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INSTITUTO SUPERIOR TÉCNICO

A Framework to Improve Pavements Design Applied to Portuguese Conditions

Bahareh Tavallaee

Supervisor: Doctor Luís Guilherme de Picado Santos

Co-supervisor: Doctor José Manuel Coelho das Neves

Thesis approved in public session to obtain the PhD degree in

Transportation Systems

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ABSTRACT

Comparing different design methods to identify the most efficient pavement design approach has always been an important challenge for the road agencies. Current pavement design method used in Portugal is mostly based on SHELL pavement design approach. This thesis aims to study and develop a framework for the consideration of a possible adoption of Mechanistic-Empirical Pavement Design Guide (ME Design), developed by AASHTO, for Portugal roads considering Portuguese conditions.

One of the main tasks in this study is to have a damage comparison between both methods. For this purpose, the performance criteria used in ME method were justified based on Portuguese conditions and experience. Three main factors – service temperature, moisture content and traffic – were also reviewed in both methods of ME and SHELL and the values were adjusted to have the similar background for both approaches.

The results of damage comparison for three selected Portuguese roads indicated that they were substantially different for the two methodologies. The AC bottom-up fatigue cracking was the main performance criterion for SHELL method while the total rutting was the one for ME method. The results also showed that SHELL method was more conservative.

The damage results were then verified by a sensitivity analysis of obtained distresses to design inputs' variations for ME Design. For this task, the results of 750 ME sensitivity analysis projects for the three selected roads with several design inputs (continuous and categorical inputs) were extracted and the tornado diagrams, normalized sensitivity index (NSI) values, and the input/output diagrams were prepared and analyzed. As result, the thickness of different layers, resilient modulus of subgrade layer and AADTT are seen as the most influential inputs in this study.

Finally, it can be said that ME Design methodology has a great potential to be used as one of the most reliable approaches to design pavements in the Portuguese technology context, despite the type of a very specific data treatment needed, once it allows a certain extent of simplification.

Keywords: Road pavement, Mechanistic Empirical Pavement Design Guide, SHELL method, performance criteria, sensitivity analysis, Portuguese conditions.

RESUMO

A comparação entre diferentes métodos para estabelecer a abordagem mais adequada ao dimensionamento dos pavimentos sempre foi um importante desafio para as administrações rodoviárias. O método de dimensionamento mais corrente em Portugal é o método da SHELL. Esta tese pretende estudar e desenvolver um quadro de aplicação do "Mechanistic-Empirical Pavement Design Guide" (método ME), desenvolvido pela AASHTO, para os pavimentos em Portugal e considerando as condições Portuguesas.

Um dos objetivos principais deste estudo é obter uma comparação do dano obtido pelos dois métodos de dimensionamento. Com este propósito, os critérios de desempenho utilizados no método ME foram justificados com base nas condições e na experiência Portuguesas. Os três principais fatores – temperatura de serviço, teor em água e tráfego – também foram analisados em ambos os métodos ME e SHELL e os valores foram ajustados para se ter uma equivalência das abordagens.

Os resultados da comparação dos danos para três estradas Portuguesas mostraram que as duas metodologias foram substancialmente diferentes. A resistência ao fendilhamento por fadiga a partir da base das camadas betuminosas foi o principal critério de desempenho no método da SHELL enquanto a deformação permanente total foi o critério mais importante no método ME. Os resultados também mostraram que o método da SHELL foi mais conservativo.

Os resultados dos danos foram então verificados através duma análise de sensibilidade das degradações obtidas das variações dos dados no método ME. Para esta tarefa, foram extraídos os resultados de 750 análises de sensibilidade ao projeto das três estradas selecionadas para vários dados de dimensionamento e, seguidamente, foram obtidos e analisados os diagramas de tornado, valores do índice de sensibilidade normalizado (NSI) e diagramas de entrada/saída. Como resultado, a espessura das diferentes camadas, o módulo de deformabilidade da fundação e o TMDA de veículos pesados foram os dados com maior influência neste estudo.

Finalmente, pode dizer-se que a metodologia ME tem um grande potencial para ser utilizada como uma das abordagens mais fiáveis para o dimensionamento de pavimentos no contexto tecnológico Português, apesar do tipo de tratamento de dados que é necessário fazer ser muito específico e permitir um certo grau de simplificação.

Palavras-chave: Pavimento rodoviário, Mechanistic Empirical Pavement Design Guide, método da SHELL, critério de desempenho, análise de sensibilidade, condições Portuguesas

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Bahareh Tavallaee

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NOTATIONS

AADT: Average Annual Daily Traffic

AADTT: Average Annual Daily Truck Traffic

AASHO: American Association of State Highway Officials

AASHTO: American Association of State Highway and Transportation Officials

ABGE: "Agregado Britado de Granulometria Extensa"

AC: Asphalt Concrete

AGPV: Average Number of Axles Groups per Vehicle

ARA: Applied Research Associates, Inc.

AVC: Automatic Vehicle Classifiers

BB: "Betão Betuminoso"

BISAR: Bitumen Stress Analysis in Roads

CBR: California Bearing Ratio

CSB: Cement Stabilized Base

CTB: Cement - Treated Base

D60: the effective grain size corresponding to 60 percent passing by weight

DDF: Directional Distribution Factor

EICM: Enhanced Integrated Climate Model

EN: "Estrada Nacional"

ER: "Estrada Regional"

ESAL: Equivalent Single Axle Load

F: Foundation

FEM: Finite Element Model

FHWA: Federal Highway Administration

Gs: specific gravity

GWT: Ground Water Table

hcd: Hourly climatic database

HDF: Hourly Distribution Factors

HMA: Hot Mix Asphalt

IC: “Itinerário Complementar”

ICM: Integrated Climate Model

IP: “Infraestruturas de Portugal”

IP: “Itinerário Principal”

IRC: Indian Road Congress

IRI: International Roughness Index

JAE: “Junta Autónoma de Estradas”

JULEA: Jacob Uzan Layered Elastic Analysis

LCPC: Laboratoire Central des Ponts et Chaussées

LDF: Lane Distribution Factor

LEF: Load equivalency factor

LTPP: Long-Term Pavement Performance

MAAT: Mean Annual Air Temperature

MACOPAV: “Manual de Conceção de Pavimentos para a Rede Rodoviária Nacional”

MAF: Monthly Adjustment Factors

MB: “Macadame Betuminoso”

MDF: Monthly Distribution Factors

M-E or ME: Mechanistic-Empirical

MEPDG: Mechanistic-Empirical Pavement Design Guide

MERRA: Modern Era Retrospective-Analysis for Research and Applications

MLET: Multi-Layer Elastic Theory

Mr: Resilient Modulus

N_{adm}: the admissible number of ESALs obtained by ME Design

NAEP80 or N_{80}^{AEP}: Accumulated Number of 80 kN Standard Axle Passages

NAVP: Accumulated Number of Heavy Vehicles

NCHRP: National Cooperative Highway Research Program

NITRR: National Institute for Transportation and Road Research

NSI: Normalized Sensitivity Index

P_{200}: Passing sieve #200 (decimal)

PCC: Portland Cement Concrete

PETE: Pavement Equivalent Temperature Model

PI: Plasticity Index (%)

PRN: Plano Rodoviário Nacional

PT: Portuguese

Pt: terminal serviceability index

QI: Quality Index

R: Reliability

RAP: Reclaimed Asphalt Pavement

RCC: Rolled - Compacted Concrete

SAQ: Sistema de Avaliação da Qualidade

SN: pavement structural number

SWCC: Soil Water Characteristic Curve

TEL: Total Equivalent Load

TRB: Transportation Research Board

TTC: Truck Traffic Classification

U.S. or US: United States

USACE: U.S. Army Corps of Engineers

VMA: Voids in Mineral Aggregate

WASHO: Western Association of State Highway Officials

WIM: Weigh in Motion

WMA: Warm Mix Asphalt

w-MAAT: weighted Mean Annual Air Temperature

w_{opt}: optimum gravimetric moisture content

wPI: PI P_{200}

w_{θ}: volumetric moisture content

α: aggression factor

$\gamma_{d\ max}$: maximum dry density

1 Introduction

1.1 Background

The road agencies aim to promote road network that is efficient, safe, secure and environmentally friendly. Roads in Portugal are defined and classified by the Plano Rodoviário Nacional (PRN, English: National Road Plan), which describes the existing and planned network of Portuguese roads. The first National Road Plan or PRN was created in 1945 – PRN45 – to take care of the deficiency of the existing road network, establishing new technical characteristics and ranking the road network. In PRN 1945, the national road network was categorized in 3 classes. The second PRN was published in 1985 – PRN85 – updated and revised due to the technological development of the vehicles and the new development methodologies, based on traffic forecasts. Introducing the second revision of PRN resulted in a National Road Network with around 10,000 km, maintaining a hierarchy in three levels. The third version of PRN – PRN2000 – was published in 1998 to respond to socio-economic development after Portugal's accession to the European Union. This plan includes about 16,500 km of road network which about 5,000 km of these roads are categorized as Regional Roads (IP, 2017).

Considering the growing rate of traffic in road network and current amount of road length in Portugal (currently about 14,313 km national road network in operation, which it takes a crucial role in transporting goods and passengers), it is important to review the pavement design approaches and adopt the more suitable one in order to optimize the quality and costs of construction, maintenance and rehabilitation of Portuguese roads. Since major roads constructed in Portugal are of flexible pavements, the focus of this study is on the flexible pavement design. However, study on the other types of pavements (rigid pavements) in Portugal is recommend for future works.

According to Infraestruturas de Portugal (IP) (Pordata, 2016), the Table 1-1 shows the extent of the national road network _mainland of Portugal for different years from 1999 until 2016.

Table 1-1:Extent of the national road network _Mainland of Portugal (Pordata, 2016)

Years	Road network (km)				
	Total	Main routes (IP)	Complementary routes (IC)	National roads (EN)	Regional roads (ER)
1999	11,991	1,368	1,037	5,059	4,528
2000	11,836	1,389	1,040	4,909	4,499
2001	12,010	1,494	1,107	4,909	4,500
2002	12,399	1,829	1,161	4,909	4,500
2003	12,589	1,949	1,229	4,910	4,500
2004	12,689	1,985	1,294	4,910	4,500
2005	12,661	1,958	1,294	4,909	4,500
2006	12,890	2,145	1,336	4,909	4,500
2007	12,902	2,198	1,387	4,911	4,406
2008	12,990	2,197	1,470	4,914	4,409
2009	13,112	2,199	1,543	4,939	4,431
2010	13,123	2,221	1,550	4,932	4,420
2011	13,411	2,330	1,717	4,945	4,420
2012	14,284	2,340	1,865	5,288	4,791
2013	14,310	2,337	1,893	5,288	4,791
2014	14,310	2,337	1,893	5,288	4,791
2015	14,310	2,337	1,893	5,288	4,791
2016	14,313	2,337	1,893	5,291	4,791

It must be pointed out that the final goal in identifying the best design approach for pavements is to develop the more sustainable and long-life product that meets the needs of the present without compromising the ability of future generations to meet their own needs (United Nations General Assembly, 1987). In other words, the sustainable pavement helps to reduce global warming potentials, energy consumption, water consumption, life cycle costs and, hazardous waste generation (Jamshidi et al., 2012). In this sense, the sustainable pavement (in this study flexible pavement) can adopt the following features:

- 100 % recyclable (RAP[1] or Reclaimed Asphalt Pavement);
- can use other recycled materials;

[1] When an asphalt concrete pavement reaches the end of its design life, the road surfacing is milled, creating a milling waste material known as Reclaimed Asphalt Pavement (RAP).

- low carbon footprint pavement;
- long life and fast resurfacing;
- porous pavement for stormwater;
- warm-mix asphalt (WMA).

For this reason, comparing different design methods to identify the most efficient design approach has always been an important challenge for the road agencies.

Current pavement design method used by Portuguese road agencies is mostly based on SHELL pavement design approach. Besides some other researches (Pereira & Pais, 2016), there is still a lack of research in Portugal to adopt new design methods, such as the Mechanistic-Empirical Pavement Design Guide (MEPDG). This thesis aims to study and develop a framework for the possible adoption (or consideration as alternative) of Mechanistic-Empirical Pavement Design Guide (MEPDG) for Portugal roads considering Portuguese conditions.

The MEPDG developed under NCHRP Project 1-37A (2004) is a set of comprehensive procedures for the analysis and design of flexible and rigid pavements (applicable for both new and rehabilitated pavements). The M-E method optimizes the structure by iterating the analysis of trial design (combination of layer types, layer thickness, and design features) for a given set of site conditions (Traffic, Climate, Material, Subgrade) until satisfying some specified performance criteria at the selected reliability levels. The AASHTOWare Pavement ME Design is a pavement design software associated with the MEPDG which involves some advances in material mechanics, axle-load spectra, and climate data for predicting pavement performance that results to produce a smoother, longer-lasting, and more cost-effective pavements (AASHTO, 2015b).

As it is shown in the Figure 1-1, a flexible pavement consists of multiple layers, namely sub-base, base, binder and surface ones. The subgrade, top of foundation, also has an important role on the overall behavior.

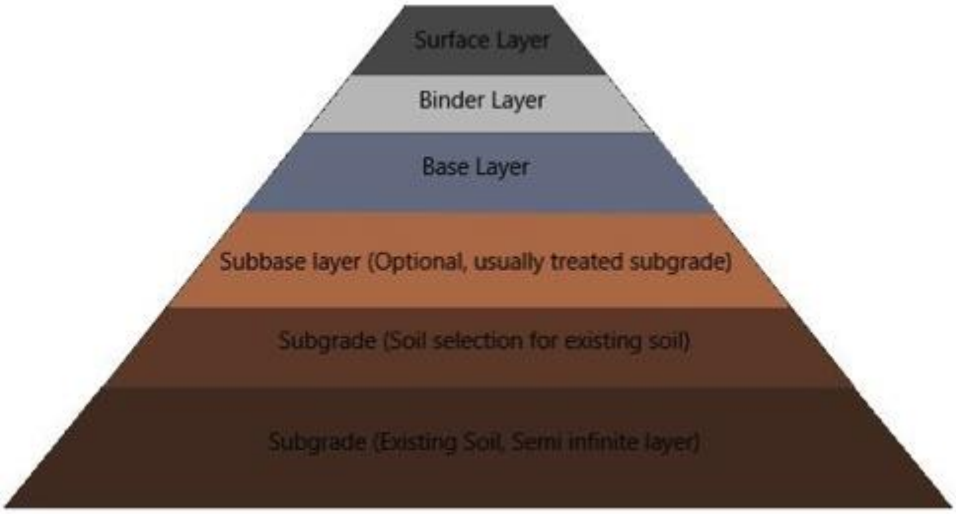


Figure 1-1: Flexible Pavement Cross-section

The structural design of pavement proposes the best estimation for thicknesses of pavement layers. A flexible pavement accommodates different kinds of materials for each layer that results in complex response when subjected to load, temperature (affects the stiffness of the asphalt layer) and moisture (affects the stiffness of the unbound granular layer and the subgrade) variations.

1.2 Statement of Research Need

Although there are several analytically based methods introduced and used as mechanistic-empirical pavement design method in the world, it should be noted that the MEPDG is the most comprehensive mechanistic-empirical method among them.

The MEPDG is considered complicated, time-consuming, and costly to apply because it requires additional information that is not typically collected by highway agencies. However, there are some important concerns for developing or adopting any new pavement design method which it seems they are well responded in MEPDG. Two main concerns for having any new pavement design method are:

- Having a flexible framework for the pavement design methodology to adjust the process based on new deterioration mechanisms or improved deterioration models.
- Considering the incremental changes in the pavement structure, material properties, traffic characteristics and climatic conditions over the design life. This approach is very useful for road administrations to monitor the evolution of pavement

conditions. The results of this monitoring can be used in the concession contracts and in the Quality Control Plans.

Additionally, there are also some other reasons particularly related to SHELL and MEPDG (ME method[1]) approaches that motivated this study in order to consider moving toward MEPDG including:

- Being concerned about overestimation (or underestimation) of distresses and consequently over (or under) designing the pavement structure in the SHELL method.
- Employing several aspects of load types variation in ME method (tire loads, axle and tire configurations, repetition of loads, distribution of traffic across the pavement, vehicle speed).
- Being applicable for both existing pavement rehabilitation and new pavement construction for ME method.
- Providing more reliable performance predictions in ME method, e.g., IRI is estimated incrementally over the entire design period by incorporating distresses such as cracking and rutting (or faulting and punchouts for rigid pavements) as major factors influencing the loss of smoothness of a pavement.
- Better characterizing materials in ME method allowing for:
    - better utilization of available materials;
    - accommodation of new materials;
    - improved definition of existing layer proportion.

These advantages motivate us to study the differences between ME method and SHELL method as an initial step toward choosing the ME method as a promising method for pavement design in Portugal or as a basis for any improvement in the Portuguese pavement design.

[1] In this study, ME method is used in the text for “MEPDG” and ME Design is used in the text for the associated software “AASHTOWare Pavement ME Design” .

1.3 Objectives

This thesis aims to study and develop a framework to facilitate the possible adoption of Mechanistic-Empirical Pavement Design Guide (MEPDG) for Portugal roads considering Portuguese conditions. The main objective of this study is to address the main challenges before adopting a new framework (MEPDG) for pavement design in Portugal. The other specific objectives of this study are:

1. Providing an overview of mechanistic-empirical pavement design method as well as current pavement design method used by Portugal (SHELL) and their main components such as traffic, climatic conditions, materials and methodology which are adopted by those methods.
2. Evaluating the available Portuguese sources for data preparation for ME method.
3. Proposing a validation framework (adjusting main factors and performance criteria) for ME design method based on Portuguese conditions. This validation framework can be considered as a benchmark for damage comparison between two methodologies (SHELL and MEPDG).
4. Identifying which method is more conservative in pavement design between the ME and SHELL methods and identifying the key distresses for each method.
5. Reviewing and documenting sensitive design inputs that influence the key distresses.

1.4 Methodology

The steps of adopted methodology in this study for evaluating the suitability of MEPDG to Portuguese conditions as well as reaching to the defined objectives are:

1. General comparisons of pavement designs (ME method and SHELL method) through the reviewing of the existing state-of-knowledge in both methods and their related components (chapter 2).
2. Data preparation of the selected roads for ME Design (Some data need validation; others need modification; some have to be converted; while some have to be correlated to the design inputs, chapter 3).

3. Evaluation and adjustment of the main factors and design criteria applied in both ME Design approach and SHELL approach for damage comparison (chapter 4).
4. Comparison of damage results for specific pavement designs between ME method and SHELL method for three selected Portuguese Pavements (chapters 4).
5. Sensitivity analysis of the MEPDG performance predictions and designs to variations of the design inputs considering Portuguese conditions. So that initially the main criteria that create the most damages in the design are analyzed (chapter 5).

1.5 Thesis Outline

This thesis is composed of six chapters and 4 appendixes for reaching the mentioned objectives under defined methodology.

Chapter 1: introduces the background of the work, the motivation of study and the statement of research need, objectives of the thesis, the adopted methodology to accomplish the thesis and finally the outline of the document.

Chapter 2: provides an overview on the evolution of flexible pavement design methodology as well as general comparisons of pavement designs (ME method and SHELL method) through the reviewing of the existing state-of-knowledge in both methods and their related components.

Chapter 3: describes the data preparation of the selected roads for ME Design and addresses the related challenges (some data need validation; others need modification; some have to be converted; while some have to be correlated to the design inputs).

Chapter 4: evaluates and adjusts the main factors and design criteria applied in both ME Design approach and SHELL approach for damage comparison in order to present the validation framework for ME method based on Portuguese conditions. This framework can be used as a benchmark for damage comparison between both methods of SHELL and ME. The chapter 4 then provides the comparison of damage results for specific pavement designs between ME method and SHELL method for three selected Portuguese Pavements

Chapter 5: contains the sensitivity analysis of the ME Design performance criteria to variations of the design inputs considering Portuguese conditions. So that initially the main criteria that create the most damages in the design are analyzed.

Chapter 6: as the final chapter presents the conclusions of this study and recommendations for future work.

Appendix 1: Pavement Performance Prediction Models.

Appendix 2: Data Preparation of the Selected Roads for ME Design, IP6, EN254.

Appendix 3: Longitudinal Profile Plans for Road Section Selections for Three Roads of IC3, IP6, EN254.

Appendix 4: Calculation of Dynamic Modulus of Flexible Layers for BISAR Based on New Service Temperature.

2 Literature Review

2.1 Overview

A pavement design method aims to determine the number, material composition and thickness of the different layers within a pavement structure through its related defined approach to distribute traffic loads efficiently whilst minimizing the whole life cost of the pavement (construction, maintenance, residual value) (Read & Whiteoak, 2003). The pavement design approaches are basically divided into the three following categories: empirical, mechanistic, and mechanistic-empirical approach. However, two design approaches of empirical and mechanistic-empirical are currently more common to use by the pavement agencies (Abaza, 2011).

This chapter reviews the evolution of flexible pavement design methodologies and then provide an overview of mechanistic-empirical pavement design method as well as current pavement design method used by Portugal (SHELL) and their main components such as traffic, climatic conditions, materials and methodology which are adopted by those methods.

2.2 Evolution of Flexible Pavement Design Methodologies

2.2.1 Introduction

Up until the 1950’s, most freight had been transported by rail or barge in the U.S., so there were no heavy traffic levels to demand a complex pavement design. Accordingly, pavement agencies designed the pavement thickness only based on their experience (Timm et al., 2014).

Today, current flexible pavement design methods can offer a reliable pavement design with the highest efficiency in compare with the suggested designs over the decades considering significant number of research studies, laboratory experimentations and collecting field data since 1950's until today. However, the methodologies and pavement design factors require constant revision in terms of updating the quality of all processes, in order to provide the better results and quality in road engineering (Pereira & Pais, 2016).

Over the decades, different pavement design methods are introduced and adopted by different countries to improve the pavement structure and the quality of ride. However, some approaches were considered as significant improvement and benchmarks in the pavement field.

As it is mentioned earlier, the pavement design approaches are basically divided into the three following categories: empirical, mechanistic, and mechanistic-empirical approach. Although a vast literature is available on flexible pavement design, in this section, the main aspects of the three design methodologies and some of their developments and improvements are mentioned.

2.2.2 Empirical Design

The empirical design approach is based on the results of experiments or experience. In other words, the correlations between inputs and outputs of an empirical process are based on observations. These correlations are examined to be reasonable (e.g. to have trends in correct directions same as actual real-world data). The empirical approach is effectively applicable when defining a scientific relationship between input and output is difficult. The empirical approaches are simple to apply and are justified based on real-world data. However, the validity of the empirical correlations is only limited to a specific circumstance that the required data are obtained in that condition (Christopher et al., 2006).

One of the most important efforts to develop the empirical pavement design procedure carried out by the U.S. Army Corps of Engineers (USACE, 1945) to design airfield pavements, for military applications during World War II. The method then adopted as a standard pavement design procedure in other parts of the world. This empirical pavement design method called California Bearing Ratio (CBR) developed by O.J. Porter for the California Highway Department in 1928 (Monismith, 2004; Pereira & Pais, 2016). Determining California Bearing Ratio (CBR) value of the soil, is used to provide the appropriate thickness of construction required above the soil for different traffic conditions using the design charts, proposed by IRC[1] (Devendra et al., 2014).

In the mid-1950's, the pavement design methods took a step forward than the CBR approach in response to need for a more robust pavement design method and consequently a stronger pavement structure. Tests on real roads with controlled traffic started in Maryland, United States in 1950 to determine the relative effects of different axle loadings. These full-scale road tests were followed by the WASHO (Western Association of State Highway Officials) road test, from 1952 to 1954, to determine the effect of different axle loadings on the cracking of the pavement, considering different thickness of the asphalt layer.

American Association of State Highway and Transportation Officials (AASHTO) carried out a series of tests called AASHO ROAD TEST (from 1958-1960 in Ottawa, Illinois) to study the effect of real traffic on the performance of pavement structures. The results of these experiments were used to develop a pavement design guide, titled as the "AASHO Interim Guide for the Design of Rigid and Flexible Pavements" issued in 1961 which is revised with major changes in 1972 and 1993. One of the major changes was the development of the pavement serviceability concept and introducing correlations between serviceability, load and thickness design of pavements which were used mainly in the AASHTO design guide (1961, 1972, 1986) and Asphalt Institute (1970) (AASHTO, 1993; Asphalt Institute, 1970; Pereira & Pais, 2016).

The first effort in Europe in developing empirical method after implementation of the AASHTO road test, was by the Road Research Laboratory of United Kingdom on implementing full-scale experimental procedures adopted in pavement design. The aim of implemented procedures was studying the long-term performance of pavement under real traffic loading in specific environment (Pereira & Pais, 2016).

[1] Indian Road Congress

2.2.3 Mechanistic Design

Unlike the empirical approach, the mechanistic design approach is based on the scientific relationships (theories of mechanics) to relate pavement structural behavior and performance to traffic loading and environmental influences. The initial credit of mechanistic design approach for flexible pavements returns to Burmister’s development during the 1940s of multilayer elastic theory to compute stresses, strains, and deflections in pavement structures.

The advantage of the mechanistic design approach is the accuracy in prediction of the response of the pavement thanks to the elasticity-based solutions by Boussinesq, Burmister, and Westergaard. However, these solutions are not able to predict the nonlinear and inelastic cracking, permanent deformation in pavement structure which is a considerable disadvantage for this approach.

As a result, the mechanistic approach requires some empirical information and relationships to relate theory to the real world of pavement performance (Christopher et al., 2006).

2.2.4 Mechanistic-Empirical Design Approach

The third approach is comprised of two parts: mechanistic part and empirical part. The mechanistic part of method is able to calculate the pavement structure response (stresses and strains) to one or more distresses (cracking and permanent deformation), as a function of the material's properties, layer thickness and loading conditions. These responses are then related to the observed performance of the pavement, corresponding to the empirical part of the method (Pereira & Pais, 2016).

Calculation of pavement structure response due to traffic loading and environmental conditions is based on the theory of mechanics (Burmister theory) implemented in several computer programs. The adequacy of design was examined through the criteria of permanent deformation and fatigue cracking introduced by several researchers.

Kerkhoven and Dormon (1953) were the first researchers suggested the use of vertical compressive strain on the top of the subgrade, as the failure criterion to reduce permanent deformation on the pavement surface. Saal and Pell (1960) proposed the use of the horizontal tensile strain at the bottom of asphalt layer to control the fatigue cracking.

After introducing mechanistic-empirical design approach by Dormon and Metcalf (1965), several mechanistic-empirical design methods were developed by researchers in different countries. Some examples of these methods were (Huang, 2004; Nikolaides, 2015; Pereira & Pais, 2016):

1. SHELL pavement design method by Claussen et al. (1977).
2. Nottingham flexible pavement design methodology by Brown (1980).
3. French method by LCPC[1] (1981).
4. Asphalt institute pavement design methodology (1981).
5. English method by Powell et al. (1984).
6. Australian flexible pavement design methodology by Austroads (2012).

On the other hand, in the United States, due to recognizing the limitations of AASHTO design guide 1993, the National Cooperative Highway Research Program (NCHRP) initiated two research projects; 1-37A and 1-40 which then resulted in documentation of the Mechanistic-Empirical Pavement Design Guide (MEPDG) (Momin, 2011).

The MEPDG developed under NCHRP Project 1-37A (2004) provides a comprehensive and computerized set of procedures for the analysis and design of new and rehabilitated flexible and rigid pavements. The design method is an iterating process to analysis the trial design (combination of layer types, layer thickness, and design features) for a given set of site conditions (traffic, climate, materials, subgrade) until satisfying some specified performance criteria at the selected reliability levels (AASHTO, 2015b).

Improving MEPDG, resulted in complete documentation and publishing the new Mechanistic Empirical Pavement Design Guide and releasing the associated software by AAHSTO in 2011 (Timm et al., 2014).

[1] Laboratoire Central des Ponts et Chaussées

Among these pavement design methods, the two approaches of mechanistic-empirical pavement design method and SHELL design method (current pavement design method used by Portugal) as well as their main components such as traffic, climatic conditions, materials are discussed in detail in this chapter.

Table 2-1 shows examples of the analytically based design procedures (Monismith, 2004).

Table 2-1: Examples of analytically based design procedures (Monismith, 2004)

Organization	Pavement Representation	Distress Modes	Design Format
Shell International Petroleum Co., Ltd., London, England	Multilayer elastic solid	Fatigue in bounded layers; Rutting: subgrade strain; Tensile stress at the bottom of soil cement	Design charts; computer program BISAR
National Cooperative Highway Research Program (NCHRP) Project 1-10B Procedure (AASHTO)	Multilayer elastic solid	Fatigue in bounded layers; Rutting	Design charts; computer program (MTC093)
The Asphalt Institute, Lexington, KY (MS-1, MS-11, MS-23)	Multilayer elastic solid	Fatigue in bounded layers; Rutting: ·subgrade strain	Design charts; computer program DAMA
Laboratoire Central des Ponts et Chaussées (LCPC)	Multilayer elastic solid	Fatigue in bounded layers; rutting	Catalogue of designs; computer program (ELIZE)
Centre de Recherches Routières, Belgium	Multilayer elastic solid	Fatigue in bounded layers; rutting	Design charts; computer program (MTC093)
National Institute for Transportation and Road Research (NITRR) South Africa	Multilayer elastic solid	Fatigue in bounded layers; rutting: ·subgrade strain ·shear in granular layers	Catalogue of designs; computer program
National Cooperative Highway Research Program (NCHRP) Project 1-26 Procedure (AASHTO)	Finite element idealization; multilayer elastic solid	Fatigue in bounded layers; rutting: ·subgrade strain	ILLI-PAVE; elastic layer programs (e.g., ELSYM)
Federal Highway Administration U.S. DOT, Washington, D.C.	Multilayer elastic or viscoelastic solid	Fatigue in bounded layers; Rutting: ·estimate at surface Serviceability (as measured by PSI)	Computer program: VESYS
University of Nottingham, Great Britain	Multilayer elastic solid	Fatigue in bounded layers; rutting: ·subgrade strain	Design charts; computer program (ANPAD)
Austroads	Multilayer elastic solid	Fatigue in bounded layers; rutting: ·subgrade strain	Design charts, computer program CIRCLY
National Cooperative Highway Research Program (NCHRP) Project 1-37A (Proposed AASHTO Guide)	Multilayer elastic	Fatigue in bounded layers; rutting: ·subgrade strain ·asphalt concrete, time hardening low temperature cracking	Computer program JULEA

2.3 MEPDG Overview

2.3.1 General Considerations

The basic principles of the M-E pavement design methodology initially conceived by Boussinesq elastic theory. However, by updating the basics, the credit could go to Burmister's solutions for two- and three-layer elastic systems responses to loads.

The heavy aircraft load impacts on airfield pavement design in II World War formed a strong motivation to have a more fundamental pavement design. The development of pavement design technology then achieved by many researches which in summary includes: the work carried out by Dr. Norman McLeod in Canada, based on ultimate strength theory for a plate load; using a modified CBR procedure by the U.S. Army Corps of Engineers; Boussinesq theory; road tests in the 1950's (e.g., the WASHO Road Test in Idaho, and the AASHO Road Test in Illinois); a comprehensive study in Canada on pavement performance in the late 1950's and early 1960's.

The improvement of pavement design knowledge has been continued as the evolution of M-E pavement design methods carried out by AASHTO from the 1970's until now. This development led to publish a comprehensive framework as Mechanistic Empirical Pavement Design Guide (MEPDG) as well as to release the associated computer program named as AASHTOWare pavement ME Design™ (formerly DARWin ME™) (Haas et al., 2007).

The MEPDG represents a significant change in the pavement design method which makes important differences between empirical design procedures (e.g. AASHTO 1993) and M-E design procedure. These differences, in terms of flexible pavement characterization, are described as follow (AASHTO, 2015b):

1. Empirical design procedure is using layer characterization (Structural Layer Coefficient) as material properties while ME design procedure is using mixture characterization (Dynamic modulus, creep compliance, tensile strength, Poisson's ratio, air voids, density, VMA, effective asphalt content, gradation, coefficient of thermal expansion, asphalt properties).

2. There is no direct tie between resilient modulus or structural layer coefficient and mix design criteria in empirical design procedure while ME design procedure provides a direct tie between materials, structural design, construction, climate, traffic, and pavement management systems.
3. There is no distress prediction in empirical design procedure while ME design procedure predicts multiple performance indicators which is used for the confirmation of design expectations.

The mechanistic-empirical pavement design method basically has the following improvements over an empirical method (AASHTO, 1993, 2015b):

- Improving the applicability to both existing pavement rehabilitation and new pavement construction.
- Applying a wide range of traffic types and loading conditions (vehicle class and load distributions).
- Improving material characterizations (e.g. dynamic modulus is incrementally generated through the pavement depth over the design life).
- Using a wide range of material properties (e.g. using dynamic modulus for asphalt mixtures, and resilient modulus for unbound materials to adequately describe the stress-dependent and environment-dependent behavior of materials).
- Improving the pavement evaluation and analysis procedures (some examples of these improvements are: improving the reliability of performance predictions by local calibration; consideration of climatic effects on pavement materials, responses, and distresses with an incremental approach over the design life).

The MEPDG optimizes the structure by iterating the analysis of trial design (combination of layer types, layer thickness, and design features) for a given set of site conditions (Traffic, Climate, Material, Subgrade) until satisfying some specified performance criteria at the selected reliability levels (AASHTO, 2015b).

The MEPDG uses a wide range of features to analyze and design a pavement in an optimum way. A summary of these features includes (Transportation Research Board, 2014):

1. Traffic: truck traffic characterization based on the distribution of axle loads for a specific axle type (i.e., axle-load spectra), hourly and monthly distribution factors, and distribution of truck classifications (i.e., the number of truck applications by FHWA vehicle class). Another traffic feature which can be analyzed by MEPDG is special axle configurations.
2. Materials: materials property characterization includes asphalt, concrete, cementitious and unbound granular materials, and subgrade soils.
3. Climate: modeling the effects of temperature, wind speed, sunshine, precipitation, and relative humidity in each pavement layer.
4. Performance prediction: using behavior models to predict pavement distresses and smoothness (IRI).
5. Input hierarchical levels are the other feature of MEPDG that suggest the agencies to use different level of data sources if they do not have detailed input data in the same level of accuracy. These input hierarchical levels are defined as follows (AASHTO, 2015b; Schwartz & Carvalho, 2007):

   - Level 1 inputs: provide the highest level of accuracy and the lowest level of uncertainty. The Level 1 is used for heavily trafficked pavements, situations with serious safety and economic consequences of early failure. It can be obtained from field or laboratory evaluation. For example, laboratory measured material properties (e.g., dynamic modulus master curve for asphalt concrete, nonlinear resilient modulus for unbound materials) and project-specific traffic data (e.g., vehicle class and load distributions) are included in this evaluation.
   - Level 2 inputs: have an intermediate level of accuracy and are closest to AASHTO Design procedure. Level 2 inputs are used when resources or testing equipment are not available for Level 1 characterization. This level of inputs can be obtained from a limited testing program or via empirical correlations or experience (possibly from an agency database) (e.g., resilient modulus estimated from CBR values).
   - Level 3 inputs: with the lowest level of accuracy are used for designs with minimal consequences of early failure (e.g., low volume roads). In this level the inputs are best estimated, or default values based on global default values or

local agency experience. (e.g., soil classification to determine the range of resilient modulus, highway class to determine vehicle class distribution).

The MEPDG recommends the pavement designer use the highest available level of inputs which should not necessary be the same hierarchical level for all inputs. The level of input selection is mainly related to roadway importance, and data collection effort costs and time (Transportation Research Board, 2014).

It should be noted, even considering the developments offered by mechanistic-empirical design method, every model related to each component of the design (traffic, climatic conditions, subgrade and pavement materials behavior) needs to be calibrated, as well as the pavement performance models. The calibration process demands more effort, time and cost which should be considered as the associated challenges with MEPDG (Pereira & Pais, 2016).

2.3.2 M-E Pavement Design Principles

Basically, the mechanistic part of M-E design uses material properties, layer thicknesses and loading conditions as the inputs to calculate the fundamental pavement responses. The empirical part of M-E design then relates these responses to observed performance (e.g., smoothness deterioration, fatigue cracking progression, rutting progression). Figure 2-1 and Figure 2-2 illustrate both parts of M-E design. Figure 2-1 shows an example of trial design for the two-layer pavement with the selected layer thicknesses (D1, D2).

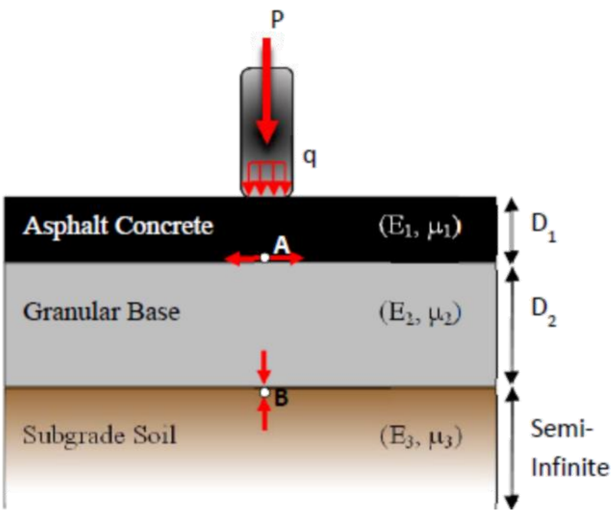


Figure 2-1: Mechanistic modeling (Timm et al., 2014)

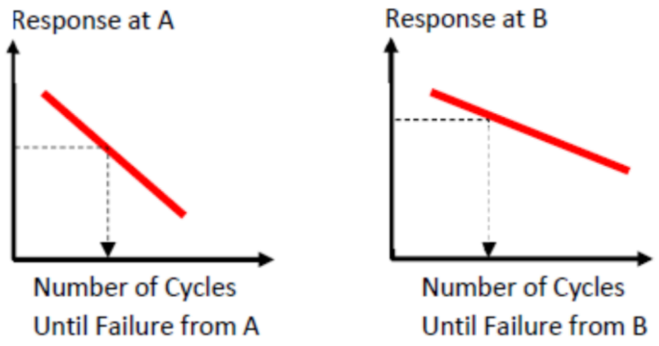


Figure 2-2: Empirical performance prediction (Timm et al., 2014)

The layer properties are characterized by their modulus (E1, E2, E3) and Poisson ratios (μ1, μ2, μ3). As for the loading conditions, a single tire load with weight (P) and contact pressure (q) has been applied to the pavement surface. Figure 2-1 also shows the critical locations in the structure; tensile strain at the bottom of the asphalt layer (A) and compressive strain on top of subgrade (B). These critical locations are respectively linked to bottom up fatigue cracking (A) and total pavement rutting (B). It should be noted that other critical locations may also be used for M-E design.

For each trial design (i.e., layer thicknesses, layer properties and loading), a mechanistic simulation is conducted to determine the pavement response at critical locations in the structure. The pavement responses are then used with empirical models to predict the number of allowable load repetitions until failure for each type of distress. Figure 2-2 shows two different empirical models for A and B, respectively (Haas et al., 2007; Timm et al., 2014).

2.3.3 M-E Design Process

In summary the following input data are required to define a trial design for selected road projects in a M-E Design Analysis:

- material properties;
- specific site subgrade support (foundation);
- loading conditions (traffic);
- environmental conditions (climate).

The M-E design method optimizes the structure by iterating the analysis of trial design (combination of layer types, layer thickness, and design features) for a given set of site conditions (Traffic, Climate, Material, Subgrade) until satisfying some specified performance criteria at the selected reliability levels. Schematically, this process is explained in the Figure 2-3.

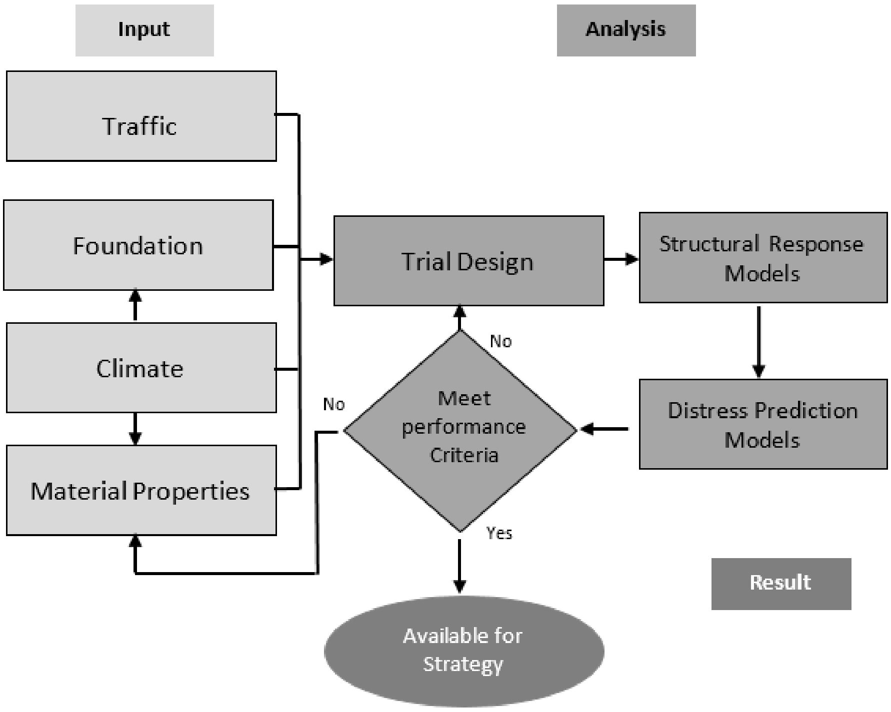


Figure 2-3: M-E flexible pavement design flow chart (Schwartz & Carvalho, 2007)

As it is illustrated in the figure, the steps of the process are defined as follows (AASHTO, 2015b):

- Proposing a trial design for specific site subgrade support, material properties, traffic loading, and environmental conditions.
- Adjusting design criteria for acceptable pavement performance at the end of the design period (i.e., acceptable levels of rutting, fatigue cracking, thermal cracking, and roughness).
- Selecting reliability level for each one of the distresses considered in the design.
- Calculating monthly traffic loading and seasonal climate conditions (temperature gradients in asphalt concrete layers, moisture content in unbound granular layers and subgrade).

- Modifying material properties in response to environmental conditions.
- Calculating structural responses (stresses, strains and deflections) for each axle type and load and for each time step throughout the design period.
- Calculating predicted distresses (e.g., rutting, fatigue cracking) at the end of each time step throughout the design period using the calibrated empirical performance models.
- Evaluating the predicted performance of the trial design against the defined design criteria at the specified reliability level. If the trial design does not meet the performance criteria, the design (thicknesses or material selection) must be modified and the calculations repeated until the design is acceptable.

2.3.4 AASHTOWare PAVEMENT ME DESIGN™

In 2008, the American Association of State Highway and Transportation Officials (AASHTO) published an interim edition of the Mechanistic-Empirical Pavement Design Guide (MEPDG): A Manual of Practice. The first mechanistic-empirical (ME) pavement design procedure document based on nationally calibrated pavement performance prediction models (AASHTO 2008). A second edition of the Manual containing updated information, additional guidance, and improved nationally calibrated models was published in 2015 (AASHTO 2015).

The time-consuming procedure of analysis (the iterating procedure) described in the MEPDG demands a computer program. As previously noted, one of the products of the NCHRP 1-37A project was the associated software. In 2011, AASHTOWare released the first version of DARWin-ME™, which was improved and rebranded to AASHTOWare Pavement ME Design™ in 2013. Several updates have been made to the software since its initial release, with the latest version (v2.5.3) released on October 16th, 2018 (ARA, 2018). Together, the MEPDG and the AASHTOWare software provide an improved process for conducting pavement analysis and for developing designs based on ME principles.

The new software is dynamically capable of evaluating the pavement design in shorter runtimes, better graphical user interface, while is able to store input values into a database. Based on MEPDG manual, the AASHTOWare Pavement ME Design™ software is comprised of a series of modules that lead the designer through the analysis procedure (Transportation Research Board, 2014).

The software can result in smoother, longer-lasting, and more cost-effective pavements by taking advantage of advances in material mechanics, axle-load spectra, and climate data for predicting pavement performance. One of the many advanced tools in ME Design software is the ability to predict pavement distresses for designed structures. The software uses the predicted performance and design performance targets to determine whether the pavement structure under evaluation meets the design reliability. The designer can then adjust the pavement structure in order to satisfy the design criteria (He et al., 2011).

The default performance prediction models provided in ME Design were calibrated primarily based on the Long-Term Pavement Performance (LTPP) database as well as some other state and Federal agency research projects in the United States (US). The ME Design software uses multiple performance criteria in its analysis including top down and bottom up (fatigue) cracking, transverse cracking, permanent deformation (rutting), and IRI (International Roughness Index) (He et al., 2011).

2.3.5 Design Input

This section describes the input variables required for ME Design.

a) Selecting Design-Performance Criteria and Reliability Level

Applying rational design-performance criteria guarantees that a pavement design will perform adequately over its design life. These threshold values are mostly based on agency policies regarding a specific pavement condition which demands some major rehabilitation activities (which can be determined either from agency's pavement management data or considerations for safety reasons). Otherwise these values should be based on the average values for a project. The distresses considered for flexible pavements are permanent deformation (rutting), “alligator” (bottom-up) fatigue cracking, “longitudinal” (top-down) cracking, thermal cracking, and roughness. The only functional distress predicted is roughness. Friction is not considered in the MEPDG methodology (AASHTO, 2015b). The prediction models for these design performance criteria are provided in Appendix 1.

The other analysis parameter of ME Design is reliability level for each performance criteria. The level of design reliability could be adjusted on the general consequence of reaching the terminal condition earlier than the design life. Design reliability (R) is defined as the probability (P) that the predicted distress will be less than the critical level over the design period (AASHTO, 2015b).

$$R = P[Distress\ over\ Design\ Period\ < Critical\ Distress\ Level] \qquad Equation\ 2\text{-}1$$

For example, if R=90 for one of the performance criteria, means that if 10 projects were designed using the ME Design software with R=90, one of those projects, on average, would crosses the threshold value of the related performance criterion at the end of the design period. The same reliability for all performance indicators is recommended by MEPDG manual.

Table 2-2 shows the suggested minimum levels of reliability for different functional classifications of the roadway (AASHTO, 2015b).

Table 2-2: Levels of reliability for different functional classifications of the roadway (AASHTO, 2015b)

Functional Classification	**Level of Reliability**	
	Urban	**Rural**
Interstate/Freeways	95	95
Principal Arterials	90	85
Collectors	80	75
Local	75	70

b) Traffic Data

Traffic data in the MEPDG has the main role for evaluating the effects of traffic loads that are applied to a pavement over the design life. The required traffic data for MEPDG are the same regardless of pavement type (flexible or rigid) or design type (new or rehabilitated). It should be noted that the equivalent single axle load (ESAL) approach used for traffic characterization in SHELL method is not needed in this method (ARA, 2004c).

The Table 2-3 shows the required traffic inputs in MEPDG:

Table 2-3: The summary of required traffic inputs in MEPDG

<table>
<tr><td colspan="4">ME Field</td></tr>
<tr><td rowspan="6">Site Specific Traffic Inputs</td><td rowspan="2">Average Annual Daily Truck Traffic (AADTT)</td><td colspan="2">Year</td></tr>
<tr><td colspan="2">Initial two-way AADTT</td></tr>
<tr><td colspan="3">Number of Lanes in Design Direction</td></tr>
<tr><td colspan="3">Percent of Trucks in Design Direction</td></tr>
<tr><td colspan="3">Percent of Trucks in Design Lane</td></tr>
<tr><td colspan="3">Operational Speed</td></tr>
<tr><td rowspan="12">WIM Traffic Data</td><td rowspan="5">Traffic Volume Adjustment Factors</td><td colspan="2">Monthly Adjustment[1]</td></tr>
<tr><td colspan="2">Road Category[2]</td></tr>
<tr><td colspan="2">AADTT Distribution by Vehicle Class[3](%)</td></tr>
<tr><td colspan="2">Hourly Truck Traffic Distribution</td></tr>
<tr><td colspan="2">Traffic Growth factors[4]</td></tr>
<tr><td rowspan="7">Axle Load Distribution Factors[5]</td><td colspan="2">Axle Load Distribution</td></tr>
<tr><td rowspan="4">Axle Type</td><td>Single Axle</td></tr>
<tr><td>Tandem Axle</td></tr>
<tr><td>Tridem Axle</td></tr>
<tr><td>Quad Axle</td></tr>
<tr><td rowspan="2">Distribution Type</td><td>Normal Distribution</td></tr>
<tr><td>Cumulative Distribution</td></tr>
<tr><td rowspan="16">General Traffic Inputs</td><td rowspan="3">Lateral traffic Wander</td><td>Mean Wheel Location (cm)</td><td></td></tr>
<tr><td>Traffic wander standard deviation(cm)</td><td></td></tr>
<tr><td>Design lane width(m)</td><td></td></tr>
<tr><td>Number axles/Truck</td><td></td><td></td></tr>
<tr><td rowspan="6">Axle Configuration</td><td>Avg. axle width (edge-to-edge outside dimension) (m)</td><td></td></tr>
<tr><td>Dual tire spacing (cm)</td><td></td></tr>
<tr><td>Tire pressure (kPa)</td><td></td></tr>
<tr><td rowspan="3">Axle spacing (cm)</td><td>Tandem Axle</td></tr>
<tr><td>Tridem Axle</td></tr>
<tr><td>Quad Axle</td></tr>
<tr><td rowspan="6">Wheelbase</td><td rowspan="3">Average axle spacing (m)</td><td>Short</td></tr>
<tr><td>Medium</td></tr>
<tr><td>Long</td></tr>
<tr><td rowspan="3">Percent of trucks</td><td>Short</td></tr>
<tr><td>Medium</td></tr>
<tr><td>Long</td></tr>
</table>

[1] All traffic volumes are assumed to be the same in all months. Input "Use Default".

[2] Includes Freeways, Principle Arterials, Collectors, Local Routes.

[3] Depends on Road Category, $VCDF_i = \frac{AADTT\ for\ class_i}{Total\ AADTT} \times 100$

[4] Includes Traffic growth function (linear, Compound), Traffic growth rate

[5] $ALDF_{ijk} = \frac{No.of\ axles\ for\ class_i,month_j,and\ load\ range_k}{Total\ No,of\ axles\ for\ class_i,month_j} \times 100$

The primary traffic data elements for the design of pavement are (ARA, 2004c):

1. Truck growth factor: which estimates future truck traffic volumes for the design period of a pavement.
2. Base year truck-traffic volume: The base year indicates the first year when the roadway segment under design is open to traffic. The following information is needed for the base year: two-way annual average daily truck traffic, number of lanes in the design direction, proportion of trucks in design direction, proportion of trucks in design lane and vehicle operational speed. All these inputs except vehicle operational speed are used to compute the number of trucks in the design lane.
3. Vehicle (truck) class distribution: which expresses the proportion of each truck class (classes 4 through 13) within the total number of trucks for the base year.
4. Hourly Distribution Factors (HDF): show the number of trucks passing over the pavement structure in each of the 24 hours of a day.
5. Monthly Distribution Factors (MDF): defines the number of trucks passing over the pavement in each month of the year.
6. Axle Load Spectra: categorizes the traffic loading in terms of the number of load applications of different axle configurations within a certain weight classification range. Axle load distribution factors need to be computed to express the proportion of total axle applications within each load interval for a given axle type and vehicle class (classes 4 to 13).
7. Axle and wheel base configurations: represent the tire, axle and vehicle wheelbase patterns for the estimation of pavement response.
8. Average Number of Axles Groups per Vehicle (AGPV): represent the average number of axles for each truck class (class 4-13) for each type of axle (single, tandem, tridem and quad).

