Response and Permanent Deformation Modelling of an Instrumented Flexible Pavement Structure

Th. Saevarsdottir* & S. Erlingsson* **
** Dept. of Civil and Environmental Engineering, University of Iceland, Reykjavik, Iceland*
*** Swedish National Road and Transport Research Institute, VTI, Linköping, Sweden*

ABSTRACT: A flexible test road structure was built and tested in an Accelerated Pavement Test to investigate its performance behaviour for a validation in a mechanistic performance scheme. Halfway through the test, water was introduced to estimate the influence increased moisture content would have on the behaviour of the structure. The raised water level had a significant effect on the behaviour of the structure with decreased resilient modulus and increased rate of accumulation of permanent deformation. Numerical analyses using the multi-layer elastic method (ERAPAVE) as well as finite element analysis (PLAXIS 3D, version 2) have been carried out to simulate the pavement responses, with the material properties based on field and laboratory testing. Thereafter, based on the numerical analyses the observed accumulation of permanent deformation of the unbound layers was modelled using work hardening material models. Generally good agreement was established between the measurements and calculations.

KEY WORDS: Unbound granular materials, water impact, accelerated pavement testing, numerical analysis, finite element analysis.

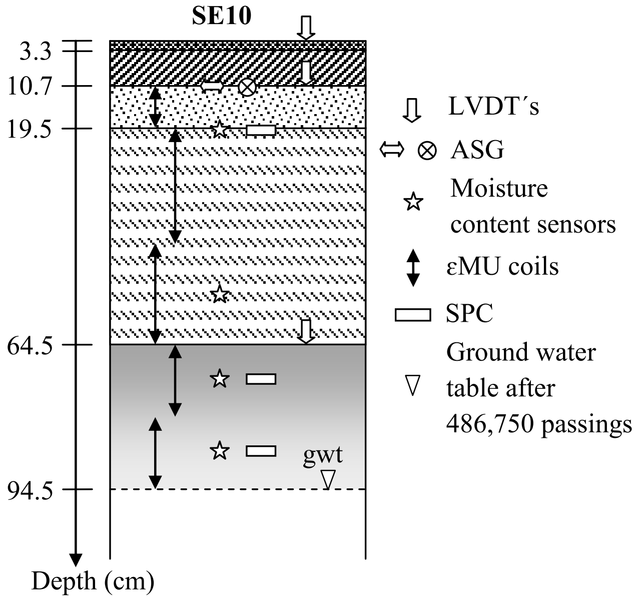
1. INTRODUCTION

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. For the development of mechanistic designing methods, the behaviour and properties of the materials used needs to be properly understood.

An Accelerated Pavement Test (APT) using a Heavy Vehicle Simulator (HVS) (referred to as SE10), was performed on a flexible test road structure at the Swedish National Road and Transport Research Institute (VTI) test facility. The aim was to get good direct measurements of stresses and strains in a flexible pavement structure and to evaluate the structure's performance under "moist" and "wet" conditions. Over one million load cycles were applied, but half way through the test the water table was raised to estimate the influence of water on the response behaviour and permanent degradation development of the structure (Wiman, 2010).

2. THE PAVEMENT STRUCTURE AND TESTING PROCEDURE

The pavement was tested at VTI full-scale indoor pavement test facility under constant environmental conditions. The structure (Figure 1) 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, fine content ~6%) and a subbase layer (0-90mm, fine content ~3%). The subgrade consisted of fine graded silty sand with a high fine content of about 25% and over 90% of the grains under 0.5mm (Wiman, 2010, Saevarsdottir & Erlingsson, 2013a & 2013b).



εMU coils (inductive coils) to measure the vertical strain (elastic and permanent), at five depth intervals and three coils at each depth range.

Soil pressure cells (SPC) to obtain vertical stresses, placed at three depths and three sensors at each depth.

Linear variable differential transducers (LVDT's) to measure the vertical deflection in relation to the bottom of the test pit.

Asphalt strain gauges (ASG) (H-bar) to obtain the horizontal strain at the bottom of the asphalt bound layers, with four gauges measured in the longitudinal and four in the transverse direction.

Moisture content sensors to measure the volumetric water content at four depths, with two sensors in SE10.

Figure 1: A cross-section of the pavement structure SE10, consisting of asphalt concrete, bituminous base, base course, subbase and subgrade, and the instrumentation.

