

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**EVALUATION OF EXISTING MECHANISTIC-
EMPIRICAL PAVEMENT DESIGN GUIDE WITH
DIFFERENT DESIGN PARAMETERS**



by
Mohammad Razeq SHAKHAN

March, 2021
İZMİR

EVALUATION OF EXISTING MECHANISTIC- EMPIRICAL PAVEMENT DESIGN GUIDE WITH DIFFERENT DESIGN PARAMETERS

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**by
Mohammad Razeq SHAKHAN**

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İZMİR

Ph.D. THESIS EXAMINATION RESULT FORM

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Mohammad Razeq SHAKHAN

EVALUATION OF EXISTING MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE WITH DIFFERENT DESIGN PARAMETERS

ABSTRACT

The Mechanistic-Empirical Pavement Design Guide (MEPDG) as the most advanced and sophisticated pavement design method was developed based on the United States conditions. Therefore, to implement this method for local conditions in other parts of the world, efforts such as local data collection, local calibration, sensitivity analysis, and local calibration should be conducted. The objective of this study was to determine the impact of different local design parameters (pavement layer thickness, HMA mixture properties, subgrade type, vehicle speed, binder grade as well as threshold values and reliability levels) on flexible pavement performance to facilitate the local calibration of the MEPDG in Izmir, Turkey. Therefore, in the first step, local data (traffic, material properties, pavement structures, and climate data) were collected and converted to the MEPDG format. In the second step, the sensitivity of local design inputs on the flexible pavement performance was determined using AASHTOWare Pavement ME Design Software 5.2.2. The analysis results revealed that: The layer thickness and vehicle speed have a strong effect on predicted pavement distresses. Total rutting and alligator cracking significantly reduced and top-down cracking highly increased with stiffer subgrade. The air voids (V_a) and effective binder content (V_{be}) in the wearing course have a strong effect on AC rutting, top-down and thermal cracking, and alligator cracking, while the alligator cracking was significantly affected from V_a and V_{be} in the AC base course. The pavement layer thicknesses increased significantly in the presence of lower threshold values and higher reliability than higher threshold values and lower reliability.

Keywords: Mechanistic-Empirical Pavement Design Guide, pavement performance, air voids, effective binder content, subgrade, threshold values

MEVCUT MEKANİSTİK-AMPİRİK ÜSTYAPI TASARIM REHBERİNİN FARKLI TASARIM PARAMETRELERİ İLE İNCELENMESİ

ÖZ

En gelişmiş ve sofistike üstyapı tasarım yöntemi olan Mekanistik-Ampirik Üstyapı Tasarım Rehberi (MEPDG), Amerika Birleşik Devletleri koşullarına göre geliştirilmiştir. Bu nedenle, bu yöntemi dünyanın diğer bölgelerindeki yerel koşullar için uygulamak istersek, yerel veri toplama, yerel kalibrasyon, duyarlılık analizi ve yerel kalibrasyon gibi çalışmalar yapılmalıdır. Bu çalışmanın amacı, farklı yerel tasarım parametrelerinin (üstyapı tabaka kalınlığı, bitümlü sıcak karışım özellikleri, taban zemin tipi, taşıt hızı, bağlayıcı miktarı ve ayrıca eşik değerleri ve güvenilirlik seviyeleri) İzmir'deki iklim koşullarına göre, esnek üstyapı performansı üzerindeki etkisini belirlemektir. Bu nedenle, ilk adımda yerel veriler (trafik, malzeme özellikleri, üstyapı tabakaları ve iklim verileri) toplanmış ve MEPDG formatına dönüştürülmüştür. İkinci adımda, yerel tasarım girdilerinin esnek üstyapı performansı üzerindeki duyarlılığı AASHTOWare Pavement ME Tasarım Yazılımı 5.2.2 kullanılarak belirlenmiştir. Analiz sonuçları; tabaka kalınlığı ve taşıt hızının, tahmin edilen üstyapı bozukluklarını üzerinde güçlü bir etkiye sahip olduğunu göstermiştir. Daha sert taban zemini koşullarında toplam tekerlek izi ve timsah sırtı çatlakları önemli ölçüde azalmış ancak boyuna çatlaklar oldukça artmıştır. Aşınma tabakasındaki hava boşlukları (V_a) ve efektif bitüm oranı (V_{be}), AC tekerlek izi, boyuna çatlaklar ve termal çatlaklar üzerinde güçlü bir etkiye sahiptir. Timsah sırtı çatlaklar bitümlü temel tabakasının V_a ve V_{be} 'nından önemli ölçüde etkilenmiştir. Daha yüksek eşik değerlerine ve daha düşük güvenilirliğe kıyasla, daha düşük eşik değerleri ve daha yüksek güvenilirlik koşullarında üstyapı tabaka kalınlıkları önemli ölçüde artmaktadır.

Anahtar Kelimeler: Mekanistik-Ampirik Üstyapı Tasarım Rehberi, üstyapı performansı, hava boşluğu, efektif bitüm oranı, taban zemini, eşik değerleri

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CHAPTER ONE

INTRODUCTION

Pavement structural design is the most complex task. Because the pavement exposes under various environmental conditions (e.g., from subzero cold to extreme hot and from dry to saturated moisture conditions) and works under different combination of traffic loading such as tire pressure, tire types, axle types, axle spacing, axle wandering, and axle loads distributions that vary throughout the day, month, and season over the whole pavement design life. The response of pavement materials to loads are also quite complex and depends on temperature, moisture, loading rate, traffic volume, and other factors. That is why the American Association of State Highway (AASHO) developed the empirical pavement design method in 1962 which was titled the AASHO Road Test. The empirical pavement design methods were updated several times in past decades, such as American Association of State Highway and Transportation Officials (AASHTO) 1972, AASHTO 1986, and AASHTO 1993. The AASHTO pavement design guides cannot be updated easily because the empirical design equations are strictly depended to original test conditions that were carried out between 1958-1960. In the other hand, the empirical pavement design methods have no enough capability to adopt significant improvements in pavement engineering, design, improved material properties, and new technologies in the construction field (AASHTO, 1972; AASHTO, 1986; AASHTO, 1993; Schwartz & Carvalho, 2007; Timm, Roobin, Tran, & Rodezno, 2014).

The improvement of material properties and performances, growing of traffic, increasing of tire pressure and axle weight, various climatic conditions, advanced construction methods, developing of personal computers and accumulation of knowledge and experiences provided enough technical infrastructure and motivation to develop a realistic and rational method for pavement structure analysis and design. Therefore, The American Association of State Highway and Transportation Officials

(AASHTO) and National Cooperative Highway Research Program (NCHRP) developed the most advanced pavement analysis and design method which is currently called the Mechanistic-Empirical Pavement Design Guide (MEPDG) in 2004. The MEPDG consist of two parts: In the mechanical part calculates pavement structural responses (e.g., stresses, strains, and deflections) based on engineering mechanics law. In the empirical part a series of empirical transfer regression equations are used to predict the pavement performances [e.g., total rutting, asphalt concrete (AC) rutting, bottom-up (alligator) cracking, top-down (longitudinal cracking, and thermal (transvers) cracking as well as international roughness index (IRI). Rutting in all layers including subgrade is called total rutting. Rutting withing asphalt layers is called AC rutting. Load related fatigue cracking which starts from the bottom of the lowest asphalt layer due to tensile strain is called bottom-up (alligator) cracking. Load related cracking which starts from top of the asphalt layer (wearing course) due to shear stresses is called top-down (longitudinal) cracking. Non-load related cracking which occurs due to repetition of cool/heat cycles in AC layers is called thermal (transverse) cracking (NCHRP, 2004a; Timm et al., 2014).

After the development of the MEPDG, state agencies in the United States of America (USA) and countries have tried to implement this method through conducting extensive research such as local data preparation (i.e., local data collection, local data analysis, and converting them to the MEPDG format, sensitivity analysis, evaluation, and validation).

1.1 Research Goal

Turkey like other countries wants to move from the existing pavement design method which was developed based on the AASHTO 1993 to the implementation of the MEPDG. In this regard, some initial works have already been done by the Turkish General Directorate of Highway like the obtaining of resilient modulus (M_r) of unbound materials, which is a crucial design input in the MEPDG (General Directorate

of Highways, 2008). The MEPDG was developed and calibrated based on the USA conditions, therefore, the direct implementation of this method for local conditions may not be suitable. Thus, the implementation of the MEPDG requires extensive efforts such as local data collection (e.g., traffic, materials properties, pavement structure, pavement distresses, and climate data) and converting them to the MEPDG format that can be used as design inputs, sensitivity analysis of local design inputs, evaluation of the pavement performance prediction models, local calibration if need be, validation, and determination of pavement performance threshold values and reliability levels (AASHTO, 2010; Pierce & Ginger, 2014).

The main objective of this thesis is to determine the level of sensitivity of local design inputs [e.g., pavement layer thickness, effective binder content (V_{be}), air voids (V_a), aggregate gradation, subgrade type, hot mixed asphalt (HMA) mixture types (e.g., dense-graded and stone mastic asphalt (SMA) mixtures), binder grades as well as pavement performance threshold values and reliability levels] on flexible pavement distress [e.g., total rutting, asphalt concrete (AC) rutting, alligator cracking, longitudinal (top-down) cracking, transverse (thermal) cracking, and international roughness index (IRI)] by the aim of facilitation and boosting of the selection of local design levels (Level 1, Level 2, and Level 3), data collection, and local calibration of the MEPDG in Izmir, Turkey.

The results of this study can be used by highway agencies and municipalities for local data collection and local calibration of the MEPDG in Izmir, Turkey. Also, the result of sensitivity analysis can assist individual pavement designers to have an optimal flexible pavement design.

1.2 Research Organization

To achieve the objective of this thesis the following activities were done:

- 1- Firstly, local design data such as traffic characteristics, HMA material properties, granular material properties, subgrade material properties, and climate data were collected from various available sources such as General Directorate of Highway official published reports, Turkish Flexible Pavement Design Guide, Turkish Highway Design Specifications, and Turkish State Meteorological Service.
- 2- Secondly, the collected local data were analyzed to check the possibility of direct use in the MEPDG. Some local data (e.g., traffic and climate) were found were developed and converted to the MEPDG format.
- 3- Finally, the local design data were used as design inputs in the AASHTOWare Pavement ME Design 2.5.5 to determine and evaluate the sensitivity of local design inputs on pavement performance. The effect of local design inputs on pavement performance was investigated by changing their values and percentages in each AC layer separately, while other design inputs were kept constant. The analysis matrix consists of three traffic levels and three subgrade types. The AASHTOWare® Pavement ME Design was run once for any element of analysis matrix per each variable.

CHAPTER TWO

LITERATURE REVIEW

In this chapter, literature reviews and the previous conducted studies on the local data collection, analysis, and converting them to the MEPDG format, sensitivity analysis, evaluation of the MEPDG pavement performance prediction models, local calibration of the MEPDG, and validation were discussed.

2.1 Empirical AASHTO Pavement Design Methods

The pavement design method is called empirical when to be developed based on experience or experiment. In other words, the relationship between traffic loading, environmental impacts, material responses, and pavement distresses is built based on field observations. In the USA, most of the goods were carried by trains or barges before the 1950s. Therefore, the traffic volume was low on highways. Thus, high engineered pavement structures were not required, and the design of pavement layer thickness solely was based on experiences. In the mid-1950s by increasing the interstate roads better-engineered pavement structures and design standards were highly required. These requirements were met by the American Association of State Highway Officials (AASHO) and were established an empirical regression equation that represents the relationship between the number of traffic load cycling and pavement structural strength, which was measured in terms of serviceability by carrying out the most important and famous road test between 1958 to 1960 in Ottawa and the Illinois States in the USA. The AASHO Road Test formed the base of AASHTO Pavement Design Guides published from 1962 to 1993 (AASHTO, 1993; Highway Research Board, 1962; Huang, 2004; NCHRP, 2004a; Schwartz & Carvalho, 2007).

The AASHTO Road Test (1958-1960) task consisted of 6 loops. The subgrade was recognized as fine-grained silty clay. The soil modulus was approximately 3000 psi.,

the maximum asphaltic layer was 6 in., maximum axle load was 48000 lb., and a total of 1.114.000 axle loads were applied. The average temperate was varied from 24.5°C to -2.8°C, the annual average precipitation was 863 mm, and the average frost depth was 711 mm. The objective of the AASHO road test was to develop a correlation between the number of load repetitions, pavement structure strength, and pavement performance. The various axle loads were represented by a single axle load (18 kips) which is called the Equivalent Single Axle Load (ESAL). The pavement structural load capacity was shown by structural Number (SN). The pavement performance was represented by the Present Serviceability Index (PSI) rating from 5 to 0. The empirical regression equation to correlate the abovementioned parameters is shown in Equation 2.1 (AASHTO, 1993; Highway Research Board, 1962; Huang, 2004; Schwartz & Carvalho, 2007).

$$\log(W_{18}) = 9.36 \times \log(SN + 1) - 0.2 + \frac{\log(4.2 - p_t) / (4.2 - 1.5)}{0.4 + 1094 / (SN + 1)^{5.19}} \quad (2.1)$$

Where;

W_{18} = Accumulated 18kip ESAL for the design life

P_t = Terminal serviceability

SN = Pavement structural number

$$PSI = 5.03 - 1.91 \times \log(1 + \overline{SV}) - 1.38 \overline{RD}^2 - 0.01 \sqrt{C + P} \quad (2.2)$$

Where;

SV = Average of slope variance in the wheel paths

RD = Total rutting depth measured in (in.)

C = Surface cracking measured in (ft²/1000 ft²)

P = Surface patching measured in (ft²/1000 ft²)

The structural number (SN) was calculated by Equation 2.3.

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \quad (2.3)$$

Where;

a_1, a_2, a_3 = Pavement structural layer coefficients (i.e., surface (0.44), base (0.33), and subbase (0.11))

D_1, D_2, D_3 = Layer thicknesses (surface course, base course, and subbase course)

Further in 1972, The AASHO Road Test empirical equation for pavement layer thickness design was revised and added the following design factors (AASHTO, 1972; Schwartz & Carvalho, 2007). To consider the environmental conditions effects on the pavement structural performance the Regional Factor (R) was added to the design equation. The values for R are shown in Table 2.1. The subgrade soil support (S_i) was added design formula to consider the influence of local subgrade soil. The S_i values varies from “1” which indicates a high plastic clay to “10” that represents crashed stone materials.

Table 2.1 Proposed values for regional factor (R) (AASHTO, 1972)

The condition of subgrade material	R
Frozen in winter	0.2–1.0
Dry in summer	0.3–1.5
Wetted in spring	4.0–5.0

In the AASHTO 1972 pavement design guide the Equation 2.4 was used.

$$\log(W_{18}) = 9.36 \times \log(SN + 1) - 0.2 + \frac{\log \left[\frac{\Delta PSI}{(4.2 - 1.5)} \right]}{0.4 + 1094 / (SN + 1)^{5.19}} + \log \frac{1}{R} (S_i - 3) \quad (2.4)$$

Where;

R = Regional factor

S_i = Soil support factor

The AASHTO 1986 pavement design guide is a revised version of the AASHTO 1972, and the revisions can be cited as follows:

- Roadbed or subgrade resilient modulus which can represent better behavior of soil,
- The effect of freezing and thaw on subgrade load bearing capacity,
- Consideration of pavement drainage conditions,
- Consideration of reliability effect.

Generally, the AASHTO 1986 and 1993 pavement design guide are similar because the same design equation was used in both methods. Only some little enhancement can be counted in the AASHTO 1993 like evaluation of existing pavement, back calculation of layered moduli for prediction of structure layer coefficients, and pavement rehabilitation methods (AASHTO, 1986; AASHTO, 1993). The design equation was used in both AASHTO 1986 and 1993 is shown as in Equation 2.5.

$$\log(W_{18}) = Z_R S_o + 9.36 \times \log(SN + 1) - 0.2 + \frac{\log \left[\frac{\Delta PSI}{4.2 - 1.5} \right]}{0.4 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \log M_R - 8.07 \quad (2.5)$$

Where;

W_{18} = 18 kips ESAL which is predicted for the whole pavement design life

Z_R = Reliability factor which is selected according to reliability percentage

S_o = Combined standard error

ΔPSI = Initial serviceability index minus terminal index serviceability index

M_R = Resilient modulus of subgrade (psi)

SN = Pavement structure number

SN: Indicates the structural strength that is calculated by Equation 2.6.

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad (2.6)$$

Which m_2, m_3 = drainage coefficients for base and subbase that ranges from 0.4-1 that in original AASHTO Road Test condition were selected 1. The other parameters have been explained before. The SN concept is shown in Figure 2.1

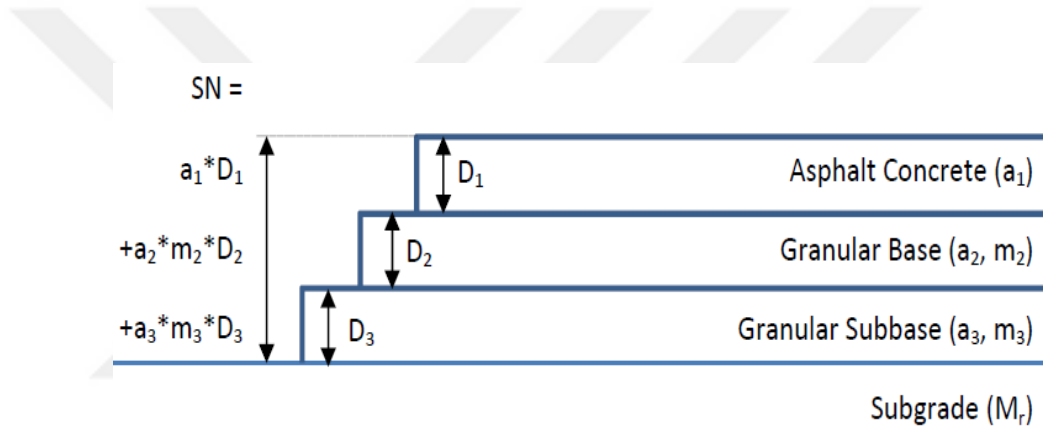


Figure 2.1 Structural number concept (Timm et al., 2014)

The Equation 2.5 is used to determine the required SN, and SN is used for the calculation of pavement layer thicknesses (Figure 2.1). The pavement thickness layers (D_1, D_2, D_3) are calculated through SN and layer coefficients (a_1, a_2, a_3) (AASHTO, 1993).

$$D_1 \geq \frac{SN_1}{a_1} \quad (2.7)$$

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \quad (2.8)$$

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 m_2 D_2}{a_3 m_3} \quad (2.9)$$

Structural layer coefficient (a_i), one of the major improvements of AASHTO 1986 is the characterizing of subgrade modulus. But resilient modulus for HMA and base/subbase unbound materials are used in term of structural layer coefficient (a_i), because the AASHTO 1986 and 1993 were developed based on the AASHTO Road Test that structural layer coefficients was used. Structural layer coefficients are empirical values that reflect the relative load-carrying capacity of materials. The recommended values are shown in Table 2.2.

Table 2.2 Structural layer coefficient (AASHTO, 1972)

Roadbed material condition	a_i
Hot mixed asphalt	0.44
Granular base	0.13
Subbase	0.11

2.2 Turkish Flexible Pavement Design Method 2008

In Turkey, the first technical specification for designing of flexible pavement was published by KGM in 1969. Then, The General Directorate of Highways (KGM) adopted the AASHTO 1972 which was titled to the Flexible Pavement Design Guide in 1984. The current Turkish flexible pavement design guide known as Flexible Pavement Design Guide 2008 that is based on the AASHTO 1986/1993 has been implemented since 2008 (General Directorate of Highways, 2008). The pavement structural design equation that is used in the Flexible Pavement Design Guide 2008 is shown in Equation 2.10.

$$\log(T_{8.2}) = Z_R S_0 + 9.36 \times \log(SN + 1) - 0.2 + \frac{\log\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.4 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \log M_R - 8.07 \quad (2.10)$$

Where;

- T8.2 = The number of repetitions of standard axle (8.2.ton) for whole design life
 Z_R = Standard normal deviation
 S_o = Reliability factor
 ΔPSI = Serviceability loss ($p_0 - p_t$)
 M_R = Resilient modulus (psi)
 SN = Structure number (in)

The required structural number (SN) for each pavement layer is illustrated in Figure 2.2. In Figure 2.2, the SN_1 shows the required SN on the top of granular base course for the calculation of HMA layer(s) thickness, the SN_2 is the required SN for the calculation of the granular base course and AC layer(s) thickness, and for calculation of total pavement structure thickness, SN_3 is required.

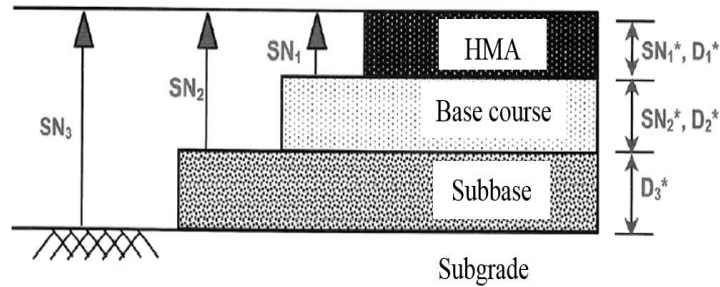


Figure 2.2 Structural number concept (General Directorate of Highways, 2008)

The thickness of layer(s) (D_1 , D_2 , and D_3) are calculated using SN, layer coefficients (a_1 , a_2 , and a_3), and drainage coefficients (m_2 and m_3). The pavement thickness is designed using Equations 2.11, 2.12, and 2.13.

$$D_1 \geq \frac{SN_1}{a_1} \quad (2.11)$$

$$D_2 \geq \frac{SN_2 - a_1 D_1}{a_2 m_2} \quad (2.12)$$

$$D_3 \geq \frac{SN_3 - a_1 D_1 - a_2 m_2 D_2}{a_3 m_3} \quad (2.13)$$

2.3 Mechanistic-Empirical Pavement Design Guide

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is an advanced and the most sophisticated pavement analysis and design method that calculates pavement structure responses and predicts different pavement performances under the combination of traffic loading and environmental conditions into consideration of various material properties. This method was developed under the NCHRP 1-37A project and calibrated using Long-Term Pavement Performance (LTPP) database in 2004 in the USA. The MEPDG was adopted by AASHTO and transferred to the software called Darwin-ME in 2008. The MEPDG consists of two models: The Mechanistic Model calculates pavement responses (stresses and strains) based on engineering mechanics law. The Empirical Model predicts pavement distresses [rutting, top-down, bottom-up, and thermal cracking as well as the International Roughness Index (IRI)] using transfer regression equations. The MEPDG adopted extensive design inputs such as traffic, material properties, climate (AASHTO, 2008; NCHRP, 2004d; Schwartz & Carvalho, 2007; Timm et al., 2014).

2.3.1 The MEPDG Design Procedure

The MEPDG is not like the AASHTO 1993 to design the pavement thickness directly using design equation. In this method the pavement layers thicknesses are analyzed and designed through an iterative process. The pavement layers thicknesses are acceptable when the predicted pavement distresses are lower than preselected pavement distresses at specific reliability level. To achieve an acceptable pavement layers thicknesses, the material properties and pavement layers thicknesses are changed. The general pavement design procedure is shown in Figure 2.3. The

pavement analysis and design steps are explained as follows (AASHTO, 2008, 2015; NCHRP, 2004a; Timm et al., 2014):

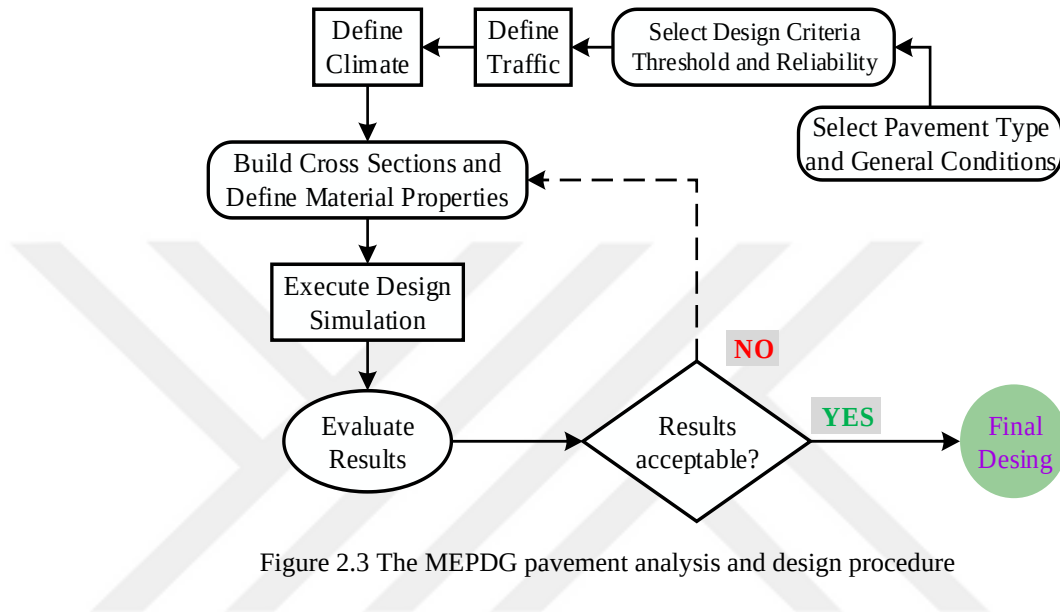


Figure 2.3 The MEPDG pavement analysis and design procedure

- 1) Selection of pavement type and general information: This is the first stages that designer can select the pavement type to be designed and select the pavement design life.
- 2) Establishment of pavement performance thresholds values: In this stage, the targeted pavement distresses or threshold values are selected. The pavement performance limit values are selected based on the local highway agency standards and experiences. The following distresses design criteria threshold values in a flexible pavement are selected in the MEPDG.
 - a) International Roughness Index (IRI), m/km
 - b) Total Rutting, mm
 - c) Thermal Cracking, m/km
 - d) Top-down Cracking, m/km
 - e) AC Rutting, mm
- 3) Selection of reliability levels: In this stage the reliability level can be selected. The reliability level percentage can be same for all pavement distresses or different

reliability level can be selected for each of distress. The reliability level percentage is selected based on highway importance level and accuracy of local calibration.

- 4) Definition of traffic data inputs: In this step traffic data are selected. The MEPDG requires extensive detailed traffic data such as traffic volume, truck monthly and hourly adjustment factors, vehicle class distribution factors, traffic growth rate, axle load distribution factors, and axle configurations.
- 5) Definition of climate data: The MEPDG requires hourly climate data (e.g., temperature, wind speed, sunshine, precipitations, and humidity) and seasonal or annual groundwater table depth to predict the impact of climate conditions on pavement performance using the Enhanced Integrated Climatic Model (EICM).
- 6) Modeling of pavement structure cross section and definition of material properties: In this stage the required pavement layer types and numbers (e.g., HMA layer(s), granular layer(s), and subgrade) and primarily layers thicknesses are selected. Also, the necessary material properties (e.g., V_a , V_{be} , Aggregate gradation, Binder grade, Poisson's ratio, etc. for HMA materials, Resilient modulus, and Poisson's ratio for unbound materials and subgrade) are defined.
- 7) Execution of simulations: After the definition and establishment of all above activities, the software is run to calculate the pavement response (stresses and strains) and predict the pavement distresses. The MEPDG calculates pavement responses under the combination of environmental effects and traffic loading into consideration of material properties and layers thicknesses. The calculated stresses and strains are used for the prediction of incremental damage over time. Then the incremental damages are transferred to pavement distresses using transfer equations.
- 8) Evaluation of results: In this stage the predicted pavement distresses are compared with targeted distresses values at selected reliability level. If any of the predicted distresses are lower than threshold values, the pavement design is rejected. But, if the predicted distresses are lower than pavement performance limit values, the design may be approved or rejected, because the main goal is to achieve an optimized pavement design. It means that the pavement design can be overestimate

for some pavement distresses while optimized for the rest of distresses. The optimized design can be achieved through adjusting of various design inputs but this step can be time-consuming and daunting task for some designers, because there are more than 100 variables with different influence on different pavement distresses. Also, it is possible that one design input in different layers has different impact on pavement distresses. However, to address this problem sensitivity analysis is required to be conducted to identify the level of sensitivity of local design inputs on pavement performance and further classify them based on their degree of sensitivity. A comprehensive sensitivity analysis may guide local designers to achieve optimized design easily.

2.3.2 The AASHTOWare Pavement ME Design

In 2004, based on the NCHRP 1-37A project the MEPDG and its software were developed. The beta-test version of the software (MEPDG v-0.9) was freely available. In 2007, based on the NCHRP 1-40D project the MEPDG was improved and the MEPDG software v1.0 was developed. In 2008, the MEPDG was transferred to the AASHTOWare series of programs and was called DARW in-ME (AASHTO, 2008). In 2013, the first commercial version of the MEPDG software was released and called AASHTOWare Pavement ME Design v1.0 (AASHTO, 2015). Furthermore, the AASHTOWare Pavement ME Design was improved in 2014, 2016, 2018 (See Table 2.4). The AASHTOWare Pavement ME Design v2.5.5 was released in 6/27/2019 (ARA Inc., 2020) and used in this research. The MEPDG software names, versions, and date of developments are listed in the Table 2.4.

Table 2.3 The MEPDG software development history

Software name and version	Release date
MEPDG v-0.9	2004
MEPDG v-1.0	2007
MEPDG v-1.1	2009

Table 2.3 continues

AASHTO Darwin ME	2011
AASHTOWare Pavement ME Design v1.0	07/01/2013
AASHTOWare Pavement ME Design v2.0	07/01/2014
AASHTOWare Pavement ME Design v2.3	07/01/2016
AASHTOWare Pavement ME Design v2.5	07/01/2018
AASHTOWare Pavement ME Design v2.5.2	08/30/2018
AASHTOWare Pavement ME Design v2.5.3	10/16/2018
AASHTOWare Pavement ME Design v2.5.4	04/08/2019
AASHTOWare Pavement ME Design v2.5.5	06/27/2019

2.3.3 Hierarchical Design Inputs

Three levels of design inputs have been established in the MEPDG based on the quality of data availability and project importance. As the MEPDG uses extensive detailed design inputs and sometimes it can be difficult that data meet the expectation, hence the hierarchical design input approach makes it easy for pavement designers to choose the national or regional default data depending on the need and the importance of the project. The design and analysis procedures and the pavement distresses prediction models are the same for all design inputs levels (AASHTO, 2008, 2010; NCHRP, 2004d). The hierarchical design is shown in Table 2.4.

Table 2.4 Hierarchical level and level of accuracy of design inputs.

Level	Level of accuracy	Input parameters
Level 1	Highest	Site features
Level 2	Intermediate	Data from agencies and predicted equations
Level 3	Lowest	Regional or global default values

Level 1: Site specific and laboratory measured material properties are required (truck class, axle load distribution, dynamic modulus, nonlinear-resilient modulus as

well as climate data). This level is proposed for heavily trafficked roads or when high reliability is required.

Level 2: Design inputs are estimated through predictive equations [dynamic modulus through Witczak predictive equation and resilient modulus is estimated from CBR values or R-values or AASHTO layer coefficient (a_i)]. This level is recommended to be used for intermediate projects.

Level 3: Design inputs are used from software database or regional default values such as (resilient modulus is selected according to soil classification, default vehicle class distribution, axle load distribution, hourly and monthly distribution, and climate data). This level is recommended for minor project of low traffic loads.

2.3.4 Pavement Performance Threshold Values and Reliability Levels

In the MEPDG, pavement performance threshold values are used as measurement scales for the predicted pavement performance to ensure that pavement will perform satisfactorily for the entire design life. The performance design criteria are determined based on the local conditions, engineering judgment, pavement management criteria, sensitivity analysis, and functional class (i.e., a higher reliability to minimize the consequence of early failure on more heavily trafficked routes). Within the context of the MEPDG, reliability of a given design is the probability that the performance of the pavement predicted for that design will be satisfactory over the time period under consideration. To describe the reliability with an example, if 10 projects were designed and constructed using AASHTOWare Pavement ME Design at 90% reliability level, 9 out of 10 projects will perform satisfactorily for the whole design life in terms of total rutting. In other word, the total rutting of one project will exceed the design limit values (AASHTO, 2008; NCHRP, 2004d; Xiao, Wang, & Qiu, 2011).

2.3.5 Pavement Performance Prediction Models

The MEPDG uses different pavement distress prediction models to predict incremental damages and pavement surface distresses such as rutting (e.g., AC rutting, base and subbase rutting, and subgrade rutting), fatigue cracking [e.g., bottom-up (alligator) and top-down (longitudinal)], and thermal (transvers) cracking. Also, the MEPDG predicts the International Roughness Index (IRI). The pavement distresses prediction models are discussed as follows.

2.3.5.1 Permanent Deformation (Rutting) Model

Rutting is a vertical consolidation and/or plastic deformation on flexible pavements which occurs in the wheel path due to traffic loading repetition and appears as small upheavals to the sides. It is one of significant distresses in flexible pavements which is highly dependent up on layer thickness, material quality, traffic repetition, traffic speed, and environment conditions. The MEPDG predicts rutting in each pavement layer. The permanent deformation in the HMA layer(s) is called AC rutting. The sum of the rut in layers (e.g., AC layers and unbound layers) and in subgrade is known as total rutting (AASHTO, 2020; NCHRP, 2004d). The total rutting is calculated using Equation 2.14.

$$RD_{Total} = RD_{AC} + RD_{gb} + RD_{sg} \quad (2.14)$$

Where;

RD_{Total} = Total rutting depth, mm

RD_{AC} = Rutting depth in HMA layer(s), mm

RD_{gb} = Rutting depth in unbound granular layer(s), mm

RD_{sg} = Rutting depth in subgrade, mm

2.3.5.1.1 *Asphalt Concrete (AC) Rutting Model.* Rutting in HMA layers is one of the most important depression types in flexible pavements. In the MEPDG, to calculate the vertical strain (ϵ_r), the AC layer is subdivided into sublayers. Then, the vertical resilient strain is used in the empirical prediction equation (Equation 2.14) to estimate the vertical plastic strain (ϵ_p). The rutting in AC layer(s) is predicted using Equation 2.15 (AASHTO, 2020; NCHRP, 2004d).

$$\Delta_{p(AC)} = \epsilon_{p(AC)} h_{AC} = \beta_{1r} k_z \epsilon_{r(AC)} 10^{k_{1r}} n^{k_{3r} \beta_{3r}} T^{k_{2r} \beta_{2r}} \quad (2.15)$$

Where;

- $\Delta p(HMA)$ = Rutting depth in HMA layer(s), in.
- $\epsilon_p(HMA)$ = Vertical plastic axial strain in the HMA layer(s), in./in.
- $\epsilon_r(HMA)$ = Elastic strain, in./in.
- $h_{(AC)}$ = Thickness of the AC layer/sublayer, in.
- n = Number of axle load repetitions
- T = Mix temperature, °F
- K_z = Depth of confinement factor
- $K_{1r,2r,3r}$ = National laboratory-derived coefficients for AC mixtures
($k_{1r}=2.4545$, $k_{2r}=3.01$, $k_{3r}=0.22$)
- $\beta_{1r}, \beta_{2r}, \beta_{3r}$ = Local calibration, as default, $\beta_{1r}=0.4$, $\beta_{2r}=0.52$, and $\beta_{3r}=1.36$

$$k_z = (C_1 + C_2 D) 0.328196^D \quad (2.16)$$

$$C_1 = -0.1039(H_{AC})^2 + 2.4868H_{AC} - 17.342 \quad (2.17)$$

$$C_2 = -0.0172(H_{AC})^2 - 1.733H_{AC} + 27.428 \quad (2.18)$$

Where;

D = Depth below the surface, in.

H_{AC} = Total AC thickness, in.

2.3.5.1.2 *Unbound Granular Materials Rutting Model*. In the MEPDG, granular material layers (e.g., base and subbase) are subdivided into sublayers. Therefore, the total rutting in each layer is equal to sum of permanent deformation in sublayers. The rutting in unbound granular layers are predicted using Equation 2.19.

$$\Delta_{gb} = \beta_{GB} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{n}\right)^\beta} \quad (2.19)$$

Where;

Δ_{gb} = Permanent deformation for the layer/sublayer, in.

n = Number of axle load repetitions

ε_0 = Intercept determined from laboratory repeated load permanent deformation tests, in/in.

ε_r = Resilient strain imposed in laboratory test to obtain material properties ε_0 , β , and ρ , in/in.

ε_v = Calculated average vertical elastic strain in the layer/sublayer, in/in.

h_{soil} = Thickness of unbound layer/sublayer, in.

k_{s1} = Global coefficients and are material dependent; $k_{s1} = 0.965$ for aggregate base granular materials, 0.965 for coarse-grained soils, 0.635 for sandy soils, and 0.675 for fine-grained soils

β_{s1} = Local calibration factor, in default condition $\beta_{s1} = 1$

$$\text{Log } \beta = -0.6119 - 0.017638(W_c) \quad (2.20)$$

$$W_c = 51.712 \left[\left(\frac{M_r}{2555} \right)^{\frac{1}{0.64}} \right]^{-0.3586 * GWT^{0.1192}} \quad (2.21)$$

$$\rho = 10^9 \left(\frac{c_0}{1 - (10^9)^\beta} \right)^{\frac{1}{\beta}} \quad (2.22)$$

$$C_0 = Ln \left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}} \right) \quad (2.23)$$

Where;

W_c = Water content (%).

E_r = Resilient modulus of the layer/sublayer (psi).

GWT = Ground water table depth (ft).

2.3.5.1.3 *Subgrade Rutting Model*. The rutting in subgrade is predicted using Equation 2.24.

$$\Delta_{sb} = 1.35 \beta_{SG} \varepsilon_v h_{soil} \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{n}\right)} \quad (2.24)$$

Where;

β_{SG} = Local calibration factor, in condition $\beta_{SG}=1.673$

2.3.5.2 *Load Related Cracking*

Load related fatigue cracking is one of the most important distress in flexible pavement. The action of repeated traffic loads causes tensile and shear stresses in the AC layers, which ultimately leads to the loss of the structural integrity of a stabilized layer. Although, the stresses and strains induced by traffic loading are lower than AC strength but the repeated loading causes fatigue in AC layer which finally leads to

cracking in the flexible pavement. Generally, it is well agreed in pavement design that traffic repeated loading bends the AC layer and creates flexural bending stresses/strain at the bottom of the AC layer. The repeated flexural bending stresses cause cracking at the bottom of the AC layer, which propagates upward and appears as an alligator cracks at the surface of the AC layer. Also, worldwide researchers have agreed that fatigue cracking can be initiated on the top of the AC layer at the wheel path and propagate downward, which is called top-down cracking. The shear stresses/strains due to high contact pressure between pavement surface and tire edge coupled with extremely aged thin AC layer are the cause of top-down cracking. The MEPDG uses only damage predicts both top-down and bottom-up fatigue cracking. The fatigue cracking prediction model is based on Miner's Law, which calculates fatigue damages in top and bottom of AC layer using Equation 2.25 (NCHRP, 2004d). Then the fatigue damage is used in empirical prediction equations model to estimate the bottom-up and top-down cracking.

$$DI = \sum \left(\frac{n}{N_{f-AC}} \right)_{j,m,l,p,T} \quad (2.25)$$

Where;

- DI = Incremental damages
- j = Axle load interval
- m = Axle load type
- l = Truck type
- p = Month
- T = Median temperature for the five temperature intervals
- n = Actual traffic repetition i .
- N_{f-AC} = Allowable traffic load repetitions

The allowable (N_i) traffic repetitions is a function of HMA mix stiffness (i.e., Dynamic modulus) and tensile strain calculated by pavement response model. In the MEPDG, the relationship between tensile strain and HMA mix stiffness are shown by Equation 2.26.

$$N_{f-AC} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{AC})^{k_{f3}\beta_{f3}} \quad (2.26)$$

Where;

- ε_t = Tensile strain at critical locations, in./in.
- E_{AC} = Dynamic modulus of HMA mixture
- $k_{f1, f2, f3}$ = National laboratory-derived model factors ($k_{f1} = 3.75$, $k_{f2} = 2.87$, and $k_{f3} = 1.46$)
- $\beta_{1f}, \beta_{f2}, \beta_{f3}$ = Local calibration factors, as default, $\beta_{f2} = 1.38$, and $\beta_{f3} = 0.88$ and β_{1f} is AC thickness dependent. For AC thickness less than 12 cm: $\beta_{1f} = 0.02050$, for AC thickness 12-30 cm, $\beta_{1f} = 5.014(H_{AC})^{-0.316}$, and for AC thickness greater than 30 cm, $\beta_{1f} = 0.001032$
- C_H = Thickness correction term, dependent on type of cracking

For bottom-up cracking:

$$C_H = \frac{1}{0.000398 + \frac{0.003602}{1 + e^{(11.02 - 3.49H_{AC})}}} \quad (2.27)$$

For top-down cracking:

$$C_H = \frac{1}{0.01 + \frac{1200}{1 + e^{(15.676 - 2.8186H_{AC})}}} \quad (2.28)$$

$$C = 10^M \quad (2.29)$$

$$M = 4.84 \left(\frac{V_{be}}{V_a + V_{be}} - 0.69 \right) \quad (2.30)$$

Where;

- ε_t = Tensile strain at critical locations, in./in.
- H_{AC} = Total thickness of the AC layers, in.
- V_{be} = Effective binder content by volume in the mixture, %
- V_a = Air voids percentage in the AC mixture, %

2.3.5.2.1 Bottom-up (Alligator) Cracking Model. Bottom-cracking or alligator cracking is a load-related cracking initiates at the bottom of the AC layer and propagates upward. Alligator cracks initially appears as multiple short, longitudinal, or transverse cracks in the wheel path that become interconnected laterally with continued truck loadings. Alligator cracking is calculated as a percent of total lane area in the MEPDG (NCHRP, 2004d). The Equation 2.31 is used in the MEPDG to predict the alligator cracking.

$$FC_{Bottom} = \left(\frac{1}{60} \right) = \left(\frac{C_4}{1 + e^{(C_1 C_1^* + C_2 C_2^* \text{Log}(DI_{Bottom} * 100))}} \right) \quad (2.31)$$

Where;

- FC_{Bottom} = Area of alligator cracking, %
- $C_{1,2,4}$ = Local calibration factors, as default, $C_4 = 6000$, $C_1 = 1$, and $C_2 = 1$
- DI_{bottom} = Incremental damage index at the bottom of the AC layers

$$C_2^* = -2.40874 - 39.748(1 + H_{HMA})^{-2.856} \quad (2.32)$$

$$C_1^* = -2C_2^* \quad (2.33)$$

2.3.5.2.2 *Top-down (Longitudinal) Cracking Model*. Longitudinal cracks initiate at the surface of the AC layer due to shear stresses/strains and propagate downward. The unit of longitudinal cracking calculated by the MEPDG is feet per mile or (meter per kilometre) including both wheel paths. The Equation 2.34 is used in the MEPDG to predict the top-down cracking (NCHRP, 2004d).

$$FC_{Top} = 10.56 \left(\frac{C_4}{1 + e^{[C_1 + C_2 \text{Log}(DI_{Top})]}} \right) \quad (2.34)$$

Where;

- FC_{Top} = Length of top-down cracks, ft/mi
- $C_{1,2,4}$ = Local calibration factors, as default, $C_4 = 1000$, $C_1 = 7$, and $C_2 = 3.5$
- DI_{Top} = Incremental damage index on the top of the AC surface

2.3.5.3 *Thermal (Transverse) Cracking Model*

Thermal cracking is a significant distress in flexible pavement that appears through a uniform, often equally spaced, and perpendicular to the direction of traffic. Generally, two major mechanisms such as low-temperature and thermal fatigue cause thermal cracking in flexible pavement. Low-temperature cracking occurs when the tensile stress due to cold temperature exceeds the tensile strength of HMA. The cracks initially appear at the surface of the AC layer and by applying additional thermal cycles, propagate downward through the entire depth of the AC layer. Thermal fatigue cracking occurs due to repetition of cool/heat cycles in the AC layer. Generally, the thermal repetitions induce lower tensile strain than AC strength but is a progressive localized fatigue damage caused by fluctuating stresses and strains in the material and a build-up of irrecoverable deformations (Al-Qadi, Hassan, & Elseifi, 2005; Biswas, Hashemian, & Bayat, 2016). In the MEPDG the thermal cracking model and transfer

function are developed according to fracture mechanics during low temperature (AASHTO, 2020; NCHRP, 2004d) as follows.

$$\Delta C = A(\Delta K)^n \quad (2.35)$$

Where;

ΔC = Change in the crack depth due to a cooling cycle

ΔK = Change in the stress intensity factor due to a cooling cycle

A, n = Fracture parameters for the AC mixture

The A and n can be obtained from the HMA mix indirect tensile strength and stiffness using Equation 2.36 and 2.37.

$$A = k_t \beta_t 10^{[4.389 - 2.52 \log(E_{HMA} \sigma_m^n)]} \quad (2.36)$$

$$n = 0.8 \left[1 + \frac{1}{m} \right] \quad (2.37)$$

Where;

k_t = Coefficient dependent to the mean annual air temperature (MAAT)

EAC = AC mixture stiffness, psi

σ_m = Fracture parameters for the AC mixture, psi

m = Laboratory coefficient derived from indirect tensile creep compliance curve

β_1 = Local calibration factor

If, MAAT > 57 °F:

$$k_t = 0.13(MAAT)^2 - 11.68(MAAT) + 244.14 \quad (2.38)$$

If, $MAAT \leq 57$ °F:

$$k_t = 3 \times 10^{-7} (MAAT)^{4.0319} \quad (2.39)$$

The stress intensity factor, K , was developed through theoretical finite element method has been incorporated in the AASHTOWare Pavement ME Design (Equation 2.40).

$$K = \sigma_{tip} \left[0.45 + 1.99(C_o)^{0.56} \right] \quad (2.40)$$

Where;

- σ_{tip} = Far-field stress from pavement response model at depth of crack tip,
psi
- C_o = Current crack length, ft

Thermal cracking amount is predicted using Equation 2.41.

$$TC = \beta_{t1} N \left[\frac{1}{\sigma_d \text{Log} \left(\frac{C_d}{H_{AC}} \right)} \right] \quad (2.41)$$

Where;

- TC = Amount of thermal cracking, ft/mil
- β_{t1} = Regression coefficient, as default, 400
- $N[Z]$ = Standard normal distribution evaluated at $[z]$
- σ_d = Standard deviation of the log of the depth of cracks in the pavement
(0.769), in.
- C_d = Crack depth, in

H_{AC} = Thickness of AC layer, in

2.3.5.4 International Roughness Index (IRI) Model

The pavement roughness (IRI) is a function of pavement distresses that accordance of pavement surface distresses results in increased roughness or reduction in smoothness. The IRI is predicted using Equation 2.42.

$$IRI = IRI_o + C_1(RD) + C_2(FC_{Total}) + C_3(TC) + C_4(SF) \quad (2.42)$$

Where;

IRI_o = Initial IRI after the pavement construction, in/mil

SF = Site factor

FC_{Total} = Area of fatigue cracking included alligator and longitudinal cracking.

