Mechanics-based Design Framework for Flexible Pavements

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Doctoral Thesis

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Abstract

Load induced top-down fatigue cracking has been recognized recently as a major distress phenomenon in asphalt pavements. This failure mode has been observed in many parts of the world, and in some regions, it was observed to be more prevalent and a primary mode of pavements failure. The analysis and design tools which are currently used to evaluate this failure mode are mainly empirical in nature and do not account properly uncertainties and variabilities effect on performance. The unavailability of effective methods has made it difficult to control and mitigate this failure mode. This paper presents a mechanics-based design framework in load and resistance factor design (LRFD) format for the top-down fatigue cracking performance evaluation of flexible pavements. This was achieved by enhancing further the hot mix asphalt fracture mechanics (HMA-FM) model through the incorporation of mixture morphology effect on key fracture properties. The partial safety factors of the various target reliabilities were formulated using a reliability analysis methodology which utilizes the first order reliability method (FORM).

Asphalt mixture morphology-based models were developed and incorporated into the mechanics-based analysis framework to characterize and evaluate aging effect on fracture energy and healing potential. These models were developed empirically exploiting the observed relation that exist between mixture morphology and these properties. The framework was calibrated and validated using pavement sections that have high quality laboratory data and well documented field performance histories. The calibration was performed on the healing potential model of the framework. The calibrated framework has been observed to predict crack initiation (CI) times which correspond well with observed performances in the field. As traffic volume was identified in having a dominant influence on predicted performance, a further investigation was performed to establish and evaluate truck traffic characterization parameters effect on predicted performance. Influence of parameters such as traffic growth rate, axle load spectra, volume adjustment factors and lateral wheel wander were investigated to establish the significance of these parameters on predicted performance.

The LRFD mechanics-based design framework was achieved by performing a reliability calibration on the mechanics-based analysis framework. The failure criterion and design period of the design framework were established by carefully evaluating the performance histories of a number of field pavement sections. Furthermore, pavement sections which have various design target reliabilities and functional requirements were used for the reliability calibration. The reliability analysis was achieved by implementing a two component reliability analysis methodology, which uses central composite design (CCD) based response surface approach for surrogate model generation and the FORM for reliability estimation. The effectiveness of the LRFD mechanics-based design framework was investigated through design examples and the results have shown clearly that the formulated partial safety factors have accounted effectively the variabilities involved in the design process. Further investigation was performed to establish the influence design inputs variabilities have on target reliabilities through case studies that combine input variabilities in a systematic way. It was observed from the results that the coefficient of variation (COV) level of the variability irrespective of the distribution type used have a significant influence on estimated target reliability.

Key Words

Mechanics-based, asphalt, fatigue, reliability, traffic, variability

Sammanfattning

Lastinducerade utmattningssprickor från ytan har nyligen identifierats som en betydande brottmekanism i asfaltbeläggningar. Denna brottyp har observerats i många delar av världen, i vissa regioner noterades det vara mer utbrett och en primär orsak till brott i vägar. Konstruktions- och analysverktygen som för närvarande används för att utvärdera denna brottyp är huvudsakligen av empirisk karaktär och tar inte hänsyn till effekter av osäkerheter och variabilitet på prestanda. Bristen på effektiva metoder har gjort det svårt att kontrollera och mildra denna brottyp i konstruktionsprocessen. Denna avhandling presenterar ett mekanik-baserad ramverk i last- och motståndsfaktorer konstruktionsformat (LRFD) för utvärdering av utmattningssprickor från ytan för asfaltsbeläggningar. Detta uppnåddes genom att ytterligare förbättra brottmekanikmodellen för varmblandad asfalt (HMA-FM) genom integreringen av blandingens morfologieffekt på brottegenskaper. De partiella säkerhetsfaktorerna för de olika tillförlitligheterna formulerades med hjälp av en tillförlitlighetsanalys där första ordningens tillförlitlighetsmetod (FORM) används.

Modeller baserade på asfaltens morfologi har utvecklats och integrerats i HMA-FM för att karakterisera effekten av åldrande och nedbrytning på brottenergi och läkandepotential. Dessa modeller har utvecklats empiriskt där den observerade relation som finns mellan en blandnings morfologi och dessa egenskaper utnyttjas. Ramverket kalibrerades och validerades med hjälp av vägar med säkerställda egenskaper och väldokumenterad prestanda. Kalibreringen utfördes på ramverkets modell för läkningspotential. Det kalibrerade ramverket har observerats förutse tider för sprickinitiering som stämde väl överens med utförande i fält. Eftersom trafikvolymen identifierades att ha ett dominerande inflytande på förväntad prestanda, utfördes vidare en utredning för att fastställa effekten av lastbilstrafikens karakteriseringsparametrar på förutsedd prestanda. Inverkan av parametrar såsom trafiktillväxt, axellastspektrum, volymjusteringsfaktorer och hjulens lateralförflyttning undersöktes för att fastställa signifikansen av dessa parmetrar på den förutsedda prestandan.

Det mekanik-baserade konstruktionsramverket utvecklades genom en tillförlitlighetskalibrering på det mekanik-baserade analysramverket. Brottkriteriet och den förväntade livstiden för konstruktionsramverket fastställdes genom att noggrant utvärdera dokumenterad prestanda för ett antal vägsektioner. Vidare har vägsektioner med olika förutbestämda pålitlighet samt funktionskrav använts för tillförlitlighetsberäkningarna. Tillförlitlighetsanalysen uppnåddes genom att implementera en tvåkomponents tillförlitlighetsanalys där en metod som kallas central composite design (CCD), vilken skapar en surrogatmodell, och FORM för beräkning av tillförlitligheten. Effektiviteten i det mekanik-baserade konstruktionsramverket undersöktes genom konstruktionsexempel och resultaten har tydligt visat att de formulerade partialkoefficienter effektivt har beaktat variabiliteten inom konstruktionsprocessen. Ytterligare undersökningar utfördes för att fastställa påverkan som indatas variabilitet har på den förutbestämda tillförlitligheten genom fallstudier där indatans variabilitet ändras systematiskt. Det observerades från resultaten att variabilitetens variationskoefficient (COV) har en signifikant inverkan på tillförlitligheten, oberoende av fördelningstyp.

Nyckelord

Mekanik-baserade, utmattningssprickor, pålitlighet, lastbilstrafiken, variabilitet

Preface

The work presented in this thesis was carried out between February 2012 and December 2016 at the Department of Civil and Architectural Engineering at KTH Royal Institute of Technology under the supervision of Prof. Björn Birgisson.

I would like to express my deepest gratitude to my supervisor Prof. Björn Birgisson for his guidance, feedbacks and encouragements throughout the project period and without which this project might not be realized. I would also like to express my sincere appreciation to Prof. Stefan Larsson and Assoc. Professors Nicole Kringos and Nils Ryden for their valuable support and encouragement.

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Yared Hailegiorgis Dinegdae

Stockholm, December, 2016

List of appended papers

Paper I

Dinegdae, Y., Onifade, I., Jelagin, D., & Birgisson, B., 2015. Mechanics-based top-down fatigue cracking initiation prediction framework for asphalt pavements, *Road Materials and Pavement Design*, 16(4) pp. 907-927

Paper II

Dinegdae, Y., & Birgisson, B., 2016. Effects of truck traffic on top-down fatigue cracking performance of flexible pavements using a new mechanics-based analysis framework, *Road Materials and Pavement Design*, DOI 10.1080/14680629.2016.1251958

Paper III

Dinegdae, Y., & Birgisson, B., 2015. Reliability-based calibration for a mechanics-based fatigue cracking design procedure, *Road Materials and Pavement Design*, 17(3) pp. 529-546

Paper IV

Dinegdae, Y., & Birgisson, B., 2016. Design inputs variabilities influence on pavement performance reliability, the 4th Chinese European Workshop-Functional Pavement Design, Delft, the Netherlands, ISBN 978-1-138-02924-8

Related publications

Dinegdae, Y., 2015. Reliability-based design procedure for flexible pavements, *Licentiate thesis*, KTH Royal Institute of Technology, Stockholm, Sweden, ISSN 1650-951X

Dinegdae, Y., & Birgisson, B., 2016. Reliability-based design procedure for fatigue cracking in asphalt pavements, *Transportation Research Record: Journal of the Transportation Research Board*, No. 2583 pp. 127-133

Dinegdae, Y., & Birgisson, B., 2016. Effect of heavy traffic loading on predicted pavement fatigue life, *8th RILEM International Conference on Mechanisms of Cracking and Deboning in Pavements*, v.13, pp.389-395

Onifade, I., **Dinegdae, Y.,** & Birgisson, B., 2016. Hierarchical approach for fatigue cracking performance evaluation in asphalt pavements, *the inaugural meeting of the Transportation Research Congress* (TRC), Beijing, China

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List of acronyms

AADTT	Annual Average Daily Truck Traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
AFOSM	Advanced First Order Second Moment
ALS	Axle Load Spectra
CCD	Central Composite Design
CDF	Cumulative Distribution Function
CI	Crack Initiation
COV	Coefficient of Variations
DCSE	Dissipated Creep Strain Energy
DDBEM	Displacement Discontinuity Boundary Element Method
ER	Energy Ratio
ESAL	Equivalent Single Axle Loads
FDOT	Florida Department of Transportation
FM	Fracture Mechanics
FOS	Factor of Safety
FORM	First Order Reliability Method
FOSM	First Order Second Moment
HDF	Hourly Distribution Factor
HMA	Hot Mix Asphalt
LRFD	Load Resistance Factor Design
LTPP	Long Term Pavement Performance
LWW	Lateral Wheel Wander
MDF	Monthly Distribution Factor
M-E	Mechanistic Empirical
MEPDG	Mechanistic Empirical Pavement Design Guide
PDF	Probability Density Function

PS	Primary Structures
PEM	Point Estimate Method
R-F	Rackwitz- Fiessler
VAF	Volume Adjustment Factors
VC	Vehicle Classification
WIM	Weigh in Motion

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1. Introduction

Pavement system is a layered structure that provides a smooth riding surface for vehicular transportation while protecting the underlying subgrade from excessive stress. Flexible pavements, which are built placing a thin hot-mix-asphalt (HMA) layer over granular base materials, are the most common type of structure in Sweden. These layers are made from materials which depending on the rate of loading, prevailing temperature and moisture conditions and stress level exhibit complex behaviour. Moreover, these properties deteriorate with age and also exhibit spatial variability which makes it very difficult to accurately capture theirs influence on pavement performance. In this era of global warming and economic uncertainty, more demand is being placed on pavement design specifications in addition to the adequacy required regarding structural capacity. A pavement design guide which is developed on the basis of fundamental material behaviour and relationships and which also takes into account the uncertainty involved in the design process is required for the challenges ahead. These guides should also be coupled with tools that perform life cycle cost analysis, life cycle assessment and risk analysis in order to deliver an optimum pavement section.

1.1 Background

Pavement analysis and design has been performed traditionally following empirical approaches. These empirical methods were developed based on test sections that were constructed at specific geographic locations with selected types of materials and structures. In addition, these empirical design equations were derived subjecting the test sections with a traffic repetition which was much lower than the total traffic which would normally be expected during a pavement design period. The American Association of State Highway Officials (AASHO) road test was a typical example (AASHO, 1962). The AASHO road test was the basis for the development of the subsequent American Association of State Highway and Transportation Officials (AASHTO) design guides (AASHTO, 86, 93). These empirical methods are insufficient for the challenges of today as current design conditions are very different from the original test sections upon which these methods were derived. Moreover, pavement performance in these empirical methods was measured using subjective criteria which are mainly obtained from users' experience and empirical data. The use of empirical methods for pavement design purpose undermines the development of new tools and models which can be used to characterize and evaluate pavement materials and subsequently pavement performance. New pavement design approaches that are developed on the basis of fundamental material behaviour and which also take into account the spatial and temporal variations of material properties are needed.