Truck Growth Factor

The truck growth rate is calculated as follows (ARA, 2004c):

$$Traffic\ Growth\ Rate\ = \left(\left(\left(\frac{AADT_{FY}}{AADT_{IY}}\right)^{(FY-IY)}\right) - 1\right) \times 100 \qquad Equation\ 2\text{-}2$$

Where

$AADT_{FY} = Aannual\ Average\ Daily\ Traffic\ for\ the\ final\ year$

$AADT_{IY} = Aannual\ Average\ Daily\ Traffic\ for\ the\ initial\ year$

$FY - IY = Difference\ between\ the\ final\ and\ initial\ year$

Base Year Truck-Traffic Volume

1. Two-way AADTT: is calculated multiplying traffic (Average Annual Daily Traffic (AADT)) volume with the percentage of heavy trucks of FHWA class 4 or higher. The result is Average Annual Daily Truck Traffic (AADTT).
2. Number of lanes: represent the number of lanes in the design direction.
3. Percent trucks in design direction: or directional distribution factor (DDF), defines the percentage of trucks in the design direction. It is usually assumed to be 50 percent when the AADT and AADTT are given in two directions. The levels of input for percent trucks in design direction are described as follows:

    - Level 1: a site-specific directional distribution factor determined from WIM, AVC, and vehicle count data (ARA, 2004c).
    - Level 2: a regional/statewide directional distribution factor determined from WIM, AVC, and vehicle count data. Estimates from trip generation models may also be used.
    - Level 3: a national average value or an estimate based on local experience.

    The default or Level 3 values for the DDF should represent the predominant type of truck using the roadway. If detailed site-specific or regional/statewide truck traffic data are unavailable, the truck DDF for the most common truck type (e.g., vehicle class 9) is suggested for use as the default value for all truck traffic.

4. Percent trucks in design lane: or truck lane distribution factor (LDF), accounts for the distribution of truck traffic between the lanes in one direction. For two-lane, two-way highways (one lane in one direction), this factor is 1.0 because all truck traffic in any one direction must use the same lane. For multiple lanes in one direction, it depends on the AADTT and other geometric and site-specific conditions. The level of input for LDF is described as follows (ARA, 2004c):

- Level 1: a site-specific lane distribution factor determined from WIM, AVC, or vehicle count data.
- Level 2: a regional/statewide lane distribution factor determined from WIM, AVC, or vehicle count data.
- Level 3: a national average value or an estimate obtained from traffic forecasting and trip generation models. An estimate based on local experience is also considered Level 3.

The default (Level 3) values recommended to use based on the LDF for the most common type of truck (vehicle class 9 trucks) is as follows:

- Single-lane roadways in one direction, LDF = 1.00.
- Two-lane roadways in one direction, LDF = 0.90.
- Three-lane roadways in one direction, LDF = 0.60.
- Four-lane roadways in one direction, LDF =0.45.

5. Operational speed (kph): or the average travel speed generally depends on many factors, including the roadway facility type (Interstate or otherwise), terrain, percentage of trucks in the traffic stream, and so on. The ME software uses 60 mph (~ 100 kph) as the default operational speed value, but this speed can be modified to reflect local/site conditions.

Traffic Volume Adjustments

1. Monthly Adjustment Factors (MAF): represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month. It depends on factors such as climate conditions, site conditions, and local economy. The following levels of input are specified (ARA, 2004c):

- Level 1: site- or segment-specific MAF for each vehicle class (classes 4 through 13) computed from WIM, AVC, or vehicle count data or trip generation models.
- Level 2: regional/statewide MAF for each vehicle class (classes 4 through 13) computed from WIM, AVC, or vehicle count data or trip generation models.

- Level 3: national MAF computed from WIM, AVC, or vehicle count data. The use of estimates based on local experience is also considered Level 3 data.

Monthly adjustment factor can be calculated by the following equation (ARA, 2004c):

$$MAF_i = \frac{AMDTT_i}{\sum_{i=1}^{12} AMDTT_i} \times 12 \qquad Equation\ 2\text{-}3$$

Where

$MAFi\ =\ monthly\ adjustment\ factor\ for\ month\ i$

$AMDTTi\ =\ average\ monthly\ daily\ truck\ traffic\ for\ month\ i$

The sum of the MAF of all months must equal 12.

2. Truck Hourly Distribution Factors (HDF): represent the percentage of the AADTT within each hour of the day. The inputs at different levels are as follows (ARA, 2004c):

- Level 1: a site- or segment-specific distribution determined from AVC, WIM, or vehicle count data.
- Level 2: a regional/statewide distribution determined from AVC, WIM, or vehicle count data.
- Level 3: the factors determined from a national data or local experience.

3. Traffic Growth Factors: at a site or segment are best estimated when a continuous traffic count data is available. The ME Design software suggests three different traffic growth functions to compute the growth or decay in truck traffic over time as follow:

$$No\ growth \qquad AADTT_X = 1.0 \times AADTT_{BY} \qquad Equation\ 2\text{-}4$$

$$Linear\ growth \qquad AADTT_X = GR \times AGE + AADTT_{BY} \qquad Equation\ 2\text{-}5$$

$$Compound\ growth \qquad AADTT_X = ADTT_{BY} \times (GR)^{AGE} \qquad Equation\ 2\text{-}6$$

Where

$AADTT_X$: *the annual average daily truck traffic at age X*,

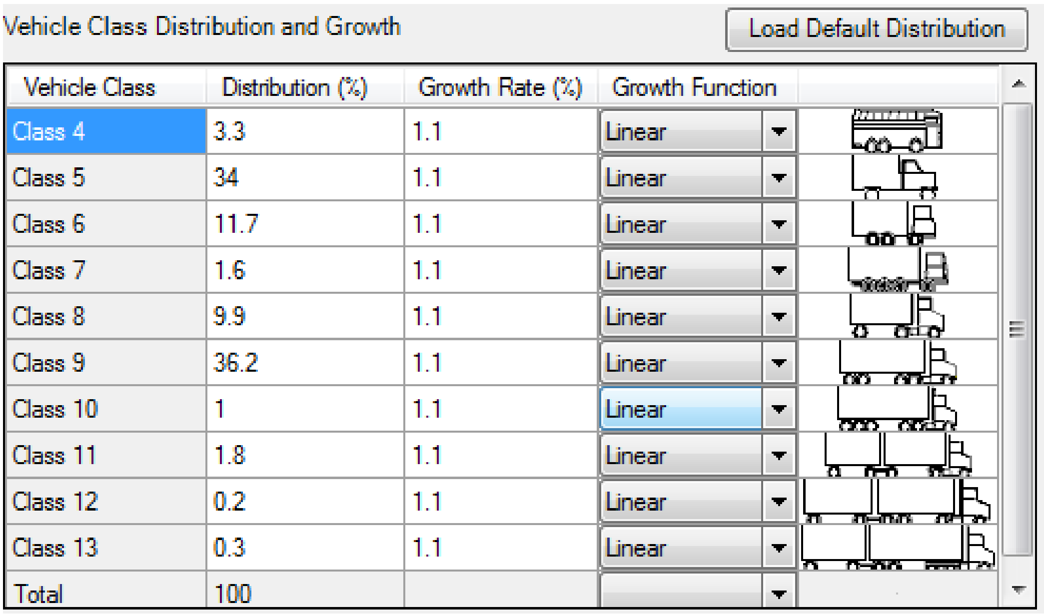
GR: *the traffic growth rate*

$AADTT_{BY}$: *the base year annual average daily truck traffic.*

4. Vehicle Class Distribution: or the average normalized truck-volume distribution represents the percentage of each truck class within the truck traffic distribution. The value is computed from data obtained of vehicle classification counting programs such as AVC, WIM, and vehicle counts. The inputs at different levels are as follows (ARA, 2004c):

    - Level 1: data obtained from site or segment specific WIM, AVC, or vehicle counts.
    - Level 2: data obtained from regional/statewide WIM, AVC, or vehicle counts.
    - Level 3: data obtained from national WIM, AVC, or vehicle counts or local experience.

The Figure 2-4 illustrates the related section for Vehicle Class Distribution and Growth in ME Design.



Vehicle Class Distribution and Growth | Load Default Distribution

Vehicle Class	Distribution (%)	Growth Rate (%)	Growth Function
Class 4	3.3	1.1	Linear
Class 5	34	1.1	Linear
Class 6	11.7	1.1	Linear
Class 7	1.6	1.1	Linear
Class 8	9.9	1.1	Linear
Class 9	36.2	1.1	Linear
Class 10	1	1.1	Linear
Class 11	1.8	1.1	Linear
Class 12	0.2	1.1	Linear
Class 13	0.3	1.1	Linear
Total	100		

Figure 2-4: Vehicle class distribution and growth (ME Design V 2.1)

The Table 2-4 shows the truck class distribution default values included in AASHTOWare Pavement ME Design software.

Table 2-4: TTC Groups used in ME Design (AASHTO, 2015b)

		Truck Class Distribution (%)									
		4	5	6	7	8	9	10	11	12	13
1	Major single-trailer truck route (type I)	1.3	8.5	2.8	0.3	7.6	74.0	1.2	3.4	0.6	0.3
2	Major single-trailer truck route (type II)	2.4	14.1	4.5	0.7	7.9	66.3	1.4	2.2	0.3	0.2
3	Major single-trailer truck route (type I)	0.9	11.6	3.6	0.2	6.7	62.0	4.8	2.6	1.4	6.2
4	Major single-trailer truck route (type III)	2.4	22.7	5.7	1.4	8.1	55.5	1.7	2.2	0.2	0.4
5	Major single and multi-trailer truck route (type II)	0.9	14.2	3.5	0.6	6.9	54.0	5.0	2.7	1.2	11.0
6	Intermediate light and single trailer truck route (type I)	2.8	31.0	7.3	0.8	9.3	44.8	2.3	1.0	0.4	0.3
7	Major mixed truck route (type I)	1.0	23.8	4.2	0.5	10.2	42.2	5.8	2.6	1.3	8.4
8	Major multi-trailer truck route (type I)	1.7	19.3	4.6	0.9	6.7	44.8	6.0	2.6	1.6	11.8
9	Intermediate light and single-trailer truck route (type II)	3.3	34.0	11.7	1.6	9.9	36.2	1.0	1.8	0.2	0.3
10	Major mixed truck route (type II)	0.8	30.8	6.9	0.1	7.8	37.5	3.7	1.2	4.5	6.7
11	Major multi-trailer truck route (type II)	1.8	24.6	7.6	0.5	5.0	31.3	9.8	0.8	3.3	15.3
12	Intermediate light and single-trailer truck route (type III)	3.9	40.8	11.7	1.5	12.2	25.0	2.7	0.6	0.3	1.3
13	Major mixed truck route (type III)	0.8	33.6	6.2	0.1	7.9	26.0	10.5	1.4	3.2	10.3
14	Major light truck route (type I)	2.9	56.9	10.4	3.7	9.2	15.3	0.6	0.3	0.4	0.3
15	Major light truck route (type II)	1.8	56.5	8.5	1.8	6.2	14.1	5.4	0.0	0.0	5.7
16	Major light and multi-trailer truck route	1.3	48.4	10.8	1.9	6.7	13.4	4.3	0.5	0.1	12.6
17	Major bus route	36.2	14.6	13.4	0.5	14.6	17.8	0.5	0.8	0.1	1.5

Table 2-5 shows the definition of 17 TTC (Truck Traffic Classification) groups included in AASHTOWare Pavement ME Design software.

Table 2-5: Definitions and descriptions for the TTC Groups (AASHTO, 2015b)

Traffic Stream	Commodities Being Transported by Type of Truck		TTC Group No.
	Multi-Trailer	**Single-Trailer and Single Unit Trucks**	
Low to None (<2%)	Relatively high amount of multi-trailer trucks (>10%)	Predominantly single-trailer trucks	5
		High percentage of single-trailer trucks, but some single-unit trucks	8
		Mixed truck traffic with a higher percentage of single-trailer trucks	11
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	13
		Predominantly single-unit trucks	16
	Moderate amount of Multi-Trailer Trucks (2 to 10%)	Predominantly single-unit trucks	3
		Mixed truck traffic with a higher percentage of single-trailer trucks	7
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	10
		Predominantly single-unit trucks	15
Low to Moderate (>2%)	Low to None (<2%)	Predominantly single-unit trucks	1
		Predominantly single-trailer trucks, but with a low percentage of single-unit trucks	2
		Predominantly single-trailer trucks with a low to moderate amount of single-unit trucks	4
		Mixed truck traffic with a higher percentage of single-trailer trucks	6
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	9
		Mixed truck traffic with a higher percentage of single-unit trucks	12
		Predominantly single-unit trucks	14
Major Bus Route (>25%)	Low to None (<2%)	Mixed truck traffic with about equal single-unit and single-trailer trucks	17

Axle Load Distribution Factors

The axle load distribution factors represent the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (classes 4 through 13). A definition of load intervals for each axle type is provided below (ARA, 2004c):

- Single axles – 3,000 lb. to 40,000 lb. at 1,000-lb intervals.
- Tandem axles – 6,000 lb. to 80,000 lb. at 2,000-lb intervals.
- Tridem and quad axles – 12,000 lb. to 102,000 lb. at 3,000-lb intervals.

The normalized axle load distribution or spectra can only be determined from weigh in motion (WIM) data. Therefore, the level of input depends on the data source (site, regional, or national). The following input levels for axle load distribution factors were defined by ME Design (ARA, 2004c):

- Level 1: the distribution factors determined based on an analysis of site- or segment specific WIM data.
- Level 2: the distribution factors determined based on an analysis of regional/statewide WIM data.
- Level 3: the default distribution factors computed from a national database such as LTPP.

General Traffic Inputs

1. Mean Wheel Location: Distance from the outer edge of the wheel to the pavement marking. The inputs at different levels are as follows (ARA, 2004c):

    - Level 1: determined through direct measurements on site-specific segments (not applicable to new alignments).
    - Level 2: determined from measurements on roadways with similar traffic characteristics and site conditions.
    - Level 3: national average value or estimates based on local experience.

2. Traffic Wander Standard Deviation: the standard deviation of the lateral traffic wander. The wander is used to determine the number of axle load applications over a point for predicting distress and performance. The different levels for traffic wander are (ARA, 2004c):

    - Level 1 – the value determined through direct measurements on site-specific segments (not applicable to new alignments).
    - Level 2 – a regional/statewide average value determined from measurements on roadways with similar traffic characteristics and site conditions (e.g., functional class, pavement type, level of service and so on).
    - Level 3 – national average value or estimates based on local experience.

3. Design Lane Width: refers to the actual traffic lane width, as defined by the distance between the lane markings on either side of the design lane. It is a design factor and may or may not equal the slab width.
4. Number of Axle Types per Truck Class: represents the average number of axles for each truck class (class 4 to 13) for each axle type (single, tandem, tridem, and quad). The inputs at different levels are as follows (ARA, 2004c):

    - Level 1 – the values determined through direct analysis of site-specific traffic data (AVC, WIM, or traffic counts).
    - Level 2 – the values determined through direct analysis of regional/statewide traffic data (AVC, WIM, or traffic counts).
    - Level 3 – the default values based on analysis of national databases such as the LTPP databases.

c) Climatic data

ME Design requires five climatic data on an hourly basis over the entire design life for the design project: hourly air temperature; hourly precipitation; hourly wind speed; hourly percentage sunshine; hourly relative humidity.

The climatic data provided in the format of .hcd file with associated station.dat file for the selected location is available to be generated and downloaded from MERRA website

(Federal Highway Administration, 2018). Hourly climatic database files or *.hcd files contain information for a specific weather station. The information in the *.hcd files are respectively associated with the following input parameters: date and hour (YYYYMMDDHH); temperature (°C); wind speed (km/h); sunshine (%); precipitation (mm); relative humidity (%).

The station.dat file contains all the hourly climatic database weather stations. Each weather station included has the following information (TRB, 2018): weather station number; weather station abbreviation; location (city/state); latitude; longitude; elevation; first date in file (YYYYMMDD).

These climatic data are mainly obtained from weather stations located near the project site. The software generates a virtual station by interpolating climatic data from selected number of nearby weather stations (up to six stations). Interpolation of climatic data from these stations is conducted by averaging data using a 1/R weighting scheme as follows (Q. J. Li et al., 2013):

$$V_m = \frac{\sum_{i=1}^{k}(V_{mi}/R_i)}{\sum_{i=1}^{k}(1/R_i)} \qquad Equation\ 2\text{-}7$$

Where:

V_m : $Calculated\ virtual\ weather\ data\ element\ for\ day\ m,$

k: $Number\ of\ weather\ stations\ selected\ for\ VWS\ interpolation,$

V_{mi}: $Value\ of\ a\ data\ element\ on\ day\ m\ for\ weather\ station\ i,$

R_i: $Distance\ of\ weather\ station\ i\ from\ the\ pavement\ project\ site.$

Additional environmental data are also required:

- longitude, latitude, elevation;
- groundwater table depth;
- drainage/surface properties:
    - surface shortwave absorptivity;
    - infiltration;

- drainage path length;
- cross slope.

The depth to a water table used in AASHTOWare Pavement ME Design software is the depth below the final pavement surface. The designer has the option to enter an annual depth to the water table or seasonal water table depths. The average annual depth could be used, unless the designer has historical data to determine the seasonal fluctuations of the water table depth.

If a subsurface drainage system is used to lower that water table, that lower depth should be used in the program, not the depth measured during the subsurface investigation.

The climate inputs are used to predict moisture and temperature distributions inside the pavement structure. Asphalt concrete stiffness is sensitive to temperature variations and unbound material stiffness is sensitive to moisture variations (AASHTO, 2015b).

d) Material properties

The large set of material properties required for ME approach is implemented in three parts of the design process: the climate model; the pavement response models; the distress models.

The material data related to climatic model are used to determine temperature and moisture profiles inside the pavement structure. The pavement response models use material properties to compute the state of stress/strain at critical locations in the structure due to traffic loading and temperature changes.

These structural responses are then used by the distress models along with complementary material properties to predict pavement performance. In this study, only new flexible pavements are evaluated. (ARA, 2004b).

The Table 2-6 summarizes the flexible pavement material properties required by the ME Design for each part of design process.

Table 2-6: Major material input considerations for flexible pavement (ARA, 2004b)

Materials Category	Materials inputs required for each part of design		
	Pavement response models	**Distress models**	**Climate model**
Hot-Mix Asphalt Materials	• Time-temperature dependent dynamic modulus (E*) of HMA mixture. • Poisson's ratio.	• Tensile strength, creep compliance, coefficient of thermal expansion.	• Surface shortwave absorptivity (only required for surface course), thermal conductivity, and heat capacity of HMA. • Asphalt binder viscosity (stiffness) characterization to account for aging.
Chemically Stabilized Materials	• Elastic modulus (E). • Resilient modulus (Mr). • Poisson's ratio. • Unit weight.	• Minimum resilient modulus, Modulus of rupture	• Thermal conductivity and heat capacity.
Unbound Base/ Subbase and Subgrade Materials	•Seasonally adjusted resilient modulus (Mr). • Poisson's ratio. • Unit weight. •Coefficient of lateral pressure.	• Gradation parameters.	• Plasticity index, gradation parameters, effective grain sizes, specific gravity, saturated hydraulic conductivity, optimum moisture contents, parameters to define the soil water characteristic curve.
Bedrock	• Elastic modulus (E). • Poisson's ratio. • Unit weight.	None	None

The Figure 2-5 shows the main three parts of the flexible pavement material properties required by the ME Design.

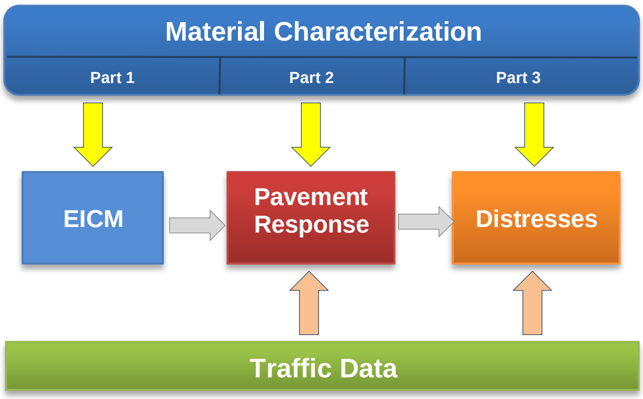


Figure 2-5: Relationship between inputs and outputs of MEPDG (Su et al., 2017).

2.3.6 Pavement Response Models

The MEPDG employs three models to predict pavement structural responses (stresses, strains, and displacements). Multi-Layer Elastic Theory (MLET or JULEA as a modified version) and the Finite Element Model (FEM) are used to compute responses due to traffic loading and the Enhanced Integrated Climate Model (EICM) is used to predict temperature and moisture histories throughout the pavement structure. The load-related analysis is accomplished with JULEA, however if non-linear behavior of unbound materials is desired (i.e., for level 1 inputs) the FEM is applied. These models are embedded in AASHTOWare Pavement ME Design software (AASHTO, 2015b).

2.3.7 MEPDG Adoption and Implementation Status in the World

Currently, several US states are moving towards incorporating the mechanistic-empirical approach into their pavement design standards. Although there are still significant number of states that uses the other pavement design methods (e.g., AASHTO 1993), it is important to mention that almost 20 states of US are currently implementing MEPDG, solely or as parallel design with AASHTO 1993 (either for both or one pavement type, flexible pavement and rigid pavement). In total about 33 states and several Canadian provinces are working on key parts of the process, including developing appropriate design inputs, establishing material and traffic databases, and training staff or consultants in the proper use of the procedure. Additionally, while the AASHTO Guide for the Local Calibration of the MEPDG was published in 2010, most agencies are actively engaged in calibrating the ME performance models to local conditions, policies, and materials. The most challenging technical aspects of implementation that are reported by the road agencies in these states are as follows (Linda Pierce & Kurt Smith, 2015):

- Compatibility of performance measures and threshold criteria.
- Designing pavement structures with features that are not included in Pavement ME or that have not been calibrated (e.g., thin PCC overlays, permeable asphalt- or cement-treated bases, geogrids and other reinforcing materials).
- Data availability.
- Characterization of traffic.
- Characterization of climate.
- Characterization of material properties.
- Back calculation analysis for characterizing existing pavement and subgrade properties.
- Sensitivity analysis of key design inputs.
- Availability of performance data to adequately perform local calibration and verification.
- Insensitivity to unbound material layer thicknesses and stiffness.
- Keeping up with version changes and the requirements to move to a newer version.

The approach of MEPDG as well as the related issues for the implementation are also studied in other countries. Following researches and works are the examples of efforts accomplished in this area for each country:

- Calibration of Distress Models from the Mechanistic–Empirical Pavement Design Guide for Rigid Pavement Design in Argentina (Bustos et al., 2011).
- Toward Implementation of the Mechanistic-Empirical Pavement Design Guide in Latin America (Delgadillo et al., 2011).
- Determination of Local Fatigue Model Calibration Used in MEPDG for Iran's Dry-No Freeze Region (Azadi et al., 2013)

- Introducing the use of mechanistic-empirical analysis method in Qatar for evaluating the performance of perpetual pavements under different levels of traffic loading (Sadek et al., 2014).
- Evaluation of Witczak E predictive models for the implementation of AASHTOWare-Pavement ME Design in the Kingdom of Saudi Arabia (Khattab et al., 2014).
- Site Specific Traffic Inputs for Mechanistic-Empirical Pavement Design Guide in Poland (Zofka et al., 2014).
- Application of the Mechanistic Empirical Pavement Design Guide to the pavement structures of the Portuguese manual (Ferreira et al., 2015)
- The Mechanistic-Empirical Pavement Design: An Egyptian (Aguib & Khedr, 2016).
- Development of a national database of asphalt material performance properties in support of perpetual pavement design implementation in Australia (Yousefdoost et al., 2018)
- Implementation initiatives of the Mechanistic-Empirical Pavement Design Guide in Lebanon to present a methodology for adoption of MEPDG in countries with insufficient design input data (Chehab et al., 2018).

2.4 SHELL Pavement Design Method

2.4.1 SHELL Overview

Introducing the SHELL pavement design method by Claussen et al. (1977) in Europe, changed the pavement design approach from the empirical to the mechanistic-empirical approach. The first version of manual for the SHELL pavement design method was published in 1963 and was updated in 1978 (by SHELL International Petroleum Company, Ltd.). The manual of 1978 contained some additional design parameters such as the pavement temperature, different types of asphalt mixes and the rut depth calculation in the asphalt layers. Then the third version of SHELL pavement design manual after several years of using and modifying the second version was introduced in 1985. One of the major improvements in this version of manual was the prediction of the permanent deformation of the asphalt layers (Pereira & Pais, 2016).

Current pavement design approach used by Portuguese road agencies is based on SHELL method and the approach applied in Portuguese pavement design manual (Junta Autónoma de Estradas (JAE), 1995). The SHELL method aims to obtain an appropriate pavement structure design by satisfying some specified performance criteria.

The Portuguese pavement design manual for the national road network (Manual de Conceção de Pavimentos para a Rede Rodoviária Nacional, MACOPAV) is used mainly in the design of new road pavements, considering traffic data, pavement foundation data, material data and general climatic data (Junta Autónoma de Estradas (JAE), 1995).

The SHELL method models the pavement structure as a linear elastic multi-layered system. The software BISAR associated with this method calculates stresses, strains and deflections at any point of the proposed pavement design under any combination of vertical and horizontal surface loads (while is able to consider slip between the pavement layers). The inputs used in this method are the material characterizations (Young's moduli and Poisson's ratios of the layers), number of the layers, thickness of the layers (except for the semi-infinite base layer), the interface shear spring compliance at each interface, the number and position of loads (SHELL, 1998).

The traffic volume used in SHELL method is expressed in terms of ESAL (or Equivalent Single Axle Load) for a standard axle (equivalent standard (80 kN) axles). The process of converting axial loads into typical axles uses equivalency factors (LEF or Load equivalency factor), determined from the equivalence equation (Power Law or AASHTO load equivalency equation) or the tables of AASHTO LEFs (Kawa et al., 1998).

$$\left(N_{EQ}\right)_i = \left(\frac{P_i}{P_0}\right)^x \qquad \textit{Equation } 2\text{-}8$$

Where

$\left(N_{EQ}\right)_i$: $equivalent\ number\ of\ passages\ of\ the\ standard\ axle\ for\ the\ axle\ i$

P_i: $load\ of\ axle\ i$

P_0: $standard\ axle\ load\ depending\ on\ pavement\ type\ (NCHRP\ Report\ 353, 1993)$

x: $constant\ depending\ on\ the\ pavement\ type$

The standard design single axle load adopted in SHELL method is 80 kN, applied through four wheels of 20 kN (dual wheel at each side of the axle) with a contact pressure of 0.6 MPa and a radius of contact area of 10.5 cm.

The expression of load in the SHELL method is illustrated in the Figure 2-6 (Lopes, 2009; Nikolaides, 2015):

Standard Axle of 80 kN: L=105 mm; p=0,6 MPa; r=105mm

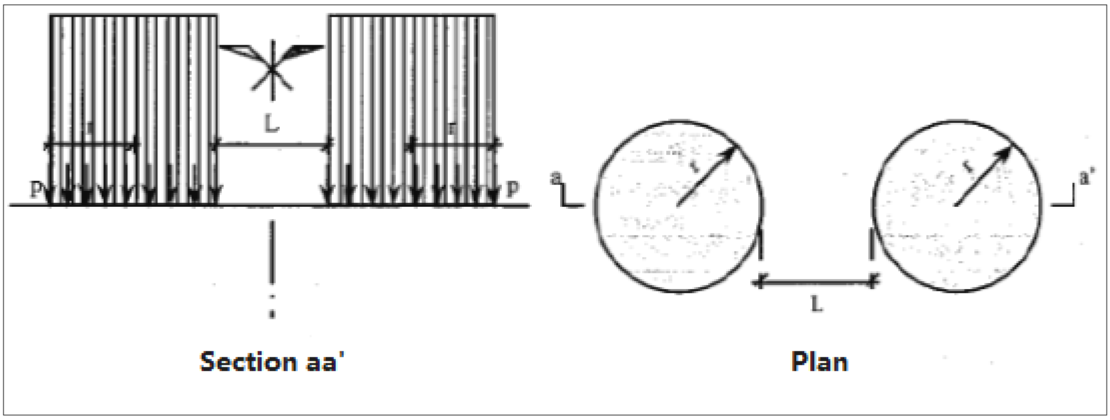


Figure 2-6: The Schematic of load for standard axles (Branco et al., 2011)

The stiffness of subgrade is calculated based on the CBR value of the soil. Subgrade modulus is calculated by the following equation (Claessen et al., 1977):

$$E = 10^7 CBR \qquad Equation\ 2\text{-}9$$

Where

E : $Subgrade\ modulus\ (Pa)$
CBR: $the\ bearing\ capacity\ of\ the\ subgrade\ (\%)$.

The stiffness of unbound material is calculated based on the following equation (Dormon & Metcalf, 1965):

$$E_2 = k\ E_3 \qquad Equation\ 2\text{-}10$$

$$k = 0.2\ {h_2}^{0.45} \qquad Equation\ 2\text{-}11$$

Where

$E2$: $stiffness\ of\ the\ granular\ layer\ or\ superior\ layer\ (MPa)$
$E3$: $modulus\ of\ the\ subgrade\ or\ inferior\ layer\ (MPa)$
$h2$: $thickness\ of\ the\ granular\ layer\ (mm)$
k: $coefficient\ function\ of\ the\ thickness\ of\ the\ granular\ layer\ (h2)$
$applicable\ in\ the\ range\ 2 < k < 4$

The climatic conditions in the SHELL method employ two main factors for the pavement design; the temperature and the moisture content or water content. The service temperature (the temperature on the pavement) is used to define the characteristics of asphalt mixes (Dynamic modulus). The temperature is calculated based on a weighted mean annual air temperature, w-MAAT, which is derived from the mean monthly air temperatures, MMAT, from a given location and is related to an effective asphalt temperature and thus to an effective asphalt stiffness (Shell, 1978). The procedure of SHELL pavement temperature calculation is explained in detail in the chapter 4.

The moisture content of the unbound layers and soil are for the definition of the drainage system of the road, as well as properties of unbound materials, mainly the subgrade CBR. There is another factor in SHELL pavement design method called “the degree of risk”. This factor is assumed in the definition of the properties of the materials, as well as in the fatigue laws used to predict distresses in the pavement structure (Pereira & Pais, 2016).

Three main performance criteria in the SHELL pavement design method, which are necessary to be satisfied are (Read & Whiteoak, 2003):

1. Fatigue cracking of the bound materials.
2. Permanent deformation of the subgrade.
3. Tensile stress at the bottom of soil cement, in case the pavement structure includes the stabilized layer.

Fatigue cracking is mainly caused by the repetitive horizontal tensile strain developed in the flexible layers under heavy traffic loads. The thicknesses of flexible layers are adjusted by this fatigue cracking criterion, and are based on traffic loads, climatic conditions, and the expected lifespan of the pavement. Accordingly, the following equation is applied for this purpose (Shell, 1978):

$$\varepsilon_t = (0.856\, V_b + 1.08)\, E^{-0.36}\, N^{-0.2} \qquad Equation\ 2\text{-}12$$

Where

ε_t: $Tensile\ strain\ at\ the\ bottom\ of\ aspahlt\ layer (m/m)$
N: $Allowable\ number\ of\ loading\ repititions\ (ESAL)\ until\ fatigue\ cracking\ failure$
V_b: $Volumetric\ percentage\ of\ bitumen\ in\ the\ asphalt\ mix$
E: $Asphalt\ concrete\ stifness\ modulus\ (Pa)$.

The other criterion applied in the SHELL design method is permanent deformation or compression strain on top of the subgrade which is employed for pavement thickness design by the following equation (Shell, 1978):

$$\varepsilon_z = k_s\, N^{-0.25} \qquad Equation\ 2\text{-}13$$

Where

ε_z: $Compression\ strain\ at\ the\ top\ of\ subgrade\ (m/m)$

N: $Allowable\ number\ of\ ESAL$

$$k_s: \begin{cases} 2.8 \times 10^{-2} for\ 50\%\ of\ survival\ probability \\ 2.1 \times 10^{-2} for\ 85\%\ of\ survival\ probability \\ 1.8 \times 10^{-2} for\ 95\%\ of\ survival\ probability \end{cases}$$

The third criterion applied in the SHELL design method is tensile stress at the bottom of soil cement, in case the pavement structure includes the stabilized layer (Shell, 1978):

$$\frac{\sigma_t}{\sigma_{rf}} = 1 + a\ log(N_{adm}) \qquad Equation\ 2\text{-}14$$

$$\sigma_{rf} = 1.5\ \sigma_{cd} \qquad Equation\ 2\text{-}15$$

Where

σ_t: $Max\ Tensile\ Stress\ at\ the\ bottom\ of\ Soil\ Cement\ (MPa)\ obtained\ by\ 80\ kN\ Standard\ Axle$
σ_{cd}: $Diametral\ Compression\ Stress\ (MPa)$
σ_{rf}: $Tensile\ Stress\ obtain\ from\ a\ flexion\ test\ \ (MPa)$
a: $Constant\ Value, -0.06 \le a \le -0.1$
N_{adm}: $Allowable\ number\ of\ ESAL$

2.4.2 BISAR Program

The BISAR program is designed to calculate the effect of vertical and horizontal stresses (shear forces at the surface) and includes an option to account for the effect of (partial) slip between the layers, through a shear spring compliance at the interface. The following inputs are required for BISAR program calculations (SHELL, 1998):

- the number of layers;
- the Young's moduli of the layers;
- the Poisson's ratios of the layers;
- the thickness of the layers (except for the semi-infinite base layer);
- the interface shear spring compliance at each interface;
- the number of loads;
- the co-ordinates of the position of the center of the loads;
- (optional) the horizontal tangential component of the load and the direction of this shear load;
- the co-ordinates of the positions for which output is required;
- one of the following combinations to indicate the vertical normal component of the load:
    - stress and load;
    - load and radius;
    - stress and radius.

2.5 Summary of Main Features for SHELL and ME

This chapter is establishing a general comparison for two pavement design methods of ME and SHELL through the reviewing of the existing state-of-knowledge in both methods and their related components. For the comparison of both methods, the following advantages and disadvantages of implementing both methods of SHELL and ME are addressed:

- As for the confirmation of design expectations, ME Design predicts more performance indicators than SHELL method. However, local calibration of the empirical models in the ME Design prior to implementation are strongly recommended by AASHTO. The calibration process requires an agency to compare historical pavement performance from existing pavements to performance predicted by the ME Design and adjust the empirical models' coefficients to minimize the error between measured and predicted performance. Although ME Design considers a wide range of performance indicators, the calibration process demands a significant amount of time, effort and cost which this process is not required for SHELL method (Haas et al., 2007).

- The ME Design requires over 100 inputs in different data parts (materials, climate and traffic). The lack of adequate data, costly and time-consuming procedure of obtaining data could be the disadvantage of this aspect.
- As it is mentioned, the ME Design requires a large set of data to model traffic, climate, materials, and pavement performance. Based on the importance of the project, different hierarchical levels of data can be used. Many designers may lack specific knowledge of the data required. Sensitivity study may assist designers in focusing on those inputs having the most effect on desired pavement performance. The task of sensitivity analysis demands considerable amount of time and effort. The calculating time of ME Design, depending on the complexity of the problem and the amount of data available, may take from 5 minute to 4 hour per simulation (Martinez-Echevarria Romero et al., 2016). This implies some challenges in terms of investment in data collection, experimental research, demanding greater cost-benefit analyses, between data cost and quality of the design process.
- One of the advantages of ME Design over SHELL method is employing several aspects of load types variation (tire loads, axle and tire configurations, repetition of loads, distribution of traffic across the pavement, vehicle speed).
- Being applicable for both existing pavement rehabilitation and new pavement construction for ME method while SHELL method is used mainly in the design of new road pavements.
- While SHELL method uses the initial properties of the materials to predict the performance for the entire life period of pavement, the ME Design applies environmental and aging effects on materials. Accordingly, pavement performances in ME method are incrementally calculated considering the evolution of traffic and

climatic conditions over time. The incremental approach provides more reliable performance predictions in ME method than SHELL method.

The Table 2-7 shows the summary of main features for both methods of SHELL and ME that are mentioned in this chapter.

Table 2-7: The summary of main features for SHELL method and ME method

Feature		SHELL method	ME method
Pavement design approach		Mechanistic-Empirical	Mechanistic-Empirical
Associated Software		BISAR	AASHTOWare PAVEMENT ME DESIGN™
Performance Criteria	Terminal IRI (m/km)	Not applied for design	✓
	Total rutting (mm)	✓	✓
	AC rutting (mm)	Not applied for design	✓
	AC bottom-up fatigue cracking (%)	✓	✓
	AC top-down fatigue cracking (m/km)	Not applied for design	✓
	AC thermal cracking (m/km)	Not applied for design	✓
	Chemically stabilized layer- fatigue fracture	✓	✓
Input hierarchical levels		Not applied for design	✓
Climate conditions	Pavement temperature	• Equivalent single service temperature	• Pavement temperature profile • Not constant in time or through depth • Internally by EICM
	Moisture content for unbound layers and soil	• Optimum moisture content value	• Moisture content profile • Not constant in time or through depth • Internally by EICM
	Required Climate data	• Moisture content of the unbound layers and soil • Mean monthly air temperature	• Hourly air temperature • Hourly precipitation • Hourly wind speed • Hourly % sunshine • Hourly relative humidity • Longitude, Latitude, Elevation • Groundwater table depth • Drainage/surface properties
Loading conditions		• ESAL (Equivalent single axle load)	• Axle-load spectra, • Hourly/monthly distribution factors, • Distribution of truck classifications, • Special axle configurations.
Incremental approach		Not applied for design	✓

3 Data Preparation for ME Design

3.1 Introduction

A comprehensive pavement design approach aims to produce the best possible structure which will be able to minimize the life cost and maximize the pavement performance. For this reason, comparing different design methods to identify the most efficient design approach has always been an important challenge for the road agencies. The main objective of this chapter is to prepare the required inputs for ME Design projects. This objective is illustrated based on three different Portuguese roads (which are already designed by SHELL method) that were selected for data preparation of ME Design. The considerations in choosing the roads are:

1. Data availability for design and construction (access to design documents).
2. Different hierarchy in road network to have different demands and applied traffic.
3. To illustrate the variability of the results for roads.

The selected roads for this study are: IC3 _Variante de Tomar_ No da Atalaia; IP6_Peniche_Atouguia da Baleia; EN254_Variante de São Miguel de Machede.

The preparation of structure design for ME design are explained in next section. The cross sections of selected roads extracted from projects and prepared for ME Design are illustrated in the Figure 3-1.

Figure 3-1: The cross sections of three selected roads

The process of data preparation and implementation in ME Design is only explained for IC3 road in this chapter. The related tables for material properties of IC3 and the data preparation process regarding two other roads of IP6 and EN254 are provided in Appendix 2.

3.2 Data Resources and Availability

The first step is to identify and organize the required data based on the importance or priority for the ME Design. In summary the input data required to define a trial design for selected road projects in ME Design software are specific site subgrade support, material properties, traffic loading and environmental conditions.

Considering the first three input data categories, some meetings and discussions has been set with "IP" (Portuguese Road Administration) to obtain and prepare required data from available databases. However, the database documents are mainly paper based (which it demands more time and effort to extract the required data from the documents). In addition, the data had some deficiencies as it was expected in some cases and required some calculations to obtain the target or demanded value otherwise it should be considered the default value in the software or mean value for the local condition in the lack of data. The data is extracted for the selected road sections and then the outliers are discarded from the other data and finally the average values are calculated to implement in the software.

Regarding the preparation of the climate data, there are four *.hcd (Hourly Climatic Database) files generated by MERRA tools (Federal Highway Administration, 2018) including climate data for four Portuguese regions which covers the full MERRA time series, 1/1/1979 to 5/31/2012, it is possible to run the shorter time periods by setting appropriate values for the Traffic Open Month and Design Life inputs in the General Information screen:

- 95001.hcd – Coimbra;
- 95002.hcd – Porto;
- 95003.hcd – Beja;
- 95004.hcd – Lisboa;
- Station.dat.

The climate data unit has been converted from US unit to the metric unit (the employed unit that has been set for the software) and some format errors in the *.hcd files have been corrected.

The current weather stations available in ME Design are only for the climate data of the United States. To apply a new weather station (i.e., four Portuguese regions) in ME Design, a new *.hcd file should be obtained and added in ME Design. For this purpose, a number unused in the station.dat file should be assigned and entered to the station.dat file list. Accordingly, four created *.hcd files have been added to the station.dat file list and the station.dat file related to the software has been replaced by the new one in the C:\Program Files\AASHTOWare\ME Design\DefaultsMetric. Then the Portuguese *.hcd files have been added to the *.hcd files in the C:\Program Files\AASHTOWare\ME Design\HCD_SI.

3.3 Road Description

For summary, the general description of selected roads is provided in the Table 3-1 (IP, 2004; Proplano - Gabinete de Estudos e Projectos Lda., 2002; Tecnofisil, 2002).

Table 3-1:General information of selected roads

General Info	Road Name		
	IC3	IP6	EN254
Type of Road	Rural Freeway	Rural Freeway	Rural Collector
Design Speed (km/h)	120	120	100
Operational Speed (km/h)	90	90	80
Transverse Road Profile (m)	[1.0x3.75x3.75x3.0] x2	[0.50x3.5x3.5x2.5]x 2	2.25x3.5x3.5x2.25
Design Type	New pavement	New pavement	New pavement
Pavement Type	Flexible pavement	Flexible pavement	Flexible pavement
Design Life	20	20	20
Base Construction	September 2004	March 2002	November 2003
Pavement Construction	May 2006	January 2003	June 2004
Traffic Opening	September 2006	June 2004	September 2004

Regarding the content of Table 3-1, the following definitions are used:

- **Design Speed:** a selected speed used to determine the various geometric design features of the roadway.

- **Operational Speed:** a speed at which a typical vehicle or the overall traffic operates. Operating speed may be defined with speed values such as the average, pace, or 85th percentile speeds (European Commission, 2016; Fitzpatrick et al., 2003).
- **Rural Freeway**: Two ways, two lanes per direction.
- **Rural Collector**: Single carriageway road, One lane per direction.
- **Base Construction:** When the base is prepared for the construction.
- **Pavement Construction:** When the pavement surface layer is placed.

3.4 Road Section Selection

For the data preparation, it is required to select some sections of the road (by checking the longitudinal profile of the road) that has the following specifications:

1. Availability of data for the selected section in quality control data and pavement structure design.
2. It is preferred that the selected length of the road would be low gradient.
3. The frequency of embankments and excavations in the selected length of the road would be more than other parts of the road.
4. The selected length of the road should be without bridge or intersections.

The priority of selection is with the availability of data. Considering the above specifications, following kms are selected for IC3 road (obtained from IP documents, plan and longitudinal profile, TMR-PE-114A Layout and TMR-PE-115 Layout): km 4+400 to 5+400; km 5+800 to 7+000. From this point and for simplification, the selected sections are always called by their road name in the text. So, IC3 road, IP6 road and EN254 road would be the applied names for the selected road sections. The longitudinal profile plans for road section selections are provided in Appendix 3.

3.5 Selection of Design-Performance Criteria and Reliability Levels

Applying rational performance criteria guarantees that a pavement design will perform adequately over its design life. The justification of the performance criteria for the selected roads is discussed in chapter 4. The reliability level for all performance criteria is considered the same (90 %). The Table 3-2 shows the default values for ME Design V 2.1 and selected values for IC3 road for the analysis parameters adjusted based on Portuguese pavement conditions.

Table 3-2: Analysis parameters

Performance Criteria	Limit		Reliability	
	Software	IC3	Software	IC3
Initial IRI (m/km)	1	1.5	-	-
Terminal IRI (m/km)	2.7	4	90	90
AC top down fatigue cracking (m/km)	378.8	378.8		
AC bottom up fatigue cracking (%)	25	50		
AC thermal cracking (m/km)	189.4	189.4		
Permanent deformation-total rutting (mm)	19	20		
Permanent deformation- AC only (mm)	6	10		
Chemically stabilized layer- fatigue fracture (%)	25	25		

3.6 Pavement Structure Design

The final proposed pavement structure design for IC3 pavement (IP, 2005) used by IP is shown in Table 3-3.

Table 3-3: Proposed pavement structure design for IC3 by the agency

Layer Type	Thickness H(m)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous Mixture	0.05	4500	0.35
Binder in Bituminous Macadam	0.09	4500	0.35
Base in Bituminous Macadam	0.11	4500	0.35
Base in ABGE[1]	0.20	300	0.35
Soil Cement Subgrade	0.20	2000	0.25
Soil Foundation	Semi infinite	60	0.4

Based on available ME Design layer options and some lack of data for MB base in IP documents, the final structure for IC3 projects is applied in ME Design. Due to the lack of data in material characteristic information for base layer, the binder layer data is used for this layer. As for the subgrade, based on software facilities "*A minimum of two unbound layers are required to correctly model subgrade moisture and drainage*". Accordingly, the

[1] "Agregado Britado de Granulometria Extensa"

last subgrade layer is divided into two layers which are now two subgrade layers with the same properties. The Table 3-4 shows the applied structure design in ME Design.

Table 3-4: Applied pavement structure design for IC3 in ME Design Software

Layer Type	Thickness H(m)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous Mixture	0.05	4500	0.35
Binder + Base in Bituminous Macadam	0.20	4500	0.35
Sandwich Granular Base in ABGE	0.20	300	0.35
Chemically Stabilized Sub grade 1: Soil Cement	0.20	2000	0.25
Sub grade 2: Soil Foundation	0.30	60	0.40
Sub grade 3: Soil Foundation	Semi infinite	60	0.40

3.7 Climatic Data

Regarding the climatic data for IC3 road, the virtual station has been created by ME design which is the combination of Coimbra and Lisboa (two regions near the road). The other climatic data includes longitude, latitude, elevation, and depth of water table (for virtual station). The coordination values for IC3 are obtained by supposing the virtual station settled in Tomar, approximately in the middle of two regions of Coimbra and Lisboa. The coordination values for this virtual station are (Free Map Tools, 2018):

- longitude : -8.4167 Decimal degree;
- latitude: 39.6 Decimal degree;
- elevation: 75 m.

The depth of water table applied in ME Design is the depth below the final pavement surface. There are two options for entering depth of water table in ME Design: an annual depth to the water table or seasonal water table depths. The more precise option is using seasonal water table depths while historical data are available to determine the seasonal fluctuations of the water table depth. However, the other option (average annual depth) can always be used. The depth of water table during construction time is needed for the ME Design. So if a subsurface drainage system is used to lower the water table, that lower depth should be used in the program, not the depth measured during the subsurface investigation (AASHTO, 2015b). The depth of water table provided for IC3 in ME Design software is considered average annual depth and default value (10 m). The Figure 3-2 illustrates the climate section of AASHTOWare Pavement ME Design.

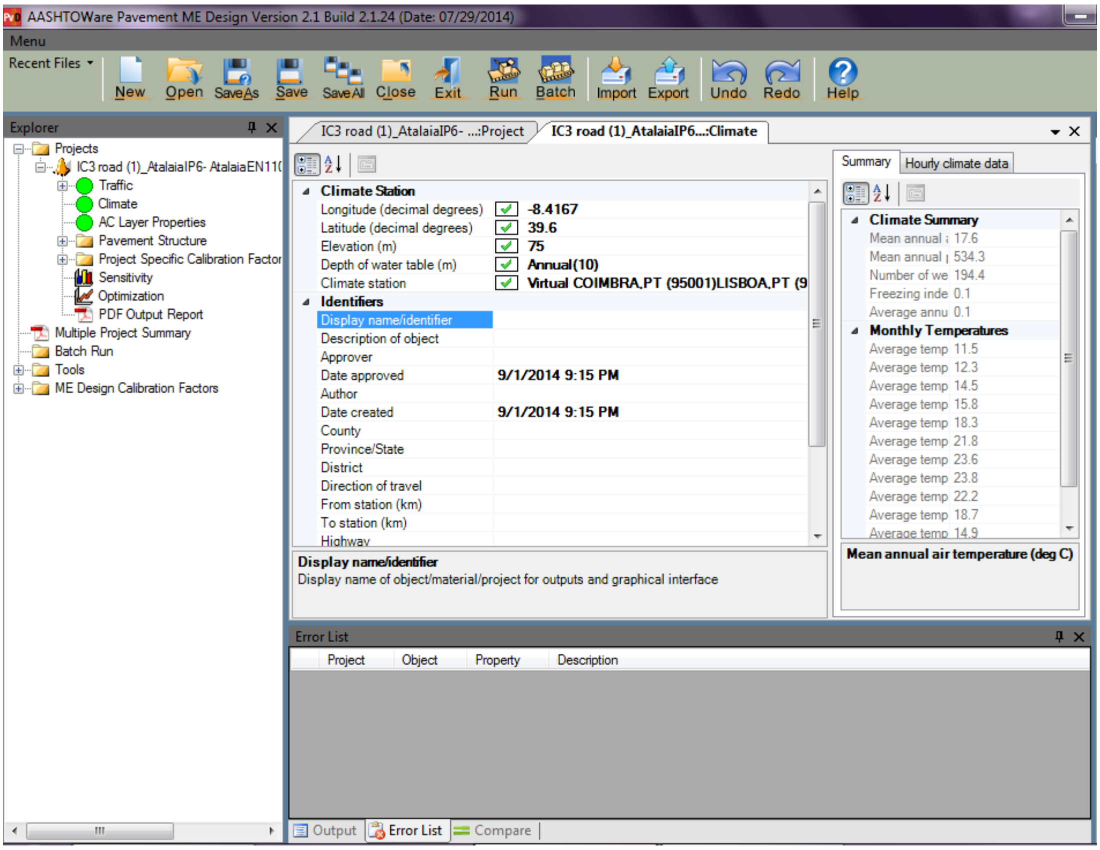


Figure 3-2: ME Design software illustration - Climate

3.8 Traffic Data

3.8.1 Site Specific Traffic Inputs

The following information are the required truck-traffic volume data for the base year. As it is mentioned earlier in chapter 2, all these inputs except vehicle operational speed are used to compute the number of trucks in the design lane. The definition of inputs is already mentioned in chapter2.

