# COMPUTATIONAL MODELING OF PERPETUAL PAVEMENTS USING THE MEPDG VERSION 0.910 SOFTWARE

***Lubinda F. Walubita, PhD***.
*Researcher – Texas Transportation Institute (TTI), Texas A&M University System, College Station, USA. Email: lfwalubita@tamu.edu*

## ABSTRACT

Because of their long structural life, many US States are exploring the use of perpetual pavements on some of their major highways. By definition, a perpetual pavement (PP) is a thick rut-resistant and fatigue (bottom-up)-resistant pavement structure designed to have a structural life in excess of 50 years, often designed for heavily-trafficked highways. During their service lives, PP structures generally require no major structural maintenance and/or rehabilitation activities, but are subject to periodic surface treatments and/or renewals. The objective of this paper is to present as a case study, the computational modeling and performance predictions of Texas PP structures using the MEPDG Version 0.910 software. The MEPDG is a mechanistic-empirical numerical software for pavement structural design analysis and performance prediction, within a given service period.

For this study, the MEPDG input data included dynamic shear rheometer (DSR) testing and dynamic modulus (DM) testing of binders and asphalt-mixture specimens, respectively. Environmental characterization was based on real-time climatic data from a local Texas weather station. Although, the pavement structures evaluated met the rutting and fatigue (bottom-up cracking) performance requirements, the MEPDG analysis indicated that proper account should also be taken of other potential distresses such as surface roughness (IRI), longitudinal (surface-down) cracking, and transverse (thermal) cracking; through among other measures, appropriate materials selection and mix-designs. The results also indicated that the surface layers should equally be of sufficient stiffness to minimize surface rutting. Overall, IRI reliability analyses indicated that at least one surface treatment or an overlay would be required within the first 23 years of service to restore the functional characteristics of the pavement among other functions. With an analysis period of over 50 years and the capability to accommodate multiple layers, the MEPDG Version 0.910 software offers promising potential for modeling perpetual pavements and should be explored further.

***Keywords:*** *perpetual pavements, rutting, fatigue, surface roughness (IRI), MEPDG*

# 1. INTRODUCTION

In an effort to construct long-lasting pavements with little future structural maintenance and/or rehabilitation, some US states are exploring the use of perpetual pavements on their heavily trafficked highways. In fact, some States such as California, Illinois, Kansas, Kentucky, Michigan, New Jersey, Ohio, Oregon, Pennsylvania, Texas, Virginia, Washington, and Wisconsin already have in-service or trial pavement structures constructed using the perpetual pavement concept (APA 2002). Literature also indicates that other countries like the United Kingdom, Germany, Canada, and China are utilizing or undertaking to apply the perpetual pavement concept on some of their heavily-trafficked highways (Nun et al. 1997).

A perpetual pavement (PP) is a long-lasting thick asphalt pavement structure with a service life in excess of 50 years without major structural rehabilitation and/or reconstruction activities; often designed for heavily-trafficked highways. However, they are subject to periodic surface maintenance and/or renewal in response to surface distresses in the upper layers of the pavement (APA 2002 and Timm et al. 2006). Deep seated structural distresses such as fatigue cracking and/or rutting are considered non-existent or if present, are very minimal. With these pavement structures, distresses and rehabilitation activities are confined to the easily accessible and replaceable surface portions of the pavement. So, when surface distresses reach critical levels, an economical solution is often to replace or simply overlay the top layers. These rehabilitation considerations are especially significant on heavily-trafficked highways where user-delay costs and traffic closures maybe prohibitive. Overall, the major benefits derived from PPs include (APA 2002):

- High structural capacity for high traffic volume and heavy loads.
- Long life with minimal or no major structural rehabilitation and/or reconstruction exercises.
- Low user-delay, reconstruction, rehabilitation, and life-cycle costs.

Because of the thicker and/or many asphalt layers, the initial construction costs for PPs are often higher than that of conventional asphalt pavements; at least over 20%. However, the above benefits will generally outweigh this effect, particularly in the long-term; thus providing a sustainable solution to the ever growing traffic for the highway agencies.

## 1.1 The Perpetual Pavement (PP) Concept

The perpetual pavement concept was derived on a mechanistic principle that thickly designed asphalt pavements with the appropriate material combinations, if properly constructed, will structurally outlive their design lives while simultaneously sustaining high traffic volumes/loads. The design philosophy is such that the pavement structure must; (1) have enough structural strength to resist structural distresses such as fatigue cracking, permanent deformation, and/or rutting; and (2) be durable enough to resist damage due to traffic forces (abrasion) and environmental effects (e.g., moisture damage).

The PP mechanistic design principle thus consists of providing enough stiffness in the upper pavement layers to prevent rutting and enough total pavement thickness and flexibility in the lowest asphalt layer to avoid bottom-up fatigue cracking. Like any other pavement structure, a solid-stable foundation is critical in supporting the pavement structure, traffic loads, and reducing seasonal-support variations due to environmental effects (e.g., freeze-thaw and moisture changes). The current mechanistic design procedure for PPs is based on two response limiting criteria:

- Horizontal tensile microstrain at the bottom of the lowest asphalt layer ($\varepsilon_t$):
  $\leq 070\ \mu\varepsilon$
- Vertical compressive microstrain on the top of subgrade layer ($\varepsilon_v$):
  $\leq 200\ \mu\varepsilon$

A perpetual pavement structure meeting these strain response criteria is considered to be structurally adequate both in terms of fatigue cracking and rutting. Otherwise, the layer thicknesses and material properties need to be modified. A mechanistic-empirical (M-E) based numerical software, PerRoad developed by Timm et al. (2004, 2006), is often used for structural analysis and layer thickness design.