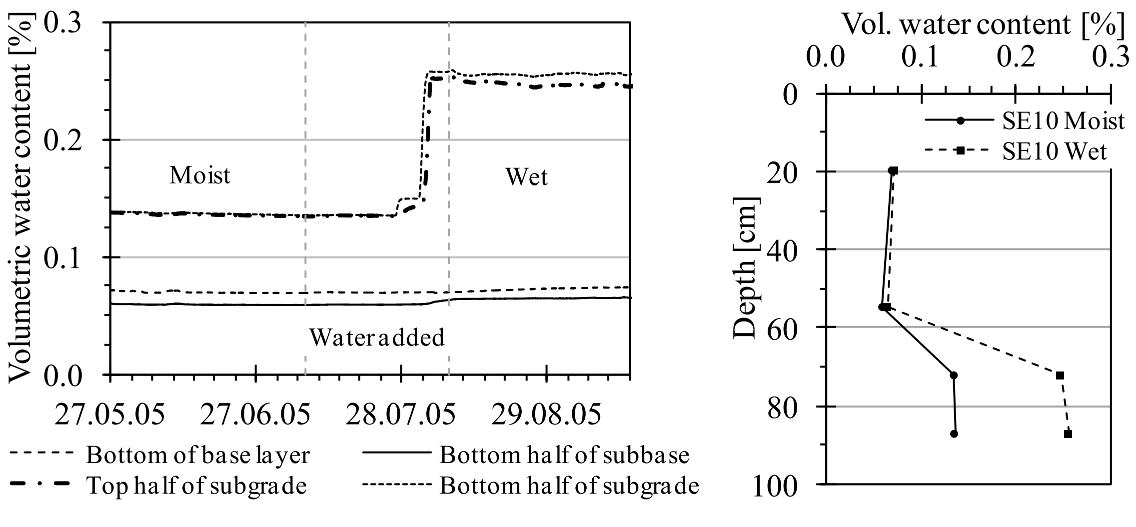


Figure 2: Change in the volumetric water content with time; and as a function of depth (measured at four depths). While the water was added no loading took place.

The test was divided into three phases but in all the phases bi-directional loading was applied; a pre-loading phase with 20,000 load repetitions applying light loading (30kN single wheel load and 700kPa tyre pressure); a response phase, where the responses were estimated from single and dual wheel configuration using various tyre pressures and axle loads and a main accelerating loading phase. The dual wheel configuration had a centre to centre spacing of 34 cm. The lateral distribution of the loading followed a normal distribution where the wander was divided into eleven segments in steps of 5cm, from plus to minus 25cm (Wiman, 2010).

In the main accelerating loading phase more than one million load cycles were applied with a dual wheel configuration, 120kN axle load and 800kPa tyre pressure. Water was added after 486,750 load repetitions, to the level of 30cm below the top of the subgrade (Figure 1). No other alternations were made and the test continued. The performance of the two states, "moist", before water was added and "wet", after water was added, were modelled. The volumetric water content (SE10), is shown in Figure 2. After introducing water the moisture content increased in all the unbound layers, despite all the moisture content sensors being above the increased groundwater table. This was probably due to capillarity suction. In the wet state; higher signals of induced vertical strain, lower signals of induced vertical stress and higher signals of induced horizontal strain at the bottom of the bituminous base were recorded. This indicates a decreased stiffness in the unbound layers or a softer structure as the groundwater table was raised (Saevarsdottir & Erlingsson, 2013a & 2013b).

3. RESPONSE MODELLING

The response modelling was modelled in two ways using a multi layer elastic theory (MLET) with the computer program ERAPAVE (Erlingsson & Ahmed, 2013) and by using the 3D finite element (FE) program, PLAXIS (3D foundation) (Brinkgreve, 2007). The bound layers and subgrade were treated as linear elastic materials, whereas the stiffness modulus for the granular materials (base and subbase) was handled as stress dependent.