1. Two-way AADTT: The values for AADTT are calculated by using the initial values obtained from traffic simulation model.
2. Number of lanes: For IC3, there are two lanes in each direction, so the number of lanes in the design direction is two.
3. Percent trucks in design direction or truck direction distribution factor (DDF): For IC3 road, a default value (Level 3) of 50 percent has been provided.

4. Percent trucks in design lane or truck lane distribution factor (LDF): The IC3 road is two lanes, two directions road. So, the LDF is equal to 0.9.
5. Operational speed (kph): Regarding IC3, the operational speed of 90 kph has been applied.

3.8.2 Traffic Volume Adjustment Factors

The traffic volume adjustment factors are already defined in chapter 2. These factors mainly obtain through the WIM traffic data. A research by Highway Research Center Auburn University indicated that a 18% variation in truck factors (i.e., MAF, HDF, LDF, DDF which will be explained in the following) creates less than 1.27 cm variation in the depth of pavement, which is not considered practically significant. In other words, if the daily truck factors for the seven days of the week and the monthly truck factors for the 12 months of the year varied less than 18% from the average yearly truck factor (ME Design default distribution), it is not necessary to calculate truck factors by day of week or month of the year. In this case, the average truck factor should be obtained based on the lightest and heaviest days and months to guarantee that it would be representative of all days and months (Turochy et al., 2005).

As for the Portuguese conditions, it is suggested to collect and study the WIM data to find if for flexible pavement, the daily truck factors for the seven days of the week and the monthly truck factors for the 12 months of the year in Portugal varied less than 18% from the average yearly truck factor.

The following part describes the justification of these factors for IC3 road in lack of WIM data:

1. Monthly Adjustment Factors (MAF): Since there is no information available to calculate the MAF, it is assumed 1.0 for all months for all vehicle classes (recommended by MEPDG).
2. Vehicle Class Distribution: or the average normalized truck-volume distribution represents the percentage of each truck class within the truck traffic distribution. The default distribution in ME Design is chosen for IC3 road (due to the lack of data

in this part). The selected distribution for IC3 illustrated in Figure 2-4 is the 9th TTC (Truck Traffic Classification) group from 17 TTC groups provided in ME Design. The TTC group of 9 represents “mixed truck traffic with about equal percentages of single-unit and single-trailer trucks” with traffic stream of “Low to Moderate (>2%)” and “Low to None (<2%)” multi trailer. The distribution values depend on road category and the value calculated for each class and obtained as:

$$VCDF_i = \frac{AADTT\ for\ class_i}{Total\ AADTT} \times 100 \qquad Equation\ 3\text{-}1$$

Where

$VCDF_i$: $Vehicle\ Class\ Distribution\ for\ class\ i$

3. Truck Hourly Distribution Factors (HDF): The HDF did not apply to IC3 pavement design. It was not required for the design type.
4. Traffic Growth Factors: The growth function for IC3 road is assumed linear for all truck classes. Regarding the growth rate, the same value of 1.1 is used for all truck classes. The calculation of growth rate for IC3 is explained in the following.

Since there was not enough data for IC3 traffic for the opening year, the average growth factor obtained by traffic simulation model (for the years 2013-2033) provided by the agency has been used in ME Design. So, the growth factor for each section is calculated as follows:

$$Traffic\ Growth\ Rate\ = \left(\left(\left(\frac{AADT_{FY}}{AADT_{IY}}\right)^{(FY-IY)}\right) - 1\right) \times 100 \qquad Equation\ 3\text{-}2$$

Where

$AADT_{FY} = Aannual\ Average\ Daily\ Traffic\ for\ the\ final\ year$

$AADT_{IY} = Aannual\ Average\ Daily\ Traffic\ for\ the\ initial\ year$

$FY - IY = Difference\ between\ the\ final\ and\ initial\ year$

The Table 3-5 shows the obtained AADT by simulation of traffic for 20 years in the following page.

Table 3-5: Annual Average Daily Traffic

2033		2028		2023		2018		2013		Opening Year	Destination	Origin	ITIN
%-HV	AADT	%-HV	AADT	%-HV	AADT	%-HV	AADT	%-HV[1]	AADT				
16.8	7558	16.9	7025	17.1	6628	17.3	6278	14.8	6099	2006	Atalaia (EN110)	Atalaia (IP6)	IC3
15.1	5152	15.2	4789	15.4	4518	15.5	4279	13.5	4156	2006	Asseiceira	Atalaia (EN110)	IC3
14.2	5494	14.3	5107	14.4	4817	14.6	4562	12.8	4431	2006	Santa Cita (EN110)	Asseiceira	IC3
20.5	5375	20.7	4998	20.9	4717	21.1	4469	17.5	4342	2013	Valdonas	Santa Cita (EN110)	IC3
25	4075	25.3	3790	25.5	3578	25.8	3391	20.6	3295	2013	A13/IC9	Valdonas	IC3
19.5	3419	19.7	3179	19.9	3000	20.1	2842	16.8	2761	2012	Alviobeira	A13/IC9	IC3

[1] Heavy vehicles

Accordingly, the Table 3-6 shows the growth factor and the related AADTT for each section.

Table 3-6: Annual Average Daily Truck Traffic for opening year

AADTT for Opening Year	% Heavy vehicles	AADT for Opening Year	Average Growth rate	Growth rate 2023-2033	Growth rate 2013-2023	Opening Year	Destination	Origin	ITIN
837	14.8	5658	1.078	1.322	0.835	2006	Atalaia (EN110)	Atalaia (IP6)	IC3
520	13.5	3855	1.080	1.322	0.839	2006	Asseiceira	Atalaia (EN110)	IC3
526	12.8	4110	1.081	1.324	0.839	2006	Santa Cita (EN110)	Asseiceira	IC3
760	17.5	4342	1.073	1.314	0.832	1/3/2013	Valdonas	Santa Cita (EN110)	IC3
679	20.6	3295	1.068	1.309	0.827	1/3/2013	A13/IC9	Valdonas	IC3
464	16.8	2761	1.075	1.316	0.834	21/12/12	Alviobeira	A13/IC9	IC3

It should be mentioned that the vehicle classification system adopted by Portugal is different than the one used by U.S.

According to Federal Highway Administration (FHWA) classification system, there are 13 classes of vehicles as shown in Figure 3-3 (Randall, 2012).

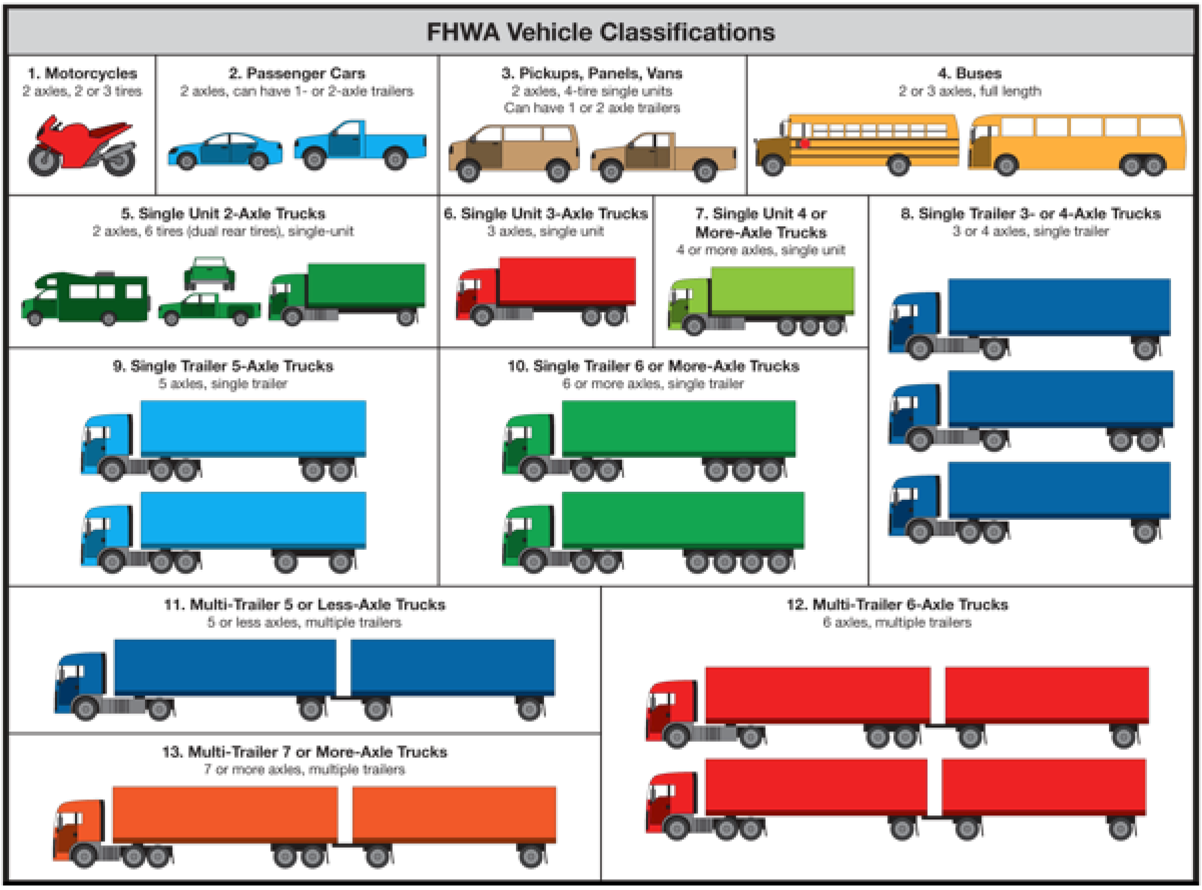


Figure 3-3: FHWA 13-Category scheme for vehicle classifications(Randall, 2012).

Based on MACOPAV, six classes of F, G, H, I, J, K among the eleven classes are introduced as the heavy classes effective in pavement design as it is shown in the Table 3-7 (Junta Autónoma de Estradas (JAE), 1995).

Table 3-7: Vehicle classification adopted by Portugal (Cordeiro, 2010)

Classes	Vehicle type
C	Motorcycles
D	Light passenger cars
E	Light commercial cars
F	Heavy commercial cars
G	Trucks with trailer
H	Trucks without trailer
J	Especial
I	Heavy Passenger cars (Buses)

As for the reference to convert the traffic counting in Portugal to FHWA classification system, the Table 3-8 can be used.

Table 3-8: Comparison of vehicle classes adopted by US and Portugal (Cordeiro, 2010)

FHWA Classification	Portuguese Classification	
Class	Class	Subclass
Class 4	Class I	I_1
		I_2
		I_3
Class 5	Class F	F_1
Class 6	Class F	F_2
		F_3
Class 7	Class F	F_4
		F_5
Class 8	Class G	G_1
	Class H	H_1
		H_2
		H_3
Class 9	Class G	G_2
		G_3
	Class H	H_4
		H_5
Class 10	Class H	H_6
Classes 11, 12 and 13	Class G	G_4
	Class H	H_7

3.8.3 Axle Load Distribution Factors

Due to the lack of WIM data for the selected roads, the default axle load distribution provided by ME Design is applied for the ME projects. The chapter 4 describes the procedure of modifying AADTT in order to have the applied traffic similar to Portuguese project while the default configuration for the axle load distribution is used by ME Design.

3.8.4 General Traffic Inputs

The following general traffic inputs are justified for IC3 road:

1. Mean Wheel Location: A default (Level 3) mean wheel location of 45.72 cm is used for IC3 road in the ME Design.
2. Traffic Wander Standard Deviation: A default (Level 3) mean value of 25.4 cm is applied for IC3 road in the ME Design.

3. Design Lane Width: The value for IC3 is 3.75m.
4. Number of Axle Types per Truck Class: Default (Level 3) estimations of the number of axle types per truck class provided in the ME design is used for IC3 road.

The Table 3-9 shows the summary of required traffic Inputs for IC3 road. In the table, input values with software default values are labeled as Default.

Table 3-9: The summary of required traffic Inputs for IC3 road

ME Field				Input Value
Site Specific Traffic Inputs	Average Annual Daily Truck Traffic (AADTT)	Year		Refer to Table 3-6
		Initial two-way AADTT		
	Number of Lanes in Design Direction			2
	Percent of Trucks in Design Direction			50
	Percent of Trucks in Design Lane			90
	Operational Speed			90
Traffic Volume Adjustment Factors		Monthly Adjustment		Default
		Road Category		Highway
		AADTT Distribution by Vehicle Class (%)		Default
		Hourly Truck Traffic Distribution		Default
		Traffic Growth factors		Table 3-6 / linear
Axle Load Distribution Factors		Axle Load Distribution		Default
		Axle Type	Single	Default
			Tandem	Default
			Tridem	Default
			Quad	Default
		Distribution Type	Normal	Default
			Cumulative	Default
General Traffic Inputs	Lateral traffic Wander	Mean Wheel Location (cm)		Default
		Traffic wander standard deviation(cm)		Default
		Design lane width(m)		3.75 m
	Number axles/Truck			Default
	Axle Configuration	Avg. axle width (edge-to-edge outside dimension) (m)		Default
		Dual tire spacing (cm)		Default
		Tire pressure (kPa)		Default
		Axle spacing (cm)	Tandem	Default
			Tridem	Default
			Quad	Default
	Wheelbase	Average axle spacing (m)	Short	Default
			Medium	Default
			Long	Default
		Percent of trucks	Short	Default
			Medium	Default
			Long	Default

3.9 Material Properties

The material properties of the layers are obtained from control quality documents for IC3 (prepared and accessed by IP). The average values (for the selected sections) are applied for obtaining material properties of IC3 road. For example, the Table 3-10 shows the material properties characterized for surface layer-IC3 pavement.

Table 3-10: Structure/Material input parameters (Flexible Pavement) for IC3 (Part1)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Necessity	Software Value	Level 1	Level 2	Level 3	
Drainage and Surface Properties						
Surface Shortwave Absorptivity (as)	SDA[1]	0.85			0.85	-
Is endurance limit applied?	SDA	False			False	-
Endurance limit (microstrain)	SDA	100			100	-
Layer interface	SDA	1 for all layers	0.2 for Layers 3 & 4 1 for other layers			-
Layers (Individual Layer Strength Properties)						
Layer (Surface Layer in BB)						
Layer thickness	Required		50			mm
Mixture Volumetrics						
Unit Weight[2]	SDA	2400	2420			Kgf/m^3
Bitumen Percentage-Pb			5.2			%
Bitumen Content-tb			5.49			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (by volume) Vb	SDA	11.6	12.2			%
Air Void at time of construction Vv[3]	SDA	7	2.8			%
Theoretical specific gravity of the mix Gt			2.49			-
Bulk or actual specific gravity of the mix Gm			2.42			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA		No			-

[1] Software default available

[2] Use as-constructed mix type specific values available from previous construction records. (IP Doc. _March 2006)

[3] Use as-constructed mix type specific values available from previous construction records.

The Table 3-11 shows the material properties characterized for surface layer-IC3 pavement.

Table 3-11: Structure/Material input parameters (Flexible Pavement) for IC3 (Part2)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Necessity	Software Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation, Sieve % Passing 19 mm 9.5 mm 4.75 mm 0.075 mm	Required		(IP Doc. _March 2006) 100.00 73.40 51.00 7.40			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA[1]	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade[2]	Required			40/50		
Indirect Tensile strength at -10 ºC, TS	SDA	DoMT[3]			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16			1.16	Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963			963	joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006			9e-006	mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06			5e-06	mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	15			%

[1] Software default available
[2] 35/50 (IP Doc. _Jan 2006)
[3] Depends on material type

The values provided in the tables are the summary of excel calculations for the selected sections. For example, from km 4+400 until km 5+400, the value for each input is obtained from IP documents (based on availability) by an interval of km 0+025 and the average value of that input is then calculated. Since there are two selected sections for IC3 (km 4+400 to km 5+400 and km 5+800 to km 7+000), the average of final obtained values for two sections is used for material properties.

For the briefness in main text, only two tables are provided as examples to show the material properties characterized for surface layer-IC3 pavement. The material properties for other layers of IC3 pavement are provided in Appendix 2

3.10 Summary

The main objective of chapter 3 was to prepare the required inputs for ME Design projects in order to address the challenges in making local databases for ME Design (e.g. availability of data for materials characterization, availability of WIM data for traffic, developing a more comprehensive library of climate data for Portugal). This objective is illustrated based on three different Portuguese roads (which are already designed by SHELL method) that were selected for data preparation of ME Design.

The considerations in choosing the roads (IC3, IP6, EN254) in this study are:

- Data availability for design and construction (access to documents).
- To represent different demands and applied traffic based on their hierarchical road network level.
- To illustrate the variability of the results for Portuguese roads.

After choosing the roads, some sections of the roads are selected to obtain the material properties. Some factors in choosing the sections are considered that among them, the availably of data in quality control data and pavement structure design documents had the priority of selection.

Regarding the justification of performance criteria based on Portuguese pavement conditions, the process of justification is discussed in chapter 4.

As for the availably of climate data for Portugal, it should be noted that the data are currently available for other regions through the related website (Federal Highway Administration, 2018). However, due to the lack of climate data for the selected roads at the time of analysis, the data were obtained by interpolating climatic data from available nearby weather stations (Lisbon, Porto, Coimbra, Beja).

The Table 3-12 shows the summary of data sources and considerations related to data preparation.

Table 3-12: The summary of data sources and availability

Required data for creating ME projects	Data Source	Availability	Modification	Lack of data
Specific site subgrade support	IP[1]	Mainly paper based documents	• The data is extracted for the selected road sections • the outliers are discarded • the average values are implemented in ME.	Software default or PT[2] default is used
Material properties	IP	Mainly paper based documents	Same as subgrade support	Software default or PT default
Traffic loading	IP	Traffic simulation model (years 2013-2033) for AADTT and growth rate	Same as subgrade support	Software default or PT default
Environmental conditions	MERRA tools	Climate data only generated for Lisbon, Coimbra, Porto and Beja at the time of analysis	• US unit to metric unit • format errors in the *.hcd files • 4 PT weather stations are added in ME	Virtual stations/ Software default

As it is mentioned earlier in this chapter, the required traffic data are composed of three main categories: site specific traffic inputs; WIM traffic data; general traffic inputs. A

[1] Portuguese Road Administration
[2] Portuguese

considerable part of traffic data from categories 2 and 3 (such as traffic volume adjustment factors, axle load distribution factors, number of axles/truck) can be obtained through the WIM data. So, it is suggested to collect and study the WIM data for Portugal.

The WIM data can also be used to find if the daily truck factors for the seven days of the week and the monthly truck factors for the 12 months of the year in Portugal varied less than 18% from the average yearly truck factor. If it would be the case, then it is not necessary to calculate truck factors by day of week or month of the year. In this case, the average truck factor should be obtained based on the lightest and heaviest days and months to guarantee that it would be representative of all days and months (Turochy et al., 2005).

4 Evaluation and Adjustment of External factors and Design Criteria

4.1 Introduction

One of the main tasks to address the challenges before adopting a new framework (ME Design) is to compare the damage results obtained by the ME method against those from the current Portuguese pavement design (SHELL method). For this comparison, addressing and justifying the main external factors and performance criteria is required to propose a validation framework for ME design method based on Portuguese conditions.

The external factors affecting the pavement performance are traffic, the environment, and the interaction of the two. The most significant environmental factors affecting pavement performance are pavement temperature and moisture content (Ongel & Harvey, 2004).

The ME Design software makes use of the enhanced integrated climatic model (EICM) to forecast future pavement temperatures and moisture contents as a function of historical weather records. These predictions are critical to the models used in the software to adjust the asphalt concrete modulus as a function of temperature and the unbound materials as a function of moisture content (Timm et al., 2014).

This validation framework can be considered as a benchmark for damage comparison between two methodologies (SHELL and MEPDG). For this reason, it is important to review and verify the following steps:

1. The value of performance criteria in ME Design will be studied and justified based on the Portuguese conditions and the reasons to choose the values will be discussed in this step.
2. Comparison between the value and effect of "service temperature" used in SHELL Method and pavement temperature obtained by EICM[1] in ME method, to set a similar background and condition for damage comparison will be accomplished in this step.

[1] Enhanced Integrated Climatic Model

3. Comparison between the value and effect of “water (moisture) content of the unbound layer and soil used in SHELL method” (effective in dynamic modulus of subgrade) and "water (moisture) content of soil used in ME method" (which depends on: water and air pressure (suction), soil type (specific surface, clay content), porosity and temperature among others.) will be accomplished in this step.
4. Aggression factor (α) calculation process and AADTT modification to apply the same "Accumulated Number of 80 kN Standard Axle Passages" used in SHELL method to ME approach.

In this chapter, three selected roads of IC3, IP6 and EN254 described in previous chapters are selected for comparison of both methods (SHELL and ME).

4.2 Justification of Performance Criteria

4.2.1 Analysis Parameters for ME Design

The Table 4-1 shows the default values of performance criteria recommended by ME Design. It is necessary to ensure that the performance criteria used by ME Design are well adjusted to apply for the adequacy check of the pavement design. This section is a study on the justification of performance criteria for Portuguese conditions in ME Design and the reasons to choose the values. In this section, the performance criteria will be justified for the three selected roads that have been mentioned in the previous chapter: IC3, IP6 and EN254.

Table 4-1: Analysis Parameters - Default values for ME Design V 2.1

Performance Criteria	Limit	Reliability
Initial IRI (m/km)	1	-
Terminal IRI (m/km)	2.7	90
AC top down fatigue cracking (m/km)	378.8	90
AC bottom up fatigue cracking (%)	25	90
AC thermal cracking (m/km)	189.4	90
Permanent deformation-total rutting (mm)	19	90
Permanent deformation- AC only (mm)	6	90
Chemically stabilized layer- fatigue fracture (%)	25	90

4.2.2 Reliability Level

One of the main analysis parameter of ME Design software is reliability level for each performance criteria. Based on MEPDG manual, there are some recommendations for choosing reliability level(AASHTO, 2015b):

- The same reliability for all performance indicators is recommended.
- For nearly all projects, the designer will require a reliability higher than 50 percent that the design will meet the performance criteria over the design life. In other words, the amount of reliability level directly relates to the importance of the project in terms of consequences of failure.
- The design reliability should be selected in balance with the performance criteria otherwise could be costly and/or impossible to obtain the desired design.

Considering these conditions, MEPDG manual provided (AASHTO, 2015b) and suggested the values in Table 4-2 for reliability levels based on the functional classification of roads that are believed to be in balance with the performance criteria.

Table 4-2: Levels of Reliability for Different Classifications of the Roadway (AASHTO, 2015)

Functional Classification	**Level of Reliability**	
	Urban	**Rural**
Interstate/Freeways/1st level national network	95	95
Principal Arterials/2nd level national network	90	85
Collectors/3rd level national network	80	75
Local	75	70

For this study and the selected road sections, the three levels of the national road network are introduced:

- 1st level national network: or “Itinerários Principais” (Principal roads) which are mainly freeways in urban and rural areas.
- 2nd level national network: or “Itinerários Complementares” (Complementary roads) which are mostly freeways in urban areas as well as rural areas, however, it could be a two-lane road where it is possible due to its demand (low to medium demand). This two-lane road has comfortable geometric characteristics (e.g. wide width) which is simply recognizable in compare to 3rd level.

- 3rd level national network: or “Estradas Nacionais” and “Estradas Regionais” (National and regional roads) that are constituted by two-lane roads in rural areas and are main distributors in urban areas.

Based on Table 4-2 and the above-mentioned categorization, the following reliability levels for the three selected roads are considered:

- **IC3:** Complementary roads based on current Portuguese road classification (Plano Rodoviário Nacional or PRN 2000) and it is part of A13 (*Autoestrada or freeway)*. So, it is considered as rural freeway. **R= 95**
- **IP6:** Principal roads based on PRN 2000, however, the cross-section of the road that is used in this study has two lanes per direction, so the road is considered rural freeway. **R= 95**
- **EN254:** National roads based on PRN 1945 and considered as rural collector. **R=75**

In this study, for more simplification, the same reliability level (**R=90**) is considered for all performance criteria for the three roads as reference for the designs.

4.2.3 Initial IRI (International Roughness Index)

The initial IRI defines the as-constructed roughness of the pavement immediately before opening of road to the traffic and has a significant impact on the long-term ride quality of the pavement (ARA, 2004d). Based on Mechanistic–Empirical Pavement Design Guide, the initial IRI value could be taken from previous years' construction acceptance records, if available (AASHTO, 2008). The recommended values by Estradas de Portugal for new Portuguese pavement in different percentage of road section are shown in Table 4-3.

Table 4-3: The admissible IRIs calculated by 100-meter sections in flexible pavements (IP, 2014).

Requirement		**Unit**	**Utilization**		
Specificities of use			**Percentage of under construction stretch**		
			50%	**80%**	**100%**
Values of admissible IRI	Surface layer	m/km	≤1.5	≤2.5	≤3.0

The table shows that the majority of new constructed pavement in Portugal (about 50% of the new section) should have the IRI less than 1.5m/km while only 30% of the road section could have the IRI between 1.5 and 2.5 m/km and the IRI for other 20 % of the road section could be between 2.5 and 3 m/km (IP, 2014).

The following values of initial IRI are extracted from IP documents for the three selected road sections:

- **IC3**: Based on IP document for IC3 (IP, 2007), the majority of IRI distribution in both direction of IC3 is in the class of $\mathbf{1.0} < \mathbf{IRI} \leq \mathbf{1.5}$ m/km (about 68.75 %).
- **IP6**: No available IRI distribution data for IP6.
- **EN254**: Based on IP document for EN254 (IP, 2004) , the majority of IRI distribution in both direction of EN254 is in the class of $\mathbf{1.0} < \mathbf{IRI} \leq \mathbf{1.5}$ m/km (about 65.83 %) and the average IRI for this road is 1.5 m/km.

In this study, for more simplification and based on the majority of IRI distribution for two roads of IC3 and EN254 as well as the other Portuguese roads based on Table 4-3, the initial IRI for three roads is considered **1.5 m/km** which can mainly satisfy the initial condition for new pavements at any national network road sections.

4.2.4 Terminal IRI

This performance criterion represents the smoothness of road at the end of design life. So, the critical value is unacceptable ride quality interpreted by highway users. IRI increases over time as a function of longitudinal cracking, transverse cracking, alligator cracking, and total rutting along with climate and subgrade factors (AASHTO, 2008).

In this study, the terminal IRI is justified based on the correlation for quality assessment of pavement (QI correlation). The $Equation$ 4-1 is an impression of QI correlation developed for Portugal, based on the AASHO Road Test and the modified PSI (present serviceability index) equation by the NDOT's PMS (Pavement Management System of the State of Nevada) (Morgado et al., 2012; Sebaaly et al., 1996):

$$QI_t = 5\, e^{-0.0002099\, IRI_t} - 0.002139\, {R_t}^2 - 0.03\, (C_3)^{0.5} \qquad Equation\ 4\text{-}1$$

Where

QI_t: $Quality\ index\ in\ year\ t\ (range\ from\ 0\ to\ 5)$;

IRI_t: $International\ Roughness\ Index\ in\ year\ t\ (mm/km)$;

R_t: $Mean\ Rut\ depth\ in\ year\ t\ (mm)$;

$$C_3: Alligator\ cracking\ area \left(\frac{m^2}{100m^2}, in\ \%\right) = \frac{section\ width \times total\ lenght\ of\ cracking\ level\ 3}{section\ width \times total\ lenght} \times 100$$

In the $Equation$ 4-1, the QI ranges from 0 (very poor) to 5 (very good). In this sense, IP defined 3 quality classes in order to facilitate the reading of this index in the sections. The classes are (Morgado et al., 2012): Class I: Good (QI ≥ 3,5); Class II - Fair (2,5 ≤ QI < 3,5); Class III - Mediocre (QI < 2,5).

Based on the evaluation system of quality for Portugal (Sistema de Avaliação da Qualidade (SAQ), IP), the minimum acceptable value of QI for urban roads is 2.5 and for national road network could be 2.0 (Morgado et al., 2012; Picado-Santos et al., 2004, 2006). In this study, for more simplification, the same terminal quality index is considered for the selected roads. The justified value is 2.5 considering the two freeways on IC3 and IP6.

The Table 4-4 shows the example of calculated IRI values in different pavement situations based on QI correlation. The recommended values for the rut depth and the alligator cracking area are only used as the example values for different pavement situations and different level of quality to calculate the IRI.

Table 4-4: IRI calculation based on QI equation for five different pavement situation and QI

Pavement Condition	**Rut depth $R_t\ (mm)$**	**% of degraded area C_3**	QI_t	$IRI_t\ (\frac{m}{km})$
Good	0	0	3.5≤x≤5	0.0≤x≤1.7
Fair	20	50	2.5≤x<3.5	0.4<x≤1.6
Mediocre	22	80	0≤x<2.5	1.3<x≤6.4

As it is shown in the table, the IRI can vary from fair to mediocre between 1.6 m/km and 6.4 m/km. Accordingly, the terminal IRI can be supposed the average value of this range (1.6 to 6.4) which is about 4 m/km. In this study for more simplification, the same terminal IRI (4 m/km) is applied for the selected road sections.

4.2.5 AC Top-Down Fatigue Cracking (Longitudinal Fatigue Cracking)

The performance criterion for top-down fatigue cracking is defined as the maximum allowable length of longitudinal cracking per kilometer of pavement that is permitted to occur over the design period. So, a critical value is reached when longitudinal cracking accelerates and closing the lanes and carrying out special repairs is needed. There are several factors that can cause and increase this distress during pavement's design life (climatic conditions, traffic, ageing, structure and construction quality). Some major factors contributing to top-down cracking that are consensually addressed by researchers are(Freitas et al., 2007; Harmelink et al., 2008; Ozer et al., 2011; Rolt, 2000):

- non-uniform tire contact stresses with transverse components;
- thermal loads;
- stiffness gradients due to ageing of binder;
- poor construction quality (e.g., segregation and compaction methods).

Top-down fatigue cracking in Portugal, as in many other temperate-climate countries, is an important surface distress in thick pavements (more than 150 mm AC pavement) (Freitas et al., 2003). On the other hand, the top-down fatigue cracking model in ME Design is recently developed in NCHRP 1-52 and the updated version of model was not available at the time of justification of criterion (Lytton et al., 2018). Accordingly, due to the situation of this model for the ME Design, the limit for this criterion is considered default threshold value in the software, which is 378.8 m/km.

4.2.6 AC Bottom-Up Fatigue Cracking (Alligator Cracking)

The performance criterion for bottom-up fatigue cracking is defined as the maximum area of alligator cracking expressed as a percentage of the total lane area that is permitted to occur over the design period. Basically, while the pavement and HMA layer deflects under wheel loads repeatedly, results in tensile strains and stresses at the bottom of the layer and accordingly this repeated bending can cause cracks to initiate at the bottom of the layer and then propagate to the surface. However, this mechanism can be accelerated by several reasons (ARA, 2004d):

- The thickness or mechanical properties of flexible layers are not appropriate for the magnitude and repetitions of the wheel loads (e.g. thin or weak layer).
- Having soft spots or areas in unbound aggregate base materials or in the subgrade soil caused by inadequate compaction or increases in moisture contents and/or extremely high ground water table (GWT).

Based on an assumption adopted by MEPDG, when the damage is 100%, cracked area is equal to 50% (Schwartz & Carvalho, 2007). It seems, the conditions for this distress are not critical in the selected roads (there is no inadequate thickness or low compaction or insufficient mechanical properties in compare with given traffic and climate conditions). In this study, considering the condition of Portuguese pavements and for more generalization, the criterion of B-U fatigue cracking is justified to be 50% of the total lane area as the maximum acceptable value.

4.2.7 Total Permanent Deformation (Rutting)

The performance criterion for total permanent deformation is defined in terms of the maximum rut depth in the wheel path. There are several factors that highly affects the rutting; AC layer thickness and AC layer modulus, traffic loading as well as the environment at the design site (ARA, 2004d). Based on the usual indication of control quality plans for the regulatory relation between the state and the motorways' concessionaires, the limit of 20mm was accepted for this criterion in this study.

4.2.8 AC Permanent Deformation

This criterion indicates the limit of rutting in asphalt layer. AC rutting is a proportion of total rutting considering the thickness and stiffness of the other layers. In this study, this criterion is justified based on 50% of total rutting criterion (i.e., 10 mm). The observation of the results of this distress by variation of AADTT (from low traffic levels to high traffic levels to include a wide range of traffic volumes) for the three roads can confirm this justification as a proper value for design. Table 4-5 shows AASHTOWare Pavement ME Design predicted AC rutting for varying truck volumes (for AADTT from 500 to 2000) for selected roads.

Table 4-5: ME Design outputs of AC rutting for varying truck volumes (Initial IRI = 1.5 m/km)

Road name		EN254	IP6	IC3	IC3	IP6	IC3	IP6	IC3	IP6
Design AADTT		~500	~500	~500	~1200	~1200	~1500	~1500	~2000	~2000
Surface/Binder Layer Thicknesses (mm)		50/100	120/160	250	250	120/160	250	120/160	250	120/160
Surface/Binder Dynamic Modulus E(MPa)		3500/3800	4200/4600	4500	4500	4200/4600	4500	4200/4600	4500	4200/4600
AC Layer Rutting (mm)	**Target**	10	10	10	10	10	10	10	10	10
	Predicted	7.84	4.58	5.41	8.1	6.81	8.83	8.83	10	8.52

4.2.9 AC Thermal Cracking

The performance criterion for thermal cracking is defined as the maximum length of transverse cracking per kilometer of pavement that is permitted to occur over the design period. Thermal cracking is a non-wheel load related cracking which is mainly caused due to cold temperature or thermal cycling (ARA, 2004d). Considering Portugal climate condition (temperate climate), this criterion is not critical for the selected roads and can be considered the suggested default value by ME Design (189.4 m/km).

4.2.10 Chemically Stabilized Layer - Fatigue Fracture

Chemically stabilized layers are high quality base materials that can support the upper pavement layers. Fatigue cracking (in chemically stabilized layers) can be created under repeating traffic loading, however having a good mixture design, structural design, and construction practices helps to minimize fatigue fracture. The effect of this fatigue cracking is the reduction of provided support to the upper pavement layers and consequently having surface distresses, especially top-down and bottom-up fatigue cracking in the asphalt surface layers. When these chemically stabilized layers are not directly located under HMA layer and are located deeper in the pavement structure (under other structural layers such as a base or sub-base course), they can be considered as constant modulus materials that are moisture insensitive. So, the effect of fatigue cracking in these layers on the HMA layer is insignificant (ARA, 2004d). In this study, IC3 pavement among the three selected roads contains a chemically stabilized layer in the structure which is a soil-cement subgrade located between sandwich granular base and soil foundation (is not directly located under AC layer). Considering the situation of this layer in the structure, the default value used by ME Design software for fatigue fracture criterion (i.e., 25 %) will be applied for this study.

4.2.11 Summary of Adjusted Limits for Performance Criteria

The Table 4-6 shows the summary of adjusted limits for performance criteria for the three selected roads in ME Design. The reliability is considered the same (90) for the three roads for performance criteria.

Table 4-6: Default values and recommended PT values for performance criteria in ME design

Performance Criteria		Limit for 20 years		
		IC3	IP6	EN254
Initial IRI (m/km)	Software default	1	1	1
	Recommended for Portuguese condition	1.5	1.5	1.5
Terminal IRI (m/km)	Software default	2.7	2.7	2.7
	Recommended for Portuguese condition	4	4	4
AC top down fatigue cracking (m/km)	Software default	378.8	378.8	378.8
	Recommended for Portuguese condition	378.8	378.8	378.8
AC bottom up fatigue cracking (%)	Software default	25	25	25
	Recommended for Portuguese condition	50	50	50
AC thermal cracking (m/km)	Software default	189.4	189.4	189.4
	Recommended for Portuguese condition	189.4	189.4	189.4
Permanent deformation-total rutting (mm)	Software default	19	19	19
	Recommended for Portuguese condition	20	20	20
Permanent deformation- AC only (mm)	Software default	6	6	6
	Recommended for Portuguese condition	10	10	10
Chemically stabilized layer-fatigue fracture (%)	Software default	25	25	25
	Recommended for Portuguese condition	25	No stabilized layer	No stabilized layer

4.3 Pavement Temperature

4.3.1 Service Temperature by SHELL method

For specified mixture and loading conditions, the HMA temperature is the primary factor controlling modulus. One of the most worldwide known methods to consider the effect of temperature, and therefore the service temperature, is SHELL method, which sets the equivalent annual service temperature. This method considers the service temperature dependent on the thickness of the bituminous layers and the equivalent annual air temperature. In other words, the temperature represented by SHELL pavement design method as service temperature, obtains through a weighted mean annual air temperature, w-MAAT, which is derived from the weighted mean monthly air temperatures, w-MMAT, from a given location and is related to an effective asphalt temperature and thus to an effective asphalt stiffness (Shell, 1978).

The Figure 4-1 shows the diagrams for obtaining service temperature by SHELL method.

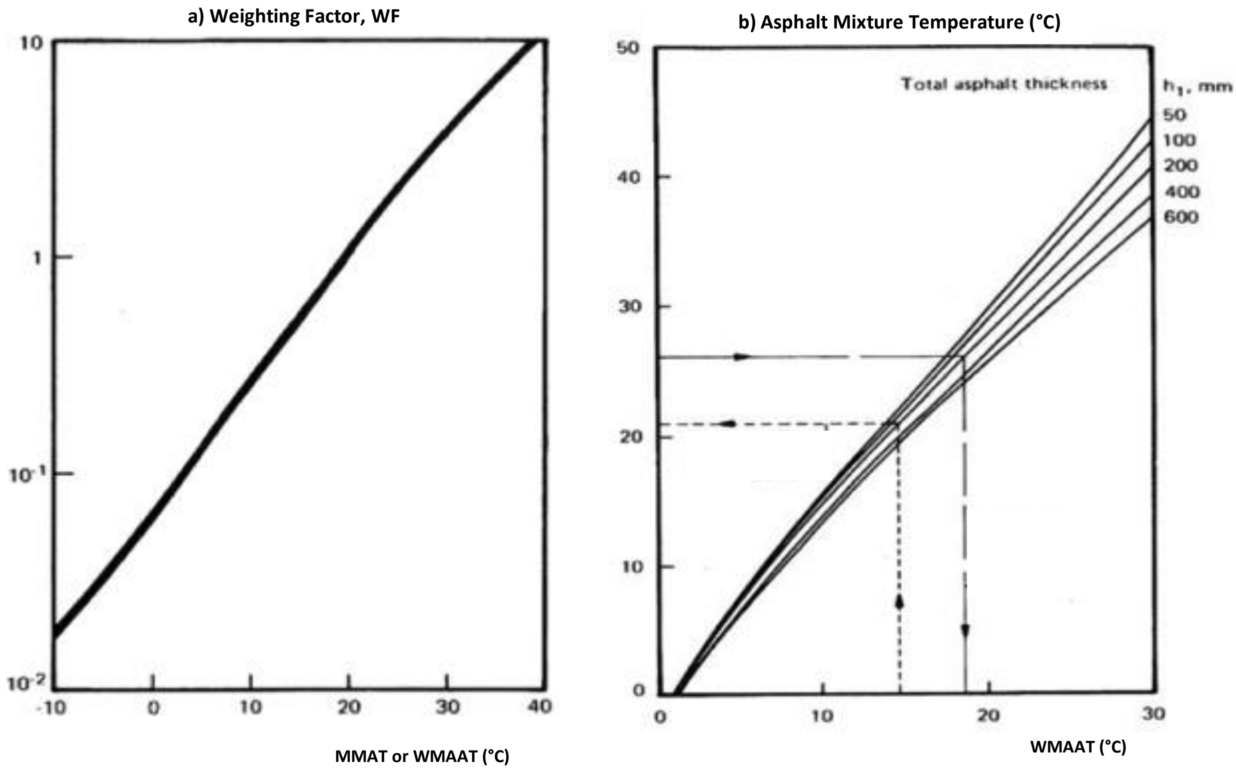


Figure 4-1: Diagrams for obtaining service temperature by SHELL method (Shell, 1978)

Diagram (a) in Figure 4-1 is weighting factor curve (the horizontal axle represents MMAT or w-MAAT (°C), the vertical axle represents weight factor) and diagram (b) is equivalent asphalt layer temperature (the horizontal axle represents the w-MAAT (°C), the vertical axle represents the equivalent asphalt layer temperature (°C), and the curves represents the asphalt layer thickness for 50, 100, 200, 400 and 600 (mm)).

Firstly, the mean monthly air temperatures are calculated and then the related weighting factor for each month can be obtained through the diagram (a). Accordingly, by having weighting factors for 12 months, the annual mean value of weighting factors can be calculated and applied in diagram (a) to obtain the weighted annual mean air temperature. Then the equivalent temperature of asphalt layer can be calculated based on the thickness of the asphalt layer and the weighted annual mean air temperature as it is shown in diagram (b) (Daibert, 2015; Shell, 1978).

4.3.2 Service temperature by EICM (applied in ME Design)

As for the MEPDG, pavement temperature is not constant in time or through depth. Temperatures throughout the pavement structure are dominated by the atmospheric conditions at the surface. The surface of the pavement is subject to more environmental effects and its temperature will fluctuate more than the temperature at the bottom of the structure. Factors affecting the top surface temperature of a pavement are: incoming shortwave radiation, reflected short-wave radiation, incoming long-wave radiation, outgoing long-wave radiation, convective heat transfer, condensation, evaporation, sublimation, precipitation; and the temperature of the layer(s) immediately beneath bound layer(s) (ARA, 2004a).

The MEPDG software subdivides the structural layers and foundation of the trial design into sublayers. The thickness of the sublayers is dependent on the material type, actual layer thickness, and depth within the pavement structure. The ICM (Integrated Climate Model) calculates the temperature and moisture conditions throughout the pavement structure on an hourly basis.

The temperatures in each HMA sublayer are combined into five quintiles (five successive groups, 20 percent each, of the calculated values) for each month of the analysis period for the load-related distresses. The frequency distribution of HMA temperatures using the ICM is assumed to be normally distributed. Figure 4-2 includes a graphical illustration of these temperature quintiles that are used in analyzing HMA mixtures.

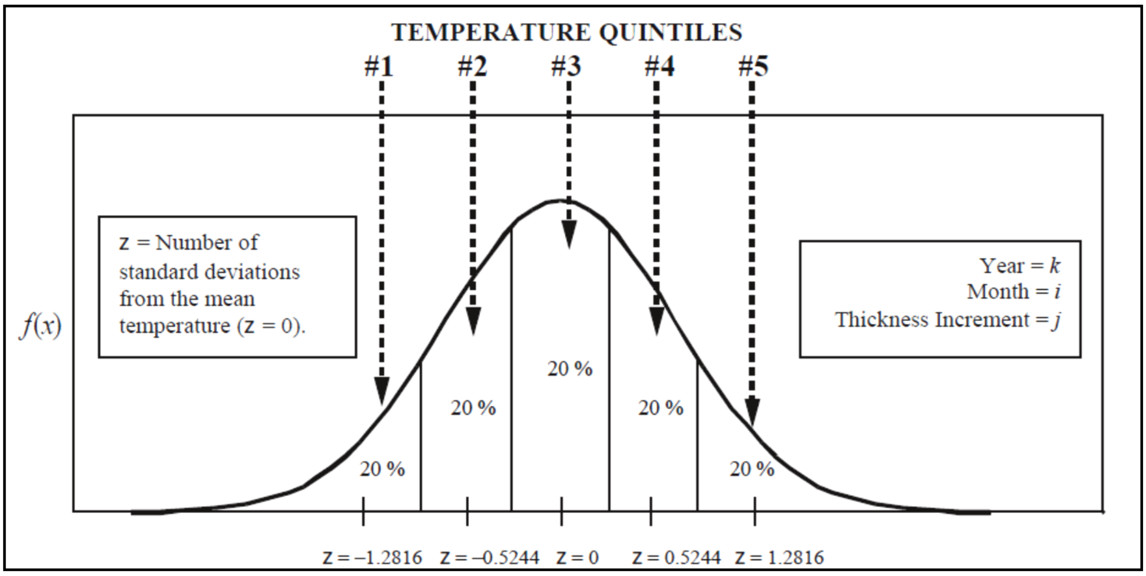


Figure 4-2: Temperature quantiles within each thickness increment of the HMA layers (AASHTO, 2015b)

The EICM provides 0.1 hours (6 minutes) temperature over the analysis period. While the EICM calculates temperature on a relatively small-time step of 0.1 hours, temperatures are output to the Design Guide summary files in two formats for flexible pavement analysis. One of them is used for rutting and fatigue analysis while the other is used for thermal fracture. As for rutting and fatigue, temperature values are required at the surface of the pavement structure and at mid-depth of all asphalt bound sub-layers. Since the first sub-layer for the asphalt is always 12.7 mm (0.5 inches), the temperatures are provided at 6.35 mm (0.25 inches) from the surface. No temperature information is generated for any other type of layer, as it is not required for the analysis (ARA, 2004a).

The average temperature within each quintile of a sublayer for each month is used to determine the dynamic modulus of that sublayer. The truck traffic is assumed to be equal within each of the five temperature quintiles. Thus, the flexible pavement procedure does not tie the hourly truck volumes directly to the hourly temperatures.

The ICM also calculates the temperatures within each unbound sublayer and determines the months when any sublayer is frozen. The resilient modulus of the frozen sublayers is then increased during the frozen period and decreased during the thaw weakening period. The ICM also calculates the average moisture content in the unbound layers for each month of the analysis period. The average monthly moisture content relative to the optimum moisture content is used to adjust the resilient modulus of each unbound sublayer for each month throughout the analysis period(AASHTO, 2015b).

4.3.3 Service Temperature by PETE (used to the application of the SHELL design method)

As it is mentioned earlier in this section, the damage comparison between SHELL method and ME method is only possible by considering the same background (service temperature) in both methods. As for the integrated climatic model in the AASHTOWare Pavement ME Design, the pavement temperature profile is internally calculated and used for calculating distresses. Therefore, it is not possible to directly compare the applied service temperature in ME with the equivalent service temperature used in SHELL projects.

For this study, the applied service temperature in SHELL method (to calculate dynamic modulus for BISAR) is considered the equivalent service temperature calculated based on a method developed for Portuguese conditions (Pavement Equivalent Temperature Model or PETE) (Picado-Santos, 2000). This method can represent a more realistic service temperature for AC layers in comparison with other approaches of computing service temperature such as SHELL, Illinois, Asphalt Institute. The PETE method uses simple meteorological data (mean monthly air temperature) and has a good accuracy for damage results in comparison with damage calculation by real temperatures. In other words, this method can produce a similar pavement temperature as the one generated by ME method, which supports the goal of having an analogous background for comparison of the design results.

The PETE model was stablished by finding the temperature, for each hour of a specific period, that gave the same damage as the real temperature distribution made. Then the linear regression between this temperature and the related air temperature for the mentioned period was verified after some cross analysis to apply for prediction of service temperature for different locations and different pavement support conditions (Picado-Santos, 2000).

According to Figure 4-3, the country is divided into 4 climatic zones, where the only difference with respect to the MACOPAV division is the separation of the Zona Media in two zones; Z. Media Sul Mondego and Z. Media Norte Mondego. In these zones the behavior of flexible pavements in terms of damage is similar (Baptista & Picado-Santos, 2000).

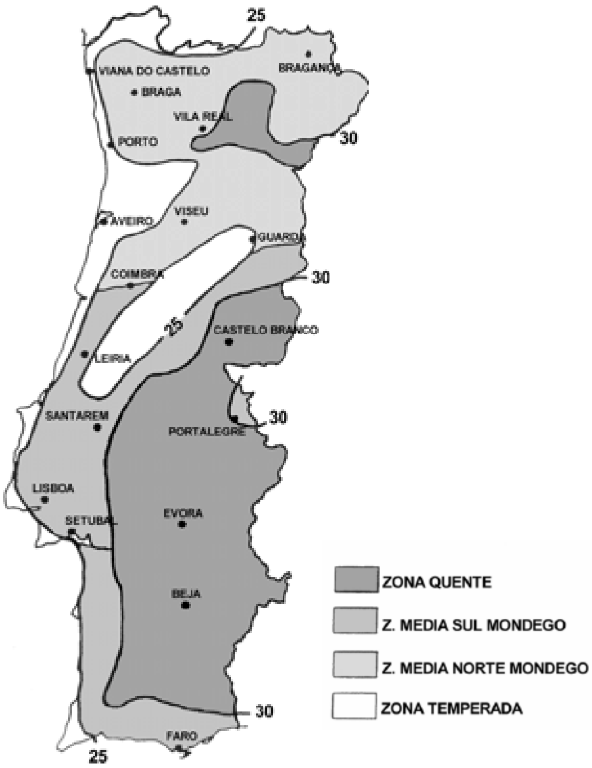


Figure 4-3: Climatic zones in Portugal (Baptista & Picado-Santos, 2000)

As for the three selected roads in this study, the pavement temperatures are obtained based on this method. These pavement temperatures can also be available through a simplification done to set a unique service temperature considering different locations in Portugal, different AC thicknesses and different pavement supports. Accordingly, this method includes tables, where the service temperature can be obtained quickly. The pavement temperature values established by PETE model, are corresponded to sections of flexible pavement with granular sub-base, in material with a large particle size of 20 cm and the remaining layers with bituminous mixtures. The pavement temperatures are provided for the different classes of traffic and for foundation classes “F2”, “F3” and “F4” with modules of 60MPa, 100MPa and 150MPa, respectively by PETE.

The corresponding tables used for this study (for F2 and F3) are Table 4-7 and

Table 4-8 (Picado-Santos, 2000).

