# Mechanistic-Empirical Modeling of Permanent Deformation in Asphalt Concrete Layers

Oscarsson, Erik

2011

# Mechanistic-Empirical Modeling of Permanent Deformation in Asphalt Concrete Layers

Erik Oscarsson

ISBN 978-91-7473-083-8

Printed in Sweden
Media-Tryck, Lund, 2011

Cover picture: Flow rutting effects shown as a permanent vertical strain distribution in the asphalt concrete layer cross section and the resulting surface profile.

Doctoral Thesis CODEN:LUTVDG/(TVTT-1040)1-208/2011

Bulletin – Lunds Universitet, Tekniska högskolan i Lund,
Institutionen för teknik och samhälle, 259

ISSN 1653-1930
ISBN 978-91-7473-083-8

Erik Oscarsson

# Mechanistic-Empirical Modeling of Permanent Deformation in Asphalt Concrete Layers

2011

*Keywords:*
Mechanistic-Empirical, modeling, permanent deformation, rutting, asphalt concrete, full-scale pavements

*Abstract:*
Three mechanistic-empirical permanent deformation models were evaluated under Swedish conditions with respect to traffic, climate and materials using accelerated pavement testing and long-term pavement performance studies. The mechanistic-empirical pavement design guide (M-E PDG), the incremental-recursive mechanistic-empirical CalME model (CalME), and the PErmanent Deformation of asphalt concrete layer for ROads (PEDRO) model generally showed both useful features and limitations. The M-E PDG results were more accurate at the lowest material input data quality level (level 3) than at the highest (level 1). The main cause was probably the demonstrated inaccuracy of the predicted dynamic modulus at level 3 compared with measured level 1 results, and the M-E PDG calibration at level 3. The CalME underestimated the permanent deformation in the semi-rigid section due to its response modeling sensitivity to overall pavement stiffness. Further, the results indicated that the relation between elastic and plastic material properties may change throughout the pavement life. The PEDRO model behavior due to lateral wander and observed field temperatures was reasonable. The zero shear rate viscosity assessment method for asphalt concrete, utilized in PEDRO, should be further evaluated. All models produced reasonable permanent deformation results although further validation and calibration is recommended before employment for pavement design purposes in Sweden.

*Citation:*
Oscarsson, Erik. Mechanistic-Empirical modeling of permanent deformation in asphalt concrete layers. Institutionen för Teknik och samhälle, Trafik och väg, 2011. Bulletin - Lunds Universitet, Lunds Tekniska Högskola, Institutionen för Teknik och samhälle, 259

*With support from:*

TRAFIKVERKET SBUF

Trafik och väg
Institutionen för Teknik och samhälle
Lunds Tekniska Högskola, LTH
Lunds universitet
Box 118, 221 00 Lund

Traffic and Roads
Department of Technology and Society
Faculty of Engineering, LTH
Lund University
Box 118, SE-221 00 Lund, Sweden

To Kristina and Vidar

# Table of contents

# List of papers

**Paper I** Oscarsson, E. (2011). Evaluation of the Mechanistic-Empirical Pavement Design Guide model for permanent deformations in asphalt concrete. International Journal of Pavement Engineering, Volume 12, Issue 1.

**Paper II** Oscarsson, E. (2011). Modelling flow rutting in in-service asphalt pavements using the Mechanistic-Empirical Pavement Design Guide. Road Materials and Pavements Design, Vol. 12, Issue 1.

**Paper III** Oscarsson, E., Popescu, L. (2011). Evaluation of the CalME permanent deformation model for asphalt concrete layers. International Journal of Pavement Research and Technology, Volume 4, Issue 1.

*The author of this thesis was responsible for the entire paper except refining the RSST-CH results and some discussion.*

**Paper IV** Said, S.F., Hakim, H., Oscarsson, E., Hjort, M. Prediction of flow rutting in asphalt concrete layers. Accepted for publication in International Journal of Pavement Engineering, January 2011.

*The contribution of the author of this thesis is participation in model development, limited diagram production, and some corrections and replies to reviewer comments.*

**Paper V** Oscarsson, E., Said, S.F. Modeling permanent deformations in asphalt concrete layers using the PEDRO model. Submitted to International Journal of Pavement Engineering, February 2011.

*The author of this thesis was responsible for the entire paper except the material testing and some model development.*

**Paper VI** Oscarsson, E., Said, S.F. Assessment of zero shear rate viscosity in asphalt concrete using frequency sweep testing. Submitted to Journal of Materials in Civil Engineering, February 2011.

*The author of this thesis was responsible for the entire paper except the material testing and some model development.*

# Summary

The continuous and increasing traffic causes deterioration in the road construction. One of the most serious distress mechanisms is permanent deformation in asphalt concrete layers. This distress appears as longitudinal surface depressions called ruts. They reduce traffic safety by causing vehicle steering difficulties, and also hydroplaning or low friction due to rain and snow. The considerable cost and resource consumption in construction and rehabilitation can be reduced by distress modeling.

In this thesis, three mechanistic-empirical permanent deformation models were evaluated under Swedish conditions with respect to traffic, climate and materials. The models are the Mechanistic-Empirical Pavement Design Guide v1.003 (M-E PDG), the incremental-recursive mechanistic-empirical model within CalME v0.82 (CalME), and the PErmanent Deformation of asphalt concrete layer for ROads (PEDRO) model. The evaluations were carried out by comparing modeled and measured permanent deformation, and by studying the model components. The models were evaluated using accelerated pavement testing achieved with a Heavy Vehicle Simulator causing deterioration in two full-scale pavement structures under controlled climate conditions. Further, the models were applied on two long-term pavement performance pavement sections on the E6 motorway at Fastarp-Heberg in Sweden. The traffic load spectrum was characterized by Weigh-In-Motion in addition to measurements of the number of vehicles. The material properties were determined using laboratory testing of dynamic modulus and to some extent inelastic testing. The temperatures in each layer were calculated using the Enhanced Integrated Climate Model.

The M-E PDG is probably the currently most widely used mechanistic-empirical methodology since it is being implemented in most US states. The model is based on response and distress modeling using vertical strains. The M-E PDG can be employed at three hierarchical levels of input data quality, where each level corresponds to the effort of obtaining the input data. The predicted dynamic modulus at level 3 was generally more than twice as large as those measured at level 1. The M-E PDG permanent deformation model employed at input data quality level 3 generally produced results in accordance with field observation. However, levels 1 resulted in overestimation of the permanent deformation. The suggested main cause for the difference between levels is the previously shown inaccuracy of the predictive dynamic modulus equation at level 3. Still, the model is calibrated using level 3 input data, which implies that this should be the level currently preferred. The predictive dynamic modulus equation is recommended to be developed in order to increase its accuracy. Further calibration at level 1 or 2 is recommended.

The CalME software v0.82, developed at University of California, contains an incremental-recursive mechanistic-empirical model called CalME hereafter. Like M-E PDG, the CalME model employs response and distress modeling although they are based on shear strains. An advantage of CalME is that the dynamic modulus adjustment is achieved not only by ageing and densification modeling but also a damage function. However, the results indicated that these considerations should be calibrated further for use in Sweden. The inelastic material properties, as tested using the repeated simple shear test at constant height, and CalME distress modeling results indicated that the relation between elastic and plastic properties may vary throughout pavement life. Therefore, the elastic-plastic relation, which is currently assumed to be constant, should be further studied. The CalME response modeling appeared sensitive to overall pavement stiffness, as shown by comparing a flexible and a semi-rigid pavement section. Relating shear stress and elastic shear strain to flow rutting in very stiff pavement sections may be difficult. CalME is currently calibrated using flexible pavements only. The calibrated model produced reasonable results using the default permanent deformation parameters.

The PEDRO model was included in the evaluation due to its newly developed material parameter assessment method. Unlike the other models, PEDRO is based on viscoelastic densification and flow. The main material property, zero shear rate viscosity (ZSV), was calculated using a new method based on the measured dynamic shear modulus and the corresponding phase angle. This model does not require destructive material testing because all permanent deformation is attributed to the linear viscoelastic Burger element. The ZSV assessment method presented ranked the asphalt concrete materials according to expectations based on dynamic modulus and phase angle. The assessed ZSV was shown to be highly sensitive to temperature. The PEDRO permanent deformation modeling indicated that this method may possibly underestimate the ZSV. The PEDRO model produced reasonable permanent deformation profiles using assessed traffic lateral wander. However, the modeled surface profile and the resulting rut depth were sensitive to the distribution of lateral wander. The PEDRO model was calibrated using both accelerated pavement testing and field observation. The ambiguous calibration results should be further validated. An advantage offered by PEDRO is that a reasonable permanent deformation depth distribution was achieved without using empirical considerations.

# Sammanfattning

Trafiken orsakar nedbrytning av våra vägar. En allvarlig nedbrytningsmekanism är permanenta deformationer i asfaltbeläggningen. Det visar sig som spårbildning, vilket är längsgående försänkningar i vägytan. Spårbildning medför en trafikfara genom att orsaka svårigheter med fordonens styrning. Dessutom orsakar det vattenplaning eller halka vid nederbörd. Den stora ekonomiska och miljömässiga kostnaden för att bygga och underhålla vägar kan begränsas genom modellering av nedbrytningen.

I denna avhandling utvärderades tre mekanistisk-empiriska modeller för permanenta deformationer under svenska förhållanden avseende trafik, klimat och material. Modellerna är "Mechanistic-Empirical Pavement Design Guide v1.003" (M-E PDG), den inkrementella och rekursiva mekanistisk-empiriska modellen inom CalME v0.82 (CalME) och "PErmanent Deformation of asphalt concrete layer for ROads" (PEDRO). Utvärderingen utfördes genom att jämföra modellerad och uppmätt permanent deformation, och genom att studera modellernas komponenter. För detta ändamål applicerades modellerna på två vägsektioner som utsattes för accelererad fullskaleprovning med en maskin kallad "Heavy Vehicle Simulator" under kontrollerade klimatförhållanden. Vidare användes modellerna på två verkliga testsektioner på motorväg E6 vid Fastarp-Heberg i Sverige. Trafiklasten karakteriserades genom mätning av verkliga axellaster med "Weigh-In-Motion" samt räkning av passerande fordon. Materialegenskaperna bestämdes genom laboratorieprovning av dynamiska moduler och delvis genom förstörande provning av oelastiska egenskaper. Temperaturerna i varje lager beräknades med hjälp av klimatmodellen "Enhanced Integrated Climate Model".

M-E PDG är troligtvis den mest använda mekanistisk-empiriska modellen eftersom den implementeras i de flesta stater i USA. Modellen baseras på beräkning av vertikal elastisk respons samt nedbrytning. M-E PDG kan användas på tre olika kvalitetsnivåer, där varje nivå motsvaras av mängden resurser som krävs för att bestämma indata. De beräknade dynamiska modulerna på nivå 3 var generellt mer än dubbelt så stora som de uppmätta på nivå 1. M-E PDG:s resultat på nivå 3 hade god överensstämmelse med uppmätta fältdata. Däremot överskattade modellen den permanenta deformationen på nivå 1. Den främsta orsaken till detta är troligen att de beräknade dynamiska modulerna på nivå 3 inte stämde överens med de uppmätta på nivå 1. Modellen är dock kalibrerad med indata på nivå 3, vilket innebär att denna nivå bör vara att föredra tills vidare. Rekommendationen är att utveckla beräkningsmodellen för dynamiska moduler så att den bättre överensstämmer med uppmätta data, samt att utföra ytterligare kalibrering på nivå 1 eller 2.

Vägdimensioneringsprogrammet CalME v0.82, utvecklat vid University of California, innehåller en inkrementell och rekursiv mekanistisk modell som här kallas CalME. Liksom M-E PDG innehåller CalME modellering av respons och nedbrytning med den skillnaden att det baseras på skjuvning istället för vertikalt tryck. En fördel med CalME är att förändringar av dynamisk modul beräknas som en funktion av åldring och förtätning men även nedbrytning. Den totala effekten visade dock att dessa förändringar bör kalibreras innan modellen används i Sverige. Provning av de oelastiska materialegenskaperna visade att relationen mellan elastiska och plastiska töjningar kan variera under vägens livslängd. Därför bör denna relation studeras vidare eftersom den nu antas vara konstant. Jämförelser mellan en flexibel och en halvstyv vägkonstruktion visade att CalME:s responsmodellering verkade vara känslig för vägkroppens totala styvhet. Därför kan det vara svårt att direkt relatera skjuvspänning och elastisk skjuvtöjning till permanent deformation i styva vägkroppar. CalME är för närvarande kalibrerad enbart för flexibla överbyggnader. Den kalibrerade modellen gav rimliga resultat när standardparametrar för permanent deformation användes.

Modellen PEDRO inkluderades i utvärderingen då en ny beräkningsmodell för materialegenskaperna nyligen utvecklades. Till skillnad från de två andra modellerna för permanent deformation baseras PEDRO på viskoelastisk förtätning och flytning. Dess huvudsakliga materialparameter, kallad zero shear rate viscosity (ZSV), beräknades genom en ny metod baserad på uppmätta dynamiska skjuvmoduler och fasvinklar. Modellen kräver ingen oelastisk materialprovning eftersom all permanent deformation beräknas genom det viskoelastiska Burger-elementet. Den nya metoden för beräkning av ZSV visade sig kunna rangordna asfaltmaterialens benägenhet till permanent deformation enligt förväntat utfall baserat på dynamiska moduler och fasvinklar. Den beräknade ZSV var känslig för ändringar i temperatur. Användningen i PEDRO indikerade att metoden eventuellt kan underskatta ZSV. PEDRO visade sig även vara känslig för trafikens sidolägesfördelning, men de beräknade ytprofilerna var rimliga vid användning av uppskattad sidolägesfördelning. PEDRO kalibrerades med både accelererad provning och fältdata från verkliga vägar. De tvetydiga kalibreringsresultaten bör följas upp med ytterligare kalibrering. En fördel med PEDRO är att en rimlig fördelning av permanent deformation mellan olika beläggningslager uppnåddes utan att empiriska korrektioner behövdes.

# List of abbreviations

| | |
|---|---|
| ACB | Asphalt Concrete Base |
| AMADEUS | Advanced Models for Analytical Design of EUropean pavement Structures |
| APT | Accelerated Pavement Testing |
| CalME | The Caltrans and Mechanistic-Empirical methodologies software |
| COST 333 | Development of New Bituminous Pavement Design Method |
| DGAC | Dense-Graded Asphalt Concrete |
| EICM | Enhanced Integrated Climate Model |
| HVS | Heavy Vehicle Simulator |
| IDT | InDirect tensile Test |
| LTPP | Long-Term Pavement Performance |
| M-E PDG | Mechanistic-Empirical Pavement Design Guide |
| MnROAD | In-service full-scale, outdoor road test track in Minnesota |
| NCHRP | National Cooperative Highway Research Program |
| PEDRO | PErmanent Deformation of asphalt concrete layer for ROads model |
| RSST-CH | Repeated Simple Shear Test at Constant Height |
| SHRP | Strategic Highway Research Program |
| SMA | Stone Mastic Asphalt |
| SNRA | Former Swedish National Road Administration (now STA) |
| STA | Swedish Transport Administration (formerly SNRA) |
| SUPERPAVE | SUperior PERforming asphalt PAVEments |
| WesTrack | Experimental full-scale, outdoor road test track in Nevada |
| WIM | Weigh-In-Motion traffic measurement system |
| ZSV | Zero Shear rate Viscosity |

# 1 Introduction

## 1.1 Background

### 1.1.1 Traffic and road deterioration

An efficient transportation system for people and goods is important for the modern society. Most transportation is carried out using roads. The public investment cost for the road network in Sweden has been approximately 0.5 % of the gross national product during the years 1996-2008, as reported by the former agency the Swedish Institute for Transport and Communications Analysis (SIKA, 2009). These investments counteract the gradual road pavement deterioration in order to maintain the standard of the network.