TC = Length of transvers cracking, ft/mil

RD = Rutting depth, in

$C_{1,2,3,4}$ = Calibration factors, as default, $C_1 = 40.0$, $C_2 = 0.400$, $C_3 = 0.008$, and $C_4 = 0.015$

2.3.6 Design Inputs for Pavement Design

The MEPDG uses extensive detailed inputs in three hierarchical levels for the designing of the pavement structures. The required design inputs are required in four main categories (e.g., traffic characteristics, climate data, pavement structure, and material properties). The required design inputs are explained as follows.

1. Traffic characteristics. Comprehensive and detailed traffic data in four categories are used in the MEPDG, which were described as follows (NCHRP, 2004c).
 - a) Base Year Traffic Information

- Design Life
 - Opening Date
 - Initial Two Way Annual Average Daily Truck Traffic (AADTT)
 - Number of Lanes in Design Direction
 - Percent Trucks in Design Direction
 - Percent Trucks in Design Lane
 - Operational Speed
- b) Traffic Volume Adjustment Factors
- Monthly Adjustment Factors
 - Vehicle Class Distribution
 - Hourly Distribution
 - Traffic Growth Factors
- c) Axle Load Distribution Factors
- d) General Traffic Inputs
- Lateral Traffic Wander
 - Number of Axles per Truck
 - Axle Configuration
2. Climate data. The climate data such as hourly temperature, windspeed, sunshine, precipitation, humidity as well as seasonal or annual groundwater table depth are required in the MEPDG.
 3. Pavement structural information. The layer types (e.g., HMA layer(s), base/subbase layer(s), and subgrade) and thicknesses are required in the MEPDG.
 4. Materials properties. The HMA, granular, and subgrade material properties are used in the MEPDG (NCHRP, 2004b). The required material properties are explained as follows.
 - a) HMA material properties. The required HMA material properties are included, mixtures volumetric (e.g., HMA specific gravity, V_a , V_{be} , Poison's ratio), mechanical properties [e.g., dynamic modulus of HMA in level 1, aggregate gradation in level 2 and level 3, binder grade (Superpave performance grade

(PG), viscosity grade, or penetration grade in level 3, binder complex shear modulus and binder phase angle in level 1 and level 2), indirect tensile strength, and creep compliance], and thermal properties (e.g., thermal conductivity, heat capacity, and coefficient of thermal).

- b) Granular and subgrade material properties. The Poisson's ratio, coefficient of lateral earth pressure, resilient modulus, CBR, R-Value, or AASHTO layer coefficient, liquid limit, plasticity index, maximum dry unit weight, optimum moisture content, specific gravity as well as sieve analysis are required.

2.3.7 Existing Data Collection, Sensitivity Analysis and Local Calibration Efforts

As mentioned before, the MEPDG is developed and calibrated based on the USA conditions using the LTPP database and predicts the pavement distresses using empirical transfer equations, therefore, the direct implementation of this method for local conditions may not be suitable. The MEPDG is implemented for local conditions through conducting activities are explained as follows:

1. Local data preparation: In this stage, the previously constructed pavement segments are selected and local data such as pavement distresses, pavement structures, material properties, traffic characteristics as well as climate data are collected. The collected local data are needed to be analyzed because some local data are not in a suitable format to can be used as design input in the MEPDG. Therefore, this data should be improved and converted to the MEPDG format.
2. Sensitivity analysis: The sensitivity analysis is done to determine the impact and level of sensitivity of design inputs on pavement performance for local conditions. As the MEPDG, requires the design inputs in three hierarchical levels, therefore, the input level can be selected based on their sensitivity levels on pavement performance. The most sensitive design data are selected in level

- 1, the moderate can be aggregated in level 2 and low sensitive may be classified in level 3.
3. Evaluation: In this stage, the local data are used as design inputs in the AASHTOWare Pavement ME Design to predict the pavement distresses. Then, the predicted pavement distresses are compared with observed distresses. If there be any significant differences, the local calibration is required to be conducted.
4. Local calibration: In this stage, the difference (bias) between observed and predicted distresses are removed and the standard error is reduced to an acceptable level based on local highway agency requirements. The removing bias and reducing of standard errors are done through adjusting local calibration factors in the transfer functions.

After the development of the MEDPG, state agencies in the USA and countries have made efforts to implement the MEPDG for local conditions. Therefore, they collected, analyzed, and developed local data, conducted evaluation, local calibration, and validation activities. They reported that data collection and preparation were laborious and long-lasting tasks. The lack and deficiency of local design data, as well as collecting in improper formats, were a common problem in those countries. Therefore, most of the countries used some default design inputs and equivalents climate data from the (A. Elshaeb, M. El-Badawy, & A. Shawaly, 2014; Alqaili & Alsoliman, 2017; Azadi, Nasimifar, & Pouranian, 2013; Caliendo, 2012; Ceylan, Coree, & Gopalakrishnan, 2009; Chhade, Mrad, Houssami, Chehab, & Asce, 2018; Ma, Wang, Zhou, & Feng, 2015; Orobio & Zaniwski, 2011; Sadek et al., 2014; Schwartz & Carvalho, 2007; Tarefder & Rodriguez-Ruiz, 2013; Yut, Mahoney, A., & Larsen, 2017; Zhang, Wang, You, & Ma, 2015; Zhao, 2011). Using default data might not lead to proper local calibration and the accuracy level of pavement performance prediction model would not be enhanced.

In 2007, Schwartz and Carvalho conducted a sensitivity analysis to evaluate the influence of local design inputs on pavement performance. In this study, a 3-layer flexible pavement was selected and analysis was performed for three traffic levels and five different environmental conditions. The results have indicated that variations of HMA thickness have significant impact on pavement performance. Results also indicated that utilization of higher performance grade binder leads to lower fatigue cracking and rutting. The high binder content results poor rutting and high fatigue performances. Results also demonstrated that when air void increases in HMA, fatigue cracking increases and the AC rutting decreases (Schwartz & Carvalho, 2007).

In 2009, the MEPDG was evaluated using local design inputs for implementation in Iowa, USA. In this study, a three-layer flexible pavement (AC surface course, AC binder course, and unbound subbase course) was analyzed with MEPDG software to determine the impact of local design inputs on pavement distresses. The sensitivity analysis results indicated that mixture volumetric properties (V_a , V_{be} , and aggregate gradation) are very sensitive on longitudinal and thermal cracking, while they have moderate impacts on AC rutting and minor influence on alligator cracking (Ceylan et al., 2009).

In 2011, the influence of design inputs on pavement performance were highlighted by conducting a sensitivity analysis. In this study, two pavement types (pavement with four AC layers and pavement with one AC layer and two granular layers) were selected. Analysis of structure 1 demonstrated that rutting is highly affected by Poisson's ratio of wearing course and resilient modulus of the subgrade, while V_{be} of layer 4, V_a of layer 3, and subgrade resilient modulus are largely effects on cracks values. Also, results of structure 2 indicated that the air void of AC layer and resilient modulus of the subgrade is the main source of rutting, while cracking is the result of V_a and V_{be} of AC layer (Orobio & Zaniewski, 2011).

In 2011, in a sensitivity analysis study, the impact of mixture factors on HMA performance was evaluated in the Clemson University, California, USA. Two aggregate gradation from two aggregate sources with two binder contents and air voids percentage were analysed. The investigations indicated that the aggregate gradation, V_a , and V_{be} have significant impacts on rutting, while the aggregate source has less effect on rutting performance. Also, research results showed that V_a and V_{be} have a high impact on mixture cracking (Zhao, 2011).

In 2017, in another study implemented in University of Connecticut, a comprehensive sensitivity analysis was performed to prepare an implementation plan for the MEPDG. Analysis was run for three traffic levels and for three subgrade types (A-7-5, A-6, A-3). The analysis results showed that the AC layer thickness is a critical parameter for rutting, top-down and bottom-up cracking, while AC thickness has only a minor effect on thermal cracking. Also, the grade of AC binder has a moderate impact on alligator and longitudinal cracking and has major effect on rutting and thermal cracking (Yut et al., 2017).

In 2012, Caliendo attempted to conduct local calibration for alligator cracking, rutting, and IRI models to promote the implementation of the MEPDG in Italy. In this regard, some of the design data were collected and converted to the MEPDG format while some others were used as defaults. Alabama's climate data was used in Italy because it was recognized that is similar to the local conditions. Italian truck traffic was found that was different from MEPDG standard in terms of truck classification, vehicle distribution, and axle-load distribution factors. Therefore, trucks were categorized into 5 classes (e.g., 4, 5, 6, 9, and 11) and axle load distributions were calculated based on new classification. Level 3 design inputs were used for materials such as dynamic modulus ($|E^*|$), resilient modulus (M_r) of granular base/subbase and subgrade, because no forensic investigation was carried out. The local calibration of the MEPDG was carried by comparing the predicted pavement distresses by MEPDG and Italian Pavement Design Method, because the pavement performance was not

available in Italy. This calibration method was significantly different from the other countries pursued methods. The study result is shown that MEPDG would overdesign compared to the Italian method for roads with high traffic level. In contrast, the MEPDG would underestimate compared to the Italian method for roads with low traffic level (Caliendo, 2012).

In 2013, The fatigue cracking (alligator and longitudinal) prediction model was evaluated for the local conditions, climate, and materials in Tehran, Iran. Local design data such as pavement structure, material properties, and pavement performance were obtained from Tehran Municipality and confirmed with forensic investigations. Tehran climate data were obtained from Meteorological Organizations in their original formats and then converted to Enhanced Integrated Climatic Model (EICM) format for use in the AASHTO Pavement ME Design. The authors provided no information about either local traffic data was used or default traffic data (level 3 entry input). As result, it was found that default calibrated transfer functions extensively overestimated the fatigue cracking, therefore, the fatigue cracking model was calibrated and local calibration factors (e.g., $\beta_{f1} = 1.0$, $\beta_{f2} = 0.8$, $\beta_{f3} = 1.5$) were suggested (Azadi et al., 2013).

In 2013, the local calibration of the MEPDG was conducted in New Mexico. In this study, local design data such as traffic, climate, pavement structure, and material properties were obtained from the New Mexico Department of Transportation. Pavement performances were measured with the qualitative rating rather than with quantitative measures (depth of rutting or length of cracks) in New Mexico which was not suitable to be used in the MEPDG. Therefore, the pavement performances data were analyzed and converted to the proper format required by MEPDG. In this study, the pavement performance prediction models were calibrated successfully (Tarefder & Rodriguez-Ruiz, 2013).

In 2014, the Egyptian climate data were collected to evaluate the impact of Egyptian climate conditions on flexible pavement performance. Climate data such as temperature, precipitation, cloud cover, humidity, and wind speed were collected from World Weather on Line website for 16 cities in Egypt. In that website, three hourly climate data were available, therefore, climate data were converted to hourly by using linear interpolation and then converted to the EICM format. A typical flexible pavement section consists of a 5 cm AC wearing surface ($V_a=7\%$, effective binder content ($V_{be}=11.0\%$, pen. 60-70 asphalt binder), 6 cm AC binder course layer ($V_a=8\%$, $V_{be}=10.2\%$, pen. 60-70 asphalt binder), 25 cm aggregate base course layer (resilient modulus; $M_r = 290$ MPa), and subgrade soil ($M_r = 100$ MPa) was selected, and analysis were done for level 3 traffic levels. The sensitivity analysis result shows that the AC rutting value was high in the south part with hot weather compared to the north part with mild weather. Also, longitudinal cracking was affected by climate conditions. It was also identified that the alligator cracking was not sensitive to all climate conditions (A. Elshaeb et al., 2014).

In 2014, to introduce the MEPDG to the State of Qatar, analysis and evaluation was done on flexible pavements for local conditions using AASHTOWare Pavement ME Design v1.1. In this study, the Needles Airport in California climate data was used, because it was found that was similar to the local climate conditions. Bound and unbound material properties and were extracted from Qatar Highway Design Manual (QHDM), but default traffic data were used for all analysis. The influence of asphalt grades (e.g., Pen. 60-70 and PG 76-10) were evaluated on conventional flexible pavement and composite flexible pavement. As result, it was found that use of PG 76-10 significantly improved the pavement performance (Sadek et al., 2014).

In 2015, sensitivity analysis and local calibration of thermal cracking were performed to propose typical pavement structure and facilitate the implementation of MEPDG in China. In this study, three pavement segments were selected to collect local data and conduct local calibration for local conditions in the northeast area of China.

The climate stations of Duluth, Minnesota; Bismarck, North Dakota; and International Falls, Minnesota were chosen as climate stations for Changping, Qitai, and Haxihuan highways, respectively. Default axle load distribution was used instead of local axle load distribution because detailed axle load distribution was not available in China. Level 3 inputs were defined and used for materials. Sensitivity analysis was carried out in MEPDG including design temperature, layer thickness, types of base and subgrade, and air voids of asphalt concrete. The result of the study revealed that thermal cracking was increased in lower temperature and local calibration of thermal cracking model was successfully done (Ma et al., 2015).

In 2015, in another study conducted in China, 20 test sections of two highways were selected to conduct the sensitivity analysis of longitudinal cracking on material parameters. The pavement structure of all sections consists of three AC layers, a stabilized aggregate base, and a lime-ash soil subbase. The analysis was done for 20-year design life at a 90% reliability level. The study results show that the impact of V_a on the wearing course is higher than underlying layers. Also, research results indicated that influence of V_{be} on the surface layer is higher than the bottom layers, while the effect of bottom layer is little more than AC middle layer (Zhang et al., 2015).

In 2017, a comprehensive local data preparation was done in Central Saudi Arabia. In this study, local design inputs including traffic (e.g., average annual daily truck traffic, vehicle types, vehicle distribution factors, growth rate, daily adjustment factors, monthly adjustment factors, axle load distribution factors, axle per truck, axle configuration, and wheelbase), pavement structure, material properties, and pavement performances were obtained from the Saudi Ministry of Transportation and hourly climate data (e.g., temperature, precipitation, humidity, wind speed, and cloud cover) were obtained from the Presidency of Meteorology and Environment in Riyadh. In this study, Saudi Arabia vehicle classification was differed from the MEPDG truck classification. Therefore, a new vehicle classification was carried based on the MEPDG format. Default axle load distribution for class 4 (buses) was used, because

the bus axle load was not measured in Saudi Arabia. Climate data was analyzed and converted to the EICM format (Alqaili & Alsoliman, 2017).

In 2018, the traffic data (e.g., vehicle classification, growth rate, and truck distribution factors) was collected and the sensitivity analysis was done for various climate condition in Lebanon. The study result revealed that the pavement's performance was not significantly affected by the variations and assumptions used in the calculations of the truck classes. It also found that the variation in the truck traffic volume, i.e., growth factor, significantly influences the predicted pavement distresses (Chhade et al., 2018).

Generally, the summary of the literature review results shows that a higher percentage of V_{be} leads to poor rutting performance vice versa good fatigue performance in the flexible pavement. Also, the lower amount of in-place air voids (V_a) leads to a reduction in pavement distresses (e.g., rutting, alligator cracking, and top-down cracking). But the effect of design factors such as V_a , V_{be} , and different aggregate particle distribution percentages in HMA mixtures in each layer (wearing course, binder course, and bituminous base course) of the pavement structure on pavement performance have not been discussed well. On the other hand, the sensitivity analyses performed in other countries were case studies that were conducted under specific local conditions, so provide a general insight regarding the impact of design factors on pavement performance. Therefore, these findings cannot be directly used in Turkey due to considerable differences, as the flexible pavement structure and number of layers, HMA mixture types, bound and unbound materials, traffic characteristics, climate conditions, construction methods, and policies are different.

CHAPTER THREE

DATA PREPARATION

In this chapter, data collection [e.g., climate data, traffic data, and material properties (e.g., HMA materials, and granular materials)] and data analysis (e.g., improving and converting of local data in the MEPDG format) were explained.

3.1 Data Collection

The MEPDG requires various and detailed local design data in three hierarchical inputs level. In this thesis, comprehensive efforts were made to collect Level 2 and Level 3 design inputs. Thus, different state roadways in North, North-South, West, and West-East part of Izmir were selected (See Table 3.1, and Figure 1.3). Furthermore, several documents such as Turkish Highway Technical Specification (General Directorate of Highways, 2013), Asphalt Mixture Laboratory Studies (Orhan, 2012), Turkish Flexible Pavement Design Guide (General Directorate of Highways, 2008), and Turkish General Directorate of Highways annual published reports [e.g., Traffic and Transportation Information and Characteristics (General Directorate of Highways, 2017), Highway Traffic Flow Characteristics and Parameters (General Directorate of Highways, 2016) and Features and Trends of Heavy Vehicle Traffic in Freight Transport on Highways (General Directorate of Highways, 2011)] were carefully overviewed and local data such as pavement structures, material properties, traffic characteristics were extracted. The climate data for Izmir was obtained from the Turkish State Meteorological Services.

Table 3.1 Selected roadway segments

Number of Roadway	Number of lanes	Direction/Location
D240	4	North
D550	4	North and South
D505	2	West
D300	4	West and East
D310	4	South



Figure 3.1 Selected roadway segments

3.1.1 Flexible Pavement Structure

Three typical pavement structures were selected for three traffic levels based on Turkish Flexible Pavement Design Guide Standard (General Directorate of Highways, 2008). The pavement layer type, numbers, and thicknesses as well as subgrade strength types (CBR) are presented in Table 3.2. Based on the design guide, the thickness of wearing course and crushed stone base course are not changed for all traffic levels and any subgrade type, while the thickness of binder course, and AC base course are changed according to traffic levels. The thickness of granular subbase course is also changed based on traffic levels and subgrade types (See Table 3.2 and Figure 3.2) (General Directorate of Highways, 2008).

Table 3.2 Pavement structure information

Layers		Average Annual Daily Truck Traffic (AADTT)		
		5000	10000	15000
HMA Wearing (mm)		50	50	50
HMA Binder (mm)		80	100	100
AC Base Course (mm)		110	110	140
Crushed Stone Base (mm)		200	200	200
Granular Subbase (mm)				
Subgrade, CBR %	5	300	300	350
	10	200	200	200
	15	200	200	200

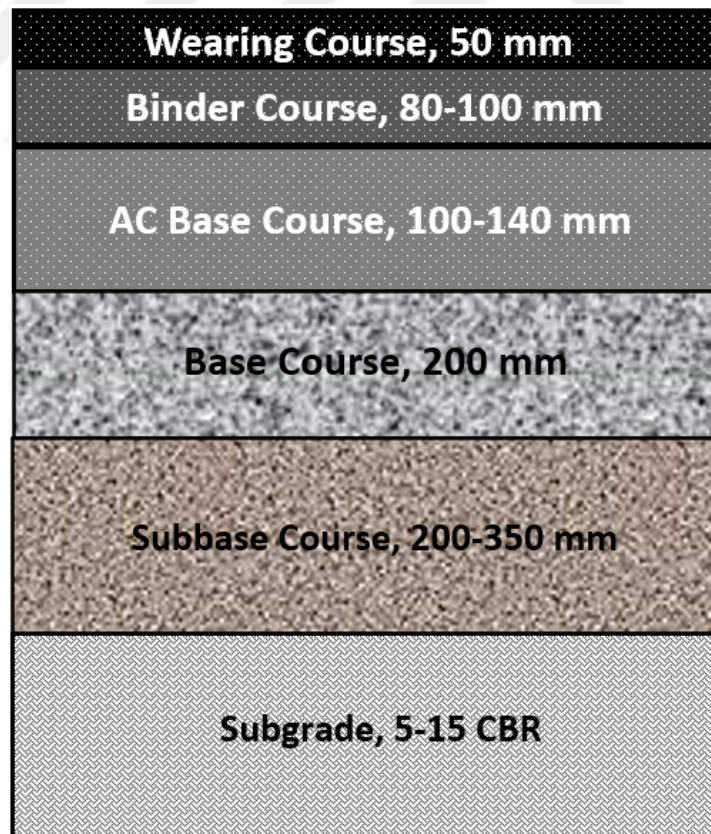


Figure 3.2 Typical flexible pavement section

3.1.2 Material Properties

The MEPDG requires various and detailed material properties in three hierarchical inputs level. In this thesis, comprehensive efforts were made to collect Level 2 and Level 3 design inputs. Therefore, several documents such as Turkish Highway Technical Specification (General Directorate of Highways, 2013), Asphalt Mixture Laboratory Studies (Orhan, 2012), and Turkish Flexible Pavement Design Guide (General Directorate of Highways, 2008) were carefully overviewed and local material properties (e.g., HMA and granular) were extracted which were explained as follows.

3.1.2.1 HMA Material Properties

The HMA material properties such as binder grade (e.g., Pen. and PG), HMA volumetric properties (e.g., maximum gravity, V_a , V_{be}), mechanical properties (e.g., binder grade and aggregate gradation) were extracted from the Turkish Highway Technical Specification (General Directorate of Highways, 2013), Asphalt Mixture Laboratory Studies (Orhan, 2012), and Binder Grade Selection Maps for Hot Mix Asphalt Pavement Roads (Saglik, Orhan, & Gungor, 2012).

3.1.2.1.1 Binder Grades. Based on the Highway Technical Specification 2013, for flexible pavement design, three types of penetration grades (e.g., 40-60, 50-70, and 70-100) are used in Turkey. But there is a bit difference between local binder gradation and the AASHTOWare Pavement ME Design binder gradation (e.g., 40-50, 60-70, 85-100). As the average of each local binder grade is within the range of the MEPDG binder gradation, therefore, no need to change and convert the local binder gradation to the MEPDG format. Also, according to the Binder Grade Selection Maps for Hot Mix Asphalt Pavement Roads (Saglik et al., 2012), different binder PG grades are used in different zones in Turkey. To determine the effect of PG grades on pavement performance, all used binder PG grades were extracted from the guide line, Table 3.3.

Table 3.3 HMA grades

Inputs	Values
Penetration grade, (0.1mm)	50-70 (40-50, 60-70) & 85-100
Performance grade, (PG)	58-22, 58-28, 58-28, 64-16, 64-22, 64-28, and 70-10, and 70-16

3.1.2.1.2 HMA Volumetric Properties. Based on the Highway Technical Specification 2013, two types of dense graded and two types stone mastic asphalt (SMA) mixtures are suggested for wearing course. For binder course only one type of dense graded and SMA is recommended. For AC base course, two types of dense graded mixtures are used, Table 3.3. Also, different air voids (V_a) and binder content are suggested for different HMA mixture types, Table 3.4.

Table 3.4 Volumetric inputs

Inputs		Wearing course		Binder course	AC base course	
		Mixture types				
		1	2	1	A	B
Specific gravity, (KN/m ³)		24	24	24	24	24
Dense graded	Binder content by weight (%)	4-7	4-7	3.5-6.5	3-5.5	3-5.5
	Air voids V _a (%)	4-5	4-5	4-6	4-6	4-6
SMA	Binder content by weight (%)	3-4	3-4	3-4		
	Air voids V _a (%)	5.8	5.8	6.5		

3.1.2.1.3 Aggregate Gradation. In level 3 design input, passing aggregate percentage from sieves (e.g., 19, 9.5, 4.75, and 0.075 mm) are required to predict the dynamic modulus of HMA using Witczak's equation (NCHRP, 2004b). Based on the Highway Technical Specification 2013, the aggregate passing percentage from sieves (e.g., 19, 9.5, 4.75, and 0.075 mm) are different for different HMA mixtures types, Table 3.5 and 3.6.

Table 3.5 Aggregate gradation for dense graded HMA mixtures

Inputs	Wearing course		Binder course	AC base course	
	Mixture types				
	1	2	1	A	B
% Passing from 19 mm sieve	100	-	90-100	60-90	70-90
% Passing from 9.5 mm sieve	72-90	80-100	70-48	43-70	55-75
% Passing from 4.75mm sieve	42-52	55-72	52-30	30-55	42-62
% Passing from 0.075mm sieve	3-8	4-8	2-6	0-7	1-8

Table 3.6 Aggregate gradation for SMA mixtures

Inputs	Wearing course		Binder course
	Mixture types		
	1	2	1
% Passing from 19 mm sieve	100	100	92-100
% Passing from 9.5 mm sieve	50-75	50-67	56-66
% Passing from 4.75mm sieve	25-40	25-33	32-42
% Passing from 0.075mm sieve	8-12	8-12	7-11

3.1.2.2 Granular Material Properties

The MEPDG guide uses resilient modulus (M_r) of granular materials to predict the pavement performance. Also, the resilient modulus (M_r) internally is predicted from CBR value or AASHTO layer coefficient (a_i). In this thesis the CBR % for crushed stone base, granular subbase and subgrade were extracted from the Highway Technical Specification, Table 3.6.

Table 3.7 CBR value of granular material and subgrade

Layer types	CBR value (%)
Base course	100
Subbase course	50
Subgrade	5, 10, and 15

3.1.3 Traffic Characteristics

The MEPDG requires extensive detailed traffic inputs which is categorized in four main parts (i.e., base year traffic information, traffic volume adjustment factors, axle load distribution factors, and general traffic inputs) (NCHRP, 2004c).

3.1.3.1 Base Year Traffic Information

Traffic information such as percent trucks in design direction, and percent in design lane were extracted from Flexible Pavement Design Guide 2008. Traffic data like AADTT, traffic growth factor, and operational speed were derived from the Transportation Information and Characteristics reports that are published annually by the General Directorate of Highways and available from 2004. In this study, the following traffic data were used, see Table 3.8.

Table 3.8 Traffic base year information

Traffic information	Inputs
Two-way AADTT	5000, 1000, and 15000
Design life (year)	20
Number of lanes	4
Percent truck in design direction	50
Percent trucks in design lane	90
Operational speed (km/hr)	65

3.1.3.2 Traffic Volume Adjustment Factors

Local traffic data such as vehicle classification, vehicle distribution factors, traffic growth factors, monthly adjustment factors, and truck hourly distribution factors were extracted from the Transportation Information and Characteristics (General Directorate of Highways, 2017), the Highway Traffic Flow Characteristics (General Directorate of Highways, 2016) and the Parameters and the Features and Trends of

Heavy Vehicle Traffic in Freight Transport on Highways surveyed between (2007-2009) (General Directorate of Highways, 2011) and (2010-2014) (General Directorate of Highways, 2014).

3.1.3.3 *Axle Load Distribution Factors*

The axle load distribution factors for single, tandem, tridem are derived from The Parameters and The Features and Trends of Heavy Vehicle Traffic in Freight Transport on Highways surveyed between (2007-2009) (General Directorate of Highways, 2011) and (2010-2014) (General Directorate of Highways, 2014).

3.1.3.4 *General Traffic Inputs*

The general traffic inputs such as axle configuration, lateral wander, and wheel base were used in this research shown in Tables 3.9, 3.10, and 3.11.

3.1.3.4.1 Axle Configuration. The axle configuration refers to the spacing between dual tire, tire pressure, average axle width, tandem, tridem, and quad axle spacing, the MEPDG default values were used in this study. See Table 3.9.

Table 3.9 Axle Configuration

Average axle width (m)	2.55
Tandem axle spacing (m)	1.31
Dual tire spacing (mm)	305
Quad axle spacing (m)	1.25
Tire pressure (kPa)	827.4
Tridem axle spacing (m)	1.5

3.1.3.4.2 *Lateral Traffic Wander*. Lateral traffic wander includes of Mean wheel location, Traffic wander standard deviation, and Design lane.

- Mean wheel location, the distance between outer edge of the wheel and the edge marking for the pavement. The default value of 460 mm is used for Izmir region.
- Traffic wander standard deviation, the default value of 254 mm is used for Izmir region.
- Design lane width, the value of 3.5 m is used for Izmir region.

3.1.3.4.3 *Wheelbase*. The spacing between steering axle and first axle in a truck is called wheelbase and divided in three ranges of short, medium, and long. See Table 3.10.

Table 3.10 Wheelbase

Axle types	Average axle spacing between axle group, m	Percent of truck, %
Short	3.66	17
Medium	4.57	22
Long	5.49	61

3.1.4 *Climate Data*

Five years climate data (2004-2018) for Izmir region were obtained in text format from the Turkish State Meteorological Services. The climate data were included hourly temperature, wind speed, cloud cover, precipitation, and humidity. The collected climate stations were shown in Table 3.11.

Table 3.11 Izmir climate stations

Station Number	Station Name	Latitude	Longitude
17180	Dikili	39.0737	26.8880
17787	Aliaga	38.7922	26.9682
17742	Konak	27.1710	53.0000
17789	Menemen	38.6237	27.0433
17219	Izmir Adnan Menderes Airport	38.2950	27.1481
17822	Odemis	38.2157	27.9642
17854	Selcuk	37.9423	27.3669
18447	Kiraz	38.2192	28.2028
18029	Tire	38.1330	27.8165
18443	Buca	38.3750	27.1953
18031	Bornova	38.4666	27.2166
17218	Cigli Airport	38.5127	27.0144
17749	Kemalpasa	38.4639	27.3705
17221	Cesme	38.3036	26.3724
18028	Urla	38.3628	26.8322
18444	Guzelbahce	38.3717	26.8908
18032	Karaburun	38.6401	26.5081

3.1.5 Pavement Performance Threshold Values and Reliability Levels

Generally, the implementation of the MEPDG consist of two main stages: In the first stage local data collection, sensitivity analysis, evaluation, local calibration, and validation are performed. In the second stage, threshold values and reliability levels are established (Pierce & Ginger, 2014; Tran, Robbins, Rodezno, & Timm, 2017; Xiao et al., 2011). The performance design criteria are determined based on the local conditions, engineering judgment, pavement management criteria, sensitivity analysis, and functional class (i.e., a higher reliability to minimize the consequence of early failure on more heavily trafficked routes) (Pierce & Ginger, 2014).

In the USA, different state agencies determined different threshold values and reliability levels. The MEPDG default threshold values and reliability level was shown in Table 3.12. The established threshold values and reliability levels by state agencies are shown in Table 3.13, 3.14, and 3.15 (Pierce & Ginger, 2014).

Table 3.12 The MEPDG default threshold values and reliability levels

Functional Class	IRI (m/km)	Rutting (mm)		Cracking			Reliability (%)
		Total	AC	Alligator (%)	Top-Down (m/km)	Thermal (m/km)	
Freeway/Interstate	2.5	10.2	6.4	10.0	379	95	95
Primary	3.2	12.7	6.6	20.0	379	133	90
Secondary	3.2	16.5	6.4	35.0	379	133	85

Table 3.13 Indiana Department of Transportation

Functional Class	IRI (m/km)	Rutting (mm)		Cracking			Reliability (%)
		Total	AC	Alligator (%)	Top-Down (m/km)	Thermal (m/km)	
Freeway/Interstate	2.53	12.7	10.16	10	NA	95	90
Primary	3.00	12.7	10.16	20	NA	114	90
Secondary	3.16	0	10.16	25	NA	133	85

Table 3.14 Colorado Department of Transportation

Functional Class	IRI (m/km)	Rutting (mm)		Cracking			Reliability (%)
		Total	AC	Alligator (%)	Top-Down (m/km)	Thermal (m/km)	
Freeway/Interstate	2.53	2.5	10.2	6.4	379	284	95
Primary	3.16	3.2	12.7	8.9	473	284	90
Secondary	3.16	3.2	15.2	12.7	568	284	85

Table 3.15 Arizona Department of Transportation

Functional Class	IRI (m/km)	Rutting (mm)		Cracking			Reliability (%)
		Total	AC	Alligator (%)	Top-Down (m/km)	Thermal (m/km)	
Freeway/Interstate	2.37	12.7	NA*	10	NA	189	97
Primary	2.37	12.7	NA	15	NA	284	95
Secondary	2.37	NA	NA	25	NA	284	90

In Turkey, the pavement performance threshold values are used for pavement maintenance purposes by The General Directorate of Highways. The pavement distresses are measured in both qualitative and quantitative scales. Qualitative measurement consists of three levels: Minor, Moderate, and Severe. Quantitative measurement units are different for different pavement surface distresses. The relation between qualitative and quantitative scales changes from distresses to distresses. For example: The total rutting measurement unit is millimeter (mm) and distinguished by lower than 15 mm, between 15-30 mm, and greater than 30 mm, which classified in minor, moderate, and severe level respectively. The measurement scale of alligator cracking is percent, which is classified into 4, 10, 30, 60, and more than 60%. Alligator cracking percentages are kept constant for all qualitative levels (e.g., minor, moderate, and severe). The measurement scale of transverse cracking is crack spacing (e.g., more than 35 m, 25-30 m, 10-25 m, 5-10 m, and lower than 5 m), while the severity of thermal cracking is recognized by crack width. Pavement with transverse cracking width smaller than 6 mm in minor level, 6-15 mm in moderate level, and higher than 15 mm in severe level are classified. The longitudinal cracking is measured in length in 50 m platform. The qualitative scale of longitudinal cracking is the same as thermal cracking. The IRI measurement scale is not existed in the Flexible Pavement Design Guide, 2008. Also, it can be noted that the flexible pavement maintenance is done based on the qualitative measurement. For example, if flexible pavement condition is classified in moderate or severe level the pavement rehabilitation needs to be applied, also, in some cases, when pavement condition is classified in severe level the reconstruction of flexible is recommended (General Directorate of Highways, 2008). See Table 3.16.

Table 3.16 Local flexible pavement maintenance threshold values

Total Rut Depth (mm)	AC Rut Depth (mm)	Alligator Cracking (%)	Top-Down Cracking (m/50m)	Thermal Cracking spacing (m/50m)
<15	NA	4.0	<50	>35
	NA	10.0	50-100	25-35
15-30	NA	30.0	100-200	10-25
	NA	60	200-250	5-10
>30	NA	> 60	>250	<5

As the purpose of pavement design using MEPDG is that pavement would satisfactorily perform to the whole design life, or in other words, pavement designed based on the MEPDG may not need major rehabilitation up to end of its design life. Therefore, in this study, the local pavement distress threshold values which are classified into minor and moderate levels were considered.

Considering the MEPDG default performance threshold values and reliability levels, state agencies performance threshold values and reliability levels, and local pavement distress limit values, the following design threshold values are extracted. These threshold values and reliability levels were used in analysis to determine the impact of them on flexible pavement performance. See Table 3.17.

Table 3.17 Purposed performance threshold values and reliability levels

Functional Class	IRI (m/km)	Rutting (mm)		Cracking			Reliability (%)
		Total	AC	Alligator (%)	Top-Down (m/km)	Thermal (m/km)	
Freeway/Interstate	2.5	10	6	10	380	190	95
Primary	3.16	15	8	20	480	285	90
Secondary	3.16	20	10	30	570	285	85

3.2 Data Analysis

Data analysis is done to improve and convert the local data to the MEPDG format because some of the local data may not be suitable for direct use in the AASHTOWare Pavement ME Design. In this study, the collected local data such as climate and traffic data were found, which were not based on the MEPDG format. Thus, to improve and convert those local data to the MEPDG format, the following steps were conducted.

3.2.1 Vehicle Classification

The MEPDG uses ten vehicle classes (4-13) which class 4 is busses and 5 through 13 are trucks and trailers. In the MEPDG, vehicles are classified based on truck types, axle numbers, and axle types (single, tandem, triple, and quad) (AASHTO, 2008, 2015). While in Turkey, vehicles are classified based on vehicle types in five groups. 1) Cars, 2) Medium duty commercial vehicles, 3) Buses, 4) Trucks, and 5) Trailers (General Directorate of Highways, 2008). This classification clearly cannot be used in the MEPDG as design inputs. Therefore, comprehensive efforts were made to develop a new truck classification to be suitable for use in the MEPDG. The observed trucks in Turkey were categorized in seven classes based on truck types, axle numbers and axle types (single, tandem, and tridem). Thus, first of all, class 1 (cars) were removed from the proposed vehicle classification. Secondly, class 2 (medium duty commercial vehicles) with 3.5-10 tons weight was merged into other classes because it consists of bus and trucks (30% bus and 70% truck). Class 3 (buses) was remained unchanged and it is equivalent to class 4 in the MEPDG. Class 4 (trucks) has been divided into three subclasses (5-7) based on axles numbers and axle types. Class 5 that represents the trailers, has been grouped into three subclasses (8-10). In the proposed truck classification: class 1 is buses, classes 2, 3, and 4 are single-unit trucks, and class 5-7 indicate the single unit trailers. The vehicle classes such as 11, 12, and 13 have not been observed in Turkey (Table 3.18) (General Directorate of Highways, 2014, 2017).

Table 3.18 Trucks classification

MEPDG	Existing	Proposed classification	Axles types/numbers	Description
4	Buses	1	1.2, 1.21, 1.22	Buses
5	Trucks	2	1.1,1.2	Two-axle, six-tire, single unit trucks
6		3	1.21, 1.22	Three-axle, single-unit trucks
7		4	11.21, 11.22, 1.122, 1.122, 1.211, 11.121, 11.122	Four or more axle, single unit trucks
8	Trails	5	1.2+11 1.2+22	Four axle, single trailer trucks
9		6	1.21+22 1.22+11, 1.22+22 1.2+111, 1.2+122, 1.2+222	Five axle, single trailer trucks
10		7	1,22+111,1,22+222	Six or more axles, single trailer trucks
11	-	-	-	Five axle, multi trailer trucks
12	-	-	-	Six axle, multi trailer trucks
13	-	-	-	Seven or more, multi trailer trucks

3.2.2 Average Annual Daily Truck Traffic (AADTT)

The Turkish General Directorate of Highways uses hundreds automatic vehicle counting devices to count traffic volume and classifies the vehicles. The average annual daily traffic (AADT) is available only for current vehicle classification (cars, medium duty commercial vehicles, buses, trucks, and trailers) which cannot be directly used for proposed truck classification. Therefore, the truck distribution factors of proposed truck classification were extracted from (General Directorate of Highways, 2014, 2017) references (Table 3.19).

Table 3.19 Truck distribution percentage for purposed truck classification

Current vehicle classification	Proposed truck classification	Truck distribution (TD) %
Buses	4	100
Trucks	5	18.56
	6	50.39
	7	30.96
Trailers	8	1.35
	9	98.36
	10	0.28

The AADTT for proposed truck classification was calculated using (Equation 3.1).

$$AADTT_{pi} = AADT_{ci} \times TD_i \quad (3.1)$$

Where;

AADTT_{pi} is average annual daily truck traffic for class i of proposed truck classification. AADT_{ci} is average annual daily traffic for class i of current vehicle classification, and TD_i is truck distribution percentage. As the medium duty commercial vehicles class forms from 30% buses and 70% trucks, therefore, its AADT was added to classes 4 and 5.

3.2.3 Vehicle Class Distribution Factors (VCDF)

The share percentage of trucks are shown by vehicle class distribution factors (VCDF) which was calculated by Equation 3.2.

$$VCDF_i = \frac{AADTT_i}{AADTT} \times 100 \quad (3.2)$$

Where;

VCDF_i is vehicle class distribution factor for class i, AADTT_i is average annual daily truck traffic for class i, and AADTT is average annual daily truck traffic for class.

At the end of calculation, the sum of VCDF must be equal to 100. As seen in Figure 3.3, the vehicle distribution percentage of vehicle classes 5,6, and 9 are high in state roads in Izmir, Turkey.

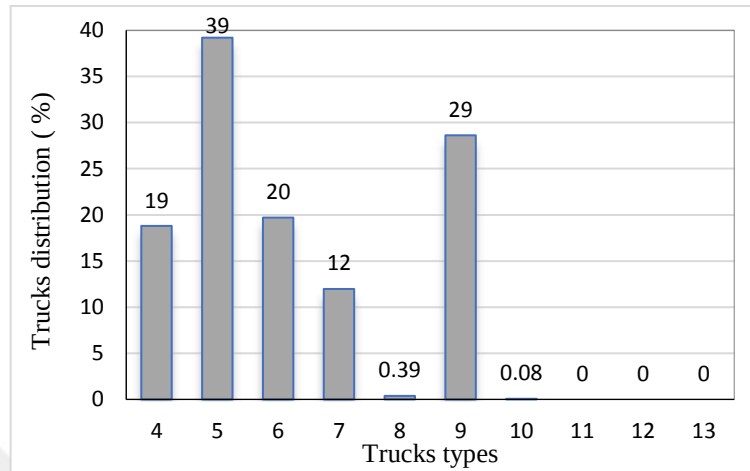


Figure 3.3 Vehicle class distribution factors

3.2.4 Monthly Adjustment Factors

The whole year traffic variation is indicated by traffic monthly adjustment factors. The monthly adjustment factors were calculated by Equation 3.3. The sum of monthly adjustment factors should be equal to 12. The monthly adjustment factors were presented in Table 3.19 and Figure 3.5 (General Directorate of Highways, 2009).

$$MAF_i = \frac{MADTT_i}{AADTT} \times 12 \quad (3.3)$$

Where;

MAF = monthly adjustment factors

MADT = monthly average daily truck traffic

i = represents the months (January to December)

Table 3.20 Trucks monthly adjustment factors

Month	Trucks classes									
	4	5	6	7	8	9	10	11	12	13
Jan	0.83	0.84	0.84	0.84	0.8	0.8	0.8	0	0	0
Feb	0.79	0.8	0.8	0.8	0.77	0.77	0.77	0	0	0
Mar	0.88	0.96	0.96	0.96	0.94	0.94	0.94	0	0	0
Apr	0.9	0.99	0.99	0.99	0.96	0.96	0.96	0	0	0
May	1.03	1.07	1.07	1.07	1.05	1.05	1.05	0	0	0
Jun	1.07	1.07	1.07	1.07	1.05	1.05	1.05	0	0	0
Jul	1.25	1.1	1.1	1.1	1.13	1.13	1.13	0	0	0
Aug	1.28	1.08	1.08	1.08	1.12	1.12	1.12	0	0	0
Sep	1.05	1.03	1.03	1.03	1.07	1.07	1.07	0	0	0
Oct	1.04	1.08	1.08	1.08	1.12	1.12	1.12	0	0	0
Nov	0.92	0.99	0.99	0.99	1.04	1.04	1.04	0	0	0
Dec	0.95	0.99	0.99	0.99	0.95	0.95	0.95	0	0	0
Total	12	12	12	12	12	12	12	0	0	0

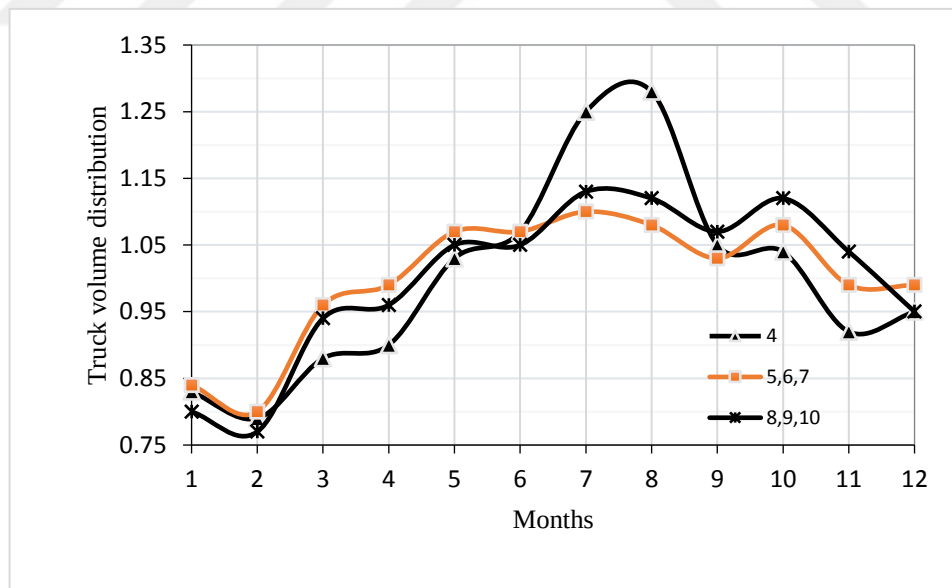


Figure 3.4 Truck monthly adjustment factors

As seen (Figure 3.5), truck traffic volumes are decreased in the winter season and are increased in the summer season.