The pavement structure was modelled in an axisymmetric analysis where the responses were calculated using a multi layer elastic theory (MLET) with a circular loading. To calculate the stress dependent stiffness modulus, a normalized form of the k - θ expression was used:

$$M_r = k_1 p_a \left(\frac{3p}{p_a} \right)^{k_2} \tag{1}$$

where k_1 and k_2 are experimentally determined constants, p is the mean normal stress level of the loading ($p = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$, σ_1, σ_2 and σ_3 are principal stresses) and p_a is a reference pressure (p_a = 100kPa) (Huang, 2004, Erlingsson, 2010, Erlingsson & Ahmed, 2013).

In the FE analysis a hardening soil model was used to calculate the stress dependency of the soil stiffness (Brinkgreve, 2007). The loading was applied with a constant tyre pressure on a square tyre imprint. The basic parameters for the soil stiffness in the hardening soil model are:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos \emptyset - \sigma'_3 \sin \emptyset}{c \cos \emptyset + p^{ref} \sin \emptyset} \right)^m \tag{2}$$

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cos \emptyset - \sigma'_1 \sin \emptyset}{c \cos \emptyset + p^{ref} \sin \emptyset} \right)^m \tag{3}$$

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cos \emptyset - \sigma'_3 \sin \emptyset}{c \cos \emptyset + p^{ref} \sin \emptyset} \right)^m \tag{4}$$

where m is the power for stress-level dependency of stiffness; E_{50}^{ref} is the secant stiffness in standard drained triaxial test; E_{oed}^{ref} is the tangent stiffness for primary oedometer loading; E_{ur}^{ref} is the unloading / reloading stiffness ($E_{ur}^{ref} = 3E_{50}^{ref}$); c is the cohesion; ϕ is the friction angle and p_{ref} is the reference confining pressure (p_{ref} = 100kPa).

4. PERMANENT DEFORMATION PREDICTION MODELLING

The accumulation of the vertical strain in the unbound pavement materials was modelled according to a procedure developed by Korkiala-Tanttu (KT) (2008) and by the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004). The main difference between the two models is that the KT model is stress based while the MEPDG model is strain dependent.

The KT model (Korkiala-Tanttu, 2008) is a simple work hardening material model for unbound material:

$$\hat{\varepsilon}_p(N) = C \cdot N^b \cdot \frac{R}{A - R} \tag{5}$$

where $\hat{\varepsilon}_p$ is the accumulated permanent strain, C is the material parameter depending on the compaction and saturation degree, N stands for the number of load repetitions, b is a shear ratio parameter depending on the material and stress state, R is the deviatoric stress ratio defined as, $R = \frac{q}{q_f} = \frac{\sigma_1 - \sigma_3}{q_0 + Mp}$, where $M = \frac{6 \cdot \sin\phi}{3 - \sin\phi}$ and $q_0 = \frac{c \cdot 6 \cdot \cos\phi}{3 - \sin\phi}$, defined by the static Mohr Coulomb failure envelope and A is the maximum value of R (here taken as 1.05).

The MEPDG model (ARA, 2004) is based on a best fit approaches from laboratory testing:

$$\hat{\varepsilon}_p(N) = \beta_1 \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^b} \varepsilon_v \tag{6}$$

where N stands for number of load repetitions; ε_0, ρ and b are regression parameters; ε_r is resilient strain imposed in a laboratory test to obtain ε_0, ρ and b; ε_v is the average vertical resilient strain in the layer as obtained from the primary response model and β_1 is a calibration factor.

In both models, KT and MEPDG, the granular layers can be divided into sublayers and the total deformation occurring in each layer is determined by summing the permanent deformation over the sublayers. The permanent deformation is thereafter gained by multiplying the permanent strain with the thickness of the layer (sublayer). The lateral wander of the traffic is further accounted for by taking the calculated principal stresses and elastic (resilient) strain representing the field conditions in the middle point of each layer over the area the wheels travel over. Hu et al. (2011) and Ahmed and Erlingsson (2013) state that a time-hardening procedure appears to provide a reasonable approach when considering the effects of various stress magnitudes on the development of rutting. This approach was used here.

5. MATERIAL PROPERTIES

The material parameters for all layers of the pavement used in the numerical analyses of its response are given in Table 1 (MLET - ERAPAVE) and Table 2 (FE - PLAXIS). When estimating the material parameters results from laboratory and field tests were considered (Table 1 and 2) (Wiman, 2010, Saevarsdottir & Erlingsson, 2013a & 2013b). In Figure 3 the results from the falling weight deflectometer (FWD) are displayed, showing good agreement between the measured and calculated values. The FWD measurements show higher deflection in the wet state than in the moist state, indicating a softer structure once water was introduced.