The main cause of road pavement deterioration is traffic, and traffic factors are becoming more serious each year. Swedish goods transport using roads, measured in ton-kilometers, increased by 60% from 1980 to 2001 (SIKA, 2005), i.e. a yearly increase of 2.3 %. The SIKA (2005) prediction for the period 2001-2020 is an increase of 30 %, which corresponds to a yearly increase of 1.4 %. Heavy vehicle developments that improve loading capacity and fuel economy lead to increases in overall vehicle weight, axle loads and tire contact pressures. Further, the use of wide-base single tires, called super-single tires, is expected to increase due to better fuel economy. Wide-base tires are widely known to cause more deterioration in asphalt concrete pavements compared to the traditional dual tire configuration (Sebaaly, 2003; Verstraeten, 1995; EAPA, 1995). Many roads in Sweden have also been constructed or reconstructed as 2+1 roads with narrow lanes, which reduces the vehicle lateral wander and accelerates the deterioration in the middle of the wheel path. Therefore, the gradual increase in traffic load is expected to continue in the future.

The comprehensive European COST 333 (1999) project ranked rutting originating in the asphalt concrete layers as the most relevant distress mechanism to consider, followed by surface cracking, longitudinal unevenness, loss of skid resistance, longitudinal cracking, and bottom-up cracking among others. Rutting is manifested by longitudinal depressions, called ruts, in each wheel path. The presence of ruts causes vehicle steering difficulties that reduce traffic safety (Sousa et al., 1991). The ruts in an impervious pavement surface will trap rain water, and occasionally snow and ice, causing hydroplaning or low friction that further reduces traffic safety (Verstraeten, 1995). Severe rutting can be accompanied by longitudinal cracks that drain free water to the underlying materials that are often sensitive to moisture. The surface profile caused by rutting suggests that a significant portion of the free water can be drained in this way if cracks exist (Sousa et al., 1991).

In addition to the traffic load, material properties and climate factors also affect the permanent deformation in asphalt concrete layers that causes rutting. The effects of traffic load can be mitigated by sufficient material stability (Verstraeten, 1995), mainly in the binder layer (STA, 2005). The stability of asphalt concrete is dependent on the mix composition and the properties of the constituents, which are mainly mineral aggregate and bituminous binder and, possibly, additives. Permanent deformation in asphalt concrete can be reduced by mineral aggregate interlocking that can be achieved by proper gradation, angularity and surface texture. A large maximum aggregate size, high angularity and surface roughness are all considered beneficial to resistance against permanent deformation (Sousa et al., 1991). Most of the mineral aggregate produced in Sweden is crushed rock material, which is a high quality component for asphalt concrete mixing. Mineral aggregate interlocking can also be achieved by stone-to-stone contact in a concept called stone mastic asphalt (SMA) (EAPA, 1998). Permanent deformation in asphalt concrete occurs mostly in the summer due to bitumen sensitivity to high temperatures. Permanent deformation may be reduced by using hard bitumen types and by limiting the bituminous binder content. However, consideration of other distress mechanisms, such as cracking, may introduce opposing requirements (Verstraeten, 1995). The bitumen types preferred in Sweden are relatively soft due to the temperate climate, as shown in the Swedish specification (SNRA, 2005). The increasing use of polymer additives can mitigate permanent deformation in asphalt concrete by reducing the dependence on temperature (Read & Whiteoak, 2003). Another development was the concept of long life pavements, sometimes called “perpetual bituminous pavements”. They are constructed with excessively thick asphalt concrete layers with the intention of keeping all forms of distress in the upper 100 mm. Therefore, rutting is concentrated to the asphalt concrete layers, while the unbound layers are protected. Long life pavements should be practically maintenance free except for periodic surface restoration (Newcomb et al., 2001). The concept is intended to increase service life from 20 years to 50 years or more.

The considerable costs and resource consumption in the construction, rehabilitation and maintenance of road pavements motivate the use of efficient distress models. The distress is calculated using input data for the traffic, materials and climate. The models can be used to design new pavements and rehabilitation measures (NCHRP, 2004), for residue assessment, and material development. Pavement design and rehabilitation is the process of selecting materials and layer thicknesses in order to produce and maintain a cost-effective pavement that will fulfill certain functional criteria during the design period. Modeling can be used to determine how often the pavements require milling and repaving, which reduces traffic accessibility. Modeling can also assess the residue value of a pavement section. Residue value assessment may be used for terminating some forms of construction and maintenance contracts. In addition, a better knowledge of distress mechanism modeling will promote the development of representative laboratory test methods that allow cost-effective mix design in the laboratory.

### 1.1.2 Rutting measurement and origin

Rut depth can be directly assessed by field measurement or interpretation of measured or modeled transversal surface profiles. The rut depth is generally defined as the maximum vertical distance between the rut bottom and a reference line. The most common reference line definitions are the 1.2 m or 1.8 m straight-edge, or a tightened wire line, as shown in Figure 1-1 and Figure 1-2 by Elkins et al., (2006).

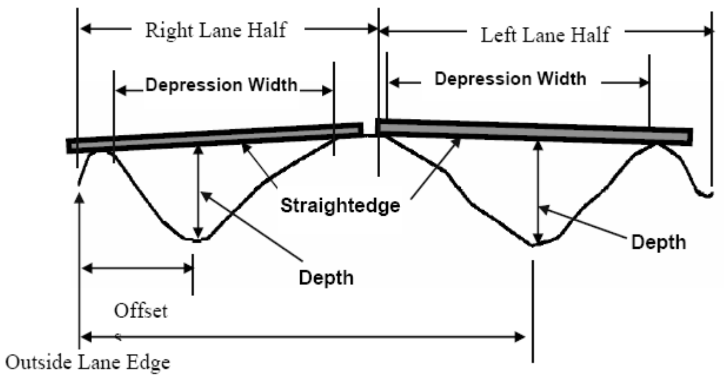


**Figure 1-1. The straight-edge rut depth measurement method (Elkins et al., 2006).**

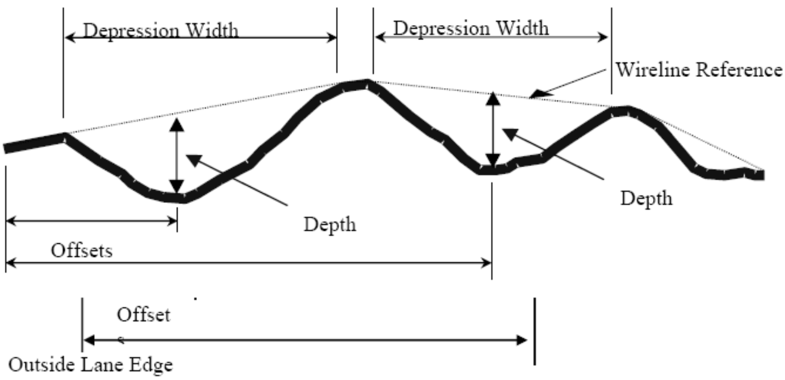


**Figure 1-2. The wire line rut depth measurement method (Elkins et al., 2006).**

Rutting can originate from the pavement surface, the asphalt concrete layers or the underlying unbound layers (Verstraeten, 1995). “Wear rutting” is abrasion of the surface, while the underlying layers remains intact, as shown in Figure 1-3. The main cause is pavement surface abrasion by studded tires during winter. Rutting due to permanent deformation in asphalt concrete layers may be accompanied with upheavals to the sides, as illustrated in Figure 1-4. Some of the factors known to induce permanent deformation in asphalt concrete layers are heavy axle loads and high tire pressure, high temperature, low speed and traffic-induced shear forces, especially at curves and intersections. Permanent deformation in unbound layers, i.e. layers composed by crushed rock and subgrade, is shown in Figure 1-5. It occurs at large depths and results in wide surface ruts. The main cause of permanent deformation in unbound layers is heavy vehicle loading. All types of rutting exhibit gradual deterioration rather than instant failure. Field rutting is typically a combination of all three rutting types. This thesis focuses on the rutting originating in asphalt concrete layers.

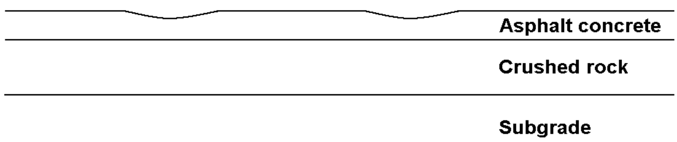


**Figure 1-3. Wear rutting on the pavement surface.**

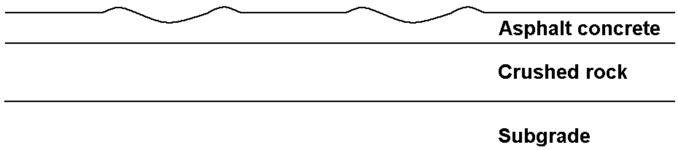


**Figure 1-4. Rutting in asphalt concrete layers.**

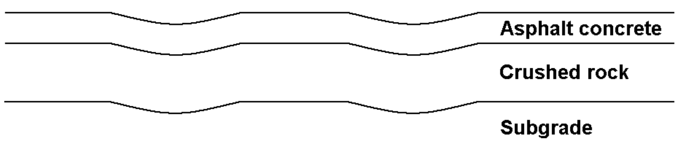


**Figure 1-5. Rutting in unbound layers.**

### 1.1.3 Permanent deformation mechanisms and stages

Permanent deformation in asphalt concrete layers comprises densification and flow rutting (Sousa et al., 1991). The initial permanent deformation mechanism is densification, i.e. post compaction or reduction of air voids. Some densification of materials should be expected, although it can be reduced by sufficient compaction during the pavement construction process. Post compaction mainly occurs during the first one or two years (Wiman et al., 2009; UCPRC, 2008), after which the asphalt concrete is dense enough to withstand further densification.

The main cause of permanent deformation in asphalt concrete layers during pavement life is the continuous material flow due to shear stress, as indicated already at the AASHO Road Test in 1962 (Sousa, 1991). The material is pressed down in the rut, laterally displaced and pressed up into upheavals on the side. Eisenmann & Hilmer (1987) studied this process by conducting accelerated pavement testing and surface profile measurements, as shown in Figure 1-6. Each line in the figure corresponds to a certain number of load applications. The asphalt concrete volume was assumed proportional to the area below the transversal surface profile. The total asphalt concrete volume was found to decrease initially due to densification. However, the main part of rutting appeared under constant volume. At the end of the test, the total asphalt concrete volume also increased, i.e. the material exhibited dilatation.

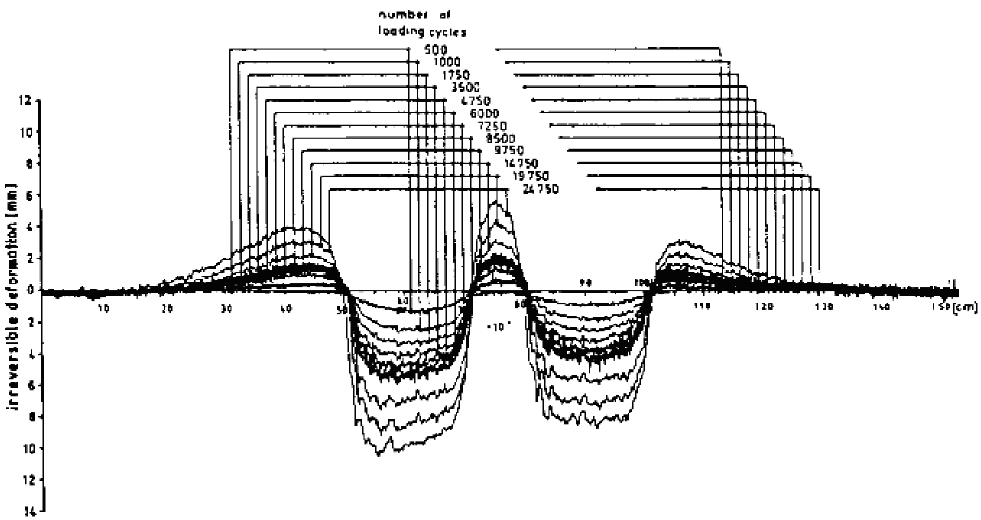


**Figure 1-6. Effect of the number of passes on transverse surface profile (Eisenmann & Hilmer, 1987).**

The three permanent deformation stages indicated by Eisenmann & Hilmer (1987) are called the primary, secondary and tertiary stages. This kind of behavior has been observed in repeated load permanent deformation tests using specimens (Zhou et al., 2004), as shown in Figure 1-7, and by means of full-scale accelerated pavement testing (Bonaquist, 1992). The primary stage develops fast due to the initial densification. The secondary stage is recognized by constant rate rutting that develops through most of the pavement life. Flow rutting due to shear stress is the main contributor, although some densification may also remain. The tertiary stage is characterized by accelerating rutting. The three permanent deformation stages may be difficult to observe on the pavement surface due to the effects of wear and structural rutting. Furthermore, the significant temperature effects may also complicate the interpretation of rutting growth (Zhou et al., 2004).

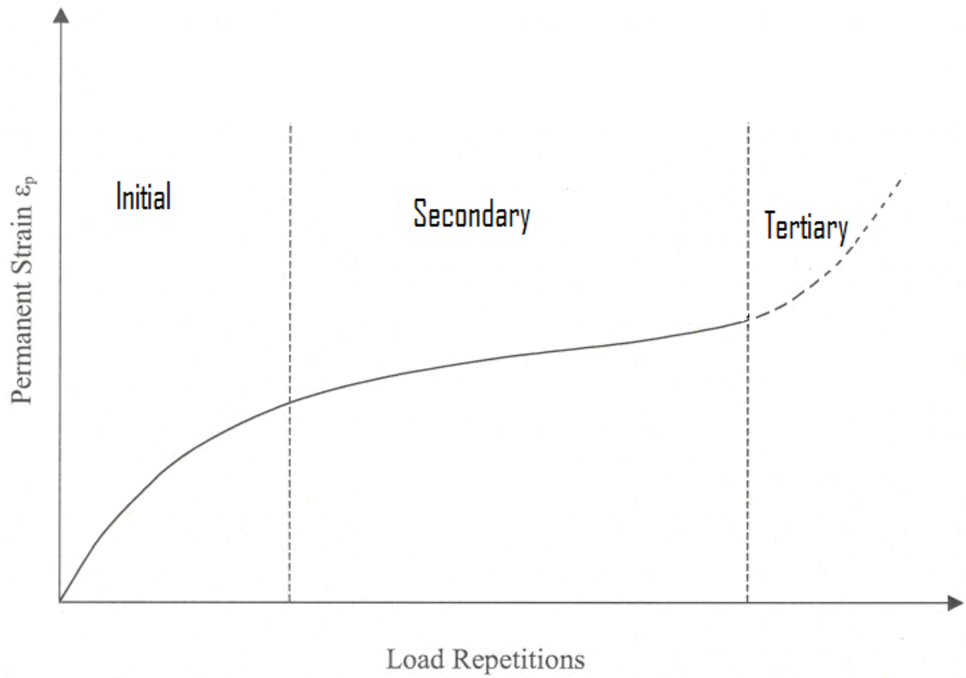


**Figure 1-7. Typical test results from repeated load test.**

### 1.1.4 Rutting model development

In the 1960s, surface rutting was mitigated by using a proper mix design, e.g. the Marshall and Hveem methods, and by limiting the vertical elastic strain in the interface between asphalt concrete and subgrade layers (Sousa et al., 1991). However, these measures were not sufficient, as the traffic load gradually increased. The first analytical models that addressed permanent deformation in asphalt concrete layers were presented by Shell in the 1970s, and several early methods were developed over the next few years. They were based on linear viscoelasticity, or elasticity combined with creep test data. The unbound material rutting was still mitigated by limiting the vertical elastic strain on the subgrade. In addition, statistical methods based on surface rutting observations were developed (Sousa et al., 1991).

