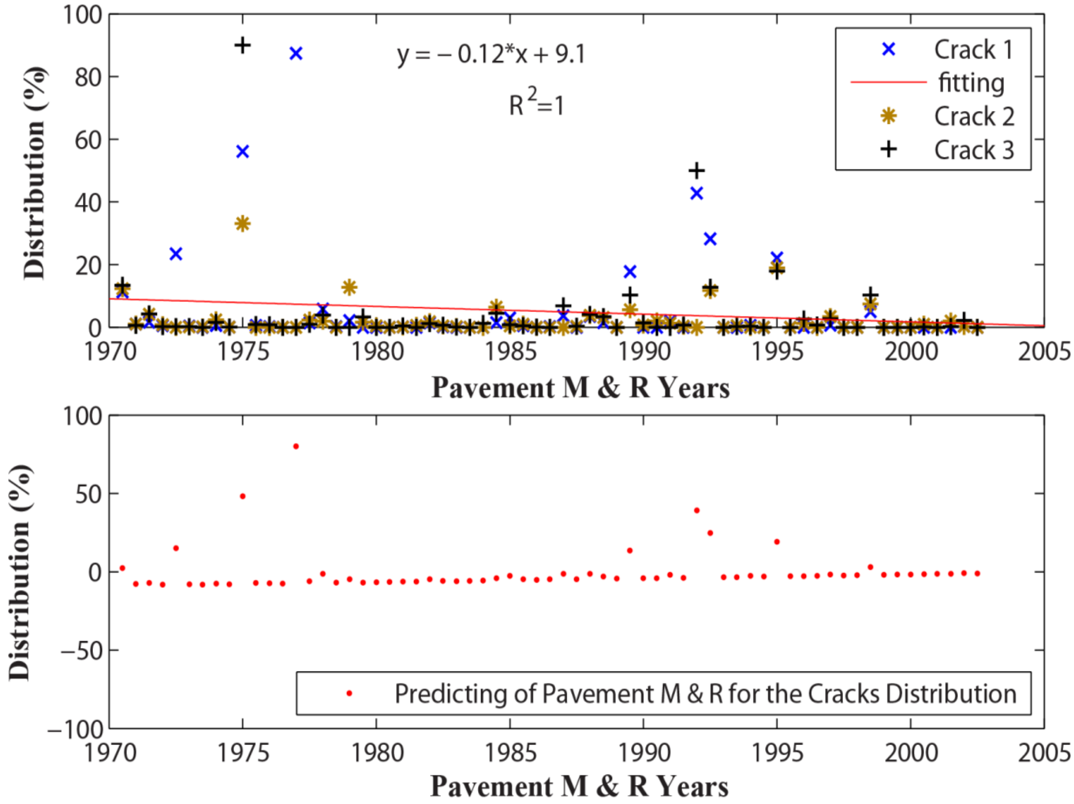
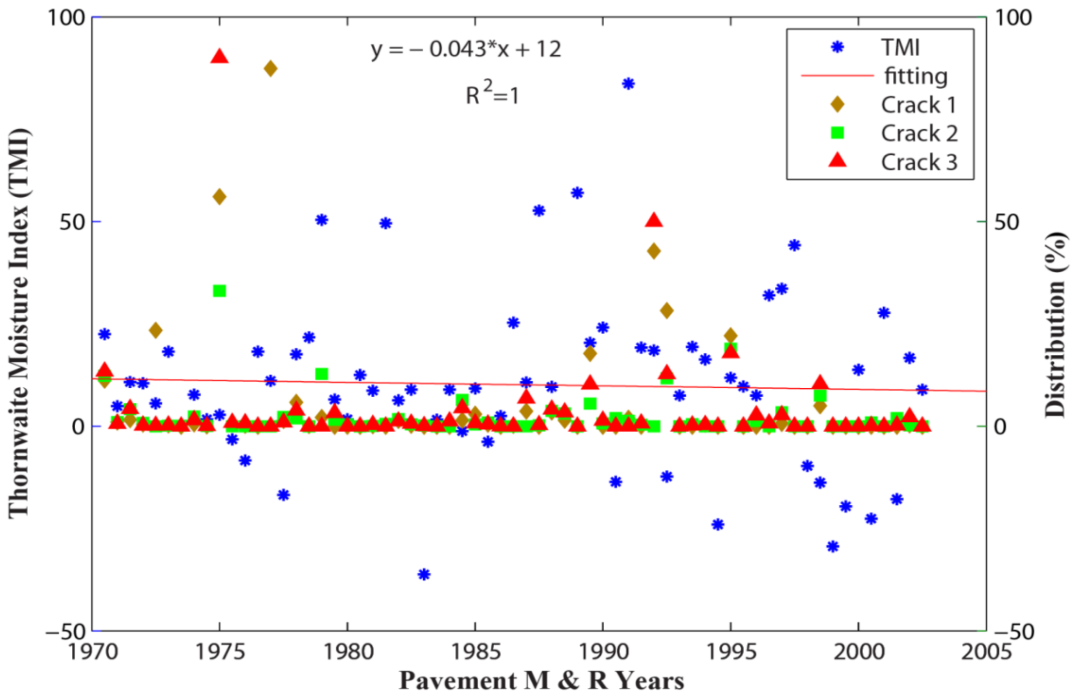
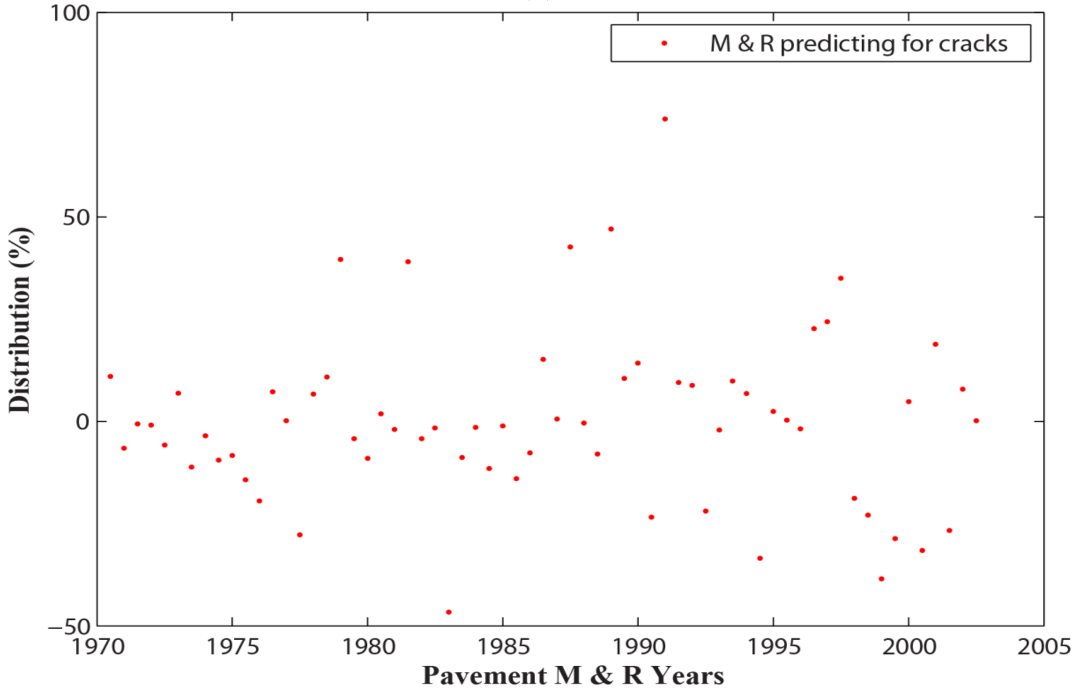


Figure 5.6 Cracks Distribution versus Pavement Maintenance and Rehabilitation Years of Seven Roads Categories on Sixty Seven Locations

The Thornwaite Moisture Index (TMI) versus pavement crack of the seven roads categories is shown in Figure 5.7. The analyses indicated that the TMI for the seal granular pavement were low in percent (i.e. about 50%) during the past 20 years (1970-1990) and then, rapidly drop below zero after 1990 onward. This indicated that the roughness, rutting and cracking of sealed asphalt pavement within the past years were everywhere as the result of environmental factors such as temperature, ages and healing. It is considered that pavement M & R performance actions were not as such effective and sustainable on those periods. Though, similar pattern and very good correlation (R^2=1) between crack type 1, crack type 2 and crack type 3 in terms of the size distribution. Austroads has recently developed road deterioration models for roughness, rutting and cracking predicting of seal granular pavement, which represents 85% of sealed pavement in Australia (Ferreira, Micaelo & Souza 2012).



(a)



(b)

Figure 5.7 (a) Thornwaite Moisture Index (TMI) versus Pavement Crack for Seven Different Types of Roads Categories from Sixty Seven Locations (b) Predicting of Pavement M & R for Cracks

An Average daily traffic versus heavy vehicles for seven different types of roads categories from sixty seven locations is shown in Figure 5.8. From the data presented, it can be seen that the heavy vehicle volume (HVV) on the traffic roads has been increased by 30 to 40% since 1990. This indicated that the wheel load of heavy trucks contribute to several forms of pavement distress. However, very good correlation (R^2=1) among the seven different types of roads categories in terms of pavement distress and fatigue as function of the heavily urban town traffic flows. The National Cooperative Highway Research Program in the U.S. (NCHRP1993) studied the effect of heavy-vehicle characteristics on pavement response and performance. The report concluded that the wheel loads of heavy – trucks can contribute to various forms of pavement distress and damage that influence sensitivity of truck wheel load. Fatigue damage varies over a range of 20:1 with typical variations in axle loads and over the same range with typical variations in pavement thickness (NCHRP1993). Similarly, Bonaquist et al. (1989) evaluated the effect of tire pressure on flexible response and performance, and the results showed increased rutting and cracking for section trafficked with the higher tire pressure of heavy-vehicle (HV).

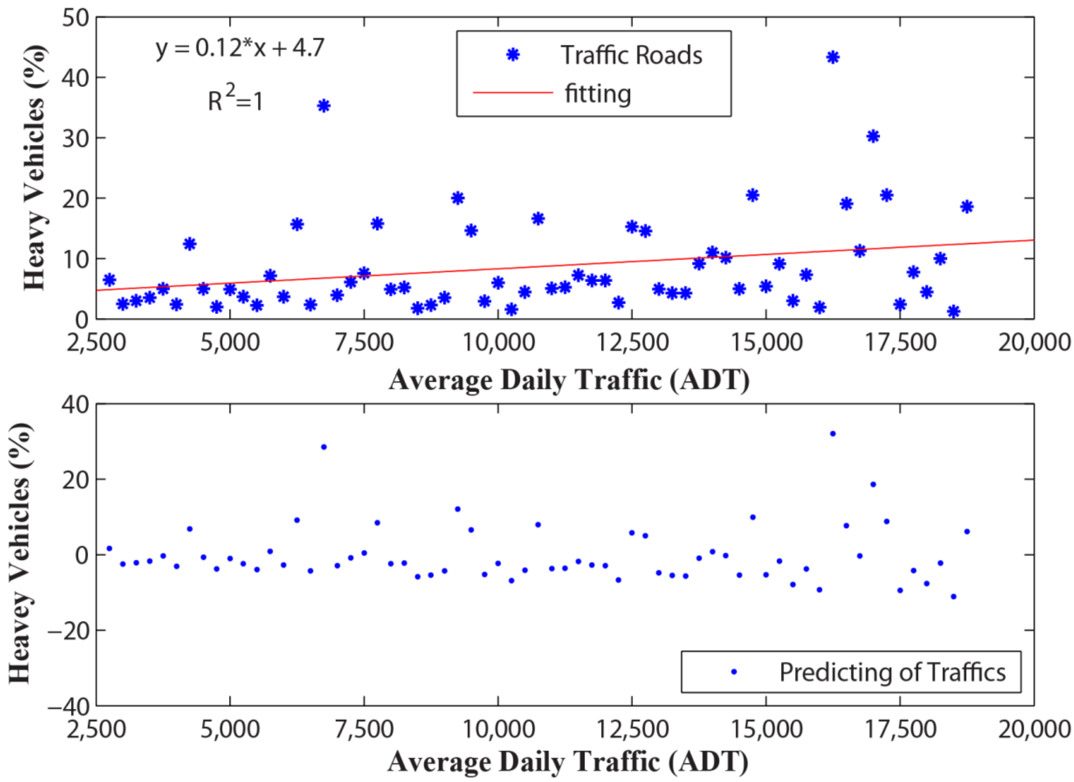


Figure 5.8 Average Daily Traffic versus Heavy Vehicles for Seven Different Types of Roads Categories from Sixty Seven Locations

The pavement construction and rehabilitation year for the surface types of asphalt concrete (AC) is shown in Figure 5.9. From the data demonstrated, it can be seen that the pavement rehabilitation for the cracks is twice in every year since 1990. These indicated that the deterioration of flexible pavement has been increased by 50 percent than 1970s due to traffic loading and environmental factors in the urban cities. While, very good correlation (R^2=1) between the construction years and the asphalt concrete rehabilitation years on the sixty seven road location as function of road surfacing. Blankenship et al. (2003) reported that it is common to see cracks reflect through a new hot-mix asphalt overlap in three to five years. The water infinity ration, freeze, thaw cycles and repeated loading cause the asphalt to deteriorate at joint and will eventually cause revelry. The reflecting structural degradation might also occurred as a result of moisture through the cracked pavement and deteriorates the base and loss of support underneath the joint concrete pavement slab.

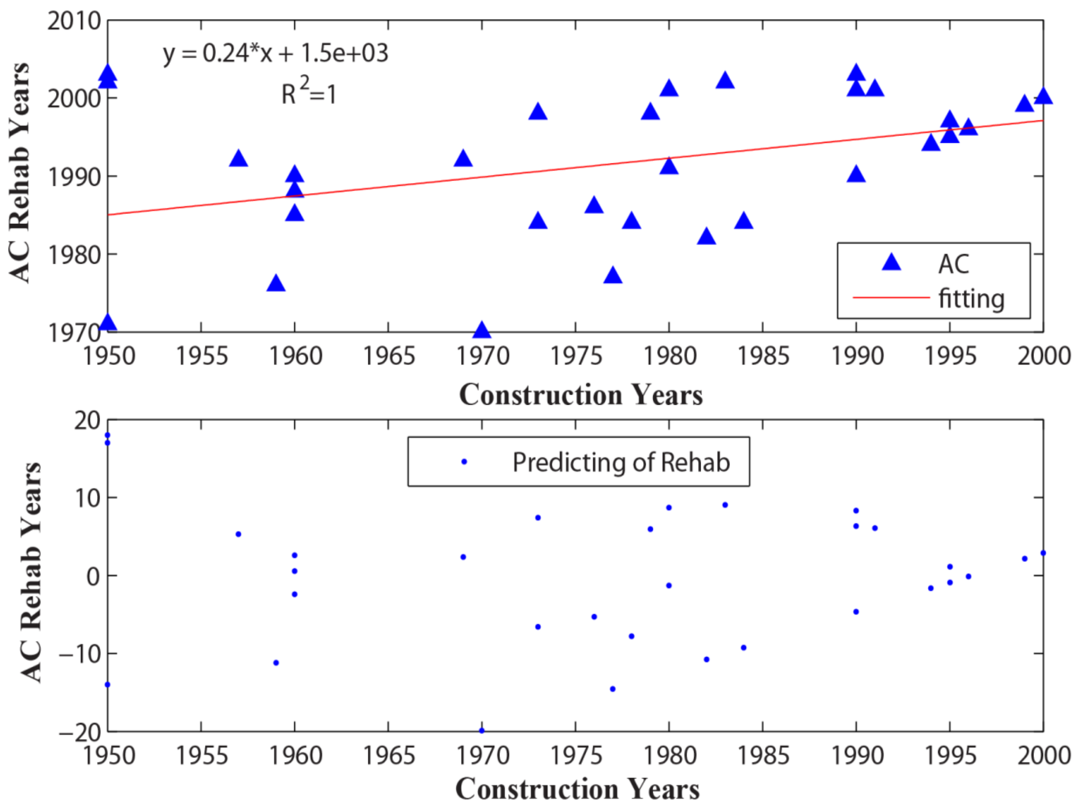


Figure 5.9 Asphalt Concrete Rehabilitation Years versus Construction Years for Different Types of Asphalt Concrete on Thirty Six Locations

Slurry Seals (SS) construction and maintenance years for different types of Asphalt Concrete on twenty five locations is shown in Figure 5.10. As it can be seen from the analyses, The SS construction and rehabilitation years are somehow persistence (R^2=1) which is the state of being quality for pavement M & R performance of the flexible pavement ver. Although several different rehabilitation strategies have been used in Western Australia, reflecting crack through joint concrete pavement overlays has been a persistence problems ever since. According to Chen et al. (2006) the performance of cracking has been related to the several premature failures. The small openings in pavement cracking retarding grid and lack of an effective bonds can causes the layer to great separation and then, cause to failures of the pavement structure.

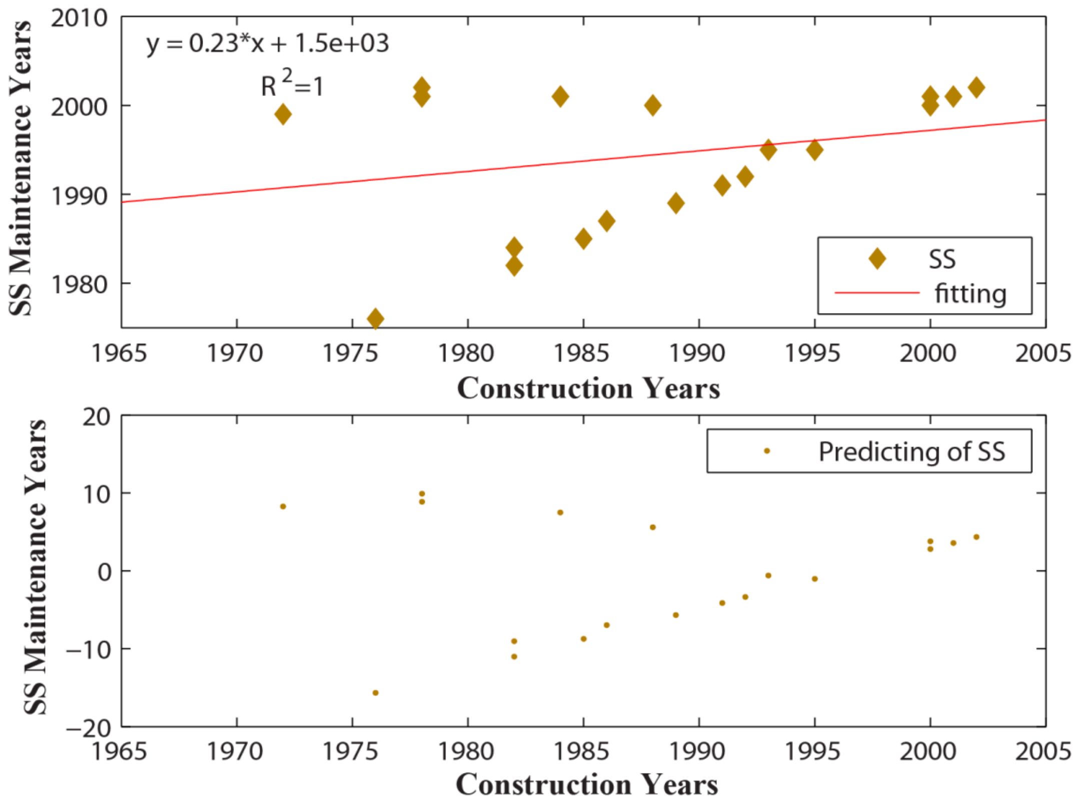


Figure 5.10 Slurry Seals Maintenance Years versus Construction Years for Different Types of Asphalt Concrete on Twenty Five Locations

5.7. CONCLUSIONS

The use of full depth of asphalt pavement to construct and rehabilitate heavily loaded urban roads has rapidly growth in Western Australia over the past 4 years. The average daily traffic (ADT) has been increased by 53.8% since 1994. Similarly, the costs for pavement M & R have also been increasing from year to year. The pavement crack distribution (i.e. crack type 1, type 2 and type 3) were above 50% all over the area since 1999 although the crack distribution for type 1 and 3 in 1984-1986 were above 85%. The deteriorate and damage of the flexible asphalt pavement might have occurred due to the increased of the traffic loading in the urban roads and environmental factor (i.e. temperature, ages and healing) on these particular years. According to FHWA guide fatigue cracking should not exceeding 25% of the total area within the first 15 years' service (Stubstad et al. 2012).

The Thomwaite Moisture Index (TMI) for the seal granular pavement were low (about 50%) during the past 20 years (1970-1990) and then, rapidly drop below zero after 1990 onward. This indicated that the roughness, rutting, and cracking of seal asphalt pavement within the past years were everywhere as the result of temperature. However, Austroads has recently developed road deterioration models for roughness, rutting, and cracking predicting of seal granular pavement which represent 85% of sealed pavement in Australia (Ferreira, Micaelo & Souza 2012).

The pavement construction and rehabilitation for the pavement cracks are twice in year since 1990 and the deterioration of flexible pavement has been increased by 50% than 1970. Blankenship et al. (2003) reported that it is common to see crack reflected through a new hot-mix asphalt overlap in three to five years. Although several different rehabilitation strategies have been used in Western Australia, reflecting crack through joint concrete pavement overlay has been a persistence problems ever since. According, to Chen et al. (2006) the performance of cracking has been related to the several premature failures and lack of an effective bonds between aggregate can causes the layer to separation and then, cause to failure of the pavement structure.

The pavement performance of the predicting model using probabilistic roads network of pavement engineering perform well in all categories and it is recommended to do the pavement work using the probabilistic network models.

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CHAPTER 6: DEVELOPING MASTER CURVES, BINDER VISCOSITY AND PREDICTING DYNAMIC MODULUS OF POLYMER-MODIFIED ASPHALT MIXTURES

An abridged version of this Chapter is published in *International Journal of Engineering and Technology*, vol. 7, no. 3, pp. 190-197, 2015.

ABSTRACT

The complex moduli relationship related mixture moduli to temperature and time rate of loading has been an integral part of several mechanistic-empirical (M-E) design procedures used throughout of the world. Seven asphalt concrete mixtures of different types of polymer modified binders (PMB) were produced in a laboratory to modify performance of asphalt mixture. The main role of this research is to evaluate the influence of these polymer modifiers on the pavement performance of asphalt mixture with the dynamic modulus, |E*| of hot-mix asphalt (HMA) mixture indicator in a laboratory test for Mainroad Western Australia and Fulton Hogan. In this study, the influence of temperature, loading frequency, and confining pressure on the dynamic characteristic of asphalt mixture were analysis, master curves of dynamic modulus of HMA mixtures were developed and data’s were interpreted. Results showed that AC10 5.7% A35P (EVA) M7 B5, AC10 5.7% C450 M10 B5 and AC10 Multi 600/700 M5 B4 mixes method were the more efficient and effective in all categories of asphalt performance measures for strength and durability of HMA as compared to others polymer modifiers. A very good correlation (R^2 = 1) was found for each polymer modifier. This suggested that laboratory test using a various temperatures and loading frequencies can improve pavement mix design, lab and field control and assurance. A strong correlation between binder viscosity and temperature [R^2 = 1] for polymer modified asphalt mixture.

Author keywords: Polymer modifier, dynamic modulus, master curve; viscosity, temperature, asphalt mixture, Western Australia.

6.1. INTRODUCTION

The new American Association of State Highway and Transportation Officials (AASHTO) Mechanistic-Empirical Pavement Design Guide (MEPDG) based on the National Cooperative Highway Research Program (NCHRP) 1-37A study uses the dynamic modulus of asphalt mixture, |E*|, as the asphalt material input in its pavement analysis (National Cooperative Highway Research Program 2001; National Cooperative Highway Research Program 2004). The concept of a dynamic modulus protocol was originally developed by Coffman and Pagen at Ohio State University in the 1960s (Dougan et al. 2003) and this test was not implemented for payment design and analysis until recently (Bayat, Kasani & Soleymani 2011). The test can be applied in a uniaxial (triaxial) condition either compression or tension. Most of the test results obtained over the past 30 years have been in compression and are generally denoted as E*. The E* test was adopted as the "Modulus Test of Choice" by the Asphalt Institute in the late 1960s by Kallas, Shook and Witczak (Dougan et al. 2003). It subsequently became an American Society of Testing and Materials (ASTM) standard in early 1970 under ASTM designation D3496 (Zeiada et al. 2011).

The dynamic modulus of hot-mix asphalt (HMA) is an important input parameter in asphalt pavement design. The mechanistic-empirical pavement design guide (MEPDG) recommends determining dynamic modulus at three levels: Level 1, Level 2 and Level 3 of analysis for predicting the performance of flexible pavement as discussed by NCHRP (National Cooperative Highway Research Program 2004). The use of a particular hierarchal input Level 1 of analysis depends on the amount of information available to the designer and the critically of the project. For example, at Level 1, the asphalt binder and the HMA are tested in the laboratory to measure dynamic modulus. However, the measurement of dynamic modulus in the laboratory is not always feasible because of the tedious experiments and it may also take several days to develop a single master curve (Azari et al. 2007; Birgisson, Sholar & Roque 2005; Singh, Zaman & Commuri 2011).

To overcome these difficult, the MEPDG recommends estimating the dynamic modulus without conducting actual modulus tests in the laboratory for Level 2 and Level 3 design (Singh, Zaman & Commuri 2011). Several prediction models are available in the literature for estimating the dynamic modulus of HMA. These models use the volumetric properties of mix, aggregate graduation, loading frequency and viscosity of an asphalt binder to predict dynamics as discussed on literature by (Al-Khateeb et al. 2006; Andrei, Witczak & Mirza 1999; Christensen Jr 2003; Loulizi, Flintsch & McGhee 2007; Singh, Zaman & Commuri 2011; Tran & Hall 2005).

Dynamic modulus testing characterizes asphalt mixture as a linear viscos-elastic material over a wide range of temperature and loading frequency. In the MEPDG, dynamic modulus testing results are used to generate a master curve for each mixture by the time-temperature superposition methodology (Bayat, Kasani & Soleymani 2011; Dougan et al. 2003; National Cooperative Highway Research Program 2004). The dynamic modulus is a fundamental asphalt mixture property and it can be used to investigate the temperature and loading frequency of hot-mix asphalt pavement because of its main application MEPDG in the form of a master curve.

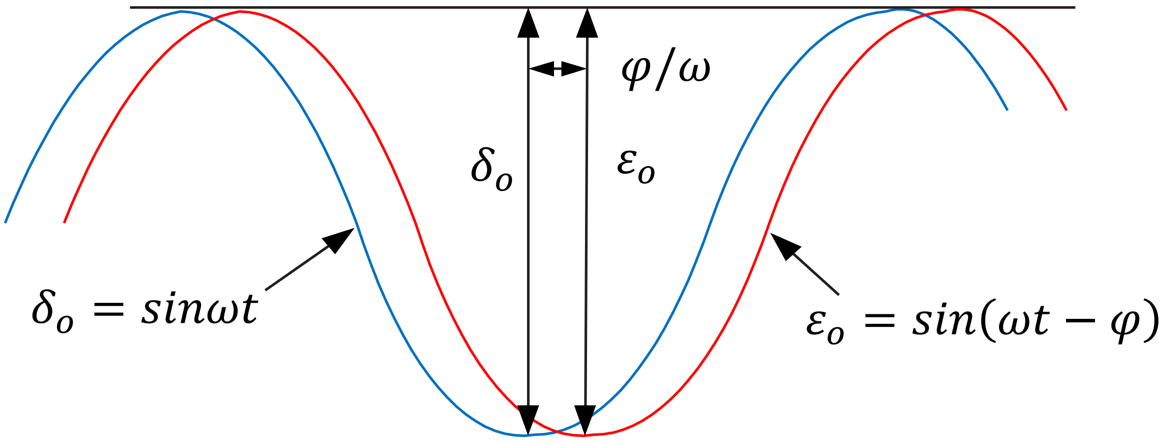


Figure 6.1 Dynamic (Complex) Modulus

The goal of this study is to evaluate the influence of polymer modified binders (PMB) on the pavement performance of asphalt mixtures for Western Australia Main Roads. Results from this research will be of great guidance in selecting modifier material for Western Australia.

6.2. METHODS AND MATERIALS

6.2.1. METHODS

For linear visco-elastic material such as HMA mixture, the stress-strain relationship under a continuous sinusoidal loading is defined as by its complex dynamic modulus (E*). This is a complex number that relates stress to strain for linear visco-elastic material subjected to continuously applied sinusoidal loading in frequency domain (Dougan et al. 2003). The complex modulus is defined as the ratio of the amplitudes of the sinusoidal stress at any given time, t and angular loading frequency, ω, $\delta_o$sin (ωt) and the amplitude of sinusoidal strain, $\varepsilon = \delta_o \sin(\omega t - \phi)$, at the same time and frequency, that result in the steady response is shown in Figure 6.1.

The dynamic (complex) modulus equation can mathematically be expressed as:

$$E^* = \frac{\delta}{\varepsilon} = \frac{\delta_0 e^{i\omega t}}{\varepsilon_0 e^{i(\omega t - \phi)}} = \frac{\delta_0 sin\omega t}{\varepsilon_0 \sin(\omega t - \phi)} \quad (6.1)$$

Where, δ_0= peak (maximum) stress; ε_0= peak (maximum) strain; ϕ = phase angle degree; ω = angular velocity; i = imaginary component of the complex modulus and t = time. Mathematically, the dynamic modulus is defined as the absolute values of the complex modulus and the equation (6.1) can be written as

$$|E^*| = \frac{\delta_0}{\varepsilon_0} \tag{6.2}$$

The primary output variable of the test is the dynamic modulus $|E^*|$, and the phase angle (ϕ), which is a direct indicator of the elastic-viscous properties of the mix or binder material. The dynamic modulus in the compression $|E^*|$ of the mix is similar in principle to the G^*, complex shear modulus of the binder developed in the SHRP and SuperPave programs at the University of California, Berkley, and Penn State University (Dougan et al. 2003). The two moduli, E^* and G^* are theoretically related through engineering mechanics by the relationships:

$$E^* = 2(1+\mu)G^* \tag{6.3}$$

In the proposed "2002 Guide for the Design of Pavement System", currently under development in NCHRP project 1-37A, the modulus of the asphalt concrete-at all analysis level of temperature and time rate of load-is determined from the a master curve constructed at a reference temperature, generally 21.1^{o}C (70F) (Dougan et al. 2003; National Cooperative Highway Research Program 2004; Schwartz 2007; Schwartz & Carvalho 2007; Witczak, Andrei & Houston 2000). Master curves are constructed using the principle of time-temperature superposition. The data at various temperatures should be shifted with respected to log of time until the curves merge in to a single smooth function. The resulting master curve of the modulus, as a function of time, formed in this manner describes the time depending of the material (Dougan et al. 2003). The amount of shift required at each temperature to form the master curve describes the temperature depending of the material. In general, the master modulus curve can be mathematically modeled by a sigmoidal function described as:

$$log\ |E^*| = \frac{\delta+\alpha}{1+e^{\beta+\gamma(\log t_r)}} \tag{6.4}$$

Where, t_r = time of loading at reference temperature; δ = minimum value of E^*; δ + α = maximum value of sigmoidal; β, γ = parameters describing the shape of sigmoidal function; and α = variable which is function of gradation. The shift factor can be shown in the following form:

$$\alpha(T) = \frac{t}{t_r} \tag{6.5}$$

where, $\alpha(T)$ = shift factor as a function of temperature; t = time of loading during test, s; t_r= time of loading at reference usually (70 ^{o}C) and T = temperature of loading cycle. The equation can be re-arranged in term of the reduced time (t_r) of the loading at the reference temperature as:

$$t_r = \frac{t}{\alpha(T)} \tag{6.6}$$

$$t = \frac{1}{2\pi f} \tag{6.7}$$

where, f is the loading frequency at desired temperature, Hz. For the sake of accuracy, a second-order polynomial relationship between the logarithm of shift factor, $\log \alpha(T)$ and the temperature is used. The relationship can be expressed as follows:

$$\alpha(T) = 10^{a(T-25)^2+b(T-25)} \tag{6.8}$$

$$log\ \alpha(T_i) = a(T_i - 25)^2 + b(T_i - 25) \tag{6.9}$$

where, $\alpha(T_i)$ is a shift factor as the function of temperature T_i; T_i is temperature of interest; a and b are a coefficient of the second-order polynomial. The dynamic modulus, E^* of the mix and the complex shear modulus of binder, G^* relationship that is described in equation (6.3) can be re-arranged as:

$$G^* = \frac{E^*}{2(1+\mu)} \tag{6.10}$$

Asphalt technicians have used the viscosity – temperature relationship or viscosity – temperature susceptibility (VTS) method of binder temperature susceptibility classification for decades (Puzinauskas 1967; Roberts et al. 1996), although it has not been used as a popular index values for this purpose. One basic definition of VTS (Rasmussen, Lytton & Chang 2002) :

$$VTS = \frac{Log[\log(\eta_{T2}) - \log[\log(\eta_{T1})]}{\log(T_2) - \log(T_1)} \tag{6.11}$$

where, T_2 and T_1 is a temperature of the binder at two known points (R = degree Rankine) and η_{T2} and η_{T1} is viscosities of the binder at the same two point (cp). The lager the magnitude of the VTS value is found to be, the more susceptible the binder is to change in viscosity with temperature. In 1967, Puzinauskas derived the VTS for over 50 binders commonly used in the United States at that time, and found the VTS value to range [based on equation (6.11)] from -3.36 to -.3.98 (Puzinauskas 1967).

Fonseca and Witczak (1996) presented a new model for prediction of the dynamic modulus of hot-mix asphalt (HMA) that included the binder viscosity as an input variable. The model include a calculation of methods for binder viscosity at a function of temperature and age (Rasmussen, Lytton & Chang 2002). These formulations were based on the VTS formula, as well as a second parameter, A. The 'A' parameter is the y – axis intercept of the log [log (viscosity)] and log (temperature) curves. The A parameter cannot be measure directly, but it can be derived from least – squares fit of viscosity – temperature data from a given binders. The basic formula for viscosity of binder can be described as (Dougan et al. 2003):

$$\eta = \frac{|G^*|}{10}\left(\frac{1}{sin_{\delta_b}}\right)^{4.8628} \tag{6.12}$$

The basic formula for VTS and A is:

$$log[log(\eta)] = A + VTS * log(T_R) \tag{6.13}$$

where η is viscosity of binder, (cp); $|G^*|$ is complex shear modulus of binder, Pa; δ_b is phase angle of binder associate with $|G^*|$, degree; A is binder (intercept) parameter and VTS is slope parameter and T_R is temperature, ^{o}Rankine.

6.2.2. MATERIALS

Types of hot-mix asphalt used on the Mainroads Western Australia network are dense graded asphalt (DGA), open graded asphalt (OGA) and stone mastic asphalt (SMA). DGA, the most common type of asphalt, provides optimal structure strength and generally good resistance to deformation. OGA is designed to drain water through the asphalt to remove excess water from the tyre/road surface. SMA is similar to OGA but has a high proportion of dust and high binder contents to achieve an improved fatigue life. SMA has a texture surface but does not drain water through its layer as does OGA (American Concrete Institute Committee 2007; Brown, Kandhal & Zhang 2004; Main Roads Western Australia 2007). All Materials selected for this project were from local sources and are originally of Western Australian pavement materials used in the industry.

In order to assess the master curve development and predict the dynamic modulus of polymer modified of hot-mix asphalt mixture, it is necessary to obtain laboratory data of different types of modifiers and characteristic of asphalt mixes. The properties of asphalt AH-70 Grade are listed on Table 1. Seven modifiers were selected for this study and the mixes descriptions for developing master curves and predicting dynamic modulus of polymer modified asphalt mixtures were: C320 M1 B3; A10E M2 B4; A15E M3 B4; Multi 600/170 M5 B4; A20E M6 B7; A35P (EVA) M7 B5; and C450 M10 B5. Each asphalt mixture consists 10 mm dense graded granite (AC10) and binder of 5.7 percent.

Table 6.1 Properties of Asphalt AH-70 Grade

Mix Design Properties	Specification Value
Penetration (25 ^oC, 100g, 5s, 0.1mm)	62.90
Ductility (5 cm/min, 15 ^oC)	160.00
Softening (ring and ball method)	52.40
Density (g/cm^3, 15 ^oC)	1.04
Flash point (^oC)	270.00
Wax content (%)	2.10
Solubility (%)	99.70
Spot test	Negative

6.2.2.1. SPECIMEN PREPARATION AND COMPACTION OF MIXES

Sample preparation and compaction temperature were obtained for each asphalt mixes using consistence test results according Australian Standard Test Methods: AS2891 and AS2150. The hot-mix asphalt (HMA) mixtures were heated for 2 hr at 170 ^oC in oven before compaction. The Gyratory compaction pressure was 600 kPa. The test sample was then compacted with gyratory compaction using the Servopac into 150 mm diameter 170 mm in height. The Servopac is a fully automated, servo-controlled, gyratory compactor designed to compact asphalt mixes by gyratory compaction. Compaction is achieved by the simultaneous actions of static compression and the shearing action resulting from the mould being gyrated through an angle about its longitudinal axis. Test specimen was cored from the center of the gyratory compacted sample. The specimen was sawn at approximately 5 mm from each sample to have the final 110 mm diameter x 160 mm height of E* test specimen. All the test specimens were compacted to about 5% air voids. The mix design for AC10 mm dense graded granite is shown in Table 6..2 was designed in according with Gyratory volumetric mix design procedures and its optimum of asphalt bender content (P_b) is 5.7%, air void is 4%, and void in mineral aggregate (VMA) are 17.7%, voids filled with asphalt (VFA) are 75%; and effective binder content (V_{beff}) is 12%.

Table 6.2 Mix Design for AC10 mm Dense Graded Granite

Mass Percentage Passing Sieve Size											
Gradation (mm)	19.0	13.2	9.5	6.7	4.75	2.36	1.18	0.6	0.3	0.15	0.075
Percentage passing by weight	100	91	76	61	48	32	21	15	8	5	3
Austroad Spec. limit	100	100	100	85-100	30-63	20-35	16-28	14-24	12-20	10-16	8-12

6.2.2.2. DYNAMIC MODULUS

The dynamic modulus test was conducted using IPC test machine, and is capable of providing a constant pressure upto 210 kPa and an environmental chamber to control testing temperatures (between -40 ^{o}C and +90 ^{o}C). Test specimens were accomplished using gluing gauge plugs onto the side of the specimen and attached a Linear Variable Differential Transducer (LVDT) to the plugs to measure the displacement. A haversine loading ($P_{dynamic}$) was adjusted in order to obtain axial strains between 75 and 125 microstrain to the specimen without impact in a cyclic manner. For each asphalt mix, 3 replicates were prepared. Since this paper is mainly focused on mixture performance at high temperature, each specimen in this study was tested at 30, 40, 50, 60, 70, 80, and 90 ^{o}C without being at low temperature. The loading frequencies were 0.1, 0.5, 1, 5, 10, 20, and 25 Hz, respectively.

6.3. RESULTS AND ANALYSIS

6.3.1. MASTER CURVE

A summary of $|E^{*}|$ versus loading frequency for different types of polymer modifiers are shown in Figure 6.2 As it has shown from the plot, the $|E^{*}|$ data are shifted using a nonlinear optimization and solve shift parameters. As the result of these seven parameters of master curves, models are fitted by almost least squares methods with matlab program in this study. From the data presented, it can be seen that all the modifiers have the same pattern and linear range to all asphalt mixes although they have a different $|E^{*}|$. This showed that the stress to strain for linear visco-elastic materials subjected to continuously applied sinusoidal loading in frequency domain at the time and frequency results a steady response and reduce typical dynamic stress level based on the various temperature. Dougan et al. (Dougan et al. 2003) discussed that the dynamic modulus that subjected to continuously applied sinusoidal loading in the frequency domain at the time and frequency results a steady response and reduce a friction at the bottom of the loading frame.

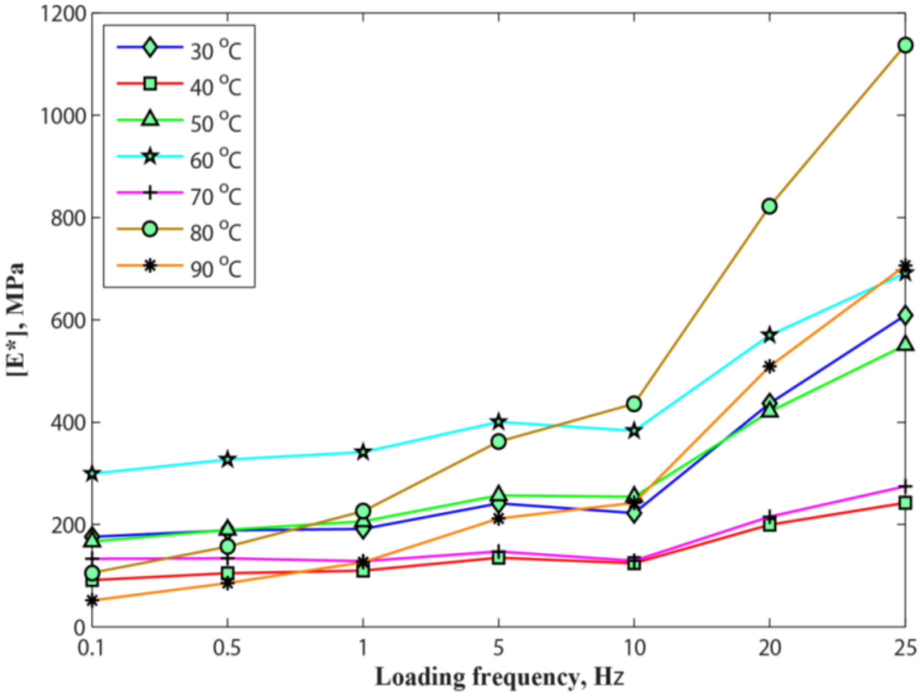


Figure 6.2 Plot of Loading Frequency versus |E*|

The shift factor $\alpha(T)$ versus temperature is given in Figure 6.3. These seven parameters are then used in equation. (6.4) to calculate $|E^*|$ of the particular asphalt mix at the temperature and loading frequency within the range in $|E^*|$ testing. $|E^*|$ master curves of all mixtures were constructed at the reference 25 °C following the principle time-temperature superposition. The data at various temperatures were shifted in line with frequency until the curves merges into a single sigmoidal function which represent the master curve using a second-order polynomial relationship between the logarithm of the shift factor, $\log \alpha(T)$ and temperature as it is shown in (Figure 6.3) and very good correlation (R^2=1) between the E^* and loading frequency with shift factor as function of temperature. Zhu et al. (Zhu et al. 2011) developed a master curves and predicting dynamic modulus of polymer modified asphalt mixture for four types of polymer modifiers and plotted shift factor α(T) versus temperature with very good correlation [R^2 = 0.9981].

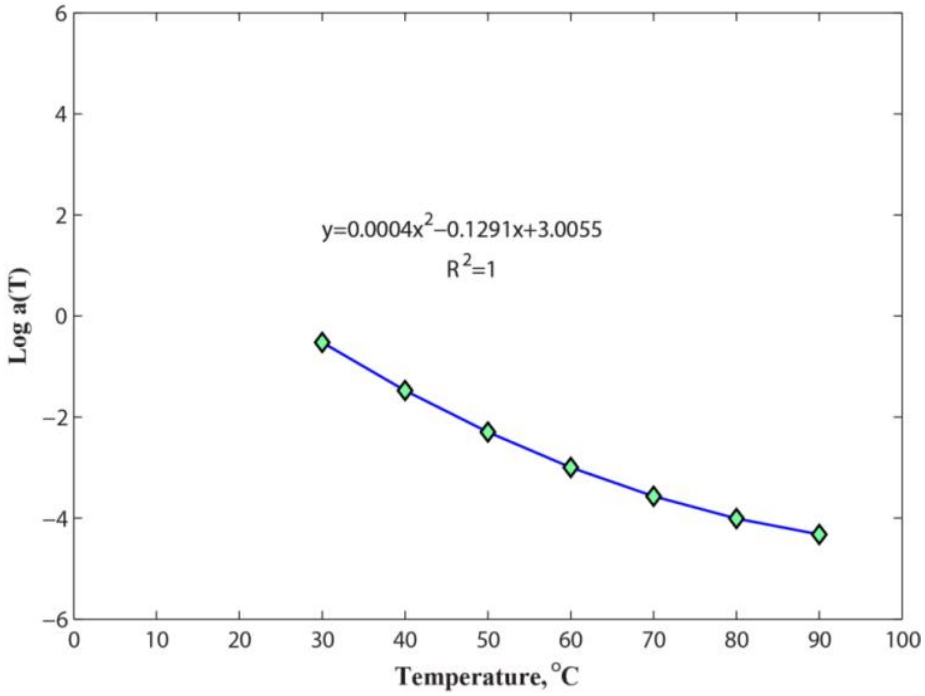


Figure 6.3 Plot of Shift Factor $\alpha(T)$ versus Temperature

Master curves and shift parameters are listed on Table 6.3. The seven shift parameters which are listed in Table 6.3 are then used in equation. (6.4) in order to calculate the $|E^*|$ of each mix at any given temperature and loading frequency within the same range that are used in the $|E^*|$ testing at reference temperature of 25 ^{o}C following the superposition.

Table 6.3 Master Curves and Shift Parameters

Mix Description	**δ**	**α**	**β**	**γ**	**a**	**b**	**%V SSD**
AC10 5.7% C320 M1 B3	0.736754	3.557723	-0.80766	0.657819	0.000819	-0.11304	5.24
AC10 5.7% A10E M2 B4	1.013825	3.160096	-0.16244	0.625139	0.000637	-0.10007	4.79
AC10 5.7% A15E M3 B4	0.923342	3.309382	-0.56845	0.496263	0.000742	-0.11319	6.35
AC10 5.7% Multi 600/170 M5 B4	1.413448	2.843524	-0.37136	0.504290	0.000723	-0.10783	6.92
AC10 5.7% A20E M6 B7	0.858570	3.392841	-0.30221	0.614766	0.000801	-0.10642	7.54
AC10 5.7% A35P (EVA) M7 B5	1.387350	2.880641	-0.74875	0.573965	0.000356	-0.11132	6.84
AC10, 5.7% C450, M10 B5	1.003145	3.232011	-0.77564	0.690726	0.000374	-0.10312	6.35

6.3.2. EFFECT OF POLYMER MODIFIERS

On the basis of this research, it is found that different polymer modifiers vary in their influences on the stiffness of mixture. Figure 6.4 compares the effect of seven different polymer modifiers on the dynamic modulus $|E^*|$. As it can be seen from plot, each of them has similar results with a similar pattern and a linear range to all. As the result of this, it has only put one figure in order to avoid repeating same figures because of the similarity (see Table 6.4). These showed that the stress dependent asphalt mix master curve using compressive dynamic (complex) modulus test data is in a good correlation with temperature. Pellinen and Witczak (Pellinen & Witczak 2002) analyzed the use of stiffness of hot-mix asphalt using a simple performance test that limit the stiffness value because of the power law and sigmoidal function.

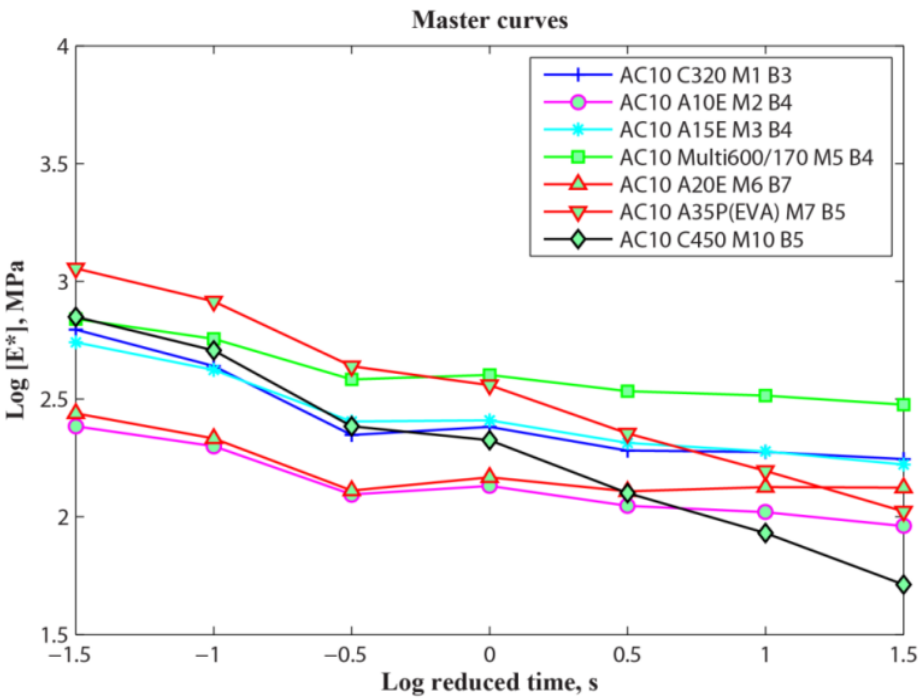


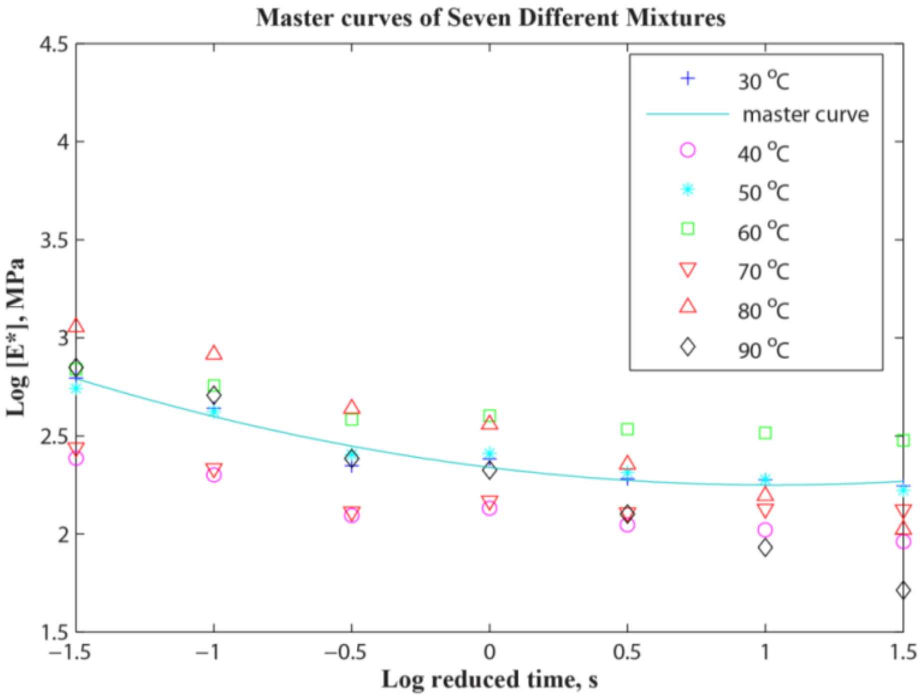
Figure 6.4 Master Curves of Seven Different Mixtures

The data at various temperature where shifted in line with frequency until the master curve merges in a single sigmoidal function, representing the master curve using a second-order polynomial relationship between the logarithm of the shift factor and the temperature (Figure 6.4). Time–temperature superposition was done by simultaneously solve the four coefficients of sigmoidal function (δ, α, β, and γ) as described in equation. (6.4) and three coefficients of the second-order polynomial (a, b, c, and c is the constant number) as equation. (6.8) and (6.9) with least squares methods.

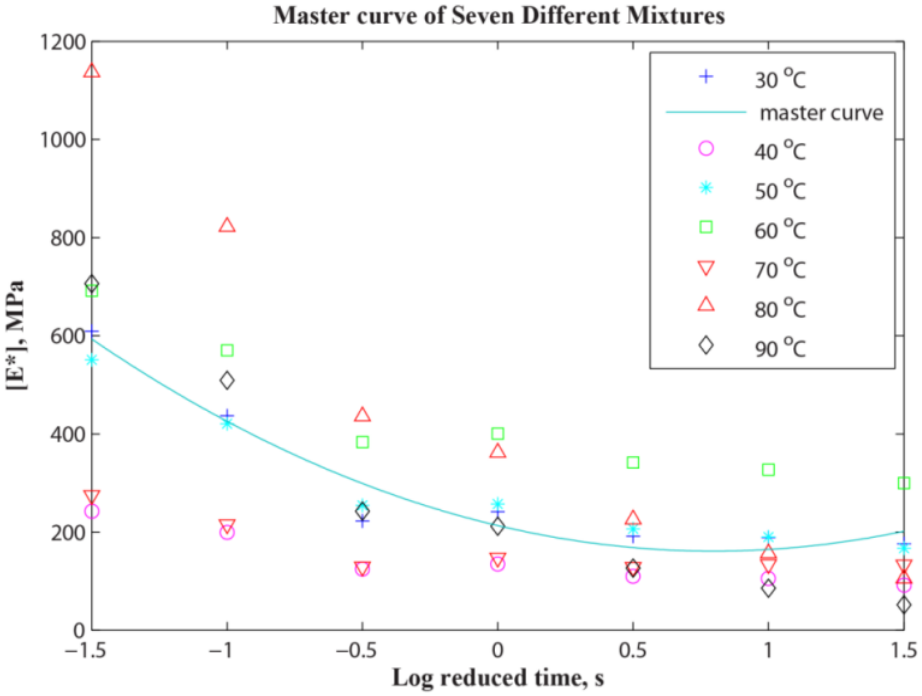
Table 6.4 Master Curves of Seven Different Mixtures

Mixes Description	log t_r(s)	log |E*| (MPa)						
C320 M1 B3	-1.0	2.79473	2.38442	2.74092	2.83997	2.43913	3.05507	2.849
A10E M2 B4	-0.3	2.64026	2.30052	2.6238	2.75591	2.33331	2.91509	2.7068
A15E M3 B4	0.0	2.34737	2.09501	2.40515	2.58353	2.11167	2.63943	2.38452
Multi600/170 M5 B4	0.7	2.38226	2.13106	2.4102	2.60249	2.16821	2.55879	2.3259
A20E M6 B7	1.0	2.28145	2.0468	2.31411	2.53373	2.10829	2.35407	2.10125
A35P (EVA) M7 B5	1.3	2.27536	2.01983	2.27816	2.51481	2.12642	2.19501	1.93154
C450 M10 B5	1.4	2.24526	1.9613	2.22218	2.47662	2.12371	2.02297	1.71278

Fitted master curves of AC10 C320 MI B3; AC10 A10E M2 B4; AC10 A15E M3 B4; AC10 Multi600/170; AC10 A20E M6 B7; AC10 A15E (EVA) M7 B5; AC10 450 M10 B7 are shown in Figure 6.5. From the data demonstrated, it can be seen that all the asphalt mixtures have a similar patterns apart Caption (a) is shown log |E*| while (b) is |E*| of the fitted master curve and has followed the linear range with the fitted master curve. These showed that the dynamic modulus master curve and shift factor are in good agreement. Garcia and Thompson (Garcia & Thompson 2007) concluded that good agreement and similar accuracy dynamic modulus obtained from laboratory test. Similarly, Kim, Seo and Momen (Kim et al. 2004) also reported that good agreement and similar accuracy dynamic modulus master curves and shift factor are obtained from laboratory test.



(a)



(b)

Figure 6.5 (a) and (b) Fitted Master Curves of AC10 C320 MI B3; AC10 A10E M2 B4; AC10 A15E M3 B4; AC10 Multi600/170; AC10 A20E M6 B7; AC10 A15E (EVA) M7 B5; AC10 450 M10 B7

6.3.3. EFFECT OF LOADING FREQUENCY AND TEMPERATURE

6.3.3.1. LOADING FREQUENCY

Loading frequency versus dynamic modulus $|E^*|$ is shown in Figure 6.6. From the data presented in the figure, it can be been that the dynamic modulus $|E^*|$ of the polymer modified mixture increase as the loading frequency increase. This showed that the dynamic modulus $|E^*|$ is small at high temperature and low frequency while it increases under the contrary condition according the principle of time-temperature superposition. For instance, take the dynamic modulus at reference temperature as shown in the figure; it is obvious that the dynamic modulus of the mixture increase with the increase of loading frequency.