3.2.5 Axle Per Trucks

The average number of axles for each truck class (class 4 to 13) for each axle type (single, tandem, and tridem) is called the number of axles per truck. The number of axles per truck was calculated for state roads in Izmir (Table 8).

Table 3.21 Number of axles per truck

Vehicle class	Single	Tandem	Tridem	Quad
4	1.96	0.04	0.00	0
5	2.00	0.00	0.00	0
6	1.00	1.00	0.00	0
7	1.33	0.33	0.68	0
8	2.00	1.00	0.00	0

3.2.6 Climate Data

In Turkey, the climate model has not yet been calibrated and the climate data files have not been created in EICM format (Öztürk, Tan, Şengün, & Yaman, 2019). Therefore, in this study, five-year (2013 to 2018) Izmir climate data was obtained from the Turkish State Metrological Service in text file. The obtained climate data was analyzed and various missing weather data was found. The missing data in hours were calculated using the average before and after the missing points. Also, the missing data in days, months, or years were adapted from the nearest other weather stations.

The climate data was converted to the text file with “.hcd” extension based on the MEPDG format. Each climatic file consists from, date (YYYY/mm/dd/hr), air temperature (°C), precipitation (mm), wind speed (m/hr), sunshine (%), and humidity (%) (Figure 3.6).

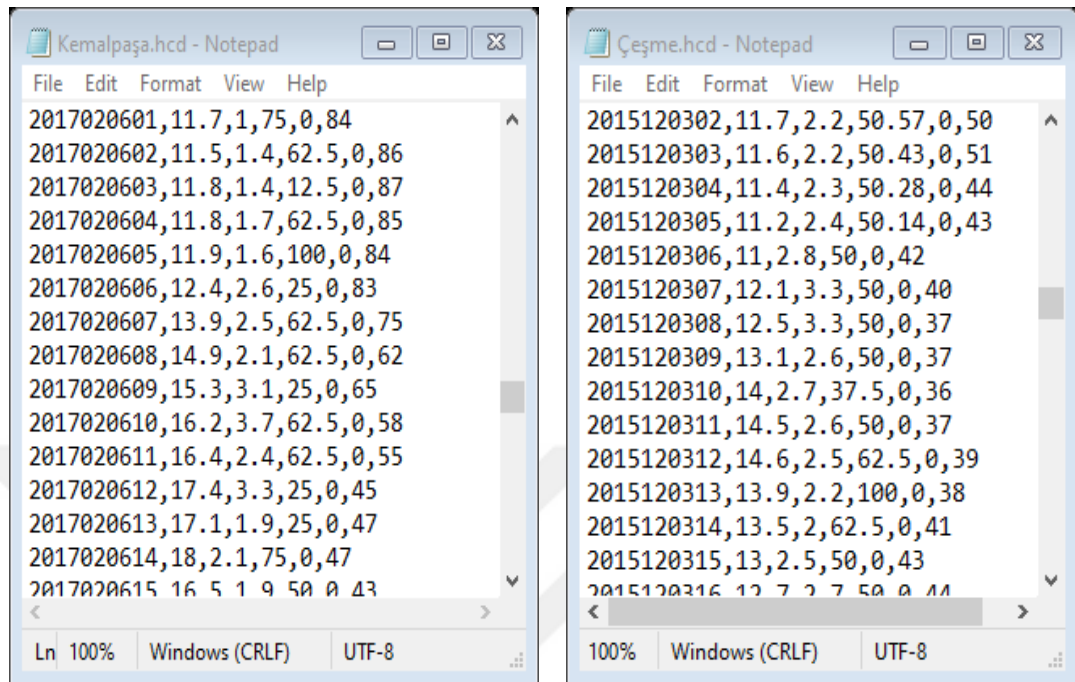


Figure 3.5 Climate data in MEPDG format (.hcd)

CHAPTER FOUR

SENSITIVITY ANALYSIS

In this chapter, the effect of local design parameters [pavement layers thickness, subgrade type, V_{be} , V_a , aggregate gradation, vehicle speed, binder grade, HMA mixture types (dense graded and SMA), threshold values and reliability levels] on flexible pavement performance [total rutting, AC rutting, alligator cracking, top-down (longitudinal) cracking, thermal (transverse) cracking as well as IRI] were evaluated using AASHTOWare Pavement ME Design v2.5.5. In order to investigate the effect of design inputs in each AC layer on flexible pavement performance, analysis was done for three traffic levels (5000, 10000, and 15000 AADTT) and three subgrade types (5, 10, and 15 CBR) at 90% reliability level for 20-year design life. The AASHTOWare Pavement ME Design was run once for any element of analysis matrix per each variable while other design inputs were kept constant. The evaluation procedure and methodology of sensitivity analysis were discussed in the following sub chapters.

4.1 Effect of Layer Thickness on Pavement Performance

Based on Turkish Flexible Pavement Design Guide, 2008, the wearing course thickness is 50 mm, binder course and AC base course thicknesses are changed based on traffic levels. Granular base course thickness is 200 mm, and granular subbase course thickness is changed according to traffic levels and subgrade types, Table 3.2. In order to determine the effect of AC layer on pavement performance; wearing course thickness was changed from 30 to 70 mm, binder course thickness was varied between 60 to 120 mm, AC base course thickness varies from 80 to 140 mm. The granular base course thickness was changed from 100 to 400 mm and subbase course thickness was changed between 100 to 400 mm, see in Table 4.1.

Table 4.1 Pavement layer thickness variation

Pavement layer type	Pavement layer thickness variation
Wearing course, mm	30-70
Binder type 1, mm	60-120
AC Base course, mm	80-140
Granular Base course, mm	100-400
Subbase course, mm	100-400

4.2 Effect of Subgrade on Pavement Performance

Generally, it is well known that the subgrade strength has a strong impact on pavement performance. But this effect can be different on various pavement distresses. To determine the effect of subgrade strength effect on pavement performance, the analysis was done for different subgrade types (5, 10, 15, 20, 25, 30, 35, 40, 45, and 50 CBR).

4.3 Effect of Vbe on Pavement Performance

According to the Highway Technical Specifications-2013, various binder content can be used for different AC layers (wearing course, binder course, and AC base course), see in Table 4.2. The Vbe percentage in wearing course varies from 8 to 15%, in binder course varies from 7 to 14% and in AC base course changes from 6 to 12% (General Directorate of Highways, 2013). It can be noted that Vbe amount is decreased from upper layer to lower layers. In another word, the higher Vbe percentage is used in wearing course and lower Vbe is suggested in AC base course. The effect of Vbe in AC layers on pavement distresses was investigated by changing their percentages in each AC layer separately, while other design parameters were remained constant. The detailed variation of Vbe are illustrated in Tables 4.2. Each variable in the Table 4.2 is indicated by specific number that each indicator represents specific Vbe percentage in each AC layer.

Table 4.2 Variation of Vbe in AC layers

Design factor	Layers	Variables indicator							
		1	2	3	4	5	6	7	8
Effective Binder Content (Vbe), %	Wearing Course	8	9	10	11	12	13	14	15
	Binder Course	7	8	9	10	11	12	13	14
	AC Base Course	6	7	8	9	10	11	12	-

4.4 Effect of Va on Pavement Performance

The Highway Technical Specifications, 2013 suggests various air voids (Va) for different AC layers (General Directorate of Highways, 2013). The detailed variation of Va is illustrated in Tables 4.3. Each variable in the Table 4.2 is indicated by specific number that each indicator represents specific Va percentage in each layer. It can be also indicated that lower Va amount is suggested for wearing course and higher Va amount for AC base course, see Table 4.3.

Table 4.3 Variation of Va in AC layers

Design factor	Layers	Variables indicator				
		1	2	3	4	5
Air Voids (Va), %	Wearing Course	3	3.5	4	4.5	5
	Binder Course	4	4.5	5	5.5	6
	AC Base Course	4	4.5	5	5.5	6

4.5 Effect of Aggregate Gradation on Pavement Performance

The Highway Technical Specifications, 2013 suggests various aggregate gradation for different mixtures types in different HMA layers. The detailed variation of aggregate particle distribution percentage in HMA mixtures are shown in Table 4.4. Each aggregate size was categorized in three classes: “Min” represents the minimum passing on sieves (19, 9.5, 4.75, and 0.075mm). “Mid” indicates the medium passing

on sieves (19, 9.5, 4.75, and 0.075mm), and Max represents the maximum passing on sieves (19, 9.5, 4.75, and 0.075mm).

Table 4.4 Aggregate gradation

Sieves (mm)	Passing	Wearing Course (%)	Binder Course (%)	AC Base Course (%)
19	Min	100	80	70
	Mid	100	90	80
	Max	100	100	90
9.5	Min	72	48	55
	Mid	81	59	65
	Max	90	70	75
4.75	Min	42	30	42
	Mid	47	41	52
	Max	52	52	62
0.075	Min	3	2	1
	Mid	5.5	4.5	4.5
	Max	8	7	8

4.6 Effect of Vehicle Speed on Pavement Performance

The asphalt is a viscoelastic material and its behavior is changed under loading rate from elastic to plastic. In other words, the response of asphalt material under rapid loading is close to elastic and under a very slow loading rate shows almost plastic behavior (Huang, 2004). This behavior is also clearly appeared in Hot Mixed Asphalt Concrete (HMA) Dynamic Modulus ($|E^*|$) that the strength of $|E^*|$ significantly decreased in slow loading rate (Ali, Irfan, Ahmed, Khanzada, & Mahmood, 2016; Bennert, 2009; Cross, Gibbe, & Aryal, 2011; Cross, Jakatimath, & KC, 2007; Iii, Mohammad, & Elseifi, 2015; Li, Zofka, & Yut, 2012). In the field, the speed of vehicles on a road varies because of various reasons such as police checking points, stop stations, speed bumps, intersections, horizontal and vertical curves, and longitudinal slopes. Therefore, the performance of flexible pavement would not be

same in the whole length of the road. Also, field observations show that for the same level of traffic and similar materials used on flexible pavement roads, the amount of pavement distress, especially rutting depth in stations and speed bumps, is significantly higher than other parts. These phenomena can be happened due to slow traffic speed on flexible pavements.

Although the vehicle speed is used as design input by the MEPDG but most of state agencies in the USA conducted the local calibration of the MEPDG average vehicle speed (Pierce & Ginger, 2014). The pavement prediction models would not be precisely calibrated if the effect of vehicle speed variation is not considered. Because the collected pavement distresses were occurred based on different vehicle speed in flexible pavements. On the other hand, if the pavement design is done for average speed and the variation of the vehicle speed is not considered, the pavement design would be led to under or overestimates.

To investigate the effect of traffic speed on flexible pavement performance, analysis was done for different vehicle speeds such as 1, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, and 70 km/hr.

4.7 Effect of Binder Grade on Pavement Performance

According to Highway Technical Specifications, 2013, and Binder Grade Selection Maps for Hot Mix Asphalt Pavement Roads (Saglik et al., 2012) different binder PG grades are used for flexible pavement design in Turkey, see in Table 3.3. Although the PG 58-16, PG 64-16, and PG 64-22 are suggested for Izmir climatic conditions by the General Directorate of Highways (Saglik et al., 2012) but in this study, different PG grades (PG 58-16, PG 58-22, PG 58-28, PG 64-16, PG 64-22, PG 64-28, PG 70-10, PG 70-16, and PG 70-22) were evaluated to completely cover the impact of different binder grade on flexible pavement performance for Izmir climatic regions. The analysis results are illustrated in Figures in Chapter 5. In the figures, each binder grade

is shown with a number such as PG 58-16 with (1), PG 58-22 with (2), PG 58-28 with (3), PG 64-16 with (4), PG 64-22 with (5), PG 64-28 with (6), PG 70-10 with (7), PG 70-16 with (8), and PG 70-22 with (9).

4.8 Effect of HMA Mixture Types on Pavement Performance

Generally, two types of dense-graded HMA mixtures (D-1 and D-2) and two types SMA mixtures (S-1 and S-2) for wearing course, one type of dense-graded HMA mixtures (D-3) and one type of SMA mixtures (S-3) for binder course, and two types of dense-graded HMA mixtures (D-4, and D-5) for AC base course are used in Turkey (General Directorate of Highways, 2013). The mixture properties (aggregate gradation, air voids, and binder content) are also different, see Table 4.5.

Table 4.5 HMA mixture types

Mixture properties	Dense-graded mixtures types					SMA mixtures types		
	Wearing course		Binder course	AC Base Course		Wearing course		Binder course
	D-1	D-2	D-3	D-4	D-5	S-1	S-2	S-3
Vbe (%)	8-14	8-14	7-13	6-12	6-12	13-14	13-14	11
Va (%)	3-5	3-5	4-6	4-6	4-6	3-4	3-4	3-4
% 19 mm sieve	100	100	80-100	60-90	70-90	100	100	92-100
% 9.5 mm sieve	72-90	80-100	70-48	43-70	55-75	50-75	67-30	56-66
% 4.75 mm sieve	42-52	55-72	52.30	30-55	42-62	20-30	25-38	32-42
% 0.075 mm sieve	3-8	4-8	2-7	0-7	1-8	8-12	8-12	7-11
Poisson's ratio	0.25	0.25	0.25	0.25	0.25	0.4	0.4	0.25

To determine the impact of HMA dense graded mixture types on pavement performance, each mixture type was investigated separately. In order to determine the effect of SMA mixture on pavement performance accurately, the evaluation was done in two steps: In the first step, as the SMA asphalt mixture is not suggested for AC base course, therefore, dense graded mixture was used for AC base course in analysis. At

the second step, the evaluation was done for only two SMA layers (wearing and binder course).

4.9 Effect of Threshold Values and Reliability Levels on Pavement Thickness

During the local calibration and implementation of the MEDPG, different pavement performance threshold values and reliability levels were determined by state agencies in the USA (Pierce & Ginger, 2014; Tran et al., 2017; Xiao et al., 2011), for more information you can refer to the Chapter Three of this dissertation. Therefore, in this study, the analysis were performed to determine the effect of threshold values and reliability levels on AC layer thickness in three steps: At the first step, the selected pavement structures (See Table 3.2) performances were evaluated for three threshold values magnitude (high, medium, and low) and three reliability levels (85, 90, and 95%) (See Table 3.17) to determine the impact of threshold values and reliability levels in terms of over or under-design on selected pavement structure. In the second step, the pavement AC layers thicknesses were adjusted to get the optimized pavement design based on predicted distresses and threshold values. In the third step, to highlight the impact of threshold values and reliability levels on pavement thicknesses, the selected AC layers thicknesses and optimized pavement layers thicknesses were compared together in terms of HMA volume.

CHAPTER FIVE

RESULTS AND DISCUSSIONS

In this chapter, the effect of local design parameters [pavement layers thicknesses, subgrade type, V_{be} , V_a , aggregate gradation, vehicle speed, binder grade, HMA mixture types (dense graded and SMA), threshold values and reliability levels] on flexible pavement performance [total rutting, AC rutting, alligator cracking, top-down (longitudinal) cracking, thermal (transverse) cracking as well as IRI] were discussed in the following sub chapters.

5.1 Effect of Layers Thicknesses on Pavement Performance

The pavement layers thicknesses play a significant role in pavement performance. But this role due to the difference in material types and properties, and placing level and thickness may be different. Therefore, in this study, each layer (wearing course, binder course, AC base course, granular base course, and subbase course) was evaluated separately. The analysis results were discussed as follows.

5.1.1 Effect of Wearing Course Thickness on Pavement Performance

The analysis was done for various wearing course thickness which was changed from 30 to 70 mm. The summary of results is shown in Table A-1.

Analysis results indicated that wearing course thickness has a significant impact on total rutting depth, see Figure 5.1. For all traffic levels and any subgrade types, the rutting depth reaches to its maximum value when wearing course thickness is 50 mm. It can be seen from Figure 5.1 that total rutting is decreased when the layer thickness is thinner or thicker than 50 mm. An increase of 20 mm in wearing course thickness (from 50 to 70 mm) leads to 13.6 % reduction in the total rutting depth, see Figure 5.1.

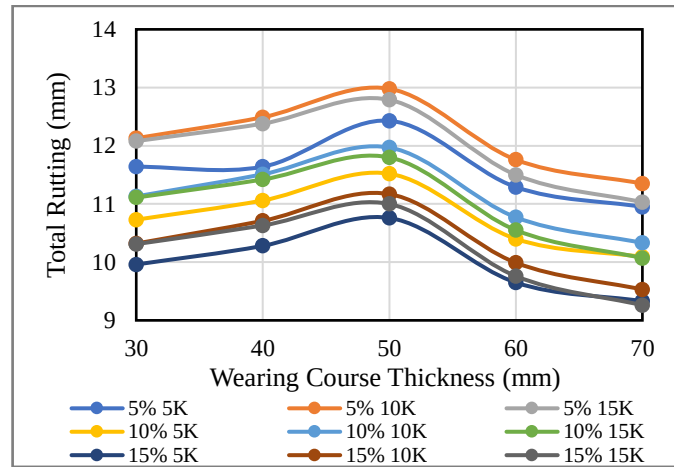


Figure 5.1 Effect of wearing course thickness on total rutting

The results demonstrated that AC rutting is significantly affected from variation of wearing course thickness, see Figure 5.2. For all traffic levels and subgrade types, the AC rutting approaches its maximum average magnitude (6.32 mm) when the wearing thickness is 50 mm. An increase of 20 mm in wearing course thickness (from 50 to 70 mm) leads to 22.6 % reduction in the AC rutting depth. Also, the results revealed that for lower thickness than 50 mm, AC rutting is also decreased, see Figure 5.2.

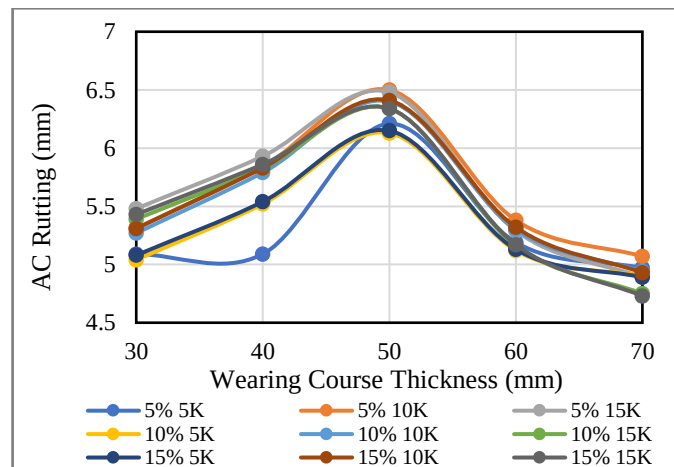


Figure 5.2 Effect of wearing course thickness on AC rutting

Analysis results revealed that top-down (alligator) cracking was largely affected from variation of wearing course thickness. For all levels of traffic and subgrade types,

a 20 mm increase in wearing course thickness (from 50 to 70 mm) would result in a 30.7 % reduction in alligator cracking, see Figure 5.3.

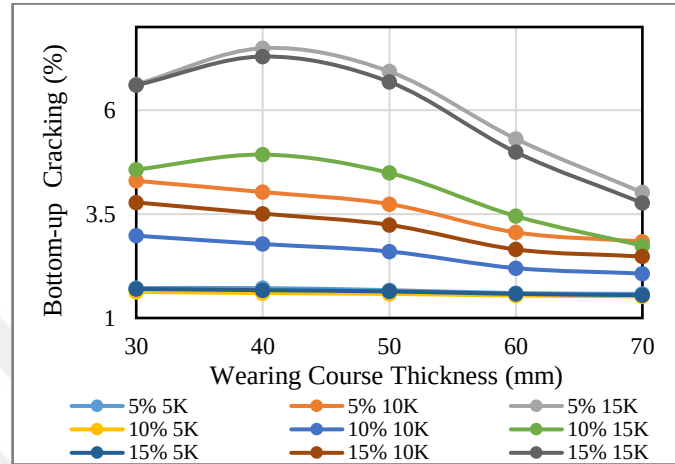


Figure 5.3 Effect of wearing course thickness on alligator cracking

Results indicated that in presence of stiffer subgrade type (10 and 15 CBR) top-down (longitudinal) cracking was strongly affected by variation of wearing course thickness. An increase in wearing thickness (40 mm) results a significant reduction in top-down cracking (39.3 %). The impact of variation of wearing course thickness illustrated in Figure 5.4.

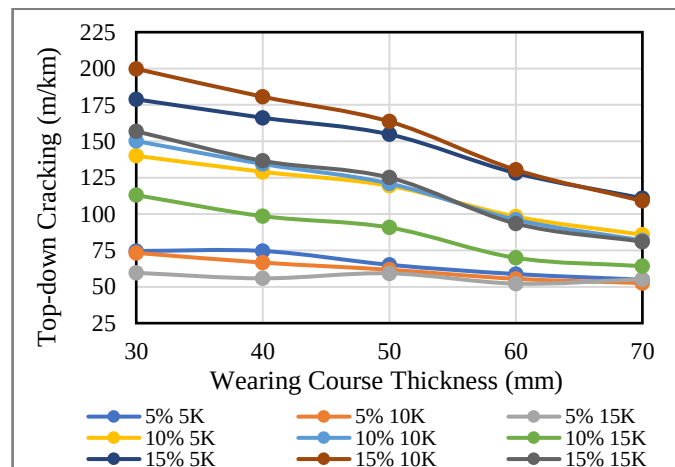


Figure 5.4 Effect of wearing course thickness on top-down cracking

Analysis results also demonstrated that thermal (transverse) cracking is very sensitive to the variation of wearing course thickness. An increase of 20 mm (from 50 to 70 mm) in the thickness of the wearing course leads to a significant reduction (29 %) in thermal cracking magnitude, see Figure 5.5.

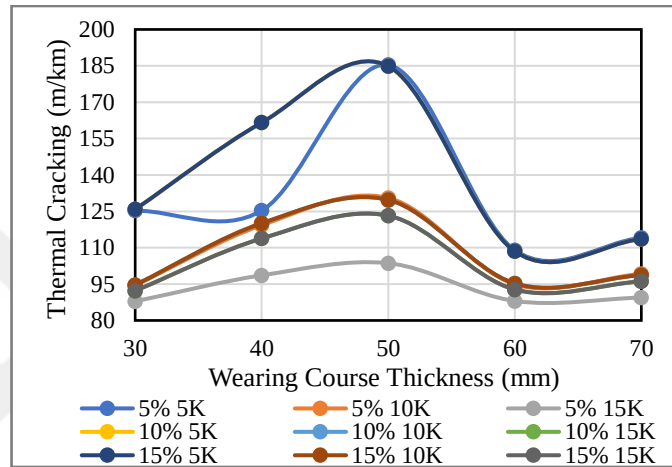


Figure 5.5 Effect of wearing course thickness on thermal cracking

5.1.2 Effect of Binder Course Thickness on Pavement Performance

In order to determine the effect of binder course thickness on pavement performance, the binder course thickness was varied from 60 to 120 mm. Results analysis are summarized in Table A.2.

Analysis results revealed that for all traffic levels and subgrade types the rutting (total and AC) is sensitive to the variation of binder course thickness. When the binder course thickness increased by 60 mm (from 60 to 120 mm) total and AC rutting magnitude are decreased by 15.9 % and 19.4 % respectively, see Figure 5.6 and Figure 5.7. Results indicated that bottom-up cracking (alligator) is extremely sensitive to the variation of binder course thickness. Alligator cracking decreased significantly (58.8%) for a 60 mm (from 60 to 120 mm) increase in the binder course thickness, see Figure 5.8.

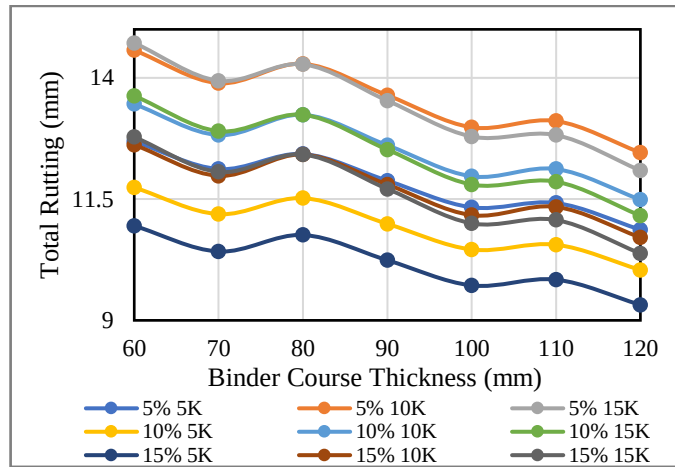


Figure 5.6 Effect of binder course thickness on total rutting

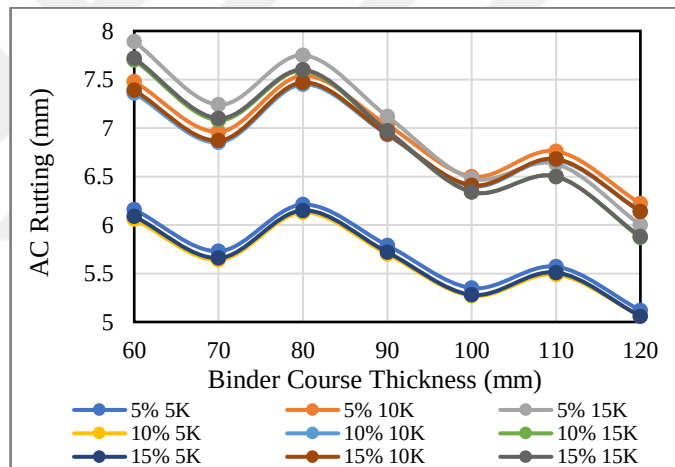


Figure 5.7 Effect of binder course thickness on AC rutting

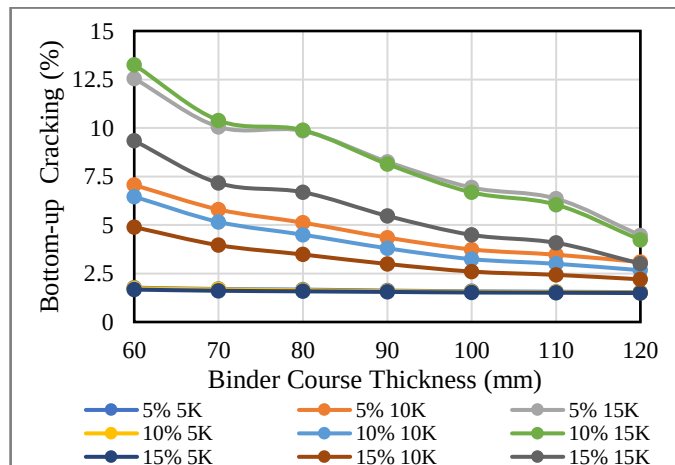


Figure 5.8 Effect of binder course thickness on alligator cracking

The analysis results also revealed that variation of binder course thickness has a strong effect on top-down (longitudinal) cracking that thicker binder course results to lower top-down cracking. An increase of 60 mm in thickness of binder course resulted to 50 % reduction in top-down cracking magnitude. Analysis results demonstrated that thermal cracking is affected by the variation of the binder course thickness for all traffic levels and subgrade types. A 14.8 % reduction in thermal cracking magnitude was happened due to an increase of 60 mm in binder course thickness, see Figure 5.10.

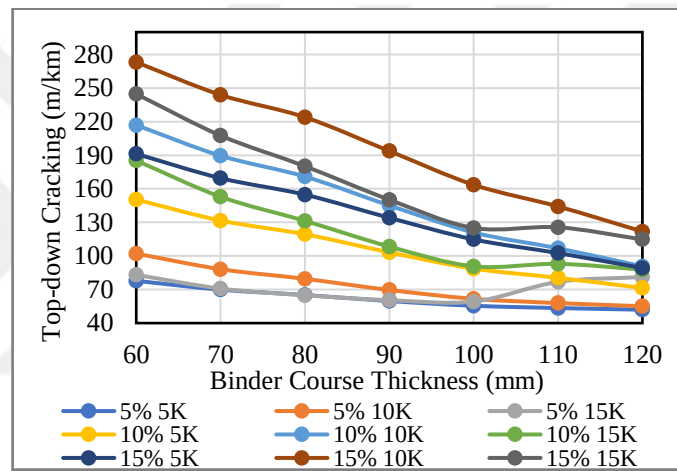


Figure 5.9 Effect of binder course thickness on top-down (longitudinal) cracking

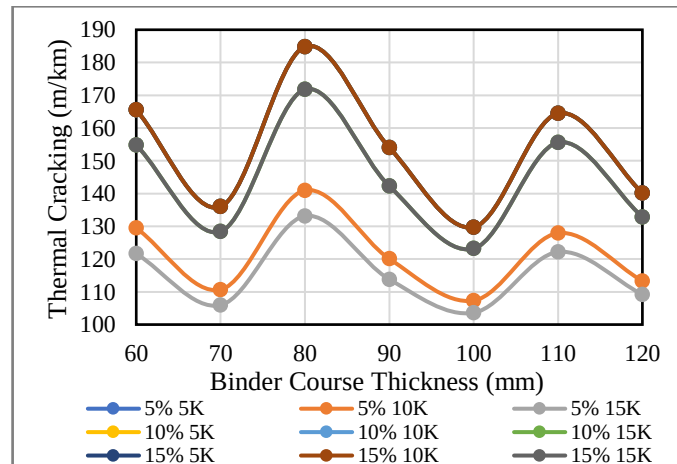


Figure 5.10 Effect of binder course thickness on thermal cracking

5.1.3 Effect of AC Base Course Thickness on Pavement Performance

To determine the effect of AC base course thickness on pavement performance, the AC base course thickness was varied from 80 to 140 mm. The summary of analysis results is illustrated in Table A.3. The analysis results revealed that total and AC rutting are significantly influenced from variation of AC base course thickness. The results showed that a 60 mm increase in AC base period thickness (from 80 to 140 mm) resulted in a 15% reduction in total rutting depth and a 17.3% reduction in AC rutting depth, see Figure 5.11 and Figure 5.12.

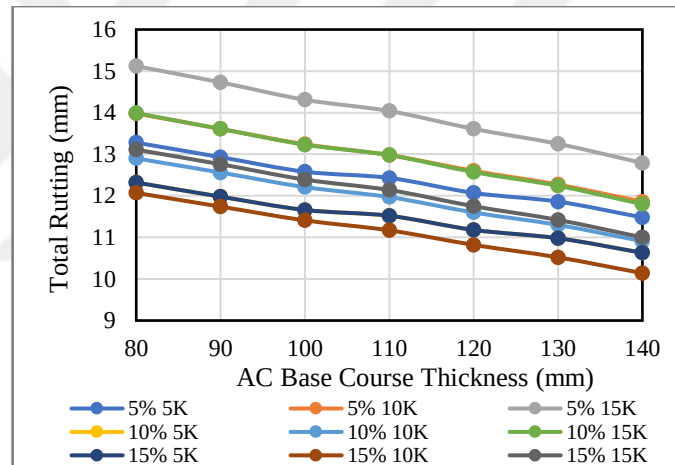


Figure 5.11 Effect of AC base course thickness on total rutting

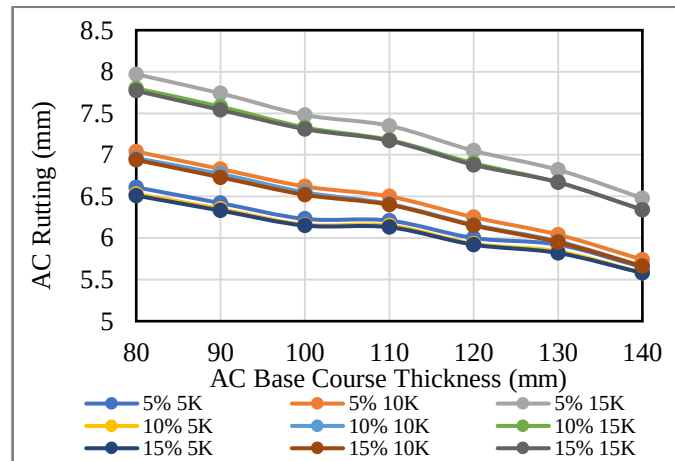


Figure 5.12 Effect of AC base course thickness on AC rutting

The analysis results also demonstrated that for high traffic level (15000 AADTT) alligator cracking is largely affected from variation of AC base thickness. Figure 5.13 shows that for high traffic level, a 42 % (80 to 140 mm) increase in AC base course thickness resulted in a 68 % reduction in alligator cracking in pavement. Figure 5.13 shows that for all traffic levels and subgrade types, a 60 mm increase in thickness of AC base course led to 62 % reduction in alligator cracking magnitude in pavement.

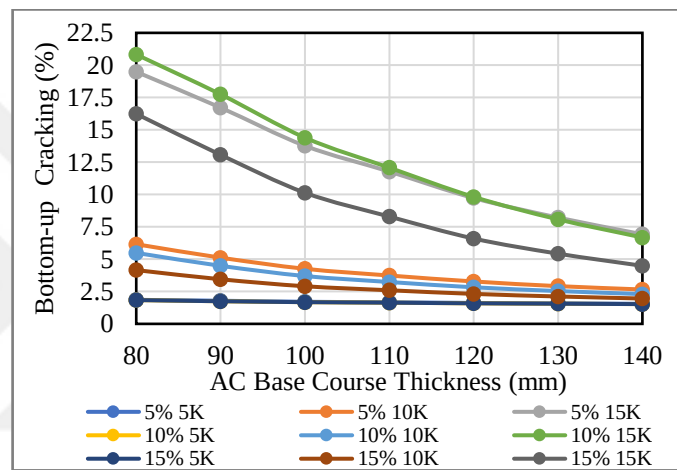


Figure 5.13 Effect of AC base course thickness on alligator cracking

The results revealed that top-down cracking is significantly affected by the variation of AC base thickness. The top-down cracking magnitude decreased in the presence of a thicker AC base course. In other words, when the AC base course thickness increased from 80 to 140 mm, the top-down cracking decreased by 52 %, see Figure 5.14. The analysis results illustrated that thermal cracking is also affected by the variation of the AC base course thickness. The thermal cracking reduced by increase of the AC base course thickness. A 60 mm increase in the thickness of the AC base course resulted in 12.2 % reduction in thermal cracking, see Figure 5.15.

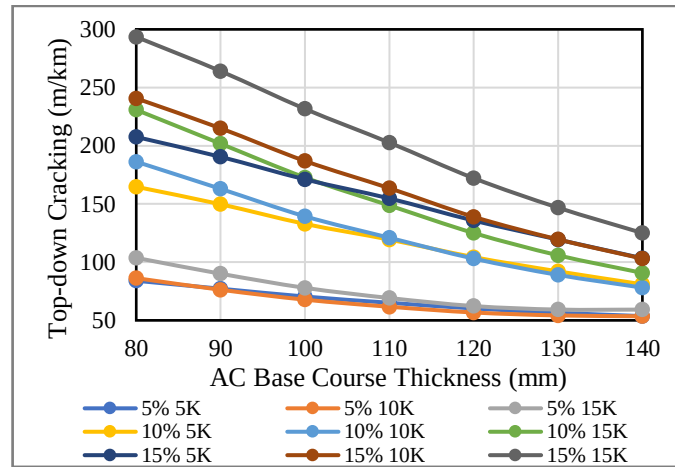


Figure 5.14 Effect of AC base course thickness on top-down cracking

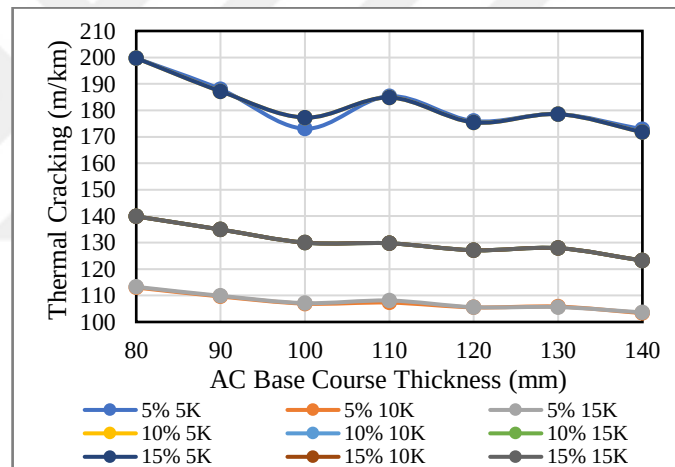


Figure 5.15 Effect of AC base course thickness on thermal cracking

5.1.4 Effect of Granular Base Course Thickness on Pavement Performance

In order to evaluate the impact of crushed stone base course thickness on pavement performance, the base course thickness was changed from 100 to 400 mm. The summary of analysis results is illustrated in Table A.4.

The analysis results demonstrated that total rutting is slightly influenced by variation of crushed stone base course thickness. The total rutting is decreased 3.2 % when the thickness of crushed stone base course increased by 300 mm (from 100 to

400 mm). On the other hand, results revealed that the AC rutting rose with an increase in the thickness of crushed stone base course. In the presence of stiffer subgrade/subbase course higher compression strain occurs on the top of AC layers which leads to higher AC rutting. A 300 mm increase in crushed stone base course resulted in 5.2 % rise in AC rutting. Since the total rutting is the sum of AC rutting, base/sub-base and subgrade rutting, so according to the results, an 8.4 % decrease in base/sub-base and subgrade rutting occurred with an increase of 300 mm in the thickness of the crushed stone base course, see Figure 5.16 and Figure 5.17.

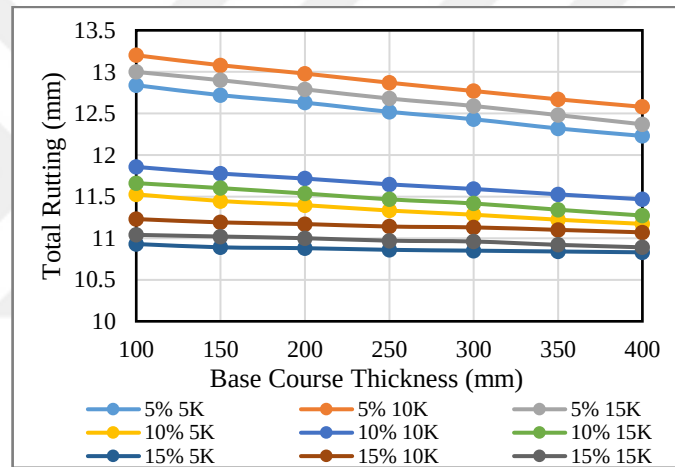


Figure 5.16 Effect of crushed stone base course thickness on total rutting

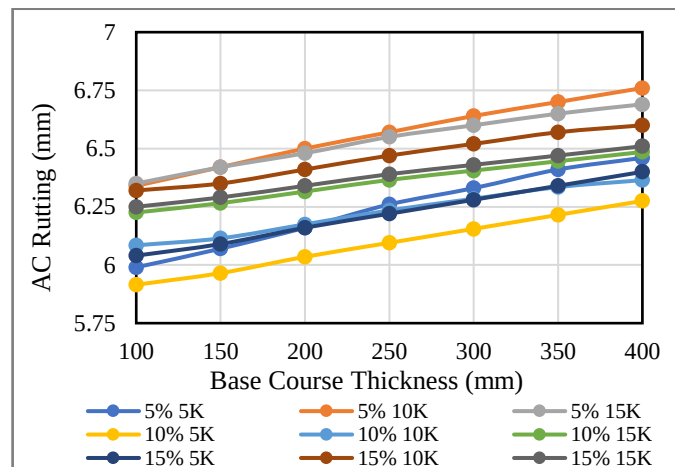


Figure 5.17 Effect of crushed stone base course thickness on AC rutting

The analysis findings revealed that alligator cracking is significantly affected by variation of crushed stone base course thickness. A decrease in crushed stone base course thickness resulted a significant increase in alligator cracking. According to the analysis results, the alligator cracking was increased by 75.4 % when the thickness of crushed stone base course decreased from 400 mm to 100 mm. See Figure 5.18.

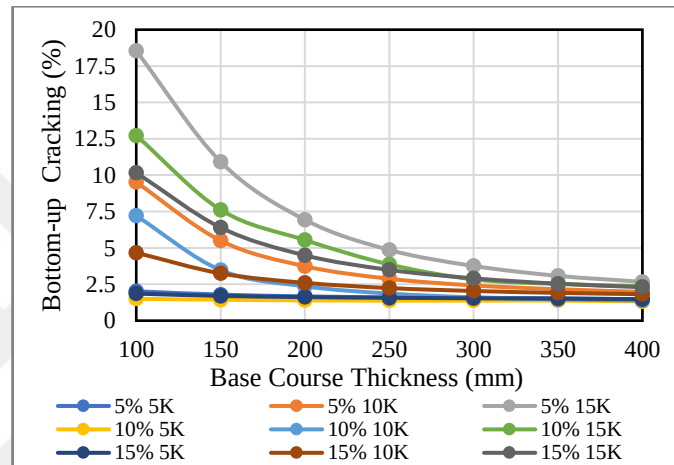


Figure 5.18 Effect of crushed stone base course thickness on alligator cracking

The analysis results indicated that top-down cracking is highly affected by variation of stone base course thickness. The top-down cracking is increased (30.7 %) with a reduction in the crushed stone base course thickness (300 mm). See Figure 5.19.

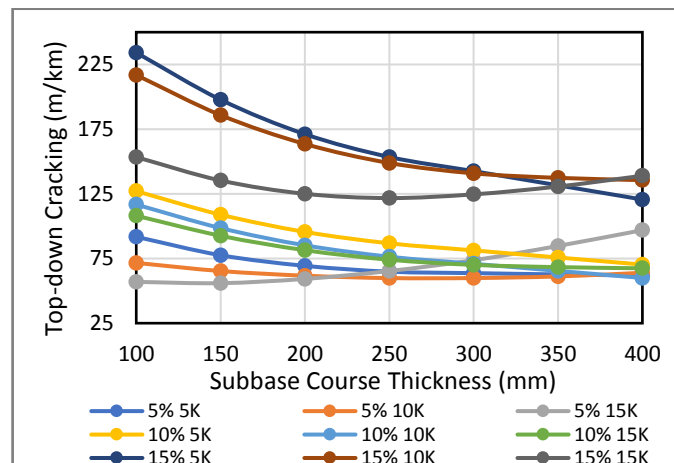


Figure 5.19 Effect of crushed stone base course thickness on top-down cracking

The analysis findings revealed that thermal (transverse) cracking is not sensitive to the variation of the crushed stone base course thickness, see Figure 5.20.

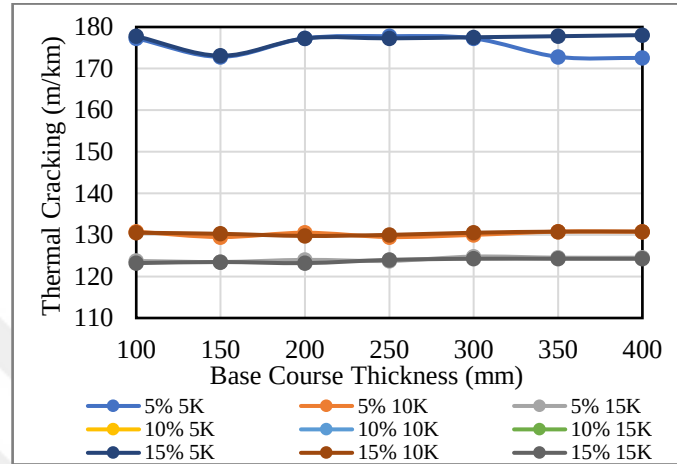


Figure 5.20 Effect of crushed stone base course thickness on thermal cracking

5.1.5 Effect of Subbase Course Thickness on Pavement Performance

In order to investigate the effect of subbase course thickness on pavement performance, the subbase course thickness was changed from 100 to 400 mm. The summary of analysis results is illustrated in Table A.5.