In the last three decades, a number of major research programs have been launched to increase our knowledge. The most notable ones are SUperior PERforming Asphalt PAVEments (SUPERPAVE) within the Strategic Highway Research Program (SHRP), and the Advanced Models for Analytical Design of European Pavement Structures (AMADEUS, 2000) performed within Development of New Bituminous Pavement Design Method (COST 333, 1999). These efforts suggest that deterioration should be modeled incrementally based on traffic characteristics, materials, structure, and climate, as shown in Figure 1-8 (AMADEUS, 2000). The shown approach is based on response modeling to calculate stresses and strains, and performance modeling to calculate the distress. The improvements of analytical, or mechanistic, modeling have eventually been implemented in practical design methodologies such as the Mechanistic-Empirical Pavement Design Guide (M-E PDG) developed by the National Cooperative Highway Research Program (NCHRP, 2004).

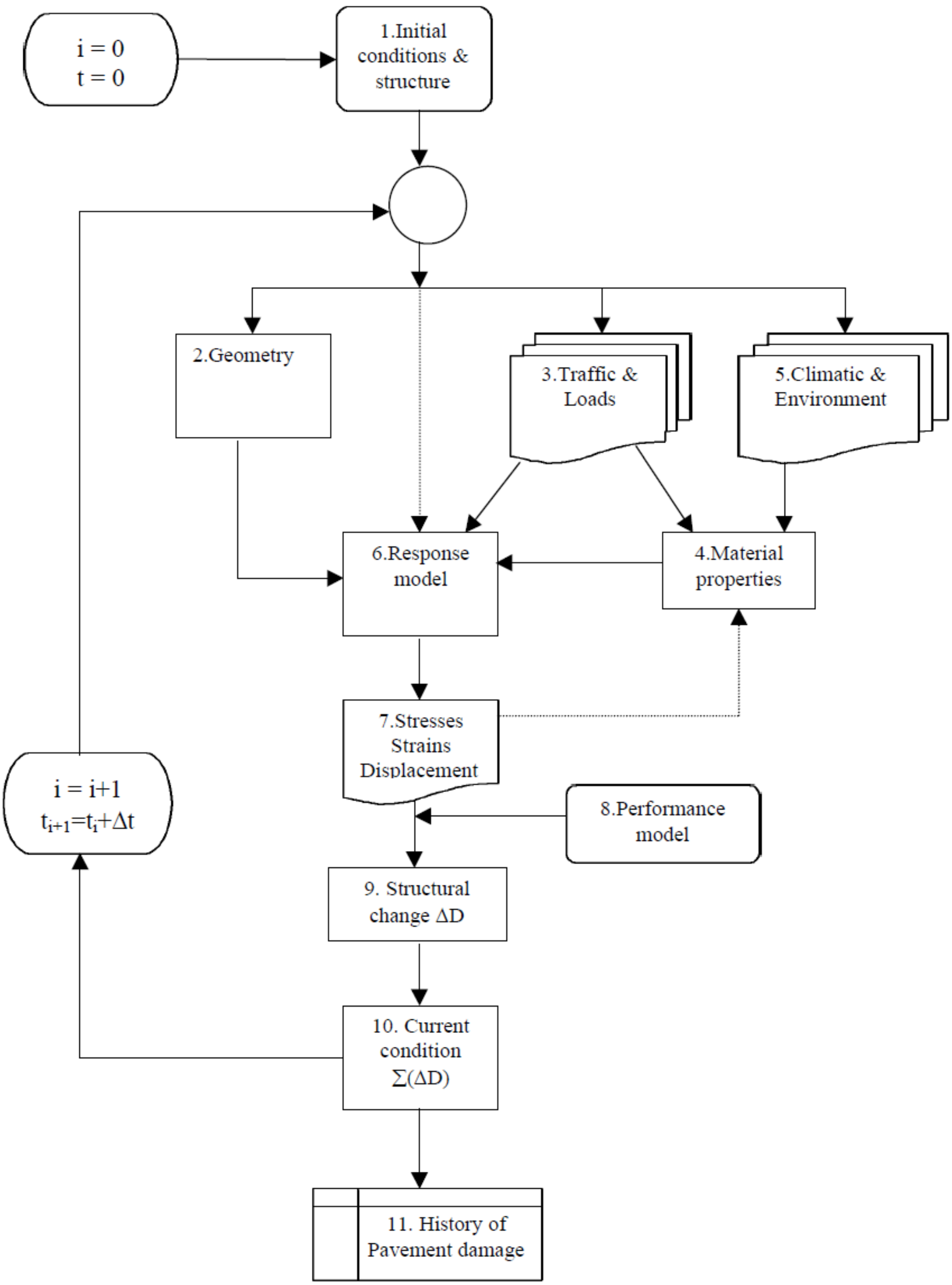


**Figure 1-8. Flowchart of an incremental deterioration model (AMADEUS, 2000).**

## 1.2 Objectives and scope

The overall objective of this doctoral thesis was to increase the knowledge in the field of mechanistic-empirical modeling of permanent deformation in asphalt concrete layers under Swedish conditions. The increase in knowledge should provide guidance for the development and implementation of new design and rehabilitation methodologies. The overall objective can be divided into elaborations on input data, modeling and calibration. The input data evaluation includes comparing and evaluating model requirements as well as alternative assessment methods and the resulting input data quality. In addition, the model approaches to continuously update the material properties as a function of traffic characteristics and climate factors were to be evaluated. The reasonableness of model behavior compared with field observations under a limited variety of conditions was to be evaluated. Further, interpretations of the model results to permanent deformation were to be compared and discussed. Another objective was to calibrate the models to adjust their results to observation. Finally, the effect and reasonableness of the calibration were to be discussed.

## 1.3 Thesis structure

The following chapter of this thesis is the method section, which describes and motivates the thesis approach in order to reach the objectives. In practice, the method used can be divided into the selection of models, pavement sections to be studied, and input data assessment methods. The background of how the models process the input data to produce the results is provided in the Model approach section, which is based on literature studies. The findings of the papers are presented and discussed in Results and discussion. The final chapter, Conclusions and recommendations, summarizes the results of this thesis.

# 2 Method

The analysis comprised both evaluation of some existing models and some model development. A literature review was carried out within the Licentiate thesis (Oscarsson, 2007). Three models were selected due to their different approaches and potentials. The evaluation and development of the models were carried out by applying them to a number of full-scale pavement test sections under various conditions. Input data regarding climate factors, traffic characteristics and material properties were collected by measurements, assessments and estimations. The model results were compared with measured data. Conclusions were drawn on the basis of the performance of the model elements as well as the overall model-to-observation correlation.

## 2.1 Model selection

The Mechanistic-Empirical Pavement Design Guide v1.003 (M-E PDG) (Papers I and II) was selected because it is probably the most widely used mechanistic-empirical methodology nowadays. The model is used, or is intended to be used, within most US states, although some may not implement it completely (Yu, 2009). The M-E PDG is based on response and distress modeling using vertical strains. Therefore, it was interesting to also evaluate the potential of shear based methodologies, such as the incremental-recursive mechanistic-empirical model developed at University of California that is packaged in the CalME software v0.82 (UCPRC, 2008; Paper III). The incremental-recursive mechanistic-empirical model, hereafter called the CalME model, employs response and distress modeling based on shear strains. Thus, the model structures of M-E PDG and CalME are largely similar, although different in some key respects. The PErmanent Deformation of asphalt concrete layer for ROads (PEDRO) model (Papers IV and V) was included due to the newly developed assessment method for its main material parameter by means of frequency sweep laboratory testing. Unlike the other models, PEDRO is based on viscoelastic densification and flow. Despite the differences between the models mentioned above, they have some common features such as climate and traffic modeling, and laboratory testing of viscoelastic material properties. The model input data were generally of the highest achievable quality unless stated otherwise.

The mechanistic-empirical models evaluated in this thesis combine the advantages of mechanistic and empirical modeling. They are based on mechanistic response modeling of linear elastic or permanent linear viscoelastic strains, as presented in Figure 2-1. The M-E PDG and CalME models continue by relating the elastic strain and plastic strain using a mechanistic-empirical distress function based on laboratory testing. Unlike the other models, the PEDRO model uses the linear viscoelastic material property called “zero shear rate viscosity” to calculate the distress. All the models are then empirically calibrated to adjust the model results to field observations. Finally, the plastic strain contribution of each sublayer is combined into total permanent deformation in the M-E PDG and CalME models. However, the M-E PDG and CalME models only calculate the maximum permanent deformation, which is interpreted as a component of surface rut depth. Unlike the other two models, PEDRO calculates the permanent deformation in multiple lateral positions. The resulting permanent deformation profile is used for rut depth interpretation.

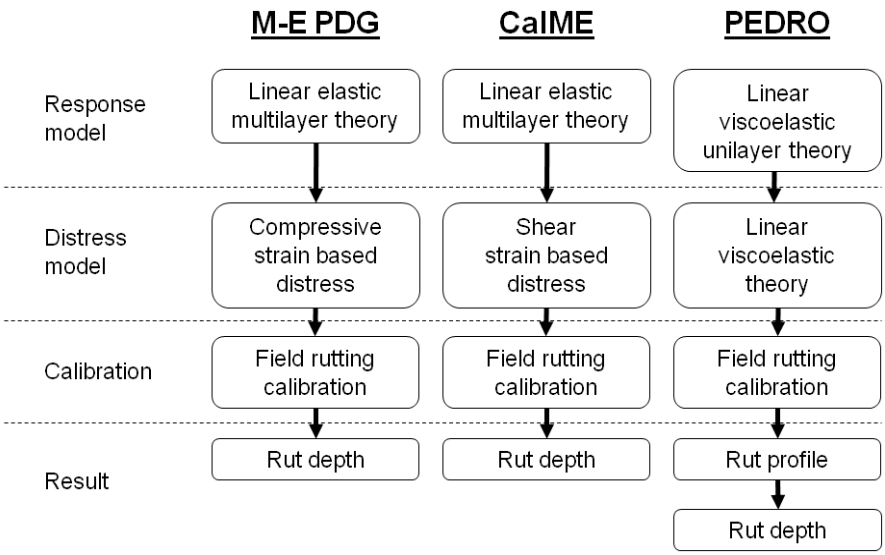


**Figure 2-1. Model structure of the models evaluated.**

All evaluated models in Figure 2-1 are incremental in accordance with the recommendation of COST 333 (1999). The time line is divided into increments, in which the calculation process is repeated to accumulate distress. The input data in each increment should therefore be representative of the period. Sufficiently small increment sizes lead to high accuracy results at the expense of high computational power. In addition, the incremental CalME approach is also recursive, since the traffic-induced damage is looped back to affect the material properties.

## 2.2 Pavement sections

The M-E PDG (Paper I) and PEDRO (Paper IV) models were employed on two full-scale pavement sections that were subjected to accelerated pavement testing (APT) using the heavy vehicle simulator (HVS), as shown in Figure 2-2. Both pavement sections were constructed with thick asphalt concrete layers, as illustrated in Figure 2-3. The main difference between these two sections was the polymer modification in the binder layer of the enhanced pavement section. The HVS testing was carried out under simplified loading conditions using constant half-axle load, tire pressure, speed and lateral wander. The accelerated pavement testing was carried out under controlled climatic conditions at a number of temperatures that were constant throughout the pavement structure. The ageing occurring in the asphalt concrete layers was very limited due to the short testing time. The materials were characterized by dynamic modulus laboratory tests in the laboratory. The modeled permanent deformation was compared with the vertical permanent deformation in each asphalt layer as measured with integrated gauges. Therefore, the structural rutting did not require consideration. This also applies to wear rutting, since the abrasion by the loading device with non-studded tires was minimal (Papers I and IV).


**Figure 2-2. The accelerated pavement testing facility called heavy vehicle simulator (Wiman, 2005).**

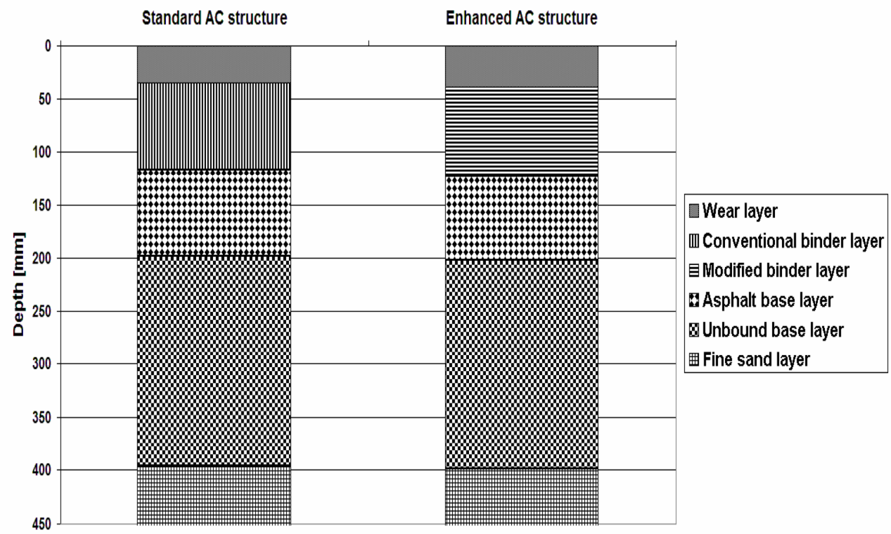


**Figure 2-3. The pavement structure of the accelerated test sections (Paper I).**

Studying the controlled deterioration achieved in full-scale APT provides both opportunities and limitations as compared with in-service pavements. APT produces fast results under controlled traffic and climate conditions, which provides simple and exact model input data. The corresponding traffic and climate input data from in-service pavements are both complex and difficult to assess. However, the simplifications of APT also reduce the realism compared with in-service pavements. Therefore, the APT was employed in the early model evaluations that were followed up by modeling in in-service pavements.

The M-E PDG (Paper II), CalME (Paper III) and PEDRO (Paper V) models were applied on two in-service pavement sections on the E6 motorway at Fastarp-Heberg in Sweden, as shown in Figure 2-4. The layer structure of the evaluated semi-rigid and flexible pavement sections are illustrated in Figure 2-5. The test sections were subjected to the complex traffic load caused by a wide range of vehicle configurations. In addition, the pavement test site experienced the seasonal and daily climatic variations that can be expected due to the location on the coast of south-western Sweden. The traffic and climate generated enormous amounts of data that were interpreted before being employed as input data. The materials and structure of the test pavement sections were determined using laboratory testing. The surface rut depth development at the test sections was well documented throughout the pavement life. The wire line method was used consistently both for field measurement and for interpreting modeled rut profiles. The wear rutting measured was subtracted from the total rutting. However, separation of permanent deformation in asphalt concrete layers and in unbound layers was not feasible, because no trench study was carried out. Therefore, Paper II and Paper III include some unbound layer permanent deformation analysis that was also employed in Paper V.