In general, a PP structure consists of but not limited to impermeable, durable, and wear resistant top layers; a stiff thick rut-resistant intermediate layer for structural strength; and a flexible fatigue-resistant bottom layer resting on a stable and high-strength foundation. The layer thicknesses are generally variable depending on the traffic loading, environmental location, and materials/mix-designs. However, the rut-resistant is often the thickest layer so as to provide sufficient load carrying capability. Figure 1 shows a typical section of a perpetual pavement structure together with some recommended layer thicknesses (APA 2002).

| | | Thickness (mm) |
|---|---|---|
| Layer 1 | High quality asphalt layer (durability & wear resistant) | 37.5 - 75 (1.5 – 3.0″) |
| Layer 2 | Stiff rut-resistant layer | 100 – 175 (4.0 – 7.0″) |
| Layer 3 | Flexible fatigue-resistant layer | 75 – 100 (3.0 – 4.0″) |
| Layer 4 | Strong pavement foundation | (variable to infinite) |

**Figure 1 Typical Perpetual Pavement Structure (APA 2002)**

While the terminology (e.g., thick-asphalt pavements, long-lasting asphalt pavements, long-life asphalt pavements, deep-strength asphalt pavements, extended life HMA pavements, and full-depth asphalt pavements) may differ from place to place, the basic concept is the same as described above (APA 2002). Texas uses the term full-depth asphalt pavement (FDAP) for PPs. Consequently, the term FDAP shall be used synonymously with the term PP in this paper to refer to perpetual pavements. Also the symbol ″ is used in this paper to represent “inches”, together with “mm”, as a dimensional unit, e.g., 1″ = 1 inch ≅ 25 mm.



[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

### 1.2 Texas Typical Full-Depth Asphalt Pavement (FDAP) Structure

Currently, the Texas Department of Transportation (TxDOT) recommends using PP structures for traffic levels in excess of 30 million equivalent single axles loads (ESALs); with a total asphalt layer thickness of at least 350 mm (≥4 inch), typically resting on a lime stabilized subgrade-base of at least 150 mm (≥6 inch) thick and supported on a well compacted natural subgrade material (TxDOT 2001, Scullion 2006, and Walubita et al. 2007). The over 350 mm total asphalt layer thickness typically consists of 4 asphalt layers; (1) 50-75 mm thick stone mastic asphalt (SMA), (2) 50-75 mm thick transitional asphalt layer, (3) ≥200 mm thick rut-resistant layer (RRL), and (4) 50-100 mm thick fatigue-resistant layer (RBL). Additionally, there is also an option for a surfacing porous friction course (PFC) of 25 to 37.5 mm (1 – 1.5″) thick for traffic noise reduction and drainage improvements.

Clearly, the Texas FDAP concept is substantially more conservative than the typical PP concept shown in Figure 1; it encompasses more layers and has a greater total pavement thickness (i.e., 350 mm minimum thickness versus 213 mm minimum thickness). Theoretically, the Texas FDAP would be expected to have better structural capacity; however, ultimate field performance is a function of many variables including materials, mix-design, and construction practices. This Texas use of PP structures is considered necessary to cope with the ever increasing traffic and to minimize the cyclic and costly structural rehabilitation/reconstruction processes. To date, there are about eight Texas FDAP projects constructed since 2001 (Walubita et al. 2007).

The current structural (thickness) design and analysis of Texas FDAP is mechanistic-empirically (M-E) based, using the Flexible Pavement System Version 19 Windows-based (FPS 19W) software (Scullion et al. 2001). FPS 19W is a linear-elastic and M-E structural response analysis software. The PerRoad software is also often used for checking the Texas PP design.

## 2. OBJECTIVES AND RESEARCH METHODOLOGY

The objective of the work contained in this paper is to provide as a case study the computational modeling and numerical performance predictions of the FDAP structures on state highway SH 114 using the MEPDG Version 0.910 software; with a focus on the rut-and fatigue-resistant asphalt layers, respectively. The second objective was to investigate the applicability of the MEPDG software for modeling Texas PPs; in particular (1) the assemblage and processing of the input data (including traffic, material properties, and environment) and (2) the interpretation of the output data. The research methodology and scope of work included laboratory testing (both binders [dynamic shear rheometer] and asphalt mixtures [dynamic modulus]) to generate input data for the MEPDG analysis.

In the paper, the SH 114 FDAP structures are briefly discussed including the initial structural design, mix design, and construction details. MEPDG numerical analyses including discussions of the input data are then presented followed by a discussion and synthesis of the findings. The paper concludes with a summary of findings and recommendations.

## 3. THE SH 114 FDAP STRUCTURES

The SH 114 FDAP project is located in the Fort Worth District, Texas (USA), on state highway SH 114; with an average daily traffic (ADT) of approximately 18000, 27.3% trucks, and a traffic growth rate of 4.5%. The SH 114 FDAP structures consist of two sections; 1) the Superpave section (about 2.7 km) designated as FW 01 is designed with Superpave mixes and 2) the Conventional section (about 0.4 km) designated as FW 02 is designed with Conventional TxDOT mixes (Walubita et al. 2007). The in situ FDAP structures are shown in Figure 2.

