Deformation Modelling of Unbound Materials in a Flexible Pavement

Saevarsdottir, Thorbjorg - Corresponding author
EFLA Consulting Engineers, Iceland, thorbjorg.saevarsdottir@efla.is
University of Iceland, Iceland, ths134@hi.is

Erlingsson, Sigurdur
VTI, Sweden
University of Iceland, Iceland

ABSTRACT
A thin flexible test road structure was built and tested in an Accelerated Pavement Test (APT) using a Heavy Vehicle Simulator (HVS) to monitor its performance behaviour. The structure was 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. In the test more than one million load cycles were applied, but mid-way the water table was raised. The raised water level had a significant effect, decreasing the resilient modulus and increasing the rate of accumulation of permanent deformation. The pavement structure has been modelled in an axisymmetric analysis where the responses have been calculated using a 2D multi layer elastic theory (MLET) as well as a 3D finite element method (FEM). The two methods agreed well with the measurements. Further was the observed accumulation of permanent deformation of the unbound layers modelled using two simple work hardening material models; a stress based model and a strain based model. The calculated permanent deformation is compared to the measured one before and after raising the ground water level. The stress based model was found to be sensitive to slight response changes but had a closer fit than the strain dependent model.

Keywords: Unbound granular materials, heavy vehicle simulator, numerical analysis, permanent deformation, rutting.

1 INTRODUCTION

The behaviour of pavement structures 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. Today most of pavement structure design is done with empirical methods, but for the development of mechanistic designing methods, the behaviour and properties of pavement materials need to be properly understood. In order to evaluate and calibrate the mechanistic models, it is important to compare the theoretical results with actual measurements in full scale pavement structures.

The many factors that must be considered in flexible pavement design are all easily within the capabilities of three dimensional (3D) finite element (FE) analysis. But FE analyses are computationally expansive and therefore the required computation time can be impractically long for a routine design (Schwartz, 2002; Loulizi et al., 2006). The multi-layer elastic theory (MLET), is commonly used in mechanistic empirical (M-E) pavement design where response calculations are performed several times. MLET is an axisymmetric analysis that can be extended by the superposition principle for multiple wheel loads.

Accelerated pavement tests (APT) of instrumented structures have increased the understanding of pavement behaviour and built a foundation for new, more sophisticated design methods. An APT using a Heavy Vehicle Simulator (HVS) was performed on an instrumented flexible test road structure (referred to as SE10) at the Swedish National Road and Transport Research Institute (VTI) test facility in 2005. At regular intervals condition surveys and pavement response measurements were performed, providing valuable data. The aim was to get reliable direct measurements of stresses and strains in a flexible pavement structure and to evaluate the structure's performance under “moist” and “wet” conditions, with the water table raised half way through the test (Wiman, 2010).

The response signals gained from the testing were compared with calculated values using MLET and a 3D FE program. The accumulation of permanent deformation and the rutting profile were further modelled using two simple work hardening material models, and the difference evaluated.

2 THE PAVEMENT STRUCTURE AND TESTING PROCEDURE

The HVS machine (HVS Mark IV) is a mobile linear full-scale accelerated road-testing machine with a heating/cooling system to keep a constant pavement temperature. A cross section of the tested structure including the instrumentation embedded within the structure can be seen in Figure 1 and the materials used are listed in Table 1 (Wiman, 2010; Saevarsdottir & Erlingsson, 2013; Saevarsdottir et al., 2014).

In the APT, the structural responses were measured from a single and dual wheel configuration with various tyre pressures and axle loads. Further, a main accelerating loading phase was carried out under constant environmental conditions at 10°C using a lateral distribution of the loading following a normal distribution. A dual wheel configuration was used with tyre type 295/80R22.5 and a centre to centre spacing of 34 cm. The dual wheel load was 60 kN and the tyre pressure 800 kPa. This resulted in a square imprint of contact width 23.5 cm and 16 cm in length corresponding in a circular loading area with a contact radius equal to 10.9 cm. In total 1,136,700 bi-directional load repetitions were applied during the test, but after 486,750 load repetitions water was gently added until it rose to the level of 30 cm below the top of the subgrade (Figure 1). This gave the opportunity to assess the influence moisture had on the response and performance of the structure as no other alternations were made (Wiman, 2010).