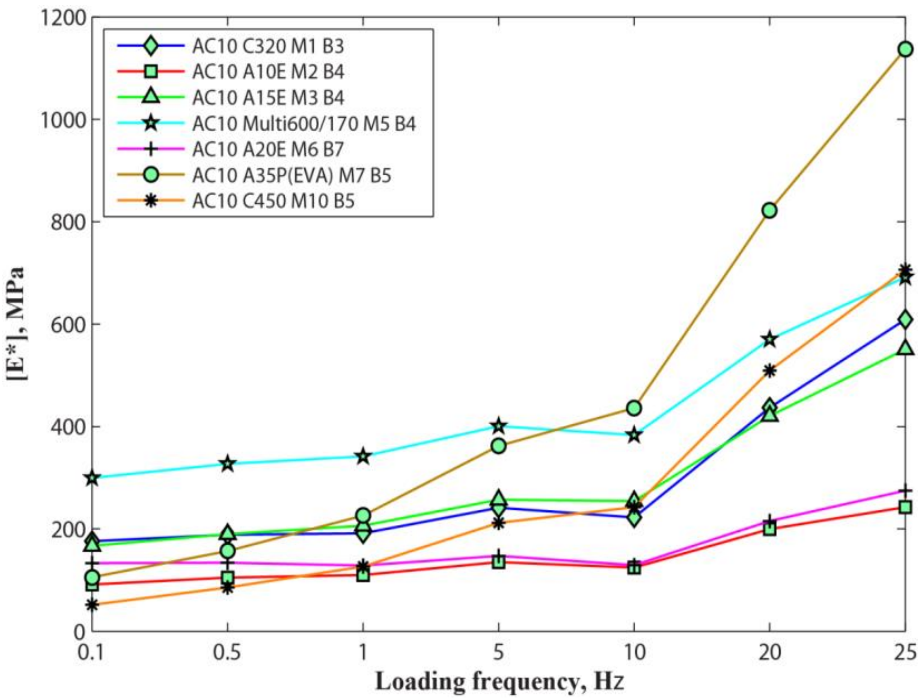


Figure 6.6 Plot of loading frequency versus |E*|

6.3.3.2. TEMPERATURE

Temperature versus $|E^*|$ are shown in Figure 6.7. From the data presented, it can be seen that mixes with AC10 A35P (EVA) modifier generally had high dynamic modulus of 1337 MPa. AC10 C450 M10 B5 and AC10 Multi 600/170 MB B4 modifiers had the second best in performance of $|E^*|$ of 706 and 691 MPa, respectively and which are lower or higher at the reduced time. This implies that the $|E^*|$ of these two modifiers are closer to the AC10 A35P (EVA) modifier according the principle of time- temperature superposition. For example, the dynamic modulus at loading frequency of 5 Hz shows that the increase in temperature will softened the asphalt binder, while the dynamic modulus $|E^*|$ is decreased. The dynamic modulus at low temperature of 30^{o}C is about twice as high temperature of 90^{o}C.

However, AC10 A10E M2 B4 and AC10 A20E M6 B7 modifiers had a very low of dynamic modulus, and these showed that the modifiers mixes might have exposed to rutting and then contribute low performance on pavement structure. While AC10 A15E M3 B7 and AC10 C320 M1 B3 modifiers are an intermediate modifiers to the principle of time-temperature superposition. However, the modifiers can be improved to rich the second level with a good aggregate asphalt mixes.

AC10 A35P (EVA), AC10 C450 M10 B5 and AC10 Multi 600/170 MB B4 polymer modifiers extremely huge with their pavement performance compared to the other asphalt mix modifiers especially both at high temperature, low frequency and low reduced time, which shows that all these three including the two intermediate modifiers can strength and stable the mixture of stiffness, and also improved the rutting resistance pavement performance according to literature (Apeagyei 2011; Garcia & Thompson 2007; Mollenhauer, Wistuba & Rabe 2009; National Cooperative Highway Research Program 2004; Schwartz 2005; Schwartz 2007; Witczak & Bari 2004; Zhu et al. 2011).

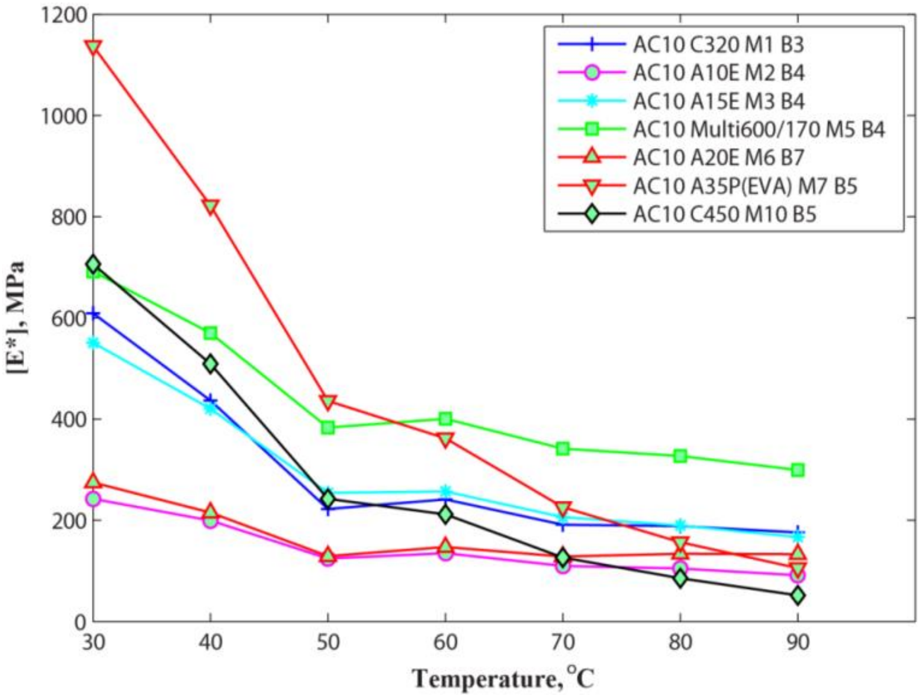


Figure 6.7 Plot of Temperature versus |E*|

6.3.3.3. BINDER VISCOSITY-TEMPERATURE SUSCEPTIBILITY

The binder data and viscosity versus temperature are shown in Table 6.5 and Figure 6.8. The complex shear modulus (G*) of binder, viscosity (ŋ), A and VTS was concurrently solved using Equation.(6.10) through (6.13). From the data presented, it can be seen that a strong correlation between binder viscosity and temperature [R^2 = 1] for polymer modified asphalt mixture. The stiffness modulus (at 2.82 T_R and 0.1Hz) ranging from 251 to 1038 MPa at phase angle of 60 degree, while binder 'A' (intercept) parameter, A = -3.5617 and the viscosity – temperature susceptibility, VTS = 10.652. This showed that a larger increase in binder viscosity implies to reducing asphalt failure temperature and improved low-temperature cracking resistance with asphalt pavement (Fabb 1974; Lewandowski 1994; Rasmussen, Lytton & Chang 2002). Since the failure temperature of asphalt is independent of the rate of cooling, it is deduced that failure occurs when the asphalt mixture attains a critical physical rate.

The Connecticut Department of Transportation (CDOT) assessed the modulus, E*, as a test method to characterize hot-mix asphalt mix design as the part of 2002 Pavement Design Guide. Results has shown a good correlation [R^2 = 0.9997] between binder viscosity -temperature (Dougan et al. 2003). Similarly, Rasmussen, Lytton and Chang (2002) has plotted VTS and found a [R^2 = 0.999].

Table 6.5 Binder Data (Viscosity –Temperature Relationship)

Temperature (^oC)	**E* (MPa)**	**G* (MPa)**	**δ**	**Temperature (Rankine)**	**Viscosity (cpoise)**	**Log Temperature (Rankine)**	**Log Log Viscosity (cpoise)**
30	601.77	1037.53	60	545.67	208.82	2.74	0.90
40	453.44	781.79	60	563.67	157.35	2.75	0.85
50	256.02	441.41	60	581.67	88.84	2.76	0.81
60	250.02	431.07	60	599.67	86.76	2.78	0.76
70	189.92	342.92	60	617.67	69.03	2.79	0.71
80	169.43	281.78	60	635.67	56.71	2.80	0.67
90	146.26	251.17	60	653.67	50.55	2.82	0.62

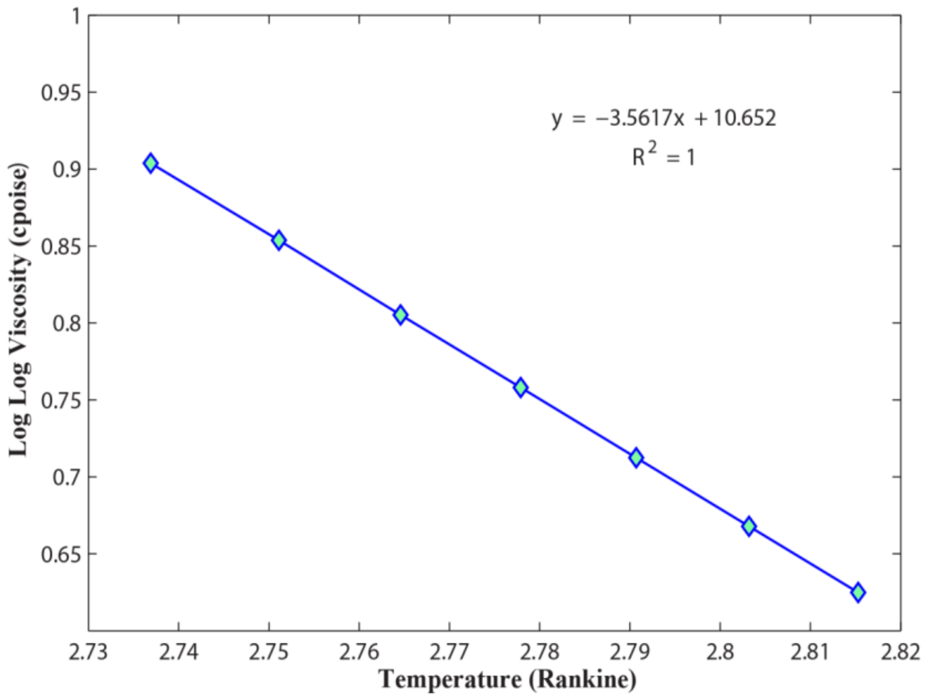


Figure 6.8 Viscosity of Binder versus Temperature for Asphalt Mixtures

6.4. CONCLUSIONS

All the asphalt polymers that are used in this research study can strength and stable the mixture stiffness of asphalt that is notable. The modification effect rank can be described as AC10 A35P (EVA) M7 B5 > AC10 450 M10 B5 > AC10 Multi 600/170 M5 B4 > AC10 C320 M1 B3 > AC10 A15E M3 B4 > AC10 A20E M6 B7 > AC10 A10E M2 B4 in this research.

The $|E^*|$ of AC10 A35P (EVA) M7 B5, AC10 450 M10 B5 and AC10 Multi 600/170 M5 B4 had been generally higher compare to the others polymer modifiers because there were mixes with aggregates, and for that matter, modifiers were functioned at high temperature and at low frequency which reduced time loading.

The good correlation (R2=1) for all the modifiers proved that the laboratory testing using various temperature can improved the rutting resistance. Dynamic modulus can be obtained as nearly as identical to the laboratory result using master curves and good correlation between the $|E^*|$ and the shifted factor as the function of temperature and time. In general, the predicting mixture performance of dynamic modulus using laboratory test with various temperature can improved the rutting and fatigue resistance of the pavement structure.

Dynamic modulus can be obtained as nearly as identical to the laboratory result using master curves and good correlation between the $|E^*|$ and the shifted factor as the function of temperature and time. A very good correlation [R^2 = 1] between binder viscosity and temperature for asphalt mixture.

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CHAPTER 7: DISTRESS IDENTIFICATION, COST ANALYSIS AND PAVEMENT TEMPERATURE PREDICTION FOR THE LONG-TERM PAVEMENT PERFORMANCE FOR WESTERN AUSTRALIA

An abridged version of this Chapter is published in *International Journal of Engineering and Technology*, vol. 7, no. 4, pp. 267-275, 2015.

ABSTRACT

Collection and analysis of pavement distress data is a significant component for effective long-term pavement performance. Accurate, consistent, and repeatable pavement distress type’s evaluation can reduce a tremendous amount of time and money that has been spending each year on maintenance and rehabilitation of existing pavement distress. The main objective of this study is to identify and quantify of surface distress in a given segment of pavement, to perform details distress rating, to predict pavement temperature and cost analysis of individual pavement distress on heavily urban roads in Western Australia (WA). Field survey were conducted from three regions in WA and two approached were used to evaluate and analysis the pavement distress. First, the probabilistic network Markov-Chains Process method was used to predict the cost analysis for individual asphalt concrete surfaced pavement distress. Second, Statistical Downscaling Model (SDSM) was used to predict pavement temperature for asphalt concrete surface pavement. Meteorological data were collected from Perth, Kalgoorlie, and Albany region in WA, and data were used to develop and validation of the model. Different types of pavement distress level were identified and color photograph illustrated the asphalt concrete surfaced pavement. Results were performed and analysis. Results from this study will be useful resource to Main Roads Western Australia, Western Australia State Highways (WASH), and other pavement related users including to the National Highway System (NHS). In addition, results can be used for pavement management systems (PMSs) purpose.

Author keywords: Pavement distress, crack identification, cost analysis, pavement temperature, pavement management, Western Australia.

7.1. INTRODUCTION

In 1987, the Strategic Highway Research Program (SHRP) began the largest and most comprehensive pavement performance in history ever-the *Long-Term Pavement Performance* (LTPP) program (Strategic Highway Research Program 1993). The *Distress Identification Manual* for the *Long-Term Pavement Performance* project was developed to provide a consistent, uniform basis for collecting distress data for the LTPP program. It will allow states and others to provide accurate, uniform, and comparable information on the condition of LTPP test sections. During the program’s

20- year life, highway agencies in United States and other Countries has been collected data on pavement condition, climate, and traffic volumes and loads from more than an thousand pavement test sections (Strategic Highway Research Program 1993). Although developed as a tool for the LTPP program, the manual has broader application. It provides a common language for describing cracks, potholes, rutting, spelling and other pavement distresses being monitored by the LTPP program. Although not specifically designed as a pavement management tool, the *Distress Identification Manual* can play an important role in a state's pavement management program by ridding reports of inconsistencies and variations caused by a lack of standardized terminology. Most pavement management program do not need to collect data at the level of detail and precision required for the LTTP program, nor are the severity level used in the manual necessary appropriate for all pavement management situations.

Hot-mix asphalt (HMA) is a viscoelastic structural material and its load carrying of the pavement varies with temperature (Nega, Nikraz & Leek 2013b; Nega et al. 2013b). While accurately determine insitu strength characteristics of flexible pavement are necessary to identify the type of pavement distress and also to predict the temperature. The majority of previously published research either on distress identification or pavement temperature has consisted predicting the annual maximum or minimum pavement temperature to recommend a suitable asphalt performance grade (Bayat, Kasani & Soleymani 2011; Deacon et al. 1994; Diefenderfer, Al-Qadi & Diefenderfer 2006; Mills et al. 2009). However, the predict of pavement temperature has not be related to the pavement distress type, identification and characterization of asphalt concrete surfaced pavement so that cost analysis of individual pavement distress can be included and also analyzed. Thus, to determine long-term pavement performance, pavement distress identification, predict pavement temperature and cost analysis of individual pavement distress are necessary.

The use of full depth asphalt pavements to construct and rehabilitate heavily loaded urban roads has rapidly grown in Western Australia (WA) over the past 5 years. In 2006/7, almost $429 million was expended on road network maintenance which made up 38% of the total road program (Main Roads Western Australia 2007). The following are some of the works undertaken during the year. Eight regionally based 10-year Term Network Contracts (TNCs) were established to provide road maintenance and rehabilitation services on the State road system and for regulatory signs and road lines on local roads. The contracts provide a range of maintenance services to help ensure that road users are provided with a safe and efficient road system and that the value of the road asset is preserved. During the year $131 million was spent on direct contract payments (Main Roads Western Australia 2007).

The main objective of this study is to identify and quantify of severity of surface distress in a given segment of pavement, to perform details distress rating, to predict pavement temperature and cost analysis of individual pavement distress for Main Roads of Western Australia so that long-term pavement performance can be achieved. This study will be useful resource to the Main Roads Western Australia (MRWA), Western Australia State Highways (WASH) and other pavement related users including to the National Highway Systems (NHS). In addition, it can be used for pavement management systems (PMSs) purposes. Figure 7.1 shows the Main Roads Networks in Western Australia, and the most common types of pavement cracks in Western Australia are shown in Table 7.1.

Figure 7.1 Main Roads Networks in Western Australia

Table 7.1 Most Common Pavement Cracking in Western Australia

Cracking Type	Defined Severity Levels
Fatigue cracking (m^2)	Yes
Block cracking (m^2)	Yes
Longitudinal cracking (m)	Yes
Reflection cracking at joint (no or m)	Yes
Transverse cracking (no or m)	Yes

7.2. METHODS

7.2.1. TRAFFIC ROAD SURVEY

Field data was conducted to collect data for evaluating the long term pavement performance in Western Australia (WA). This data was collected in Perth, Kalgoorlie and Albany between January and March 2014. Data were collected by the author and staff from Curtin University in Western Australia.

Thirty six roads survey were used to identified and characterized the types of pavement distresses and Distress Identification Manual for Long-Term Pavement Performance by Strategic Highway Research Program (Strategic Highway Research Program 1993) was used as a guidance. depth, width, and length measurements of the pavement distress were taken from each asphalt concrete surfaced pavement roads.

7.2.2. PAVEMENT NETWORK MANAGEMENT TOOLS

Linear and non-linear programming models are the two main types of algorithms utilized by researchers in developing pavement management optimization models (Gao, Chou & Wang 2010). In linear programming models, key assumptions of all functions that includes objective and constrain function are consider as linear. However, in non-linear programming, this assumption does not accumulate at all (Hillier & Lieberman 2010). Abaza and Ashur (Abaza & Ashur 1999) developed their model based non-linear programming. Pavement condition prediction models are significant component of pavement optimization models. These are two types of prediction models: deterministic models and probabilistic models. According to Butt et al. (1994), the pavement deterioration rates are often “uncertain”, frequently used the probabilistic model based on the Markov process approach to evaluate and analysis the pavement condition (Chen et al. 1996).

7.2.2.1. NON-LINEAR MODEL ALGORITHM

The non-linear model for pavement maintenance and rehabilitation optimization is formulated as follows (Abaza & Ashur 1999; Gao, Chou & Wang 2010):

Minimize

$$\sum_{t=1}^{T} \sum_{j`=1}^{5} S_{tj} X_j LC_J \tag{7.1}$$

Subject to State transition constrains:

$$S_{tj} = \sum_{t=1}^{5} S_{t-1i} \{1 - X_i) DN_{ij} + X_i P_{ij}\} \, for \; all \; t = 2, \ldots T; j = 1,2, \ldots 5 \tag{7.2}$$

Non-negativity constraints:

$$X_i \geq 0 \; for \; all \; i = 1, \ldots 5 \tag{7.3}$$

Sum to one constraints:

$$\sum_{k=0}^{4} X_{jk} = 1 \; for \; \; i = 1, \ldots 5 \tag{7.4}$$

Target condition constraints:

$$S_{Tj} \leq e_{Tj} \; for \; selected \; j \tag{7.5}$$

Budget constrains:

$$\sum_{j=1}^{5} S_{tj} X_j LC_J \leq B_t \; f \; or \; t = 1, \ldots t \tag{7.6}$$

where S_{tj} is the proportion of pavement in state j at year t; X_i is proportion of pavement i receiving treatment; T is number of analysis years; C_J is unit cost of applying treatment to pavement in state j; DN_{ij} is probability that receiving no treatment moves from i to state j; P_{ij} is probability that pavement receiving new treatment transit from state i to state j; e_{Tj} is upper limit of proportion of pavement in condition j in final year T; and B_t is maximum available budget in year t.

7.2.3. STATISTICAL DOWNSCALE MODEL

Statistical Downscale Model (SDSM) is multiple regression based tool proposed by Wilby, Dawson and Barrow (2002) to describe the linkage between coarse scale General Circulation Model (GCM) daily climate predictors and daily maximum or minimum temperature of selected station. SDSM is a combination of the stochastic weather generator approach and a transfer function model with high performance in capturing future inter-annual variability (Wilby, Dawson & Barrow 2002). In downscaling the GCM predictors, SDSM develops inter relationship between predictor (i.e. daily minimum temperature, maximum temperature, rainfall) and predictand (GCM variables). To select the most appropriate GCM predictors, SDSM provides linear correlation analysis by percentage of explained variance (E %), correlation matrix and scatter plots.

SDSM model is calibrated and validated in monthly basis for three selected regions by considering the daily maximum temperature and minimum temperature as the predictand variables. Initially 26 NCEP variables are subjected for predictor selection and by scatter plot, correlation analysis, explained variance facilities most appropriate predictors are selected.

7.2.3.1. STUDY AREA AND DATA SETS

Three airports located in Western Australia are subjected to this study. These locations are highly urbanized and road network is highly grown. To obtain the high resolution daily maximum and minimum temperature for these regions SDSM model is employed to downscale GCM predictors. Daily maximum and minimum temperature of each site were obtained from Bureau of Meteorology (BoM), Australia and used as the predictand variable in SDSM model. National Centre for Environmental Prediction (NCEP) re analyzed data are used as the predictors in SDSM model calibration and validation. In future temperature downscaling Canadian Global Climate Model (CGCM3) data under A2 scenario for the period of (1961-2100) are employed. The details of study area are shown in Table 7.2.

Table 7.2 Details of Study Area

Region	Latitude	Longitude	Observed max/min temp. period
Perth airport	31.9522° S	115.8589° E	1961-1990
Kalgoorlie	30.7487° S	121.4658° E	1961-1990
Albany	35.0228° S	117.8814° E	1971-2000

7.3. CRACK IDENTIFICATION AND CHARACTERISTICS

7.3.1. FATIGUE CRACKING

Fatigue cracking, also known as alligator cracking, is single crack or a series of interconnected cracks caused by fatigue failure of the asphalt concrete (Oregon Department of Transportation 2010). They are the result of repetitive traffic loads (wheel paths), and high deflection often due to wet bases or subgrade but also maybe present anywhere in the lane due to traffic wander. These types of cracking can also lead to potholes and pavement disintegration. A series of interconnected cracks characterizes in early stages of development. It eventually develops into many-sided, sharp-angled pieces, usually less than 0.3 m (1 ft) on the longest side. Characteristically has chicken wire/alligator pattern in later stages (Strategic Highway Research Program 1993). Longitudinal cracks occurring in the wheel path are rated as fatigue cracking.

An area of cracks with no or only a few connecting cracks, where a crack are not spalled or sealed and with no pumping is evident are considered as low severity fatigue cracking, whereas, if an area of interconnected cracks are forming a complete pattern, where cracks may be slightly spalled or sealed with no pumping is evident are defined as moderate severity fatigue cracking. However, where sections of an area are moderately or severely spalled, multiple interconnected cracks are forming a complete pattern, pieces are missing or move when subjected to traffic or cracks may be sealed and pumping may be evident across the entire pavement roadway are described as high severity fatigue cracking (Oregon Department of Transportation 2010; Roberts et al. 1996; Roberts, Mohammad & Wang 2002; Strategic Highway Research Program 1993). This type of failure cannot be treated with crack sealing and/or filling.

7.3.2. BLOCK CRACKING

Block cracking is a pattern of cracks that divides the pavement into approximately rectangular pieces. Block cracking is a pattern cracks that divide the pavement into approximately rectangular pieces or blocks. Block cracking, unlike fatigue cracking, will occur throughout of the pavement width, not only in the wheel paths. The blocks range in size from an approximately 0.1sq.m to 10sq.m. (1 sq. ft to 100 sq. ft) (Strategic Highway Research Program 1993). These cracks are the result of age hardening of the asphalt coupled with shrinkage during cold weather, and can be effectively treated with crack sealants.

7.3.3. LONGITUDINAL CRACKING

Longitudinal cracks are cracks that are predominantly parallel to pavement's centerline. Location within lane (wheel path versus non-wheel path) is significant. These are caused by thermal stress and/or traffic loading (Strategic Highway Research Program 1993). They occur frequently either at joint between adjacent travel lanes or in between a travel lane and the shoulder, where the hot-mix asphalt density is lower and air voids are higher (Roberts et al. 1996). Majority cracks are within 25 mm (1 in) of skip strip or fogs strip/edge of pavement or within 25 mm (1 in) of the middle of the lane (Oregon Department of Transportation 2010). Cracks may meander into the wheel path, but generally stay out of the wheel path.

Longitudinal cracking sometimes can be associated with raveling, poor adhesion or stripping. Longitudinal cracks which occur in the wheel path and cracks less than mean width 6 mm (0.25) should be rated as low severity fatigue cracking. The cracks range from mean width of 6 mm (0.25 in) to 19 mm (0.75 in) should be also rated as moderate severity longitudinal cracking whereas, if it is greater than mean width 19 mm (0.75 in) and then, it should be rated as high severity longitudinal cracking (Strategic Highway Research Program 1993). There are two types of longitudinal cracking: wheel path and non-wheel path longitudinal cracking.

7.3.4. REFLECTION CRACKING AT JOINT

Reflection cracking is a crack in asphalt concrete overlay surfaces that occur **o**ver joints in concrete pavements. These cracks are caused either by cracks or other discontinuities movement with an underling pavement surface that propagate up due to movement at the crack (Smith & Romine 1999; Smith 1991; Smith et al. 1984). An unsealed crack with a mean width of less than 6 mm (0.25 in.); or a sealed crack with sealant material in good condition and with a width that cannot be determined has low severity, and any crack with a mean width greater than 6 mm (0.25 in.) and less than 19 mm (0.75 in.) can be considered as medium severity, and this may also associated with low severity random cracking (Strategic Highway Research Program 1993; 1994a). Any crack with a mean width greater than or equal 19 mm can develop adjacent moderate to high severity random cracking. They are two types of reflection cracking: transverse and longitudinal reflection cracking.

7.3.5. TRANSVERSE CRACKING

Transverse cracking is cracks that are predominantly perpendicular to pavement centerline, and are not located over Portland cement concrete joints. Transverse cracks are generally caused by thermally induced shrinkage at low temperature. When the tensile stress due to shrinkage exceeds the tensile strength of the hot-mix asphalt pavement surface and then, crack occur (Oregon Department of Transportation 2010; Roberts et al. 1996). These cracks can be effectively treated with crack sealants. An unsealed crack with a mean width of less than 6 mm (0.25 in.); or a sealed crack with sealant material in good are described as low severity, and any crack with a mean width greater than 6 mm (0.25 in.) and less than 19 mm (0.75 in.) can be considered as medium severity, and this may also associated with low severity random cracking. Any crack with a mean width greater than or equal 19 mm can develop adjacent moderate to high severity random cracking (Strategic Highway Research Program 1993; 1994a).

7.4. PAVEMENT TEMPERATURE

Characterization of the insitu strength performance of highways constructed using hot-mix asphalt (HMA) is difficult because of viscoelastic behavior (Al-Qadi et al. 2009; Nega, Nikraz & Leek 2013b). These component materials exhibiting various properties contribute to complex mechanical behaviour of HMA, which can be characterised as elastic viscos elastic, and plastic under different condition such as temperature, load application and aging (Dibike et al. 2001; Gopalakrishnan & Kim 2011; Nega et al. 2013b). Diefenderfer, Al-Qadi and Diefenderfer (2006) highlighted highways that are subjected to heavy loading can cause significant damage capacity of the pavement varies with temperature.

Asphalt is a viscoelastic material, which means that its stiffness is dependent on temperature and rate of loading. The fatigue damage, or cracking of an asphalt pavement caused traffic load is influenced by the stiffness properties of the mix and distribution of stresses and strain through this layer. The level of tensile strain in asphalt is dependent on temperature and this effect can be considered in terms of the influence of temperature on mix stiffness (Koole, Valkering & Stapel 1989; Rao Tangella et al. 1990; Walubita 2006). Deacon et al. (1994) investigated the effect of temperature on pavement life and development of temperature equivalency factors for fatigue, performed controlled strain, flexural fatigue tests at four temperature ranging from 5^oC to 25^oC (Baburamani 1999; Rao Tangella et al. 1990). The initial flexural stiffness and the slope of the initial strain-fatigue life were found to be sensitive to temperature.

7.5. PAVEMENT MANAGEMENT SYSTEMS

Pavements are an important part of highway transportation infrastructure that constitutes an enormous investment of public funds. A tremendous amount of time and money is spent each year on construction of new pavements as well as on maintenance and rehabilitation (M&R) of existing pavement. To maximize benefits and minimize overall costs, a systematic and scientific approach is needed to manage pavements (Lee, Park & Mission 2013). Pavement management systems (PMSs) provide consistent, objective, and systematic procedures to determine priorities, schedule allocating resources and budgeting for pavement M&R (Federal Aviation Administration 2006). Typical unit costs and expected life typical pavement maintenance treatments are shown in Table 7.3.

Table 7.3 Typical Unit Costs and Expected Life of Typical Pavement Maintenance Treatments

Treatment	Code	Expected Life of Treatment			
		Cost/m^2	Min	Average	Max
Crack sealing	CS	$1.50	2	3	5
Fog seals	FS	$1.50	2	3	4
Slurry seals	SS	$10.00	3	5	7
Microsurfacing	MS	$10.00	3	7	9
Chip seals	CS	$8.76	3	5	7
Asphalt overlay DGA 30 mm	AS30	$17.63	2	5	10
Asphalt overlay DGA 40 mm	AS40	$23.58	2	5	10
Asphalt overlay DGA 60 mm	AS60	$35.33	2	5	10
Asphalt overlays DGA 90 mm	AS90	$48.35	2	5	10
Asphalt overlays SMA 30 mm	SMA30	$24.12	2	5	10
Asphalt overlays SMA 40 mm	SMA40	$29.56	2	5	10
Asphalt overlay SMA 60 mm	SMA60	$45.07	2	5	10
Asphalt overlay SMA 90 mm	SMA90	$59.85	2	5	10
Asphalt overlay SAMI DGA 30 mm	SAS30	$28.45	2	7	12
Asphalt overlay SAMI DGA 40 mm	SAS40	$34.40	2	7	12
Asphalt overlay SAMI SMA 30 mm	SSMA30	$34.94	2	7	12
Asphalt overlay SAMI SMA 40 mm	SSMA40	$40.38	2	7	12

Note: The costs would be expected to vary with size and/or location of job. The expected lives would also very depending on the traffic loading and environmental conditions (such as temperature, aging, healing and resting).

Pavement engineering management systems uses the systems approach to provide a unified treatment of pavement design, testing, construction, maintenance, evaluation, and restoration (Haas & Hudson 1978; Nega, Nikraz & Leek 2013a). Improving road safety through proper pavement engineering and maintenance should be one of the major objective of pavement management systems (Tighe et al. 2000). When pavement are evaluated in terms of safety, a number of factor related to pavement engineering properties are raised, such as pavement geometric design, pavement materials and mix design, pavement surface properties, shoulders type and pavement color and visibility (Tighe et al. 2000). A good pavement engineering management system requires an accurate and efficient pavement performance (Butt et al. 1987) so that prediction models based on the Pavement Condition Index and the age of the pavement can be developed.

7.6. RESULTS AND ANALYSIS

7.6.1. DISTRESS IDENTIFICATION AND CHARACTERISTICS

A summary of most pavement distress and characteristic types of asphalt concrete surfaced pavements of Western Australia are shown in Table 4. From the data presented, it can be seen that the majority of asphalt surfaced pavements roads have fatigue, longitudinal and transverse cracking as compared to others types of distress. The crack mean widths of these are also high. This showed that the annual daily traffic (ADT) in heavily loaded urban roads has been increasing to cause all these pavement distress. The Strategic Highway Research Program (Strategic Highway Research Program 1993) identified the pavement distress with asphalt concrete surfaced pavements into five main categories: cracking (fatigue, block, edge, longitudinal, reflection and transverse cracking); patching and potholes; surface deformation; surface defects and miscellaneous.

(Guyer 2009) evaluated pavement thickness that must be design to withstand the anticipated traffic roads for the design life of pavement. Increasing the grow weight by as little 10 percent can equivalent to increase the volume of traffic by as much as 300 to 400 percent and imposed largely a fatigue, longitudinal and transverse effect on the flexible pavement as a rapidly increased number of loads repetition per vehicle operation.

Table 7.4 Distress Types of Asphalt Concrete Surfaced Pavement in Western Australia

Road Name	Mix Type during Construction	Cracking Type	Defined Severity Levels	Crack widths (mm)
Welshpool	AC14 -75 Blow	Transverse Cracking	Yes	20.1
Mills	AC14 -75 Blow	Fatigue Cracking	Yes	20.3
Kurnall	AC14 -75 Blow	Reflection Cracking	Yes	10.6
Dowd	AC14 -75 Blow	Fatigue Cracking	Yes	19.3
Carousel	AC10 -50 Blow	Longitudinal Cracking	Yes	20.3
Carden	AC14 -50 Blow	Longitudinal Cracking	Yes	18.6
Montrose	AC10 -35 Blow	Block Cracking	Yes	10.6
Metcalf	AC10 -35 Blow	Potholes	Yes	240.4
High	AC10 -75 Blow	Transverse Cracking	Yes	11.7
Bannister	AC14 -75 Blow	Fatigue Cracking	Yes	18.4
Vinicombe	AC14 -75 Blow	Longitudinal Cracking	Yes	19.4
Riley	AC10 -50 Blow	No Cracking	Yes	0

Distress with asphalt concrete surfaced pavements of high severity fatigue cracking is shown in Figure 7.2. This longitudinal fatigue crack has a mean width of 20 mm and occurs in areas where subjected to repeated traffic loading (wheel paths). In the Distress Identification Manual for Long-Term Pavement Performance by Strategic Highway Research Program (Strategic Highway Research Program 1993) described an area of moderately or severely spalled interconnected crack forming with a complete pattern as high severity cracking and cracks should immediately be sealed. Oregon Department of Transportation (Oregon Department of Transportation 2010) on the Pavement Distress Survey Manual has reported a single longitudinal fatigue should be considered to have a width of 12 mm (0.5 in.). If different severity levels exist with an area that cannot easily be distinguished and then, it should use highest severity level.

Figure 7.2 High Severity Fatigue Cracking

A moderate block cracking of asphalt concrete surfaced pavement area is shown in Figure 7.3. This crack has a mean width of 11 mm. From the distress area, it can be seen that cracks divided the pavement surface into approximately rectangular pieces, and typically occurred throughout the pavement width, and not just in the wheel paths. Cracks with a mean width > 6 mm (0.25 in.) and ≤ 19 mm (0.75 in) can be considered as moderate severity block cracking (Strategic Highway Research Program 1993).

Figure 7.3 Moderate Severity Block Cracking

A high severity longitudinal cracking of distress asphalt concrete surfaced pavements area is shown in Figure 7.4. This crack has a mean width of 20 mm. From the data presented, it can be seen that cracks are predominantly parallel to pavement centerline, which is located within the lane (wheel path versus non-wheel path) is significant. In the Distress Identification Manual for Long-Term Pavement Performance (LTPP) developed by Strategic Highway Research Program (Strategic Highway Research Program 1993) and Pavement Distress Survey Manual developed by Oregon Department of Transportation (Oregon Department of Transportation 2010) described any crack with a mean width > 19 mm (0.75 in.) is considered as high severity longitudinal cracking while any crack ≤ 19 mm as adjacent moderate to high severity random cracking.

Figure 7.4 High Severity Longitudinal Cracking

Asphalt concrete surfaced pavement with moderate severity reflection cracking is shown in Figure 7.5. This crack has a mean width of 11 mm. From the distress area, it can be viewed that cracks in the asphalt concrete are in the overlay surface, which was at joints. Any cracks with a mean width > 6 mm (0.25 in) and ≤ 19 mm (0.75 in.) is considered as moderate severity reflection cracking (Smith & Romine 1999; Strategic Highway Research Program 1993).

Figure 7.5 Moderate Severity Reflection Cracking

Pavement distress with moderate severity transverse cracking is shown in Figure 7.6. This crack has a mean width of 16 mm. From the data presented, it can be viewed that cracks are predominantly perpendicular to pavement centerline, and are not actually located over Portland cement joints. According to (Oregon Department of Transportation 2010; Smith & Romine 1999; Strategic Highway Research Program 1993), any crack with a mean width > 6 mm (0.25 in) and ≤ 19 mm (0.75 in.) is considered as moderate severity transverse cracking. Figure 7.7 shown asphalt concrete surfaced pavement with no cracking.

Figure 7.6 Moderate Severity Transverse Cracking

Figure 7.7 Pavement with No Cracking

7.6.2. COST ANALYSIS

A summary of cost analysis for road expenditure of Main Roads Western Australia is shown in Figure 7.8. From the data presented, it can be seen that road maintenance had high cost of Aus $581.475 million as compared to the others expenditure. Results are shown that a tremendous amount of time and many spent each year on maintenance and rehabilitation of existing pavement as well as on construction new pavements. (Lee, Park & Mission 2013) recommended a systematic and scientific approach to maximum benefits and minimize overall costs so that long-term pavement performance will be managed and achieved.

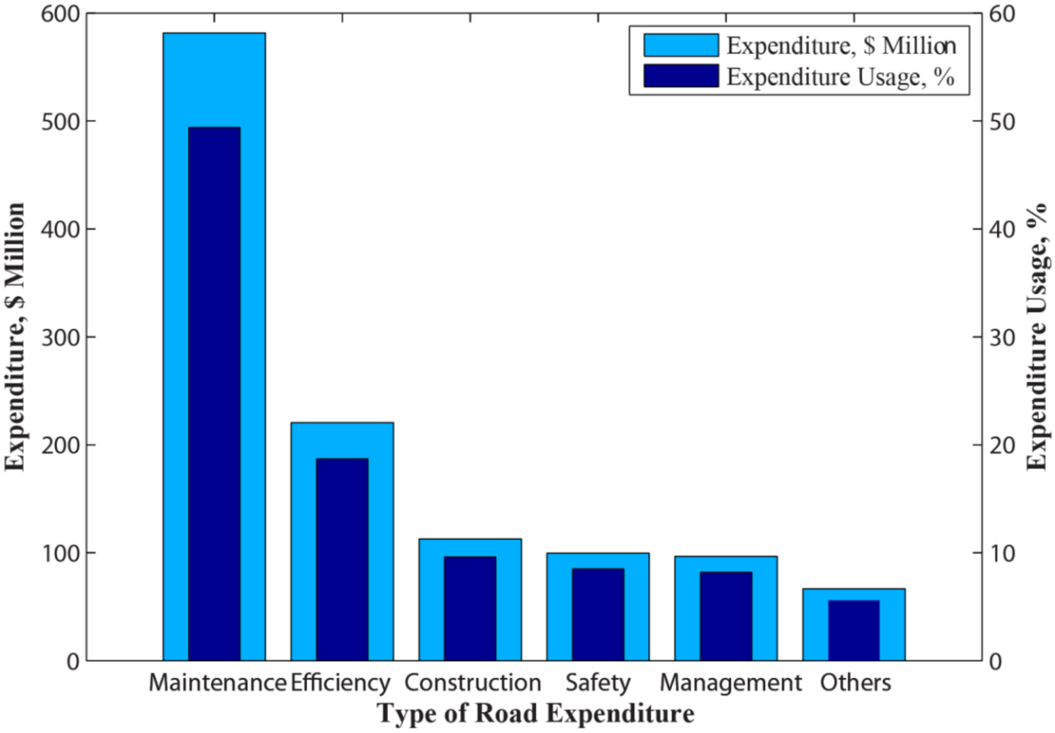
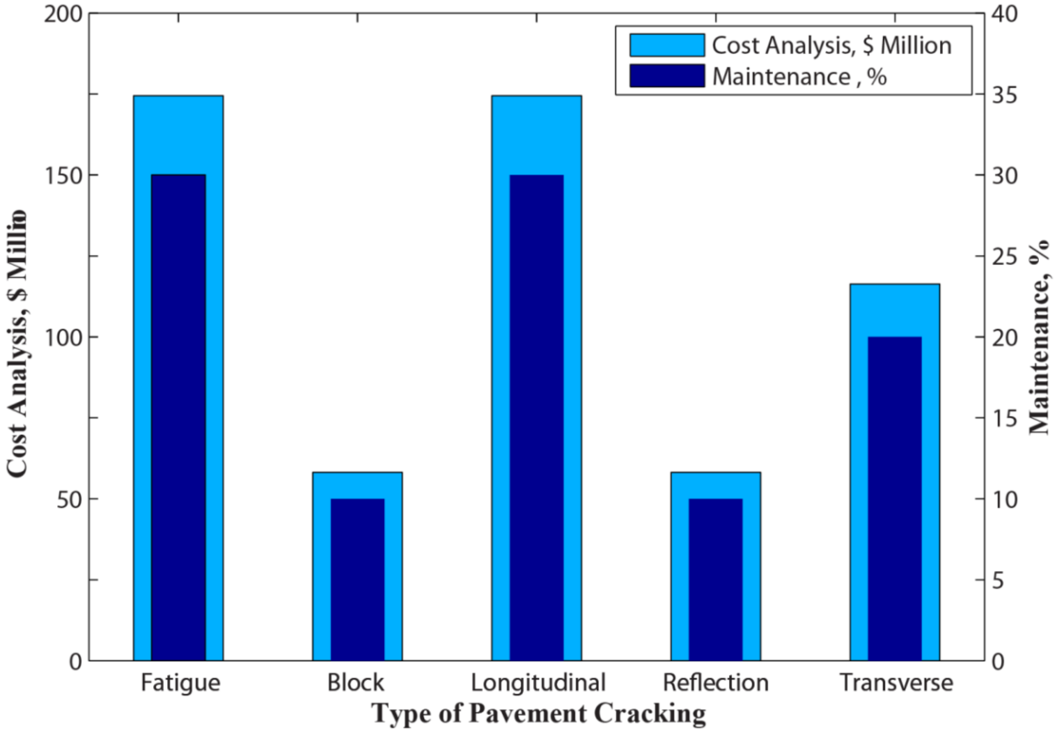


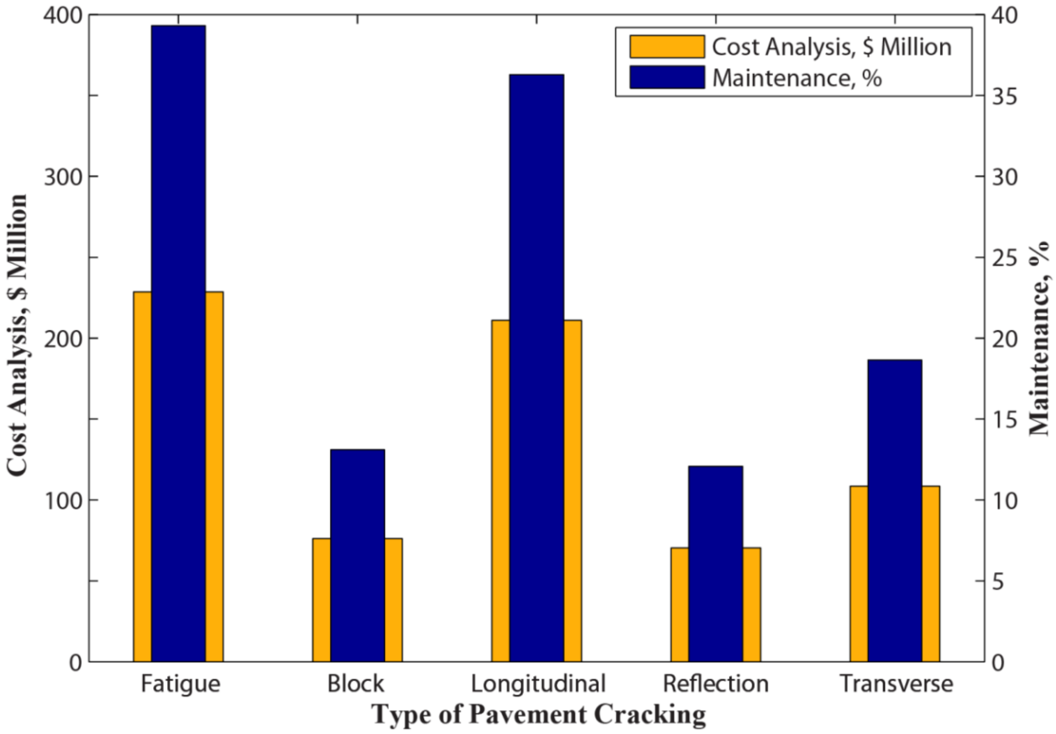
Figure 7.8 Road Expenditure Cost Analysis for Main Roads Western Australia

Cost analysis predicting non-linear model using probabilistic network chain process for different type of cracking of asphalt concrete surfaced pavement are shown Figure 7.9. From the data demonstrated, it can be seen that all cost analysis predicting have a similar patterns apart Caption (a) is shown for a year 2011 while Caption (b) and (c) for 2012 and 2013, repectively. From the predict model analysis, it can be seen that the cost for fatigue and longitudinal cracking are high and similar in pattern as compared to block, reflection and transverse cracking. This indicates that a tremendous amount of time and money has been spending to fatigue and longitudinal cracking maintenance and rehabilitation. Deterioration of flexible pavement can be increased because of traffic loading and environmental factors in a heavily urban roads According the FHWA guide fatigue cracking should not exceeding 25 percent of the total area within the first 15 years' service (Stubstad et al. 2012). Pavement management systems (PMSs) provide consistent, objective, and systematic procedures to determine priorities, schedule allocating resources and budgeting for pavement M&R (Federal Aviation Administration 2006; Nega, Nikraz & Leek 2013a).



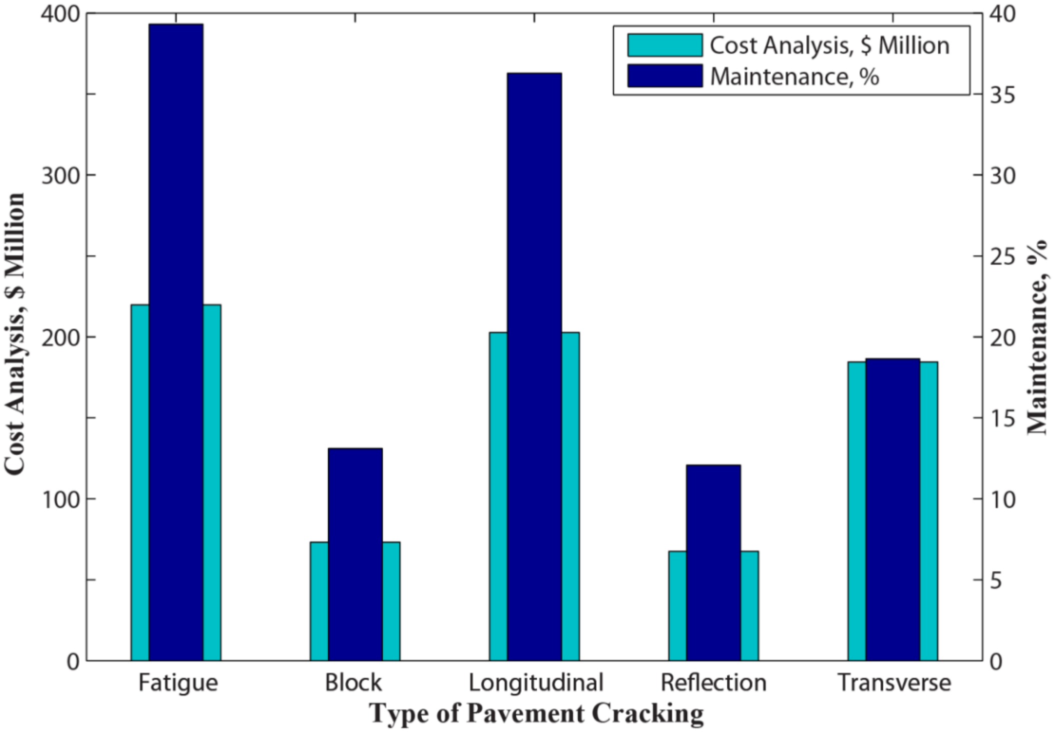
Cost Analysis, $ Million
Maintenance , %
Cost Analysis, $ Millio
Maintenance, %
Type of Pavement Cracking
Fatigue
Block
Longitudinal
Reflection
Transverse
200
150
100
50
0
40
35
30
25
20
15
10
5
0

(a)



Cost Analysis, $ Million
Maintenance, %
Cost Analysis, $ Million
Maintenance, %
Type of Pavement Cracking
Fatigue
Block
Longitudinal
Reflection
Transverse
400
300
200
100
0
40
35
30
25
20
15
10
5
0

(b)



(c)

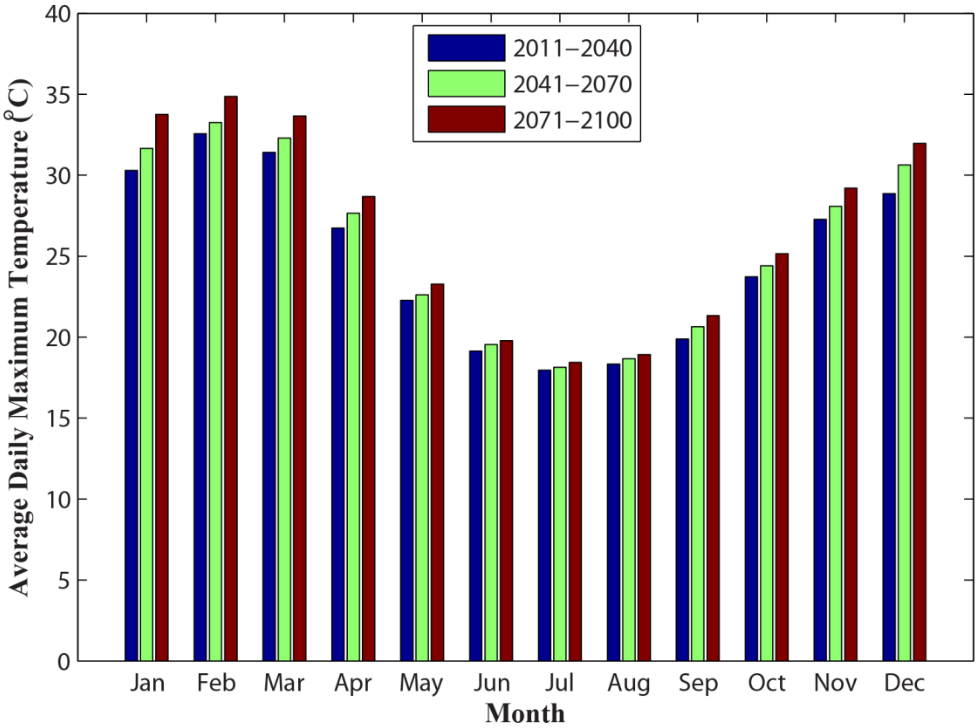
Figure 7.9 Caption (a), (b) and (c) Cost Analysis for Different Type of Pavement Cracking of Asphalt Concrete Surfaced Pavement in Year; 2011, 2012 and 2013 of Western Australia

7.6.3. CALIBRATION AND VALIDATION OF SDSM MODEL

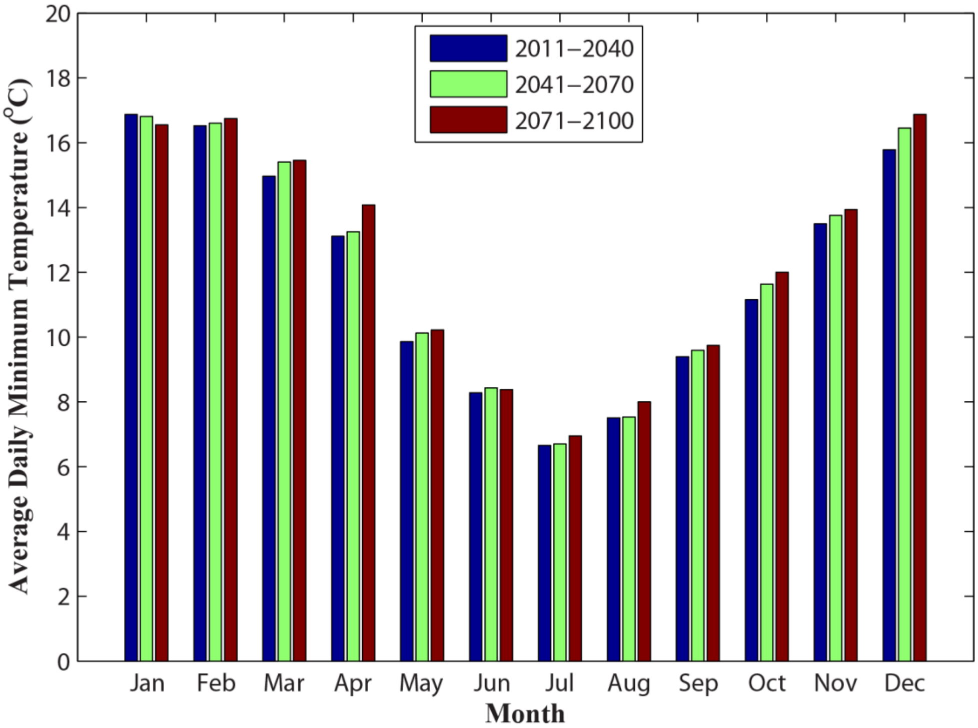
The future pavement temperature predicted using the SDSM model in downscaling GCM temperature for the selected regions are shown in Figure 7.10. From the data performed, it can be seen that all future temperature predicted for all selected regions have almost a similar results and followed similar patterns. Thus to avoid a repetition, results has presented only for Perth heavily urban roads regions. Caption (a) is shown average daily maximum temperature while Caption (b) is average daily minimum temperature. From the predicted analysis, it can be seen that future maximum and minimum daily temperature forecast for Perth region shows increasing trend for the period of 2011-2040 while it shows a decrease for the period of 2071-2100.

The predicted model shows a significant increment of daily maximum and minimum daily temperature for summer months (December to March). For example, January has an average daily maximum temperature of 30 ^{o}C for the period 2011-2040 while 32 ^{o}C and 34 ^{o}C for 2040-2070 and 2071-2100, repectively. Therefore, this temperature increment should be taken into account for the sensitive flexible pavement design process so that long-term pavement performance can be achieved. However, average daily minimum temperature in January does not show increasing tread but decreasing in tread. This showed that minimum temperature increment takes low value as compared to maximum temperature increment, and this temperature variation in a large range with a short period of time can affect the flexible pavement design and pavement performance.

Mills et al. (2006) described that temperature variation in a huge range can highly affect the performance of pavement infrastructure, and create different type of pavement distress. Similarly, (Mills et al. 2009) analyzed the effect of temperature variation for flexible pavement design, and recommended that pavement engineers should take into consideration to the temperature variations during pavement design. Maintenance and rehabilitation (M&R) to the pavement distress should require earlier in the design life.



(a)



(b)

Figure 7.10 Prediction of Future Daily Average Maximum and Minimum Pavement Temperature

7.7. CONCLUSIONS

Distress identification, prediction of cost analysis for pavement distress and pavement temperature for long-term pavement performance has been achieved. The Markov Chain process (non-linear model) approach and the statistical downscale (SDSM) model can be used to evaluate and analysis the pavement temperature for long-term pavement performance. It is highly recommended to use a systematic and scientific approach to maximum benefits and minimize overall costs so that long-term pavement performance will be achieved.

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CHAPTER 8: SIMULATION OF SHAKEDOWN BEHAVIOR FOR FLEXIBLE PAVEMENT'S UNBOUND GRANULAR LAYER

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ABSTRACT

Full-depth asphalt concrete pavements are generally designed to control fatigue cracking and reduce potential rutting when subjected to repeated heavy traffic loads. A particular interesting question is whether a limit load exists below which excitation shakedown in the sense that the granular layer does not accumulate further deformation. Although pavement design guides give more weight to asphalt concrete layer failures, granular failure may not be ignored; especially for thin layers and/or heavy load. The behavior of granular layers used in base and, sub-base layers of flexible pavement is complicated due to its nonlinear elastoplastic response when subjected to dynamic traffic loading. The objective of this paper is to present a simplified simulation model for the Shakedown behavior of granular layer in flexible pavement. This method is integrated with Mohr-Coulomb criterion, which is used and applied to simulate the response of unbound granular layers to dynamic loading in a numerical analysis. The results of analysis are then compared to simplify the results of modeling without considering shakedown effects and then, the conclusions are drawn.

Author keywords: Dynamic analysis; flexible pavement; Mohr Coulomb criterion; numerical simulation; shakedown; unbound granular layer (UGL).

8.1. INTRODUCTION

A structural formation of the elastic-plastic problem and/or shakedown theory has been adopted to account for this complex response in the form given by Martin (1981; 1984). This framework is essentially identical to that used by (Cohn, Maier & Grierson 1979; Zarka & Casier 1979). The formation was presented in discrete terms, in a manner which can readily be identified with finite element approximations (Martin et al. 1987).