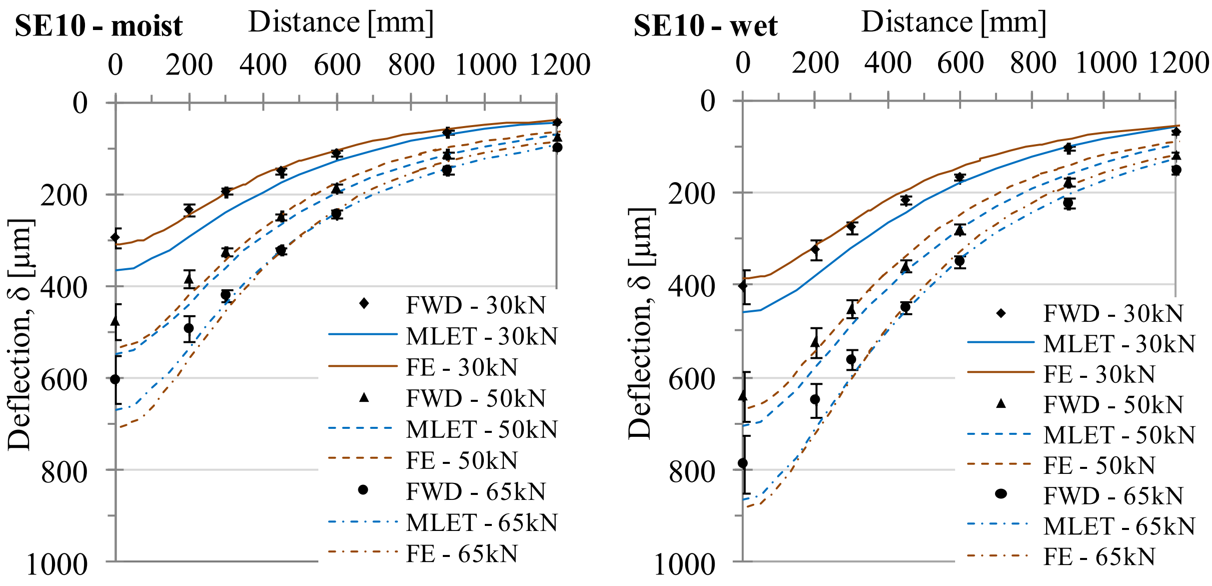


Figure 3: Comparison between FWD measurements, back calculations performed with MLET (ERAPAVE) and FE (PLAXIS), using 30, 50 and 65kN load intensity for "moist" (left) and "wet" (right) state.

Table 1: Material parameters in the response analyses using MLET (ERAPAVE).

		FWD	HVS	FWD & HVS		
		E/M_r [MPa]	E/M_r [MPa]	k_1 [-]	k_2 [-]	γ [kN/m^3]
Asphalt concrete	Moist	6000	3500	-	-	24
	Wet					
Bituminous base	Moist	6000	3500	-	-	24
	Wet					
Unbound base	Moist	-	-	500	0.6	20
	Wet			400		
Unbound subbase – upper half	Moist	-	-	1450	0.6	19
	Wet			1150		
Unbound subbase – lower half	Moist	-	-	2850	0.6	19
	Wet			1550		
Subgrade	Moist	80	50	-	-	16
	Wet	60	45			

In table, E – Young modulus of bound materials; M_r – Resilient stiffness of unbound materials; γ – Unit weight.

The material parameters for the HVS testing were optimized for dual wheel configuration, under the centre of one of the tyres, with a 120kN applied axle load and 800kPa tyre pressure. Following calculations (Chapter 6 and 7) were made under the same conditions. The Poisson´s ratio (v) was set as 0.35 for all layers. When performing a stress dependent analysis (Equations 1 to 4), k_2 and m were set as a constants while k_1, $E_{50}{}^{ref}$, $E_{oed}{}^{ref}$ and $E_{ur}{}^{ref}$ reduced with increased moisture content (Rahman & Erlingsson, 2012; Li & Baus, 2005). Comparing the material parameters used in the MLET and the FE analysis, it can be noticed that the ratio between moist and wet is similar as well as the ratio between the upper and lower half of the subbase. The ratio between the base and subbase layers is on the other hand not the same.