The “moist” case simulates a normal field situation with the groundwater table at great depth. The “wet” case simulates a supposedly worst case scenario, with water running in the trenches.

3 RESPONSE MODELLING

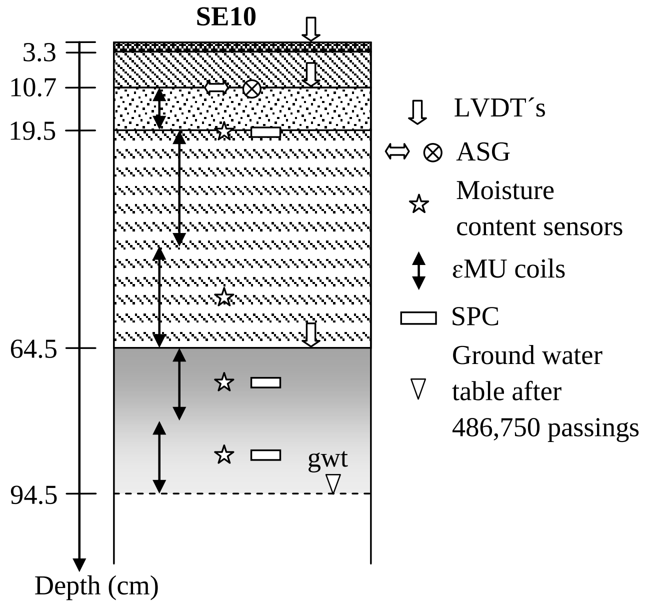
The responses were modelled using the computer program ERAPAVE (Erlingsson & Ahmed, 2013) which is MLET based and the 3D finite element (FE) program PLAXIS (3D foundation, version 2) (Brinkgreve, 2007). The bound layers and subgrade were treated as linear elastic materials, where the stiffness of the bound layers was adjusted according to the ambient temperature. The stiffness modulus for the granular materials (base and subbase) was treated as stress dependent.

The responses were calculated using MLET with a circular loading area (Figure 2) and extended by applying the superposition principle for the dual wheel configuration (Huang, 2004; Erlingsson & Ahmed, 2013). To calculate the stress dependent stiffness modulus, E_r, a normalized form of the k - θ expression was used (May & Witczak, 1981; Uzan, 1985; Lekarp et al., 2000a; Erlingsson, 2010):

$$E_r = k_1 p_{ref} \left(\frac{3p}{p_{ref}} \right)^{k_2} \tag{1}$$

where k_1 and k_2 are experimentally determined constants; p is the mean normal stress ($p=⅓(\sigma_1+\sigma_2+\sigma_3)$; σ_1, σ_2 and σ_3 are principal stresses) and p_{ref} is a reference pressure (p_{ref} = 100 kPa).

In the FE analysis a hardening soil (HS) model was used 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).



εMU coils - strain measuring units
vertical strain (recoverable and permanent)
SPC - soil pressure cells
vertical stresses
LVDT's - linear variable differential transducers
vertical deflection
ASG (H-bar) **-** asphalt strain gauges
horizontal strain at the bottom of the asphalt bound layers
Laser beam
surface profile
Moisture content sensors
volumetric water content

Figure 1 A cross-section of the pavement structure SE10, including the instrumentation embedded within the structure. The structure consisted of asphalt concrete, bituminous base, granular base course, subbase and subgrade.

Table 1 The materials used in the pavement structure SE10

Layer	Thick-ness [cm]	Description	Maximum aggregate size, d_{max} [mm]	Fine content [%]	Optimum gravimetric water content [%]
AC Asphalt concrete wearing course	3.3	AC pen 70/100	16	-	-
BB Bituminous base	7.4	AC pen 160/220	32	-	-
BC Base course	8.8	Unbound crushed rock (granite)	32	~ 6	4-5
Sb Subbase	45	Unbound crushed rock (granite)	90	~ 3	4-5
Sg Subgrade*	~235	Fine graded silty sand	4	25	13

**over 90% of the grains under 0.5 mm*

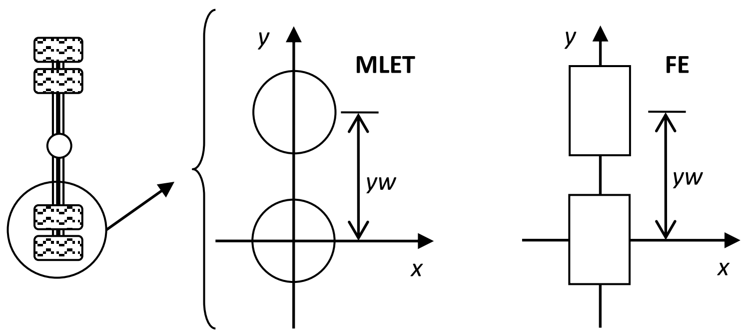


Figure 2 Plan view of the dual wheel setup in MLET (circular loading area) and in FE (square tyre imprint). The size of the imprint of the two setups is the same.