**Figure 2-4. The E6 motorway at Fastarp-Heberg with profile measuring device (Wiman et al., 2002).**

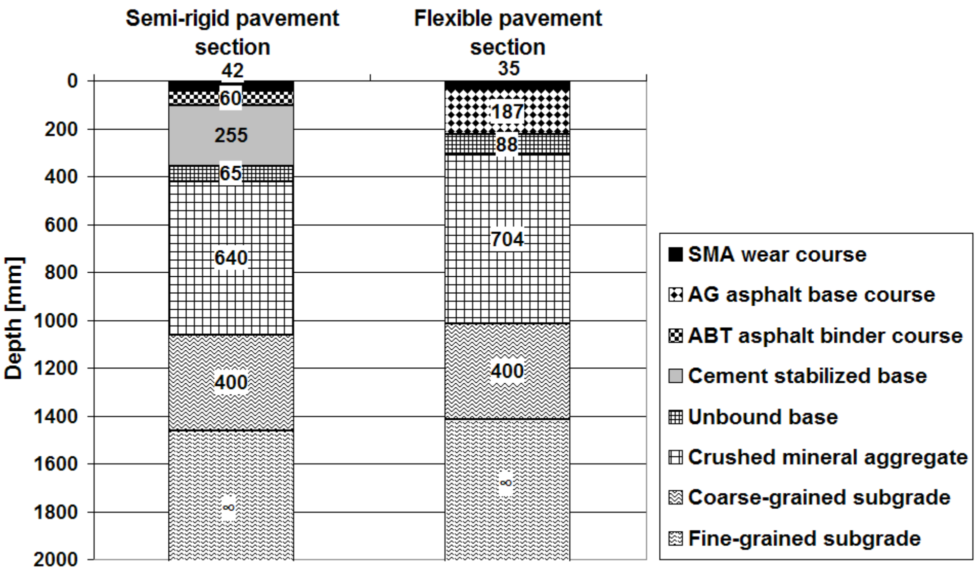


**Figure 2-5. The pavement structure of the in-service test sections (Paper II).**

## 2.3 Climate factors

The main climatic factor for permanent deformation is temperature. The temperature in the HVS test pavement sections was controlled using a climate chamber. Consequently, the temperature was considered constant without any gradient throughout the pavement layer structure (Papers I and IV). The hourly temperature in each sublayer of the E6 motorway pavement section was modeled using the Enhanced Integrated Climate Model (EICM) developed by Larson & Dempsey (2003). EICM is a widely used one-dimensional model for heat and moisture flow due to climatic factors such as air temperature, wind speed, solar radiation, cloud cover and rainfall. The climate model input data at Fastarp-Heberg were measured by the Swedish Transport Administration, the Swedish Meteorological and Hydrological Institute and the Geological Survey of Sweden (Papers II, III and V). All climate data corresponded to the highest level of input data quality, level 1, in the M-E PDG model.

## 2.4 Traffic characteristics

Measurements and assessments of traffic characteristics such as heavy vehicle flow, axle weight, axle configuration, tire contact pressure, speed and lateral wander were carried out to provide input data for the permanent deformation models. The traffic data corresponded to the highest level of input data quality, level 1, in the M-E PDG model. The loadings in the HVS test were applied using a controlled loading device equipped with a wide-base single tire. The loading device moved bi-directionally with controlled speed and normally distributed lateral wander (Papers I and IV).

The heavy vehicle flow on the E6 motorway at Fastarp-Heberg was measured using on-site pneumatic tubes or inductive loops throughout the pavement life (Papers II, III and V). Field characterization of heavy vehicle traffic passing on the E6 motorway at Fastarp-Heberg was carried out using Weigh-In-Motion (WIM) measurement. The main data extracted from WIM data were axle load spectrum, hourly distribution and vehicle speed. The WIM data were collected in 2004 near Kungsbacka and in 2005-2007 near Löddeköpinge, located 95 km north and 130 km south of the test sections, respectively. The heavy vehicle load spectra were considered sufficiently similar to allow interpolation of the data. The extensive WIM data recorded in one week were processed and summarized to fit each model. The axle load spectrum was determined for single axles only for PEDRO (Papers IV and V) and also organized into axle groups, i.e. steering, single, tandem, tridem axles for CalME and M-E PDG (Papers I, II and III). In addition, the axle groups were attributed to certain heavy vehicle types for M-E PDG (Papers I and II).

## 2.5 Material properties

The material properties were determined in order to provide input data for the permanent deformation models. Asphalt concrete characterization can be divided into specimen production, dynamic modulus testing or assessment, destructive testing, and assessment of material properties adjustments throughout pavement life. The unbound material properties were defined using the gradation for M-E PDG (Papers I and II), and by using falling weight deflectometer data for CalME (Paper III).

The asphalt concrete specimens used for material testing were cored from the pavement sections (Papers I-V). Three replicate specimens were used in all dynamic modulus tests, while 2-3 specimens were available for destructive test purposes. The specimens tested in Papers I and IV were cored from an area not trafficked by the loading device at approximately the same time as the HVS testing. The specimens obtained from the E6 Fastarp-Heberg motorway sections (Papers II, III and V) were cored from between the wheel tracks after 12 years of traffic. Those specimens were thus considered affected by a combination of ageing and limited densification and damage.

The dynamic modulus testing was carried out in indirect tensile mode and in shear mode. The M-E PDG dynamic modulus input data should be measured by means of compressive uniaxial testing using 150 mm high cylindrical specimens according to the AASHTO TP62-03 (2006) standard. However, it was not feasible to core specimens with the specified height from considerably thinner asphalt concrete layers of the selected pavement sections. Therefore, the uniaxial test was replaced with the indirect tensile test using a viscoelastic solution developed by Kim et al. (2004). The indirect tensile test data was used for M-E PDG and CalME (Papers I, II and III) and shear test data for PEDRO (Papers IV, V and VI). The time-temperature superposition principle was employed to shift the dynamic modulus data in each frequency sweep laterally according to its temperature. The dynamic modulus data were modeled using a parameter-controlled sigmoidal function in order to produce master curves (Pellinen, 2009).

The dynamic modulus data of the cored asphalt concrete specimens were adjusted to account for ageing, densification and damage as compared to their initial state. All the models evaluated are based on the initial material input data at the time of the pavement construction, or at the opening of traffic. In Papers I and IV, the specimens were cored from an area unaffected by traffic loading after the HVS test was completed, and they were therefore considered representative of the initial pavement materials. In contrast, the in-service pavement sections at the Fastarp-Heberg on the E6 motorway had been trafficked for 12 years before the cores were obtained from between the wheel tracks. Ageing adjustments of the dynamic modulus data were therefore considered important. In Paper II, several adjustments were carried out simultaneously. The M-E PDG dynamic modulus testing should be carried out using gyratory compacted, short-term aged specimens (NCHRP, 2004). The compaction differences, long-term ageing and possible damage effects were accounted for by normalizing the dynamic modulus data according to results from previous testing. The stiffness adjustments in Papers III and V were carried out according to Swedish specification (SNRA, 2005) based on an ageing study by Said (2005).

The M-E PDG can be employed at three hierarchical levels of input data quality. Each level corresponds to the degree of effort in obtaining the input data. The model analysis at the three levels is similar, although the intention is to develop different model elements to improve the utilization of high quality input data (NCHRP, 2004). In addition to laboratory measurements at level 1, dynamic modulus was also assessed using the M-E PDG integrated predictive equation at level 3 (Paper II). The predictive equation is a function of mix composition, binder viscosity and loading frequency (NCHRP, 2004). The level 3 analysis for asphalt concrete was carried out to compare the measured dynamic modulus with the predicted one and to evaluate the effects on the M-E PDG permanent deformation model.

Destructive testing was carried out to provide parameters for the CalME distress model (Paper III). A limited number of specimens were tested using the repeated simple shear test at constant height (RSST-CH) according to the AASHTO T320-03 (2006) standard. The RSST-CH data were interpreted using the CalME gamma function. The M-E PDG distress function was predetermined (Papers I and II). The PEDRO model (Papers IV and V) require no destructive testing.

# 3 Model approach

The models evaluated simulate the permanent deformation development using a number of model elements. The elements in the models evaluated are response modeling, distress modeling, rut depth interpretation and field calibration. The three models are similar in performing the calculations incrementally in multiple sublayers.

## 3.1 Response modeling

The models evaluated are based on the elastic or viscoelastic strain response in the pavement layers due to the moving traffic load. The traffic data measured are divided into axle load spectra. Each axle load is distributed over a number of circular load areas on the pavement surface, and the lateral wander distribution is accounted for. The elastic or viscoelastic strain response is modeled as a part of the models evaluated.

### 3.1.1 Axle load configuration

The heavy vehicle axles are modeled as a half axle load in each rut. The half axle load may be modeled as a wide-base tire or a pair of dual tires. The dual tire configuration is the most common type, which is also reflected in the traditionally modeled equivalent standard axle according to Swedish specification (SNRA, 2005), as shown in Figure 3-1. However, the use of wide-base tires is expected to increase due to their fuel economy advantage (EAPA, 1995). Wide-base tires are known to cause more permanent deformation in flexible pavements than dual tires (Sebaaly, 2003; Verstraeten, 1995; EAPA, 1995). This is illustrated in Figure 3-2 and Figure 3-3 based on accelerated pavement testing, as reported by Bonaquist (1992).

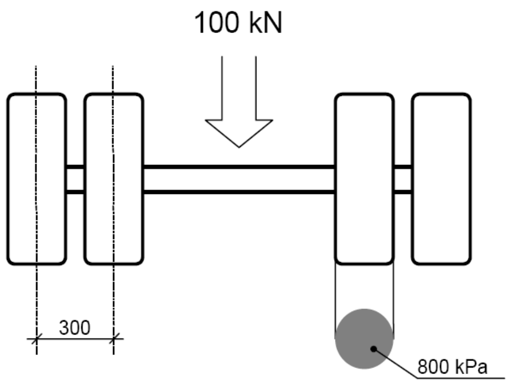


**Figure 3-1. The Swedish equivalent standard axle load (SNRA, 2005).**

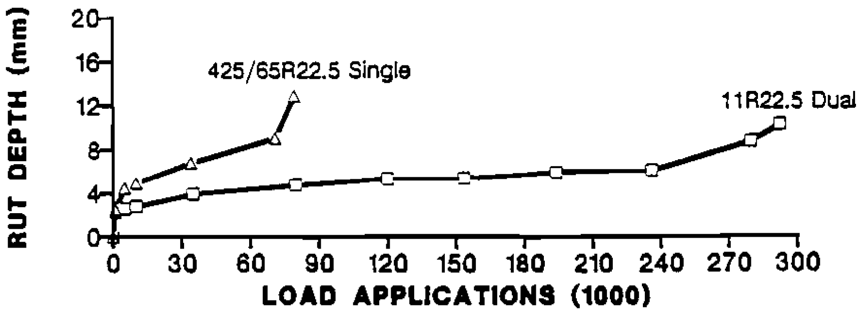


**Figure 3-2. Rut depth in an 89 mm thick asphalt concrete structure using wide-base single tires, and dual tires (Bonaquist, 1992).**

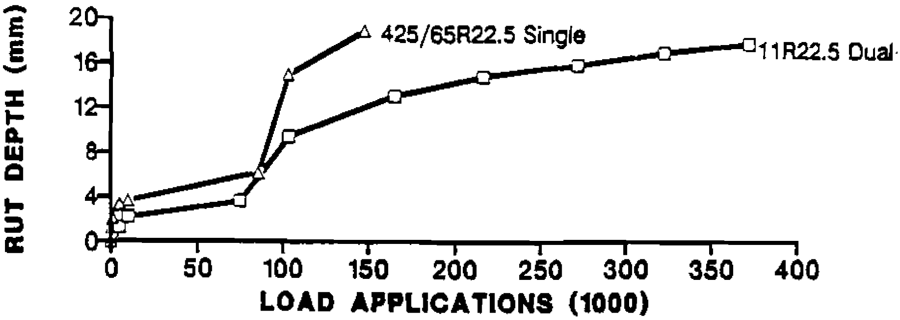


**Figure 3-3. Rut depth in a 178 mm thick asphalt concrete structure using wide-base single tires, and dual tires (Bonaquist, 1992).**

The M-E PDG and CalME currently model dual tires only, except for the steering axles (Papers I, II and III). PEDRO was employed using single tires only in accordance with the HVS testing (Paper IV) and using dual tires only in the field (Paper V). All the models evaluated could be developed to account for any portion of single and dual tires, although this may increase the required computation power.

### 3.1.2 Tire contact pressure

Each tire mounted on the half-axle is traditionally modeled as a circular, uniformly distributed load with a constant contact pressure equal to the mean tire inflation pressure. The Weigh-In-Motion (WIM) measurements define the load data for each passing axle. The tire-to-pavement contact pressure is traditionally assumed to be constant and equal to the mean tire inflation pressure in order to calculate the contact area radius. However, the modeled elastic response has been shown to be considerably more sensitive to variation in tire inflation pressure than load variation, as illustrated in Figure 3-4 and Figure 3-5 (Eisenmann & Hilmer, 1987). Therefore, variable tire inflation pressure could be accounted for, although it may be difficult to assess. All the models evaluated use the traditional approach (Papers I-V). However, CalME can optionally calculate a variable tire pressure based on variable load and constant load radius, in order to reduce the required computation power (Paper III).

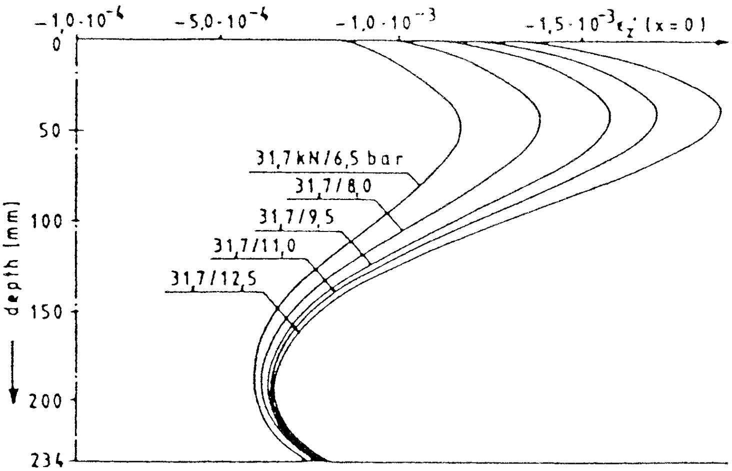


**Figure 3-4. Vertical elastic strain as a function of depth for some tire inflation pressures using a constant load level (Eisenmann & Hilmer, 1987).**

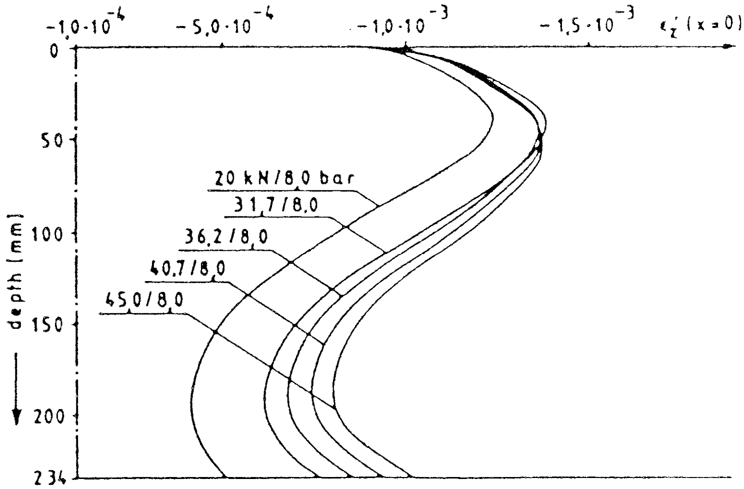


**Figure 3-5. Vertical elastic strain as a function of depth for some load levels using a constant tire inflation pressure (Eisenmann & Hilmer, 1987).**

The traditional tire modeling approach described above results in larger elastic strain than modeling more realistic, non-circular and non-uniformly distributed contact pressure. The effects of the model assumptions were extensively reviewed based on previous research and investigations by Luo & Prozzi (2005). They measured the actual contact pressures and modeled them using multiple, uniformly distributed circles instead of a single one. Contrary to initial expectations, the traditional approach results in larger compressive elastic strains as compared to using the actual contact pressures. Therefore, the accuracy could be increased by modeling a large number of uniformly distributed load circles rather than a single one. However, that would require more computation power and better assessments of contact pressure data.