**(1) FW 01: Superpave Section**

| Layer | Material | Binder + Aggregate | Thickness (mm) |
|---|---|---|---|
| Layer 1: Impermeable load carrying surface layer | ½″ HDSMA | 6.8% PG 20-28 + Igneous/Granite | 50 (2″) |
| Layer 2: Transitional layer | ¾″ SFHMAC | 4.2% PG 76-22 + Limestone | 75 (3″) |
| Layer 3: Stiff rut-resistant Layer (RRL) | 1″ SFHMAC | 4.0% PG 70-22 + Limestone | 325 (13″) |
| Layer 4: Flexible fatigue-resistant layer (RBL) | ¾″ SFHMAC | 4.2% PG 64-22 + Limestone | 100 (4″) |
| Layer 5: Base | Stabilized subgrade | 6.0% lime treated | 200 (8″) |
| Subgrade | | | ∞ |

**(2) FW 02: Conventional Section**

| Layer | Material | Binder + Aggregate | Thickness (mm) |
|---|---|---|---|
| Layer 1: Impermeable load carrying surface layer | ½″ HD SMA | 6.8% PG 20-28 + Igneous/Granite | 50 (2″) |
| Layer 2: Transitional layer | TxDOT Type C | 4.4% PG 70-22 + Limestone | 75 (3″) |
| Layer 3: Stiff rut-resistant Layer (RRL) | TxDOT Type B | 4.5% PG 64-22 + Limestone | 325 (13″) |
| Layer 4: Flexible fatigue-resistant layer (RBL) | TxDOT Type C | 5.3% PG 64-22 + Limestone | 100 (4″) |
| Layer 5: Base | Stabilized subgrade | 6.0% lime treated | 200 (8″) |
| Subgrade | | | ∞ |

**Figure 2 The SH 114 FDAP Structures**

In Figure 2, HDSMA stands for heavy-duty stone mastic (matrix) asphalt and SFHMAC stands for stone-filled hot-mix asphalt concrete. The preceding number in the materials column, e.g., ½″ in front of HDSMA refers to the nominal maximum aggregate size (NMAS) such as ½″ (or 12.5 mm) NMAS. PG refers to performance-graded binder (AI 1996a). TxDOT Type B and C are conventional TxDOT coarse to dense graded 22 mm (⅞″ ) and 16 mm (⅝″) NMAS mixes, respectively (TxDOT 2004). Note that NMAS is defined as one sieve size larger than the first sieve to retain more than 10% of the material (aggregate).



[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

The FPS 19W initial structural thickness design including material characteristics are shown in Figure 2 with a total pavement thickness of about 750 mm (30″); 550 mm (22″) consisting of four asphalt layers and 200 mm of the 6.0% lime treated subgrade as the base. For conservative purposes, the following material properties were used in the initial FPS 19W structural design: subgrade (E=63 MPa, ν=0.45), base (Layer 5, E=117 MPa, ν=0.45), RBL (E=117 MPa, ν=0.35), RRL (E=5175 MPa, ν=0.35), and Layers 1 and 2 (E=3450 MPa, ν=0.35); where E and ν are the elastic modulus and Poisson's ratio, respectively. The greater thickness (325 mm) of Layer 3 (1″ SFHMAC and TxDOT Type B) contributes to this layer's rut resistance as well as preventing bottom-up crack propagation. The structures were checked with the PerRoad software and met the PP mechanistic response criteria; $\varepsilon_t$ (Layer 4 [RBL] ≅ 60 με) < 70 με and $\varepsilon_v$ (subgrade ≅ 149 με) < 200 με (Timm 2004 and Walubita et al. 2007).

The current Texas FDAP mixture designs are based on the Superpave volumetric design system at 100 gyrations to achieve 4.0% air voids (AV) (i.e., 96% density), except for the RBL designed at 97% density (Scullion 2006 and Walubita et al. 2007). The binder contents (by total mix weight) are shown in Figure 2 and indicate the highest and lowest binder contents for the ½″ HDSMA and 1″ SFHMAC layers, respectively. TxDOT typically uses at least 6.0% PG 76-22 binder for SMA mixes with the exception of this particular case where the Contractor switched to PG 70-28 on site (TxDOT 2004). In general, the RBL should be designed with considerable binder content to contribute to its fatigue resistance characteristics and hence the term Rich Bottom Layer; the opposite is also true for the RRL.

The RRLs (Layer 3), in particular the 1″ SFHMACs, are typically designed with a dense to coarse aggregate gradation (25 mm NMAS) and are required to pass below the Superpave "restricted zone" (Figure 3) (AI 1996b). With a good aggregate interlock and stone-on-stone contact, this coarse aggregate gradation contributes to this layer's rut resistance characteristics.

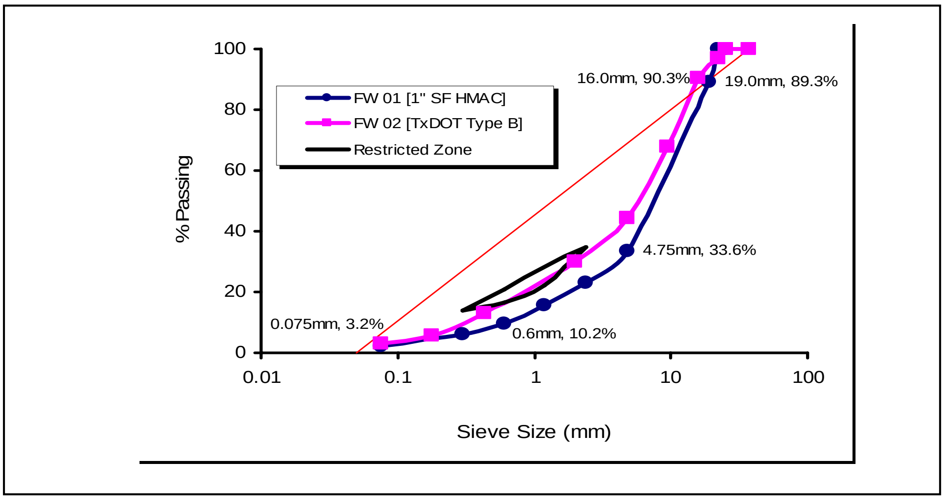



**Figure 3 Aggregate Gradations**

Note also from Figure 3 that the 1″ SFHMAC is relatively coarser than the TxDOT Type B mix based on the percentage retained aggregates, which is typically determined as follows:

$$\%Retained = 100\text{-}\%Passing \quad \text{(Eq. 1)}$$

Based on this coarse gradation, traditional wisdom would suggest that the 1″ SFHMAC would be expected to be more rut-resistant than the TxDOT Type B mix; which in practice is however not always the case, as aggregate interlock also plays an equally significant role.