Table 4-7: Service Temperatures for an F2 Foundation Class (E = 60MPa) (Picado-Santos, 2000)

Location	Total thickness of bituminous layer (cm)						
	16	22	25	28	30	32	Maximum
	T6	T5	T4	T3	T2	T1	
Bragança	26,4	27.3	26.9	26.5	26.7	26.4	27.3
Viana do Castelo	26.0	26.6	26.3	26.0	26.1	26.0	26.6
Chaves	27.2	28.0	27.5	27.0	27.1	26.8	28.0
Braga	26.6	27.3	26.9	26.5	26.6	26.4	27.3
Mirandela	28.8	29.5	28.7	28.3	28.2	27.2	29.5
Miranda do Douro	26.8	27.6	27.2	26.8	26.9	26.7	27.6
Vila Real	27.0	27.7	27.3	26.9	27.0	26.8	27.7
Porto – P. Rubras	25.0	25.6	25.4	25.3	25.1	25.2	25.6
S. Bárbara	29.4	30.0	29.3	29.3	28.9	28.5	30.0
Porto – S. Pilar	25.5	26.1	25.9	25.6	25.5	25.6	26.1
Bigorne	23.6	24.4	24.3	24.3	24.2	24.1	24.4
F. Castelo Rodrigo	27.1	27.8	27.5	27.0	27.1	26.8	27.8
Viseu	27.1	27.8	27.4	26.9	27.0	26.7	27.8
S. Jacinto	24.4	24.8	24.8	24.7	24.6	24.7	24.8
Caramulo	24.6	25.3	25.2	25.0	24.9	25.0	25.3
Guarda	23.8	24.6	24.5	24.4	24.3	24.4	24.6
Mira	25.5	26.0	25.8	25.5	25.4	25.5	26.0
Coimbra	27.5	28.2	27.7	27.3	27.4	27.1	28.2
Mont.-o-Velho	26.1	26.7	26.4	26.1	26.2	26.0	26.7
Fundão	28.0	28.7	28.2	27.7	27.8	27.6	28.7
Castelo Branco	29.2	29.8	29.2	29.1	28.7	28.3	29.8
Alcobaça	26.3	26.9	26.5	26.3	26.4	26.2	26.9
Tancos	28.5	29.1	28.6	28.0	28.1	27.8	29.1
Cabo Carvoeiro	24.1	24.5	24.5	24.4	24.3	24.4	24.5
Portalegre	27.8	28.6	28.0	27.6	27.7	27.5	28.6
Santarém	28.4	28.9	28.4	27.9	28.0	27.6	28.9
Ota	27.8	28.5	27.9	27.4	27.5	27.3	28.5
Mora	28.9	29.6	28.8	28.9	28.3	28.1	29.6
Elvas	29.9	30.5	29.5	29.7	29.0	28.8	30.5
Cabo da Roca	23.9	24.3	24.3	24.3	24.2	24.2	24.3
Lisboa	27.5	28.2	27.6	27.4	27.4	27.2	28.2
Évora	28.3	28.9	28.4	27.9	28.0	27.6	28.9
Setúbal	28.0	28.5	28.0	27.5	27.6	27.5	28.5
Sesimbra	26.6	27.2	26.9	26.5	26.7	26.5	27.2
Beja	29.5	30.1	29.4	29.4	28.9	28.5	30.1
Sines	24.7	25.1	25.0	24.9	24.9	24.9	25.1
Zambujeira	25.9	26.4	26.1	25.9	25.7	25.9	26.4
V. Real S. António	29.0	29.5	28.8	28.9	28.3	28.1	29.5
Praia da Rocha	27.5	28.2	27.6	27.4	27.4	27.2	28.2
Faro	28.3	28.8	28.3	27.9	27.9	27.6	28.8

Table 4-8: Service Temperatures for an F3 Foundation Class (E = 100MPa) (Picado-Santos, 2000)

Location	**Total thickness of bituminous layer (cm)**						
	12	**18**	**21**	**24**	**26**	**28**	**Maximum**
	T6	**T5**	**T4**	**T3**	**T2**	**T1**	
Bragança	25.0	25.8	26.7	26.6	26.4	26.1	26.7
Viana do Castelo	25.2	25.5	26.2	26.1	25.9	25.8	26.2
Chaves	25.9	26.6	27.5	27.3	26.9	26.7	27.5
Braga	25.6	26.0	26.9	26.7	26.5	26.2	26.9
Mirandela	27.7	28.0	28.9	28.7	28.2	28.0	28.9
Miranda do Douro	25.3	26.1	27.1	26.9	26.7	26.4	27.1
Vila Real	25.7	26.3	27.3	27.1	26.7	26.6	27.3
Porto – P. Rubras	24.3	24.6	25.2	25.2	25.1	25.0	25.2
S. Bárbara	28.6	28.7	29.7	29.2	28.9	28.5	29.7
Porto – S. Pilar	24.7	25.1	25.7	25.7	25.5	25.4	25.7
Bigorne	22.1	23.0	23.1	23.8	23.8	23.8	23.8
F. Castelo Rodrigo	25.6	26.4	27.4	27.2	26.8	26.6	27.4
Viseu	25.8	26.4	27.4	27.2	26.8	26.6	27.4
S. Jacinto	23.7	24.0	24.0	24.5	24.5	24.4	24.5
Caramulo	23.4	24.0	24.0	24.8	24.7	24.7	24.8
Guarda	22.1	23.1	23.2	24.0	24.0	24.0	24.0
Mira	24.7	25.0	25.6	25.6	25.4	25.3	25.6
Coimbra	27.0	27.1	27.9	27.6	27.3	27.0	27.9
Mont.-o-Velho	25.4	25.7	26.3	26.2	26.0	25.9	26.3
Fundão	27.0	27.3	28.3	28.0	27.7	27.4	28.3
Castelo Branco	28.3	28.6	29.4	29.0	28.7	28.3	29.4
Alcobaça	25.5	25.8	26.6	26.4	26.2	26.0	26.6
Tancos	27.8	27.9	28.8	28.5	28.0	27.8	28.8
Cabo Carvoeiro	23.5	23.7	23.7	24.2	24.2	24.1	24.2
Portalegre	26.9	27.2	28.1	27.8	27.6	27.3	28.1
Santarém	27.7	27.8	28.7	28.3	27.8	27.7	28.7
Ota	27.2	27.3	28.1	27.8	27.5	27.2	28.1
Mora	28.1	28.2	29.1	28.7	28.3	28.0	29.1
Elvas	29.0	29.1	30.1	29.6	29.0	28.9	30.1
Cabo da Roca	23.4	23.6	23.6	24.0	24.0	24.0	24.0
Lisboa	27.1	27.1	27.8	27.6	27.3	27.1	27.8
Évora	27.5	27.7	28.6	28.3	27.8	27.7	28.6
Setúbal	27.4	27.5	28.3	27.9	27.6	27.4	28.3
Sesimbra	25.8	26.1	26.9	26.7	26.5	26.3	26.9
Beja	28.7	28.8	29.8	29.3	28.9	28.6	29.8
Sines	24.2	24.4	24.3	24.8	24.7	24.7	24.8
Zambujeira	25.2	25.5	26.1	26.0	25.8	25.7	26.1
V. Real S. António	28.4	28.5	29.2	28.7	28.4	28.1	29.2
Praia da Rocha	27.1	27.1	27.8	27.5	27.3	27.1	27.8
Faro	27.8	27.8	28.6	28.3	27.8	27.7	28.6

Table 4-9 shows the calculated equivalent pavement temperature based on PETE method for the selected Portuguese pavements.

Table 4-9: Equivalent service temperature based on PETE approach for the selected roads

Road Name	Location	AC thickness (cm)	Subgrade Modulus Mr (MPa)	PETE temperature °C
IC3	Tancos	25	60	28.6
IP6	Tancos	28	60	28
EN254	Evora	15	80	28

4.4 Moisture Content

The variation of moisture content has significant effect on stiffness or resilient modulus of unbound layers, subgrade layer and foundation. Stabilized layers are considered insensitive to moisture content, and the resilient modulus or stiffness of these layers can be held constant over time (AASHTO, 2015b).

The variation of moisture content that occur after construction of the pavement section generally falls into three categories:

1. Increase or decrease from the initial condition (typically near optimum) to the equilibrium or average condition.
2. Seasonal fluctuation about the average or normal moisture condition due to infiltration of rainfall through cracks in the bound layer(s) and due to fluctuations in the groundwater table (GWT) in the absence of freeze/thaw.
3. Variations in moisture content due to freeze/thaw.

Due to the climatic condition of Portugal (specifically for the three selected roads), the category 3 is not discussed in this study. On the other hand, the study by Witczak et al. (2000) shows that the effect on resilient moduli, M_R, due to Categories 1 could be quite significant. However, Category 2 results, i.e., seasonal changes in moisture in the absence of freeze/thaw, were found to produce typically insignificant changes in M_R. Accordingly, it would be reasonable for ME method to assume that there are no cracks in newly constructed pavements and that the GWT does not fluctuate during the design period. Given these assumptions and conditions, the role of the EICM with respect to moisture content is limited to the prediction of changes under Categories 1 in this study. Therefore, the EICM should predict the equilibrium moisture contents (Zapata & Houston, 2008).

The related tasks in EICM model that includes the effect of moisture content on flexible pavements are as follows (ARA, 2004a):

- Records the user supplied resilient modulus, M_R, of all unbound layer materials at an initial or reference condition. Generally, this will be at or near the optimum water content and maximum dry density. In this study, the user supplied resilient modulus is the value provided by Portuguese road projects (the applied value in SHELL method for selected roads).
- Evaluates the expected changes in moisture content, from the initial or reference condition, as the subgrade and unbound materials reach equilibrium moisture condition. Also evaluates the seasonal changes in moisture contents.
- Evaluates the effect of changes in soil moisture content with respect to the reference condition on the user entered resilient modulus, M_R.

Accordingly, the difference of applying moisture content factor in both methods of SHELL and ME can be described as:

- In SHELL method, the optimum moisture content is a single value used to calculate resilient modulus of soil and unbound layers.
- In ME method, the EICM calculates moisture profile (prediction of changes from optimum moisture content to equilibrium moisture content which is not constant in time or through the depth pf pavement) in order to calculate the equilibrium resilient modulus (internally). However, the initial resilient modulus used in ME method is the user supplied value obtained by optimum moisture content (same as SHELL method).

As for the ME method, the amount of changes in moisture content (which creates the difference between initial resilient modulus and final resilient modulus) could be insignificant due to:

1. Deep water table (changes in moisture content for more than 6 meter is insignificant).
2. Good external drainage (less changes in moisture content with good external drainage, assumption: considered for the selected roads).
3. Balance between monthly precipitations and evaporations (for temperate climate conditions seems to be balanced).
4. Good internal drainage (no data for the selected roads in this part. assumption: considered for the selected roads).

So, given the above assumptions and conditions, the changes of resilient modulus due to the changes in moisture content can be considered insignificant in this study.

On the other hand, The EICM predicts the moisture content based on the soil water characteristic curve (SWCC). Several mathematical equations have been proposed to represent the SWCC. The EICM uses the Fredlund and Xing (1994) equation, to predict the volumetric moisture content (w_{θ}) from the soil matric suction (h). However, the authors of EICM version 2.6 (Witczak et al., 2000) concluded that soil suction and SWCCs simply couldn't be measured with great precision at the present time. Therefore, they concluded that the SWCC could probably be estimated from D60 (the effective grain size corresponding to 60 percent passing by weight) or wPI (PI P_{200}) about as accurately as it can be measured, unless the laboratory or person making the measurement is highly experienced (Bayomy & Salem, 2005).

So, three main parameters that have main role in calculation of moisture profile in EICM model are:

- the maximum dry density ($\gamma_{d\ max}$);
- specific gravity (G_s);
- the optimum gravimetric moisture content (w_{opt}) of the compacted unbound material.

The other parameters can internally be computed by EICM model. The values applied for maximum dry density and optimum gravimetric moisture content in ME projects for the three roads (IC3, IP6, EN254) are the provided values by Portuguese projects (applied in SHELL method). The value for specific gravity is determined from P_{200} (Passing sieve #200 (decimal) and PI (Plasticity Index (%)) of the layer. The P_{200} is obtained from gradation information extracted from Portuguese road Projects. The PI values for soil for IC3 road and IP6 road are extracted from Portuguese documents. The PI for other cases is considered default value recommended by ME Design which the values are reasonable for Portuguese technical specifications of IP for earthworks. It should be noted that, the sensitivity analysis indicates that G_s increased when PI, w_{opt}, and $\gamma_{d\ max}$ increased, with PI the most significant parameter in the relationship and $\gamma_{d\ max}$ the least significant (Zapata & Houston, 2008).

As a result, the conditions for moisture content in both methods of SHELL and ME can be considered similar.

4.5 AADTT Modification (Calculation for IC3 road)

As it is mentioned earlier in the chapter 2, the approaches of applying traffic in both methods of SHELL and ME are different. The main difference of these approaches is defined as follows:

1. Equivalent single axle loads (ESALs) (which is used in SHELL method): This approach converts wheel loads of various magnitudes and repetitions (“mixed traffic”) to an equivalent number of “standard” or “equivalent” loads (can be obtained by the “Generalized Fourth Power Law”).
2. Load spectra (which is used in ME method): This approach characterizes loads directly by number of axles, configuration and weight. It does not involve conversion to equivalent values.

The load spectra approach has the potential to be more accurate in its load characterization. However, it demands more traffic data for the calculation. The traffic data required for ME Design are explained in detailed in MEPDG manual (AASHTO, 2015b). Accordingly, four basic types of traffic data are required for pavement structural design (ARA, 2004c):

1. Traffic volume-base year information.
2. Traffic volume adjustment factors:

   a. Monthly adjustment;
   b. Vehicle class distribution;
   c. Hourly truck distribution;
   d. Traffic growth factors.
3. Axle load distribution factors.
4. General traffic inputs:
   a. Number axles/trucks;
   b. Axle configuration;
   c. Wheel base.

Considering the availability of traffic data for Portuguese conditions, some of the above-mentioned inputs are supposed as the default values for IC3 road design in ME software (e.g. Axle load distribution factors and traffic volume adjustments (except traffic growth rate)). Accordingly, the applied AADTT in ME Design is modified to have the applied traffic similar to Portuguese project in SHELL method. This AADTT modification would result in similar conditions for both design approaches to prepare them for desired comparison.

The Accumulated Number of 80 kN Standard Axle Passages (NAEP80) assigned for IC3 project in the IP document is equal to 3.71E+07 standard axle. Accordingly, this value should be applied in the ME Design projects. The following section describes the procedure to employ the NAEP80 of project (3.71E+07 standard axle) in the ME Design software.

Aggression Factor or Damage Factor is defined as the number of standard axles which is equivalent to a passage of a heavy vehicle (associated with a particular category) or converts the set of heavy vehicles on standard axles. Consequently, the desired accumulated number of 80 kN standard axles (N_{80}^{AEP}), is calculated according to the aggression factor, the average annual daily tuck traffic, the annual growth rate, lane distribution factor, directional distribution factor and pavement design period in years, by the following expression (equation 4-2):

$$N_{80}^{AEP} = \alpha \, N_{AVP} \qquad Equation\ 4\text{-}2$$

Where

$$N_{AVP} = 365. AADTT. C\ .P. L. D \qquad Equation\ 4\text{-}3$$

$$C = \frac{(1+t)^P - 1}{P.t} \qquad Equation\ 4\text{-}4$$

$N_{80}^{AEP}: Accumulated\ Number\ of\ 80\ kN\ Standard\ Axle\ Passages$

$\alpha: Aggression\ Factor$

$N_{AVP}: Accumulated\ Number\ of\ Heavy\ Vehicles$

$AADTT: Average\ Annual\ Daily\ Truck\ Traffic$

$C: Growth\ Factor$

$P: Design\ Period\ in\ years$

$L: Truck\ lane\ distribution\ factor\ (LDF)$

$D: Directional\ distribution\ factor\ (DDF)$

$t: Average\ Annual\ Growth$

The equation 4-5 is used to obtain the aggression factor based on the applied traffic input data in ME Design software:

$$\alpha = \frac{\sum_{Class=4}^{Class=13}(Total\ Equivalent\ Load\ (80\ kN) \times Vehicle\ (truck) class\ distribution)}{100} \qquad Equation\ 4\text{-}5$$

The Table 4-10 shows the AADTT distribution for each vehicle class for IC3 applied in ME Design.

Table 4-10: AADTT distribution for each vehicle class for IC3 road in ME Design

Vehicle Class	Distribution (%)
Class 4	3.3
Class 5	34
Class 6	11.7
Class 7	1.6
Class 8	9.9
Class 9	36.2
Class 10	1
Class 11	1.8
Class 12	0.2
Class 13	0.3
Total	100

The other factor of defined equation for aggression factor is total equivalent load (80 kN) which is calculated as:

$$Total\ equivalent\ load\ (80kN)_{class\ i} = \sum_{j=Single}^{Quad}\Big((Number\ of\ j\ axles\ per\ truck_{class\ i})\ \ (Total\ equivalent\ load_{j\ axle\ and\ class\ i})\Big) \qquad Equation\ 4\text{-}6$$

For the first part of $Equation$ 4-6, the Table 4-11 shows the distribution of axles per truck for each class. Regarding the second part of the $Equation$ 4-6, there are two traffic data required for ME Design which are used to calculate the value of total Equivalent load for j axle, class i.

The first one is the Axle Load Distribution Factors which represent the percentage of the total axle applications within each load interval for a specific axle type (single, tandem, tridem, and quad) and vehicle class (classes 4 through 13). A definition of load intervals for each axle type is provided below (ARA, 2004c):

- Single axles – 3,000 lb to 41,000 lb at 1,000-lb intervals.
- Tandem axles – 6,000 lb to 82,000 lb at 2,000-lb intervals.
- Tridem and Quad axles – 12,000 lb to 102,000 lb at 3,000-lb intervals.

The applied unit for ME project created for this study is SI unit, however due to the accuracy of calculation with origin unit for ME Design, the US unit for load intervals is used to calculate the aggression factor in this document.

Table 4-11: Distribution of axles per truck for each class for IC3 road in ME Design software

Vehicle	Single	Tandem	Tridem	Quad
Class 4	1.62	0.39	0	0
Class 5	2	0	0	0
Class 6	1.02	0.99	0	0
Class 7	1	0.26	0.83	0
Class 8	2.38	0.67	0	0
Class 9	1.13	1.93	0	0
Class 10	1.19	1.09	0.89	0
Class 11	4.29	0.26	0.06	0
Class 12	3.52	1.14	0.06	0
Class 13	2.15	2.13	0.35	0

There are tables for Single, Tandem, Tridem and Quad axle load distribution in ME Design which the calculation of total equivalent load, related to these axle load distribution tables, is accomplished and the results are used to obtain aggression factor.

The Table 4-12 shows the axle load distribution for single axle per year in summary.

Table 4-12: Single axle load distribution table for IC3 in ME Design Software

Number	Month	Class	Total	k=1	k=2	k=3	k=4	k=5	k=6	...	k=39
				3000	4000	5000	6000	7000	8000	...	41000
m=1	January	4	100	0.52	1.66	1.95	3.78	6.19	10.9	...	0
m=2	February	4	100	0.52	1.66	1.95	3.78	6.19	10.9	...	0
m=3	March	4	100	0.52	1.66	1.95	3.78	6.19	10.89	...	0
m=4	April	4	100	0.51	1.66	1.95	3.78	6.19	10.9	...	0
m=5	May	4	100	0.52	1.66	1.95	3.78	6.2	10.91	...	0
m=6	June	4	100	0.52	1.66	1.95	3.78	6.19	10.88	...	0
m=7	July	4	100	0.53	1.66	1.95	3.78	6.19	10.9	...	0
m=8	August	4	100	0.53	1.66	1.95	3.78	6.19	10.9	...	0
m=9	September	4	100	0.54	1.66	1.95	3.78	6.19	10.89	...	0
m=10	October	4	100	0.54	1.66	1.95	3.78	6.19	10.89	...	0
m=11	November	4	100	0.52	1.66	1.95	3.78	6.19	10.91	...	0
m=12	December	4	100	0.53	1.66	1.95	3.78	6.19	10.89	...	0
...	...	...	...	...	...	...	...	...	...	...	0
m=12	December	13	100	2.47	7.4	3.45	4.95	6.24	8.13	...	0

For the calculation of total equivalent load, it is required to convert the load intervals to estimated equivalent load (ESAL or equivalent single axle load). For this purpose, the AASHTO LEF tables for Load Equivalency Factors to calculate the total equivalent load (80 kN) for each class is used. The assumptions to properly use the LEF tables based on Portuguese conditions are (AASHTO, 1993) (Appendix D of AASHTO):

- terminal serviceability index (pt) = 2.5;
- pavement structural number (SN)[1] = 5.0 for flexible pavements.

So, the regression equations (4-7 to 4-9) for calculation of the equivalent load (80 kN) based on these tables are obtained as[2]:

For Single axle LEF:

$$y = 1e^{-5}.x^{3.8565} \qquad Equation\ 4\text{-}7$$

[1] The use of SN=5 for the determination of the 80 kN single axle equivalence factors will normally give results that are sufficiently accurate for design purpose (AASHTO 1993, Appendix D, Page D2)
[2] Since the quad axle does not have any distribution in the vehicle classes, it is not considered for the regression equation.

$$R^2 = 0.9996$$

For Tandem Axle LEF:

$$y = 7e^{-7}.x^{4.0316} \qquad Equation\ 4\text{-}8$$

$$R^2 = 0.9996$$

For Tridem Axle LEF:

$$y = 1e^{-7}.x^{4.1739} \qquad Equation\ 4\text{-}9$$

$$R^2 = 1$$

Where

x: $Load\ for\ j\ axle, class\ i\ in\ kips\ per\ day$

y: $Equivalent\ load\ for\ j\ axle, class\ i$

The equivalent load for each class per month is calculated as:

$$Equivalent\ Load\ for\ j\ axle, Class\ i, month\ m = \frac{\sum_{k=1}^{n}(LEF_k\ \ for\ j\ axle,class\ i,month\ m \times Axle\ Load\ Value_k\ for\ j\ axle,class\ i,month\ m)}{100} \qquad Equation\ 4\text{-}10$$

Where

m: $month$

k: $the\ related\ column\ for\ axle\ load\ interval\ in\ month\ m\ as\ well\ as\ axle\ load\ value$

n: $the\ last\ column\ in\ each\ axle\ type$

j: $type\ of\ axle$

Accordingly, the single axle load distribution table based on the equivalent load is modified as it is shown in Table 4-13.

Table 4-13: Sample Table for Equivalent Single Axle Load Distribution for IC3 in ME Design

Load Interval				**k=1**	**k=2**	**k=3**	**k=4**	**k=5**	**k=6**	**...**	**k=39**
Load in lb				**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**...**	**41000**
Load in kips				**3**	**4**	**5**	**6**	**7**	**8**	**...**	**41**
LEF for Single axle				**0.000692**	**0.002098**	**0.004961**	**0.010022**	**0.01816**	**0.030393**	**...**	**16.58445**
No.	**Month**	**Class**	**Total**								
m=1	**Jan.**	**4**	**100**	0.52	1.66	1.95	3.78	6.19	10.9	...	0
m=2	**Feb.**	**4**	**100**	0.52	1.66	1.95	3.78	6.19	10.9	...	0
m=3	**Mar.**	**4**	**100**	0.52	1.66	1.95	3.78	6.19	10.89	...	0
m=4	**Apr.**	**4**	**100**	0.51	1.66	1.95	3.78	6.19	10.9	...	0
m=5	**May**	**4**	**100**	0.52	1.66	1.95	3.78	6.2	10.91	...	0
m=6	**Jun.**	**4**	**100**	0.52	1.66	1.95	3.78	6.19	10.88	...	0
m=7	**Jul.**	**4**	**100**	0.53	1.66	1.95	3.78	6.19	10.9	...	0
m=8	**Aug.**	**4**	**100**	0.53	1.66	1.95	3.78	6.19	10.9	...	0
m=9	**Sept.**	**4**	**100**	0.54	1.66	1.95	3.78	6.19	10.89	...	0
m=10	**Oct.**	**4**	**100**	0.54	1.66	1.95	3.78	6.19	10.89	...	0
m=11	**Nov.**	**4**	**100**	0.52	1.66	1.95	3.78	6.19	10.91	...	0
m=12	**Dec.**	**4**	**100**	0.53	1.66	1.95	3.78	6.19	10.89	...	0
...	**...**	**...**	**...**	...	...	...	...	...	...	...	0
m=12	**Dec.**	**13**	**100**	2.47	7.4	3.45	4.95	6.24	8.13	...	0

The Table 4-14 shows the monthly results for the equivalent load in single axle distribution.

Table 4-14: Monthly Results for Equivalent Single Axle Load Distribution for IC3 in ME Design

Load Interval				k=1	k=2	k=3	k=4	k=5	...	k=39	Monthly Result
Load in lb				3000	4000	5000	6000	7000	...	41000	
Load in kips				3	4	5	6	7	...	41	
LEF for Single axle				0.000692	0.002098	0.004961	0.010022	0.01816	...	16.58445	
Month	Month	Class	Total								
m=1	Jan.	4	100	0.52	1.66	1.95	3.78	6.19	...	0	0.215
m=2	Feb.	4	100	0.52	1.66	1.95	3.78	6.19	...	0	0.215
m=3	Mar.	4	100	0.52	1.66	1.95	3.78	6.19	...	0	0.215
m=4	Apr.	4	100	0.51	1.66	1.95	3.78	6.19	...	0	0.215
m=5	May	4	100	0.52	1.66	1.95	3.78	6.2	...	0	0.215
m=6	Jun.	4	100	0.52	1.66	1.95	3.78	6.19	...	0	0.215
m=7	Jul.	4	100	0.53	1.66	1.95	3.78	6.19	...	0	0.215
m=8	Aug.	4	100	0.53	1.66	1.95	3.78	6.19	...	0	0.215
m=9	Sept.	4	100	0.54	1.66	1.95	3.78	6.19	...	0	0.215
m=10	Oct.	4	100	0.54	1.66	1.95	3.78	6.19	...	0	0.215
m=11	Nov.	4	100	0.52	1.66	1.95	3.78	6.19	...	0	0.215
m=12	Dec.	4	100	0.53	1.66	1.95	3.78	6.19	...	0	0.215
...	...	...	...	...	...	...	...	...	...	...	...
m=12	Dec.	13	100	2.47	7.4	3.45	4.95	6.24	...	0	0.180

The second required traffic data used to calculate the value of total equivalent load for j axle, class i is Truck Traffic Monthly Adjustment Factors which represent the proportion of the annual truck traffic for a given truck class that occurs in a specific month. In other words, the monthly distribution factor for a specific month is equal to the monthly truck traffic for the given class for the month divided by the total truck traffic for that truck class for the entire year. Truck traffic monthly adjustment factors (MAF) depend on factors such as adjacent land use, the location of industries in the area, and roadway location (urban or rural). The "Truck Traffic Monthly Adjustment Factors" for IC3 road and the other selected roads in ME Design is considered default value of 1 for all classes (4 to 13) and all months of year (January to December). So, the total equivalent load for class i, j axles for one year is defined as:

$$Total\ Equivalent\ Load_{for\ j\ axle,class\ i\ \ per\ year} = \sum_{m=1}^{12} \left(Equivalent\ Load\ _{for\ j\ axle,Class\ i,month\ m} \times monthly\ adjustment\ factor_{for\ month\ m}\ \right) Equation\ 4\text{-}11$$

The Table 4-15, Table 4-16 and Table 4-17 show the results for the TEL- Total Equivalent Load (80 kN) in each class for each axle type:

Table 4-15: Total Equivalent Load (80 kN) in each class for single axle

	Total Class 4	**Total Class 5**	**Total Class 6**	**Total Class 7**	**Total Class 8**	**Total Class 9**	**Total Class 10**	**Total Class 11**	**Total Class 12**	**Total Class 13**
TEL	2.6	1.1	2.1	4.6	1.8	1.6	1.6	2.8	2.2	2.2

Table 4-16: Total Equivalent Load (80 kN) in each Class for Tandem Axle

	Total Class 4	**Total Class 5**	**Total Class 6**	**Total Class 7**	**Total Class 8**	**Total Class 9**	**Total Class 10**	**Total Class 11**	**Total Class 12**	**Total Class 13**
TEL	3.8	2.3	5.3	10	2.1	5.7	7.6	4.5	4.6	8.3

Table 4-17: Total Equivalent Load (80 kN) in each Class for Tridem Axle

	Total Class 4	**Total Class 5**	**Total Class 6**	**Total Class 7**	**Total Class 8**	**Total Class 9**	**Total Class 10**	**Total Class 11**	**Total Class 12**	**Total Class 13**
TEL	2.4	8.7	9.1	12	11	1.2	5.1	2.3	7.4	13

Accordingly, the Table 4-18 shows the total equivalent load (80 kN) for each class:

Table 4-18: Total Equivalent Load (80 kN) for each Class

		Total equivalent Load by Reg. from LEF Table
Total class 4	1.62Sigle+0.39Tandem	5.66
Total class 5	2Single	2.22
Total class 6	1.02Single+0.99Tandem	7.44
Total class 7	1Single+0.26Tandem+0.83Tridem	17.69
Total class 8	2.38Sinlge+0.67Tandem	5.80
Total class 9	1.13Single+1.93Tandem	12.86
Total class 10	1.19Sinlge+1.09Tandem+0.89Tridem	14.72
Total class 11	4.29Single+0.26Tandem+0.06Tridem	13.11
Total class 12	3.52Single+1.14Tandem+0.06Tridem	13.22
Total class 13	2.15Single+2.13Tandem+0.35Tridem	26.89

Consequently, the aggression factor (α) based on the aggression factor equation (equation 4-5) and the AADTT distribution table (Table 4-10) is calculated as it is shown in Table 4-19:

Table 4-19: Aggression Factor Calculation for IC3 road

	Distribution (D)	**Total Equivalent Load (TEL)**	**D×TEL**	**Aggression Factor**
Total class 4	3.3	5.66	18.68	
Total class 5	34	2.22	75.59	
Total class 6	11.7	7.44	87.06	
Total class 7	1.6	17.69	28.31	
Total class 8	9.9	5.80	57.44	
Total class 9	36.2	12.86	465.64	
Total class 10	1	14.72	14.72	
Total class 11	1.8	13.11	23.60	
Total class 12	0.2	13.22	2.64	
Total class 13	0.3	26.89	8.07	
Total	100		781.74	=781.74/100=7.82

Finally, N_{AVP} (Accumulated Number of Heavy Vehicles) and therefore AADTT are calculated based on the equations 4-2 and 4-3. The new modified AADTT will be used for the IC3 pavement design.

$$AADTT_{New} = \frac{N}{365\,.\,\alpha.\,C.\,P.\,D\,.\,L} = \frac{3.71E + 07}{7.82 \times 1.11 \times 20 \times 0.5 \times 0.9} = 1299.5 \cong 1299$$

4.6 IC3 Pavement Design by BISAR

Based on the new calculated service temperature (PETE) for AC layers, the new dynamic modulus is calculated and applied in SHELL method.

The dynamic modulus is calculated using equations 4-12 and 4-13:

$$E_m = 10^A (For\ bitumen\ stiffness\ (Sb) between\ 5\ MPa\ and\ 1000\ MPa) \qquad Equation\ 4\text{-}12$$

$$A = \frac{S89+S68}{2}(log\, Sb - 8) + \frac{S89-S68}{2}|log\, Sb - 8| + Sm108 \qquad Equation\ 4\text{-}13$$

Where

$$\begin{cases} S89 = 1.12 \times \dfrac{(Sm3109 - Sm108)}{log\, 30} \\ S68 = 0.6 \times log \dfrac{1.37 \times {v_b}^2 - 1}{1.33 \times v_b - 1} \\ Sm3109 = 10.82 - \dfrac{1.342 \times (100 - v_a)}{v_a + v_b} \\ Sm108 = 8 + 5.68 \times 10^{-3} \times v_a + 2.135 \times 10^{-4} \times {v_a}^2 \end{cases}$$

v_a: $Aggregate\ content\ by\ volume$ (%)

v_b: $Effective\ Binder\ Content\ (by\ volume)\ at\ time\ of\ construction$ (%)

E_m: $Dynamic\ modulus\ of\ bituminous\ mixture\ (Pa)$

Sb: $Bitumen\ Stiffness\ (Pa)$

While bitumen stiffness is calculated using equation 4-14:

$$Sb = 1.157 \times 10^{-7} \times tc^{-0.368} \times 2.718^{-IPen} \times (Tab_r - T)^5 \qquad Equation\ 4\text{-}14$$

Where

Sb: $Bitumen\ Stiffness\ (Pa)$

tc: $Loading\ Time\ (s) = 1/vt$

vt: $Average\ traffic\ speed\ for\ heavy\ vehicle\ (km/h)$

$$IPen: Index\ of\ bitumen\ penetration\ = \frac{20 \times Tab_r + 500 \times \log pen25r - 1951.55}{Tab_r - 50 \times \log pen25r + 120.15}$$

$$T: Service\ temp\ by\ PETE\ method\ (°C)$$

$$Tab_r: retained\ softening\ point\ (Ring\ and\ Ball)\ (°C)\ = 99.13 - 26.35 \log pen25r$$

$$pen25: Bitumen\ Penetration\_25°C\ (mm)$$

$$pen25r: Retained\ bitumen\ penetration\_25°C\ (mm)\ = 0.65 \times pen25$$

Table 4-20 shows the summary of dynamic modulus calculation based on new service temperatures for AC layers. The related tables for this calculation are provided in appendix 4.

Table 4-20: Summary of dynamic modulus calculation based on new service temperatures for AC layers

AC layer name	**IC3**		**IP6**			**EN254**	
	Surface in BB	Binder + Base in MB	Surface in BB	Binder in BB	Base in MB	Surface in BB	Binder in MB
Thickness (cm)	5	20	6	6	16	5	10
Penetration Grade	35/50	35/50	60/70	60/70	60/70	50/70	50/70
Vb %	12.22	9.94	11.12	11.12	10.34	11.57	9.28
Va %	84.97	86.52	84.14	84.12	85.66	82.11	85
T (°C)	28.6	28.6	28	28	28	28	28
IPen	-0.06	-0.06	-0.14	-0.14	-0.14	-0.12	-0.12
Tabr (°C)	61.85	61.85	56.29	56.29	56.29	57.21	57.21
tc (s)	0.01	0.01	0.01	0.01	0.01	0.02	0.02
Sb (MPa)	25.13	25.13	12.03	12.03	12.03	11.72	11.72
Em (MPa)	4160.83	5187.2	2452.52	2448.87	2952.1	1948.68	2876.96

The final proposed pavement structure design for IC3 (applied in BISAR 3.0 and ME design Software) is demonstrated in the Table 4-21 (IP, 2005):

Table 4-21: Applied pavement structure design for IC3 in BISAR 3.0 and ME Design

Layer Type	Thickness H(m)	Dynamic Modulus E(MPa)	Poisson's Ratio	Spring Compliance(m^3/N)
Surface in Bituminous mixture	0.05	4161	0.35	0.00E+00
Binder+ Base in Bituminous Macadam[1]	0.20	5187	0.35	0.00E+00
Sandwich Granular Base in ABGE	0.20	300	0.35	2.00E-09
Chemically Stabilized Subgrade 1: Soil Cement	0.20	2000	0.25	2.00E-09
Subgrade 2: Soil Foundation [2]	0.30	60	0.40	0.00E+00
Subgrade 3: Soil Foundation	Semi infinite	60	0.40	0.00E+00

The final proposed structure is applied in IC3 BISAR project. The expression of load in the BISAR for IC3 road design is based on 80 kN standard axle. Accordingly, the configuration of loads for IC3 BISAR project as well as the co-ordinates of the positions for the outputs are adjusted and the results are obtained for IC3 road. Based on obtained results by BISAR for IC3 road the following admissible number of ESALs is calculated:

1. The admissible number of ESALs for tensile strain at the bottom of Asphalt Layer (MB Layer)
2. The admissible number of ESALs for compression strain on top of the subgrade
3. The admissible number of ESALs for tensile stress at the bottom of soil cement

The equation 4-15 used to determine the damage in percentage:

$$Damage = \frac{NAEP80}{N_{adm}} \qquad Equation\ 4\text{-}15$$

Where

[1] Since there was not enough material characteristic information for MB base, the MB Binder data has been used for this layer.

[2] Based on ME Design software facilities "A minimum of two unbound layers are required to correctly model subgrade moisture and drainage". Accordingly, the last subgrade layer is divided into two layers which are now two subgrade layers with the same properties.

N_{adm}: $the\ admissible\ number\ of\ ESALs\ obtained\ by\ ME\ Design$

$NAEP80$: $Accumulated\ number\ of\ 80\ kN\ standard\ axle\ passages$

1. The equation 4-16 is used to obtain the admissible number of ESALs for tensile strain at the bottom of Asphalt Layer (MB Layer) (Shell, 1978):

$$\varepsilon_t = (0.856\,.\,V_b + 1.08)\,.\,E^{-0.36}.\,N^{-0.2} \qquad Equation\ 4\text{-}16$$

Where:

ε_t: $Tensile\ strain\ at\ the\ bottom\ of\ aspahlt\ layer (m/m)$
N: $Allowable\ number\ of\ loading\ repititions\ (ESAL)\ until\ fatigue\ cracking\ failure$
V_b: $Volumetric\ percentage\ of\ bitumen\ in\ the\ asphalt\ mix$
E: $Asphalt\ concrete\ stifness\ modulus\ (Pa)$.

The Table 4-22 shows the damage calculation results for this tensile strain:

Table 4-22: BISAR N calculation for tensile strain at the bottom of asphalt layer (MB Layer)

Data type for the equation	Values
Tensile Strain at the bottom of Asphalt Layer (MB Layer)	6.61E-05
Effective Binder Content (by volume) at time of construction V_b (%)	9.94
Asphalt concrete stiffness modulus E (Pa)	5.19E+09
Admissible number of ESALs	2.09E08
Damage % (3.71E+07/2.09E08) %	18

2. The equation 4-17 is used to obtain the admissible number of ESALs for compression strain on top of the subgrade (Shell, 1978):

$$\varepsilon_z = k_s\,.\,N^{-0.25} \qquad Equation\ 4\text{-}17$$

Where

ε_z: $Compression\ strain\ at\ the\ top\ of\ subgrade\ (m/m)$
N: $Allowable\ number\ of\ ESAL$

$$k_s: \begin{cases} 2.8 \times 10^{-2} for\ 50\%\ of\ survival\ probability \\ 2.1 \times 10^{-2} for\ 85\%\ of\ survival\ probability \\ 1.8 \times 10^{-2} for\ 95\%\ of\ survival\ probability \end{cases}$$

The Table 4-23 shows the damage calculation results for this compression strain:

Table 4-23: BISAR N calculation for compression strain on top of the subgrade

Data type for the equation	**Values**
Compression strain on top of the subgrade	-1.08500E-04
k_s	1.80000E-02
Admissible Number of ESALs	7.57E08
Damage % (3.71E+07/7.57E08) %	5

3. The equation 4-18 is used to obtain the admissible number of ESALs for tensile stress at the bottom of soil cement (Shell, 1978):

$$\frac{\sigma_t}{\sigma_{rf}} = 1 + a\,log(N_{adm}) \qquad Equation\ 4\text{-}18$$

$$\sigma_{rf} = 1.5 \times \sigma_{cd} \qquad Equation\ 4\text{-}19$$

Where

σ_t: $Max\ Tensile\ Stress\ at\ the\ bottom\ of\ Soil\ Cement\ (Mpa)\ obtained\ by\ 80\ kN\ Standard\ Axle$

σ_{cd}: $Diametrical\ Compression\ Stress\ (Mpa)$

σ_{rf}: $Tensile\ Stress\ obtain\ from\ flexural\ strength\ test\ (Mpa)$

a: $Constant\ Value, -0.06 \leq a \leq -0.1$

N_{adm}: $Allowable\ number\ of\ ESAL$

The Table 4-24 shows the damage calculation results for this tensile stress:

Table 4-24: BISAR N calculation for tensile stress at the bottom of soil cement

Data type for the equation	**Values**
Tensile Stress at the bottom of Soil Cement MPa	9.42E-02
Compression Stress (Diametrical) MPa (IP, 2005)	0.25
Tensile Stress obtain from Compression Stress	0.375
Constant Value (mean value supposed)	-0.08
Admissible Number of ESALs	22.96E08
Damage % (3.71E+07/22.96E08) %	2

4.7 Damage Results for SHELL and ME Design

By verification of important design factors for both approaches, the desired comparison between two design approaches is then possible. The Table 4-25 shows distress prediction results obtained by ME Design for the three selected roads. The target value in the Table 4-25 is the threshold value supposed by user for ME project. The predicted value is the value obtained by ME Design after analyzing the proposed pavement structure based on defined input data. Accordingly, the damage is equal to the predicted value divided by the target value. As it is noted in the Table 4-25, there are no values for IP6 and EN254 in distress type of chemically stabilized layer-fatigue fracture. The reason is that the pavement structures for IP6 and EN254 do not have stabilized layer.

Table 4-25: Distress prediction summary for IC3, IP6 and EN254

Distress Type	**Road**	**Distress @ Specified Reliability**		**Reliability (%)**	
		Target	Predicted	Target	Achieved
Terminal IRI (m/km)	IC3	4.00	2.95	90	99.89
	IP6		3.24		99.37
	EN254		2.81		99.96
Permanent deformation – total pavement (mm)	IC3	20.00	17.04		99.25
	IP6		26.74		19.30
	EN254		15.15		99.98
AC bottom-up fatigue cracking (%)	IC3	50.00	1.49		100.00
	IP6		1.79		100.00
	EN254		1.62		100.00
AC thermal cracking (m/km)	IC3	189.40	5.15		100.00
	IP6		5.15		100.00
	EN254		5.15		100.00
AC top-down fatigue cracking (m/km)	IC3	378.80	51.79		100.00
	IP6		84.01		100.00
	EN254		239.94		97.9
Permanent deformation – AC only (mm)	IC3	10.00	8.29		98.55
	IP6		7.99		99.12
	EN254		4.63		100.00
Chemically stabilized layer - fatigue fracture (%)	IC3	25.00	0.38		100.00
	IP6	-	-	-	-
	EN254	-	-	-	-

The Table 4-26 shows the obtained damages for the three selected roads. The results indicate that for the similar data in terms of the fundamental characteristics of pavement, traffic and weather conditions, the obtained damages are substantially different for the two methodologies.

Table 4-26: Result comparison of damage calculation for IC3, IP6 and EN254

Distress Type	**Road**	**Distress @ Specified Reliability**		**N**	**D $_{ME\ Design}$ %**	**D $_{SHELL}$ %**
		Target	Predicted	N (ME Design) =NAEP80	Predicted/target	$\frac{\mathbf{NAEP80}}{\mathbf{N_{adm}}}$
Terminal IRI (m/km)	IC3	4.00	2.95	3.71E+07	74	-
	IP6		3.24	4.952E+07	81	-
	EN254		2.81	4.90E+06	70	-
Permanent deformation - total pavement (mm)	IC3	20.00	17.04	3.71E+07	85	5
	IP6		26.74	4.952E+07	134	162
	EN254		15.15	4.90E+06	76	41
AC bottom-up fatigue cracking (%)	IC3	50.00	1.49	3.71E+07	3	18
	IP6		1.79	4.952E+07	4	110
	EN254		1.62	4.90E+06	3	150
AC thermal cracking (m/km)	IC3	189.40	5.15	3.71E+07	3	-
	IP6		5.15	4.952E+07	3	-
	EN254		5.15	4.90E+06	3	-
AC top-down fatigue cracking (m/km)	IC3	378.80	51.79	3.71E+07	14	-
	IP6		84.01	4.952E+07	22	-
	EN254		239.94	4.90E+06	63	-
Permanent deformation - AC only (mm)	IC3	10.00	8.29	3.71E+07	83	-
	IP6		7.99	4.952E+07	80	-
	EN254		4.63	4.90E+06	46	-
Chemically stabilized layer - fatigue fracture (%)	IC3	25.00	0.38	3.71E+07	2	2
	IP6	-	-	-	-	-
	EN254	-	-	-	-	-

4.8 Summary and Conclusions

This chapter is mainly presenting a study on justification of performance criteria, main environmental and traffic factors for Portuguese conditions in ME Design. This step is one of the important steps to set a similar background and conditions for damage comparison between both design methods. Consequently, the damage comparison can help to propose a validation framework for ME design method based on Portuguese conditions.

In this study, the performance criteria have been justified based on Portuguese conditions and the reasons to choose the limits have been described. Three main factors of service temperature, moisture content and traffic are then reviewed in both methods of ME and SHELL and the values are adjusted to have a similar background for both approaches.

The service temperatures or the equivalent pavement temperatures in the SHELL method has been calculated and adjusted based on a proposed Portuguese method for pavement temperature calculation (PETE method) to have a similar effect as generated pavement temperature profile in ME Design by EICM. So, the new dynamic modules have been calculated and used in SHELL method based on new service temperatures.

Three main parameters that have main role in calculation of moisture profile in EICM model (the maximum dry density, specific gravity and the optimum gravimetric moisture content of the compacted unbound material) have been evaluated to be similar to the available data for Portuguese projects.

The traffic volume (AADTT) as one of the main factors based on the available NAEP80 of the Portuguese projects has been edited. So, the modification of AADTT would result in similar conditions for both design approaches to prepare them for desired comparison.

Finally, accomplishing these justification steps makes it possible to compare the damage results in both approaches and then the sensitivity of main design criteria in both methods can be evaluated to the variations of the design inputs considering Portuguese conditions.

The results indicate that for similar data in terms of the fundamental characteristics of the pavement, traffic and weather conditions, the obtained damages are substantially different for the two methodologies.

The results in the Table 4-26 for the IC3 road show that the AC bottom-up fatigue cracking (by 18%) is the dominant damage and accordingly is the main criterion in the design by SHELL method while the total permanent deformation (by 85 %) can be the dominant criterion for the design by ME method. The project results (obtained by SHELL method from IP documents) also confirm that the AC bottom-up fatigue cracking (by 63 %) is the main damage in the design, but the values are different from the current values due to some differences in the extracted and adjusted data (e.g. pavement structure adjustment) for this study. Nevertheless, it can be said that the dominant criterion for both methodologies (SHELL method and ME method) are not the same (although similar conditions have almost been set for this comparison).

As the Table 4-26 shows, considering the obtained data for the IP6 road from the IP (Infraestruturas de Portugal) documents, there is a failure in the satisfaction of total permanent deformation (by 162 % in SHELL method and 134 % in ME method). So, in this case the dominant criterion for both methodologies (SHELL method and ME method) are the same. The dominance of the total permanent deformation is also confirmed by project results (by 28 % for total permanent deformation). Still the project results and the obtained results by this study regarding SHELL method are different (that could be due to the layer adjustment in the structure).

The results for the EN254 road are of the same type of the ones for IC3 road (Table 4-26). So that the AC bottom-up fatigue cracking (by 150 %) is the dominant damage and accordingly is the main criterion in the design by SHELL method while the total permanent deformation (by 76 %) is the dominant criterion for the design by ME method. The project results (obtained by SHELL method from IP documents) also confirm that the AC bottom-up fatigue cracking (by 43 %) was the dominant damage.

The results for the three roads show that SHELL method is more conservative regarding the total permanent deformation while ME method is more conservative in terms of AC bottom-up fatigue cracking. In other word, the dominant performance criterion in SHELL method is AC bottom-up fatigue cracking while total permanent deformation is the dominant and main performance criterion in ME method.

As it is also shown in Table 4-26, Terminal IRI is the second dominant performance criterion for ME Design, however terminal IRI is mostly dependent on total rutting based on the IRI model described in $Equation\,4\text{-}20$. So, the dominancy of this criterion is due to the dominancy of total rutting and can be considered as a subset of total rutting in procedure of sensitivity analysis (AASHTO, 2015b).

$$IRI = IRI_0 + 0.0150(SF) + 0.400(FC_{Total}) + 0.0080(TC) + 40.0(RD) \qquad Equation\ 4\text{-}20$$

Where

IRI_0 : $Initial\ IRI\ after\ construction, in./mi,$
$SF = Site\ factor,$
$FC_{Total} = Area\ of\ fatigue\ cracking$
$TC = Length\ of\ transverse\ cracking\ ft/mi$
$RD = Average\ rut\ depth, in.$

Considering the IC3 road, the damage values for the chemically stabilized layer - fatigue fracture are similar and accordingly both methods show the similar trend in estimation of this criterion, even though the total damage was very low in both cases.

In addition, with the most matched initial conditions and for final design decision, SHELL method is more conservative than ME method. In other words, ME method can roughly constitute the closer estimation for the proper design.

It should be noted that these results are the initial results that should be verified by the sensitivity analysis of obtained distresses to design inputs' variations for ME Design. Accordingly, the next chapter initially evaluates the sensitivity of the two main criteria (total permanent deformation and AC bottom-up fatigue cracking) to variations of the design inputs considering Portuguese conditions.

5 Sensitivity Analysis

5.1 Introduction

As it is mentioned in previous chapters, it is important to evaluate the suitability of MEPDG to Portuguese conditions before adopting the MEPDG as a new design approach for Portuguese pavements. For this purpose, the following tasks are required to be accomplished:

1. General comparisons of pavement designs (ME method and SHELL method) that is explained in the chapter 2.
2. Data preparation of the selected roads for ME Design (explained in chapter 3)
3. Evaluation and adjustment of the main factors and design criteria applied in both ME Design approach and SHELL approach for damage comparison that is explained in chapter 4.
4. Comparison of damage results for specific pavement designs between ME method and SHELL method for three selected roads that was discussed in chapter 4.
5. Sensitivity analysis of the MEPDG performance predictions and designs to variations in the design inputs considering Portuguese conditions. So that initially the main criteria that create the most damages in the design (total permanent deformation and AC bottom-up fatigue cracking) will be analyzed.

MEPDG sensitivity studies were started immediately after the initial release of the MEPDG in 2004 and the studies are still in progress and development. Therefore, some sensitivity results changed from one version to the next as software error corrections, model recalibrations, and other changes were implemented.

ME Design requires over 100 inputs to apply traffic, climate condition, materials, and pavement performance in software to estimate pavement distresses over the design life of the pavement. Considering this amount of inputs, having the knowledge of main required data for the project is essential. Sensitivity study provides the designers, the relative sensitivity of the models used in ME Design to inputs relating. So, the designers can focus on those inputs having the most effect on desired pavement performance (Q. Li et al., 2011). This assessment is achieved by running the same project several times while changing the value of an input and evaluating the results to quantify the sensitivity of predicted pavement performance.

Sensitivity analysis can be performed in two main categories: (1) sensitivity of predicted performance to design inputs; and (2) sensitivity of predicted performance to calibration coefficients. Performing the first sensitivity analysis helps to identify the suitable input level (e.g., Level 1, 2 or 3) for each design input. The second sensitivity analysis is applied to identify the most significant coefficients among all calibration coefficients in order to have a more effective calibration (Q. Li et al., 2011).

The analysis reported here calculates the sensitivity of MEPDG flexible pavement performance predictions to some design input variations using AASHTOWare Pavement ME Design™ version 2.1. It should be noted that the second sensitivity analysis was not considered in this study, since the available inputs for the selected roads are not enough (field data, level 1, laboratory data) for the calibration. So, accomplishing the second type of sensitivity analysis and performance coefficient calibration are recommended for future works.

For the sensitivity analysis in this study, only new-construction scenarios are considered. This parametric study is for three real cases of flexible pavement in different locations of Portugal with different climate conditions and traffic levels. Due to the limitation of time in workplan and software availability (expiration time) and based on some previous sensitivity studies (R. Li, 2013), the variables selected for this sensitivity study are as follow:

- Two-way AADTT.
- Thickness for all layers.
- Air voids for flexible layers.
- Effective Binder content for flexible layers.
- Resilient modulus for base, sub base and subgrade layers (in following sections, the term of unbound modulus used in text, tables and diagrams represents the resilient modulus for unbound layer in this chapter same as the term used in the sensitivity section of ME Design for resilient modulus).