Table 2: Material parameters in the response analyses using FE (PLAXIS).

		FWD	HVS	FWD & HVS						
		E_{ref} [MPa]	E_{ref} [MPa]	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	m [-]	c [kPa]	ϕ [°]	γ [kN/m^3]
Asphalt concrete	Moist	6000	3500	-	-	-	-	-	43	24
	Wet									
Bituminous base	Moist	6000	3500	-	-	-	-	-	43	24
	Wet									
Unbound base	Moist	-	-	115	108	345	0.6	40	43	20
	Wet			92	86	276		36		
Unbound subbase – upper half	Moist	-	-	180	168	540	0.6	40	43	19
	Wet			142	133	426		36		
Unbound subbase – lower half	Moist	-	-	360	336	1080	0.6	40	43	19
	Wet			194	181	582		36		
Subgrade	Moist	80	50	-	-	-	-	-	43	16
	Wet	60	45							

In table, E_{ref} – Young modulus; γ – Unit weight.

In Tables 3 and 4 the parameters used in the permanent deformation predictions are given. The parameters used in the KT model are estimated based on the materials being used as well as compaction, saturation degree and stress state where appropriate. The cohesion, c, was reduced by 10% from moist to wet state in the base and subbase layers and by 50% in the subgrade (Theyse, 2002; Matsushi & Matsukura, 2006). In the MEPDG model the material properties ρ, b and $\varepsilon_0/\varepsilon_r$ were obtained using equations developed by ARA (2004), the gravimetric water content (W_c) was based on measurements and the calibration factor β_1 was calibrated to get a reasonable resemblance between the measured and calculated values.

Table 3: Material parameters of the unbound layers used to predict the permanent deformation with the KT model.

		c [kPa]	ϕ [°]	C [10^{-4}] [-]	b [-]
Unbound base (granular)	Moist	40	43	1.1	0.35
	Wet	36		75	0.05
TOP Unbound subbase (granular)	Moist	40	43	0.6	0.33
	Wet	36		40	0.05
BOTTOM Unbound subbase (granular)	Moist	40	43	0.5	0.31
	Wet	36		30	0.05
Subgrade	Moist	14	35	0.05	0.55
	Wet	7		1.5	0.3

Table 4: Material parameters of the unbound layers used to predict the permanent deformation via MEPDG model.

		W_c [%] *	b [-]	ρ [-]	$\varepsilon_0/\varepsilon_r$ [-]	β_1 [-]
Unbound base (granular)	Moist	3.3	0.214	1778	21.2	0.41
	Wet	3.4	0.213	1833	21.2	0.47
TOP Unbound subbase (granular)	Moist	3.0	0.216	1624	21.2	0.70
	Wet	3.2	0.215	1704	21.2	1.02
BOTTOM Unbound subbase (granular)	Moist	3.0	0.216	1624	21.2	0.70
	Wet	3.4	0.214	1789	21.2	1.02
Subgrade 64.5-94.5cm	Moist	7.7	0.179	8036	22.6	1.0
	Wet	16.1	0.127	467485	29.2	8.3
Subgrade 94.5-144.5cm	Moist	7.7	0.179	8036	22.6	0.9
	Wet	18.4	0.116	1996957	32.5	6.0
Subgrade 144.5-194.5cm	Moist	7.7	0.179	8036	22.6	0.8
	Wet	18.4	0.116	1996957	32.5	4.0
Subgrade 194.5-244.5cm	Moist	7.7	0.179	8036	22.6	0.7
	Wet	18.4	0.116	1996957	32.5	2.0

* The optimum gravimetric water content for the base and subbase is around 4-5% whilst for the subgrade it is around 13%.

6. RESPONSE ANALYSIS

The measured (MM) and calculated (MLET; FE) induced vertical strains for both "moist" and "wet" states are shown in Figure 4. The vertical lines are the average of measured vertical strains over the depth interval whilst the dotted lines represent the calculated strain, using MLET (ERAPAVE) and FE analysis (PLAXIS). Some difference is observed between the FE and the MLET calculations but both methods give reasonably good agreement between response measurements and calculations.