In terms of construction and placement, the critical thicker RRLs (1″ SFHMAC [25 mm NMAS] and TxDOT Type B [22 mm NMAS]) were compacted in lift thicknesses of 100 mm (100 mm + 112.5 mm + 112.5 mm = 325 mm) and 125 mm ( 125 mm + 125 mm +75 mm = 325 mm), respectively (Walubita et al. 2007). The compaction rolling sequence were as follows; 1″ SFHMAC - 2 vibratory passes for the breakdown roller, 3 pneumatic passes for the second (intermediate) roller, and 1 vibratory pass plus 1 static pass for the finishing roller and; TxDOT Type B - 5 vibratory and 2 static passes for the breakdown roller, 8 pneumatic passes for the second (intermediate) roller, and 1 vibratory pass plus 1 static pass for the finishing roller. The average compaction mat temperature was 152 °C and the target compaction density was 96% for all other asphalt layers except the RBL at 97%; following typical compaction and rolling practices. The high RBL density also contributes to its fatigue-resistance and impermeability characteristics.

## 4. THE MEPDG VERSION 0.910 SOFTWARE

Computational modeling and numerical performance predictions in terms of cracking, rutting, and surface roughness (international roughness index [IRI]) were accomplished with the MEPDG software Version 0.910. Note that this new MEPDG software has the capability to handle multiple layers over a 50-year analysis period, which is advantageous for analyzing PPs. Details of the MEPDG software can be found elsewhere (AASHTO 2006).

The MEPDG is an M-E based numerical software for pavement structural design analysis and performance prediction, within a given service period (AASHTO 2006). The MEPDG adopts two major aspects of M-E based material characterization; pavement response properties and major distress/transfer functions. Pavement response properties are required to predict states of stress, strain, and deformation within the pavement structure when subjected to external wheel loads and thermal stresses. These properties for assumed elastic material behavior are the elastic modulus (E) and Poisson's ratio ($\nu$). The major MEPDG distress/transfer functions for asphalt pavements are load-related fatigue fracture, permanent deformation, rutting, and thermal cracking.

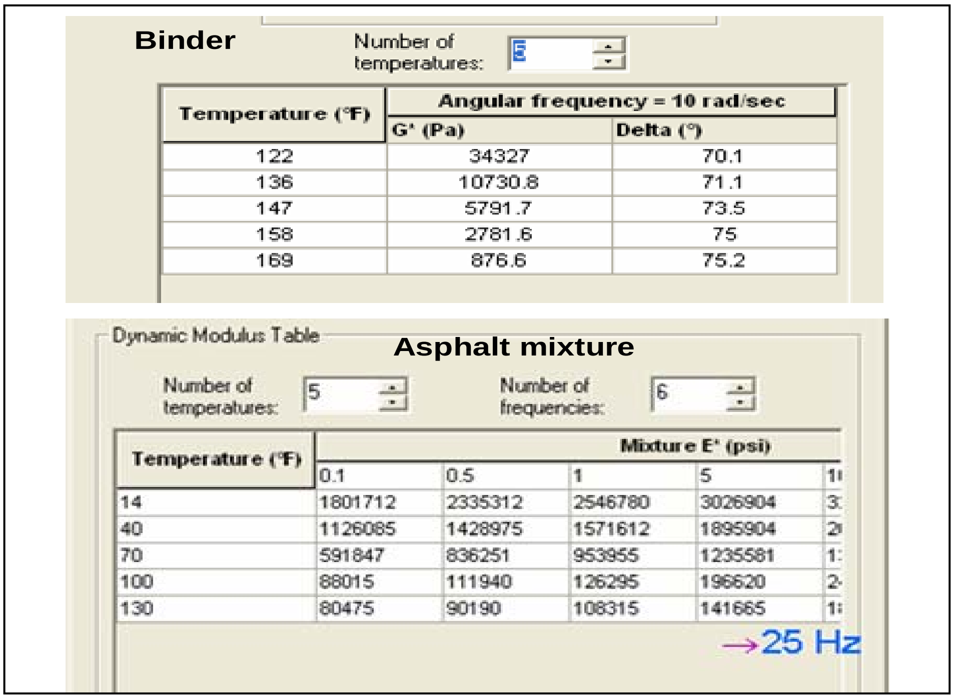


[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

## 4.1 MEPDG Input Data

In terms of the input data, the MEPDG utilizes a hierarchical system for both material characterization and analysis (AASHTO 2006). This system has three material property input levels. Level 1 represents a design philosophy of the highest achievable reliability, and Levels 2 and 3 have successively lower reliability, respectively. In addition to the typical volumetrics, Level 1 input requires laboratory measured binder and asphalt mixture properties such as the shear and dynamic modulus, respectively; whereas Level 3 input requires only the PG binder grade and aggregate gradation characteristics. Level 2 utilizes measured binder shear modulus properties and aggregate gradation characteristics.