The results revealed that total cracking is not sensitive to the variation of the subbase course thickness while the AC rutting is affected by the subbase thickness. According to the analysis findings, when the subbase course thickness was varied from 100 to 400 mm the total rutting was decreased by 0.2 % and AC rutting was increased by 5 %. Also, the compression of total rutting and AC rutting show that an increase in the subbase course thickness resulted a 5.2 % reduction in base/subbase and subgrade rutting magnitude. The analysis results of the impact of subbase thickness on total and AC rutting are shown in Figure 5.21 and Figure 5.22, respectively.

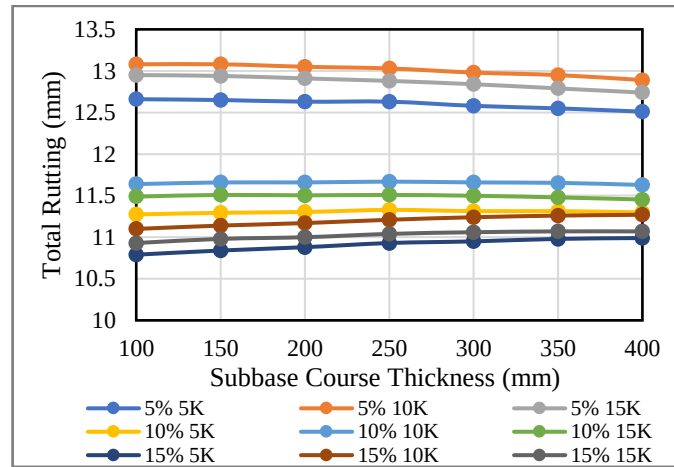


Figure 5.21 Effect of crushed subbase course thickness on total rutting

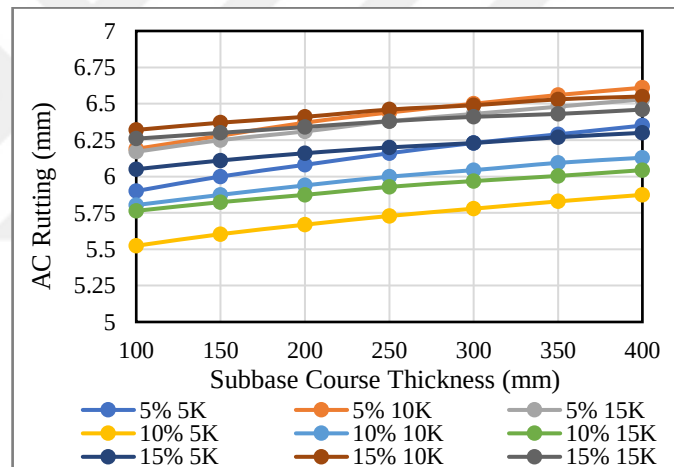


Figure 5.22 Effect of crushed subbase course thickness on AC rutting

The analysis results demonstrated that variation of the subbase thickness has a significant effect on alligator cracking. A 300 mm decrease in the subbase thickness resulted a severely increase (72.5 %) in the alligator cracking magnitude, see Figure 5.23. Also results indicated that top-down cracking is sensitive to the variation of subbase course thickness. The subbase course thickness increase (from 100 to 400 mm) resulted a minor reduction (7.1 %) in the top-down cracking, see Figure 5.24. The analysis results also revealed that variation of subbase course thickness has not effects on thermal cracking, see Figure 5.25.

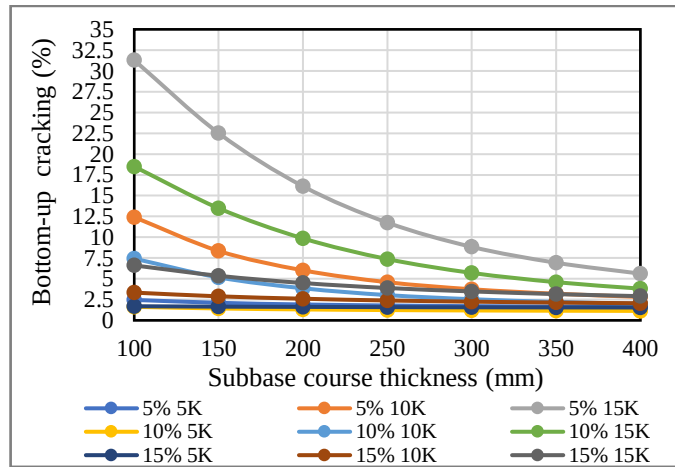


Figure 5.23 Effect of crushed subbase course thickness on alligator cracking

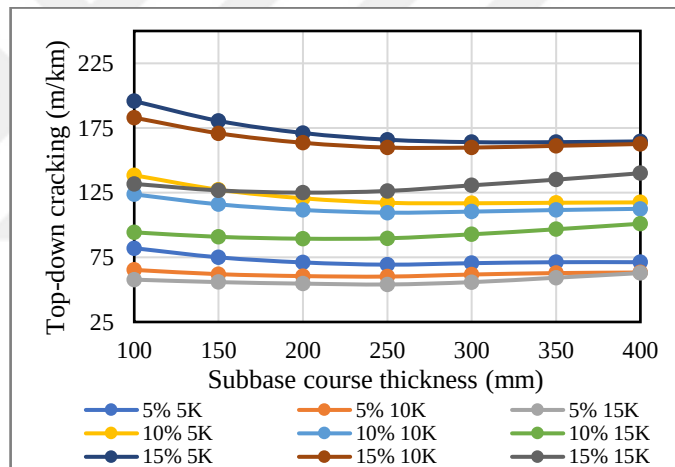


Figure 5.24 Effect of crushed subbase course thickness on top-down cracking

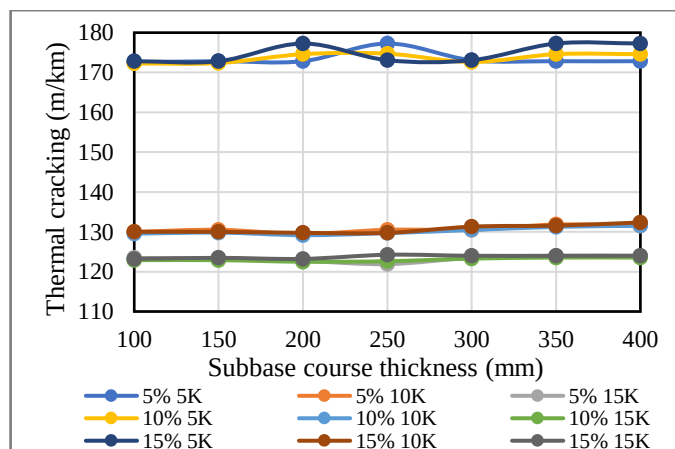


Figure 5.25 Effect of crushed subbase course thickness on thermal cracking

5.2 Effect of Subgrade Types on Pavement Performance

Generally, it is well known that the subgrade strength has a strong influence on pavement performance but this impact can be different on various pavement distresses. In this study, to determine the impact of subgrade type on different pavement distresses, analysis was done for 5, 10, 15, 20, 25, 30, 35, 40, 45, 50 CBR. The summary of the analysis findings is illustrated in Table A.6.

The analysis results revealed that subgrade type has a strong effect on total rutting depth. The total rutting depth significantly decreased in the presence of stiffer subgrade types. Results showed that the variation of subgrade strength (from 5 to 50 CBR) resulted a high reduction (28.8 %) total rutting depth. The results are illustrated in Figure 5.26.

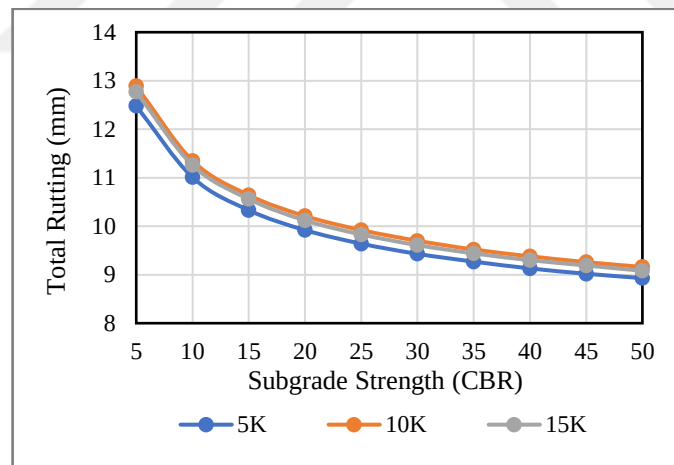


Figure 5.26 Effect of subgrade strength on total rutting

The analysis results demonstrated that subgrade type has a no major effect on AC rutting depth. The AC rutting depth slightly increased (1.1 %) when the subgrade strength was changed from 5 to 50 CBR value. The results findings are shown in Figure 5.27.

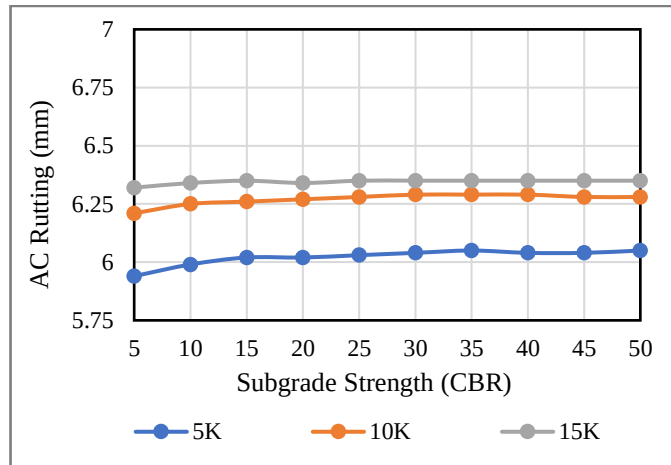


Figure 5.27 Effect of subgrade strength on AC rutting

The analysis findings indicated that alligator cracking was strongly affected by the variation of the subgrade type. Lower subgrade type resulted in a higher alligator cracking. An increase in subgrade strength (from 5 to 50 CBR) led to a significant reduction (75.5 %) in alligator cracking. In the other word, in the presence of stiffer subgrade type the alligator cracking approaches to zero, see Figure 5.28.

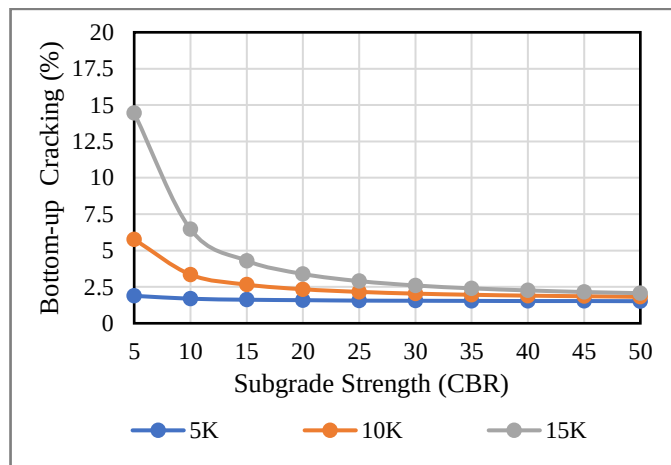


Figure 5.28 Effect of subgrade strength on alligator cracking

The analysis results demonstrated that thermal cracking was not affected from the variation of the subgrade type. The analysis findings are shown in Figure 5.29.

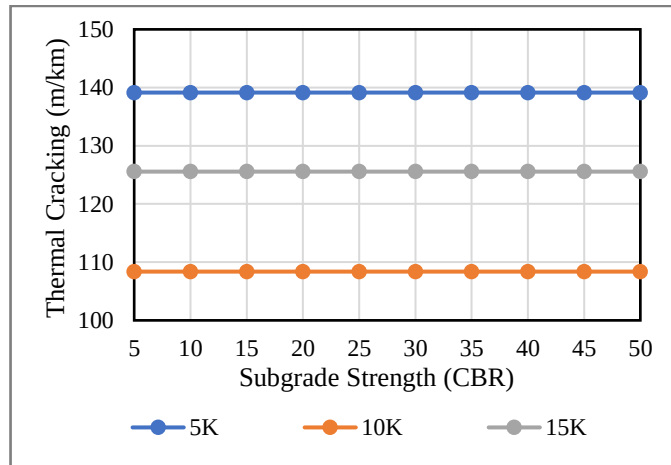


Figure 5.29 Effect of subgrade strength on thermal cracking

The analysis results also indicated that the subgrade type has very high effect on top-down (longitudinal) cracking magnitude. The stiffer subgrade type resulted in higher top-down cracking. An increase in subgrade strength (from 5 to 50 CBR) led to 79.3 % rise in the top-down cracking, see Figure 5.30. The existence of stiffer subgrade leads in higher tensile shear stress on the top of the wearing course in flexible pavement and consequently, the top-down cracking is increased.

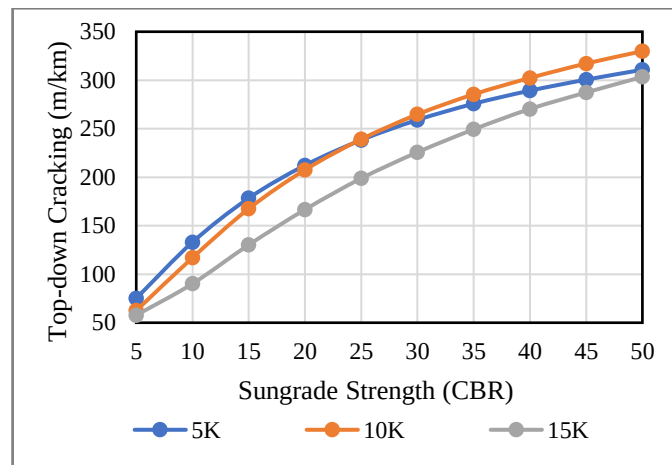


Figure 5.30 Effect of subgrade type on top-down cracking

5.3 Effect of Vbe on Pavement Performance

According to the Highway Technical Specifications, 2013, different binder contents are suggested to be used in different AC layers (wearing, binder, and AC base course). Therefore, in this study, the impact of variation of Vbe in each AC layer on flexible pavement performance was evaluated separately and discussed in the following subtitles (5.3.1, 5.3.2, and 5.3.3). The summary of analysis findings is shown in Table A.7, Table A.8, and Table A.9.

5.3.1 Effect of Vbe in Wearing Course on Pavement Performance

In order to determine the effect of the effective binder content by volume (Vbe) in wearing course on pavement performance, the amount of Vbe was changed from 8 to 15 %.

Analysis findings showed that total rutting has low sensitivity to the variation of Vbe in wearing course. For all traffic levels and any subgrade type, increasing the amount of Vbe in the wearing course from 8 to 15 % led to an 8.1 % increase in the total rutting depth, see Figure 5.31.

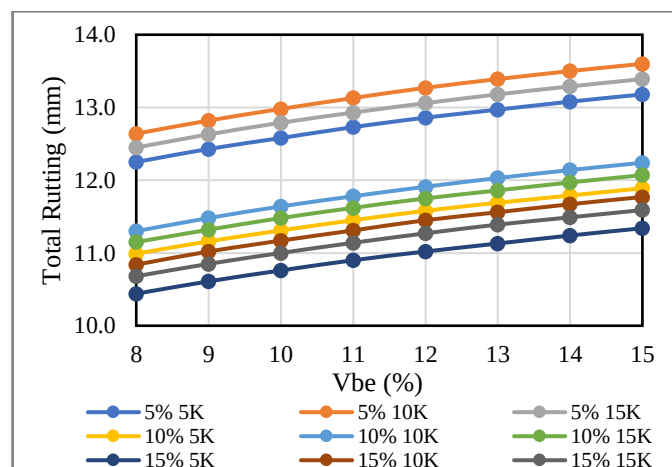


Figure 5.31 Effect of Vbe in the wearing course on total rutting

The analysis results revealed that AC rutting is sensitive to variation of Vbe in the wearing course. The increase of Vbe percentage HMA mixture of wearing course resulted in an increase in AC rutting depth. Increasing the amount of Vbe in the wearing course from 8 to 15 % resulted to an 15 % increase in the AC rutting depth, see Figure 5.32.

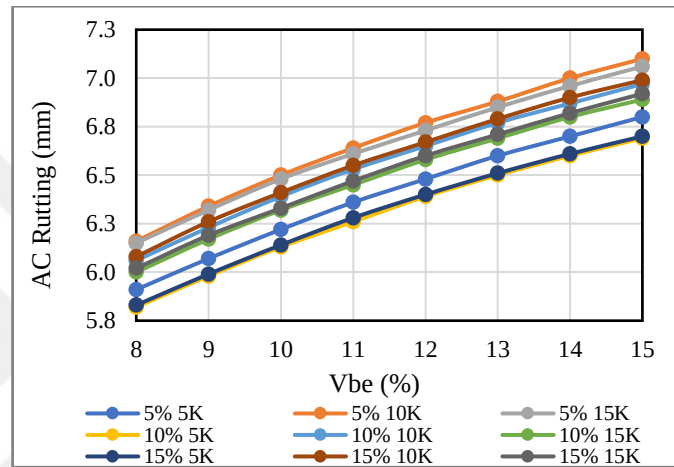


Figure 5.32 Effect of Vbe in the wearing course on AC rutting

The analysis results demonstrated that variation of Vbe in wearing course has impact on alligator cracking. Increasing of Vbe in wearing course was resulted to higher alligator cracking, see Figure 5.33. The Figure 5.33 shows that for the medium and high traffic levels (10000 and 15000 AADTT) the variation of Vbe in wearing course has higher effect on alligator cracking. Increasing binder content in HMA mixtures reduces the $|E^*|$ of HMA mixtures, and ultimately, the strength of the HMA mixture is decreased. As the alligator cracking starts from the bottom of the AC layer (here AC base course) and then propagates to upper AC layers (binder and wearing course), therefore, the use of low-strength material in the upper AC layers (wearing and binder course) may boost the cracking spread and ultimately, lead to higher alligator cracking in the pavement. Also, results revealed that thermal cracking is sensitive to the Vbe amount in the wearing course. An increase of Vbe in the wearing course results in a reduction in thermal cracking, see Figure 5.34. Also, results

indicated that top-down cracking is sensitive to the variation of V_{be} in the wearing course. The increasing of V_{be} in wearing course resulted in a significant reduction (39.7 %) in top-down cracking, see Figure 5.35.

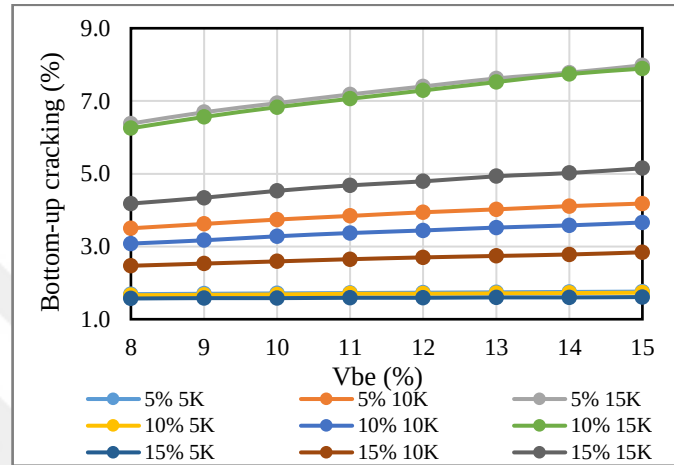


Figure 5.33 Effect of V_{be} in the wearing course on alligator cracking

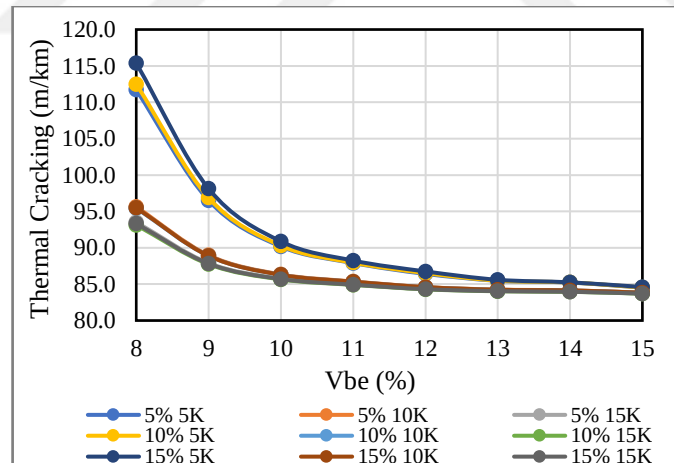


Figure 5.34 Effect of V_{be} in the wearing course on thermal cracking

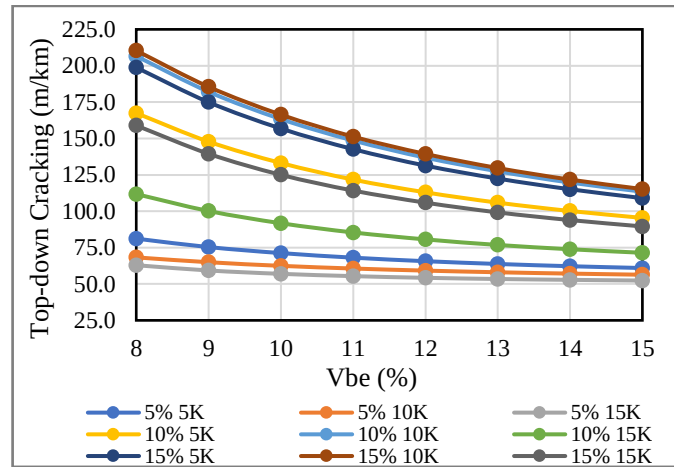


Figure 5.35 Effect of Vbe in the wearing course on top-down cracking

The analysis results indicated that IRI is not sensitive to the variation of the Vbe in wearing course, see Figure 5.36.

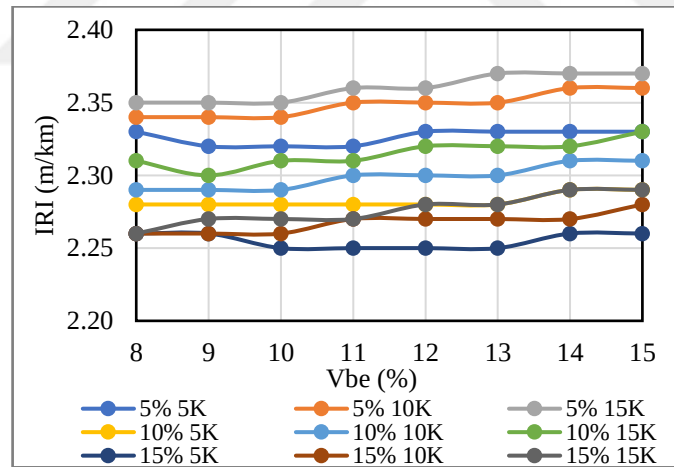


Figure 5.36 Effect of Vbe in the wearing course on IRI

5.3.2 Effect of Vbe in Binder Course on Pavement Performance

In order to determine the Vbe in binder course on pavement performance, the amount of Vbe was changed from 7 to 14 %.

The analysis results revealed that total rutting is affected from the variation of the Vbe in binder course, higher Vbe amount in the binder course led to an increase in total rutting magnitude, see Figure 5.37. Also, results demonstrated that for all traffic levels and any subgrade types, the AC rutting is sensitive to the variation of the Vbe amount in the binder course. The AC rutting is risen by increasing of Vbe amount in the binder course. A reduction in Vbe amount (14 - 7 %) led to 15.8 % decrease in AC rutting, see Figure 5.38.

The results demonstrated that thermal cracking is not sensitive to the variation of Vbe in the binder course, see Figure 5.39.

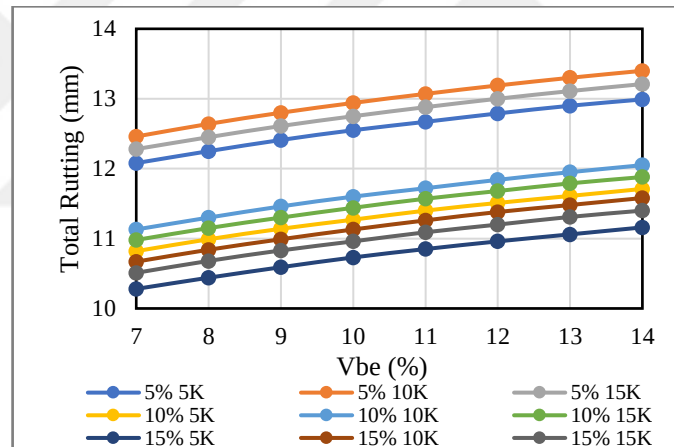


Figure 5.37 Effect of Vbe in the wearing course on total rutting

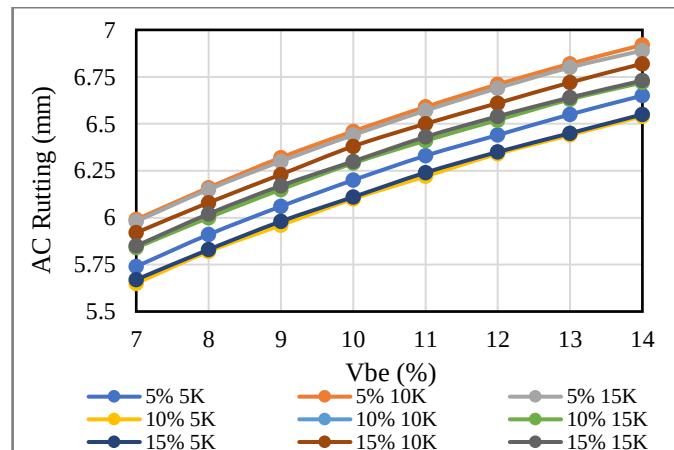


Figure 5.38 Effect of Vbe in the wearing course on AC rutting

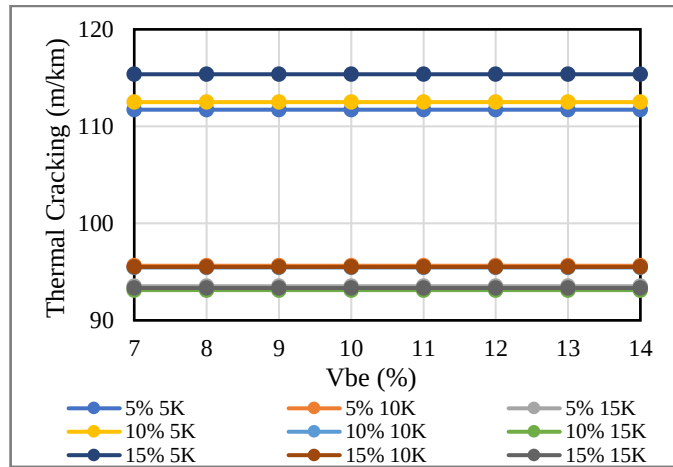


Figure 5.39 Effect of Vbe in the wearing course on thermal cracking

The results demonstrated that alligator cracking is sensitive to variation of Vbe amount in the binder course. Increasing of Vbe led to slightly increase in alligator cracking, see Figure 5.40. As the alligator cracking starts from the bottom of the AC layer (here AC base course) and then propagates to upper AC layers (binder and wearing course), therefore, the use of low-strength material in the upper AC layers (wearing and binder course) may boost the cracking spread and ultimately, lead to higher alligator cracking in the pavement.

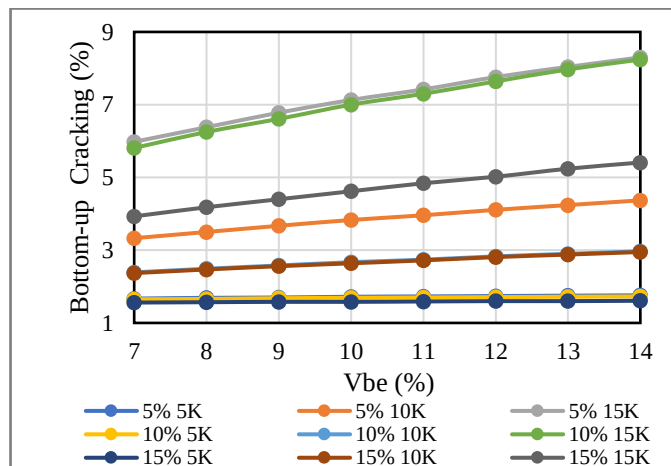


Figure 5.40 Effect of Vbe in the wearing course on alligator cracking

The analysis results revealed that for low and medium traffic levels (5000 and 10000 AADTT), top-down cracking is less sensitive to the variation of V_{be} in the binder course. For high traffic level (15000 AADTT), top-down cracking is increased by the increase of V_{be} in the binder course, see Figure 5.41. Generally, increasing of V_{be} from 7 to 14 % led to 25 % increase in top-down cracking magnitude.

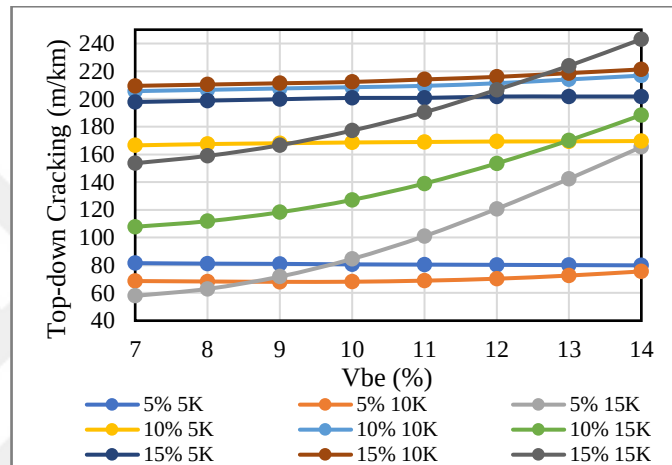


Figure 5.41 Effect of V_{be} in the wearing course on top-down cracking

5.3.3 Effect of V_{be} in AC Base Course on Pavement Performance

To evaluate the impact of V_{be} in AC base course on pavement performance, the amount of V_{be} varied from 6 to 12 %.

The analysis findings indicated that for all traffic levels and any subgrade type, total rutting, AC rutting, and thermal cracking are not sensitive to the variation of V_{be} in the AC base course. The analysis results illustrated in Figure 5.42, Figure 5.43, and Figure 5.44.

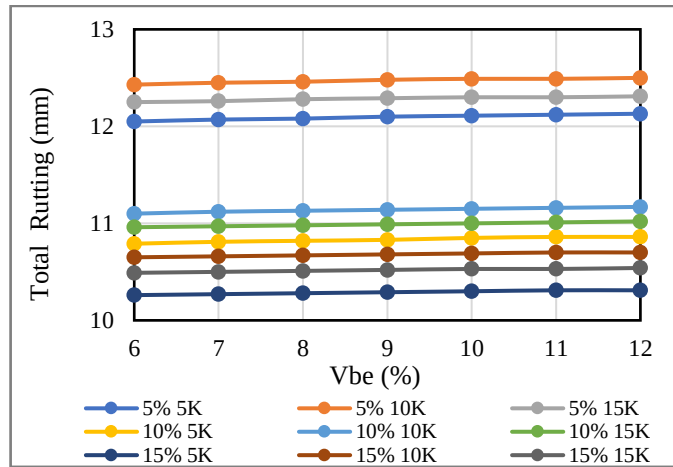


Figure 5.42 Effect of Vbe in the wearing course on total cracking

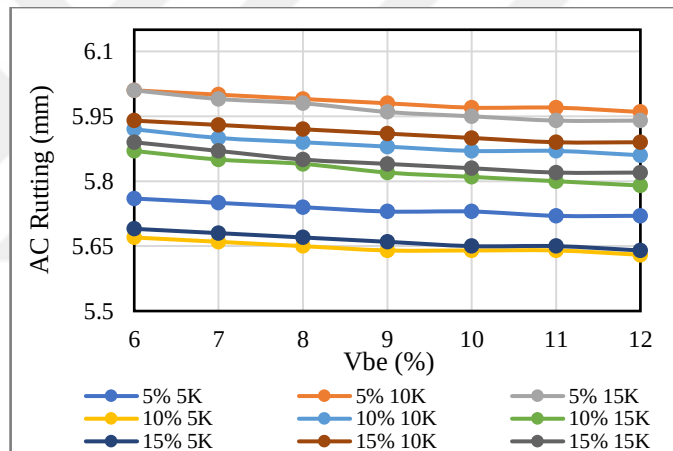


Figure 5.43 Effect of Vbe in the wearing course on AC rutting

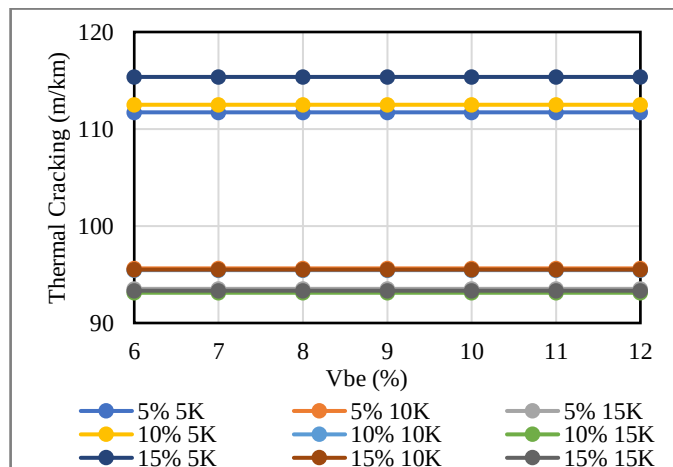


Figure 5.44 Effect of Vbe in the wearing course on thermal cracking

The analysis results revealed that alligator cracking is extremely sensitive to change in Vbe percentage in the AC base course. Decreasing of Vbe amount in AC base course resulted in a significant and rapid increase in alligator cracking. In other hand, the Figure 5.45 shows that for all traffic levels and any subgrade type, alligator cracking was almost approached to zero in the presence of higher Vbe amount (more than 9 %) in AC base course. Generally, it can be resulted from Figure 5.45 that 4 % reduction of Vbe amount (10 – 6 %) in the AC base course led to almost 94 % increase in alligator cracking.

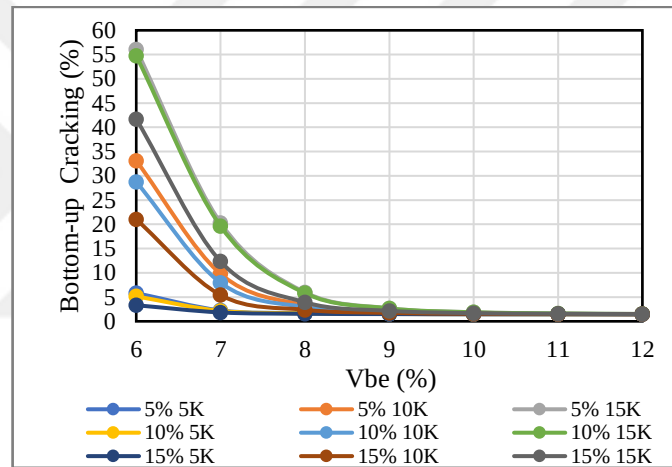


Figure 5.45 Effect of Vbe in the wearing course on thermal cracking

The results demonstrated that top-down cracking is less sensitive to the variation of Vbe in the AC base course. Increasing of Vbe percentage led to an increase in top-down cracking magnitude. Generally, when the Vbe amount increased from 6 to 12 % in AC base course the top-down cracking was increased by 16.6 %. See Figure 5.46. Also, results indicated that IRI is affected by the variation of Vbe in AC base course. The IRI was reduced by an increase in Vbe amount, see Figure 5.47.

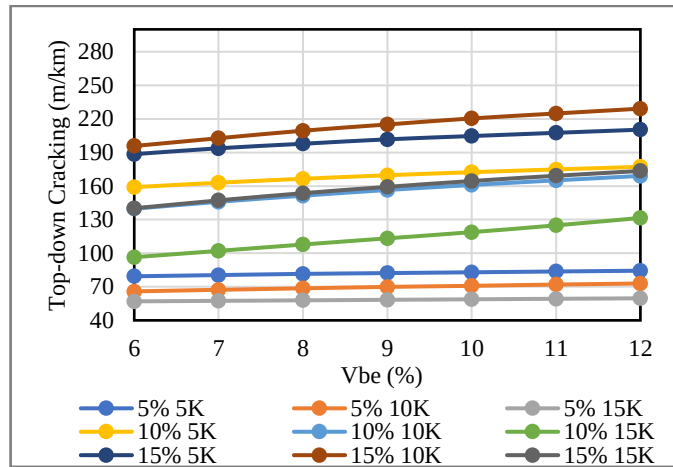


Figure 5.46 Effect of Vbe in the wearing course on thermal cracking

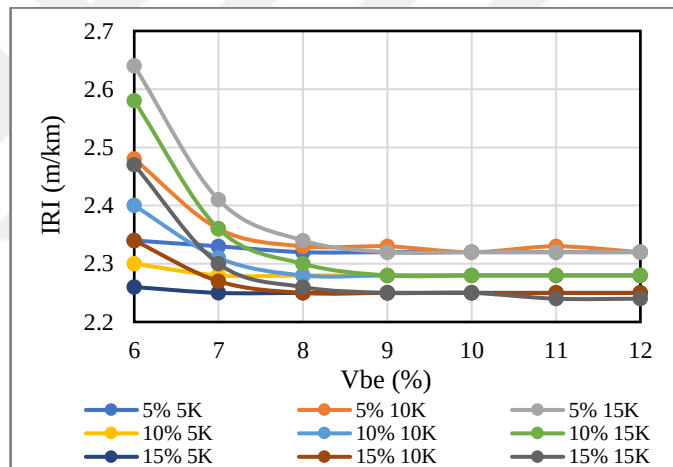


Figure 5.47 Effect of Vbe in the wearing course on IRI

5.4 Effect of Va on Pavement Performance

According to the Highway Technical Specifications, 2013, different air voids are suggested to be used in different AC layers (wearing, binder, and AC base course). Therefore, in this study, the impact of variation of Va in each AC layer on flexible pavement performance was evaluated separately and discussed in the following subtitles (5.4.1, 5.4.2, and 5.4.3). The summary of analysis findings is shown in Table A.10, Table A.11, and Table A.12.

5.4.1 Effect of Va in Wearing Course on Pavement Performance

The analysis results demonstrated that total rutting, AC rutting, and alligator cracking are less sensitive to the variation of Va in the wearing course. These pavement distresses are slightly increased by an increase in the Va amount in the wearing course. See Figure 5.48, Figure 5.49, and Figure 5.50. Also, results revealed that variation of Va in the wearing course has a strong effect on thermal cracking. The thermal cracking is rapidly reduced when Va amount decreased from 3 to 4 percent. The effect of Va on the thermal cracking was decreased for higher Va percentages (4.5 to 5 %). Generally, a 2 % increase in Va led to 62.8 % reduction in thermal cracking, see Figure 5.51

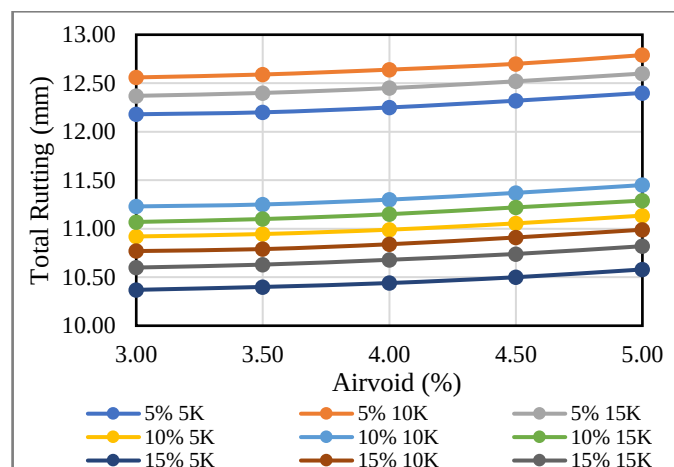


Figure 5.48 Effect of Va in the wearing course on total rutting

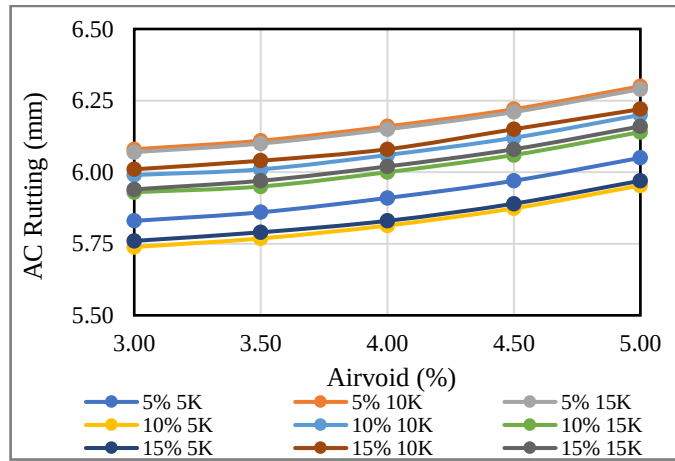


Figure 5.49 Effect of Va in the wearing course on AC rutting

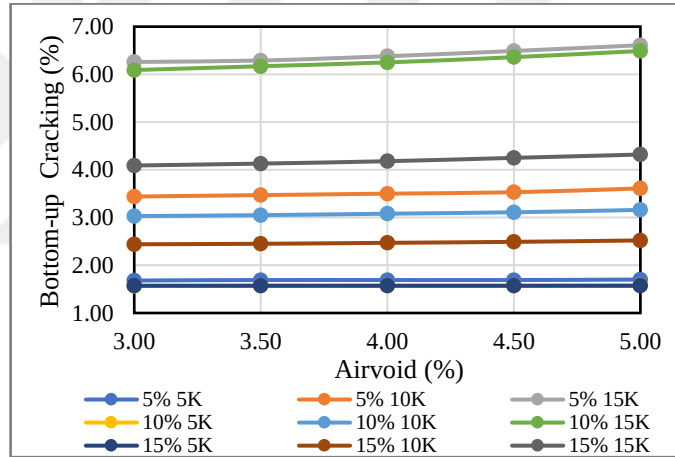


Figure 5.50 Effect of Va in the wearing course on alligator cracking

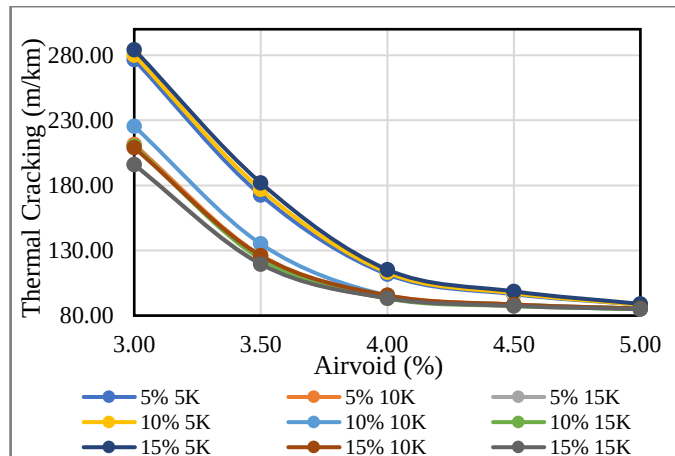


Figure 5.51 Effect of Va in the wearing course on thermal cracking

Results demonstrated that top-down cracking is significantly affected by variation of Va percentage in the wearing course. Increasing of Va amount led to a significant and rapid increase in top-down cracking, see Figure 5.52. Generally, for all traffic levels and any subgrade type, when the air voids amount in the wearing course increased from 3 to 5 % the top-down cracking average magnitude was increased by 85 %.