The mechanistic empirical pavement design guide (MEPDG) and other similar methods were developed to address the short comings that exist in empirical design approaches (ARA Inc., 2004; WSDOT, 2011; MnPAVE, 2011; PMS Object, 2008). These methods compute the pavement response at critical locations through principles of engineering mechanics while modelling the pavement layers with either linear elastic analysis or finite element method. Pavement performance is evaluated in these methods through failure modes that capture the actual phenomena. In addition, these mechanistic empirical (M-E) methods have included many new variables to characterize material properties, climate conditions and traffic inputs and account for factors such as aging and reliability. Even if M-E methods present a paradigm shift in pavement analysis and design, there are still some issues which need to be addressed and resolved. One of the issues is the way pavement performance is evaluated, which is based on empirical transfer functions that relate the critical strain or stress to the number of loading cycles to failure. These empirical functions are developed without considering the actual failure mechanism and require an extensive amount of data for model calibration and validation, which would delay the timely implementation of new material models and design tools. In addition, reliability is incorporated mainly through empirical approaches that do not propagate the uncertainties involved in the design process.

The development of mechanics-based design approaches that eliminate the empiricism associated with the existing design tools is the main focus of current research efforts. These design approaches determine pavement response and damage accumulation on the basis of fundamental material behaviour and material mechanics. The hot mix asphalt fracture mechanics (HMA-FM) which uses the critical condition concept and an energy threshold criterion to evaluate asphalt mixtures performance is a good example (Roque et al., 1999). The HMA-FM model was incorporated into a top-down cracking design tool for asphalt pavements through a parameter termed energy ratio (ER), which was observed to successfully distinguish cracked pavements from those that did not crack (Wang et al., 2007). Nevertheless, the ER method did not account effects of factors such as aging and healing properly, and reliability was incorporated through an empirical approach. Zou and Roque (2009) have further enhanced the HMA-FM model with the incorporation of material models that characterize asphalt mixtures damage and fracture properties in a better manner (e.g., AC stiffness, fracture energy, healing potential, tensile strength). Further research on asphalt mixtures fatigue resistance has shown that mixture morphology, which governs and controls aging characteristics, can play a critical role in AC pavements long-term cracking performance (Onifade et al., 2013 and Kumar Das et al., 2013). This thesis further enhances and develops the HMA-FM into a mechanics-based analysis framework through the incorporation of mixtures morphology influence on fracture and damage properties. Furthermore, a reliability calibrated deterministic design approach for the fatigue cracking performance evaluation of asphalt pavements is examined.

The development of a mechanics-based analysis and design framework for pavement performance evaluation will provide many benefits and advantages to the pavement industry. In addition to optimizing pavement sections for structural, economic and environmental conditions, these analysis and design methods are expected to provide the following benefits:

- The evaluation of pavement performance using fundamental material behaviour and material mechanics can facilitate the development and timely implementation of pavement analysis and design tools, which fosters innovation in the pavement industry.
- The incorporation of asphalt mixture properties in the design process provides the opportunity to better utilize available materials and the introduction of novel materials.
- The accurate prediction of pavement performances allows the implementation of pavement management strategies that facilitate timely intervention during maintenance requirements.

1.2 Research objectives and scope

The primary objective of this PhD thesis was to develop a mechanics-based design framework for the top-down fatigue cracking performance evaluation of asphalt concrete (AC) pavements. The main tasks were the following:

- Developing a mechanics-based analysis framework for the top-down fatigue cracking performance evaluation of AC pavements.
- Evaluate the significance of truck traffic characterization parameters on top-down fatigue cracking performance of AC pavements.
- Developing a reliability-calibrated design procedure for the top-down fatigue cracking evaluation of AC pavements
- Investigate input parameters variabilities impact on estimated target reliability

As the main focus area of this research was to develop a mechanics-based design framework for the top-down fatigue cracking performance evaluation of flexible pavements, other distress modes such as rutting, bottom-up fatigue cracking and thermal cracking were not included. The new material models were developed empirically exploiting the observed relation that exist between parameters of interest.

2. Pavement performance evaluation

A literature review of previous studies which are considered essential in the development of a reliability-based design framework for the top-down fatigue cracking performance evaluation of asphalt concrete pavements is presented in the subsequent sections. The main focus of the survey was the topics of top-down fatigue cracking, truck traffic characterization, reliability methods, pavement reliability and pavement design inputs variabilities.

2.1 Top-down fatigue cracking

Load-induced fatigue cracking is one of the major failure modes considered in the design of flexible pavements. It is caused by the repeated application of traffic loading and manifested as longitudinal cracking along the wheel path. The widely accepted assumption is that fatigue cracking normally initiates at the bottom of the AC layer and propagates further into the surface (i.e. bottom-up fatigue cracking) due to bending induced flexural tensile stresses. Therefore, most mechanistic-empirical (M-E) models estimate the fatigue life of asphalt pavements using the tensile strain at the bottom of the bound layer. A general pavement fatigue life prediction model is presented as follows:

$$N_f = Ck_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \left(\frac{1}{E} \right)^{k_3} \tag{1}$$

where N_f number of repetitions to fatigue cracking, ε_t tensile strain at the bottom of the AC layer, E stiffness of the material, k_1, k_2, k_3 laboratory regression coefficients and C laboratory to field adjustment factor.

It is now well recognized that load induced top-down fatigue cracking where cracking initiates at the surface of the AC layer and propagates downward commonly occurs in flexible pavements. This phenomenon has been observed in many parts of the United States as well as in places such as China, Japan, and Europe. In places like Florida, USA, top-down fatigue cracking has been reported to be more prevalent, accounting almost 90% of all the fatigue cracking failures. Once initiated, top-down fatigue cracks widen up during downward propagation which allows water infiltration into the underlying layers, weakening the pavement structure and eventually causing structural failure. The presence of top-down fatigue cracking also contributes to the increase in surface roughness which causes reduction in pavement serviceability.

The failure mechanism of top-down fatigue cracking cannot be explained by the traditional approach which was used to explain bottom-up fatigue cracking. The unavailability of models which are developed on the basis of the actual mechanism that result in this type of cracking has made it difficult to control effectively this failure mode in the design process. Researchers have tried to develop hypotheses which explain potential mechanisms and key factors that result in the development of top-down fatigue cracking (Collop & Roque, 2004; Mollenhauer & Wistuba, 2012; Myers, Roque, & Ruth, 1998; Wang et al., 2003; Zou, Roque, & Byron, 2012). Experimental investigations were also carried out to identify key mixture properties which can be used to evaluate the susceptibility of HMA mixtures to this kind of failure (Baek, Underwood, & Kim, 2012; Chen et al., 2012; Roque, Zhang, & Sankar, 1999). Analytical preliminary models were also developed that have a potential to predict the initiation and propagation of top-down fatigue cracks (Myers and Roque, 2002; Roque, Zhang, and Sankar, 1999; Yoo and Al-Qadi, 2008). Nevertheless, not that much have been done to evaluate and validate these hypotheses, test methods and predictive models. Models which can be used to analyse and design pavement sections for top-down fatigue cracking have been also developed and incorporated into design guides.

2.1.1 MEPDG surface-down fatigue cracking model

One of the structural distresses considered in the mechanistic-empirical pavement design guide (MEPDG) was surface-down fatigue (longitudinal) cracking. The mechanisms which are attributed in the design guide for causing this type of distress are surface tensile stresses and strains which are induced due to wheel load, and shearing near the edge of tire from radial tires with high contact pressure. Severe aging of the asphalt mixture in combination with high contact pressure has also been suggested as one possible mechanism (ARA Inc., 2004). The design guide considers surface-down fatigue cracking through the incorporation of a preliminary model that relates surface tensile strain which is induced due to the combined effects of load and age hardening with fatigue life. The model was observed to have an excellent agreement with field performances and was calibrated using pavement sections from the long term pavement performance data base (LTPP). The fatigue cracking prediction is normally performed using the cumulative damage concept suggested by Miner's as follows:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \tag{2}$$

where D damage, T total number of periods, n_i actual traffic for period i and N_i allowable failure repetitions for the same period.

Roque et al. (2011) evaluated the top-down fatigue cracking performance of a number of field pavement sections using the predictive model incorporated in the MEPDG and reported that the model predicts crack initiation times which are much longer than the observed performances in the field.

2.1.2 Energy ratio (ER) method

The hot mix asphalt fracture mechanics (HMA-FM) model, which was developed at the University of Florida, can predict the initiation and propagation of top-down fatigue cracks in asphalt pavements. A fundamental crack growth law was developed in the model on the basis that asphalt mixtures have a limit or threshold dissipated creep strain energy ($DCSE_f$) which governs mixture resistance for fracture (Zhang et al., 2001; Roque et al., 2002). A critical condition that leads to crack initiation or propagation is reached once the damage in asphalt mixture due to the repeated application of traffic loading equals or exceeds the threshold or limit dissipated creep strain energy. HMA-FM based crack growth simulator was developed using the displacement discontinuity boundary element method (DDBEM), which is capable of predicting the relative cracking performance of asphalt mixtures of the same age (Sangpetngam, Birgisson & Roque 2003; Sangpetngam, Birgisson & Roque 2004).

Roque et al. (2004) after a detailed analysis and evaluation of a number of field pavement sections in Florida using the HMA-FM model derived a parameter called energy ratio (ER), which is defined by dividing the

$DCSE_f$ of a mixture with the minimum dissipated creep strain energy ($DCSE_{min}$) as follows:

$$ER = \frac{DCSE_f}{DCSE_{\min}} \tag{3}$$

The ER method was used to evaluate the cracking performance of a number of pavement sections in Florida and was observed to successfully distinguish pavement sections which exhibited cracking from those that did not. Figure 1 presents a graphic illustration of $DCSE_f$ and $DCSE_{min}$.

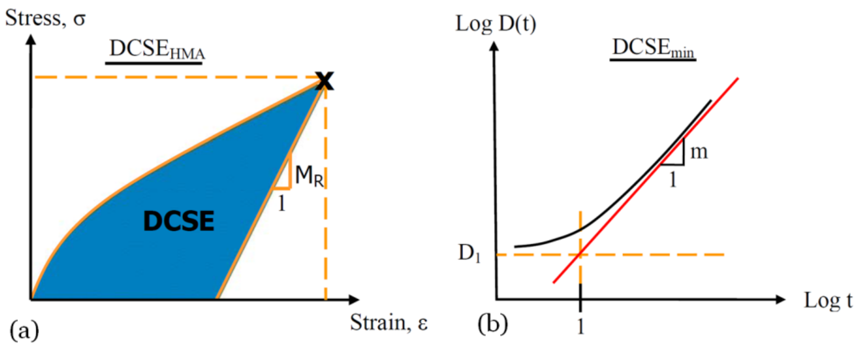

Figure 1. Graphic illustration of (a) $DCSE_f$ and (b) the creep compliance curve and $DCSE_{min}$

A predictive equation for the $DCSE_f$, which has been correlated to mixtures top-down cracking resistance in the field, was obtained on the basis of the creep rate in tension at time t=1000s. The following equation is proposed to estimate the change in $DCSE_f$ with asphalt aging.

$$DCSE_f = c_f S_t \frac{mD_1}{10^{3(1-m)}} \tag{4}$$

where c_f is a function of binder viscosity and equals 6.9×10^7, S_t, m and D_1 are tensile strength and creep compliance parameters of the asphalt mixtures.

The $DCSE_{min}$, which is the minimum dissipated creep strain energy required to produce a 50.8mm crack, is determined using the creep compliance parameters and tensile strength of the asphalt mixture, and the maximum tensile stress induced in the pavement structure (Wang et al.,2007). The following equation is used to determine the $DCSE_{min}$.

$$DCSE_{\min} = \frac{m^{2.98} D_1}{f(S_t, \sigma_{\max})} \tag{5}$$

The tensile strength (S_t) is related with the maximum tensile stress (σ_{max}) at the bottom of the AC layer using the following equation:

$$f(S_t, \sigma_{\max}) = \frac{6.36 - S_t}{33.44 \cdot \sigma_{\max}^{3.1}} + 2.46 \cdot 10^{-8} \tag{6}$$

The ER parameter was calibrated for different levels of traffic and reliability and incorporated into a Level 3 M-E pavement design tool for the top-down fatigue cracking performance evaluation of asphalt pavements for Florida condition. The design tool incorporates material property predictive models that estimate the evolution in material properties such as dynamic modulus, tensile strength and creep compliance parameters with age. The design scenario in the ER method is to determine a pavement structure which satisfies the required optimum energy ration (ER_{opt}) value at the end of the pavement design life for the specified traffic and reliability levels (Wang et al., 2006).