### 3.1.3 Lateral wander

Lateral wander is the transversal distribution of vehicle passages over the lane width. The vehicle distribution mitigates densification and flow rutting both by reducing the traffic passing in the rut, and by increasing the portion that may press down the upheavals. Field measurements have shown that lateral wander can be modeled using a normal distribution as a function of lane width, which is the primary factor. Additional factors that may have an impact are weather, vehicle type, traffic conditions, road characteristics, the surrounding environment and time of day, week, month and year (Buiter et al., 1989). All the models evaluated take lateral wander directly into account by applying a normally distributed traffic load in their mechanistic analysis.

### 3.1.4 Pavement structure

Linear elastic or viscoelastic response modeling can be applied on a single semi-infinite layer or on a multilayer structure. The modeled structure is considered unaffected throughout the pavement life in the models evaluated. The M-E PDG and CalME use the linear elastic multilayer approach to determine the elastic strains caused by a moving load. The main input data is the elastic modulus and Poisson's ratio, in addition to layer thicknesses. The elastic material properties of the viscoelastic asphalt concrete material are modeled using the laboratory-determined master curve combined with temperature and loading frequency data. The PEDRO model is based on Björklund's (1984) approach for linear viscoelastic response in a single semi-infinite layer due to a moving load. The linear viscoelastic material input data are zero shear rate viscosity and Poisson's ratio.

### 3.1.5 Loading frequency

Loading frequency modeling is required in order to model the elastic material properties based on viscoelastic material properties. The exact loading frequency in a viscoelastic material should be calculated using a detailed Fourier analysis (Al-Qadi et al., 2009). However, the models evaluated use approximate methods that generally model loading time as a function of vehicle speed and sublayer depth in CalME (Ullidtz et al., 2006a) and also material properties of the above layers in M-E PDG (NCHRP, 2004). The loading time is calculated in each sublayer, as shown in Figure 3-6 from the M-E PDG (NCHRP, 2004). The approximate loading time is then converted into loading frequency by inverting either the loading time (NCHRP, 2004) or the product of 2π and the loading time (Ullidtz et al., 2006a).

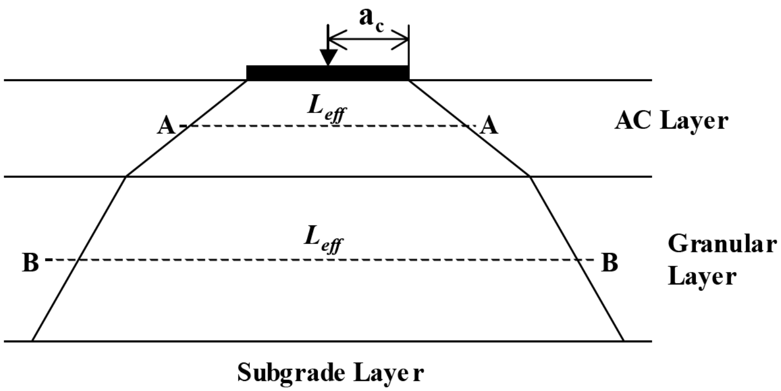


**Figure 3-6. Effective length approximation in each sublayer (NCHRP, 2004).**

### 3.1.6 Material properties adjustment

Asphalt concrete properties are continuously developing due to ageing, densification and damage. “Ageing” is a generic term for a number of processes that hardens the bitumen and increase its viscosity. The main processes are oxidation, evaporation, exudation, and physical hardening (Read & Whiteoak, 2003). Most asphalt concrete stiffening due to ageing appears early in the pavement life. “Densification” refers to the traffic-induced reduction of air voids that have been found to increase the dynamic modulus in asphalt concrete specimens in laboratory (Karlstrand & Neander, 2006). Traffic may also induce damage, which can be defined as the gradual reduction of stiffness due to micro-cracking (UCPRC, 2008). It should be noted that ageing and densification increase the dynamic modulus, while damage reduces it.

The models evaluated perform several dynamic modulus adjustments in order to account for ageing, densification and damage. The M-E PDG employs the Global Ageing System, which, in turn, includes models for short-term ageing, surface ageing, air void adjustment, and viscosity-depth modeling (NCHRP, 2004). However, the M-E PDG does not continuously adjust stiffness due to damage. CalME uses an empirical model based on time, and alternatively also temperature, to account for ageing and densification. The damage function for asphalt concrete layers is based on a fatigue model, thus making the CalME model recursive. PEDRO was employed using an empirical ageing model based on time, and a separate densification function. In conclusion, all the models evaluated include various stiffness adjustments sub-models.

## 3.2 Distress modeling

Distress modeling relates the structural response model results to plastic strain. The distress models are generally dependent on similar factors, as shown in Equations (1) to (3). The M-E PDG distress model, or "constitutive equation", is a function of the number of loadings, temperature and the elastic compressive strain determined by means of the response model, as shown in Equation (1). Its inelastic material properties are pre-determined on the basis of repeated load laboratory testing. The CalME distress models, i.e. the "gamma function" and the previously used "power function", are similar to that of the M-E PDG. However, the modeled stress mode is shearing instead of compression, and temperature dependence is exchanged for shear stress dependence, as shown in Equation (2). It should be noted that the CalME permanent deformation model depends on the shear stress at 50 mm depth, regardless of the sublayer depth. The inelastic material properties are determined for each material using the repeated simple shear test at constant height (RSST-CH). The PEDRO model distress function is based on the viscoelastic material property called zero shear rate viscosity (ZSV). The ZVR in each sublayer is determined using a method based on frequency sweep shear testing data. In addition, the distress function is dependent on the number of loadings, as shown in Equation (3). It would be interesting to perform a comprehensive sensitivity analysis to evaluate the effect of these variables.

$$\varepsilon_p^{M-E\,PDG} = f(N, T, \varepsilon_e, I) \qquad (1)$$

$$\gamma_p^{CalME} = f(N, \tau, \gamma_e, I) \qquad (2)$$

$$\varepsilon_{ve}^{PEDRO} = f(N, T, I) \qquad (3)$$

where

$\varepsilon_p$: Plastic strain
$\varepsilon_e$: Elastic strain
$\varepsilon_{ve}$: Viscoelastic strain
$\gamma_p$: Plastic shear strain
$\gamma_e$: Elastic shear strain
$\tau$: Shear stress at 50 mm depth
N: Number of load repetitions
T: Temperature
I: Inelastic material properties

## 3.3 Rut depth interpretation

The permanent deformation is calculated by numerical integration of the plastic strain over the total asphalt layer thickness. The permanent deformation is incrementally accumulated throughout pavement life. The M-E PDG and CalME interpret the accumulated permanent deformation in the middle of the wheel track as rut depth. However, CalME limits the permanent deformation in asphalt concrete layers to a maximum depth of 100 mm below the pavement surface. The PEDRO procedure differs by calculating the permanent deformation based on the total asphalt concrete layer thickness in a number of lateral positions in order to obtain a rut profile. The rut depth from asphalt concrete layers is then interpreted according to the wire line method by adding the maximum positive and minimum negative permanent deformations from the wheel track middle and upheavals, respectively.

## 3.4 Field calibration

The models evaluated are field calibrated in order to empirically increase the correlation with observed rut depth. The purpose of calibration factors is to remove any bias, i.e. systematic errors produced by the mechanistic-empirical model elements. The field calibration data for the models are derived from accelerated pavement testing and long-term pavement performance (LTPP) programs. The results of the M-E PDG calibration effort for the asphalt concrete permanent deformations model are shown in Figure 3-7 (NCHRP, 2006).

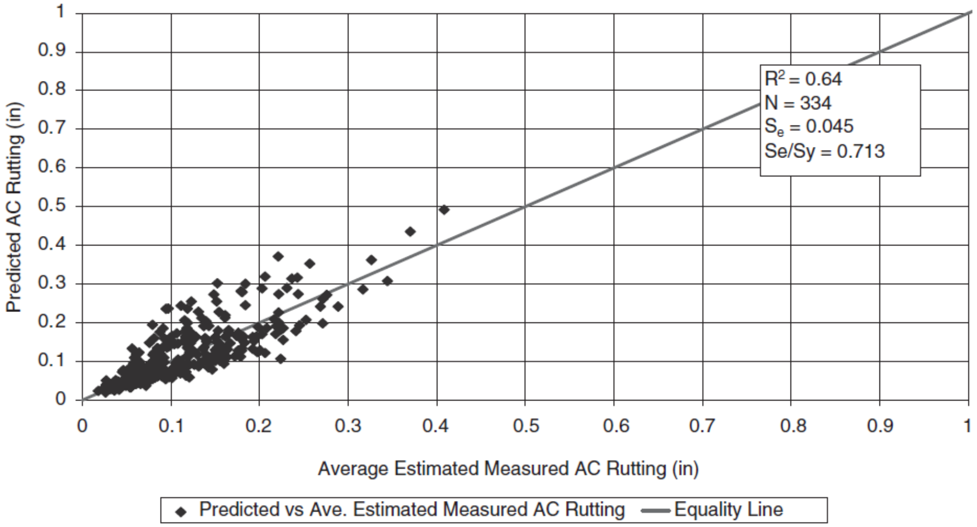


**Figure 3-7. Permanent deformation calibration for asphalt layers (NCHRP, 2006).**

The field calibration is carried out using calibration factors that may be separated from, or integrated with, the distress model for plastic strain. All the models evaluated use separate calibration factors. In addition, the M-E PDG employs calibration factors that are integrated into the distress model, in which they control the overall result, temperature dependence and dependence on the number of loadings.

# 4 Results and discussion

## 4.1 Material properties

Characterization and modeling of material properties are integral parts of the permanent deformation models. There are a number of different methods for specimen production, material testing and material adjustments throughout the pavement life. The models should generally be used within their limitations, i.e. by using the methods that they prescribe. However, alternative input data collection methods may also be used, although they may alter the results.

### 4.1.1 Dynamic modulus due to specimen production methods

All the asphalt concrete specimens intended for dynamic modulus testing were cored from the pavements because they were considered more representative than laboratory compaction methods. The M-E PDG specifically prescribes that dynamic modulus testing specimens should be produced using gyratory compaction. Laboratory compaction methods, including gyratory compaction, are known to produce stiffer specimens than field compaction (Airey et al., 2006). Replacing gyratory compaction with pavement coring would therefore produce specimens weaker than intended, which would result in over-prediction of permanent deformation in Paper I, considering that the M-E PDG model is laboratory-calibrated using the stiffer gyratory compacted specimens (NCHRP, 2004). The dynamic modulus data derived in Paper II were calibrated using results from gyratory compacted specimens carried out by Oscarsson (2007). In contrast to M-E PDG, the CalME and PEDRO models rely on field compacted specimens rather than gyratory compacted specimens. CalME employed mainly field compacted specimens in the calibration effort (Ullidtz et al., 2006a and 2006b), although some slab compacted specimens were also included. Consequently, Paper III tested field compacted specimens only. So far, the newly developed PEDRO has only been employed with field compacted cores (Papers IV and V). In sum, the specimens should be produced using the method prescribed by the model. The field compacted specimens used in Paper I may have led to some over-prediction of rutting in the asphalt concrete layers.

### 4.1.2 Dynamic modulus interpretation

The dynamic modulus data may be interpreted by means of various methods before being employed in the modeling. The most common procedure is to fit the data to a master curve without any assumptions. However, in Paper III the dynamic modulus data were interpreted into master curves using the minimum dynamic modulus assumption of 100 MPa according to the CalME developers (UCPRC, 2008). This is based on back-calculated falling weight deflectometer data that produce asphalt concrete dynamic modulus values higher than 100-200 MPa at any temperature. The proposed cause is the differences in boundary conditions of specimens in a laboratory test compared to those in the field (Ullidtz et al., 2006a). The UCPRC (2008) also noted that the modulus should at least correspond to that of the aggregate alone. The effect of the assumption is a master curve overestimation compared to measured dynamic modulus data, as shown in the example in Figure 4-1. It may appear inappropriate to correct a proposed structural problem using an empirical material adjustment. Nonetheless, the model should be used under the conditions it was developed and calibrated for (Paper III).

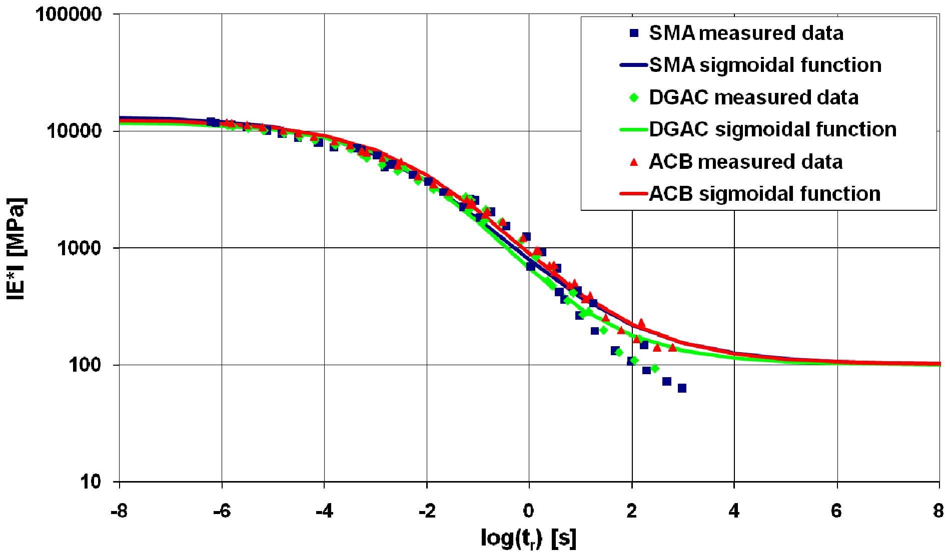


**Figure 4-1. Example of CalME master curve fitting of dynamic modulus test data (Paper III).**

## 4.1.3 Dynamic modulus assessment

The asphalt concrete dynamic modulus can be assessed according to the volumetric properties defining the constituents and their proportions. The M-E PDG at input data quality levels 2 and 3 employs a useful predictive equation as a function of aggregate gradation, viscosity and bitumen content, air void content and loading frequency (NCHRP, 2004). The predictive equation was calibrated using a vast range of mix types. Paper II investigated the difference between measured and predicted dynamic modulus data. The measured data was produced with the indirect tensile test using specimens that were field cored from an area not trafficked on a 12 years old pavement while the M-E PDG prescribes the uniaxial test using gyratory compacted, short-term aged specimens. Therefore, the dynamic modulus data derived in Paper II were calibrated with uniaxial test results using equally composed gyratory compacted specimens, as reported by Oscarsson (2007). This calibration method assumes that the dynamic modulus data of all layers are equally affected by compaction differences, long-term ageing and damage effects, regardless of depth, air void content and binder properties. The results indicated a serious over-prediction of the dynamic modulus in all asphalt concrete layers used in Paper II, as shown in Figure 4-2 and Figure 4-3. The ratio between the predicted dynamic modulus and the one measured ranged from 1.95 to 3.52. Other evaluations have shown that the predictive equation produces reasonable results at high dynamic modulus values and over-predicts at low dynamic modulus values (Azari et al., 2007; Dongré et al., 2005). Consequently, further investigation of the predictive dynamic modulus equation would be valuable.