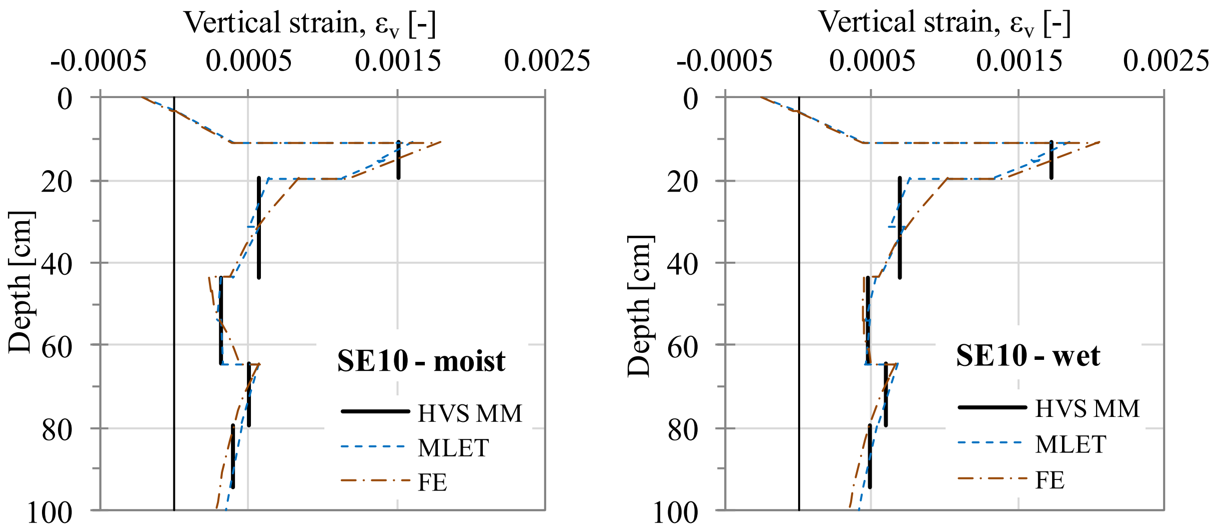


Figure 4: Vertical resilient strain as a function of depth (moist state - left; wet state – right).

As water was added, the moisture content of the unbound layers increased, causing the stiffness of the unbound layers to decrease and higher strains to be registered. The volumetric water content increased with depth (Figure 2) and therefore both the difference in vertical strain and the difference in the resilient modulus were higher in the bottom of the subbase than in the base (Table 1 & Figure 4). With increased fine content in the subgrade material compared to the subbase, the increased moisture content should have more effect on the resilient response in the subgrade than in the subbase but that was not found to be the case here.

7. PERMANENT DEFORMATION

The measurement (MM) of accumulated permanent deformation as a function of load repetition and the predicted deformation are displayed in Figure 5, using the KT-model and in Figure 6, using the MEPDG model. The responses were gained from MLET (ERAPAVE) and from the FE (PLAXIS) analysis.