2.1.3 Enhanced HMA-FM method

A simplified fracture energy-based approach for crack initiation prediction was developed and integrated into the HMA-FM based crack propagation model in order to form the top-down cracking performance model (NCHRP, 2010). The top-down cracking performance model was developed on a critical condition concept, which specifies that crack initiates or propagates under a critical loading, environmental and healing conditions. Another key feature of the performance model is the fact that the effect of transverse thermal stress is included in the overall top-down fatigue cracking performance evaluation. The mechanisms which are attributed in the model for crack initiation are bending mechanism, which governs crack initiation in thin to medium thickness HMA layers and near-tire mechanism, which explains crack initiation in thicker pavements (NCHRP, 2010). The model was calibrated using field pavement sections from Florida and was found in delivering acceptable crack initiation times.

The top-down cracking performance model incorporates material property predictive models that account for the near surface mixture properties change with aging. AC stiffness aging model, fracture energy aging model and the healing potential models are some of the predictive material models which are incorporated into the performance model.

The asphalt stiffness aging model was developed on the basis of the global aging model and the Witczak and Fonseca (1996) dynamic modulus model (Mirza & Witczak, 1995). The mode considers the stiffness gradient within the AC layer which is induced by temperature fluctuation and aging. Equation 7 was used to consider the aging effect on mixture stiffness.

$$\left|E^*\right|_t = \left|E^*\right|_o \frac{\log \eta_t}{\log \eta_o} \tag{7}$$

where $|E^*|_t$ and $|E^*|_o$ are aged and original conditions dynamic modulus values respectively which are estimated at a loading time of 0.1s. The binder viscosities at aged (η_t) and unaged (η_o) conditions are estimated at a reference temperature of 10°C.

The dissipated creep strain energy limit aging model predicts the evolution in the dissipated creep strain energy limit value of the asphalt mixture. It generally decreases at a decreasing rate with age and reaches some minimum value after a sufficiently long time. The following equation was suggested to determine the evolution in $DCSE_f$ with age and depth.

$$DCSE_f(t,z) = \mathrm{FE}_f(\mathrm{t,z}) - \left[\frac{(S_t(t,z))^2}{2 \cdot S(t,z)}\right] \tag{8}$$

where $DCSE_f(t, z)$ and $FE_f(t, z)$ are dissipated creep strain energy and facture energy values respectively. $S_t(t, z)$ and $S(t, z)$ are tensile strength and stiffness values of the AC layer.

A simplified empirically-based healing model which has three components: maximum healing potential aging model, a daily-based healing criterion and a yearly-based healing criterion was developed and integrated into the performance model to predict the evolution in healing properties. Equation 9 presents the maximum healing potential surface aging model (h_{ym}) model.

$$h_{ym}(t,z) = 1 - \left[S_n(t)\right]^{FE_i/1.67} \quad (9)$$

where FE_i is initial fracture energy and $S_n(t)$ is normalized stiffness.

The top-down fatigue cracking performance model predicts crack initiation time using a parameter termed normalized damage accumulation ($DCSE_{norm}$). This parameter is defined by dividing the remaining dissipated creep strain energy after considering healing effects ($DCSE_{remain}$) with the corresponding limit $DCSE_f$. The threshold for crack initiation is when $DCSE_{norm}$ = 1.0 as shown in the following equation.

$$DCSE_{norm}(t) = \frac{DCSE_{remain}(t)}{DCSE_f(t)} \geq 1.0 \quad (10)$$

The performance model considers both load and thermal induced damages for the computation of $DCSE_{remain}$, which is obtained using the following equation for each time interval, Δt:

$$DCSE_{remain}(\Delta t) = (1 - h_{dn}) \cdot \left[n \cdot (DCSE_L / cycle) + DCSE_T(\Delta t)\right] \quad (11)$$

where n is number of load cycles in the time interval Δt.

The top-down cracking performance model uses a crack growth model which in conjunction with the material property models and thermal response model predicts the increase in crack depth with time. The model uses a displacement discontinuity boundary element (DDBE) program to predict the load-induced stresses ahead of the crack tip. Meanwhile, the stress intensity factor (SIF) of an edge crack was applied to the thermal stresses predicted using the thermal stress model to estimate the near-tip thermal stresses (Sangpetngam, 2003). The load and thermal stresses are then used to compute the induced damage in the same manner as the crack initiation model and the crack started to grow when $DCSE_{norm}$ = 1.0.

2.1.4 Asphalt mixtures morphology

There are several studies which have attempted to establish experimental methods that can identify key mixture properties for evaluating the susceptibility of asphalt mixtures to top-down fatigue cracking (Baek, Underwood, & Kim, 2012; Chen et al., 2012). One of the properties which have been investigated for its influence on mixtures fracture performance is mixtures morphology (Onifade et al., 2013; Kumar Das et al., 2013). Mixtures morphology, which governs the sizes, interconnectivity and distribution of pores that are partially filled with bitumen and small filler particles, can have a significant influence on aggregate interlock and mixtures performance as it influence the level of oxidative aging in the mixture.

Lira et al. (2012) developed an asphalt morphological framework that can identify and quantify key morphological parameter in a fundamental way. The framework was developed on the basis of aggregate packing arrangements and aggregate gradations and can quantify parameters that determine mixtures ability to transfer stresses. One of the morphological parameters identified in this framework is the primary structure coating thickness (PS coating thickness), which is a mix of bitumen and small filler particles that coats the load bearing or primary structure of the aggregates. Onifade et al., (2013) and Kumar Das et al., (2013) have observed that the amount of PS coating thickness can influence the cracking performance of asphalt mixtures.

2.2 Truck traffic characterization

Most pavement analysis and design specifications characterize and incorporate truck traffic using simplified empirical-based approaches. The equivalent single axle load (ESAL) characterization where a single factor is used to represent the multitude of applied traffic loads is the most well-known and has been implemented in many design specifications. The American Association of State Highway and Transport Officials (AASHTO) pavement design procedure and the French pavement design manual (LCPC) are typical examples where the mixed traffic effect is converted into ESALs using factors such as the equivalent axle load factors (EALF) and the coefficient of traffic aggressiveness (CAM) respectively (AASHTO, 1993; LCPC, 1994). An axle EALF indicates the level of damage a particular axle load and axle configuration induces relative to a standard axle and its magnitude depends on factors such as the pavement type, thickness and terminal failure conditions.

The ESALs traffic characterization has its origin in the 1950s American Association of State Highway Officials (AASHO) road test and has been since the basis for many pavement design procedures including the

AASHTO design guides (AASHO, 1962; AASHTO 1993). The AASHTO (1972) design guide derived a set of EALFs for different axle loads and axle configuration based on empirically developed equations and experience. The ESALs approach of converting the mixed traffic stream into a single factor was found to be insufficient for conditions which are significantly different from the original conditions upon which the performance observations were made. It also does not utilize available traffic data, which makes it inconsistent with the state of the practice recommended by the federal highway administration (FHWA) (2001). Rauhut, Lytton & Darter (1984) and Hajek (1995) have also showed that the use of ESALs can limit pavement design accuracy and recommended a pavement design based on actual axle load statistics and vehicle classification data.

Mechanistic-empirical pavement design procedures require a comprehensive traffic characterization approach that reflects accurately the diverse effect traffic loads have on pavement performance. For this reason, these design procedures characterize expected traffic using the magnitude, configuration and frequency of axle loads (MnPAVE, 2005; NCHRP, 2004; Timm & Young, 2004). This way of traffic characterization allows M-E pavement design procedures to compute pavement response and damage accumulation for the entire axle load distribution and eventually predict load related distresses for new and rehabilitated pavements. Moreover, the impact of traffic characterization parameters such as traffic growth rate, volume adjustment factors, seasonal and hourly traffic variations and lateral wheel wander on pavement performance can be evaluated and established (NCHRP, 2004).

A hierarchical approach for the development of traffic inputs was adopted in the MEPDG as it is not possible to obtain accurate future traffic characterization for some design scenarios which is due to unavailability of traffic data that has been collected over the years. Therefore, three broad level of traffic data inputs (Level 1 through 3), which are mainly defined by the amount of available traffic data, were recommended in the design guide. Level 1 inputs are considered to be the most accurate and are obtained from weigh-in-motion (WIM) stations which are placed directly on the project site or other similar roads. In the case of Level 2 inputs, regional or state wide WIM data are used to develop the required traffic inputs. Level 3 inputs are considered to be the least accurate and are developed averaging state-wide or nation-wide WIM data (NCHRP, 2004). The subsequent sections present the traffic characterization inputs which are required for the performance evaluation of pavements in a M-E approach.

2.2.1 Vehicle class distribution

Vehicle class distribution represents the normalized annual percentage of each truck class within the annual average daily truck traffic (AADTT). The FHWA collects and organizes traffic data for pavement design purposes using 13 standard vehicle class types. For pavement design purpose only traffic inputs from vehicle class types (4-13) are required as induced damage by passenger cars (i.e., vehicle classes 1-3) is assumed to be negligible (NCHRP, 2004). The MEPDG recommends 17 different truck traffic classification (TTC) groupings for pavement design purpose which can be used depending on functional requirements and local economy. A study which was conducted using the LTPP traffic database has shown that the annual vehicle class distribution factors do not change over time, and if there is a change it is mainly due to random variation than a variation due to changing truck site conditions (NCHRP, 1999).

2.2.2 Axle load spectra

Axle load spectra, which represent the percentage of the total axle load application within each load interval, are one of the inputs required for traffic characterization in M-E design guides (Lu & Harvey 2006). Swan et al. (2008) reported that axle load spectra, traffic volume and type of vehicles are the most significant factors affecting pavement performance. For pavement design purposes, the MEPDG and other M-E design specifications recommend axle load spectra of each axle configuration (Single, Tandem and Tridem) and vehicle class types (4-13) (NCHRP, 1999).

Axle load spectra are normalized on annual basis and as such it is imperative to study the year to year variations that exist within the data. A study which was performed on the data that was used to generate the default Level 3 axle load spectra inputs for the MEPDG has shown that there was no significant variation on annual basis (NCHRP, 1999). Based on a five year WIM data, Cunagin (2013) reported that the axle load spectra at a given WIM site do not vary significantly on annual basis.

Nevertheless, both studies have observed substantial variation among the various WIM sites.

2.2.3 Volume adjustment factors

The traffic volume of highways shows variation on time basis, which can have an effect on pavements performance. Thus, M-E pavement design specifications recommend adjustment factors that are provided to reflect the monthly and hourly variations of the traffic volume. Monthly distribution factors (MDF) are required to adjust the seasonal variation of the traffic volume whereas the hourly distribution factors (HDF) are needed to take into account the hourly variations. The hourly distribution factors can have a significant impact on predicted performance in the case when pavement performance and damage are computed on hourly basis (Zou & Roque, 2009).

Monthly distribution factors represent the proportion of the annual truck traffic that occurs in a specific month and its values are site specific and depend on factors such as the local economy and climate conditions (NCHRP, 1999). Monthly distribution factors are assumed to remain constant during the design period but in reality these factors are expected to change from year to year.

The hourly distribution factors represent the expected percentage of the AADTT within each hour of the day. For Level 3 pavement design, the MEPDG recommends default values that were obtained from the LTPP traffic database. These factors exhibit higher than the average percentage values for the hours 10 -15 (NCHRP, 1999).

2.2.4 Traffic lateral wheel wander

Repetitive traffic loadings are not applied at a specific unique location on the pavement surface which is due to the lateral wandering effect of the wheel. Studies have shown that the lateral wheel wander parameter affects prediction of distresses within the pavement system (Blab and Litzka, 1995; Wu and Harvey, 2008). Therefore, M-E pavement design procedures consider the lateral wheel wander effect in truck traffic characterization. The MEPDG models the lateral wheel wander with a normal distribution, which is the same distribution observed by Erlingsson, Said and McGarvey (2012) based on field measurements.

