Performance Modelling of Flexible Pavements Tested in a Heavy Vehicle Simulator

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Faculty of Civil and Environmental Engineering
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Dissertation submitted in partial fulfilment of a
Philosophiae Doctor Degree in Civil Engineering

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Abstract

Due to the complex behaviour of pavements most traditional pavement design is done with empirical methods that are based on long-term experience. Due to their simplicity, one of their limitations is that they cannot be extrapolated with confidence beyond the conditions on which they are based. New mechanistic designing methods are being developed to predict the behaviour of road structures. The behaviour depends on many factors such as the applied loads, the materials used, the thickness of the layers and the environmental conditions. One of the main limitations today is the influence environmental conditions (such as temperature, frost/thaw variations and moisture) have on the materials and deterioration of road structures, but these factors are very important for thin flexible pavements common in the Northern Periphery. To model the behaviour adequately it is important to compare the results of mechanistic (numerical) analyses with actual measurements in a full scale structures. Here accelerated pavement tests (APT), where instrumented pavement structures were tested using a heavy vehicle simulator (HVS), were used to examine the influence increased moisture content has on road structures and the accuracy of repeated tests was estimated.

Keywords:

Accelerated pavement testing, heavy vehicle simulator, unbound granular materials, performance prediction, pavement response, permanent deformation, water impact.

Útdráttur

Aðferðir við burðarþolshönnun vega hafa til langs tíma byggt á reynslu. Hönnunin er, þrátt fyrir einfaldleikann, takmörkuð og niðurstöðurnar einhæfar og ógegnsæjar og því erfitt að aðlaga hana að óvenjulegum og nýjum aðstæðum. Víða er verið að þróa nýjar aflfræðilegar hönnunaraðferðir sem spá fyrir um niðurbrotshegðun vega. Hegðunin er háð mörgum ólíkum þáttum svo sem álagi, efnisvali, þykkt laga og umhverfisþáttum. Í dag er helsti veikleiki aflfræðilegra hönnunaraðferða takmörkuð þekking á áhrifum ýmissa umhverfisþátta, einkum hitastigs, frosts/þíðu skipta og raka, á efniseiginleika mismunandi laga vegarins og hver tengsl niðurbrot vegarins og umhverfisþáttanna eru. En þessir þættir eru mjög mikilvægir þegar verið er að skoða þunnar vegbyggingar sem eru mjög algengar á norðurslóðum. Aflfræðileg hönnunaraðferð í vegagerð byggir á því að aflfræði sé beitt við að ákvarða svörun vegbygginga við mismunandi hjólaálagi í ákveðnu loftslagi og síðan er beitt reynslusamböndum er byggja á tilraunaniðurstöðum við að ákvarða áhrif álagspúlsanna á vegbygginguna (þ.e. hrörnun vegbyggingarinnar) sem fall af tíma. Þess vegna er mikilvægt að bera saman niðurstöður reikninga við mælingar gerðar í vegbyggingum í fullri stærð. Hröðuð álagspróf (APT) þar sem vegbygging var prófuð með þungum bílhermi (HVS) og svörun mæld sem fall af tíma var notuð til að kanna og greina hvaða áhrif þungumferð og aukin raki hefðu á vegbygginguna. Nákvæmni endurtekinna prófanna var einnig áætlaður.

Lykilhugtök:

Hröðuð álagspróf, þungur bílhermir, óbundin malarefni, frammistöðumat vega, svörun vegbygginga, niðurbeygjumyndun vegbygginga, áhrif vatns.

List of publications

This dissertation is based on the following publications:

Paper I:

Saevarsdottir, Th. & Erlingsson, S. (2012). Evaluation of Flexible Pavement Structure in an Accelerated Pavement Test. In Harvey, J., Jones, D., Mateos, A. & Al-Qadi, I. (eds.) *Advances in Pavement Design Through Full-scale Accelerated Pavement Testing*. Taylor & Francis Group, London, pp. 237-248.

Paper II:

Saevarsdottir, Th., Erlingsson, S. & Carlsson, H. (2014). Instrumentation and performance modelling of Heavy Vehicle Simulator Tests. Submitted to *International Journal of Pavement Engineering*.

Paper III:

Saevarsdottir, Th. & Erlingsson, S. (2013). Water impact on the behaviour of flexible pavement structures in an accelerated test. *Road Materials and Pavement Design*. 14/2, 256-277. doi:10.1080/14680629.2013.779308.

Paper IV:

Saevarsdottir, Th. & Erlingsson, S. (2013). Effect of moisture content on pavement behaviour in a heavy vehicle simulator test. *Road Materials and Pavement Design*. 14/1, Special Issue EATA 2013, 274-286. doi:10.1080/14680629.2013.774762.

Paper V:

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Peer reviewed conference publications not included in this dissertation:

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Saevarsdottir, Th. & Erlingsson, S. (2013). Response and permanent deformation modelling of an instrumented flexible pavement structure. *The 9th International Conference on the Bearing Capacity of Roads, Railways and Airfields (BCRRA 2013), 25-27 June 2013*. Trondheim, Norway.

Saevarsdottir, Th., Erlingsson, S. & Carlsson, H. (2014). Heavy Vehicle Simulator Tests at VTI. *The International Society for Asphalt Pavements Conference (ISAP 2014), 1-5 June 2014*. Raleigh, North Carolina, USA.

To my children Anthony Sævar, Jóhanna Lilja and Alex Þór, my husband Phil, my parents Jóhanna Lilja and Sævar and my late grandparents Einar and Sigga for their support and belief in education.

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Abbreviations

3D – Three dimensional.
A – Maximum value of the failure ratio R.
a – Minimum of $\log(M_R/M_{Ropt})$ (equation 11).
a – Regression parameter (equation 21).
AASHTO – American Association of State Highway and Transportation Officials.
AC – Asphalt concrete.
APT – Accelerated Pavement Testing.
ASG – Asphalt Strain Gauges.
b – Maximum of $\log(M_R/M_{Ropt})$ (equation 11).
b – Regression parameter (equation 21).
BC – Base course.
c – Cohesion.
C – Material parameter.
d_{max} – Maximum diameter.
DP – Disruption potential.
E – Young's modulus / soil stiffness.
ε_0 – Material parameter.
E_{50} – Triaxial modulus.
E_{50}^{ref} – Secant stiffness in a standard drained triaxial test.
E_{oed} – Oedometer modulus.
E_{oed}^{ref} – Tangent stiffness for primary oedometer loading.
E_{ur} – Un-/reloading modulus.
E_{ur}^{ref} – Unloading / reloading stiffness ($E_{ur}^{ref} \cong 3E_{50}^{ref}$).
FE- Finite Element.
FWD – Falling Weight Deflectometer.
GWT – Ground Water Table.
h – Height.
HMA - Hot Mix Asphalt.
HVS – Heavy Vehicle Simulator.
k_1, k_2 and k_3 – Experimentally determined constants.
k_m – Regression parameter.
KT – Korkiala-Tanttu.
LVDT - Linear Variable Differential Transducers.
m – Power for stress-level dependency of stiffness.
M-E – Mechanistic Empirical.
MEPDG – Mechanistic Empirical Pavement Design Guide.
MLET – Multi Layer Elastic Theory.
MM – Measurements.
M_R – Resilient modulus.
M_{Ropt} – Resilient modulus at a reference condition.

N – Load repetitions.
N_{eq} – Equivalent number of load repetitions.
p – Normal stress ($p = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$.
p_a – Reference pressure (p_a = 100kPa).
q – Deviator stress ($q = (\sigma_1 - \sigma_3)$).
q_f – Deviator stress at failure ($q_f = (q_0 + m \cdot p)$).
R – Failure ratio ($R = q/q_f$).
RLT – Repeated Load Triaxial Test.
S – Degree of saturation at a given time.
Sb – Subbase.
Sg – Subgrade.
S_{opt} – Degree of saturation at a reference condition.
SPC – Soil Pressure Cell.
T&L – Tseng & Lytton
VTI – The Swedish National Road and Transport Research Institute.
w – Moisture content.
W_c – Water content.
z – Thickness.

β – Material parameter.
β_1 – Calibration factor.
Δt – Small time steps
$\hat{\delta}_p$ – Permanent deformation.
ε – Axial strain.
εMU – Strain Measuring Unit.
ε_p – Accumulated plastic strain.
ε_r – Recoverable (resilient) strain.
ε_v – Vertical strain.
θ – Bulk stress ($\theta = (\sigma_1 + 2\sigma_3)$).
v – Poisson's ratio.
ρ – Material parameter.
σ – Axial stress.
σ_1, σ_2 and σ_3 – Principal stresses.
σ_d – Deviator stress ($\sigma_d = \sigma_1 - \sigma_3$).
τ_{oct} – Octahedral shear stress ($\tau_{oct}^2 = \frac{1}{9}\left((\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2\right)$).
ϕ – Friction angle.

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1 Introduction

The purpose of pavements is to provide a safe and comfortable ride, and to be able to provide that its surface must be smooth (minimum distortions) and skid resistant. Pavements must have an adequate serviceability level over a reasonable time period with minimum deterioration caused by insufficient strength in the pavement layers and subgrade, increasing traffic and traffic loads as well as various environmental conditions. The most important mechanical properties for function and endurance of base and subbase layers of pavement structures, often made from unbound course aggregates, are their strength, stiffness and their ability to resist permanent deformations. These layers should provide strength, spread the applied load and protect the subgrade from extensive loading and deformations (Doré & Zubeck, 2009; Magnúsdóttir et al., 2002).

Due to the complex behaviour of pavements most traditional pavement design is done with empirical methods that are developed and based on long-term experience. The behaviour of pavements is complex and depends on many factors such as the axle/wheel loading configurations, the materials used, the thickness of the layers and the environmental conditions. The main limitation of empirical methods is that they cannot be extrapolated with confidence beyond the conditions on which they are based. For the development of mechanistic designing methods to proceed, the behaviour and properties of the materials to be used in the structure have to be properly understood as the models need to capture realistically the structural responses under various traffic loadings and environmental conditions. In thin pavements the granular base and subbase layers show a complex elasto-plastic behaviour under external loading. For a validation it is therefore important to compare the results of numerical analyses with actual measurements in full scale pavement structures to assure that the models applied capture the correct structural behaviour. The Accelerated Pavement Test (APT) is a test performed on full scale instrumented test roads where the magnitude and location of the applied loads, the number of load repetitions and the environmental conditions are controlled. At regular intervals condition surveys and pavement response measurements are performed, providing valuable data. The development of APT with instrumented pavement structures has increased the understanding of pavement behaviour and built a foundation for new, more sophisticated design methods (Nokes et al., 2012; du Plessis et al., 2006; Brown, 2004; Willis, 2008).

At the Swedish National Road and Transport Research Institute (VTI) test facility flexible road structures have been built and tested in an APT using a Heavy Vehicle Simulator (HVS) for the last 15 years. The objectives have been to investigate pavement responses and permanent deformation to understand pavement performance behaviour. This information can be used to validate mechanistic performance schemes. Two of these tests were performed at VTI's test facility in Linköping in 2005 (referred to as SE10) and 2009 (referred to as SE11) and are examined in this dissertation. The HVS machine is a linear full-scale accelerated road-testing machine with a heating/cooling system. The pavements were constructed by normal road construction machinery in a test pit that is 3m deep, 5m wide and 15m long, with the length of the tested structures 6m. The construction of the structures was substantially the same, with a 10 cm asphalt bound layer over a granular crushed base and subbase resting on a fine graded silty sand subgrade. The tests were performed under the same conditions in order to investigate the accuracy of repeated tests. Among other purposes, the aim was to get good direct measurements of stresses and strains in a thin pavement structure and an evaluation of the structure's performance under "moist" and "wet" conditions. The "moist" case simulates a standard situation where the groundwater table is at great depth with minor spatial moisture transfer and very limited evaporation, whilst the "wet" case simulates a worst case where the water is running in the trenches. About 1.1 million load cycles were applied, but after about half a million, the water table was raised giving the opportunity to estimate the influence of water on the structure's response and performance. The structures were instrumented to measure stress, strain and deflection responses as a function of load repetitions as well as permanent deformation manifested on the surface as rutting. A moving tyre load was applied using lateral wander with a dual wheel configuration, with a total axle load of 120kN and a tyre pressure of 800kPa. At regular intervals condition surveys and pavement response measurements were performed, providing valuable data (Wiman, 2006 & 2010).

The pavement structures were instrumented to measure their responses and performance: εMU coils to measure the vertical strain (elastic and permanent); soil pressure cells to obtain vertical stress; linear variable differential transducers to measure the vertical deflection in relation to the rigid bottom of the test pit; asphalt strain gauges to obtain the horizontal strain at the bottom of the asphalt bound layers; laser beam to measure the surface profile and moisture content sensors to measure the volumetric water content (Wiman, 2001 & 2006 & 2010).

As mentioned earlier there are many factors that must be considered when designing a flexible pavement that affects the structure's behaviour. These requirements are all easily within the capabilities of three dimensional (3D) finite element (FE) analysis but FE analyses are computationally expensive due to the inherent procedures involved. The required computation time may therefore be impractically long for a routine design (Schwartzs, 2002; Loulizi et al., 2006). Another method, multi-layer elastic theory (MLET), is commonly used in mechanistic empirical (M-E) pavement design where the response calculations are performed several times. MLET is an axisymmetric analysis that can be extended by the superposition principle for multiple wheel loads. Its main assumptions are that each layer is homogeneous and isotropic and the material is weightless and infinite in areal extent (Huang, 2004; Erlingsson & Ahmed, 2013; Erlingsson, 2007).

The response signals gained from the testing are compared with calculated values using MLET with a circular tyre imprint and a 3D FE program with a square tyre imprint. The accumulation of permanent deformation and the rutting profile were further modelled using two simple work hardening material models, one stress dependent developed by Korkiala-Tanttu (KT) (2008, 2009) and the other strain dependent as presented in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004). A "time-hardening" procedure was used to take into account the effects of various stress cases, such as changing axle loads and environmental conditions, on the development of rutting (Lytton et al., 1993; Hu et al., 2011; Ahmed & Erlingsson, 2012). The difference between these two methods was evaluated.

The response, performance and accumulation of permanent deformation have been modelled to demonstrate a typical data process. From the analysis a performance prediction can be carried out as a function of load repetitions to evaluate the performance of new road concepts and maintenance strategies. A life cycle cost analysis comparison between feasible designs can be done and used to decrease costs and environmental impact, and a maintenance scheme can be planned (Wiman & Erlingsson, 2008).

1.1 Objectives

The objectives of this work included several things. One was to estimate the influence increased moisture content had on flexible pavement structures and to model the influence. The responses, including vertical strain, vertical stress, tensile strain at the bottom of the asphalt layer and permanent deformation, were evaluated in both "moist" and "wet" states and a rutting profile drawn. The responses were gained by using a 3D FE analysis as well as the MLET procedure and the difference in the two methods was discussed. Accumulation of permanent deformation and the rutting profile were modelled using two simple work hardening material models, one stress dependent developed by Korkiala-Tanttu (KT) (2008, 2009) and the other strain dependent as presented in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004). An assessment of these two procedures was made. Two structures were tested under the same conditions and with the same test procedure to investigate the accuracy of repeated tests. The accuracy of the instrumentation used was also validated.

2 Background

Pavement structures are extensive linear structures built over various geomorphologic, geologic and climate environments. From the top down a pavement structure distributes the load from heavy vehicles to the subgrade soil, avoiding excessive deformations and controlling environmental effects on the structure's bearing capacity. From the bottom up the pavement structure should minimize environmental-related stresses and displacements, such as differential frost heave and post consolidation. The structure needs to fulfil its role to maintain good structural and functional performance (Doré & Zubeck, 2009).

In this dissertation flexible pavements were considered. Typical flexible pavements consist of: a 2.5-25 cm thick surface layer, which in most cases is an hot mix asphalt (HMA) (or asphalt concrete); a 10-30 cm thick base layer that typically consists of unbound (or unstabilized) aggregate material and a 15-50 cm thick subbase layer that generally is of "local" unbound aggregate material. The surface, base and subbase layers are sitting on the natural compacted soil or a filling called subgrade. The required thickness of each layer within the structure varies widely as it depends on many things such as the materials, the magnitude, number and configuration of the traffic load, the environmental conditions, and the desired service life of the pavement (Mamlouck, 2006; Huang, 2004; Doré & Zubeck, 2009).

In flexible pavements the granular base and subbase layers have a structural purpose in the overall pavement performance. The load is distributed over a small area at the surface and as the depth increases the load is distributed over a larger area causing the stress to decrease with depth. Therefore it is important to use better quality material closer to the surface compared to deeper in the structure. Every time a load is applied and removed on flexible pavements, it experiences a localized deformation and the pavement layers rebound. After each load application a very small amount of deformation can remain permanently and accumulate as more load applications are applied, causing the structure to deteriorate and fail its service criteria (Mamlouck, 2006; Lekarp et al., 2000a; Huang, 2004).