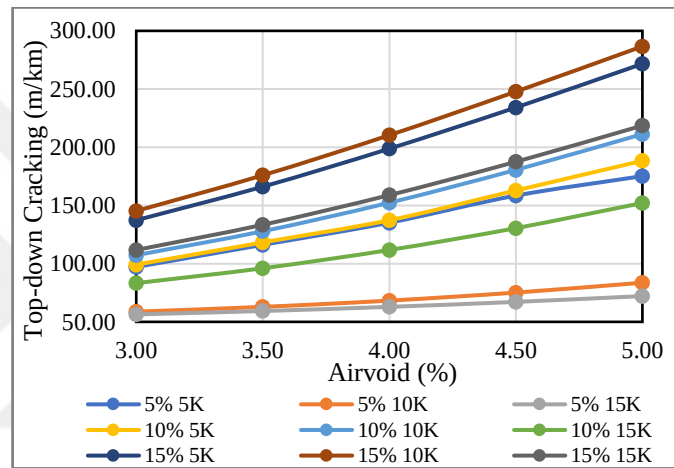


Figure 5.52 Effect of Va in the wearing course on top-down cracking

5.4.2 Effect of Va in Binder Course on Pavement Performance

The analysis findings revealed that for all traffic levels and any subgrade type, variation of Va in binder course has just a little effect on total rutting, AC rutting, alligator cracking, and top-down cracking. Figure 5.53 and Figure 5.54 show that a 2 % increase in Va content in the binder course led to 2.4 and 4.8 % increase in total and AC rutting magnitude respectively. Also, analysis results showed that increasing of Va led to 8.3 % rise in alligator cracking and 2.8 % increase in top-down cracking, see Figure 5.55 and Figure 5.56.

The analysis results indicated that for all traffic and subgrade types, thermal cracking was not sensitive to the variation of the Va in the binder course. See Figure 5.57.

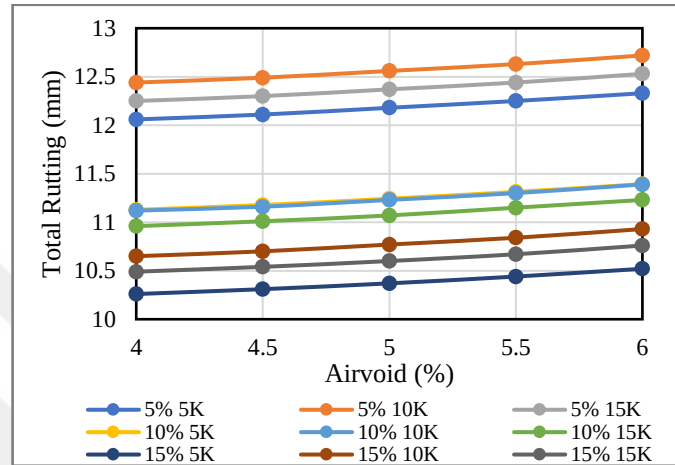


Figure 5.53 Effect of Va in binder course on total rutting

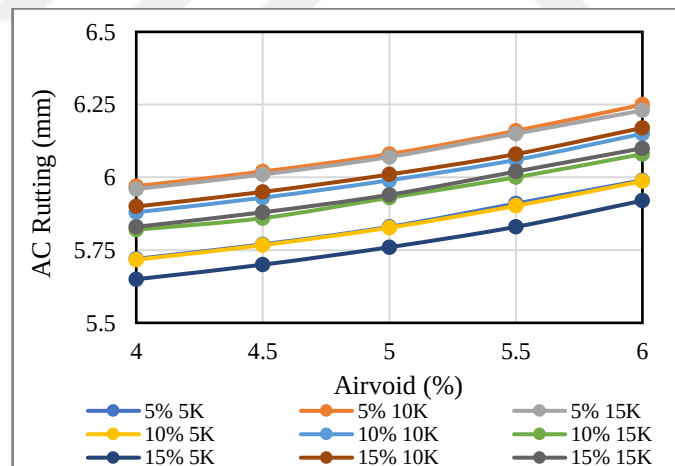


Figure 5.54 Effect of Va in binder course on AC rutting

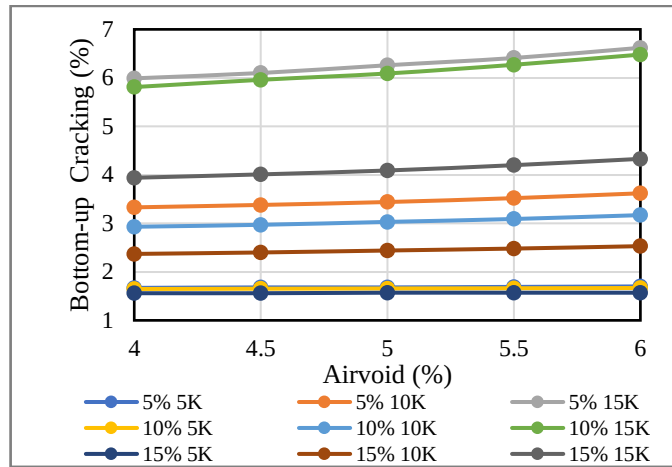


Figure 5.55 Effect of Va in binder course on alligator cracking

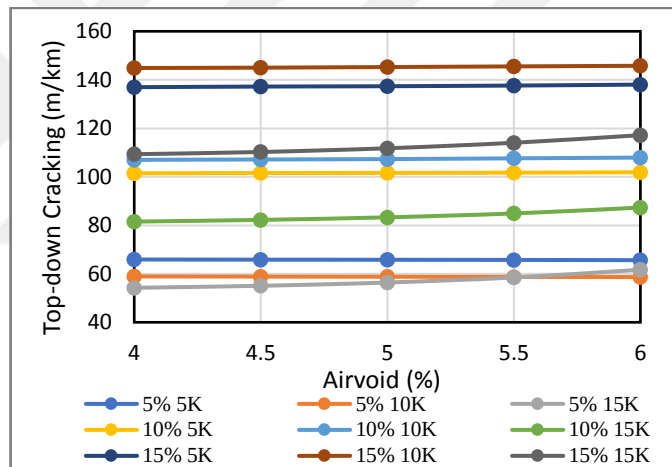


Figure 5.56 Effect of Va in binder course on alligator cracking

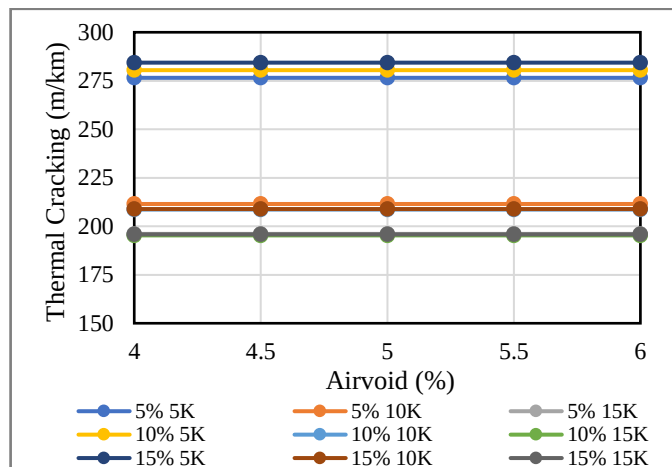


Figure 5.57 Effect of Va in binder course on alligator cracking

5.4.3 Effect of V_a in AC Base Course on Pavement Performance

Based on Highway Technical Specifications, 2013, the recommended air voids (V_a) in the AC base course is higher than air voids in wearing and binder course. The air void is changed from 4 to 6 % in AC base course. The analysis results indicated that for all traffic levels and subgrade types, the variation of V_a in AC base course almost has no effect on total and AC rutting, thermal cracking, and top-down cracking. See Figure 5.58, Figure 5.59, and Figure 5.60, and Figure 5.61.

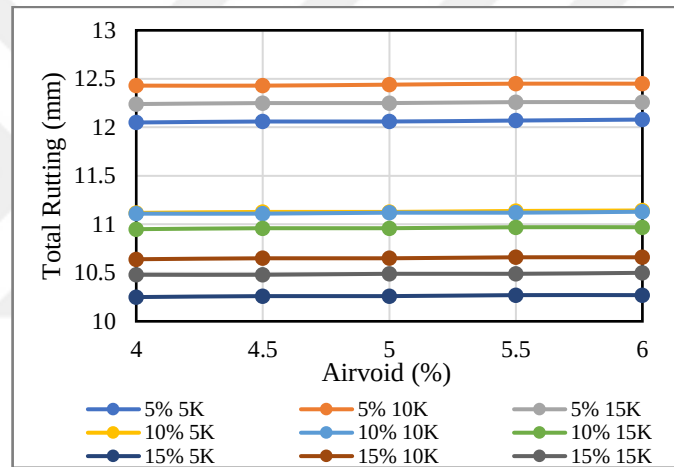


Figure 5.58 Effect of V_a in AC base course on total rutting

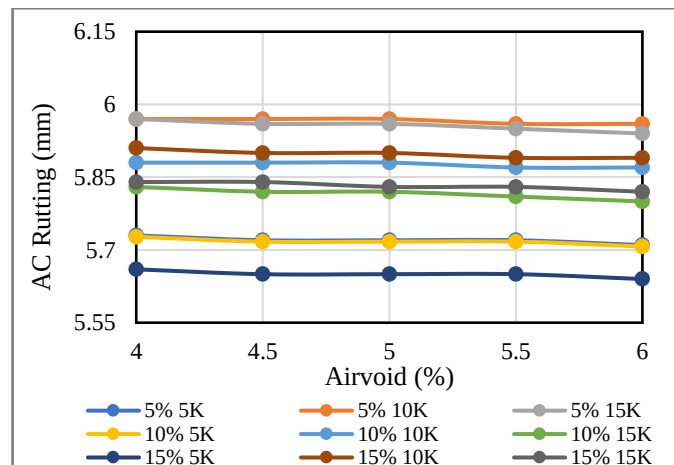


Figure 5.59 Effect of V_a in AC base course on AC rutting

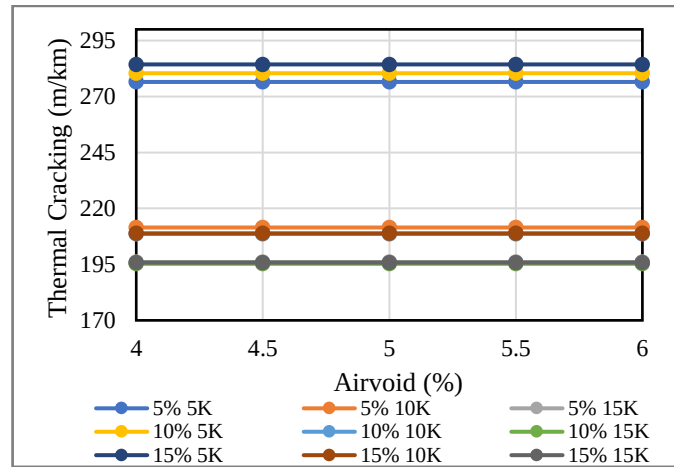


Figure 5.60 Effect of Va in AC base course on thermal cracking

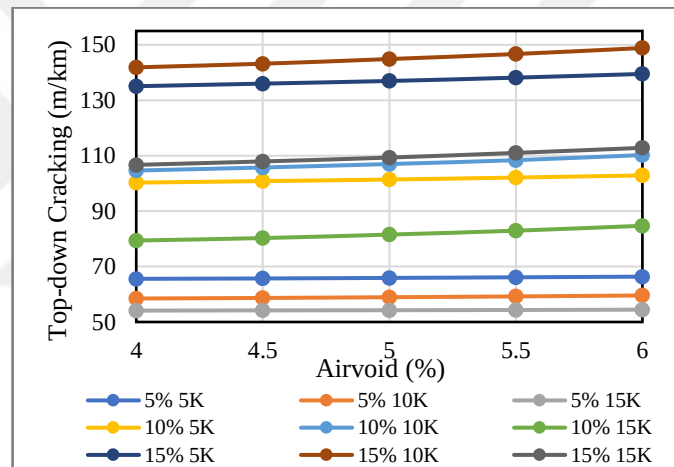


Figure 5.61 Effect of Va in AC base course on top-down cracking

The analysis findings demonstrated that alligator cracking is severely affected by Va change in the AC base course. Alligator cracking rose rapidly when air voids increased from 5 to 6 % in the AC base course, see Figure 5.62. Generally, for all traffic levels and subgrade types, a 2 % reduction in air voids amount in the AC base layer resulted in a 90 % reduction in alligator cracking average magnitude, see Figure 5.62. It can be deduced from Figure 5.62 that when the air voids in the AC base layer is reduced to less than 4 %, almost no alligator cracking occurs in flexible pavement.

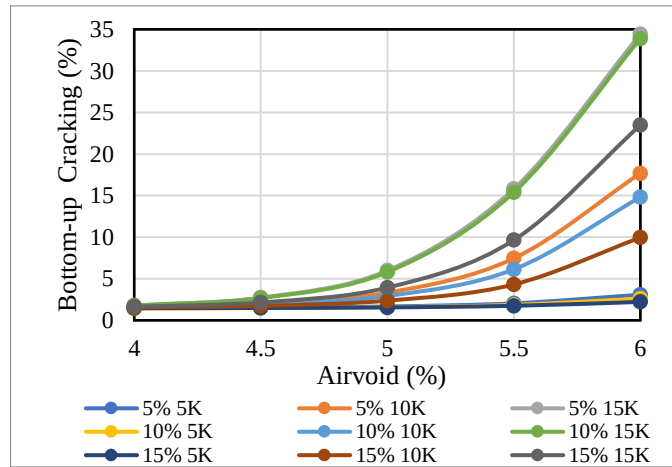


Figure 5.62 Effect of Va in AC base course on alligator cracking

5.5 Effect of Aggregate Gradation on Pavement Performance

The Turkish Highway Technical Specifications suggests various aggregate gradation for different mixtures types in different HMA layers. In this study, the effect of percentage of aggregate passing the 0.075, 4.75, 9.5, and 19mm sieves on pavement performance were investigated and discussed in the following subtitles. Also, the analysis results are shown in Table A.11, Table A.12, Table A.13, and Table A.14. In the Figures 5.63 to 5.82, the percentage of aggregate passing the 0.075, 4.75, 9.5, and 19mm sieves are shown with “Min”, “Mid”, and “Max”. See Table 4.

5.5.1 Effect of Percentage of Aggregate Passing the 0.075 mm sieve

The analysis results indicated that total and AC rutting are sensitive on the percentage of aggregate passing the 0.075 mm sieve. For all traffic levels and subgrade types, increasing of the percentage of aggregate passing the 0.075 mm sieve resulted in a reduction in total and AC rutting, see Figure 5.63 and Figure 5.64.

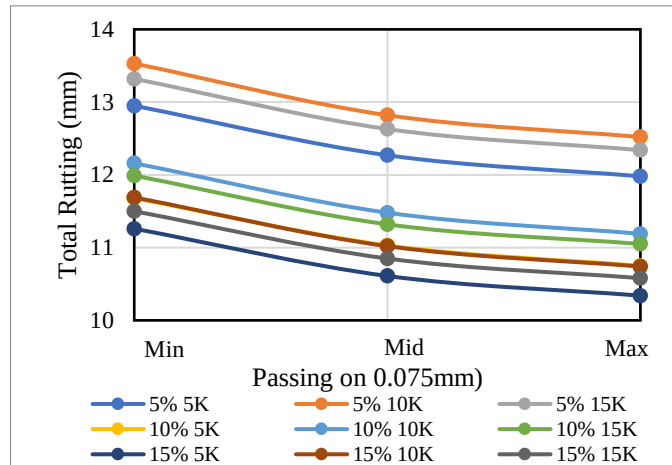


Figure 5.63 Effect of aggregate passing the 0.075 mm sieve on total rutting

Also, results demonstrated that alligator cracking is affected by variation of aggregate passing the sieve 0.075 mm. When the percentage of aggregate passing the 0.075 mm sieve increased the alligator cracking is reduced. See Figure 5.65. According to analysis results, thermal cracking is not sensitive to the variation of aggregate passing the 0.075 mm sieve. See Figure 5.66.

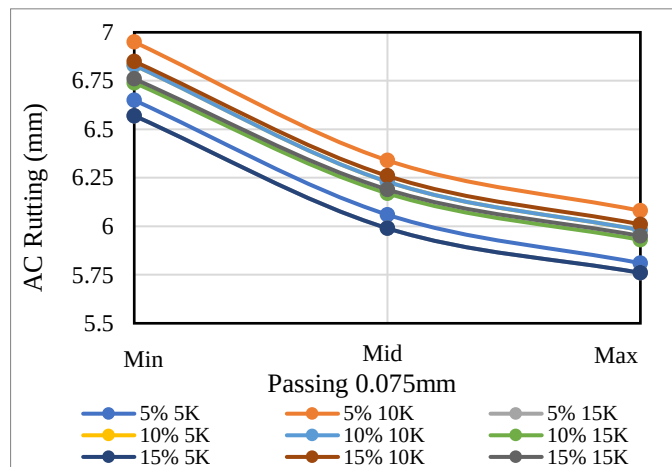


Figure 5.64 Effect of aggregate passing the 0.075 mm sieve on AC rutting

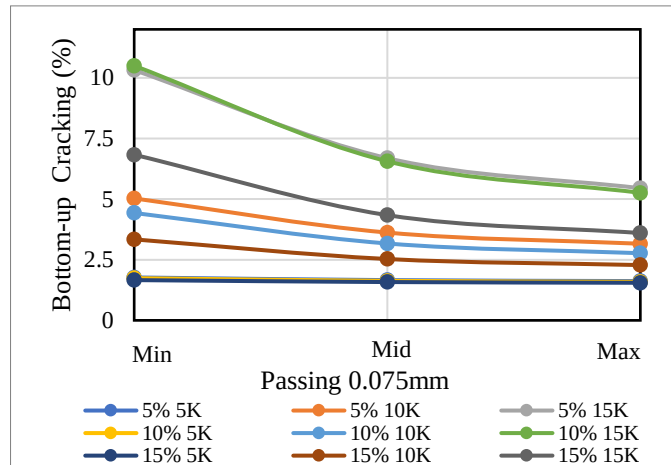


Figure 5.65 Effect of aggregate passing the 0.075 mm sieve on alligator cracking

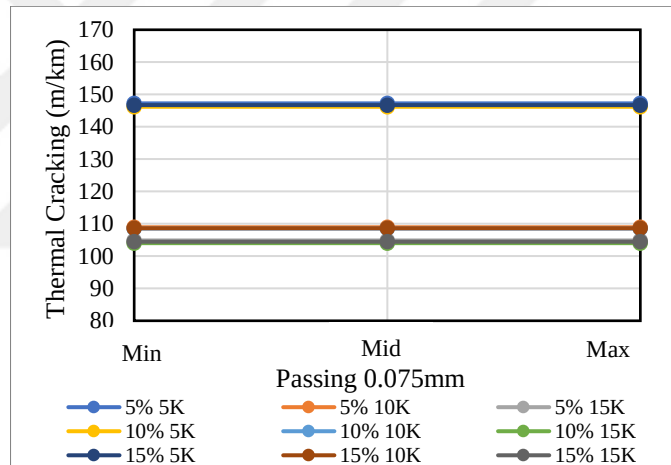


Figure 5.66 Effect of aggregate passing the 0.075 mm sieve on alligator cracking

The analysis results indicated that top-down cracking is sensitive to the variation of aggregate passing the sieve 0.075 mm. For all traffic level and any subgrade types top-down cracking was increased by increasing of the percentage of aggregate passing the 0.075 mm sieve. See Figure 5.67.

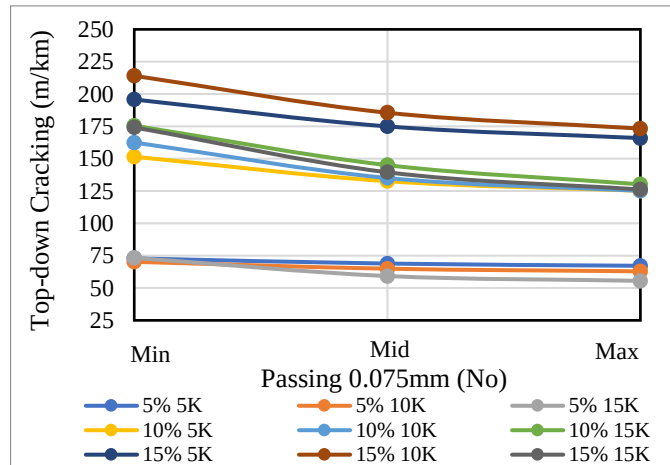


Figure 5.67 Effect of aggregate passing the 0.075 mm sieve on top-down cracking

5.5.2 Effect of Percentage of Aggregate Passing the 4.75 mm sieve

The analysis results revealed that total and AC rutting is sensitive to the variation of aggregate passing the sieve 4.75 mm. For all traffic level and any subgrade types total and AC rutting were increased by increasing of the percentage of aggregate passing the 4.75 mm sieve. See Figure 5.67 and Figure 5.68. The results also indicated that thermal cracking is not sensitive to the variation of the percentage of aggregate passing the 4.75 mm sieve, see Figure 5.69. Results demonstrated that for high traffic levels (15000 AADTT) variation of aggregate passing the 4.75 sieve has a significant effect on alligator cracking. See Figure 5.70. Also, results revealed that top-down cracking is affected by the variation of aggregate passing the 4.75 sieve. Increasing aggregate passing the 4.75 mm sieve resulted in a reduction in top-down cracking, see Figure 5.71.

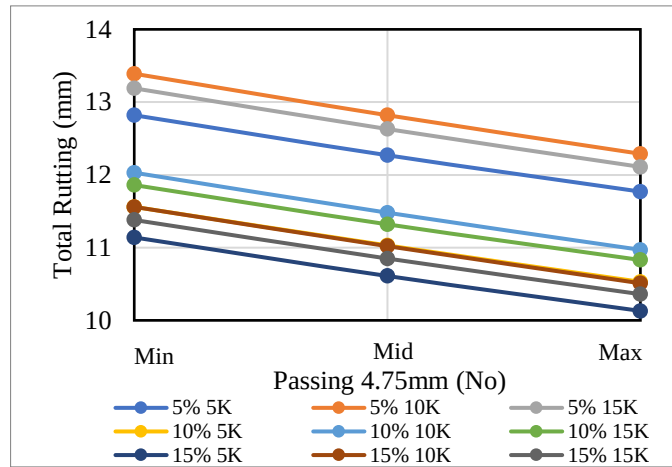


Figure 5.68 Effect of aggregate passing the 4.75 mm sieve on total rutting

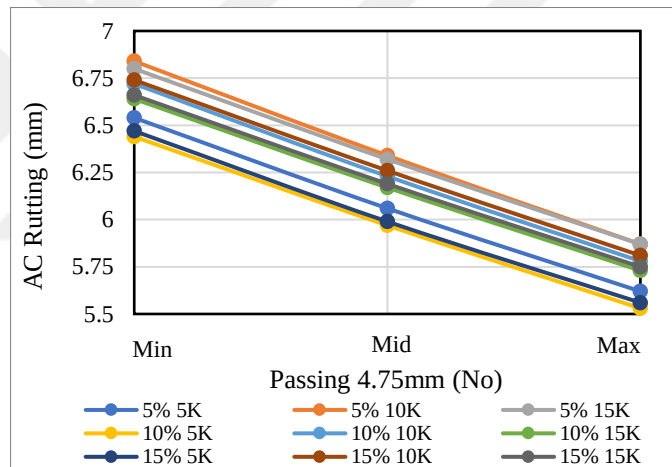


Figure 5.69 Effect of aggregate passing the 4.75 mm sieve on AC rutting

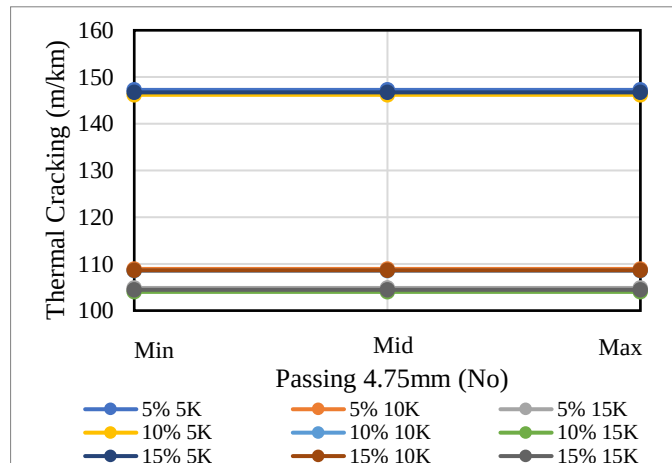


Figure 5.70 Effect of aggregate passing the 4.75 mm sieve on thermal cracking

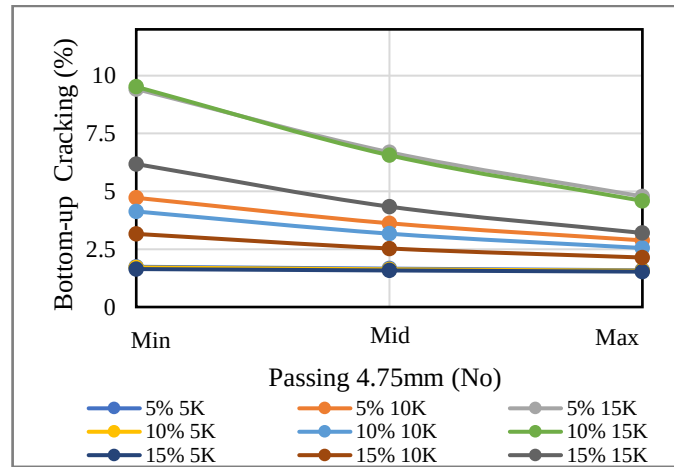


Figure 5.71 Effect of aggregate passing the 4.75 mm sieve on alligator cracking

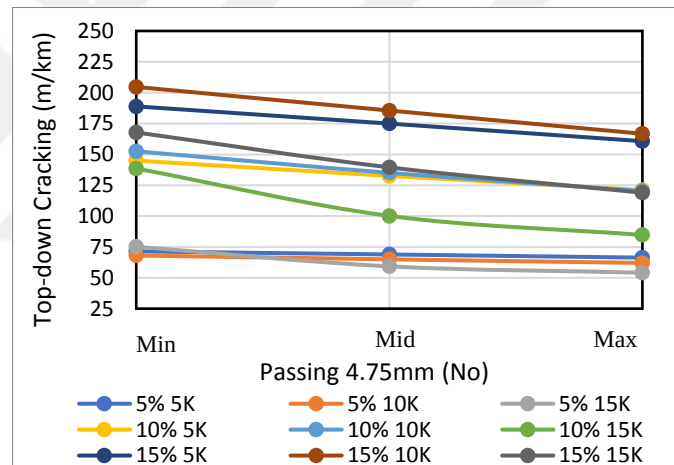


Figure 5.72 Effect of aggregate passing the 4.75 mm sieve on top-down cracking

5.5.3 Effect of Percentage of Aggregate Passing the 9.5 mm sieve

The analysis results indicated that variation of percentage of aggregate passing the 9.5 mm sieve effects the total and AC rutting magnitude. Total and AC rutting are increased by an increase in aggregate passing the 9.5 mm sieve. See Figure 5.72 and Figure 5.73. The results revealed that thermal cracking was not affected by variation of percentage of aggregate passing the 9.5 mm sieve. See Figure 5.74.

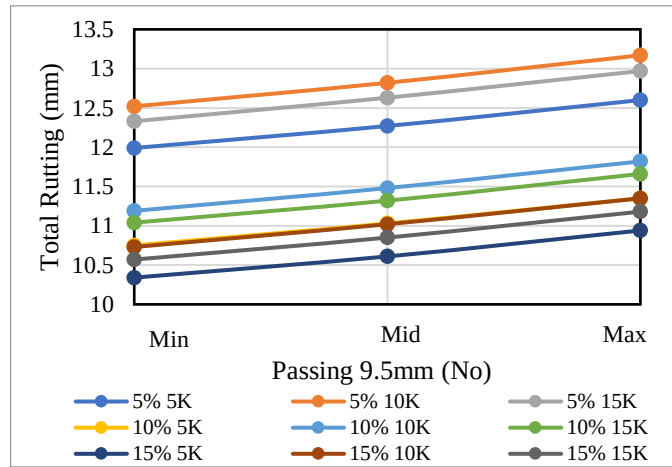


Figure 5.73 Effect of aggregate passing the 9.5 mm sieve on total rutting

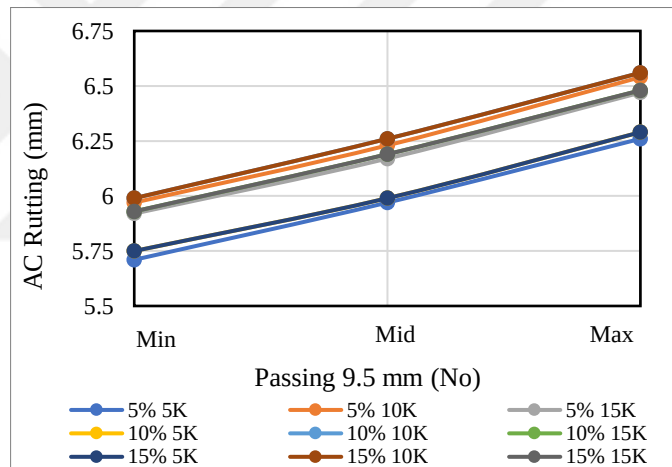


Figure 5.74 Effect of aggregate passing the 9.5 mm sieve on AC rutting

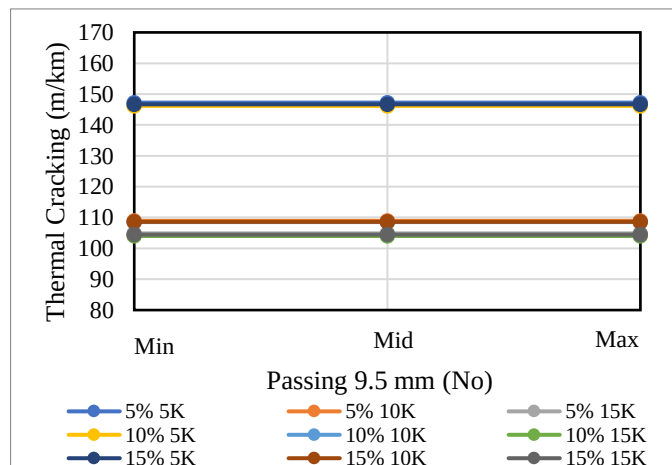


Figure 5.75 Effect of aggregate passing the 9.5 mm sieve on thermal cracking

The analysis results revealed that alligator cracking and top-down cracking were sensitive to the variation of percentage of aggregate passing the 9.5 mm sieve. Alligator cracking and top-down cracking are increased by an increase in aggregate passing the 9.5 mm sieve. See Figure 5.75 and Figure 5.76.

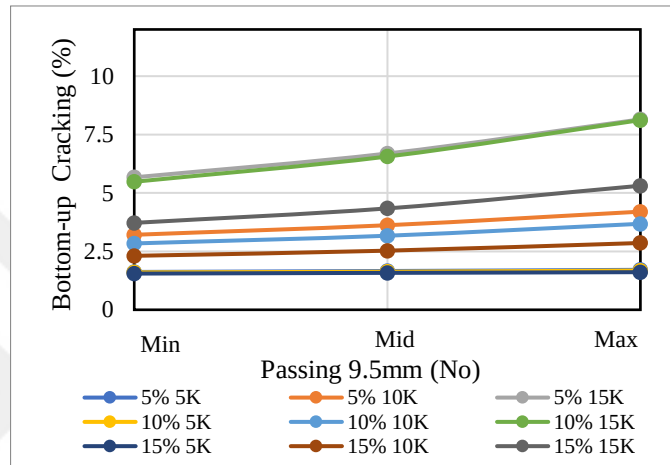


Figure 5.76 Effect of aggregate passing the 9.5 mm sieve on alligator cracking

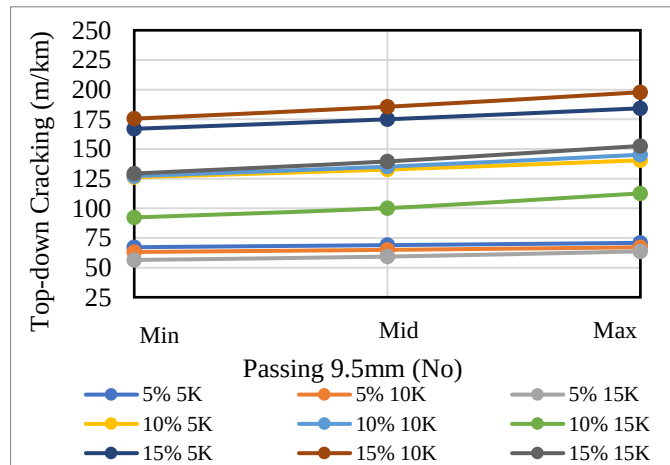


Figure 5.77 Effect of aggregate passing the 9.5 mm sieve on top-down cracking

5.5.4 Effect of Percentage of Aggregate Passing the 19 mm sieve

The analysis results indicated that for all traffic level and subgrade types, total and AC rutting were increased by increasing of aggregate passing the 19 mm sieve. See Figure 5.77, and 5.78.

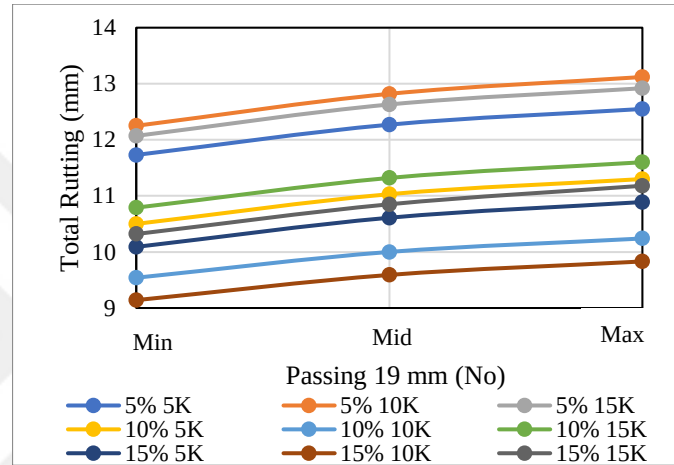


Figure 5.78 Effect of aggregate passing the 19 mm sieve on total rutting

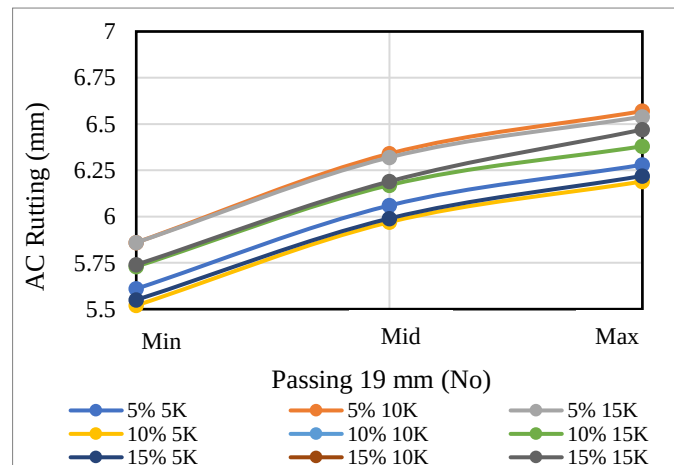


Figure 5.79 Effect of aggregate passing the 19 mm sieve on AC rutting

Also, analysis results indicated that thermal cracking was not sensitive to variation of the aggregate passing the 19 mm sieve. See Figure 5.79.

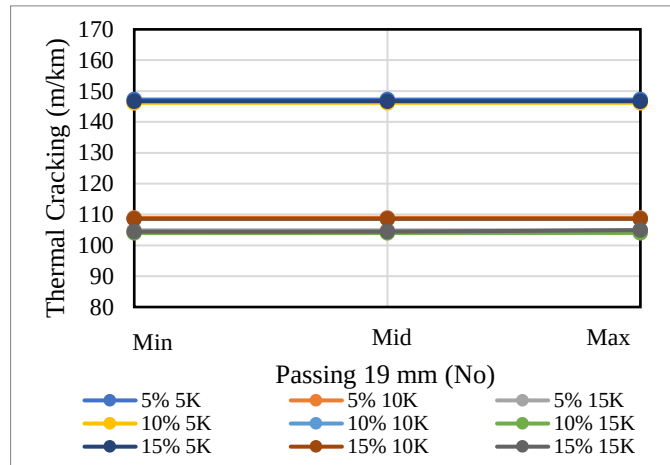


Figure 5.80 Effect of aggregate passing the 19 mm sieve on thermal cracking

The analysis results revealed that alligator cracking and top-down cracking were sensitive to the variation of percentage of aggregate passing the 19 mm sieve. Alligator cracking and top-down cracking are increased by an increase in aggregate passing the 9.5 mm sieve. See Figure 5.80 and Figure 5.81.

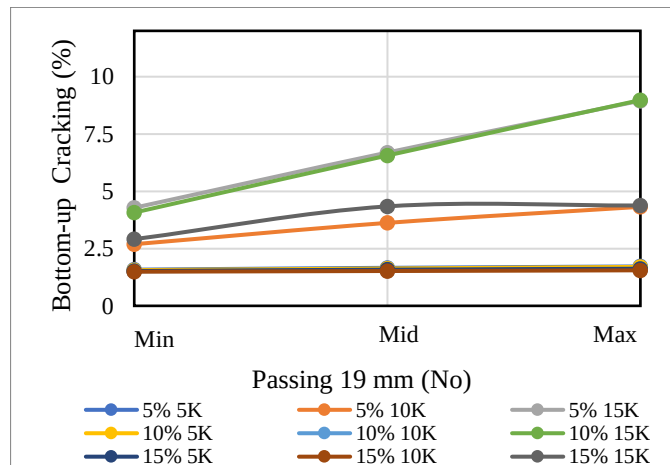


Figure 5.81 Effect of aggregate passing the 19 mm sieve on alligator cracking

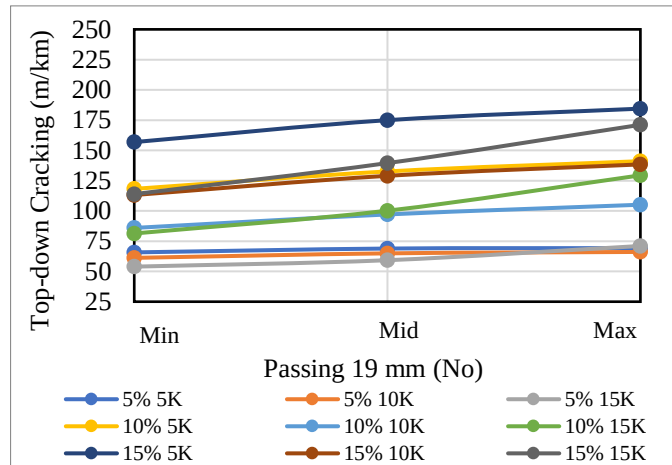


Figure 5.82 Effect of aggregate passing the 19 mm sieve on top-down cracking

5.6 Effect of Vehicle Speed on Pavement Performance

To investigate the effect of traffic speed on flexible pavement performance, analysis was done for different vehicle speeds such as 1, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, and 70 km/h. The summary of results is shown in Table A.15. The analysis results revealed that total rutting was extremely affected by variation of vehicle speed. Generally, for all traffic levels and subgrade types, total rutting was increased by 58.4 % when vehicle speed decreased from 75 to 1 km/h. See Figure 5.83.

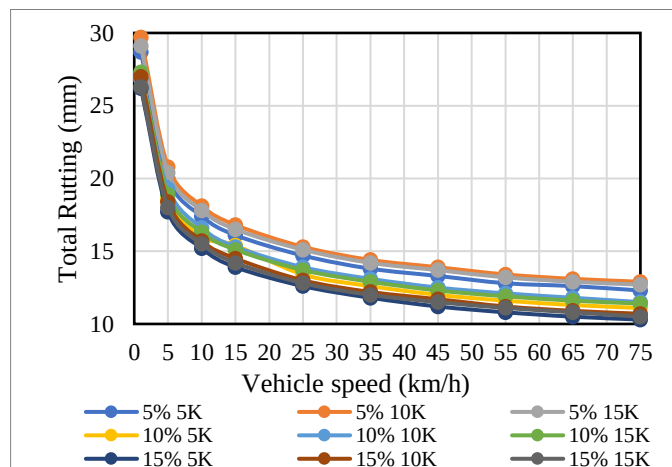


Figure 5.83 Effect of vehicle speed on total rutting

The results demonstrated that variation of vehicle speed has a significant effect on AC rutting. For all traffic levels and any subgrade type, when vehicle speed decreased from 75 to 1 km/h the AC rutting was increased by 71.6 %. Figure 5.84 shows that AC rutting is increased rapidly for traffic speeds bellow 25 km/h. Also, results indicated that for all traffic levels and subgrade types, the variation of vehicle speed has a negligible effect on base/subbase and subgrade rutting. Figure 5.85 shows that for all traffic levels and subgrade types, decreasing traffic speed from 75 to 1 km/h results in a little increase (8.8 %) in base/subbase and subgrade rutting.

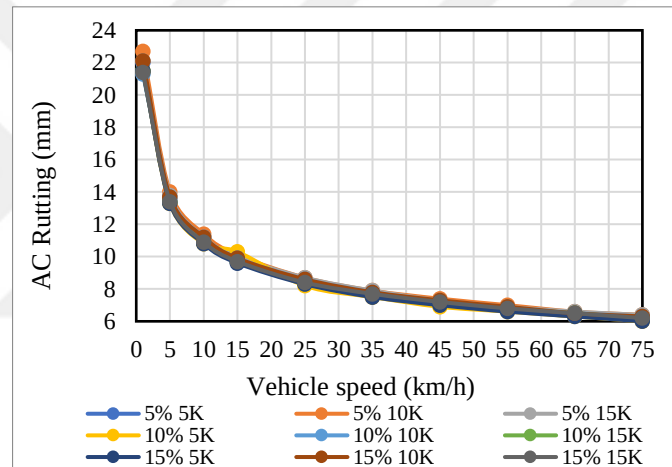


Figure 5.84 Effect of vehicle speed on AC rutting

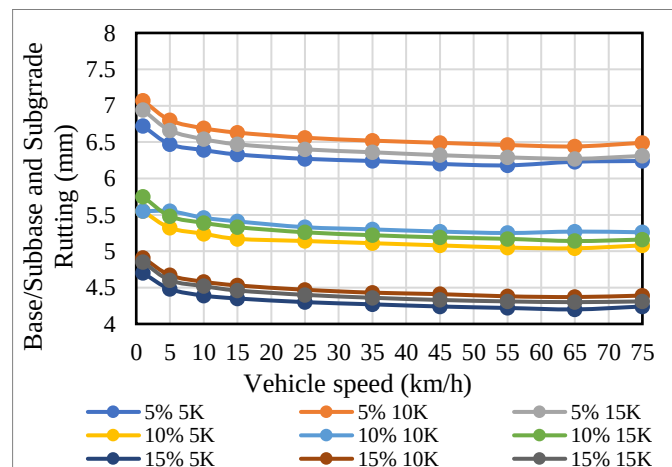


Figure 5.85 Effect of vehicle speed on base/subbase and subgrade rutting

The analysis results revealed that alligator cracking is highly sensitive to the variation of vehicle speed. Generally, for all traffic levels and subgrade types, reducing vehicle speed from 75 to 1 km/h led to an 84 % increase in alligator cracking. See Figure 5.86. Also, results indicated that thermal cracking was not affected by the variation of traffic speed, see Figure 5.87.