2.3 Reliability analysis

Structural reliability, which estimates the reliability or probability of failure of a given structural system during its design period, has been the focus of many researchers. There are several reliability analysis methods that can be used depending on the complexity of the reliability problem, the number of random variables involved and the uncertainty associated with these random variables. The first step in a reliability analysis problem is to establish the basic variables (X_i) and the performance function that defines the relationships among them. The performance equation can be described analytically as follows:

$$Z = g(X_1, X_2, ..., X_n) \tag{12}$$

The failure surface or limit state that defines the boundary between the safe and failure regions in the design parameter space is also needs to be defined. The condition that Z=0 is usually used as a limit state but depending on the reliability problem different conditions can be used to define the limit state. The limit state function, which can be either an explicit or implicit function of the random variables, plays an important role in reliability analysis. Reliability for the case when Z<o defines failure can be given as follows:

$$R = 1 - p_f = p\{g(x) \geq 0\} = \int_{g(x)\geq 0} f_X(x)dx \tag{13}$$

where $f_x(x)$ is the joint probability density function for the basic random variables X_1, X_2,….,X_n and the integration is performed over the region g()>0.

In order to solve the reliability problem as defined in equation 13, it is necessary first to obtain the joint probability density function of the random variables, which for most cases is impossible to obtain. For few special cases numerical integration can be used to obtain an exact solution. Monte Carlo based simulations can also be used to solve the reliability analysis problem.

There are several analytical methods which can be used to obtain an approximate solution to the integral in Equation 13. These methods are easy to use and are mainly divided into two groups namely first order reliability method (FORM) and second order reliability method (SORM).

There are a variety of FORM methods which can be applied depending on the complexity of the reliability analysis problem.

2.3.1. Monte Carlo simulation

The Monte Carlo simulation method is a straightforward approach that requires only a basic working knowledge of probability and statistics for evaluating the reliability of complicated engineering systems. Monte Carlo simulation requires the performance function to be defined either implicitly or explicitly with the random variables, and the random variables probabilistic characteristic to be quantified in terms of their probability density functions (*pdf*). The simulation is performed based on numerical experimentation where the performance function is evaluated deterministically for a large number of realizations of the basic random variables X, i.e. x_i=1,2,...n to determine whether each of the outcomes fulfils the requirement on the limit state conditions. For the case when g(x) ≤0 defines the limit state function, if a realization of the random variables does not fulfil this requirement then it will be considered as a 'failure condition'. Probability of failure (p_f) is estimated by dividing the number of simulation cycles for the condition g(x) ≤ 0 (N_f) with the total number of simulation cycles (N) as follows:

$$p_f = \frac{N_f}{N} \tag{14}$$

The accuracy of the probability of failure estimated using Equation 14 will mainly depend on the total number of simulation cycles. The estimated probability of failure would increase as the number of cycle increases and would approach the true value as N approaches infinity. Ayyub and Haldar (1985) proposed a method to evaluate the number of required cycles by evaluating the COV of the estimated probability of failure as follows:

$$COV(p_f) = \delta_{p_f} = \frac{\sqrt{\dfrac{(1-p_f)p_f}{N}}}{p_f} \tag{15}$$

2.3.2. Advanced First Order Second Moment (AFOSM)

The advanced first order second moment (AFOSM) method which was proposed by Hasofer and Lind (1974) has overcome the lack of invariance observed in the Cornell (1969) reliability index for mechanically equivalent performance functions. The AFOSM approach solves the invariance problem by transforming the random variables from the original coordinate system into a reduced coordinate system and constructing a linear approximation to the performance function. The normal random variables are reduced into standard normal variables as follows:

$$X_i^{'} = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}} \tag{16}$$

where X'_i is a standard normal variable with zero mean and unit standard deviation and X_i is a normal variable with μ_{Xi} mean and σ_{Xi} standard deviation.

The safety or reliability index (β_{HL}) which is defined in the reduced space as the minimum distance from the origin of the axes to the limit state surface (failure surface) is computed as follows:

$$\beta_{HL} = \sqrt{(x'^{*})^{t}\left(x'^{*}\right)} \tag{17}$$

where **x'*** and **x*** are vectors which represent the values of all the random variables i.e. X_1, X_2,…,X_n at the design point or checking point in the original and reduced coordinate systems respectively.

For a two random variable problem (R and S) that is defined by a linear performance function, the AFOSM methodology for solving the reliability problem can be explained with the help of Figure 2. Figure 2(a) and 2(b) present the two random variables in the original and reduced coordinate systems respectively. As can be seen in Figure 2(b), the position of the failure surface relative to the origin defines the reliability of the system. The Hasofer-Lind reliability index β_{HL} is invariant as the geometric shape and the distance from the origin remain constant regardless of the form in which the limit state equation is written.

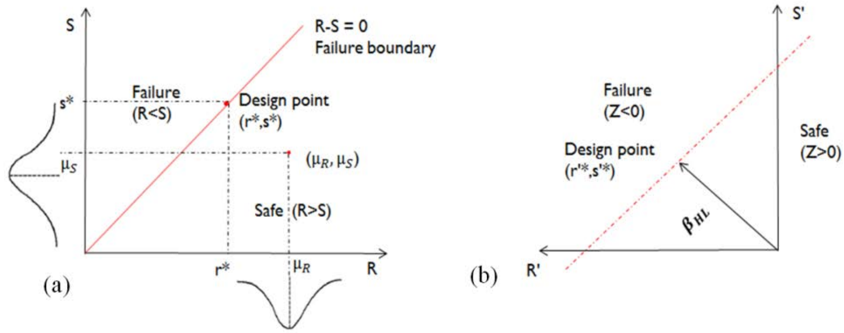

Figure 2. Hasofer Lind reliability index a) original coordinate space and b) standard normal space

A first order approximation of the failure probability can be obtained with the Hasofer-Lind reliability index using Equation 18. This is the integral of the standard normal density function along the ray joining the origin and the minimum distance point **x'***. This point represents the worst combinations of the stochastic variables and named the design point or the most probable point (MPP) of failure.

$$p_f = \Phi(-\beta_{HL}) \tag{18}$$

Finding the minimum distance in the case when the reliability problem involves many random variables and the limit state is non-linear becomes an optimization problem. Rackwitz (1976) suggested an algorithm which solves the Hasofer-Lind reliability index and the design point by constructing a linear approximation to the performance function at every search point till the minimum distance from the origin is obtained.

2.3.3. First order reliability method (FORM)

The Hasofer-Lind reliability index can be exactly related to the probability of failure only for the case when all the random variables are normal and statistically uncorrelated and the limit state function is linear. To correct these shortcomings, Rackwitz and Fiessler (1978) and Chen and Lind (1983) included information regarding the distributions of the random variables into the algorithm, which is applicable for both linear and non-linear performance functions.

Rackwitz and Fiessler (1976) proposed a two-parameter equivalent normal approach to transform the non-normal random variables into normal random variables. This transformation was achieved by imposing the conditions that the cumulative distribution functions and the probability density functions of the actual variables and the equivalent normal variables should be equal at the checking point (x_1*,x_2*,....,x_3*) on the failure surface. For a statistically independent non-normal variable equating its cumulative density function (CDF) with an equivalent normal variable at the checking point results in:

$$\Phi\left(\frac{x_i^* - \mu_{X_i}^N}{\sigma_{X_i}^N}\right) = F_{X_i}(x_i^*) \tag{19}$$

where Φ ( ) and $F_x(x_i*)$ are the CDF of the standard normal variate and the original non-normal variable at the checking point respectively and, μ_X^N and σ_X^N are the mean and standard deviation of the equivalent normal variable at the checking point respectively.

The condition that the probability density functions (PDF) of the original variable and the equivalent normal variable at the checking point should be equal results in:

$$\frac{1}{\sigma_{X_i}^N}\phi\left(\frac{x_i^* - \mu_{X_i}^N}{\sigma_{X_i}^N}\right) = f_{X_i}(x_i^*) \tag{20}$$

where ϕ( ) and f_{Xi} are the *pdfs* of the equivalent standard normal and the original non-normal random variables respectively.

Rackwitz and Fiessler (1978) suggested an algorithm which uses a Newton-Raphson type recursive formula to find the design point. This method linearizes the performance function at each iteration point and instead of solving for the reliability index directly it uses the derivatives of the performance function to obtain the next iteration point. The Rackwitz-Fiessler algorithm for many cases converges fast and has been widely used in structural reliability problems.

The main steps of the Rackwitz and Fiessler (R-F) algorithm can be described as follows:

1. Define the appropriate performance or limit state function and failure criterion.

$$Z = g(X_1, X_2, ...X_n) \le 0 \tag{21}$$

2. Assume initial values of the design point. The mean values are normally taken as initial point.

$$x_i^* = (\mu_{x_1}, \mu_{x_2}, ..., \mu_{x_n}) \tag{22}$$

3. Compute the equivalent normal mean and standard deviation values of non-normal variables using Equations 19 and 20.
4. Transfer the random variables into standard normal variables using Equation 16.
5. Compute the partial derivatives which are the components of the gradient vector of the performance function in the equivalent standard normal space by the chain rule of differentiation:

$$\frac{\partial g}{\partial X_i^{'}} = \frac{\partial g}{\partial X_i}\frac{\partial X_i}{\partial X_i^{'}} = \frac{\partial g}{\partial X_i}\sigma_{X_i}^N \tag{23}$$

The direction cosines, which are the components of the corresponding unit vector of the performance function, are computed as follows:

$$\alpha_i = \frac{\left(\frac{\partial g}{\partial X_i^{'}}\right)^*}{\sqrt{\sum\left(\frac{\partial g}{\partial X_i^{'}}\right)^{2*}}} = \frac{\left(\frac{\partial g}{\partial X_i}\right)^* \sigma_{X_i}^N}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial X_i}\sigma_{X_i}^N\right)}} \tag{24}$$

6. Compute the new design points in the equivalent standard normal space using the recursive formula as follows:

$$x_{i+1}^{'*} = \frac{1}{\left|\nabla g(x_i^{'*})\right|^2}\left[\nabla g(x_i^{'*})^t x_i^{'*} - g(x_i^{'*})\right]\nabla g(x_i^{'*}) \tag{25}$$

7. Compute the distance to this new design point from the origin as

$$\beta = \sqrt{\sum_{i=1}^{n}(x_i^{'*})^2} \tag{26}$$

8. Compute the new design values in the original space as follows

$$x_i^* = \mu_{X_i}^N + \sigma_{X_i}^N x_i^{'*} \tag{27}$$

Compute and check the convergence criterion for g() for this new design point. If the convergence criterion is not satisfied then repeat steps 3 through 8 until convergence is achieved.

2.4 Pavement reliability

Pavement analysis and design has been performed traditionally following a deterministic approach where a design parameter is represented with a unique deterministic value (Shell, 1978; PMS Objekt, 2008; Austroads 2004). However, in reality, pavement design and construction involve many uncertainties, variabilities and approximations which arise from various sources. The combined effect of these uncertainties will have a significant influence on predicted pavement performance (Darter, McCullough & Brown, 1972; Noureldin et al., 1994, Timm, Birgisson & Newcomb, 1998; Kenis & Wang, 2004).

Several studies have been performed since the 1970s which attempt to address the question of reliability for pavement application (Lamer & Moavenzadeh, 1971; Darter, Hudson, & Brown, 1973). Most of these research have been performed either with Monte Carlo simulation or analytical reliability analysis tools such as the first order second moment (FOSM), the point estimate methods (PEM) and the first order reliability method (FORM) (e.g., Chua, Kiureghian, Monismith, 1992; Timm et al., 1999; Kim, Harichandran & Buch, 1998; Kim & Buch, 2003; Retherford & McDonald, 2010).

The role of reliability analysis in pavement design is to compute the probability that a pavement structure will perform as planned during its design period. For the case when pavement performance is evaluated with traffic, reliability can be defined as the probability that the total expected traffic load applications will not exceed the traffic load capacity of the pavement structure. In terms of distress, the same reliability can be defined as the probability that any particular type of distress will remain within the permissible level during the design period (AASHTO, 1993; Kulkarni, 1994). The performance function or safety margin (SM) in terms of traffic can be defined as follows:

$$SM = N - n \tag{28}$$

where N is the allowable traffic load repetition that the pavement can withstand before failure and n is the actual traffic load repetition which is expected during the design period.