The stress pattern induced in a pavement due to a moving wheel load is complex and hard to simulate in a laboratory (Figure 2.1). In unbound layers, the vertical and horizontal stresses are positive while the shear stress is reversed as the load passes causing a rotation of the principal stress axes. Under traffic loading the granular pavement layers show a nonlinear and time-dependent elastoplastic response, and therefore the resilient response is normally defined by the resilient modulus, M_R. The resilient behaviour of unbound granular materials is affected by several factors such as stress level, density, grading and maximum grain size of the material, aggregate type and shape of the particles, where the effect of the stress parameters is the most significant (Lekarp et al., 2000a).

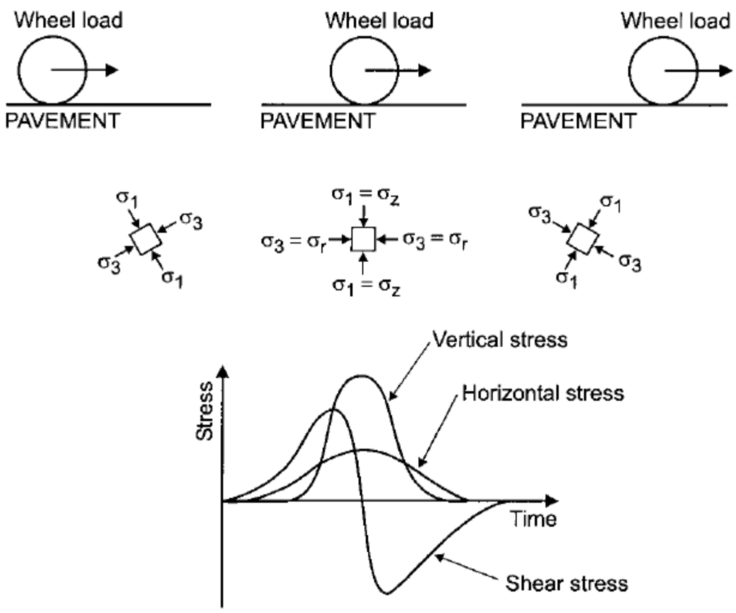

Figure 2.1 Stresses beneath a rolling wheel load (Lekarp et al., 2000a).

2.1 Mechanistic empirical prediction

Due to the complexity of the problem traditional pavement design is still largely done with empirical methods. As empirical methods are developed for certain types of pavement constructions under specific environmental conditions and with specific traffic loads applied the methods cannot be extrapolated with confidence beyond the conditions on which they are based. For decades researchers have been trying to combine the theoretical principles of soil mechanics and the simplicity required for routine analysis of material response in developing mechanistic designing methods.

To be able to develop mechanistic designing methods the behaviour and properties of the materials need to be properly understood under various environmental conditions, and the response of the granular layers (base and subbase) under various traffic loadings has to be taken into account and modelled realistically. In Figure 2.2 a simple flow chart of a mechanistic empirical performance prediction process is displayed (Ahmed & Erlingsson, 2013). The design life of the pavement is divided into small time steps, Δt, with specific material properties, climatic and traffic conditions. The input parameters are introduced into a response model that performs structural analysis of the stresses, strains and deformations occurring in the structure under the conditions applied within the time step. The calculated responses are thereafter input parameters to a performance prediction model which estimates the distresses occurring within the time step. The distresses are accumulated with time to obtain the performance history of the pavement. The loop is repeated until the incremented time step reaches the desired design life of the structure. New mechanistic designing methods give the designers the ability to choose between road sections and view their performance as a function of time. The designer can evaluate new road concepts and maintenance strategies by assessing the total cost of constructing, maintaining and operating different road constructions under various conditions. A life cycle cost assessment comparison between investment alternatives of feasible designs can then be performed to decrease costs and the environmental impact of road structures.

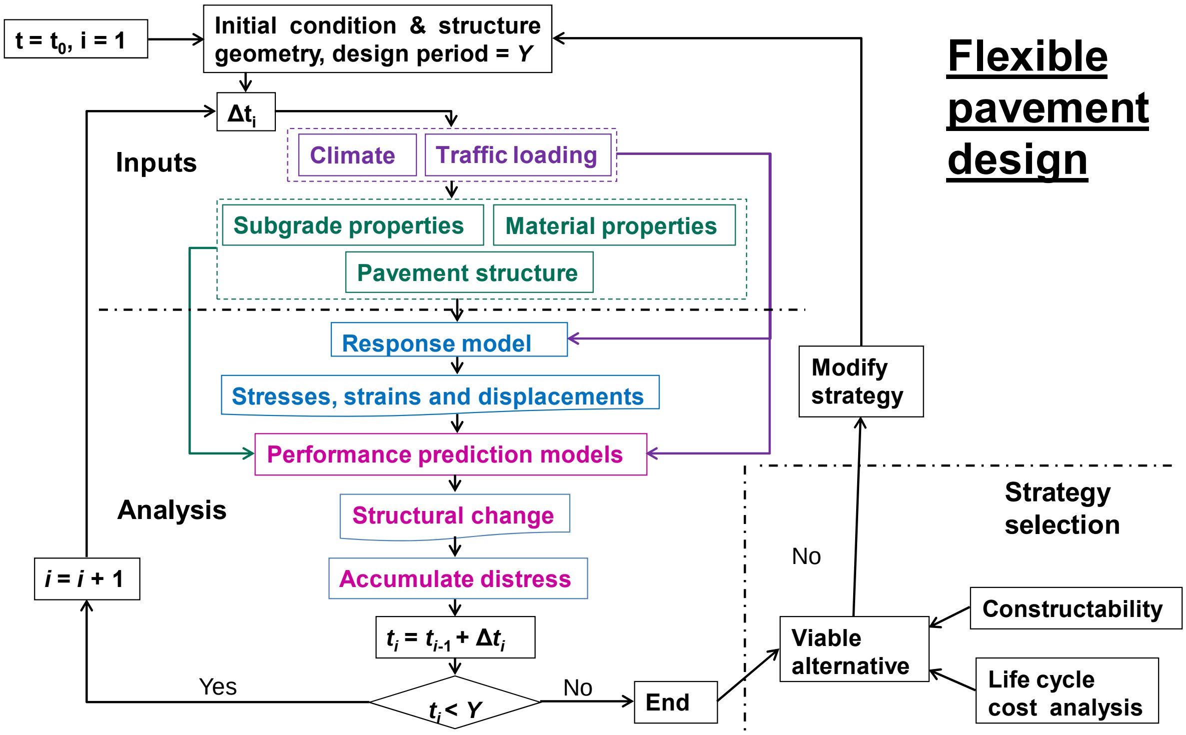

Figure 2.2 A simple flow chart of the Mechanistic Empirical Performance prediction process.

2.1.1 Climatic effects on pavement structures

It is a big task to design pavement structures and take into consideration different environment and climate conditions, especially when the conditions are hard to estimate. With a better understanding of the effect different conditions have on the structure the effect of constantly changing conditions would be easier to estimate. Environmental changes that have already taken place in the last few decades due to climate change include, for example: more fluctuating changes in precipitation and in most areas an increase in the number of cloudbursts, even in areas where total precipitation has decreased; warmer temperatures that have caused significant changes on the cryosphere, reduction of the snow cap in most places in the northern hemisphere with a thinner ice layer in permafrost areas and an increase in the thickness of the soil layer on top, thus greatly influencing soil stability and drainage; changed timing of the seasons as spring arrives earlier and autumn later. The future is hard to predict but it is believed that the process already started will continue (Doré & Zubeck, 2009; Björnsson, 2008; Mortimer et al., 2007). One of the main limitations today is lack of knowledge of the effect various environmental conditions have on the behaviour of pavement structures.

2.2 Accelerated Pavement Testing (APT)

With the complex elasto-plastic behaviour under external loading described earlier, it is important to compare the results of numerical analyses with actual measurements of stresses, strains and deflections in a full scale pavement structure. APT simulates the effects of long term in-service loading in a compressed time period. The APT is a test performed on full scale instrumented test roads, where the magnitude and location of the applied loads, the number of load repetitions and the environmental conditions are controlled. At regular intervals condition surveys and pavement response measurements are performed giving valuable validation data for the mechanistic designing methods. In most cases APT programs are compensated with other laboratory testing programs. The development of APT with instrumented pavement structures has increased the understanding of pavement behaviour but APT generates knowledge over a wide spectrum (Figure 2.3). APT has built a foundation for more sophisticated design methods (Nokes et al., 2012; du Plessis et al., 2008; NCHRP, 2004; Metcalf, 1996; Kumara, 2005; Korkiala-Tanttu, 2008). In this dissertation an HVS, an APT device, was used for testing instrumented flexible pavement structures (Paper II – Saevarsdottir et al., 2014).

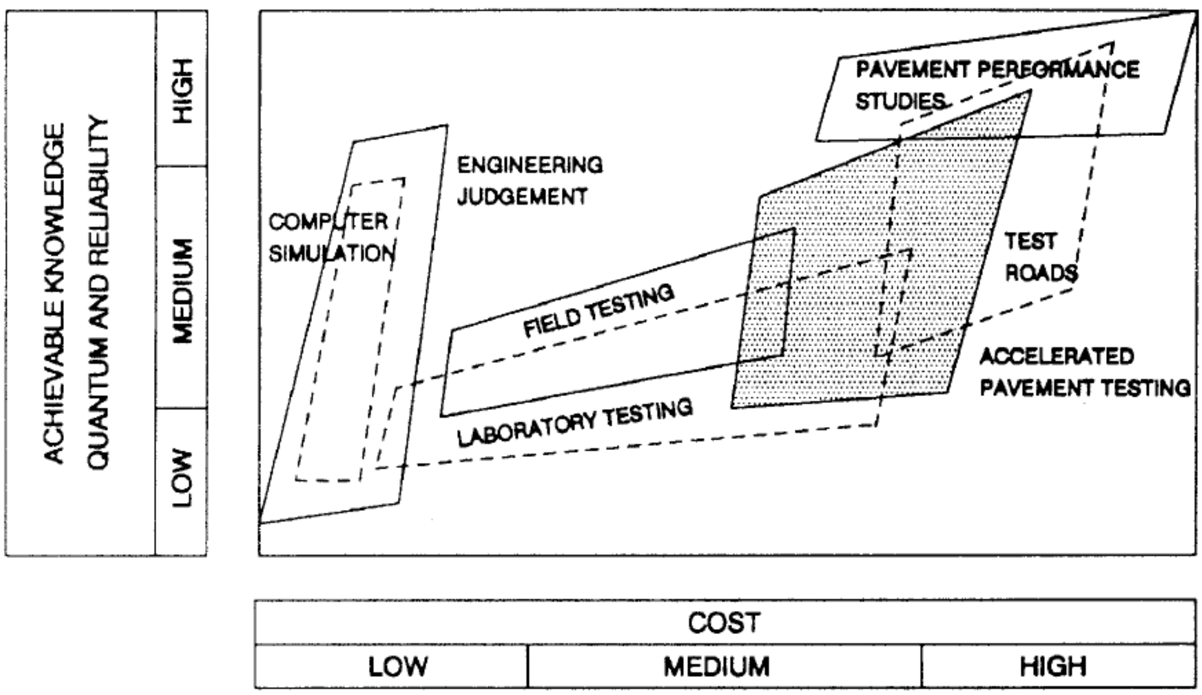

Figure 2.3 Interrelationship between pavement engineering facets that collectively and individually contribute to knowledge (NCHRP, 2004).

2.2.1 The Heavy Vehicle Simulator (HVS)

The HVS machine (Figure 2.4) operated at VTI (HVS Mark IV) is a mobile linear full-scale accelerated road-testing machine with a heating/cooling system to maintain a constant air and pavement temperature during testing. The HVS can be powered by diesel fuel as well as electricity and is therefore independent of external power. The unit is 23 m long, 3.7 m wide, 4.2 m high and weighs 50 tonnes, but its main technical characteristics are:

- Number of loadings up to 25000 per 24 hours (bi-directional) with a maximum speed of the loading wheel as 12 km/h.
- Dual and single loading wheels.
- Lateral wander of the centre of the loading wheel is up to 0.75 m.
- The load can be applied in one or both directions, with the wheel load varying between 30 and 110 kN.

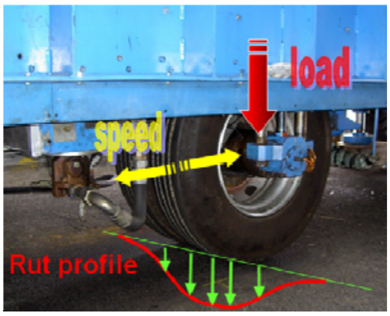

Figure 2.4 The HVS machine operated at VTI in Sweden.

Tested pavements, both in the field and at VTI's test facility, are constructed by normal road construction machinery. The VTI indoor full scale test facility has three test pits that are 3m deep, 5m wide and 15m long, but the length of the monitored structure is normally 6m (Wiman 2006, 2010; Wiman & Erlingsson, 2008).

2.2.2 Instrumentation of pavement structures in an APT

Pavement instrumentation is an important tool to monitor the in situ performance and the health of pavements (e.g. the pavement materials) and quantitatively measure pavement systems responses to various loading cases. Parameters that are often measured include strains, stresses, deflections, moisture and temperature. Various instruments (sensors) are available where the reliability, accuracy and cost vary significantly (Weinmann et al., 2004; NCHRP, 2004 & 2012).

It is desirable to have many sensors embedded in the test sections of full-scale pavement experiments. Too many sensors, however, may significantly disturb the quality of the soil they are to measure (e.g. density and moisture) and decrease the reliability of the test results. Special consideration is required when constructing instrumented test sections as misalignment and damage to the instrumentation from the construction process or premature failure of test sections due to inadequate compaction of instrumented test sections are known problems. The sensor installation requires extreme care so that the soil surrounding the sensors correctly represents the soil within the layer to be measured (Cortez & Janoo 2008; NCHRP 2004 & 2012).

In this dissertation the pavement structures were instrumented to measure their responses and performance: εMU coils to measure the vertical strain (elastic and permanent); soil pressure cells (SPC) to obtain vertical stress; linear variable differential transducers (LVDT) to measure the vertical deflection in relation to the rigid bottom of the test pit;

asphalt strain gauges (ASG) to obtain the horizontal strain at the bottom of the asphalt bound layers; a laser beam to measure the surface profile; and moisture content sensors in some structures to measure the volumetric water content (Wiman 2010 & 2006 & 2001).

2.2.3 Falling Weight Deflectometer (FWD)

In most cases APT programs are compensated with other laboratory testing equipment, and one of these tests is FWD. FWD is a widely used non-destructive testing device that is used to measure the mechanical response of various pavement structures under a given dynamic load intensity. FWD is used when performing structural testing for pavement rehabilitation projects, in various research projects, and for failure detection of pavement structures (Doré & Zubeck, 2009; Huang, 2004).

The device is commonly mounted on trailers or testing vehicles and includes a mechanical loading system, a falling weight, and a measurement system. The loading system induces load pulses that mimic the vertical loading component of a heavy single wheel travelling at highway speed. The haversine shaped load pulses are caused by a falling weight on a rubber buffer attached to a stiff loading plate. The deflection of the pavement surface caused by the load pulse is recorded by a set of geophones at an increasing distance from the loading plate. Two types of analyses can be carried out with the deflection data; an analysis based on deflection basin-shape indicators and backcalculation of a pavement layer module (Doré & Zubeck, 2009).

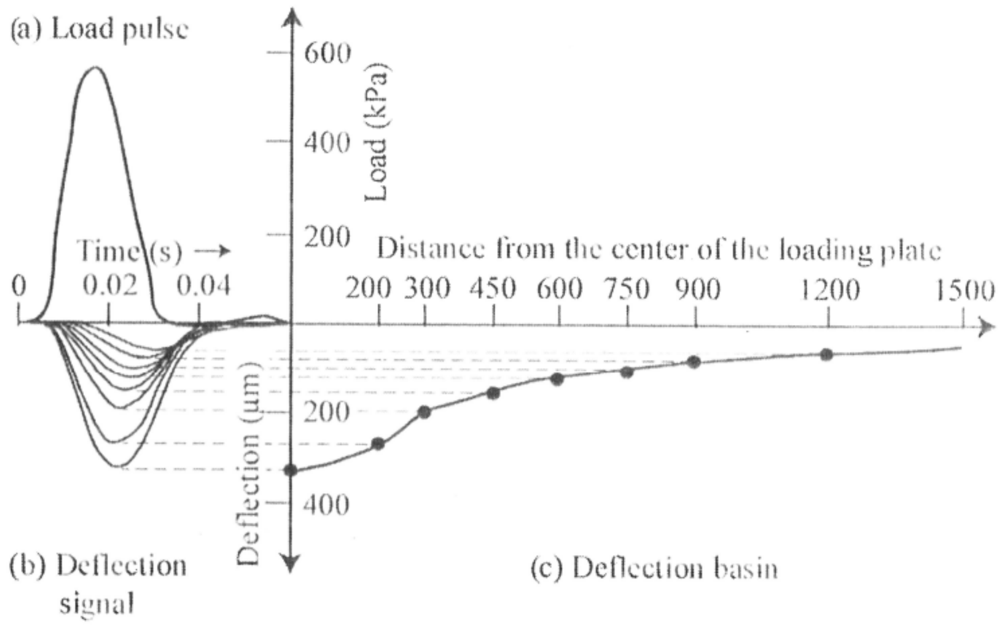

Figure 2.5 Falling weight deflectometer loading and related pavement response. (Doré & Zubeck, 2009).