- Aggregate gradation for flexible layers (one of the important material properties needed to calculate dynamic modulus for input level 2 and 3 in ME method. It should be noted that the Level 3 estimates of dynamic modulus from mix gradation, volumetric, and binder characteristics).
- Environmental conditions (different locations including several environmental factors like air temperature, precipitation, sunshine…) (only for IC3).
- Water table depth (only for IC3).
- Vehicle class distribution (only for IC3).

The following tasks should be done in this chapter:

- The priority of distresses for analysis based on damage calculation will be identified.
- The influence of inputs on main criteria (characterized by damage calculation) will be analyzed by preparing related diagrams for each input (related to each main distress).

5.2 Sensitivity of ME Indicators to Design Inputs for Selected Road Projects

5.2.1 General Considerations

One of the important steps in MEPDG implementation for the road agencies is evaluating the sensitivity of obtained distresses to design inputs' variations. So, the results can be used to improve the procedure, and help to define priorities for the implementation and calibration tasks.

AASHTOWare Pavement ME Design™ can create sensitivity projects based on the defined primary projects to analyze the sensitivity of performance predictions for some design inputs with adjustable variation range and increment number. For this parametric study, the suggested properties in the sensitivity section can adjust between allowable and/or recommended minimum and maximum values by ME Design and the increments are adjusted based on the type pf input and the necessity (25 increments are the maximum adjustable value in the ME Design). When percentage variation around the reference value is not possible, distinct cases are selected for comparison purposes (i.e., mixture type, vehicle class distribution, climate conditions, etc.).

Figure 5-1 shows the illustration of related section in ME Design for some sensitivity analysis.

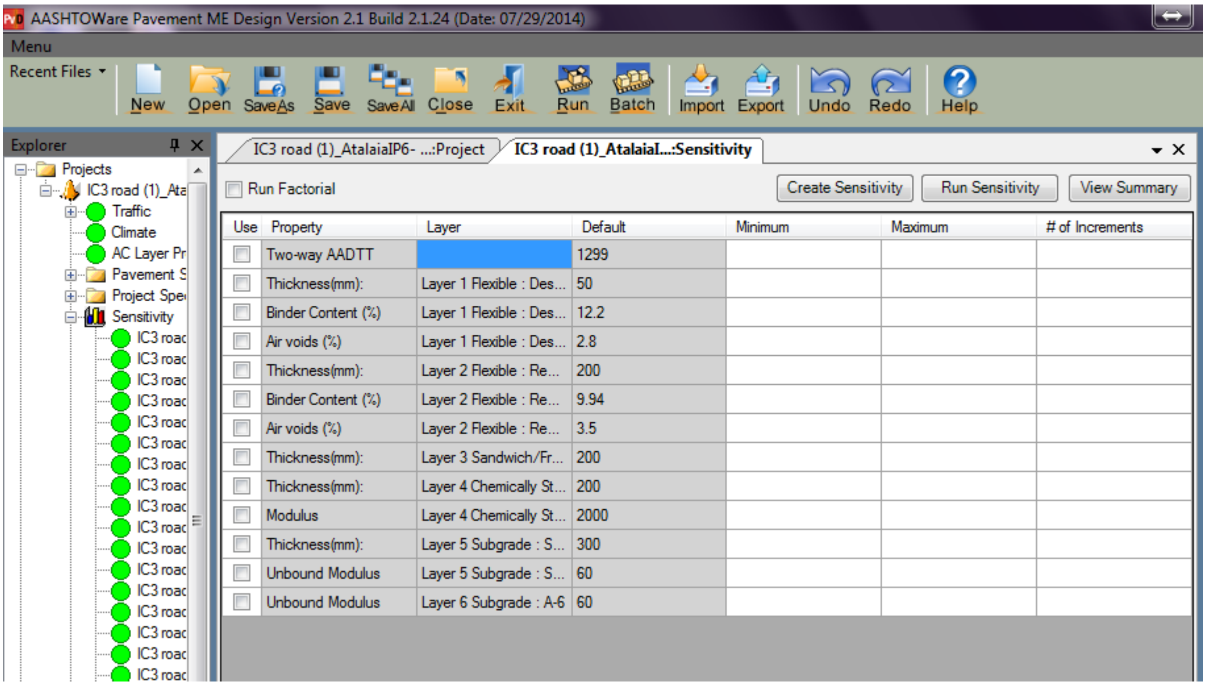


Figure 5-1: AASHTOWare Pavement ME Design™ sensitivity analysis

As it is mentioned earlier in this chapter, the AASHTOWare Pavement ME Design™ version used for this study is 2.1, however, it should be noted that the current version available for the ME Design™ is 2.5.3. There are some changes in the recent versions of ME Design™ (specifically in Ver. 2.2) including some revises in the calibration coefficients (for behavior models) and the pavement design types that may affect the results. For example, one of the additional design types included in the ME Design™ 2.2 that make some concerns toward the IC3 road project (which is categorized as a flexible pavement in Ver. 2.1) is defined as follows(AASHTO, 2015a):

New Semi-Rigid Pavement: A semi-rigid pavement is composed of a flexible layer (e.g., HMA) and a rigid layer (e.g., cement - treated base [CTB], cement stabilized base [CSB], rolled - compacted concrete [RCC], or lean mix concrete).

This additional design type considers IC3 pavement as a semi-rigid pavement composed of different structure and layers leading to different performance behavior and different analysis results. Due to maintain the uniformity of the study and the results, it is preferred to accomplish the study including sensitivity analysis with the ME Design™ 2.1.

The Table 5-1 shows the summary of information for selected roads for this sensitivity analysis.

Table 5-1: Summary of information for the selected flexible pavements in Portugal

Location (Virtual Station)		Tomar	Tomar	Evora
Road name		IC3	IP6	EN254
Construction year		2006	2003	2004
Design life (year)		20	20	20
AADTT		1299	1734	154
Surface	**Type**	Bituminous Mixture	Bituminous Mixture	Bituminous Mixture
	Thickness (mm)	50	60	50
	Symbol[1]	L1	L1	L1
Binder	**Type**	-	Bituminous Mixture	Bituminous Macadam
	Thickness (mm)	-	60	100
	Symbol	-	L2	L2
Base	**Type**	(Binder+Base) Bituminous Macadam	Bituminous Macadam	ABGE[2]
	Thickness (mm)	200	160	200
	Symbol	L2	L3	L3
Subbase	**Type**	Sandwich Granular Base in ABGE	ABGE	Crushed Gravel
	Thickness (mm)	200	200	200
	Symbol	L3	L4	L4
Subgrade 1	**Type**	Soil Cement	-	-
	Thickness (mm)	200	-	-
	Symbol	L4	-	-
Subgrade 2	**Type**	Soil (A-6)	Treated Soil	Natural Soil (soil selection)
	Thickness (mm)	300	200	200
	Symbol	L5	L5	L5
Subgrade 3	**Type**	Soil (A-6)	Treated Soil	Natural Soil (soil selection)
	Thickness (mm)	Semi infinite	Semi infinite	Semi infinite
	Symbol	L6	L6	L6

[1] Applied name in sensitivity study
[2] "Agregado Britado de Granulometria Extensa" (Extensive crushed aggregate)

The Table 5-2 shows the selected ME inputs for this sensitivity study with the range of variation for the selected roads.

Table 5-2: Inputs used in the sensitivity analysis for material properties and traffic loading

Variable Name	IC3				IP6				EN254			
	Base	Min	Max	# Incr.[1]	Base	Min	Max	# Incr.	Base	Min	Max	# Incr.
Two-way AADTT	1299	909	1689	12	1734	1213	2255	12	154	107	201	12
L1 thickness (mm)	50	25.4	100	25	60	158.3	500	25	50	25.4	500	25
L2 thickness (mm)	200	158.3	500	25	60	158.3	500	25	100	25.4	500	25
L3 thickness (mm)	200	100	500	25	160	272.2	500	25	200	120.3	500	25
L4 thickness (mm)	200	100	500	25	200	25.4	500	25	200	25.4	500	25
L5 thickness (mm)	300	63.4	500	25	200	25.4	500	25	200	63.4	500	25
L3 Resilient modulus (MPa)	300	210	390	12	-	-	-	-	290	203	377	12
L4 Resilient modulus (MPa)	2000	1400	2600	12	130	91	169	12	120	84	156	12
L5 Resilient modulus (MPa)	60	42	78	12	60	42	78	12	80	56	104	12
L6 Resilient modulus (MPa)	60	42	78	12	60	42	78	12	80	56	104	12
L1 Air voids (%)	2.8	1.96	3.64	12	5	3.5	6.5	12	6.3	4.41	8.19	12
L2 Air voids (%)	3.5	2.45	4.55	12	5	3.5	6.5	12	5.7	3.99	7.41	12
L3 Air voids (%)	-	-	-	-	5	3.5	6.5	12	-	-	-	-
L1 Binder Content (%)	12.2	8.54	15.86	12	11	7.7	14.3	12	11.6	8.12	15.08	12
L2 Binder Content (%)	9.94	6.96	12.92	12	11	7.7	14.3	12	9.3	6.51	12.09	12
L3 Binder Content (%)	-	-	-	-	10	7	13	12	-	-	-	-

[1] Number of increments

The inputs of AADTT, modulus, air voids and binder content in Table 5-2 vary from -30 % to 30 % with the intervals of 5 %. It should be noted that the variations of inputs are not applied simultaneously, and each input is studied separately while the other inputs in ME design are considered default or base value of the project. As it is mentioned earlier, the term of unbound modulus used in text, tables and diagrams represents the resilient modulus for unbound layer in this chapter same as the term used in the sensitivity section of ME Design for resilient modulus (Figure 5-1).

The logic behind the selection of these inputs comes from the results and findings of sensitivity studies which were accomplished since the release of the original research version of the MEPDG (Version 0.7) in July 2004 by several researchers. Those sensitivity evaluation studies typically focused on the sensitivity of some performance behaviors and/or some input parameters. While there are some problems with previous sensitivity evaluations in terms of scope or adopted approach, their results can lead us to have a proper decision in choosing the most important input parameters for the current sensitivity analysis among the several ME inputs. So, the analysis would be more efficient in the available time (R. Li, 2013).

The other inputs studied in the sensitivity analysis are categorical types rather than continuous. These distinct cases with their related variations are mentioned in the Table 5-3. These inputs are also studied separately while the other ME inputs considered without variation. Details of the traffic inputs such as axle load distributions, seasonal and daily traffic distributions, axle geometric configuration, tire pressure, traffic speed and traffic growth rates are not considered in this study.

Table 5-3: Inputs used in the sensitivity analysis for distinct cases

Input Type	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**
Aggregate gradation type	Base (Project value)	Dense graded	Free drainage	Open graded	-
Environmental conditions	Porto	Coimbra	Lisbon	Beja	Tomar (Base)
Water table depth (m)	5.5	10 (Base)	15	20	-
Vehicle Class Distribution	Vehicle Dist. 1, TTC[1]=9 (Base)	Vehicle Dist.2, TTC=11	Vehicle Dist.3, TTC=3	-	-

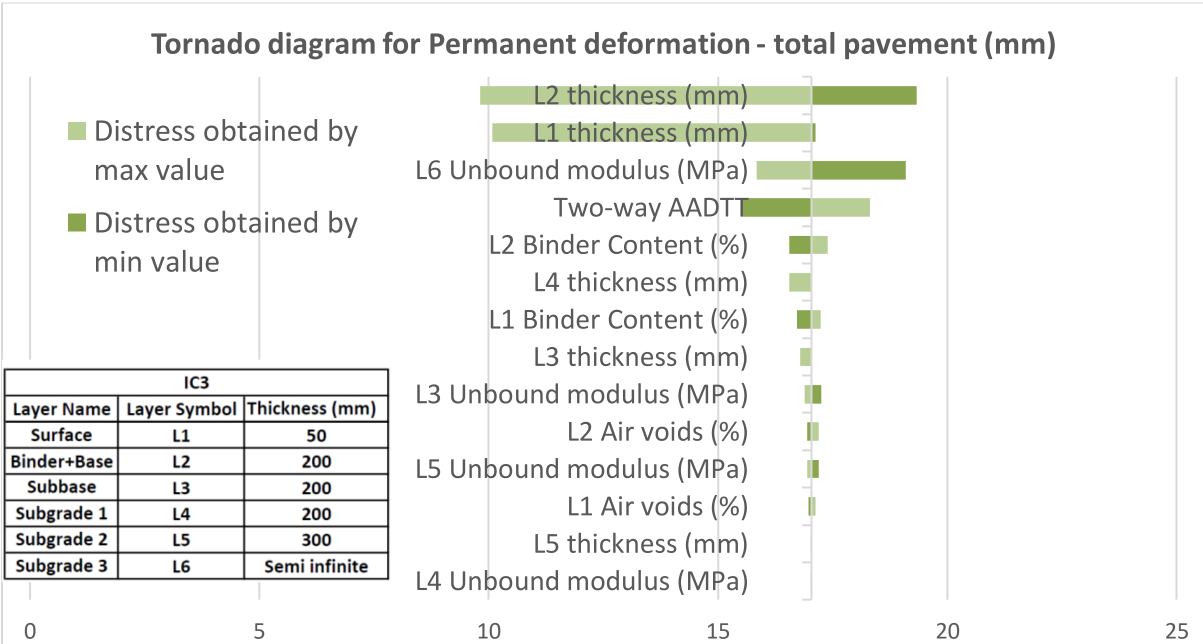
[1] Truck Traffic Classification

5.2.2 IC3 _Sensitivity Analysis

- **Sensitivity of Continuous Inputs**

Based on damage results, the sensitivity of the three main criteria (total permanent deformation, AC bottom-up fatigue cracking and stabilized layer fatigue fracture) to variations of the design inputs will be evaluated in this section. The prepared tornado charts in this section show the priority and importance of continuous inputs to study their effect on performance behavior for IC3 pavement.

Based on Figure 5-2, the variations of thickness for layer 2, thickness for layer 1, Resilient modulus of layer 6 and AADTT respectively show the most difference between maximum and minimum value in the total permanent deformation for the adopted ranges.



IC3		
Layer Name	Layer Symbol	Thickness (mm)
Surface	L1	50
Binder+Base	L2	200
Subbase	L3	200
Subgrade 1	L4	200
Subgrade 2	L5	300
Subgrade 3	L6	Semi infinite

Figure 5-2: Tornado chart for the sensitivity analysis of total permanent deformation_IC3 road

The Figure 5-3 shows that the variations of thickness for layer 1, thickness for layer 2, binder content of Layer 2 and air voids layer 2 respectively show the most difference between maximum and minimum value in AC bottom-up fatigue cracking.

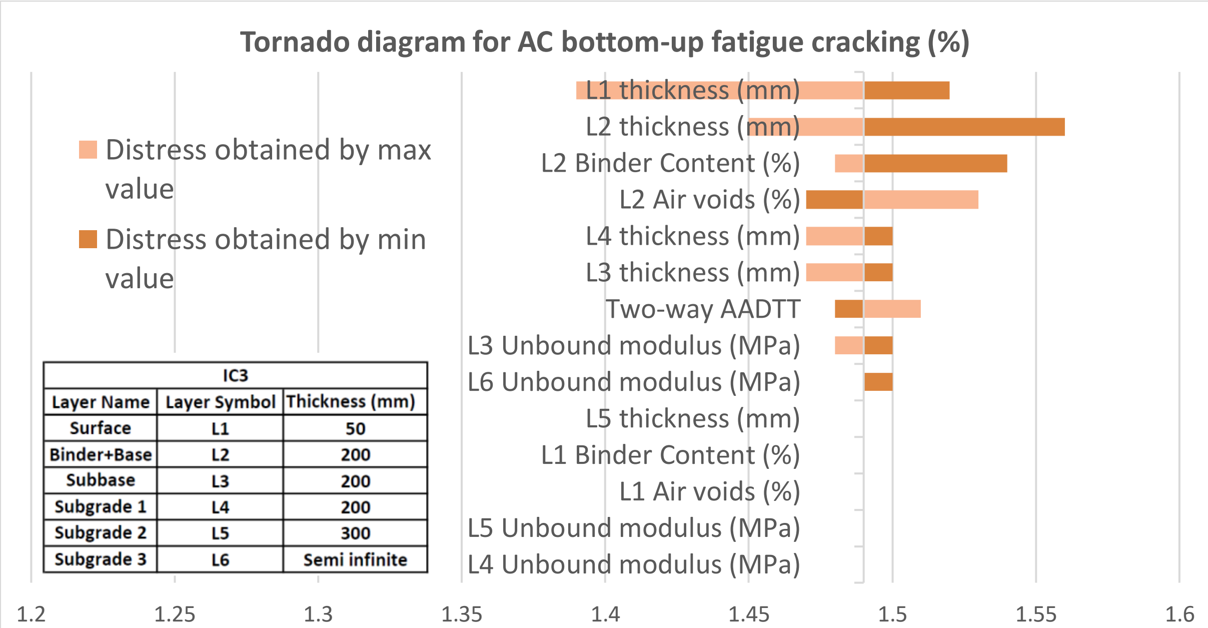


Figure 5-3: Tornado chart for the sensitivity analysis of AC bottom-up fatigue cracking_IC3 road

Regarding Figure 5-4, the variation of AADTT, thickness for layer 4, thickness for layer 2 and thickness for layer 1 to fatigue fracture will be studied.

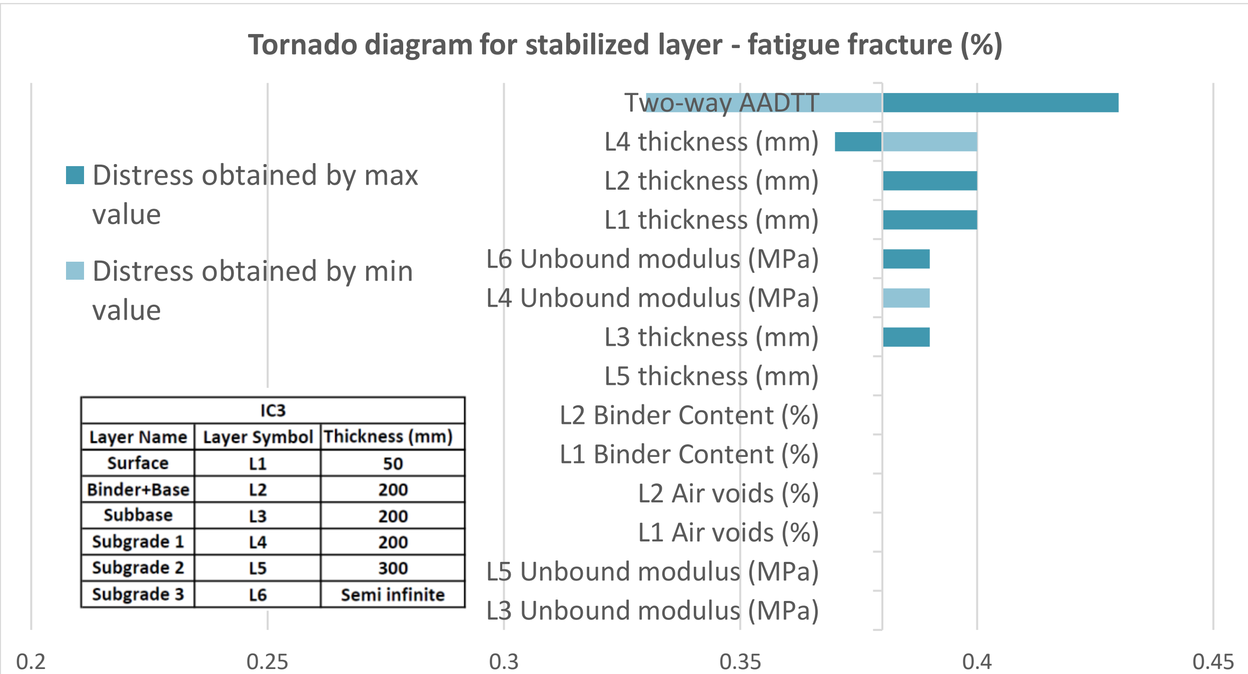


Figure 5-4: Tornado chart for the sensitivity analysis of Stabilized layer fatigue fracture_IC3 road

The variations' effect of the input data is explained by a chosen metric in this study. The adopted metric for quantifying the sensitivity of performance behaviors to inputs in this study is a normalized sensitivity index (NSI) which is defined as the percentage change of predicted distress relative to its design limit caused by a given percentage change in the design input (R. Li, 2013; Schwartz et al., 2014, 2013).

$$NSI = \frac{\Delta Y_{ji}.X_{ki}}{\Delta X_{ki}.DL_j} \qquad Equation\ 5\text{-}1$$

Where

$\Delta X_{ki}: Change\ in\ design\ input\ k\ about\ X_{ki}$

$\Delta Y_{ji}: Change\ in\ predicted\ distress\ j\ corresponding\ to\ \Delta X_{ki}$

$DL_j: design\ limit\ for\ distress\ j$

For example, if NSI = -0.3 for the sensitivity of total permanent deformation to thickness of surface layer shows that a 10 % reduction in thickness will increase total rutting by 3 % of its design limit of 20 mm. In other words, the 10 % reduction in thickness will increase rutting by (-0.10) × (-0.3) × 20 = 0.6 mm. Based on this, the following design input sensitivity categories are defined for the sensitivity results:

- hypersensitive (NSI≥ 5);
- very sensitive (1 ≤ NSI < 5);
- sensitive (0.1 ≤ NSI < 1);
- insensitive (NSI< 0.1).

In this sensitivity study, the analyzing of relative magnitudes of the NSI values are more desirable than their precise values in order to better evaluate which pavement design inputs are most effective.

Based on the definition for NSI, the max normalized sensitivity index for the most effective inputs in Figure 5-2, Figure 5-3 and Figure 5-4 are calculated as it is shown in Table 5-4.

Table 5-4: Ranking of New HMA design inputs by max NSI value for IC3

Design Input	**Max NSI Value (%)**			**Max NSI**	**Category of Sensitivity**
	Total permanent deformation	**AC bottom-up fatigue cracking**	**Stabilized layer fatigue fracture**		
L2 Thickness	-0.61	-0.01	0.01	-0.61	Sensitive
L1 Thickness	-0.30	0.00	0.00	-0.30	Sensitive
L6 unbound modulus	-0.30	0.00	0.00	-0.30	Sensitive
AADTT	0.25	0.00	0.01	0.25	Sensitive
L2 Binder Content	0.08	0.00	0.00	0.08	Insensitive
L1 Binder Content	0.05	0.00	0.00	0.05	Insensitive
L4 Thickness	-0.03	0.00	0.00	-0.03	Insensitive
L2 Air voids	0.03	0.00	0.00	0.03	Insensitive
L5 Thickness	-0.01	0.00	0.02	0.02	Insensitive
L1 Air voids	0.02	0.00	0.00	0.02	Insensitive
L3 Thickness	0.01	0.00	0.00	0.01	Insensitive
L3 unbound modulus	0.00	0.00	0.00	0.00	Insensitive
L4 unbound modulus	0.00	0.00	0.00	0.00	Insensitive
L5 unbound modulus	0.00	0.00	0.00	0.00	Insensitive

Thickness of Layer 2 (Binder + Base Layer)

The thickness of layer 2 is varied between 158.29 mm and 500 mm. The max NSI value for the variation of thickness L2 is -0.61 to total rutting as it is shown in Table 5-4. Increasing the thickness of layer 2, decreases the total rutting with the average NSI of -0.40. However, total permanent deformation in the bigger values of thickness L2 is less sensitive to the thickness L2 variations.

The Figure 5-5 shows the mentioned trend.

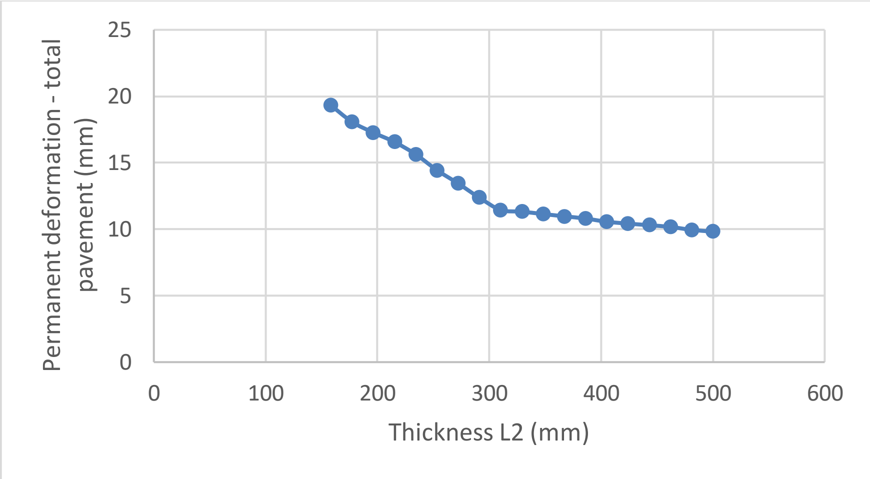


Figure 5-5: Sensitivity of total rutting to second Layer thickness variation_IC3.

Thickness of Layer 1 (Surface Layer)

The thickness of layer 1 is varied between 25.4 mm and 500 mm. The max NSI value for the variation of thickness L1 is -0.3 to total rutting as it is shown in Table 5-4.

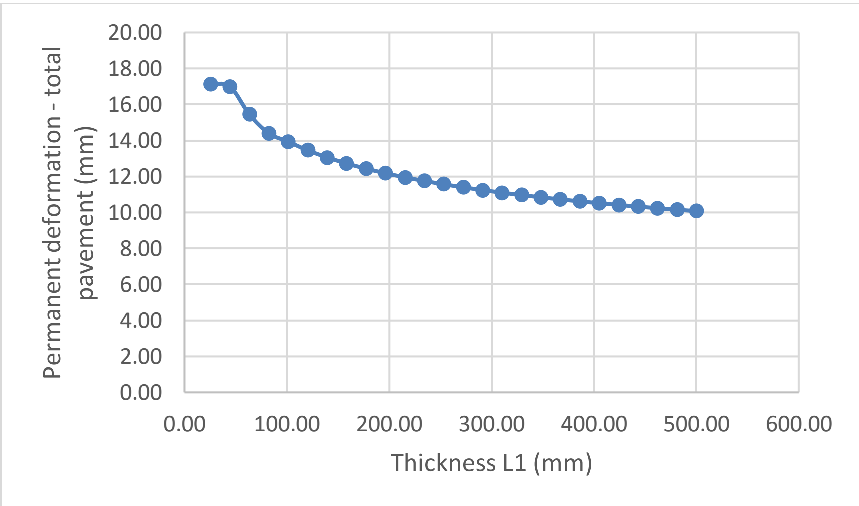


Figure 5-6: Sensitivity of total rutting to first Layer thickness variation_IC3.

As Figure 5-6 shows, increasing the thickness of layer 1, decreases the total rutting with the average NSI of -0.08 which does not imply significant changes. It should be noted that total permanent deformation in the bigger values of thickness L1 is even less sensitive to the thickness L1 variations.

Resilient Modulus of layer 6 (Subgrade)

The resilient modulus of layer 6 is varied between 42 MPa and 78 MPa. The max NSI value for the variation of resilient modulus L6 is -0.3 to total rutting as it is shown in Table 5-4. As Figure 5-7 shows, increasing the resilient modulus L6, decreases the total rutting with the average NSI of -0.3. It should be noted that total permanent deformation in the bigger values of resilient modulus L6 is less sensitive to the resilient modulus L6 variations.

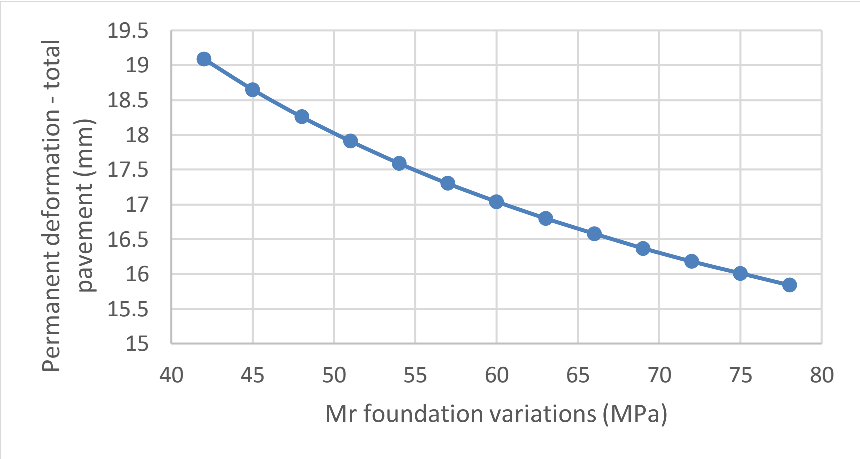


Figure 5-7: Sensitivity of total rutting to Resilient Modulus of layer 6 (subgrade)_IC3.

AADTT

The AADTT is varied between 909 and 1689. The max NSI value for the variation of AADTT is 0.25 to total rutting as it is shown in Table 5-4. As Figure 5-8 shows, increasing the AADTT, increases the total rutting with the average NSI of 0.2. However, total permanent deformation in the bigger values of AADTT is less sensitive to the AADTT variations.

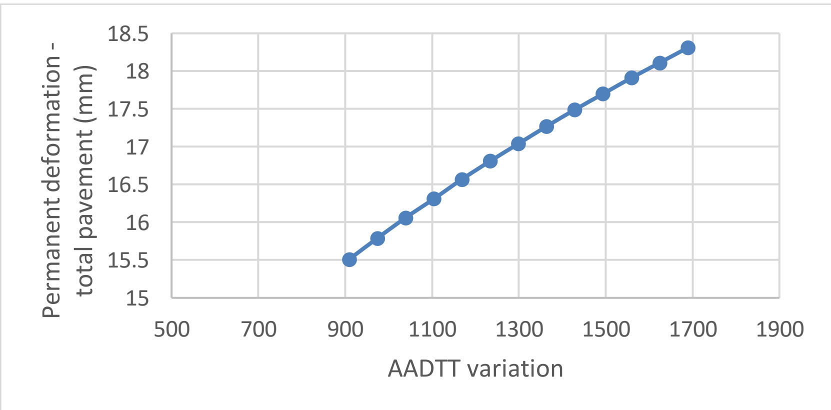


Figure 5-8: Sensitivity of total rutting to AADTT_IC3.

- **Sensitivity of Categorical Inputs**

The other types of inputs considered for the sensitivity study are categorical inputs as it is mentioned in Table 5-3. For each input, four different cases are studied for the three selected roads. Due to time limitations, the aggregate gradation type is studied for all three roads and the other three inputs are studied only for IC3.

Aggregate Gradation Type for Flexible Layers

Based on the availability of modulus data in the three roads, the calculation of dynamic modulus would be for level 3 data which includes typical rheological properties of the asphalt binder grade (e.g. penetration grade) and the aggregate gradation of the asphalt concrete mixture. So, it is important to study the sensitivity of main criteria to the variation of aggregate gradation of flexible layers. It should be noted that levels 2 and 3 data for estimating dynamic modulus provide reasonable results only for conventional Hot Mix Asphalt mixtures (AASHTO, 2014). In this study, the applied asphalt for all the three selected roads are conventional.

The Table 5-5 shows the selected cases for aggregate gradation types for two flexible layers of IC3.

Table 5-5: The selected cases for aggregate gradation types for flexible layers of IC3

Sieve Gradation mm	Percent passing for Layer 1				Percent passing for Layer 2			
	Case1 (Base)	Case2 (Dense graded)	Case3 (Open graded)	Case4 (Free drainage)	Case1 (Base)	Case2 (Dense graded)	Case3 (Open graded)	Case4 (Free drainage)
19	100	100	100	100	99.1	100	100	100
9.5	73.4	80	72.5	55	72	80	72.5	55
4.75	51	60	45	22.5	40.4	60	45	22.5
0.075	7.4	7	5	4.5	8.3	7	5	4.5

Sensitivity of Total Rutting to Different Gradation

Increasing the gap between the aggregate particles or reducing the density of gradation increases the total rutting.

Figure 5-9 and Figure 5-10 show that the type of gradation in the mixture of flexible layers slightly influences total rutting (however, it is not so significant).

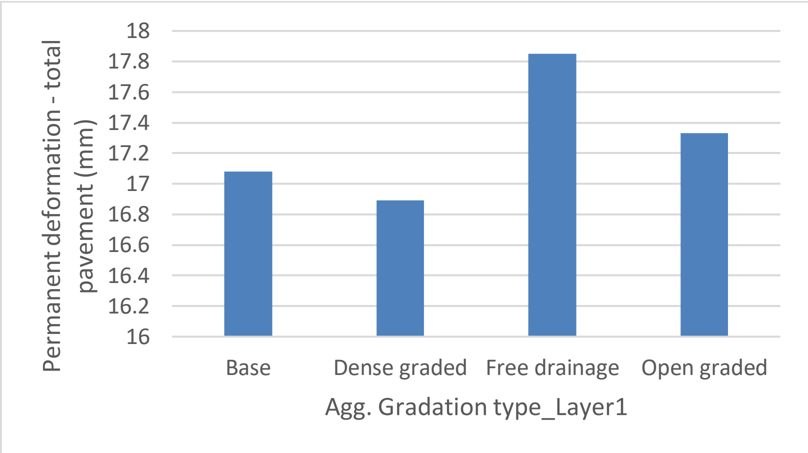


Figure 5-9: Sensitivity of total rutting to aggregate gradation layer 1_IC3.

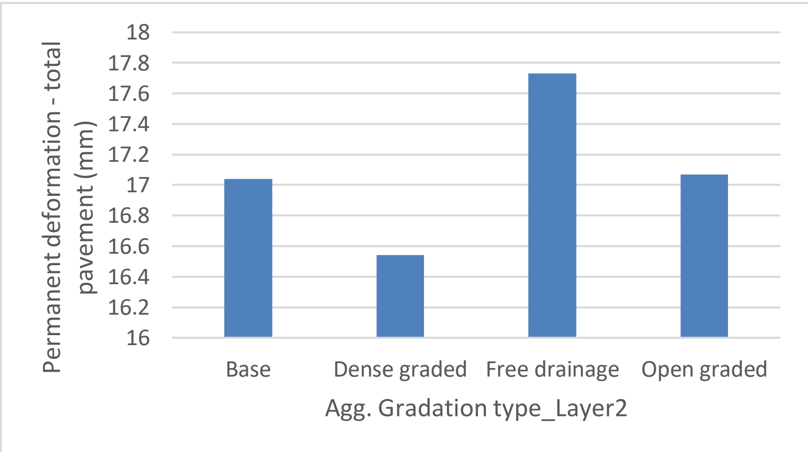


Figure 5-10: Sensitivity of total rutting to aggregate gradation layer 2_IC3.

Sensitivity of Stabilized Layer - Fatigue Fracture to Different Gradation

The changes in the gradation type does not have any significant effect on the stabilized layer - fatigue fracture.

Sensitivity of AC Bottom-Up Fatigue Cracking to Different Gradation

The changes in gradation type (density) is only significant when the gap between the aggregate particles are too much or the density of gradation is very low. So, increasing the gaps, increase the AC B-U fatigue cracking.

The Figure 5-11 and Figure 5-12 show the mentioned trend for AC B-U cracking.

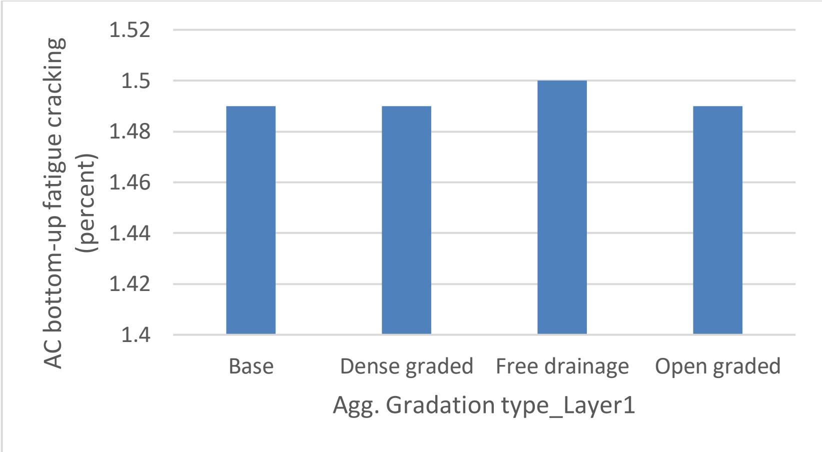


Figure 5-11: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 1_IC3.

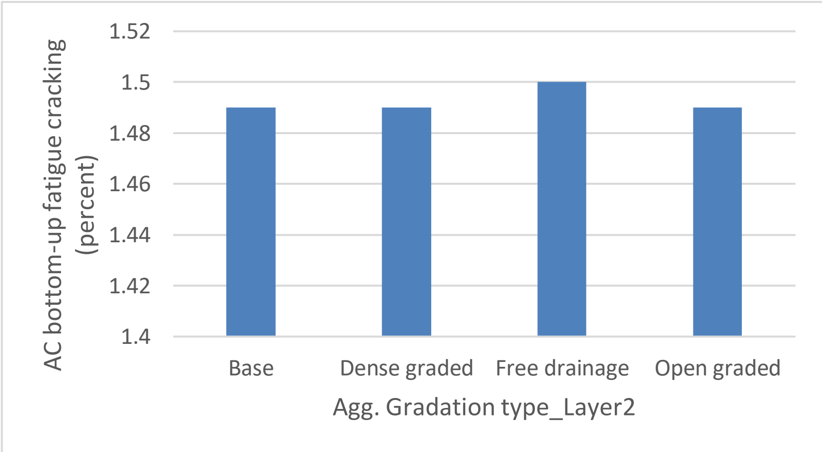


Figure 5-12: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 2_IC3.

Environmental conditions

The climate impacts are analyzed by simulating four different locations within Portugal for IC3 road (It should be noted that if there were more climate data for other regions of Portugal, it would create the better diversity of climate conditions. However, only the following four locations were available at the time of sensitivity analysis). These locations are: Porto; Coimbra; Lisbon; Beja.

As for the variation of climate conditions, ME approach provided the results for the sensitivity analysis that practically are a bit away from the expectations about these regions. It could be that the .hcd files are not well calibrated to the real climate conditions of

Portuguese regions in some factors or there are some exceptional cases in climate conditions of these regions over the 33 years data records (i.e., these exceptional cases do not generally affect the climate conditions of regions while their effect is considered in ME Design). Figure 5-13 is an example of the sensitivity results of total rutting for the regions.

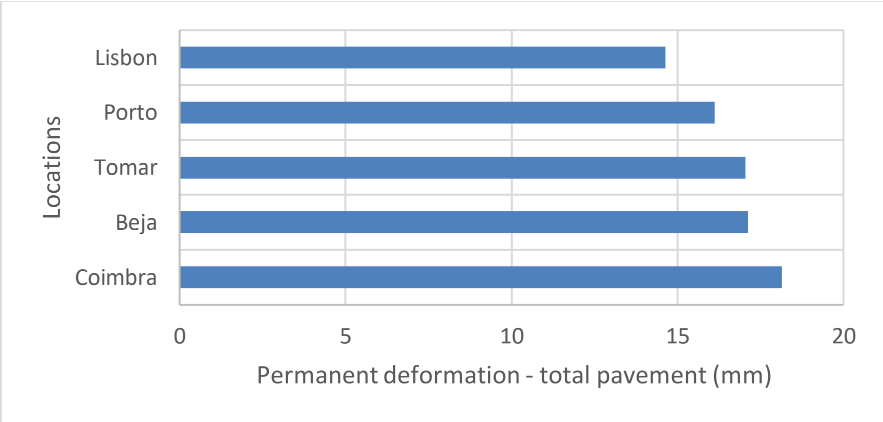


Figure 5-13: Sensitivity of total rutting to different environmental conditions_IC3.

Water Table Depth

For this input, three different scenarios besides the base value for IC3 project are considered for sensitivity analysis. The Table 5-6 shows the depth to water table for each case.

Table 5-6: Selected depths for water table depth Sensitivity Analaysis_IC3.

Case No.	Case1	Case2	Case3	Case4
Water table depth (m)	5.5	10	15	20

The water table depth is an important input because ME Design has the capability, using the EICM, to estimate changes in the resilient modulus of the unbound layers and foundation soils over time. However, for most pavement designs, water table depths greater than 6 meters below the planned surface elevation will have a minimal effect of the pavement distress predictions. It should be noted that the ME Design only predicts the effects of water moving upward into the pavement layers from ground water tables located close to the surface (AASHTO, 2015b). As it is expected, the sensitivity results indicate the changes of water table depth do not show any significant changes in total rutting or AC B-U fatigue cracking.

Vehicle Class distribution

Three different default distributions for IC3 road are selected as it is shown in Table 5-7 and Table 5-8.

Table 5-7: Selected Vehicle Class Distribution_IC3

Vehicle Distribution .1 Principal Arterials_other, TTC=9	Vehicle Distribution .2 Principal Arterials_other, TTC=11	Vehicle Distribution .3 Principal Arterials_other, TTC=3
Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks.	Mixed truck traffic with a higher percentage of single-trailer trucks.	Predominantly single-trailer trucks.

Table 5-8: Number of axles per truck class and vehicle distribution by traffic level _IC3

Vehicle Class	Axle Type			Distribution by traffic (%)		
	Single	Tandem	Tridem	TTC 3	TTC9	TTC11
Class 4	1.62	0.39	0.00	0.9	3.3	1.8
Class 5	2.00	0.00	0.00	11.6	34	24.6
Class 6	1.02	0.99	0.00	3.6	11.7	7.6
Class 7	1.00	0.26	0.83	0.2	1.6	0.5
Class 8	2.38	0.67	0.00	6.7	9.9	5
Class 9	1.13	1.93	0.00	62	36.2	31.3
Class 10	1.19	1.09	0.89	4.8	1	9.8
Class 11	4.29	0.26	0.06	2.6	1.8	0.8
Class 12	3.52	1.14	0.06	1.4	0.2	3.3
Class 13	2.15	2.13	0.35	6.2	0.3	15.3

As it is shown in Table 5-8, classes 5 and 9 have the major parts in the distributions, representing respectively single unit trucks and single trailer trucks. The results in the Figure 5-14 show that the increase in the percentage of single trailer trucks and decrease in the percentage of single unit trucks have a significant influence on the total permanent deformation (so the change from dist. 1 to dist. 3 increases the total rutting).

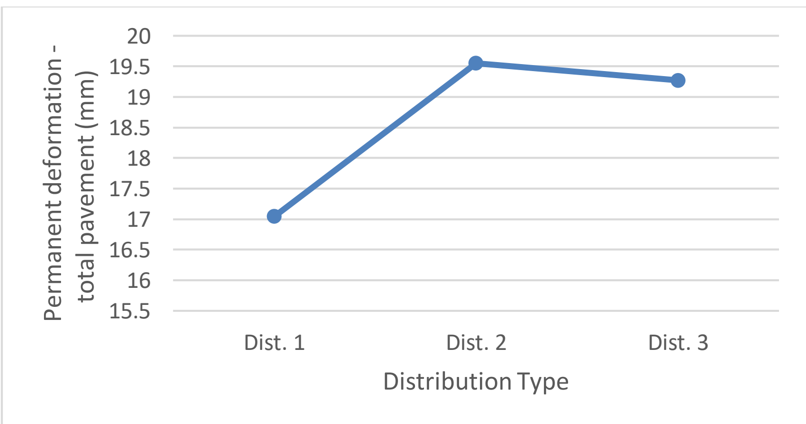


Figure 5-14: Sensitivity of total rutting to different vehicle class distributions_IC3.

There are no significant changes in the AC B-U fatigue cracking.

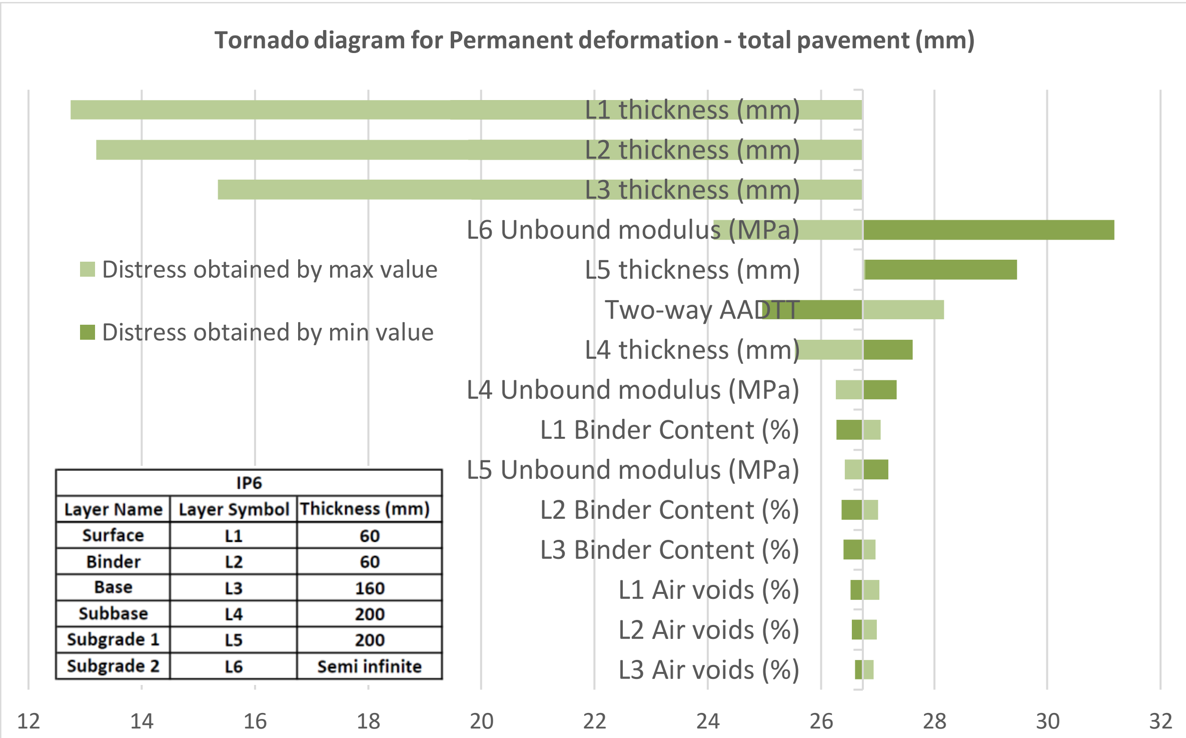
5.2.3 IP6_Sensitivity Analysis

The sensitivity of total permanent deformation and AC bottom-up fatigue cracking to variations of the design inputs will be evaluated for IP6 road.

- **Sensitivity of Continuous Inputs**

The tornado charts in this section show the priority of continuous inputs to study their effect on performance behavior for IP6 pavement.

Based on the Figure 5-15, the variations of thickness for the three first layers, modulus of layer 6, thickness L5 and AADTT have the most effectiveness in the total permanent deformation.



IP6		
Layer Name	Layer Symbol	Thickness (mm)
Surface	L1	60
Binder	L2	60
Base	L3	160
Subbase	L4	200
Subgrade 1	L5	200
Subgrade 2	L6	Semi infinite

Figure 5-15: Tornado chart for the sensitivity analysis of total permanent deformation_IP6 road

The Figure 5-16 shows that the variations of binder for layer 3, thicknesses for layers (1, 2, 4, 3) , air voids for layer 3, thickness for layer 5, AADTT, and modulus of layer 4 have the most effectiveness in AC bottom-up fatigue cracking.

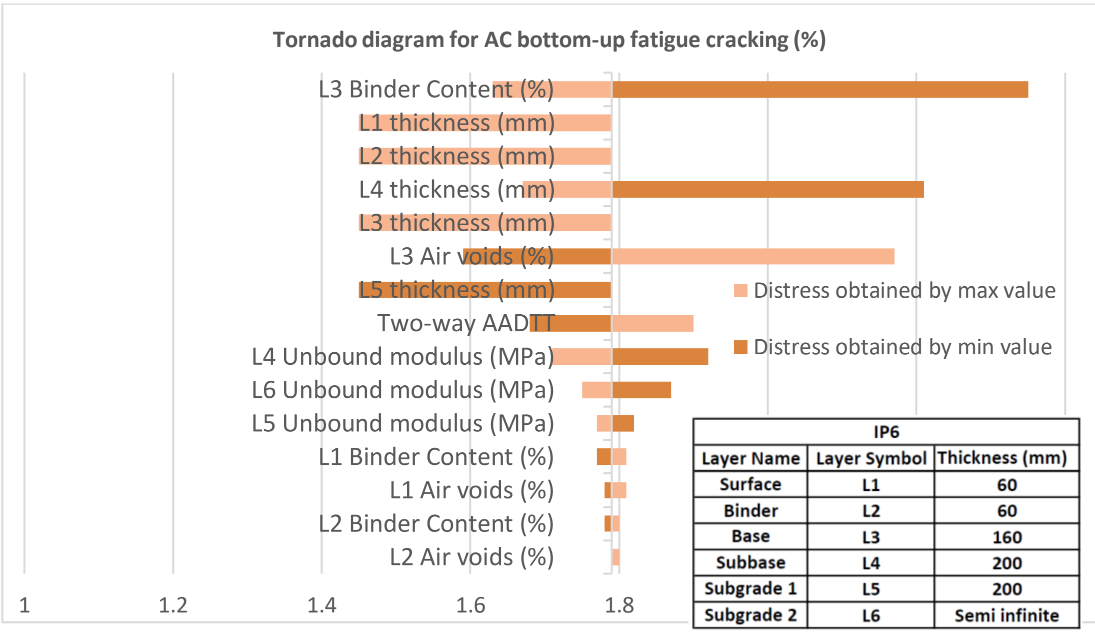


Figure 5-16: Tornado chart for the sensitivity analysis of AC bottom-up fatigue cracking_IP6 road

The Table 5-9 shows the max NSI (normalized sensitivity index) for the most effective inputs in Figure 5-15 and Figure 5-16 for total rutting and AC B-U fatigue cracking.

Table 5-9: Ranking of New HMA design inputs by max NSI value for IP6.

Design Input	**MAX NSI for inputs (%)**		**Max NSI**	**Category of Sensitivity**
	Total rutting	**AC bottom-up cracking**		
L6 unbound modulus	-0.70	0.00	-0.70	Sensitive
L3 Thickness	-0.49	-0.01	-0.49	Sensitive
AADTT	0.30	0.01	0.30	Sensitive
L1 Thickness	-0.22	0.00	-0.22	Sensitive
L2 Thickness	-0.21	0.00	-0.21	Sensitive
L5 Thickness	-0.18	0.01	-0.18	Sensitive
L4 unbound modulus	-0.10	0.00	-0.10	Sensitive
L5 unbound modulus	-0.10	0.00	-0.10	Sensitive
L1 Binder Content	0.08	0.00	0.08	Insensitive
L4 Thickness	-0.07	-0.01	-0.07	Insensitive
L2 Binder Content	0.06	0.00	0.06	Insensitive
L3 Binder Content	0.06	-0.04	0.06	Insensitive
L1 Air voids	0.05	0.00	0.05	Insensitive
L2 Air voids	0.04	0.00	0.04	Insensitive
L3 Air voids	0.03	0.03	0.03	Insensitive

Resilient Modulus of layer 6 (Subgrade)

The resilient modulus of layer 6 is varied between 42 MPa and 78 MPa. The max NSI value for the variation of resilient modulus L6 is -0.7 to total rutting as it is shown in Table 5-9. As Figure 5-17 shows, increasing the resilient modulus L6, decreases the total rutting with the average NSI of -0.6. It should be noted that total permanent deformation in the bigger values of resilient modulus L6 is less sensitive to the resilient modulus L6 variations.