In this study, Level 1 was used and the binder complex shear modulus was determined from dynamic shear rheometer (DSR) testing of rolling thin film-oven (RTFO) short-term aged binder samples; measured at 10 rad/s, and includes the phase angle and various representative test temperatures as the MEPDG input data. These binder data are used in the MEPDG software to predict asphalt mixture aging during analysis. For the asphalt mixtures, the actual dynamic modulus (DM) input data for MEPDG Level 1 analysis are the test temperatures, the test loading frequencies, and the respective measured modulus (|E*|) values; determined from the DM test. Figure 4 is an example of input screens for the Level 1 material properties that were generated from DSR and DM testing of binders and asphalt mixtures, respectively.




| Temperature (°F) | Angular frequency = 10 rad/sec | |
|---|---|---|
| | G* (Pa) | Delta (°) |
| 122 | 34327 | 70.1 |
| 136 | 10730.8 | 71.1 |
| 147 | 5791.7 | 73.5 |
| 158 | 2781.6 | 75 |
| 169 | 876.6 | 75.2 |

| Temperature (°F) | Mixture E* (psi) | | | |
|---|---|---|---|---|
| | 0.1 | 0.5 | 1 | 5 |
| 14 | 1801712 | 2335312 | 2546780 | 3026904 |
| 40 | 1126085 | 1428975 | 1571612 | 1895904 |
| 70 | 591847 | 836251 | 953955 | 1235581 |
| 100 | 88015 | 111940 | 126295 | 196620 |
| 130 | 80475 | 90190 | 108315 | 141665 |

**Figure 4 Example of MEPDG Level 1 Material Properties**

Details of the DSR and DM tests together with the test results for the SH 114 materials are published elsewhere (Walubita et al. 2007). These tests were conducted for each layer/material shown in Figure 2 and were input individually (i.e., per layer) in the MEPDG software (Walubita et al. 2007). Modulus values of 117 MPa and 63 MPa were used for the base and subgrade, respectively. For traffic, an average annual daily traffic of 18000 with a traffic growth rate of 4.5% (compound growth) was utilized. The truck composition was taken as 27.3% in the design direction and 100% in the design lane at 95% reliability level. Environmental characterization was based on real-time climatic data from the Alliance airport in Fort Worth, Texas (US). Typical distress failure criteria consistent with TxDOT tolerable limits were used, for an analysis period of up to 50 years (TxDOT 2003 and Walubita et al. 2007).

Summarized, the basic MEPDG input data include the general project information, traffic, climate (environment), pavement structure (structural design and material properties), distress failure limits, pavement design life, and a design reliability level (AASHTO 2006).

### 4.2 MEPDG Analysis and Output Data

During execution, the MEPDG software predicts performance at any age of the pavement for a given pavement structure and traffic level under a particular environmental location (AASHTO 2006). The MEPDG predicted performance is then matched against predefined performance criteria at a given reliability level and design life. If the predefined performance criteria or analysis parameters are not met, the following options are feasible to improve the results:

- Reviewing/modifying the input data including the pavement structure (thicknesses), material properties, traffic, environment, reliability level, pavement design life, and analysis parameters (distress failure limits); or
- Changing the asphalt mix-design and/or the material types.

## 5. MEPDG LEVEL 1 RESULTS

The MEPDG Level 1 results are summarized in Table 1 and include both the distress and reliability predictions (English units[a]). In Table 1, the percentage reliability predictions (in parentheses) represent the probability percentage of the pavement not performing to expectations, i.e., the percentage chance of the distress exceeding the target threshold. For instance, there is 0% probability that both pavement sections will have major bottom-up fatigue related problems during their service life. Or in other words, there is 0% probability that bottom-up fatigue cracking will exceed the 25% threshold at 95% design reliability. The 95% design reliability implies that 5% (also in parentheses) chance of failure or exceeding the target (distress) threshold is allowable

With the exception of IRI, no major distresses were predicted on both pavement structures. In fact, Table 1 shows no evidence of bottom-up fatigue cracking (0% probability); thus meeting the PP status. Also, the total pavement rutting is acceptably within the 18.75 mm (0.75″) design limit on both sections; thus no major rutting problems could be theoretically expected from these pavement structures.



[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

**Table 1 MEPDG Level 1 Distress Analysis**

| Performance Criteria | | DT | RT | Distress Predicted |
|---|---|---|---|---|
| **FW 01: Superpave Section** | | | | |
| 1 | Terminal IRI (in/mi) | 172 | 95(5%) | **210.8 (80%)** |
| 2 | AC Surface Down Cracking (Longitudinal Cracking) (ft/500) | 1000 | 95 (5%) | 3.4 (4%) |
| 3 | AC Bottom-Up Cracking (Alligator Cracking) (%) | 25 | 95 (5%) | 0 (0%) |
| 4 | AC Thermal Fracture (Transverse Cracking) (ft/mi): | 1000 | 95 (5%) | 1 (6%) |
| 5 | Permanent Deformation (AC Only),(in) | 0.50 | 95 (5%) | 0.42 (11%) |
| 6 | Permanent Deformation (Total Pavement) (in) | 0.75 | 95 (5%) | 0.59 (10%) |
| **FW 02: Conventional Section** | | | | |
| 1 | Terminal IRI (in/mi) | 172 | 95(5%) | **215.2 (82%)** |
| 2 | AC Surface-Down Cracking (Longitudinal Cracking) (ft/500) | 1000 | 95 (5%) | 7.7 (9%) |
| 3 | AC Bottom-Up Cracking (Alligator Cracking) (%) | 25 | 95 (5%) | 0 (0%) |
| 4 | AC Thermal Fracture (Transverse Cracking) (ft/mi) | 1000 | 95 (5%) | 1 (6%) |
| 5 | Permanent Deformation (AC Only),(in) | 0.50 | 95 (5%) | **0.51 (53%)** |
| 6 | Permanent Deformation (Total Pavement) (in) | 0.75 | 95 (5%) | 0.69 (13%) |
| DT = distress target; RT = reliability target (%); AC = asphalt concrete; IRI = international roughness index; Units: 1 ft = 1 feet ≅ 0.305 m; 1 mi = 1 mile ≅ 1609 m; 1 in = 1 inch ≅ 25 mm. | | | | |