2.3 Resilient response

2.3.1 Linear material model

In a linear elastic material the strain increases linearly in relation to the increase in stress. By using Hook's law deformations are assumed to be linear elastic, resulting in the deformation parameters to be independent of the stress level. This is the simplest form to describe the resilient deformation behaviour of unbound granular materials, e.g. as linear elastic. Young's modulus (or the modulus of elasticity), E, is found by using Hooke's law:

$$E = \frac{\sigma}{\varepsilon} \tag{1}$$

where σ is the axial stress and ε is the axial strain. It has been found experimentally that the ratio between the lateral and axial strain in a linear elastic material is a constant, known as Poisson's ratio, v (Ugural & Fenster, 2003; Kolisoja, 1997).

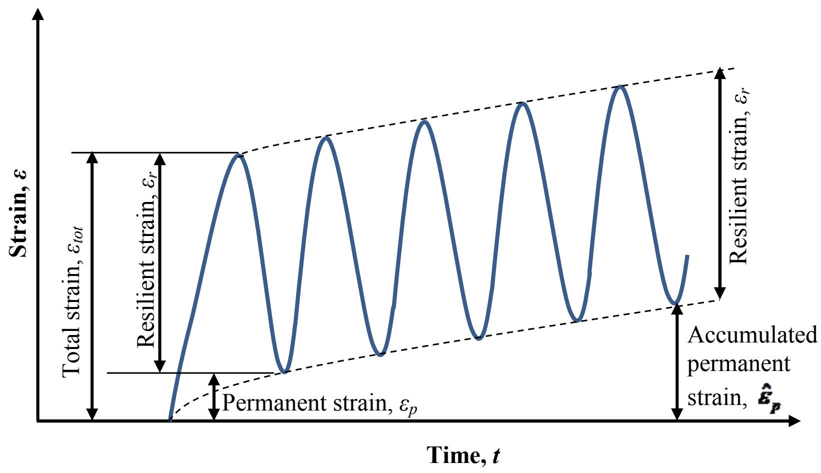

Figure 2.6 Strains under cyclic loading (Rahman, 2014).

Most paving materials are not elastic as the materials experiences some permanent deformation after each load application, e.g. granular layers show a nonlinear and time-dependent elasto-plastic response under traffic loading. If the load is small compared to the strength of the material and repeated often, the deformation under each individual load repetition is almost completely recoverable and is therefore often considered as elastic (Figure 2.6). Therefore, the resilient response of granular materials is usually defined by the resilient modulus (M_R) and Poisson's ratio (v). The resilient modulus, M_R, is an elastic modulus based on the recoverable strain under repeated loads, i.e. an elastic modulus to be used with the elastic theory (Huang, 2004; Lekarp et al. 2000a). The resilient modulus is defined as:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2}$$

where σ_d is the deviator stress (σ_1- σ_3) and ε_r is the recoverable strain.

For the stress-strain relationship of unbound granular materials, elastic material behaviour can be assumed by the generalized Hook's law (Erlingsson, 2007):

$$\sigma_{ij} = D_{ijkl}\varepsilon_{kl} \tag{3}$$

where σ_{ij} is the stress tensor, D_{ijkl} are the elastic constants and ε_{kl} is the elastic stress tensor. In a 2D axisymmetric analysis using cylindrical co-ordinates the elastic stress-strain relationship becomes (Erlingsson, 2007):

$$\begin{bmatrix} \sigma_r \\ \sigma_\theta \\ \sigma_z \\ \tau_{rz} \end{bmatrix} = \frac{M_r}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{bmatrix} \varepsilon_r \\ \varepsilon_\theta \\ \varepsilon_z \\ \gamma_{rz} \end{bmatrix} \tag{4}$$

2.3.2 Nonlinear material models

The resilient modulus, M_r, is a mechanical property of unbound pavement materials which strongly depends on the level of stress applied. Therefore the stress-strain relationship needs to be modelled as accurately as possible with constitutive laws. The problem is complex and it is therefore difficult to combine the theoretical principles of soil mechanics with the simplicity required. A number of measurements must be done to get a range of average normal stresses, $p = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$ where σ_1, σ_2 and σ_3 are principal stresses, and deviator stresses, $q = (\sigma_1 - \sigma_3)$, that different axles cause for an accurate estimate of a stress-strain relationship. The most common and simplest relationships to describe the stress dependency of the stiffness modulus are the k - θ expressions, where the base constitutive relationships is Huang's (2004) simple k-θ model that was first presented by Brown and Pell (1967) (May & Witczak, 1981; Uzan, 1985; Gomes-Correia et al., 1999; Kolisoja, 1997; Lekarp et al., 2000a):

$$M_r = k_1\theta^{k_2} \tag{5}$$

In a normalized form this equation is frequently written as:

$$M_r = k_1 p_a \left(\frac{3p}{p_a}\right)^{k_2} \tag{6}$$

where k_1 and k_2 are experimentally determined constants; θ is the bulk stress ($\theta = 3p$); and p_a is a reference pressure, p_a = 100kPa. This relationship has been shown to be able to capture the main behaviour characteristics of unbound granular materials under various rolling wheel loading situations (Huang 2004; Erlingsson 2007).

In the 3-D case according to MEPDG the resilient modulus is estimated by using a generalized constitutive model (ARA, 2004; Doré & Zubeck, 2009; Kolisoja, 1997):

$$M_r = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \quad (7)$$

where τ_{oct} is the octahedral shear stress ($\tau_{oct}^2 = \frac{1}{9}\left((\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2\right)$); and k_1, k_2, k_3 are regression constants obtained by fitting resilient modulus test data to the constitutive model. It is often assumed that k_3=0 for course material, returning the base constitutive relationship.

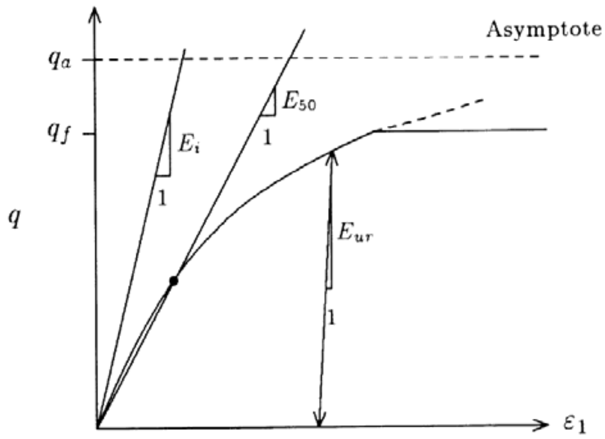

Figure 2.7 Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test (Schanz et al., 1999).

A hardening soil (HS) model is also available to calculate the stress dependency of the soil stiffness (Brinkgreve, 2007). The plastic strains are calculated by introducing a multi-surface yield criterion and the hardening is assumed to be isotropic, dependent on the plastic shear and the volumetric strain (Schanz et al., 1999). The basic parameters for the soil stiffness, E, in the hardening soil model are (Figure 2.7):

$$E_{50} = E_{50}^{ref} \left(\frac{c \cdot \cos\phi - \sigma_3' \cdot \sin\phi}{c \cdot \cos\phi + p_{ref} \cdot \sin\phi} \right)^m \quad \text{Triaxial modulus} \quad (8)$$

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cdot \cos\phi - \sigma_1' \cdot \sin\phi}{c \cdot \cos\phi + p_{ref} \cdot \sin\phi} \right)^m \quad \text{Oedometer modulus} \quad (9)$$

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cdot \cos\phi - \sigma_3' \cdot \sin\phi}{c \cdot \cos\phi + p_{ref} \cdot \sin\phi} \right)^m \quad \text{Un-/reloading modulus} \quad (10)$$

where m is the power for stress-level dependency of stiffness; E_{50}^{ref} is the secant stiffness in a standard drained triaxial test; E_{oed}^{ref} is the tangent stiffness for primary oedometer loading; E_{ur}^{ref} is the unloading / reloading stiffness ($E_{ur}^{ref} \cong 3E_{50}^{ref}$); c is the cohesion and ϕ is the friction angle (Brinkgreve, 2007). The triaxial modulus, E_{50}, mainly controls the shear yield surface and the oedometer modulus, E_{oed}, the cap yield surface. In fact, E_{50}^{ref} largely controls the magnitude of plastic strains that are related to the shear yield surface and E_{oed}^{ref} controls the magnitude of plastic strains that originate from the yield cap (Schanz et al., 1999).

2.3.3 Factors affecting the resilient modulus

Lekarp et al. (2000a) point out that there are several factors that affect the resilient response of granular materials such as moisture, stress level, density, grading and maximum grain size of the material, aggregate type and shape of the particles.

Stress level

The stress level has a high impact on resilient properties of granular materials, as noted earlier. The resilient modulus of untreated granular materials is highly dependent on the confining pressure and sum of principal stresses; when the stress level increases the resilient modulus increases. When comparing the deviator or shear stress to confining pressure, it has less influence on the material stiffness (Lekarp et al. 2000a; Kolisoja, 1997).

Stress history affects the resilient behaviour of granular materials as the material densifies and particles rearrange under repeated stress application. The effect of stress history can be neglected if the applied stresses are low enough to prevent considerable permanent deformation in the material. As the number of load cycles increases the resilient modulus increases (Lekarp et al. 2000a).

The Poisson's ratio is influenced by the stress state applied (Kolisoja, 1997; Ekblad & Isacsson, 2006). Triaxial test results with constant confining pressure and variable confining pressure do not agree if the Poisson's ratio decreases or increases with increasing ratios of deviator stress to confining pressure (Lekarp et al. 2000a). According to the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004) the effect of the Poisson's ratio on pavement response is not significant and therefore it is often assumed as a constant value.

Moisture content

The resilient response of dry and partially saturated granular materials is often similar, but as the moisture content increases it reduces the resilient modulus of granular materials, their frictional strength and resistance to deformation (Ekblad, 2007; Lekarp et al., 2000a & 2000b; Rahman & Erlingsson, 2012; Theyse, 2002; Charlier et al., 2009; Salour & Erlingsson, 2013). In the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004) it is stated that the change in moisture content is the most important factor for the amount of rutting of unbound materials, as increased moisture content causes a decrease in the resilient modulus. If all other conditions remain the same, increased moisture content will lead to a greater elastic (resilient) strain and therefore more rutting.

One of the reasons for a reduced resilient modulus with increased moisture content is due to the lubricating effect of water, causing lower inter-particle forces. Another reason is that, if the soil has a low conductivity, excess pore water pressure might accumulate with repeated loading, causing the effective stresses to decrease, leading to a reduction in the strength and stiffness of the material and less resistance to permanent deformation (Lekarp et al., 2000a & 2000b; Dawson & Kolisoja, 2004; Cary & Zapata, 2011).

In the stress dependent material layers where equation 6 is used, the k_2 factor is close to being a constant, whereas the k_1 factor reduces as the moisture content increases. Rahman and Erlingsson (2012) did a series of RLT tests (Figure 2.8), and concluded that k_1 decreases as the moisture content increases while k_2 is less affected. Li and Baus (2005) came to the same conclusion when investigating the mechanical properties of unbound granular materials in full-scale cyclic and static laboratory plate load tests. They further stated that the degree of saturation, compaction effort and soil gradation have a significant impact on the k_1 values but only a minor or no impact on the k_2 values.

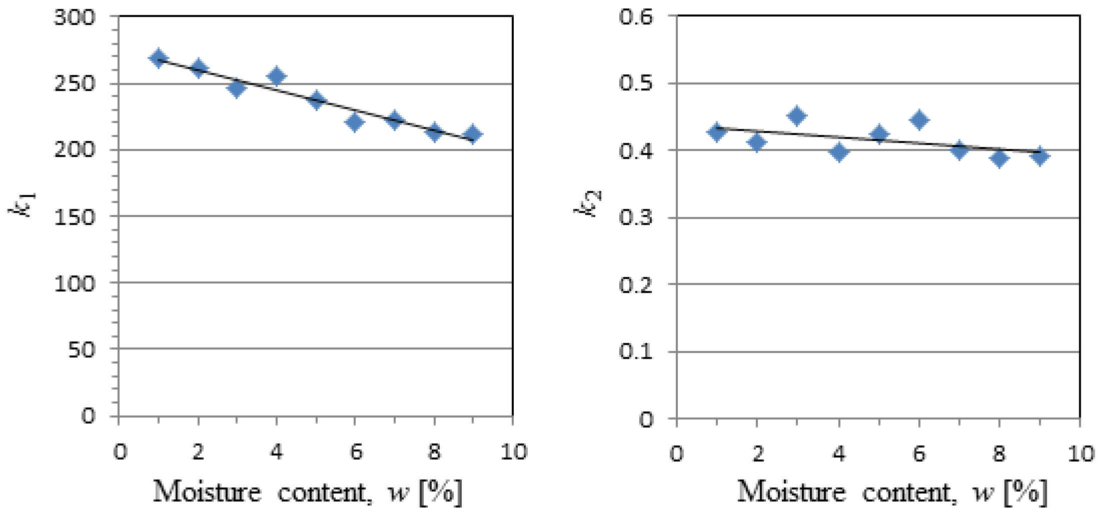

Figure 2.8 The parameters k_1 and k_2 as a function of moisture content (Rahman & Erlingsson, 2012).

Several models, mostly empirical in nature, have been developed to take variation of moisture conditions into account when estimating the resilient soil response (Cary & Zapata, 2011). Cary and Zapata (2011) have developed a model predicting the resilient response as a function of external stress state and matric suction, independent of moisture variation. In MEPDG (ARA, 2004) an M_R-Moisture model is given to express the variation in M_R with moisture content (Figure 2.9):

$$\log_{10}\frac{M_R}{M_{Ropt}} = a + \frac{b-a}{1+EXP(\ln\frac{-b}{a} + k_m \cdot (S - S_{opt}))} \tag{11}$$

where (S-S_{opt}) is the variation in degree of saturation in decimals (S – degree of saturation at a given time; S_{opt} – degree of saturation at a reference condition), M_{Ropt} is the resilient modulus at a reference condition, M_R is the resilient modulus at a given time, a is the minimum of log(M_R/M_{Ropt}), b is the maximum of log(M_R/M_{Ropt}) and k_m is a regression parameter.

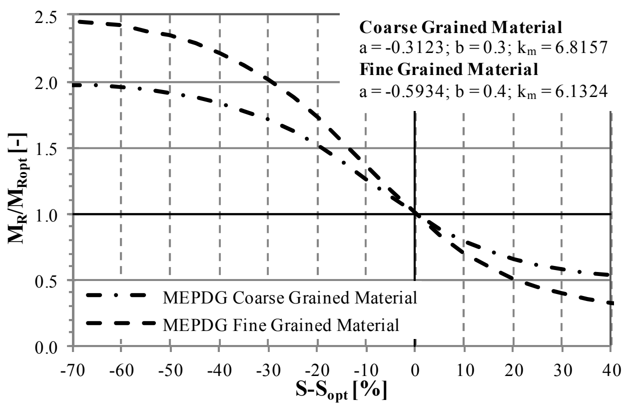

Figure 2.9 The resilient modulus ratio plotted against change in degree of saturation.

Density

It is well known that increasing density of a granular material changes its response to static loading to become stiffer and stronger. As the density increases, the number of contact points increase, causing a decrease in the average contact stress corresponding to a certain external load and therefore less deformation in particle contacts, resulting in a higher resilient modulus. The resilient modulus therefore generally increases with increasing density. For partly crushed material the influence was found to be more than for fully crushed material where the resilient modulus remained almost unchanged. At high stress levels the density (or state of compaction) has been found to have an insignificant influence on the resilient modulus (Lekarp et al., 2000a; Kolisoja, 1997).

Aggregate type and grading

It is well known that the Young modulus varies between different rock types but it is hard to separate the effects of the elastic properties from the effects of the other factors. It is impossible to produce material specimens having identical grain size distribution and shape. Mineralogical composition of aggregate particles can vary depending on the grain size of the aggregate and in the case of crushed aggregates it may have lower strength and stiffness than the original intact rock material (Kolisoja, 1997). Crushed aggregate (with angular / subangular particles), has been reported to have better load spreading properties and a higher resilient modulus than uncrushed gravel (subrounded / rounded particles) (Lekarp et al., 2000a). Yideti et al. (2013a), state that in unbound granular material porosity of the primary structure material plays a major role in the resilient modulus performance and with increasing porosity the resilient modulus decreases.

A grading curve normally presents the grain size distribution of granular materials, which often contain a large number of particles of different sizes. Generally it is believed that the resilient modulus decreases as the amount of fine content increases. If the amount of fines and other small grains is relatively small, it does not affect the ability of larger particles to touch each other and transmit the loads. On the other hand, if the amount of fines and other small grains is so high that it overfills the empty spaces between the large particles, it affects direct contact between the large particles. This results in coarse grained material floating in the fine grained material, where the fine grained material dominates the macroscopic behaviour of the material. In Figure 2.9 it can be seen that the fine grained material is affected more by change in moisture content than the coarse grained material. It has been found that the resilient modulus increases with increasing maximum particle size. Particle size distribution has some influence on the materials stiffness but is believed to be of minor significance (Lekarp et al., 2000a; Kolisoja, 1997).