The realistic analysis of pavement performance requires an approach that recognized the incremental mode of failure of such structure when subjected to repeated moving loads (Ghadimi, Nega & Nikraz 2015), and that theory of structural shakedown provides such an approach (Sharp 1985). The shakedown concept is examined in pavement material engineering to simulate the unbound granular materials (UGM) used in flexible pavement layers (Raad, Weichert & Haidar 1989; Sharp & Booker 1984; Sharp 1985; Werkmeister 2006).

The shakedown concept has been used to describe the behaviour of conventional engineering structure under repeated cyclic loading (Werkmeister, Dawson & Wellner 2001). The interesting question is whether a critical stress level exists between stable and unstable condition in pavement structure. According to the 'shakedown concept', this level is termed as the 'shakedown limit'. It was originally developed to analyze the behavior of pressure vessels to cyclic thermal loading. And then later, the theory was applied to analyze the behavior of metal surface under repeated rolling and/or sliding load (Werkmeister, Dawson & Wellner 2001). The concept maintains four categories of material response are under repeated loading: purely elastic, elastic shakedown, plastic shakedown, and incremental collapse (Werkmeister, Dawson & Wellner 2001). The shakedown concept is used to indicate the behavior of UGM based on the data by AASHO experiment (AASHTO1986a; Sharp 1983; Werkmeister, Dawson & Wellner 2001), while the method of limit analysis was to indicate the lower and upper bound limits of UGM in shakedown condition.

Collins and Boulbibane (1998) applied the upper-bound theorem (Kolter's theorem) to analyze different types of wear mechanism of surfaces subjected to repeated sliding or rolling contacts. It investigated the critical shakedown load associated with various failure mechanisms such as subsurface and surface slip, and rut formation using various nonlinear optimization techniques including qaasi-Newton and simulates annealing. While, Yu and Hossain (1998) analyzed the lower-bound shakedown formation of layered pavements using a linear approximation of the Mohr-Coulomb yield criterion.

Brown, Yu, Juspi and Wang (2012) also validated the laboratory modelling of two wheel tracking devices for lower-bound shakedown subjected to Mohr-Coulomb criterion that was described earlier by Brown, Yu and Juspi (2008) for single-layer system of soil or granular material, in which they demonstrated that Yu's theory gave reliable results. In this study, application of moving-wheel load to a series of two- and three layered pavement system which involved a various soils and unbound granular materials, were used to determine a 'shakedown' condition. And then, data were used to assess the validity of applying three-dimensional lower-bound shakedown theory to pavement as a basis for improving pavement design.

Habiallah, Chazallon and Hornych (2004) presented an elastoplastic simplified model based on the shake down theory for flexible pavements. This method is usually used to describe the characteristic behavior of unbound granular materials by introducing a yield surface taking into account the influence of the pressure on the mechanical behaviour of the granular media. Chazallon, Hornych and Mouhoubi (2006) developed a new elastoplastic model for the long-time behaviour of unbound granular material in flexible pavement taken into account in both isotopic and kinematic harding. In this model, modified Boyce model (Boyce 1980; Boyce, Brown & Pell 1976) was implemented in order to account for sand response, which takes into account the influence of the initial void ratio and the mean stress on the mechanical behavior is considered. And then, Allou, Chazallon and Hornych (2007) implemented the constrictive mathematical material models that was presented by Habiballah and Chazallon (2005) into a finite elements modeling (FEM) simulation of a low volume traffic road.

Based on repeated load triaxial tests, a further general procedure has been developed by Chazallon et al. (2009) for the determination of the material parameters of constitutive model in order to integrate the previous studies (Allou, Chazallon & Hornych 2007; Chazallon, Hornych & Mouhoubi 2006) in a FEM modelling by taking into account the Boyce model (Boyce 1980; Boyce, Brown & Pell 1976) and shakedown for UGM layers. Another, 2D-FE simulation is conducted by Ling and Lin (2003) where the response of reinforced asphalt pavement concrete under plane strain model condition subjected to monotonic loading is investigated. The simulation is run through PLAXIS program and UGM elastoplastic behaviour is considered through Mohr-Coulomb Criterion (Ghadimi, Nega & Nikraz 2015) in order to obey the associated and non-associated flow rules.

Saad, Mitri and Poorooshasb (2006) investigated the numerical simulation to evaluate the benefits of integrating a high modulus into the pavement foundation and/or design criteria. The fatigue resistance of the pavement system is evaluated through the maximum tensile strain transmitted to the bottom of the asphalt concrete layer whereas the maximum compressive strain transmitted to the top of subgrade is used in conjection with the maximum vertical surface deflection to evaluate the rutting resistance of the pavement. Material elastoplastic behavior is considered through Drucker-Plager model and simulation is run by ADINA.

Ghadimi, Nega and Nikraz (2015) integrated a simulation of shakedown behavior in pavement granular layer in new constitutive model based on Mohr-Coulomb Criterion in FEM simulation in ABAQUS that was initially the experimental for UGM shakedown presented by Chazallon et al. (2009). Insufficient evidences are currently available to confirm the reliability of the proposed linkage between shakedown ranges that are defined either repeated load triaxial (RLT) test or in-situ performance of UGM.

The main objective of this paper is to present a simplified simulation model for the Shakedown behavior of unbound granular layer (UGL) in flexible pavement. This method is integrated with Mohr-Coulomb criterion, which is used and applied to simulate the response of unbound granular layers to dynamic loading in a numerical analysis.

8.2. BACKGROUD

8.2.1. SHAKEDOWN APPLICATION

Sharp and Booker (1984) and Sharp (1985) were one of the first researchers who introduced the shakedown theory or application into pavement design. Sharp (1985) and Brett (1987) observed that many pavement sites in the field do shakedown rather than deteriorate continually based on observations from the AASHTO road tests (AASHTO1986a) and a number of road sections in New South Wales (Tao et al. 2010). Their early observations provided some early field support to the application of shakedown theory in pavement analysis and pavement design. Specially, The studies on the wear of layered surface by Anderson and Collins (1995) and (Wong, Kapoor & Williams 1997) revealed the particular relevance of shakedown theory and/or application to pavement analysis (Tao et al. 2010).

The shakedown application for rate-independent material has been applied to discrete structure for many years, but has only recently applied successfully to continua. A notable success is the one of the upper-bound theorem (Koiter’s theorem) to analysis different types of wear mechanisms of surfaces subjected to repeated sliding or rolling contacts (Collins & Boulbibane 1998; Sharp & Booker 1984; Tao et al. 2010; Werkmeister 2004).

8.2.2. SHAKEDOWN CONCEPT AND PAVEMENT DESIGN

For a successful pavement design, the pavement must be able to resist the accumulation of permanent deformation (Werkmeister et al. 2002). The permanent deformation of Unbound Granular Material (UGM) and other material load to irreversible deformation at the pavement surface. Thus in practice, a pavement construction should be designed in such a way that no any small, permanent deformation appear in each layer. For design purpose, this implies that maximum load level which is associated with a solely resilient response must be known and then not exceeded, if uncontrolled permanent deformations are to be prevented. This has raised the possibility of existence of a critical stress level between stable and unstable condition in a pavement.

According to the 'shakedown concept', this is termed as the 'shakedown limit' (Werkmeister, Dawson & Wellner 2001; Werkmeister et al. 2002). The shakedown concept has been used to describe the behavior of conventional engineering structure under repeated cyclic loading. The concept mainly maintains that are four categories of material response under repeated loading: purely elastic, elastic shakedown, plastic shakedown, incremental collapse or ratcheting (Johnson 1986; Werkmeister, Dawson & Wellner 2001). The inelastic design procedures are based on shakedown principles. Although the provisions are limited to bridges comprising compact shapes (Barker et al. 1996).

8.2.3. CALCULATIONS OF SHAKEDOWN LIMIT

Estimation of the shakedown limit load can be found using two applications of extremism shakedown principles for elastic and/or perfectively plastic structures (Lubliner 1990). The lower-bound theorem due to Melan states that if any equilibrium residual stress distribution can be found, which together with the stress field produced by the passage of a load does not exceed the yield condition at any time, then shakedown occur (Collins & Boulbibane 1998). To implement this theorem the elastic stress field induced in the layered pavement must be calculated. According to Collins and Boulbibane (1998), this is readily achieved using a finite element procedure or a standard package such as BIRAR or CIRCLY. The problem of finding the shakedown limit load can then be formulated as a linear programming problem.

8.3. UNBOUND GRANULAR MATERIALS

Lekarp and Dawson (1998) introduced a new model to increase the understanding of permanent deformation behaviour of unbound granular materials. The existing numerical models are verified by a series of laboratory repeated load tests to accumulated permanent axial strain at given number of cycles as a function of applied stresses, taking into consideration that the maximum shear ration and the length of the stress path into p-q space. In a similar way, Werkmeister, Dawson and Wellner (2001) investigated the permanent deformation behaviour of granular materials and the shakedown concept using triaxial tests and provided a powerful material assessment and pavement design tool for engineering unbound pavement bases. For more detail of this concept as applied to pavement, see (Collins & Boulbibane 2000; Collins & Boulbibane 1998; Sharp & Booker 1984).

A new simple design approach that uses test results from the repeated load triaxial apparatus to establish the risk level of permanent deformation in the unbound granular layer (UGL) in pavement construction under consideration of the seasonal effects was developed by Werkmeister et al. (2003). From this study data, a serviceability limit line 'plastic shakedown limit' stress boundary for unbound granular materials (UGM) was defined for different moisture contents. Below this line, the material has stable behaviour. The serviceability limit line was applied in a finite element (FE) program, FENLAP, to predict whether stable behaviour occurs in the UGM. To calculate the stress in the UGL, a nonlinear elastic model (Dresden model) was implemented into the FE program. The effects of channing moisture content during spring thaw period and asphalt temperature on pavement structural response were investigated.

8.3.1. MODELS FOR FLEXIBLE PAVEMENT MATERIALS

The flexible pavement generally includes an asphalt (a bituminous wearing course), unbound granular base and subgrade layers. The asphalt is considered as visco-elastic and thermo-sensitive materials. At low temperature, they can be considered as purely elastic. The UGM exhibits elastoplastic behavior without viscous. In current pavement research, their mechanical behavior is studied with repeated load triaxial tests (Chazallon et al. 2009). These tests allow studying either the short-term resilient behavior or the long-term behavior where plastic strain occurs. Subgrade soils are very important material to support highway. Current AASHTO pavement design procedures recommend the resilient modulus of subgrade soils or pavement design and analysis. Laboratory repeated triaxial loading tests and field falling weight deflectometer (FWD) are two common way to characterize the resilient modulus of subgrade soils. Subgrades are more susceptible to moisture change.

Flexible pavements, particularly when unsurfaced or thinly surfaced, granular layer play an important structure role in the overall performance of the pavement structure. Consequently, to establish more natural pavement design and construction criteria, it is essential that the response of granular layers under traffic loading by thoroughly understood and taken into consideration.

8.4. DESCRIPTION OF CONSTITUTIVE MODELS

The formulation and solution of incremental problem in elastic-plastic solid is a fundamental problem in plasticity. Powerful techniques are available in finite element methods for the solution of this class of problem, and solutions can be carried out fairly routinely in standard finite element codes.

The analyses presented here tries to take into account the elastic materials where the applied repeated stress is sufficient small such that no element of the material enters the yield condition because the loading and unloading path in stress-strain excursion is the same. In other words, there is no any residual strain produced in loading cycles. From the first stress-strain excursion, all deformations are fully recovered. However, in case of elastoplastic, there would be some residual strain in each loading cycles if the amplitude of loading exceeds the yield criterion.

When the applied repeated stress is slightly less than that required to producing a rapid incremental collapse, it is called “plastic shakedown” (Werkmeister, Dawson & Wellner 2001). This implies that a finite amount of energy is absorbed by the material on each stress-strain excursion. Once a purely resilient response has been obtained, the material is said, “shakedown”, and the maximum stress level at which this condition is achieved is termed the “plastic shakedown limit”. The solution for such kind of material is through introducing constitutive models. The oldest and still the most useful widely applied constitutive models to model UGM response of flexible pavement layers (FPL) are Mohr-Coulomb criterion and Drucker-Prager (Werkmeister, Dawson & Wellner 2001; Yu 1994; 2006; Yu & Hossain 1998).

The yield criterion proposed by Coulomb (1773) is in term of shear stress τ and normal stress σ_n acting on a plane (Yu 2006). It suggests that the yield begin as long as the shear stress and the normal stress satisfy the following equation:

$$|\tau| = c + \sigma_n \, tan\phi \tag{8.1}$$

where c and ϕ are the cohesion and angle of internal friction for soil. Mohr-Coulomb yield, which is equation (2) and Drucker-Prager's, (equation 3), yield surface on a deviatoric plane is shown in Figure 8.1.

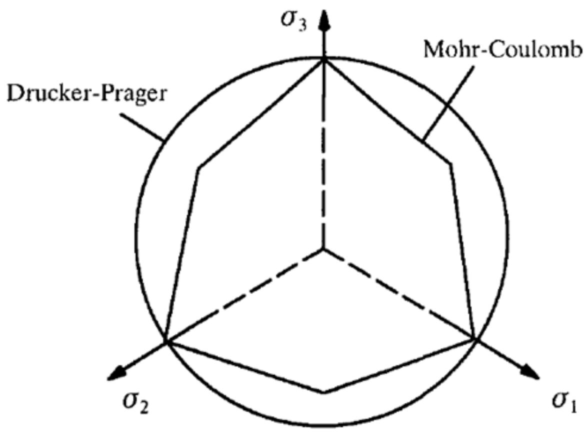

Figure 8.1 Mohr-Coulomb and Drucker-Prager's yield surface on a deviatoric plane

In terms of the principle stress, Coulomb's yield criterion equation can be expressed as:

$$f = \sigma_1 - \sigma_3(\sigma_1 + \sigma_3)sin\phi - 2c \; cos\phi = 0 \quad for \; \sigma_1 \geq \sigma_2 \geq \sigma_3 \tag{8.2}$$

where σ_1, σ_2 and σ_3 are three principle stress; and c and ϕ and are cohesion and angle of internal friction, respectively. A plastic potential function is generally obtained by using a dilation angle φ.

$$g = \sigma_1 - \sigma_3(\sigma_1 + \sigma_3)sin\varphi = constant \quad for \; \sigma_1 \geq \sigma_2 \geq \sigma_3 \tag{8.3}$$

where φ is angle of dilation. The von Mises yield criterion is not suitable for modelling the yielding of frictional material as it does not include the effect of mean stress as observed in experiment (Yu 2006). To overcome this limitation of the von Mises yield function, Drucker and Prager (1952) proposed the following revised function for frictional soils

$$f = \sqrt{J_2} - aI_1 - k = 0 \tag{8.4}$$

$$a = \frac{2sin\phi}{\sqrt{3}(3-sin\phi)} \text{ and } k = \frac{6c\ cos\phi}{\sqrt{3}(3-sin\phi)} \tag{8.5}$$

where I_1 and J_2 are first invariant principle stress and second invariant of deviator stress tensor, respectively, and a and k are material constants for use in analysis. Soil or rock rarely behaves as a linear elastic material. So a better approach would be to model them as nonlinear elastic-plastic material. The fully define the elastic plastic stress-strain relation for a geomaterials, it is necessary to specify a nonlinear incremental elastic stress-strain relation. Two methods may be used to construct a nonlinear elastic stress-relation. They are known as Green's hyperelastic and hypoelastic theory (Yu 1994; 2006). The theory of hyperelastic assumes that there is exists a *strain energy function*, $U_s(\varepsilon_{ij})$ and a complementary energy function, $U_c(\sigma_{ij})$, such that

$$\sigma_{ij} = \frac{\partial U_s}{\partial \varepsilon_{ij}}\ \ and\ \varepsilon_{ij} = \frac{\partial U_c}{\partial \varepsilon_{ij}} \tag{8.6}$$

The above equation (8.5) yield a one-to-one relationship between stress and strain can be expresses in the rate forms of this stress-strain relationship as equation given below:

$$\dot{\sigma}_{ij} = \frac{\partial^2 U_s}{\partial \varepsilon_{ij}\partial \varepsilon_{kl}}\dot{\varepsilon}_{kl} = D_{ijkl}\dot{\varepsilon}_{kl} \tag{8.7}$$

$$\dot{\varepsilon}_{ij} = \frac{\partial^2 U_s}{\partial \sigma_{ij}\partial \sigma_{kl}}\dot{\sigma}_{kl} = C_{ijkl}\dot{\sigma}_{kl} \tag{8.8}$$

In the above equation (8.7) and (8.8), the thermodynamic laws are always satisfied because energy cannot generate through load cycles, and materials are identical for loading and unloading. However, In the theory of hypoelasticity, it is assumed that the incremental stress and strain tensors are linearly variable material moduli that are functions of the current stress or strain state (Yu 1994; 2006). Mathematically the equation can be expressed as:

$$\dot{\sigma}_{ij} = \dot{D}_{ijkl}(\sigma_{mn})\dot{\varepsilon}_{kl} \tag{8.9}$$

From equation (8.7) and (8.9), complete relation between a stress and strain rate for an elastic-perfectly plastic solid may be express as following equation given below:

$$\acute{\sigma}_{ij} D_{ijkl}^{ep} \dot{\varepsilon}_{kl} \tag{8.10}$$

where D_{ijkl}^{ep} is the elastic-plastic stiffness matrix (Yu 1994). Equation (8.10) can be express in terms of tensor of elastic coefficient for both perfectively elastic and plastic material if the $\acute{\sigma}_{ij} = \dot{\varepsilon}_{ij}$, which are mentioned on above equation (8.7) and (8.8), respectively. And then, equation can be written as:

$$\sigma_{ij}^{n} = C_{ijkl}^{n} \varepsilon_{kl}^{n} \tag{8.11}$$

One of the first interpretations that can be applied in shakedown concept is to describe the variation of permanent triaxial deformation or limit the state of plastic strain, ε^{p}, with the number of cyclic loading, N. The relationship of these can mathematically described with an equation as:

$$\varepsilon_{i}^{p} = f_{i}(N) \tag{8.12}$$

Equation (8.12) can be differentiating in terms of loading time, t, and be expressed as:

$$\frac{\partial \varepsilon^{p}}{\partial t} = \frac{\partial f_{i}(N)}{\partial N} \varepsilon_{0}^{p} \tag{8.13}$$

where ε^{p} and N are plastic strain and number of cyclic loading, respectively. f is the yield function that determines the direction of plastic strain. Under plastic strain field, the minimal and maximal cyclic loading is a monotonic periodic scalar function, which varies between 0 and 1. The expression can be written as following:

$$\begin{cases} N = 1 \Rightarrow \dfrac{\partial f_{i}(1)}{\partial N} = 1 \\ N = \infty \Rightarrow \dfrac{\partial f_{i}(1)}{\partial N} = 0 \end{cases} \tag{8.14}$$

By using an elastic strain-strain relation, we can determine the elastic strain rate from equation (8.11) and (8.13) as

$$\partial\varepsilon_{ij}^{e} = C_{ijkl}\partial\sigma_{kl} \Rightarrow C_{ijkl} = \frac{\partial\varepsilon_{ij}^{e}}{\partial\sigma_{ij}} \tag{8.15}$$

where C_{ijkl} and $\partial\varepsilon_{ij}^{e}$ are elastic compliance matrix and elastic strain rate, respectively. The total stain (elastic-plastic strain) is the sum of the elastic and plastic part, and the equation can be expressed by

$$\partial\varepsilon_{ij} = C_{ijkl}\partial\sigma_{kl} + \frac{1}{c}\frac{\frac{\partial g}{\partial\sigma_{ij}}}{\frac{\partial f}{\partial\sigma_{ij}}\frac{\partial g}{\partial\sigma_{ij}}}\partial f \tag{8.16}$$

Equation (8.15) can be further written as

$$\partial\varepsilon_{ij} = C_{ijkl}^{ep}\partial\sigma_{kl} \tag{8.17}$$

And then, finally the elastic-plastic compliance matrix can also be written further one step from equation (8.16) and (8.17) as the following given equation

$$C_{ijkl}^{ep} = C_{ijkl} + \frac{1}{c}\frac{\frac{\partial g}{\partial\sigma_{ij}}\frac{\partial f}{\partial\sigma_{kl}}}{\frac{\partial f}{\partial\sigma_{ij}}\frac{\partial g}{\partial\sigma_{ij}}} \tag{8.18}$$

From the shakedown concept, Habiballah and Chazallon (2005) designated the $S_{ij\,min}^{el}(x)$ and $S_{ij\,max}^{el}(x)$ as the minimal and maximal deviatoric elastic stress. Thus the periodic actual elastic stress can be expressed in term of time of loading as:

$$S_{ij}^{el}(x,t) = S_{ij\,min}^{el}(x\ ) + \Lambda(t)\Delta S_{ij}^{el}(x) \cdots \ \Delta S_{ij}^{el}(x) = S_{ij\,max}^{el}(x)$$
$$-S_{ij\,min}^{el}(x) \tag{8.19}$$

where, $S_{ij\,min}^{el}(x)$ and $S_{ij\,max}^{el}(x)$ are the minimal and maximal deviatoric elastic stress, respectively. $S_{ij}^{el}(x,t)$ and $\Delta S_{ij}^{el}(x)$ are the periodic actual elastic stress and loading amplitude in the order given. Finally, using equation (8.14), (8.18) and (8.19), the shakedown constitutive model either for *elastic* or *plastic* shakedown has the ability of modification for both cyclic loading and stress state (normal and shear stress) in relation to the asphalt surface layers.

Equation (8.20) designates the newly developed constitutive model for the shakedown UGM of flexible pavement:

$$\left(C_{ijkl}^{n_i}\right)^{\dot{\hat{S}}} = \left(\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{ep} + \left(1-\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{\hat{e}} + \left(1-\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{e} \tag{8.20}$$

$$\left(C_{ijkl}^{n_i}\right)^{\dot{\hat{S}}} = \left(\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{ep} + \left(1-\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{\hat{e}} + \left(1-\frac{1}{c}\cdot\frac{\partial f_i(N)}{\partial N}\right)\left(C_{ijkl}^{n_i}\right)^{e}$$

where $C^{\hat{S}}$ and C^{ep} are denote the shakedown and elastoplastic constitutive models of the unbound granular materials, respectively. $C^{\hat{e}}$ and C^{e} are the inelastic and purely elastic constitutive models, in the order given. The superscript n and N are the rate and the number of cyclic loading. The letter c is a material constant that refer to the hardening of the UGM, and if there no residuals in the process and then, c can be denotes as, $(c = 1)$ and be used as multipliers. The shakedown constitutive model, $\widehat{\boldsymbol{S}}$, is gradually slowdown from elastoplastic constitutive model, $\boldsymbol{C^{ep}}$, to purely elastic constitutive model, $\boldsymbol{C^{e}}$. There is an inelastic, $\boldsymbol{C^{\hat{e}}}$, stage in between the changing process before it reaches purely elastic (i.e. 100% elastic).

8.4.1. 2D-FE MODELLING OF FLEXIBLE PAVEMENT LAYERS

This constitutive model is examined against the laboratory data that was done by Habiballah and Chazallon (2005). In addition, constitutive model is also compared to Chazallon et al. (2009) and Ghadimi, Nega and Nikraz (2015) constitutive models. In this step, authors were conducted and they allowed using their laboratory results to verify the performance of this constitutive model. In this 2D-FE numerical simulation, a repeated load triaxial cell is used and simulated with axisymmetric model using ABAQUS. And then, A stages of stress state configuration for flexible pavements is set according to (Chazallon, Hornych & Mouhoubi 2006; Habiballah & Chazallon 2005) and simulated on the model.

Numerical simulation of developed plastic strain with number of load repetitions against the laboratory test that was published by Habiballah and Chazallon (2005) is shown in Figure 2. The material properties of unbound granular material that was used for verification are demonstrated in Table 1. The simulation has a triaxial cylindrical 20 cm height and 10 cm diameter sample and is modeled with low number of 100 cycle's linear quadrilateral element of CAX4R. The different stress ratio q/p is defined by the ratio of deviatoric stress ($q = \sigma_1 - \sigma_3$) to normal stress or confining pressure, ($p = (\sigma_1 + 2\sigma_3)/3$) (Chazallon, Hornych & Mouhoubi 2006; Habiballah & Chazallon 2005).

Table 8.1 Material Properties of UGM Used for Verification

UGM properties	Elastic moduli (*E*), MPa		Poisson's ratio (ν)
Elastic	*E* = 200		ν = 0.3
Plastic	ϕ = 44^{o}	c = 12.26 kPa	Ψ dilation = 39^{0}

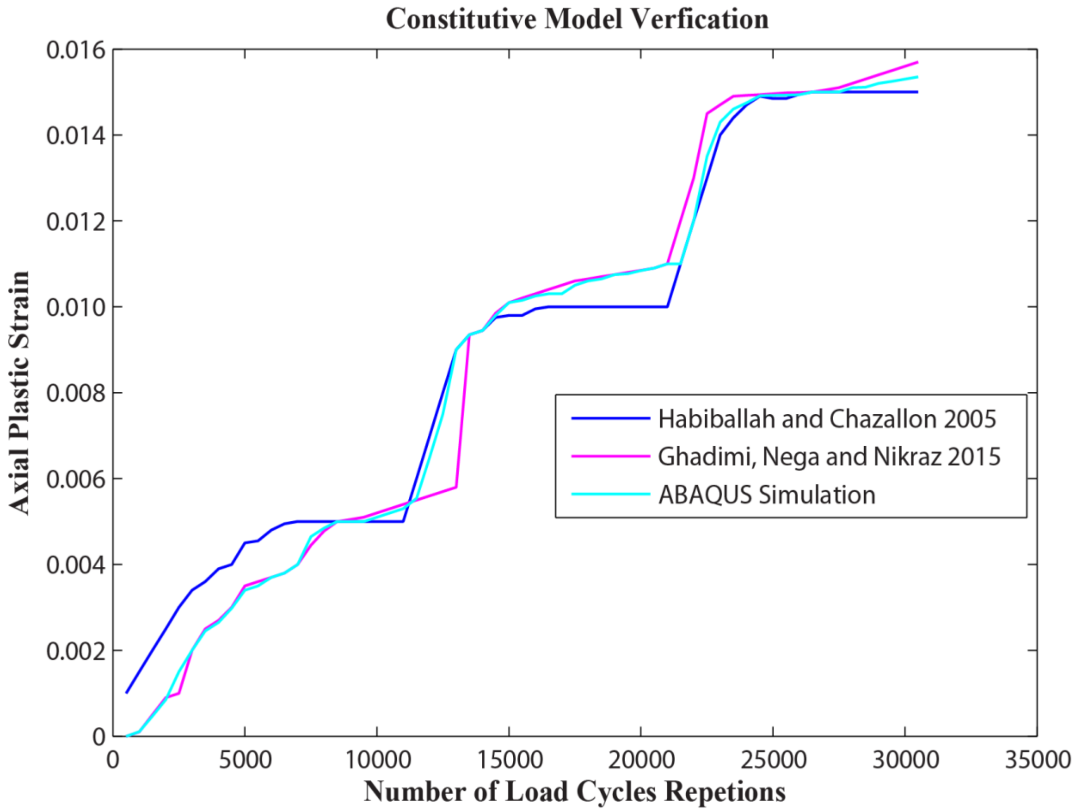

Figure 8.2 Simulation of Developed Plastic Strain against the Laboratory Test (Verification)

Numerical simulation of three layered flexible pavement: Asphalt, Base (UGM) and Subgrade layers are modeled in ABAQUS and MATLAB. The simulation is modeled with 740 kPa UDP (uniformly distributed pressure) on radius of 9.0 cm over the circular area. The pressure and loading area are selected and were modeled with total medium of 10 m x 20 m according the Australian Standard for design loads on structure ((Austroads 2004; 2012). Figure 8.3 shows the FEM model of the three layered flexible pavement. As it can be seen from the developed model, the loading tire is not affected by the boundary condition. This showed that the design load on the structure has a capacity to resist deformation because it was design in the way that boundary condition of flexible pavement layers should not affected by repletion of cyclic traffic loading.

The dimension of this model is 100- R x 200- R (loading radius) both in horizontal and vertical direction, repectively. These dimensions are within the ranges of the previous research studies that were recommended. For example, Kim, Tutumluer and Kwon (2009) modeled an ABAQUS numerical simulation for axisymmetric model with a medium of 140-R in vertical and 20-R in horizontal. They suggested that effect of boundary condition would be neglected if it has to be modeled with this medium. Similarly, Ghadimi, Nega and Nikraz (2015) modeled with a medium of 111-R and 222-R in horizontal and vertical direction, in the given order, in ABAQUS simulation, and they are satisfied without come of the results. However, Duncan, Monismith and Wilson (1968) and Huang (1993) suggested the boundaries need to be at 50- times R in vertical and 12 time R in horizontal direction of the layered flexible pavement for modeling.

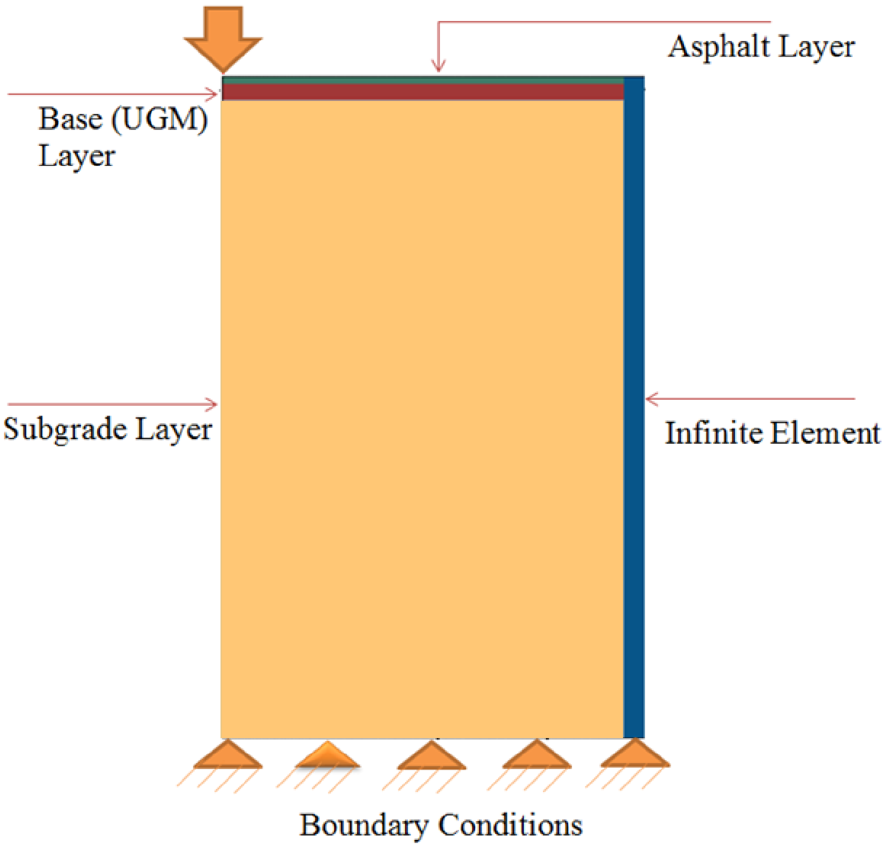

Figure 8.3 FEM Model of Three Layered Flexible Pavement

The asphalt pavement concrete is modeled with a linear elastic material thickness of 20 cm. The subgrade layer is modeled according to (Chen & Saleeb 1982; Drucker & Prager 1952; Drucker, Prager & Greenberg 1952). An axisymmetric infinite element does not represent the true solution behavior as being infinite in the far field. Due to the linear behavior in the field, the infinite element does a harmonic behavior with the finite element in a layered half-space under surface pressure.

For the base layer (UGM), two constitutive models: Mohr-Coulomb criterion and Shakedown theory are carried out for a thickness of 50 cm. In the first numerical simulation, the constitutive models is modeled according to Mohr-Coulombs criterion (Coulomb 1773) and then, In the second simulation according the application of shakedown theory to pavement design stated by (Collins & Boulbibane 1998; Collins & Boulbibane 2000) so that the effect of shakedown in flexible pavement will be fully understand. The models consists 15000 element in total, which is also includes CAX4R and CINAX4 elements for the layered medium and infinite space, respectively. An efficient returning mapping algorithms that was introduced by Clausen, Damkilde and Anderson (2007) was inserted as unbound granular materials constitutive model, and was also coded in UMAT under ABAQUS simulation. The material properties used for simulation of the layered flexible pavement is shown in Table 8.2.

Table 8.2 Material Properties for Flexible Pavement Model

Pavement Layer	**Elastic Moduli (*E*), MPa**	**Poisson's Ratio (ν)**	**Density (kg/m^3)**	**Angle of Internal Friction, ϕ (Degree)**	**Angle of Dilation, Ψ (Degree)**	**Cohesion, c (kPa)**
Asphalt 20 cm	2800	0.40	2200	0	0	0
Base 50 cm	500	0.30	1800	35	17	7
Subgrade	50	0.35	1700	20	15	7

The dynamic finite element simulation has modeled by implicit in ABAQUS to reveal the formulation processes of primarily. The loading is presumed to be a haversine periodical pressure in 7s. It has a 0.1s loading time and followed by rest period of 0.9s. The loading cycles are shown in Figure 8.4. From the modeled data presented, it can be seen that the pressure load of tire is gradually increased to reach a maximum peak and then, decreased to zero. This showed that the strain continues to develop due to unloading stress distribution and then, followed by 0.9s rest period. Roman, Roger and Walla (1989) evaluated the effect of tire pressure on flexible pavement response and performance. The data showed little effect due to tire pressure at all load levels. On the other hand, on the basis of classical fatigue models, the increased cracking was found to result primarily from the combined effects of higher pavement temperature and thinner pavement structure.

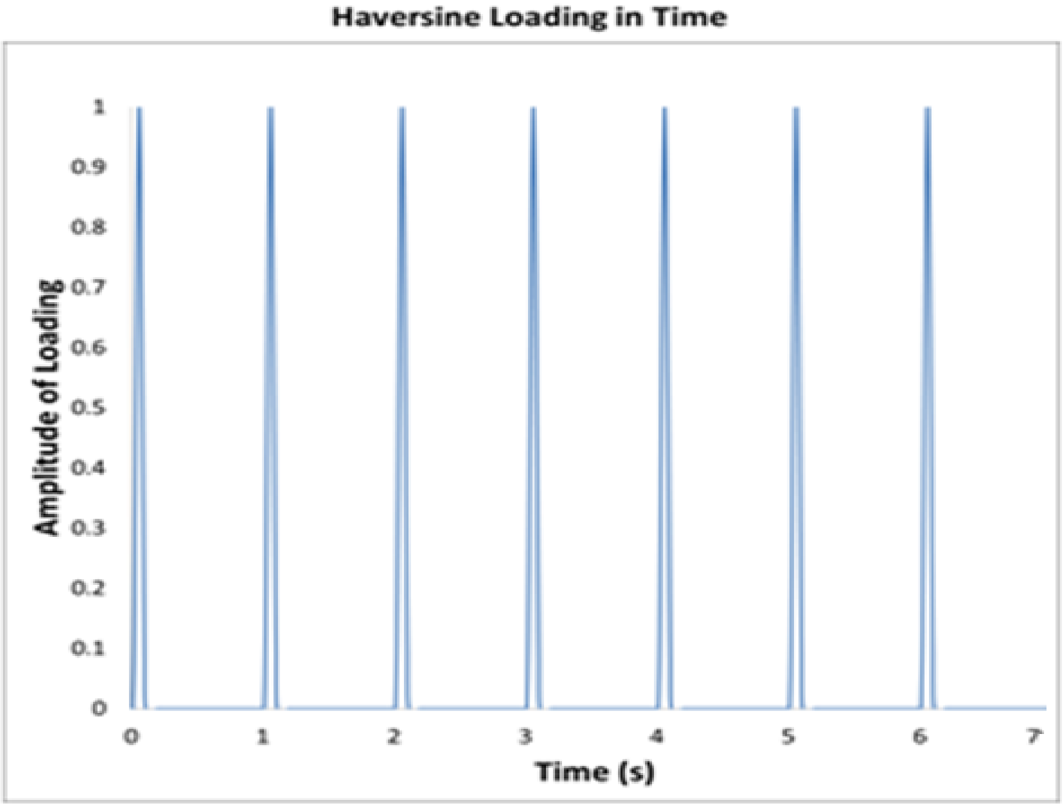

Figure 8.4 Haversine Periodic Loading versus Time

8.4.2. SHAKEDOWN ANALYSIS FOR FLEXIBLE PAVEMENT UGM LAYERS

The shakedown concept has been used to describe the behavior of conventional engineering under cyclic loading. The resulting permanent deformation, which accumulated with the repeated load, was described and compared with types of response usually described by the shakedown approach. The method of description could provide a power material assessment and pavement design tool for engineering UGM pavement bases.

The total vertical strain that was developed in this model for both Mohr-Coulomb criterion and Shakedown constitutive model is shown in Figure 8.5 and 8.6, respectively. As it can be seen from analysis, the contours' range of the shakedown model has less strain (i.e. $-9.04\text{x}10^{-5}$ to $3.67\text{x}10^{-5}$) as compared to Mohr-Coulomb criterion model (i.e. $-1.21\text{x}10^{-5}$ to $1.34\text{x}10^{-5}$). It is should be notice from the analysis that the vertical strain is design of flexible pavement is one of the critical which the rutting of the pavement is controlled.

Therefore, understanding of the shakedown concept and also give a sound consideration for its effect during pavement design might reduce the risk level of permanent deformation in the UGL in pavement construction. If shakedown concept has ignored, it might lead to pavement overdesign and then, rutting will control the criterion. Collins and Boulbibane (2000) analyzed the mechanical response of an unbound pavement to the repeated loading of traffic, and pavement is modeled as a layered elastic/plastic structure by the concept of shakedown theory. Results demonstrated that the concept of shakedown theory can lead to design procedures in which the base course thickness can be deduced as a function of the applied and the strength and stiffiness properties of subgrade and base course.

The main limitation of the model is the use of nonhardening, perfectly-plastic and Mohr-Coulomb model. There are two avenue of research, which would make the procedures more realistic: continue with perfectively-plastic bur regard it as in the classical critical state and extended the classical shakedown theory to the more complex plasticity models in response to cyclic loading (Collins & Boulbibane 2000). They finally summarized that despite the shortcoming, it is argued that the concept and technique of shakedown theory has much to offer to the pavement engineers. Similarly, Habiballah and Chazallon (2005) summarized the rutting of flexible pavement which occurs in the UGLs put in evidence the lack of precision in the design method. They described the shakedown theory method can be an alternative to the step-by-step method for the UGM behavior modelling under large cycle is complex.

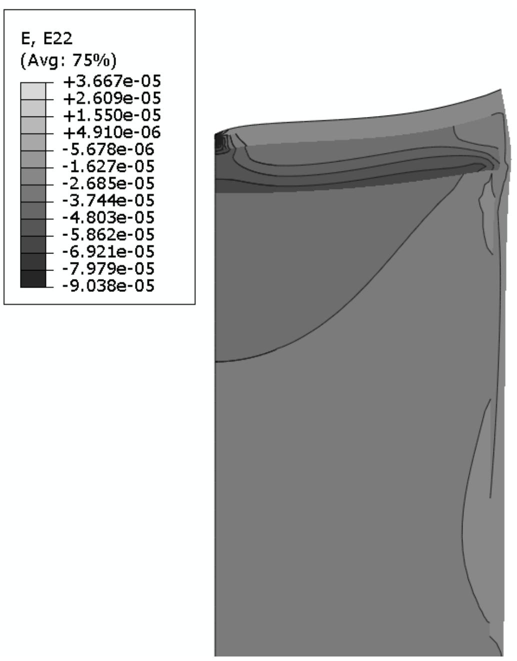

Figure 8.5 Vertical Strain in Shakedown Model

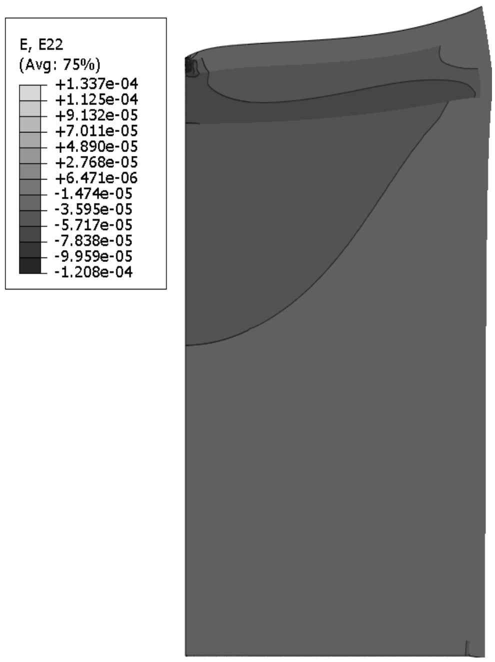

Figure 8.6 Vertical Strain in Mohr- Coulomb Model

From Figure 8.1, mathematically the direction of equivalent plastic strain flow rate for Mohr-Coulomb criterion plasticity can be expressed by the following equation:

$$\mathrm{d}\varepsilon^{pl} = \frac{\mathrm{d}\varepsilon^{-pl}}{g}\frac{\partial G}{\partial \sigma} \quad \ldots .where\ g\ can\ be\ written\ as{:}\ g = \frac{1}{c}\ \sigma{:}\ \frac{\partial G}{\partial \sigma} \qquad (8.21)$$

By substitution g into equation (8.21) and also integrating and then, the equation (8.21) can be simplify as:

$$\varepsilon^{-pl} = \int \frac{1}{c}\sigma :\ \mathrm{d}\varepsilon^{pl} \qquad (8.22)$$

where c is the constant material cohesion; σ is stress tensor and $\mathrm{d}\varepsilon^{pl}$ is plastic strain increment tensor. It is understandable that the magnitude of equivalent plastic strain is determine by the above equation in each element. The distributions of equivalent plastic strain for Mohr-Coulomb and Shakedown model are shown in Figure 8.7 and 8.8. As it can be seen from model, shakedown model has had a high equivalent plastic strain (i.e. 4.08×10^{-4}) as compared to equivalent plastic strain (7.95×10^{-4}) in Mohr-Coulomb materials. This indicates that the effects of shakedown behavior are more than the material in Mohr-Coulomb even if a reasonable difference of equivalent plastic strain distributions is acceptable for both models. It is understandable that the element under the beneath of asphalt layer might have caused a high vertical stress during the repletion of cyclic loading. Of particular important in this case is whether a given a pavement structure will experience accumulation of plastic strain or whether the increase in plastic strain will cease to occur, there by loading to stable or shakedown.

Boulbibane et al. (2005) reviewed a new mechanistic approach to unbound pavement design based on the shakedown theory. Results show minimum shakedown load are found by varying the geometry of the proposed failure mechanisms, and this is done for unbound pavement laying a clay subgrade. Similarly, Chazallon et al. (2009) presented a finite element program for the modelling of rutting of two flexible pavement with shakedown theory and FEM. The program incorporates a permanent deformation model for unbound granular material based on the concept of the shakedown theory. Comparisons of model predication with result of cyclic triaxial test were fairly good result have been obtained.

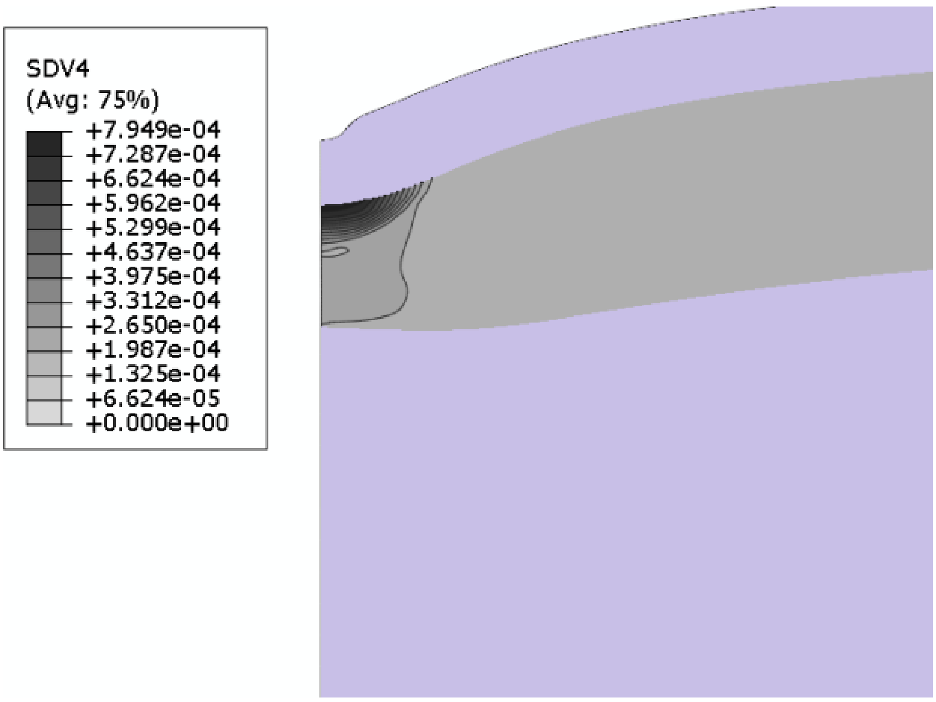

Figure 8.7 Equivalent Plastic Strain in Mohr-Coulomb Model

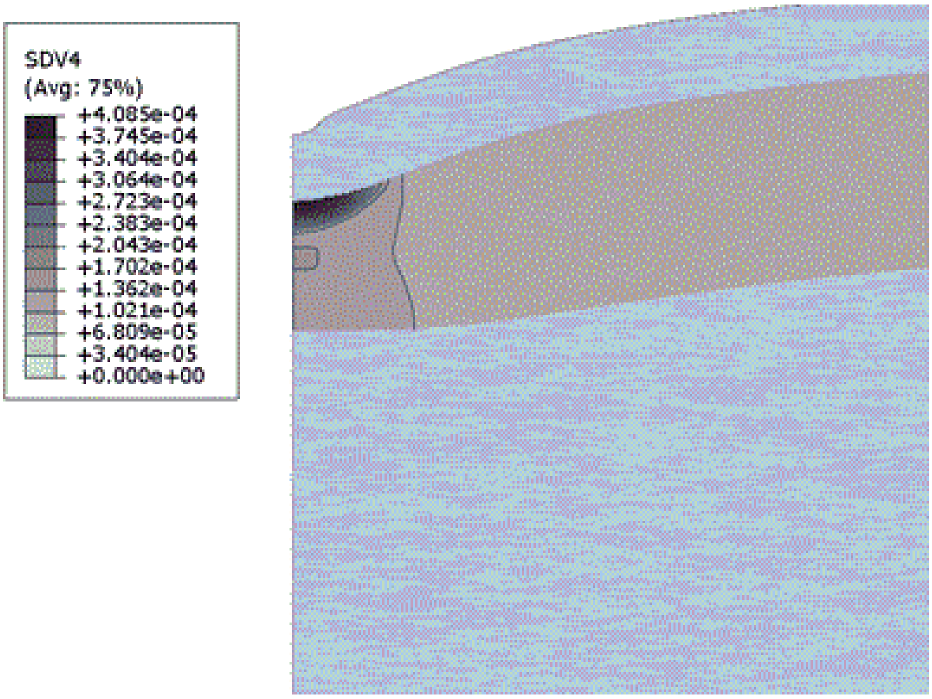

Figure 8.8 Equivalent Plastic Strain in Shakedown Model

To investigate the particular effect of shakedown behavior more adequately, another element (i.e. second element) was selected from the beneath of asphalt concrete layer at the center of loading. From the investigation, this element had high vertical stress during loading cycles. Figure 8.9 shows the development of equivalent plastic strain during loading time for both Mohr-Coulomb and Shakedown at the center of this element. As it can be seen from the from the presented figure, the equivalent plastic strain in shakedown model is less developed as compared to the Mohr-Coulomb model within 7s time period of dynamic analysis. If it was simulated for a longer loading time, the effect of shakedown could behave differently. The maximum peak of accumulated plastic strain in Mohr-Coulomb has reached 2.70x10^{-4} while 2.50x10-^{4} in shakedown models.

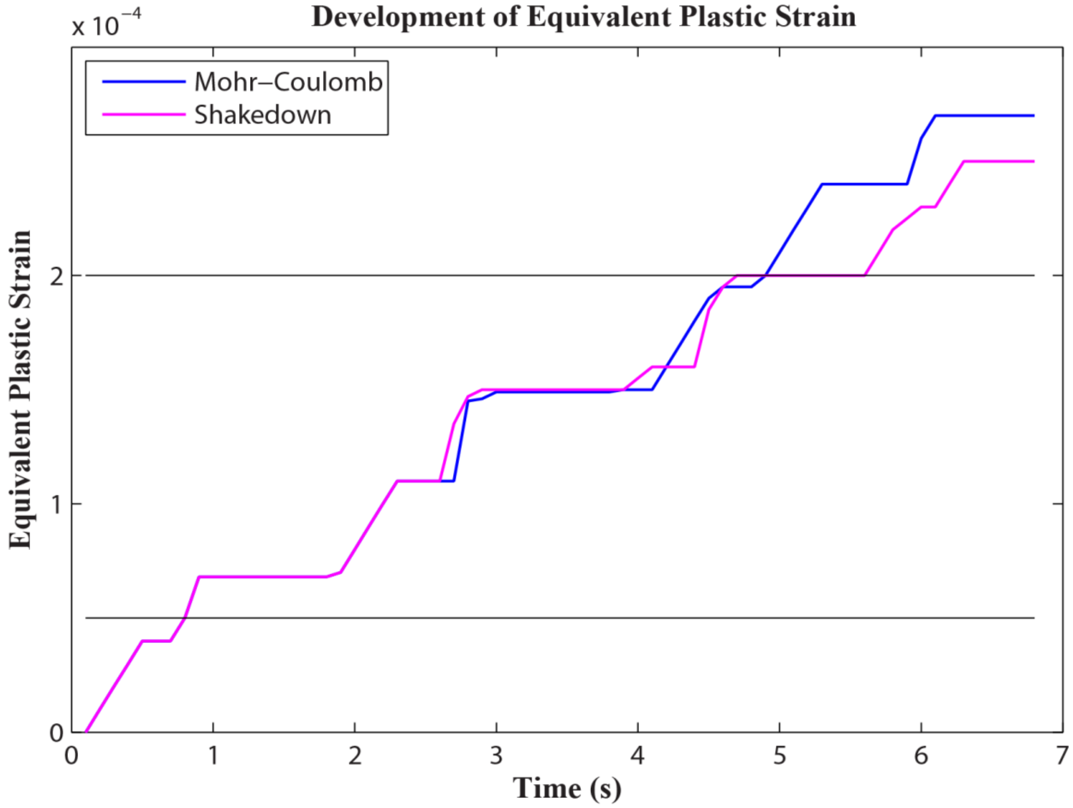

Figure 8.9 Equivalent Plastic Strain versus Time of Mohr-Coulomb and Shakedown

Stress –stain curve are an extremely important of a material mechanical properties in modern engineering materials. They are important graphical measure of hysteresis loop if the loads are high enough to induce characteristic of plastic shakedown behavior, while engineering stress-strain curve are used within elastic limit. As the result of these, the plastic shakedown behavior can easily understood if and if the hysteresis loop of stress-strain curve are taken into consideration. The stress-stain hysteresis's loop for shakedown model (i.e. for second element below the asphalt concrete layer) is shown in Figure 8.10. As it can be seen from the hysteresis loop, the behavior of the material is declining towards to the elastic behavior. This shows the material shakedown response is different from the material in Mohr-Coulomb plastic criterion. It can also be understood that the materials behavior in this model is stiffer because of increasing in loading cycles.

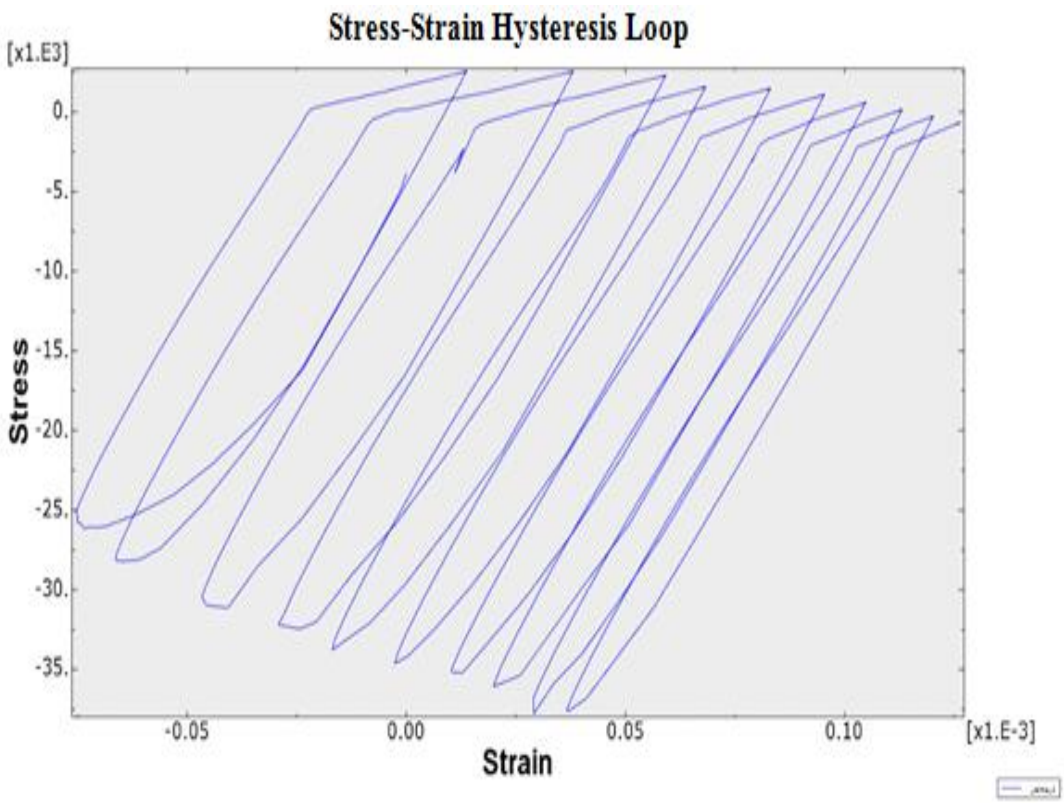

Figure 8.10 Stress-Strain Hysteresis Loop during Shakedown

8.5. CONCLUSIONS

This paper presents a numerical simulation model for shakedown behavior for flexible pavement's unbound granular layer (UGL). This method is integrated with Mohr-Coulombs criterion, which is used and applied to simulate the response of unbound granular material (UGM) to dynamic loading in a numerical analysis. A new constitutive model is developed based on Mohr-Coulomb criterion and Drucker-Plager method for flexible pavement unbound granular materials (UGM).

The new developed constitutive model is capable of considering shakedown effects in base UGL and predicting the various types of observed flexible pavement failure and the effect of various design parameters. The implementation of this new developed constitutive model is verified against published results of laboratory test data measured shakedown for UGM. This model also compared with others a simplified model studies of pavement unbound granular layers based on the shakedown theory and Mohr-Coulomb criterion for verification.

This constitutive model is numerically implemented in dynamic FE simulation in ABAQUS and results are compared to Mohr-Coulomb criterion. Results has shown 50% reduction of vertical plastic stain of the base (UGL) layer with shakedown model as compared to Mohr-Coulomb after 7s repetion of cyclic loading. This justified that it is necessary to give a sound consideration to shakedown effect in structure of flexible pavement layers and various design parameters because this justification can provide a power material assessment and pavement design tool for engineering UGM pavement bases. If shakedown effect has neglected in shakedown concept and pavement design, the risk level of pavement deformation in the UGL in pavement construction might critically increase.