The loading was modelled with constant pressure on a square imprint (Figure 2). The basic parameters for the soil stiffness, E, in the hardening soil model are:

Triaxial modulus:

$$E_{50} = E_{50}^{ref}\left(\frac{c\cdot\cos\phi - \sigma_3'\cdot\sin\phi}{c\cdot\cos\phi + p_{ref}\cdot\sin\phi}\right)^m \quad (2)$$

Oedometer modulus:

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cdot\cos\phi-\sigma_1'\cdot\sin\phi}{c\cdot\cos\phi+p_{ref}\cdot\sin\phi}\right)^m \quad (3)$$

Un-/reloading modulus:

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cdot\cos\phi-\sigma_3'\cdot\sin\phi}{c\cdot\cos\phi+p_{ref}\cdot\sin\phi}\right)^m \quad (4)$$

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 ($E_{ur}^{ref} = 3E_{50}^{ref}$); c is the cohesion and ϕ is the friction angle (Brinkgreve, 2007).

4 THE RESPONSE BEHAVIOUR

Results from indirect tension tests (ITT) of the bituminous layers, repeated load triaxial (RLT) tests of the unbound materials, plate load (PL) tests and falling weight deflectometer (FWD) tests were considered when estimating the material parameters used in the numerical analyses of the pavement responses (Table 2) (Wiman, 2010; Saevarsdottir & Erlingsson, 2013). The material parameters were optimized for a dual wheel configuration, under the centre of one of the tyres, with an 800 kPa tyre pressure and a 60 kN applied dual tyre load. Poisson´s ratio (v) was set as a constant of 0.35 for all the layers. When performing the stress dependent analysis, k_2 and m were set as a constant while k_1, E_{50}^{ref}, E_{oed}^{ref} and E_{ur}^{ref} reduced with increased moisture content (Li & Baus, 2005; Rahman & Erlingsson, 2012). The cohesion, c, was reduced by 10% from “moist” to “wet” state (Theyse, 2002). When comparing the MLET and the FE analysis the ratio between the material parameters in “moist” and “wet” states was similar and between the upper and lower half of the subbase, but between the base and subbase layers some difference was observed. This might be related to the difference in calculation methods as MLET makes some assumptions that the FE method does not make. The FE analysis (square tyre imprint) should capture the behaviour of the asphalt layer better than an MLET analysis (circular loading area), but the difference between FE and MLET becomes insignificant deeper into the structure (Helwany et al., 1998; Huang, 2004; Saevarsdottir & Erlingsson, 2014).

All the figures in this section are from the main accelerated loading phase and the registration taken under the centre of one of the wheels.

In Figure 3, the vertical lines are the average of measured (MM) vertical strains over the depth interval (see Figure 1) whilst the dotted lines represent the calculated strain, using MLET and FE analysis. Some difference was observed between FE and MLET but both methods gave reasonably good agreement between response measurements and calculations. The increased moisture content in the “wet” state caused the stiffness of the unbound layers to decrease and higher strains and lower stresses to be registered (Saevarsdottir & Erlingsson, 2013).

Typical registrations measurements (MM) and calculated (MLET; FE) induced vertical strains and stresses for both “moist” and “wet” states are shown in Figures 4 and 5. The registrations were taken approximately in the middle of the “moist” and “wet” phases respectively. The calculated values were gained by moving the applied static load in short increments. In Figure 4 the vertical strain is displayed over two depth intervals; over the base layer (”moist”) and over the top of the subgrade (”wet”) (Figure 1). In Figure 5 the vertical stress is shown at two depths; at the bottom of the base (”moist”) and in the subgrade (”wet”) (Figure 1). No significant difference was observed between the calculated values using MLET and FE and both methods showed reasonable agreement with the measurements. The calculated vertical stress had some tendency to be underestimated in the subgrade.