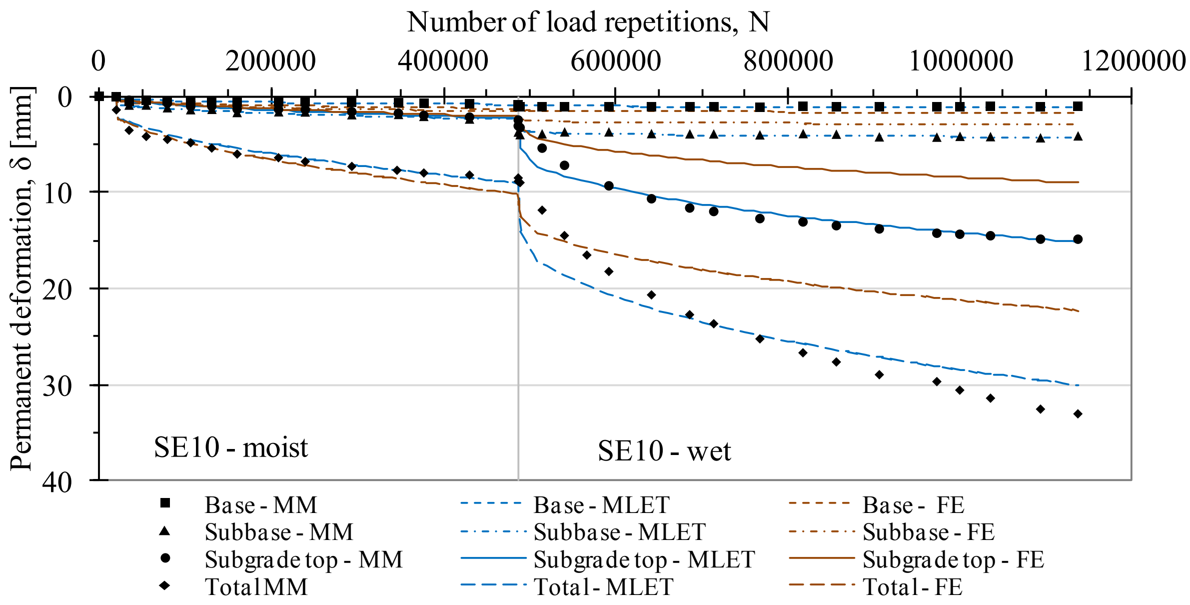


Figure 5: Permanent deformation in the unbound layers as a function of load repetitions; the calculated values were obtained by using the KT model.

In Figures 5 and 6 the permanent deformation is shown for the base course, the subbase, the top 30cm of the subgrade and the total deformation of the structure. The measured deformation of the asphalt bound layers was less then 1mm and therefore not taken into account. Some difference was observed between the FE and MLET calculations. Reasonable results were gained when using the MEPDG model but when using the FE analysis in the KT model the "wet" deformation was underestimated in all layers except the base. This is probably due to underestimated stress values in the "wet" state of the FE analysis. In all layers the raised groundwater table accelerated the development of the permanent deformation, but the base layer showed the smallest increase whilst the subgrade showed the largest and greatest extent of increase in permanent deformation.

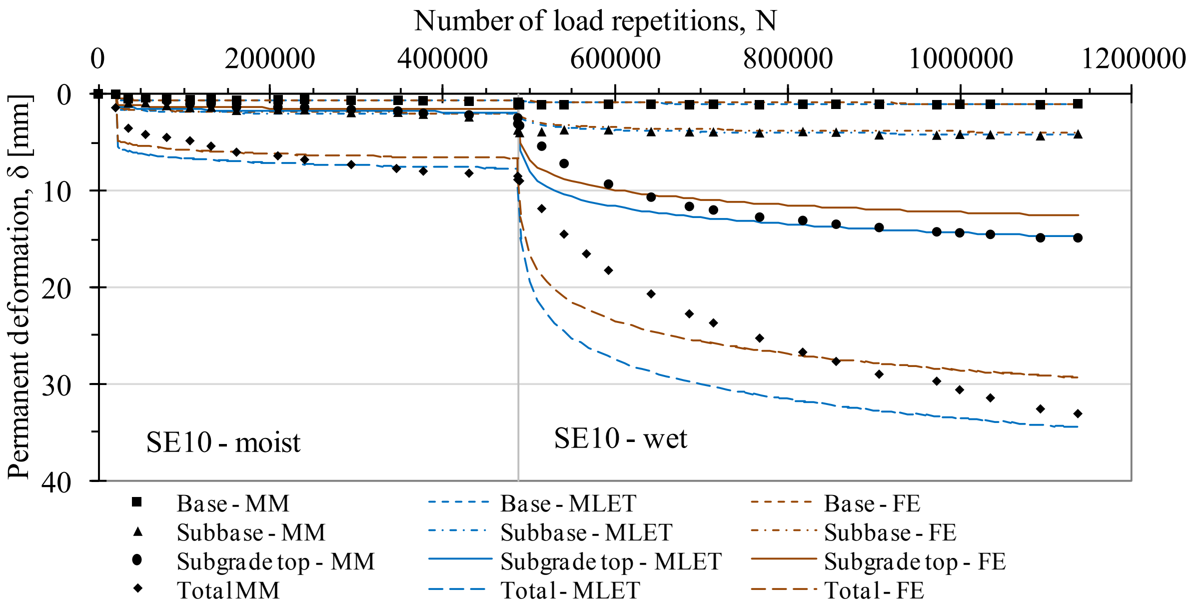


Figure 6: Permanent deformation in the unbound layers as a function of load repetitions; the calculated values were obtained by using the MEPDG model.