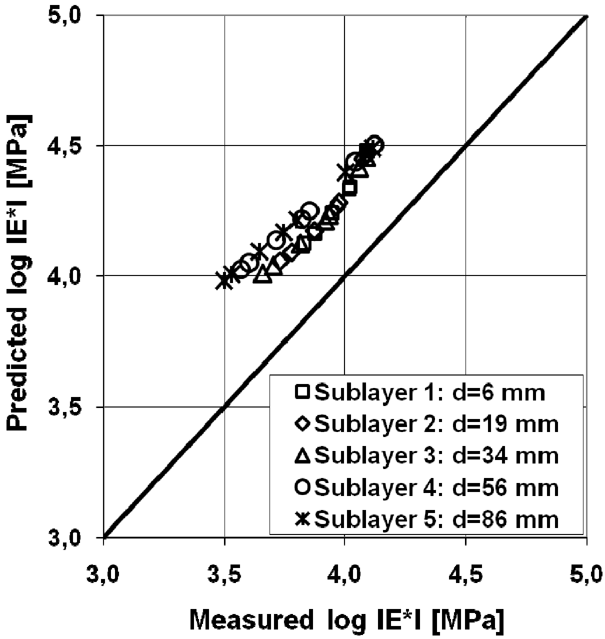


**Figure 4-2. Comparison of M-E PDG predicted and measured dynamic modulus data in asphalt concrete sublayers at different depths in the semi rigid pavement section (Paper II).**

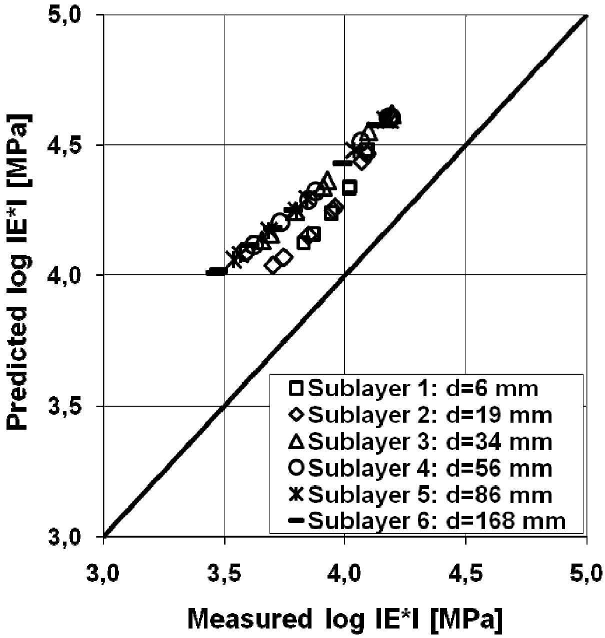


**Figure 4-3. Comparison of M-E PDG predicted and measured dynamic modulus data in asphalt concrete sublayers at different depths in the flexible pavement section (Paper II).**

### 4.1.4 Zero shear rate viscosity assessment

The zero shear rate viscosity (ZSV) can be assessed using a number of methods. For bituminous binders, the main methods are extrapolation of dynamic viscosity, application of the Cross model on dynamic viscosity data, and creep and recovery tests with or without obtaining a steady state (Anderson et al., 2007). The application of those ZSV assessment methods on asphalt concrete may cause difficulties due to material differences as compared with bituminous binders. The difference lies in the internal mineral aggregate structure in asphalt concrete. Nilsson and Isacsson (2001) employed a ZSV assessment method based on cyclic triaxial creep tests without reaching the steady state. However, the reported results appeared inconsistent with field performance.

A new ZSV assessment method was developed based on the dynamic shear modulus and phase angle, and a calculation process using the simplified Cross model (Sybilski D., 1996) in a linear viscoelastic Burger element (Paper VI). Paper V showed that the ZSV ranged over more than 3 to 4 decades on the log scale due to field temperatures from -5 °C to +35 °C. The assessed ZSV was employed as the main material parameter in the PEDRO model (Papers IV and V).

The ZSV is inversely proportional to the permanent deformations modeled with PEDRO. Linear viscoelasticity was expected to account for only a portion of the total permanent deformation, since viscoplasticity effects were not included. This expected behavior was generally found in Paper IV. However, Paper V showed that the calculated permanent deformation in several instances exceeded that observed in the accelerated testing or in the field, which may be the result of inaccuracies in any of the materials, climate and traffic input data. One possible cause is that the ZSV assessment method developed in Paper VI underestimated the zero shear rate viscosity (Paper V). Nevertheless, the ZSV assessment method appeared to rank the asphalt concrete materials correctly according to expectations based on their dynamic modulus and phase angle. Further validation of the ZSV assessment method presented would be interesting.

### 4.1.5 Material properties adjustments

The models can account for dynamic modulus increase due to ageing and densification, and for decrease due to damage. The M-E PDG Global Ageing System continually increases the dynamic modulus for short-term ageing, surface ageing, air void ageing adjustment, and viscosity-depth modeling (NCHRP, 2004), as shown in Table 4-1. The resulting M-E PDG adjustment is positive since no adjustment for damage is made. CalME adjusts the dynamic modulus using a positive ageing and densification function, as well as a negative recursive damage function. The combined CalME dynamic modulus effect was negative, i.e. the damage function was stronger than the ageing and densification functions (Paper III). However, long-term pavement performance (LTPP) studies have shown that during the five first years, stiffness increased by 18 %, 38 % and 69 % for the stone mastic asphalt (SMA), dense-graded asphalt concrete (DGAC) and asphalt concrete base (ACB) layers, respectively (Wiman et al., 2002). The inclusion of a recursive damage model in CalME is an advantage, although further developments are recommended in order to correspond to observed performance in Sweden. The PEDRO model adjusts for ageing and post compaction only, resulting in an increase in the dynamic modulus. It would be interesting to study and calibrate each known dynamic modulus effect separately.

**Table 4-1. Adjustment factors for dynamic modulus data**

| Cause | M-E PDG | CalME | PEDRO |
|---|---|---|---|
| Ageing | + | + | + |
| Densification | + | + | + |
| Damage | | - | |
| Result | + | - | + |

The + and - signs refer to dynamic modulus increase and decrease, respectively.

Ageing, densification and damage influence the permanent deformation properties in two ways, one of which is by controlling stiffness that determines the elastic strain response, and the other being the viscoelastic strain in PEDRO and the elastic-plastic distress functions in the M-E PDG and CalME models. The latter two models use elastic-plastic strain relations that remain constant throughout pavement life. However, results from Paper III suggest that the plastic-elastic strain relation was affected in the limited number of 12-year-old specimens cored between the wheel tracks. The repeated simple shear test at constant height (RSST-CH) using aged specimens produced significantly lower permanent strain than the default WesTrack material (Paper III). This is illustrated in Figure 4-4, which is based on the RSST-CH results and the assumptions $\tau = \tau_{ref}$ and $\gamma_e$ = 1000 MPa in an imaginary asphalt concrete slab. Comparing with model results, the CalME model seriously underestimated rut depth when using input data produced with the RSST-CH and 12-year-old specimens, while CalME simulations with WesTrack data resulted in good predictions. Assuming that CalME is reasonably calibrated, the results suggest that the aged specimens were affected by ageing, densification or damage. Therefore, the relation between elastic and plastic properties is recommended to be further investigated.

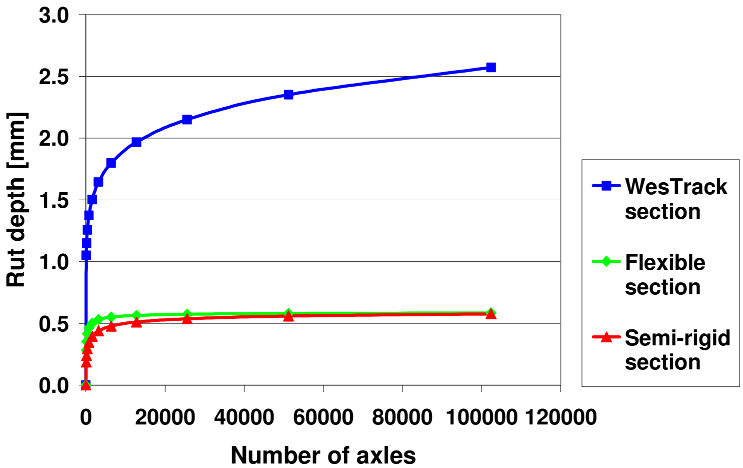


**Figure 4-4. Example of permanent deformation in asphalt concrete layers based on RSST-CH results (Paper III).**

# 4.2 Response modeling

## 4.2.1 Lateral wander

Updated lateral wander field measurements would provide better input data for the permanent deformation models. The PEDRO model displayed a strong sensitivity to lateral wander, as shown in Figure 4-5 and Figure 4-6 (Paper V). Traffic that is normally distributed with a 290 mm standard deviation produced only 36 % of the flow rutting and 58 % of the densification as compared to traffic with no lateral wander. The model's sensitivity suggests that field lateral wander should be thoroughly assessed. Instead, it was estimated on the basis of an empirical model based on lane width (Buiter et al., 1989). Furthermore, the lateral wander is normally assumed to be constant throughout pavement life, although rutting is known to reduce lateral wander by channelization (Harvey & Popescu, 2000). Further research on field lateral wander would therefore increase the benefits of viscoelastic modeling.

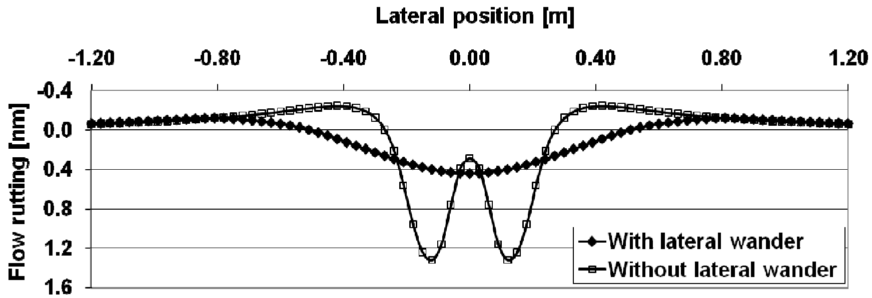


**Figure 4-5. The principal PEDRO flow rutting profile caused by traffic with and without lateral wander (Paper V).**

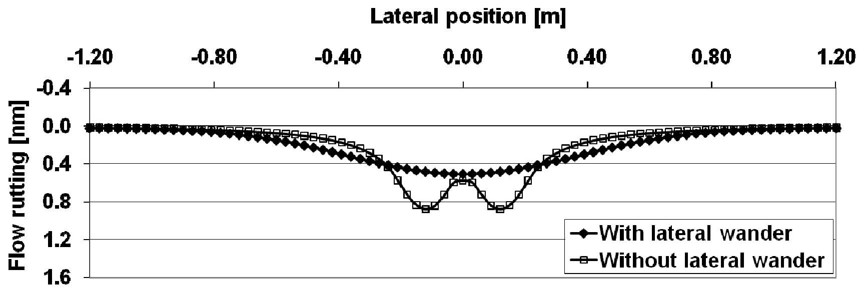


**Figure 4-6. The principal PEDRO densification profile caused by traffic with and without lateral wander (Paper V).**

### 4.2.2 Linear elastic multi-layer strain modeling

The linear elastic multi-layer strain modeling is dependent on the structural properties of the pavement. However, some investigators have considered permanent deformation in asphalt concrete layers to be a question of material stability rather than structural properties (Verstraeten, 1995). Therefore, the elastic multi-layer strain may not be entirely representative of permanent deformation in asphalt concrete. One such unwarranted effect of permanent deformation modeling based on elastic response was presented in Paper III. The CalME modeling in the flexible section resulted in reasonable estimates in comparison with observation, while the permanent deformation in the semi-rigid section was seriously underestimated, as shown in Figure 4-8 and Figure 4-7. This observation was made for both simulations with measured and default WesTrack permanent deformations parameters. The cause, according to Equation 2, is that shear stresses and/or elastic shear strains were considerably smaller in the semi-rigid section than in the flexible section. This behavior is probably due to the stiff cement-treated base layer in the semi-rigid section. Consequently, CalME appears very sensitive to overall pavement stiffness. Shear stress and elastic shear strain may be difficult to relate to flow rutting in very stiff pavement sections. It should be noted that CalME is not calibrated for semi-rigid pavements. However, calibration using a single factor cannot reasonably fit the CalME model to observations both in flexible and semi-rigid pavements. In contrast, the M-E PDG response model appeared more robust to overall pavement stiffness, as the results for the semi-rigid and flexible sections were more reasonable compared with field measurements, as shown in Figure 4-9 and Figure 4-10 (Paper II).

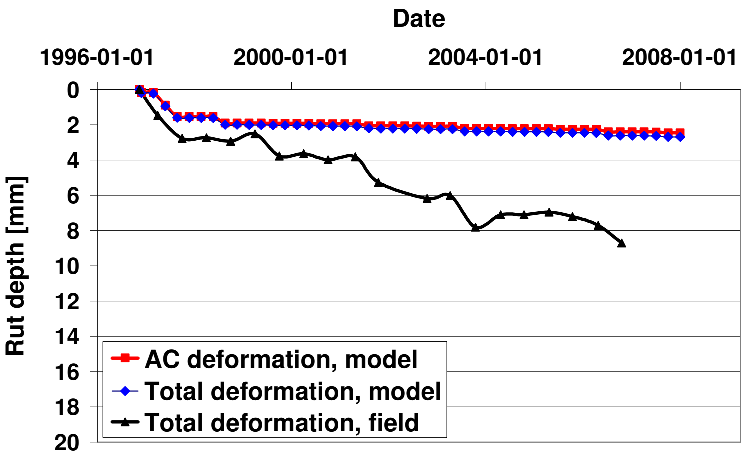


**Figure 4-7. CalME rut depth in the semi-rigid test section using WesTrack permanent deformation parameters (Paper III).**

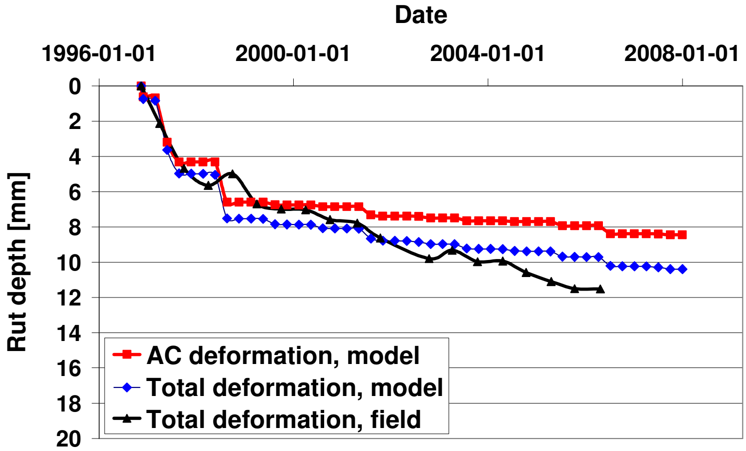


**Figure 4-8. CalME rut depth in the flexible test section using WesTrack permanent deformation parameters (Paper III).**

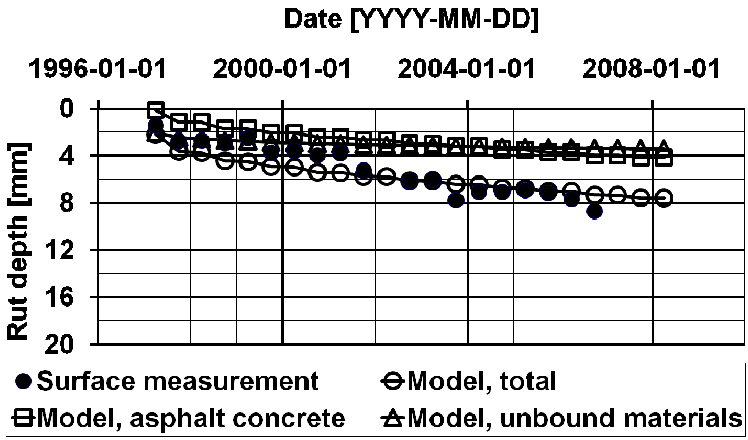


**Figure 4-9. M-E PDG rut depth in the semi-rigid test section using national calibration at level 3 (Paper II).**

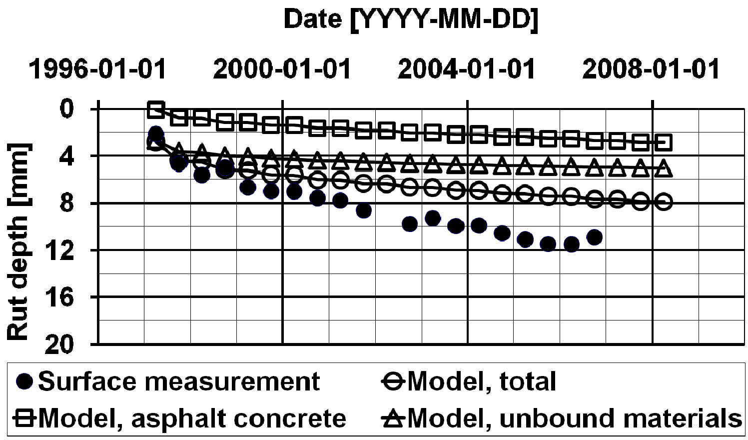


**Figure 4-10. M-E PDG rut depth in the flexible test section using national calibration at level 3 (Paper II).**

# 4.3 Distress modeling

## 4.3.1 Model mechanisms

Permanent deformation modeling should ideally reproduce the mechanisms identified, i.e. initial densification but mainly flow rutting throughout pavement life, as described in Section 1.1.3. These mechanisms account for the primary and secondary permanent deformation stages. Tertiary stage modeling is not included, since it is considered difficult to assess. In addition, tertiary stage permanent deformation generally results in larger rut depth than would be acceptable in practice. Therefore, modeling the primary and secondary stages is considered sufficient (NCHRP, 2004). These two stages are modeled with one single function by M-E PDG (NCHRP, 2004) and CalME (Ullidtz et al., 2006b). The PEDRO model offers an advantage by modeling the densification and flow rutting mechanisms separately in the response model. This feature of PEDRO requires that Poisson's ratio can be assessed. The resulting permanent deformation curve shapes from the models evaluated are similar, as shown in Figure 4-11. The fast development of the primary stage is followed by constant rate rutting that continues through the remaining pavement life.