Figure 5 is a detailed result of the MEPDG permanent deformation and rutting analyses of the pavement structures. Figure 5 shows that the asphalt layers on the Conventional section, predominantly in the top layers, appear to exhibit a potential for permanent deformation. Both Table 1 and Figure 5 show that the predicted permanent deformation (12.75 mm [0.51″]) slightly exceeds the 12.5 mm (0.5″) design limit with about a 53% chance of occurrence. This was attributed to the use of the softer PG binder grades which resulted in relatively lower mixture moduli values (less stiff) on the Conventional section. For example, the measured |E*| values at 10 Hz, 21 °C were 7328 MPa and 4464 MPa for the ¾″ SFHMAC (Layer 2, Superpave section) and Type C (Layer 2, Conventional section), respectively (Walubita et al. 2007).

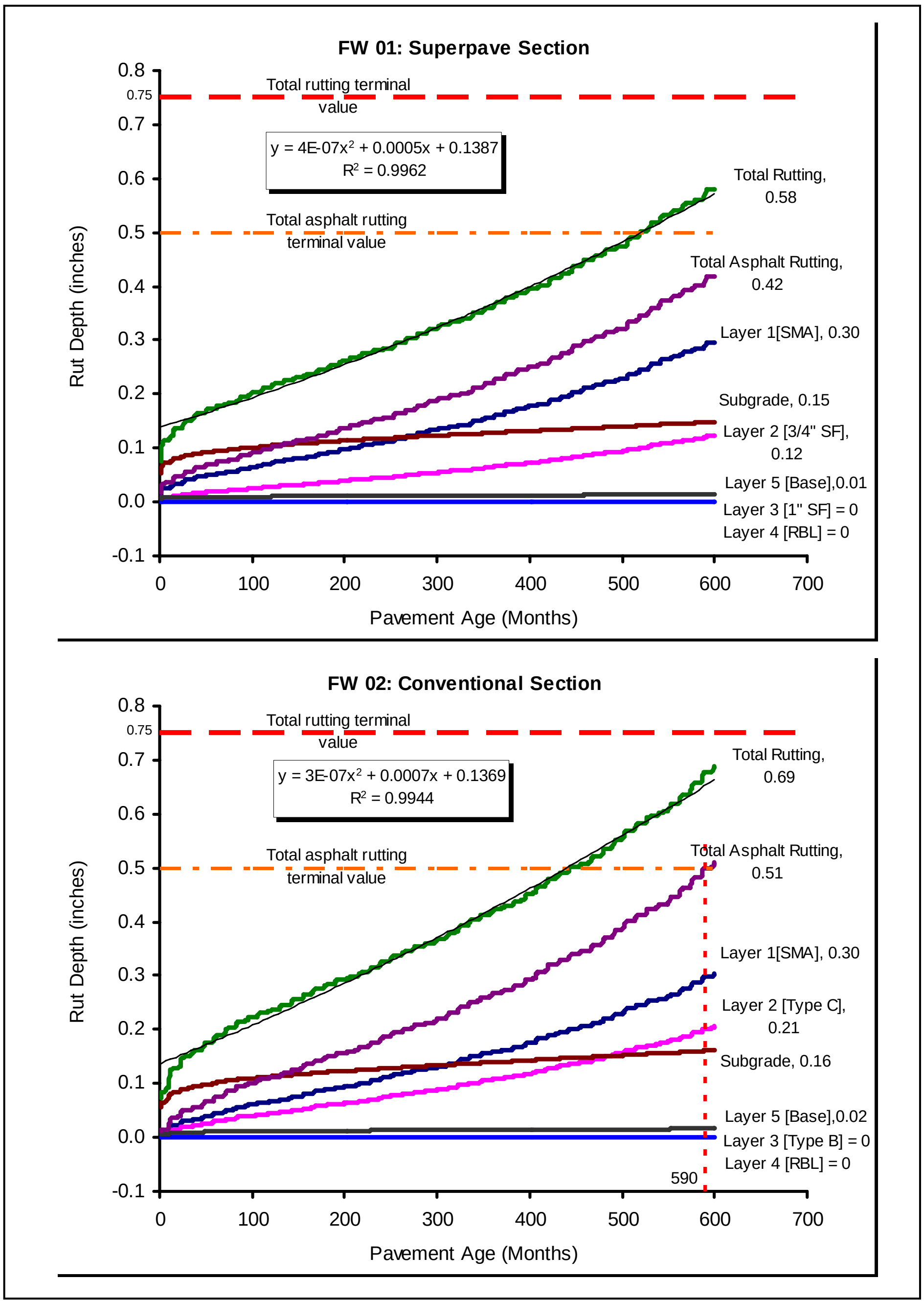



**Figure 5 MEPDG Permanent Deformation and Rutting Predictions**
(1 inch ≅ 25 mm; ¾″ SF = ¾″ SFHMAC [19 mm NMAS]; 1″ SF = 1″ SFHMAC [25 mm NMAS])



[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

From the results shown in Table 1 and Figure 5, the following are evident:

- As seen in Figure 5, both sections show no permanent deformation in Layers 3 (RRL) and 4 (RBL). The stiff RRLs are designed to be rut-resistant and so no major permanent deformation would be expected. At 10 Hz, 21 °C, the measured RRL |E*| values were 9412 MPa and 6155 MPa for 1″ SFHMAC (Superpave section) and TxDOT Type B (Conventional section), respectively. The non-existence of permanent deformation in the flexible RBLs (Layers 4, mean |E*| = 3957 MPa) is attributed to load shielding from the upper layers in the MEPDG analysis.
- Some permanent deformation is evident in Layers 1 and 2; with the Conventional section exhibiting comparatively more for Layer 2 and equivalently for Layer 1 (due apparently to similar material characteristic properties). Note that the same ½″ HDSMA was used on both sections for Layer 1. Total asphalt layer permanent deformation is considerable more on the Conventional section. This was attributed to the use of relatively softer binders that resulted in less stiff asphalt mixtures. At 10 Hz, 21 °C, the measured average |E*| values were 6250 MPa and 4611 MPa for the Superpave and Conventional sections, respectively (Walubita et al. 2007).
- Both sections show some rutting in the subgrade but very marginal in the base (lime treated subgrade). However, the Conventional section has more total rutting (about 1.2 times); indicating higher rut susceptibility than the Superpave section. Second order polynomial relationships (Eq.s 2 and 3) derived from Figure 5 approximated that the total pavement rutting will reach the 18.75 mm (0.75″) terminal value well in excess of the 30-year initial design life for both sections. In the field however, this may not necessarily be the case as indicated by the reliability predictions in Table 1. In fact, Table 1 shows at most 13% probability of excessive rutting above the design threshold.

$$Total\ Rut_{Depth(FW\,01)} = 4\times10^{-7}t^2 + 0.0005t + 0.1387 \qquad \text{(Eq. 2)}$$

$$Total\ Rut_{Depth(FW\,02)} = 3\times10^{-7}t^2 + 0.0007t + 0.1369 \qquad \text{(Eq. 3)}$$

Where $t$ is the pavement age in months.

- Based on 95% reliability predictions (i.e., 5% allowable, Table 5), both sections show possible occurrence of transverse (thermal fracture) cracking but with longitudinal (surface-down) cracking for the Conventional section only.
- Because of possibly similar climatic and environmental conditions, a similar level of reliability was predicted for thermal fracture (transverse cracking), indicating that there is about 6% chance of excessive transverse cracking above the threshold limit.
- Both sections failed the IRI distress criterion (Table 5) with about 80% probability of exceeding the design threshold limit. In fact, reliability predictions indicated that the IRI will reach critical levels approximately in the 23rd of service life; suggesting that at least one surface treatment or an overlay should be done before this time. Figure 6 shows these results graphically, with the Superpave and Conventional sections almost overlapping each other for both reliability and IRI predictions.

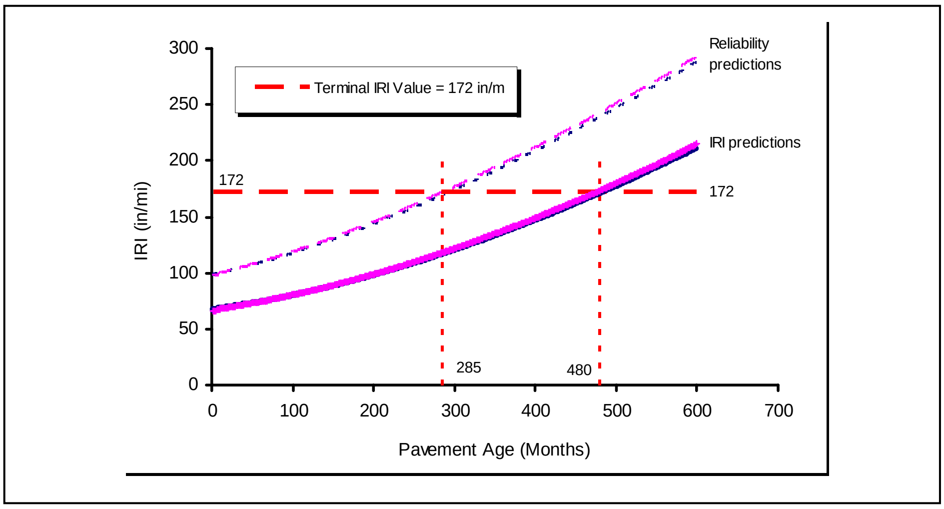



**Figure 6 MEPDG IRI Predictions (1 in ≅ 25 mm; 1 mi ≅ 1609 m)**

Note that although the total asphalt layer permanent deformation may be critical based on the 12.5 mm (0.5″) terminal value in particular for the Conventional section, the total pavement rutting on both sections (14.5 mm [0.58″] for the Superpave and 17.25 mm [0.69″] for the Conventional) is within the 18.75 mm (0.75″) terminal value. Therefore, one surface treatment or an overlay will theoretically be required at least before the 23rd year of service based on the IRI distress analysis. In the field however, and as indicated by the MEPDG reliability predictions, different results may be observed. Note also that the MEPDG predicted performance and subsequent analyses subjectively depends on the selected design threshold limits. These results nonetheless provide an analytical indication of the critical distresses and the expected performance.

## 6. DISCUSSION AND SYNTHESIS OF RESULTS

While both structures met the PP requirement with neither signs of major bottom-up fatigue cracking nor total pavement rutting, the MEPDG analyses indicated that the Superpave is relatively a better structure than the Conventional section, particularly with respect to permanent deformation. Based on laboratory testing, the Superpave mixtures were found to be much stiffer with a mean |E*| value 1.4 times higher than the Conventional mixtures. This difference in performance is attributed to the differences in material type, mix-designs, aggregate characteristics, and material properties (Figure 2).