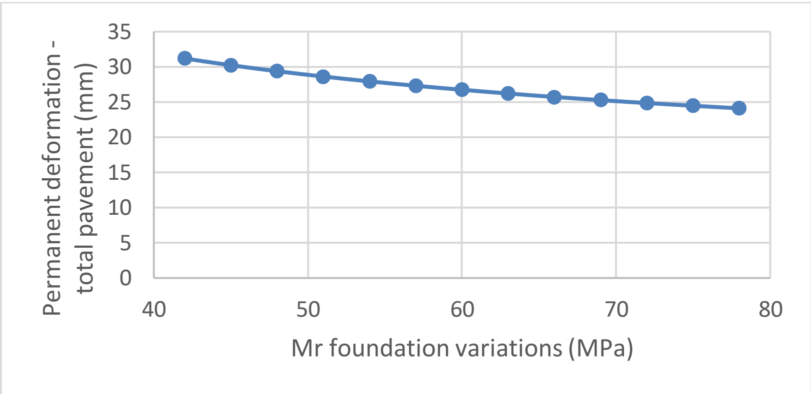


Figure 5-17: Performance sensitivity to Resilient Modulus of layer 6 (subgrade)_IP6.

Thickness of layer 3 (Base in Bituminous Macadam)

The thickness of layer 3 is varied between 272.192 mm and 500 mm. The max NSI value for the variation of thickness L3 is -0.49 to total rutting as it is shown in Table 5-9. As Figure 5-18 shows, increasing the thickness of layer 3, decreases the total rutting with the average NSI of -0.35. It should be noted that total permanent deformation in the bigger values of thickness L3 is less sensitive to the thickness L3 variations.

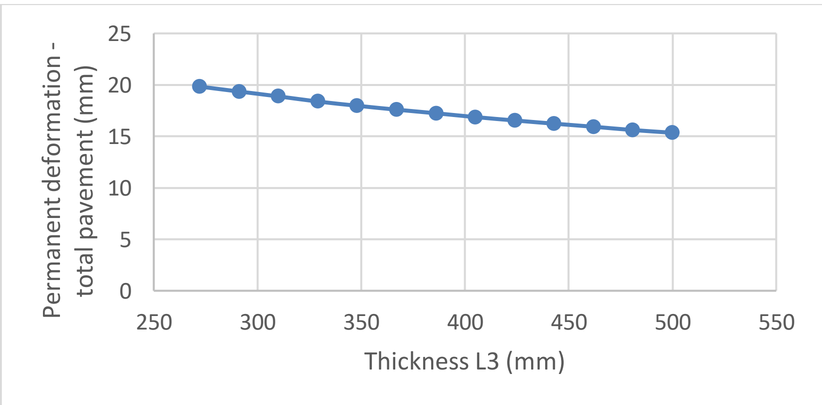


Figure 5-18: Sensitivity to third Layer thickness_IP6.

AADTT

The AADTT is varied between 1213 and 2255. The max NSI value for the variation of AADTT is 0.3 to total rutting as it is shown in Table 5-9. As Figure 5-19 shows, increasing the AADTT, increases the total rutting with the average NSI of 0.26. However, total permanent deformation in the bigger values of AADTT is less sensitive to the AADTT variations.

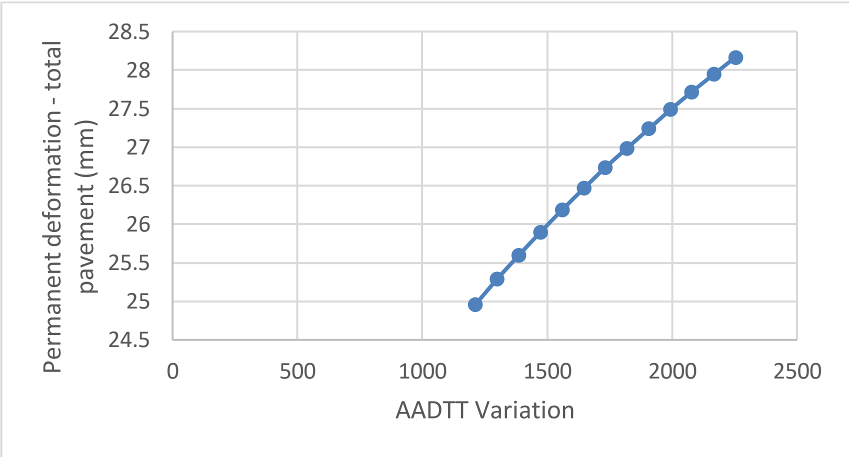


Figure 5-19: Sensitivity to AADTT_IP6.

Thickness of layer 1 (Surface Layer)

The thickness of layer 1 is varied between 158.288 mm and 500 mm. The max NSI value for the variation of thickness L1 is -0.22 to total rutting as it is shown in Table 5-9. As Figure 5-20 shows, increasing the thickness of layer 1, decreases the total rutting with the average NSI of -0.14. It should be noted that total permanent deformation in the bigger values of thickness L1 is less sensitive to the thickness L1 variations.

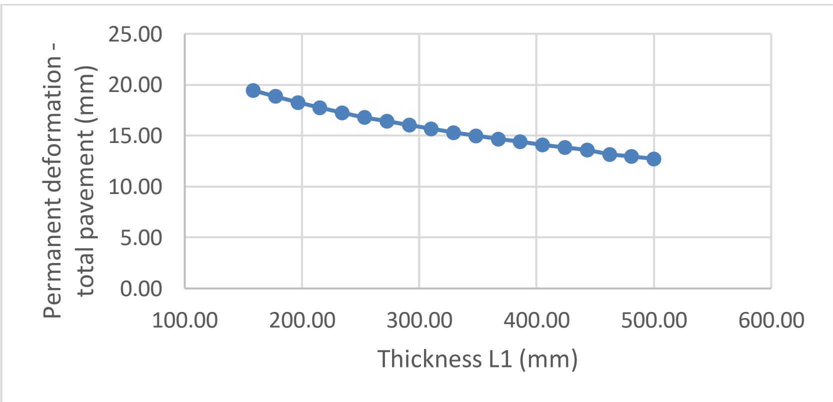


Figure 5-20: Sensitivity to Surface Layer thickness_IP6.

Thickness of layer 2 (Binder Layer)

The thickness of layer 2 is varied between 158.288 mm and 500 mm. The max NSI value for the variation of thickness L2 is -0.21 to total rutting as it is shown in Table 5-9 . As Figure 5-21 shows, increasing the thickness of layer 2, decreases the total rutting with the average NSI of -0.13. It should be noted that total permanent deformation in the bigger values of thickness L2 is less sensitive to the thickness L2 variations.

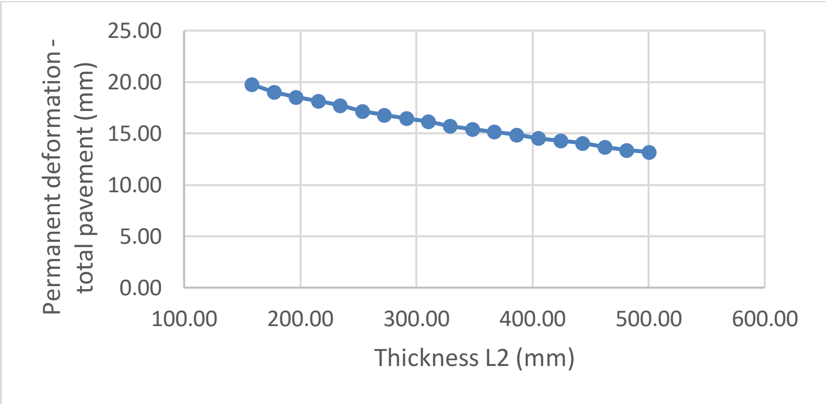


Figure 5-21: Sensitivity to second Layer thickness_IP6.

Thickness of layer 5 (Subgrade)

The thickness of layer 5 is varied between 25.4 mm and 500 mm. The max NSI value for the variation of thickness L5 is -0.18 to total rutting as it is shown in Table 5-9. As Figure 5-22 shows, increasing the thickness of layer 5, decreases the total rutting with the average NSI of -0.01. It should be noted that total permanent deformation in the bigger values of thickness L5 is insensitive to the thickness L5 variations.

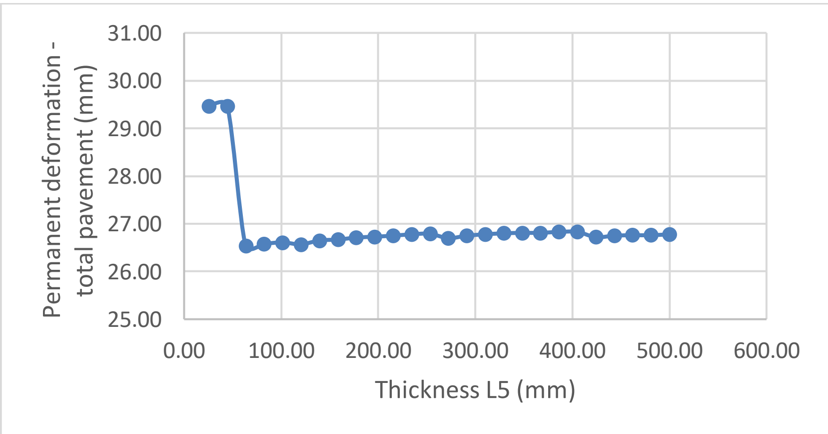


Figure 5-22: Sensitivity to thickness L5_IP6.

Resilient Modulus of layer 4 (Subbase)

The resilient modulus of layer 4 is varied between 91 MPa and 169 MPa. The max NSI value for the variation of resilient modulus L4 is -0.1 to total rutting as it is shown in Table 5-9. As Figure 5-23 shows, increasing the resilient modulus L4, decreases the total rutting with the average NSI of -0.1.

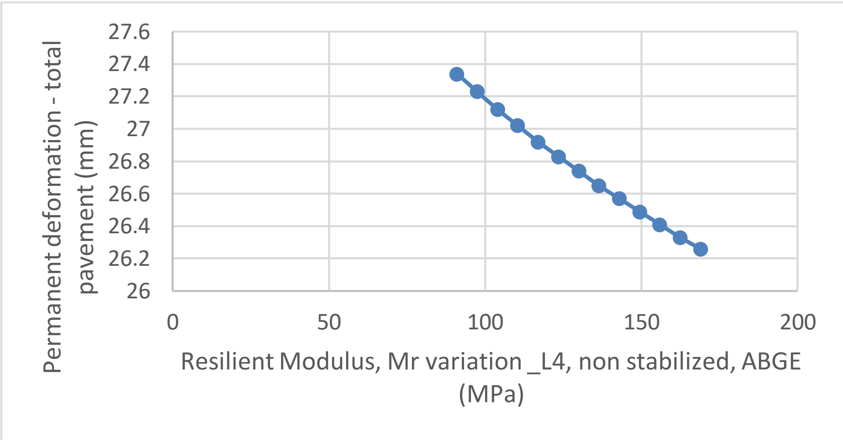


Figure 5-23: Sensitivity to Resilient Modulus L4_IP6.

Resilient Modulus of layer 5 (Subgrade)

The resilient modulus of layer 5 is varied between 42 MPa and 78 MPa. The max NSI value for the variation of resilient modulus L5 is -0.1 to total rutting as it is shown in Table 5-9. As Figure 5-24 shows, increasing the resilient modulus L5, decreases the total rutting with the average NSI of -0.1.

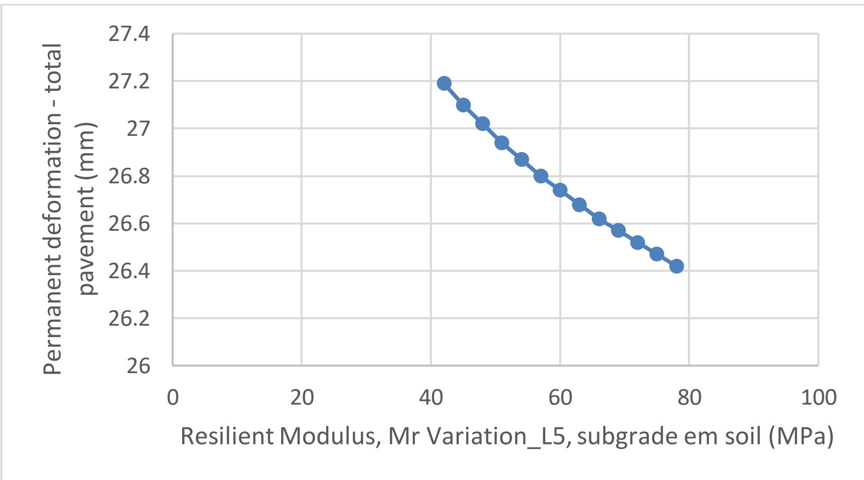


Figure 5-24: Sensitivity to Resilient Modulus L5_IP6.

- **Sensitivity of Categorical Inputs**

Aggregate Gradation Type for Flexible Layers

Table 5-10 shows the selected cases for aggregate gradation types for three flexible layers of IP6.

Table 5-10: The selected cases for aggregate gradation types for flexible layers of IP6

Percent passing	Case No.	Gradation (Sieve) mm			
		19	**9.5**	**4.75**	**0.075**
Layer 1 (Surface)	**Case1 (Base)**	100	74.7	45.4	6.4
	Case2 (Dense graded)	100	80	60	7
	Case3 (Open graded)	100	72.5	45	5
	Case4 (Free drainage)	100	55	22.5	4.5
Layer 2 (Binder)	**Case1 (Base)**	97.1	76	49.9	5.7
	Case2 (Dense graded)	100	80	60	7
	Case3 (Open graded)	100	72.5	45	5
	Case4 (Free drainage)	100	55	22.5	4.5
Layer 3 (Base)	**Case1 (Base)**	87.6	62.9	47.2	27.3
	Case2 (Dense graded)	100	80	60	7
	Case3 (Open graded)	100	72.5	45	5
	Case4 (Free drainage)	100	55	22.5	4.5

Sensitivity of Total Rutting to Different Gradation

Figure 5-25 shows that the type of gradation in the mixture of flexible layers slightly influences on total rutting (However, it is not so significant).

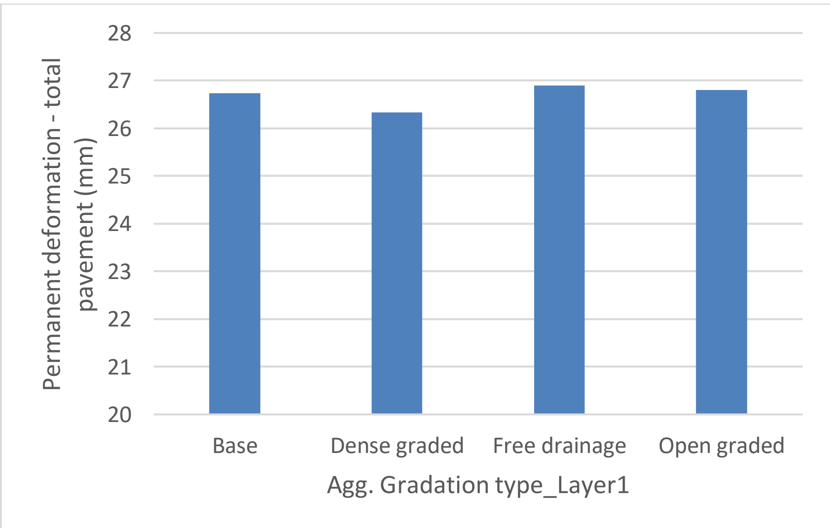


Figure 5-25: Sensitivity of total rutting to aggregate gradation layer 1_IP6.

Figure 5-26 and Figure 5-27 also show the same trend as Figure 5-25 shows.

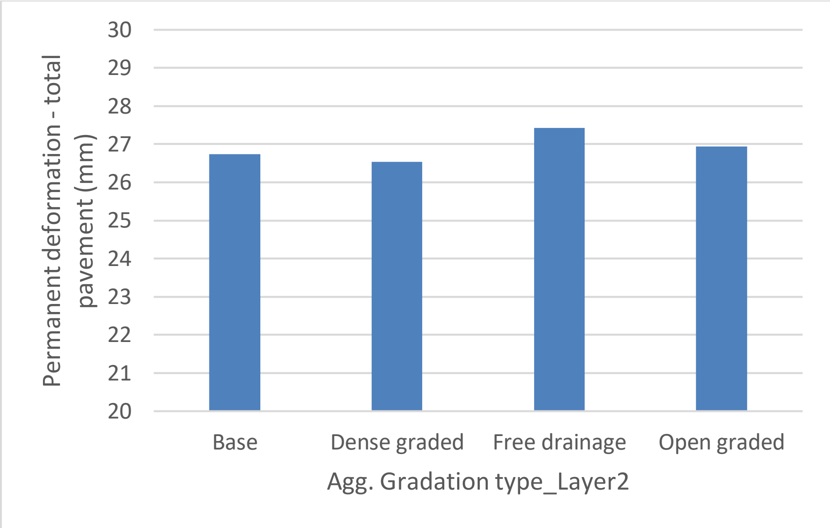


Figure 5-26: Sensitivity of total rutting to aggregate gradation layer 2_IP6.

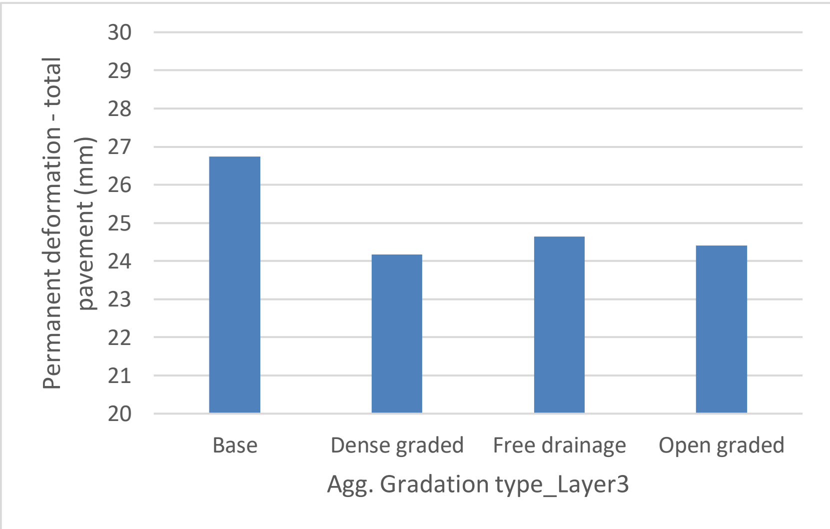


Figure 5-27: Sensitivity of total rutting to aggregate gradation layer 3_IP6.

Increasing the gap between the aggregate particles or reducing the density of gradation increases the total rutting.

Sensitivity of AC Bottom-Up Fatigue Cracking (%) to Different Gradation

The changes in gradation type (density) is only significant when the gap between the aggregate particles are too much or the density of gradation is very low. So, increasing the gaps, increase the AC B-U fatigue cracking.

The Figure 5-28 shows the sensitivity of AC B-U fatigue cracking to variation in aggregate gradation of surface layer.

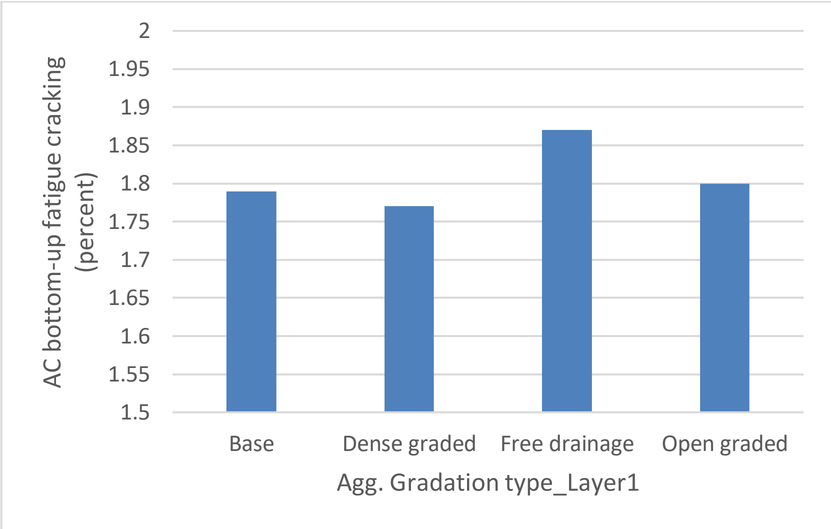


Figure 5-28: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 1_IP6.

The Figure 5-29 shows the sensitivity of AC B-U fatigue cracking to variation in aggregate gradation of binder layer.

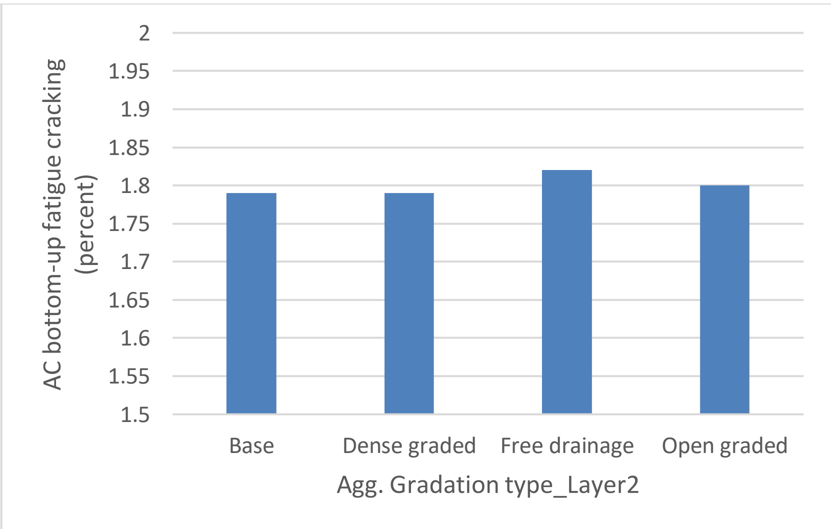


Figure 5-29: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 2_IP6.

The Figure 5-30 shows the sensitivity of AC B-U fatigue cracking to variation in aggregate gradation of base layer.

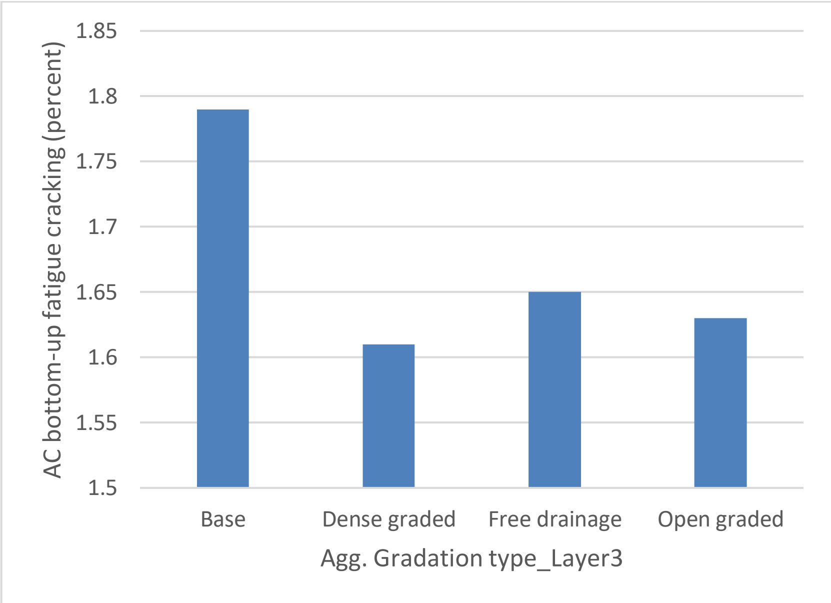


Figure 5-30: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 3_IP6.

5.2.4 EN254_ Sensitivity Analysis

The sensitivity of total permanent deformation and AC bottom-up fatigue cracking to variations of the design inputs will be evaluated for EN254 road.

- **Sensitivity of Continuous Inputs**

The tornado charts in this section show the priority and importance of continuous inputs to study their effect on performance behavior for EN254 pavement. Then, the variations' effect of the most effective inputs for total rutting and AC B-U fatigue cracking is shown based on the max NSI (normalized sensitivity index).

Based on Figure 5-31, the variations of thickness for the layers 2, 1, modulus of layer 6, thickness layer 3 and AADTT have the most effectiveness in the total permanent deformation.

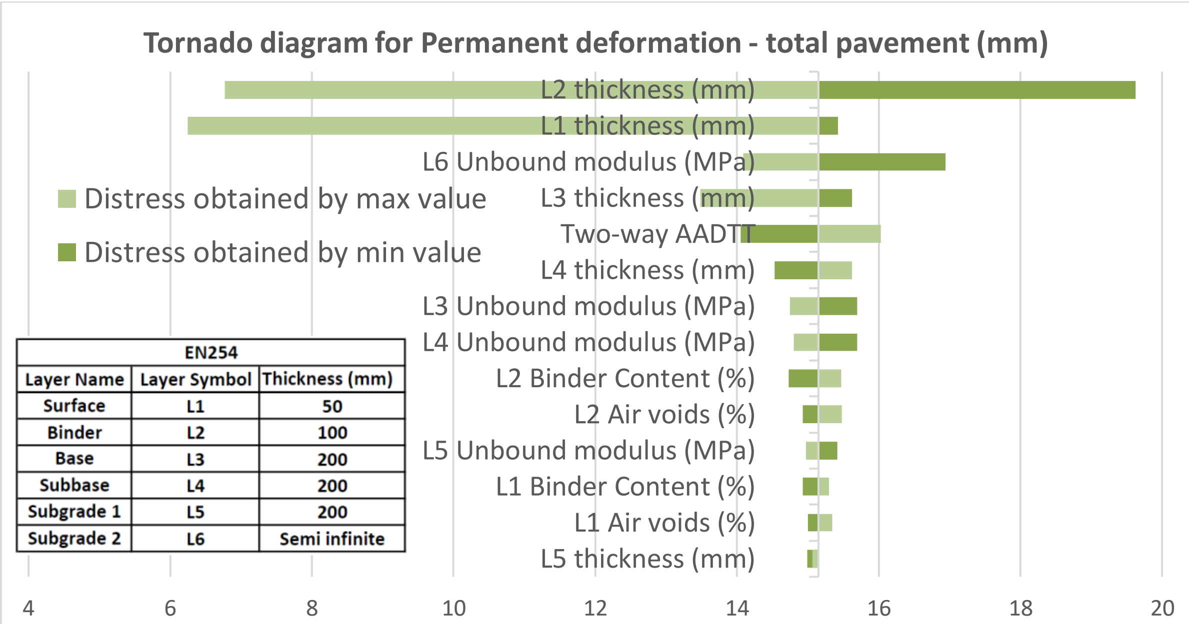


Figure 5-31: Tornado chart for the sensitivity analysis of total permanent deformation_EN254 road.

Figure 5-32 shows that the variations of binder content for layer 2, air voids layer 2, thickness layer 2 and 1 have the most effectiveness in AC bottom-up fatigue cracking.

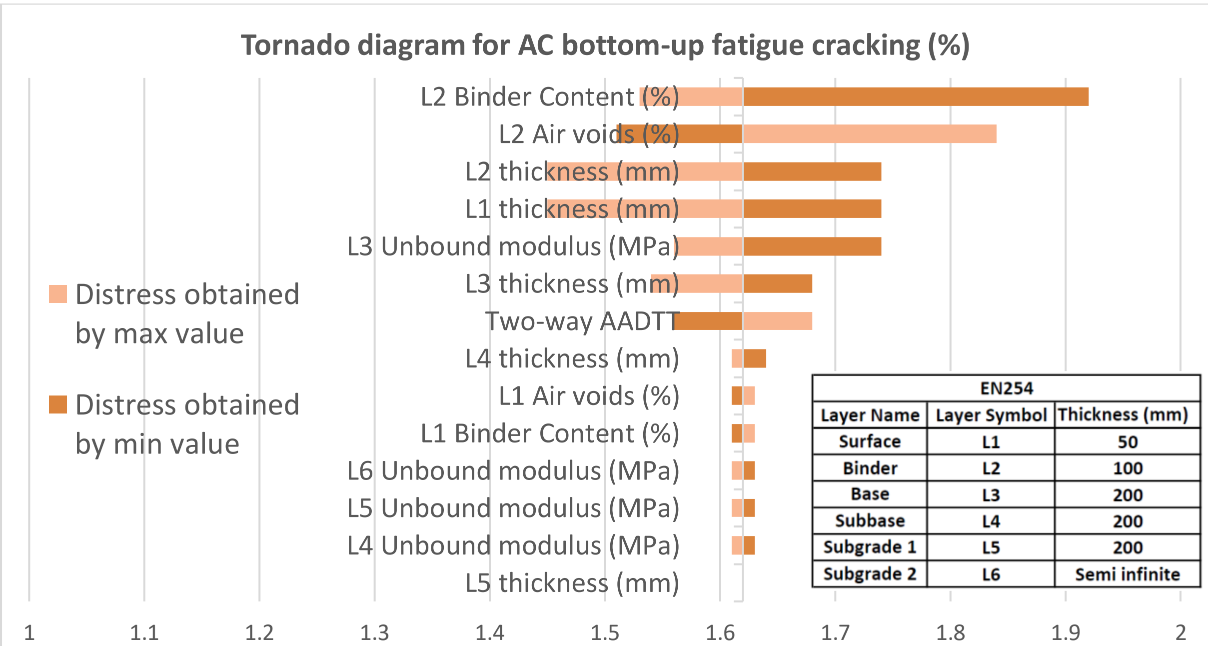


Figure 5-32: Tornado chart for the sensitivity analysis of AC bottom-up fatigue cracking_EN254 road.

The Table 5-11 shows the max NSI (normalized sensitivity index) for the most effective inputs in Figure 5-31 and Figure 5-32 for total rutting and AC B-U fatigue cracking.

Table 5-11: Ranking of New HMA design inputs by max NSI value for EN254.

Design Input	Max NSI for inputs (%)		Max NSI	Category of Sensitivity
	Total permanent deformation	AC bottom-up fatigue cracking		
L2 Thickness	-0.30	-0.01	-0.30	Sensitive
L6 unbound modulus	-0.30	0.00	-0.30	Sensitive
L3 Thickness	-0.22	-0.01	-0.22	Sensitive
AADTT	0.18	0.00	0.18	Sensitive
L1 Thickness	-0.17	0.00	-0.17	Sensitive
L5 Thickness	-0.16	0.00	-0.16	Sensitive
L4 Thickness	-0.12	0.00	-0.12	Sensitive
L3 unbound modulus	-0.10	0.00	-0.10	Sensitive
L4 unbound modulus	-0.10	0.00	-0.10	Sensitive
L2 Binder Content	0.07	-0.02	0.07	Insensitive
L2 Air voids	0.06	0.01	0.06	Insensitive
L1 Binder Content	0.04	0.00	0.04	Insensitive
L1 Air voids	0.03	0.00	0.03	Insensitive
L5 unbound modulus	0.00	0.00	0.00	Insensitive

Thickness of layer 2 (Binder Layer)

The thickness of layer 2 is varied between 25.4 mm and 500 mm. The max NSI value for the variation of thickness L2 is -0.3 to total rutting as it is shown in Table 5-11. As Figure 5-33 shows, increasing the thickness of layer 2, decreases the total rutting with the average NSI of -0.17. It should be noted that total permanent deformation in the bigger values of thickness L2 is less sensitive to the thickness L2 variations.

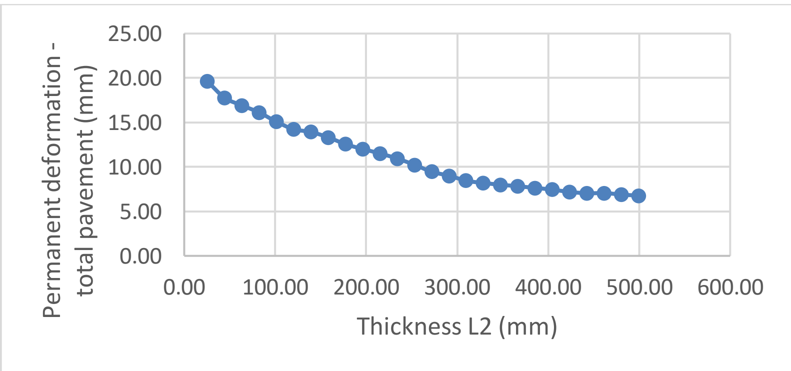


Figure 5-33: Sensitivity to second Layer thickness_EN254.

Resilient Modulus of layer 6 (Subgrade)

The resilient modulus of layer 6 is varied between 56 MPa and 104 MPa. The max NSI value for the variation of resilient modulus L6 is -0.3 to total rutting as it is shown in Table 5-11. As Figure 5-34 shows, increasing the resilient modulus L6, decreases the total rutting with the average NSI of -0.2. It should be noted that total permanent deformation in the bigger values of resilient modulus L6 is less sensitive to the resilient modulus L6 variations.

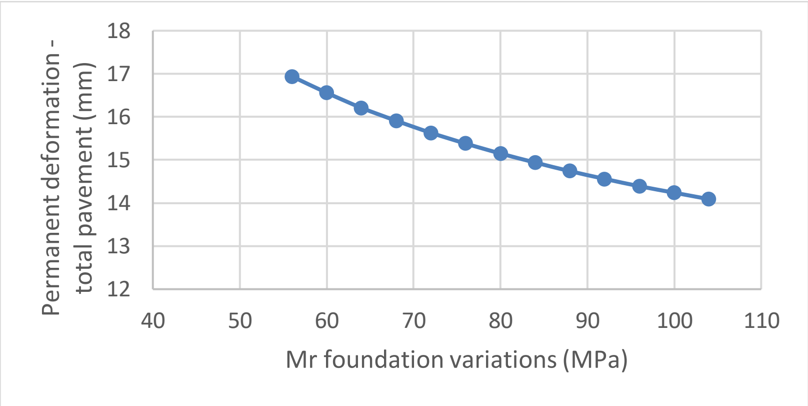


Figure 5-34: Sensitivity to Resilient Modulus of layer 6 (subgrade)_EN254.

Thickness of layer 3 (Base in ABGE)

The thickness of layer 3 is varied between 120.32 mm and 500 mm. The max NSI value for the variation of thickness L3 is -0.22 to total rutting as it is shown in Table 5-11. As Figure 5-35 shows, increasing the thickness of layer 3, decreases the total rutting with the average NSI of -0.08. It should be noted that total permanent deformation in the bigger values of thickness L3 is less sensitive to the thickness L3 variations.

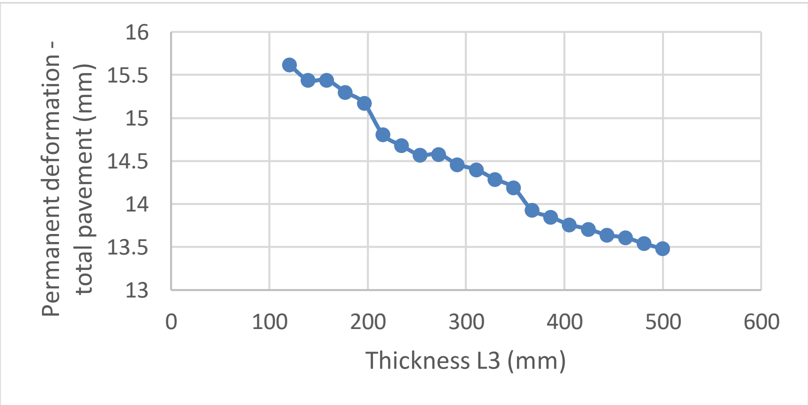


Figure 5-35: Sensitivity to thickness L3_EN254.

AADTT

The AADTT is varied between 107 and 201. The max NSI value for the variation of AADTT is 0.18 to total rutting as it is shown in Table 5-11. As Figure 5-36 shows, increasing the AADTT, increases the total rutting with the average NSI of 0.16. However, total permanent deformation in the bigger values of AADTT is less sensitive to the AADTT variations.

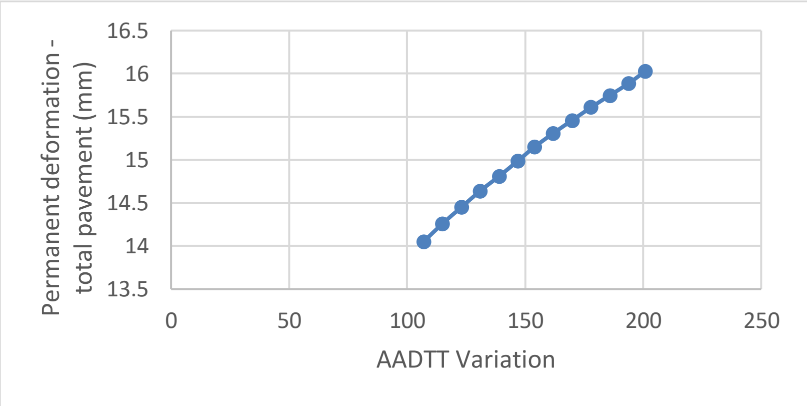


Figure 5-36: Sensitivity to AADTT_EN254

Thickness of layer 1 (Surface layer)

The thickness of layer 1 is varied between 25.4 mm and 500 mm. The max NSI value for the variation of thickness L1 is -0.17 to total rutting as it is shown in Table 5-11. As Figure 5-37 shows, increasing the thickness of layer 1, decreases the total rutting with the average NSI of -0.08. It should be noted that total permanent deformation in the bigger values of thickness L1 is insensitive to the thickness L1 variations.

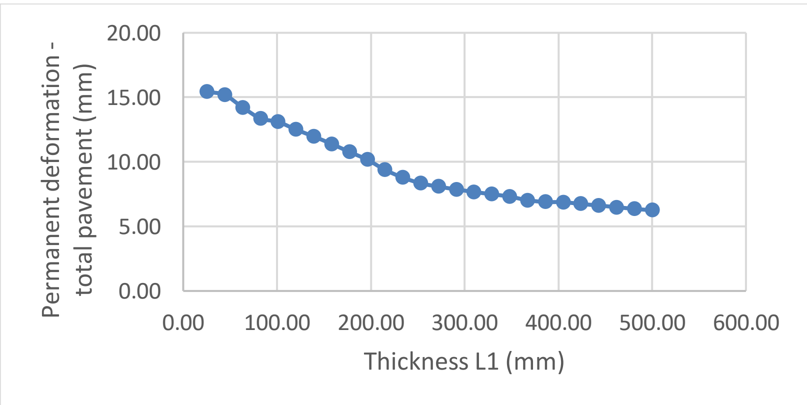


Figure 5-37: Sensitivity to thickness L1_EN254.

Thickness of layer 5 (Subgrade)

The thickness of layer 5 is varied between 63.368 mm and 500 mm. The max NSI value for the variation of thickness L5 is -0.16 to total rutting as it is shown in Table 5-11. As Figure 5-38 shows, increasing the thickness of layer 5, does not have significant effect on total permanent deformation.

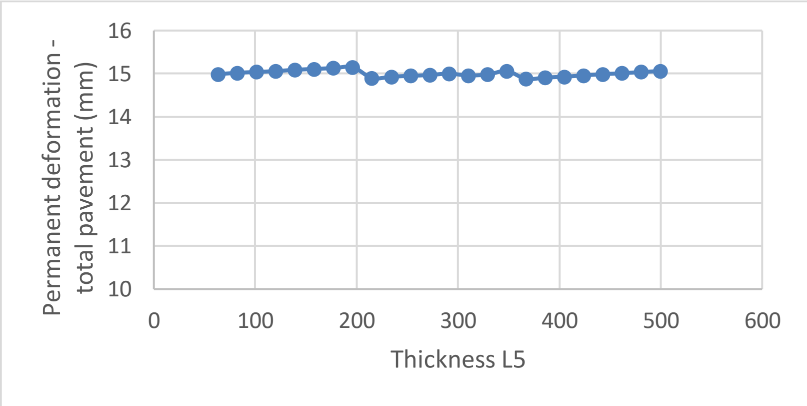


Figure 5-38: Sensitivity to thickness L5_EN254.

Thickness of layer 4 (Sub base)

The thickness of layer 4 is varied between 25.4 mm and 500 mm. The max NSI value for the variation of thickness L4 is -0.12 to total rutting as it is shown in Table 5-11. As Figure 5-39 shows, increasing the thickness of layer 4, in some points decreases the total rutting however, in general increases the total rutting.

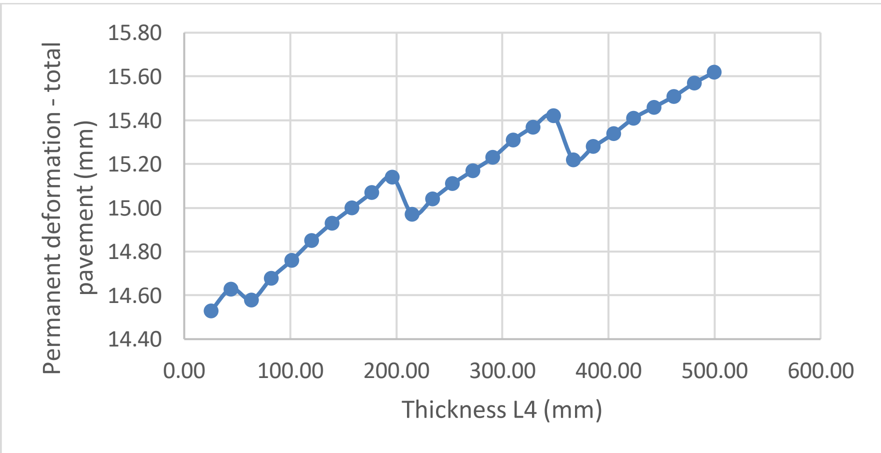


Figure 5-39 : Sensitivity to thickness L4_EN254.

Resilient Modulus of layer 3 (Base)

The resilient modulus of layer 3 is varied between 203 MPa and 377 MPa. The max NSI value for the variation of resilient modulus L3 is -0.1 to total rutting as it is shown in Table 5-11. As Figure 5-40 shows, increasing the resilient modulus L3, decreases the total rutting with the average NSI of -0.1.

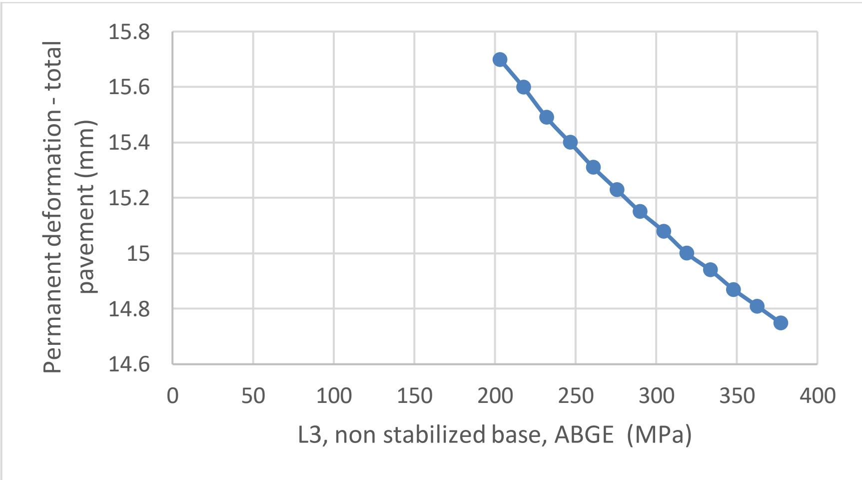


Figure 5-40: Sensitivity to resilient modulus L3_EN254.

Resilient Modulus of layer 4 (Sub base)

The resilient modulus of layer 4 is varied between 84 MPa and 156 MPa. The max NSI value for the variation of resilient modulus L4 is -0.1 to total rutting as it is shown in Table 5-11. As Figure 5-41 shows, increasing the resilient modulus L4, decreases the total rutting with the average NSI of -0.1.

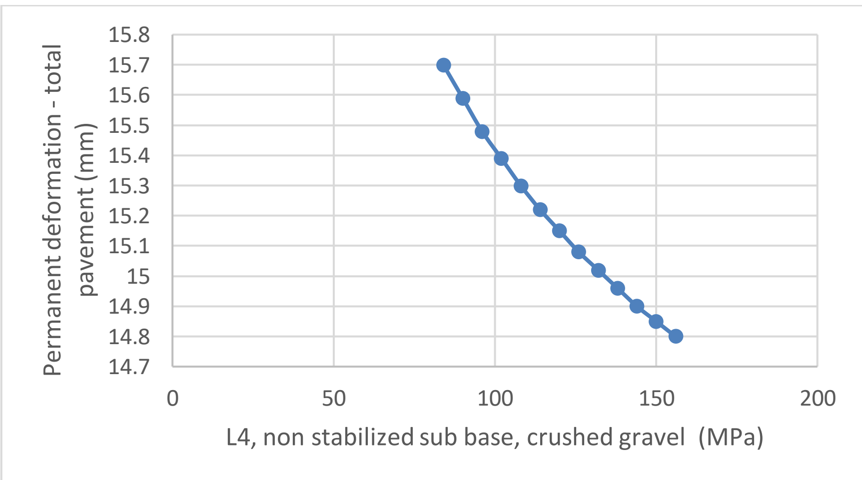


Figure 5-41: Sensitivity to resilient modulus L4_EN254.

- **Sensitivity of Categorical Inputs**

Aggregate Gradation Type for Flexible Layers

Table 5-12 shows the selected cases for aggregate gradation types for first flexible layer of EN254[1].

Table 5-12: The selected cases for aggregate gradation types for flexible layer of EN254

Percent passing	Case No.	Gradation			
		19mm Sieve	9.5mm Sieve	4.75mm Sieve	0.075mm Sieve
Layer 1	Case1 (Base)	99.5	74.9	48.5	3.3
	Case2 (Dense graded)	100	80	60	7
	Case3 (Open graded)	100	72.5	45	5
	Case4 (Free drainage)	100	55	22.5	4.5

Sensitivity of Total Rutting to Different Gradation

Figure 5-42 shows that the type of gradation in the mixture of flexible layers slightly influences on total rutting (However, it is not so significant). Increasing the gap between the aggregate particles or reducing the density of gradation increases the total rutting.

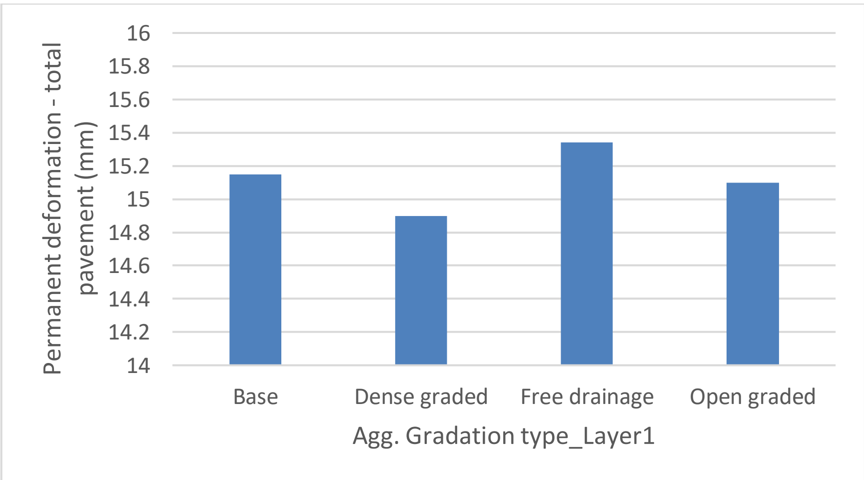


Figure 5-42: Sensitivity of total rutting to aggregate gradation layer 1_EN254.

[1] Due to time limitation for software availability, only first flexible layer is studied.

Sensitivity of AC Bottom-Up Fatigue Cracking to Different Gradation

Figure 5-43 shows that the changes in gradation type (density) are only significant when the gap between the aggregate particles is too much or the density of gradation is very low. So, increasing the gaps, increase the AC B-U fatigue cracking.

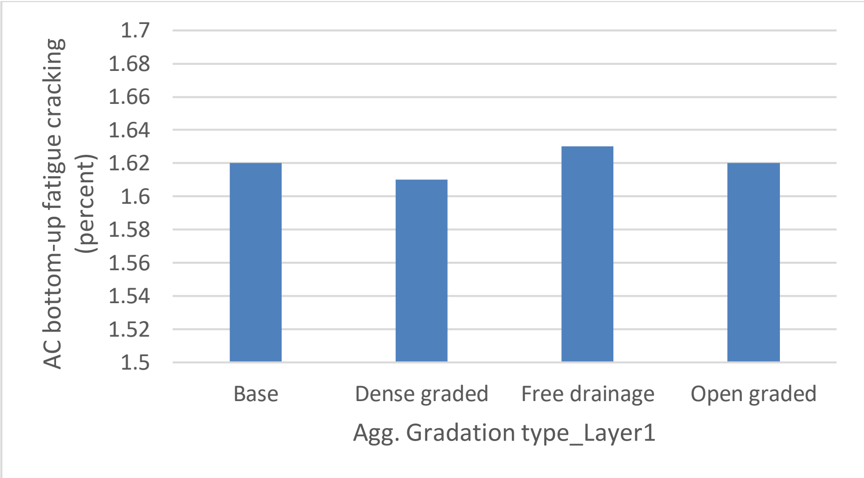


Figure 5-43: Sensitivity of AC B-U fatigue cracking to aggregate gradation layer 1_EN254.

5.3 Conclusions

The sensitivity of ME Design predicted pavement performance to design inputs has been evaluated based on the past sensitivity analysis researches and the approach of individual sensitivity analysis. In the individual approach, each potentially sensitive design input identified through the past studies was varied individually for each of three selected roads to assess quantitatively the local sensitivity of the ME predicted distresses to the design input.

Regarding the quantifying the effect of inputs on distresses, there is no sensitivity metric that is uniquely best for all variables and all purposes. The adopted metric for quantifying the sensitivity of performance behaviors to inputs in this study is a normalized sensitivity index (NSI) which is defined as the percentage change of predicted distress relative to its design limit caused by a given percentage change in the design input.