Table 2 Material parameters used in the response analyses of the HVS tested structure.

Layer	State	MLET & FE			MLET	FE				
		E	*γ*	k_2 & m [-]	k_1 [-]	E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	*c* [kPa]	*Φ* [°]
Asphalt concrete	Moist	3500	24	-	-	-	-	-	-	43
	Wet									
Bituminous base	Moist	3500	24	-	-	-	-	-	-	43
	Wet									
Unbound base	Moist	-	20	0.6	500	115	108	345	40	43
	Wet				400	92	86	276	36	
Unbound subbase – upper half	Moist	-	19	0.6	1450	180	168	540	40	43
	Wet				1150	142	133	426	36	
Unbound subbase – lower half	Moist	-	19	0.6	2850	360	336	1080	40	43
	Wet				1550	194	181	582	36	
Subgrade	Moist	50	16	-	-	-	-	-	-	35
	Wet	45								

In table, E – Young modulus of bound materials and resilient stiffness of unbound materials [MPa]; γ – Unit weight [kN/m^3]; E_{50}^{ref}, E_{oed}^{ref} and E_{ur}^{ref} – [MPa].

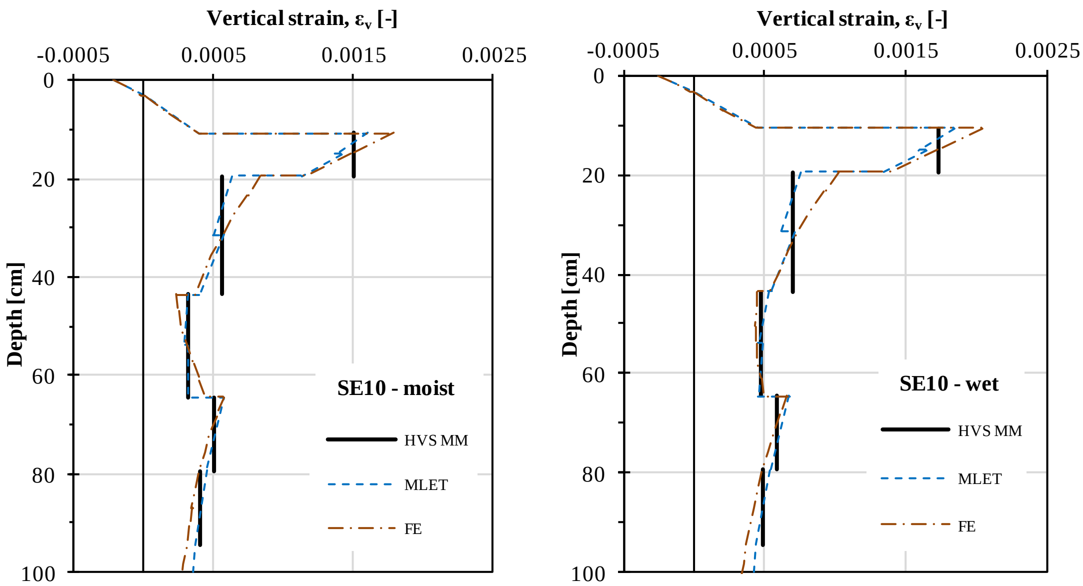


Figure 3 Vertical resilient strain as a function of depth ("moist" state - left; "wet" state – right).

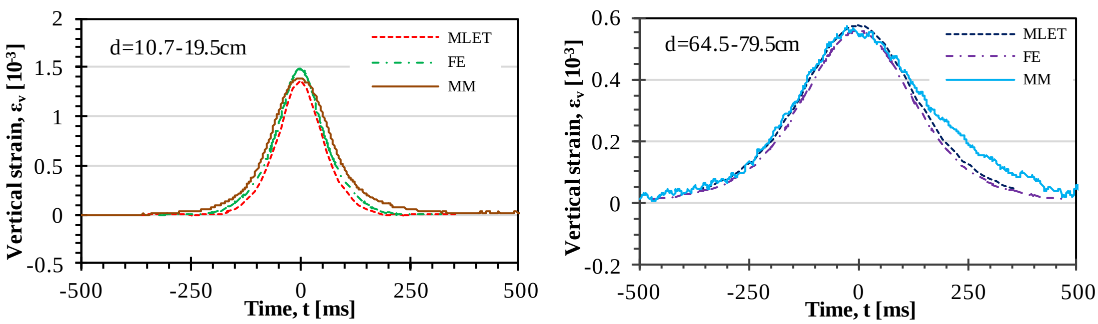


Figure 4 Induced vertical strain registration of εMU sensors and calculations, for depth ranges 10.7-19.5 cm ("moist") and 64.5-79.5 cm ("wet").