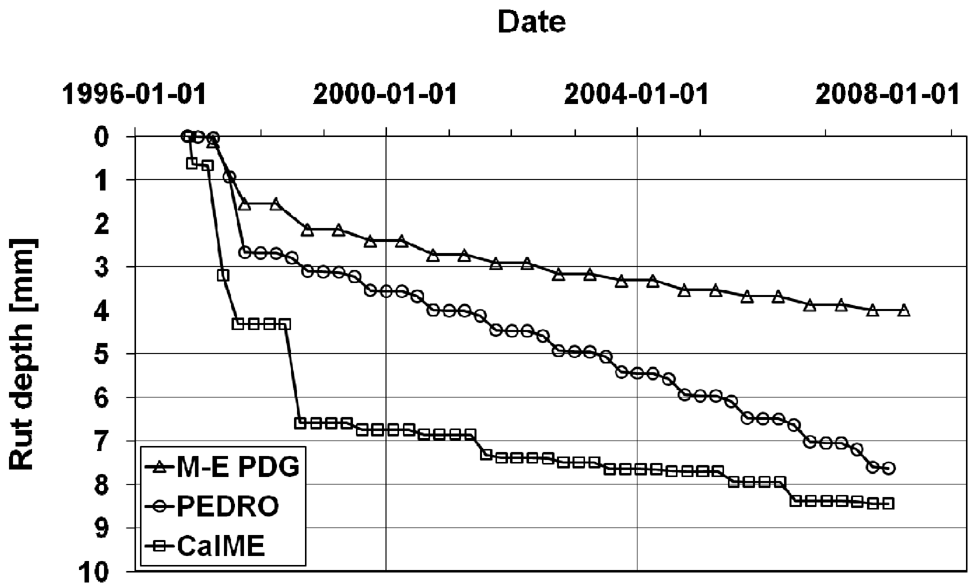


**Figure 4-11. Principal model curve shape for permanent deformation in asphalt concrete layers in the flexible test section.**

## 4.3.2 Model versatility

The distress model should account for permanent deformation behavior that is not fully modeled in the response model. One increasingly important method to reduce permanent deformation in asphalt concrete layers is polymer modification. Polymer addition can mitigate permanent deformation by increasing overall stiffness or rebalancing the elastic and viscous strain components (Read & Whiteoak, 2003), i.e. reducing the phase angle. Permanent deformation models should ideally account for both these effects of polymer modification. The M-E PDG elastic response model takes the dynamic modulus into account. However, the phase angle is not addressed in the pre-determined distress function (Paper I). Similarly, CalME accounts for the dynamic modulus in the response model. The parameters of the CalME distress function are obtained for each asphalt mix type using the RSST-CH (Paper III). If the RSST-CH can be considered representative of permanent deformation in the field, this procedure will account for phase angle changes due to polymer modification. The PEDRO approach is different in that the zero shear rate viscosity assessment considers both dynamic modulus and phase angle (Papers IV and V).

## 4.4 Rut depth interpretation

The models evaluated generally accumulate the permanent strain over the total asphalt concrete layer depth by numerical integration using sublayers. However, the M-E PDG and CalME models adjust the depth distribution of permanent deformation empirically. M-E PDG employs an empirical function, the so-called "$k_1$ factor", that concentrates the permanent strain to the top 100 mm of asphalt concrete according to observations from seven sections on the MnRoad pavement test track (NCHRP, 2004), as shown in Figure 4-12 (Paper I). CalME achieves a similar effect by simply restricting the permanent strain accumulation in asphalt concrete layers to the top 100 mm (Ullidtz et al., 2006a; Ullidtz et al., 2006b). The PEDRO permanent deformation distribution over depth, which does not include any empirical correction, is very similar to that of the M-E PDG shown in Figure 4-12 (Paper V). This behavior is supported by previous studies, in which very small permanent deformations in asphalt concrete layers have been found below 100 mm depth (Brown & Cross, 1992; Harvey & Popescu, 2000; NCHRP, 2004). The pavement sections subjected to HVS testing in Paper I, however, showed that approximately one third of all permanent deformations occurred in the asphalt concrete base (ACB) layer located below 117 mm. This could be related to the standard selection of a soft binder with penetration 160/220 in the ACB layer. However, the unrealistically constant temperature, i.e. lack of a temperature gradient, produced an artificial depth distribution of permanent deformation in the HVS test. Therefore, Paper I results cannot be used to conclude that significant permanent deformation occurs at depths larger than 100 mm. In conclusion, the effect of the M-E PDG and CalME depth limitations appears reasonable. Nevertheless, a mechanistic-empirical methodology should ideally not be restricted by any purely empirical depth limitations.

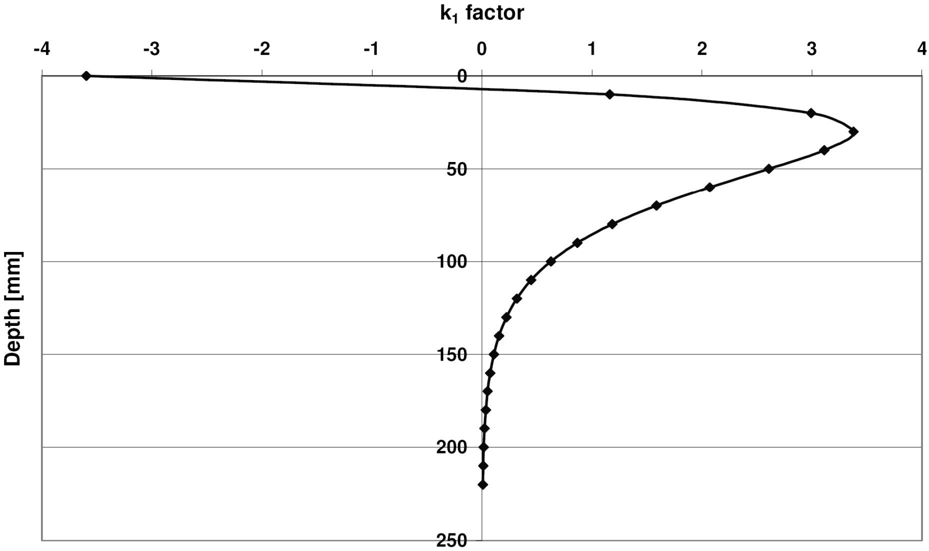


**Figure 4-12. The $k_1$ factor as a function of depth in a 220 mm asphalt concrete pavement (Paper I).**

# 4.5 Field calibration

The models evaluated were field calibrated in order to remove any model bias by empirically adjusting model results to observation. The placement, number and magnitude of the calibration factors can be considered measures of the importance of empirical considerations in a model. Calibration will typically increase the model's accuracy when applied on pavement sections similar to the calibration sections, and the calibration efforts carried out in this thesis were limited. Therefore, calibration factors intended for general use should include a wider range of pavement sections constructed with different structures and materials, and subjected to various traffic loads and climates.

The M-E PDG field calibration effort using HVS testing carried out in Paper I showed that bias can effectively be eliminated. The M-E PDG asphalt concrete permanent deformation model employed at input data quality level 1 was assigned the calibration factors $\beta_{r1}$ equal to 0.30 or 0.50, $\beta_{r2}$ equal to 1.50 or 1.45, and $\beta_{r3}$ equal to 0.70 or 0.60 for the standard and enhanced pavement structures, respectively. The low magnitude of the overall linear $\beta_{r1}$ calibration factor may have been caused by the use of specimens produced by coring instead of gyratory compaction, as discussed in Section 4.1.1. The $\beta_{r2}$ factor increased the modeled temperature susceptibility, and the $\beta_{r3}$ factor made the model less dependent on the number of load repetitions (Paper I). This may be the result of differences in material, structure and climate as compared to the pavements for which M-E PDG is calibrated. However, the multiple calibration factors integrated in the distress model can also effectively conceal model inconsistencies and deviations from the prescribed input data assessment methods. For example, the testing of cored field specimens instead of the prescribed gyratory compacted specimens in Paper I may have resulted in a lower dynamic modulus and therefore more permanent deformation. Still, the calibration can effectively adjust model results to observation, and the calibration factors in Paper I were therefore evaluated in Paper II. The calibration factors for standard structures, i.e. $\beta_{r1} = 0.30$, $\beta_{r2} = 1.50$ and $\beta_{r3} = 0.70$ was found to adjust model results to observation as well as with national calibration, i.e. $\beta_{r1} = \beta_{r1} = \beta_{r1} = 1$. NCHRP (2004) recommends that the M-E PDG be regionally calibrated. Therefore, it would be interesting to further evaluate the regional calibration factors found in this study.

The M-E PDG model employed using asphalt concrete input data at level 3 produced reasonable results, while level 1 over-predicted the permanent deformation. All other data were input at the highest possible level with respect to the field section documentation available (Paper II). The systematic difference between the levels was a factor of approximately 2.9 and 3.1 in the flexible and semi-rigid sections, respectively, as shown in Figure 4-13 based on modeling using national calibration in Paper II. The suggested cause for the difference between levels 1 and 3 is the over-prediction by the predictive equation for dynamic modulus, as discussed in Section 4.1.3. The dynamic modulus is the main asphalt concrete property, which is measured at level 1 and predicted at level 3. The ratio between the level 3 and level 1 dynamic modulus ranged from 1.95 to 3.52, as shown in Figure 4-2 and Figure 4-3. Therefore, the over-prediction of the predictive equation for dynamic modulus at level 3 appears to be the main cause for the difference in modeled permanent deformation at level 1 and 3 (Paper II). The M-E PDG is currently calibrated at level 3 according to its documentation in Appendix EE (NCHRP, 2004). Therefore, level 3 is the input quality level currently preferred. Further development is recommended to include harmonization of the levels and a calibration effort at level 1 or 2 (Paper II).

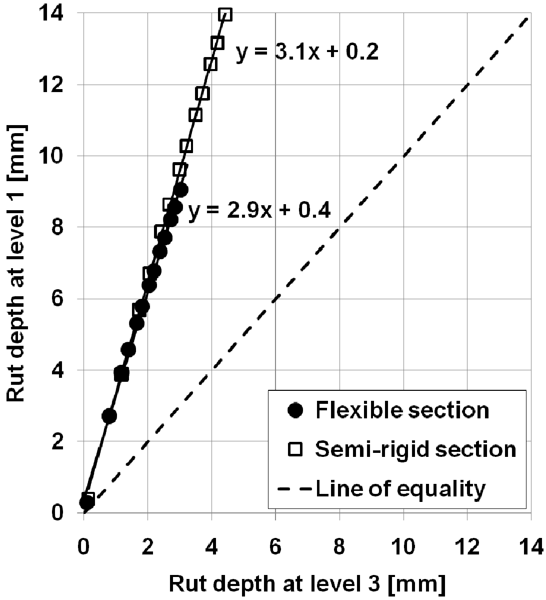


**Figure 4-13. Comparison between M-E PDG results at level 1 and 3 using national calibration.**

No changes in field calibration were carried out for CalME in Paper III, since the model produced reasonable results for the flexible pavement section using default data. However, CalME seriously underestimated the asphalt concrete permanent deformation in the semi-rigid section, which is most likely due to the sensitivity of the shear response model to the overall pavement stiffness, as discussed in Section 4.2.2. Consequently, further calibration using a single calibration factor would not fit both the flexible and semi-rigid sections in Paper III.

The PEDRO model was successfully calibrated using accelerated pavement testing (APT) and field observations. The temperature-controlled calibration factors found in Paper IV generally increased the modeled permanent deformation in order to correlate with the APT results. In contrast, the calibration performed in Paper V using a single calibration factor reduced the modeled permanent deformation results to field observation. The calibration factor was 0.7 for both the densification and the flow rutting mechanisms in the semi-rigid section. The calibration factors for the flexible section were 2 for densification and 0.7 for flow rutting. The model should be further evaluated using the 0.7 calibration factor. However, since the PEDRO calibration results were based on surface rut depth data that was adjusted for the wear and structural rutting modeled with the CalME and M-E PDG models, the PEDRO model calibration factors should be used with caution.

# 5 Conclusions and recommendations

The objective of this thesis was to evaluate the M-E PDG, CalME and PEDRO permanent deformation models for asphalt concrete layers under Swedish conditions. It has been shown that each model is composed of a collection of input data assessment methods, adjustments of input data, model elements, and calibration. Each model approach was shown to be characterized by both useful features and limitations. The models focus on the primary and secondary permanent deformation stages.

The results in this thesis were based on a limited number of pavement sections and limited to Swedish conditions with respect to traffic, climate and materials. The input data evaluation focused on the material properties since the input data on traffic characteristics and climate factors were identical in the models evaluated. This thesis did not include any comprehensive sensitivity analysis. However, it would be interesting to further study model behavior using a wide range of traffic, climatic and material input data. All models generally produced reasonable permanent deformation results although further validation and calibration is recommended before employment for pavement design purposes in Sweden.

## 5.1 The M-E PDG model

The M-E PDG can be employed at three hierarchical levels of input data quality. Each level corresponds to the effort of obtaining the input data. The main M-E PDG material parameter, dynamic modulus, was successfully measured at input data quality level 1 using indirect tensile testing of field-cored specimens. However, field cores required a correction due to the normally higher dynamic modulus of the prescribed gyratory compacted specimens. The predicted dynamic modulus at level 3 was more than twice as large as that measured at level 1 in this work. Therefore, the predictive dynamic modulus should be further developed in order to be a useful tool for pavement design. The M-E PDG incrementally adjusts the dynamic modulus throughout pavement life according to ageing and densification, while damage is not taken into consideration. It would be interesting to study the effects of ageing, densification and damage on dynamic modulus separately. The M-E PDG does not employ destructive testing data or phase angle data from the modeled asphalt materials.