2.3.4 Response modelling

Many techniques are available for determining responses in flexible pavement systems. Here two methods were examined and used: a 3D FE analysis and an MLET method. A major advantage of MLET is computation efficiency but it cannot take all the factors that affect flexible pavement structures into account like the 3D FE analysis (Schwartzs, 2002; Loulizi et al., 2006; Huang, 2004; Erlingsson and Ahmed, 2013; Erlingsson, 2007; ARA, 2004).

Multi Layer Elastic Theory (MLET)

The MLET is a widely used response model in pavement engineering. The theory was first developed for two and three layered systems by Burmister (1943 & 1945) and extended to *n*-layered systems with increased computer usage (Figure 2.10). The basic assumptions to be satisfied are (Huang, 2004):

- Each layer must be homogeneous, isotropic and linearly elastic with a Young modulus (E), a Poisson ratio (v) and a finite thickness (z), apart from the lowest layer that has infinite thickness.
- The material is assumed to be weightless and infinite in areal extent.
- The load is applied on a circular area with uniform pressure.
- Continuity conditions have to be satisfied at layer interfaces with the same vertical stress, shear stress, vertical displacement and radial displacement.

MLET is an axisymmetric single wheel analysis that can be extended by the superposition principle for multiple wheel loads. The pavement material nonlinearity effect has been incorporated into MLET solutions in a successive approximated way, but the spatial variation of stiffness in a realistic manner is impossible to include with the fundamental axisymmetric MLET formulation. A point has to be selected to represent the entire nonlinear layer. In most cases in pavement design only the most critical stress, strain, or deflection are of interest and a point near to the applied load can be selected. However if responses at different points are wanted it becomes difficult to use MLET for analysing nonlinear materials (Huang, 2004; ARA, 2004).

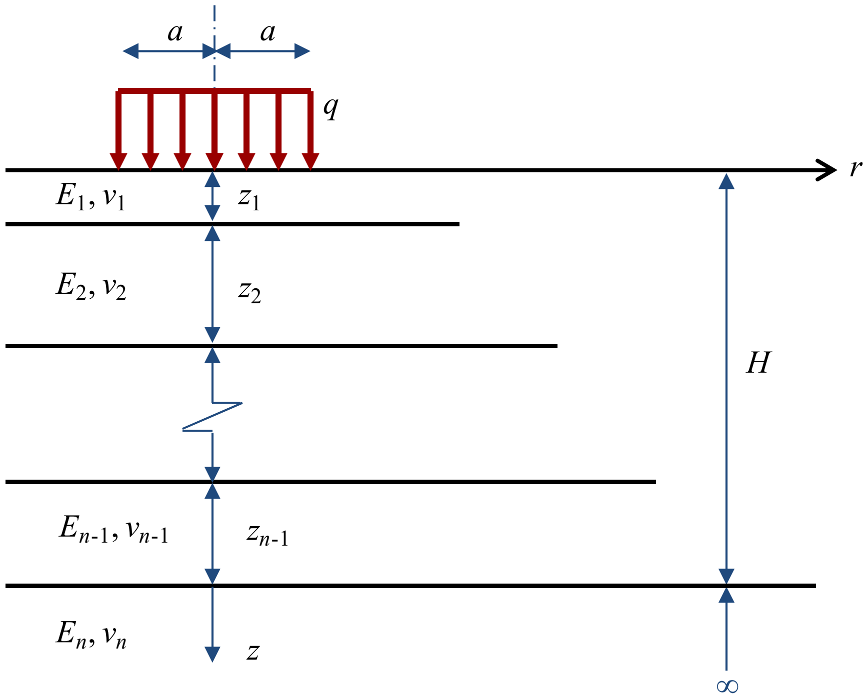

Figure 2.10 An n-layered system.

Finite Element (FE)

Duncan et al. (1968) first used the FE method for analysis of flexible pavements. The method has not been widely used for routine design purposes despite its many qualities. The reasons behind this lack of use can be many, such as the long computer time and storage required and the fact that many geotechnical FE programs only apply static loading whereas pavement structures experience moving traffic loads, a condition that is more complicated. The material models embedded within the geotechnical FE programs are often developed for static loading and not for repetitive cyclic loading. The pavement analysis has to be simplified for these reasons (Huang, 2004; Korkiala-Tanttu & Laaksonen, 2004).

FE methods can simulate a wide variety of nonlinear material behaviour as well as linear elasticity. Stress-dependent stiffness and no-tension conditions for unbound materials can be treated. The strength of an FE analysis is that it meets some of the limitations of MLET solutions (ARA, 2004).

2.4 Pavement performance

Pavement performance is defined as the change (deterioration or distress) of pavement conditions with time or traffic. As noted earlier, each traffic load application leaves its mark on the structure that accumulates with time. There are several different types of distresses that can occur and accumulate over the years such as rutting, fatigue cracking, material disintegration, roughness and bleeding (Mamlouck, 2006; Doré & Zubeck, 2009; ARA, 2004). In this dissertation the main focus was on the permanent deformation and rutting. Pavement failure happens when one or more of the distresses reach an unacceptable level. There are several factors that affect the performance of flexible pavements (Figure 2.11), such as the traffic (traffic load volume, tyre pressure, load magnitude, vehicle speed, wheel / axle configuration, channelized traffic etc.), the soil and pavement materials (including change in material properties with time), the environmental conditions (moisture & temperature and their interaction); the construction and maintenance practice; and combinations of all of the above (Mamlouck, 2006; Huang, 2004; Doré & Zubeck, 2009).

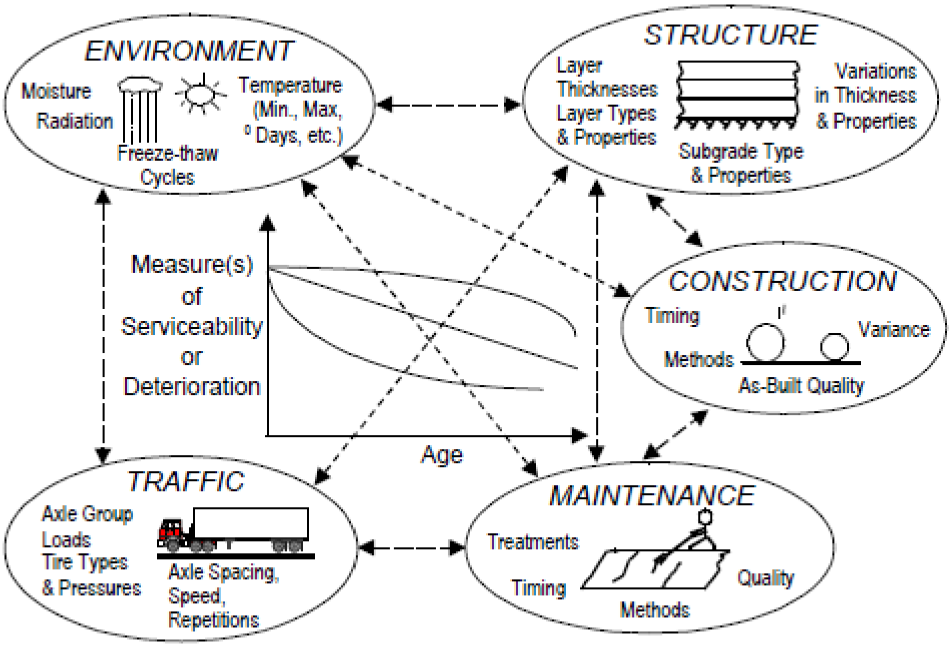

Figure 2.11 Factors affecting road performance (Haas, 2001).

2.4.1 Permanent deformation / rutting

The permanent deformations of structural layers and subgrade in low volume roads and heavily loaded fields have a significant influence on a pavements deterioration and service life. Rutting is defined as "permanent deformation in the wheel path" by Mamlouk (2006) or "depressions of the wheel paths as a result of traffic load" by Doré and Zubeck (2009). Pavement rutting is not desirable. It can become a safety hazard as lateral manoeuvrability of vehicles is often reduced, the risk of skidding on ponding water and ice increases, fuel consumption of pavement users increases, and the structural capacity of the pavement structure decreases as water concentrates on the surface and soaks into the pavement structure. Furthermore, the asphalt layers crack can when unbound layers rut underneath them, and the rutting is seldom uniform, causing unevenness of the pavement surface (Doré & Zubeck, 2009; Dawson & Kolisoja, 2004).

Rutting can be limited to the asphalt layer (unstable HMA, densification of HMA or studded tyre wear), but in thinner flexible pavements it is usually also caused by permanent deformation in the unbound material layers or a combination of the two. There are several factors that affect the amount of rutting and a number of reasons for the occurrence of rutting. Material properties, mix design and in-service conditions are the factors that mainly affect the permanent deformation of HMA mixes. Key factors influencing the accumulation rate of permanent deformation in unbound granular materials include grain size distribution of the material, degree of compaction, moisture content and stress conditions (especially the intensity of shear stresses) (Doré & Zubeck, 2009; Dawson & Kolisoja, 2004; Korkiala-Tanttu, 2008; ROADEX, 2011; Mamlouk, 2006).

Rutting is a major problem in Northern Europe, where flexible pavements are common along with harsh environmental conditions.

2.5 Permanent strain response

When designing flexible road pavements their serviceability needs to be acceptable. One of the design criteria is to limit a structure's rut development or deflection. This is fairly easy to measure but extremely complex to predict. It is not sufficient to characterize the pavement materials but the impact of environmental conditions and appropriate stress distribution also need to be estimated during the pavements service life. The deformation response of unbound granular materials under repeated traffic loading is defined by a resilient response (load carrying ability of the pavement) and a permanent strain response, which characterizes the long-term performance, including rutting (Figure 2.12). There are several methods available but most of them have an empirical nature that is hard to extrapolate with confidence beyond the conditions on which they are based (Lekarp et al. 2000b; Erlingsson, 2012).

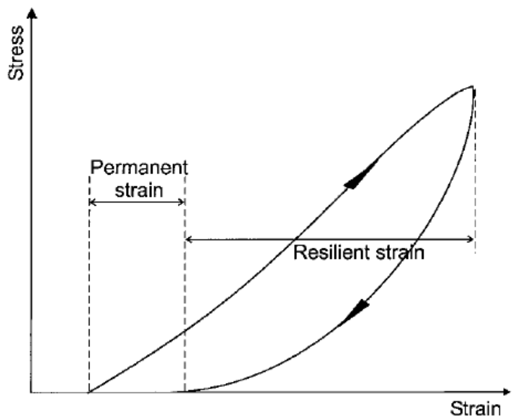

Figure 2.12 Resilient and permanent strains in granular materials during one cycle of load application (Lekarp et al., 2000a & 2000b).

2.5.1 Factors affecting permanent strain response

Despite the limited research, it has been shown that several factors affect the plastic behaviour and permanent strain response of granular materials (Lekarp et al., 2000b):

Stress

Development of permanent deformation in granular materials is highly affected by the stress level. In unbound granular materials the permanent deformation is governed by some form of stress ratio consisting of deviatoric and confining stresses. There is a close relation between the permanent deformation behaviour of granular materials and the stress history.

Each load application gradually stiffens the material, reducing the proportion of permanent to resilient strains during posterior loading cycles. The effect of stress reorientation on permanent strain is not fully understood, probably due to the fact that repeated load triaxial testing does not allow for change in the direction of principal stresses. It is believed that principal stress reorientation results in larger permanent strains (Lekarp et al., 2000b; Korkiala-Tanttu, 2008).

Number of load applications

The number of load cycles is an important factor when analysing the long-term behaviour of granular materials. The permanent deformation gradually grows as the number of load application increases, i.e. each load application contributes a small increment to the accumulation of strain (Lekarp et al., 2000b; Sweere, 1990).

Moisture content

Increased moisture content (above optimum) causes positive pore water pressure to develop under rapidly applied loads that reduces the effective stress and therefore the permanent deformation resistance of the material. Measurements by Korkiala-Tanttu et al. (2003) showed that even small changes in moisture content can have a dramatic effect on the formation of permanent deformation. The primary reason might be matric suction, depending on the mineralogical characteristics of the material. Erlingsson (2010), Li and Baus (2005), and Salour and Erlingsson (2013) reported that increased water content significantly increased the vertical deflection. Rahman and Erlingsson (2012), Gidel et al. (2001) and Uthus et al. (2006) obtained the same results and observed that once the moisture content reached or went over an optimum value the permanent deformation increased dramatically and the material collapsed (Figure 2.13).

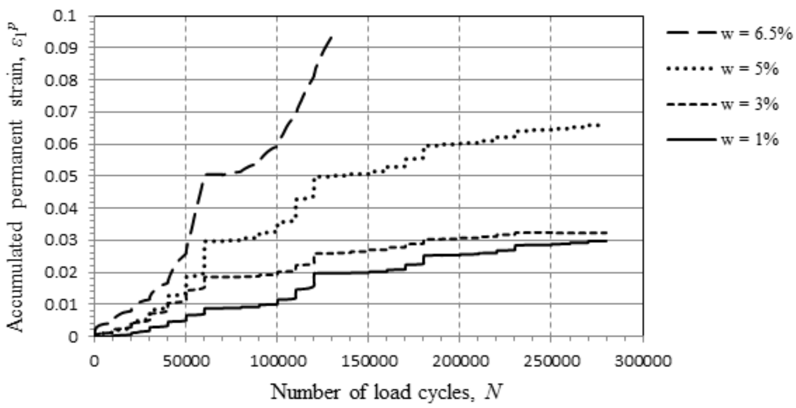

Figure 2.13 Accumulation of permanent strain with load repetitions in a multi-stage repeated load triaxial test at four different moisture contents (w), for a typical Swedish crushed base course material (Rahman & Erlingsson, 2012).

Moisture content has a complex effect on the deformation behaviour of granular materials. The amount of influence depends on the moisture content but also on the grain size distribution and on the electrochemical properties of the material, which are based on the mineralogical composition (Kolisoja, 1997; Uthus 2007).

Aggregate type, density and grading

The effect of density or degree of compaction is important for the long-term behaviour of granular materials. Increasing density increases the resistance to permanent deformation under repetitive loading. The grading curve and amount of fines is believed to influence the resistance to plastic strains. Yideti et al. (2013b) have identified a new parameter entitled disruption potential (DP) which is defined as the ratio of the volume of potentially disruptive fine material over the free (available) volume within the primary structure material within unbound granular materials. They showed that most stable unbound granular materials with a DP ranging from 0.5-0.9 exhibited the best performance in terms of resistance to permanent deformation. This indicates that some or a limited amount of fine granular material is desirable to resist deformation but an extensive amount of fine material decreases the resistance. Angular materials (e.g. crushed stone) undergo smaller plastic deformations then gravel with rounded particles as the angular material has higher angle of shear resistance due to better particle interlock (Lekarp et al., 2000b).

2.5.2 Permanent strain modelling

Material will experience permanent deformation when repeated loading is applied. This deformation is due to sag, compaction or distortion of the material as well as friction wear. It is hard to estimate these quantities separately, but it has been proven that the speed of permanently built up deformation under repeated loads wears off as the number of applied load repetitions increases. The accumulated plastic strain, $\hat{\varepsilon}_p$, is therefore related to the number of applied load repetitions as well as the magnitude and layout of the applied load, i.e. to the actual stress regime (Magnúsdóttir et al., 2002; Erlingsson, 2012). The accumulated plastic strain, $\hat{\varepsilon}_p$, can be express as:

$$\hat{\varepsilon}_p(N) = f(N) \cdot g(p_{\max}, q_{\max}) \tag{12}$$

where *f(N)* is a function based on curve fitting of repeated load triaxial test (RLT) data and $g(p_{max}, q_{max})$ is a function relating to the actual stress regime induced from the load on the pavement surface to the Mohr-Coulomb failure criteria of soil materials, shown in Figure 2.14 where $m = \dfrac{6\sin\phi}{3-\sin\phi}$ and $s = q_0 = \dfrac{6 \cdot c \cdot \cos\phi}{3-\sin\phi}$.