8. CONCLUSIONS

Analysis of a flexible pavement structure tested in an APT with an HVS machine is presented here. More than one million load cycles were applied but halfway through the test the water table was raised, giving the opportunity to estimate the influence of water on the response and performance of the structure. The responses as well as the permanent deformation were monitored and compared with calculated values.

The responses 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. Two methods were used to calculate the responses, MLET using ERAPAVE and FE analysis using PLAXIS. Generally good agreements were found between the measured responses and calculated values. The raised water level had a significant effect on the structure as it increased the water content in the unbound material layers, causing the resilient stiffness to reduce.

The accumulation of permanent deformation of the unbound layers was modelled using two simple work hardening material models, one developed by Korkiala-Tanttu and one presented in the MEPDG, both models gave reasonable results. All the unbound layers showed increased permanent deformation as the water content increased, with the most dramatic increase in the subgrade where the largest increase in the water content was observed.

ACKNOWLEDGEMENTS

The work described in this paper was sponsored by the Icelandic Road Administration (ICERA), The University of Iceland Research Fund and Ludvig's Storr Culture and Development Fund. Data was used from the Swedish National Road and Transport Research Institute (VTI), the testing was performed in collaboration with The Swedish Transport Administration (TRV).

REFERENCES

Ahmed, A. W. & Erlingsson, S. (2013). Evaluation of permanent deformation models for unbound granular materials using accelerated pavement tests. *Road Materials and Pavement Design*. 14/1, 178-195. doi:10.1080/14680629.2012.755936.

ARA Inc. (2004). "*Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, Final report, NCHRP 1-37A*." Transportation Research Board of the National Academies, Washington, D.C., USA.

Brinkgreve, R.B.J. (2007). PLAXIS 3D foundation, version 2, manual. PLAXIS, Delft, Netherlands.

Erlingsson, S. (2010). Impact of Water on the Response and Performance of a Pavement Structure in an Accelerated Test. *Road Materials and Pavement Design*. 11/4, 863-880.

Erlingsson, S. & Ahmed, A. W. (2013). Fast layered elastic response program for analysis of flexible pavement structures. *Road Materials and Pavement Design*. 14/1, 196-210. doi:10.1080/14680629.2012.757558.

Hu, S., Zhou, F. & Scullion, T. (2011). Development, Calibration and Validation of a New M-E Rutting Model for HMA Overlay Design and Analysis. *Journal of Materials in Civil Engineering*. 23/2, 89-99.

Huang, Y. H. (2004). *Pavement Analysis and Design*. 2nd edition. Upper Saddle River, New Jersey, USA: Pearson Education Inc., Prentice Hall and Education Inc.

Korkiala-Tanttu, L. (2008). *Calculation method for permanent deformation of unbound pavement materials.* Espoo, Finland: Ph.D. thesis, VTT Technical Research Centre of Finland.

Li, T. & Baus, R. L. (2005). Nonlinear Parameters for Granular Base Materials from Plate Tests. *Journal of Geotechnical and Geoenvironmental Engineering*. 131/7, 907-913.

Matsushi, Y. & Matsukura, Y. (2006). Cohesion of unsaturated residual soils as a function of volumetric water content. *Bulletin of Engineering Geology and the Environment.* 65/4, 449-455.

Saevarsdottir, Th. & Erlingsson, S. (2013a). Effect of moisture content on pavement behaviour in a heavy vehicle simulator test. In press, *Road Materials and Pavement Design*. doi:10.1080/14680629.2013.774762.

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

Rahman, S. & Erlingsson, S. (2012). Moisture Sensitivity of Unbound Granular Materials. *Proceedings of the 4th European pavement and asset management conference (EPAM4), 5-7 September 2012*. Malmö, Sweden, CD-ROM.

Theyse, H. L. (2002). *Stiffness, Strength and Performance of Unbound Aggregate Material: Application of South African HVS and Laboratory Results of California Flexible Pavements*. Report for the California Pavement Research Program, University of California, Pavement Research Cener.

Wiman, L. G. (2010). *VTI rapport 628, Accelererad provning av vägkonstruktioner; Referensöverbyggnad enligt ATB Väg*. Linköping, Sweden: Swedish National Road and Transport Research Institute (väg- och transportforskningsinstitut – VTI). (*In Swedish*)