In order to solve the reliability problem stated in Equation 28, the statistical properties of the safety margin need to be established, which are the outcomes of the design inputs that are used to compute the allowable and expected traffic repetitions. In the case when the variabilities of the underlying design inputs are defined with the full probability distribution approach, the safety margin variability would be represented by the joint probability distribution function of these design inputs and the probability of failure (p_f), as can be seen in Figure 3, is the area under the curve to the left of the origin.

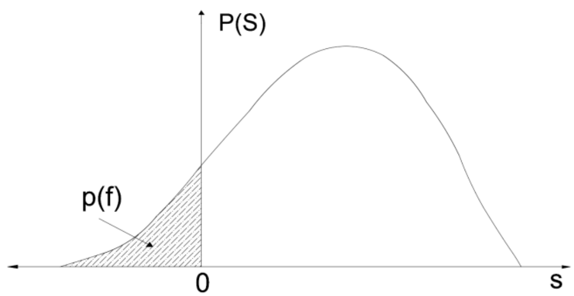

Figure 3. Illustration of the probability distribution function of the safety margin

2.4.1 Reliability application for pavement design

Pavement design specifications which incorporate reliability have been developed to address the effect of uncertainties on pavement performance. Harr (1987) estimated the failure risk associated with pavement performance evaluations using the factor of safety (FOS) approach where a single factor is used to represent the overall uncertainty involved in the design process. The AASHTO design procedure has also incorporated a reliability design factor (Fr) as a positive spacing parameter between the allowable and expected traffic repetitions (AASHTO, 1993). These factors are mainly determined through engineering judgement or experience and might not ensure designs of uniform reliability as the various random variables influence on predicted performance is quite different.

M-E pavement design procedures incorporate mechanistic models that provide a more rational and realistic methodology for pavement response and damage computations. In addition, these design procedures consider input variabilities and uncertainties effect on predicted performance through reliability. In the MEPDG, the individual distresses (e.g., rutting, fatigue cracking and thermal cracking for flexible pavements) are the focus of interest and these values are considered to be normal random variables. The design reliability for each individual distress modes is based on the standard error of each individual model estimates that is obtained through the calibration process. This standard error is a combination of input variability, model error and variability introduced due to construction. The design guide recommends a desired level of reliability with acceptable level of distress at the end of the design life for each individual distress modes (ARA Inc., 2004).

The Florida cracking design tool incorporates empirically derived reliability factors to account for traffic and pavement structural effects on optimum energy ratio values (Wang et al., 2006). These reliability factors were derived in conjunction with existing design procedure which enables the newly developed factors to incorporate the risk level implied in the existing design procedure. This also minimizes the design deviation that can happen between the new and existing design procedures. Further attempts have also been done to decouple the structural (φ) and traffic effects (γ) from the optimum energy ratio values (ER_{opt}).

$$ER_{opt} = \frac{\gamma}{\varphi} \tag{29}$$

2.4.2 Load and resistance factor design (LRFD)

A reliability analysis based on a probabilistic method of uncertainty propagation allows the design procedure to account for variabilities and uncertainties effect on predicted performance. Nevertheless, probabilistic-based pavement design involves the repeated application of reliability analysis tools such as the first order reliability method (FORM), which are used to evaluate and assess the reliability of trial designs until estimated reliability converges to the targeted reliability. This approach may not be suitable for designs that are carried out on trial basis but can be used to develop reliability-based codified design procedures. The load and resistance factor design (LRFD) approach, where the uncertainty and influence of each parameter is considered separately has been adopted in many design specifications to provide a user friendly reliability-based design approach.

In LRFD procedure, partial safety factors which are derived through reliability analysis are assigned to each significant parameter based on its degree of influence, associated variability and on the level of safety required. Most structural design specifications have already adopted the LRFD format (AASHTO1997b; ACI, 1995; AISC, 1986; & OHD, 1991). However, the adoption of the LRFD procedure for pavement design is at its early stage and there are limited studies regarding its development and implementation (Kim, Harichandran & Buch, 1998; Kim and Buch, 2003). By providing pavement sections of uniform performance level, the LRFD procedure provides a proper platform for life cycle cost (LCC) comparisons. Moreover, it enables the design engineer to choose appropriate safety level for a given pavement design taking into account functional requirements and consequence of failure.

The LRFD procedure is based on the concept that the reliability associated with a given design equation should correspond to a certain degree of structural safety denoted by a certain target reliability. It is based on the limit state design concept and performance is evaluated by comparing the factored resistance with the sum of the factored loads (AASHTO, 1997a). A typical LRFD format can be expressed as follows:

$$\phi R_n \geq \sum \gamma_i Q_{n,i} \tag{30}$$

where ϕ is the partial safety factor for the resistance, γ_i are the partial safety factor for the load effects, R_n and $Q_{n,i}$ are the nominal or mean values of the resistance and the load effects, respectively.

2.5 Design inputs variabilities

Pavement analysis and design involves many inputs and factors that are uncertain or variable in nature. These uncertainties and variabilities can be attributed to three major sources. The first source is due to the inherent variability of design inputs which is caused by the spatial variability in material and cross sectional properties and improper measurements. Uncertainty is also introduced into the design process due to model bias. The models that are used to determine pavement response or damage accumulation are mainly an idealized form of the much complex pavement behaviour. The third source of uncertainty, statistical uncertainty, is concerned with incomplete information which is due to finite sample size (Huang, 2004).

Statistical characterization of variabilities and uncertainties is a prerequisite for any reliability analysis. Parameters that are identified through a parametric or sensitivity analysis as having a significant influence on predicted performance needs to be modelled as random variables in the design process. For pavement analysis and design these parameters can be modulus and thickness of layers, environmental inputs, traffic characterization inputs and asphalt mixture characterization parameters. Probabilistic method of uncertainty propagation requires variabilities to be modelled with the full distributional approach where the probability density function in addition to the mean and variance of the random variables are required.

Table 1 . A summary of pavement design inputs variabilities

Parameter	Pdf	COV range [%]	Reference
Asphalt thickness	Log-Normal	-	Kim and Burati (1988)
	Normal	3-12	Timm et al. (1999)
	Normal	3-12	Noureldin et al. (1994)
	-	7-8	Kenis and Wang (2004)
	Normal	3-25	Bush (2004)
	Normal	10	Darter et al. (1973)
Base Modulus	Log-Normal	15-50	Timm et al. (1999)
	Normal	10-30	Noureldin et al. (1994)
	-	23-25	Kenis and Wang (2004)
	Log-Normal	5-60	Bush (2004)
Traffic	Normal	-	Sun and Hudson (2005)
	Log-Normal	30-42	Maji and Das (2008)
	Log-Normal	42	AASHTO (1985)
	Extreme type-I	-	Timm et al. (1999)

A literature survey has revealed that there are only few studies which have been conducted regarding the variability associated with pavement design inputs as most of the research effort has been devoted to the characterization and evaluation of these parameters. Table 1 presents the variability associated with the design parameters which are considered to have a significant influence on the top-down fatigue cracking performance of asphalt pavements. The variability of these design inputs were reported using the coefficient of variation (COV), which is obtained by dividing the standard deviation with the mean value.

3. Pavement sections and traffic data

A number of field pavement sections and asphalt mixtures were used in the development of the mechanics-based analysis and design framework. In addition, traffic data from weigh in motion (WIM) stations were studied and analysed to obtain representative traffic inputs for Florida conditions. The subsequent section presents the pavement sections and the traffic inputs which were used for this purpose.

3.1 Pavement sections

Pavement sections that have high quality field and laboratory data were selected for model development, calibration and validation purposes (Paper I, II & III). A total of 28 pavement sections from the state of Florida, USA, which have well-documented field performance histories and comprise of state roads and interstate with a wide range in traffic, mixture types, materials, structures and climate inputs were used for this purpose. The Field performance histories of these pavement sections were obtained from the Florida department of transportation (FDOT) database. A summary of the sources that were used to obtain the design inputs for the selected pavement sections are provided below.

Table 2. A summary of information of pavement sections from source 1

Section	County	Core age (year)	Traffic/year (MESALS)	Year opened for traffic	Observed CI year
I75-1A	Charlotte	15	0.57	1988	1998
I75-1B	Charlotte	14	0.56	1989	2000
I75-3	Lee	15	0.67	1988	1999
I75-2	Lee	14	0.58	1989	2001
SR80-1	Lee	16	0.22	1987	2001
SR80-2	Lee	19	0.21	1984	1999

Source 1: six pavement sections which were part of the group that was used to evaluate the enhanced HMA-FM model (NCHRP, 2010) were selected for this study. Further information regarding mixtures gradation and volumetric properties was obtained from a study undertaken by Jajliardo (2003). Table 2 presents information regarding section name,

location, traffic volume and observed crack initiation (CI) time of these sections.

Source 2: six pavement sections that were part of the study to evaluate loading effects on surface cracking of pavements in asphaltic mixtures were included in this study (Garcia, 2002). These sections were also part of the study undertaken by Jajliardo (2003) for developing specification criteria to mitigate top-down cracking.

Source 3: five pavement sections that were part of the Florida SuperPave monitoring project were selected for this study. In the SuperPave monitoring project, researchers evaluated construction and filed performance data to establish a reliable and appropriate performance based specifications for the new generation of asphalt mixtures (Roque et al., 2011).

*Source 4: a*dditional 11 pavement sections were obtained from the study conducted by the Texas Transportation Institute (TTI) researchers in partnership with the FDOT engineers. The research team evaluated field pavement sections in order to compile a database of information on mixture, structure, traffic and other related design variables. The database was used to verify the predictions from the mechanistic-empirical pavement design program (MEPDG) and to perform model calibration for local conditions (Oh & Fernando, 2008).

3.2 Traffic inputs

Traffic data from various sources were analysed to obtain traffic inputs which are representative of Florida conditions. Traffic inputs such as vehicle class distribution, axle load spectra, volume adjustment factors and numbers of axle per truck were developed for this purpose (Paper II). These traffic inputs can be considered as Level 3 inputs as they are representative of the average Florida conditions. Data from various sources, including the long term pavement performance (LTPP) database and publications from Florida State University were used for developing these traffic inputs (Cunagin et al., 2013; Kwigizile, 2004; NCHRP, 1999). The following section presents the developed traffic characterization inputs for Florida conditions.

3.2.1 Vehicle class distribution

Representative vehicle class distribution factors for Florida interstate and state road conditions were developed using Weigh in Motion (WIM) data from NCHRP (1999) and Kwigizile (2004). These WIM sections are located in different parts of Florida and the data set, a total of 15, was averaged to obtain representative average condition for Florida interstate and state roads. The vehicle class distribution was not observed to vary on annual bases but there was significant variation among different WIM sites.

3.2.2 Axle load spectra

The annual normalized load spectra for each axle configurations (Single, Tandem and Tridem) and vehicle class (VC 4-13) for Florida conditions were obtained by analysing WIM data from the LTPP database and a study undertaken by Cunagin et al. (2013). The LTPP database was collected for a five year period and the data showed no consistent year to year variation. Cunagin et al. (2013) analysed a five year WIM data which was collected from different locations in Florida. These data sets were used to develop representative average axle load spectra values for Florida conditions. Figure 4 presents vehicle classes 6, 8, and 9 single axle load spectra.

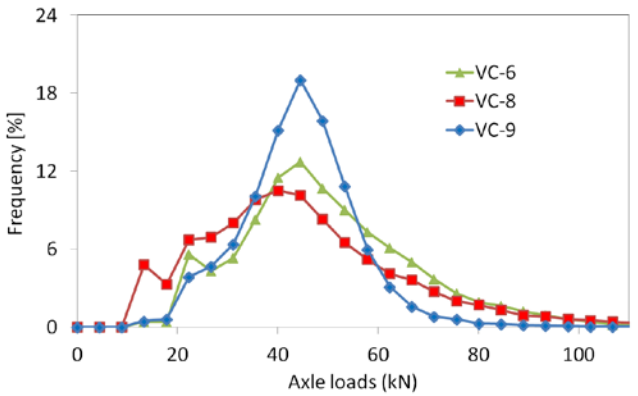

Figure 4. Representative single axle load spectra distributions for vehicle classes 6, 8 and 9

3.2.3 Monthly and hourly distribution factors

The representative monthly distribution factors for Florida conditions were developed using data from Cunagin et al. (2013). These factors were developed using average daily traffic volume of VC-9 from WIM site 9901. The hourly distribution factors were developed by studying and analysing data from Kwigizile (2004). Hourly vehicular volume data from seven WIM sites were used for this purpose. Kwigizile (2004) collected three days of hourly vehicular volume data in a one month period which was later averaged to determine the HDFs of the studied highways. Figure 5 presents the representative monthly and hourly distribution factors for Florida conditions.