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CHAPTER 9: DYNAMIC ANALYSIS OF FALLING WEIGHT DEFLECTOMETER TEST RESULTS FOR STRENGTH OF CHARACTER OF PAVEMENT LAYER MODULI

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ABSTRACT

Falling weight deflectometer (FWD) testing has been extensively done in the past to evaluate structural condition based on engineering and also determine the layer moduli of flexible pavements. Any setting implementation of modulus for pavement layers that obtained from the backcalculation process may not sometimes be accurate even if the computed and measured deflection basin has fulfilled the standard to match within a certain tolerable limits. Immeasurable interpretation has been involved for a decades in obtaining moduli of these pavement layers. On top of these, guidelines and tools are provided in order to calculate flexible pavements layer moduli. However, FWD interpretation has increasingly become challenging from time to time because of repeated heavily traffic urban roads, which have experienced multiple traffic shift and milling operations and overlays. The characteristics of pavement structure such as variation in pavement thickness, pavement layer damage as the result of fatigue and rutting, and changes in pavement temperature and other factors can overwhelm the deflection data to have the greatest influence on fatigue damage of flexible pavement than those induced by structure layer stiffness. The main objective of this study is to analyze the FWD test results for strength of character of flexible pavement layer moduli in Western Australia so that allowable loads for existing pavement structures can be determine. FWD measurement was taken from 7 locations of traffic roads including core data, roadway data and pavement distress information. The results of FWD test of these traffic roads are compared for consistency and assurance of existing pavement structures of Western Australia and then, the conclusions are drawn.

Author keywords: Flexible pavement; Layer moduli; Falling weight; Deflectometer; Deflection basin; Dynamic analysis; Analytical model; Western Australia

9.1. INTRODUCTION

Falling weight deflectometer (FWD) testing has been as integral part of the condition assessment of flexible pavement for a decade. Various approach and procedures FWD deflection analysis have been developed in several studies (Xu, Ranjithan & Kim 2002b). Most of these procedures and guidances for FWD deflection do not account either the dynamic loading effect or nonlinear material behavior. Although few producers do account for these effects, but field implementation of these procedures is very difficult because of complex nature of both user's inputs and mechanisms that involves sound understanding in underlying concept. Development of a reliable procedures and guidances for flexible pavement layer moduli condition assessment is a challenging task owning to the large number of factor to be considered (interaction, distresses, severity and so forth) (Xu, Ranjithan & Kim 2002b).

Sebaaly, Mamlouk and Davies (1986) evaluated the dynamic analysis of falling weight deflectometer data by using a multi-degree of freedom elastodynamic analysis which is based on a Fourier synthesis of a solution for periodic loading elastic or viscoelastic moduli layered strata. The results indicate that inertial effects are important in the prediction of the pavement response. Xu, Ranjithan and Kim (2002b) developed a mechanistic relationship between FWD deflection and asphalt pavement layer condition indicates. Results identify three types of layer condition indicators: deflection basin parameters (DBPs), effective layer moduli, and stresses and strain. Xu, Ranjithan and Kim (2002a) also presented a new condition assessment criterial for flexible pavement layer using FWD from field data. Nondestructive condition assessment criteria were developed for application in conjunction with the condition evaluation indicted that are estimated based on falling weigh deflectometer deflection.

Kim and Park (2002) developed a mechanistic empirical method for assessing pavement layer condition and estimated the remaining life of flexible pavement using multi-load level falling weight deflectometer (FWD) deflection. Synthetic deflection database was generated using FEM and a stress-dependent soil model. The results indicate that the condition assessment criteria for asphalt pavement using multi-load level FWD deflection can estimate the base and subgrade layer moduli condition. While AC layer modulus were found to be better indicators than deflection basin parameters. Appea and Al-Qadi (2000) also assessed falling weight deflectometer data for stabilized flexible pavements. The performance and structural condition of nine flexible pavement test sections that was built in Bedford, Virginia, have been monitored for past 5 years using FWD. The nine flexible pavements had three groups with aggregate base layer moduli thickness of 100, 150, and 200 mm. The deflection basins obtained from the flexible pavement testing have been analyzed using the ELMOD backcalculated program to find the pavement structural capacity and to defect changes in the aggregates resilient modulus. The analysis shows a reduction in the backcalculated resilient modulus of the

100 mm thick base layer, which was 33 percent reduction over past 5 years for non-stabilized section as compared with the geosynthetically stabilized section.

Magnuson, Lytton and Briggs (1991) compared the computer predictions and field data for dynamic analysis of FWD data using the SCALPOT computer program. Physical properties of the pavement were generated by a trial- and-error backcalculation. The asphaltic concrete surface layer was represented as a three-parameter viscoelastic medium elastic solid. Results indicated a good agreement between experimental and computer-predicated response, which was obtained using the backcalculated pavement layer properties.

A study was conducted to develop methods for using FWD measurements to determine moduli of in site pavement material sand to compare FWD-estimated moduli with laboratory-measured values in order to achieve consistent input to thickness design procedures (Frazier 1991). A three-layer pavement model was used to characterized flexible pavement and simple procedures were developed to account for seasonal variations and effective moduli values for granular base-subbase and subgrade soils from limited FWD measurement (Frazier 1991). There were large differences between FWD moduli and laboratory moduli from triaxial testing (AASHTO T274). However, good agreement was demonstrated between FWD and laboratory values (AASHTO T274) moduli for subgrade soils even if characterization of granular base-subbase was difficult.

The use of falling weight deflectometer data has become one of the primarily means of characterization of insitu structural properties of flexible pavement. Park, Buch and Chatti (2001) analyzed the effect layer temperature prediction model deflection to determine the AC modulus. Results suggested that the model could be adapted to all seasons and other climate region.

The search of solutions to the problems of determining the effective structure number (SN) of flexible pavement based on interpretation of FWD deflection is not new. Most methods rely on the intrinsic relationship between measured deflection parameters and layer coefficients or moduli and thickness of the flexible pavement subgrade system (Hoffman 2003). The idea of related load-deformation response (FWD) deflection basin to structural parameters (SN) is appealing since it conveys a fundamental relationship of classical mechanics. Livneh and Goldberg (2001) assessed road formation and foundation by comparison of use of falling weight deflectometer and light drop weight (LDW). It is obvious that falling weight deflectometer is a suitable device for stiffness measurements but is sometimes sophisticated for formation and foundation testing (Livneh & Goldberg 2001).

The falling weight deflectometer (FWD) has been used in the evaluation of material properties of pavement system for decades. The load amplitude and frequency content are intended to provide pavement deformation levels similar to those induced by truck wheal loads in heavily urban traffic loading. Interpretation of the insitu measured data is normally based on elastic solution and does not take into account the possible existence of localized nonlinearities. Chang, Roesset and Stokoe II (1992) investigated the nonlinear effects in falling weight deflectometer tests using both a linear and nonlinear solution with the generalized cap model to reproduce the nonlinear soil behavior. The material nonlinearities were found to be important for FWD tests on flexible pavement where the subgrade is relatively soft and where the pavement is thin. FWD is commonly considered to provide estimates of material properties for levels of load similar to those exerted by truck model as discussed by Uddin et al. (1985b; 1985a).

The main objective of this study is to analyze the FWD test results for strength of character of flexible pavement layer moduli so that allowable loads for existing pavement structure can be determine; to evaluate FWD deflection data for pavement maintenance and rehabilitation (M&R); to demonstrate a proper interpretation of FWD data for the flexible pavement sections that have been experienced multiple milling operation and overlay; and to show multiple FWD measurements along the length of the heavily traffic urban roads so that it can be identify pattern that are critical to the proper interpretation of the FWD deflection data of Western Australia roads. Design of typical FWD configuration; location of loading plate; geophones and measured deflection basin is shown in Figure 1.

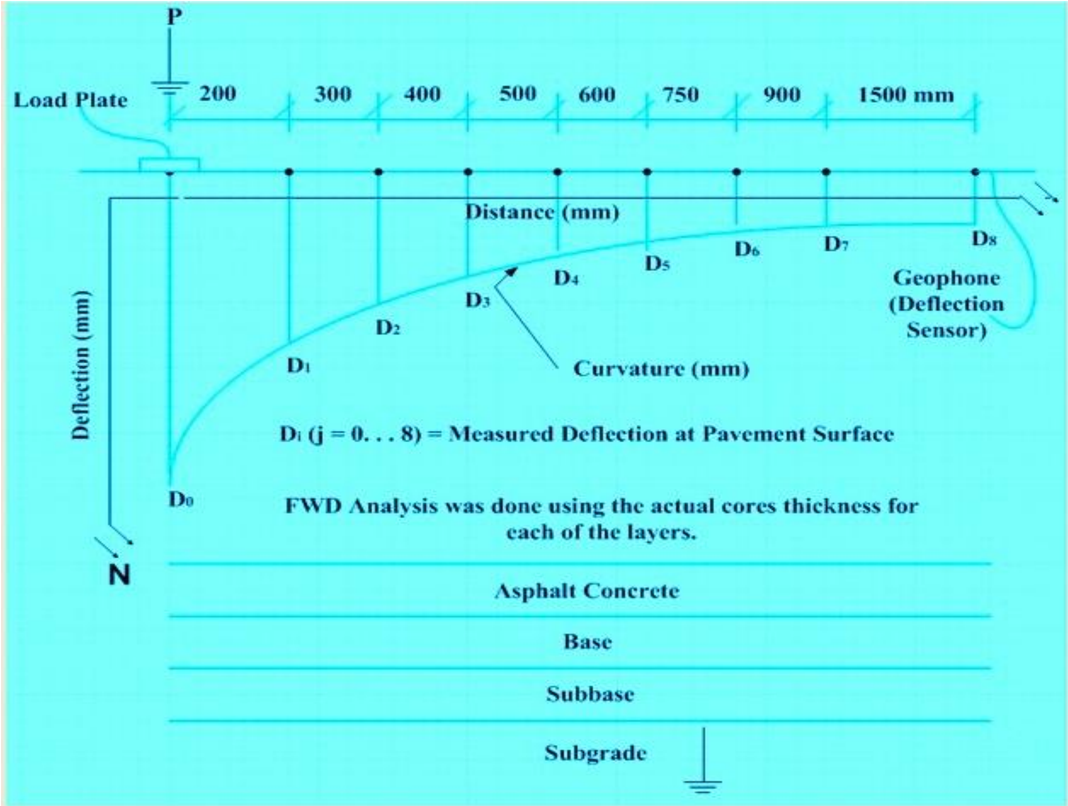

Figure 9.1 Design of Typical FWD Configuration, Location of Loading Plate, Geophones and Measured Deflection Basin

9.2. BACKGROUND

9.2.1. CHARACTERISTICS OF PAVING MATERIALS

The most often used materially for the flexible pavement surface layer are asphalt concrete and Portland cement concrete. The loading defendant behavior of these materials is not interest in this study because they are considerably stiffer than the base and subgrade material (soils). However, it is necessary to point out that loading rate would affect the dynamic modulus of the asphalt pavement mixtures, especially in high temperature.

9.2.2. ANALYTICAL MODEL AND APPROACHES

Falling weight deflectometer (FWD) testing has been extensively done in the past to assess structural condition and determine the model of flexible pavement layer. The set of modulus value for pavement layers obtained from the backcalculation may not be accurate even though the computed and measured deflection basin may match within tolerable limit (Mehta & Roque 2003). Extensive data interpretation is involved in obtaining the layer moduli of these pavements. For example, guidelines and tools are provided for calculating layer moduli of flexible pavement. However, FWD interpretation has become challenging because more roads have experience now and then with several milling operation and overlays. The characteristics of the flexible pavement structure (damage layers, variation in pavement thickness, and change of pavement temperature) can overwhelm the deflection data to have more significant effect than those induced by structural layer moduli stiffiness as summarized by Mehta and Rogue (2003).

The insitu material values of flexible pavement layer moduli are remarkable parameters in determination of allowable loads for pavement structures, overlays thickness and assessment of pavement rehabilitation and maintenance (M&R). Many researchers have specified that stiffiness of pavement determined through nondestructive testing (NDT) is a fundamental method of determining effective layer moduli (ELM) (Ceylan et al. 2005; Rada, Richter & Stephanos 1992; Thompson 1992; Xu, Ranjithan & Kim 2002b). The use of FWD deflection measurements to characterize the capacity of pavement structure and also determine the flexible pavement layer moduli has increasingly become predominant (Macneal 1982; Mehta & Roque 2003; Nazarian & Chai 1992). Some studies have focused on algorithm to interpret deflection data from specific structure. While the accuracy of the structure and its characteristics such as damaged layers, variation of thickness and change in temperature, can overwhelm the deflection data can have a significant effect than those induced by structure layer moduli stiffness (Mehta & Roque 2003).

9.2.3. BACKCALCULATION OF FLEXIBLE PAVEMENT

There are several source of error in the backcalculated moduli besides the nonlinearity of the stress-strain relation of the material in pavement layers (Lytton 1989). These error, which are introduced by deflection calculation model and its presumed constitutive relations, are systematic and thus cannot be eliminated or reduced by repeated measurement or calculations (Lytton 1989; Ullidtz & Coetzee 1995). Lytton (1989) discussed only random error in computing layer moduli that can be reduced or eliminated, and the source of these are in the measurements that are made (both force and deflection), and in the spatial variation of the materials in the layers. It will be little value to the American Society for Testing and Materials (ASTM) to attempt to set standards on pavement deflection testing or on modulus backcalculation procedures without first making a careful assessment of magnitude of both the random and systematic errors.

Similarly, Mehta and Rogue (2003) reported backcalculation is the 'inverse' problem of determining material properties of pavement layers from its response to surface loading. No direct, closed-form solution is currently available to determine the layer moduli of a multi layered system given the surface and layer thickness. Artificial neural networks (ANNs) provide a fundamentally new approach to backcalculation of flexible pavement layer moduli from falling-weight deflectometer deflection basins (Meier & Rix 1994). An ANN is a highly interconnected collection of single processing elements that can be trained to approximate a complex, nonlinear function through repeated exposure example of the function.

Several computer program such as ADAM, BISDEF, BOUSDEF, CHEVDEF, COMDEF, DBCONPAS, ELMOD, ELSDEF, EVERCALC, ILLI-BACK, ISSEM4, MODCOMP2, MODULUS, OAF, PADAL, and WESDEF, have been developed for backcalculation analyses (Kang 1998). Each of this computer programs for backcalculation employs a particular forward model and a specific task of backcalculation scheme. Kang (1998) summarized that an error in backcalculation may occur due several factors such as nonuniform pressure, distribution, temperature and moisture gradients, improper loading position to the edge of the pavement, variation of material properties and nonlinearity, selection of an improper forward model, and deflection-matching algorithms.

A new method was developed to backcalculate flexible pavement layer moduli from pavement surface deflection measured by a FWD using genetic algorithms (Kameyama et al. 1997). It was found that layer moduli can be accurately estimated by backcalculation using this method and with surface deflection calculated by the theory of elasticity for layer structures as the input condition.

9.3. FALLING WEIGHT DEFLECTOMETER

The falling weight deflectometer is a portable apparatus used for measuring the deflection of a road surface under the influence of a load pulse of very short time duration (Bohn et al. 1972). This pulse simulates a fast moving wheel load, and it can be adjusted to correspond to any desired wheel load of 5 tons (5000 kPa) or less. FWD deflectometer device appears to have originated in Europe. It is consist essentially of a large mass that is constrained to fall vertically under gravity on to a spring-loaded plate resting on pavement surface (Sebaaly, Davis & Mamlouk 1985).

Pavement characterization is one critical important in the allocation of resources to rehabilitate the rapidly deteriorating highway infrastructure. Several mechanical devices are now available to the highway engineer for this purpose such as the falling weight deflectometer (FWD), California bearing ratio (CBR), dynaflect, road rater (RR). Sebaaly, Davis and Mamlouk (1985) reported that the dynaflect and the road rater impart vibrating loads to the surface of the flexible pavement. Interpretation of deflection data remains somewhat problematic, however, since spurious resonances in the pavement strata may be induced (Mamlouk & Davies 1984) and the loading condition do not accurately reflect traffic loadings. Nevertheless, field studies have shown that the FWD yields good correlations with pavement deflections induced by traffic loading (Sebaaly, Davis & Mamlouk 1985; Sebaaly, Mamlouk & Davis 1986). According to Sebaaly, Davis and Mamlouk (1985), the dynamic analysis of falling weight deflectometer comprise in two distinct parts: determination of the dynamic motion of FWD and evaluation of the pavement response.

A falling weight deflectometer is a device that applies on impulsive load to a pavement surface and the deflection response is recorded at a series of radial point. The level of impact load, loading duration, and area are adjusted in such a way that it corresponds to the actual loading by a standard truck moved on a in-service load as defined by Sharma and Das (2008). The problem of backcalculation of pavement layer moduli of flexible pavement from FWD deflection data is truly a complex one and efforts are being made to involve a generalized approach to impact analysis (May & Von Quintas 1994; Sharma & Das 2008) that can be backcalculate the insitu layer moduli in a accurate and efficient manner without time consuming.

Various approach such as closed formal solution, database search, optimization techniques and regression equation have been implemented (Anderson 1989; Chou & Lytton 1991; Harichandran et al. 1993; Roque, Ruth & Sedwick 1998). However, the difficulties faced by researcher when using the backcalculation approach of layer moduli of asphalt are still unmeasurable due to complexity of asphalt of flexible pavement materials, variation in thickness, various change in temperature and change in traffic loading with time. Although a number of backcalculation software and studies on their comparative performance are available in published literature (Fwa & Rani 2005; Goktepe, Agar & Lav 2006; Shoukry & William 1999; Ullidtz & Coetzee 1995). The seed moduli chosen for backcalculation analysis of multilayer flexible pavements can have significant impacts on the performance of backcalculation software and sometimes, the final solutions of the backcalculated moduli (Fwa & Rani 2005). All backcalculation provide internally generated seed moduli for backcalculation analysis. However, the internally generated seed moduli does not always produces satisfactory results (Fwa & Rani 2005).

Among all nondestructive testing (NDT) methods, the falling weight deflectometer (FWD) method is the most widely or popular technique used (Goktepe, Agar & Lav 2006). FWD can successfully simulate traffic loads and it can also produce a huge amount of deflection data in a short period of time as reviewed by (Bianchini & Bandini 2010; Hoffman & Thompson 1982; Saltan, Tigdemir & Karasahin 2002). FWD measures the time-domain deflection on numerous road section and are used to backcalculate mechanical pavement properties using specific software involving forward and backcalculation directions (Goktepe, Agar & Lav 2006).

Fleming (1998) compared the falling weight deflectometer (FWD) with the German dynamic plate (GDP). They used plate load test (PLT) and the conventional falling weight deflectometer as benchmarks for comparison, and correlation ratio of about 0.5 was obtain between the GDP and FWD moduli (Alshibli, Abu-Farsakh & Seyman 2005). Livneh and Goldberg (2001) suggested that the dynamic modulus of the GDP is about 0.3-0.4 times the conventional FWD modulus.

9.3.1. DYNAMIC DEFLECTION DEVICE

Vibratory devices: dynaflect and road rater apply a stead state sinusoidal load to the pavement surface, where the falling weight deflection (FWD) such as dynatest FWD, kuab FWD, and phoenix FWD applies on impulse load to the pavement (Kang 1998). The FWD is the most popular used to provide estimates of pavement material properties for the load level by moving wheel load (Hoffman & Thompson 1982; Uddin & McCullough 1989; Uddin et al. 1985b; Uddin et al. 1985a; Ullidtz 1987). Dynamic analyses are generally not as popular as the static analyses in practice mainly due to running time concern (Kang 1998). A number of elastodynamic computer program have been developed based on the Kausel's formulation (Kausel 1981) for a horizontal layer system. In most cases, static and dynamic analysis of pavement system assume the existence of horizontal layers with different material properties that extend to infinity horizontally as discussed by Kang (1998). A rigorous evaluation of pavements may require consideration of other several factors such as the layer thickness virility (Kang 1998) and nonlinear characteristics of the paving material (Chang 1991; Chang, Roesset & Stokoe II 1992). In these cases, a finite element method (FEM) approach is required to study these variables on the response of flexible pavement.

9.4. METHODS AND MATERIALS

9.4.1. METHODS

Various computer programs are available to perform backcalculation analysis. In this study, the BISDEF computer program is used for backcalculation analysis. Bush III and Alexander (1985) developed the BISDEF computer program to handle multiple loads and to consider different interface layer condition. Burmister (1943) investigated the load (means a method to determine stresses, strains and displacement) in order to develop a flexible pavement layered moduli theory and found an exact solution for the boundary stresses on center of a circular, which was uniformly distributed load acting on the surface of a three-layer and half space (Appea & Al-Qadi 2000; Appea 2003). To obtain a Burmister (1943) type of solution, it is necessary to perform an integration using digital computers (Appea 2003):

$$D = F\left[\int_0^{\infty} f\,(e^{2mh}, e^{-2mh}, h) \cdot J_o(mr) \cdot J_1(mr) \cdot dm\right] \tag{9.1}$$

where, D is the deflection; F is Bessel's function of $J_o(mr)$; f is Bessel's function of $J_1(ma)$; h is layer thickness; r is radial distance from the load axis; m is parameter; and a loading radius. The nonlinear least squares optimization method was then used to minimize the sum of the squared relative difference to solve the following problem (Sivaneswaran, Kramer & Mahoney 1991):

$$f(E,h) = \frac{1}{n}\sum_{i=1}^{n}\left\{\frac{d_i^c(E,h)-d_i^m}{d_i^m}\right\} \tag{9.2}$$

The error location *i* is defined as follows:

$$r_i(E,h) = \frac{d_i^c(E,h)-d_i^m}{d_i^m} \tag{9.3}$$

After multiplying by the constant *n* for any convenience and then, Equation (9.3) can be expressed as follows:

$$f(E,h) = \sum_{i=1}^{n}\left(r_i(E,h)\right)^2 = r^T r \tag{9.4}$$

where, $r = \{r_1, r_2, r_3 \cdots\cdots r^n\}$ the relative error and T is the transpose function. The gradient of the criterion function is $\nabla f = 2Ar$; $A = \{\nabla r_1, \nabla r_2, \cdots \nabla r_n\}$; and the Hessian integral estimates equation can be written as follows:

$$H = \nabla^2 f = 2AA^T + 2\sum_{i=1}^{n} r_i \nabla^2 r_i \tag{9.5}$$

The gradient and Hessian are the respective multidimensional equivalents of the slope and curvature of a one-dimensional function. In this formulation, the first part of the Hessian is known as soon as the gradient ∇f has been evaluated. Since $r^T r$ is being minimized, the relative errors are often error. A good approximation to the Hessian may be made by neglected the second part (Appea 2003):

$$H = 2AA^T \tag{9.6}$$

All flexible pavement materials are assumed to be homogenous, isotropic, and linear-elastic except for the subgrade, which is assumed to exhibit nonlinear response (Al-Qadi, Brandon & Bhutta 1997; Al-Qadi et al. 1994; Appea 2003) is defined as follows:

$$E_0 = C_0 \left(\frac{\sigma_1}{\sigma}\right)^n \tag{9.7}$$

where, E_0 is the surface modulus; σ_1 is major principle stress; σ is reference stress; andC_0, n are constants, and n is negative. The FWD deflection basin was also characterized by analyzing the centroid (r_x, r_y) of the deflection basin. It had been determined that a flexible pavement having a deflection basin with higher ratio of r_x to r_y would represent a pavement with better loading distribution ability and higher relative stiffness as discussed by Appea and Al-Qadi (2000). The r_x/r_y ratio can be calculated as follows:

$$N = \frac{r_x}{r_y} \tag{9.8}$$

$$r_x = \frac{\sum_i^n A_i x_i}{A} \tag{9.9}$$

$$r_y = \frac{\sum_i^n A_i y_i}{A} \tag{9.10}$$

where, A_i is the area of each element under deflection basin; x_i is centroid distance of each element from the first sensor along the x-axis; y_i is vertical centroid distance from each element along y-axis; A is total area of the deflection basin and n is the sum number of element under the deflection basin.

To determine the modulus values, the flexible pavement system is modeled as a layered system. In the computer program, the modulus of the surface layer was assigned and then, the material is assumed as linear elastic. The elastic pavement layered moduli analysis is an approximation because all the flexible pavement layers are nonlinear elastic or viscoelastic. The incremental advantage of conducting nonlinear or viscoelastic analysis over elastic analysis will be compromised by the inherent approximation involved in the backcalculation process (Appea 2003; Mehta & Roque 2003). BISAR can handle horizontal applied loads and also allows for variation in strain transfer at pavement interacts.

Several researchers have developed models for temperature-modulus correction by addressing the issue of enumerated. However, many of these models have been based on a statistical analysis data that was obtain from a limited range types of asphalt mixture or flexible pavement profile. For instance, Johnson and Baus (1992) recommended the temperature correction formula based on an approximation from Asphalt Institute (AI) as follows:

$$\lambda_E = 10^{-0.0002175(70^{1.886} - T^{1.886})} \tag{9.11}$$

where, T is the temperature (°F) and λ_E is correction factor. Other researchers including Ullidtz (1987) developed a model based on results from the AASHTO Road test data. Ullidtz's correction model is shown as:

$$\lambda_E = \frac{1}{3.177 - 1.673 \log T} \ for\ T > 1℃ \tag{9.12}$$

Baltzer and Jansen (1994) and Kim, Hibbs and Lee (1995) developed a correction model based on statistical analysis of backcalculated moduli and measured hot-mix asphalt (HMA) temperature as follows:

$$\lambda_E = 10^{m(T-20)} \tag{9.13}$$

where, m = 0.018 by Baltzer and Jansen and 0.0275 by Kim, Hibbs and Lee.

9.4.2. MATERIALS

FWD deflection data was collected from seven project sites, and every site has fifteen locations. The test sections were an approximation of 7 km on hundred and five locations that were being investigated for the flexible pavement structural performance. Falling weight deflectometer was used to evaluate and identify pavement characteristic and pavement structural properties that have strongly been influenced pavement performance and functionality. Hard cores of the asphalt pavement concrete layer were taken from each of these locations during FWD testing.

Inertial force is considered in the pavement structure analysis using the FWD test; and the asphalt density in situ was generally around 2400 kg/m3, the roadbase around 2150 kg/m3 and the subbase around 1850 kg/m3. These densities were at Marshall or maximum dry density. Generally, the asphalt is compacted to around 97% of Marshall Density, the roadbase to around 96% of maximum dry density (MDD) and the subbase to around 96% of MDD according to the Australian Standard (2003). While after traffic ticking, density is expected to increase slightly. A summary of pavement material properties and thickness of the profile at seven project sites that were investigated is shown in Table 9.1.

Table 9.1 Summary of Material Properties and Thickness of the Profile at the Site 1-7

Site No	Temperature During Testing (^oC)		Altitude	Route	Station (km)	Design Thickness (cm)		
	Air	Surface				AC	Base	Subbase
1	19.8	17.8	39.0	Dongara Rd	0.0-1.15	20	30	40
2	30.5	35.6	8.7	Barker St	0.0-1.15	20	30	40
3	18.7	25.0	15.6	Armadale Rd	0.0-1.15	20	30	40
4	29.8	37.7	22.1	Burlington St	0.0-1.15	20	30	40
5	39.8	31.4	25.5	Nicholson Rd	0.0-1.15	20	30	40
6	28.7	39.9	12.9	Star St	0.0-1.15	20	30	40
7	25.7	27.2	12.9	Orrong Rd	0.0-1.15	20	30	40

9.5. RESULTS AND ANALYSIS

The range of asphalt concrete effective surface modulus values and pavement temperature are shown in Table 9.2. The FWD measurements were taken during a day time, this no pavement crack or visible possibility damage was observed on the mixed asphalt cores. From the FWD measurements data, it can be seen that the variation of the modulus values of the project appeared to be an artifact of the backcalculation investigation process. The variation of deflection with all project followed the same trend as that of the deflections away from pavement load. This showed that the deflection under the load is a representative of surface layer moduli, and this trend appears to indicate that the response of the inflexible pavement is dominated by lower layer and then, made the pavement surface layer moduli along with the location to be identical. Thus results suggest that stress-dependent nature of lower can be of significant for overall behaviour of the flexible pavement. Mehta and Roque (2003) evaluated FWD for determination of layer moduli of flexible pavements using BISDEF computer program that was used for backcalculation analysis of seven project. Results indicated that the variation of deflection under the load versus project location followed the same trend as that deflection away from the load. Mehta's also viewed that the response of the pavement system that was dominated by lower layer and surface layer moduli was also the same along with the location. Similarly, de Almeida, Brown and Thom (1994) also analyzed similar to the Mehta and Roque (2003).

Table 9.2 Range of Effective Surface Layer Modulus Values during First Iteration for Site

Project/Site Number	Effective Surface Layer Modulus (GPa)			Pavement Temperature (^oC)
	Minimum	Maximum	Average	
1	0.20	0.65	0.38	34.9
2	0.35	5.86	3.17	35.5
3	0.61	14.93	8.20	35.3
4	0.35	5.78	3.13	35.2
5	1.14	16.06	8.70	34.9
6	0.45	12.35	5.21	35.2
7	1.05	16.18	8.70	35.1

The effective layer modulus of the combined base and subbase layer and the subgrade for project 1 is shown in Figure 9.2. From the FWD modeled data presented, it can be seen that the combined base and subbase layer modulus and the subgrade modulus of flexible pavement has followed the same trend of the deflection at various locations. This showed that the measured and computed deflectioin basin adre approximately similar. These results are clearly indicated that the FWD data interpretatioin of the comparative quality of the base and subbase, and subgrade are appropriate. Although a insignificant error between the measured and predicated basin might have been occurred. This showed that the design load on the structure has a capcity to resist deformation because it was design in the way that boundary conditon of the flexible paveme nt layers should not affected by repletion of cyclic traffic loading. The average effective layer moduli for base and subbase was 0.39 GPa while 0.33 GPa for subgrade. Mehta and Roque (2003) evaluted the FWD data for deformation of layer moduli of pavement, and results indicated the variation of deflection under the load versus project followed the same trend as that the deflection. The average effective layer moduli for base and subase was about 0.38 GPa and 0.33 GPa for subgrade layer.

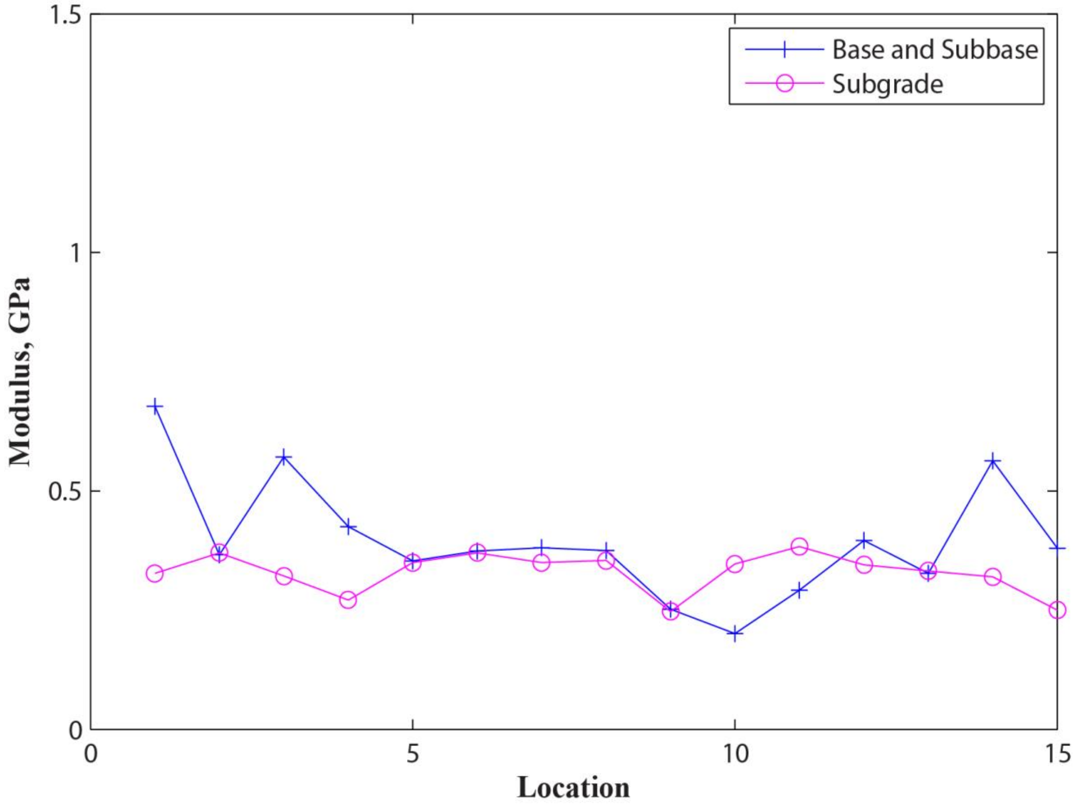

Figure 9.2 Effective layer moduli versus location for site 1

Figure 9.3 and 9.4 show the effective layer moduli of combined base and subbase layer and subgrade layer for project 2 and 3, respectively. As it can be seen from the FWD measured and predict, the combined base and subbase layer modulus are followed the same trend of the deflection at various locations in both projects while the subgrade layer modulus was fluctuating to match the farthest deflection. This showed that the FWD data a definitive interpretation quality of the base and subbase moduli would have been inappropriate due to insignificant error on subgrade moduli because the measured and computed deflection basin was remain approximately the same. The influence of overburden and pore pressures might have locked in the horizontal stresses on the insitu stiffness so that subgrade modulus was locked to match deflection. de Almida, Brown and Thom (1994) recommended the influence of overburden, pore pressures and a rigid bottom to include during analysis because error can influence the flexible pavement layer moduli from being progressing to match a furthest deflection with other layers.

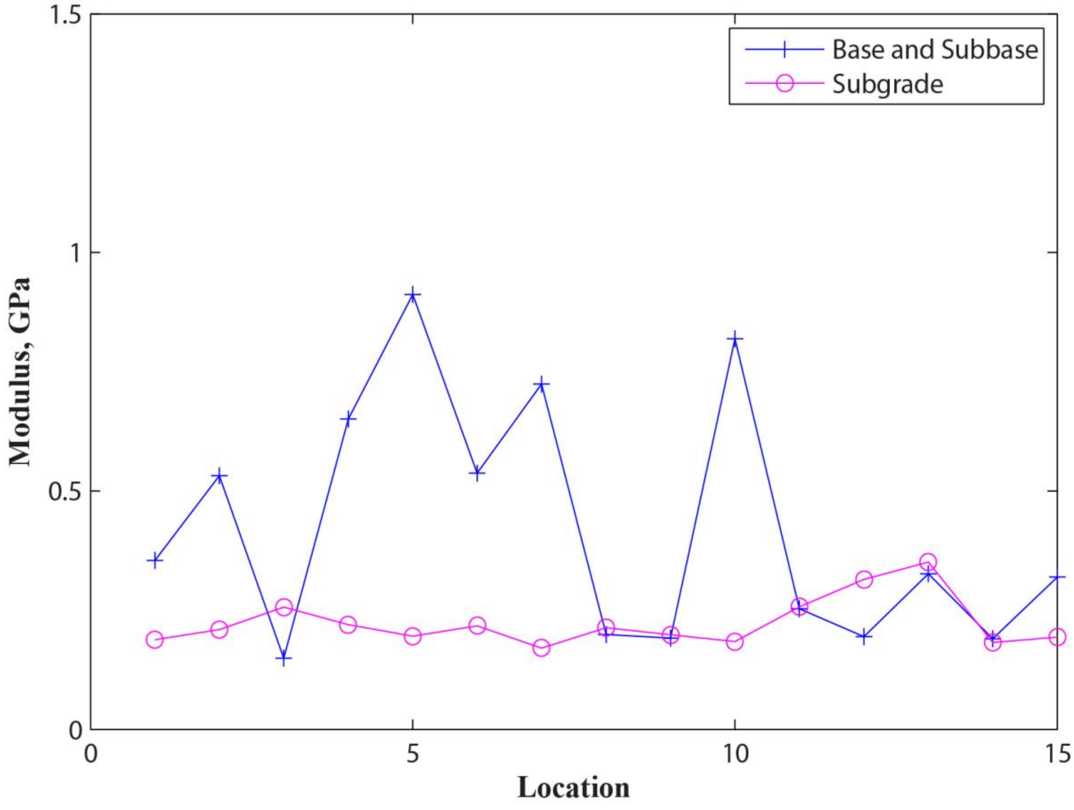

Figure 9.3 Effective layer moduli versus location for site 2

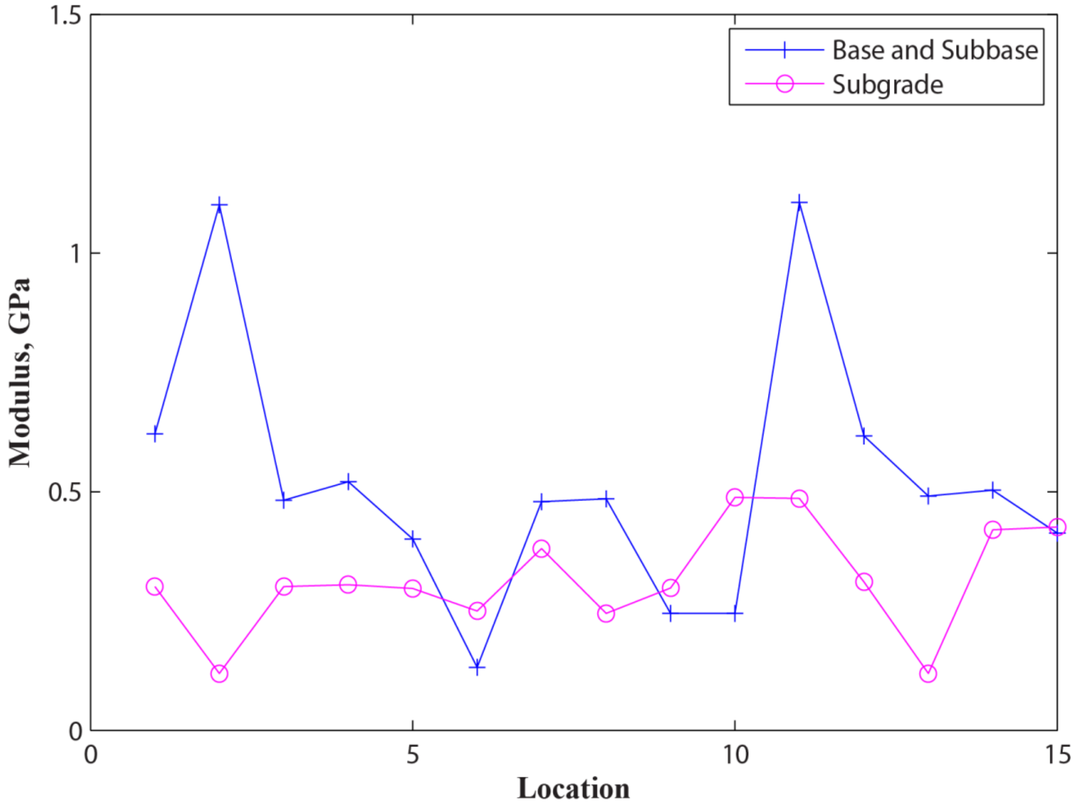

Figure 9.4 Effective layer moduli versus location for site 3

Effective layer moduli of combined base and subbase layer and subgrade modulus for project 4 and 5 are shown in Figure 9.5 and 9.6, repectively. From the FWD data and predicted model presented, it can be seen that both the combined base and subbase and subgrade pavement layer modulus are on the trend of deflection at various locations. This indicated that the strain continuous to develop due to unloading stress distribution and then, to a rest period while small error might have occurred on the subgrade. It was determined that neither the combined base and subbase layers nor the subgrade layers independently affected the deflection basin due to pavement dynamic load. However, subgrade usually contribute 60 to 80% of the total center deflection, therefore a small error might have occurred in the moduli of the other layers and a definitive interpretation. It is appears that thick and stiff asphalt concrete layers and stiff subgrade were causing the deflection basin so that it would be insensitive either to the combined base and subbase layer or subgrade moduli (Appea 2003; Ghadimi, Nega & Nikraz 2015; Mehta & Roque 2003). Ghadimi, Nega and Nikraz (2015) and Appea (2003) discussed subgrade moduli that usually contribute 60 to 80% of the total center deflection. Therefore a small error in determination of subgrade modulus can lead to a very large error in the moduli of the other layers.

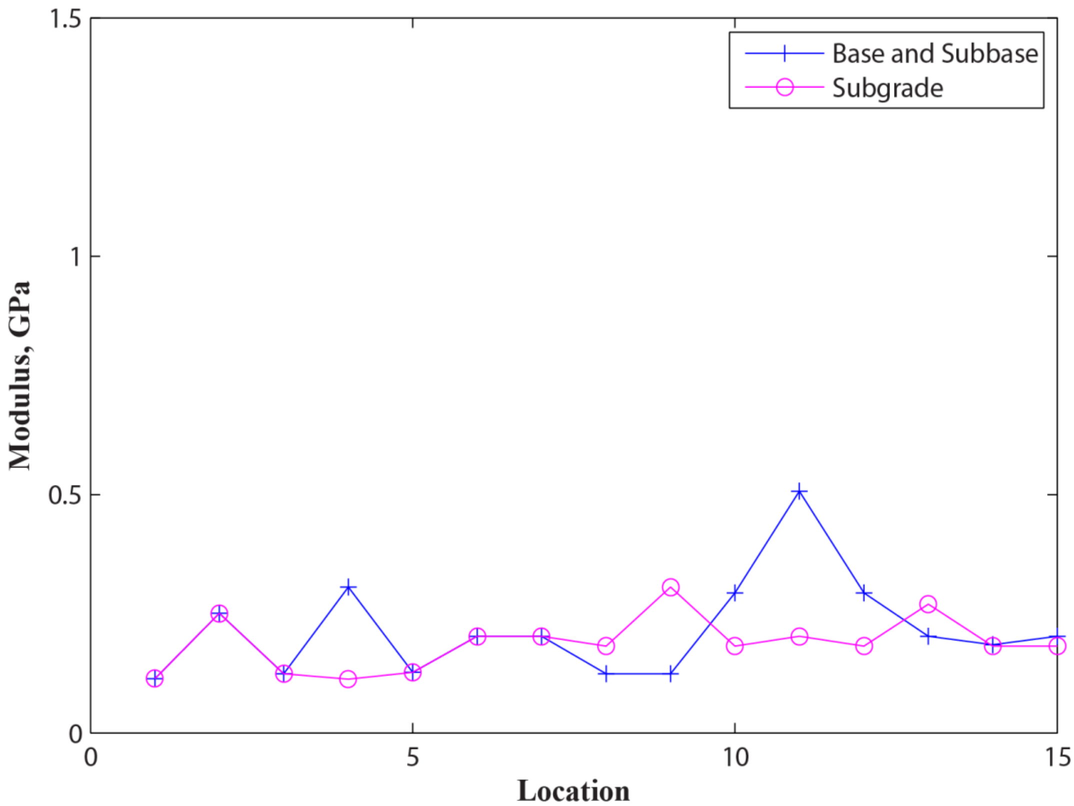

Figure 9.5 Effective layer moduli versus location for site 4

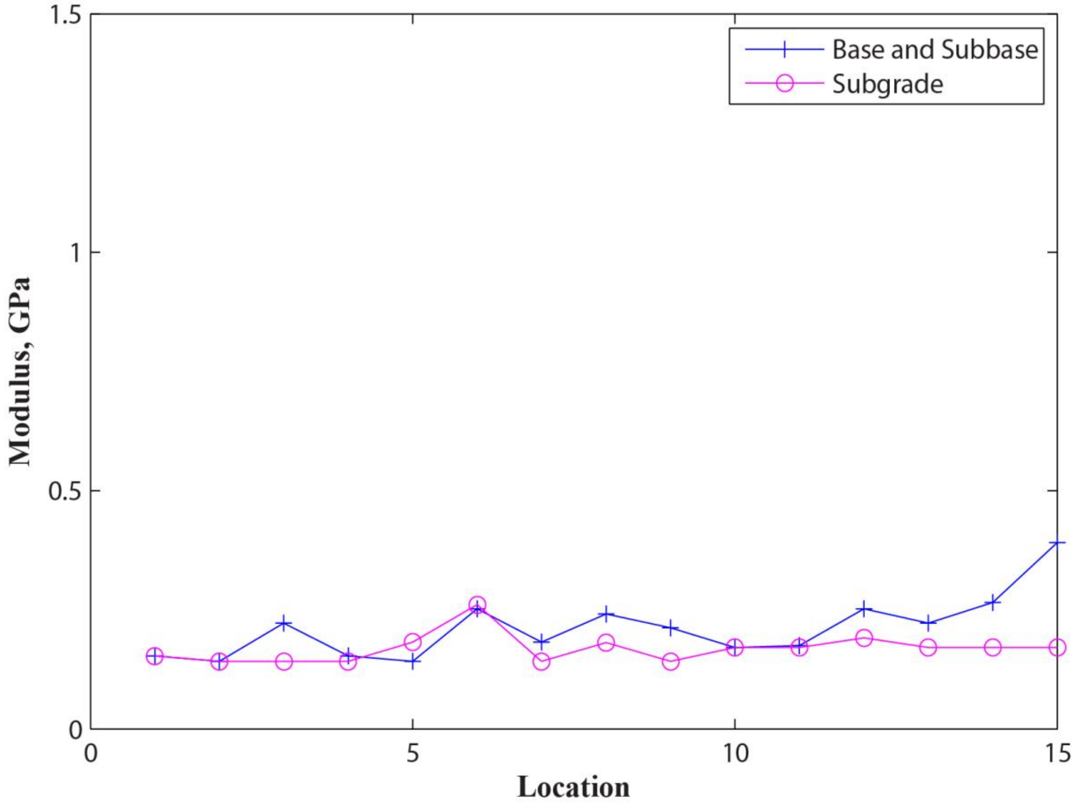

Figure 9.6 Effective layer moduli versus location for site 5

Figure 9.7 shows the effective layer moduli of base and subgrade layer for project 6. The analysis was repeated with the modulus values of these particularly two layer. From the figure presented, it can be seen that the base layer modulus was computed with subgrade layer modulus to remark a similar trend of the deflection at the various locations. The average deflection for base layer modulus was 0.27 GPa while 0.23 GPa for subgrade layer. This showed that the modulus values of these layers were locked to enable to program to give a robust solution and multiple FWD test measurements and predicting were the key in ascertaining the consistency of the modulus values. Mehta and Rogue (2003) and Ghadimi, Nega and Nikraz (2015) repeated the analysis base and subgrade modulus values because of the subbase layer modulus was kept constant. Mehta's effective layer moduli for base layer was approximately 0.26 GPa while 0.22 GPa for subgrade at thirty locations of investigation.

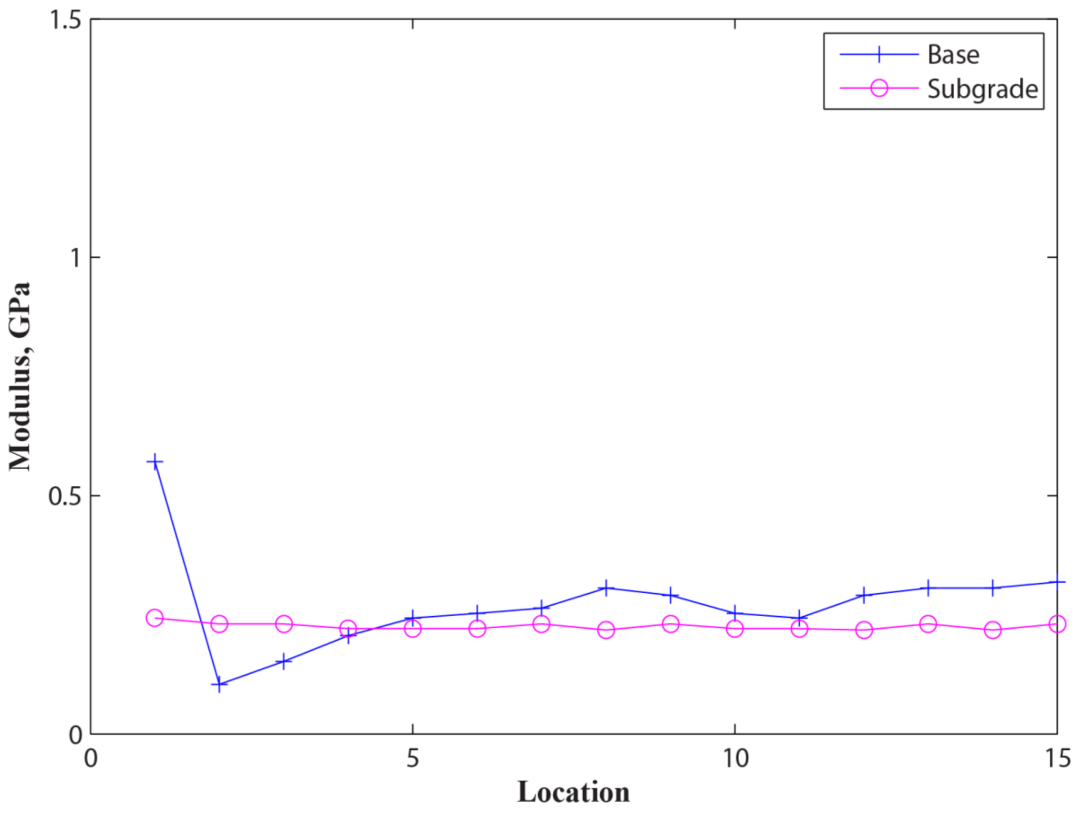

Figure 9.7 Effective layer moduli versus location for site 6

Figure 9.8 shows the effective layer moduli of base, subgrade and damage binder course for project 7. As it can be seen from the FWD analysis, the base and subgrade modulus were closer to each other in the computed deflection, which was 0.19 GPa for base layer and 0.20 GPa for subgrade layer, and had a similar trend of variation at all locations. However, the binder course had two sections: damage and undamaged section. This difference in deflection could be due to high continuing stresses at a depth or lack of load transfer to the bottom layers due to the damage asphalt concrete layer. Oliveira et al. (2009) compared between fatigue lines of damage and undamaged binder course specimen of flexible pavement. Results indicated that the number of load application required to cause a reduction in initial modulus by 50% was defined as a number of cycles of failure. The main reason for the difference (damage and undamaged) behavior of the asphalt mixes is the strain and stress controlled testing. Similarly, Nega, Nikraz and Leek (2013b), Nega et al. (2013a) and Mehta and Rogue (2003) observed cores that indicated cracking below some of the cores and matched reasonably well with effective layer moduli of the damaged binder course at the bottom of the cores.

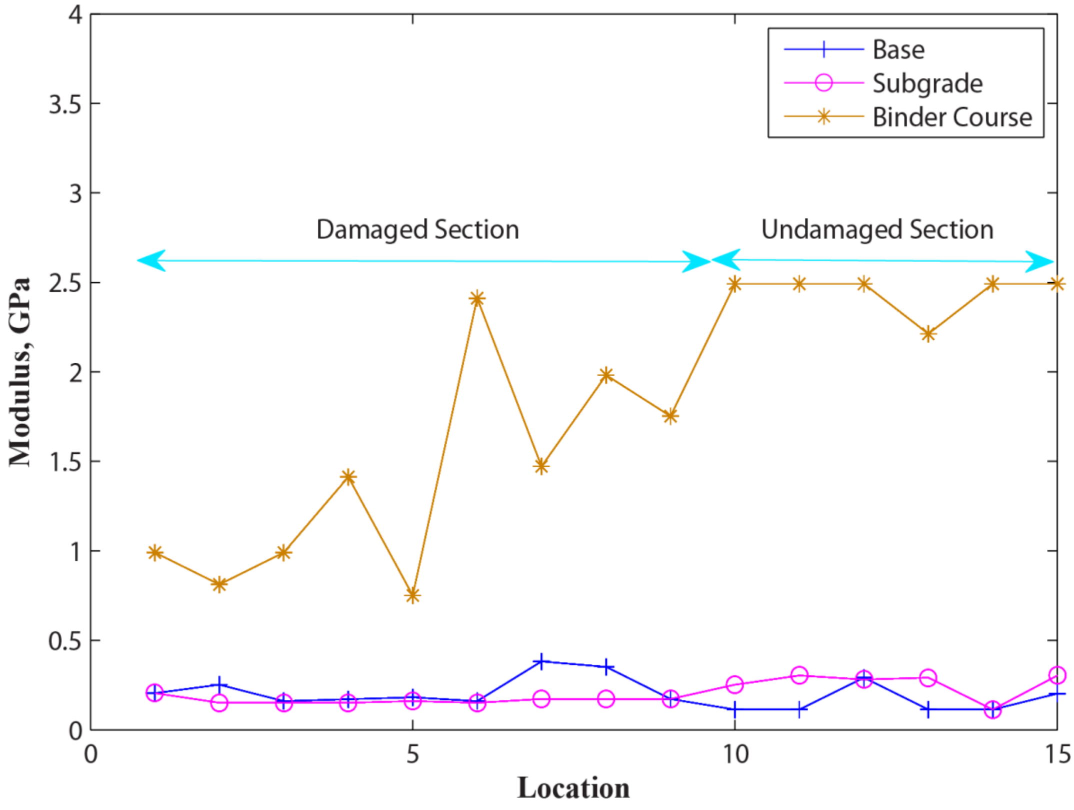

Figure 9.8 Effective layer moduli versus location for site 7

The subgrade was modeled as a finite-thickness, homogenous, linear-elastic layer place on top of a bedrock and dynamic deflection basin were obtained by computed the deflection at the sixteen geophone locations using a variable subgrade depth with an average modulus of 236 MPa and 40 kN loads distributed over area of (0, 200, 300, 400, 500, 600, 750, 900 and 1500 mm). The thickness of the subgrade used was (0 -3 m/0-0.15ch). A poisson's ratio of 0.30 was assumed for the subgrade (granular). The deflections obtained for each model were normalized to 700 kPa using the following equation:

$$D_i^* = \frac{L_a}{L_b} D_i \tag{9.14}$$

where, D_i^* is the normalized deflection (mm) in sensor i (sensor located from the center of the applied load L); D_i is the deflection (mm) in sensor i (sensor located from the center of the of the applied load); L_a is the load level in target a and L_b is the load level applied during test b.

$$\frac{L_a}{L_b} = \frac{1}{D_0} \tag{9.15}$$

where, D_0 is the center load deflection (sensor 0). Assuming a semi-infinite space, the theoretical pressure distribution under a rigid plate that is used FWD testing can be expressed as (Ullidtz 1998):

$$q(r) = \frac{q_a}{2(a^2 - r^2)^{0.5}} \tag{9.16}$$

where q is the applied pressure; a is radius of the plate and r is the distance from the center of the plate. From equation (9.14) and (9.15), a simplify equation can be written as follow:

$$D_i^* = \frac{D_i}{D_0} \tag{9.17}$$

If the solution for a point load a homogenous half-space in integrated over the area of the rigid plate of the FWD with the distribution pressure that was given by equation (9.16) and then, the maximum deflection equation is given as (Appea 2003):

$$D_0 = \frac{\pi(1-\mu^2)q_a}{2E} = \frac{(1-\mu^2)p}{20E} \tag{9.18}$$

where, E is modulus of elasticity; μ is the poisson's ratio and p is applied load.

The dynamic deflection basin at the various distances from the load center from nonlinear and linear-elastic behavior for different sections, are shown in Figure 9.9 through 9.15. As it can be seen from the FWD linear and nonlinear analysis, the measured deflection fell down between the calculated deflection from the nonlinear and linear elastic behavior for all sections for all projects/sites. It can also be observed that the measured deflection approximately followed a similar trend for all project (project 1-7). However, the maximum deflection at D_0 (sensor 0) for all sections for project1, 4 and 5 are high (between range of 320-550, 550-910 and 300-800 µm, respectively) as compared to other deflection (see Figure 9.9, 9.12 and 9.13). Their behavior also looked like nonlinear behavior. While the maximum deflection (sensor 0) for project 2, 3, 6, and 7 ranged an approximate average of 400 to 650 µm (see Figure 9.10, 9.11, 9.14 and 9.15).

The modular ration between the subgrade and the rigid foundation can create some influence on deflection in some of the layer system. However, in this case, it is totally negligible because the variation in the subgrade modules was lesser than the difference between the subgrade modulus and rigid foundation. It should be understood that some of the sections have the same moduli, which is included in the same calculation for linear-elastic and nonlinear analysis; and as the results of these, measured deflection fall down between calculated linear and nonlinear values for most of the section for all projects.

Appea (2003) analyzed two-layer system analysis of subgrade. The deflection results from seven sensor spacing's (0, 305, 452, 609, 914, 1219, and 1524 mm) were calculated using the KENLAYER software for twelve sections (section A-L). Results were shown that some of the sections had the same moduli and used the values in order to calculate for linear elastic as well as nonlinear analysis. The maximum deflection range at D_0 (sensor 0) for majority section were approximately between 200 to 500 μm with a similar deflection trends.