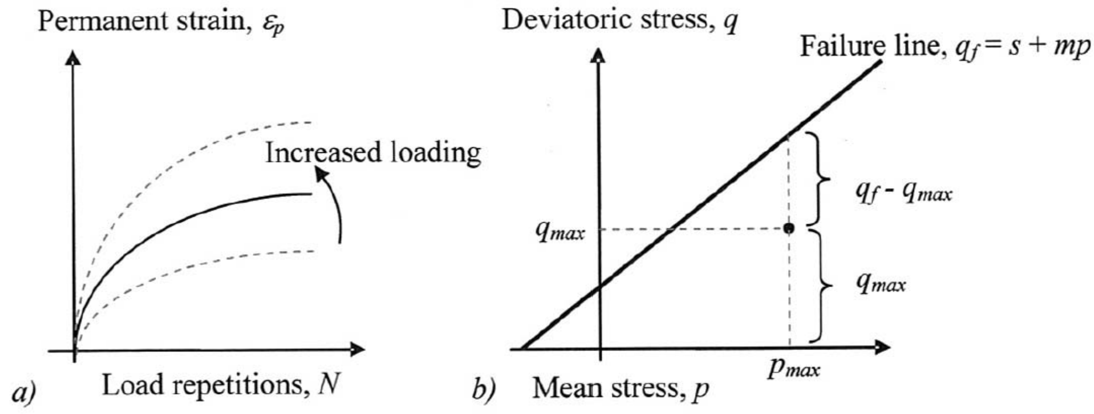

Figure 2.14 An empirical approach to model the permanent deformation development in unbound aggregate materials. a) A curve fitting f(N) approach b) the curve is scaled through comparison with how close the actual stress regime lies to the Mohr-Coulomb failure envelope (Erlingsson, 2012).

In most of the available models each layer, j, can be divided into sublayers, i, and the deformation manifested on the surface is calculated by adding up the permanent deformation, $\hat{\delta}_p$, in the layers:

$$\hat{\delta}_p = \sum_{i=1}^{n} \sum_{j=1}^{m} \hat{\varepsilon}_{p_{ij}} \cdot \Delta h_{ij} \tag{13}$$

where $\hat{\varepsilon}_{p_{ij}}$ and Δh_{ij} are the average plastic strain and thickness of sublayer i of layer j, n is the total number of layers and m is the total number of sublayers within each layer.

The AASHTO model or the Tseng & Lytton method (T&L method) are commonly used to characterize permanent deformation of the pavement materials (Tseng & Lytton, 1989):

$$\hat{\varepsilon}_p = \varepsilon_0 \cdot e^{-\left(\frac{\rho}{N}\right)^{\beta}} \tag{14}$$

where $\hat{\varepsilon}_p$ is the permanent strain and the material parameters ε_0, ρ and β are estimated by fitting a curve that relates permanent strains to loading cycles often obtained by RLT tests. The values of ε_0, ρ and β depend on the type of material and its physical properties as well as the testing conditions.

According to MEPDG the deformation in unbound materials is found by modifying the T&L method (ARA, 2004):

$$\frac{\delta_p(N)}{h} = \hat{\varepsilon}_p = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \cdot e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v \tag{15}$$

where ε_r is resilient strain imposed in lab test to obtain ε_0, ρ and β (in/in), ε_v is the average vertical resilient strain in the layer (from the primary response model) (in/in), β_1 is a calibration factor for unbound granular and subgrade materials. After numerous modifications a reasonably calibrated relationship was gained using the following models (ARA, 2004):

$$\log\beta = -0.6119 - 0.017638W_c \tag{16}$$

$$C_0 = \ln\left(\frac{a_1 E_r^{b_1}}{a_9 E_r^{b_9}}\right) \tag{17}$$

$$\rho = 10^9\left[\frac{C_0}{\left(1-\left(10^9\right)^{\beta}\right)}\right]^{1/\beta} \tag{18}$$

$$\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = \frac{e^{(\rho)^{\beta}} \cdot a_1 E_r^{b_1} + e^{\left(\rho/10^9\right)^{\beta}} \cdot a_9 E_r^{b_9}}{2} \tag{19}$$

$$W_c = 51.712\left[\left(\frac{\varepsilon_r}{2555}\right)^{\frac{1}{0.64}}\right]^{-0.3586 \cdot GWT^{0.1192}} \tag{20}$$

where W_c is the water content (%), a_1 = 0.15, a_9 = 20.0, b_1 = 0.0, b_9 = 0.0, E_r is the resilient modulus of the layer/sublayer (psi) and GWT is the ground water table depth (ft).

Korkiala-Tanttu (2005 & 2008) has developed a new analytical, relatively simple, nonlinear elasto-plastic material deformation model for unbound materials. The model is developed using the theory behind the static loading material model and then extended to fit the dynamic loading cases. It can take into account the number of passes, the capacity of the material and its stress state. It is an extended version of Sweere's (1990) equation, suggesting that the relation of accumulated permanent deformation and number of passes is an exponential function:

$$\hat{\varepsilon}_p(N) = a \cdot N^b \qquad \text{Sweere's equation} \tag{21}$$

where a and b are regression parameters.

$$\hat{\varepsilon}_p = C \cdot N^b \, \frac{R}{A-R} \qquad \text{Korkiala-Tanttu's (KT) model} \tag{22}$$

In the model the yielding and shear strains are described with the failure ratio, R; when R is closer to failure the deformations are larger. Here R is defined as the ratio between deviatoric stress, q, and deviatoric stress at failure, $q_f = q_0 + m \cdot p$, where p is hydrostatic stress (kPa). Parameter A is the maximum value of the failure ratio R, which theoretically is 1 and that should be used if non-linear elasto-plastic material models are being used but if linear elasto-plastic models are applied $R/_{A-R}$ can increase to indefinite values as R approaches 1, and therefore A should be chosen as roughly 1.05. The material parameter C depends on various factors such as stress, the material, its degree of compaction and water content. The value of the parameter b, is calculated from laboratory tests and deformation measurements of HVS tests but it gives the damping shape of the permanent deformation curve: if b=1, $\hat{\delta}_p$ is linearly dependent on N, but if b is small the deformations are not primarily dependent on N. Parameter b mainly depends on the stress state and failure ratio, but the degree of compaction and water content also has some effect (Korkiala-Tanttu, 2008).

Gidel et al. (2001) presented a new approach to relate the permanent axial deformation with the number of load cycles. Repeated multistage triaxial tests on unbound granular materials were used to develop the model:

$$\hat{\varepsilon}_p = \varepsilon^0 \cdot \left[1-(N)^{-B}\right] \cdot \left(\frac{l_{\max}}{p_a}\right)^n \cdot \frac{1}{\left(m + \frac{s}{p_{\max}} - \frac{q_{\max}}{p_{\max}}\right)} \tag{23}$$

where ε^0, B and n are model parameters, $l_{\max} = \sqrt{p_{\max}^2 + q_{\max}^2}$ and p_a = 100kPa.

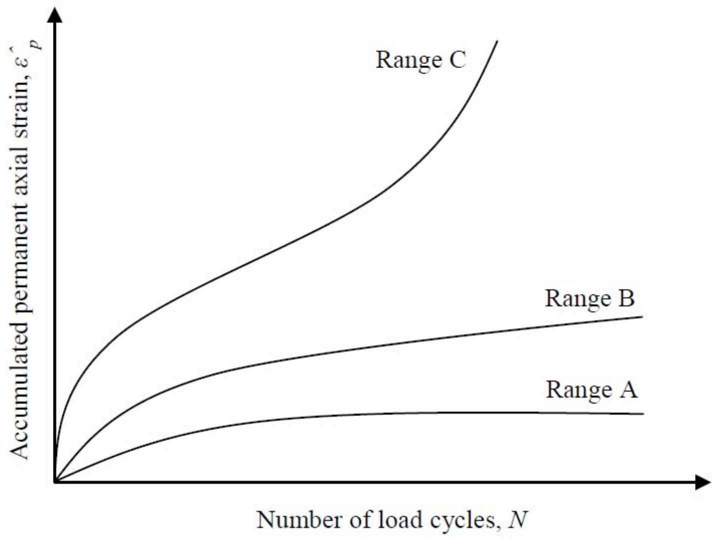

Figure 2.15 Different types of permanent deformation behaviour, depending on stress level (Erlingsson & Rahman, 2013).

The Dresden model takes into account the different deformation behaviour in the shakedown ranges A, B and C (Figure 2.15) (Werkmeister et al., 2003 & 2004). The model is based on the stress-dependent Huurman model (1996) (Werkmeister et al. 2003):

$$\hat{\varepsilon}_p(N) = A\left(\frac{N}{1000}\right)^B + C\left(\exp\left(D\frac{N}{1000}\right)-1\right) \tag{24}$$

where A, B, C and D are model coefficients.

Many more models have been presented and are summarized, for example in Lekarp et al. (2000b). For this dissertation only two models were used, the stress dependent KT model and the strain dependent MEPDG model.

2.5.3 Time hardening

Most of the performance models do not take into account the variable stress paths that may be involved in reality. The stress state of pavement structures changes, for example, when the axle loads (such as lateral wander, various axle loads and configurations) and environmental conditions (such as moisture content and temperature) change. A "time-hardening" procedure can be used when taking into account the effects of different stress variations on the accumulation of permanent deformations and rutting development (Lytton et al., 1993; Hu et al., 2011; Erlingsson, 2012; Ahmed & Erlingsson, 2013; Erlingsson & Rahman, 2013).

The time-hardening approach takes into account the effect of the stress history. This is done by calculating the equivalent number of load repetitions at the beginning of each load step ($N_{eq,j}$) for any given stress path, j, to attain an equal amount of, for example, permanent strain as was accumulated from the previous stress paths. The equivalent number of load repetitions is used to modify the total number of load repetitions (N) that is then used to calculate the accumulation of permanent strain for the current stress path (Figure 2.16):

$$\hat{\varepsilon}_{p_j} = \hat{\varepsilon}_{p_{j-1}} + \hat{\varepsilon}_{p_j}(N=1) \cdot \left[\left(N_{eq_j} + \Delta N \right)^b - N_{eq_j}^b \right] \tag{25}$$

where ΔN is the load repetitions for the current stress level, $\hat{\varepsilon}_{p_j}(N=1)$ is the permanent strain for the jth stress level for N=1 and $\hat{\varepsilon}_{p_{j-1}}$ is the permanent strain at the end of the previous stress path (j-1). $N_{eq,j}$ can be calculated from:

$$N_{eq_j} = \left[\frac{\hat{\varepsilon}_{p_{j-1}}}{\hat{\varepsilon}_{p_j}(N=1)} \right]^{\frac{1}{b}} \qquad \text{KT-model} \tag{26}$$

$$N_{eq_j} = \rho \left(-\ln \left(\frac{\hat{\varepsilon}_{p_{j-1}}}{(\varepsilon_0 / \varepsilon_r) \cdot \varepsilon_{v_j}} \right) \right)^{-\left(1/b\right)} \qquad \text{MEPDG-model} \tag{27}$$

In the case where the current stress path is significantly lower than the previous one, the $N_{eq,j}$ approaches infinity and the assumption is made that no accumulation of permanent strain is derived from the current stress path.

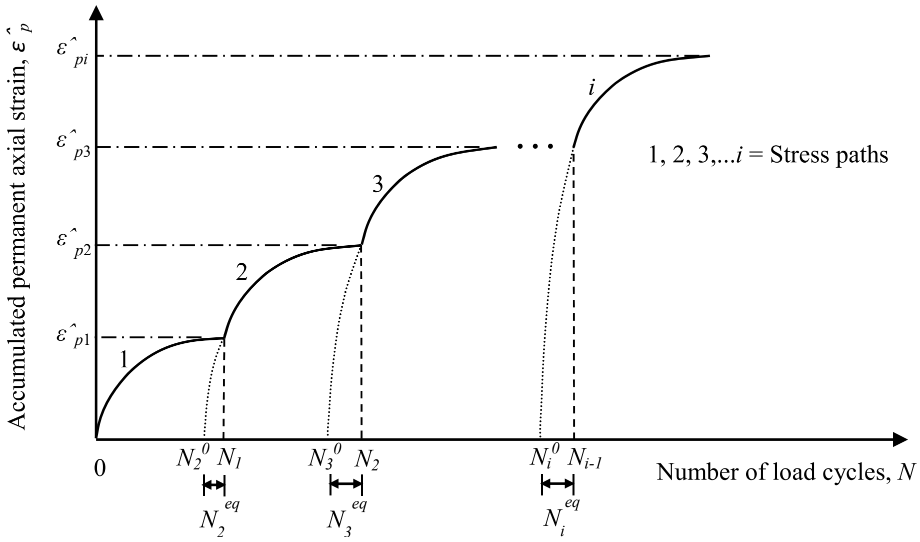

Figure 2.16 Schematic figure of the time hardening approach (Erlingsson & Rahman, 2013).

3 Summary of key findings

In this dissertation mainly two test road structures, referred to as SE10 and SE11, have been examined (Figure 3.1). The construction of the structures was substantially the same and they were instrumented to measure stress, strain and deflection responses as a function of load repetitions as well as permanent deformation manifested on the surface (rutting). They were tested with an HVS to investigate their performance behaviour as well as the accuracy of repeated tests. The performance behaviour can thereafter be used for validation in a mechanistic performance scheme. The structures were tested in "moist" and "wet" states, as the groundwater level was raised half-way through the test, giving the opportunity to estimate the influence of water on the response and performance of the structures. The "moist" case was to simulate a situation where the groundwater table is at great depth with little spatial moisture transfer and very limited evaporation, whilst the "wet" case simulated supposedly the worst case with water running in the trenches (Wiman, 2010). The responses as well as the permanent deformation were monitored and compared with calculated values. The accuracy of repeated tests was estimated and normally good correlation was found between the two tests.

The structures consisted of a Hot Mix Asphalt (HMA), divided into a surface course (AC pen 70/100; d_{max}=16mm) and a bituminous road base (AC pen 160/220; d_{max}=32mm). Under the asphalt were two layers of unbound crushed rock (granite), a base layer (0-32mm SE10 / 0-45mm SE11) and a subbase layer (0-90mm) resting on a subgrade. The subgrade consisted of silty sand with a high fine content of about 25% and over 90% of the grains under 0.5mm. The subbase had a fine content of 3% whilst the base material had a fine content around 6%. The optimum gravimetric water content for the base and subbase is around 4-5% whilst for the subgrade it is approximately 13% (Wiman 2010).

The HVS tests were divided into three phases with bidirectional loading applied in all the phases (Wiman 2010): *a pre-loading phase*, with 20,000 load repetitions applying light loading (30kN single wheel load (60kN full axle load) and 700kPa tyre pressure) and evenly distributed wheel passes in the lateral direction to achieve even compaction; *a response phase*, where the responses were estimated from a single and dual wheel configuration using various tyre pressures and axle loads; and a *main accelerating loading phase*, with more than one million load cycles applied, dual wheel configuration, centre to centre spacing of 34 cm, tyre type 295/80R22.5, 60kN dual wheel load (120kN full axle load), 800kPa tyre pressure, constant environmental conditions at 10°C temperature and a lateral distribution of the loading followed a normal distribution (Figure 3.2). Approximately half-way through the tests water was gently added into the structures until the water level was 30cm below the top of the subgrade (Figure 3.1, Figure 3.3 and Figure 3.4). As no other alterations were made this gave the opportunity to assess the influence increased moisture had on the response and performance of the structures (Wiman, 2010).

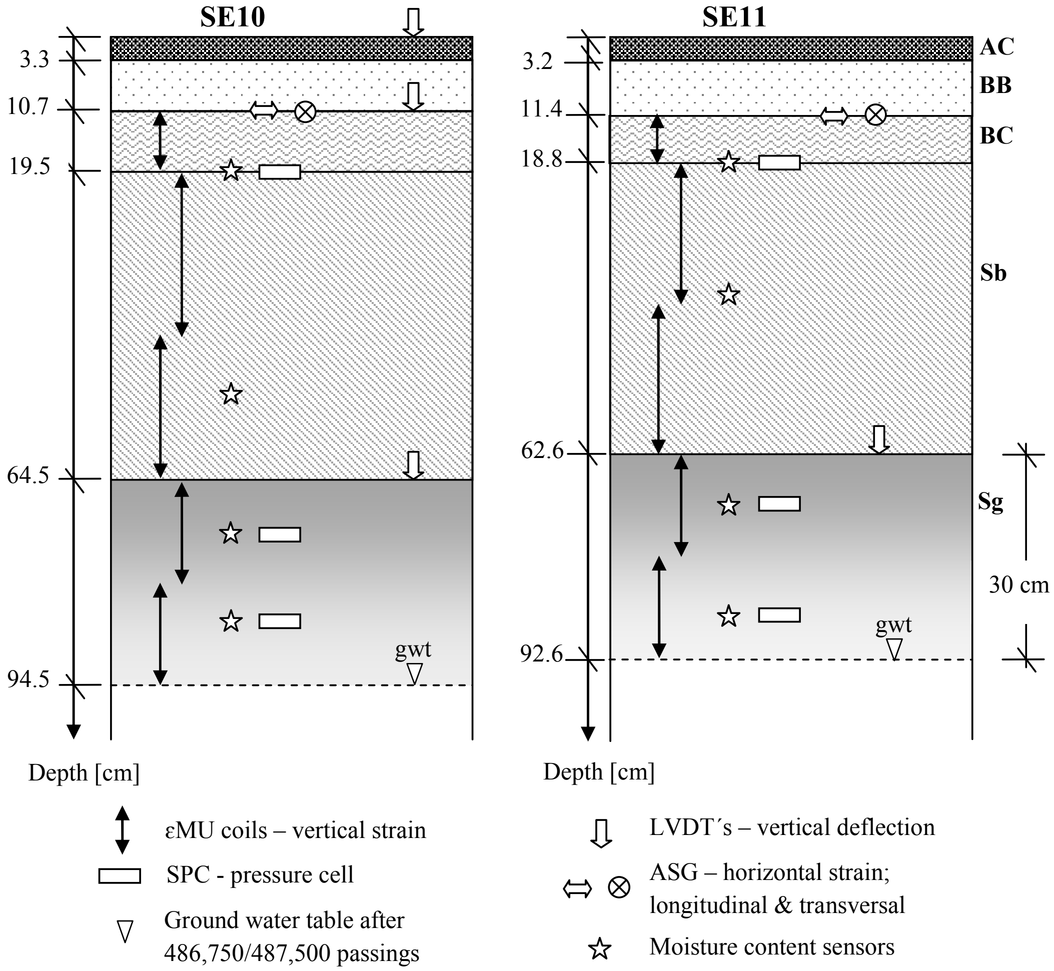

Figure 3.1 Cross sections of pavement structures SE10 and SE11 including the vertical location of the instrumentation. In figure: AC - asphalt concrete, BB – bituminous base, BC – granular base course, Sb – granular subbase and Sg – subgrade.