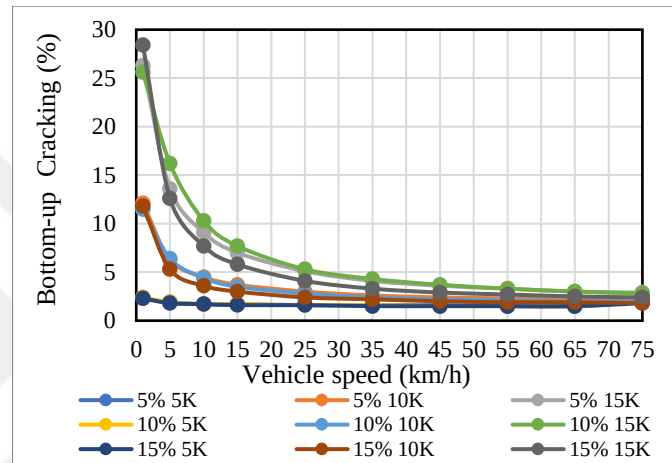


Figure 5.86 Effect of vehicle speed on alligator cracking

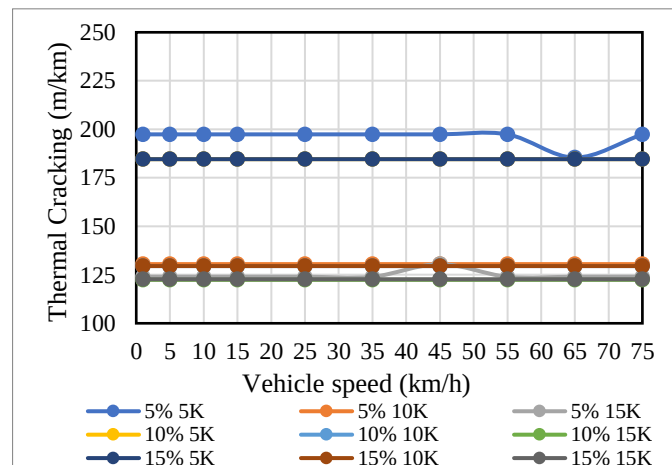


Figure 5.87 Effect of vehicle speed on thermal cracking

The results demonstrated that variation of traffic speed has a significant impact on top-down cracking. Figure 5.88 shows that for traffic speeds higher than 15 km/h the top-down cracking is not changed much but for speeds lower than 15 km/h, top-down

cracking is increased very rapidly. Generally, when traffic speed decreased from 75 to 1 km/hr top-down cracking increased by 94 %. See Figure 5.88

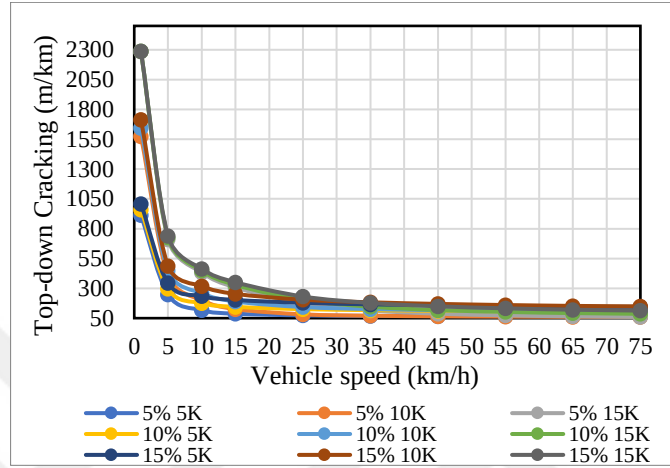


Figure 5.88 Effect of vehicle speed on top-down cracking

5.7 Effect of Binder Grade on Pavement Performance

To determine the impact of binder grade on pavement performance different binder performance grade were evaluated. The analysis results are illustrated in Figures 5.89, 5.90, 5.91, 5.92, and 5.93. In the figures, each binder grade is shown with a number such as PG 58-16 with (1), PG 58-22 with (2), PG 58-28 with (3), PG 64-16 with (4), PG 64-22 with (5), PG 64-28 with (6), PG 70-10 with (7), PG 70-16 with (8), and PG 70-22 with (9). Also, the summary of results is shown in Table A.18.

The analysis results indicated that total, and AC rutting are significantly affected by binder grade types. For all traffic levels and subgrade types, when PG 70-10 and PG 70-16 is used as binder the flexible pavement showed better performance in terms of total and AC rutting compared to other PG grades (PG 58-16, PG 58-22, PG 58-28, PG, 64-16, PG 64-22, PG 64-28, and PG 70-22). See Figure 5.89 and Figure 5.90.

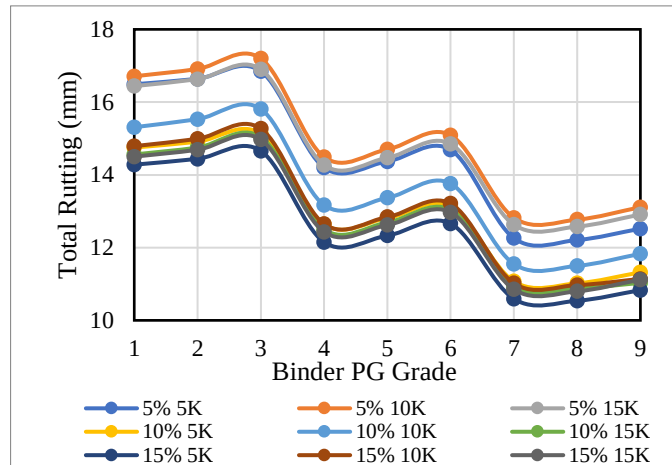


Figure 5.89 Effect of binder grade on total rutting

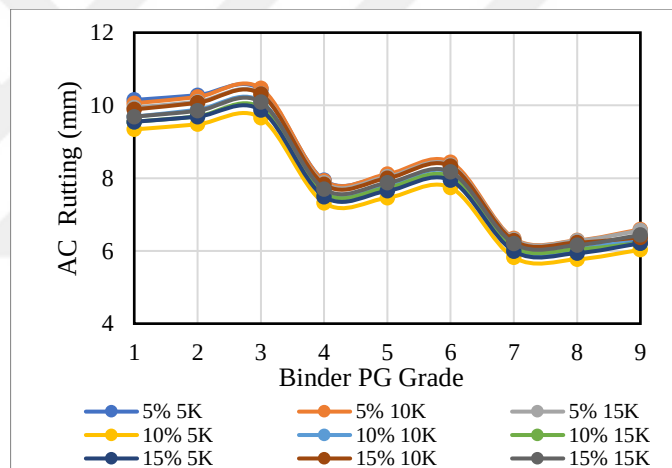


Figure 5.90 Effect of binder grade on AC rutting

The results indicated that alligator, top-down, and thermal cracking are affected by binder grade types. For all traffic levels and subgrade types, when PG 70-10 and PG 70-16 is used as a binder, the flexible pavement showed better performance in terms of alligator and top-down cracking compared to other binder grades (PG 58-16, PG 58-22, PG 58-28, PG, 64-16, PG 64-22, PG 64-28, and PG 70-22). See Figure 5.91 and Figure 5.92. Also, results revealed that thermal cracking decreased when PG 64-28 is used as binder grade, see Figure 5.93.

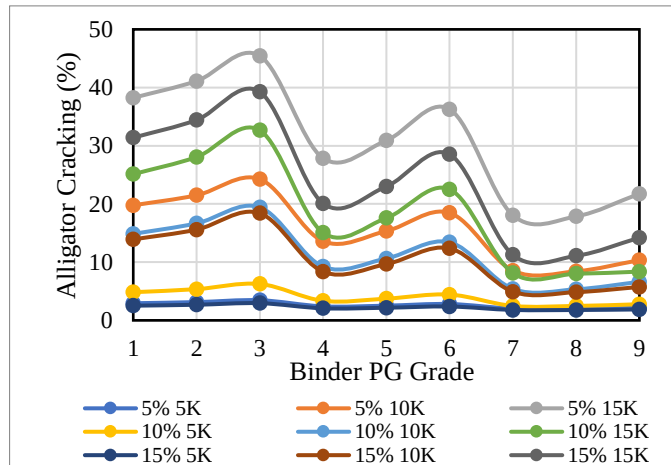


Figure 5.91 Effect of binder grade on alligator cracking

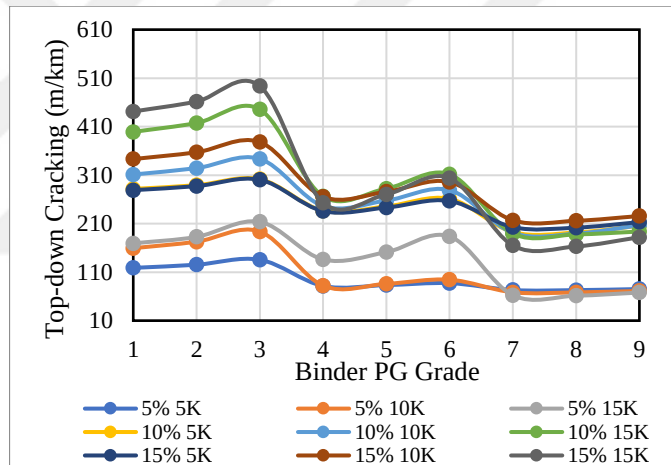


Figure 5.92 Effect of binder grade on top-down cracking

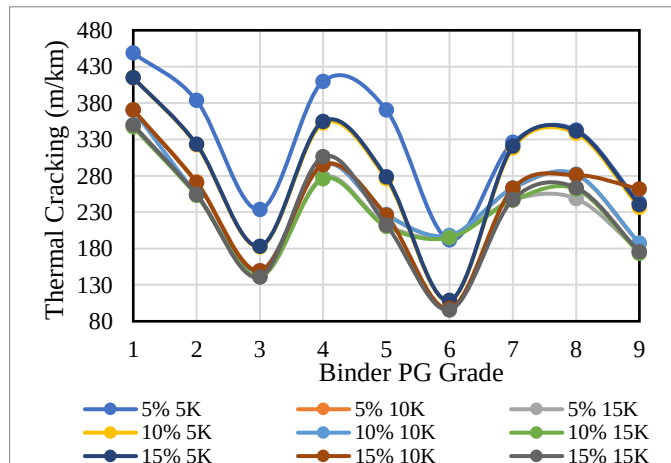


Figure 5.93 Effect of binder grade on thermal cracking

The General Directorate of Highways has suggested that PG 64-16 can be used for vast majority of areas in the Izmir climatic zone. But, results of this study revealed that flexible pavement would have better performance when PG 70-10 and PG 70-16 are used as binder. Also, the comparison of predicted dynamic modulus of the HMA D-1 mixture for all PG grades showed that dynamic modulus for PG 70-10 and PG 70-16 are higher than for other grades at high temperature (54.4 °C). Figure 5.94 showed that in the presence of low temperature (-10 °C), the dynamic modulus of HMA D-1 mixture for all binder grades were almost equal. But with increasing temperature, the dynamic modulus for all PG grades was decreased at different rates. The PG 70-16 and PG 70-10 had the lowest reduction rates, respectively. See Figure 5.94. On the other hand, the comparison of predicted dynamic modulus of HMA D-1 mixture for Pen. and PG grades demonstrated that the dynamic modulus of D-1 mixture for Pen. 85-100 and PG 64-16 are almost equal. While the dynamic modulus for Pen. 40-50 and Pen. 60-70 were close to PG 70-16 and PG 70-10 respectively. The dynamic modulus for different binder grades (penetration and PG) are illustrated in Figure 5.95.

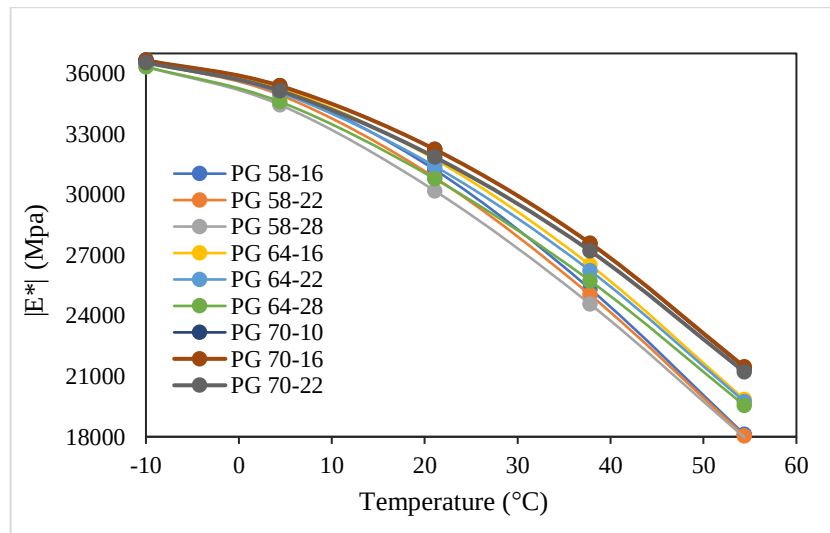


Figure 5.94 Predicted dynamic modulus of HMA D-1 mixture for different PG grades

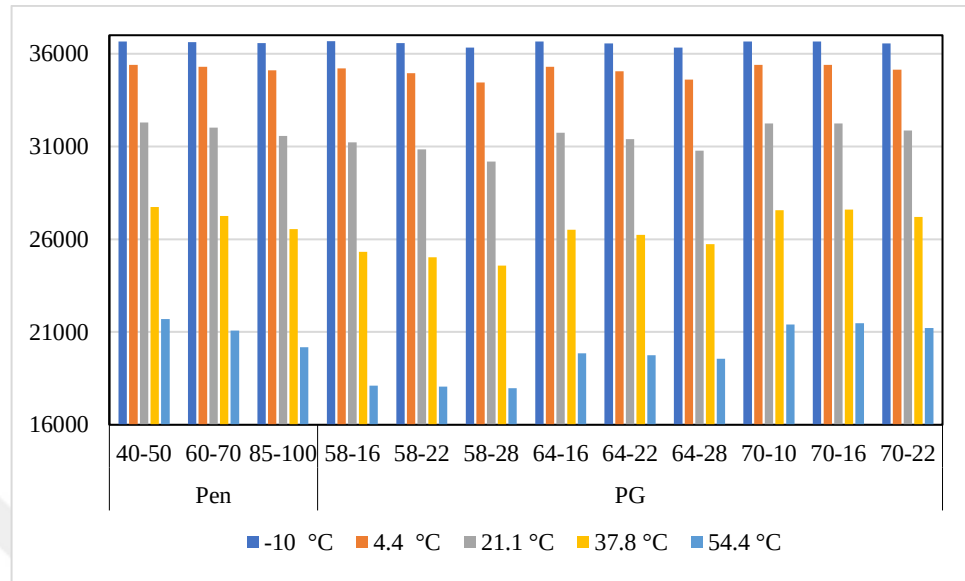


Figure 5.95 Predicted dynamic modulus of HMA D-1 mixture for different binder grades

5.8 Effect of Mixture Type on Pavement Performance

In order to determine the impact of different mixture types on pavement performance two HMA dense graded mixtures (D-1 and D-2) and two types of SMA mixtures were analyzed. The summary of analysis results was illustrated in Table A. 19. Also, the analysis results were discussed in details as follows.

The analysis results demonstrated that for all traffic and subgrade levels, total and AC rutting depth are lower for SMA mixtures (S-1 and S-2) compared to dense graded HMA mixtures (D-1 and D-2). See Figure 5.96 and Figure 5.97. This result is based on field observation and behavior of SMA pavement. Also, results revealed that alligator cracking in SMA mixtures is high which is contrary to the SMA pavement performance. See Figure 5.98. One of the main reasons can be the placing of dense graded AC base course with higher V_a amount and lower V_{be} percentage as well as lower Poisson's ratio. Also results indicated that top-down cracking is significantly reduced in SMA pavement, see Figure 5.99.

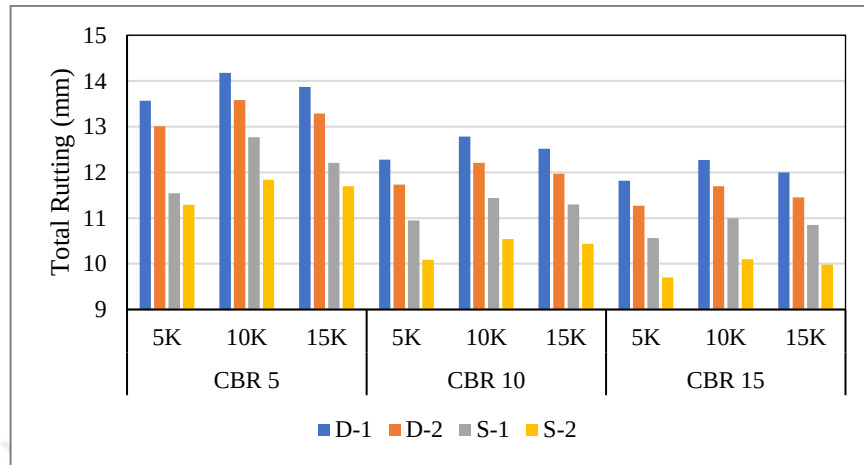


Figure 5.96 Effect of mixture types on total rutting

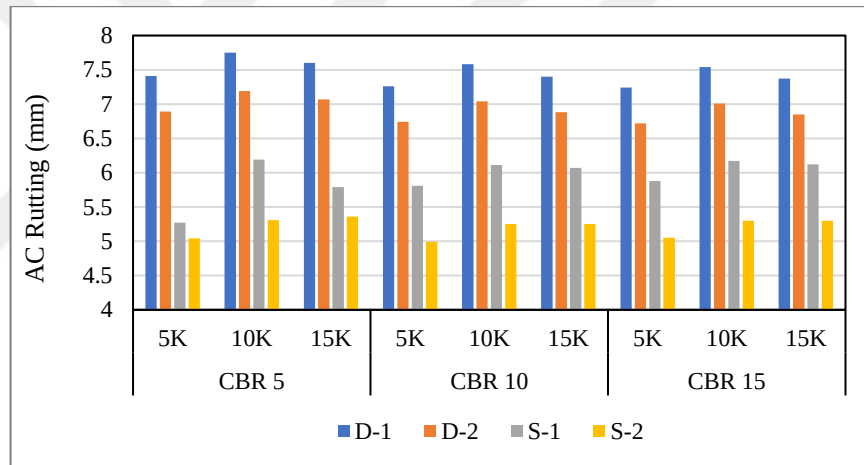


Figure 5.97 Effect of mixture types on AC rutting

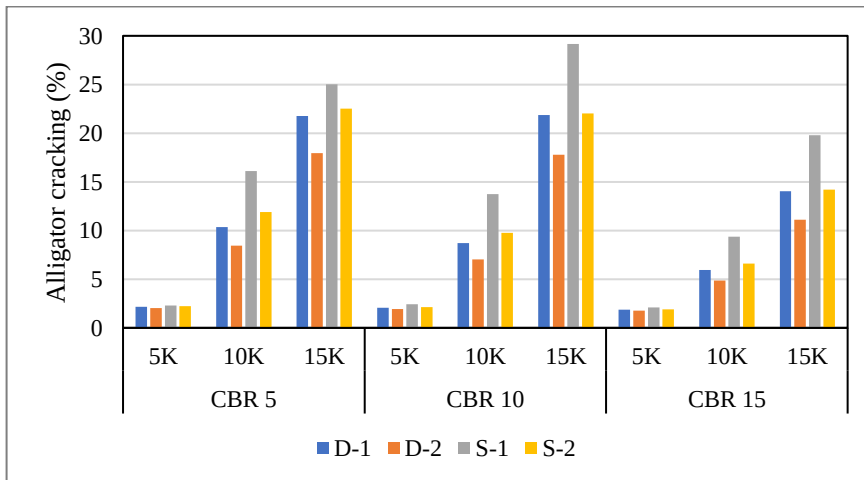


Figure 5.98 Effect of mixture types on alligator cracking

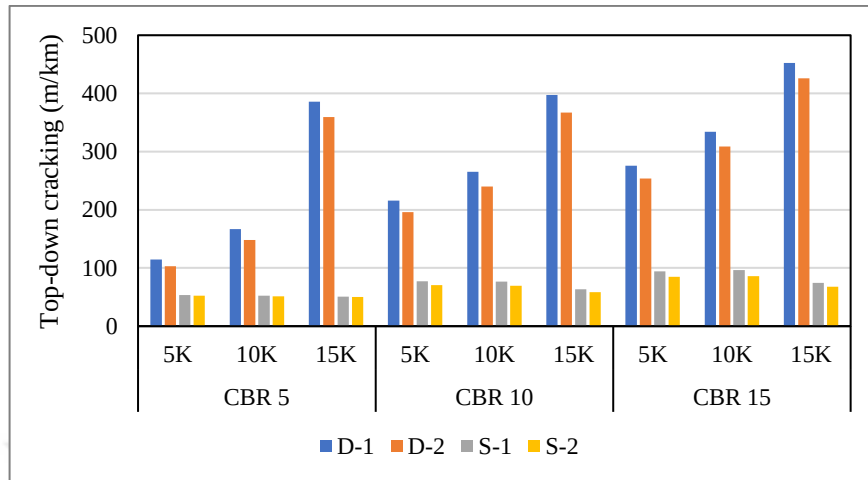


Figure 5.99 Effect of mixture types on top-down cracking

Also results demonstrated that thermal cracking is high in SMA mixtures compared to dense graded mixtures, see Figure 5.100. The existing of low air voids and higher Poisson's ration in the wearing course would lead to higher thermal cracking in the pavement. See Figure 5.

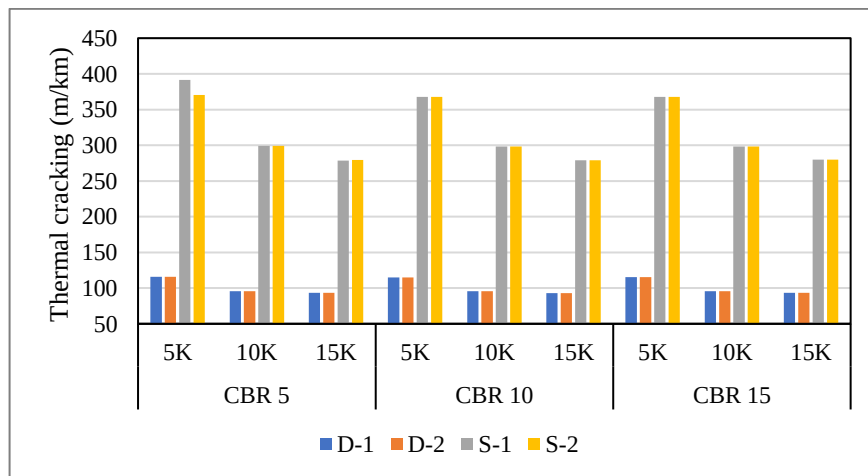


Figure 5.100 Effect of mixture types on thermal cracking

5.9 Effect of Threshold Values and Reliability Levels on Pavement Thickness

To determine the effect of threshold values and reliability levels on pavement thickness, analysis was done in three steps (85 % reliability with higher threshold values, 90 % reliability with medium threshold values, and 95 % with lower threshold values). The analysis was conducted for three different subgrade types (CBR values of 5, 10, and 15 %). The analysis results were discussed in detail in the following sub titles. Also, the summary of analysis results was illustrated in Table A.20, Table A.21, and Table A.22.

5.9.1 Effect of Threshold Values on Pavement Thickness for Lower Subgrade Type (CBR Value of 5 %)

The analysis results indicated that for all traffic levels and lower subgrade type, the same pavement structure is significantly oversized for 85 % reliability level and higher threshold values in terms of total and AC rutting. See Figure 5.101 and Figure 5.102. Also, results revealed that pavement structure is over design in terms of total and AC rutting at 90 % reliability level and medium threshold values. Results demonstrated that pavement structure for lower subgrade type is under-designed in terms of total and AC rutting at 90 % reliability level and for lower threshold values.

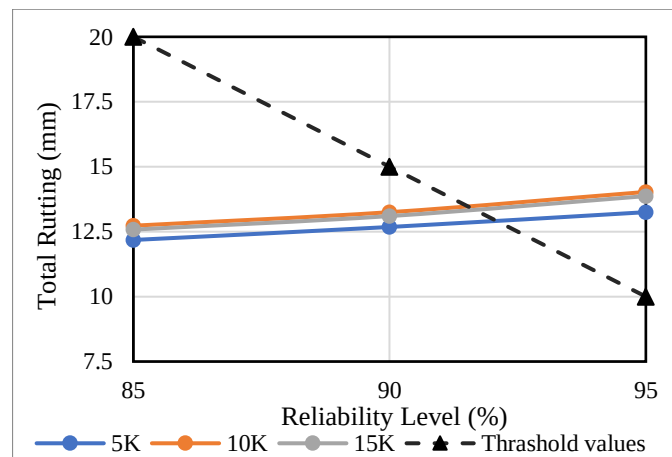


Figure 5.101 Effect of threshold values and reliability levels on total rutting for CBR 5 %

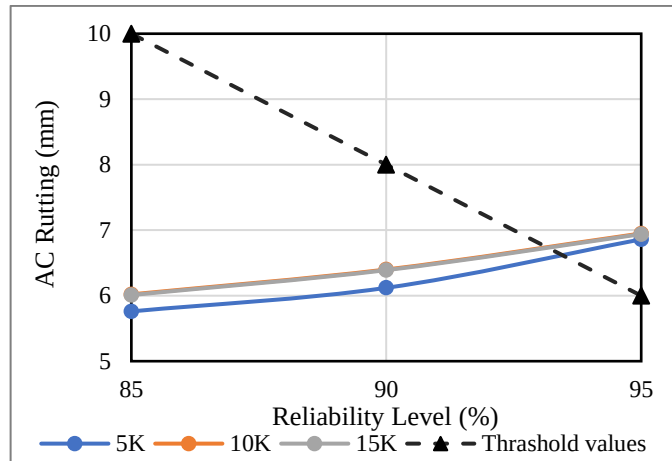


Figure 5.102 Effect of threshold values and reliability levels on AC rutting for CBR 5 %

The analysis results indicated that pavement structure in terms of alligator cracking is overestimated at 85 % reliability and for lower threshold values. Also, according to the analysis results, the pavement structure for lower subgrade type (CBR value of 5 %) and higher traffic level in terms of alligator cracking is optimized designed but for other cases the pavement structure is oversized. For all traffic levels and lower subgrade type (CBR value of 5 %) the pavement structure in terms of alligator cracking is underestimated at 95 % reliability level and lower threshold values. See Figure 5.103.

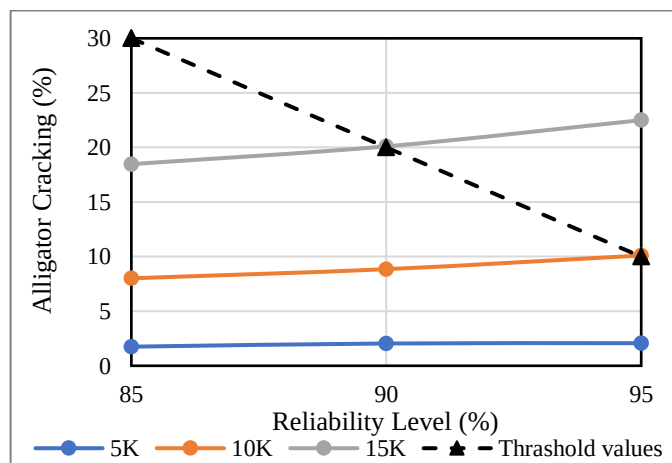


Figure 5.103 Effect of threshold values and reliability levels on alligator cracking for CBR 5 %

The analysis results demonstrated that for all traffic levels and lower subgrade type (CBR value of 5 %), the pavement structure in terms of thermal and top-down cracking at all reliability level and for any threshold values are oversized. See Figure 5.104 and Figure 5.105.

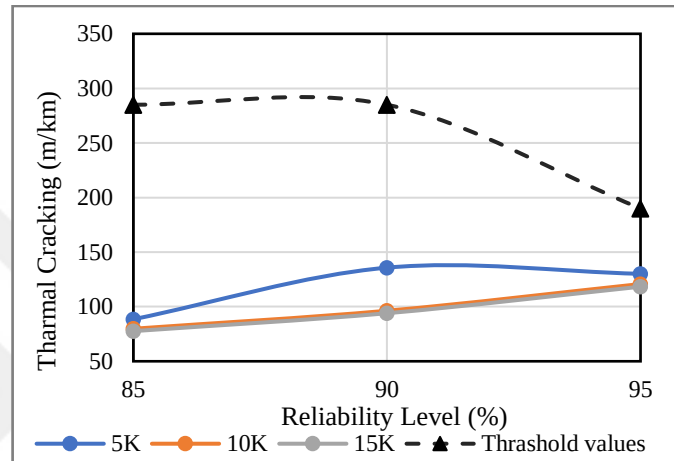


Figure 5.104 Effect of threshold values and reliability levels on thermal cracking for CBR 5 %

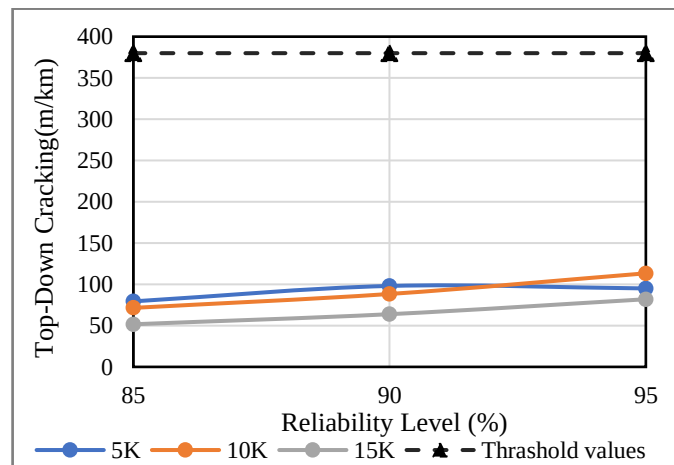


Figure 5.105 Effect of threshold values and reliability levels on top-down cracking for CBR 5 %

5.9.2 Effect of Threshold Values on Pavement Thickness for Medium Strength Subgrade Type (CBR 10 %)

The analysis results demonstrated that for all traffic levels and medium strength subgrade type, the pavement structure in terms of total and AC rutting at 85 and 90 % reliability levels and for higher and medium threshold values are overdesigned. While the pavement structure for lower threshold values at 95 % reliability level is underestimated. See Figure 5.106 and Figure 5.107.

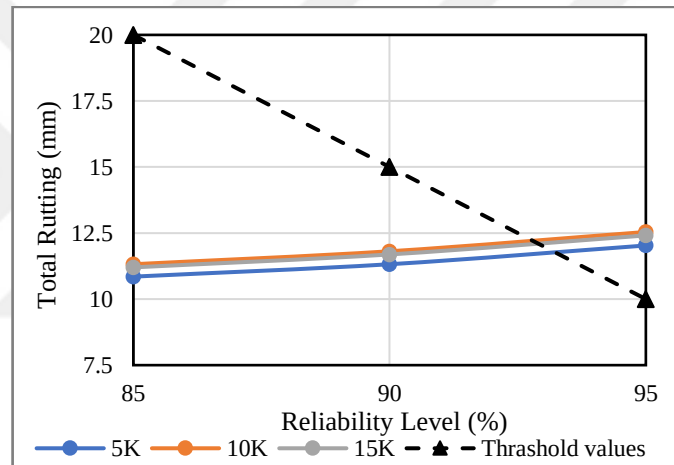


Figure 5.106 Effect of threshold values and reliability levels on total rutting for CBR 10 %

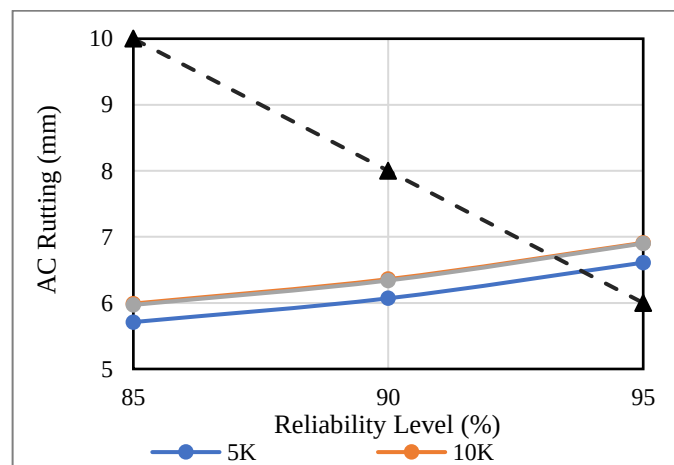


Figure 5.107 Effect of threshold values and reliability levels on AC rutting for CBR 10 %

The analysis results revealed that pavement structure in terms of alligator cracking is overestimated for all traffic levels and medium strength subgrade type at 85 and 90 % reliability levels and for higher and medium threshold. Also, results revealed that for lower threshold values at high reliability level the pavement structure is under-designed for high traffic level (15000 AADTT) but for the same conditions the pavement structure for lower and medium traffic levels (5000 and 10000 AADTT) is overdesigned. See Figure 5.108.

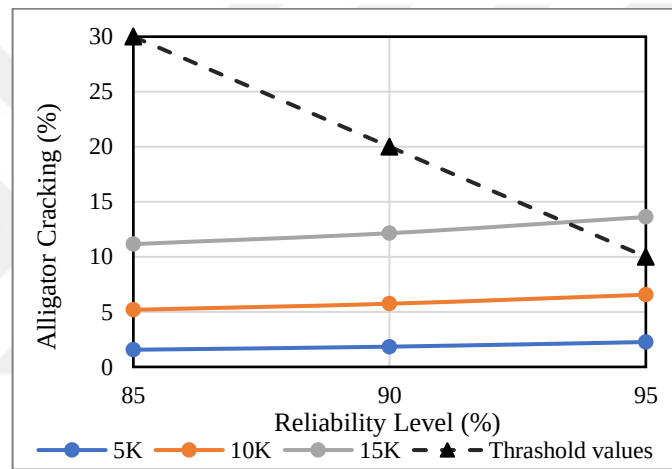


Figure 5.108 Effect of threshold values and reliability levels on alligator cracking for CBR 10 %

The analysis results also indicated that for all traffic levels and medium strength subgrade type (CBR value of 10 %), the pavement structure in terms of thermal and top-down cracking at all reliability levels and for any threshold values are overdesigned. See Figure 5.109 and Figure 5.110.

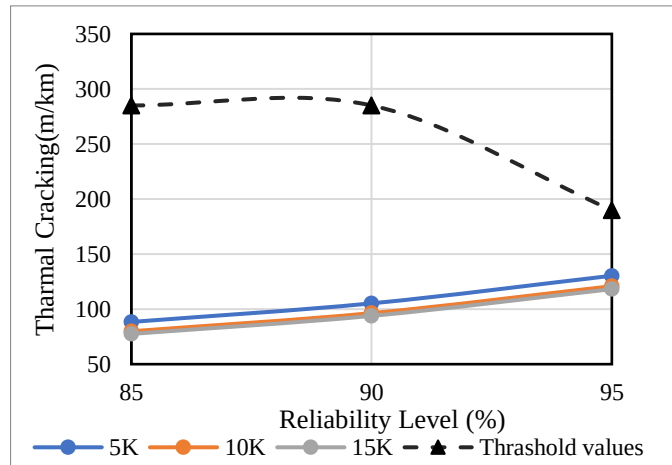


Figure 5.109 Effect of threshold values and reliability levels on thermal cracking for CBR 10 %

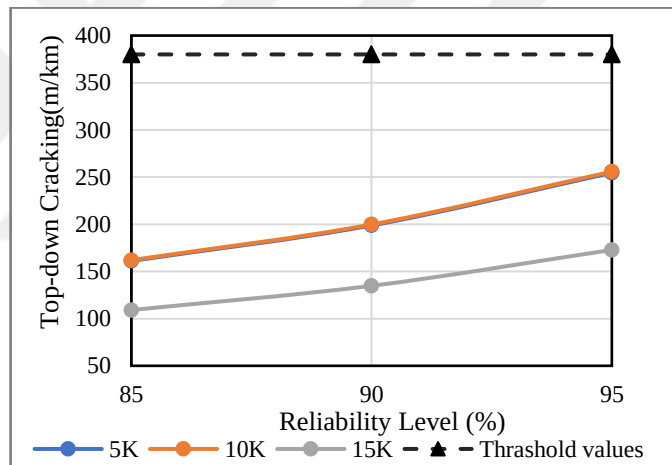


Figure 5.110 Effect of threshold values and reliability levels on top-down cracking for CBR 10 %

5.9.3 Effect of Threshold Values on Pavement Thickness for Stiffer Subgrade Type (CBR 15 %)

The analysis results indicated that for all traffic levels and stiffer subgrade type (CBR value of 15 %), the pavement structure in terms of total and AC rutting at 85 and 90 % reliability levels and for higher and medium threshold values are oversized. While the pavement structure for lower threshold values at 95 % reliability level is underestimated. See Figure 5.111 and Figure 5.112.

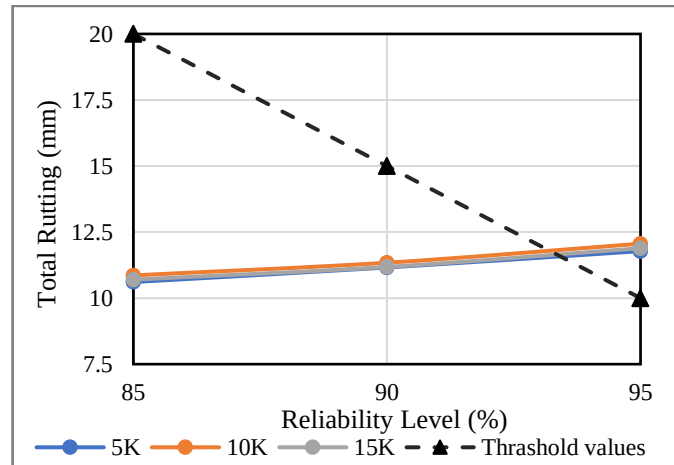


Figure 5.111 Effect of threshold values and reliability levels on total rutting for CBR 15 %

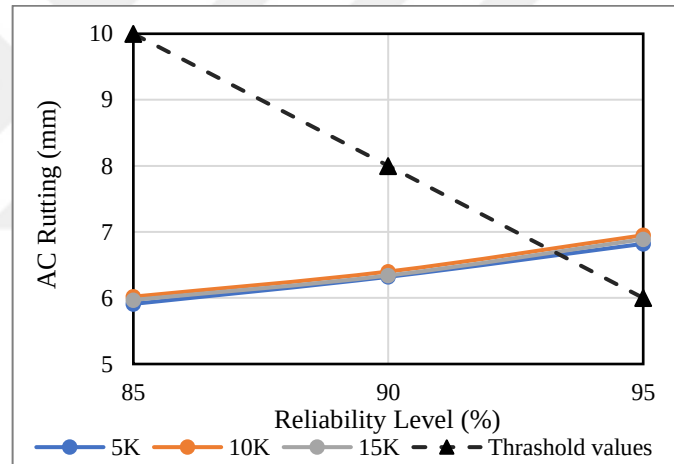


Figure 5.112 Effect of threshold values and reliability levels on AC rutting for CBR 15 %

The analysis results also indicated that for all traffic levels and stiffer subgrade type (CBR value of 15 %), the pavement structure in terms of alligator cracking at 85 and 90 % reliability levels and for any threshold values are overdesigned. But for lower threshold values and at 95 % reliability levels the pavement structure is optimized designed for high traffic level (15000 AADTT) and underestimated for low and medium traffic levels (5000 and 10000 AADTT). See Figure 5.113.

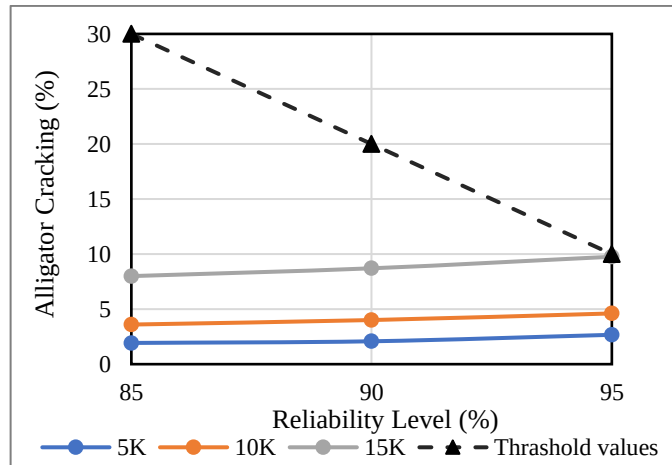


Figure 5.113 Effect of threshold values and reliability levels on alligator cracking for CBR 15 %

The analysis results demonstrated that for all traffic levels and stiffer subgrade type (CBR value of 15 %), the pavement structure in terms of thermal and top-down cracking at all reliability levels and for any threshold values are overdesigned. See Figure 5.114 and Figure 5.115.

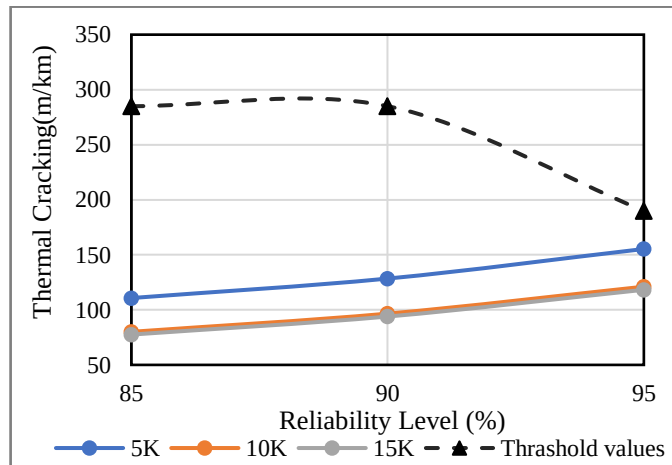


Figure 5.114 Effect of threshold values and reliability levels on thermal cracking for CBR 15 %

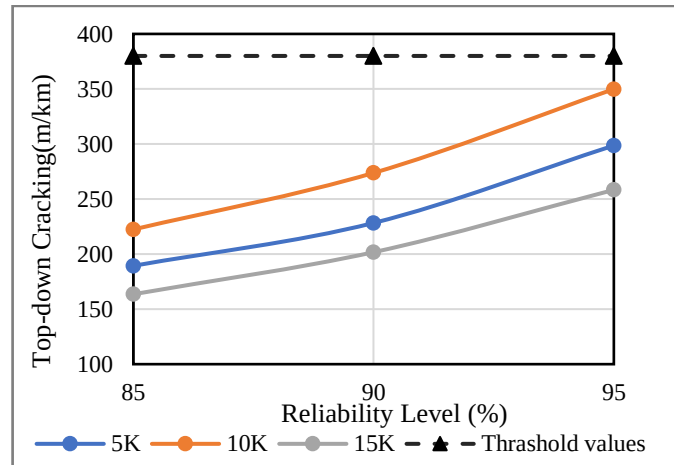


Figure 5.115 Effect of threshold values and reliability levels on top-down cracking for CBR 15 %

5.9.4 Optimized Pavement Design

To achieve the optimized pavement design, the pavement layer thicknesses (wearing, binder, and AC base course) were changed to reduce the difference between the predicted distresses and the targeted values as minimum as possible. The summary of analysis results was illustrated in Table 5.1. Table A.23, Table A.24, and Table A.25.

The analysis results demonstrated that for all traffic levels and subgrade types the pavement thickness increased when threshold values changed from higher to medium and reliability increased from 85 to 90 %. While, a change in threshold values (from medium to low) and increasing reliability level (from 85 to 95 %) resulted in a significant increase in the pavement thickness, see Table 5.1. Figure 5.112 shows that changing design performance values from higher to medium and increasing reliability level from 85 to 90 % resulted in 16.73 % increase in the HMA average volume for all traffic levels and subgrade types. Also, it can be deduced from Figure 5.116 that when the threshold values changed from medium to lower and reliability level increased from 90 to 95 %, the HMA average volume increased by 35.69 %. And when threshold

values changed from high to low and reliability level changed from 85 to 95 %, the HMA average volume was increased by 52.4 %, see Figure 5.116.