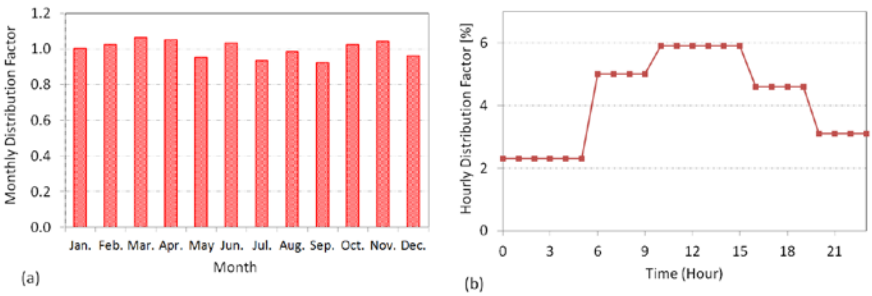

Figure 5. Representative (a) MDFs and (b) HDFs

4. Summary of appended papers

In this thesis, a mechanics-based design framework was developed for the top-down fatigue cracking performance evaluation of flexible pavements. This was achieved by performing a reliability calibration on a mechanics-based analysis framework, which predicts top-down fatigue cracking initiation time on the basis of HMA-FM. Axle load spectra truck traffic characterization along with its associated parameters were integrated into the analysis framework in order to improve the prediction accuracy while the sensitivity of the design framework relative to variations in inputs variabilities was also evaluated. The subsequent sections present a summary of the papers which are appended in this thesis.

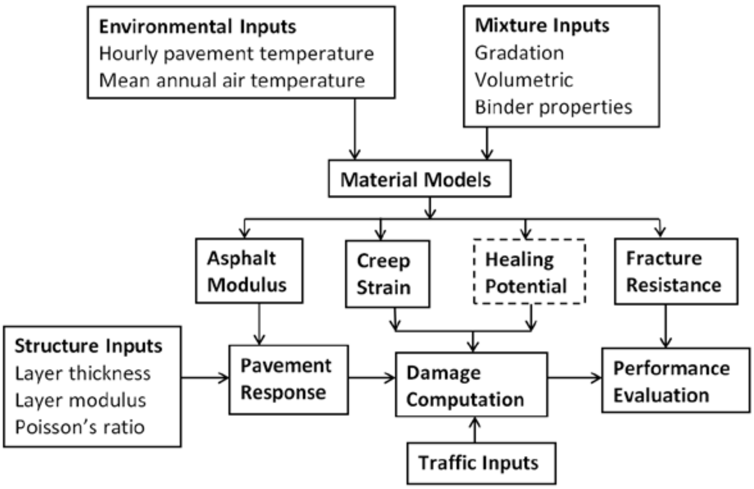

Figure 6. Flowchart for the mechanics-based analysis framework

4.1 Mechanics-based analysis framework (Paper I)

The mechanics-based analysis framework predicts top-down fatigue cracking initiation time in asphalt concrete pavements through mechanistic principles. HMA-FM was selected for further enhancement as it has a threshold concept that can easily be used to determine fracture in asphalt materials. Moreover, it allows the incorporation of material property predictive models which can be used to obtain the aged values of key damage and fracture properties. Paper I and Paper II present a detail description of the analysis framework along with its various components. The general outline of the mechanics-based analysis framework is presented in Figure 6.

The input module of the analysis framework provides all the information that is required for pavement performance evaluation. These inputs which are obtained either from laboratory tests or design specifications are required in the material models to determine the evolution in key fracture properties with age. Hourly air temperature which is a key input in the creep strain and asphalt stiffness material models is obtained by averaging a five year hourly air temperature from the nearest local weather station.

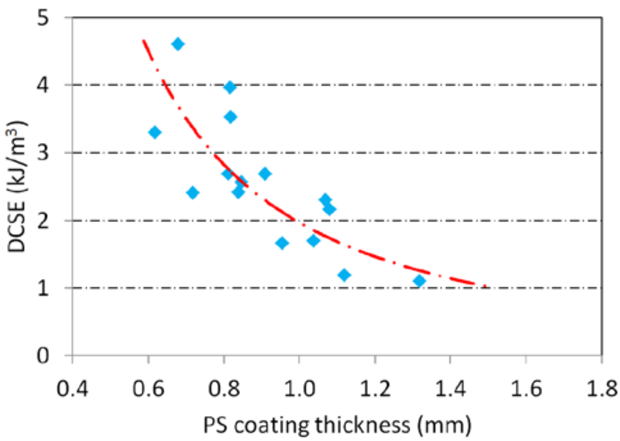

Figure 7. Relationship between $DCSEl_{im}$ and PS coating thickness (t_{ps})

Material models which utilize mixture morphology were developed to quantify the evolution in mixtures dissipated creep strain energy limit ($DCSE_{lim}$) and healing potential (h_{ym}). The morphology based predictive model for $DCSE_{lim}$ was achieved in two steps. First, a relationship between PS coating thickness and $DCSE_{lim}$ was established using data from 15 pavement sections. Figure 7 presents the observed relationship between PS coating thickness and $DCSE_{lim}$. Based on the observed relationship and using additional asphalt mixtures, a non-linear regression was undertaken to establish Equation 31, which relates $DCSE_{lim}$ with PS coating thickness (t_{ps}) and age (t).

$$DCSE_{\lim} = k_1 \left(\frac{1}{t_{ps}} \right)^{k_2} \left(\frac{1}{t} \right)^{(k_3 + k_4 \cdot \log(t_{ps}))} \tag{31}$$

where k_1= 2.38, k_2 = 0.79, k_3 = 0.33 and k_4 =0.12.

The healing potential model (h_{ym}) which is a simplified equation that accounts for mixtures morphology was developed on the basis of previous research. The developed model predicts yearly healing potential values using PS coating thickness (t_{ps}), initial dissipated creep strain energy ($DCSE_i$) and time of year (t). Figure 8 presents how PS coating thickness values influence the level of mixtures healing potential. An unknown aging factor k, that determines the healing potential evolution with age, was introduced to be used later for model calibration. Equation 32 presents the developed healing potential equation.

$$h_{ym}(t) = 1 - \left(\left[\exp\left(\frac{t_{ps}}{t} \right)^{-DCSE_i} \right]_{norm} \right)^k \tag{32}$$

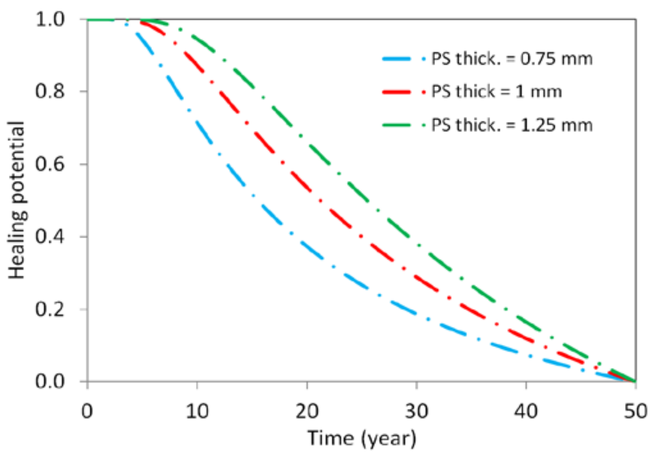

Figure 8. Relationship between h_{ym} and PS coating thickness

The developed framework was calibrated and validated using field pavement sections that have high quality laboratory and field performance data. From preliminary calibration results it was observed that it might not be possible to obtain an optimum aging factor that can effectively accounts for all the design conditions. Therefore, the model calibration was performed by dividing the pavement sections into two groups. Traffic volume which was observed to have a significant influence on predicted performance was used to divide the pavement sections into two categories: low volume traffic roads and medium to high volume roads. A traffic volume of 100,000 ESALs/year was used as a criterion for this purpose. Figure 9 presents the prediction results for the medium to high traffic volume roads. As can be seen in Figure 9, there is a general agreement between the two values even if there is a discrepancy for some pavement sections. Equation 33 presents the optimized healing potential equation for medium to high volume traffic roads.

$$h_{ym}(t) = 1 - \left(\left[\exp\left(\frac{t_{ps}}{t}\right)^{-DCSE_i} \right]_{norm} \right)^{15.5 \cdot t_{ps} + 3.35} \tag{33}$$

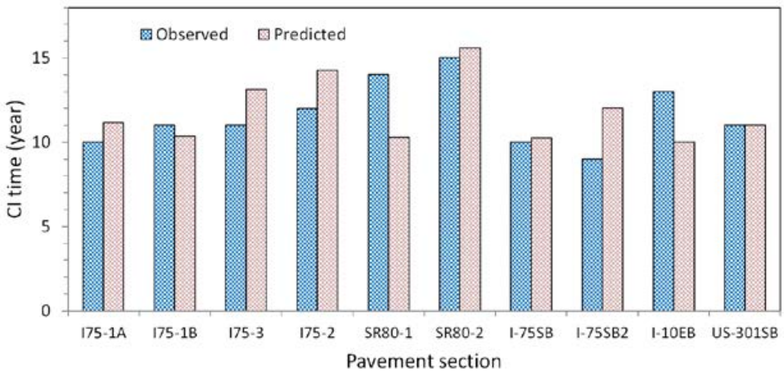

Figure 9. Predicted and observed CI times for medium to high volume roads

The validation for the medium to high volume traffic category model was performed on 12 pavement sections, which were obtained from various counties in Florida. Figure 10 presents the comparison between the observed and predicted crack initiation values for the validation sections. As can be seen in Figure 10, the analysis framework was able in predicting crack initiation times which are relatively in general agreement to the observed performances in the field.

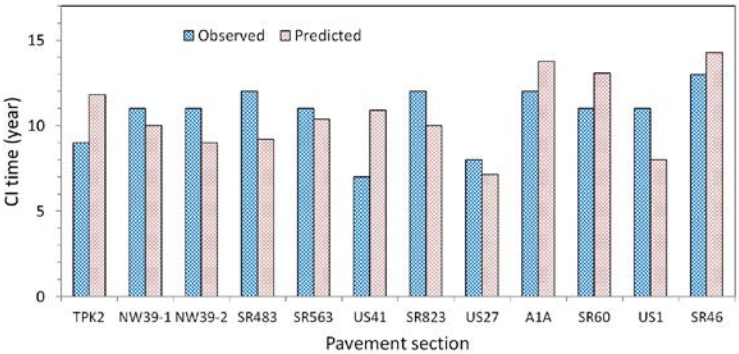

Figure 10. Predicted and observed CI times for medium to high volume roads

4.2 Evaluation of truck traffic effects (Paper II)

Traffic loading was identified in having a significant impact on the top-down fatigue cracking performance of flexible pavements. Most of the pavement analysis and design tools for the top-down fatigue cracking performance evaluation of asphalt pavements use a simplified traffic characterization approach in the form of ESALs. A comprehensive traffic characterization approach that reflects accurately the diverse effect traffic loads have on pavement performance is needed in order to accurately determine pavement response and damage accumulation. A traffic characterization approach as recommended in the traffic monitoring guide (FHWA, 2011) was followed for the integration of truck traffic into the mechanics-based analysis framework. Paper II presents the traffic integration approach which was adopted to incorporate truck traffic characterization parameters (e.g., axle load spectra, volume adjustment factors, monthly and hourly distribution factors and lateral wheel wander) into the analysis framework and the impact these parameters have on predicted performance.

A flowchart which depicts the approach adopted to integrate axle load spectra and its associated parameters into the analysis framework is presented in Figure 11. The flowchart requires as an input annual average daily truck traffic (AADTT), monthly distribution factors (MDF), hourly distribution factors (HDF), vehicle class distribution, axle load spectra for each vehicle class and axle configuration and the number of axle per truck.

The flowchart, utilizing these inputs and Monte Carlo simulation generates for every hour the volume of expected traffic loading with theirs associated contact locations.