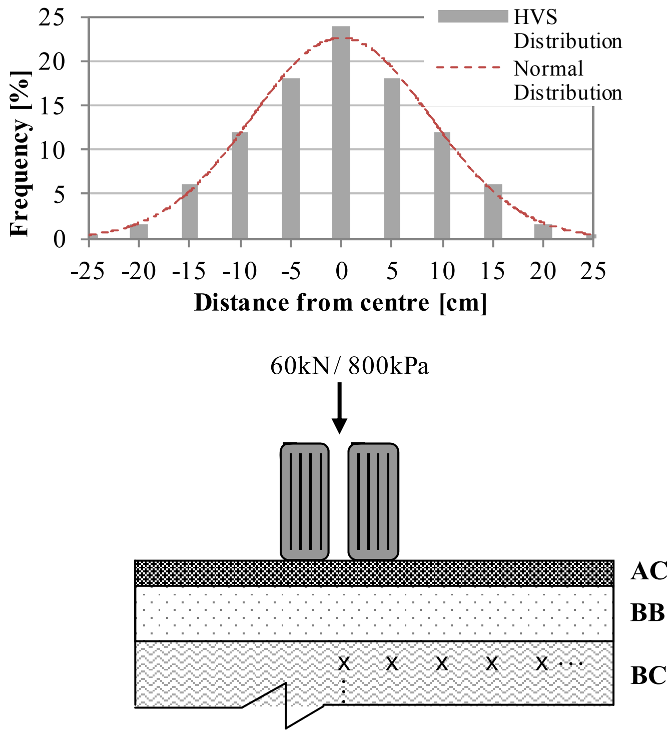

Figure 3.2 The distribution of the centre of loading (lateral wander).

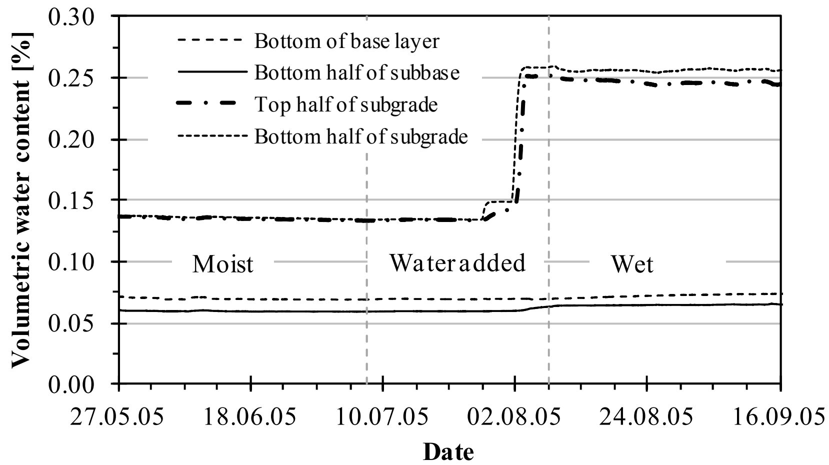

Figure 3.3 Change in the volumetric water content for SE10 with time; while the water was added no loading took place.

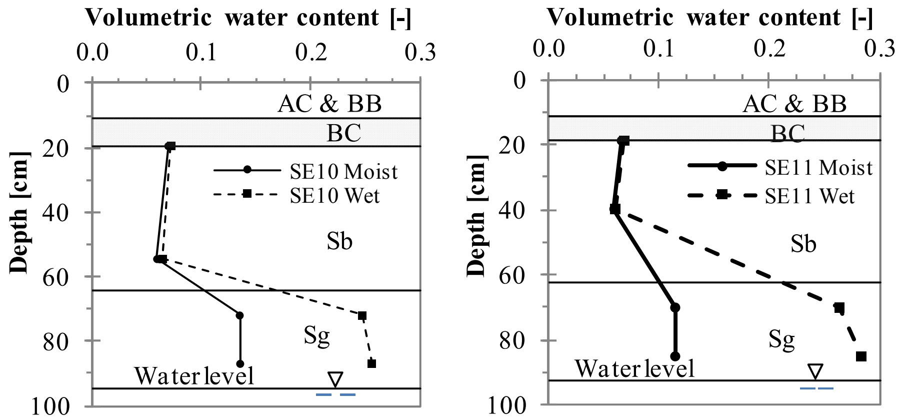

Figure 3.4 The volumetric water content as a function of depth for SE10 and SE11.

3.1 The instrumentation

The pavement structures were instrumented with εMU coils (inductive coils) to measure the vertical strain (recoverable and permanent); soil pressure cells (SPC) to obtain vertical stresses; linear variable differential transducers (LVDT's) to measure the vertical deflection; asphalt strain gauges (ASG) (H-bar) to obtain the horizontal strain at the bottom of the asphalt bound layers; a laser beam to measure the surface profile; and moisture content sensors to measure the volumetric water content (Wiman 2010). The instrumentation and its accuracy are described in Paper II – Saevarsdottir et al. (2014).

The response signals were generally good with the exception that some of the εMU coils showed some noise in the signal. In those cases a moving average was used. The signals as well as their comparison with calculations are shown in Paper V – Saevarsdottir and Erlingsson (2014).

Acceptable accuracy was found of the testing equipment. Better agreement was normally gained for vertical strains than stresses. Generally it is more difficult to measure the vertical stresses in granular materials because of the complex inherent inter-granular interactions between the aggregate particles and the sensors. The variation of the measurements for all sensors decreased as the depth increased, most likely due to the fact that the influence of the dynamic loading diminishes with depth as well as having finer and more homogenous material in the subgrade. The LVDTs showed in general reasonable measurements, but they are known to be accurate and reliable. The ASG sensors performed fairly well but these were the only meters where a better performance was gained in a previous test compared to SE10. The moisture content sensors showed good performance in all layers. The difference between sensors at the same depth indicated that two sensors out of three often have good correlation. This might be related to the fact that unbound materials can be hard to measure with many factors influencing the results, such as compaction and the quality of the contact between the material and the meter. Therefore it is recommended that at least 3 sensors be used at each depth.

3.2 The response behaviour of the structures

The response signals were monitored and compared with calculated values using a 2D axisymmetric MLET method (SE10 and SE11) and a 3D FE method (SE10). The responses of the structures were calculated using a linear material model for the bitumen bound layers and the subgrade and a nonlinear stress dependent model for the base and subbase layers (Paper I – Saevarsdottir & Erlingsson, 2012). When the material parameters were estimated, several laboratory and field tests were considered including FWD. The results of the FWD measurements can be seen in Figure 3.5 for SE10 and Figure 3.6 for SE11. The material parameters used are listed in Paper III – Saevarsdottir and Erlingsson (2013a) and Paper IV – Saevarsdottir and Erlingsson (2014).

Generally good agreements were found between the measured responses and calculated values using both MLET and FE methods with no significant difference between them, despite different models used to calculate the stress dependence of the base and subbase layers as well as using a circular loading area in MLET and a square loading area in the FE analysis. It is likely that the difference between the two methods was more pronounced in the uppermost part of the structure, but that could not be observed here.

In Figure 3.7 the vertical strain for SE10 from the FE analysis is shown and in Figure 3.8 it is compared with the MLET calculations and the measurements (MM). In Figure 3.9 the measured (MM) vertical strain is compared with calculations using the MLET method. The figures indicate a good repeatability of the HVS testing procedure and a good agreement between the measurement and calculations. Figure 3.10 and Figure 3.11 show the vertical induced stress as a function of depth for SE10 and SE11 respectively. The measurements (MM) and calculations (MLET, FE) show good correlation. In Figure 3.12 the measured and calculated tensile strain at the bottom of the asphalt bound layers is shown for both SE10 and SE11. In the figure the relationship between calculations and measurements are found to be reasonable. On the other hand this does not tell the full story as the tensile strain in the transversal direction reduced as the groundwater table was raised in both SE10 and SE11. In the longitudinal direction some of the measurements were rather stable between moist and wet states or showed a slight increase. The tensile strain was in all cases higher in the longitudinal direction compared to the transversal. The reduction between moist and wet state needs to be investigated further. This is described in detail in Paper II – Saevarsdottir et al. (2014).

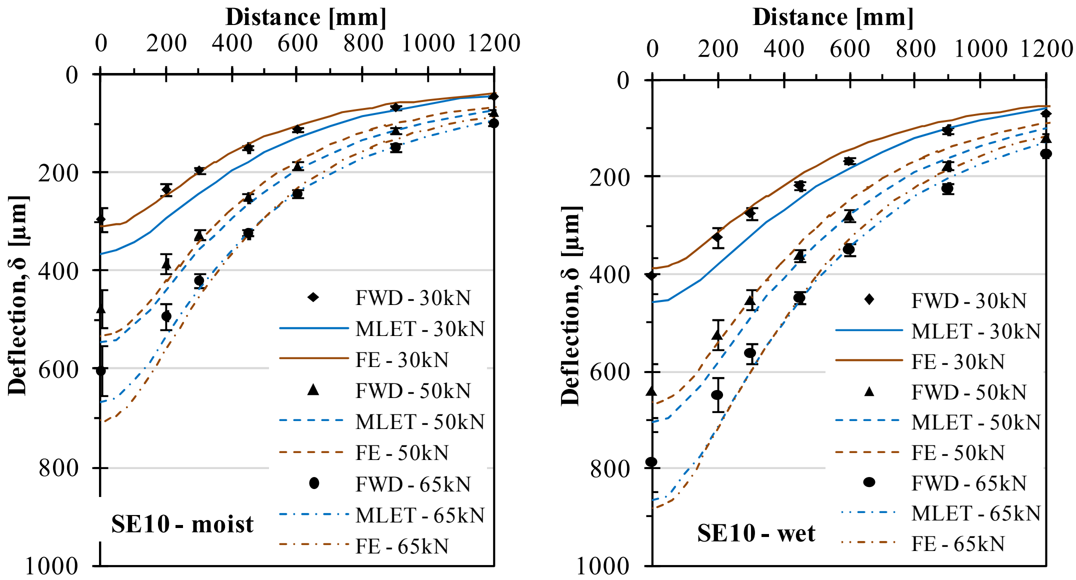

Figure 3.5 Comparison between FWD measurements and back calculations using MLET and FE analysis, using 30, 50 and 65kN load intensity for "moist" and "wet" states for pavement structure SE10.

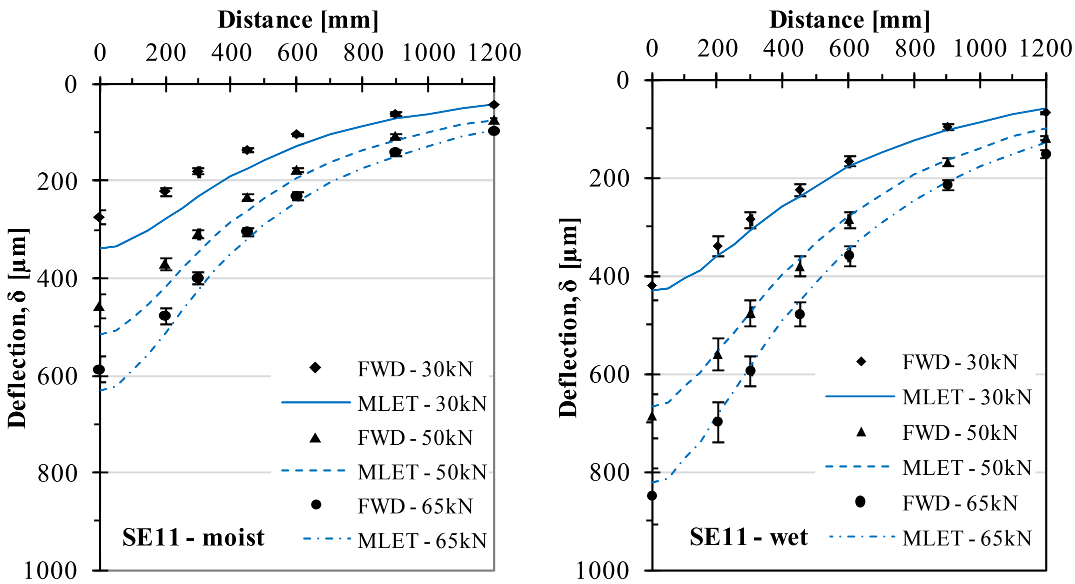

Figure 3.6 Comparison between FWD measurements and back calculations using MLET analysis, using 30, 50 and 65kN load intensity for "moist" and "wet" states for pavement structure SE11.

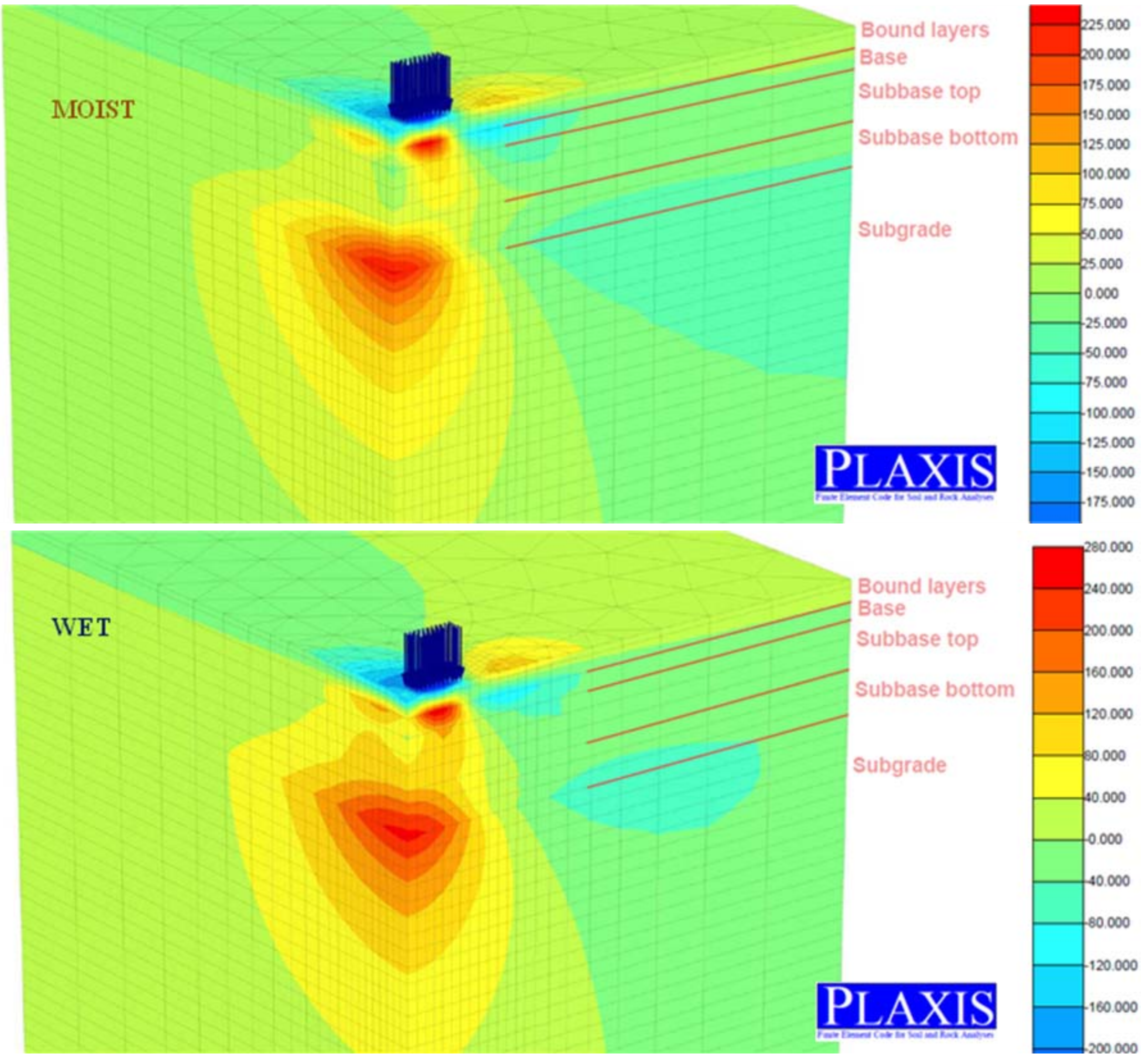

Figure 3.7 Calculated vertical resilient strain as a function of depth using a FE analysis (PLAXIS). Both "moist" (top) and "wet" (bottom) states are shown for test SE10 for a dual wheel configuration, 60kN dual wheel load (120kN axle load) and 800kPa tyre pressure.