The M-E PDG permanent deformation model employed at input data quality level 3 generally produced results in accordance with field observation. However, the modeled permanent deformation was significantly larger at level 1 than at level 3. The main cause suggested is the previously shown inaccuracy of the predictive dynamic modulus equation at level 3. However, the model is calibrated using level 3 input data, which implies that this should be the level currently preferred. It is recommended that the predictive dynamic modulus equation be further developed in order to increase its accuracy and that calibration at level 1 or level 2 be carried out. The regional M-E PDG model calibration effort carried out at level 1 using full-scale accelerated testing resulted in an increase in temperature susceptibility and a reduced susceptibility to the number of loadings. However, modeling permanent deformation in the motorway sections displayed no significant effect of the regional calibration as compared to national calibration. It would be interesting to further evaluate the regional calibration factors found in this study. Calibration and other empirical consideration can effectively adjust the results of the model to observation. However, it would be interesting to achieve the required model behavior using a higher degree of mechanistic model elements. The M-E PDG model empirically places the main portion of permanent deformation in asphalt concrete in the upper 100 mm in accordance with field studies and the other models evaluated in this study. It would be interesting to achieve the depth distribution mechanistically rather than empirically.

## 5.2 The CalME model

The dynamic modulus input data were successfully determined by indirect tensile testing using field-cored specimens. The incremental CalME dynamic modulus adjustment is achieved by an increasing empirical ageing and densification function as well as a decreasing damage function. The overall CalME dynamic modulus adjustment resulted in a continuous decrease, although a long-term pavement performance (LTPP) study showed an increase. Inclusion of the recursive damage model is an advantage, even though the overall dynamic modulus adjustments did not correspond to the Swedish field data in this study. The asphalt concrete stiffness models for ageing, densification and damage should be further developed and calibrated, and CalME would benefit from expansion of the materials library. The inelastic material properties of each asphalt concrete material were tested using the destructive repeated simple shear test at constant height (RSST-CH). The RSST-CH results and CalME distress modeling results indicated that the relation between elastic and plastic properties may vary throughout pavement life. Therefore, further investigation is recommended since the elastic-plastic relation is currently assumed to be constant.

The CalME response modeling appeared sensitive to overall pavement stiffness, as shown by comparing a flexible and a semi-rigid pavement section. It may be difficult to relate shear stress and elastic shear strain to flow rutting in very stiff pavement sections. CalME is currently calibrated using flexible pavements only. Further calibration comprising both flexible and semi-rigid sections would require using more than a single calibration factor. The calibrated model produced reasonable results using the default WesTrack permanent deformation parameters. The CalME model empirically distributes the permanent deformation in asphalt concrete in the upper 100 mm in accordance with field studies and the other two models evaluated in this study. However, it would be interesting to model the depth distribution of permanent deformation mechanistically rather than empirically.

## 5.3 The PEDRO model

The PEDRO material input data were based on the dynamic shear modulus testing using field-cored specimens. The model adjusts the dynamic modulus throughout pavement life according to ageing and densification. The main PEDRO material property, zero shear rate viscosity (ZSV), was calculated using a new method based on the measured dynamic shear modulus and the corresponding phase angle. This model does not require destructive material testing, because all permanent deformation is attributed to the linear viscoelastic Burger element. The ZSV assessment method presented ranked the asphalt concrete materials correctly according to expectations based on dynamic modulus and phase angle. The assessed ZSV was shown to be highly sensitive to temperature. The PEDRO permanent deformation modeling indicated that this method may underestimate the ZSV. Therefore, further validation and research on the ZSV assessment method is recommended.

The PEDRO model produced reasonable permanent deformation profiles using assessed traffic lateral wander. However, the modeled surface profile and the resulting rut depth were sensitive to the distribution of lateral wander. Therefore, further research on lateral wander assessment methods is recommended. The PEDRO model was calibrated using both accelerated pavement testing and field observation. The model results were generally increased or decreased to correspond to the results from accelerated pavement testing and field observation, respectively. The ambiguous calibration results should be further validated. The PEDRO densification and material flow models place the main portion of permanent deformation in asphalt concrete in the upper 100 mm in accordance with field studies. One advantage offered by PEDRO is that the depth distribution was achieved without using empirical considerations.

# 6 Acknowledgements

The author gratefully acknowledges the support given in terms of financing, supervision, research data and equipment, discussions, and moral support. This work was funded by the Swedish Transport Administration (Trafikverket), which was formerly called Swedish National Road Administration (Vägverket), and the Development Fund of the Swedish Construction Industry (SBUF). The project management was provided by Skanska Teknik, which is my employer. The work was carried out at the Department of Technology and Society at Lund University in collaboration with the Swedish National Road and Transport Research Institute (VTI).

I would like to express my gratitude to my supervisor Dr. Monica Berntman for all her guidance and good advice. My special thanks go to my co-supervisor Dr. Safwat Said at the Swedish National Road and Transport Research Institute (VTI) for sharing his invaluable knowledge and experience, and to Dr. Richard Nilsson at Skanska Teknik for his helpfulness in providing theoretical and practical support.

This thesis relied heavily on access to research data and laboratory facilities. The Swedish National Road and Transport Research Institute (VTI) provided data from the SE09 Heavy Vehicle Simulator (HVS) test, field measurements and laboratory testing, as well as access to laboratory equipment. In addition, the access to laboratory facilities and support at the Skanska Teknik laboratory in Malmö and at the Pavement Research Center at the University of California is also gratefully acknowledged.

The work has benefited from continuous discussions with professionals from various organizations. I would like to thank Jan Hansson at Skanska Teknik and Sven Agardh at Sweco for our fruitful discussions. My discussions with John Harvey, Per Ullidtz, Rongzong Wu, Erik Denneman and Frank Farshidi during my stay at the Pavement Research Center at the University of California (UCPRC) were also very rewarding. The technical support given by Leif G. Wiman, Hassan Hakim, Andreas Waldemarsson and their colleagues at the Swedish National Road and Transport Research Institute (VTI) is also gratefully acknowledged. The comments given at the final seminar by Robert Karlsson at VTI, Roger Nilsson at Skanska Teknik och Projekteringsledning and Christian Busch at Grontmij Carl Bro were very clarifying. Other contributors I wish to thank are Kenneth Olsson at Skanska Teknik, and Jesper Elsander, Anders Huvstig and Carl-Gösta Enocksson at the Swedish Transport Administration (Trafikverket) for their advice.

Finally, I want to thank those people who have given me moral support during the completion of what I consider to be the most difficult and time-consuming task of my life to this point. The encouraging and welcoming atmosphere at the Department of Technology and Society at Lund University, the Skanska Teknik laboratories and the Pavement Research Center at the University of California have kept my spirit high. I wish to thank family and friends for their invaluable support and encouragement.

Erik Oscarsson

*Lund, February 2011*

# 7 References


AASHTO TP62-03 (2006). AASHTO Provisional Standards, 2006 Edition, Standard TP62-03: Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures. American Association of State Highway and Transportation Officials, Washington D.C.

AASHTO T320-03 (2006). AASHTO Provisional Standards, 2006 Edition, Standard T320-03: Standard Method of Test for Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures using the Superpave Shear Tester (SST). American Association of State Highway and Transportation Officials, Washington D.C.

Airey, G.D., Collop, A.C., Zoorob, S.E., Hunter, A.E. (2006). Comparison of Field and Laboratory Compacted Asphalt Mixtures. 10th International conference on asphalt pavements, ISAP, Quebec.

Al-Qadi I.L., Elseifi M.A., Jun Yoo P., Dessouky S.H., Gibson N., Harman T., D'Angelo J., Petros K. (2009). Accuracy of Current Complex Modulus Selection Procedure from Vehicular Load Pulse: NCHRP Project 1-37A Mechanistic-Empirical Pavement Design Guide. Transportation Research Record Vol. 2087, pp. 81-90.

AMADEUS (2000). Advanced Models for Analytical Design of European Pavement Structures, Final report RO-97-SC.2137. European Commission, 2000.

Anderson D.A., Le Hir Y.M., Planche J.-P., Martin D., Shenoy A. (2007). Zero Shear Viscosity of Asphalt Binders. Transportation Research Record, Vol. 1810, pp. 54-62.

Azari H., Al-Khateeb G., Shenoy A., Gibson N. (2007). Comparison of Simple Performance Test E* of Accelerated Loading Facility Mixtures and Prediction E*, Transportation Research Record, Vol. 1998, pp. 1-9.

Björklund, A. (1984). Creep induced behaviour of resurfaced pavements: Some studies with the aid of beams subjected to a temperature wave and loaded under plane strain conditions. Swedish National Road & Traffic Research Institute (VTI), Report 271A.

Bonaquist, R. (1992). An Assessment of the Increased Damage Potential of Wide Based Single Tires. Proceedings of the 7th International Conference on Asphalt Pavements, Vol. 3, pp. 1-16.

Brown E. R., Cross S. A. (1992). A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements. National Center for Asphalt Technology, Report 92-5.

Buiter R., Cortenraad W.M.H., van Eck A.C., Van Rij H. (1989). Effects of transverse distribution of heavy vehicles on thickness design of full-depth asphalt pavements. Transportation Research Record, Vol. 1227, pp. 66-74.

COST 333 (1999). Development of a new bituminous pavement design method. European commission, COST 333, Final report of the action.

Dongré R., Myers L., D'Angelo J., Paugh C., Gudimettla J. (2005). Field Evaluation of Witczak and Hirsch Models for Predicting Dynamic Modulus of Hot-Mix Asphalt, Journal of the Association of Asphalt Paving Technologists, Vol. 74, pp. 381-442.

EAPA (1995). Heavy duty pavements: The arguments for asphalt. European Asphalt Pavement Association.

EAPA (1998). Heavy duty surfaces: The arguments for SMA. European Asphalt Pavement Association.

Eisenmann, J., Hilmer, A. (1987). Influence of Wheel Load and Inflation Pressure on the Rutting Effect at Asphalt-Pavements - Experiments and Theoretical Investigations. Sixth International Conference on the Structural Design of Asphalt Pavements, Vol. I, Ann Arbor.

Elkins, G. E., Schmalzer, P., Thompson, T., and Simpson, A. (2006). Long-Term Pavement performance Information Management System Pavement Performance Database User Guide, Federal Highway Administration, Report No. FHWA-RD-03-088.

Harvey, J., Popescu, L. (2000) Rutting of Caltrans Asphalt Concrete and Asphalt-Rubber Hot Mix Under Different Wheels, Tires and Temperatures – Accelerated Pavement Testing Evaluation. Pavement Research Center, University of California, Draft report Prepared for California Department of Transportation

Karlstrand N. and J. Neander (2006). Efterpackningens effekt på bindlagret. Thesis, In Swedish, Linköping University.

Kim, Y.R., Seo, Y., King, M., Momen, M. (2004). Dynamic modulus testing of asphalt concrete in indirect tension mode. Transportation Research Record, Vol. 1891, pp. 163-173.

Larson G.E., Dempsey B.J. (2003). Enhanced integrated climate model (EICM) v3.0, Applied Research Associates, ERES Division & Department of Civil Engineering, University of Illinois.

Luo, R., Prozzi, J.A. (2005). Evaluation of the Joint Effect of Wheel Load and Tire Pressure on Pavement Performance. Texas Transportation Institute, Texas A&M University System, Report SWUTC/05/167245-1.

NCHRP (2004). Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. National Cooperative Highway Research Program, Report 1-37A.

NCHRP (2006). Changes to the Mechanistic-Empirical Pavement Design Guide Software Through Version 0.900, July 2006. National Cooperative Highway Research Program, Results Digest 308.

Newcomb, D.E., Buncher, M., Huddleston, I.J. (2001). Concepts of Perpetual Pavements. Transportation Research Board, Transportation Research Circular 503: Perpetual Bituminous Pavements.

Nilsson R.N., Isacsson U. (2001). Flexible pavement response as influenced by degree of compaction. In Nilsson R.N., Viscoelastic pavement response analysis using VEROAD. Doctoral dissertation, Royal Institute of Technology, Stockholm, 2001.

Oscarsson, E. (2007). Prediction of Permanent Deformations in Asphalt Concrete using the Mechanistic-Empirical Pavement Design Guide. Licentiate's Thesis, Department of Technology and Society, Lund University, Bulletin 236.

Pellinen, T.K. (2009). Complex modulus characterization of asphalt concrete. In: Kim Y.R. (eds.) Modeling of asphalt concrete, 1st Edition. McGraw-Hill Construction.

Read J., Whiteoak D. (2003). The Shell bitumen handbook 5th Edition, Shell UK Oil Products Limited, Thomas Telford Publishing. London.

Said S.F. (2005). Ageing Effect on Mechanical Characteristics of Bituminous Mixtures, Transportation Research Record 1901, pp. 1-9.

Sebaaly P.E. (2003). Determination of Pavement Damage From Super-single and Singled-out Dual Truck Tires. National Cooperative Highway Research Program, Report 1-36.

SIKA (2005). Prognoser för godstransporter år 2020. Statens institut för kommunikationsanalys. Report 2005:9.

SIKA (2009). SIKA Basfakta 2008: Övergripande statistik om transportsektorn. Statens institut för kommunikationsanalys, Report 2009:28.

SNRA (2005). Allmän teknisk beskrivning för vägkonstruktion (ATB Väg), In Swedish. Swedish Transport Administration (formerly Swedish National Road Administration), Report VV 2005:112.

Sousa J. B., Craus J., Monismith C.L. (1991). Summary report on permanent deformation in asphalt concrete. Strategic Highway Research Program, Report SHRP-A/IR-91-104.

Sybilski D. (1996). Zero-Shear Viscosity of Bituminous Binder and Its Relation to Bituminous Mixture's Rutting Resistance, Transportation Research Record, Vol. 1535, pp. 15-21.

Yu, H.T. (2009). Updates on Mechanistic Empirical Pavement Design Guide (MEPDG), The Project Development Conference in Boise. The Idaho Transportation Department.

UCPRC (2008). Software documentation for CalME v0.8. Pavement Research Center, University of California.

Ullidtz P., Harvey J., Tsai B.-W., Monismith C. (2006a). Calibration of Incremental-Recursive Flexible Damage Models in CalME Using HVS Experiments, Pavement Research Center, University of California, Report UCPRC-RR-2005-06.

Ullidtz P., Harvey J., Tsai B.-W., Monismith C. (2006b). Calibration of CalME models using WesTrack Performance Data, Pavement Research Center, University of California, Report UCPRC-RR-2006-14.

Verstraeten, J. (1995). Bituminous materials with a high resistance to flow rutting. PIARC technical committee on flexible roads.

Wiman, L.G., Hermelin, K., Ydrevik, K., Hultqvist, B.-Å., Carlsson, B., Viman, L., Eriksson, L. (2002). Prov med olika överbyggnadstyper: Observationssträckor på väg E6, Fastarp-Heberg, Resultatrapport efter 5 års uppföljning, 1996-2001. The Swedish National Road & Transport Research Institute (VTI), Report 52-2002, In Swedish.

Wiman L.G. (2005). VTT/VTI. Presentation at the Heavy Vehicle Simulator International Alliance (HVSIA) Meeting in August 2005, Pretoria, South Africa.

Wiman L.G., Carlsson H, Viman L, Hultqvist B-Å. (2009). Prov med olika överbyggnadstyper. Uppföljning av observationssträckor på väg E6 Fastarp-Heberg, 10-årsrapport, 1996-2006. The Swedish National Road & Transport Research Institute, Report 632-2009, In Swedish.

Zhou, F., Scullion, T., Sun, L. (2004). Verification and Modeling of Three-Stage Permanent Deformation Behavior of Asphalt Mixes. Journal of Transportation Engineering, Vol. 130, No. 4, pp. 486-494.