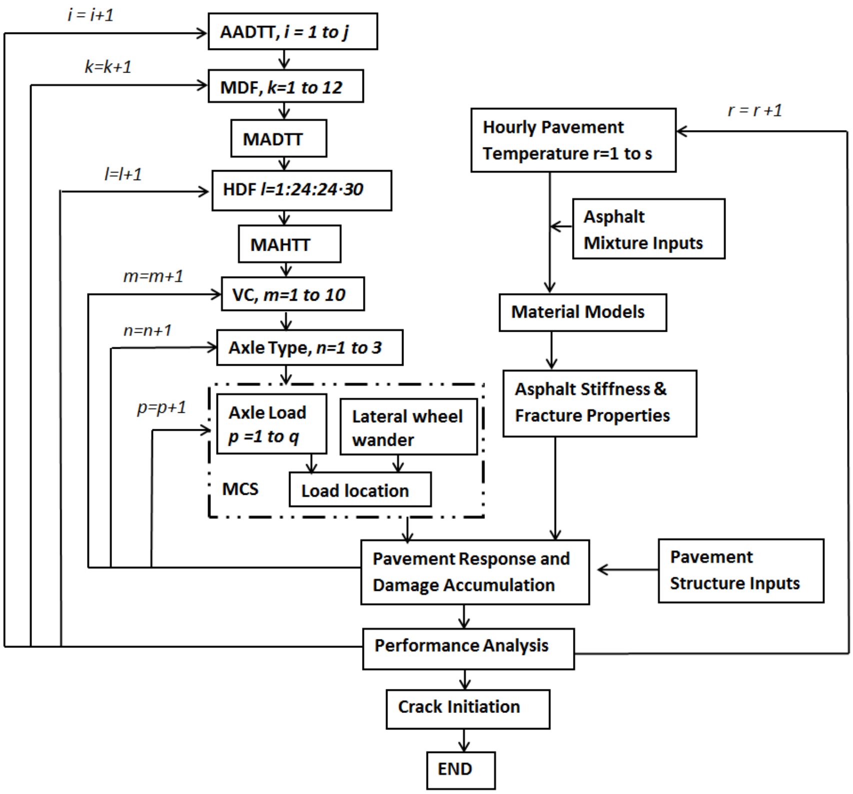

Figure 11. Flowchart for truck traffic integration into the mechanics-based analysis framework

The use of axle load spectra (ALS) for pavement performance computations requires more time and computational resources than the ESALs approach. Thus, a design single axle loads spectra that organize the loads with 8kN bin and which includes the tandem and tridem axle load effects through superposition was developed. A constant contact pressure was assumed as this was a more realistic depiction of the actual traffic conditions (Morton et al., 2004). Figure 12 presents the crack initiation times of pavement sections which were obtained using the ESALs and axle load spectra characterizations. For all the pavement sections studied, the axle load spectra characterization resulted in early crack initiation times. Zhao et al., (2012) have also observed that top-down fatigue cracking to be more sensitive to axle load spectra traffic characterization. Nevertheless, a closer look at the result shows that for some pavement sections the ESAL characterization delivers better approximation to the observed performance than the axle load spectra characterization.

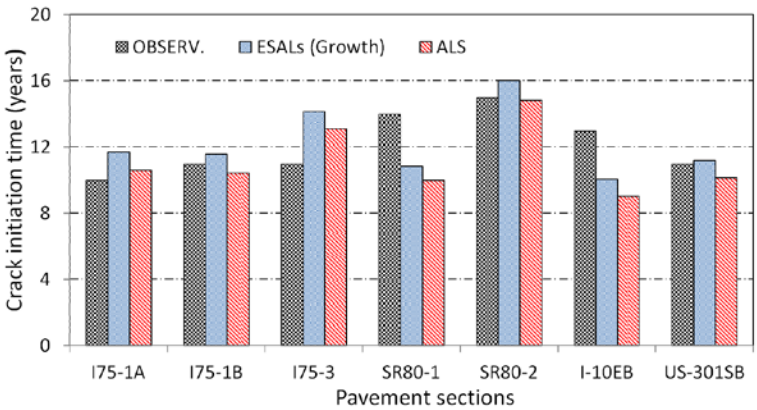

Figure 12. Impact of axle load spectra (ALS) on predicted fatigue cracking performance

The lateral wheel wander (LWW) effect was integrated into the axle load spectra traffic characterization so as to establish its impact on predicted fatigue cracking performance. A normal distribution with a standard deviation of 25.4cm was used to model the wheel wander variability, which was used as an input in the Monte Carlo simulation (NCHRP, 1999). Figure 13 presents the crack initiation times of the studied pavement sections which were obtained using axle load spectra with and without lateral wheel wander effects. It is obvious from the results that the difference between the two predictions is negligible. This anomaly can be explained by the fact that the mechanics-based analysis framework searches the maximum surface tensile stress on a wider area and changing the load location point might not have that much impact on the maximum tensile stress value.

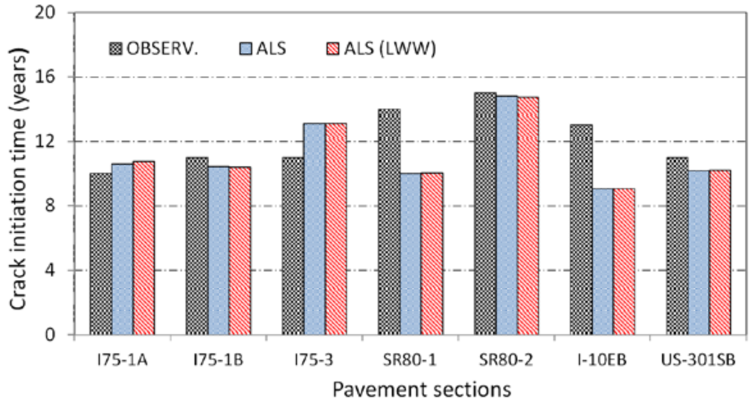

Figure 13. Impact of axle load spectra with lateral wheel wander (ALS, LWW) on predicted fatigue cracking performance

The monthly distribution factors (MDF) and the hourly distribution factors (HDF) were incorporated into the axle load spectra characterization so as to establish the impact volume adjustment factors have on predicted fatigue cracking performance. Figure 14 presents the crack initiation times of the studied pavement sections which were obtained using the axle load spectra characterization with and without volume adjustment factors.

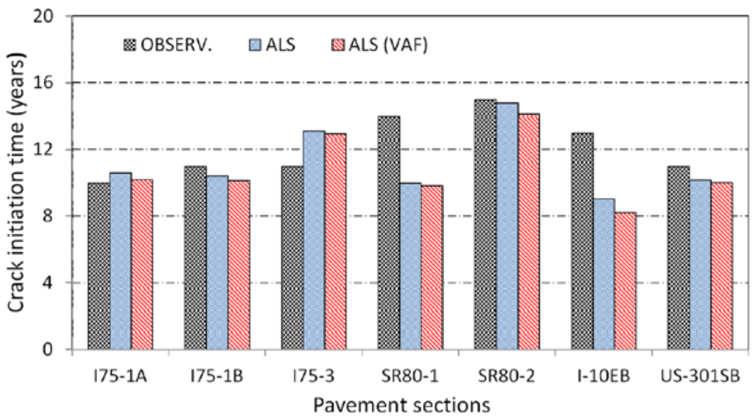

Figure 14. Impact of axle load spectra (ALS) with volume adjustment factors (VAF) on predicted fatigue cracking performance

4.3 Reliability-based calibration (Paper III)

The load and resistance factor design (LRFD) format was used to develop a mechanics-based design approach for the top-down fatigue cracking performance evaluation of flexible pavements. Figure 15 presents the flowchart for the mechanics-based design framework. The failure criterion and the design period for the design framework were determined based on field performance histories of pavement sections in Florida. A detailed evaluation of the model calibration and validation pavement sections field performance histories revealed that on average crack initiates on the 10th year and failure occurs after an additional 5 years in service. Therefore, crack imitation as a failure criterion and 15 years as a design period was used to establish the design framework.

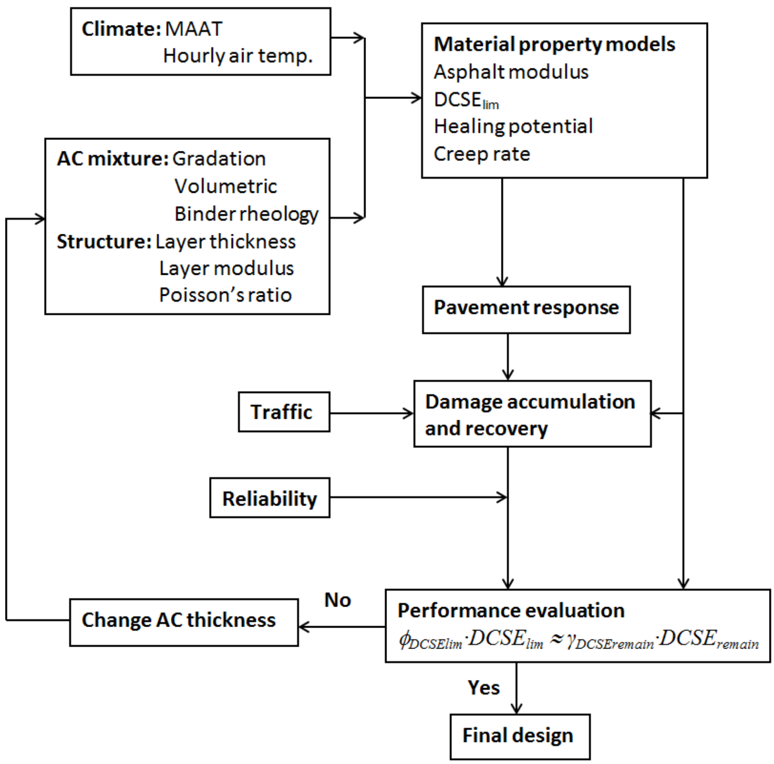

Figure 15. Flow chart for mechanics-based design framework

A two-component reliability analysis methodology as can be seen in Figure 16 was implemented in this study to compute pavement reliabilities and to formulate partial safety factors. The first component through a central composite design (CCD) based approach generates a surrogate model to represent accumulated damage. The need for response surface approach arises due to the fact that the performance function in the mechanics based analysis framework is not a closed form analytical expression of the input parameters which is due to the incorporation of multi-layered linear elastic analysis for pavement response computation. Once the performance function is established, the second component computes the associated reliability using the Rackwitz-Fiessler (R-F) algorithm.

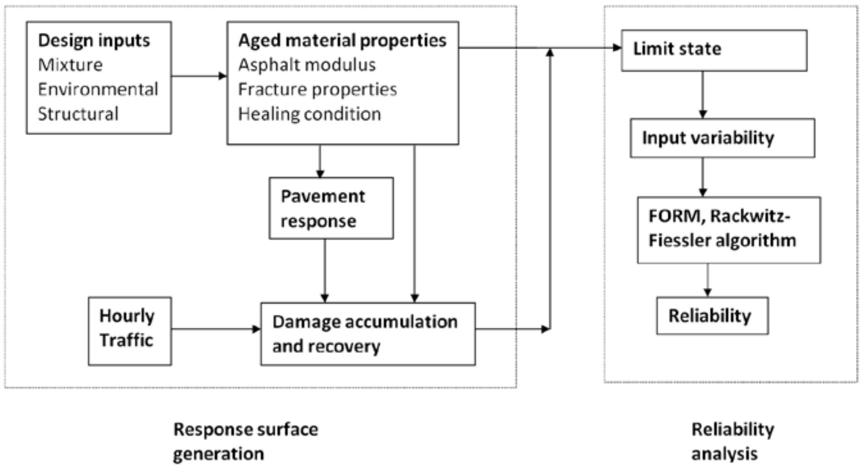

Figure 16. Two-component reliability analysis methodology

The generated surrogate model which is a second degree polynomial was checked to examine its adequacy and to ensure that it provides a good approximation to the actual model. Figure 17 presents a comparison between $DCSE_{remain}$ values which were predicted by the actual and the surrogate models. As can be seen in Figure 17, the $DCSE_{remain}$ values predicted by the RSM model are in an excellent agreement with the expected results from the actual model.