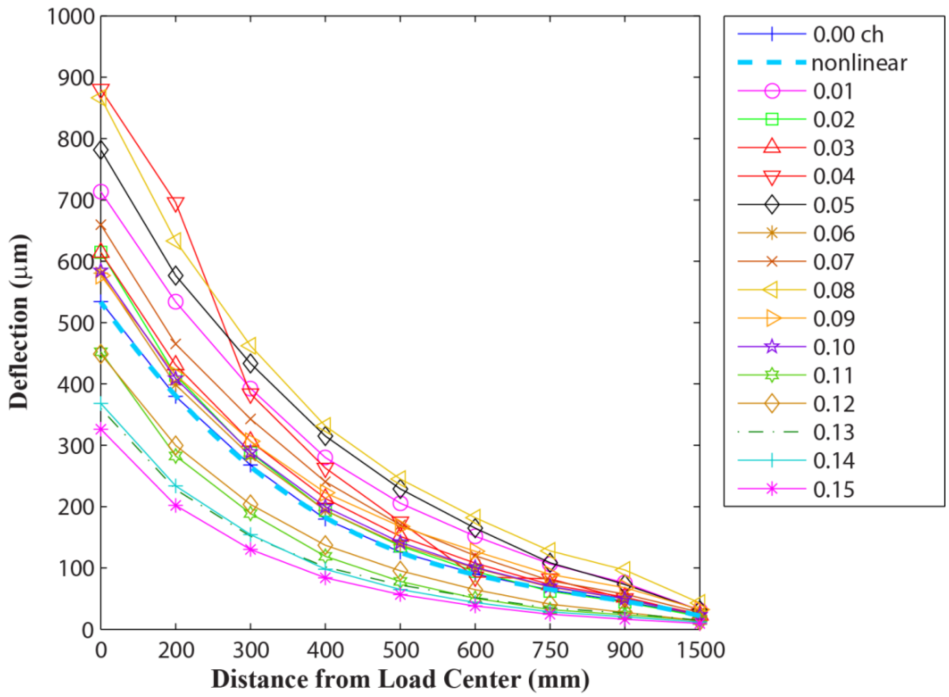

Figure 9.9 Dynamic Deflection Basin at the Various Locations for Site 1 from Nonlinear Analysis

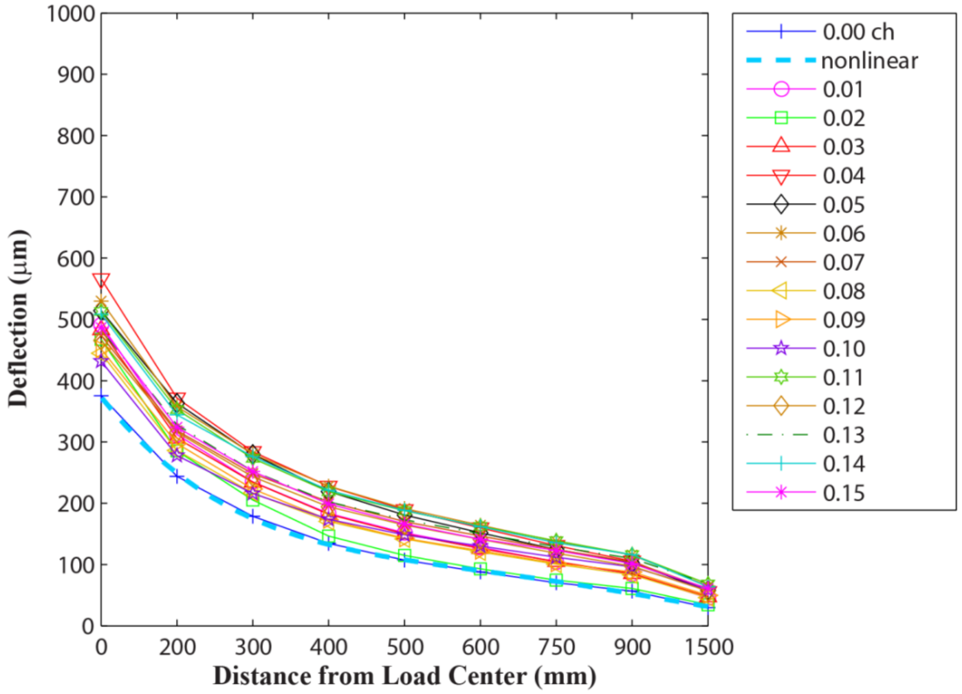

Figure 9.10 Dynamic Deflection Basin at the various Locations for Site 2 from Nonlinear Analysis

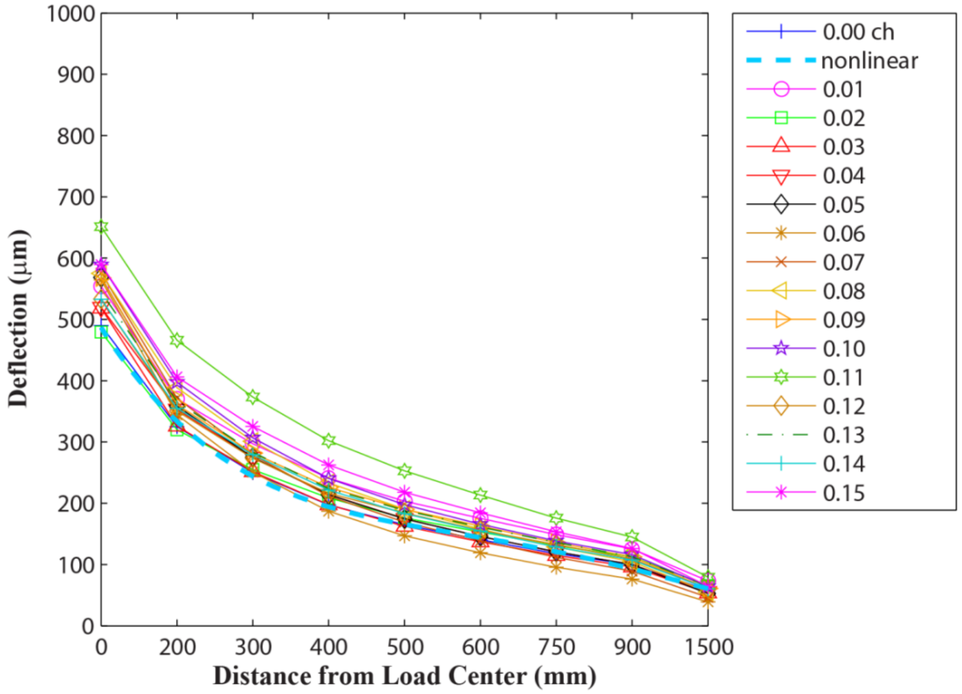

Figure 9.11 Dynamic Deflection Basin at the various Locations for Site 3 from Nonlinear Analysis

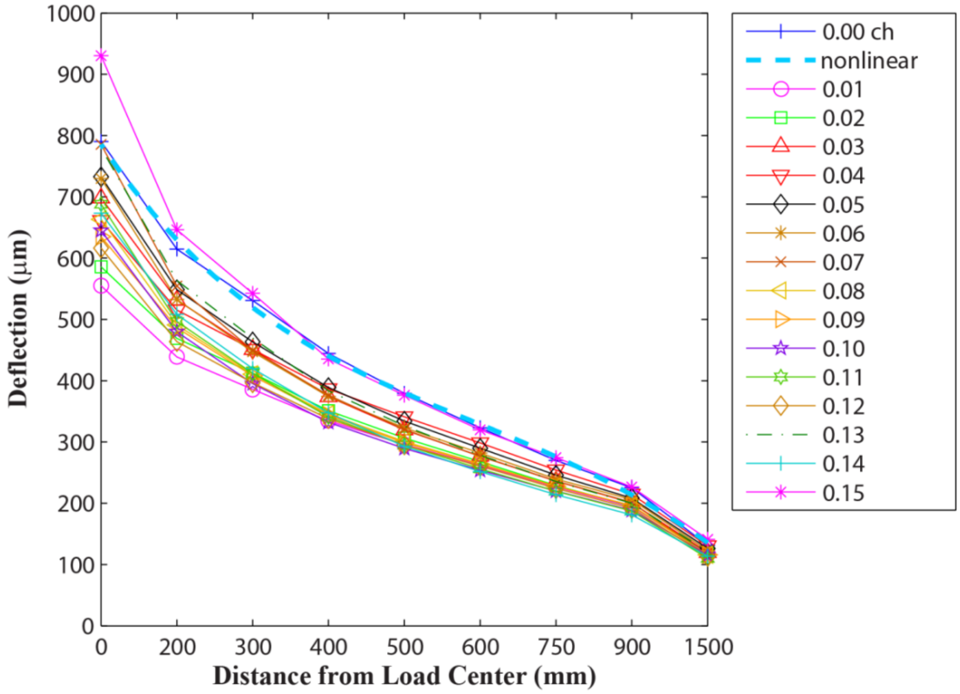

Figure 9.12 Dynamic Deflection Basin at the various Locations for Site 4 from Nonlinear Analysis

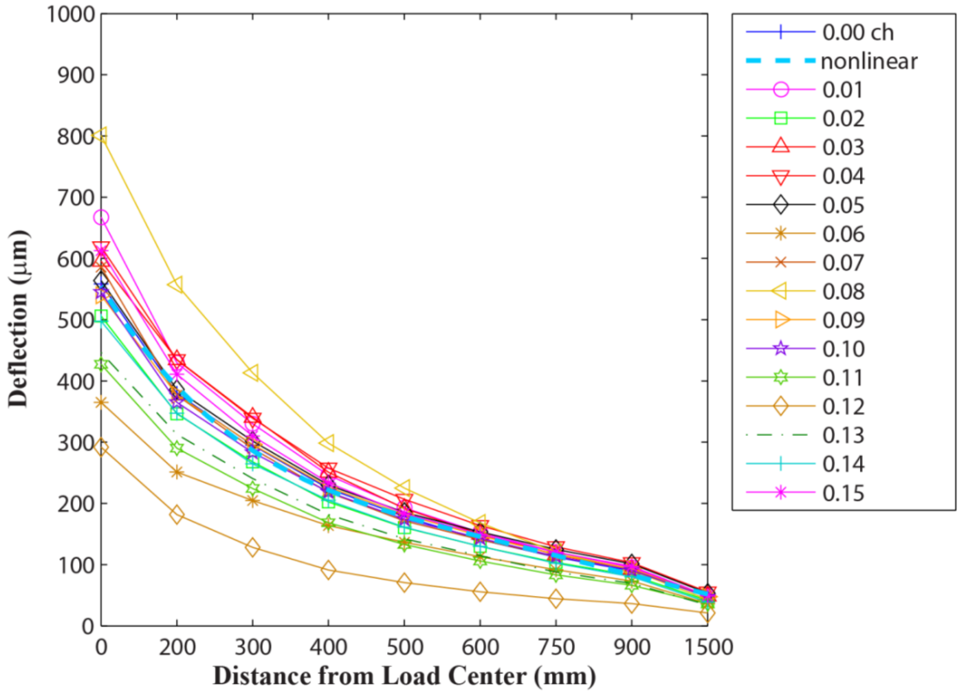

Figure 9.13 Dynamic Deflection Basin at the various Locations for site 5 from Nonlinear Analysis

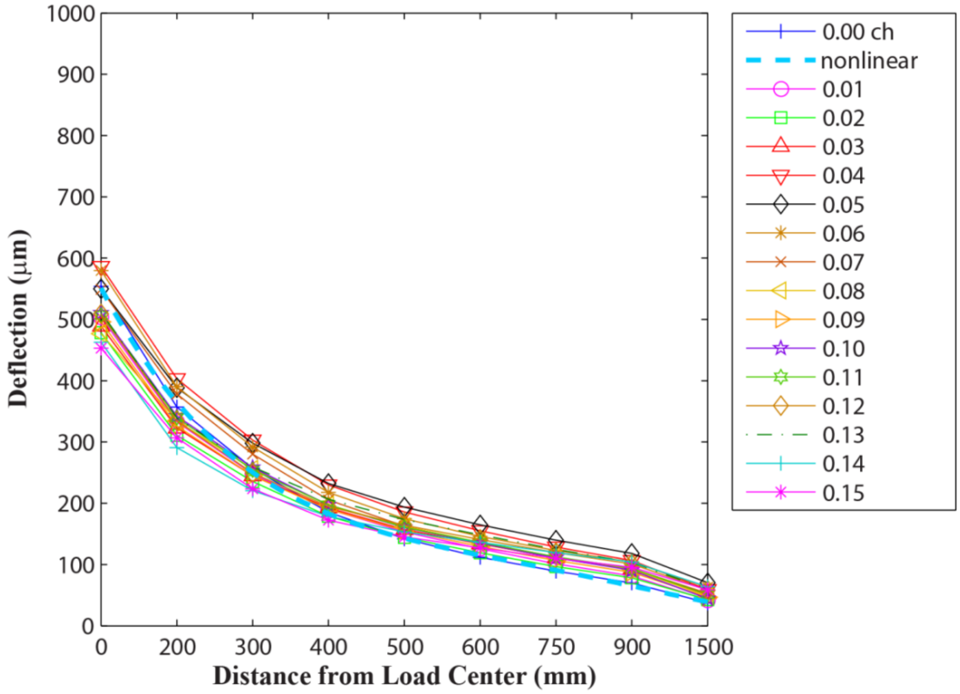

Figure 9.14 Dynamic Deflection Basin at the various Locations for site 6 from Nonlinear Analysis

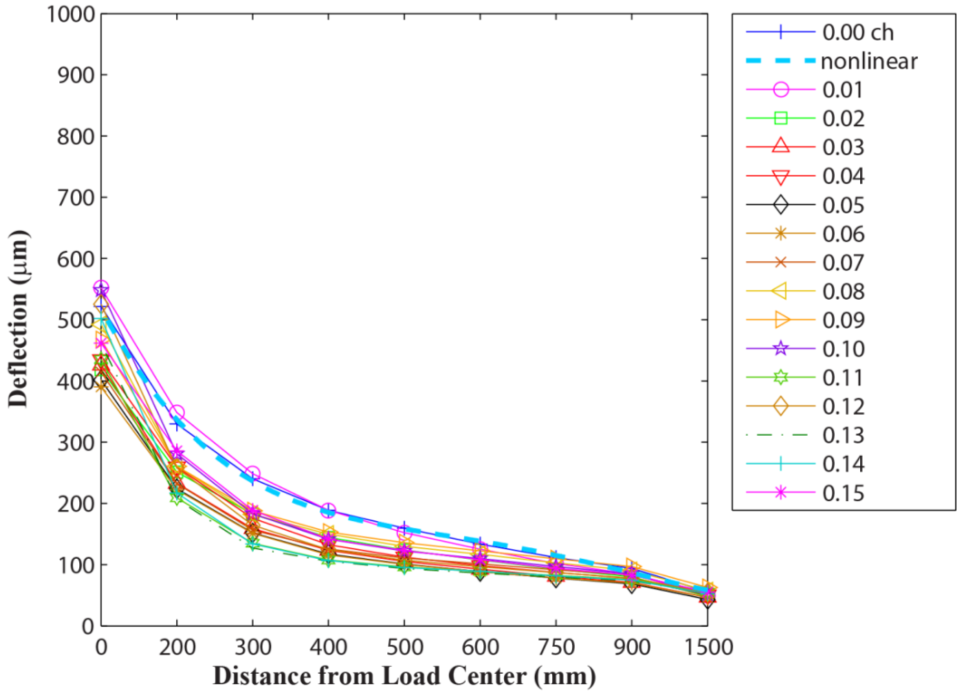

Figure 9.15 Dynamic Deflection Basin at the various Locations for site 7 from Nonlinear Analysis

A summary of the subgrade analysis using invariance constraints approach and the procedure proposed by Ullidtz (1987) to determine the present of a stiff layer (depth to stiff layer) was investigated with ELSYM5 software (equation 9.16 and 9.17 applied).

The average subgrade moduli using different analysis method is shown in Figure 9.16. From the subgrade analysis using different approach, it can be seen that results of the different approach are generally in agreement. Although differences were observed in some section, in particular Section D from linear elastic analysis method. The percentage difference for this Section is 22% for the linear elastic analysis. The average results obtained from the layered elastic analysis does not differ much from the results the apparent subgrade formulae (figure 15). They show a similar trend, with Section D having the highest moduli and Section G having a little bit lower moduli. But all sections have reasonable results. A stiffer layer is possible in section B, C, E, F and G, and the depth to stiff layer is estimated at about 4.27 m (about 14 ft); this is agree with predictions from Ullidtz method (Ullidtz 1987), which is suggested the presence of a shallow stiffer at around 4.57 m (about 15 ft)..

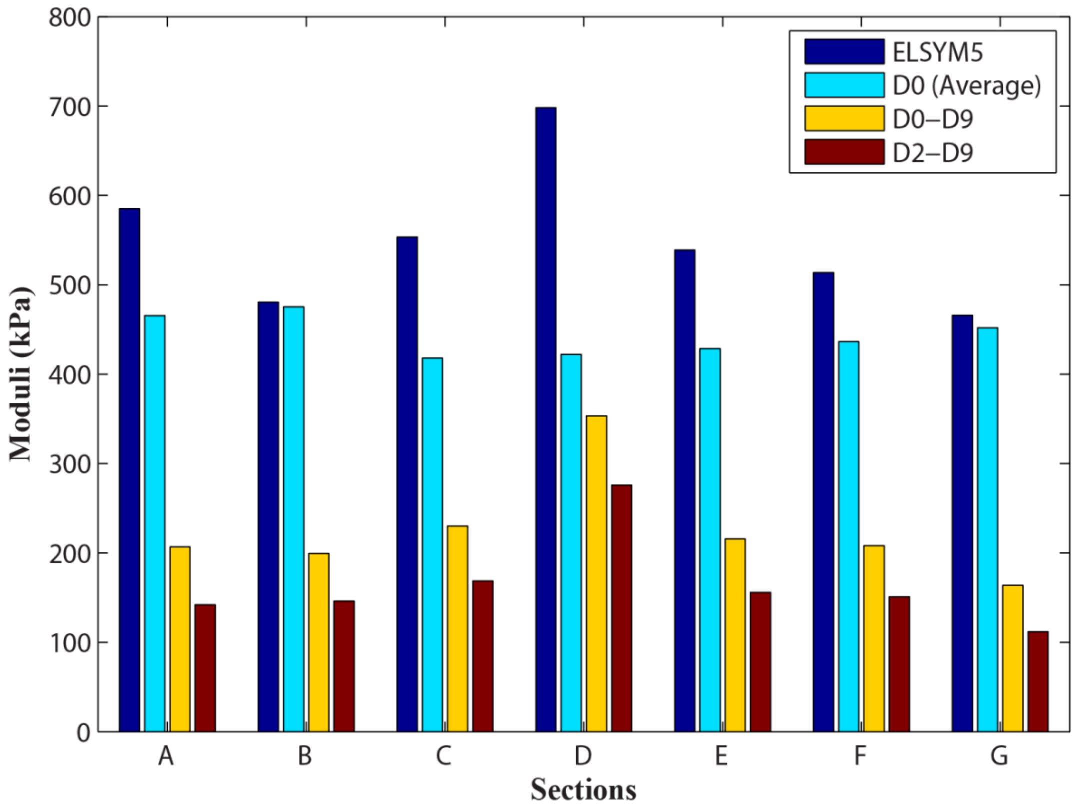

Figure 9.16 Average subgrade moduli determined using different analysis method

A comparison of average vertical strain at the top subgrade layer using WESLEA (WES), Finite Element Method (FEM) and Method of Equivalent Thickness (MET) is shown in Figure 9.17. This influence lines for vertical strain at top of subgrade as measured by four different gauges are also among as the predicated by three different response models: WES, FEM and MET. From the model presented, it can be seen that the vertical strain at the top of subgrade using WESLEA, Finite Element Method and Method of Equivalent Thickness multi-layer computer program analysis were closed one to another in computed deflection with a similar trend of variation at various distance. This showed a reducing in vertical surface deflection and the critical tensile strain in the asphalt concrete layer. In this case, MET is seen to result in best prediction as compared to others prediction. However, all the predictions are quite reasonable; and different analyses methods are enable to give a robust solution.

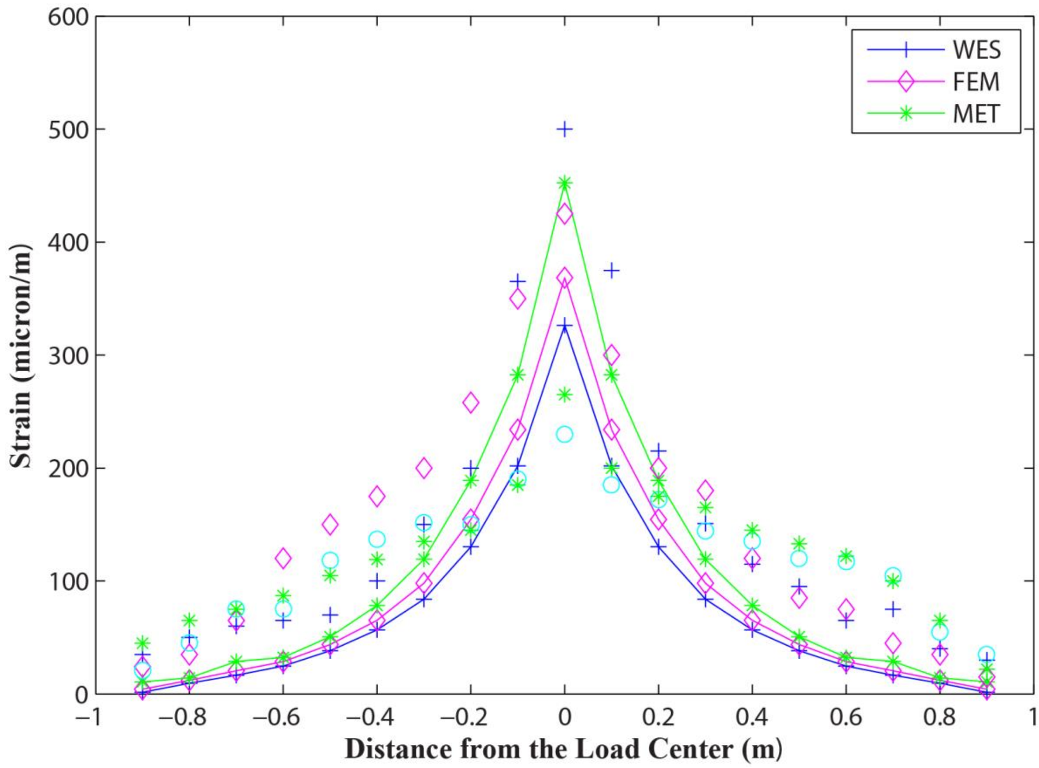

Figure 9.17 Measured and calculated vertical strain in the subgrade

Ullidtz, Zhang and Baltzer (1999) evaluated a pavement response and performance of models from instrumental tests in Danish Road Testing Machine. The pavement was instrumented with gauges for measuring stresses and strains at the critical positions. Layer moduli were determined from FWD testing using different backcalculation procedures. Then, stresses and strain were calculated at the position of the instruments, and compared to the values. Results indicated the vertical stress on the subgrade is overestimated by the linear elastic method, but underestimated for two non-linear subgrades. The vertical strain in the subgrade, which is an important design parameter, is also underestimated by a factor of two by the linear elastic methods. However, the horizontal strain at the bottom of the asphalt layer reasonable well. The influence lines for vertical strain at top of subgrade as measured by gauges and predicted by different types of response models (i.e. WES, FEM, and MET) also showed that the MET is seen the best predictor as compared to WES and FEM.

9.6. CONCLUSIONS

The dynamic analysis of falling weight deflectometer test and predicting for the strength of character of flexible pavement layer moduli has achieved and algorithms for interpretation of the deflection basin has improved. The variation of moduli of the all layer along the length of sections for majority of the projects were accurate and consistencies with measured and computed predicting. However, some of the projects had some inconsistences in modulus values along the length of the section. Although the results are reasonable but consideration should be taken to fix varied along the section during analysis.

The Normalized dynamic deflection basin at the various distances from the load center from nonlinear and linear-elastic behavior for different section for all the projects obtained a good fit between the measured and computed deflection, and also followed a similar trend. While consideration should be taken when the length of subgrade modulus section was locked or fluctuating in progress to match the deflection because the length of the section along its layer can create incompatible results and lack accuracy either in measured or predict of the layer modulus.

A stiffer layer of the subgrade moduli is possible in section B, C, E, F and G, and the depth to stiff layer is estimated at about 4.27 m (about 14 ft); this is agree with predictions from Ullidtz method (Ullidtz 1987), which is suggested the presence of a shallow stiffer at around 4.57 m (about 15 ft).

In general, the results of this research demonstrated that it is often difficult to analyze FWD deflection data and the effect of layer condition to obtain a unique set of layer moduli which will conform to field condition. This is particularly true when backcalculation procedures like BISDEF are used exclusively for layer moduli determination. Backcalculation procedures are enhanced when one or more of the layer moduli can be determined accurately by other means such as BISAR and WESLEA generated deflection.

On the top of BISDEF, BISAR and WESLEA can be used to predict the effect of layer condition and layer moduli which can be also used for verified by comparing the measure deflections with BISAR and WESLEA generated deflection. Badu-Tweneboah et al. (1989) investigated flexible pavement layer moduli from Dynaflect and FWD deflections using a linear elastic multilayer computer program (BISAR) to generate deflection for different combination of layer thickness and moduli. The results demonstrated that it is often difficult to analyze Dynaflect and FWD deflection data to obtain a unique set of layer moduli, which will conform to field condition and to analyze the effect of layer condition even if the results are reasonable.

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CHAPTER 10: NEW NUMERICAL SIMULATION FOR STRESSES AND STRAINS CHARACTERISTICS IN FLEXIBLE PAVEMENT USING THREE-DIMENTIONAL NONLINEAR FINITE ELEMENT ANALYSIS

An abridged version of this Chapter is submitted to *Journal of Transportation Engineering*, American Society of Civil Engineers on 7 December 2015 (under review).

ABSTRACT

A new numerical simulation is developed to obtain a various structural parameters: stresses, strain, and displacement of five-layers of flexible pavement using three-dimensional nonlinear finite element (FE) method in ABAQUS. The main purpose of this study is to develop a new simulation of an accurate and effective solution for stresses and strains characteristics problems in flexible pavement. New constitutive model is developed based on theory of Hooke's law (three-dimensional) for stress and strain. Also, 40 kN wheel load to represent a set of dual tires was assumed to be uniformly distributed over the contact area between tire/pavement surface and simulated using linear viscoelastic and nonlinear viscoelastic material. In addition, four different tire inflation pressures were used (350, 490, 630 and 700 kPa). Results showed that the new developed constitutive model for stress-strain behavior and/or characteristics is capable of considering the effects of stress and strain in pavement layers and predicting of various type of observed in flexible pavement failure and the effects of various pavement design parameters. The implementation of this new developed constitute model is verified against published results of laboratory test data. Results has also shown 40% reduction of vertical plastic strain of each layers of flexible pavement with nonlinear viscoelastic materials as compared with linear viscoelastic material after 7,500 s repeating time of 30, 000 cyclic loading.

Author keywords: New simulation; stresses; strains; pavement analysis; flexible pavement; finite element; nonlinear; three dimensional; tire contact area; tire inflation pressure; cyclic loads

10.1. INTRODUCTION

Pavements are deceptively very complex structure system that involves the interaction of numerous variables. The pavement performance has been influenced by several factors such as traffic loading (stresses and strains as various changes with wheel and axle load traveling at different speeds), the environmental (temperature and moisture), material properties and construction practice. Pavement analysis and design procedures depend heavily on empirical relationships based on long-term experience and field tests such as the American Association of State Highway Official (AASHO) Road Test (AASHO1962; HRB1961).

The AASHO Road Test was performed in the late 1950s, is the basis for most pavement design procedures that use the American Association of State Highway and Transportation Officials (AASHTO) methods of equivalent factors (AASHTO1972; Helwany, Dyer & Leidy 1998). The relationships between traffic loading and pavement performance obtained from the AASHO Road Test are recognized to apply only to the condition under which they were developed. The relative damage to pavement caused by new vehicle characteristics and configurations may be different from that caused by the axle loads used at the AASHO Road Test (AASHTO1986a; Helwany, Dyer & Leidy 1998).

There have been a numbers to derive new load equivalent factor in order to account for vehicle characteristics other factor that were not considered in the AASHO Road Test. Many of these studies have produced new load equivalency factors that are intended to replace the AASHTO equivalencies which are currently used by most highway agencies (AASHTO1972; 1986a). The method for load equivalency factor determination into two major categories (Helwany, Dyer & Leidy 1998): (1) empirical methods, which use observed loading and distress data to estimate pavement damage; (2) mechanistic methods, which incorporate pavement (primary) response parameter such stress, strain and deflection to estimate pavement damage.

The mechanistic (primary) response parameters of pavement can be analytical evaluated using damage prediction models. These parameters include the vertical strain on the top the subgrades, the tensile strain at the bottom of the pavement, the surface vertical deflection, and the tensile stress in a concrete pavement (Helwany, Dyer & Leidy 1998). There are several multi-layer computer programs such as KENLAYER, ELSYM5, CHEVRON, EVERSTRS, WESLEM, ILLI-PAVE, DAMA, MnPAVE, BISAR, CIRCLY5, MICHPAVE, and ABAQUS that are available for solution of the stresses and strains or “boundary values” problems of a multi-layered pavement system (AASHTO2008; Elshaer 2009; Harichandran, Baladi & Yeh 1989; Harichandran, Baladi & Yeh 1990; Huang 1993; Kopperman, Tiller & Tseng 1986; Michelow 1963; NCHRP2002; Nega, Nikraz & Al-Qadi 2015b; Nega, Nikraz & Al-Qadi 2015a; Raad & Figueroa 1980; Van Cauwelaert et al. 1989). These programs are divides in two major categories: (1) finite element method, and (2) layered elastic theory. A variety of material constitutive models such as linear elastic, non-linear elastic, viscoelastic, and elasto-viscoplastic models can be employed to describe the behavior of the pavement materials.

10.2. STRESS AND STRAIN IN FLEXIBLE PAVEMENT

10.2.1. HOMOGENOUS MASS

The simple was to characterize the behavior of a flexible pavement under the wheel loads is to consider it as a homogeneous half-space (Huang 1993). A half-space has an infinitely large area and an infinite depth with a top plane on which the loads are applied. The original Boussinesq (1885) theory was based on a concentrated load applied on an elastic half-space (AASHTO1993; 2008; Huang 1993). The stresses, strain and deflections due to concentrated load can be integrated to obtain those due to a circular loaded area. As the matter of the fact, before the development of layered theory by Burmister (1943), much attention was given to Boussinesq solutions because they were the only ones available for a decades (Huang 1993). The theory can be used to determine the stresses, strain and deflections in the subgrade if the modulus ratio between the pavement and the subgrade is close to unity as exemplified by a thin asphalt surface a thing granular base. If the modulus ration is much greater than unity, the equation must be modified, as demonstrated by the earlier Kansas design method (KSHC1974). Pavement structural analysis includes three issues: (1) material characterization; (2) theoretical model for structural response; (3) environmental conditions.

10.2.2. MATERIAL CHARACTERIZATION

According to Yoder and Witczak (1975), three aspects of material behavior are typically consider for pavement analysis and design: (1) relationship between the stress and strain (linear or nonlinear; (2) time depending of strain under a contact load (viscous or non-viscous); and (3) degree to which the material can recover strain after stress removal (elastic or plastic).

10.2.3. THEORETICAL RESPONSE MODEL

The theoretical response models for the pavement are typically based on a continuum mechanics approach (Elshaer 2009; Nega, Nikraz & Al-Qadi 2015b). These models can either a closed-formed analytical solution or a numerical approach. Since the passed decades, various theoretical response models have been developed from analytical solutions such as Boussinesq equations based on elasticity to 3D-dimensional dynamic finite element methods (FEM) (Helwany, Dyer & Leidy 1998; Huang 1993).

10.2.4. ENVIRONMENTAL CONDITION

Environmental condition can have a great impact on pavement performance. The most common environmental factors in pavement structural analysis are: temperature, aging, resting period and healing, and frosting as the result of moisture variation. Specially, when high moisture content is combining with low temperature, it can lead to both frost heaves (during freezing) and loss of subgrade support (during thaw) and then, significantly weakening the structural capacity of the pavement and then, structural damage and pre-mature structural failures can occur.

10.3. FLEXIBLE PAVEMENTS

Flexible pavement in general consist of on asphalt-bound surface course or layer on top of unbound base and subbase granular layers over the subgrade soil. In some cases, the subbase and/ or base layers may be absent (e.g. full depth asphalt pavement), while in other the base and or subbase layers may be stabilized using cementitions or bituminous admixture. Drainage layers may also be provides to remove water quickly from the pavement structure. Some common variations of flexible pavement systems are shown in Figure 10.1. Full depth asphalt pavements are used primarily for flexible pavements subjected to very heavy traffic loadings.

Hot mix asphalt concrete produced by an asphalt plant is the most common surface layer material for flexible pavement, especially for moderately to heavily trafficked highways. Dense-graded (i.e., well graded with a low void ratio) aggregates with a maximum aggregate of about 25 mm (1 in.) are most commonly used in hot mix asphalt concrete, but a wide variety of other types of gradation (e.g. gap-graded) have also been used successfully for special condition (Christopher, Schwartz & Boudreau 2006). The Superpave procedure had become the standard for asphalt mixture design, although country and local government agencies may still use the older Marshall and Hveem mix design procedures (The Asphalt Institute 1984). The asphalt surface layer in a flexible pavement may be divided in sub-layers. Typical sub-layers, proceedings from the top down ward, are as follows (Christopher, Schwartz & Boudreau 2006):

- ***Seal coat*:** A thin asphaltic surface treatment used to increase (or restore) the water and skid resistance of the road surface. Seal coats may be covered with aggregate when used to increase resistance.
- ***Surface course*** (also called the ***wearing course***): The top must sublayer (in the absence of a seal coat) of the pavement. This is typically constructed of dense graded asphalt concrete. The primarily design objectives for the surface course are water proofing, skid resistance, rutting, resistance, and smoothness.
- ***Binder course*** (also called the ***asphalt base course***): The hot mix asphalt (HMA) layer immediately below the surface course. The base course generally and often a lower asphalt content than the surface courses. A binder course may be used as part of a thick asphalt layer either for economy (the lower quality asphalt concrete in the binder course had a lower material cost than the higher-asphalt content concrete in the surface course) or if the overall thickness of the surface layer is too great to be paved in one lift.

Figure 10.1 Some common variations of flexible pavement sections (NCHRP 1-37A,2002)

Thin liquid bituminous coatings can be also as additional materials on top of stabilized base layer and on untreated aggregate base layer as follows:

- Tack coat: Applied on top of stabilized base layers and between lifts in thick asphalt concrete surface layers to promote bonding of the layers.
- Prime coat: Applied on untreated aggregate base layer to minimize flow of asphalt cement from the asphalt concrete to the aggregate base and to promote a good interface bond. Prime coats are often used to stabilize the surface of the base to support the paving construction activities above. Cutback asphalt (asphalt cement blended with a petroleum solvent) is typically used because of its greater deep penetration.

Christopher, Schwartz and Boudreau (2006) reported that proper compaction of asphalt concrete during construction is critical for satisfactory flexible pavement performance. Improper compaction can lead to excessive rutting (permanent deformation) in the asphalt layer due to densification under traffic; cracking or raveling of the asphalt concrete due to embrittlement of the bituminous binder from exposure to air and water; and failure of the underlying unbound layers due to excessive infiltration of surface water. Typically construction specification require field compaction levers of 90% or more of the theoretical maximum density for mixture (Christopher, Schwartz & Boudreau 2006). Layers of unbound material below the asphalt concrete layer must be constructed properly in order to achieve to the overall objectives of pavement performance. Nega, Nikraz and Al-Qadi (2015b) also summarized that the design and analysis of flexible pavement required a knowledge of the stresses and displacement due to a variety of surface loading condition.

10.4. FINITE ELEMENT METHOD

The finite element modelling approaching offers the best method of analysis for multilayered pavement system. The two- and three- dimensional and/ or axisymmetric finite element models have different formation and also consider different directional component of stress and strain. There-dimensional and finite element analysis can consider all three- dimensional response components and should predict more accurate pavement response.

Chen et al. (1995) analyzed the effect of high inflation pressure and heavy axle load on flexible pavement performance by using three-dimensional finite element model. Al pavement structure pavements were assumed to be homogenous and linear elastic. Results obtained from their studies were compared to another elastic layer program ELSYM5 (Kopperman, Tiller & Tseng 1986), for a uniform circular pressure and had a close agreement between two models.

Helwany, Dyer and Leidy (1998) studied three- layer flexible pavement system subjected to different type of loading. Axle loading with different tire pressure, different configurations, and different speed were conducted in two-dimensional finite element computer program DACSAR (Deformation Analysis Considering Stress Anisotropy and Reorientation and three-dimensional finite element computer program NIKE3D

(Nonlinear Implicit Three-Dimensional Finite Element, by Lawrence Livermore National Laboratory). Various material constitutive models such as linear elastic, nonlinear elastic, and viscoelastic were employed in these analyses. As a preliminary analysis, the analytical solutions for the one layer system, using DASCAR and NIKE3D, agree with Boussinesq's solution. This study showed that finite element modelling of pavement could extremely useful to predict accurate pavement structure response.

Shoukry and William (1999) used three- dimensional finite element model to backcalculate moduli of pavement structures and were compared the results with predications of backcalculation program such as MODCOMP, MODULUS, and EVERCALC (Kim 2007). The measured deflections were obtained from falling weight deflectometer (FWD) test. All the pavement layers were modeled as linear layers and 8-noded solid brick element (C3D8) were used. This three-dimensional finite element analysis has a fair agreement with the backcalculated layer moduli.

Wang (2001) investigated the response of flexible pavement structures with various pavement material properties, model dimensions, and different types of loadings using three-dimensional finite element analysis. He developed an effective meshing tool for three-dimensional model incorporating multiple layers, interlayer debonding and slip, and various loading. The effect of base material nonlinearity was also studied with stress-dependent K-θ model and the effect of spatially varying tire and/ or pavement contact pressure on pavement surface. He concluded the spatially varying tire/pavement pressure affected the response of flexible pavement significantly.
ABAQUS^{TM}, ANSYS^{TM}, and ADINA^{TM} are the general-purpose finite element programs that can provide proper analysis of various engineering problems. Pavement structures modeling has significantly developed and/or advanced in recent year however, pavement analysis with general-purpose program has not frequently been applied to flexible pavement. At present time, only a few researchers have investigated to nonlinear flexible pavement response using the general-purpose of finite element.

Zaghloul and White (1993) evaluated pavement response under FWD loading for flexible pavement using three-dimensional analysis in the ABAQUS^{TM}, A number of material models were modeled as viscoelastic and granular materials (i.e., base/subbase), were modeled using Drucker-Prager models. The Cam-Clay model was used for subgrade soils. Both static and dynamic loading analyses were conducted to predict elastic and plastic pavement responses. This capability help explain pavement response under various loading conditions and for different material characteristics. They found that their model was capable to simulating truck and/ or loads and realistic deformation prediction (Kim 2007), were obtained.

ABAQUS^{TM} was used for dynamic loading response analysis by Uddin, Zhang and Fernandez (1994). They investigate effect of dynamic loading for cracked pavement comparing response with static loading for a linear elastic system and the significant of three-dimensional finite element simulation of pavement. They found that the corresponding static deflection under the linear elastic solution remain higher than the dynamic deflections for a cracked pavement. They also properly simulated longitudinal and transverse cracks on surface by special gap element in ABAQUS^{TM} element set.

Chen et al. (1995) have made a comprehensive study of various finite element pavement analysis programs and showed that the results from ABAQUS^{TM} were compared to those from other program. An attempt of ABAQUS^{TM} finite element program was made from infinite element in the vertical direction for a linear analysis. The results from ABAQUS yielded the lowest tensile strain compared with other program in the linear case.

Kuo, Hall and Darter (1995) developed three-dimensional finite element model for concrete pavement called 3DPAVE. 3DPAVE used the ABAQUS^{TM} program to overcome many of the inherent limitation of two-dimensional finite element models. They performed the feasibility study to find the most appropriate element for two-dimensional and three-dimensional model. The three-dimensional ABAQUS^{TM} finite element modeling was conducted in various loading cases such as interior loading and edge loading cased with C3D27R type ABAQUS^{TM} element. To investigate the effect separation between layers, interface behavior, dowel bars and aggregate interlock were also modeled by ABAQUS^{TM}, element and/or material keyword library. Dense liquid foundation and elastic solid foundation solution were modeled by FOUNDATION and BRICK elements, respectively. Comparison between three-dimensional finite element modeling and full scale field test data prove that 3DPAVE model properly solved for pavement behavior.

As opposed to the relatively simple pavement layered elastic or inelastic theory, numerical simulation using finite element method (FEM) can be a very complex and costly analysis software tool. However, the application of finite element (FE) analysis technique allow as a more accurate simulation of flexible pavement problems. This method can include almost all controlling parameters of dynamic loading and discontinues such as cracks and shoulder joints, viscoelastic can nonlinear elastic behavior, infinite and stiff foundations, system damping, quasistatic analysis, crack propagation, and others (Loulizi, Al-Qadi & Elseifi 2006). For the last past two decades, FE techniques have been successfully used to simulate different type of pavement problems that could not be modeled using the simpler multilayer elastic theory (Elseifi 2003; Loulizi, Al-Qadi & Elseifi 2006; Uddin, Zhang & Fernandez 1994; Zaghloul & White 1993).

According to Desai (1979) description, the formulation and application of the FE methods are divided in to eight basic steps: (1) discretize the structure into suitable number of small elements; (2) select approximation models for the unknown quantities; (3) define the stress-strain constitutive equations, which describe the response such as strain and displacement of system to the applied force; (4) define the element behavior equations: (5) assemble element equation and introduce boundary conditions from which the equation and introduce boundary conditions from which the equations describing the behavior of the entire problem can be obtained; (6) solve the problem of nodal displacement; (7) calculate other function such as stresses, moments and shear force based on the assumed constitutive model equations; and (8) finally, interpret results and mesh refinement from which the problem output is evaluated and then, decide if necessary mesh refinement in needed (Al-Qadi, Elseifi & Leonard 2003; Al-Qadi et al. 2004; Loulizi, Al-Qadi & Elseifi 2006).

It is important to realize that in the FE analysis method, the level of accuracy that is obtained depends on different factors, which also includes the degree of mesh element refinement such as element dimensions, the order of elements in particular higher-order elements usually improve accuracy, and the location of the evaluation.

10.5. LAYERED ELASTIC THEORY

The layered elastic theory is the tool most often used to calculate flexible pavement response to traffic or trucking loading. This is mainly due to its simplicity and to the fact that pavement engineers have been exposed to it since 1940s. For the first time, Burmister developed a closed-form solution for a two-layered linearly elastic half-space problem and then, which was later extended to a three-layer system (Burmister 1943; 1945; Huang 1993; Loulizi, Al-Qadi & Elseifi 2006).

Since the two-and three-layered elastic theory development, with the advance in technology, the theory has been extended to deal with multi-layer system and a large number of computer programs have been developed (Al-Qadi, Elseifi & Leonard 2003; Elshaer 2009). The major assumptions of the layered elastic theory are the following (Huang 1993): (1) each layer is assumed homogenous, isotropic, and linear elastic; (2) all materials are weightless, which means, no inertia effort is considered; (3) all layered are assumed to be infinite in lateral extent; (4) all layers have a finite thickness except for the subgrade, which is assumed to be infinite; (5) pavement system are located statically over a uniform circular area; and (6) the compatibility of stresses and strains is assumed to be satisfied at all layer interface.

The layered elastic theory is based on classical theory of elasticity. The theoretical procedures is well described in details on literature by Huang (1993). However, a few equations are presented below for clarity and understanding. The classical theory of elasticity assumes a stress function, ϕ, which satisfies the governing differential equation:

$$\nabla^4 \phi = 0 \tag{10.1}$$

where, ϕ is the assumed function and

$$\nabla^4 = \left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2}\right)\left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2}\right) \tag{10.2}$$

where, r and z are the cylindrical coordinates for radial and vertical directions, respectively. Once the stress function is found, the stresses and displacements can be determined using the following equations:

$$\sigma_z = \frac{\partial}{\partial z}\left[(2-\nu)\nabla^2\phi - \frac{\partial^2}{\partial z^2}\phi\right] \tag{10.3}$$

$$\sigma_r = \frac{\partial}{\partial z}\left(\nu\nabla^2\phi - \frac{\partial^2}{\partial r^2}\phi\right) \tag{10.4}$$

$$\sigma_t = \frac{\partial}{\partial z}\left(\nu\nabla^2\phi - \frac{1}{r}\frac{\partial\phi}{\partial r}\right) \tag{10.5}$$

$$\tau_{rz} = \frac{\partial}{\partial r}\left[(1-\nu)\nabla^2\phi - \frac{\partial^2\phi}{\partial z^2}\right] \tag{10.6}$$

$$\omega = \frac{1+\nu}{E}\left[(1-2\nu)\nabla^2\phi + \frac{\partial^2\phi}{\partial r^2} + \frac{1}{r}\frac{\partial\phi}{\partial r}\right] \tag{10.7}$$

$$u = \frac{-1+\nu}{E}\left(\frac{\partial^2\phi}{\partial r\partial z}\right) \tag{10.8}$$

where, σ_z is stress in the vertical or z direction; σ_r is stress in radial or r direction; σ_t is the stress in the tangential or t direction; τ_{rz} is shear stress; ω is the displacement in the vertical or z direction; u is the displacement in radial or r direction; and ν is the poisson's ratio. Several softwares are available to compute stresses, strains and displacement in flexible pavement system using the layered elastic theory.

10.5.1. PAVEMENT RESPONSE MODELS: LINEAR ELASTIC MATERIAL

Structural analysis in pavement has been greatly developed since the initial studies carried out by Boussinesq in which soils were modeled as a linear-elastic material (Boussinesq 1885). Boussinesq's theory was then extended to multilayer elastic model due to the work of Burmister (Burmister 1945) and Schiffman (Schiffman 1962). Boussinesq's equations were original developed for a static point load. And then later, Boussinesq' equations were further extended by other researcher for a unfirming distributed by integration (Newmark 1947; Sanborn & Yoder 1967).

Boussinesq's equations are seldom used as the main pavement analysis and design theory today. However, Boussinesq' theory is still considered a useful tool for pavement analysis and it provides the basic for several method that are being currently used (Elshaer 2009; Huang 1993; Trauner 2003). Yoder and Witczak (1975) suggested that Boussinesq theory can be used to estimate subgrade stresses; strain , and deflections when the modulus of base and the subgrade are close. Ullidtz (1998) also suggested that pavement surface modulus, and the equivalent “modulus of elasticity for layered soil using weighted or weighted modulus” calculated from measured surface deflections based on Boussinesq’s equations, can be used as an overall stiffness of pavement. Pavement surface deflection response under axle load distribution along various layers is shown in Figure 10.2.

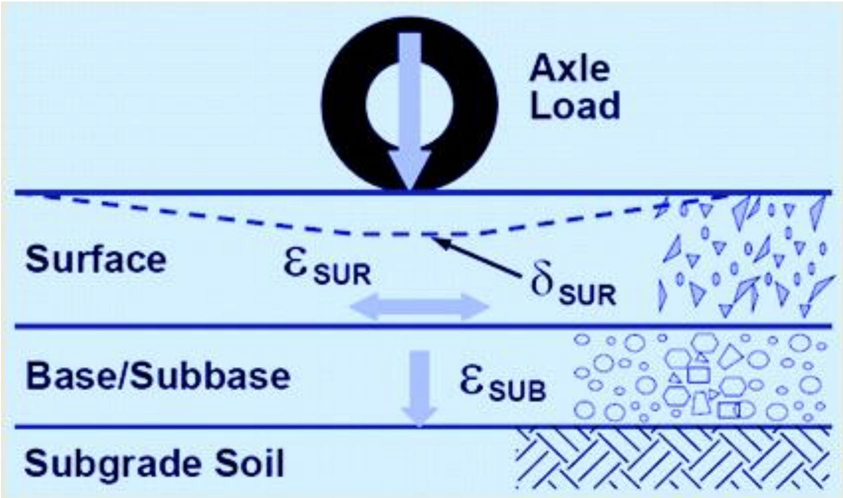
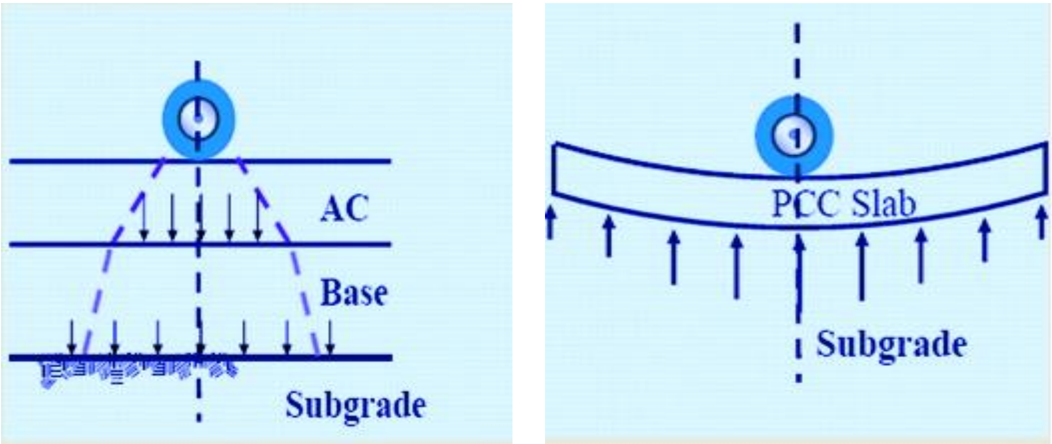


Figure 10.2 Surface deflection response under axle load distribution along various layers

In the analysis of flexible pavement, the axle loads on the surface of the pavement layers produce two types of strains, which are believed to be the most critical for analysis and design purpose. These are the horizontal tensile strains ε_t at the bottom of the asphalt or bitumen layer, and the vertical compression strain ε_z at the top of subgrade layer. If the horizontal tensile strain or tangential strains ε_t are substantial or excessive, cracking on the surface layer will occur and then, pavement will fail due to fatigue; which mean the pavement will fail due to fatigue cracking. If the vertical compression strains ε_z are substantial, permanent deformations are noticed at the surface of flexible pavement structure and then, pavement will fail due to rutting because the subgrade layers are overloading.

Flexible and rigid pavement responses to traffic loading are different. A flexible pavement supports traffic by distributing the traffic load to the subgrade. The design approach for these structures is to provide a thickness of pavement which will limit loads transmitted to subgrade to acceptable levels. A rigid pavement utilizes the flexural strength of a Portland cement concrete slab for support of traffic loads structural design is based on providing a thickness of slab sufficient to limit flexural stresses to below failure limit. The structural response for asphalt concrete (AC) and Portland cement concrete (PCC) of flexible and rigid pavement, respectively are shown in Figure 10.3.



(a) Flexible Pavement (b) Rigid Pavement

Figure 10.3 Structural response of flexible and rigid pavement during model analysis

There are different modes of failure in flexible pavement. Flexible pavement is constructed with high durability and surface skid resistance under in-service condition. It is expected to exhibit a minimum possible cracking and rutting in flexible pavement layers. A large stresses and strains are produced with thicker layers that carried out a higher flexural stress than thinner layer when it is subjected to large concentrated loads (Gupta & Kumar 2014). Rutting and decreased fatigue life of flexible pavement may attribute due to shortcoming of application of the pavement analysis and design, and lack of knowledge of in-service in flexible pavement.

The flexible pavement may consist of a relatively thin wearing surface built over a base course and subbase course; they rest upon the compacted subgrade. In contrast, rigid pavement are made up on Portland cement concrete and may or may not have a base course between the pavement and subgrade (Yoder & Witczak 1975). The essential difference between the two types of pavement, flexible and rigid, is the manner in which they distributed the load over the subgrade. The rigid pavement, because of its rigidity and high modulus of elasticity, tends to distribute the load over the relatively wide area of soil; thus, a major portion of structure capacity is supplied by slab itself.

The major factor considered in the design of rigid pavements is the structure strength of the concrete. For this reason, minor variations in subgrade strength have a little influence upon the structural capacity of the pavement. Asphalt pavements may possess stiffness as Portland cement concrete pavements. This is true when stabilized materials are used in any of the pavement component or if for example, relatively thick asphaltic concrete layers are used. On the other hand if very thin surface are used (for example, surface treatment), the pavement can be consider to be flexible (Yoder & Witczak 1975).

The loading-carrying capacity of a true flexible pavement is brought about by load-distributing characteristics of the layered system. Flexible pavements consist of a series of layers with the highest-quality materials at or near the surface. Hence, the strength of a flexible pavement is the result of building up thick layer and, thereby, distributing the load over the subgrade, rather than by the bending action of slab. The thickness design of the pavement is influenced by the strength of the subgrade. If an asphalt pavement has high stiffness, it may behave essentially a rigid pavement and fatigue of surface or of any pavement component may become critical. In these cases, For example, full-depth asphalt pavements are used in certain situations. This type of pavement undoubtedly approaches the rigid condition and the classical method for designing flexible pavement no longer apply (Yoder & Witczak 1975). The same is true if a cementing agent is used as a stabilizing additive in the base or subbase.

10.6. FLEXIBLE PAVEMENT RESPONSE MODELS

10.6.1. SINGLE LAYER MODEL

Boussinesq (1885) was the first to examine the pavement's response to a load. A series of equation was proposed by Boussinesq to determine stresses, strains, and deflection in homogenous, isotropic, linear elastic half-space with modulus, E, and Passion's ratio, ν. subjected to a static point load, p. Several models are based on Boussinesq equations (Frohlich 1934; Huang 1993), who described the distribution of stress in an elastic, homogenous, isotropic, weightless, semi-infinite solid medium due to a force applied to a point in that medium. The Boussinesq vertical stress component (σ_z) on the point N (Figure 4) is given by:

$$\sigma_z = \frac{3P}{2\pi}\frac{z^3}{R^5} = \frac{3P}{2\pi R^2} cos^3\beta \tag{10.9}$$

where, P is th vertical point load, z is the depth below the surface, and R and β are polar coordinates since $R^2 = r^2 + z^2$ (when, x, y, and z are coordinates of point N since $r^2 = x^2 + y^2$ (Figure 4), the stress at the center line of the load can be expressed as:

$$\sigma_z = \frac{3P}{2\pi z^2} \tag{10.10}$$

The normal vertical stress component, σ_z on point N can be expressed as:

$$\sigma_z = \frac{3P}{2\pi z^2}\frac{1}{[1+(r/z)^2]^{5/2}} \tag{10.11}$$

The Boussinesq vertical stress coefficient (K) is given by:

$$K = \frac{3}{2\pi}\frac{1}{[1+(r/z)^2]^{5/2}} = \frac{0.474}{[1+(r/z)^2]^{5/2}} \tag{10.12}$$

where, Equation (10.10) can be written as:

$$\sigma_z = K(P/z^2) \tag{10.13}$$

The K – values can be presented graphically for different values of z and r since($K = f(r/z)$, where by Equation (10.13) may be used to calculate the vertical and normal stress, which is caused by a point load, P at any point N in the medium restricted by the assumptions (Jumikis 1967). The load and stress in a cylindrical coordinate system is shown in Figure 10.4.