Based on the IRI distress with about 80% probability of exceeding the design threshold limit, the MEPDG analyses indicated that a surface treatment maybe required by the 23rd year of service. In practice, this means that at least one surface treatment or an overlay would be required within the first 23 years of service to restore the pavement functional characteristics; which is not uncommon for most typical asphalt pavements. However, these MEPDG predictions will be verified in the ongoing field performance monitoring program of the SH 114 highway (Walubita et al. 2007).



[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.

## 7. SUMMARY AND RECOMMENDATIONS

The findings and recommendations drawn from this study are summarized as follows:

- The SH 114 pavement structures exhibit no potential for bottom-up fatigue cracking, but may be subject to surface rutting, surface roughness, longitudinal, and thermal cracking during their service life; with the Conventional section exhibiting more susceptibility. Data showed that material type and mix-design characteristics are critical to the performance of perpetual pavements and should be well accounted for in the design, but obviously without compromising constructability and structural integrity.
- The MEPDG software predicted no permanent deformation in the intermediate rut-resistant layers, but indicated potential for permanent deformation in the top asphalt layers; emphasizing the fact that the top layers must be equally designed with considerable stiffness to prevent surface rutting.
- Based on the surface roughness analysis, the MEPDG indicated that at least one surface treatment or an overlay would be required within the first 23 years of service to restore the functional characteristics of the pavements.
- While perpetual pavements are designed to be rut-and fatigue-resistant, account should be taken of other potential distresses such as surface roughness, longitudinal (surface down) cracking, and transverse (thermal) cracking through among other measures appropriate materials selection and mix-designs.
- With an analysis period of over 50 years and the capability to accommodate multiple layers, the MEPDG Version 0.910 software offers promising potential for modeling and analyzing perpetual pavements, whose intermediate/lower layers typically have expected service lives in excess of 50 years.

Currently, monitoring and performance evaluation of the Texas perpetual pavements including laboratory testing and non-destructive field testing is ongoing. The results will form a basis for supplementing and validating this study’s findings.

## 8. ACKNOWLEDGEMENTS

The author thanks TxDOT and the Federal Highway Administration (FHWA) for their support in funding this research study and all those who helped during the course of this research work. In particular, the constructive comments/suggestions provided by Joseph S. Mayunga in the preparation of this paper are gratefully acknowledged.

## 9. DISCLAIMER

The contents of this paper reflect the views of the author who is responsible for the facts and accuracy of the data presented herein and do not necessarily reflect the official views or policies of any agency or institute. This paper does not constitute a standard, specification, nor is it intended for design, construction, bidding, or permit purposes. Trade names were used solely for information and not for product endorsement.

## 10. REFERENCES


AI. (1996a) - Asphalt Institute*: Superpave Asphalt Binder Specification - Performance Graded Asphalt Binder Specification and Testing, Superpave Series No. 1 (SP-1)*, Lexington, Kentucky, US.

AI. (1996b) - Asphalt Institute*: Superpave Mix Design, Superpave Series No. 2 (SP-2)*, Lexington, Kentucky, US.

APA. (2002) - Asphalt Pavement Alliance: Perpetual Pavements – A Synthesis, APA 101. Maryland, US.

MEPDG Version 0.910. (2007) - *NCHRP 1-37A, Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,* http://www.trb.org/mepdg, Accessed March 2007.

Nunn, M.E., A. Brown, D. Wetson, and J. C. Nicholls. (1997) - Design of Long-Life Flexible Pavements for Heavy Traffic, Report No. 250, Transportation Research Laboratory (TRL), Berkshire, UK.

Timm, D. H. A (2004) - Guide to PerRoad 2.4 for PerRoad Perpetual Pavement Software, National Center for Asphalt Technology (NCAT), Auburn University, Alabama, US.

Timm, D. H., and D. E. Newcomb. (2006) - Perpetual Pavement Design for Flexible Pavements in US, *International Journal of Pavement Engineering (IJPE*), Vol. 7. No. 2, pp. 111-119 (June)

Scullion, T. (2006) - Perpetual Pavements in Texas: The State of the Practice, Technical Report FHWA/TX-05/0-4822-1, Texas Transportation Institute (TTI), College Station, Texas, US.

Scullion, T., and Liu, W. (2001) - "Flexible Pavement Design System FPS 19W: User's Manual," Technical Report 0-1869-1, TTI, College Station, Texas, US.

Scullion, T., and W. Liu. (2003) - "MODULUS 6.0 for Windows: User's Manual," Technical Report 0-1869-2, TTI, College Station, Texas, US.

TxDOT. (2001) - Memorandum on Full-depth Asphalt Pavements, Flexible Pavement Design Task Force Implementation, Texas Department of Transportation (TxDOT), Austin, Texas, US (April 23).

TxDOT. (2003) - "*Condition of Texas Pavements. PMIS Annual Report, FY 2001 – 2003*", Texas Department of Transportation (TxDOT), Materials & Pavement Section, Austin, Texas, US.

TxDOT. (2004) - *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*, Austin, Texas, US.

Zhou, F., S. Hu, and T. Scullion. (2006) - Integrated Asphalt (Overlay) Mix Design with Balancing Both Rutting and Cracking Requirements, Technical Report FHWA/TX-05/5123-1, TTI, College Station, Texas, US.

Zhou, F. and Scullion, T. (2005) - Overlay Tester: *A Rapid Performance Related Crack Resistance Test*, Technical Report FHWA/TX-05/0-4467-2, TTI, College Station, US, pp. 85.

Walubita, L. F. and Scullion, T. (2007) - Perpetual Pavements in Texas: The Fort Worth SH 114 Perpetual Pavement in Wise County, Draft Technical Report FHWA/TX-05/0-4822-2, TTI, College Station, Texas, US.




[a] In this study, the MEPDG software utilized English units and so majority of the results are in English units.