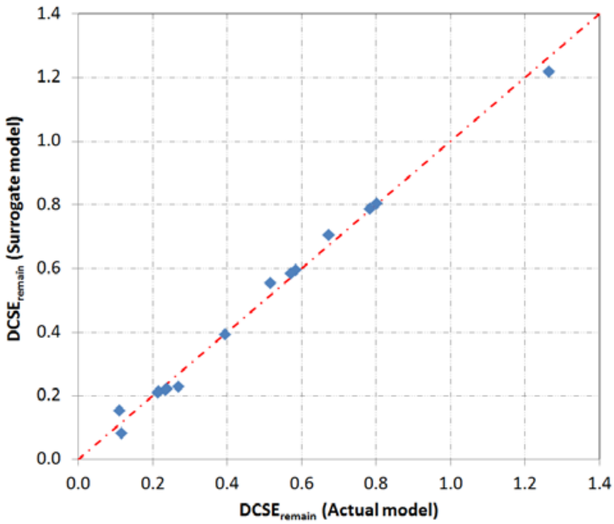

Figure 17. DCSE$_{remain}$ values as predicted by the actual and RSM models

As it was not computationally efficient to model all the parameters as random variables, a parametric study was conducted to determine the parameters which have a dominant effect on predicted top-down fatigue cracking performance. Based on the parametric study results, design inputs such as DCSE$_{lim}$, asphalt layer thickness (H_{AC}), base modulus (E_B), and traffic volume (ESALs) were modelled as random variables. The partial safety factors were formulated using field pavement sections that have a wide range in design inputs and functional requirements. Table 3 presents the computed partial safety factors for the various target reliabilities.

Table 3. Partial safety factors for the design procedure

Target Reliability (%)	$\phi_{DCSE_{lim}}$	ϕ_{Hac}	ϕ_{Eb}	γ_{ESALs}	γ_{Global}
75	0.902	0.977	0.890	1.169	1.501
80	0.882	0.958	0.89	1.244	1.682
85	0.854	0.948	0.864	1.268	1.871
90	0.823	0.941	0.843	1.422	2.255
95	0.780	0.914	0.798	1.502	2.842

Illustrative design examples were performed to show how the mechanics-based design framework can be used to design new pavement sections for top-down fatigue cracking. Section I75-1A was redesigned for various target reliabilities (75% - 95%), which was achieved by optimizing the AC layer thickness while keeping the other parameters constant. The actual reliabilities of the optimized pavement sections were obtained using the two component reliability analysis methodology. Figure 18 compares the actual and target reliabilities for these sections. As can be seen in the figure there is an excellent agreement between the two reliabilities and the discrepancy which is observed for reliabilities 75% and 80% can be explained by the fact that those partial factors were formulated using few pavement sections.

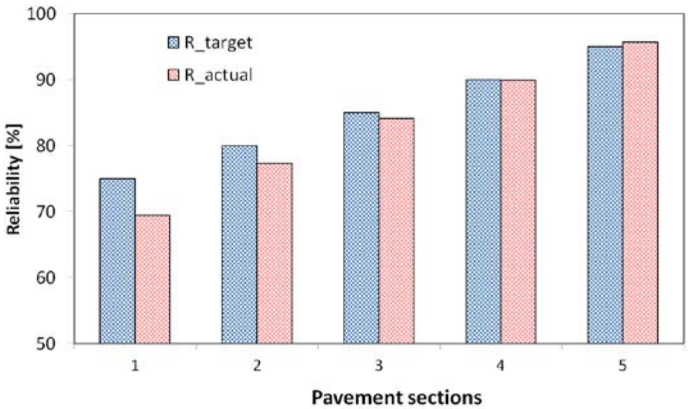

Figure 18. Comparison between target and calculated reliabilities

4.4 Evaluation of inputs variabilities (Paper IV)

The variability of pavement design inputs and parameters effect on estimated target reliability in the case when actual variabilities are different from the variability conditions assumed in the mechanics-based design procedure was investigated in Paper IV. The variabilities of the damage accumulation and the damage rate, which was defined by divided $DCSE_{remain}$ with $DCSE_{lim}$, were also investigated. Ten case studies which combine COV levels and distribution types systematically were investigated. These cases were established based on literature survey and the three COV levels used in the analyses reflect low, average and high variability conditions. The distribution functions were modelled with normal and lognormal distribution as these were the widely reported type of density functions in the literature. Table 4 and 5 present the cases which were investigated for establishing the influence of COV level.

Table 4. Design inputs variabilities

Parameter	Distribution	COV [%]		
		Case 1	Case 2	Case 3
H_{AC}	Normal	5	10	15
E_B	Normal	15	30	45
n	Normal	30	40	50
$DCSE_{lim}$	Normal	25	35	45

Table 5. Design inputs variabilities

Parameter	Distribution	COV [%]		
		Case 4	Case 5	Case 6
H_{AC}	Lognormal	5	10	15
E_B	Lognormal	15	30	45
n	Lognormal	30	40	50
$DCSE_{lim}$	Lognormal	25	35	45

A pavement section which was designed using the mechanics-based design framework for a target reliability of 90%, was used to establish the influence of the various variability cases on the target reliability. The reliability for the case when actual variabilities are different from the assumed conditions was estimated using the two component reliability analysis methodology. Figure 19 present the estimated target reliabilities for the three COV levels in the case when normal and lognormal distribution functions were used respectively to model the probability density functions. As can be seen in the figures irrespective of the type of distribution functions used, the level of COV used to model the variabilities influence the estimated target reliabilities significantly.

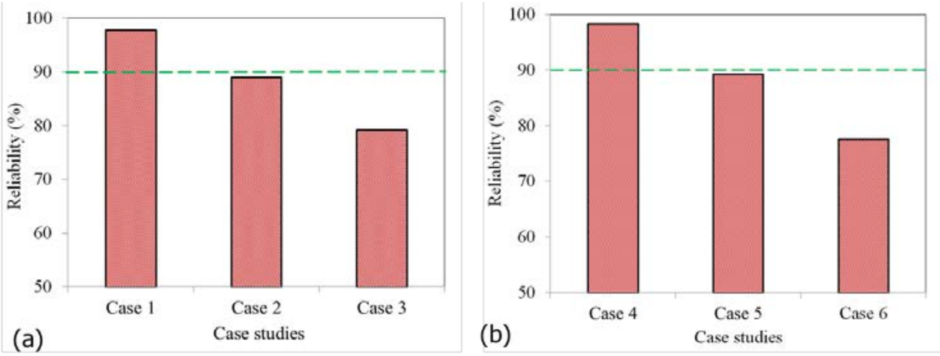

Figure 19. Influence of COV levels on target reliability: (a) normal pdf, (b) log-normal pdf

The variabilities of the damage accumulation ($DCSE_{acum}$) and damage rate were established by performing Monte Carlo simulation on the 10 cases. Figure 20 presents the Monte Carlo generated distribution of $DCSE_{acum}$ for case 7 fitted with a lognormal probability density function. A chi squared goodness of fit test which was performed on $DCSE_{acum}$ frequency distributions was not conclusive enough regarding which type of probability density function best fits the Monte Carlo generated distributions. The $DCSE_{acum}$ for practical purposes can be modelled with a lognormal distribution with a COV range of 39.13%-90.33%.

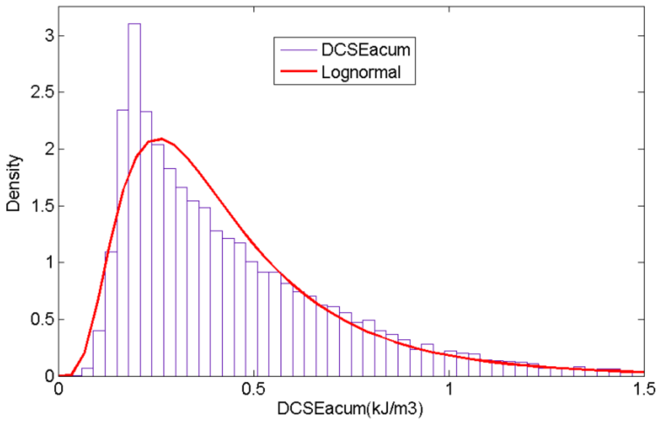

Figure 20. Monte Carlo generated $DCSE_{acum}$ fitted with log-normal distribution

5. Conclusions

A mechanics-based design framework for the top-down fatigue cracking performance evaluation of flexible pavements was presented in this thesis. The design framework was developed by performing a reliability calibration on the mechanics-based analysis model, which is a crack initiation prediction model developed on the basis of HMA-FM and mixture morphology. Furthermore, traffic characterization parameters and design inputs variabilities effect on predicted performance was evaluated and presented. The following conclusions have been made based on the findings from this thesis.

- The asphalt mixture morphology based models have been successful in explaining and predicting the evolution in $DCSE_{lim}$ and mixture healing potential with age.
- The mechanics-based analysis framework can predict crack initiation times that are in a good agreement with observed field performances.
- The axle load spectra traffic characterization induced more damage and resulted in early crack initiation times while the impact of the lateral wheel wander on predicted performance was relatively insignificant.
- For practical purposes, the use of ESALs is acceptable as it has been observed for some pavement sections in delivering better approximation to the observed field performance than the axle load spectra characterization.
- The methodology implemented for the reliability computation and partial safety factors formulation proves to be efficient in handling the uncertainty of the design inputs and their overall effect on predicted performance.
- The CCD based surrogate model which was employed to generate $DCSE_{remain}$ was shown to provide an excellent approximation to the true model while the FORM based R-F algorithm was shown to be very applicable for pavement reliability evaluation.
- The suggested LRFD framework for the mechanics-based design framework was successful in delivering designs of uniform reliability while considering the inherent variability of the design inputs. It can be used to supplement existing procedures for the design of new,

existing and rehabilitated pavement sections for top-down fatigue cracking.

- The analyses have shown that the COV values used in modelling the variabilities of input parameters have a considerable influence on estimated target reliability irrespective of the probability density functions used.
- The variability of pavement life can be modelled best with both normal and lognormal distribution functions with a wide range in COV values. In the case of accumulated damage, the variability was observed to follow a relatively narrow COV range and neither normal nor log-normal distribution functions succeeded in fitting the Monte Carlo generated frequency spectrum. Nevertheless, for practical purposes this variability can be assumed to follow a lognormal distribution function.

6. Recommendations for future studies

The modelling of top-down fatigue cracking failure in asphalt pavements is a very difficult task as it involves many factors, material properties and design conditions. For the mechanics-based analysis framework development many simplifications and assumptions were made which are expected to affect the accuracy of predicted results. The morphology-based models were developed empirically using a packing theory framework which does not consider the shape, texture and mineral content of the aggregates. Pavement response is computed using a multi-layered linear elastic analysis tool which ignores the viscous properties of the asphalt layer and the nonlinear behaviour of the unbound layers. Therefore, the mechanics-based analysis framework needs to be enhanced with the incorporation of the above mentioned issues and other important factors that might affect the predicted results.

The reliability calibration was achieved using a limited number of field pavement sections, which may not be enough to represent the many design conditions and variabilities that exist in practice. Furthermore, the variabilities of the design inputs which are considered to have a dominant influence on predicted performance was established using a literature survey that might not be large enough to characterise the actual variabilities of these design inputs. It is highly recommended that a calibration which is based on a large set of representative pavement sections is necessary in order to make the mechanics-based design framework more robust and accurate.

The database which was used to obtain the traffic characterization inputs for Florida conditions needs to be extended so as to reflect accurately the state-wide traffic variations. Further analyses including seasonal climate variation effect on unbound granular layers and subgrade properties might be necessary to fully assess and understand the impact traffic volume adjustment factors have on predicted fatigue cracking performance. Moreover, a recalibration of the mechanics-based analysis framework using the axle load spectra approach is necessary in order to capture accurately the impact axle load spectra and its associated parameters have on top-down fatigue cracking performance.

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PAPER I

Dinegdae, Y., Onifade, I., Jelagin, D., & Birgisson, B., 2015. Mechanics-based top-down fatigue cracking initiation prediction framework for asphalt pavements, *Road Materials and Pavement Design*, 16(4) pp. 907-927

PAPER II

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PAPER III

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PAPER IV

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