In this study, the relative magnitudes of the NSI values are more important than their precise values in order to better evaluate which pavement design inputs are most important. The selected inputs for the sensitivity analysis were in the two groups of continuous inputs and categorical inputs. The results of sensitivity analysis after creating and analyzing about 750 ME projects for the three selected roads with several design inputs (continuous and categorical inputs) have been extracted and the tornado diagrams, NSI values, and the input/output diagrams have been prepared. Although it is difficult to extract broad conclusions that apply to all pavements under all conditions and on the other hand, the level of input data used in this sensitivity study (e.g. traffic distribution data, dynamic modulus data, water table depth…) was mostly level 3 (extracted from previous records or software default value). However, some trends are consistently observed in the sensitivity results that are mostly expected:

- As expected, the magnitude of the sensitivity values for total permanent deformation (which is the dominant performance criterion for ME method) to the variation of inputs are constantly higher than the values for other main distresses in this study (AC bottom-up fatigue cracking and Stabilized layer fatigue fracture).
- The thickness of different layers, resilient modulus of layer 6 (subgrade) and AADTT are the most influential inputs in this study.
- Increasing the AADTT, increases the total permanent deformation. However, total permanent deformation in the bigger values of AADTT is less sensitive to the AADTT variations.
- Increasing the thickness of layers, decreases the total permanent deformation. However, total permanent deformation in the bigger values of thickness is less sensitive or insensitive to the thickness variations.
- Increasing the resilient modulus of unbound layers, decreases the total permanent deformation. It should be noted that total permanent deformation in the bigger values of resilient modulus could be less sensitive to the resilient modulus variations.
- The type of gradation in the mixture of flexible layers slightly influences on total rutting (However, it is not so significant). Increasing the gap between the aggregate particles or reducing the density of gradation increases the total rutting.

The overall trends from sensitivity analysis in this chapter are summarized in Table 5-13.

Table 5-13: Summary of sensitivity analysis results

	Design Input	**Total rutting**	**AC bottom-up fatigue cracking**	**Stabilized layer fatigue fracture**
Continuous Inputs	Higher two-way AADTT	↑	→	→
	Bigger L1 thickness	↓	→	→
	Bigger L2 thickness	↓	→	→
	Bigger L3 thickness	→	→	→
	Bigger L4 thickness	→	→	→
	Bigger L5 thickness	→	→	→
	Bigger L3 Resilient modulus	→	→	→
	Bigger L4 Resilient modulus	→	→	→
	Bigger L5 Resilient modulus	→	→	→
	Bigger L6 Resilient modulus	↓	→	→
	More L1 Air voids	→	→	→
	More L2 Air voids	→	→	→
	More L3 Air voids	→	→	→
	Bigger L1 Binder Content	→	→	→
	Bigger L2 Binder Content	→	→	→
	Bigger L3 Binder Content	→	→	→
Categorical Inputs	More gap graded aggregate L1	↑	↗	→
	More gap graded aggregate L2	↑	↗	→
	Deeper water table depth	→	→	→
	Vehicle class distribution with higher % of heavier vehicle class	↑	→	→

Legend:
L1: Surface
L2: Binder+ Base
L3: Subbase
L4, L5, L6: are respectively subgrade 1, subgrade 2, subgrade 3

↑ It is sensitive and usually increases. ↓ It is sensitive and usually decreases.

↗ It is less sensitive and slightly increases. → It is insensitive and usually unchanged.

The results of categorical inputs indicate:

- The results of sensitivity analysis for the variation of climate conditions was not well adjusted to the real climate conditions of Portuguese regions. So, it is required to investigate the effect of climate conditions in a broader range of weather stations and even individually for each climate factor to indicate the effect of climate conditions in detail.

- The changes of water table depth do not show any significant changes in total rutting or AC B-U fatigue cracking (It has minimal effect in depths lower than 6 m). It should be noted that the effect of water table would be through the changes in the resilient modulus of the unbound layers and foundation soils over time and consequently these changes can affect the total rutting.
- The results of vehicle class distribution variation show that the increase in the percentage of single trailer trucks (higher vehicle classification) and decrease in the percentage of single unit trucks (lower vehicle classification) have a significant influence on the total permanent deformation while there are no significant changes in the AC B-U fatigue cracking. So, as expected for the total rutting, the higher the level of vehicle classification and traffic loading, the more effect on increasing of total rutting is noticeable.

6 Conclusions and recommendations

6.1 Summary

This thesis aims to study and develop a framework for the possible adoption of Mechanistic-Empirical Pavement Design Guide (MEPDG) for Portugal roads design considering Portuguese conditions.

The first step in this work was to study the differences between ME method and SHELL method as an initial step toward choosing the ME method as a promising method for pavement design in Portugal or as a basis for any improvement in the Portuguese pavement design. The comparison of both methods in chapter 2 was stablished through the reviewing of the existing state-of-knowledge in both methods and their related components.

The second step was evaluating the available Portuguese sources for data preparation for ME method. In other words, the second step addressed the challenges in making local databases for ME Design (e.g. availability of data for materials characterization, availability of WIM data for traffic, developing a more comprehensive library of climate data for Portugal). For this purpose, the required data for ME method were described and the approaches to obtain the data were then explained by preparing data for the three different Portuguese roads (chapter 3). Some factors in choosing the roads and the sections were considered in this study which among them, the availably of data in quality control data and pavement structure design documents had the priority. For this study, three roads were selected for data preparation, damage comparison and sensitivity analysis:

- IC3 _Variante de Tomar_ No da Atalaia (sections: km 4+400 to 5+400; km 5+800 to 7+000);
- IP6_Peniche_Atouguia da Baleia (section: km 2+500 to 3+500);
- EN254_Variante de São Miguel de Machede (section: km 0+000 to 1+400).

To create ME projects of the selected sections and applying Portuguese conditions in the ME projects, the following categories of data were considered for the availability and modifications:

- specific site subgrade support (data source: IP or Infraestruturas of Portugal[1]);
- material properties (data source: IP);
- traffic loading (data source: IP);
- environmental conditions (data source: MERRA tools).

The first two categories of data were obtained through the paper-based documents of IP. The data had some deficiencies as it was expected in some cases and required some calculations to obtain the target or demanded value otherwise it should be considered the default value in the software or mean value for the local condition in the lack of data. Regarding the traffic data, AADTT and growth rate was obtained by traffic simulation model (years 2013-2033). Since the other traffic data (such as traffic volume adjustment factors, axle load distribution factors, number of axles/truck) were not available for the selected road sections, they were considered the ME default. It should be noted that these traffic data can be obtained through the WIM data. So, it is suggested to collect and study the WIM data for Portugal.

The generated data by MERRA tools for climate were limited to four cities of Lisbon, Coimbra, Porto and Beja at the time of analysis. However, it is notable that the climate data are currently available for other regions of Portugal through the related website (Federal Highway Administration, 2018). The generated files for climate data (*.hcd files) were modified based on metric units and the correct format. The files were then added to the climate database of ME Design. Finally, the climate data for the selected sections were obtained by interpolating climatic data from available nearby weather stations (Lisbon, Porto, Coimbra, Beja).

One of the main tasks in this study is to have a damage comparison between both methods of ME and SHELL. For this purpose, the performance criteria used in ME method are justified based on Portuguese conditions. Additionally, three main factors of service temperature, moisture content and traffic were reviewed in both methods of ME and SHELL and the values were adjusted to have the similar background for both approaches. (Chapter4).

[1] Portuguese Road Organization

The damage comparison had an important role in identifying which method is more conservative in pavement design (ME method or SHELL method) as well as identifying the key distresses for each method. The results of this comparison are explained in the next subsection (conclusions).

The damage results are then verified by the sensitivity analysis of obtained distresses to design inputs' variations for ME Design. For this task, the results of 750 ME sensitivity analysis projects for the three selected roads with several design inputs (continuous and categorical inputs) are extracted and the tornado diagrams, normalized sensitivity index (NSI) values, and the input/output diagrams are prepared and analyzed (Chapter 5). So, Chapter 5 reviewed and documented sensitive design inputs that influence the key distresses and smoothness the most.

6.2 Conclusions

The comparison of both methods (SHELL and ME) addressed the following conclusions related to the main differences in implementation of either SHELL method or ME method for pavement design:

- While SHELL method considers the following three performance criteria to be satisfied for the design: fatigue cracking of the bound materials; permanent deformation of the subgrade; tensile stress at the bottom of soil cement, ME method improved the pavement evaluation process by predicting more performance indicators. Accordingly, the following performance criteria are considered in ME method as the confirmation of design expectations: terminal IRI; total rutting, AC rutting, AC bottom-up fatigue cracking, AC top-down fatigue cracking; AC thermal cracking; chemically stabilized layer- fatigue fracture. However, AASHTO manual strongly recommended that the empirical models of these performance criteria should be locally calibrated in the ME Design prior to implementation. The calibration process requires an agency to compare historical pavement performance from existing pavements to performance predicted by the ME Design and adjust the empirical models’ coefficients to minimize the error between measured and predicted performance. The calibration process would be a costly, time-consuming process that demands a significant amount of effort. As an advantage for the SHELL method, this process is not required for the implementation of the method.

- Demanding a large set of data in different parts (materials, climate and traffic) is the other feature of ME method and the difference of two methods. However, the lack of adequate data, costly and time-consuming procedure of obtaining data could be the disadvantage of this aspect for ME method.
- The other feature of ME method is employing over 100 inputs data from different hierarchical levels based on the road importance, and data collection effort costs and time. So, focusing on the inputs that have the most effect on desired pavement performance is important for the designers to have more effective and comprehensive selection of data from different hierarchical levels for a project. For this purpose, the sensitivity analysis of ME performance predictions and designs to variations of the design inputs is required. However, the task of sensitivity analysis demands considerable amount of time and effort. This implies some challenges in terms of investment in data collection, experimental research, demanding greater cost-benefit analyses, between data cost and quality of the design process.
- One of the advantages of ME Design over SHELL method is employing several aspects of load types variation (tire loads, axle and tire configurations, repetition of loads, distribution of traffic across the pavement, vehicle speed) which supports to have more accurate and realistic environment for the loading conditions of pavement over the design life.
- The other advantage of ME method is the applicability for both existing pavement rehabilitation and new pavement construction for ME method while SHELL method is used mainly in the design of new road pavements.
- The other feature of ME method and one of the main reasons for using the large set of climate data in ME method is applying the incremental effect of environmental factors in material and accordingly in pavement performances. In other words, while SHELL method uses the initial properties of the materials to predict the performance for the entire life period of pavement, the ME Design applies environmental and aging effects on materials over design life. So, pavement performances in ME method are incrementally calculated considering the evolution of traffic and climatic

conditions over time. As an advantage for ME method, the incremental approach provides more reliable performance predictions in ME method than SHELL method.

The results presented in this thesis in damage comparison show that SHELL method was more conservative, regarding the total permanent deformation, while ME method was more conservative in terms of AC bottom-up fatigue cracking. In other words, the dominant performance criterion in SHELL method was AC bottom-up fatigue cracking while total permanent deformation was the dominant and main performance criterion in ME method. It should be noted that Terminal IRI was the second dominant performance criterion for ME Design, however terminal IRI is mostly dependent on total rutting based on the IRI model. So, the dominancy of this criterion is due to the dominancy of total rutting and can be considered as a subset of total rutting in procedure of sensitivity analysis.

Regarding the damage values for the chemically stabilized layer-fatigue fracture, both approaches of SHELL and ME showed similar trend in estimation of this criterion. In addition, with the most matched initial conditions, SHELL method was more conservative than ME method. In other words, ME method seemed to can roughly constitute the closer estimation for the design.

The results of sensitivity analysis after creating and analyzing about 750 ME projects for the case studies with several design inputs consistently indicated some trends that are mostly expected:

- As expected, the magnitude of the sensitivity values for total permanent deformation (which is the dominant performance criterion for ME method) to the variation of inputs are constantly higher than the values for other main distresses in this study (AC bottom-up fatigue cracking and Stabilized layer fatigue fracture).
- The thickness of different layers, resilient modulus of subgrade layer and AADTT are the most influential inputs in this study.
- Increasing the AADTT, increases the total permanent deformation. However, total permanent deformation in the bigger values of AADTT is less sensitive to the AADTT variations.

- Increasing the thickness of layers, decreases the total permanent deformation. However, total permanent deformation in the bigger values of thickness is less sensitive or insensitive to the thickness variations.
- Increasing the resilient modulus of unbound layers, decreases the total permanent deformation. It should be noted that total permanent deformation in the bigger values of resilient modulus could be less sensitive to the resilient modulus variations.
- The type of gradation in the mixture of flexible layers slightly influences on total rutting (However, it is not so significant). Increasing the gap between the aggregate particles or reducing the density of gradation increases the total rutting.
- The results of sensitivity analysis for the variation of climate conditions was not well adjusted to the real climate conditions of Portuguese regions. So, it is required to investigate the effect of climate conditions in a broader range of weather stations and even individually for each climate factor to indicate the effect of climate conditions in detail.
- The changes of water table depth do not show any significant changes in total rutting or AC B-U fatigue cracking (It has minimal effect in depths lower than 6 m). It should be noted that the effect of water table would be through the changes in the resilient modulus of the unbound layers and foundation soils over time and consequently these changes can affect the total rutting.
- The results of vehicle class distribution variation show that the increase in the percentage of single trailer trucks (higher vehicle classification) and decrease in the percentage of single unit trucks (lower vehicle classification) have a significant influence on the total permanent deformation while there are no significant changes in the AC B-U fatigue cracking. So, as expected for the total rutting, the higher the level of vehicle classification and traffic loading, the more effect on increasing of total rutting is noticeable.
- As an outcome, can be concluded that while total rutting has the dominant effect on the layer design thicknesses in ME method, this distress seems to be dependent on the specific pavement structure and resilient modulus value (i.e. subgrade Mr). Accordingly, the variation of subgrade resilient modulus affects the layer design thickness (especially surface layer or HMA layer).

6.3 Recommendations and Future Works

The steps recommended for future works to evaluate the suitability of MEPDG for Portuguese conditions could be:

- Assessing the conditions of the laboratory and field equipment required for higher level of design input to achieve higher design accuracy and acquire the required equipment to develop a testing program for sensitive design inputs. Consequently, there would be no lack of data to achieve high design levels and/or a less conservative yet appropriate design.
- Local calibration and validation of distress models to obtain more accurate results for each Portuguese region based on the related traffic loading and climatic conditions.
- Evaluating the suitability of MEPDG for rehabilitation and maintenance of Portuguese pavements.
- Providing a framework for necessary training of the MEPDG to road agencies.
- Developing a framework for obtaining required traffic data (load spectra, weigh in motion (WIM) and automatic vehicle classifiers (AVC) data) for Portuguese regions.
- Besides the university studies in Portugal for the adoption of ME approach, it would be useful to have a cost-benefit analysis of pavement structures carried out by the road agencies in Portugal in order to compare the pavement design costs for both methods of SHELL and ME in different stages of procedure including data preparation, analysis, design and implementation for new and rehabilitated pavements, etc.
- Implementing a study focusing on the effects of different RAP binder sources and RAP contents in Portugal on the rheological properties of virgin asphalt incorporating extracted RAP binder. This study can be done for other recycled materials used in Portuguese pavements.

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Appendixes

Appendix 1: Pavement Performance Prediction Models

The first appendix provides the related pavement performance prediction models that have been used in ME Design.

1. AC Fatigue Cracking model

$$N_{f-HMA} = k_{f1}(C)(C_H)\,\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \qquad Eq.A-1$$

Where:

N_{f-HMA}: *Allowable number of axle − load applications for a flexible pavement and HMA overlays,*

ε_t: *Tensile strain at critical locations and calculated by the structural response model, in./in.,*

E_{HMA}: *Dynamic modulus of the HMA measured in compression, psi,*

k_1, k_2, k_3: *Global field calibration parameters (from the NCHRP* $1-40D$ *recalibration;* $k_1 = 0.007566, k_2 = 3.9492, k_3 = 1.281$)

$\beta_{f1}, \beta_{f2}, \beta_{f3}$: *Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to* 1.0.

$$C = 10^M \qquad Eq.A-2$$

$$M = 4.84\left[\frac{V_{be}}{V_a+V_{be}} - 0.69\right] \qquad Eq.A-3$$

Where:

C_H = *Thickness correction term, dependent on type of cracking.*

V_{be}: *Effective binder content by volume* (%)

V_a: *Air voids* (%)

For bottom-up or alligator cracking:

$$C_H = \frac{1}{0.000398 + \frac{0.003602}{1+e^{(11.02-3.49H_{HMA})}}} \qquad Eq.A-4$$

For top-down or longitudinal cracking:

$$C_H = \frac{1}{0.01 + \frac{12.00}{1+e^{(15.676-2.8186 H_{HMA})}}} \qquad Eq. A - 5$$

Where:

$H_{HMA} = \ Total\ HMA\ thickness, in.$

2. AC Rutting Model

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}} \qquad Eq. A - 6$$

Where:

ε_p: $Accumulated\ permanent\ or\ plastic\ axial\ strain\ in\ the\ HMA$

$layer/sublayer, (in/in).$

ε_r: $Resilient\ or\ elastic\ strain\ calculated\ by\ the\ structural\ response$

$model\ at\ the\ mid-depth\ of\ each\ HMA\ sublayer, (in/in)$

T: $Mix\ or\ pavement\ temperature$ (°F)

N: $number\ of\ axle\ load\ repetitions$

k_1, k_2, k_3: $Global\ field\ calibration\ parameters\ (from\ the\ NCHRP\ 1$

$-40D\ recalibration;\ k1r\ = -3.35412, k2r\ =\ 0.4791, k3r\ =\ 1.5606)$

$\beta_{r1}, \beta_{r2}\ , \beta_{r3}$: $Local\ or\ mixture\ field\ calibration\ constants;$

$for\ the\ global\ calibration, these\ constants\ were\ all\ set\ to\ 1.0.$

$$k_z = (C_1 + C_2 \times depth) \times 0.328196^{depth} \qquad Eq. A - 7$$

$$C_1 = -0.1039 \times H_{ac}^2 + 2.4868 \times H_{ac} - 17.342 \qquad Eq. A - 8$$

$$C_2 = 0.0172 \times H_{ac}^2 - 1.7331 \times H_{ac} - 27.428 \qquad Eq. A - 9$$

Where:

k_z: $Depth\ confinement\ factor$

Depth: depth below the surface, in

H_{ac}: $Total\ HMA\ thickness, in.$

3. Thermal Fracture Model

$$C_f = 400 \times N\left(\frac{\log C/h_{ac}}{\sigma}\right) \qquad Eq.A-\ 10$$

Where:

C_f: $Observed\ amount\ of\ thermal\ cracking\ (ft/500ft)$

k: $refression\ coefficient\ determined\ through\ field\ calibration$

$N(z)$: $standard\ normal\ distribution\ evaluated\ at\ (z)$

σ: $standard\ deviation\ of\ the\ log\ of\ the\ depth\ of\ cracks\ in\ the\ pavements$

C: $crack\ depth\ (in)$

h_{ac}: $thickness\ of\ asphalt\ layer\ (in)$

$$\Delta C = (k \times \beta_t)^{n+1} \times A \times \Delta K^n \qquad Eq.A-\ 11$$

$$A = 10^{(4.389-2.52\times \log(E\times\sigma_m\times n))} \qquad Eq.A-\ 12$$

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: $fracture\ parameters\ for\ the\ asphalt\ mixture$

E: $mixture\ stiffness$

σ_M: $undamaged\ mixture\ tensile\ strenght$

β_t: $Local\ or\ mixture\ calibration\ factor.$

k : $Global\ calibration\ Coefficient\ for\ each\ input\ level\ (Level\ 1\ =\ 5.0; Level\ 2\ =\ 1.5;\ and\ Level\ 3\ =\ 3.0),$

4. Subgrade Rutting Model

$$\delta_a(N) = \beta_{s1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right| \qquad Eq.A-\ 13$$

Where:

δ_a: $Permanent\ or\ plastic\ deformation\ for$

$the\ layer/sublayer, in.,$

N: $Number\ of\ axle - load\ applications,$

ε_v: $Average\ vertical\ resilient\ or\ elastic\ strain\ in\ the$

$layer/sublayer\ and\ calculated\ by\ the\ structural\ response$

$model, in/in.,$

ε_0: $Intercept\ determined\ from\ laboratory\ repeated\ load$

$permanent\ deformation\ tests, in/in.,$

ε_r: $Resilient\ strain\ imposed\ in\ laboratory\ test\ to\ obtain$

$material\ properties\ eo, e, and\ \rho, in/in.,$

h : $Thickness\ of\ the\ unbound\ layer/sublayer, in.,$

$k1$: $Global\ calibration\ coefficients;$

$k1 = 1.673\ for\ granular\ materials\ and$

$1.35\ for\ fine - grained\ materials, and$

$\beta s1 =\ Local\ calibration\ constant\ for\ the\ rutting\ in\ the$

$unbound\ layers;\ the\ local\ calibration\ constant\ was\ set\ to$

$1.0\ for\ the\ global\ calibration\ effort.$

$$log\,\beta = -0.61119 - 0.017638(W_c) \qquad Eq.A-\ 14$$

$$\rho = 10^9\left(\frac{C_0}{\left(1-(10^9)^{\beta}\right)}\right)^{\frac{1}{\beta}} \qquad Eq.A-\ 15$$

$$C_0 = ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}}\right) = 0.0075 \qquad Eq.A-\ 16$$

Where

β, ρ: $material\ properties$

$W_c = Water\ Content, \%$

$M_r = Resilient\ modulus\ of\ the\ unbound\ layer\ or\ sublayer, psi$

$a_{1,9} = Regression\ constants;\ a_1 = 0.15\ and\ a_9 = 20.0$

$b_{1,9} = Regression\ constants;\ b_1 = 0\ and\ b_9 = 0.0$

5. AC Top Down Cracking Model

$$FC_{top} = \left(\frac{C_4}{1+e^{(C_1 - C_2 \times log_{10}(Damage))}}\right) \times 10.56 \qquad Eq. A-\ 17$$

Where:

$FC_{top} =\ Length\ of\ longitudinal\ cracks\ that\ initiate\ at\ the\ top\ of\ the\ \ HMA\ layer, ft/mi,$

$Damage\ =\ Cumulative\ damage\ index\ near\ the\ top\ of\ the\ HMA\ surface,$

$C1,2,4\ =\ Transfer\ function\ regression\ constants;\ C1 =\ 7.00;\ C2 =\ 3.5;\ and\ C4 =\ 1{,}000.$

$$AC\ Cracking\ Top\ Standard\ Deviation: 200\ +\ \frac{2300}{1+e^{(1.072-2.1654\times log_{10}(FC_{top}+0.0001))}}$$

6. AC Bottom Up Cracking Model

$$FC_{Bottom} = \left(\frac{C4}{1+e^{(C_1\times C_1' + C_2 \times C_2'\ log_{10}(D\times 100))}}\right) \times \left(\frac{1}{60}\right) \qquad Eq. A-\ 18$$

Where:

$FC_{Bottom}: Area\ of\ alligator\ cracking\ that\ initiates\ at\ the\ bottom\ of$

$the\ HMA\ layers, \%\ of\ total\ lane\ area,$

$D: Cumulative\ damage\ index\ at\ the\ bottom\ of\ the\ HMA\ layers, and$

$C1,2,4\ =\ Transfer\ function\ regression\ constants;$

$(C4 =\ 6{,}000;\ C1 = 1.00;\ and\ C2 = 1.00.)$

$$C_2' = -2.40874 - 39.748 \times (1 + h_{ac})^{-2.856} \qquad Eq. A-\ 19$$

$$C_1' = -2 \times C_2' \qquad Eq. A-\ 20$$

where:

$h_{ac}: Total\ HMA\ thickness, in.$

$AC\ Cracking\ Bottom\ Standard\ Deviation: 1.1 + \frac{13}{1 + e^{(7.57 - 15.5 \times log_{10}(FC_{Bottom} + 0.0001))}}$

7. CSM Fatigue Model

$$N_f = 10^{\left(\frac{k_1 \beta_{c1} \left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{c2}}\right)} \qquad Eq. A - 21$$

Where:

$N_f: Allowable\ number\ of\ axle - load\ applications\ for\ a\ semi - rigid\ pavement,$

$\sigma t\ = Tensile\ stress\ at\ the\ bottom\ of\ the\ CTB\ layer, psi,$

$MR\ :\ 28 - day\ modulus\ of\ rupture\ for\ the\ CTB\ layer, psi.$

$(Note: Although\ the\ MEPDG\ requires\ that\ the\ 28$
$- day\ modulus\ of\ rupture\ be\ entered\ for\ all\ cementitious\ stabilized$

$layers\ of\ semi - rigid\ pavements, the\ value\ used\ in\ all$

$calculations\ is\ 650\ psi, irregardless\ of\ the\ value\ entered\ into$

$the\ MEPDG\ software)$

$k1{,}2\ :\ Global\ calibration\ factors; these\ values\ are\ set\ to\ 1.0\ in$

$the\ software.$

$\beta c1, c2: Local\ calibration\ constants;\ these\ values\ are\ set\ to\ 1.0\ in$

$the\ software,$

8. CSM Cracking Model

$$FC_{ctb} = C_1 + \frac{C_2}{1 + e^{C_3 - C_4(Damage)}} \qquad Eq. A - 22$$

Where:

$FCctb = \ Area\ of\ fatigue\ cracking, sq\ ft,$

$Damage: Cumulative\ damage\ index\ of\ the\ CTB\ or\ cementitious\ layer$

$C1,2,3,4\ =\ Transfer\ function\ regression\ constants;\ C1 = 1.0,$

$C2 = 1.0, C3 = 0, and\ C4 = 1{,}000, (non\ calibrated\ values)$

$CSM\ Standard\ Deviation: CTB * 1$

9. IRI Flexible Pavements Model

$$IRI = IRI_0 + 0.0150(SF) + 0.400(FC_{Total}) + 0.0080(TC) + 40.0(RD) \qquad Eq. A-\ 23$$

Where:

$IRI_0\ :\ Initial\ IRI\ after\ construction, in./mi,$

$SF\ =\ Site\ factor,$

$FC_{Total}\ =\ Area\ of\ fatigue\ cracking\ (combined\ alligator,$

$longitudinal, and\ reflection\ cracking\ in\ the\ wheel\ path),$

$percent\ of\ total\ lane\ area. All\ load\ related\ cracks\ are$

$combined\ on\ an\ area\ basis—\ length\ of\ cracks\ is\ multiplied$

$by\ 1\ ft\ to\ convert\ length\ into\ an\ area\ basis,$

$TC\ =\ Length\ of\ transverse\ cracking\ (including\ the\ reflection$

$of\ transverse\ cracks\ in\ existing\ HMA\ pavements), ft/mi, and$

$RD\ =\ Average\ rut\ depth, in.$

$$SF = Age[0.02003(PI + 1) + 0.007947(Precip + 1) + 0.000636(FI + 1)] \qquad Eq. A-\ 24$$

Where:

$SF\ =\ Site\ factor,$

$Age\ =\ Pavement\ age, yr,$

$PI\ =\ Percent\ plasticity\ index\ of\ the\ soil,$

$FI\ =\ Average\ annual\ freezing\ index, °F\ days, and$

$Precip\ =\ Average\ annual\ precipitation\ or\ rainfall, in.$

Appendix 2: Data Preparation of the Selected Roads for ME Design, IC3, IP6, EN254

A. 1 Material Properties for IC3

Two tables for required material properties of surface layer of IC3 pavement are provided in chapter 3, section 3-2-9. The related tables of material properties for other layers of IC3 pavement are provided in this part.

Table A - 1: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 3)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Layers (Individual Layer Strength Properties)						
Layer (Binder Layer + Base Layer in MB)						
Layer thickness	Required		200			mm
Mixture Volumetrics						
Unit Weight[1]	SDA[2]	2400	2455 (IP Doc. _March 2006)			Kgf/m^3
Bitumen Percentage-Pb			4.2 (IP Doc. _March 2006)			%
Bitumen Content-tb			4.38			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (by volume) at time of construction Vb	SDA	11.6	9.95			%
Air Void at time of construction Vv[3]	SDA	7	3.5			%
Theoretical specific gravity of the mix Gt			2.545(IP Doc. _March 2006)			-
Bulk or actual specific gravity of the mix Gm			2.455(IP Doc. _March 2006)			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA	0.35 A: -1.63 B: 3.84e-6	No			-

[1] Use as-constructed mix type specific values available from previous construction records.
[2] Software default available
[3] Use as-constructed mix type specific values available from previous construction records.

Table A - 2: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 4)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation 19 mm Sieve % Passing 9.5 mm Sieve % Passing 4.75 mm Sieve % Passing 0.075 mm Sieve % Passing	Required		(IP Doc. _March 2006) 99.1 72 40.4 8.3			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA[1]	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			40/50		

[1] Software default available

Table A - 3: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 5)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	**Software Default Value**	**Level 1**	**Level 2**	**Level 3**	
Indirect Tensile strength at -10 °C , TS (new HMA surface; not required for existing HMA layers)	SDA[1]	DoMT[2]			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9E-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5E-06				mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	13.48			%

[1] Software default available
[2] Depends on material type

Table A - 4: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 6)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Layer (Sandwiched Granular Base)						
Unbounded Material (type)	Required		ABGE			-
Thickness	Required		200			mm
Strength Properties						
Poisson's Ratio	SDA[1]	0.35	0.35			-
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required		300			MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes	SDA	DoMT[2]	IP doc. May 2006, km 7+600 100 98.9 93.7 78 69.1 60.1			%
Liquid Limit	SDA	DoMT				-
Plasticity Index	SDA	DoMT				-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2320 (IP doc. Nov. 2005)			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	6.5(IP doc. Nov. 2005)			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

[1] Software default available
[2] Depends on material type

Table A - 5: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 7)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Layer (Chemically Stabilized Base)						
General Properties						
Material Type	Required		Soil Cement			-
Layer thickness	Required		200			mm
Unit Weight	SDA[1]	DoMT[2]	1935 (IP documents Average)			Kgf/m^3
Poisson's Ratio	SDA	DoMT	0.25			-
Strength Properties						
Elastic/Resilient Modulus	SDA	13790	2000			MPa
Minimum Elastic/Resilient Modulus	SDA	689.5				MPa
Modulus of Rupture	SDA	4.48				MPa
Thermal Properties						
Thermal Conductivity	SDA	2.16				Watt/meter-Kelvin
Heat Capacity	SDA	1172.3				joule/kg-Kelvin

[1] Software default available
[2] Depends on material type

Table A - 6: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 8)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Layer (Subgrade1+Subgrade2)						
Unbounded Material (type)	Required		Soil Foundation			
Thickness	Required		300+Semi Infinite			mm
Strength Properties						
Poisson's Ratio	SDA[1]	0.35	0.4			
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				60	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes			(IP documents Average) 2'' 100.0 1'' 98.8 3/4'' 98.4 1/2'' 97.5 3/8'' 96.7 N4 94.8 N10 87.0 N20 76.2 N40 59.1 N80 47.0 N200 38.9			%

[1] Software default available

Table A - 7: Structure/Material input parameters (Flexible Pavement) for IC3 (Part 9)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Necessity	Software Default Value	Level 1	Level 2	Level 3	
Liquid Limit	SDA[1]	DoMT[2]	33.4(IP documents Average)			-
Plasticity Index	SDA	DoMT	12.6(IP documents Average)			-
Is layer compacted?	SDA	DoMT	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2000(IP documents Average)			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	12.2(IP documents Average)			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

A. 2 IP6

In this section, the data preparation for IP6 road is explained in different parts (material, climate, traffic, trial design).

A.2.1 Road Description

Transverse Road Profile (Proplano - Gabinete de Estudos e Projectos Lda., 2002):

- 0+000/3+640 - [0.50x3.5x3.5x2.5 (m)]x2: Two ways, Two lanes per direction (freeway)
- 3+640/10+500 - 2.5x3.5x3.5x2.5 (m): Single carriageway road, One lane per direction
- 10+500/18+090 - [1.0x3.5x3.5x2.5 (m)]x2: Two ways, Two lanes per direction (freeway)

Design Speed: 120 km/h for freeway, 100 km/h for single carriageway road

Operational Speed: 90 km/h for freeway, 80 km/h for single carriageway road

[1] Software default available
[2] Depends on material type

Length of Road: 18+090 KM

Design Type: New pavement; Pavement Type: Flexible pavement; Design Life: 20 Years

Base Construction: March 2002; Pavement Construction: January 2003; Traffic Opening: June 2004

A.2.2 Road Section Selection

As it is noted in chapter 3, the priority of selection is with the availability of data. Accordingly, the following KMs has been selected for IP6 highway: km 2+500 to 3+500

A.2.3 Selection of Design-Performance Criteria and Reliability Level

The Table A - 8 shows the values for the analysis parameters adjusted based on Portuguese pavement conditions.

Table A - 8: Analysis Parameters - Default values for ME Design V 2.1 and selected values for IP6 Project

Performance Criteria	Limit		Reliability	
	Software	IP6	Software	IP6
Initial IRI (m/km)	1	1.5	-	-
Terminal IRI (m/km)	2.7	5	90	90
AC top down fatigue cracking (m/km)	378.8	378.8		
AC bottom up fatigue cracking (%)	25	50		
AC thermal cracking (m/km)	189.4	189.4		
Permanent deformation-total rutting (mm)	19	20		
Permanent deformation- AC only (mm)	6	10		
Chemically stabilized layer- fatigue fracture (%)	25	25		

A.2.4 Design Structure

The final proposed pavement structure design for IP6 achieved by IP documents is as follows:

Table A - 9: Proposed pavement structure design for IP6 by the agency

Layer Type	Thickness H(cm)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous mixture	6	4200	0.35
Binder in Bituminous mixture	6	4200	0.35
Base in Bituminous Macadam	16	4600	0.35
Sub Base in ABGE	20	130	0.40
Soil Foundation	Semi infinite	60	0.45

Based on available ME Design layer options, the final structure which is used in the IP6 project is as follows:

Table A - 10: Applied pavement structure design for IP6 in ME Design Software

Layer Type	Thickness H(cm)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous mixture	6	2453	0.35
Binder in Bituminous mixture	6	2449	0.35
Base in Bituminous Macadam	16	2952	0.35
Sub Base in ABGE	20	130	0.40
Subgrade 1 in soil	20	60	0.45
Subgrade 2 in soil	Semi infinite	60	0.45

A.2.5 Climatic Data

Regarding the climatic data for IP6 road, the virtual station has been created by ME design software which is the combination of Coimbra and Lisboa (two regions near the road).

The other climatic data includes Longitude, Latitude, Elevation and Depth of water table (for virtual station). The coordination values for IP6 are obtained by supposing the virtual station settled in Tomar, approximately in the middle of two regions of Coimbra and Lisboa. The values are:

- Longitude : -8.4167 Decimal degree
- Latitude: 39.6 Decimal degree
- Elevation: 75 m

The depth of water table provided for IP6 project in ME Design software is considered average annual depth and default value (10 m).

A.2.6 Traffic Data

- Truck Growth Factor

To calculate the truck growth rate for IP6, it was not possible to obtain more data for IP6 traffic from the opening year, so the average growth factor obtained for the software. The initial values are created by traffic model provided by the agency.

The Table A - 11 shows the annual average daily traffic or AADT calculated based on the values of traffic simulation model.

Table A - 11: Annual Average Daily Traffic

ITIN	Origin	Destination	Opening Year	2013		2018		2023		2028		2033	
				% Heavy vehicles	AADT	% Heavy vehicles	AADT	% Heavy vehicles	AADT	% Heavy vehicles	AADT	Heavy vehicles	AADT
IP6	Peniche	Atouguia da Baleia	2004	6	15577	6.3	16043	6.3	16952	6.2	17983	6.2	19362
IP6	Atouguia da Baleia	Serra D' El-Rei	2004	6	5934	6.3	6112	6.3	6458	6.2	6851	6.2	7376
IP6	Serra D' El-Rei	A-da-Gorda (A8/IP6)	2006	8	6912	8.6	7118	8.6	7520	8.5	7976	8.4	8585

So, based on the Table A - 12, the growth factor for each section is as follows:

Table A - 12: Annual Average Daily Truck Traffic for Opening Year

ITIN	Origin	Destination	Opening Year	Growth rate 2013-2023	Growth rate 2023-2033	Average Growth rate	AADT for Opening Year	% Heavy vehicles	AADTT for Opening Year
IP6	Peniche	Atouguia da Baleia	2004	0.84949141	1.33814	1.093816	14124	6	847.44
IP6	Atouguia da Baleia	Serra D' El-Rei	2004	0.849801995	1.33799	1.093896	5380	6	322.8
IP6	Serra D' El-Rei	A-da-Gorda (A8/IP6)	2006	0.846634917	1.333314	1.089974	6407	8	512.56

- Base Year Truck-Traffic Volume

1. Two-way AADTT: The values for this part are calculated before in Table A - 12

2. Number of lanes: For IP6 (selected section), there are two lanes in each direction, so the number of lanes in the design direction is two.
3. Percent trucks in design direction: For IP6 road, a default value (Level 3) of 50 percent has been provided.
4. Percent trucks in design lane: The default (Level 3) values recommended to use based on the LDF for the most common type of truck (vehicle class 9 trucks) is as follows:

    - Single-lane roadways in one direction, LDF = 1.00.
    - Two-lane roadways in one direction, LDF = 0.90.
    - Three-lane roadways in one direction, LDF = 0.60.
    - Four-lane roadways in one direction, LDF =0.45.

    The selected section of IP6 road has two lanes in each direction. So, the LDF is equal to 0.9.

5. Operational speed (kph): Regarding IP6, the operational speed of 90 kph has been applied.

- Traffic Volume Adjustments
    - Monthly Adjustment Factors (MAF): Since there is no information available to calculate the MAF, it is assumed 1.0 for all months for all vehicle classes (recommended by MEPDG).
    - Vehicle Class Distribution: The default distribution in ME Design is chosen for IP6 road
    - Truck Hourly Distribution Factors (HDF): The HDF did not apply to IP6 pavement design. It was not required for the design type.
    - Traffic Growth Factors: The growth function for IP6 road is assumed linear for all truck classes. Regarding the growth rate, as it is noted before, the same value of 1.1 is used for all truck classes.
- Axle Load Distribution Factors

Regarding IP6 road, since there is no WIM data available at this moment, the default axle load distribution used by ME Design is applied for the design.

To briefness the Table A - 13 is provided for IP6 Traffic inputs. In the table, input values with software default values are labeled as Default.

Table A - 13: Traffic Inputs for IP6

<table>
<tr><th colspan="4">ME Field</th><th>Input Value</th></tr>
<tr><td rowspan="6">Site Specific Traffic Inputs</td><td rowspan="2">Average Annual Daily Truck Traffic (AADTT)</td><td colspan="2">Year</td><td rowspan="2">Refer to Table A - 12</td></tr>
<tr><td colspan="2">Initial two-way AADTT</td></tr>
<tr><td colspan="3">Number of Lanes in Design Direction</td><td>2</td></tr>
<tr><td colspan="3">Percent of Trucks in Design Direction</td><td>50</td></tr>
<tr><td colspan="3">Percent of Trucks in Design Lane</td><td>90</td></tr>
<tr><td colspan="3">Operational Speed</td><td>90</td></tr>
<tr><td rowspan="12">WIM Traffic Data</td><td rowspan="5">Traffic Volume Adjustment Factors</td><td colspan="2">Monthly Adjustment</td><td>Default</td></tr>
<tr><td colspan="2">Road Category</td><td>Highway</td></tr>
<tr><td colspan="2">AADTT Distribution by Vehicle Class (%)</td><td>Default</td></tr>
<tr><td colspan="2">Hourly Truck Traffic Distribution</td><td>Default</td></tr>
<tr><td colspan="2">Traffic Growth factors</td><td>Refer to Table A - 12/ linear</td></tr>
<tr><td rowspan="7">Axle Load Distribution Factors</td><td colspan="2">Axle Load Distribution</td><td>Default</td></tr>
<tr><td rowspan="4">Axle Type</td><td>Single Axle</td><td>Default</td></tr>
<tr><td>Tandem Axle</td><td>Default</td></tr>
<tr><td>Tridem Axle</td><td>Default</td></tr>
<tr><td>Quad Axle</td><td>Default</td></tr>
<tr><td rowspan="2">Distribution Type</td><td>Normal Distribution</td><td>Default</td></tr>
<tr><td>Cumulative Distribution</td><td>Default</td></tr>
<tr><td rowspan="16">General Traffic Inputs</td><td rowspan="3">Lateral traffic Wander</td><td>Mean Wheel Location (cm)</td><td></td><td>Default</td></tr>
<tr><td>Traffic wander standard deviation(cm)</td><td></td><td>Default</td></tr>
<tr><td>Design lane width(m)</td><td></td><td>3.7 m</td></tr>
<tr><td>Number axles/Truck</td><td></td><td></td><td>Default</td></tr>
<tr><td rowspan="6">Axle Configuration</td><td>Avg. axle width (edge-to-edge outside dimension) (m)</td><td></td><td>Default</td></tr>
<tr><td>Dual tire spacing (cm)</td><td></td><td>Default</td></tr>
<tr><td>Tire pressure (kPa)</td><td></td><td>Default</td></tr>
<tr><td rowspan="3">Axle spacing (cm)</td><td>Tandem Axle</td><td>Default</td></tr>
<tr><td>Tridem Axle</td><td>Default</td></tr>
<tr><td>Quad Axle</td><td>Default</td></tr>
<tr><td rowspan="6">Wheelbase</td><td rowspan="3">Average axle spacing (m)</td><td>Short</td><td>Default</td></tr>
<tr><td>Medium</td><td>Default</td></tr>
<tr><td>Long</td><td>Default</td></tr>
<tr><td rowspan="3">Percent of trucks</td><td>Short</td><td>Default</td></tr>
<tr><td>Medium</td><td>Default</td></tr>
<tr><td>Long</td><td>Default</td></tr>
</table>

A.2.7 Material Properties

The material properties of the layers have been obtained from control quality documents for IP6 prepared and accessed by IP. The average values (for the selected section) are applied in the following table and accordingly in ME Design software. For example, for the selected kms of 2+500 until 3+500, the value for each input has been obtained from IP documents (based on availability) by an interval of 0+025 and the average value of that input are then calculated. The Table A - 14 to Table A - 22 are the summary of excel calculations for the selected section.

Table A - 14: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 1)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Default Value	Level 1	Level 2	Level 3	
Drainage and Surface Properties						
Surface Shortwave Absorptivity (as)	SDA[1]	0.85				-
Is endurance limit applied?	SDA	False				-
Endurance limit (micro strain)	SDA	100				-
Layer interface	SDA	1 for all layers				-
Layers (Individual Layer Strength Properties)						
Layer (Surface Layer in BB)						
Layer thickness	Required		60			mm
Mixture Volumetrics						
Unit Weight	SDA	2400	2409[2]			Kgf/m^3
Bitumen Percentage- Pb			4.8			%
Bitumen Content-tb			5.04			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (Vb)	SDA	11.6	11.09			%
Air Void (Vv)	SDA	7	4.74			%
Theoretical specific gravity of the mix Gt			2.53			-
Bulk or actual specific gravity of the mix Gm			2.41			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA	0.35 A: -1.63 B: 3.84e-6	No			-

[1] Software default available
[2] IP Doc. _Nov. 2003

Table A - 15: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 2)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation **19 mm Sieve % Passing** **9.5 mm Sieve % Passing** **4.75 mm Sieve % Passing** **0.075 mm Sieve % Passing**	Required		100 74.7 45.4 6.4			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA	21.1	21.1			ºC

Table A - 16: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 3)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	**Software Default Value**	**Level 1**	**Level 2**	**Level 1**	
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			60/70		
Indirect Tensile strength at -10 ºC, TS (new HMA surface; not required for existing HMA layers)	SDA	DoMT[1]			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06				mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	16			%

[1] Depends on material type

Table A - 17: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 4)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Layers (Individual Layer Strength Properties)						
Layer (Binder Layer_BB)						
Layer thickness	Required		60			mm
Mixture Volumetrics						
Unit Weight	SDA	2400	2400(IP Doc. _Jan 2004)			Kgf/m^3
Bitumen Percentage-Pb			4.8			%
Bitumen Content-tb			5.04			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (by volume) at time of construction Vb	SDA	11.6	11			%
Air Void at time of construction Vv	SDA	7	4.76			%
Theoretical specific gravity of the mix Gt			2.52(IP Doc. _Jan 2004)			-
Bulk or actual specific gravity of the mix Gm			2.4(IP Doc. _ Jan 2004)			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA	0.35 A: -1.63 B: 3.84e-6	No			-

Table A - 18: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 5)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation **19 mm Sieve % Passing** **9.5 mm Sieve % Passing** **4.75 mm Sieve % Passing** **0.075 mm Sieve % Passing**	Required		97.1 76 49.9 5.7			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			60/70		
Indirect Tensile strength at -10 ºC, TS (new HMA surface; not required for existing HMA layers)	SDA	DoMT			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06				mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	13.54			%

Table A - 19: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 6)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Layers (Individual Layer Strength Properties)						
Layer (Base Layer_MB)						
Layer thickness	Required		160			mm
Mixture Volumetrics						
Unit Weight	SDA	2400	2375			Kgf/m^3
Bitumen Percentage-Pb			4.4			%
Bitumen Content-tb			4.6			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (by volume) at time of construction Vb	SDA	11.6	10			%
Air Void at time of construction Vv	SDA	7	4			%
Theoretical specific gravity of the mix Gt			2.5			-
Bulk or actual specific gravity of the mix Gm			2.4			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA	0.35 A: -1.63 B: 3.84e-6	No			-

Table A - 20: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 7)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation **19 mm Sieve % Passing** **9.5 mm Sieve % Passing** **4.75 mm Sieve % Passing** **0.075 mm Sieve % Passing**	Required		87.6 62.9 47.2 27.3			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			60/70		
Indirect Tensile strength at -10 ºC, TS (new HMA surface; not required for existing HMA layers)	SDA	DoMT			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06				mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	15			%

Table A - 21: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 8)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Layer (Granular Base_ABGE)						
Unbounded Material (type)	Required		ABGE			-
Thickness	Required		200			mm
Strength Properties						
Poisson's Ratio	SDA	0.35	0.4			-
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				130	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes	SDA	DoMT	100 90.9 72.3 52.9 44.0 37.7			%
Liquid Limit	SDA	DoMT				-
Plasticity Index	SDA	DoMT				-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2339 (IP doc. Avg. Nov. 2003, Jan. 2004)			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	5(IP doc. Avg. Nov. 2003, Jan. 2004)			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

Table A - 22: Structure/Material input parameters (Flexible Pavement) for IP6 (Part 9)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Layer (Subgrade1+Subgrade2)						
Unbounded Material (type)	Required		Soil Foundation			
Thickness	Required		200+Semi Infinite			mm
Strength Properties						
Poisson's Ratio	SDA	0.35	0.45			
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				60	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes			2'' 100 1'' 100 3/4'' 99.2 1/2'' 97.4 3/8'' 96.2 N4 94.3 N10 92.7 N20 88 N40 86.9 N80 80.8 N200 68			%
Liquid Limit	SDA	DoMT	34.8(IP doc. July 2003)			-
Plasticity Index	SDA	DoMT	18.4(IP doc. July 2003)			-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2324(IP doc. July 2003)			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	4.5(IP doc. July 2003)			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

A. 3 EN254 Road

In this section, the data preparation for EN254 road is explained in different parts (material, climate, traffic, trial design).

A.3.1 Road Description

Transverse Road Profile:

0+000/ 3+739.897- 2.25x3.5x3.5x2.25 (m): Single carriageway road, One lane per direction

Design Speed: 100 km/h

Operational Speed: 80 km/h

Length of Road: 3+739.897km

Design Type: New pavement; Pavement Type: Flexible pavement; Design Life: 20 Years

Base Construction: November 2003; Pavement Construction: June 2004; Traffic Opening: September 2004

As for the road section selection, the following kms have been selected for EN254: km 0+000 to km 1+400.

A.3.2 Selecting Design-Performance Criteria and Reliability Level

The Table A - 23 shows the values for the analysis parameters adjusted based on Portuguese pavement conditions.

Table A - 23: Analysis Parameters - Default values for ME Design V 2.1 and selected values for En254

Performance Criteria	Limit		Reliability	
	Software	En254	Software	En254
Initial IRI (m/km)	1	1.5	-	-
Terminal IRI (m/km)	2.7	4	90	90
AC top down fatigue cracking (m/km)	378.8	378.8	90	90
AC bottom up fatigue cracking (%)	25	50	90	90
AC thermal cracking (m/km)	189.4	189.4	90	90
Permanent deformation-total rutting (mm)	19	20	90	90
Permanent deformation- AC only (mm)	6	10	90	90
Chemically stabilized layer- fatigue fracture (%)	25	25	90	90

A.3.3 Design Structure

The final proposed pavement structure design for EN254 achieved by IP documents is as follows:

Table A - 24: Proposed pavement structure design for EN254 by the agency

Layer Type	Thickness H(m)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous mixture	0.05	3500	0.35
Binder in Bituminous Macadam	0.10	3800	0.35
Base in ABGE	0.20	290	0.35
Sub Base in Crushed Gravel	0.20	120	0.35
Soil Foundation	Semi infinite	80	0.40

Based on available ME Design layer options, the final structure which is used in the En254 project is as follows:

Table A - 25: Applied pavement structure design for EN254 in ME Design

Layer Type	Thickness H(m)	Modulus E/M_r(MPa)	Poisson's Ratio
Surface in Bituminous mixture	0.05	1949	0.35
Binder in Bituminous Macadam	0.10	2877	0.35
Base in ABGE	0.20	290	0.35
Sub Base in Crushed Gravel	0.20	120	0.35
Subgrade 1 in soil	0.20	80	0.40
Subgrade 2 in soil	Semi infinite	80	0.40

A.3.4 Climatic Data

Regarding the climatic data for En254 road, the virtual station has been created by ME design software which is the combination of Beja and Lisboa (Two regions near the road). The other climatic data includes longitude, latitude, elevation and depth of water table (for virtual station). The coordination values for En254 are obtained by supposing the virtual station settled in Evora, approximately in the middle of two regions of Beja and Lisboa. The values are: Longitude: -7.9 Decimal degree; Latitude: 38.5667 Decimal degree; Elevation: 275 m.

The depth of water table provided for EN254 project in ME Design software is considered average annual depth and default value (10 m).

A.3.5 Traffic Data

- Truck Growth Factor (growth rate)

The Table A - 26 shows AADT calculated based on the values of traffic simulation model.

Table A - 26: Annual Average Daily Traffic

ITIN	Origin	Destination	Opening Year	2013		2018		2023		2028		2033	
				AADT	% Heavy vehicles	AADT	% Heavy vehicles	AADT	% Heavy vehicles	AADT	% Heavy vehicles	AADT	% Heavy vehicles
EN254	Évora	S Miguel de Machede	2004	2916	6	3003	6.3	3173	6.3	3366	6.2	3625	6.2
EN254	S Miguel de Machede	Redondo Sul	2004	2802	8	2886	8.6	3048	8.6	3233	8.5	3480	8.4

So, the average growth factor obtained by the Table A - 27 is applied for the software.