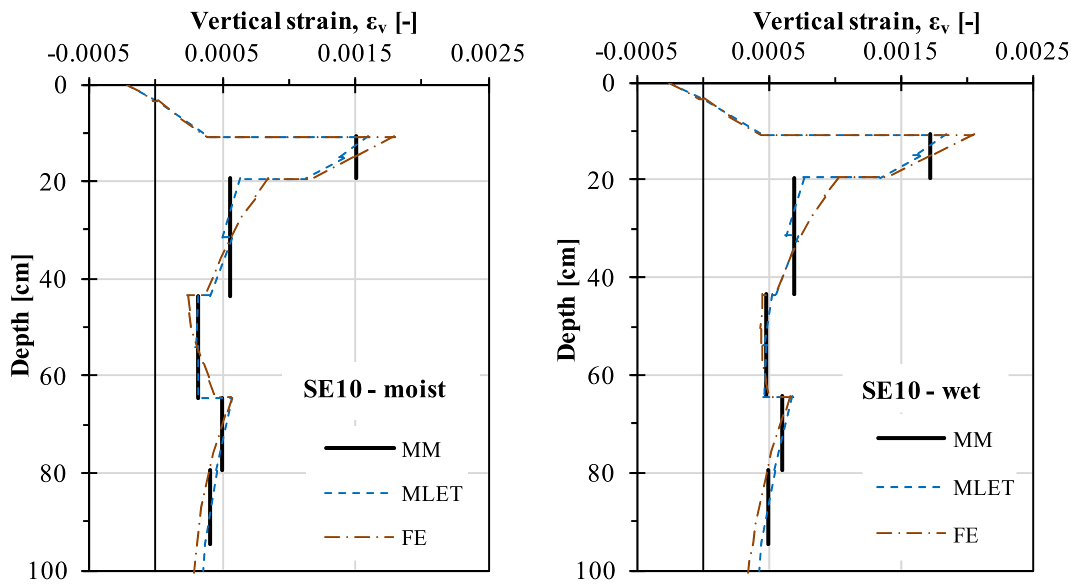

Figure 3.8 Vertical resilient strain as a function of depth for "moist" (left) and "wet" (right) states for test SE10 for a 60kN dual wheel load and 800kPa tyre pressure.

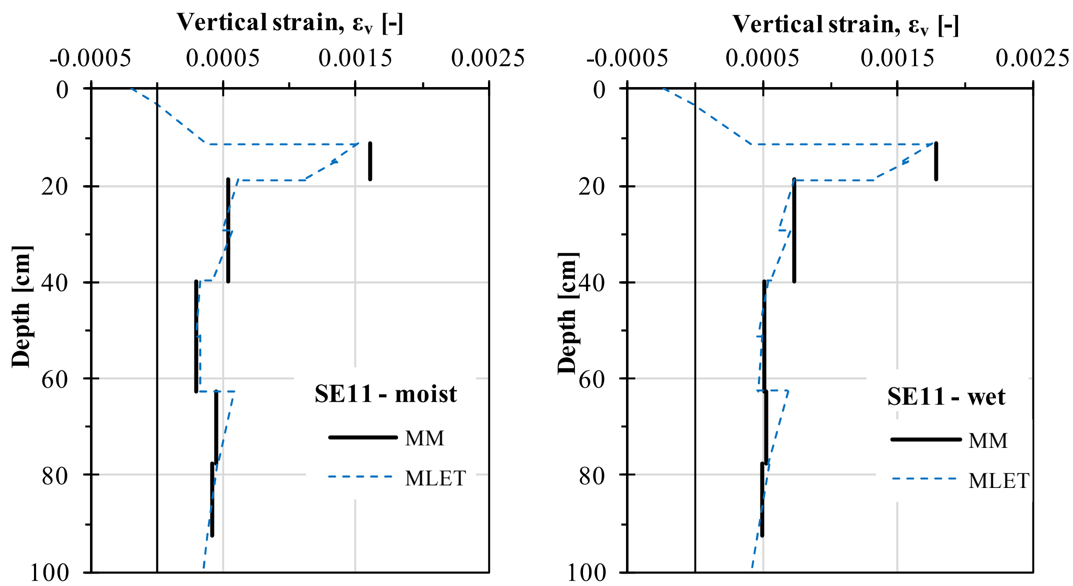

Figure 3.9 Vertical resilient strain as a function of depth for "moist" (left) and "wet" (right) states for test SE11 for a 60kN dual wheel load and 800kPa tyre pressure.

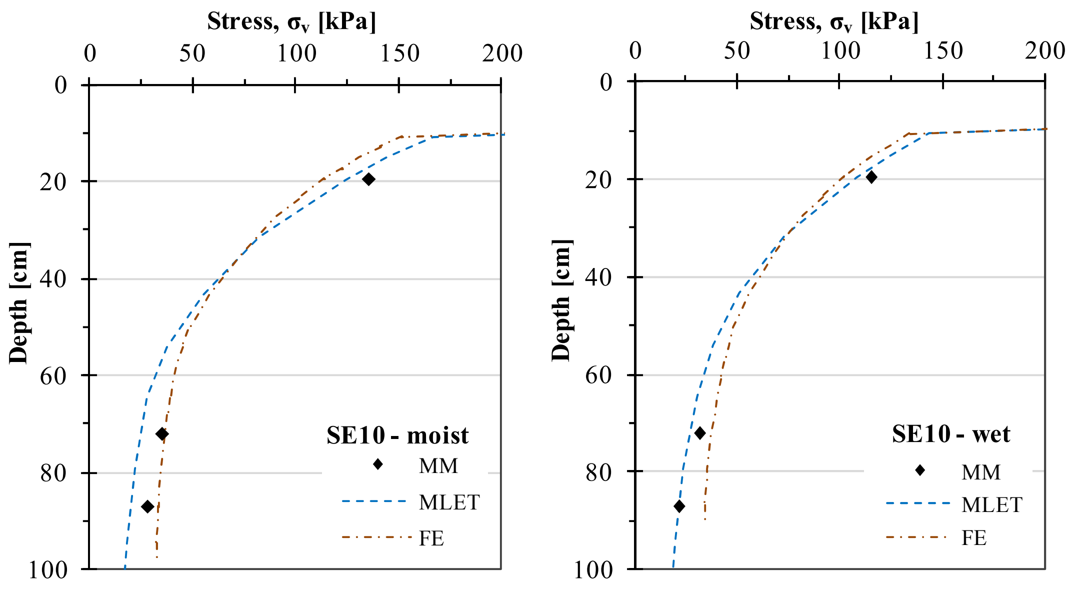

Figure 3.10 Vertical induced stress as a function of depth for "moist" (left) and "wet" (right) states for test SE10 for a 60kN dual wheel load and 800kPa tyre pressure.

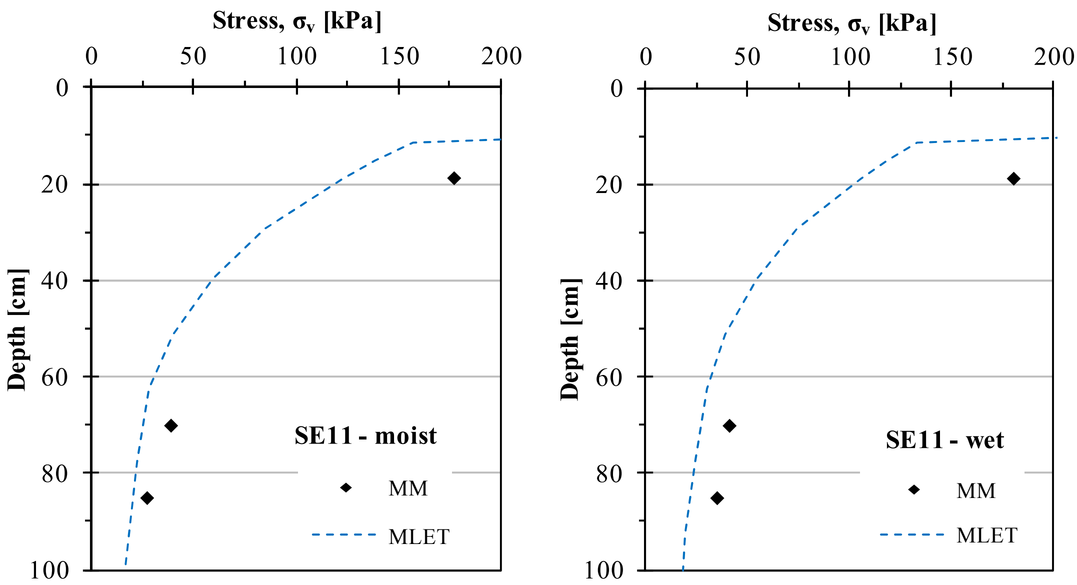

Figure 3.11 Vertical induced stress as a function of depth for "moist" (left) and "wet" (right) states for test SE11 for a 60kN dual wheel load and 800kPa tyre pressure.

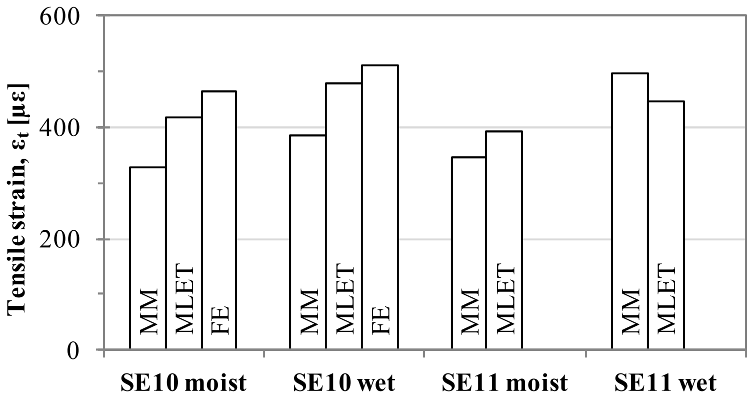

Figure 3.12 Measured and calculated horizontal tensile strain at the bottom of the asphalt bound layers.

The raised water level had a significant effect on the structures, as more vertical strain and less vertical stress were experienced in the unbound layers after the groundwater table was raised. This indicated a lower resilient stiffness of the unbound material layers or a softer structure once the water was introduced. The change in the vertical strain and the resilient modulus was found to be higher in the subbase than in the subgrade, despite a larger increase in the water content and a higher fine content in the subgrade material compared to the subbase. As pointed out in section 2.3.3 there are several other factors that affect the resilient response of granular materials. A possible explaination might be that the subgrade is significantly weaker than the subbase, causing a tension effect in the subbase as it does not provide sufficient support. This would be an effect similar to that experienced in a simply supported beam unit. It could also be that the lubricant effect of the water is more pronounced in the coarse subbase material that allows more resilient movements between individual particles, compared to the dense graded subgrade material that restricts such elastic movements.

3.3 The accumulation of permanent deformation

The accumulation of permanent deformation of the unbound layers was modelled using two simple work hardening material models where lateral wander (Figure 3.2) and change in moisture content (Figure 3.3, Figure 3.4) were accounted for by using the “time hardening” procedure (Figure 2.16). One of the models was a stress dependent model developed by Korkiala-Tanttu (KT) and the other one is strain dependent as presented in the MEPDG. The material parameters used are listed in Paper IV – Saevarsdottir & Erlingsson (2013b). The calculated deformation using the KT model with the responses gained from both MLET and FE analyses is compared to the measured values for SE10 in Figure 3.13. In Figure 3.14 the permanent deformation is calculated using the MEPDG model and the responses again gained from both FE and MLET analysis and compared to the measured values for SE10. In Figure 3.13 and Figure 3.14 the deformation is shown over the base course, the subbase layer, and over the top 30cm of the subgrade material, as well as the total deformation. The permanent deformation of SE11 is shown in Figure 3.15 where the calculations are obtained using the KT and the MEPDG model but all the responses were obtained using the MLET analysis. In Figure 3.15 the total deformation is not included. The scattering of the measured deformation in Figure 3.15 was probably due to unsatisfactory bonding between the sensors and the material in SE11. There was some indication of swelling in the subbase layer, first after introducing the raised water level while the base layer experiences limited change between the moist and wet states. The subgrade layer showed more deformation in the moist state compared to SE10 but less deformation was observed in the wet state. The measurements in SE11 showed the difference between the moist and wet states but the values were considerably lower than in SE10.

All the unbound layers showed increased permanent deformation with increased water content. The most dramatic increase was in the subgrade where the largest increase in the water content was observed. When estimating the accumulated deformation with the KT model (equation 21) the parameter b was reduced as the water content increased, but Korkiala-Tanttu (2005) found the opposite. When using the MEPDG model (equation 14), b slightly reduced as the water content increased. The value of the parameter C in the KT model increases with increased water content but here the increase was significantly higher than reported in the literature. This needs to be investigated further (Paper III – Saevarsdottir & Erlingsson, 2013a).

When predicting the total deformation the material parameters for the subgrade did not have to be altered when using the KT model (equation 21), but when using the MEPDG model (equation 21) the calibration factor β_1 had to be changed. The β_1 reduced with depth and the reduction was more in the wet state than in the moist state. The reason for the change in β_1 is not known and needs further investigation, but possible explanations are increased compaction or increased lateral pressure that in turn increases the interlocking between the material particles with depth (Paper IV – Saevarsdottir & Erlingsson, 2013b).

When using the MEPDG model (Figure 3.14) similar results were obtained when using the MLET and FE analyses. Despite the MLET and FE analyses returning similar stress response results, the change in the amount of permanent deformation when using the KT model (Figure 3.13) was considerably larger, indicating some sensitivity to slight changes in the input values (Paper V – Saevarsdottir & Erlingsson, 2014).

In Figure 3.16 cross sections of the rutting profile after different numbers of load repetitions are visible, two cross sections are from the moist state (293,500 and 486,750 load repetitions) and three in the wet state (566,477; 767,400 and 1,136,700). The calculated profile was gained by using both the KT and the MEPDG model and the responses were used from both MLET and FE analyses. In none of the cases did the calculated rutting profile reach the same shape of the measured one, as the calculated amount of rutting was more than the measured one a meter away from the centre of the tyres. The MEPDG model does not resemble well the amount of rutting in the initial stages of the wet state but has a reasonable correlation in the moist state and to the maximum value at the end of the wet state. The KT model had a reasonably good correlation with the measurements after various numbers of load repetitions when using the responses calculated with MLET but the maximum rutting was not reached at the end of the test when responses from the FE analysis were used. Both these models are semi-empirical and more work is required to improve their accuracy for various load and environmental situations (Paper V – Saevarsdottir & Erlingsson, 2014).

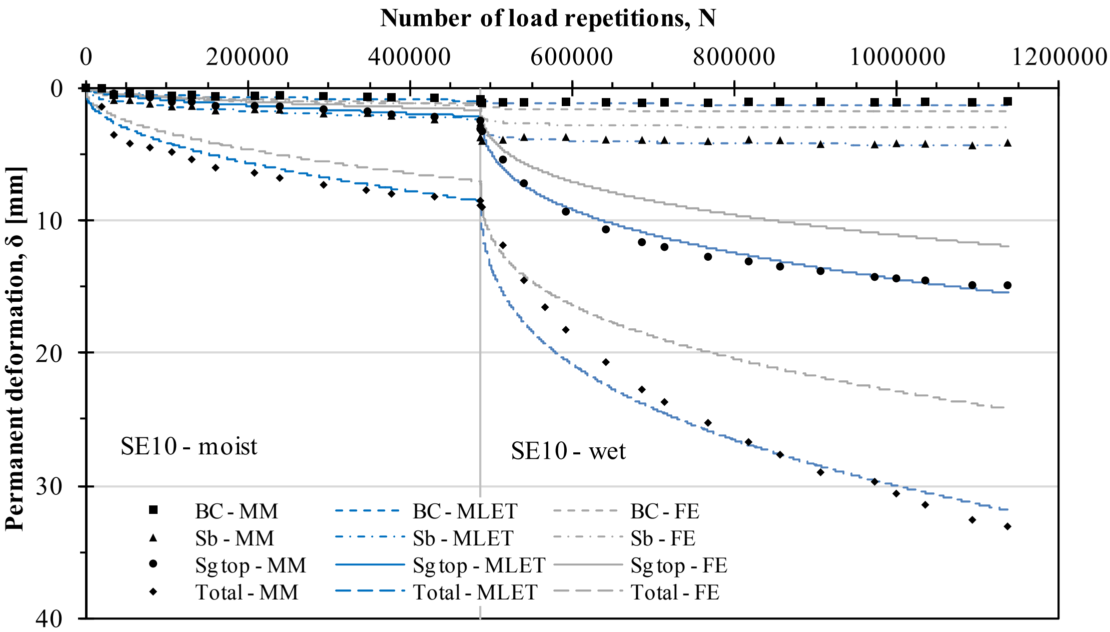

Figure 3.13 Permanent deformation in the unbound layers and the total deformation as a function of load repetitions; calculated values were obtained from the KT model.