Table 5.1 Effect of threshold values and reliability on pavement thickness

Subgrade CBR	Traffic (AADTT)	Pavement Layer type, mm	Predicted Pavement Layer Thickness @ Specified Reliability level			HMA Volume at 1 Sqm at Specified Reliability, M ³		
			85%	90%	95%	85%	90%	95%
5%	5K	Wearing course	40	40	80	0.04	0.04	0.08
		Binder course	60	80	100	0.06	0.08	0.1
		AC base course	0	0	120	0.00	0.00	0.12
	10K	Wearing course	50	50	80	0.04	0.04	0.08
		Binder course	60	60	120	0.05	0.06	0.11
		AC base course	0.0	100	140	0.00	0.1	0.15
	15K	Wearing course	50	50	80	0.05	0.06	0.08
		Binder course	100	110	120	0.1	0.1	0.12
		AC base course	110	140	160	0.11	0.12	0.16
10%	5K	Wearing course	40	40	60	0.04	0.04	0.06
		Binder course	50	80	100	0.05	0.08	0.1
		AC base course	0	0	120	0.00	0.00	0.12
	10K	Wearing course	50	50	80	0.04	0.04	0.07
		Binder course	60	60	100	0.05	0.06	0.11
		AC base course	0.0	100	120	0.00	0.08	0.14
	15K	Wearing course	50	50	80	0.05	0.06	0.08
		Binder course	80	100	110	0.08	0.1	0.1
		AC base course	100	120	140	0.11	0.12	0.14
15%	5K	Wearing course	40	40	60	0.04	0.04	0.06
		Binder course	60	70	90	0.06	0.08	0.1
		AC base course	0.0	0	110	0	0	0.11
	10K	Wearing course	50	50	60	0.04	0.04	0.06
		Binder course	60	60	100	0.05	0.05	0.1
		AC base course	0.0	100	140	0.06	0.08	0.14
	15K	Wearing course	50	50	80	0.05	0.06	0.08
		Binder course	80	100	100	0.06	0.1	0.1
		AC base course	100	110	120	0.08	0.1	0.12

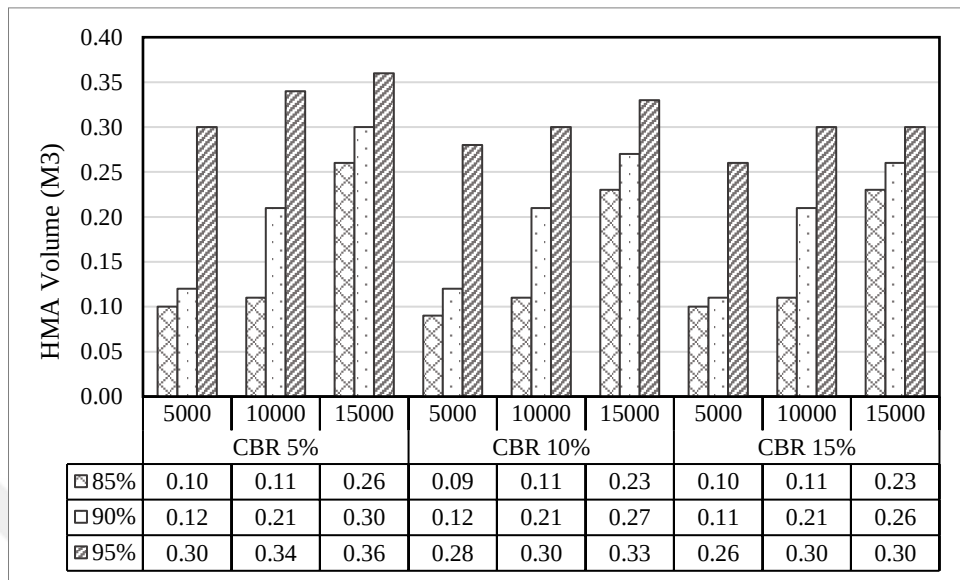


Figure 5.116 Effect of threshold values and reliability level on the HMA volume

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

The objective of this thesis was to determine the level of sensitivity of local design inputs [e.g., pavement layer thickness, effective binder content (Vbe), air voids (Va), aggregate gradation, subgrade type, hot mixed asphalt (HMA) mixture types (e.g., dense-graded and stone mastic asphalt (SMA) mixtures), binder grades as well as pavement performance threshold values and reliability levels] on flexible pavement distress using the AASHTOWare Pavement ME Design 2.5.5 by the aim of facilitation and boosting of the selection of local design levels (Level 1, Level 2, and Level 3), data collection, and local calibration of the MEPDG in Izmir, Turkey. The results of this study are the output of AASHTOWare Pavement ME Design 2.5.5 and valid for Izmir local conditions. The conclusion of this study was explained as follows:

1. Generally, an increase in wearing course thickness from 50 to 70 mm resulted in reduction by 13.6 %, 22.6 %, 30.7 %, 39.3 %, and 29% in total rutting, AC rutting, alligator cracking, top-down cracking, and thermal cracking, respectively.
2. The increase of binder course thickness from 60 to 120 mm resulted in a reduction by 15.9 %, 19.4 %, and 14.8 % in total and AC rutting, and thermal cracking respectively. A reduction of 50 % in top-down cracking and 58.8 % in alligator cracking happened when binder course thickness increased by 60 mm.
3. The increase of AC base course thickness from 80 to 140 mm resulted in a reduction in total rutting, AC rutting, thermal cracking, alligator cracking, and top-down cracking by 15, 17.3, and 12.2, 62, and 52 % respectively.
4. The increase of crushed base course thickness from 100 to 400 mm resulted in 3.2 % decrease in total rutting depth and 5.2 % increase in AC rutting depth. Also, alligator cracking is decreased by 75.4 % and top-down cracking decreased by 30.70 %. The subbase course thickness increasing (100-400 mm) resulted in 5.2, 72.5, and 7.1 % reduction in rutting in base course, subbase course and subgrade.

- Increasing the thickness of subbase course by 300 mm resulted in 72.5 and 7.1 % reduction in alligator cracking and top-down cracking, respectively. Also, increase of 300 mm increase in subbase course thickness led to 5 % increase in AC rutting.
5. The subgrade type has strong effect on pavement performance. The predicted total rutting decreased by 28.8 % when subgrade strength changed from 5 to 50 CBR. The AC rutting slightly increased for an increase in subgrade strength. The subgrade type has not major impact on the AC cracking and thermal cracking. Increasing in subgrade strength from 5 to 50 CBR led to a reduction by 75.5 % in alligator cracking and an increase of 79.3 % in top-down cracking.
 6. The increasing Vbe percentage (8-15 %) in the wearing course resulted in an average increase by 8.1 % and 15 % in total and AC rutting respectively. Increasing Vbe amount in the wearing course resulted in a significant reduction in thermal and top-down cracking by 16.5 and 39.7 % respectively. Alligator cracking is slightly increased with increase of Vbe percentage in the wearing course.
 7. The average total and AC rutting increased by 8.1 and 15.8 % with increase of Vbe percentage in binder course from 7 to 14 %. The increasing Vbe amount in binder course led to a slight increase in predicted alligator cracking in the selected flexible pavement structures. Top-down and thermal cracking is not sensitive to the variation of the Vbe in binder course.
 8. The Vbe in AC base course has negligible impact on rutting (total and AC), thermal cracking, and top-down cracking, while the alligator cracking highly decreased by 94.7 % with increasing Vbe amount (6-12 %) in the AC base course. For higher Vbe amount (>9 %), alligator cracking almost approach to zero.
 9. The increase of air voids (Va) in the wearing course from 5 – 3 % resulted in: A slight increase by 1.9 and 3.6 % in the predicted total and AC rutting respectively. A significantly increase by 85.1 % in top-down cracking. A highly reduction (62.8 %) in thermal cracking. The variation of Va in wearing course has negligible effect on alligator cracking.

10. The increase of V_a from 4- 6 % in binder course led to an increase by 2.4 and 5 % in total rutting and AC rutting. The variation of V_a in binder course has a negligible impact on alligator cracking, top-down cracking, and thermal cracking.
11. The variation of V_a in AC base course has a light effect on predicted total and AC rutting, thermal cracking, and top-down cracking. One percent decrease (6-5 %) resulted in 79.5 % reduction in alligator cracking. For lower V_a amount (<4.5 %), alligator cracking almost approach to zero.
12. The increase of passing aggregate the sieves 0.075 and 4.75 mm resulted in reduction in predicted rutting (total and AC) and cracking (alligator and top-down). Thermal cracking is not sensitive to the variation of aggregate particle distribution in HMA mixture.
13. The reduce of passing aggregate the sieves 9.5 and 19 mm led a reduction in predicted rutting and cracking (alligator and top-down). Thermal cracking is not affected by variation of the aggregate gradation.
14. The vehicle speed has strong impact on predicted pavement distresses. The speed reduction (75 to 25 km/h) resulted in 16.9 and 26.7 % increase total rutting and AC rutting respectively. While, speed reduction (25 to 1 km/h) led to 49.8 and 61.1 % increase in total rutting and AC rutting magnitudes respectively. The reduction of traffic speed (15 to 1 km/h) led to 77.6 and 221.3 % increase in alligator cracking and top-down cracking.
15. The predicted pavement distresses (total rutting, AC rutting, alligator cracking, top-down cracking, and thermal cracking) reduced when stiffer binder grade used. The PG 70-10 and PG 70-26 showed better pavement performance than PG 58-16, 58-22, and PG 64-16 for Izmir climatic condition.
16. The comparison dense graded HMA and SMA mixtures showed that predicted total and AC rutting is significantly lower in SMA pavement. The using of SMA in the wearing and binder course did not result in a reduction in predicted alligator cracking when the AC base course was dense graded HMA mixture. The predicted thermal cracking is high in the SMA pavement then dense graded HMA pavement.

17. The threshold values and reliability level have strong impact on pavement thickness. When the threshold values changed from higher to medium, and the reliability level increased from 85 to 90 %, the pavement thickness slightly increased. While the threshold values changed from medium to low and the reliability level increased from 85 to 95 %, the pavement thickness increased significantly.

The summary of the conclusion of this study showed in Table 6.1. In Table 6.1, the level of sensitivity of design inputs on predicted pavement performance: Higher than 75 % showed by "EHS" (extreme highly sensitive), 35-75 % showed by "HS" (highly sensitive), 15 – 35 % represented by "S" (sensitive), 5-15 % showed by "LS" (low sensitive), and lower than 5 % showed by "NS" (non-sensitive). Also, in Table 6.1, "NA" means not applicable.

Table 5.2 Sensitive level of design inputs on pavement performance

Distress type	Layer Types	Layer thickness	Air voids	Effective binder Content	Passing aggregate from sieves (mm)				Subgrade type	Vehicle speed	Binder grade	Mixture type
					0.075	4.75	9.5	19				
Total Rutting (mm)	Wearing course	LS	LS	LS	LS	LS	LS	LS	S	HS	S	NS
	Binder course	S	NS	LS								
	AC base course	S	NS	NS								
	Base course	LS	NA	NA								
AC Rutting (mm)	Subbase course	LS	NA	NA	LS	LS	LS	LS	NS	HS	HS	LS
	Wearing course	S	S	S								
	Binder course	S	LS	S								
	AC base course	S	NS	NS								
Alligator Cracking (%)	Base course	LS	NA	NA	HS	HS	S	HS	EHS	EHS	HS	LS
	Subbase course	LS	NA	NA								
	Wearing course	S	LS	S								
	Binder course	HS	LS	S								
Top-down Cracking (m/km)	AC base course	HS	EHS	EHS	S	S	LS	LS	EHS	EHS	HS	LS
	Base course	EHS	NA	NA								
	Subbase course	EHS	NA	NA								
	Wearing course	HS	EHS	HS								
Thermal Cracking (m/km)	Binder course	HS	NS	S	NS	NS	NS	NS	NS	NS	S	NS
	AC base course	S	NS	S								
	Base course	LS	NA	NA								
	Subbase course	LS	NA	NA								
	Wearing course	S	HS	S	NS	NS	NS	NS	NS	NS	S	NS
	Binder course	S	NS	NS								
	AC base course	LS	NS	NS								
	Base course	LS	NA	NA								
	Subbase course	LS	NA	NA	NS	NS	NS	NS	NS	NS	S	NS
	Wearing course	S	HS	S								
	Binder course	S	NS	NS								
	AC base course	LS	NS	NS								
	Base course	LS	NA	NA	NS	NS	NS	NS	NS	NS	S	NS
	Subbase course	LS	NA	NA								
	Wearing course	S	HS	S								
	Binder course	S	NS	NS								
	AC base course	LS	NS	NS	NS	NS	NS	NS	NS	NS	S	NS
	Base course	LS	NA	NA								
	Subbase course	LS	NA	NA								
	Wearing course	S	HS	S								

6.2 Recommendations

According to the results and conclusion of the study, the following recommendation for selection of the level of local design inputs and collection, local calibration as well as flexible pavement design in Izmir, Turkey are suggested.

- In regions where the rutting is a prevalent problem in the flexible pavements, it is recommended that mixture factors in wearing course and binder course should be selected as Level 1. In other words, dynamic modulus should be determined in the laboratory using site-specific materials.
- The in-place air voids and effective binder content should be collected through forensic study, for local calibration of the MEPDG in Izmir, Turkey.
- In the areas where the rutting, alligator cracking, and top-down cracking is not a big problem but thermal cracking is the predominant distress, the dynamic modulus and tensile strength of the wearing course mixture should be determined through laboratory works. Besides Level 1 mixture design inputs should be selected.
- For those flexible pavements that only alligator cracking is a prevalent distress the dynamic modulus of the AC base course layer should be determined through laboratory works.
- It is recommended that pavement distresses should be collected considering traffic speed to increase the accuracy of local calibration and avoid from under or over pavement design.
- It is strongly recommended that traffic speed should be considered in pavement design for those parts of the highways that traffic speed is slower than 25 km/hr.
- Although Turkish Highway Technical Specifications recommends higher air voids and lower binder content in the AC base course layer, but the study results recommend lower in-place air voids and higher binder content in the AC base course layer.
- As this study was conducted for level 2 and level 3, therefore, it is recommended that a comprehensive sensitivity analysis should be done for level 1 design inputs.

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APPENDICES

APPENDIX A: AASHTOWare ANALYSIS RESULTS

Table A.1 Effect of wearing course thickness on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Wearing course thickness (mm)				
			30	40	50	60	70
Total Rutting (mm)	5%	5K	11.64	11.64	12.43	11.29	10.95
		10K	12.13	12.49	12.98	11.76	11.35
		15K	12.08	12.38	12.79	11.5	11.03
	10%	5K	10.73	11.06	11.52	10.4	10.09
		10K	11.13	11.51	11.97	10.77	10.33
		15K	11.11	11.42	11.8	10.55	10.07
	15%	5K	9.96	10.28	10.76	9.65	9.33
		10K	10.32	10.71	11.17	9.99	9.53
		15K	10.31	10.63	11	9.76	9.26
AC Rutting (mm)	5%	5K	5.09	5.09	6.21	5.19	4.97
		10K	5.31	5.84	6.5	5.38	5.07
		15K	5.48	5.93	6.48	5.29	4.89
	10%	5K	5.04	5.52	6.13	5.12	4.9
		10K	5.27	5.79	6.4	5.3	4.94
		15K	5.39	5.83	6.34	5.16	4.75
	15%	5K	5.08	5.54	6.15	5.13	4.89
		10K	5.31	5.83	6.41	5.32	4.93
		15K	5.43	5.86	6.34	5.17	4.73
Bottom-up Cracking (%)	5%	5K	1.72	1.72	1.67	1.6	1.58
		10K	4.3	4.03	3.74	3.06	2.84
		15K	6.62	7.49	6.93	5.31	4.02
	10%	5K	1.63	1.6	1.58	1.54	1.52
		10K	2.98	2.78	2.6	2.2	2.07
		15K	4.57	4.93	4.49	3.45	2.74
	15%	5K	1.7	1.67	1.64	1.58	1.55
		10K	3.78	3.51	3.24	2.65	2.48
		15K	6.6	7.29	6.68	4.99	3.77
Thermal Cracking (m/km)	5%	5K	125.3	125.3	185.4	109	114.1
		10K	94.48	119.3	130.5	94.93	99.21
		15K	87.9	98.61	103.6	88.03	89.44
	10%	5K	125.8	161.6	184.8	108.5	113.6
		10K	94.64	120.1	129.7	95.27	98.71
		15K	92.24	113.8	123.2	92.76	96.13
	15%	5K	125.8	161.6	184.8	108.5	113.6
		10K	94.64	120.1	129.7	95.27	98.71
		15K	92.24	113.8	123.2	92.76	96.13
Top-down Cracking (m/km)	5%	5K	74.61	74.61	65.11	58.76	54.83
		10K	73.31	66.61	61.7	55.49	52.55
		15K	59.59	55.81	59.24	52.18	55.14
	10%	5K	140	129	119.4	98.23	85.76
		10K	150.1	134.4	121	96.09	81.89
		15K	113	98.55	90.74	69.9	64.18
	15%	5K	178.9	166.2	154.8	128.2	110.9
		10K	199.8	180.6	163.6	130.4	109
		15K	156.8	136.6	125	93.44	80.99

Table A.2 Effect of binder course thickness on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Binder course thickness (mm)						
			60	70	80	90	100	110	120
Total Rutting (mm)	5%	5K	12.69	12.12	12.43	11.88	11.33	11.42	10.87
		10K	14.57	13.89	14.29	13.64	12.98	13.11	12.46
		15K	14.72	13.94	14.28	13.53	12.79	12.82	12.09
	10%	5K	11.74	11.19	11.52	10.99	10.46	10.56	10.04
		10K	13.46	12.82	13.24	12.61	11.97	12.12	11.49
		15K	13.63	12.9	13.24	12.52	11.8	11.86	11.16
	15%	5K	10.95	10.42	10.76	10.24	9.72	9.84	9.32
		10K	12.62	11.98	12.42	11.8	11.17	11.34	10.71
		15K	12.78	12.06	12.42	11.71	11	11.07	10.38
AC Rutting (mm)	5%	5K	6.16	5.73	6.21	5.79	5.35	5.57	5.12
		10K	7.48	6.96	7.54	7.03	6.5	6.76	6.22
		15K	7.89	7.24	7.75	7.12	6.48	6.63	6
	10%	5K	6.06	5.64	6.13	5.7	5.27	5.49	5.06
		10K	7.36	6.85	7.45	6.93	6.4	6.67	6.14
		15K	7.7	7.08	7.59	6.96	6.34	6.5	5.87
	15%	5K	6.09	5.66	6.15	5.72	5.28	5.51	5.06
		10K	7.39	6.87	7.47	6.94	6.41	6.68	6.14
		15K	7.72	7.1	7.6	6.97	6.34	6.5	5.88
Bottom-up Cracking (%)	5%	5K	1.77	1.7	1.67	1.62	1.58	1.56	1.54
		10K	7.06	5.8	5.12	4.35	3.74	3.47	3.09
		15K	12.55	10.06	9.85	8.26	6.93	6.36	4.46
	10%	5K	1.74	1.68	1.64	1.59	1.56	1.54	1.52
		10K	6.46	5.16	4.5	3.8	3.24	3	2.67
		15K	13.26	10.39	9.89	8.14	6.68	6.05	4.23
	15%	5K	1.67	1.61	1.58	1.55	1.52	1.51	1.49
		10K	4.89	3.96	3.48	2.99	2.6	2.43	2.2
		15K	9.34	7.16	6.69	5.47	4.49	4.08	3.01
Thermal Cracking (m/km)	5%	5K	13.26	10.39	9.89	8.14	6.68	6.05	4.23
		10K	129.5	110.7	141	120.1	107.3	127.91	113.29
		15K	121.6	106	133.1	113.8	103.6	122.16	109.16
	10%	5K	165.5	136	184.8	154	129.7	164.47	140.18
		10K	165.5	136	184.8	154	129.7	164.47	140.18
		15K	154.8	128.4	171.8	142.3	123.2	155.59	132.87
	15%	5K	165.5	136	184.8	154	129.7	164.47	140.18
		10K	165.5	136	184.8	154	129.7	164.47	140.18
		15K	154.8	128.4	171.8	142.3	123.2	155.59	132.87
Top-down Cracking (m/km)	5%	5K	77.74	69.9	65.11	59.74	55.49	53.47	51.98
		10K	102.1	88.18	79.56	69.61	61.7	57.9	55.07
		15K	83.13	70.86	64.88	60.5	59.24	76.83	81.17
	10%	5K	150.4	131.6	119.4	103.2	88.7	80.46	71.42
		10K	216.9	189.5	170.9	145.1	121	106.84	90.74
		15K	185.2	152.9	131.4	108.4	90.74	92.93	87.84
	15%	5K	191.2	169.5	154.8	134.1	114.8	102.58	89.21
		10K	273.1	243.9	224	193.8	163.6	144.23	121.87
		15K	244.7	207.6	180.2	150.3	125	125.61	114.8

Table A.3 Effect of AC base course thickness on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	AC base course thickness (mm)						
			80	90	100	110	120	130	140
Total Rutting (mm)	5%	5K	13.28	12.93	12.58	12.43	12.07	11.86	11.48
		10K	13.98	13.61	13.24	12.98	12.6	12.27	11.86
		15K	15.12	14.73	14.31	14.04	13.61	13.25	12.79
	10%	5K	12.32	11.98	11.66	11.52	11.18	10.98	10.63
		10K	12.9	12.56	12.21	11.97	11.6	11.3	10.91
		15K	13.99	13.61	13.23	12.98	12.57	12.24	11.8
	15%	5K	12.32	11.98	11.66	11.52	11.18	10.98	10.63
		10K	12.07	11.74	11.41	11.17	10.82	10.52	10.14
		15K	13.11	12.76	12.39	12.14	11.75	11.42	11.00
AC Rutting (mm)	5%	5K	6.61	6.42	6.23	6.21	6	5.92	5.65
		10K	7.04	6.83	6.62	6.5	6.25	6.04	5.74
		15K	7.97	7.74	7.48	7.35	7.05	6.82	6.48
	10%	5K	6.53	6.35	6.16	6.15	5.93	5.84	5.58
		10K	6.96	6.77	6.55	6.41	6.16	5.96	5.66
		15K	7.8	7.58	7.33	7.18	6.9	6.67	6.34
	15%	5K	6.51	6.33	6.15	6.13	5.92	5.82	5.58
		10K	6.94	6.73	6.52	6.4	6.15	5.95	5.66
		15K	7.77	7.54	7.31	7.17	6.88	6.67	6.34
Bottom-up Cracking (%)	5%	5K	1.86	1.79	1.71	1.67	1.62	1.59	1.55
		10K	6.16	5.12	4.26	3.74	3.28	2.92	2.66
		15K	19.49	16.72	13.77	11.78	9.73	8.22	6.93
	10%	5K	1.84	1.75	1.68	1.64	1.59	1.56	1.53
		10K	5.49	4.49	3.7	3.24	2.83	2.54	2.3
		15K	20.83	17.75	14.4	12.08	9.82	8.08	6.68
	15%	5K	1.84	1.75	1.68	1.64	1.59	1.56	1.53
		10K	4.16	3.45	2.91	2.6	2.32	2.12	1.96
		15K	16.24	13.07	10.14	8.3	6.6	5.43	4.49
Thermal Cracking (m/km)	5%	5K	199.7	188	173.1	185.4	176	178.6	172.8
		10K	113	109.6	106.9	107.3	105.5	105.8	103.3
		15K	113.3	109.9	107.2	108.1	105.6	105.6	103.6
	10%	5K	199.7	187.2	177.3	184.8	175.4	178.6	171.8
		10K	139.9	135	130	129.7	127.1	127.9	123.2
		15K	139.9	135	130	129.7	127.1	127.9	123.2
	15%	5K	199.7	187.2	177.3	184.8	175.4	178.6	171.8
		10K	139.9	135	130	129.7	127.1	127.9	123.2
		15K	139.9	135	130	129.7	127.1	127.9	123.2
Top-down Cracking (m/km)	5%	5K	84.01	77.2	70.47	65.11	60.23	56.5	53.67
		10K	86.28	76.28	67.78	61.7	56.75	54.24	53.53
		15K	103.7	90.06	77.93	69.19	62.29	59.26	59.24
	10%	5K	164.8	149.8	132.9	119.4	104.3	92.09	80.99
		10K	186.3	163	139.4	121	103.1	89.04	78.11
		15K	230.9	201.8	172.7	148.7	125	105.9	90.74
	15%	5K	207.6	190.5	171.1	154.8	135.7	119.4	103.4
		10K	240.7	215	186.9	163.6	139	119.5	103.1
		15K	293.5	264.1	231.7	202.7	172.2	146.8	125

Table A.4 Effect of granular base course thickness on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Base course thickness (mm)						
			100	150	200	250	300	350	400
Total Rutting (mm)	5%	5K	12.84	12.72	12.63	12.52	12.43	12.32	12.23
		10K	13.2	13.08	12.98	12.87	12.77	12.67	12.58
		15K	13	12.9	12.79	12.68	12.59	12.48	12.37
	10%	5K	11.53	11.45	11.4	11.33	11.28	11.22	11.17
		10K	11.86	11.78	11.72	11.65	11.59	11.53	11.47
		15K	11.66	11.6	11.54	11.47	11.42	11.34	11.27
	15%	5K	10.93	10.89	10.88	10.86	10.85	10.84	10.83
		10K	11.23	11.19	11.17	11.14	11.13	11.1	11.07
		15K	11.04	11.02	11	10.97	10.96	10.92	10.89
AC Rutting (mm)	5%	5K	5.99	6.07	6.16	6.26	6.33	6.41	6.46
		10K	6.34	6.42	6.5	6.57	6.64	6.7	6.76
		15K	6.35	6.42	6.48	6.55	6.6	6.65	6.69
	10%	5K	6.04	6.09	6.16	6.22	6.28	6.34	6.4
		10K	6.32	6.35	6.41	6.47	6.52	6.57	6.6
		15K	6.25	6.29	6.34	6.39	6.43	6.47	6.51
	15%	5K	6.04	6.09	6.16	6.22	6.28	6.34	6.4
		10K	6.32	6.35	6.41	6.47	6.52	6.57	6.6
		15K	6.25	6.29	6.34	6.39	6.43	6.47	6.51
Bottom-up Cracking (%)	5%	5K	2.02	1.79	1.67	1.6	1.57	1.54	1.46
		10K	9.52	5.5	3.74	2.88	2.42	2.16	1.98
		15K	18.55	10.92	6.93	4.86	3.76	3.08	2.67
	10%	5K	1.507	1.437	1.397	1.377	1.377	1.377	1.327
		10K	7.207	3.473	2.363	1.863	1.603	1.473	1.383
		15K	12.73	7.62	5.54	3.865	2.84	2.53	2.36
	15%	5K	1.86	1.7	1.62	1.57	1.54	1.51	1.48
		10K	4.67	3.25	2.6	2.24	2.04	1.91	1.82
		15K	10.17	6.4	4.49	3.49	2.91	2.54	2.3
Thermal Cracking (m/km)	5%	5K	177.3	172.8	177.3	177.8	177.3	172.8	172.6
		10K	130.8	129.5	130.5	129.5	130	130.8	130.8
		15K	123.7	123.5	124	123.7	124.8	124.5	124.5
	10%	5K	61.51	61.38	61.64	61.51	62.03	61.9	61.9
		10K	88.54	86.19	88.28	88.28	88.41	88.54	88.67
		15K	64.9	64.77	64.51	64.64	64.9	65.03	65.03
	15%	5K	177.8	173.1	177.3	177.3	177.5	177.8	178.1
		10K	130.5	130.3	129.7	130	130.5	130.8	130.8
		15K	123.2	123.5	123.2	124	124.3	124.3	124.3
Top-down Cracking (m/km)	5%	5K	91.76	77.56	69.33	64.82	63.68	62.7	61.77
		10K	71.61	65.31	61.7	59.86	59.9	61.13	63.96
		15K	56.92	55.94	59.24	65.33	73.68	84.71	96.92
	10%	5K	127.1	108.9	95.56	86.75	81.27	75.78	70.3
		10K	116.7	98.54	85.21	76.4	70.91	65.43	59.94
		15K	108.1	92.57	81.43	74.07	70.06	68.32	67.44
	15%	5K	234.2	197.8	171.1	153.5	142.5	131.6	120.6
		10K	216.9	185.9	163.6	148.9	140.8	137.4	135.6
		15K	153.4	135.3	125	121.7	124.6	130.7	139

Table A.5 Effect of subbase course thickness on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Subbase course thickness (mm)						
			100	150	200	250	300	350	400
Total Rutting (mm)	5%	5K	12.66	12.65	12.63	12.63	12.58	12.55	12.51
		10K	13.08	13.08	13.05	13.03	12.98	12.95	12.89
		15K	12.95	12.94	12.91	12.88	12.84	12.79	12.74
	10%	5K	11.27	11.29	11.3	11.33	11.31	11.31	11.3
		10K	11.64	11.66	11.66	11.67	11.66	11.65	11.63
		15K	11.49	11.51	11.5	11.51	11.5	11.48	11.45
	15%	5K	10.79	10.84	10.88	10.93	10.95	10.98	10.99
		10K	11.1	11.14	11.17	11.21	11.24	11.26	11.27
		15K	10.93	10.98	11	11.04	11.06	11.07	11.07
AC Rutting (mm)	5%	5K	5.9	6	6.08	6.16	6.23	6.29	6.35
		10K	6.19	6.28	6.37	6.44	6.5	6.56	6.61
		15K	6.17	6.25	6.31	6.38	6.43	6.48	6.53
	10%	5K	5.524	5.604	5.669	5.729	5.779	5.829	5.874
		10K	5.804	5.874	5.939	5.999	6.044	6.094	6.129
		15K	5.764	5.824	5.874	5.929	5.969	6.004	6.044
	15%	5K	6.05	6.11	6.16	6.2	6.23	6.27	6.3
		10K	6.32	6.37	6.41	6.46	6.49	6.53	6.55
		15K	6.26	6.3	6.34	6.38	6.41	6.43	6.46
Bottom-up Cracking (%)	5%	5K	2.46	2.11	1.91	1.79	1.71	1.66	1.62
		10K	12.41	8.35	6.02	4.59	3.74	3.2	2.82
		15K	31.29	22.52	16.13	11.76	8.84	6.93	5.62
	10%	5K	1.629	1.429	1.314	1.239	1.189	1.159	1.134
		10K	7.424	5.169	3.859	3.039	2.544	2.219	1.989
		15K	18.5	13.49	9.859	7.374	5.709	4.594	3.814
	15%	5K	1.7	1.65	1.62	1.59	1.57	1.56	1.55
		10K	3.34	2.89	2.6	2.39	2.25	2.14	2.06
		15K	6.61	5.36	4.49	3.89	3.48	3.16	2.91
Thermal Cracking (m/km)	5%	5K	172.6	172.8	172.8	177.3	173.1	172.8	172.8
		10K	130	130.5	129.5	130.5	130.5	131.8	131.6
		15K	123.5	123.2	122.7	121.9	123.5	124	124
	10%	5K	172.2	172.4	174.6	174.7	172.6	174.6	174.6
		10K	129.5	129.8	129.2	129.7	130.5	131.2	131.5
		15K	122.9	122.9	122.5	122.6	123.3	123.5	123.5
	15%	5K	172.8	172.8	177.3	173.1	173.1	177.3	177.3
		10K	130	130	129.7	129.7	131.3	131.6	132.4
		15K	123.2	123.5	123.2	124.3	124	124	124
Top-down Cracking (m/km)	5%	5K	82.06	74.99	71.05	69.33	70.47	71.24	71.24
		10K	65.27	61.97	60.48	60.11	61.7	62.82	63.28
		15K	57.75	55.83	54.68	54	55.81	59.24	62.82
	10%	5K	138.5	127.3	120.6	117.2	116.8	117.2	117.5
		10K	123.7	116	111.6	109.5	110.3	111.6	112.5
		15K	94.31	90.83	89.41	89.71	92.79	96.7	101
	15%	5K	195.8	180.5	171.1	165.9	164.1	164.1	164.6
		10K	183	170.9	163.6	159.9	159.9	161.2	162.6
		15K	131.8	126.7	125	126.3	130.7	135.1	140

Table A.6 Effect of subgrade types on pavement performance

Distress Types	Traffic (AADTT)	Subgrade type (CBR %)									
		5	10	15	20	25	30	35	40	45	50
Total Rutting (mm)	5K	12.48	11.01	10.33	9.92	9.64	9.43	9.27	9.13	9.02	8.93
	10K	12.89	11.35	10.64	10.21	9.92	9.7	9.52	9.38	9.26	9.16
	15K	12.77	11.26	10.56	10.12	9.83	9.61	9.44	9.3	9.19	9.08
AC Rutting (mm)	5K	5.94	5.99	6.02	6.02	6.03	6.04	6.05	6.04	6.04	6.05
	10K	6.21	6.25	6.26	6.27	6.28	6.29	6.29	6.29	6.28	6.28
	15K	6.32	6.34	6.35	6.34	6.35	6.35	6.35	6.35	6.35	6.35
Bottom-up cracking (%)	5K	1.89	1.69	1.62	1.59	1.56	1.55	1.54	1.53	1.53	1.52
	10K	5.77	3.35	2.66	2.34	2.16	2.04	1.96	1.9	1.86	1.82
	15K	14.45	6.47	4.3	3.39	2.9	2.6	2.4	2.26	2.15	2.07
Thermal Cracking (m/km)	5K	139.14	139.14	139.14	139.14	139.14	139.14	139.14	139.14	139.14	139.14
	10K	108.38	108.38	108.38	108.38	108.38	108.38	108.38	108.38	108.38	108.38
	15K	125.56	125.56	125.56	125.56	125.56	125.56	125.56	125.56	125.56	125.56
Top-down Cracking (m/km)	5K	75.17	133.01	178.41	212.26	238.31	259.07	275.76	289.24	300.61	310.76
	10K	62.47	117.03	167.56	207.56	239.14	264.79	285.47	302.35	317.2	329.92
	15K	57.77	90.23	130.25	166.63	198.8	225.73	249.36	270.35	287.38	303.46

Table A.7 Effect of Vbe in wearing course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Vbe in wearing course (%)							
			8	9	10	11	12	13	14	15
Total Rutting (mm)	5%	5K	12.3	12.4	12.6	12.7	12.9	13.0	13.1	13.2
		10K	12.6	12.8	13.0	13.1	13.3	13.4	13.5	13.6
		15K	12.5	12.6	12.8	12.9	13.1	13.2	13.3	13.4
	10%	5K	11.0	11.2	11.3	11.5	11.6	11.7	11.8	11.9
		10K	11.3	11.5	11.6	11.8	11.9	12.0	12.1	12.2
		15K	11.2	11.3	11.5	11.6	11.8	11.9	12.0	12.1
	15%	5K	10.4	10.6	10.8	10.9	11.0	11.1	11.2	11.3
		10K	10.8	11.0	11.2	11.3	11.5	11.6	11.7	11.8
		15K	10.7	10.9	11.0	11.1	11.3	11.4	11.5	11.6
AC Rutting (mm)	5%	5K	5.9	6.1	6.2	6.4	6.5	6.6	6.7	6.8
		10K	6.2	6.3	6.5	6.6	6.8	6.9	7.0	7.1
		15K	6.2	6.3	6.5	6.6	6.7	6.9	7.0	7.1
	10%	5K	5.8	6.0	6.1	6.3	6.4	6.5	6.6	6.7
		10K	6.1	6.2	6.4	6.5	6.7	6.8	6.9	7.0
		15K	6.0	6.2	6.3	6.5	6.6	6.7	6.8	6.9
	15%	5K	5.8	6.0	6.1	6.3	6.4	6.5	6.6	6.7
		10K	6.1	6.3	6.4	6.6	6.7	6.8	6.9	7.0
		15K	6.0	6.2	6.3	6.5	6.6	6.7	6.8	6.9
Bottom-up Cracking (%)	5%	5K	1.7	1.7	1.7	1.7	1.7	1.7	1.8	1.8
		10K	3.5	3.6	3.7	3.8	3.9	4.0	4.1	4.2
		15K	6.4	6.7	6.9	7.2	7.4	7.6	7.8	8.0
	10%	5K	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7
		10K	3.1	3.2	3.3	3.4	3.4	3.5	3.6	3.7
		15K	6.3	6.6	6.8	7.1	7.3	7.5	7.7	7.9
	15%	5K	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
		10K	2.5	2.5	2.6	2.7	2.7	2.7	2.8	2.8
		15K	4.2	4.3	4.5	4.7	4.8	4.9	5.0	5.2
Thermal Cracking (m/km)	5%	5K	111.7	96.5	90.2	87.9	86.4	85.5	85.2	84.5
		10K	95.6	89.0	86.3	85.4	84.6	84.3	84.2	83.8
		15K	93.5	87.9	85.8	85.0	84.3	84.1	84.0	83.7
	10%	5K	112.5	96.9	90.3	88.0	86.5	85.5	85.2	84.5
		10K	95.5	88.9	86.3	85.3	84.6	84.2	84.1	83.8
		15K	93.1	87.7	85.7	84.9	84.3	84.0	84.0	83.7
	15%	5K	115.4	98.1	90.9	88.2	86.8	85.6	85.3	84.6
		10K	95.5	88.9	86.3	85.3	84.6	84.2	84.1	83.8
		15K	93.3	87.8	85.7	84.9	84.3	84.0	84.0	83.7
Top-down Cracking (m/km)	5%	5K	81.2	75.4	71.2	68.1	65.7	63.8	62.2	61.0
		10K	68.3	65.0	62.5	60.6	59.2	58.1	57.1	56.4
		15K	62.9	59.3	57.0	55.4	54.2	53.4	52.8	52.4
	10%	5K	167.4	147.8	133.2	121.7	113.0	105.9	100.2	95.4
		10K	206.6	182.1	163.3	148.6	136.8	127.5	119.7	113.4
		15K	111.8	100.2	91.8	85.4	80.6	76.8	73.9	71.4
	15%	5K	198.8	175.0	156.8	142.7	131.4	122.5	115.1	109.0
		10K	210.4	185.5	166.4	151.3	139.4	129.7	121.7	115.3
		15K	158.9	139.5	125.0	114.2	105.9	99.2	93.9	89.6
IRI (m/km)	5%	5K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
		10K	2.3	2.3	2.3	2.4	2.4	2.4	2.4	2.4
		15K	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	10%	5K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
		10K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
		15K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	15%	5K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
		10K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
		15K	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3

Table A.8 Effect of Vbe in binder course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Vbe in binder course (%)							
			7	8	9	10	11	12	13	14
Total Rutting (mm)	5%	5K	12.08	12.25	12.41	12.55	12.67	12.79	12.9	12.99
		10K	12.46	12.64	12.8	12.94	13.07	13.19	13.3	13.4
		15K	12.28	12.45	12.61	12.75	12.88	13	13.11	13.21
	10%	5K	10.82	10.99	11.14	11.27	11.4	11.51	11.61	11.71
		10K	11.13	11.3	11.46	11.6	11.72	11.84	11.95	12.05
		15K	10.98	11.15	11.3	11.44	11.57	11.68	11.79	11.88
	15%	5K	10.28	10.44	10.59	10.73	10.85	10.96	11.06	11.16
		10K	10.67	10.84	10.99	11.13	11.26	11.38	11.48	11.58
		15K	10.51	10.68	10.83	10.96	11.09	11.2	11.31	11.4
AC Rutting (mm)	5%	5K	5.74	5.91	6.06	6.2	6.33	6.44	6.55	6.65
		10K	5.99	6.16	6.32	6.46	6.59	6.71	6.82	6.92
		15K	5.98	6.15	6.3	6.44	6.57	6.69	6.8	6.89
	10%	5K	5.65	5.82	5.96	6.1	6.22	6.34	6.44	6.54
		10K	2.39	2.49	2.58	2.67	2.74	2.83	2.9	2.97
		15K	5.84	6	6.15	6.29	6.41	6.52	6.63	6.72
	15%	5K	5.67	5.83	5.98	6.11	6.24	6.35	6.45	6.55
		10K	5.92	6.08	6.23	6.38	6.5	6.61	6.72	6.82
		15K	5.85	6.02	6.17	6.3	6.43	6.54	6.64	6.73
Bottom-up Cracking (%)	5%	5K	1.67	1.69	1.7	1.72	1.73	1.74	1.75	1.76
		10K	3.33	3.5	3.67	3.83	3.96	4.11	4.24	4.37
		15K	5.98	6.38	6.78	7.13	7.42	7.76	8.04	8.3
	10%	5K	1.65	1.66	1.68	1.69	1.7	1.71	1.72	1.73
		10K	2.39	2.49	2.58	2.67	2.74	2.83	2.9	2.97
		15K	5.81	6.25	6.61	7	7.3	7.64	7.97	8.24
	15%	5K	1.56	1.57	1.58	1.58	1.59	1.6	1.6	1.61
		10K	2.37	2.47	2.56	2.64	2.72	2.81	2.88	2.95
		15K	3.93	4.18	4.4	4.62	4.84	5.02	5.24	5.41
Thermal Cracking (m/km)	5%	5K	111.7	111.7	111.7	111.7	111.7	111.7	111.7	111.7
		10K	95.63	95.63	95.63	95.63	95.63	95.63	95.63	95.63
		15K	93.52	93.52	93.52	93.52	93.52	93.52	93.52	93.52
	10%	5K	112.5	112.5	112.5	112.5	112.5	112.5	112.5	112.5
		10K	95.45	95.45	95.45	95.45	95.45	95.45	95.45	95.45
		15K	93.12	93.12	93.12	93.12	93.12	93.12	93.12	93.12
	15%	5K	115.4	115.4	115.4	115.4	115.4	115.4	115.4	115.4
		10K	95.5	95.5	95.5	95.5	95.5	95.5	95.5	95.5
		15K	93.33	93.33	93.33	93.33	93.33	93.33	93.33	93.33
Top-down Cracking (m/km)	5%	5K	81.53	81.17	80.99	80.64	80.46	80.28	80.1	79.92
		10K	68.61	68.28	68.03	68.11	68.88	70.28	72.56	75.54
		15K	57.9	62.92	71.8	84.54	101	120.7	142.4	165.3
	10%	5K	166.5	167.4	168	168.6	169	169.3	169.4	169.6
		10K	205.7	206.6	207.6	208.5	209.5	211.3	214.1	216.9
		15K	107.8	111.8	118.2	127.2	139	153.5	170.1	188.4
	15%	5K	197.8	198.8	199.8	200.8	200.8	201.8	201.8	201.8
		10K	209.5	210.4	211.3	212.3	214.1	215.9	218.6	221.3
		15K	153.6	158.9	166.6	177.2	190.4	206.6	224	243.1
IRI (m/km)	5%	5K	2.32	2.33	2.33	2.34	2.34	2.34	2.35	2.35
		10K	2.33	2.34	2.34	2.34	2.35	2.35	2.36	2.36
		15K	2.35	2.35	2.35	2.36	2.36	2.37	2.37	2.38
	10%	5K	2.28	2.28	2.29	2.29	2.29	2.3	2.3	2.3
		10K	2.28	2.29	2.3	2.3	2.3	2.31	2.31	2.31
		15K	2.3	2.3	2.31	2.31	2.32	2.32	2.33	2.33
	15%	5K	2.25	2.26	2.26	2.26	2.27	2.27	2.27	2.27
		10K	2.25	2.26	2.26	2.27	2.27	2.27	2.27	2.28
		15K	2.26	2.26	2.27	2.27	2.28	2.28	2.29	2.29

Table A.9 Effect of Vbe in AC base course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Vbe in binder 2 (%)						
			6	7	8	9	10	11	12
Total Rutting (mm)	5%	5K	12.05	12.07	12.08	12.1	12.11	12.12	12.13
		10K	12.43	12.45	12.46	12.48	12.49	12.49	12.5
		15K	12.25	12.26	12.28	12.29	12.3	12.3	12.31
	10%	5K	10.79	10.81	10.82	10.83	10.85	10.86	10.86
		10K	11.1	11.12	11.13	11.14	11.15	11.16	11.17
		15K	10.96	10.97	10.98	10.99	11	11.01	11.02
	15%	5K	10.26	10.27	10.28	10.29	10.3	10.31	10.31
		10K	10.65	10.66	10.67	10.68	10.69	10.7	10.7
		15K	10.49	10.5	10.51	10.52	10.53	10.53	10.54
AC Rutting (mm)	5%	5K	5.76	5.75	5.74	5.73	5.73	5.72	5.72
		10K	6.01	6	5.99	5.98	5.97	5.97	5.96
		15K	6.01	5.99	5.98	5.96	5.95	5.94	5.94
	10%	5K	5.67	5.66	5.65	5.64	5.64	5.64	5.63
		10K	5.92	5.9	5.89	5.88	5.87	5.87	5.86
		15K	5.87	5.85	5.84	5.82	5.81	5.8	5.79
	15%	5K	5.69	5.68	5.67	5.66	5.65	5.65	5.64
		10K	5.94	5.93	5.92	5.91	5.9	5.89	5.89
		15K	5.89	5.87	5.85	5.84	5.83	5.82	5.82
Bottom-up Cracking (%)	5%	5K	5.82	2.25	1.67	1.53	1.48	1.46	1.46
		10K	33.12	9.78	3.33	2.00	1.65	1.53	1.49
		15K	56.07	20.29	5.98	2.69	1.87	1.62	1.52
	10%	5K	5.15	2.15	1.65	1.52	1.48	1.46	1.46
		10K	28.76	7.95	2.93	1.89	1.61	1.52	1.48
		15K	54.74	19.61	5.81	2.67	1.87	1.62	1.52
	15%	5K	3.32	1.83	1.56	1.49	1.46	1.46	1.45
		10K	21.02	5.41	2.37	1.74	1.56	1.5	1.47
		15K	41.71	12.34	3.93	2.16	1.69	1.55	1.49
Thermal Cracking (m/km)	5%	5K	111.72	111.72	111.72	111.72	111.72	111.72	111.72
		10K	95.63	95.63	95.63	95.63	95.63	95.63	95.63
		15K	93.52	93.52	93.52	93.52	93.52	93.52	93.52
	10%	5K	112.5	112.5	112.5	112.5	112.5	112.5	112.5
		10K	95.45	95.45	95.45	95.45	95.45	95.45	95.45
		15K	93.12	93.12	93.12	93.12	93.12	93.12	93.12
	15%	5K	115.37	115.37	115.37	115.37	115.37	115.37	115.37
		10K	95.5	95.5	95.5	95.5	95.5	95.5	95.5
		15K	93.33	93.33	93.33	93.33	93.33	93.33	93.33
Top-down Cracking (m/km)	5%	5K	79.38	80.46	81.53	82.24	82.95	83.66	84.36
		10K	65.88	67.31	68.61	69.9	70.86	71.99	72.93
		15K	57.01	57.43	57.9	58.36	58.82	59.28	59.71
	10%	5K	159.03	162.98	166.51	169.64	172.38	174.85	177.19
		10K	139.76	145.91	151.39	156.47	160.96	165.11	169.06
		15K	96.42	102.1	107.77	113.29	118.81	124.89	131.5
	15%	5K	188.49	193.75	197.79	201.75	204.67	207.56	210.38
		10K	195.78	202.74	209.45	215.03	220.43	224.85	229.15
		15K	140.16	147.2	153.63	159.27	164.52	169.18	173.5
IRI (m/km)	5%	5K	2.34	2.33	2.32	2.32	2.32	2.32	2.32
		10K	2.48	2.36	2.33	2.33	2.32	2.33	2.32
		15K	2.64	2.41	2.34	2.32	2.32	2.32	2.32
	10%	5K	2.3	2.28	2.28	2.28	2.28	2.28	2.28
		10K	2.4	2.31	2.28	2.28	2.28	2.28	2.28
		15K	2.58	2.36	2.3	2.28	2.28	2.28	2.28
	15%	5K	2.26	2.25	2.25	2.25	2.25	2.25	2.25
		10K	2.34	2.27	2.25	2.25	2.25	2.25	2.25
		15K	2.47	2.3	2.26	2.25	2.25	2.24	2.24