Table A - 27: Annual Average Daily Truck Traffic for opening year

ITIN	Origin	Destination	Opening Year	Growth rate 2013-2023	Growth rate 2023-2033	Average Growth rate	AADT for Opening Year	% Heavy vehicles	AADTT for Opening Year
EN254	Évora	S Miguel de Machede	2004	0.84822	1.34068	1.09445	2644	7	185
EN254	S Miguel de Machede	Redondo Sul	2004	0.84507	1.33429	1.08968	2542	8	203

- Base year truck-traffic volume

1. Two-way AADTT: The values for this part are calculated before in Table A - 27.
2. Number of lanes: For EN254 (selected section), there are one lane in each direction, so, the number of lanes in the design direction is one.
3. Percent trucks in design direction: For EN254 road, a default value (Level 3) of 50 percent has been provided.

4. Percent trucks in design lane: The default (Level 3) values recommended to use based on the LDF for the most common type of truck (vehicle class 9 trucks) is as follows:

    - Single-lane roadways in one direction, LDF = 1.00.
    - Two-lane roadways in one direction, LDF = 0.90.
    - Three-lane roadways in one direction, LDF = 0.60.
    - Four-lane roadways in one direction, LDF =0.45.

    The En254 road has one lane in each direction. So, the LDF is equal to 1.00.

5. Operational speed (kph): Regarding EN254, the operational speed of 80 kph has been applied.

- Traffic Volume Adjustments
    - Monthly Adjustment Factors (MAF): Since there is no information available to calculate the MAF, it is assumed 1.0 for all months for all vehicle classes (recommended by MEPDG).
    - Vehicle Class Distribution: The default distribution in ME Design is chosen for EN254 road (since there is no enough data in this part).
    - Truck Hourly Distribution Factors (HDF): The HDF did not apply to En254 pavement design. It was not required for the design type.
    - Traffic Growth Factors: The growth function for EN254 road is assumed linear for all truck classes. Regarding the growth rate, as it is noted earlier in this chapter, the same value of 1.1 is used for all truck classes.
- Axle Load Distribution Factors

Regarding EN254 road, since there is no WIM data available at this moment, the default axle load distribution used by ME Design is applied for the design.

To briefness the Table A - 28 is provided for En254 Traffic inputs. In the table, input values with software default values are labeled as Default.

Table A - 28: Traffic Inputs for EN254

ME Field				Input Value
Site Specific Traffic Inputs	Average Annual Daily Truck Traffic (AADTT)	Year		Refer to Table A - 27
		Initial two-way AADTT		
	Number of Lanes in Design Direction			1
	Percent of Trucks in Design Direction			50
	Percent of Trucks in Design Lane			100
	Operational Speed			80
WIM Traffic Data	Traffic Volume Adjustment Factors	Monthly Adjustment		Default
		Road Category		Single carriageway road
		AADTT Distribution by Vehicle Class (%)		Default
		Hourly Truck Traffic Distribution		Default
		Traffic Growth factors		Refer to Table A - 27 / linear
	Axle Load Distribution Factors	Axle Load Distribution		Default
		Axle Type	Single Axle	Default
			Tandem Axle	Default
			Tridem Axle	Default
			Quad Axle	Default
		Distribution Type	Normal Distribution	Default
			Cumulative Distribution	Default
General Traffic Inputs	Lateral traffic Wander	Mean Wheel Location (cm)		Default
		Traffic wander standard deviation(cm)		Default
		Design lane width(m)		3.5 m
	Number axles/Truck			Default
	Axle Configuration	Avg. axle width (edge-to-edge outside dimension) (m)		Default
		Dual tire spacing (cm)		Default
		Tire pressure (kPa)		Default
		Axle spacing (cm)	Tandem Axle	Default
			Tridem Axle	Default
			Quad Axle	Default
	Wheelbase	Average axle spacing (m)	Short	Default
			Medium	Default
			Long	Default
		Percent of trucks	Short	Default
			Medium	Default
			Long	Default

A.3.6 Material Properties

The material properties of the layers are obtained from control quality documents for En254 prepared and accessed by IP. The average values (for the selected section) are applied in ME Design. For example, for the selected KMs of 0+000 until 1+400, the value for each input is obtained from IP documents (based on availability) by an interval of 0+025 and the average value of that input are then calculated. The Table A - 29 to Table A - 35 are the summary of excel calculations for the selected sections.

Table A - 29: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 1)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Drainage and Surface Properties						
Surface Shortwave Absorptivity (as)	SDA[1]	0.85				-
Is endurance limit applied?	SDA	False				-
Endurance limit (microstrain)	SDA	100				-
Layer interface	SDA	1 for all layers				-
Layers (Individual Layer Strength Properties)						
Layer (Surface Layer in BB)						
Layer thickness	Required		50			mm
Mixture Volumetrics						
Unit Weight	SDA	2400	2343			Kgf/m^3
Bitumen Percentage-Pb			5.1			%
Bitumen Content-tb			5.37			%
Bitumen Unit Weight G_b					10.3	Kgf/m^3
Aggregate Unit Weight G_a					27	Kgf/m^3
Effective Binder Content (by volume) Vb	SDA	11.6	11.6			%
Air Void at time of construction Vv	SDA	7	6.3			%
Theoretical specific gravity of the mix Gt			2.501			-
Bulk or actual specific gravity of the mix Gm			2.343			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA		No			-

[1] Software default available

Table A - 30: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 2)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation (mm) **19** **9.5** **4.75** **0.075**	Required		99.5 74.9 48.5 3.3			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			60/70		
Indirect Tensile strength at -10 ºC, TS	SDA	DoMT			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06				mm/mm/deg C
Voids in mineral aggregate ($V_a\ or\ V_{MA}$)	SDA	18.6	17.9			%

Table A - 31: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 3)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Layers (Individual Layer Strength Properties)						
Layer (Binder Layer_MB)						
Layer thickness	Required		100			mm
Mixture Volumetrics						
Unit Weight	SDA	2400	2325			Kgf/m^3
Bitumen Percentage-Pb			4			%
Bitumen Content-tb			4.17			%
Bitumen Unit Weight G_b					10.3 (Supposed)	Kgf/m^3
Aggregate Unit Weight G_a					27 (Supposed)	Kgf/m^3
Effective Binder Content (by volume) at time of construction Vb	SDA	11.6	9.3			%
Air Void at time of construction Vv	SDA	7	5.7			%
Theoretical specific gravity of the mix Gt			2.466			-
Bulk or actual specific gravity of the mix Gm			2.325			-
Poisson's Ratio			0.35			
Is Poisson's ratio calculated?	SDA	0.35 A: -1.63 B: 3.84e-6	No			-

Table A - 32: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 4)

ME Field	Type of Input		Input Value (Average of two selected sections)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Mechanical Properties						
Dynamic modulus, E_{HMA} (new HMA layers)						MPa
Asphalt Mix: Aggregate Gradation **19 mm Sieve % Passing** **9.5 mm Sieve % Passing** **4.75 mm Sieve % Passing** **0.075 mm Sieve % Passing**	Required		98.6 71.7 44.9 6			%
NCHRP 1-37A viscosity-based E* predictive model						
Reference Temperature	SDA	21.1	21.1			ºC
Asphalt Binder						
if Superpave Binding Grading (level 1 & 2)						ºC
High Temp	Required					
Low Temp	Required					
if Conventional Viscosity Grade (level 2 & 3)						-
Viscosity Grade	Required					
if Conventional Penetration Grade (level 2 & 3)						-
Penetration Grade	Required			60/70		
Indirect Tensile strength at -10 ºC, TS (new HMA surface; not required for existing HMA layers)	SDA	DoMT			2.59 (Default)	MPa
Creep Compliance	SDA	DoMT				1/GPa
Thermal Properties						
Thermal Conductivity of Asphalt	SDA	1.16				Watt/meter-Kelvin
Heat Capacity of Asphalt	SDA	963				joule/kg-Kelvin
Is Thermal Contraction calculated?						-
Mix coefficient of thermal contraction	SDA	9e-006				mm/mm/deg C
Aggregate coefficient of thermal contraction	SDA	5e-06				mm/mm/deg C
Voids in mineral aggregate (V_a or V_{MA})	SDA	18.6	15			%

Table A - 33: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 5)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Layer (Granular Base_ABGE)						
Unbounded Material (type)	Required		ABGE			-
Thickness	Required		200			mm
Strength Properties						
Poisson's Ratio	SDA	0.35	0.35			-
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				290	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes	SDA	DoMT	1'' 81.7 3/4'' 65.1 #4 32.3 #10 23.2 #40 11.4 #200 3.8			%
Liquid Limit	SDA	DoMT				-
Plasticity Index	SDA	DoMT				-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2411			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	4.6			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

Table A - 34: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 6)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Value	Level 1	Level 2	Level 3	
Layer (Granular Sub Base _Crushed Gravel)						
Unbounded Material (type)	Required		Crushed Gravel			-
Thickness	Required		200			mm
Strength Properties						
Poisson's Ratio	SDA	0.35	0.35			-
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				120	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes	SDA	DoMT	1'' 78.6 3/4'' 67 N4 30.7 N10 24 N40 12.8 N200 4.4			%
Liquid Limit	SDA	DoMT				-
Plasticity Index	SDA	DoMT				-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2404			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	3.7			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

Table A - 35: Structure/Material input parameters (Flexible Pavement) _EN254 (Part 7)

ME Field	Type of Input		Input Value (Average)			Input Unit
	Requirement	Software Default Value	Level 1	Level 2	Level 3	
Layer (Subgrade1+Subgrade2)						
Unbounded Material (type)	Required		Soil Foundation			
Thickness	Required		200+Semi Infinite			mm
Strength Properties						
Poisson's Ratio	SDA	0.35	0.4			
Coefficient of Lateral Pressure	SDA	0.5				k0
Resilient Modulus	Required				80	MPa
Sieve (Gradation and other engineering properties)						
Percent Passing for requested sieve sizes			3/4'' 100 N10 87 N40 40.4 N200 11.6			%
Liquid Limit	SDA	DOMT				-
Plasticity Index	SDA	DoMT				-
Is layer compacted?	SDA	Compacted	YES			-
Maximum Dry Unit Weight	SDA	DoMT	2100			Kgf/m^3
Specify Gravity of Soils	SDA	DoMT				m/hr
Saturated Hydraulic Conductivity	SDA	DoMT				-
Optimum Gravimetric Water Content	SDA	DoMT	8.1			%
Soil Water Characteristic Curve Parameter (af, bf, cf, hr)	SDA	DoMT				-

Appendix 3: Longitudinal Profile Plans for Road Section Selections for the Roads IC3, IP6, EN254

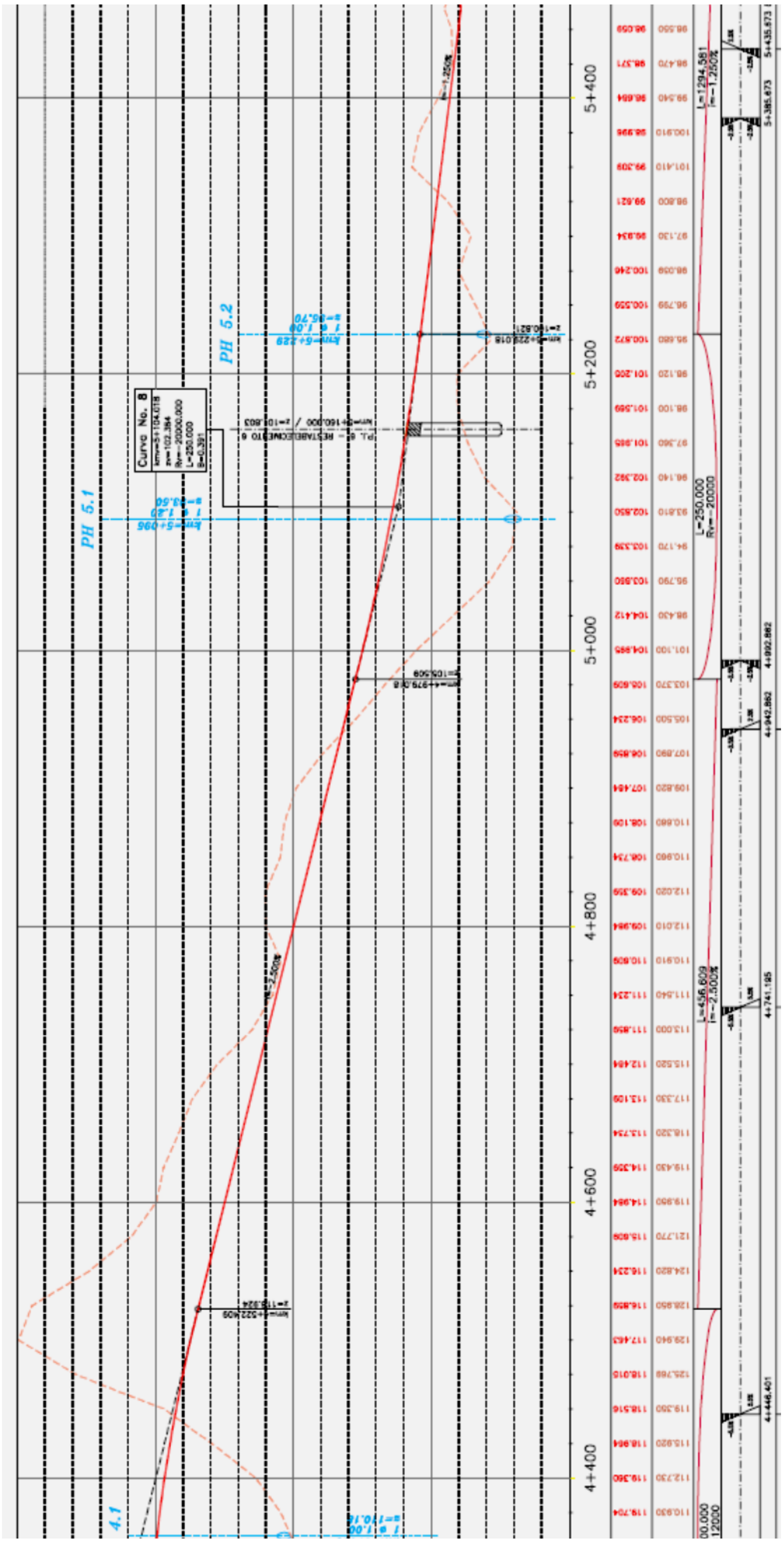

Figure A- 1: Longitudinal profile plan for km 4+400 to 5+400_IC3

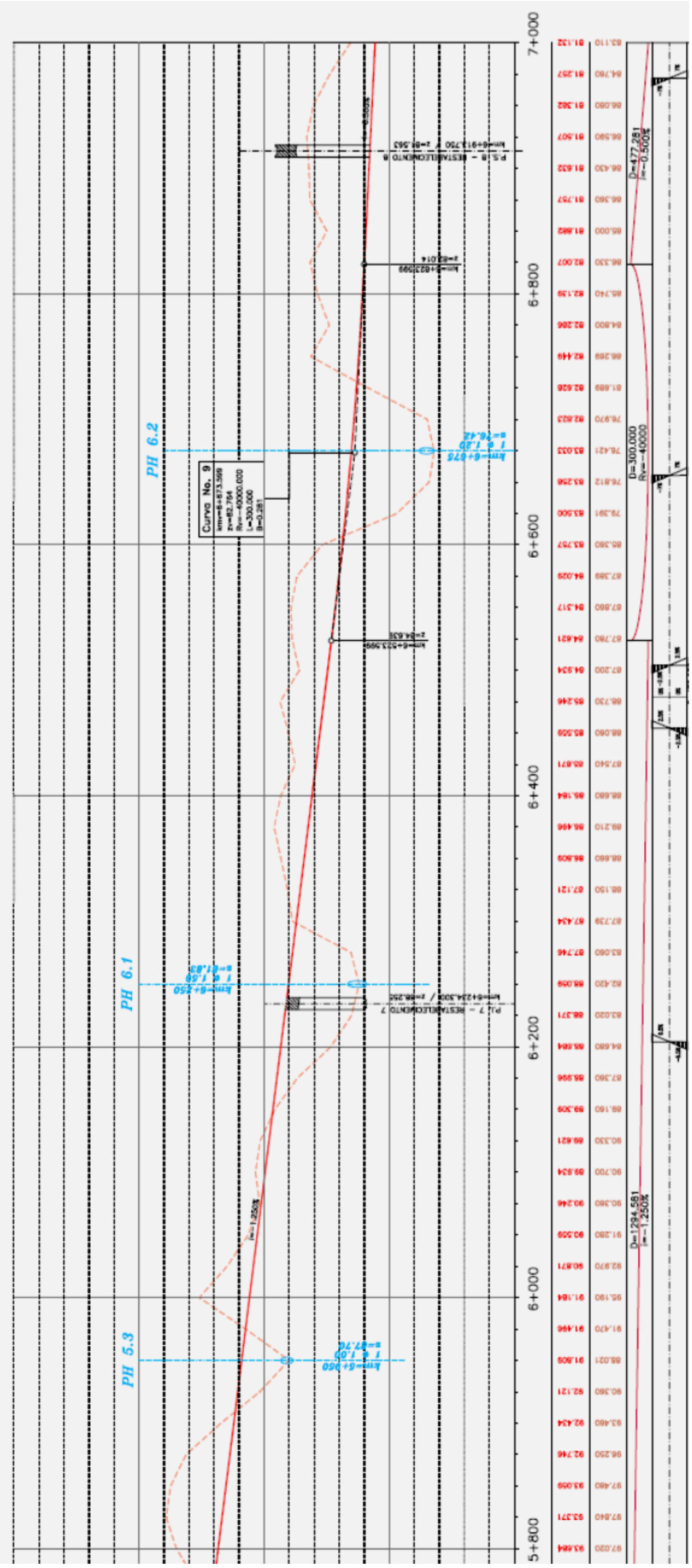

Figure A- 2: Longitudinal profile plan for km 5+800 to 7+000_IC3

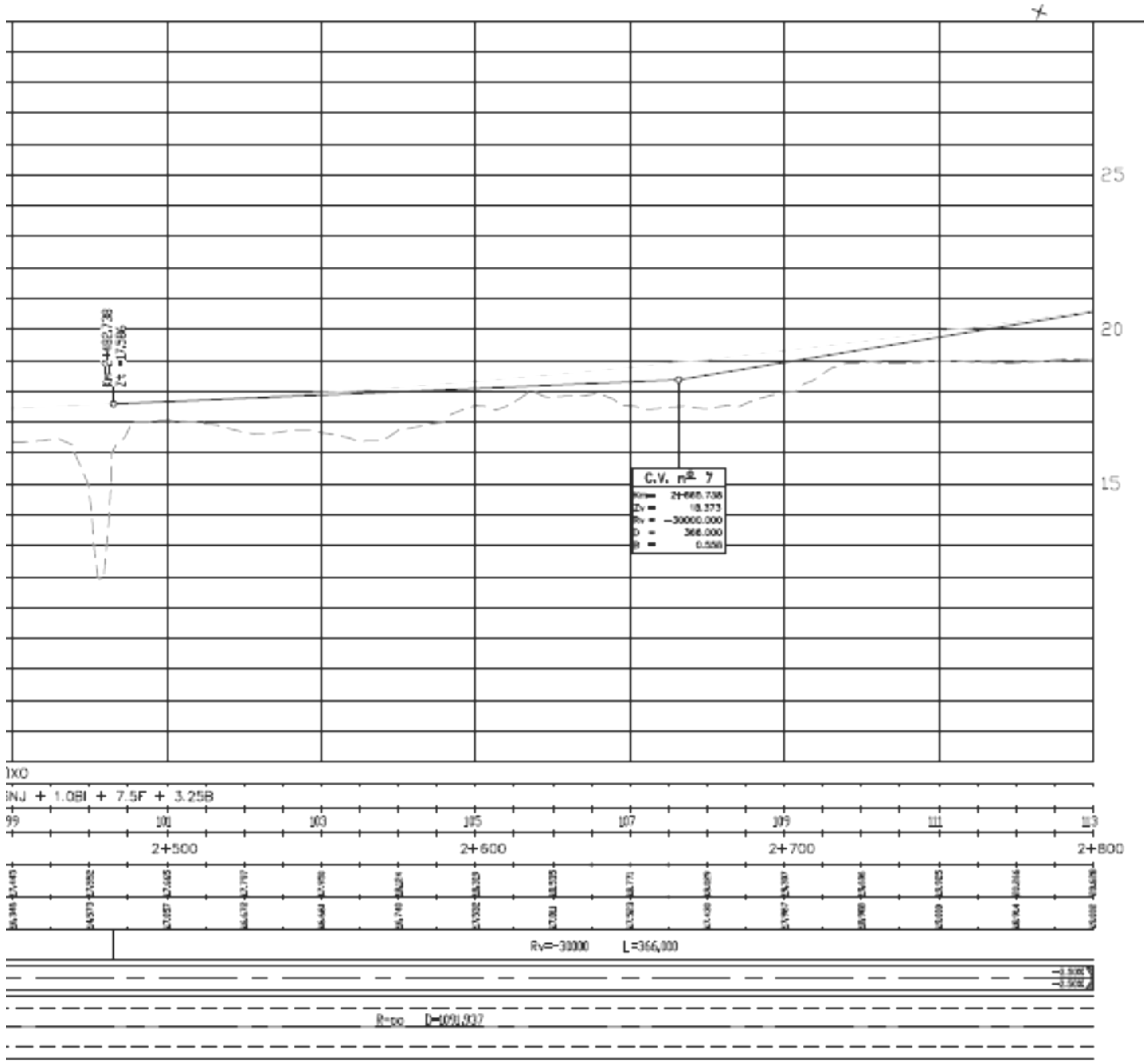

Figure A- 3: Longitudinal profile plan for km 2+500 to 2+800_IP6

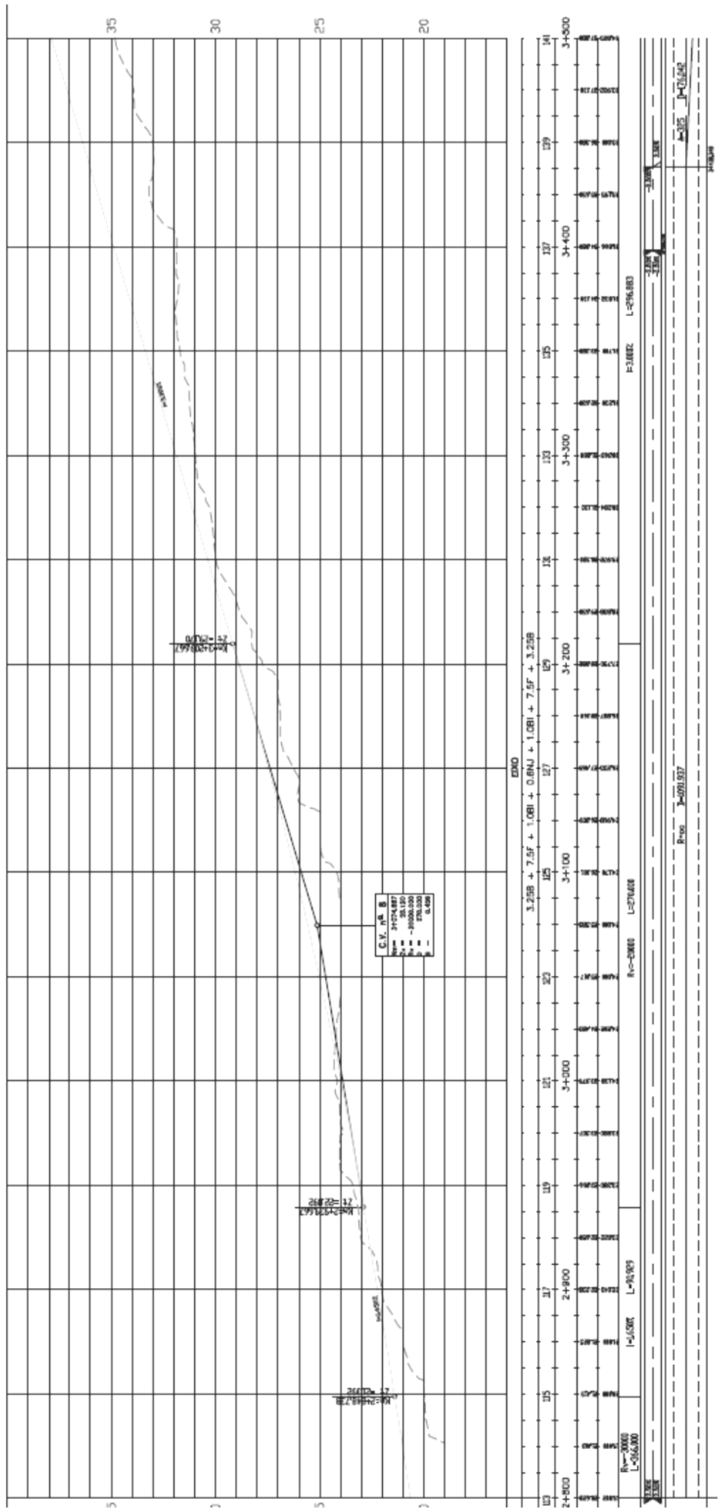

Figure A- 4: Longitudinal profile plan for km 2+800 to 3+500_IP6

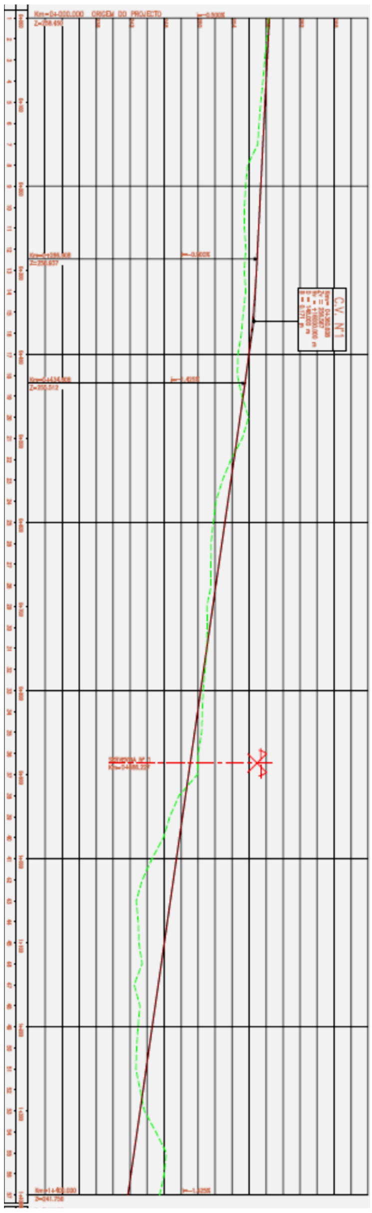

Figure A- 5: Longitudinal profile plan for km 0+000 to 1+400_EN254

Appendix 4: Calculation of Dynamic Modulus of Flexible Layers for BISAR Based on New Service Temperature

This appendix provides the results of the associated calculations mentioned in the section 4.6 for the selected roads.

Table A - 36: Structure design before dynamic modulus calculation for PETE temperature_IC3

Layer Type	Thickness H(m)	Modulus E/M_r (MPa)	Poisson's Ratio
Surface in Betão Betuminoso	0.05	4500	0.35
Binder + Base in Macadam Bituminous	0.20	4500	0.35
Sandwich Granular Base in ABGE	0.20	300	0.35
Chemically Stabilized Sub grade 1: Soil Cement	0.20	2000	0.25
Sub grade 2: Soil Foundation [1]	0.30	60	0.4
Sub grade 3: Soil Foundation	Semi infinite	60	0.4

Table A - 37: New dynamic modulus calculation for SHELL_IC3

Vb, Va and VMA calculation		
Layer name	BB layer	MB layer
Layer thickness	5.00	20.00
gb (kN/m3) bitumen unit weight	10.30	10.30
ga (kN/m3) agg. Unit weight	27.00	27.00
Pb % bitumen percentage	5.20	4.20
tb % bitumen content	5.49	4.38
Effective Binder Content (by volume) Vb %	12.22	9.94
Theoretical specific gravity of the mix Gt	2.49	2.55
Bulk or actual specific gravity of the mix Gm	2.42	2.46
Air Void at time of construction Vv %	2.81	3.54
Agg. Content by volume Va %	84.97	86.52
Voids in mineral aggregate (VMA) %	15.03	13.48
Bitumen Stiffness_Sb calculation		
Penetration Grade	35/50	35/50
Service temp by PETE method_TS - (°C)	28.60	28.60
Penetration_ P25 - (0,1 mm)	40.00	40.00
Softening point (Ring and ball) _Tab - (°C)	52.00	52.00
P25r - (0,1 mm)	26.00	26.00
Tabr - (oC)	61.85	61.85
IPen	-0.06	-0.06
Average traffic speed for heavy vehicle_V (km/h)	80.00	80.00
t - (s)	0.01	0.01
Bitumen Stiffness_Sb - (MPa)	25.13	25.13
Dynamic modulus calculation for SHELL (5 to 1000 MPa)		
Vb %	12.22	9.94
Va %	84.97	86.52
Sm108	10.02	10.09
Sm3109	10.61	10.63
S68	0.68	0.62
S89	0.45	0.41
A	9.62	9.71
Em - (Pa)	4160830275.68	5187200138.81
Em - (MPa)	4160.83	5187.20

Table A - 38: Structure design after dynamic modulus calculation for PETE temperature_IC3

Layer Type	Thickness H(m)	Modulus E/M_r (MPa)	Poisson's Ratio
Surface in Betão Betuminoso	0.05	4161	0.35
Binder + Base in Macadam Bituminous	0.20	5187	0.35
Sandwich Granular Base in ABGE	0.20	300	0.35
Chemically Stabilized Sub grade 1: Soil Cement	0.20	2000	0.25
Sub grade 2: Soil Foundation [1]	0.30	60	0.40
Sub grade 3: Soil Foundation	Semi infinite	60	0.40

Project: BISAR results_IC3_based on new dynamic modulus, **Calculated**: 5/16/2018 21:19

Table A - 39: Structure Configurations for IC3 Road

Layer Number	Thickness (m)	Modulus Elasticity (MPa)	Poisson's Ratio	Shear Compliance (m^3/N)
1	0.05	4160.00	0.35	0.00
2	0.20	5190.00	0.35	0.00
3	0.20	300.00	0.35	0.00
4	0.20	2000.00	0.25	0.00
5	0.30	60.00	0.40	0.00
6	---	60.00	0.40	---

Table A - 40: Load Configurations for IC3 Road

Load Number	Vertical Load (kN)	Vertical Stress (MPa)	Horz. (Shear) Load (kN)	Horz. (Shear) Stress (MPa)	Radius (m)	X-Coord. (m)	Y-Coord. (m)	Shear Angle (Degrees)
1	20.00	0.58	0.00	0.00	0.11	0.00	-0.16	0.00
2	20.00	0.58	0.00	0.00	0.11	0.00	0.16	0.00

Table A - 41: BISAR Results for IC3 Road

Position No.	Layer No.	Coordinate (m)			Stress (MPa)			Strain (µstrain)			Displacement (µm)		
		X	Y	Z	XX	YY	ZZ	XX	YY	ZZ	UX	UY	UZ
1	2	0.00	0.00	0.25	0.44	0.33	-0.04	66.10	37.10	-61.00	0.00	0.00	250.00
2	2	0.00	0.16	0.25	0.44	0.36	-0.04	62.70	42.90	-62.10	0.00	6.41	245.00
3	4	0.00	0.00	0.65	0.09	0.09	-0.01	37.10	34.00	-27.60	0.00	0.00	222.00
4	4	0.00	0.16	0.65	0.09	0.08	-0.01	35.90	31.10	-26.00	0.00	5.20	219.00
5	5	0.00	0.00	0.65	0.00	0.00	-0.01	28.10	26.10	-109.00	0.00	0.00	222.00
6	5	0.00	0.16	0.65	0.00	0.00	-0.01	27.30	24.20	-104.00	0.00	4.01	219.00

Damage Calculation Results for BISAR_IC3

Table A - 42: BISAR Damage Calculation for Tensile Strain at the bottom of Asphalt Layer (MB Layer) _IC3

Tensile Strain at the bottom of Asphalt Layer (MB Layer)	6.61E-05
Effective Binder Content (by volume) at time of construction Vb (%)	9.94
Asphalt concrete stiffness modulus E (Pa)	5.19E+09
Admissible number of ESALs	2.09E+08
Damage % (3.71E+07/2.09E08) %	18

Table A - 43: BISAR Damage Calculation for Compression strain on top of the subgrade_IC3

Compression strain on top of the subgrade	-1.08500E-04
Ks for 95% of survival probability	1.80000E-02
Admissible number of ESALs	7.6E+08
Damage % (3.71E+07/7.574798E08) %	5

Table A - 44: BISAR Damage Calculation for Tensile Stress at the bottom of soil cement_IC3

Tensile Stress at the bottom of soil cement MPa	9.42E-02
Compression Stress (Diametrical) MPa (based on EP docs)	0.25
Tensile Stress obtain from Compression Stress	0.375
a constant value (mean value is considered)	-0.08
Admissible number of ESALs	2.3E+09
Damage % (3.71E+07/22.96149E08) %	2

Table A - 45: Results obtained by IP project_IC3

Criterion		**Strain**	**NAEP80**	**Damage**
Tensile Strain	Project Value	93E-06	3.71E+07	63 %
	ADM_BISAR	82E-06	5.92E+07	
Compression Strain	Project Value	231E-06	3.71E+07	10 %
	ADM_BISAR	129E-06	3.79E+08	

Table A - 46: Calculation of new AADTT for ME Design project _IC3

Road Section Name	Truck lane distribution factor (LDF)	Directional distribution factor (DDF)	Class type	Average Annual Growth Rate	Design Period in years	Growth factor	Aggression Factor (obtained from another sheet_US unit)	Accumulated Number of 80 kN Standard Axle Passages	New Average Annual Daily Truck Traffic $= \frac{N}{365 \times \alpha \times C \times P \times D \times L}$	Obtained AADTT from IP project
	L	D	T	t	P	C	Alpha	N (dim,80)	$AADTT_{new}$	$AADTT_{IP}$
IC3 road (1) _AtalaiaIP6-AtalaiaEN110	0.9	0.5	T2	0.011	20	1.11	7.82	3.71E+07	1299	837
IC3 road (2) _Atalaia EN110_Asseiceira	0.9	0.5	T3	0.011	20	1.11	7.82	3.71E+07	1299	520
IC3 road (3) _Asseiceira-Santa Cita (EN110)	0.9	0.5	T3	0.011	20	1.11	7.82	3.71E+07	1299	526
IC3 road (4) _Santa Cita (EN110)-Valdonas	0.9	0.5	T3	0.011	20	1.11	7.82	3.71E+07	1299	760
IC3 road (5) _Valdonas-A13-IC9	0.9	0.5	T3	0.011	20	1.11	7.82	3.71E+07	1299	679
IC3 road (6) _A13-IC9-Alviobeira	0.9	0.5	T4	0.011	20	1.11	7.82	3.71E+07	1299	464

Table A - 47: Structure design before dynamic modulus calculation for PETE temperature_IP6

Layer Type	**Thickness H(m)**	**Modulus E/M_r (MPa)**	**Poisson's Ratio**
Surface in Betão Betuminoso	0.06	4200	0.35
Binder in Betão Betuminoso	0.06	4200	0.35
Base in Macadam Bituminous	0.16	4600	0.35
Sub Base in ABGE	0.20	130	0.40
Subgrade 1 in soil	0.20	60	0.45
Subgrade 2 in soil	Semi infinite	60	0.45

Table A - 48: New dynamic modulus calculation for SHELL_IP6

Vb, Va and VMA calculation			
Layer name	Surface	Binder	Base
Layer thickness	6.00	6.00	16.00
gb (kN/m3) bitumen unit weight	10.30	10.30	10.30
ga (kN/m3) agg. Unit weight	27.00	27.00	27.00
Pb % bitumen percentage	4.80	4.80	4.40
tb % bitumen content	5.04	5.04	4.60
Effective Binder Content (by volume) Vb %	11.12	11.12	10.34
Theoretical specific gravity of the mix Gt	2.53	2.52	2.50
Bulk or actual specific gravity of the mix Gm	2.41	2.40	2.40
Air Void at time of construction Vv %	4.74	4.76	4.00
Agg. Content by volume Va %	84.14	84.12	85.66
Voids in mineral aggregate (VMA) %	15.86	15.88	14.34
Bitumen Stiffness_Sb calculation			
Penetration Grade	60/70	60/70	60/70
Service temp by PETE method_TS - (°C)	28.00	28.00	28.00
Penetration_ P25 - (0,1 mm)	65.00	65.00	65.00
Softening point (Ring and ball) _Tab - (°C)	52.00	52.00	52.00
P25r - (0,1 mm)	42.25	42.25	42.25
Tabr - (oC)	56.29	56.29	56.29
IPen	-0.14	-0.14	-0.14
Average traffic speed for heavy vehicle_V (km/h)	80.00	80.00	80.00
t - (s)	0.01	0.01	0.01
Bitumen Stiffness_Sb - (MPa)	12.03	12.03	12.03
Dynamic modulus calculation_SHELL (5 to 1000 Mpa)			
Vb %	11.12	11.12	10.34
Va %	84.14	84.12	85.66
Sm108	9.99	9.99	10.05
Sm3109	10.60	10.60	10.62
S68	0.65	0.65	0.63
S89	0.46	0.46	0.43
A	9.39	9.39	9.47
Em - (Pa)	2.45E+09	2.45E+09	2.95E+09
Em - (MPa)	2.45E+03	2.45E+03	2.95E+03

Table A - 49: Structure design after dynamic modulus calculation for PETE temperature

Layer Type	**Thickness H(m)**	**Modulus E/M_r (MPa)**	**Poisson's Ratio**
Surface in Betão Betuminoso	0.06	2453	0.35
Binder in Betão Betuminoso	0.06	2449	0.35
Base in Macadam Bituminous	0.16	2952	0.35
Sub Base in ABGE	0.20	130	0.40
Subgrade 1 in soil	0.20	60	0.45
Subgrade 2 in soil	Semi infinite	60	0.45

Project: BISAR results_IP6_based on new dynamic modulus, **Calculated:** 5/16/2018 21:31

Table A - 50: Structure Configurations for IP6 Road

Layer Number	Thickness (m)	Modulus Elasticity (MPa)	Poisson's Ratio
1	0.06	2.45E+03	0.35
2	0.06	2.45E+03	0.35
3	0.16	2.95E+03	0.35
4	0.20	1.30E+02	0.40
5	0.20	6.00E+01	0.45
6	Semi infinite	6.00E+01	0.45

Table A - 51: Load Configurations for IP6 Road

Load Number	**Vertical Load (kN)**	**Vertical Stress (MPa)**	**Horz. (Shear) Load (kN)**	**Horz. (Shear) Stress (MPa)**	**Radius (m)**	**X-Coord. (m)**	**Y-Coord. (m)**	**Shear Angle (Degrees)**
1	2.00E+01	5.77E-01	0.00E+00	0.00E+00	1.05E-01	0.00E+00	-1.58E-01	0.00E+00
2	2.00E+01	5.77E-01	0.00E+00	0.00E+00	1.05E-01	0.00E+00	1.58E-01	0.00E+00

Table A - 52: BISAR results for IP6 road

Position No.	**Layer No.**	**Coordinate (m)**			**Stress (MPa)**			**Strain (µstrain)**			**Displacement (µm)**		
		X	**Y**	**Z**	**XX**	**YY**	**ZZ**	**XX**	**YY**	**ZZ**	**UX**	**UY**	**UZ**
1	3	0.00	0.00	0.28	0.45	0.36	-0.03	114.00	73.20	-107.00	0.00	0.00	351.00
2	3	0.00	0.16	0.28	0.44	0.37	-0.03	108.00	76.60	-105.00	0.00	12.00	343.00
3	5	0.00	0.00	0.48	0.00	0.00	-0.02	110.00	95.60	-242.00	0.00	0.00	311.00
4	5	0.00	0.16	0.48	0.00	0.00	-0.02	105.00	86.40	-227.00	0.00	14.60	305.00

Damage Calculation Results for BISAR_IP6

Table A - 53: BISAR Damage Calculation for Tensile Strain at the bottom of Asphalt Layer (MB Layer) _IP6

Tensile Strain at the bottom of Asphalt Layer (MB Layer)	1.14E-04
Effective Binder Content (by volume) at time of construction Vb (%)	10.34
Asphalt concrete stiffness modulus E (Pa)	2.95E+09
Admissible number of ESALs	4.5E+07
Damage (4.95E+07/4.5E+07) %	110

Table A - 54: BISAR Damage Calculation for Compression strain on top of the subgrade_IP6

Compression strain on top of the subgrade	-2.41900E-04
Ks for 95% of survival probability	1.80000E-02
Admissible number of ESALs	3.07E+07
Damage (4.95E+07/3.07E+07) %	162

Table A - 55: Results obtained by IP project_IP6

Criterion		**Strain**	**NAEP80**	**Damage**
Tensile Strain	Project Value	112E-06	4.95E+07	16 %
	ADM_BISAR	77.2E-06	3.17E+08	
Compression Strain	Project Value	215E-06	4.95E+07	28 %
	ADM_BISAR	156E-06	1.77E+08	

Table A - 56: Calculation of new AADTT for ME Design _IP6

Road Section Name	Truck lane distribution factor (LDF)	Directional distribution factor (DDF)	Class type	Average Annual Growth Rate	Design Period in years	Growth factor	Aggression Factor (obtained from another sheet_US unit)	Accumulated Number of 80 kN Standard Axle Passages	New Average Annual Daily Truck Traffic $= \frac{N}{365 \times \alpha \times C \times P \times D \times L}$	Obtained AADTT from IP project
	L	**D**	**T**	**t**	**P**	**C**	**Alpha**	**N (dim,80)**	**AADTTnew**	**AADTTIP**
IP6_Peniche_Atouguia da Baleia	0.9	0.5	T2	0.011	20	1.11	7.82	4.95E+07	1735	847

Table A - 57: Structure design before dynamic modulus calculation for PETE temperature_EN254

Layer Type	Thickness H(m)	Modulus E/M_r (MPa)	Poisson's Ratio
Surface in Betão Betuminoso	0.05	3500	0.35
Binder in Macadam Bituminous	0.10	3800	0.35
Base in ABGE	0.20	290	0.35
Sub Base in Crushed Gravel	0.20	120	0.35
Subgrade 1 in soil	0.20	80	0.40
Subgrade 2 in soil	Semi infinite	80	0.40

Table A - 58: New dynamic modulus calculation for SHELL_EN254

Vb, Va and VMA calculation		
Layer name	BB layer	MB layer
Layer thickness	5.00	10.00
gb (kN/m3) bitumen unit weight	10.30	10.30
ga (kN/m3) agg. Unit weight	27.00	27.00
Pb % bitumen percentage	5.10	4.00
tb % bitumen content	5.37	4.17
Effective Binder Content (by volume) Vb %	11.57	9.28
Theoretical specific gravity of the mix Gt	2.50	2.47
Bulk or actual specific gravity of the mix Gm	2.34	2.33
Air Void at time of construction Vv %	6.32	5.72
Agg. Content by volume Va %	82.11	85.00
Voids in mineral aggregate (VMA) %	17.89	15.00
Bitumen Stiffness_Sb calculation		
Penetration Grade	50/70	50/70
Service temp by PETE method_TS - (°C)	28.00	28.00
Penetration_ P25 - (0,1 mm)	60.00	60.00
Softening point (Ring and ball) _Tab - (°C)	50.00	50.00
P25r - (0,1 mm)	39.00	39.00
Tabr - (oC)	57.21	57.21
IPen	-0.12	-0.12
Average traffic speed for heavy vehicle_V (km/h)	50.00	50.00
t - (s)	0.02	0.02
Bitumen Stiffness_Sb - (MPa)	11.72	11.72
Dynamic modulus calculation_SHELL (5 to 1000 Mpa)		
Vb %	11.57	9.28
Va %	82.11	85.00
Sm108	9.91	10.03
Sm3109	10.56	10.61
S68	0.66	0.61
S89	0.50	0.44
A	9.29	9.46
Em - (Pa)	1.9487E+09	2.8770E+09
Em - (MPa)	1948.68	2876.96

Table A - 59: Structure design after dynamic modulus calculation for PETE temperature_EN254

Layer Type	Thickness H(m)	Modulus E/M_r (MPa)	Poisson's Ratio
Surface in Betão Betuminoso	0.05	1949	0.35
Binder in Macadam Bituminous	0.10	2877	0.35
Base in ABGE	0.20	290	0.35
Sub Base in Crushed Gravel	0.20	120	0.35
Subgrade 1 in soil	0.20	80	0.40
Subgrade 2 in soil	Semi infinite	80	0.40

Project: BISAR results_EN254_based on new dynamic modulus, **Calculated:** 5/16/2018 21:36

Table A - 60: Structure Configurations for EN254 Road

Layer Number	Thickness (m)	Modulus Elasticity (MPa)	Poisson's Ratio
1	0.05	1.95E+03	0.35
2	0.10	2.88E+03	0.35
3	0.20	2.90E+02	0.35
4	0.20	1.20E+02	0.35
5	0.20	8.00E+01	0.40
6	Semi infinite	8.00E+01	0.40

Table A - 61: Load Configurations for EN254 Road

Load Number	Vertical Load (kN)	Vertical Stress (MPa)	Horz. (Shear) Load (kN)	Horz. (Shear) Stress (MPa)	Radius (m)	X-Coord. (m)	Y-Coord. (m)	Shear Angle (Degrees)
1	2.00E+01	5.77E-01	0.00E+00	0.00E+00	1.05E-01	0.00E+00	-1.58E-01	0.00E+00
2	2.00E+01	5.77E-01	0.00E+00	0.00E+00	1.05E-01	0.00E+00	1.58E-01	0.00E+00

Table A - 62: BISAR results for EN254 road

Position No.	Layer No.	Coordinate (m)			Stress (MPa)			Strain (µstrain)			Displacement (µm)		
		X	Y	Z	XX	YY	ZZ	XX	YY	ZZ	UX	UY	UZ
1	2	0.00	0.00	0.15	0.55	0.25	-0.10	174.00	29.90	-132.00	0.00	0.00	396.00
2	2	0.00	0.16	0.15	0.65	0.52	-0.12	177.00	117.00	-184.00	0.00	11.80	385.00
3	5	0.00	0.00	0.55	0.00	0.00	-0.02	133.00	116.00	-307.00	0.00	0.00	277.00
4	5	0.00	0.16	0.55	0.00	0.00	-0.02	127.00	102.00	-285.00	0.00	17.50	270.00

Damage Calculation Results for BISAR_EN254

Table A - 63: BISAR Damage Calculation for Tensile Strain at the bottom of Asphalt Layer (MB Layer) _EN254

Tensile Strain at the bottom of Asphalt Layer (MB Layer)	1.77E-04
Effective Binder Content (by volume) at time of construction Vb (%)	9.28
Asphalt concrete stiffness modulus E (Pa)	2.88E+09
Admissible number of ESALs	3.27E+06
Damage (4.90E+06/3.27E+06) %	150

Table A - 64: BISAR Damage Calculation for Compression strain on top of the subgrade_EN254

Compression strain on top of the subgrade	-3.07000E-04
Ks for 95% of survival probability	1.80000E-02
Admissible number of ESALs	1.18E+07
Damage (4.90E+06/1.18E+07) %	41

Table A - 65: Results obtained by IP project_EN254

Criterion		**Strain**	**NAEP80**	**Damage**
Tensile Strain	Project Value	163	4.90E+06	71%
	ADM_BISAR	152	6.94E+06	
Compression Strain	Project Value	383	4.90E+06	14%
	ADM_BISAR	233	3.56E+07	

Table A - 66: Calculation of new AADTT for ME Design project _EN254

Road Section Name	Truck lane distribution factor (LDF)	Directional distribution factor (DDF)	Class type	Average Annual Growth Rate	Design Period in years	Growth factor	Aggression Factor (obtained from another sheet_US unit)	Accumulated Number of 80 kN Standard Axle Passages	New Average Annual Daily Truck Traffic $= \frac{N}{365 \times \alpha \times C \times P \times D \times L}$	Obtained AADTT from IP project
	L	**D**	**T**	**t**	**P**	**C**	**Alpha**	**N (dim,80)**	**AADTTnew**	**AADTTIP**
EN254 road (1) _Evora-S Miguel de Machedel_t.o.2004	1	0.5	T6	0.011	20	1.11	7.82	4.90E+06	154	185
EN254 road (2) _S Miguel de Machede-Redondo Sul_t.o.2004	1	0.5	T5	0.011	20	1.11	7.82	4.90E+06	154	203

Damage comparison between ME Design and SHELL method for three roads of IC3, IP6 and EN254

Table A - 67: Damage Comparison_SHELL_ME_IC3 road

Distress Type	Distress @ Specified Reliability		N	$D_{ME\ Design}$ %	D_{SHELL} %
	Target	Predicted	N (ME Design) =NAEP 80	Predicted/target	NAEP80/Nadm
Terminal IRI (m/km)	4	2.95	3.71E+07	74	-
Permanent deformation - total pavement (mm)	20	17.04	3.71E+07	85	5
AC bottom-up fatigue cracking (percent)	50	1.49	3.71E+07	3	18
AC thermal cracking (m/km)	189.4	5.15	3.71E+07	3	-
AC top-down fatigue cracking (m/km)	378.8	51.79	3.71E+07	14	-
Permanent deformation - AC only (mm)	10	8.29	3.71E+07	83	-
Chemically stabilized layer - fatigue fracture (percent	25	0.38	3.71E+07	2	2

Table A - 68: Damage Comparison_SHELL_ME_IP6 road

Distress Type	Distress @ Specified Reliability		N	$D_{ME\ Design}$ %	D_{SHELL} %
	Target	Predicted	N (ME Design) =NAEP 80	Predicted/target	NAEP80/Nadm
Terminal IRI (m/km)	4	3.24	4.95E+07	81	-
Permanent deformation - total pavement (mm)	20	26.74	4.95E+07	134	162
AC bottom-up fatigue cracking (percent)	50	1.79	4.95E+07	4	110
AC thermal cracking (m/km)	189.4	5.15	4.95E+07	3	-
AC top-down fatigue cracking (m/km)	378.8	84.01	4.95E+07	22	-
Permanent deformation - AC only (mm)	10	7.99	4.95E+07	80	-

Table A - 69: Damage Comparison_SHELL_ME_EN254 road

Distress Type	Distress @ Specified Reliability		N	$D_{ME\ Design}$ %	D_{SHELL} %
	Target	Predicted	N (ME Design) =NAEP 80	Predicted/target	NAEP80/Nadm
Terminal IRI (m/km)	4	2.81	4.90E+06	70	-
Permanent deformation - total pavement (mm)	20	15.15	4.90E+06	76	41
AC bottom-up fatigue cracking (percent)	50	1.62	4.90E+06	3	150
AC thermal cracking (m/km)	189.4	5.15	4.90E+06	3	-
AC top-down fatigue cracking (m/km)	378.8	239.94	4.90E+06	63	-
Permanent deformation - AC only (mm)	10	4.63	4.90E+06	46	-