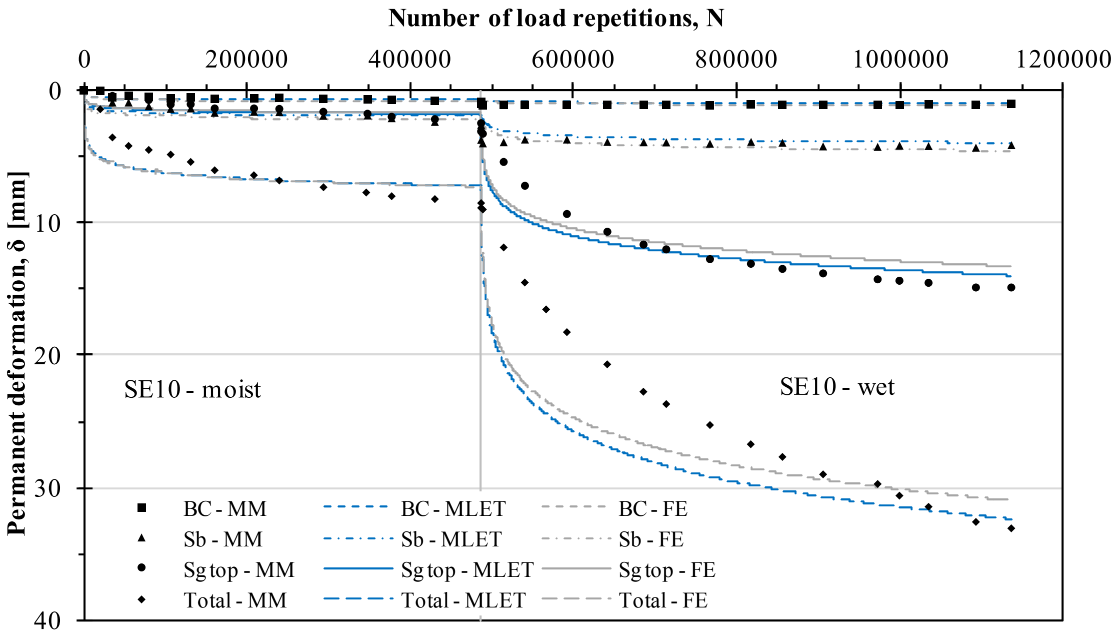

Figure 3.14 Permanent deformation in the unbound layers and the total deformation as a function of load repetitions; calculated values were obtained from the MEPDG model.

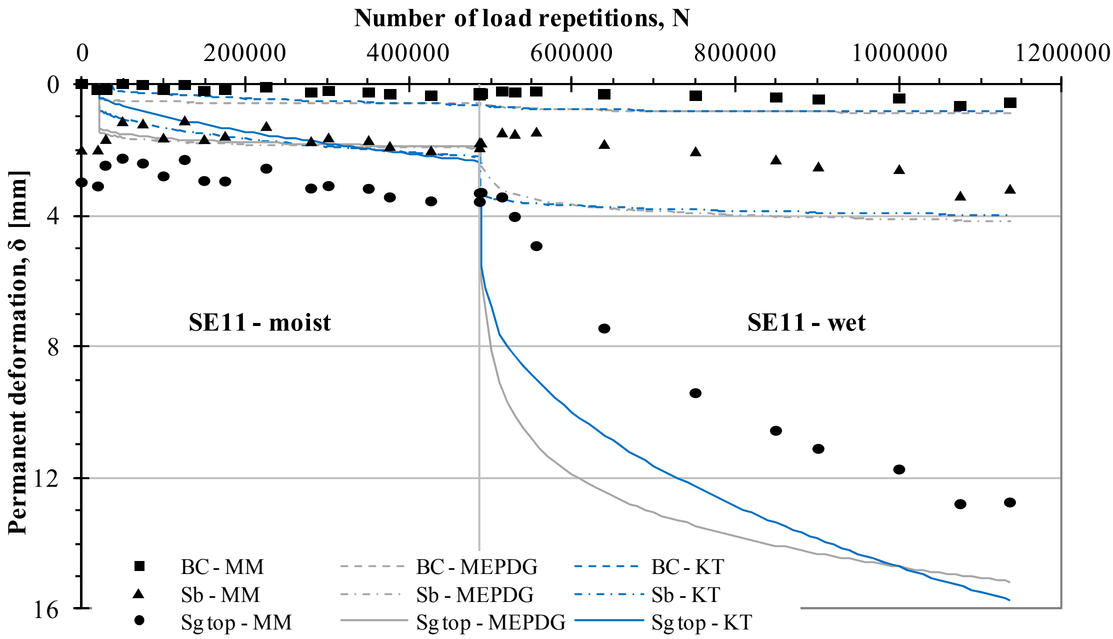

Figure 3.15 Permanent deformation in the unbound layers of SE11 as a function of load repetitions; the calculated values were obtained by using the KT model.

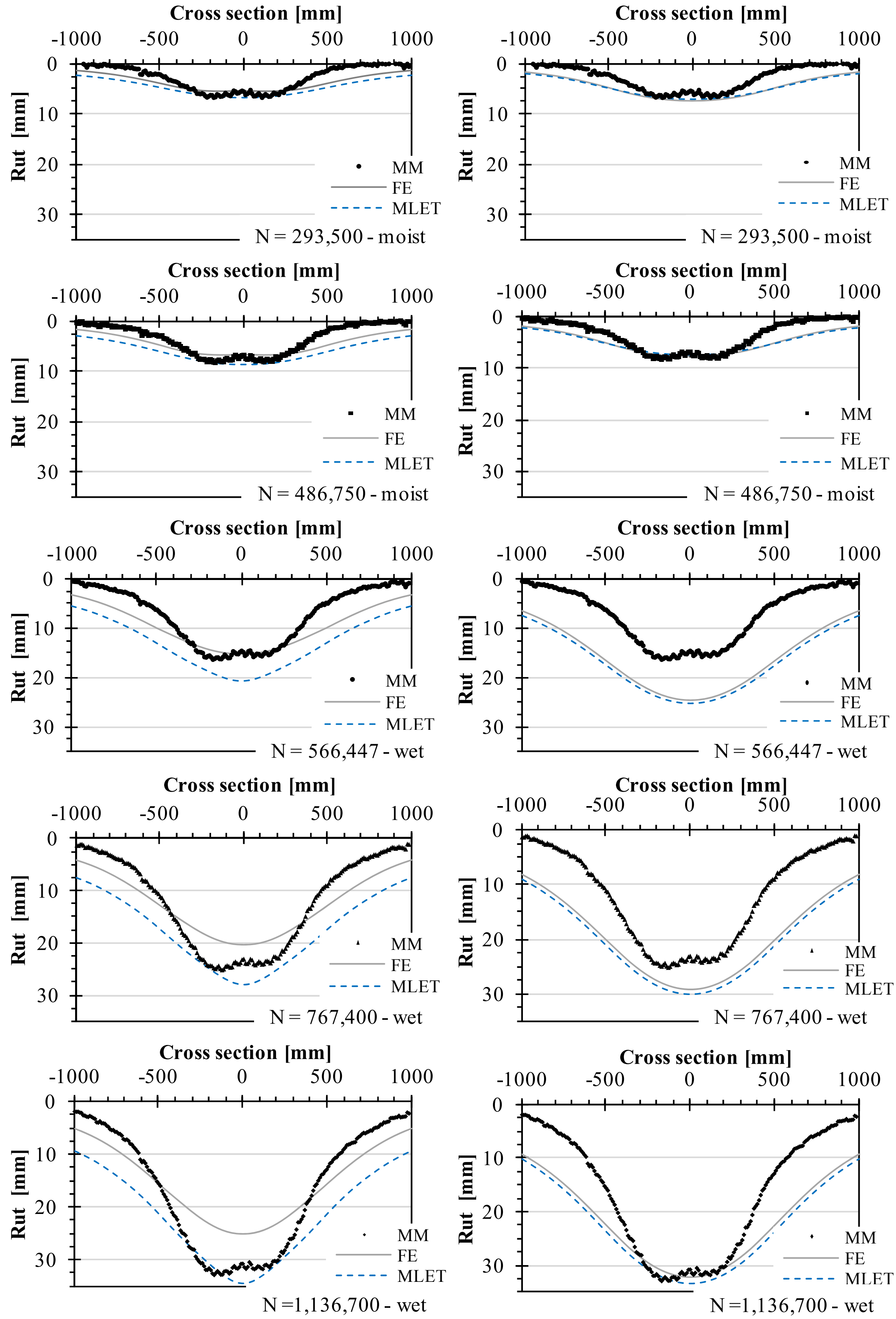

Figure 3.16 Cross section of the rutting profile of SE10 after different numbers of load repetitions. The calculated values were obtained by using the KT model (left) and the MEPDG (right).

4 Conclusion and future work

Instrumented flexible pavement structures tested in an APT with an HVS machine have been analysed here. The HVS test was performed on full scale test roads, where the magnitude and location of the applied loads, the number of load repetitions and the environmental conditions were controlled. At regular intervals condition surveys and pavement response measurements were performed, providing valuable data.

The HVS tests were divided into three phases, a pre-loading phase, a response phase and a main accelerating loading phase. In the response phase various traffic loads were applied to the structures but in the main accelerated testing phase a dual tyre with constant load and pressure was applied. In the main accelerated testing phase the environmental conditions were on the other hand changed. Half-way through the test the water level was increased significantly. That might become a more frequent event with climate change as the time between severe rainfalls might shorten. The increased water level gave the opportunity to estimate what influence increased moisture content would have on the behaviour of the pavement structures as a whole as well as individual material layers within the structure. The raised water level had a significant effect, with increased water content in the unbound material layers causing reduced resilient stiffness and increased accumulation of permanent deformation.

The testing procedure was found to be good, as two equivalent flexible pavement structures were tested and comparable results gained. When examining the instrumentation two out of three sensors showed similar results, which was assuring that valid measurements were being made in the unbound materials. Unbound materials are hard to measure with many factors influencing the results such as compaction and the quality of the contact between the material and the measuring instruments.

The results from HVS (APT) tests can be used to improve the design of road structures by providing a performance prediction as a function of load repetitions for different materials, structures, climate conditions and traffic. The HVS machine can be used to evaluate new road concepts and maintenance strategies by assessing the total cost of constructing, maintaining and operating various road constructions under different conditions. A life cycle cost analysis comparison between investment alternatives of feasible designs can then be performed, which can be used to decrease costs and environmental impact of new road structures, as well as reconstruction and repairs to old structures.

4.1 The response behaviour

The response signals were monitored and compared with calculated values using a 2D axisymmetric MLET method and a 3D FE method. The bitumen bound layers and subgrade were treated as linear elastic materials whilst a nonlinear stress dependent model was used for the base and subbase layers. Generally good agreements were found between the measured responses, from the instruments within the structures (HVS) and from FWD measurements, and calculated values using both methods with no significant difference between them. More difference between the two methods was expected as MLET and FE use different models to calculate the stress dependence of the base and subbase layers as well as using a circular loading area in MLET and a square loading area in the FE analysis. It is likely that the difference between the two methods was more pronounced in the uppermost part of the structure, but that could not be observed here.

The results showed clearly the great influence increased moisture content had on the pavement structures. All the unbound layers showed higher signals of induced vertical strain and lower signals of induced vertical stress, despite some of the upper layers having only a minimum increase in moisture content. All the moisture content sensors were placed above the groundwater table but all showed increased moisture content with the increased water level. The increased moisture content caused reduced resilient modulus in all the unbound layers, resulting overall in a softer structure.

There are a few aspects regarding the responses of the pavement structures as the water table was raised that need further investigation. The change in the vertical strain and the resilient modulus was found to be higher in the subbase compared to the subgrade, despite a larger increase in the water content and a higher fine content in the subgrade material compared to the subbase. A possible explanation is that the subgrade material might be too weak to support the reasonably good quality subbase material. The base layer had a lower stiffness compared to the subbase material. The materials showed similar performance in laboratory tests and were obtained from the same quarry so it is assumed that the relatively thin base layer might not reach sufficient compaction. Another possibility is that the meters are not representative of the material layer they are to measure but FWD measurements had a reasonable agreement with back calculated values. The same trend was observed between the top and bottom of the subbase layer where higher stiffness was found in the bottom half despite the higher induced stress values.

Generally the tensile strain increases as the moisture content increases. In SE10 and SE11 this did not happen as in most cases the transversal tensile strain reduced as the groundwater table was raised. The longitudinal tensile strain meanwhile was reasonably constant between moist and wet states. When examining results from FWD measurements over the sensors the same trend was observed for 30, 50 and 65kN applied load intensities. There are many factors that affect the tensile strain such as the wheel configuration, the tyre type and pressure, the material properties, the speed of the loading wheel and the applied load but this unexpected reduction in tensile strain needs to be investigated further.

4.2 The permanent deformation

The accumulation of permanent deformation increased significantly when the groundwater table was increased. The higher level of the ground water table caused increased moisture content in all the unbound material layers. The increase in accumulation of permanent deformation was visible in all the unbound material layers but the increase was most significant in the subgrade layer where the highest increase in moisture content was recorded.

The accumulation of permanent deformation of the unbound layers was modelled using two simple work hardening material models where lateral wander and change in moisture content were accounted for by using the "time hardening" procedure. One of the models was a stress dependent model developed by Korkiala-Tanttu (KT) (equation 21) and the other one was a strain dependent presented in the MEPDG (equation 14). Both models gave reasonable correlation with measured values, both before and after raising the ground water table. Both these models are semi-empirical and more work is required to improve their accuracy for various road materials, load and environmental situations but this work is a step towards that direction.

When using the MEPDG model, similar results were gained when using the MLET and FE analyses. The MLET and FE analyses showed similar stress response results but the change in the amount of permanent deformation when using the KT model was considerably larger, indicating some sensitivity to slight changes in the input values. When estimating the accumulated deformation with the KT model the parameter b was reduced as the water content increased, though the opposite behaviour had previously been found. The value of the parameter C increases with increased water content but in this research the increase was significantly larger than reported in the literature. When predicting the total deformation, the calibration factor β_1 in the MEPDG model had to be changed depending on the depth in the subgrade layer. The β_1 reduced with depth and the reduction was higher in the wet state compared to the moist state. Possible explanations are increased compaction or increased lateral pressure that in turn increases the interlocking between the material particles with depth, but this needs to be investigated further.

Both models gave reasonable results when looking at the maximum rutting value at the end of the moist and wet states. The calculated rutting profile, however, did not reach the same shape as the measured one, as the calculated amount of rutting was more than the measured one a metre away from the centre of the tyres. The MEPDG model does not resemble well the amount of rutting in the initial stages of the wet state while the KT model had a reasonably good correlation with the measurements after various numbers of load repetitions, when using the responses calculated with MLET.

4.3 Limitations and future studies

In every research there are some limitations. In this case one of the main limitations was the number of tested and analysed pavement structures. It would have been interesting to look at more varied sections with, for example, different subgrade material to see how that would change the behaviour of the subbase material, and to see how the tensile strain at the bottom of the asphalt bound layers would behave if the base course were strengthened and various axle and tyre configurations, loads and tyre pressures were applied. More tests are also required to better validate the performance models. The main emphasis in this work was on responses and permanent deformation of flexible pavement structures but flexible pavements experience several other distresses such as fatigue cracking, material disintegration, and roughness that were not considered here.

The raised water level had a significant effect on the structural behaviour as it increased the water content in the unbound material layers, thus reducing the resilient stiffness and increasing the accumulation of permanent deformation. There are several other factors that affect the resilient response and accumulation of permanent deformation of granular materials. These factors include stress level, density, grading of the material, the aggregate type and the shape of the particles. Further investigations are needed to find the relationship between water and these factors as well as the combined effect on the resilient properties and permanent deformation of unbound materials.

It is hard or impossible to estimate how much, if any, influence the instrumentation has on the structure as limited measurements can be done without any instrumentation. Here FWD measurements were performed and compared with the response measurements gained from the pavement structure but it would be interesting to do more non-destructive tests for comparison and validation of the instruments used.

The application of the results of laboratory tests of the material properties over to in-field properties is an endless battle. The laboratory and FWD results were used to estimate the material properties of the material layers in the pavement but as such factors as compaction and moisture content have a great influence on the parameters this is really hard to do.

Here an APT was being performed and therefore ageing of the materials was not taken into account and the structures might not have had time to heal between load repetitions. The test was controlled and the pavement structures did not undergo the various environmental conditions applied to an operative structure. The controlled environment, on the other hand, does provide an idea of the effect some limited factors have on a pavement structure without the influence of other affecting factors. Ageing, various environmental conditions, various load applications as well as other factors affecting a pavement structure need to be considered in an M-E designing procedure. It would be interesting to compare a functional pavement structure that has been monitored with a comparable structure tested with an APT to validate the models as well as the model parameters for various seasonal and traffic conditions.

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Appended paper