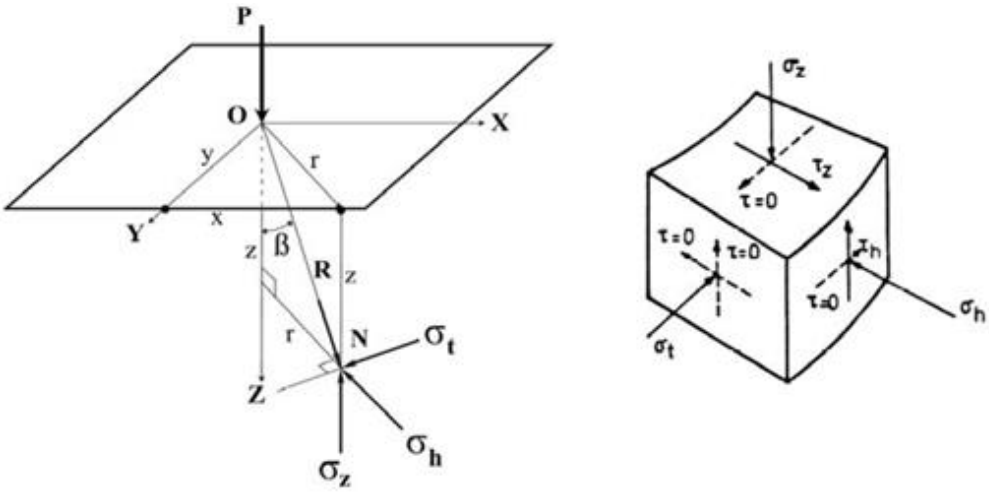


Figure 10.4 Load and stress in a cylindrical system (After Boussinesq (1885))

From the above load and stress in a cylindrical coordinate system; vertical stress, radial stress, tangential stress, and shear stress can be calculating using the formula as:

$$\sigma_z = \frac{P}{2\pi}\frac{3z^3}{(r^2+z^2)^{5/2}} \tag{10.14}$$

$$\sigma_r = \frac{P}{2\pi}\left[\frac{3r^3z}{(r^2+z^2)^{5/2}} - \frac{1-2\mu}{r^2+z^2+z\sqrt{r^2+z^2}}\right] \tag{10.15}$$

$$\sigma_t = \frac{P}{2\pi^{(1-2\mu)}}\left[\frac{z}{(r^2+z^2)^{5/2}} - \frac{1}{r^2+z^2+z\sqrt{r^2+z^2}}\right] \tag{10.16}$$

$$\sigma_z = \frac{P}{2\pi}\frac{3rz^3}{(r^2+z^2)^{5/2}} \tag{10.17}$$

Frohlich (1934) introduced the concentration factor, ν, into the Boussinesq equation for vertical stress component (Equation 10.9) to account for the non-elastic behavior of soil (Trauner 2003) so that theory of elasticity can be predicted the stress distribution in a satisfactory manner. Thus the concentration factor, ν, which was introduced into the Boussinesq equation for vertical stress component (Eq. 10.9), can be written as:

$$\sigma_z = \frac{\nu P}{2\pi r^2}cos^{\nu}\beta \tag{10.18}$$

where, $\nu = 3$ describe the distribution in a perfect elastic isotropic mass according to Boussinesq. The concentration factor does not represent a soil physical property but is related to the soil type and soil moisture content (Trauner 2003), and contact area of the applied load and contact stress (Horn & Lebert 1994; Koolen 1994). Figure 10.5 shows one-layer system component of stresses under asymmetric loading.

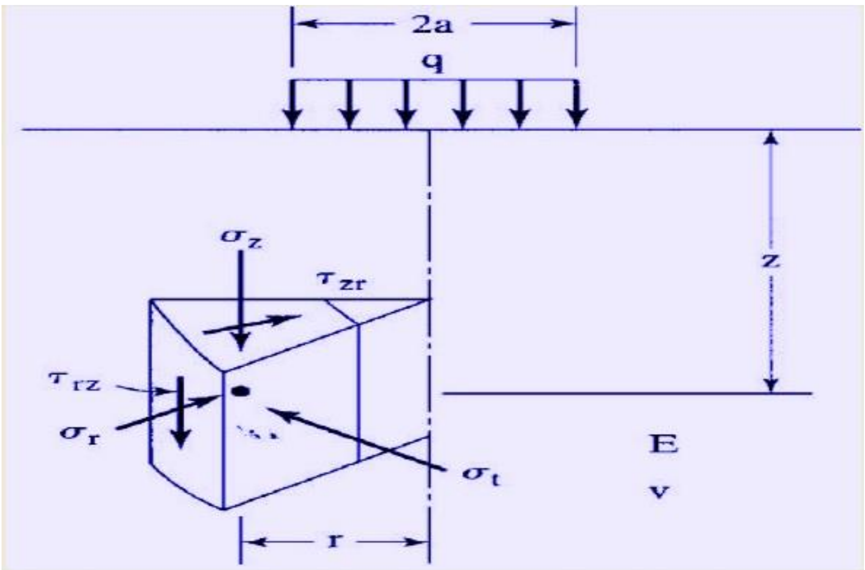


Figure 10.5 One-layer system component of stresses under asymmetric loading

10.6.2. TWO-LAYER ELASTIC MODELS

The exact case of a two-layer system is full depth asphalt pavement construction in which a thick layer of HMA is placed directly on the subgrade. If a pavement is composed of three-layer which means; a compose of an asphalt course, a granular base course, and subgrade, it is necessary to combine the base course and the subgrade into a single layer for computing the stresses and strains in the asphalt layer or to combine the asphalt surface course and base course for computing the stresses and strains in the subgrade (Huang 1993).

Pavement systems have a layered structure with better material on top and cannot be represented by a homogenous mass as assumed in Boussinesq's theory (Boussinesq 1885). As the results of this, a better theory is needed to analysis the behavior of pavements, and Burmister layer theory is more appropriate. Burmister (1943) was first developed a solutions for two layer system and then extended them to a three-layer system (Burmister 1945). With the advance of computers, the theory can be applied to multilayer system with a number of layers (Huang 1967; 1968a; 1993). Figure 10.6 shows the traffic wheel load distribution along various layers. The stress, strain, displacement and the boundary and continuity condition for Burmister two-layer system in a flexible pavement are shown in Figure 10.7 and 10.8, respectively. The basic assumptions for all Burmister's models to be satisfied are:

1. The pavement system consists of several layers; and each layer is homogenous, isotropic, and linearly elastic with an elastic modulus, E and poisson's ratio, ν.
2. Each layer has a uniform thickness, h and infinite dimensions in all horizontal direction, and resting on a semi-infinite elastic half-space.
3. The pavement system should be free of stresses and deformations before application of external loads.
4. All the layers and the dynamic effects are assumed to be weightless and negligible.
5. A uniform pressure, q is applied on the surface over a circular area of radius, a.
6. Continuity conditions are satisfied at the layer interfaces as indicated by the same vertical stress, shear stress, vertical displacement, and radial

displacement. For frictionless interface, the continuity of shear stress and radial displacement is replaced by zero shear stress at each side of the interface.

Vertical Stress Distribution

The vertical stress on the top of subgrade is an important factor in pavement analysis and design. The main function of a pavement is to reduce the vertical stress on the subgrade so that undesirable pavement deformation will not occur. Huang (1968a; 1993) pointed out that the allowable vertical stress on a given subgrade depends on the strength or modulus of the subgrade. As the matter of the fact, the combine effect of stress and strength, and the vertical compressive strain has been used most frequently as a pavement design criterion. This design criterion is only valid or implies for highway and airfield (airport) pavements because the vertical strain is caused by primarily by the vertical stress, while the effect of horizontal stress in the flexible pavement is relative small or insignificant (Huang 1993). However, the design of railroad trackbeds is not the same as highway and airfield pavement. It is based on vertical stress instead of vertical strain, because the majority of horizontal stress caused by the distribution of wheel loads through rails and ties over a large area and it makes the vertical strain as a poor indicator of the vertical stress (AASHTO2008; Huang 1967; 1968a; 1993; Ullidtz 1987).

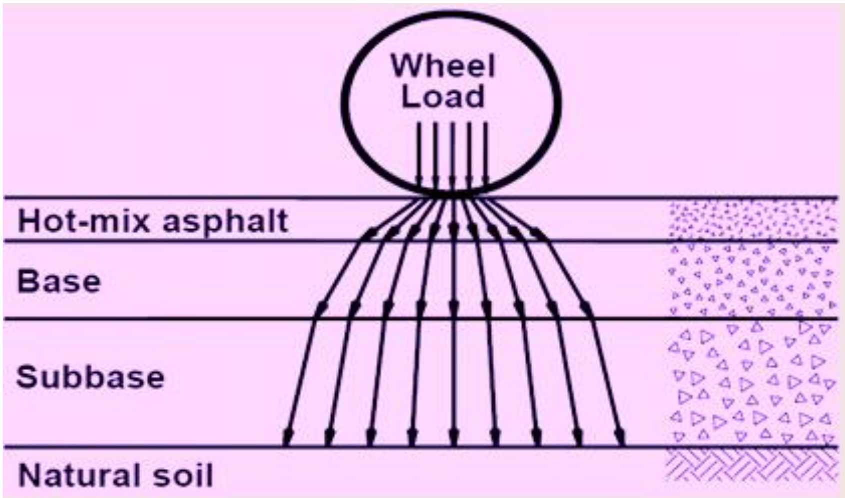


Figure 10.6 Distribution of traffic wheel load along various layers

The Burmister's layer theory on stresses in a two-layer system depend on the elastic modulus ratio $(E1/E2)$ and thickness of the pavement layer to radius of contact area

(h_1/a) (Burmister 1958). The effect of a pavement layer on the distribution of vertical stresses under the center of a circular loaded area is shown in Figure 10.7. The vertical stress distribution curve or chart on the pavement layer is applicable to the case when the thickness, h_1 of the layer 1 is equivalent to the radius of contact area, $(h_1/a = 1)$. A Poisson ratio, ν of 0.5 is applicable for all these layers as assumed by Burmister (1943; 1958) It can be seen that the vertical stresses decrease with the increase in modulus ratio. At the pavement-subgrade interface, vertical stress is about 68% of the applied pressure if $(E2/E1) = 1$ as indicated by Boussinesq' stress distribution (Boussinesq 1885), and reduced s to about 8% of the applied pressure if $(E1/E2) = 100$ (Huang 1993).

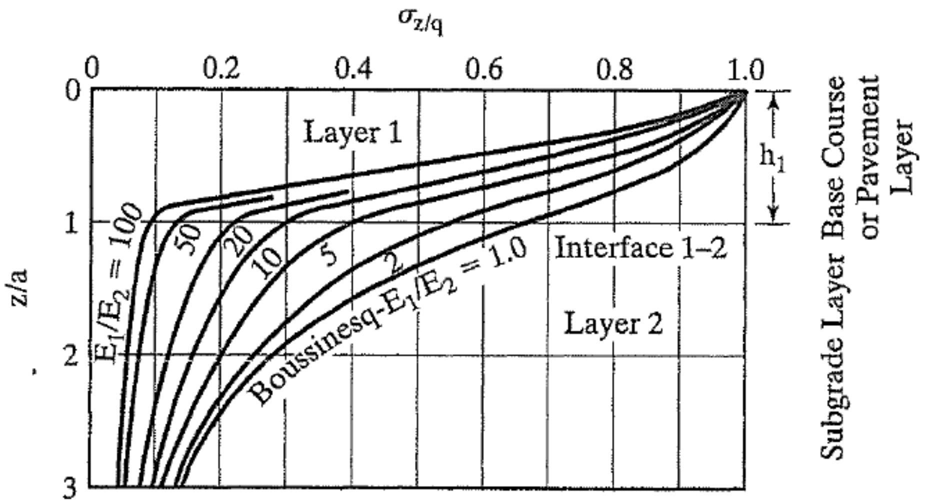


Figure 10.7 Vertical stress distribution in a two-layer system (After Burmister (1958))

Vertical Surface Deflection

Pavement design consists of two broad categories: (!) design of the paving mixture, and (2) structural design of the pavement structural component. These two design steps must go hand to hand. The structural design of pavement is basically different from the structural design of bridge and building in that the pavement structure lies exposed upto the ground surface (Yoder & Witczak 1975)., hence, is greatly influence by environmental factor. Likewise, a highway, for example, will cross many different soil deposits and become necessary for the design engineer to select in a rational manner a design values representative of the area under question.

Vertical surface deflections have been used as a criterion of pavement design, and can be used to determine the surface deflection for two-layer system in flexible pavement (Figure 10.8). Burmister derived the stress and displacement equation for two-layer pavement system from the mathematical equation theory of elasticity for the three-dimensional problem (Burmister 1943) that was solved by Love (1927) and Timoshenko (1923). Burmister assumed the poisson's ratio, ν to be 0.5, and he realized that the stress and deflection were dependent on the ratio of the moduli of subgrade to the pavement $(E1/E2) = 1$ and the thickness of the pavement layer to radius (bearing area) (h_1/a). For pavement analysis and design application purpose, the surface deflection is expressed in terms of the deflection factor, F_w as:

$$w_0 = \frac{1.5qa}{E_2} F_2 \quad \text{.... For flexible load bearing} \tag{10.19}$$

$$w_0 = \frac{1.18qa}{E_2} F_2 \quad \text{.... For rigid load bearing} \tag{10.20}$$

where, w_0 is the surface deflection at the center of a circular uniform loading, q is the uniform pressure on surface of circular area, a is the radius of circular area, E_2 is the elastic modulus of subgrade layer, and F_2 is the deflection factor. The deflection factor is a function of $(E1/E2)$ and (h_1/a).

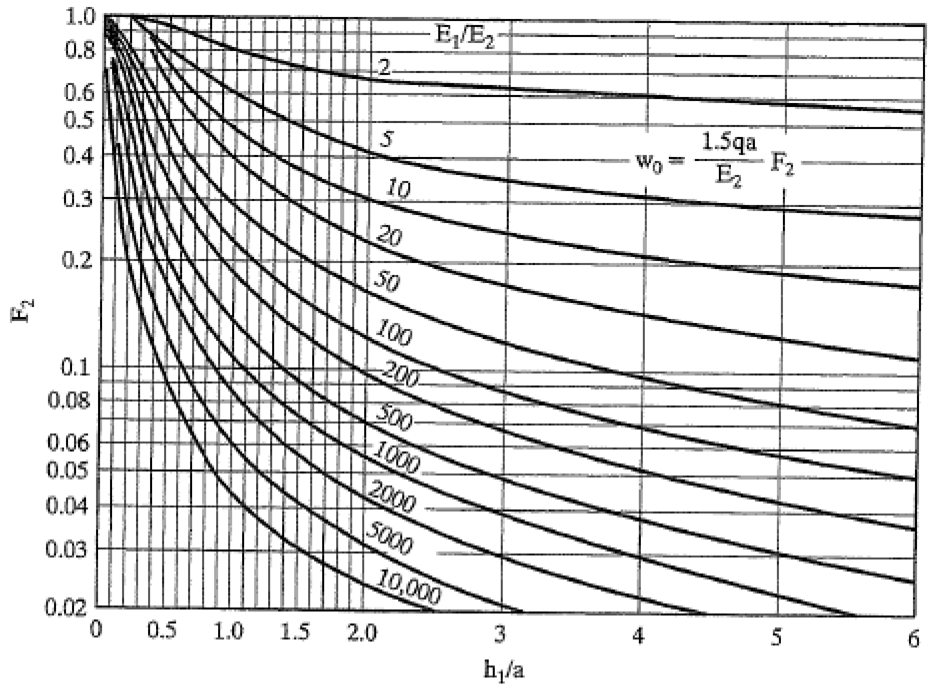


Figure 10.8 Vertical surface deflections for two-layer systems (After Burmister (1943))

The effect of pavement thickness and elastic modulus ratio $(E1/E2)$ on the vertical stress, σ_c at the pavement-subgrade interface under the center of a circular loaded area is shown in Figure 10.9. For a given uniform applied pressure, the vertical stress increases with the increase in contact radius and decrease with the increase in thickness, h_1. The reason that the ratio a/h_1 instead of (h_1/a) was used for the purpose of preparing influence charts (Huang 1969b) for two-layer elastic foundations (Huang 1993).

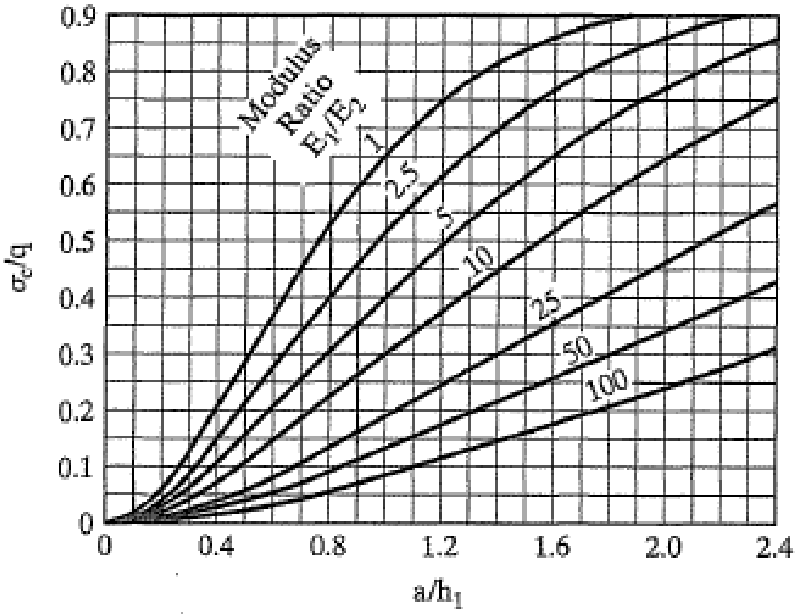


Figure 10.9 Vertical interface stress for two-layer systems (After Huang (1969b; 1993))

10.6.3. MULTI-LAYER ELASTIC MODEL

Burmister (1945) derived analytical expressions for the stresses and displacement to a three-layer on actual pavement system. Similarly, Acum and Fox (1951) also presented a normal and radial stresses in three-layer system at the interfaces. Later, Acum and Fox's study was extended by Jones (1962) to include a wider range of similar parameters. And then, Peattie (1962) developed Jones's study to high level of pavement analysis and design for pavement engineers before modern computer was widely available. Schiffman (1962) developed a general solution to analysis of stresses and displacement in a N-layer elastic system (Figure 10.10). Schiffman's analytical theory provides a solution for the problem in determination of stresses and displacements of a multi-layer pavement systems to a non-uniform normal surface loads, tangential surface loads, rigid and semi-rigid and plate bearing load (Elshaer 2009). An asymmetric cylindrical coordinate system for a multi-layer elastic system is shown in Figure 10.11

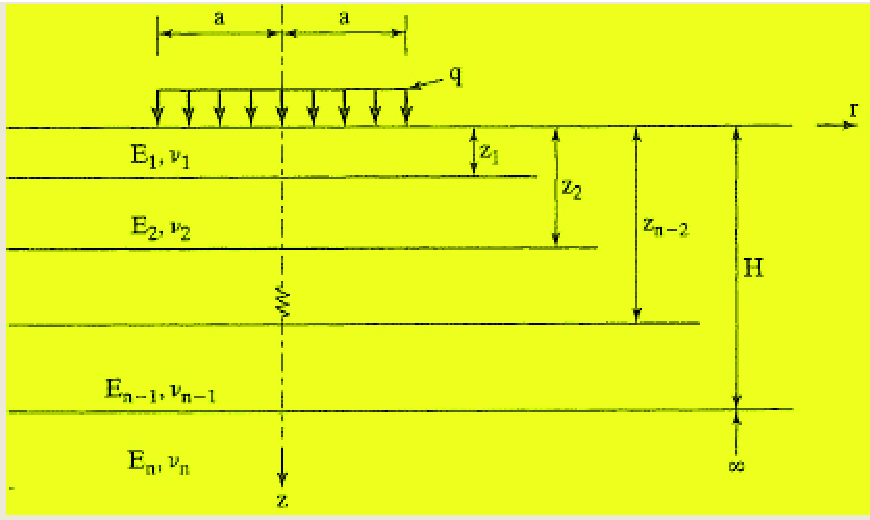


Figure 10.10 An n-layer elastic system in cylindrical coordinates

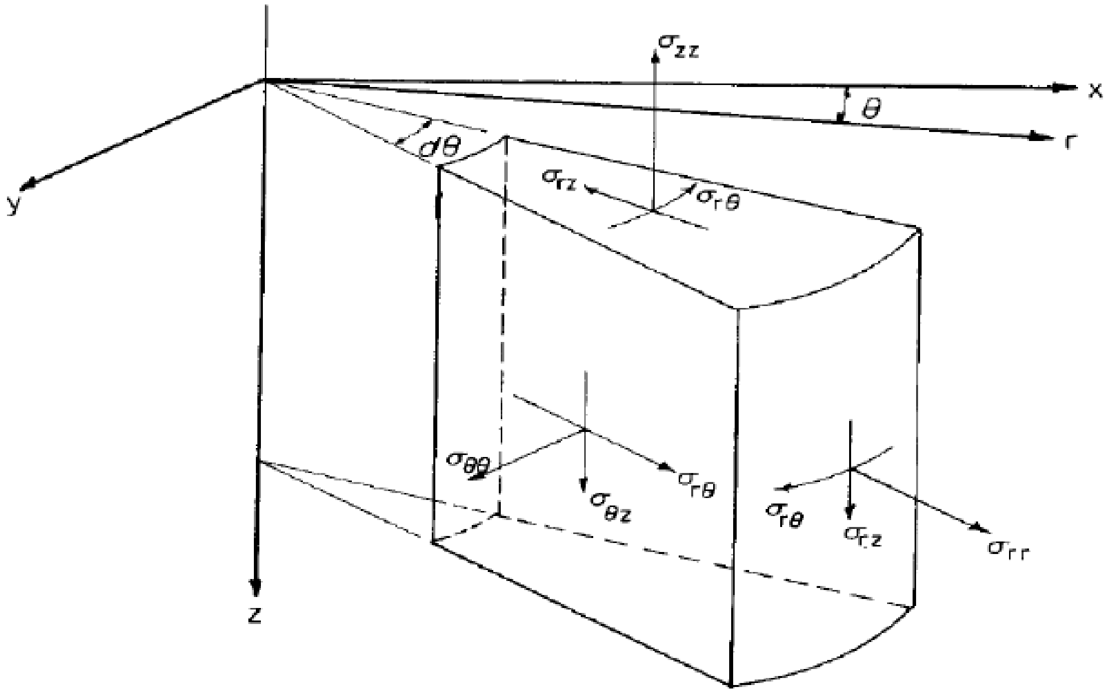


Figure 10.11 Superposition of Stress for Multi-Layer System (After Schiffman (1962))

10.7. LOADING AND DAMAGE OF FLEXIBLE PAVEMENT

A flexible pavement is a multilayer structure composed of asphalt surface layer and combined unbound aggregate roadbase on a subgrade of natural soil. Under service conditions the stress state or stress distribution is idealized as depicted as shown in Figure 3. Moving wheel loads cause the top and bottom of the pavement to shift rapidly from compression to tension, and fatigue cracks arise from the repeated tensile strains. According to elastic layer theory, the maximum strain is at the bottom of the asphalt surface layer (Ullidtz 1987). Most pavement analysis and design models are therefore based on straining at the bottom of the asphalt layer to predict pavement performance with respect to fatigue cracking (Al-Qadi, Elseifi & Leonard 2003; Nega, Nikraz & Al-Qadi 2015a; Nega et al. 2015). Cumulative vertical strain on the pavement layer and the subgrade produce the deformation that results in rutting (Huang et al. 2011).

The stress and strain characteristic response of flexible pavement layer due to traffic urban loading (i.e. vehicular loading) has been extensively studied (Al-Qadi et al. 2009; Hartman, Gilchrist & Walsh 2001; Mulungye, Owende & Mellon 2007; Zaghloul & White 1993; Zhou & Scullion 2002), and the results depict a delayed lateral strain relaxation which varies with lateral position of wheel of load on a pavement.

Pavement failure is determined by criteria based on longitudinal rutting or fatigue cracking in the wheel tracks. However, large elastic displacements on thin pavements with weak foundations cause fatigue failure or cracking that undermine the substructure before appreciable rutting has occurred, hence, fatigue cracking is the limiting criterion (Nega et al. 2015; Nega et al. 2013a). Structural performance of a flexible pavement is therefore primarily affected by factors that influence the critical tensile strain at the bottom of the surfacing layer. For any give pavement attributes, the axle load, axle configuration, suspension type and tyre inflation pressure will all affect the magnitude and distribution of stresses, strains, and deflection in its structure (Al-Qadi, Wang & Tutumluer 2010; Huang 1993; Huang 2004; Ullidtz 1987).

Pavement fracture defects occur because of traffic loading, environmental effect such as influence of temperature and moisture damage, characterization of pavement materials, and construction inadequacies. Cracking that only occur in the wheel path is considered due to the influence of traffic loading, while cracking observed outside the wheel path, across the full width of a flexible pavement is considered due to environmental factors (Helwany, Dyer & Leidy 1998; Huang 1993; Huang 2004). The occurrence of mechanical fatigue crack is related to traffic action. For example, fatigue of the asphalt layer is imitate on the top surface of flexible pavement and grown downwards (Loulizi, Al-Qadi & Elseifi 2006; Rahman, Mahmud & Ahsam 2011). The main cause of the cracks is the high contact stresses which are induced on the asphalt surface due to the tyre-surface interaction (Huang 2004; Ullidtz 1998; Ullidtz 2002).

Currently, increasing traffic volume, weight and tyre pressure on the urban traffic roads highways have been become a problem and researchers have highlighted the shortcomings associated with empirical methods for evaluating and predicting the in-service behavior of asphalt mixtures (Al-Qadi et al. 2009; Mulungye, Owende & Mellon 2007; Nega, Nikraz & Al-Qadi 2015b; Owende et al. 2001). In mechanistic methods used in the analysis of layered pavement system under traffic load, the pavement layers are considered as homogenous, linear elastic, and the traffic loading is considered at static. In this case, the mechanistic methods are very accurate and reliable with any things if the pavement subgrade system behaves as a linear elastic system (Nega, Nikraz & Al-Qadi 2015b; Owende et al. 2001).

The use of multi-layer elastic theory together with static loading is a rational approach as compared with the old empirical pavement analysis and design method (Huang 1993; Owende et al. 2001). However, in real situation, heterogeneous pavement layers behave far these ideal conditions and are subjected to dynamic and cyclic loading. For such kind of loading behavior, researchers diverted their attention to the finite element (FE) method, which provides a better understanding and solution in the dynamic analysis of flexible pavements, while considering the heterogeneity and nonlinearity at the same time (Mulungye, Owende & Mellon 2007; Owende et al. 2001). With the availability of high speed mulita-layer computers, finite element method using ABAQUS software can handle a very complex geometry, boundary conditions and material properties which are related to model pavements without any problem (Hadi & Bodhinayake 2003; Helwany, Dyer & Leidy 1998)

There is a lack of stress, strain, and deflection analysis under moving loads. In the existing flexible pavement analysis and design method of asphalt pavement concrete, equivalency simplified vehicle load is commonly known as a static uniform load. Static uniform load is most commonly used on pavement structure for mechanical analysis and calculating, and that is basically reasonable on the condition of low speed and small load. As the matter of the fact, the moving vehicle on road produces a complex vertical and horizontal force to flexible pavement. Actually there is a great difference between static loads and moving loads mode on asphalt concrete under the fast-moving vehicle loads, the response of asphalt pavement structure cannot described by static mechanical characteristics.

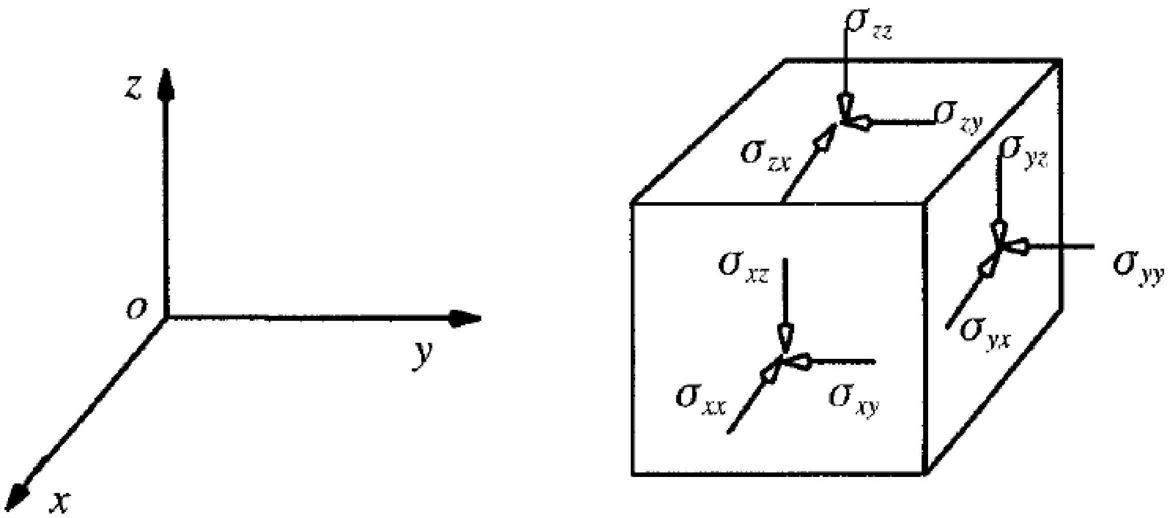


Figure 10.12 Stress state for three-dimensional element

10.8. DESCRIPTION OF CONSTITUTIVE MODEL FOR NONLINEAR STRESS AND STRAIN BEHAVIOR

In three-dimensions, the elastic normal stress-strain relation takes the following simple form (from Figure 10.12) as:

$$\varepsilon_{xx} = \frac{1}{E}\left[\sigma_{xx} - \nu(\sigma_{yy} + \sigma_{zz})\right] \qquad (10.23)$$

$$\varepsilon_{yy} = \frac{1}{E}\left[\sigma_{yy} - \nu(\sigma_{xx} + \sigma_{zz})\right] \qquad (10.24)$$

$$\varepsilon_{zz} = \frac{1}{E}\left[\sigma_{zz} - \nu(\sigma_{xx} + \sigma_{yy})\right] \qquad (10.25)$$

The shear stresses are related to shear strain by the following relations:

$$\varepsilon_{xy} = \frac{1+\nu}{E}\sigma_{xy} = \frac{\sigma_{xy}}{2G} \qquad (10.26)$$

$$\varepsilon_{yz} = \frac{1+\nu}{E}\sigma_{yz} = \frac{\sigma_{xz}}{2G} \qquad (10.27)$$

$$\varepsilon_{xz} = \frac{1+\nu}{E}\sigma_{xz} = \frac{\sigma_{xz}}{2G} \qquad (10.28)$$

where, E and ν are material constants known as the Young's modulus and Poisson's ratio respectively, and G is shear modulus of the material. The state of stress at a point is defined by a stress tensor with nine stress components. However, only six of them, three normal stresses (σ_{xx}, σ_{yy}, and σ_{xz}) and three shear stresses (σ_{xy}, σ_{yz}, and σ_{xz}), are independent due to moment equilibrium. These stresses need to satisfy three equation of equilibrium (Fung 1965; Malvern 1969; Prager 1961; Spencer 1980):

$$f(\sigma_{ij}) = f(I_1, I_2, I_3) = 0 \tag{10.29}$$

where, σ_{ij} is the stress tensor, and the subscripts i and j take integral values of 1, 2, and 3; I_1, I_2, and I_3 are the first, second, and third stress invariant, respectively; and f is a specific function for a certain asphalt material and must be determined experimentally in a laboratory. In this study, the stress invariant can be defined by Hooke's law (Hooke 1978):

$$I_1 = \sigma_{xx} + \sigma_{yy} + \sigma_{zz} \tag{10.30}$$

$$I_2 = \sigma_{xx}\sigma_{yy} + \sigma_{yy}\sigma_{yy} + \sigma_{zz} - \sigma_{xy}^2 - \sigma_{yz}^2 - \sigma_{xz}^2 \tag{10.31}$$

$$I_3 = \sigma_{xx}\sigma_{yy}\sigma_{zz} - \sigma_{xx}\sigma_{yz}^2 - \sigma_{yy}\sigma_{xy}^2 - \sigma_{zz}\sigma_{xy}^2 + 2\sigma_{xy}\sigma_{yz}\sigma_{xz} \tag{10.32}$$

Once stress rate is on the yield surface, the most widely theory is to assume that the plastic strata rate (or increment) can be determined by following formula (Hill 1950; Melan 1938; von Mises 1928):

$$d\varepsilon_{ij}^p = d\lambda \frac{\partial g}{\partial \sigma_{ij}} \tag{10.33}$$

where, $d\lambda$ is a positive scalar, and

$$g = g(\sigma_{ij}) = g(I_1, I_2, I_3) = 0 \tag{10.34}$$

where, g is a plastic potential, which is assumed to be as the yield surface. Equation (10.32) is referred to as a plastic flow rule that basically defines ratios of components of plastic strain rate. This plastic flow rule was based on the observation by de Saint-Venont (1870) that for metals the principal axes of the plastic strain rate coincide with those of the stress (Yu 2006). This is assumed as a coaxial, which has been the foundation of almost all the plastic models used in engineering. The plastic potential approaches in finite number of linearly independent limiting values as the stress point approaches the single point was in question. Thus Koiter (1953) proposed a generalized flow rate as follows:

$$d\varepsilon_{ij}^{P} = \sum_{i=1}^{n} d\lambda_i \frac{\partial g_1}{\partial \sigma_{ij}} \tag{10.35}$$

where, $d\lambda_i$ is non negative and $\partial g_1 / \partial \sigma_{ij}$ is the linearly independent gradients. Suppose plastic strain rate $d\varepsilon_{ij}^{p}$ is given and the corresponding stress state σ_{ij} determined from the normality rule and the yield criterion, is represented by a point P in the stress space and then, if σ_{ij} is an arbitrary strate of stress represented by a point P* inside the yield surface, then the different between the incremental plastic works done and by the two stress on the actual plastic strain rate is:

$$dW_P = \left(\sigma_{ij} - \sigma_{ij}^{*}\right) d\varepsilon_{ij}^{P} \tag{10.36}$$

Assumed the equation (10.36) represents the scale product two vectors and then, the yield surface is strictly convex, the angle between these vectors is acute and the scalar product is positive. Then, equation (14) can be written as

$$\left(\sigma_{ij} - \sigma_{ij}^{*}\right) d\varepsilon_{ij}^{P} \geq 0 \tag{10.37}$$

This condition, due to von Mises (1928) and Hill (1948; 1950), is called as maximum plastic theory. The most common measures include the total plastic work per unit volume, the accumulated plastic strain (Hill 1950), the volumetric plastic strain rate (Schofield & Wroth 1968; Yu 1998), or a combination of volumetric and shear plastic strain rates (Yu & Yuan 2005). The yield surface for a strain-hardening or softening asphalt material is also called the loading surface. By integrating equation (10.37), mathematically, plastic deformation can be expressed as follows:

$$f\left(\sigma_{ij} \varepsilon_{ij}^{P}\right) = 0 \tag{10.38}$$

where, ε_{ij}^{P} is the plastic strain tensor. If the yield surface does not change with history, and then, the material is under a perfect plastic. This is a special case of strain-hardening material. For a perfect plastic material, the behavior of is elastic when the stress rate lies inside the yield surface. Plastic strain will occur as long as the stress states lies on travel along the yield surface. The complete stress condition for plastic and elastic can be stated as

$$Elastic: f\left(\sigma_{ij}\right) < 0 \;\; or \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} < 0 \tag{10.39}$$

$$Plastic: f\left(\sigma_{ij}\right) = 0 \;\; and \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} = 0 \tag{10.40}$$

By substituting equation (10.38) to equation (10.39) and (10.40), the complete stress condition for plastic and elastic for a strain-harding material can be written as:

$$Elastic: f\left(\sigma_{ij}, \varepsilon_{ij}^{P}\right) < 0 \;\; or \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} \leq 0 \tag{10.41}$$

$$Plastic: f\left(\sigma_{ij}, \varepsilon_{ij}^{P}\right) = 0 \;\; and \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} \leq 0 \tag{10.42}$$

In this study, df is evaluated only with respect to the increments in the stress component as by Kachanov (1974). For solving the boundary values problems on stress and strain characteristics behavior in flexible pavement involving elastic-plastic behavior, it is very important to clearly determine what behavior will result from a further stress increment when stress is already on the yield surface. The assumed three possible conditions that can occur or exist are:

$$Unloading: f\left(\sigma_{ij}, \varepsilon_{ij}^{P}\right) = 0 \;\; and \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} < 0 \tag{10.43}$$

$$Neutral\ loading: f\left(\sigma_{ij}, \varepsilon_{ij}^{P}\right) = 0 \;\; and \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} = 0 \tag{10.44}$$

$$Loading: f\left(\sigma_{ij}, \varepsilon_{ij}^{P}\right) = 0 \;\; and \; df = \frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} > 0 \tag{10.45}$$

In this study, a general expression for plastic strains can be assumed based on loading conditions from equation (10.43) - (10.45) as Hill (1950) showed a general expression for plastic stratus as

$$d\varepsilon_{ij}^{P} = G_{ij} df \tag{10.46}$$

where, G_{ij} is a symmetric tensor, which is supposed to be a function of the stress component. This last assumption is very significant because the ratios of component of the plastic strain rate are function of the current stress but not of the stress rate. These can be satisfied by assumption G_{ij} as the following form

$$G_{ij} = h\frac{\partial g}{\partial \sigma_{ij}} \tag{10.47}$$

where, h and g are scale function of the stress tensor. As the same time, g is also known as a plastic potential. By substituting equation (10.47) into equation (10.46), the plastic strain rate can be determined as follows:

$$d\varepsilon_{ij}^{P} = h\frac{\partial g}{\partial \sigma_{ij}}df \tag{10.48}$$

This was first used by Melan (1938). Drucker (1952; 1959) introduced a fundamental stability postulates. Although Drucker's stability postulate is a generalized of simple facts and are valid for certain classes of asphalt pavement materials properties and is not a statement of any thermodynamics materials principles (Yu 2006), as it is often presented (Green and Naghdi 1965). However, in this study, it has taken into consideration because the Drucker's stability postulate can show to lead into two important inequalities (Drucker 1952; 1960):

$$\left(\sigma_{ij} - \sigma_{ij}^{*}\right)d\varepsilon_{ij}^{P} \geq 0 \qquad \textit{the same as Equation } (10.36) \tag{10.49}$$

$$d\sigma_{ij}d\varepsilon_{ij}^{P} \geq 0 \tag{10.50}$$

The first simple kinematic hardening model was proposed by Prager (1955). This classical model of decision making assumes that the yield surface keeps its original shape and size, and moves into the direction of plastic strain rate tensor. Mathematically, this can be described as linear rule:

$$da_{ij} = cd\varepsilon_{ij}^{P} \tag{10.51}$$

where, c is a constant material. Whilst Prager's model is reasonable for one-dimensional problems, it does not seem to give consistence predictions for two-and three-dimensional case (Ziegler 1959). The reason is that the yield function takes different forms for one-, two-, and three- dimensional case. To overcome this kind of limitation, Ziegler criterion is considered. Ziegler (1959) suggested that the yield surface should move in the direction as determined by the vector $\sigma_{ij} - a_{ij}$. Mathematically, Ziegler's model can be expresses as follows:

$$da_{ij} = d\mu\left(\sigma_{ij} - a_{ij}\right) \quad (10.52)$$

where, $d\mu$ is a material constant. In order to determine the complete relation between stress and strain for elastic-plastic flexible pavement materials, in this study, need to assume the *consistency* condition by Prager (1949). For perfectly plastic materials, consistency condition means that the stress state remains on yield surface. For strain-hardening materials, consistency means that during plastic flow the stress state must remain on the subsequent yield surface or loading surface (Yu 2006). For isotropic harden material, the yield function can described as:

$$f\left(\sigma_{ij}, a\right) = 0 \quad (10.53)$$

Then in this study, Prager's consistency condition requires. Using partial differential with respect to σ_{ij} and a of equation (10.52), we get

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} + \frac{\partial f}{\partial a} da = 0 \quad (10.54)$$

Since the hardening parameter is a function of plastic strain, so the consistency condition, equation (10.54) can be further written as follows:

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} + \frac{\partial f}{\partial a}\frac{\partial a}{\partial \varepsilon_{ij}^{P}} d\varepsilon_{ij}^{P} = 0 \quad (10.55)$$

For perfectly plastic material, the second term of equation (10.54) should be zero. These for a complete stress-strain relation for plastic and hardening materials, the total strain rate into elastic and plastic strain can be expresses as

$$d\varepsilon_{ij} = d\varepsilon_{ij}^{e} + d\varepsilon_{ij}^{P} \tag{10.56}$$

Hooke's law (Hooke 1678) used to link the stress rate with elastic strain rate by elastic stiffness matrix, D_{ijkl} as follows:

$$d\sigma_{ij} = D_{ijkl} d\varepsilon_{kl}^{e} = D_{ijkl}(d\varepsilon_{kl} - d\varepsilon_{kl}^{P}) \tag{10.57}$$

To accumulate non-associated to plastic flow, equation (10.57) can be written further as

$$d\sigma_{ij} = D_{ijkl}\left(d\varepsilon_{kl} - d\lambda \frac{\partial g}{\partial \sigma_{kl}}\right) \tag{10.58}$$

By substituting equation (10.58) into equation (10.55), which is the consistency condition, in this study obtain as:

$$d\lambda = \frac{1}{H}\frac{\partial f}{\partial \sigma_{ij}} D_{ijkl} de_{kl} \tag{10.59}$$

where, H is given by

$$H = \frac{\partial f}{\partial \sigma_{ij}} D_{ijkl} \frac{\partial g}{\partial \sigma_{kl}} - \frac{\partial f}{\partial a}\frac{\partial a}{\partial \varepsilon_{ij}^{P}}\frac{\partial g}{\partial \sigma_{ij}} \tag{10.60}$$

By substituting equation (10.59) into equation (10.58), the new constitutive model for stress and strain characteristic behavior in flexible can be written as:

$$d\sigma_{ij} = D_{ijkl}^{ep} d\varepsilon_{kl} \tag{10.61}$$

The elastic-plastic stiffness, D_{ijkl}^{ep} is defined as

$$D_{ij}^{ep} = D_{ijkl} - \frac{1}{H} D_{ijmn} \frac{\partial g}{\partial \sigma_{mn}} \frac{\partial f}{\partial \sigma_{pg}} D_{pgkl} \tag{10.62}$$

The above constative is valid for both strain-hardening and perfect plastic material. For perfect plastic material, yield surface is remaining unchanged so that equation can be written in a simplify way as

$$H = \frac{\partial f}{\partial \sigma_{ij}} D_{ijkl} \frac{\partial g}{\partial \sigma_{kl}} \tag{10.63}$$

For kinematic hardening material, the yield function can be expressed as:

$$f = f\left(\sigma_{ij} - a_{ij}\right) - R_0 = 0 \tag{10.64}$$

where, a_{ij} is the coordinate of the center of yield surface and it can also consider as back stress tensor from the center of the loading at that yield surface. From the kinematic hardening law and the thermodynamic theory point of view, in this study consider the kinematic hardening law which was proposed by Prager into equation (10.64) and can be written as:

$$da_{ij} = cd\varepsilon_{ij}^{P} = cd\lambda \frac{\partial g}{\partial \sigma_{ij}} \tag{10.65}$$

where, g is a plastic potential. Assumed Prager's consistency condition applied to the yield function into equation (10.64), and applied a partial derivation to it, and then, equation (10.64) becomes:

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} + \frac{\partial f}{\partial a_{ij}} da_{ij} = 0 \tag{10.66}$$

By applying one more derivation to equation (10.66) in terms of σ_{ij} and a_{ij}, the equation (10.65) can be written as

$$\frac{\partial f}{\partial \sigma_{ij}} = -\frac{\partial f}{\partial a_{ij}} \tag{10.67}$$

This is the final assumed form of the yield function of equation (10.64).

Substitute equation (10.65) into equation (10.66) and then, equation (10.66) can be rewritten as:

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} + \frac{\partial f}{\partial a_{ij}} cd\lambda \frac{\partial g}{\partial \sigma_{ij}} = 0 \tag{10.68}$$

Equation (10.68) can further be arranged for simplicity as

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} = -\frac{\partial f}{\partial a_{ij}} cd\lambda \frac{\partial g}{\partial \sigma_{ij}} \tag{10.69}$$

Substitute equation (10.67) into equation (10.69) and then, equation (10.69) can be rewritten as:

$$\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} = \frac{\partial f}{\partial \sigma_{ij}} cd\lambda \frac{\partial g}{\partial \sigma_{ij}} \tag{10.70}$$

The plastic multiplier, $d\lambda$ can be calculated by the equation as follows

$$d\lambda = \frac{1}{c}\left[\frac{\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij}}{\frac{\partial f}{\partial \sigma_{ij}}\frac{\partial g}{\partial \sigma_{ij}}}\right] = \frac{1}{c}\frac{\partial f}{\frac{\partial f}{\partial \sigma_{ij}}\frac{\partial g}{\partial \sigma_{ij}}} \tag{10.71}$$

By using an elastic stress-strain characteristics relation in flexible pavement, the elastic strain rate can be determine as

$$d\varepsilon_{ij}^{e} = C_{ijkl} d\sigma_{kl} \tag{10.72}$$

where, C_{ijkl} is the elastic compliance matric. Using the equation (10.71) and (10.72), the total strain rate (which is the sum of elastic and plastic parts) can be written as:

$$d\varepsilon_{ij} = C_{ijkl} d\sigma_{kl} + \frac{1}{c}\frac{\frac{\partial g}{\partial \sigma_{ij}}}{\frac{\partial f}{\partial \sigma_{ij}}\frac{\partial g}{\partial \sigma_{ij}}} df \tag{10.73}$$

For clarity and simplicity, equation (10.73) can further written as

$$d\varepsilon_{ij} = C_{ijkl}^{ep} d\sigma_{kl} \tag{10.74}$$

By substituting equation (10.73) into equation (10.74), equation (10.74) can be written as

$$C_{ijkl}^{ep} d\sigma_{kl} = C_{ijkl} d\sigma_{kl} + \frac{1}{c}\frac{\frac{\partial g}{\partial \sigma_{ij}}}{\frac{\partial f}{\partial \sigma_{ij}}\frac{\partial g}{\partial \sigma_{ij}}} df \tag{10.75}$$

The elastic-plastic compliance matrix C_{ijkl}^{ep} can be calculated by rearranging and simplifying equation (10.75) as follows:

$$C_{ijkl}^{ep} = C_{ijkl} + \frac{1}{c}\frac{\frac{\partial g}{\partial \sigma_{ij}}\frac{\partial f}{\partial \sigma_{kl}}}{\frac{\partial f}{\partial \sigma_{ij}}\frac{\partial g}{\partial \sigma_{ij}}} \tag{10.76}$$

It is clearly understood that equation (10.76) can be inverted so that it can give elastic-plastic stiffness matrix as

$$d\sigma_{ij} = \left[C_{ijkl}^{ep}\right]^{-1} d\varepsilon_{kl} = D_{ijkl}^{ep} d\varepsilon_{kl} \tag{10.77}$$

where, C_{ijkl}^{ep} is the elastic-plastic compliance matrix, and D_{ijkl}^{ep} is the elastic-plastic stiffness matrix. For a linear and isotropic elastic material, the stress and strain relation as indicated in Figure 1 and Hooke's law (i.e., equation (10.23) to (10.28) can be expressed in a total form as follows

$$\varepsilon_{ij} = \frac{1+\nu}{E}\sigma_{ij} + \frac{\nu}{E}\sigma_{kk}\delta_{ij} \tag{10.78}$$

$$\sigma_{ij} = \frac{E}{1+\nu}\varepsilon_{ij} + \frac{\nu E}{(1-\nu)\,(1+2\nu)}\varepsilon_{kk}\delta_{ij} \tag{10.79}$$

When elastic material in a flexible pavement is applied in conjection with a plastic material in a model, it is important to have the elastic stress-strain behavioral relation written in a rate form (Yu 2006) as:

$$\dot{\varepsilon}_{ij} = \frac{1+\nu}{E}\dot{\sigma}_{ij} + \frac{\nu}{E}\dot{\sigma}_{kk}\delta_{ij} \tag{10.80}$$

$$\dot{\sigma}_{ij} = \frac{E}{1+\nu}\dot{\varepsilon}_{ij} + \frac{\nu E}{(1-\nu)\,(1+2\nu)}\dot{\sigma}_{kk}\delta_{ij} \tag{10.81}$$

In equation (10.78) to (10.81), the Hooke's elastic stress-strain relations are expressed in terms of Young's modulus, E and Poisson's ratio,ν. Alternatively, in this study, they can be expressed in terms of bulk modulus, K and shear modulus, G as follows:

$$\dot{\varepsilon}_{ij} = \frac{1}{9K}\dot{\sigma}_{ij}\delta_{ij} + \frac{1}{2G}\dot{S}_{ij} \tag{10.82}$$

$$\dot{\sigma}_{ij} = K\dot{\varepsilon}_{kk}\delta_{ij} + 2G\dot{e}_{ij} \tag{10.83}$$

where,

$K = \frac{E}{2\,(1-2\nu)}$, $G = \frac{E}{2\,(1+2\nu)}$, $S_{ij} = \sigma_{ij} - \frac{\sigma_{kk}}{3}\delta_{ij}$, and

$$e_{ij} = \varepsilon_{ij} - \frac{\sigma_{kk}}{3}\delta_{ij} \tag{10.84}$$

Yu (2006) recommended that soil or rock is rarely behaves as linear elastic matrix so a better approach would be to model them as a nonlinear elastic-plastic material. The Green hyperelastic and hypoelastic theory are also considered in this study so that the elastic-plastic stress and strain relation relationship can fully define and understand (for example, when there is a nonlinear, incremental and elastic stress-strain relation in asphalt pavement material properties). A hyperelastic or Green elastic material is a type of constitutive model for ideally elastic material for which the stress-strain relationship derives from a strain energy density function. The hyperelastic material is a special case of Cauchy elastic material (Ogden 1984). The simplest hyperelastic material model is the Saint Venont-Kirchhoff model, which is just an extension of linear elastic material model to the nonlinear regime. The model has the form:

$$S = \lambda \, tr\,(E)1 + 2\mu E \tag{10.85}$$

where, S is the second Piola-Kirchhoff stress, E is the Lagrangian Green strain, and λ and μ are the Lame' constants. The theory of hyperelasticity assumes that there is exists a *strain energy function,* $U_s(\varepsilon_{ij})$ and *complementary energy function,* $U_c(\sigma_{ij})$, such as

$$\sigma_{ij} = \frac{\partial U_s}{\partial \varepsilon_{ij}} \qquad \text{and} \qquad \varepsilon_{ij} = \frac{\partial U_s}{\partial \sigma_{ij}} \tag{10.86}$$

The above equation is yield a one-to-one relation between the stress and strain characteristic behavior in flexible pavement. The rate form of this stress-strain relation is assumed for this purpose by the following equation as

$$\dot{\sigma}_{ij} = \frac{\partial^2 U_s}{\partial \varepsilon_{ij}\, \partial \varepsilon_{kl}} \dot{\varepsilon}_{kl} = D_{ijkl} \dot{\varepsilon}_{kl} \tag{10.87}$$

$$\dot{e}_{ij} = \frac{\partial^2 U_c}{\partial \sigma_{ij}\, \partial \sigma_{kl}} \dot{\sigma}_{kl} = C_{ijkl} \dot{\sigma}_{kl} \tag{10.88}$$

where, C_{ijkl} and D_{ijkl} are compliance and stiffness matrix, respectively and the superposed dots is indicate time derivatives. A key feature of hyperelasticity is that no energy can be generated through load cycles and therefore thermodynamic laws are always satisfied. The main objection to the theory of hyperelastic is that it may require many material constants, which could difficult or expensive to determine in practice (Yu 2006). Another major limitation of the theory is that it cannot model load history dependency as equation (10.86) and (10.87) imply that tangent moduli of the material are identical for loading and unloading. Applications of this type of theory of soil modelling are given by Houlsby (1985) and Borja, Tamagnini and Amorosi (1997).

In order to solve the tangent moduli of material problem for equation (10.87) and (10.88), which was described in some literatures that the loading and unloading are identical and it has also nonlinear material behavior its nature (i.e., material characterization), In this study we assumed if de Saint Venant-Kirchhoff and Prager's consistency condition applied to it and then, hyperelastic nonlinear material behavior can be model as linear elastic material. Thus, equation (10.87) and (10.88) can be written as:

$$f\left(d\dot{\sigma}_{ij}\right) = \lambda \left[\frac{\partial^2 U_s}{\partial \varepsilon_{ij}\, \partial \varepsilon_{kl}}\right] \dot{\varepsilon}_{kl} + 2\mu \left[\frac{\partial^2 U_s}{\partial \varepsilon_{ij}\, \partial \varepsilon_{kl}}\right] \dot{\varepsilon}_{kl} = 0 \text{ and}$$

$$df = \frac{\partial f}{\partial \dot{\sigma}_{ij}} \partial \dot{\sigma}_{ij} > 0 \qquad (10.89)$$

$$f\left(d\dot{\varepsilon}_{ij}\right) = \lambda \left[\frac{\partial^2 U_c}{\partial \sigma_{ij}\, \sigma_{kl}}\right] \dot{\sigma}_{kl} + 2\mu \left[\frac{\partial^2 U_c}{\partial \sigma_{ij}\, \partial \sigma_{kl}}\right] \dot{\sigma}_{kl} = 0 \text{ and}$$

$$df = \frac{\partial f}{\partial \dot{\varepsilon}_{ij}} \partial \dot{\varepsilon}_{ij} > 0 \qquad (10.90)$$

where, λ and μ are the Lame' constants. The above equations are considered that it may not require many material constants and it was also assumed that it can model the loading history because the tangent moduli of the material for loading and unloading are different, and it is assumed that the material matrix could be a function for both stress and strain. It should be noted that the nonlinear material characterizes behavior has changed to a linear elastic material in this case. Thus the hyperelasticity is not guaranteed that thermodynamic laws are always satisfied at this step. Bear in mind that hyperelasticity is always used to satisfied the thermodynamic laws because the theory says that no energy can be generated through load cycle when the stress-strain relationship is based on derives from a strain energy density function.

Mathematically, Equation (10.89) and (10.90) can also furtherly rewritten (superimposed or inverted) as

$$df\left(\dot{\sigma}_{ij}, \dot{\varepsilon}_{ij}\right) = \left\{\frac{\partial^2 U_s}{\partial \varepsilon_{ij}\, \partial \varepsilon_{kl}}\right\} \dot{\varepsilon}_{kl} + \left\{\frac{\partial^2 U_c}{\partial \sigma_{ij}\, \partial \sigma_{kl}}\right\} \dot{\sigma}_{kl} \qquad (10.91)$$

It has assumed that the above equation is linear elastic material. Therefore, the materials can be furtherly described by a new relation form:

$$\dot{T} = D \vdots \dot{F} \vdots F \tag{10.92}$$

where, T is a stress measure, D is the material property tensor, F is the deformation gradient, and F is a stress strate for the linear elastic behavior, and superposed dots indicate time derivative ($T = \dot{\sigma}_{ij}\ or\ \dot{\varepsilon}_{ij}$; $\partial^2 U_s/\partial\varepsilon_{ij}\,\partial\varepsilon_{kl} = D_{ijkl}$; $\dot{F} = \dot{\varepsilon}_{kl}$; and $F = \partial^2 U_c/\partial\delta_{ij}\,\partial\varepsilon_{kl} = \sigma_{mn}$).

Hypoelastic material is an elastic material that has a constitutive model independent of finite strain measures except in the linearized case. The hypoelastic material is district of hyperelastic material, in that, except under special circumstance, they cannot be derived from a strain energy density function (Ogden 1984). In the theory of hypoelasticity, it is assumed that the incremental stress and strain tensors are linearly related by variable material moduli that are functions of the current stress or strain behavior. Mathematically, it can be expresses as

$$\dot{\sigma}_{ij} = D^e_{ijkl}(\sigma_{mn})\dot{\varepsilon}_{kl} \tag{10.93}$$

where, D^e_{ijkl} is the elastic stiffness matrix and it could be a function for both stress and strain. More details of stresses and strains constitutive material properties including the separation of recoverable and irrecoverable material components for flexible pavement can be found in (Brown 1996; Burmister 1943; Burmister 1945; Huang et al. 2011; Huang 1967; Huang 1968a; Huang 1993; Huang 2004; Masad et al. 2009; Ogden 1984; Rahman, Mahmud & Ahsam 2011; Yu 2006).

10.9. 3D-NONLINEAR FINITE-ELEMENT ANALYSIS

The finite –element method is well suited for analyzing pavement-tire interaction problems involving material nonlinearities and complex loading and such analysis proceeds by defining the characteristics of each pavement layers, including linear elastic, nonlinear, viscoelastic, plastic and viscoplastic material characterization (Nega, Nikraz & Al-Qadi 2015b). With the development of the powerful finite-element method, a break-through was made in the analysis of flexible and rigid pavement (Huang 2004). Cheung and Zienkiewicz (1965) developed finite-element methods for analyzing slabs on elastic foundation of both liquid and solid types.

The methods were applied to jointed slabs on liquid foundations by Huang and Wang (1973; 1974) and on solid foundations by Huang (1974). The general purpose 3-D finite-element package ABAQUS (1993) was used in simulating pavements (i.e. flexible and composite) involving nonlinear subgrade under dynamic loading (Zaghloul & White 1993; 1994). Nega, Nikraz and Al-Qadi (2015b) developed a new elastoplastic model for long time shakedown behavior of unbound granular material in flexible pavement using 2-D linear finite-element in ABAQUS taken into account in both isotopic and kinematic harding.