Table A.10 Effect of Va in wearing course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Wearing course air voids (%)				
			3.00	3.50	4.00	4.50	5.00
Total Rutting (mm)	5%	5K	12.18	12.20	12.25	12.32	12.40
		10K	12.56	12.59	12.64	12.70	12.79
		15K	12.37	12.40	12.45	12.52	12.60
	10%	5K	10.92	10.95	10.99	11.06	11.14
		10K	11.23	11.25	11.30	11.37	11.45
		15K	11.07	11.10	11.15	11.22	11.29
	15%	5K	10.37	10.40	10.44	10.50	10.58
		10K	10.77	10.79	10.84	10.91	10.99
		15K	10.60	10.63	10.68	10.74	10.82
AC Rutting (mm)	5%	5K	5.83	5.86	5.91	5.97	6.05
		10K	6.08	6.11	6.16	6.22	6.30
		15K	6.07	6.10	6.15	6.21	6.29
	10%	5K	5.74	5.77	5.81	5.87	5.95
		10K	5.99	6.01	6.06	6.12	6.20
		15K	5.93	5.95	6.00	6.06	6.14
	15%	5K	5.76	5.79	5.83	5.89	5.97
		10K	6.01	6.04	6.08	6.15	6.22
		15K	5.94	5.97	6.02	6.08	6.16
Bottom-up Cracking (%)	5%	5K	1.68	1.69	1.69	1.69	1.70
		10K	3.44	3.47	3.50	3.53	3.61
		15K	6.26	6.29	6.38	6.49	6.61
	10%	5K	1.57	1.57	1.57	1.57	1.58
		10K	3.03	3.05	3.08	3.11	3.16
		15K	6.09	6.17	6.25	6.36	6.49
	15%	5K	1.57	1.57	1.57	1.57	1.57
		10K	2.44	2.45	2.47	2.49	2.52
		15K	4.09	4.13	4.18	4.25	4.32
Thermal Cracking (m/km)	5%	5K	276.51	172.57	111.72	96.44	88.37
		10K	211.48	126.34	95.63	88.55	85.41
		15K	196.07	120.08	93.52	87.48	84.99
	10%	5K	280.07	176.92	113.19	97.17	88.34
		10K	225.45	135.10	95.45	88.50	85.39
		15K	210.25	123.50	93.12	87.38	84.93
	15%	5K	284.34	181.97	115.37	98.61	89.02
		10K	208.87	126.08	95.50	88.53	85.39
		15K	195.81	119.55	93.33	87.48	84.96
Top-down Cracking (m/km)	5%	5K	97.20	116.20	135.20	158.50	175.24
		10K	58.80	62.92	68.28	75.17	83.66
		15K	56.41	59.34	62.92	67.18	72.18
	10%	5K	99.20	118.30	137.36	162.88	188.39
		10K	107.31	127.87	152.39	180.59	211.32
		15K	83.30	96.09	111.77	130.53	152.14
	15%	5K	137.36	166.16	198.80	234.20	271.73
		10K	145.27	175.97	210.40	247.82	286.73
		15K	111.77	133.43	158.91	187.55	218.65

Table A.11 Effect of Va in binder course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Air voids in binder course (%)				
			4	4.5	5	5.5	6
Total Rutting (mm)	5%	5K	12.06	12.11	12.18	12.25	12.33
		10K	12.44	12.49	12.56	12.63	12.72
		15K	12.25	12.3	12.37	12.44	12.53
	10%	5K	11.13	11.18	11.24	11.31	11.39
		10K	11.12	11.16	11.23	11.3	11.39
		15K	10.96	11.01	11.07	11.15	11.23
	15%	5K	10.26	10.31	10.37	10.44	10.52
		10K	10.65	10.7	10.77	10.84	10.93
		15K	10.49	10.54	10.6	10.67	10.76
AC Rutting (mm)	5%	5K	5.72	5.77	5.83	5.91	5.99
		10K	5.97	6.02	6.08	6.16	6.25
		15K	5.96	6.01	6.07	6.15	6.23
	10%	5K	5.717	5.767	5.827	5.902	5.987
		10K	5.88	5.93	5.99	6.06	6.15
		15K	5.82	5.86	5.93	6.00	6.08
	15%	5K	5.65	5.7	5.76	5.83	5.92
		10K	5.9	5.95	6.01	6.08	6.17
		15K	5.83	5.88	5.94	6.02	6.1
Bottom-up Cracking (%)	5%	5K	1.67	1.68	1.68	1.69	1.7
		10K	3.33	3.38	3.44	3.52	3.62
		15K	5.99	6.1	6.26	6.41	6.62
	10%	5K	1.647	1.652	1.657	1.662	1.667
		10K	2.93	2.97	3.03	3.09	3.17
		15K	5.81	5.96	6.09	6.27	6.48
	15%	5K	1.56	1.56	1.57	1.57	1.57
		10K	2.37	2.4	2.44	2.48	2.53
		15K	3.94	4.01	4.09	4.2	4.33
Thermal Cracking (m/km)	5%	5K	276.5	276.5	276.5	276.5	276.5
		10K	211.5	211.5	211.5	211.5	211.5
		15K	196.1	196.1	196.1	196.1	196.1
	10%	5K	280.5	280.5	280.5	280.5	280.5
		10K	208.6	208.6	208.6	208.6	208.6
		15K	195.3	195.3	195.3	195.3	195.3
	15%	5K	284.3	284.3	284.3	284.3	284.3
		10K	208.9	208.9	208.9	208.9	208.9
		15K	195.8	195.8	195.8	195.8	195.8
Top-down Cracking (m/km)	5%	5K	65.9	65.84	65.79	65.73	65.63
		10K	58.93	58.86	58.8	58.74	58.65
		15K	54.24	55.07	56.41	58.49	61.68
	10%	5K	101.5	101.6	101.6	101.7	101.9
		10K	107	107.2	107.3	107.6	107.9
		15K	81.53	82.24	83.3	84.89	87.32
	15%	5K	137	137.2	137.4	137.6	138
		10K	144.9	145	145.3	145.5	145.8
		15K	109.3	110.2	111.8	114	117.2

Table A.12 Effect of Va in AC Base course on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Base course thickness (mm)				
			4	4.5	5	5.5	6
Total Rutting (mm)	5%	5K	12.05	12.06	12.06	12.07	12.08
		10K	12.43	12.43	12.44	12.45	12.45
		15K	12.24	12.25	12.25	12.26	12.26
	10%	5K	11.12	11.13	11.13	11.14	11.14
		10K	11.11	11.11	11.12	11.12	11.13
		15K	10.95	10.96	10.96	10.97	10.97
	15%	5K	10.25	10.26	10.26	10.27	10.27
		10K	10.64	10.65	10.65	10.66	10.66
		15K	10.48	10.48	10.49	10.49	10.5
AC Rutting (mm)	5%	5K	5.73	5.72	5.72	5.72	5.71
		10K	5.97	5.97	5.97	5.96	5.96
		15K	5.97	5.96	5.96	5.95	5.94
	10%	5K	5.727	5.717	5.717	5.717	5.707
		10K	5.88	5.88	5.88	5.87	5.87
		15K	5.83	5.82	5.82	5.81	5.8
	15%	5K	5.66	5.65	5.65	5.65	5.64
		10K	5.91	5.9	5.9	5.89	5.89
		15K	5.84	5.84	5.83	5.83	5.82
Bottom-up Cracking (%)	5%	5K	1.48	1.53	1.67	2.05	3.08
		10K	1.62	2.02	3.33	7.46	17.69
		15K	1.79	2.73	5.99	15.82	34.43
	10%	5K	1.438	1.478	1.583	1.868	2.618
		10K	1.59	1.91	2.93	6.14	14.83
		15K	1.78	2.68	5.81	15.41	33.9
	15%	5K	1.46	1.49	1.56	1.75	2.22
		10K	1.54	1.74	2.37	4.32	9.96
		15K	1.64	2.16	3.94	9.66	23.49
Thermal Cracking (m/km)	5%	5K	276.5	276.5	276.5	276.5	276.5
		10K	211.5	211.5	211.5	211.5	211.5
		15K	196.1	196.1	196.1	196.1	196.1
	10%	5K	280.4	280.4	280.4	280.4	280.4
		10K	208.6	208.6	208.6	208.6	208.6
		15K	195.3	195.3	195.3	195.3	195.3
	15%	5K	284.3	284.3	284.3	284.3	284.3
		10K	208.9	208.9	208.9	208.9	208.9
		15K	195.8	195.8	195.8	195.8	195.8
Top-down Cracking (m/km)	5%	5K	65.57	65.71	65.9	66.12	66.36
		10K	58.47	58.67	58.93	59.22	59.57
		15K	54.11	54.18	54.24	54.33	54.44
	10%	5K	100.3	100.8	101.4	102.1	102.9
		10K	104.6	105.7	107	108.4	110.2
		15K	79.38	80.28	81.53	82.95	84.71
	15%	5K	135.1	136	137	138.2	139.5
		10K	141.9	143.2	144.9	146.7	148.9
		15K	106.7	107.9	109.3	111	112.8

Table A.13 Effect of percentage of aggregate passing the 0.075 mm sieve on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Passing 0.075mm (%)		
			Min (1)	Mid (2)	Max (3)
Total Rutting (mm)	5%	5K	12.95	12.27	11.98
		10K	13.53	12.82	12.52
		15K	13.32	12.63	12.34
	10%	5K	11.68	11.03	10.75
		10K	12.16	11.48	11.19
		15K	11.99	11.32	11.05
	15%	5K	11.26	10.61	10.34
		10K	11.69	11.02	10.74
		15K	11.5	10.85	10.58
AC Rutting (mm)	5%	5K	6.65	6.06	5.81
		10K	6.95	6.34	6.08
		15K	6.57	5.99	5.76
	10%	5K	6.83	6.23	5.98
		10K	6.83	6.23	5.98
		15K	6.74	6.17	5.93
	15%	5K	6.57	5.99	5.76
		10K	6.85	6.26	6.01
		15K	6.76	6.19	5.95
Bottom-up Cracking (%)	5%	5K	1.77	1.66	1.62
		10K	5.03	3.62	3.16
		15K	10.32	6.69	5.45
	10%	5K	1.74	1.63	1.59
		10K	4.43	3.17	2.77
		15K	10.5	6.56	5.26
	15%	5K	1.66	1.58	1.55
		10K	3.34	2.53	2.28
		15K	6.83	4.34	3.6
Thermal Cracking (m/km)	5%	5K	147.24	147.24	147.24
		10K	108.92	108.92	108.92
		15K	104.85	104.85	104.85
	10%	5K	146.19	146.19	146.19
		10K	108.53	108.53	108.53
		15K	104.01	104.01	104.01
	15%	5K	146.71	146.71	146.71
		10K	108.61	108.61	108.61
		15K	104.46	104.46	104.46
Top-down Cracking (m/km)	5%	5K	72.93	68.92	67.2
		10K	70.28	64.96	62.9
		15K	73.31	59.28	55.57
	10%	5K	151.51	132.6	125.18
		10K	162.51	135.06	125.18
		15K	175.5	145	130.2
	15%	5K	195.78	174.97	165.93
		10K	214.1	185.54	173.28
		15K	174.18	139.5	126.18

Table A.14 Effect of percentage of aggregate passing the 4.75 mm sieve on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Passing 4.75mm (%)		
			Min (1)	Mid (2)	Max (3)
Total Rutting (mm)	5%	5K	12.82	12.27	11.77
		10K	13.39	12.82	12.29
		15K	13.19	12.63	12.11
	10%	5K	11.56	11.03	10.53
		10K	12.03	11.48	10.97
		15K	11.86	11.32	10.83
	15%	5K	11.14	10.61	10.13
		10K	11.56	11.02	10.51
		15K	11.38	10.85	10.36
AC Rutting (mm)	5%	5K	6.54	6.06	5.62
		10K	6.84	6.34	5.87
		15K	6.8	6.32	5.87
	10%	5K	6.44	5.97	5.53
		10K	6.72	6.23	5.78
		15K	6.64	6.17	5.73
	15%	5K	6.47	5.99	5.56
		10K	6.74	6.26	5.81
		15K	6.66	6.19	5.75
Bottom-up Cracking (%)	5%	5K	1.74	1.66	1.59
		10K	4.72	3.62	2.88
		15K	9.42	6.69	4.78
	10%	5K	1.71	1.63	1.57
		10K	4.13	3.17	2.55
		15K	9.52	6.56	4.59
	15%	5K	1.64	1.58	1.53
		10K	3.16	2.53	2.14
		15K	6.18	4.34	3.2
Thermal Cracking (m/km)	5%	5K	147.24	147.24	147.24
		10K	108.92	108.92	108.92
		15K	104.85	104.85	104.85
	10%	5K	146.19	146.19	146.19
		10K	108.53	108.53	108.53
		15K	104.01	104.01	104.01
	15%	5K	146.71	146.71	146.71
		10K	108.61	108.61	108.61
		15K	104.46	104.46	104.46
Top-down Cracking (m/km)	5%	5K	71.42	68.92	66.34
		10K	68.07	64.96	62.03
		15K	75.17	59.28	54.11
	10%	5K	145.01	132.6	121.15
		10K	152.51	135.06	120.27
		15K	138.57	100.17	84.71
	15%	5K	188.91	174.97	160.71
		10K	204.67	185.54	166.75
		15K	167.79	139.5	118.96

Table A.15 Effect of percentage of aggregate passing the 9.5 mm sieve on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Passing 9.5mm (%)		
			Min (1)	Mid (2)	Max (3)
Total Rutting (mm)	5%	5K	11.99	12.27	12.6
		10K	12.52	12.82	13.17
		15K	12.33	12.63	12.97
	10%	5K	10.75	11.03	11.35
		10K	11.19	11.48	11.82
		15K	11.04	11.32	11.66
	15%	5K	10.34	10.61	10.94
		10K	10.73	11.02	11.35
		15K	10.57	10.85	11.18
AC Rutting (mm)	5%	5K	5.71	5.97	6.26
		10K	5.97	6.23	6.54
		15K	5.92	6.17	6.47
	10%	5K	5.75	5.99	6.29
		10K	5.99	6.26	6.56
		15K	5.93	6.19	6.48
	15%	5K	5.75	5.99	6.29
		10K	5.99	6.26	6.56
		15K	5.93	6.19	6.48
Bottom-up Cracking (%)	5%	5K	1.62	1.66	1.71
		10K	3.21	3.62	4.2
		15K	5.67	6.69	8.15
	10%	5K	1.6	1.63	1.68
		10K	2.84	3.17	3.68
		15K	5.48	6.56	8.12
	15%	5K	1.55	1.58	1.61
		10K	2.31	2.53	2.86
		15K	3.72	4.34	5.31
Thermal Cracking (m/km)	5%	5K	147.24	147.24	147.24
		10K	108.92	108.92	108.92
		15K	104.85	104.85	104.85
	10%	5K	146.19	146.19	146.19
		10K	108.53	108.53	108.53
		15K	104.01	104.01	104.01
	15%	5K	146.71	146.71	146.71
		10K	108.61	108.61	108.61
		15K	104.46	104.46	104.46
Top-down Cracking (m/km)	5%	5K	67.22	68.92	70.86
		10K	63.16	64.96	67.14
		15K	56.45	59.28	63.7
	10%	5K	126.18	132.6	140.43
		10K	127.17	135.06	145.14
		15K	92.26	100.17	112.53
	15%	5K	166.98	174.97	184.26
		10K	175.53	185.54	197.79
		15K	129.27	139.5	152.51

Table A.16 Effect of percentage of aggregate passing the 19 mm sieve on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Passing 19 mm (%)		
			Min (1)	Mid (2)	Max (3)
Total Rutting (mm)	5%	5K	11.73	12.27	12.55
		10K	12.25	12.82	13.12
		15K	12.07	12.63	12.92
	10%	5K	10.5	11.03	11.3
		10K	9.54	10	10.24
		15K	10.79	11.32	11.6
	15%	5K	10.09	10.61	10.89
		10K	9.14	9.59	9.83
		15K	10.32	10.85	11.18
AC Rutting (mm)	5%	5K	5.61	6.06	6.28
		10K	5.86	6.34	6.57
		15K	5.86	6.32	6.54
	10%	5K	5.52	5.97	6.19
		10K	4.76	5.14	5.33
		15K	5.73	6.17	6.38
	15%	5K	5.55	5.99	6.22
		10K	4.78	5.16	5.35
		15K	5.74	6.19	6.47
Bottom-up Cracking (%)	5%	5K	1.58	1.66	1.72
		10K	2.69	3.62	4.32
		15K	4.28	6.69	8.95
	10%	5K	1.56	1.63	1.7
		10K	1.51	1.55	1.59
		15K	4.07	6.56	8.98
	15%	5K	1.52	1.58	1.62
		10K	1.49	1.52	1.55
		15K	2.91	4.34	4.38
Thermal Cracking (m/km)	5%	5K	147.24	147.24	147.24
		10K	108.92	108.92	108.92
		15K	104.85	104.85	104.85
	10%	5K	146.19	146.19	146.19
		10K	108.53	108.53	108.53
		15K	104.01	104.01	104.01
	15%	5K	146.71	146.71	146.71
		10K	108.61	108.61	108.61
		15K	104.46	104.46	104.93
Top-down Cracking (m/km)	5%	5K	65.73	68.9	69.27
		10K	61.22	64.94	66.1
		15K	53.95	59.28	71.05
	10%	5K	118.22	132.6	141.09
		10K	85.93	97.08	105.11
		15K	81.35	100.17	129.41
	15%	5K	156.84	174.97	184.48
		10K	113.14	128.99	138.43
		15K	113.89	139.5	171.13

Table A.17 Effect of traffic speed on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Vehicle Speed (km/hr)											
			1	5	10	15	25	35	45	55	65	75		
Total Rutting (mm)	5%	5K	28.7	20.0	17.4	16.1	14.7	13.8	13.3	12.8	12.6	12.3		
		10K	29.7	20.8	18.1	16.8	15.3	14.4	13.9	13.4	13.1	12.9		
		15K	29.1	20.4	17.8	16.5	15.1	14.2	13.7	13.2	12.9	12.7		
		5K	27.0	18.6	16.0	15.4	13.4	12.6	12.0	11.6	11.3	11.1		
	10%	10K	26.8	19.3	16.6	15.3	13.9	13.1	12.5	12.1	11.8	11.5		
		15K	27.3	18.9	16.3	15.1	13.7	12.9	12.3	11.9	11.6	11.4		
		5K	26.2	17.7	15.2	13.9	12.6	11.8	11.2	10.8	10.5	10.3		
		10K	27.0	18.4	15.7	14.5	13.0	12.2	11.7	11.2	10.9	10.7		
	15%	15K	26.3	18.0	15.5	14.2	12.8	12.0	11.5	11.1	10.8	10.5		
		5K	22.0	13.5	11.0	9.7	8.4	7.6	7.1	6.7	6.3	6.1		
		10K	22.7	14.0	11.4	10.1	8.7	7.9	7.4	7.0	6.6	6.4		
		15K	22.1	13.8	11.2	10.0	8.7	7.9	7.3	6.9	6.6	6.4		
AC Rutting (mm)	10%	5K	21.5	13.3	10.8	10.3	8.2	7.5	6.9	6.6	6.3	6.0		
		10K	21.3	13.7	11.2	9.9	8.6	7.8	7.2	6.8	6.5	6.3		
		15K	21.5	13.4	10.9	9.7	8.4	7.7	7.1	6.8	6.5	6.2		
		5K	21.5	13.3	10.8	9.6	8.3	7.5	7.0	6.6	6.3	6.0		
	15%	10K	22.1	13.7	11.2	9.9	8.6	7.8	7.3	6.9	6.5	6.3		
		15K	21.4	13.4	10.9	9.7	8.4	7.7	7.2	6.8	6.5	6.2		
		5K	2.3	1.9	1.7	1.7	1.6	1.6	1.6	1.5	1.5	1.9		
		10K	12.1	6.3	4.5	3.7	3.0	2.6	2.4	2.3	2.2	2.1		
	5%	15K	26.3	13.6	9.1	7.0	5.1	4.1	3.6	3.3	3.0	2.9		
		5K	2.4	1.9	1.7	1.7	1.6	1.6	1.5	1.5	1.5	1.9		
		10K	11.5	6.4	4.4	3.5	2.8	2.4	2.2	2.1	2.0	2.0		
		15K	25.6	16.2	10.3	7.7	5.3	4.3	3.7	3.3	3.0	2.8		
Bottom-up Cracking (%)	15%	5K	2.3	1.8	1.7	1.6	1.6	1.5	1.5	1.5	1.5	1.8		
		10K	11.8	5.3	3.6	3.0	2.4	2.2	2.0	1.9	1.9	1.8		
		15K	28.4	12.6	7.7	5.8	4.1	3.3	2.9	2.7	2.5	2.4		

Table A.17 continues

[illegible]

Table A.18 Effect of binder grade on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Binder Grade									
			PG 58-16	PG 58-22	PG 58-28	PG 64-16	PG 64-22	PG 64-28	PG 70-10	PG 70-16	PG 70-22	
Total Rutting (mm)	5%	5K	16.49	16.64	16.85	14.2	14.4	14.7	12.26	12.21	12.52	
		10K	16.71	16.91	17.2	14.49	14.7	15.1	12.82	12.77	13.11	
		15K	16.44	16.63	16.9	14.27	14.5	14.9	12.63	12.58	12.91	
	10%	5K	14.75	14.91	15.13	12.64	12.81	13.14	11.07	11.02	11.32	
		10K	15.31	15.53	15.81	13.17	13.37	13.76	11.55	11.5	11.83	
		15K	14.56	14.75	15.04	12.5	12.69	13.04	10.93	10.88	11.03	
	15%	5K	14.28	14.44	14.66	12.15	12.33	12.66	10.59	10.54	10.83	
		10K	14.79	14.99	15.27	12.65	12.84	13.21	11.02	10.97	11.14	
		15K	14.5	14.69	14.97	12.43	12.62	12.97	10.85	10.8	11.12	
AC Rutting (mm)	5%	5K	10.15	10.28	10.44	7.94	8.1	8.4	6.06	6.01	6.28	
		10K	10.06	10.23	10.47	7.93	8.1	8.4	6.34	6.29	6.59	
		15K	9.94	10.1	10.32	7.87	8.0	8.4	6.32	6.28	6.56	
	10%	5K	9.34	9.48	9.66	7.32	7.46	7.74	5.82	5.77	6.03	
		10K	9.69	9.88	10.11	7.65	7.81	8.15	6.11	6.06	6.36	
		15K	9.55	9.7	9.94	7.59	7.74	8.04	6.1	6.05	6.21	
	15%	5K	9.55	9.69	9.87	7.49	7.65	7.94	5.99	5.94	6.21	
		10K	9.89	10.07	10.31	7.83	8	8.33	6.28	6.23	6.37	
		15K	9.69	9.85	10.09	7.7	7.87	8.17	6.2	6.16	6.44	
Bottom-up Cracking (%)	5%	5K	2.94	3.14	3.46	2.4	2.5	2.8	2.02	2.01	2.15	
		10K	19.77	21.49	24.27	13.6	15.4	18.5	8.55	8.43	10.34	
		15K	38.24	41.12	45.45	27.86	30.9	36.3	18.06	17.87	21.75	
	10%	5K	4.84	5.36	6.25	3.36	3.73	4.43	2.48	2.48	2.77	
		10K	14.86	16.66	19.43	9.22	10.64	13.42	5.41	5.34	6.62	
		15K	25.12	28.06	32.65	15.05	17.58	22.52	8.12	8.01	8.38	
	15%	5K	2.52	2.68	2.95	2.07	2.17	2.38	1.79	1.78	1.88	
		10K	13.91	15.59	18.41	8.35	9.67	12.38	4.92	4.85	5.74	
		15K	31.39	34.43	39.26	20.1	23.01	28.53	11.26	11.09	14.2	

Table A.18 continuous

Thermal Cracking (m/km)	5%	5K	448.87	383.58	233.42	409.7	370.5	191.9	325.86	342.84	242.82
		10K	370.52	271.55	149.58	295.57	226.4	197.6	262.93	281.47	186.93
		15K	349.63	254.83	141.23	278.08	212.3	195.7	246.74	248.56	176.75
	10%	5K	414.92	322.47	182.23	352.24	275.72	108.01	318.29	338.4	236.81
		10K	370.52	254.31	149.33	295.57	226.37	197.88	263.19	281.99	187.45
		15K	347.02	253	141.49	275.99	210.17	194.93	246.21	261.62	173.35
	15%	5K	414.92	323.52	183.01	354.85	278.34	108.58	320.64	341.53	240.21
		10K	370.52	270.76	149.06	295.05	225.58	97.51	262.67	280.95	261.88
		15K	349.63	254.31	140.71	306.28	212.26	95.55	247.26	263.71	175.7
Top-down Cracking (m/km)	5%	5K	118.81	125.32	135.6	81.53	83.3	87.3	73.12	72.93	75.17
		10K	159.39	172.72	193.76	82.06	85.9	94.4	68.48	68.25	71.24
		15K	169.39	182.72	213.76	136.15	151.3	183.7	62.35	61.48	67.94
	10%	5K	281.67	289.22	301.74	236.69	245.5	261.23	192.64	191.82	205.65
		10K	311.28	324.47	343.61	246.26	257.61	279.06	190.99	189.44	206.61
		15K	399.18	417.39	445.61	266.24	282.31	311.85	189.64	187.87	193.75
	15%	5K	279.06	287.38	300.59	235.87	243.15	256.86	202.74	201.75	213.19
		10K	343.17	357.16	378.16	265.5	275.78	295.92	216.87	215.93	225.73
		15K	441.23	461.77	493.61	253.15	270.37	304.04	164.99	163.1	181.9

Table A.19 Effect of mixture types on pavement performance

Distress type	Subgrade (CBR)	Traffic (AADTT)	Dense graded mixtures		SMA mixtures	
			D-1	D-2	S-1	S-2
Total Rutting (mm)	CBR 5	5K	13.57	13.01	11.54	11.29
		10K	14.18	13.58	12.77	11.84
		15K	13.87	13.29	12.21	11.7
	CBR 10	5K	12.28	11.73	10.95	10.09
		10K	12.78	12.21	11.44	10.54
		15K	12.52	11.97	11.3	10.44
	CBR 15	5K	11.82	11.27	10.56	9.7
		10K	12.27	11.7	11	10.1
		15K	12	11.45	10.85	9.98
AC Rutting (mm)	CBR 5	5K	7.41	6.89	5.27	5.04
		10K	7.75	7.19	6.19	5.31
		15K	7.6	7.07	5.79	5.36
	CBR 10	5K	7.26	6.74	5.81	4.99
		10K	7.58	7.04	6.11	5.25
		15K	7.4	6.88	6.07	5.25
	CBR 15	5K	7.24	6.72	5.88	5.05
		10K	7.54	7.01	6.17	5.3
		15K	7.37	6.85	6.12	5.3
Bottom-up Cracking (%)	CBR 5	5K	2.16	2.02	2.28	2.22
		10K	10.36	8.45	16.09	11.88
		15K	21.75	17.96	25.03	22.52
	CBR 10	5K	2.07	1.94	2.42	2.12
		10K	8.7	7.03	13.74	9.77
		15K	21.85	17.77	29.17	22.04
	CBR 15	5K	1.88	1.78	2.11	1.9
		10K	5.94	4.85	9.36	6.61
		15K	14.04	11.12	19.8	14.21
Thermal Cracking (m/km)	CBR 5	5K	115.63	115.63	391.42	370.52
		10K	95.63	95.63	299.23	299.23
		15K	93.52	93.52	278.34	279.64
	CBR 10	5K	115.11	115.11	367.91	367.91
		10K	95.45	95.45	298.44	298.44
		15K	93.12	93.12	278.86	278.86
	CBR 15	5K	115.37	115.37	367.91	367.91
		10K	95.5	95.5	298.44	298.44
		15K	93.31	93.31	279.9	279.9
Top-down Cracking (m/km)	CBR 5	5K	114.65	103.22	53.49	52.64
		10K	166.86	148.22	52.64	51.19
		15K	385.82	359.28	50.58	50.26
	CBR 10	5K	215.95	195.8	77.19	70.47
		10K	265.5	239.92	76.46	69.27
		15K	397.56	367.35	63.48	58.3
	CBR 15	5K	275.76	253.9	94.1	84.89
		10K	334.29	308.56	96.26	85.76
		15K	452.43	425.87	74.24	67.86

Table A.20 Effect of threshold values and reliability levels on pavement thickness (CBR 5)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Criterion Satisfied?
			Targeted	Predicted	Targeted	Achieved	(Pass, Fail)
Total Rutting (mm)	5	5K	20	12.18	85	100	Pass
		10K		12.73		100	Pass
		15K		12.58		100	Pass
		5K	15	12.68	90	100	Pass
		10K		13.25		98.25	Pass
		15K		13.1		98.56	Pass
		5K	10	13.25	95	52.71	Fail
		10K		14.03		40.27	Fail
		15K		13.87		42.74	Fail
AC Rutting (mm)	5	5K	10	5.76	85	100	Pass
		10K		6.02		99.99	Pass
		15K		6.01		99.99	Pass
		5K	8	6.12	90	99.48	Pass
		10K		6.4		99.01	Pass
		15K		6.39		99.03	Pass
		5K	6	6.86	95	85.78	Fail
		10K		6.95		84.54	Fail
		15K		6.94		84.69	Fail
Bottom-up Cracking (%)	5	5K	30	1.75	85	100	Pass
		10K		8.01		100	Pass
		15K		18.47		99.72	Pass
		5K	20	2.04	90	100	Pass
		10K		8.85		100	Pass
		15K		20.1		89.74	Fail
		5K	10	2.07	95	100	Pass
		10K		10.1		94.69	Fail
		15K		22.5		40.46	Fail
Thermal Cracking (m/km)	5	5K	500	88.48	85	100	Pass
		10K		79.78		100	Pass
		15K		77.61		100	Pass
		5K	350	135.75	90	100	Pass
		10K		96.28		100	Pass
		15K		94.01		100	Pass
		5K	250	130.11	95	99.96	Pass
		10K		120.75		99.98	Pass
		15K		118.33		99.98	Pass
Top-down Cracking (m/km)	5	5K	1000	79.35	85	100	Pass
		10K		71.48		100	Pass
		15K		51.64		100	Pass
		5K	1000	98.06	90	100	Pass
		10K		88.35		100	Pass
		15K		63.84		100	Pass
		5K	1000	95.03	95	100	Pass
		10K		113.35		100	Pass
		15K		81.93		100	Pass
IRI(m/km)	5	5K	3.16	2.2	85	99.89	Pass
		10K		2.24		99.84	Pass
		15K		2.29		99.75	Pass
		5K	3.16	2.33	90	99.87	Pass
		10K		2.36		99.84	Pass
		15K		2.41		99.75	Pass
		5K	2.5	2.48	95	95.47	Pass
		10K		2.53		94.3	Fail
		15K		2.59		92.86	Fail

Table A.21 Effect of threshold values and reliability levels on pavement thickness(CBR 10)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Criterion Satisfied? (Pass, Fail)
			Targeted	Predicted	Targeted	Achieved	
Total Rutting (mm)	10	5K	20	10.85	85	100	Pass
		10K		11.32		100	Pass
		15K		11.2		100	Pass
		5K	15	11.32	90	99.93	Pass
		10K		11.81		99.8	Pass
		15K		11.69		99.84	Pass
		5K	10	12.03	95	72.71	Fail
		10K		12.54		64.93	Fail
		15K		12.41		66.98	Fail
AC Rutting (mm)	10	5K	10	5.71	85	100	Pass
		10K		5.99		99.99	Pass
		15K		5.97		99.99	Pass
		5K	8	6.07	90	99.53	Pass
		10K		6.36		99.08	Pass
		15K		6.34		99.12	Pass
		5K	6	6.61	95	89.02	Fail
		10K		6.91		85.09	Fail
		15K		6.9		85.33	Fail
Bottom-up Cracking (%)	10	5K	30	1.56	85	100	Pass
		10K		5.19		100	Pass
		15K		11.16		100	Pass
		5K	20	1.84	90	100	Pass
		10K		5.74		100	Pass
		15K		12.15		99.94	Pass
		5K	10	2.26	95	100	Pass
		10K		6.56		99.92	Pass
		15K		13.62		77.32	Fail
Thermal Cracking (m/km)	10	5K	500	88.36	85	100	Pass
		10K		79.88		100	Pass
		15K		77.48		100	Pass
		5K	350	105.27	90	100	Pass
		10K		96.39		100	Pass
		15K		93.88		100	Pass
		5K	250	130.33	95	99.96	Pass
		10K		120.86		99.98	Pass
		15K		118.19		99.98	Pass
Top-down Cracking (m/km)	10	5K	1000	161.16	85	100	Pass
		10K		161.97		100	Pass
		15K		109.13		100	Pass
		5K	1000	198.8	90	100	Pass
		10K		199.8		100	Pass
		15K		134.79		100	Pass
		5K	1000	254.59	95	100	Pass
		10K		255.87		100	Pass
		15K		172.82		100	Pass
IRI(m/km)	10	5K	3.16	2.15	85	99.93	Pass
		10K		2.18		99.91	Pass
		15K		2.2		99.88	Pass
		5K	3.16	2.26	90	99.93	Pass
		10K		2.29		99.91	Pass
		15K		2.32		99.88	Pass
		5K	2.5	2.43	95	96.28	Pass
		10K		2.46		95.74	Pass
		15K		2.49		95.11	Pass

Table A.22 Effect of threshold values and reliability levels on pavement thickness(CBR 15)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Criterion Satisfied?
			Targeted	Predicted	Targeted	Achieved	(Pass, Fail)
Total Rutting (mm)	15	5K	20	10.61	85	100	Pass
		10K		10.86		100	Pass
		15K		10.7		100	Pass
		5K	15	11.16	90	99.94	Pass
		10K		11.34		99.92	Pass
		15K		11.18		99.94	Pass
		5K	10	11.78	95	76.63	Fail
		10K		12.06		72.7	Fail
		15K		11.89		75.27	Fail
AC Rutting (mm)	15	5K	10	5.91	85	99.99	Pass
		10K		6.02		99.99	Pass
		15K		5.97		99.99	Pass
		5K	8	6.32	90	99.16	Pass
		10K		6.4		99.01	Pass
		15K		6.34		99.13	Pass
		5K	6	6.82	95	86.31	Fail
		10K		6.95		84.54	Fail
		15K		6.89		85.43	Fail
Bottom-up Cracking (%)	15	5K	30	1.93	85	100	Pass
		10K		3.6		100	Pass
		15K		8.0		100	Pass
		5K	20	2.08	90	100	Pass
		10K		4.01		100	Pass
		15K		8.71		100	Pass
		5K	10	2.67	95	100	Pass
		10K		4.61		100	Pass
		15K		9.76		95.79	Pass
Thermal Cracking (m/km)	15	5K	500	110.73	85	99.83	Pass
		10K		80.05		100	Pass
		15K		77.53		100	Pass
		5K	350	128.43	90	100	Pass
		10K		96.57		100	Pass
		15K		93.93		100	Pass
		5K	250	155.31	95	99.83	Pass
		10K		121.06		99.98	Pass
		15K		118.24		99.98	Pass
Top-down Cracking (m/km)	15	5K	1000	189.34	85	100	Pass
		10K		222.38		100	Pass
		15K		163.56		100	Pass
		5K	1000	228.31	90	100	Pass
		10K		273.76		100	Pass
		15K		201.75		100	Pass
		5K	1000	298.63	95	100	Pass
		10K		349.91		100	Pass
		15K		258.36		100	Pass
IRI(m/km)	15	5K	3.16	2.15	85	99.93	Pass
		10K		2.14		99.94	Pass
		15K		2.16		99.92	Pass
		5K	3.16	2.26	90	99.93	Pass
		10K		2.26		99.94	Pass
		15K		2.28		99.92	Pass
		5K	2.5	2.43	95	96.33	Pass
		10K		2.43		96.42	Pass
		15K		2.45		96.05	Pass

Table A.23 Optimized design (CBR 5 %)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Cration satisfied?
			Targeted	Predicted	Targeted	Achieved	(Pass, Failed)
Total Rutting (mm)	5	5K	20	15.53	85	99.85	Pass
		10K		14.67		99.97	Pass
		15K		13.8		99.99	Pass
		5K	15	14.8	90	91.58	Pass
		10K		14.28		94.7	Pass
		15K		12.21		99.65	Pass
		5K	10	9.85	95	95.92	Pass
		10K		9.91		95.58	Pass
		15K		9.76		96.4	Pass
AC Rutting (mm)	5	5K	10	5.65	85	100	Pass
		10K		5.63		100	Pass
		15K		6.8		99.83	Pass
		5K	8	5.39	90	99.94	Pass
		10K		6.32		99.16	Pass
		15K		5.48		99.92	Pass
		5K	6	3.73	95	100	Pass
		10K		3.7		100	Pass
		15K		3.46		100	Pass
Bottom-up Cracking (%)	5	5K	30	19.5	85	96.36	Pass
		10K		20.6		95.9	Pass
		15K		25.65		92.79	Pass
		5K	20	10.24	90	99.65	Pass
		10K		19.64		90.61	Pass
		15K		18.25		94.18	Pass
		5K	10	2.0	95	100	Pass
		10K		2.12		100	Pass
		15K		2.22		100	Pass
Thermal Cracking (m/km)	5	5K	285	107.49	85	100	Pass
		10K		75.89		100	Pass
		15K		79.8		100	Pass
		5K	285	125.82	90	100	Pass
		10K		101.33		100	Pass
		15K		87.74		100	Pass
		5K	190	111.09	95	99.99	Pass
		10K		110.92		99.99	Pass
		15K		110.33		99.99	Pass
Top-down Cracking (m/km)	5	5K	380	190.72	85	90.58	Pass
		10K		251.89		100	Pass
		15K		85.83		100	Pass
		5K	380	229.15	90	100	Pass
		10K		235.04		100	Pass
		15K		61.58		100	Pass
		5K	380	66.56	95	100	Pass
		10K		66.3		100	Pass
		15K		67.93		100	Pass
IRI(m/km)	5	5K	3.16	2.35	85	99.58	Pass
		10K		2.31		99.67	Pass
		15K		2.35		99.58	Pass
		5K	3.16	2.4	90	99.77	Pass
		10K		2.41		99.74	Pass
		15K		2.37		99.81	Pass
		5K	2.5	2.37	95	97.26	Pass
		10K		2.38		97.22	Pass
		15K		2.37		97.27	Pass

Table A.24 Optimized design (CBR 10 %)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Criterion satisfied?
			Targeted	Predicted	Targeted	Achieved	(Pass, Failed)
Total Rutting (mm)	10	5K	20	13.61	85	100	Pass
		10K		12.93		100	Pass
		15K		13.79		99.99	Pass
		5K	15	13.58	90	97.54	Pass
		10K		13.58		97.37	Pass
		15K		12.42		99.41	Pass
		5K	10	9.88	95	95.75	Pass
		10K		9.73		96.55	Pass
		15K		9.79		96.24	Pass
AC Rutting (mm)	10	5K	10	5.14	85	100	Pass
		10K		5.45		100	Pass
		15K		7.81		98.58	Pass
		5K	8	5.61	90	99.88	Pass
		10K		6.86		97.65	Pass
		15K		6.86		97.65	Pass
		5K	6	4.74	95	99.73	Pass
		10K		4.65		99.8	Pass
		15K		4.46		99.91	Pass
Bottom-up Cracking (%)	10	5K	30	21.38	85	95.06	Pass
		10K		24.37		92.54	Pass
		15K		28.17		88.22	Pass
		5K	20	18.66	90	91.76	Pass
		10K		18.63		92.13	Pass
		15K		19.92		90.18	Pass
		5K	10	2.02	95	100	Pass
		10K		2.36		100	Pass
		15K		4.66		100	Pass
Thermal Cracking (m/km)	10	5K	285	77.53	85	100	Pass
		10K		76.41		100	Pass
		15K		88.23		100	Pass
		5K	285	110.15	90	100	Pass
		10K		106.34		100	Pass
		15K		95.08		100	Pass
		5K	190	112.06	95	99.99	Pass
		10K		112.37		99.99	Pass
		15K		110.7		99.99	Pass
Top-down Cracking (m/km)	10	5K	380	274.99	85	98.55	Pass
		10K		320.12		99.35	Pass
		15K		305.26		99.98	Pass
		5K	380	339.02	90	97.56	Pass
		10K		365.48		90.15	Pass
		15K		205.63		100	Pass
		5K	380	138.03	95	100	Pass
		10K		82.75		100	Pass
		15K		81.75		100	Pass
IRI(m/km)	10	5K	3.16	2.28	85	99.75	Pass
		10K		2.29		99.73	Pass
		15K		2.35		99.57	Pass
		5K	3.16	2.36	90	99.82	Pass
		10K		2.38		99.8	Pass
		15K		2.37		99.81	Pass
		5K	2.5	2.36	95	97.41	Pass
		10K		2.36		97.43	Pass
		15K		2.38		97.2	Pass

Table A.25 Optimized design (CBR 15 %)

Distress type	Subgrade, CBR (%)	Traffic (AADTT)	Pavement distresses @ specified reliability		Reliability (%)		Criterion satisfied?
			Targeted	Predicted	Targeted	Achieved	(Pass, Failed)
Total Rutting (mm)	10	5K	20	13.03	85	100	Pass
		10K		12.32		100	Pass
		15K		14.21		99.97	Pass
		5K	15	13.02	90	98.8	Pass
		10K		12.39		99.5	Pass
		15K		12.32		99.47	Pass
		5K	10	9.92	95	95.53	Pass
		10K		9.78		96.28	Pass
		15K		10.0		95	Pass
AC Rutting (mm)	10	5K	10	5.19	85	100	Pass
		10K		5.69		100	Pass
		15K		8.14		97.63	Pass
		5K	8	5.67	90	99.85	Pass
		10K		6.05		99.56	Pass
		15K		7.17		96.2	Pass
		5K	6	5.14	95	99.11	Pass
		10K		5.02		99.35	Pass
		15K		5.03		99.34	Pass
Bottom-up Cracking (%)	10	5K	30	20.74	85	95.52	Pass
		10K		18.62		97.29	Pass
		15K		29.9		85.12	Pass
		5K	20	17.74	90	92.93	Pass
		10K		19.61		90.6	Pass
		15K		16.11		97.09	Pass
		5K	10	1.99	95	100	Pass
		10K		2.67		100	Pass
		15K		7.1		99.87	Pass
Thermal Cracking (m/km)	10	5K	285	77.56	85	99.98	Pass
		10K		76.11		100	Pass
		15K		90.9		100	Pass
		5K	285	110.67	90	100	Pass
		10K		92.13		100	Pass
		15K		96.57		100	Pass
		5K	190	113.79	95	99.99	Pass
		10K		111.23		99.99	Pass
		15K		111.11		99.99	Pass
Top-down Cracking (m/km)	10	5K	380	302.39	85	99.98	Pass
		10K		320.5		97.5	Pass
		15K		310.2		99.21	Pass
		5K	380	376.25	90	90.5	Pass
		10K		355.21		99.65	Pass
		15K		330.4		97.5	Pass
		5K	380	232.61	95	100	Pass
		10K		152.74		100	Pass
		15K		176.45		100	Pass
IRI(m/km)	10	5K	3.16	2.25	85	99.81	Pass
		10K		2.22		99.85	Pass
		15K		2.36		99.54	Pass
		5K	3.16	2.33	90	99.86	Pass
		10K		2.32		99.87	Pass
		15K		2.33		99.86	Pass
		5K	2.5	2.35	95	97.59	Pass
		10K		2.35		97.6	Pass
		15K		2.38		97.14	Pass