The presented nonlinear viscoelastic and plastic model has been implement into the well-known commercial FE code ABAQUS ™ via the user material subroutine UMAT, and the validation has been conducted by comparing with creep –recovery experimental data at different stress levels and various temperature (ABAQUS™, version 8, Habbit, Karlsson and Sorensen, Inc., Providence, R.I., 2008) and creep-recovery experimental data at different stress level (i.e. stiffness and shear) as the function of cyclic loading (Owende et al. 2001). Figure 10.13 and 10.14 are an example of predication of creep recovery experimental data at different stiffness and shear level as function of cyclic loading, respectively. These figures show that the model predictions have a reasonable agreement with experimental data.

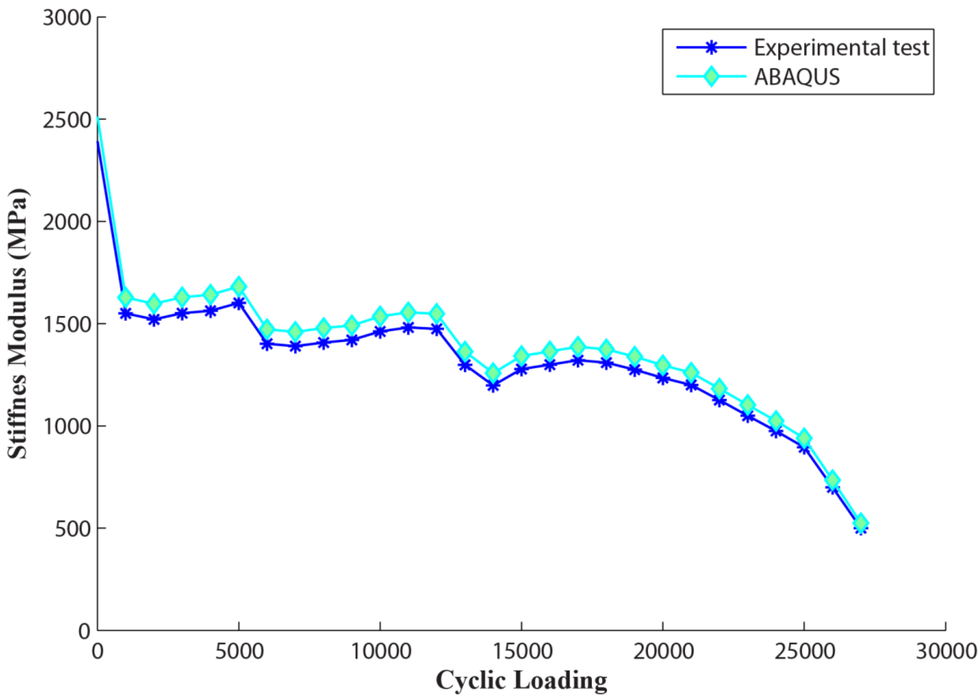


Figure 10.13 Comparison of experimental stiffness modulus of asphalt mixtures with finite element model analysis in ABAQUS

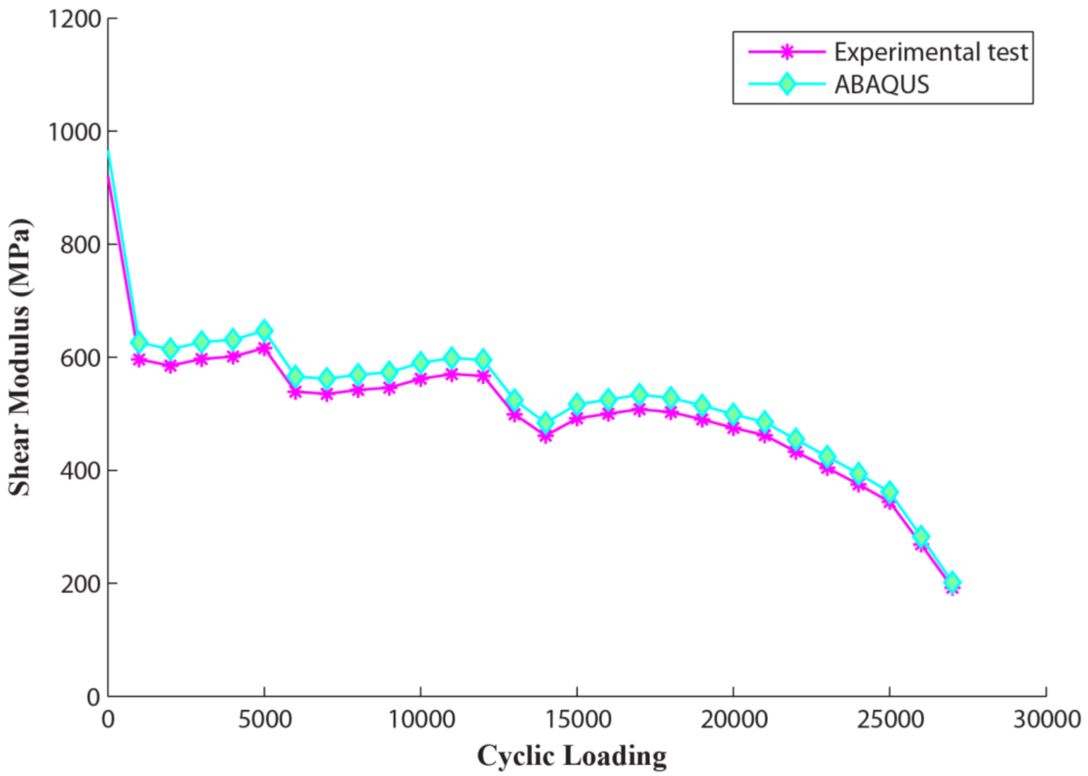


Figure 10.14 Comparison of experimental shear modulus of asphalt mixtures with finite element model analysis in ABAQUS

10.9.1. FINITE ELEMENT SIMULATIONS OF A PAVEMENT STRUCTURE

This section includes the description of a three-dimensional finite element (FE) model of a pavement section, and the use of this model to simulate flexible pavement response under repeated loading. In addition, this section also summarizes procedures for the identification of the material constraints associated with the constitutive model outline in the previous section.

10.9.1.1. GEOMETRY OF FLEXIBLE PAVMENT STRUCTURE, FE MODEL AND IN SITE DATA FOR MODEL VERIFICATION

The flexible pavement structure contains a finite-element discretization of a five-layer system having a 50 mm-thick AC layer, a 150 mm-thick unbound base layer, a 250 mm-thick unbound subbase layer, a 75 mm-thick compacted subgrade layer and an infinite natural subgrade is considered in this study. The pavement layers are overlaid on an infinite natural subgrade of peat soil. Typical cross-section of conventional flexible pavement setup: load of dual tires, material properties, pavement thickness and center space between the dual tires is shown in Figure 10.15. The illustration depicts of typical cross-section of five-layer linear elastic of the flexible pavement model is also considered a viscoelastic on the pavement surface area. Falling Weight Deflection (FWD) experimental data from seven main roads and creep test from which the model verification was derived with permission to make sure the implemented to assure integrity of the in situ experimental data used in the verification of the finite element. The details experiments can be found (Mulungye, Owende & Mellon 2007; Nega, Nikraz & Al-Qadi 2015a; Owende et al. 2001).

The 40 kN wheel load to represent a set of dual tires is assumed to be uniformly distributed over the contact area between tire and flexible pavement surface (Figure 10.15). The size of contact area depends on the contact pressure and the contact pressure is assumed as equal to the tire pressure.

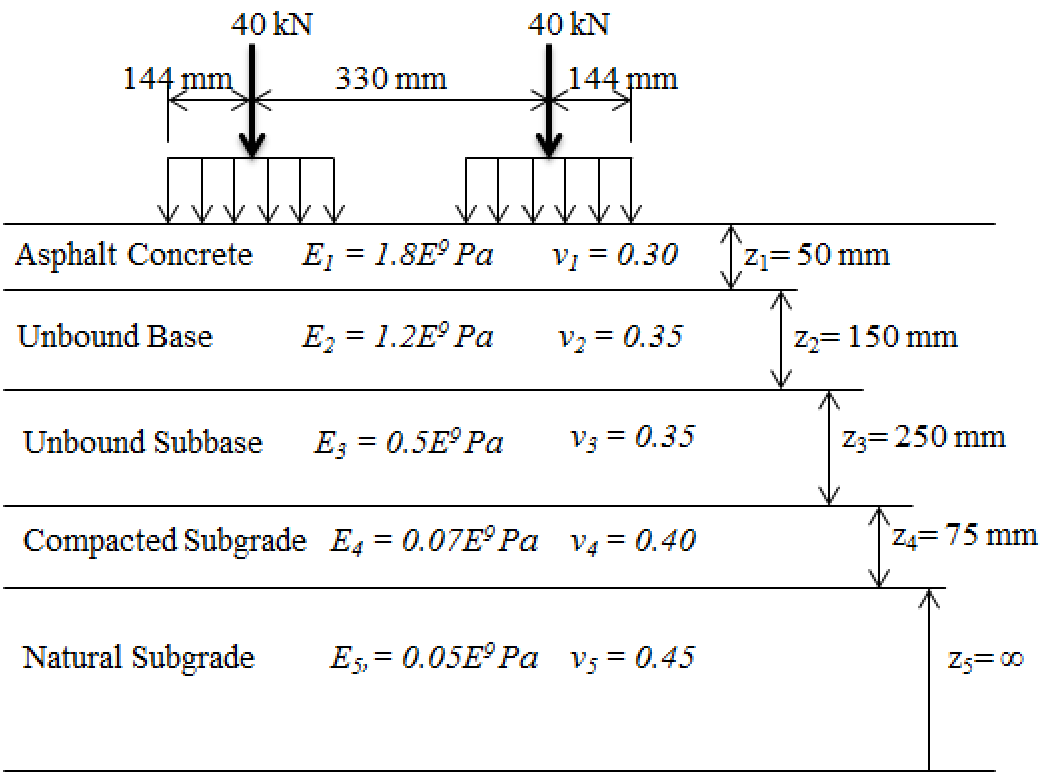


Figure 10.15 Typical cross-section of conventional flexible pavement setup: load of dual tires, material properties, pavement thickness and center space between the dual tires

10.9.1.2. FE PAVEMENT MATERIAL PROPERTIES

A constitutive law is required for pavement material in the nonlinear finite element simulation. The asphalt surface is viscoelastic and elastoplastic material of high complexity (Finn et al. 1986; Nega, Nikraz & Al-Qadi 2015b; Nega et al. 2013b; Uddin, Zhang & Fernandez 1994). It is exhibits sensitivity to both temperature and rate of loading (Deacon et al. 1994; Diefenderfer, Al-Qadi & Diefenderfer 2006; Mulungye, Owende & Mellon 2007). However, many methods for analysis of pavement employ simple linear elastic theory as a first approximation to the evaluation of the pavement response model. The elastic material properties of flexible pavement layers are tabulated in Table 10.1.

Viscoelasticity for the asphalt surface is also considered in this study, and the results compared to the corresponding pavement response of linear elastic material model. The viscoelastic material curve fitting tool in ANSYS (ANSYS 1999) was used to determine the material constraints constants of the prony series expansion for shear modulus option from the experimental data. The Viscoelastic data in Table 2 was obtained from creep and/ or bending fatigue tests, which was performed at temperature 25^{o}C with a void content of 7% and a frequency of 5 Hz. The experiment data in cycles were converted to time and then, stiffness modulus was converted to shear modulus. It should be noted that the stiffness and shear modulus equations are included constitutive model equation in this study.

Table 10.1 Material Properties for Flexible Pavement Model

Pavement Layer	Moduli , E (Pa)	Poisson's Ratio (v)	Thickness (mm)	Density, p (kg/m^3)
Asphalt concrete	1.80E^9	0.30	50	2400
Unbound base	1.20E^9	0.35	150	2150
Unbound Subbase	0.50E^9	0.35	250	1850
Compacted subgrade	0.07E^9	0.40	75	1700
Natural subgrade	0.05E^9	0.45	Infinite	1600

Table 10.2 Viscoelastic Prony Coefficient from Four Point Bending Fatigue Test (Creeping test)

Cyclic Loading	Time (s)	Stiffness Modulus, E (MPa)	Shear Modulus, G (MPa)
10	2.5	2536	975
20	5.0	2383	916
30	7.5	2440	938
40	10.0	2437	937
50	12.5	2469	950
60	15.0	2495	960
70	17.5	2561	985
80	20.0	2528	972
90	22.5	2475	952
100	25.0	2392	920
200	50.0	1550	596
300	75.0	1521	585
400	100.0	1551	597
500	125.0	1563	601
550	137.5	1601	616
950	237.5	1402	539
1050	262.5	1390	535
1150	287.5	1408	542
1250	312.5	1420	546
1350	337.5	1462	562
1450	362.5	1481	570
1550	387.5	1474	567
2550	637.5	1298	499
3550	887.5	1198	461
4550	1137.5	1278	492
4590	1147.5	1299	500
6090	1522.5	1321	508
7590	1897.5	1309	503
9090	2272.5	1275	490
10950	2737.5	1235	475
12590	3147.5	1200	462
14590	3647.5	1126	433
16590	4147.5	1050	404
18590	4647.5	975	375
20000	5000.0	895	344
25000	6250.0	700	269
30000	7500.0	500	192

In this study, linear elastic and viscoelastic theory is used and the vehicle speed is directly related to duration of cyclic loading and as the same time, the resilient and shear modulus of each paving materials are considered so that material selected can communicate with the vehicle speed. The greater the speed the larger the modulus, and the smaller the strain is in a flexible pavement. Figure 10.16 shows sketch of simulated pavement structure for cyclic loading. Table 10.3 shows the measured static, wheel load, tire contact area and average contact dual tire inflation pressures

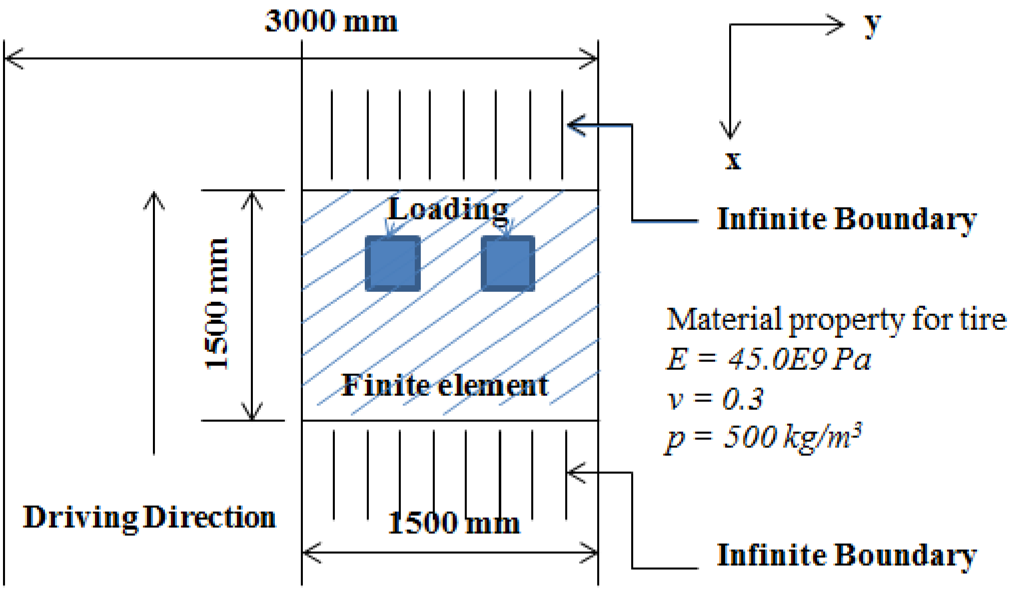


Figure 10.16 Sketch of simulated pavement structure for cyclic loading

Table 10.3 Measured Static, Wheel Load, Tire Contact Area and Mean Contact Pressure for Experimental Truck

Tire Pressure (kPa)	Wheel Loading (kN)			Contact Area (m^2)		Mean Contact Pressure (kPa)		
	Front	Middle	Rear	Front	Rear	Front	Middle	Rear
350	28.4	21.6	22.6	0.059	0.045	484	476	498
490	28.4	21.6	22.6	0.050	0.042	571	516	539
630	28.4	21.6	22.6	0.044	0.039	648	552	578
700	28.4	21.6	22.6	0.040	0.038	715	573	599
350	31.6	34.4	35.2	0.067	0.067	474	510	522
490	31.6	34.4	35.2	0.055	0.062	578	555	568
630	31.6	34.4	35.2	0.048	0.056	661	610	624
700	31.6	34.4	35.2	0.044	0.051	720	677	693
350	31.6	44.6	46.1	0.067	0.085	475	524	518
490	31.6	44.6	46.1	0.055	0.077	580	578	572
630	31.6	44.6	46.1	0.048	0.070	663	636	629
700	31.6	44.6	46.1	0.044	0.063	722	708	699

10.9.2. TIRE CONTACT AREA AND ASSOCIATED STRESS

In the mechanistic method of design, it is necessary to know the contact area between tire and pavement so the axle load can be assumed to be uniformly distributed over the contact area. Huang (1993; 2004) discussed that the size of contact area depends on the contact pressure. When the wall of tires is in compression, the sum of vertical force due to wall and tire pressure must be equal to the force as a result of contact pressure. Similarly, for high-pressure tires, the contact pressure is small than tire pressure due to the wall of the tires pressure is in a tension. However, in a pavement design, the contact pressure is generally assumed to be equal to the tire pressure because of heavier axle load have higher tire pressures and more destructive effects on pavements. Thus the use of tire pressure as the contact pressure is therefore on the safe side as recommended by Huang's.

The contact area can be represented by two semicircles and a rectangle as shown in Figure 10.17a. The acceptable approximation shape of contact area for each tire is composed of a rectangle and two semicircles, having length L and width 0.6L. This shape of two semicircles and rectangle is converted to a single rectangle as suggested by Huang (2004) having a contact area of $0.5227L^2$ and a width o f0.6L as shown in Figure 10.17b. The area of contact $A_c = \pi(0.3L)^2 + (0.4L)(0.6L) = 0.5227L^2$ or

$$L = \sqrt{\frac{A_c}{0.5227}} \tag{10.21}$$

The contact area, A_c is obtained by dividing the Load on each tire by tire inflation pressure as:

$$L = \frac{P}{P_i} \tag{10.22}$$

where, P is the wheel load of a dual tires and P_i is the tire inflation pressure of dual tires. Since$L = 0.330\, m$, $\left(L = \sqrt{(P/P_i)/0.5227} = \sqrt{(40/700)/0.5227} = 0.330\right)$, the contact area has the dimension of $0.288\, m \times 0.198\, m$ as shown in Figure 10.17. The contact area shown in Figure 10.13a was previously used by PCA (1966) for design of rigid pavement (Huang 2004). The current PCA (1984) method is based on the finite element procedures, and a rectangular area is assumed with length of 0.8712L and width 0.6L, which was the same area of $0.5227L^2$ as shown in Figure 10.17a and b.

When layered theory is used for flexible pavement design, it is assumed that each tire has a circular contact area, and this assumption is not corrected, although the error incurred is believed to be small as discussed by Huang's. To simplify the analysis of flexible pavement, the same contact area as duals is frequently used in this study to represent a set of dual tires, instead of using two circular areas as suggested by Huang's. The dimension of tires contact used for flexible pavement design is shown in Figure 10.18.

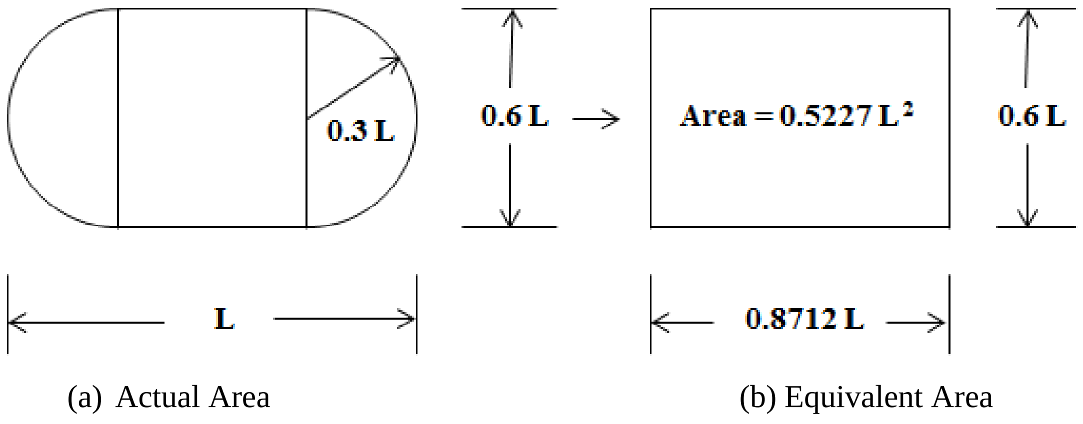


Figure 10.17 Dimension of tire contact area between tire and pavement surface

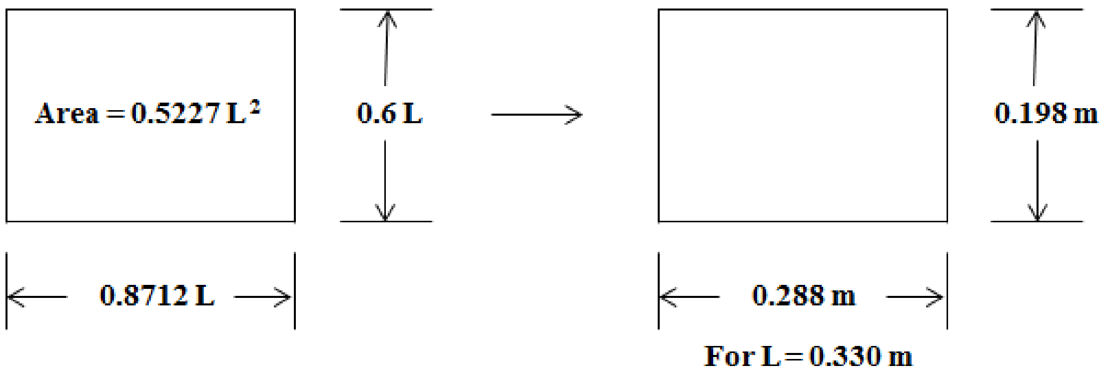


Figure 10.18 Dimension of tire contact area used for flexible pavement design

The loading configuration allow developing a symmetric FE model edge of the mesh in order to represent the middle of pavement the others opposite side of vertical edge is also fixed in horizontal direction over the whole pavement section while the bottom of the FE mesh is fixed on both horizontal and vertical direction representing a very stiff layer. The FE model employed the infinite element (CIN3D8) in ABAQUS to representing the infinite boundary in the driving direction.

The three-dimensional FE mesh contains a 8-node, first-order quadratic element with reduce-integration (C3D8R) (linear) and a 20-node quadratic reduced-integration (C3D20R) (nonlinear) was used for this model in ABAQUS because they are not susceptible to locking, even when subjected to complicated state of stress. Therefore, these element types are generally the best choice for most general stress and/ or displacement simulations. As the result of this appropriate choice of element type, in this study, all the elements have converged because of hourglassing that are used through the width.

The size of an element is 0.0508 m x 0.0508 m and 0.0254 m x 0.0254 m within the course and fine mesh. From the FE model area, it can be assumed that the FE mesh can contains around 14,500 elements and 520 infinite elements. This is a rough assumption to preload the model in ABAQUS. Figure 10.19 and 10.20 show the assembly of five-layers of flexible pavement and the three-dimensional nonlinear FE mesh.

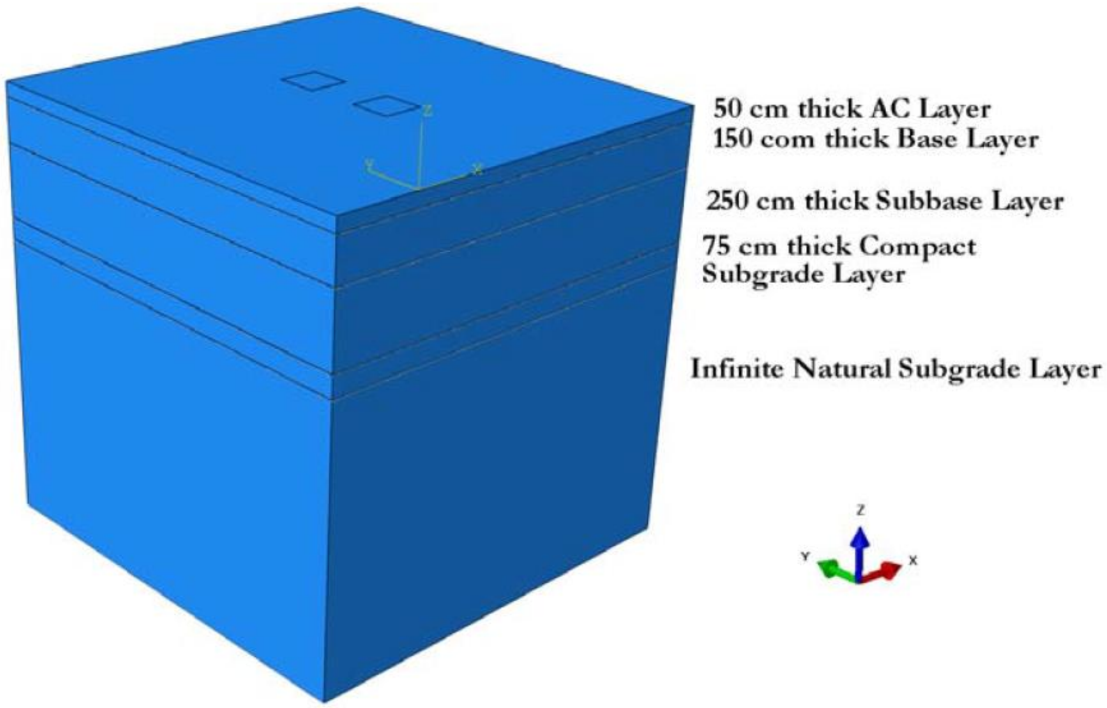


Figure 10.19 Assembly of five-layers of flexible pavement (3D-FE) model analysis for stresses and strains characteristic behavior applying Dual tires-The Dual –Axle Analysis

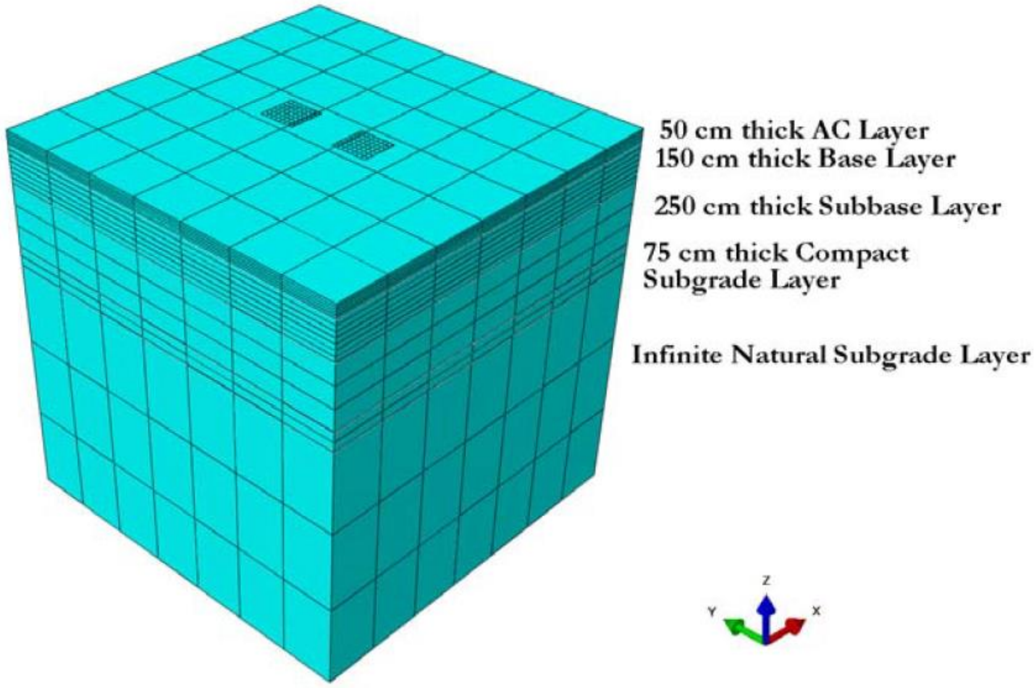
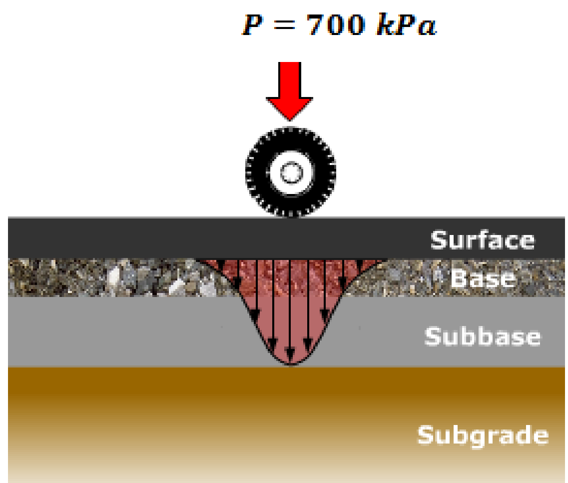
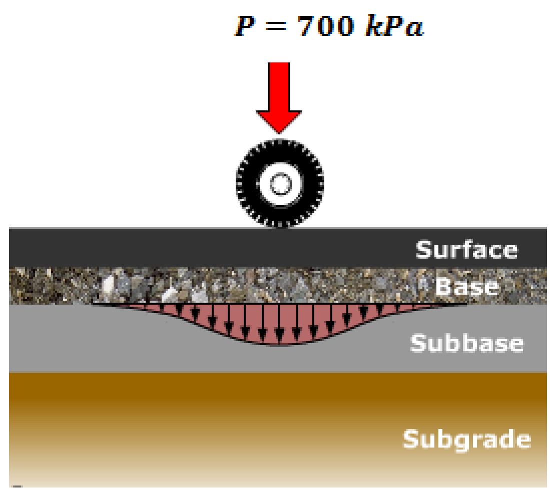


Figure 10.20 Nonlinear finite element mesh (three-dimensional) of flexible pavement using hexagonal continuum reduced elements (C3D20R) for Dual Tires-The Dual –Axle Analyses.

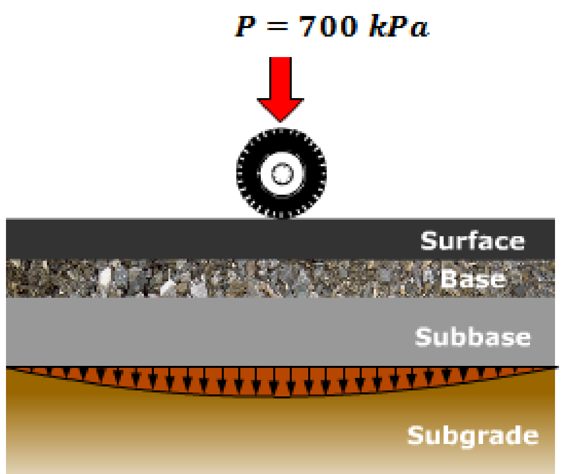
The sketch of simulated for loading area of the dual tire and distribution of tire pressure in each layer of the flexible pavement is shown in Figure 10.21. The loading is applied on the asphalt surface as shown in Figure 10.22 with maximum tire inflation pressure of 700 kPa according to Australia Standard (Australian Standard 1995; Austroads 2012) and distributed over the contract length (Hua & White 2002; Huang et al. 2011; Hunter, Airey & Harireche 2007). This loading represents the average value of an actual truck loading. For simplicity, dotted idealized curve in Figure 10.23 is used to represent the contact pressure distribution in the finite element simulations. The loading area is 198 mm in width and 288 mm in length for each tire. Within each cycle, the loading is applied with duration of 0.008 s to simulate the vehicle speed 95 km/h and then, the load is removed for 1.0 s as the concept is shown as schematic form in Figure 10.24. The cyclic time is a total of 10 second with a loading of 0.1 s and resting 0.9 s.



(a)



(b)



(c)

Figure 10.21 Sketch of Simulated for the loading area of the dual tire

where, P is the tire inflation pressure.

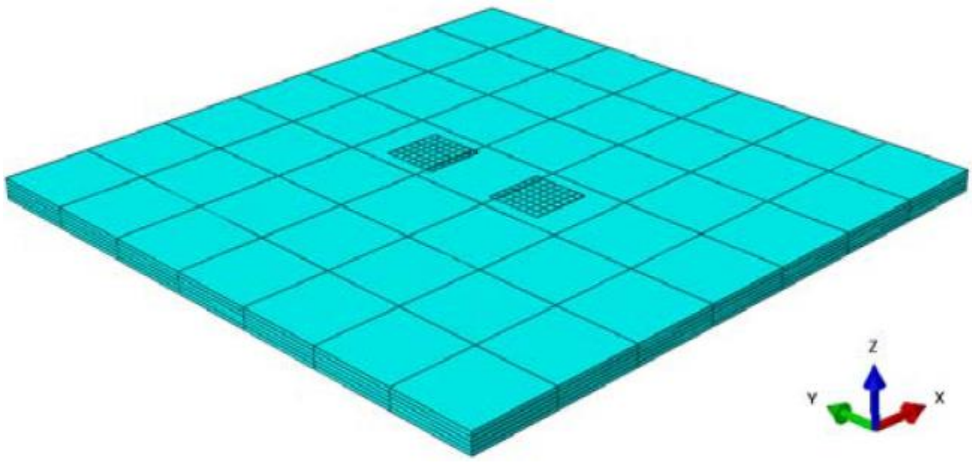


Figure 10.22 Plane view of asphalt surface (AC) layer showing the area of applied load

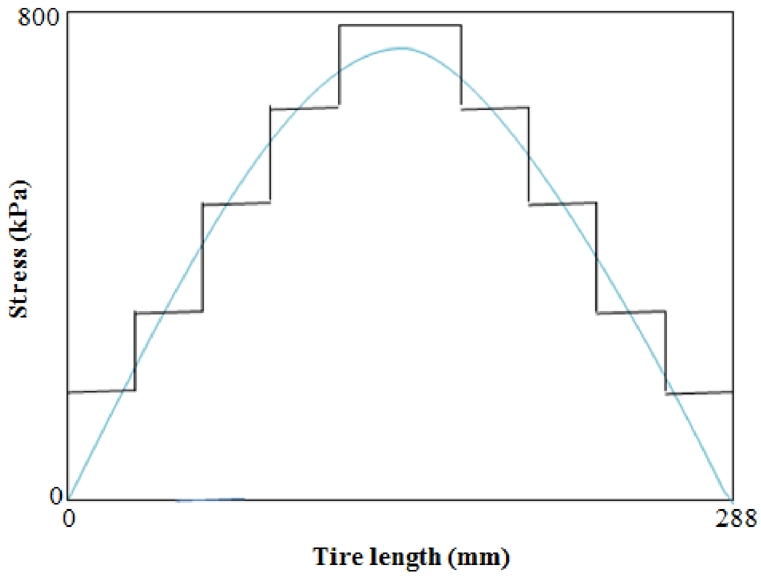


Figure 10.23 Assumed loading pressure distribution over the tire contact

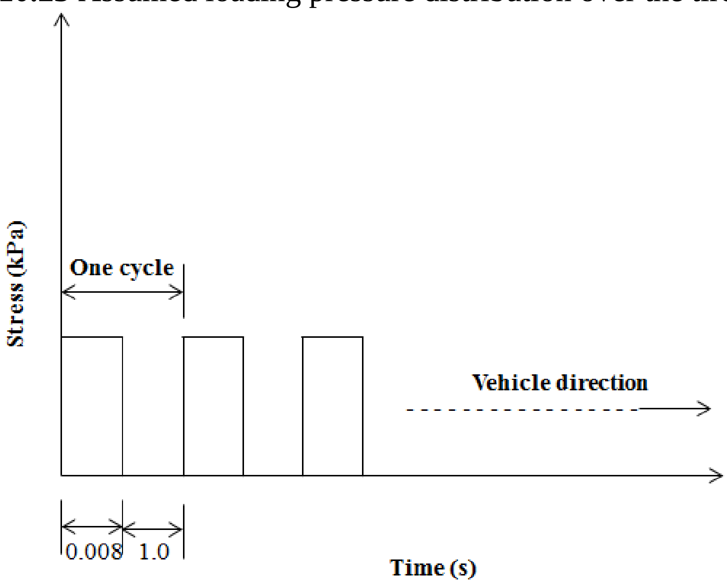


Figure 10.24 Sketch for simulated vehicle speed for each cyclic loading

The developed three-dimensional FE model (undeformed shape) using ABAQUS to represent a five-layer of flexible pavement structure and also to simulate linear viscoelastic and nonlinear viscoelastic response under repeated loading (dual tires) at different temperature is shown in Figure 10.25. The deformed shapes for both linear and nonlinear viscoelastic model are shown in Figure 10.26 and 10.27, respectively. Control of Mises stress on deformed shape of flexible pavement using a linear viscoelastic and a nonlinear viscoelastic model for the dual tires analysis are shown in Figure 10.25 and 10.26. As it can be seen from the analysis, the contours' range of the linear viscoelastic model has low maximum stress (i.e., 2.17 x 10^{-7} to 1.56 x 10^{6}) as compared to nonlinear viscoelastic model (i.e., 2.51 x 10^{-1} to 1.57 x 10^{6}). This indicated that the vertical strain on the top of the subgrade layer and the tensile strain at the bottom of the asphalt surface in this simulation are about 9% less in stiffness than that predicted by nonlinear viscoelastic analysis. The simulations shows that tensile viscoelastic strain accumulate at the pavement surface (i.e., on linear viscoelastic material) could be the cause for distorted surface as the result of vertical strain on the top of compact subgrade, and a phenomenon that could be associated with cracking of asphalt pavement.

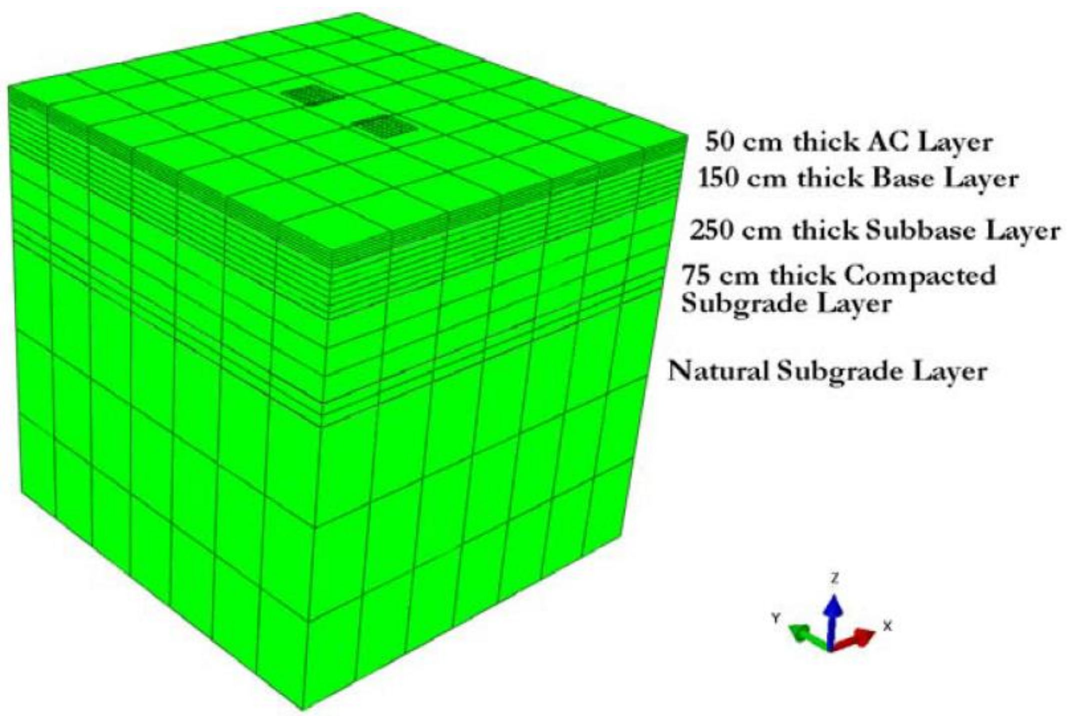


Figure 10.25 Undeformed shape of flexible pavement using linear viscoelastic and nonlinear viscoelastic finite-element discretization (three-dimensional) model for the Dual-Tires Analyses

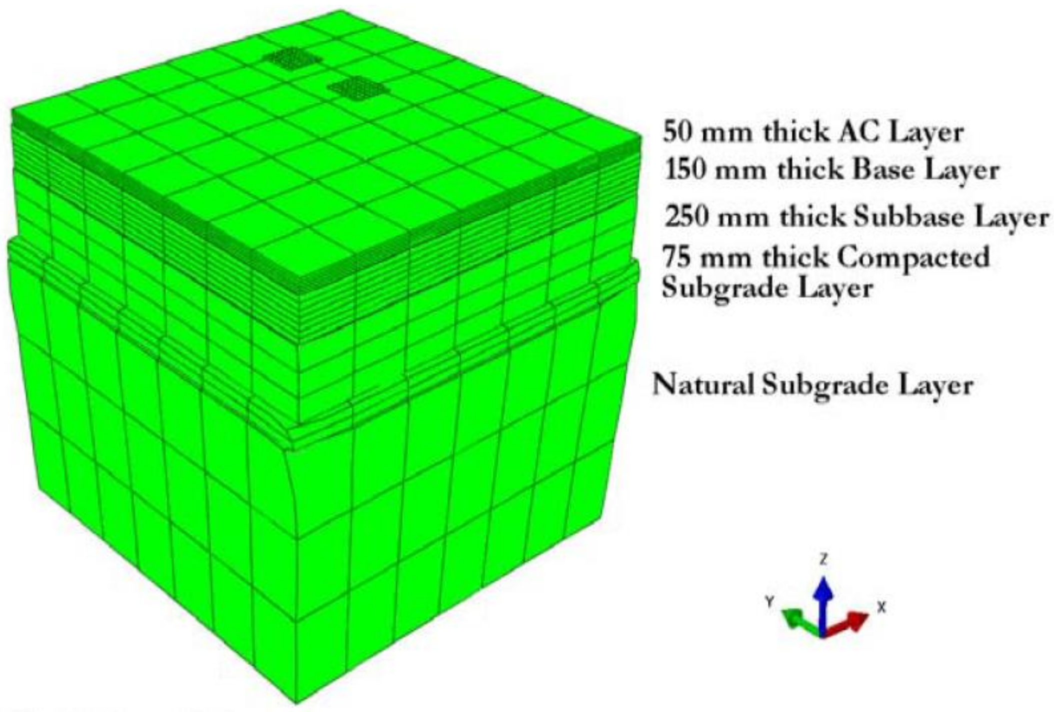


Figure 10.26 Deformed shape linear viscoelastic and plastic finite-element discretization (three-dimensional) model for the Dual-Tires Analyses

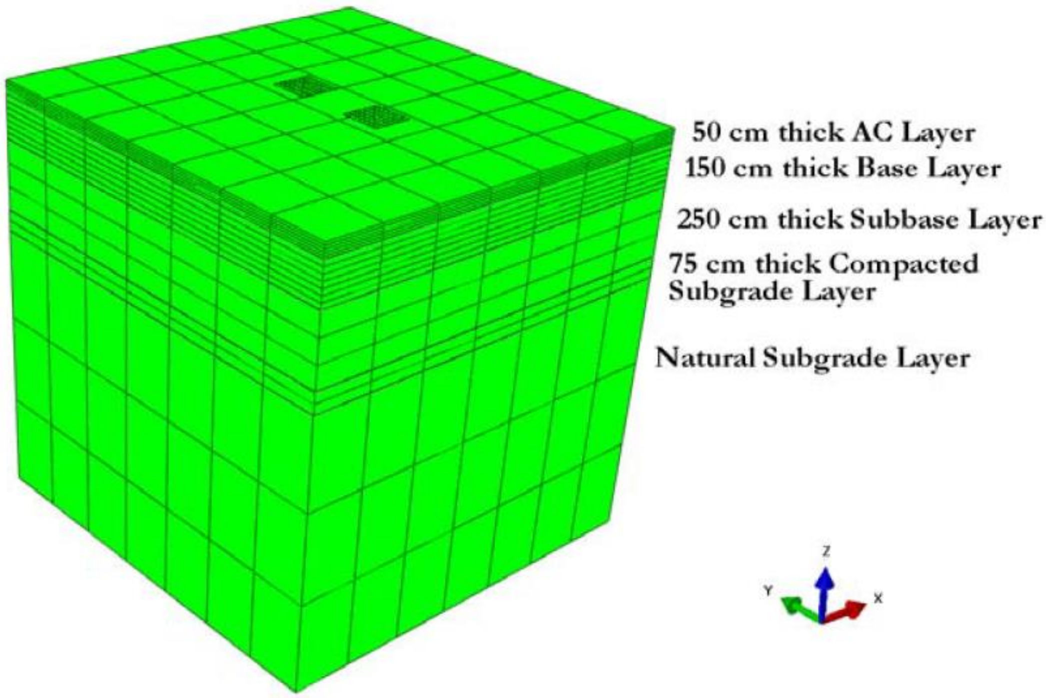


Figure 10.27 Deformed shape nonlinear viscoelastic and plastic finite-element discretization (three-dimensional) model for the Dual-Tires Analyses

Even if a reasonable difference of maximum stress distribution is acceptable for both models, including the nonlinear geometric effects in simulation reduces a vertical deflection and/ or vertical strain on top subgrade and tensile strain at the bottom of asphalt surface. The nonlinear viscoelastic analysis includes the effect of stiffening and the direction of the pressure, and neither of these effects is included in the linear viscoelastic simulation. Another, a particular important in this case is whether a given flexible pavement structure will experience high stress that will lead to accumulation of viscoplastic strain on increase viscoplastic strain will cease to occur, and then loading from dual tires to stable the linear viscoelastic material. Permanent vertical strain in non-stress dependent case is significant lower than the permanent in stress dependent case.

The fundamental difference between a linear and nonlinear material response is depend on pavement material types. Although most pavement materials are nonlinear, the use of a linear model will suffice provided the stress states are low. Noted that both developed simulations whether it is linear viscoelastic or nonlinear viscoelastic models hand it very well and there was not a problem of converging during iteration. Despite this a little significant different and as the result of these, the tensile viscoelastic strain on the top of compacted subgrade surface with the linear might develops at the sides of the applied load of the dual tires due to asphalt mixture heave associated with permanent deformation.

Furthermore, nonlinear viscoelastic behavior determined by linear relaxation time spectrum and that the nonlinear effect of stress is to alter the intrinsic time-dependent scale of material. It should be notice from analysis that vertical strain is design of flexible pavement is one of the critical, which the rutting of the pavement is controlled. Therefore, understanding of stress-strain relationship concept both in linear viscoelastic and nonlinear viscoelastic material give a sound consideration for its effect during pavement design might reduce the risk level of permanent deformation and/ or rutting in flexible pavement construction. If the stress-strain characteristic in pavement analysis and design has ignored, it might lead to pavement overdesign and then, permanent deformation or rutting will control the criterion.

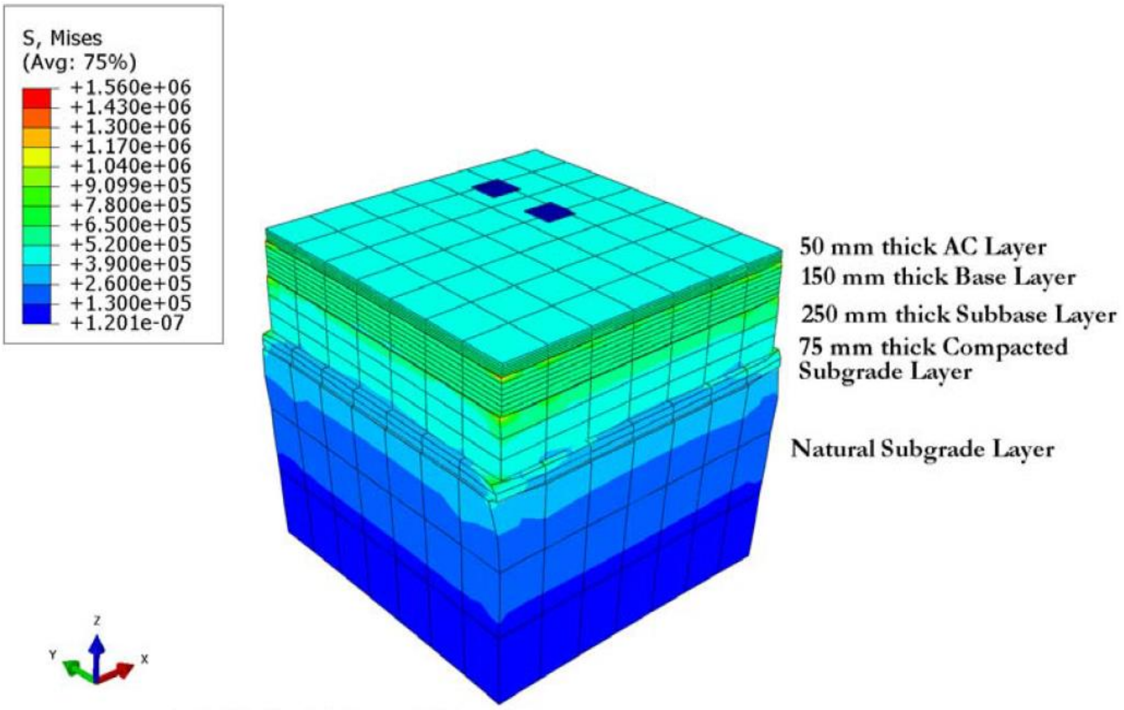


Figure 10.26 Contour plot of Mises stress on deformed shape linear viscoelastic and plastic finite-element discretization (three-dimensional) model for the Dual-Tires Analyses

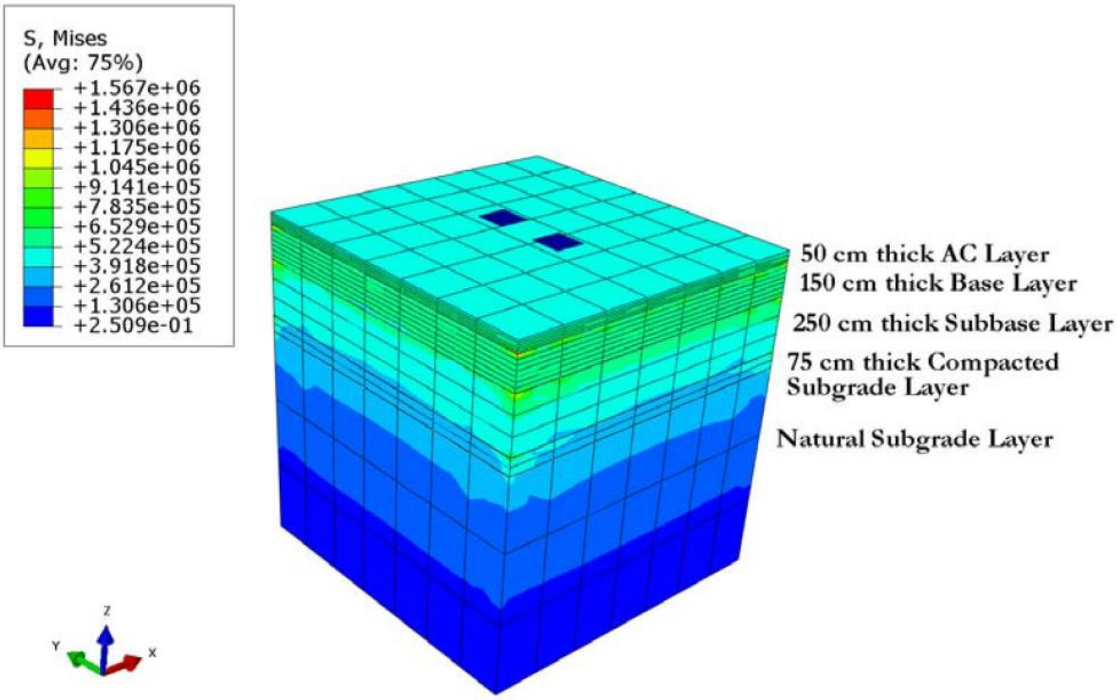


Figure 10.27 Contour plot of Mises stress on deformed shape nonlinear viscoelastic and plastic finite-element discretization (three-dimensional) model for the Dual-Tires Analyses

Huang, et al. (2011) developed a nonlinear viscoelastic-viscoplastic constitutive model in order to represent the response of asphalt mixture under different temperatures and rates of loading. They developed a three-dimensional FE model using ABAQUS to represent a three-layer pavement structure and also to simulate the viscoelastic and viscoplastic response under repeated loading at different temperatures. The demonstrated results indicated that the capacity of the model is simulating influence of temperature on permanent deformation and in predicting viscoelastic and viscoplastic strain distribution in the asphalt layer. The simulations showed the tensile viscoplastic strain accumulate at the pavement surface, a phenomenon that could be associated with cracking of asphalt pavement. Results also showed high pavement temperature and tensile viscoplastic strain developed at the sides of the applied due to asphalt mixture heave associated with permanent deformation and dilution.

Similarly, Helwany, Dyer and Leidy (1998) illustrated the useful of the finite-element in analysis of three-layer pavement system subjected to different types of loading. The method was able of simulating the observed response of pavements subjected to axle loads with different tire pressure, axle loads with different configuration, and axle loads traveling at different speed. A variety of material constitutive model such as linear elastic, nonlinear elastic, and viscoelastic are employed in the analyses to describe a behavior of the pavement materials. Results showed increase in the vertical stress at the bottom of the AC layer as the tire approaches the element in question and reached its maximum at tire was directly above the element, and then decreasing as the tire recodes. It was noted the vertical stress at the bottom of the AC layer corresponding to a vehicle speed of 8 km/hr was approximately 7% greater than that corresponding to a vehicle speed of 105 km/hr. They recommended that for a more analysis of the pavement, the viscous parameter of the AC layer and other pavement layers should be determined from laboratory tests.

10.9.3. TIRE/PAVEMENT CONTRACT STRESSES AND STRAINS, DISPLACEMENT UNDER THE MOVING LOAD AND SOME EFFECTS ON PAVEMENT

When the material is loaded the state of stress, the state of strain at that point is a time-dependent. In high-strong pavement viscoelastic material, large stresses are accompanied by small stress, and in a week pavement material, large strain results from small stress (Nega, Nikraz & Leek 2013b; Nega, Nikraz & Leek 2013c). Stress can be described in way comparable to description of stress. When the state stress is described by normal stresses and shear stresses, the components of deformation and/ or displacement are normal strain and shear strain. The stress, strain and deflection distribution response in a different layered of flexible pavement as simulated vehicle speed under repeated loading at different time of cyclic loading are shown in Figure 10.28 to 10.30, respectively.

From the simulation 3-D finite-element presented, it can be seen the load (i.e. from dual tires) was uniformly distributed over pavement contact area and also had a better match with the loading time obtained from the vehicle at average speed. This indicated that the cyclic loading time does not affect its material properties but improve the uniform vertical condition including within asphalt surface at the dual tires center. In addition, it improved the load/contact stress idealization modeling. Also, at low levels of stress ratios the accumulation of permanent strain cease, resulting in an equilibrium state because the materials properties are match with the simulated vehicle speed at different cyclic loading time. It is demonstrated that this new developed simulation method provides an appropriate and realistic analysis of prediction of stress, strain and deflection distribution in the pavement layers.

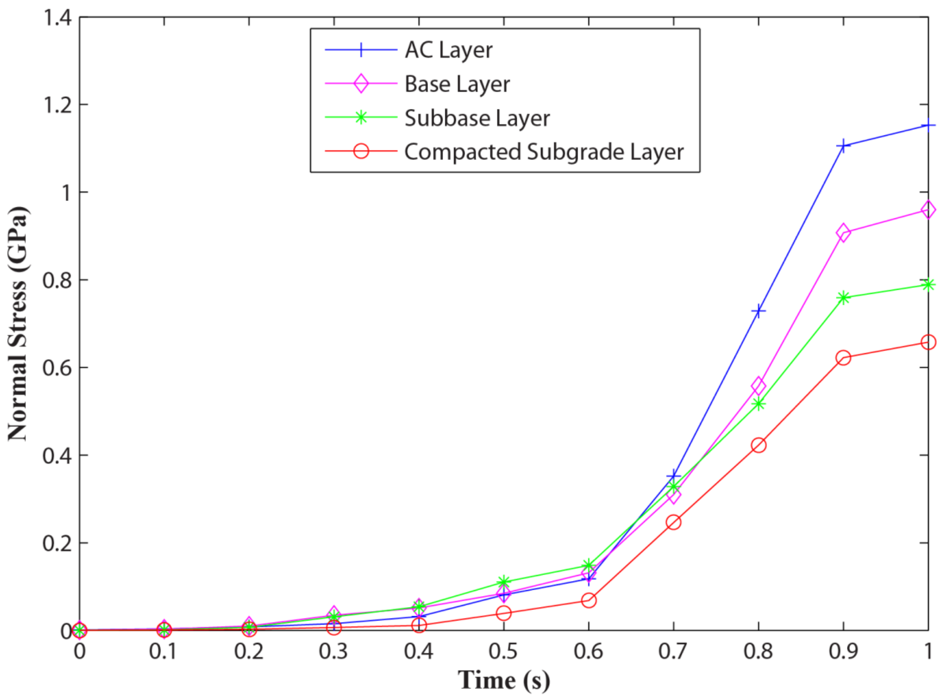


Figure 10.28 Normal stress of flexible pavement layers as the function of time

Furthermore, the pavement layers computed deflection with a similar trend variation at different cyclic loading time, and these also showed a reducing vertical surface deflection and critical tensile strain in asphalt concrete layer including the others pavement layers. The displacement distribution (Figure 10:30) in flexible pavement layers as the function of time due to applied load was quite reasonable and it is also within the standard just like the stress-strain behavior. All the predictions are quite reasonable also matched with the laboratory experiment. This finding may have important implication for design of relatively thin asphalt surface layers for pavement.

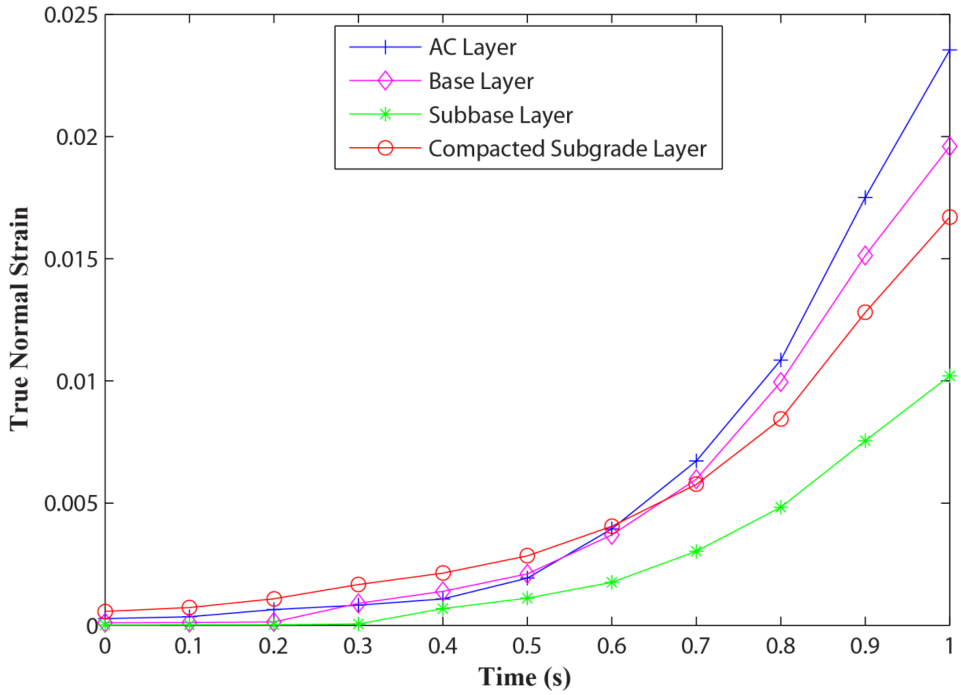


Figure 10.29 True normal strain of flexible pavement layers as the function of time

Brown and Pell (1967) investigated the stresses, strains and deflection in a layered flexible pavement in laboratory subjected to dynamic loads. The complete stress and strain distribution set up were in different layered of flexible pavement under dynamic load. The load was uniformly distributed over a circular contact area. In situ measurements of stress and strain were made using pressure and strain cell. Stress and strain distributions were determined by moving the load. The demonstrated results indicated stresses showed a good agreement with theory in both system (i.e., match of layered pavement materials with the moving dynamic load). However, strain being dependent on modulus was less easy to predict theoretical insitu values of modulus were stress dependent for both different flexible pavement materials. Two layers system results compared less favorably with theory but important values of tensile horizontal stress above the interface and vertical strain below.

Lekarp and Dawson (1998) presented a state of are on modeling permanent deformation behavior of unbound granular materials. The existing numerical models are verified by series of laboratory repeated load tests. A new model is introduced expressing the accumulated permanent axial stress at any given number of cycles as function of applied stresses, taking into account the maximum shear stress ratio and the length of stress path in p-q space. Result showed similarities with the concept of stress-strain relationship theory. At low level of stress, ratios the accumulation of permanent strain cease, resulting in an equilibrium state. The stresses, strains and deflection in a layered flexible pavement are always time-dependent and also subjected to dynamic load and vehicle speed.

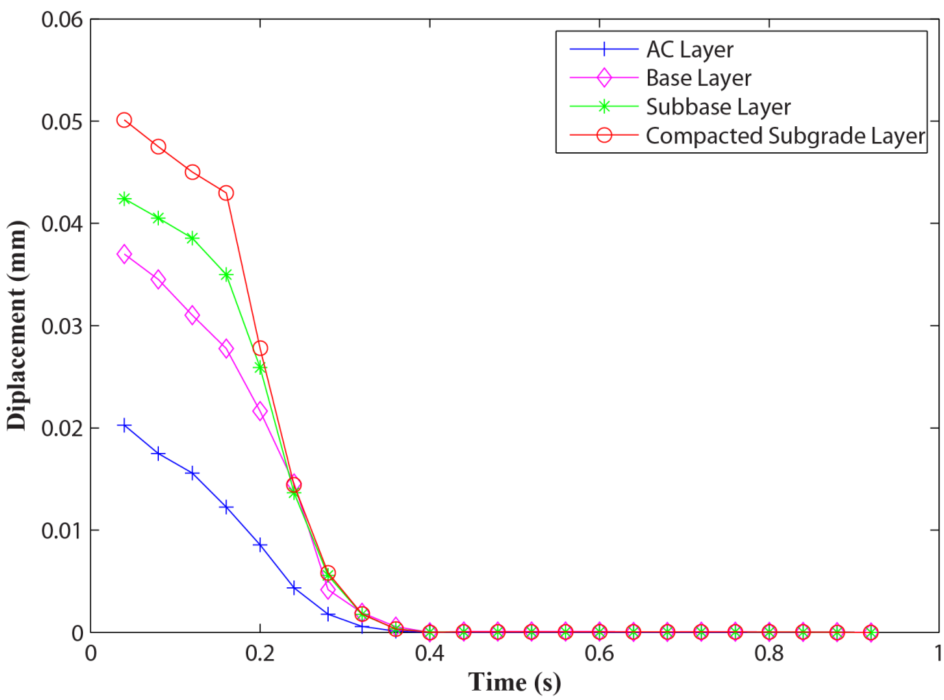


Figure 10.30 Displacement of flexible pavement layers as the function of time due to applied load

Tseng and Lytton (1989) also presented a method to predict the permanent deformation (rutting) in flexible pavement using a mechanistic-empirical model material characterization. Three permanent deformation parameters are developed through testing to simply represent the curved relationship between permanent strain and the number of load cycles. Equation were developed by regression analysis which determine how these parameters are affected by the material properties, environmental condition (moisture and temperature), and the stress state. These relationships are important in calculating and/ or predicting permanent deformation of pavement layers since the relationship between permanent deformation and cycles of load from the laboratory that are significantly different from field condition. Results showed that a reasonable agreement with measured results, and also provide an appropriate and realistic analysis of prediction of permanent deformation. Figure 10.31 shows the comparison of experiment stiffness and shear modulus of asphalt mixtures with simulated stiffness and shear modulus in finite-element analysis in ABAQUS. As

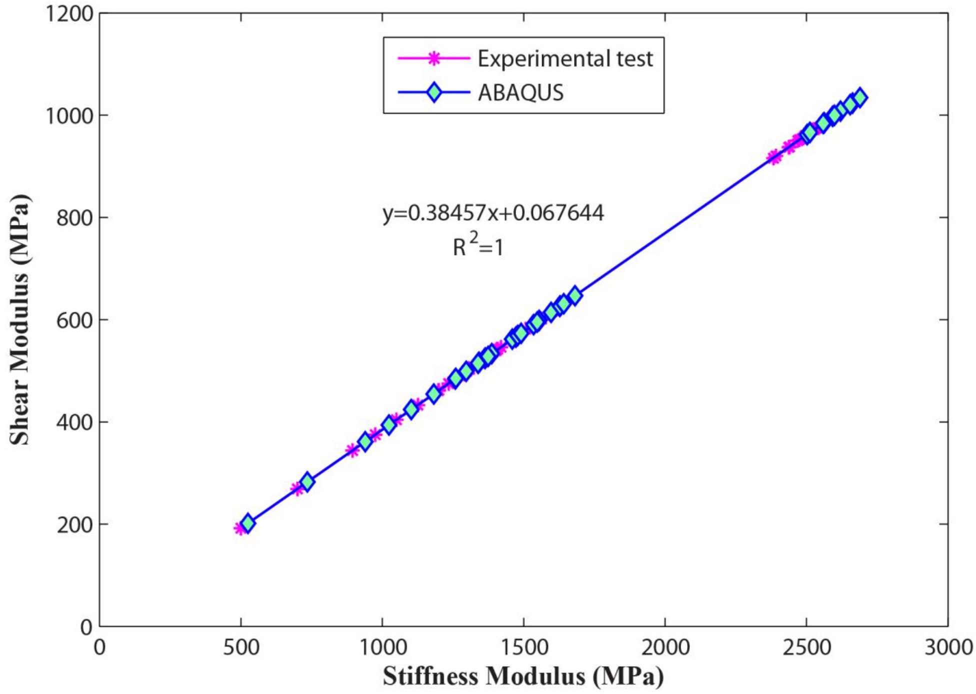


Figure 10.31 Comparison of experimental stiffness modulus and shear modulus of asphalt mixtures with finite element model analysis in ABAQUS

It can be seen from the presented analysis, an accurate correlation between the measured and predicting analysis (FE in Abaqus). Both the experiment (measured) and FE-analysis were with very good correlation (R^2 = 1) with a criteria for subjective class of goodness of pavement material properties. This increase in stiffness and shear modulus implies to reducing asphalt pavement failure and improved low stiff and shear cracking resistance with flexible pavement structure. Pellinen and Witczak (2002) analyzed the use of stiffness and shear of flexible pavement (hot-mix asphalt) using simple performance test that limit the stiffness values because of the power law and sigmoidal function and found good correlation (R^2 =0.9981).

Average comparisons of actual stress and strain variation for different contact area tires/pavement are shown in Figure 10.32 and 10.33, respectively. As it can be seen from the experiment result and finite-element in Abaqus, there are a good correlation between the two analyses. The maximum principle stress variation (Figure 10.32) increase number of elements in the analysis for different contact between tire/pavement.

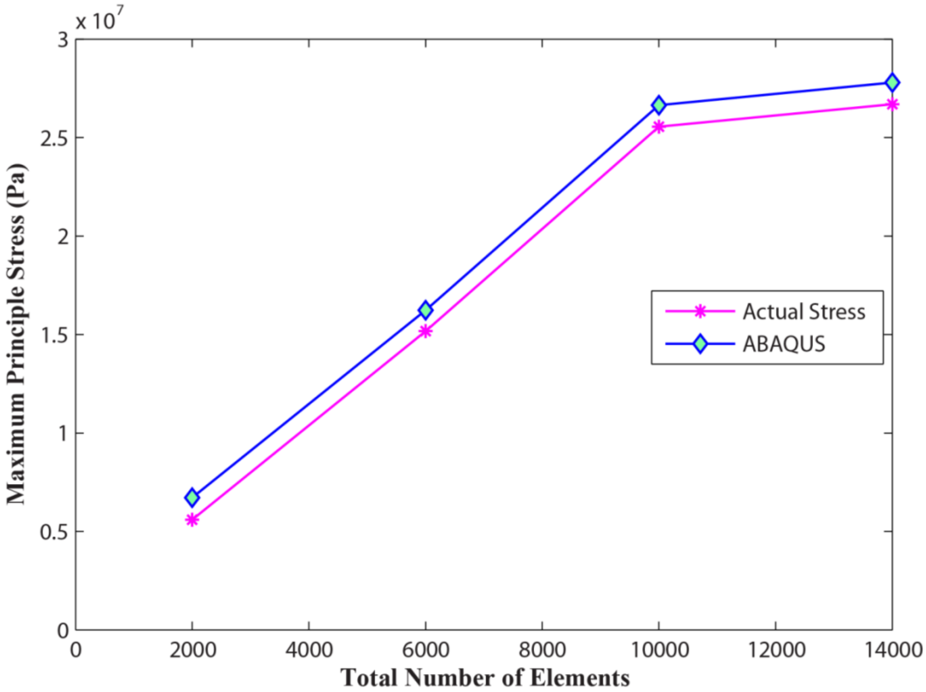


Figure 10.32 Comparison of actual stress variation for different contact area of dual tires in asphalt pavement multi-layers with finite element model analysis in ABAQUS

While maximus strain (Figure 10.33) is decrease with the increase of elements. Also, it can be noted that the predicting in Abaqus has great values for both stress and strain as compared to the measured one (i.e., experiment). In addition, increasing the number of element increase the degree of accuracy. Rahman, Mahmud and Ahsam (2011) analyzed stress-strain characteristics of flexible pavement using 3-D finite element application for predicting mechanical behavior and pavement performance subjected to various traffic factors. Different axle configuration, tire imprint area and tire inflation pressure were investigated to analyses the considerable impact on pavement damage initiation from fatigue and pavement deformation. The flexible pavement layer modeling was done in Abaqus. Results indicated that the actual tire imprint area has greater value for stress and strain than all other possible tire contact area. They recommended that using tire imprint area suitable instead of circular, rectangular or ellipse area; and increasing the number of element increase the degree of accuracy.

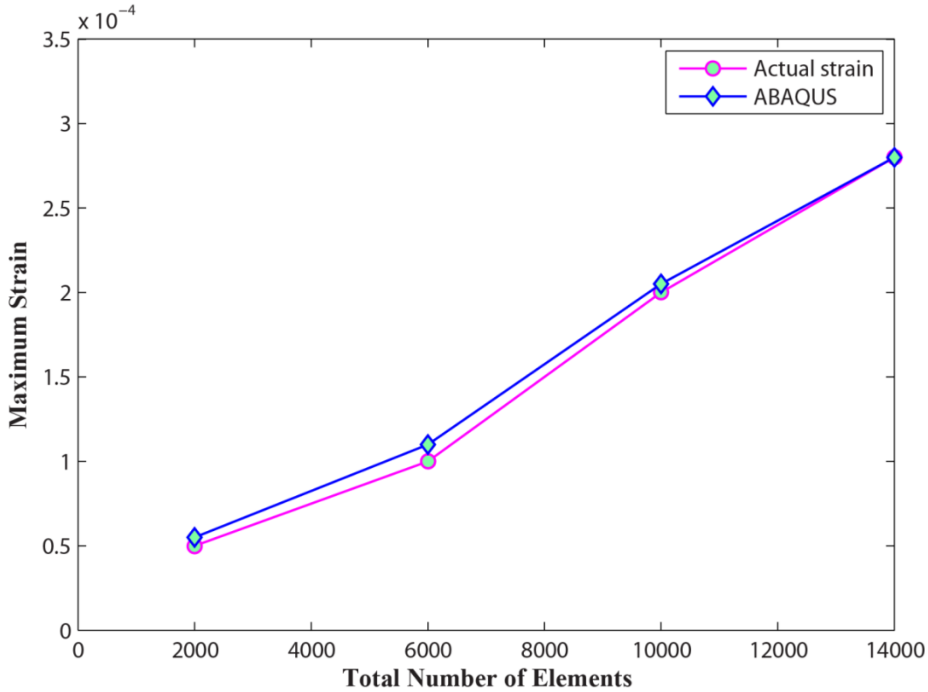


Figure 10.33 Comparison of actual strain variation for different contact area of dual tires in asphalt pavement multi-layers with finite element model analysis in ABAQUS

However, Huang (1993; 2004) suggested to use a rectangular contact area when layered theory used for flexible pavement design. Huang's also discussed that when layered theory is used for flexible pavement design, to assume that each tire has a circular contact area is not correct even if the error is small. Huang's converted two semicircles and a rectangular shape of contact area for each tire to a single rectangle contact area of $0.5227L^2$ and a width of 0.6L (Huang 1993; 2004). Similarly, a single rectangle contact area was used by (Hadi & Bodhinayake 2003; Huang et al. 2011; Mulungye, Owende & Mellon 2007) to simulate tire/pavement surface of flexible pavement using three-dimensional finite element analysis.

Summary of average normal stress and strain variation versus time for different contact area of dual tires/pavement from experimental test and FE-predicted analyses are shown in Figure 10.34 and 10.35, respectively. As it can be seen from comparison of measured and predicted model analysis, the stress (Figure 10.34) increase with the increase of time while strain (Figure 10.35) decrease as time increases and both of them followed a similar trend to one another.

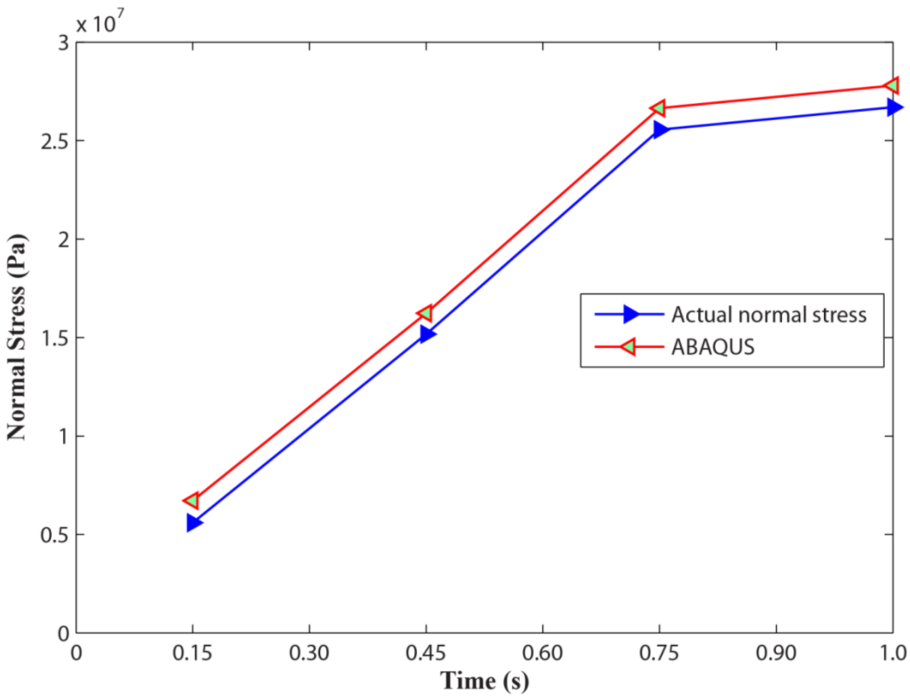


Figure 10.34 Normal stress versus time for asphalt pavement multi-layers

In addition, there is a reasonable agreement with the measured and calculated. It also provides an appropriate and realistic analysis of prediction of stress and strain of layered flexible pavement. Furthermore, stresses and strains from the finite-element analysis were significantly higher than the stresses and strains than that were measured in the laboratory test. This indicates the finite element method is well suited for pavement analysis because of its versatility, and can be used as primary response parameters of pavement. It should be notice from this analysis that stress are determined from incremental strain at each integration points. An accurate correlation of stress-strain curve (Figure 10.36) between experiment test and finite element analysis in Abaqus was also demonstrated in this new developed simulation.

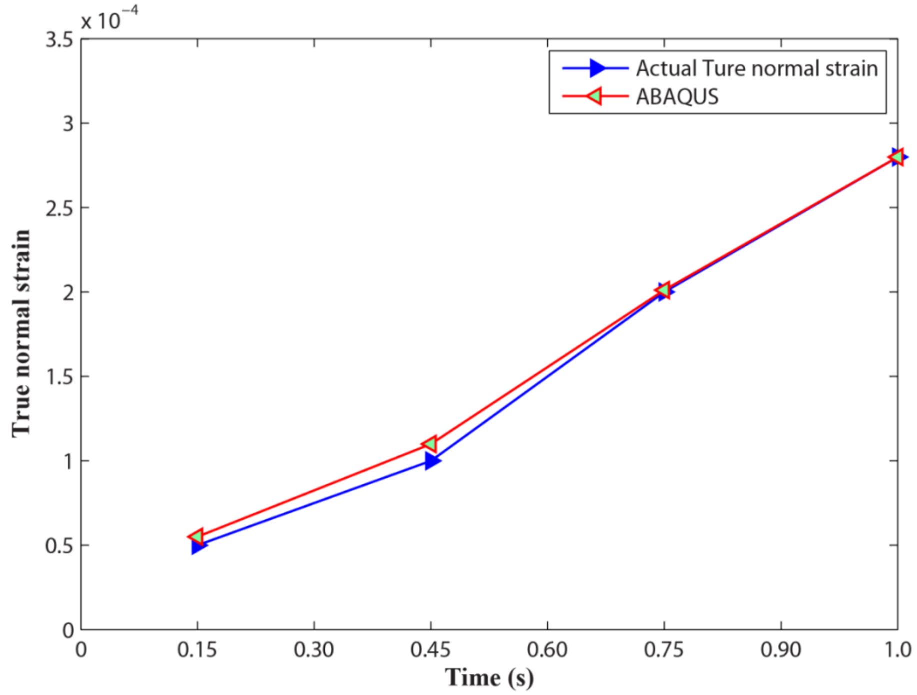


Figure 10.35 Normal strain versus time for asphalt pavement multi-layers

Helwany, Dyer and Leidy (1998) illustrated the usefulness of finite-element method in the analysis of three-layer pavement system subjected to different types of loading. The method was capable of simulating the observed response of pavement subjected to axle loads with different tire pressure loads with different configurations, and axle load travelling at different speed. A variety of material constitutive model as linear elastic (i.e., linear stress-strain behavior), nonlinear elastic (nonlinear stress-strain behavior), and viscoelastic (i.e., effect of viscoelastic behavior) are in the analysis to describe the behavior of pavement materials. Results demonstrated an accurate correlation between the calculated and measured.

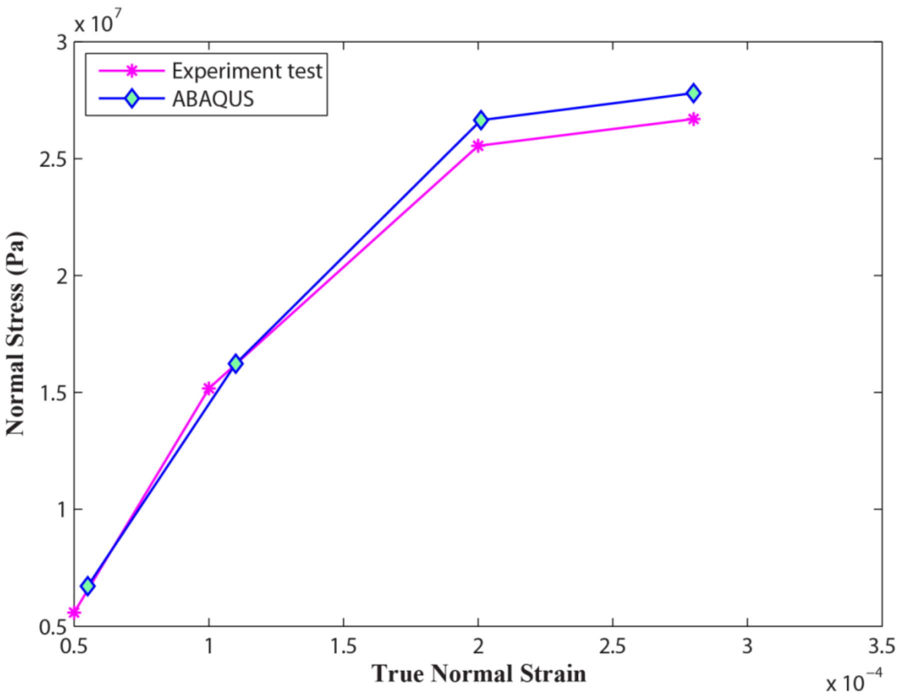


Figure 10.36 Comparison of experimental stress-strain curve for asphalt pavement mulita-layers with finite element model analysis

10.9.4. EFFECT OF TIRE INFLATION PRESSURE ON PAVEMENT SURFACE

Variation in tire inflation pressure changes the size of the contact patch and the tire vertical stiffness. Damage potential may increase by approximately 100% with 140 kPa increase in tire pressure at low pressure (350-490 kPa) approximately 50% at high pressure (490-630 and 630-700/770 kPa) for both single and dual wheels (Mulungye, Owende & Mellon 2007). The tire-pavement pressure distribution is generally known to be complex and affected by tire type. There are many consistencies in the data from various experiment studies measuring the distribution of contact pressure between tire and pavement (Helwany, Dyer & Leidy 1998). Simplifying assumptions have been used in literatures, including the use of a circular contact area with contact pressure equal to the tire pressure.

Figure 10.37 and 10.38 is depict the trend of measured and predicting load repetitions from FE model with respect of stress (Figure 10.37) and strain (Figure 10.37) under the steering wheel, respectively. The values are obtained from nodes (FE-predicting in Abaqus) corresponding to experimental truck. As it can be seen from both analyses, there is 40% increase in stress (Figure 10.37) with the predicting due to varied contact pressure than the contact pressure that was measured in experiment. Both linear and nonlinear relationship exists in the analysis and that indicated the static load application with linear and nonlinear analysis. However strain value (Figure 10:37) does not vary significantly for spatially varied pressure. This showed that when strain is applied as pavement design criteria, failure, N_f or fatigue for wheel load application, uniform contact pressure does not affect the analysis.

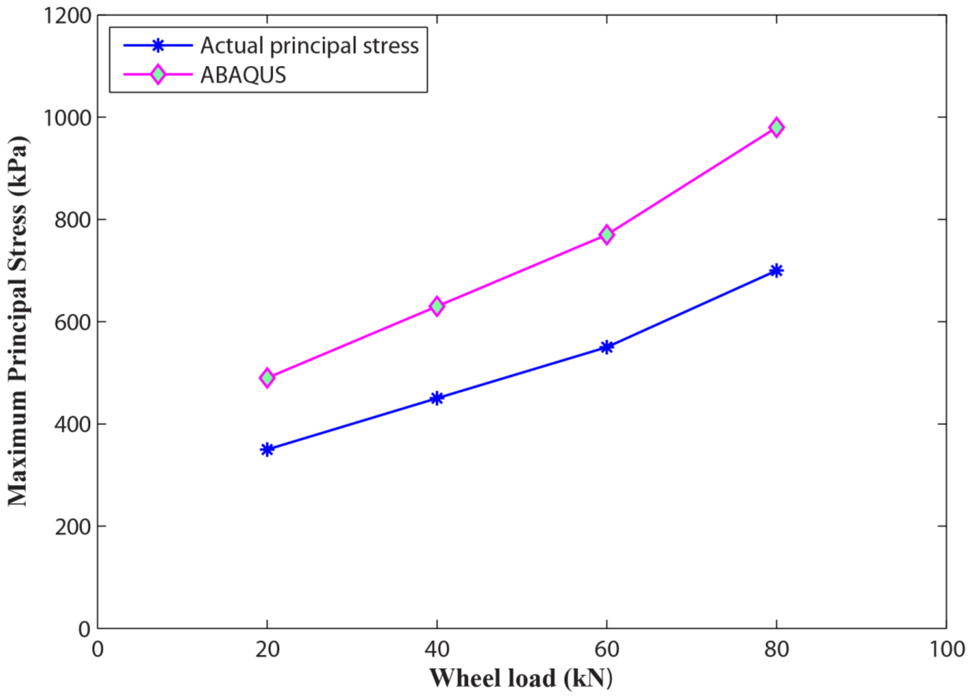


Figure 10.37 Stress comparison for different dual tires inflation pressure of pavement multi-layer with finite element model analysis

It is shown that fatigue life, N_f generally decrease with an increase in tire inflation pressure and the magnitudes of reduction were highest in the 350-490 kPa range. Load repetitions to failure, N_f, with respect of to the lateral strain was predicted to increase 90% on average for the pressure decrease from 490 to 350 kPa and almost 100% for wheel loads equivalent to a fully laden truck, a trend that has also been observed in other available evidence (Gillespie et al. 1992). The impact of tire inflation pressure on fatigue life with respect to longitudinal strain under steering was similar, but at less severe strain influences (higher N_f) (Mulungye, Owende & Mellon 2007).

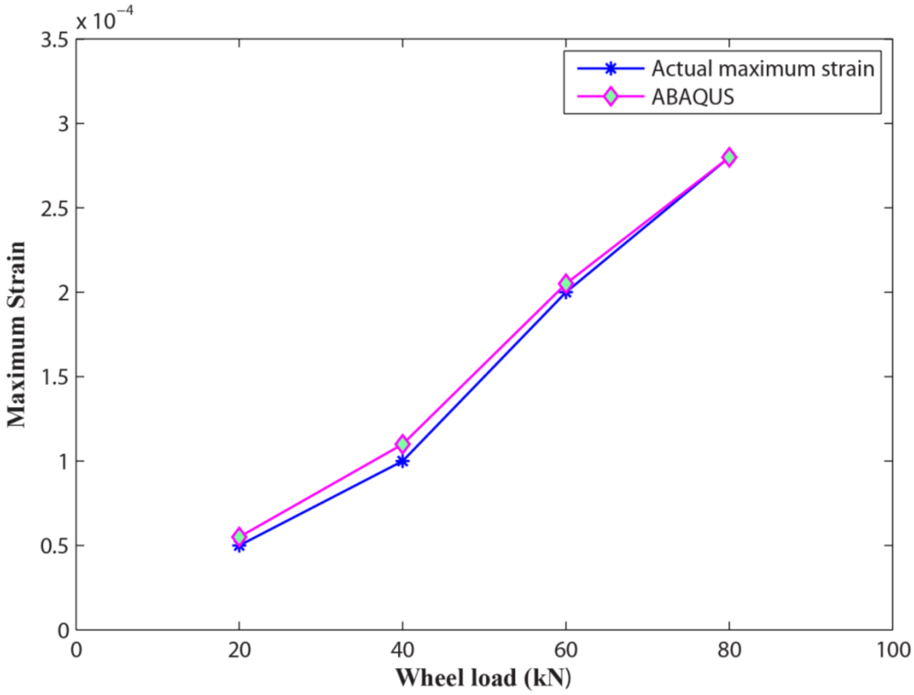


Figure 10.38 Strain comparison for different dual tires inflation pressure of pavement multi-layers with finite element model analysis

The tire-pavement pressure distribution is generally known to be complex and affected tire type. There are many consistencies in the data from various experiments studies measuring the distribution of contact pressure between tire and pavement. Simplifying assumptions have been used in literatures, including the use of a circular contact with contact pressure equal to the tire pressure.

10.10. CONCLUSIONS

The primary response of parameters of pavements of pavement, required a stress and strain characteristics/behavior prediction models, can be analytically evaluated using linear and nonlinear finite element (FE) method. The FE method in ABAQUS is well suited for pavement analysis because of its versatility. This study illustrated the significant of such method in analysis of five-layer pavement subjected to different type of cyclic loading. The method was able to simulate observed response of pavement subjected to wheel loads with different tire inflation pressure and wheel loads travelling with different speeds. This method is integrated with three-dimensional (3-D) FE analysis, which was used to simulate the linear viscoelastic and nonlinear viscoelastic materials of flexible pavement.

A new constitutive model is developed for the stress and strain behavior, and a variety of constitutive models such as a linear elastic, linear viscoelastic and nonlinear viscoelastic were employed in the analysis to describe the effect of stress and strain behavior and/or characteristics in pavement materials. The new developed constitutive model for stress-strain behavior and/o characteristics is capable of considering the effects of stress and strain in pavement layers and predicting of various type of observed in flexible pavement failure and the effects of various pavement design parameters. The implementation of this new developed constitutive model is verified against published results of laboratory test data (Owende et al. 2001; Nega, Nikraz & Al-Qadi 2015a) measured to evaluate pavement layers from creep test and falling weight deflectometer (FWD) test, respectively. This model also compared with other's models studies of pavement layers based on the nonlinear viscoelastic and viscoplastic theory for verification.

This constitutive model is numerically implemented in dynamic nonlinear FE simulation in ABAQUS and results are compared to stress and strain behavior, including displacement using linear viscoelastic and nonlinear viscoelastic. Results has shown 40% reduction of vertical plastic strain of each layers of flexible pavement with nonlinear viscoelastic material as compared with linear viscoelastic material after 7,500 s repeating time of 30,000 cyclic loading. This justified that it is necessary to give a sound consideration to the effect of stress and strain (its complexity) behavior, including nature of displacement in structure of flexible pavement layers and various design parameters because this justification can provide a powerful material assessment and pavement design tool for flexible pavement layers. If the effect of stress and strain complex behavior and /or displacement in pavement layers concept and design has neglected, the risk level of rutting or pavement structure failure or deformation in flexible pavement layer in a pavement contruction might critically increase.

The linear viscoelastic model has low maximum stress range (2.17×10^{-7} to 1.56×10^{6}) as compared to nonlinear viscoelastic model (2.5×10^{-1} to 1.57×10^{6}). This indicate that vertical strain (VE) on the top of the subgrade layer and the tensile strain at the bottom of the asphalt surface in this simulation are about 9% less stiffness as compared to nonlinear viscoelastic analyses. The simulation shows that tensile viscoelastic strain accumulate at the pavement surface (i.e., on linear viscoelastic material) could be the cause for distorted surface as the result of VE on the top of compacted subgrade and a phenomenon that could be associated with cracking of asphalt pavement. In general, the study demonstrated the important of accurate materials characterization on predicting the stress and stress characteristics of flexible pavement layer.

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CHAPTER 11: DISCUSSION, CONCULSIONS AND FUTURE WORK

The focus of this dissertation was to prove and improve the understanding and application of numerical models and experimental techniques of characterization of asphalt mixtures and/or mixes for accurate and appropriate performance of pavement structure. The key feature of the numerical models developed and advanced experimental techniques benchmarked in this dissertation was the ability to develop, validate, and simulate the characterization of asphalt mixtures of flexible pavement. Simulating the asphalt material characterization to construct, sustain, and restore the heavily urban roads where damage has been always induced by traffic loading and environmental factors, and modify the use of full depth asphalt to construct and rehabilitate the heavily loaded urban roads are very important. Since the effect of stress and strain behavior, including displacement of flexible pavement control its functionality and pavement design life. Therefore, numerical models and advance experiment techniques are significant for asphalt pavement to have function and good performance by solving the invalid strategies. The dissertation has shown insight into nine main research questions.

Comparing the finite difference of different types of asphalt mixtures and material characterization of flexible pavement, and determine an appropriate pavement thickness using experimental techniques, and demonstrated an equation that the methods mainly differ in how they spatially variable in material characterization and in how they represent in performance ranking to strength and durability of asphalt pavement is very important. Also, comparing finite difference and finite element solution to two-and three-dimensional and developed a new constitutive models and demonstrated an equations that chosen methods and boundaries of flexible pavement layers are a key solutions for managing the stresses and strain, including the displacement of the road for an accurate and an appropriate of pavement performance, which has been caused by traffic loads and environmental factor (temperature, moisture, ageing…etc and so forth).

Comparing these methods proved that experimental techniques and finite element (FE) numerical modeling incorporate with broader algorithms and various different types of assessment of pavement performance used to evaluate the engineering properties of asphalt mixtures with various types of materials based on engineering principles and pavement material characterization of asphalt mixtures so that two key factors (aggregate graduation and asphalt binders) that influence the engineering properties of flexible pavement can solved. A direct comparison of two specific method finite element method and experimental techniques that were used indicated that the two-and three-finite element algorithm and some specific individual asphalt mixes from the experiments were better to maintain the integrity of the numerical solution when using coarse numerical discretization. Therefore, FE solutions and some asphalt mixes from the experiments can provide an improved to development and validation of characterization of asphalt mixtures of flexible pavement.

This comparison of numerical and experimental techniques has helped alleviate, to some degree, along-standing debate within the variably saturated pavement modeling community. In the past, when researchers have justified their use of either a finite difference or FE discretization, in particular with comparison finite element analysis with laboratory experiment, often the reason cited have been week. For example, Clement, Wise and Molz (1994) suggested with no substantial justification that finite element models were less likely to be accepted by the public as compared to finite difference or laboratory experimental test. Similarly, An opposed to relatively simple pavement layered elastic or inelastic theory, numerical simulation using finite element method (FEM) can be very complex and costly analysis software (Mehta & Roque 2003). This kind of statement should have properly qualified, and has the potential to confuse. It should be acknowledged that there are obvious situations where the choice between a finite element model and laboratory experiment are clear.

Helwany, Dyer and Leidy (1998) acknowledged that finite element modeling of pavement, if validated, can be extremely useful, because it can be used directly to estimate primary response parameters without resorting to field experiment, which may be costly. Similarly, Nega, Nikraz and Al-Qadi (2015b) summarized that finite–element method is well suited for analyzing tire/pavement interaction problems involving material nonlinearities and complex loading and such analysis proceeds by defining the characteristics of each pavement layers, including linear elastic, nonlinear, viscoelastic, plastic and viscoplastic material characterization. With the development of the powerful finite-element method, a break-through was made in the analysis of flexible and rigid pavement (Huang 2004). Cheung and Zienkiewicz (1965) developed finite-element methods for analyzing slabs on elastic foundation of both liquid and solid types.

The outcome of the analysis in Chapter 2 with current move adapted mechanistic-empirical in design of pavement structure and different types of asphalt mixes, state-of-the-are mechanistic methodologies using engineering characterization of hot-mix asphalt (HMA) mixture were analyzed and compared the different types of asphalt mixes in terms of performance and relative advantage of one asphalt mix over another asphalt mixes to determine the best mixes. This work was unique because the different types of asphalt mixes that were produced in the laboratory tests relied upon the entire asphalt mixes (aggregate and asphalt binder) performance based on engineering principles and ethics. For example, HMA can be described as a multiphase heterogeneous material composed of a viscoelastic asphalt binder, irregular rigid aggregate particle in high volume fraction, and small percentage air voids (Gopalakrishnan & Kim 2011) in concise paper defining asphalt mixes. These component materials exhibiting various properties contribute to complex mechanical behavior of HMA, which can be characterized as viscoelasticity and plastic under different condition such as temperature, load application and aging (Dibike et al. 2001). Results showed that AC20-75 and AC14-75 Blow asphalt mixes were more effective and durable in pavement performance as compared to other mixes. They can strength and stable the mixture stiffness of asphalt that is notable, and can be lead to more accurate solutions.

There are obvious possibilities to extend the work presented in Chapter 2. For example, there are other issues not address such as choosing an appropriate mix design (latest), maintenance and management of asphalt mixes. The performance of asphalt mixture is dependent on these components to construct and rehabilitate a heavily loaded urban road including modifying the performance of asphalt mixtures (AASHTO1986a; Austroads 2006; 2008). They discussed that choosing a proper mix design and maintain the entire system for accuracy and an appropriate asphalt mixes design is highly ineffective due to mismanagement of the asphalt mixes, and effective management of the asphalt mixes is also difficult if an asphalt mix is not properly designed. Similarly, several types of moduli that have been used to represent the stiffness of asphalt concrete mixture (Khan et al. 1998) and asphalt pavement materials are an important material property of any mechanistically based on design/analysis procedures of flexible pavement (FHWA2002). This is important for further analysis since asphalt mixes performance is depend on proper and appropriate mix design and maintenance.

The pavement materials characterization of hot-mix asphalt mixes was presented in Chapter 3. The results clearly demonstrated that AC20-75 (20 mm thick dense graded granite under intermediate mix) and AC14-75 (14 mm thick dense graded granite under intermediate mix) asphalt mixes blow again performed significantly better than the other mixes: performing high in tensile strength, resilient modulus, wheel tracking, binder content, Marshall compaction, and air voids content test that were taken to analyses each asphalt mixtures. However, AC14-50 (14 mm thick dense graded granite asphalt concrete under intersection mix), AC7-50 (7 mm thick open graded granite asphalt concrete), and SMA7-50 (7 mm thick granite stone mastic asphalt) blow asphalt mixes were slightly poor in resilient modulus and none of these asphalt mixes tested exceeded the Australian standard limit of 5500 MPa (Australian Standard 2008; Main roads Western Australia 2010). This indicated that these particular mixes might have a partially crusted aggregate that contributed to achieve poor performance in resilient modulus.

Hick and Monismith (1981) noted the resilient modulus of partially crushed aggregate decrease with an increase in fine contents, while the modulus increased for crushed aggregate with increasing in fine content. Also, these asphalt mixes that performed poor in resilient modulus had also high percentage of binder content. And increase in binder contents also reduces the frictional contact of between aggregate particles. Beyond a certain values, further increase in binder content reduce the frictional contact between aggregate particles and the overall stiffness and stability of the asphalt (Austroads 2006). At low percentage, added binder content increase the mix cohesion and strength (Anderson, Walker & Turner 1999).

There are obvious possibilities to extend the work presented in Chapter 3. For instance, there is one further issue not address in detail in this dissertation such as surface treatment and uniform-sized aggregate (Barskale & Itani 1989). Some of the disadvantage of a gravel surface pavement can be eliminated at moderate cost by surface treatment so that the frictional contact between aggregate and mix cohesion during asphalt mix would be strong although the ability to easily regard the surface is lost. A surface treatment is single or multiple application of a liquid asphalt spray followed by spreading and rolling in of a uniform-size aggregate surface treatments during asphalt mixes in quite practical for low volume urban traffic roads. Graded and compacted earth-surface pavement may sometime be used for tire/pavement loading which are light and infrequent, and provide some stability, reduce dust and erosion problem during asphalt mixes.

Chapter 4 presents an evaluation and validation of characterization methods for fatigue performance of asphalt mixes. The results showed that AC20-75 asphalt mix blow again performed significantly better than the other asphalt mixes: performing high in indirect tensile modulus, dynamic creep, wheel tracking, and aggregate gradation test that were taken to analyses each asphalt mixtures. And AC14-75 asphalt mix was the second rank in all performance parameters. This showed that these asphalt mixes are high resistance to permanent deformation or rutting and have low pavement distress. However, AC14-50, AC7-50, and SMA7-50 asphalt mixes blow singingly poor performance in dynamic creep (compression stress) and wheel tracking, and asphalt mixes also had high minimum slope of strain. This indicated that the asphalt mixes are low resistance to permanent deformation or rutting, and also have high pavement stress. There was also a sudden steep change in slope after 8,000 cycles for SMA7-50 blow. This may be attributed to the stripping of aggregate. No stripping was however, observed after 10, 000 cycles to other asphalt mixes. Modification of the asphalt mixes depends on the development and validation of characterization methods that are used for pavement fatigue design life. However, modification rank can be described as AC20-75 Blow > AC14-75 Blow > AC14-50 Blow > AC7-50 Blow > SMA7-50 Blow for all the laboratory experiments.

There are obvious possibilities to extend the work presented in Chapter 4. For example, there are a few further issues not address in detail in this dissertation such as a proper compaction temperature, a specific range of asphalt density, rate of cooling and characterization of indigenous asphalt mixes. For example, low air temperatures, high winds, and night construction create adverse conditions for hot-mix asphalt paving (Chadbourn et al. 1996). This presents a risk for road owners and contractors. To achieve optimum load-bearing and weathering characteristics, an asphalt mix must be compacted to a specific range of density, and the time required for hot-mix asphalt to reach the proper compaction temperature to achieve this density decreases with an increased rate of cooling (Chadbourn et al. 1996). Similarly, there is a strong need for further investigation, initially through a detailed laboratory characterization of indigenous asphalt mixes with conventional, multigrade and modified binders of the effect of volumetric properties, rest period, healing, and ageing, and multi-axle loading effects on fatigue performance (Baburamani 1999; SHRP1994a).

The work in Chapter 5 dealt with a comparison of pavement network management tools and it's probabilistic of pavement engineering. A general discussion and comparison of the fundamental numerical techniques was used to solve the linear model for pavement maintenance and rehabilitation (pavement M&R) relevant to Western Australia roads networks. The analyses present an unbiased comparison of the relative advantages of the linear model base on the Probabilistic Network Markov-Chains Process and Chapman-Kolmogorov optimization pavement management tool with the past 30 years Lao Road Design Manual (LRDM) data inventory. The linear model techniques, and utilizes the comparison results to present a new and improved modelling strategy for simulating pavement M&R shows promise as a research tool for future use. The results showed that the pavement performance of the predicting model using probabilistic network process (i.e. Linear) perform well in all categories as compared to the past 30 years LRDM data inventory. This work is a novel and will draw into appropriate and effective pavement engineering management system to account for proper pavement design, preliminary planning, future pavement M & R networks, service life and functionality. Some of the limitations and short coming of Chapter 2, in particular, pavement maintenance and management of asphalt mixes can be solved with Chapter 5.

The dynamic (complex) moduli relationship related mixture moduli to temperature and time rate of loading has been an integral part of several mechanistic-empirical (M-E) design procedures against a laboratory-scale model is presented in the following. The developing master curves, binder viscosity, and predicting dynamic modulus of polymer-modified asphalt mixtures was presented for the first time in Chapter 6. Seven asphalt concrete mixtures of different types of polymer modified binders (PMB) were produced in a laboratory to analyze the influence of polymer modifiers on a pavement performance of asphalt mixtures with the dynamic (complex) modulus $|E^{*}|$ of hot-mix asphalt (HMA) mixtures. The governing equation and analytical model were used as a part of the analysis of the laboratory data. The influence of temperature, loading frequency, and confining pressure on the dynamic characteristic of asphalt mixture were analyzed; master curves of dynamic modulus of HMA mixtures were developed and data's were interpreted.

Results showed that AC10 5.7% A35P (EVA) M7 B5, AC10 5.7% C450 M10 B5 and AC10 Multi 600/700 M5 B4 mixes method were the more efficient and effective in all categories of asphalt performance measures for strength and durability of HMA as compared to others polymer modifiers. A very good correlation (R^2 = 1) was found for each polymer modifier and a strong correlation between binder viscosity and temperature [R^2 = 1] for polymer modified asphalt mixture. This work was unique because it suggested that laboratory test using a various temperatures and loading frequencies can improve pavement mix design, lab and field control and assurance. For example, The Connecticut Department of Transportation (CDOT) assessed the modulus, E*, as a test method to characterize hot-mix asphalt mix design as the part of 2002 Pavement Design Guide. Results has shown a good correlation [R^2 = 0.9997] between binder viscosity -temperature (Dougan et al. 2003). Similarly, Rasmussen, Lytton and Chang (2002) has plotted VTS and found a [R^2 = 0.999].

Chapter 7 presents a pavement distress identification, cost analysis, and pavement temperature prediction for long-term pavement performance. Collecting and analysis of pavement distress data is a significant component for effective long-term pavement design life. Accurate, consistent, and repeatable pavement distress type's evaluation can reduce a tremendous amount of time and money that has been spending each year on maintenance and rehabilitation of existing pavement distress. This study conducted a field survey and collected a pavement distress data on significant components for effective long-time pavement performance. Governing equation and non-linear model: Probabilistic Network Markov-Chain Process method and the Statistical Downscale Model (SDSM) model were used to predict the cost analysis, asphalt surface pavement distress, and pavement temperature for individual asphalt concrete surface pavement distresses The model used to describe a new procedure in order to assess the worthiness of test problems to be used as a benchmark for pavement management system (PMSs) for Mainroads Western Australia. Results clearly demonstrated that the Markov Chain process (non-linear model) approach and the statistical downscale (SDSM) model can be used to evaluate the pavement temperature for long-term pavement performance. It is highly recommended to use a systematic and scientific approach to maximum benefits and minimize overall costs so that long-term pavement performance will be achieved. Some of the limitations and short comings that were mentioned in Chapter 2 and 3 can be solved with Chapter 7.

The work in Chapter 8 dealt explicitly with simulation of shakedown behavior for flexible pavement's unbound granular layer (UGL) using finite element method in ABAQUS and MATLAB. The eighth chapter focuses upon the limitation problem of full depth asphalt concrete pavements that are generally designed to control fatigue cracking and reduce potential rutting when subjected to repeat heavily traffic roads. A new constitutive model was developed for shakedown effect, pavement distress, impact of temperature and loading frequencies in pavement (analytical and the non-linear numerical model problem) in this chapter. To achieve this purpose, a new constitutive model based on Mohr-Coulomb criterion and Drucker Plager method for flexible pavement of unbound granular material (UGM) was developed.

This method (Chapter 8) was novel because it integrated with Mohr-Coulomb Criterion, which is used and applied to simulate the response of unbound granular layer to dynamic loading in a numerical analysis. Analyses showed that the new developed constitutive model is capable of considering shakedown effect in base UGL and predicting the various flexible pavement failures (such as pavement distress problem in Chapter 7) and effect of various design parameters (such as impact of temperature and loading frequencies in pavement in Chapter 6). The implementation of this new developed constitutive model is verified against published results of laboratory test data measured shakedown for UGM and was also coded in UMAT under ABAQUS simulation. Results has also shown 50% reduction of vertical plastic strain of the base (UGL) layer with shakedown model as compared to Mohr-Coulomb after 7 s repetion of cyclic loading. This work was unique since the numerical solutions were compared to results against the laboratory data that was done by Habiballah and Chazallon (2005). In addition, constitutive model was also compared to Chazallon et al. (2009) and Ghadimi, Nega and Nikraz (2015) constitutive models that were evaluated for this work.

Dynamic analysis of falling weigh deflectometer test results for strength of character of pavement layer moduli were presented in Chapter 9. Falling weight deflectometer (FWD) testing has extensively been done in the past to evaluate structural condition for consistency and assurance of existing pavement structure based on engineering principles and also determine the layer moduli of flexible pavement. The moduli can be input into semi-empirical mechanistic equation to estimate the remaining life of the pavement system. FWD measurements were taken from 7 locations of traffic roads including core data, roadway data and pavement distress information. Governing equation and BISDEF and ELSYM5 computer software were used to analysis backcalculation moduli based on FWDs data. In addition, This dissertation also validate the backcalculated moduli theoretically and through instrumental response on top of asphalt pavement mixtures because FWD data interpretation has increasingly become challenging from time to time as result of repeated heavily traffic urban roads.

The work in Chapter 9 was also considered the inertia force in the pavement structure analysis using the FWD test; and the density in site was generally around 2400 kg/m^3, the roadbase around 2150 kg/m^3, and the subbase around 1850 kg/m^3. These densities were at Marshall or maximum dry density. Generally, the asphalt was compacted to 97% of maximum dry density, the roadbase to around 96% of maximum dry density (MDD), and the subbase to around 96% of MDD according to Australian standard in 2003 (Australian Standard 2003). Furthermore, this work was unique since subgrade was modeled as a finite-thickness, homogenous, linear-elastic layer placed on top of a bedrock and dynamic deflection basin were obtained as by computed the deflection at the sixteen geophone locations using a variable subgrade depth with an average modulus of 236 MPa and 40 kN loads distributed over area of (0, 200, 300, 400, 600, 750, 900 and 1500 mm). The thickness of the subgrade used was (0-3 m) with a Poisson's ratio of 0.3. The deflections obtained for each model were normalized to 700 kPa.

Finally, Results were compared to WESLEA (WES), Finite Element Method (FEM), and Method of Equivalent Thickness (MET). The results clearly demonstrated that the flexible pavement layer moduli and algorithms for interpretation of the deflection have improved. The variation of the moduli of all layer along the length of sections for majority of the roads were accurate and consistence with measured and computed predicting. Although some of the projected roads had some inconsistence in modulus values along the length of section but results are reasonable. The influence lines for vertical strain at top of subgrade as measured by gauges and predicted by different types of response (i.e., WES, FEM, and MET) also showed that the MET is seen the best predictor as compared to WES and FEM.

Finally, the tenth chapter focuses on multilayer solutions that calculate stresses, strains, and displacement in flexible pavement structures that have been caused by traffic surface loading. Most elastic layer computer software applications have been developed for mainframe computers. A numerical analysis technique has been used in order to obtain a various structural parameters such as stresses, strains and deflections or displacements of flexible pavement layers using three-dimensional finite element analysis in ABAQUS. This dissertation (Chapter 10) describes the multilayer solution that calculates stress, strain and displacement in flexible pavement structure caused by traffic surface loading that have been a problem in existence for several decades.

The work in Chapter 10 dealt explicitly with new numerical simulation for stresses and strains characteristics in flexible pavement using linear and nonlinear (three-dimensional) finite element analysis in ABAQUS. The primary response of parameters of pavements of pavement, required a stress and strain characteristics/behavior prediction models, can be analytically evaluated using linear and nonlinear finite element (FE) method. The FE method in ABAQUS is well suited for pavement analysis because of its versatility. This dissertation illustrated the significant of such method in analysis of five-layers pavement subjected to different type of cyclic loading. The method was able to simulate observed response of pavement subjected to wheel loads with different tire inflation pressure and wheel loads travelling with different speeds. This method was also integrated with three-dimensional (3-D) FE analysis, which was used to simulate the linear viscoelastic and nonlinear viscoelastic materials of flexible pavement.

A new constitutive model is developed for the stress and strain behavior, and a variety of constitutive models such as a linear elastic, linear viscoelastic and nonlinear viscoelastic were employed in the analysis to describe the effect of stress and strain behavior and/or characteristics in pavement materials. This work was unique and novel because the new developed constitutive model for stress-strain behavior and/o characteristics is capable of considering the effects of stress and strain in pavement layers and predicting of various type of observed in flexible pavement failure and the effects of various pavement design parameters. The implementation of this new developed constitutive model is verified against published results of laboratory test data (Owende et al. 2001; Nega, Nikraz & Al-Qadi 2015a) measured to evaluate pavement layers from creep test and falling weight deflectometer (FWD) test, respectively and was also coded in UMAT under ABAQUS simulation. This model also compared with other’s models studies of pavement layers based on the nonlinear viscoelastic and viscoplastic theory for verification.

This constitutive model in Chapter 10 is numerically implemented in dynamic nonlinear FE simulation in ABAQUS and results are compared to stress and strain behavior, including displacement using linear viscoelastic and nonlinear viscoelastic. Furthermore, the linear viscoelastic model (Chapter 10) has low maximum stress range (2.17 x 10^{-7} to 1.56 x 10^{6}) as compared to nonlinear viscoelastic model (2.5 x 10^{-1} to 1.57 x 10^{6}). This indicate that vertical strain (VE) on the top of the subgrade layer and the tensile strain at the bottom of the asphalt surface in this simulation are about 9% less stiffness as compared to nonlinear viscoelastic analyses. The simulation shows that tensile viscoelastic strain accumulate at the pavement surface (i.e., on linear viscoelastic material) could be the cause for distorted surface as the result of VE on the top of compacted subgrade and a phenomenon that could be associated with cracking of asphalt pavement. In general, the study demonstrated the important of accurate materials characterization on predicting the stress and stress characteristics of flexible pavement layer.

Results has also shown 40% reduction of vertical plastic strain of each layers of flexible pavement with nonlinear viscoelastic material as compared with linear viscoelastic material after 7,500 s repeating time of 30,000 cyclic loading. This justified that it is necessary to give a sound consideration to the effect of stress and strain (its complexity) behavior, including nature of displacement on pavement structure of flexible pavement layers and various design parameters because this justification can provide a powerful material assessment and pavement design tool for flexible pavement layers. If the effect of stress and strain complex behavior and /or displacement in pavement layers concept and design has neglected, the risk level of rutting or pavement structure failure or deformation in flexible pavement layer in a pavement contruction might critically increase. In general, some of the limitation and short coming of (Chapter 2-4 and Chapter 9) can be solved using Chapter 10.

This dissertation has enabled further insight into complex three-dimensional nature of characterization of asphalt mixes and flexible pavement analysis and design. The basic numerical and laboratory tools used in the study have provided a significant and novel contribution to the understanding of characterization of asphalt mixes and flexible pavement design. It is hoped that the contribution to understanding from the this dissertation shall be integrated into future body of knowledge so that some of the problems associated with a poor understanding of characterization of asphalt mixes and appropriate design of flexible pavement including pavement management tool and pavement distress with the current finite element method in ABAQUS at present time can be alleviated.

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