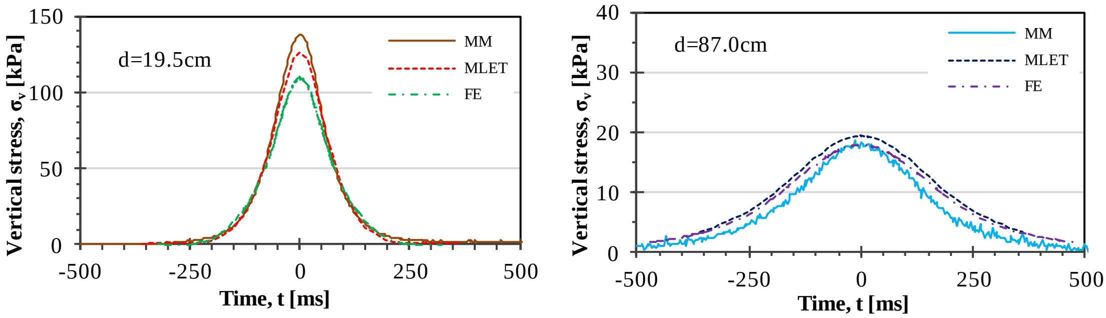


Figure 5 Induced vertical stress registration of SPC sensors and calculations, at depths of 19.5 cm ("moist") and 87 cm ("wet").

5 PERMANENT DEFORMATION PREDICTION MODELLING

Various models exist to evaluate permanent strain/deformation of unbound materials (Lekarp et al, 2000b; ARA, 2004; Korkiala-Tanttu, 2008). Here the accumulation of the vertical strain in the unbound pavement materials was modelled using two models: a procedure developed by Korkiala-Tanttu (KT) (2008, 2009) which is a stress based and by the Mechanistic-Empirical Pavement Design Guide (MEPDG) (ARA, 2004) a strain dependent model. The structure was kept at a constant temperature of 10°C resulting in insignificant permanent deformations within the asphalt layer and therefore not taken into account (Ahmed & Erlingsson, 2014; Hornych et al., 2013; Loulizi et al., 2006).

The KT model (Korkiala-Tanttu, 2008 & 2009) is a simple work hardening material model for unbound materials:

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

where $\hat{\varepsilon}_p$ is the accumulated permanent strain; C is a material parameter depending on compaction and saturation degree; N is the number of load repetitions; b is a shear ratio parameter depending on the material and stress state; R is the deviatoric stress ratio:

$$R = \frac{q}{q_f} = \frac{\sigma_1 - \sigma_3}{q_0 + Mp}$$

where

$$M = \frac{6 \cdot \sin\phi}{3 - \sin\phi} \qquad q_0 = \frac{c \cdot 6 \cdot \cos\phi}{3 - \sin\phi}$$

defined by the static Mohr Coulomb failure envelope and A is the maximum value of R, which theoretically is 1 but as R approaches 1 the expression *R/(A-R)* can increase to indefinite values; A has therefore been taken as 1.05.

The MEPDG model (ARA, 2004) is based on a best fit approach from laboratory testing, where a simple three parameter work hardening model has been used for unbound materials:

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

where ε_r is resilient strain imposed in a laboratory test to obtain ε_0, ρ and b (ρ, b and $\varepsilon_0/\varepsilon_r$ are obtained by using equations developed by ARA (2004)); 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.

The permanent deformation was gained by multiplying the permanent strain in the middle of each layer with its thickness. The total permanent deformation was gained by dividing each layer into sub-layers and sum up the deformation of the sublayers.

A "time-hardening" procedure was used when taking into account the effects of lateral wander and different water content, on the development of rutting (Lytton et al., 1993; Ahmed & Erlingsson, 2013; Rahman & Erlingsson, 2013).

Table 3 Material parameters of the unbound layers used to predict the permanent deformation with the KT model, calibrations factors are found with best fit approach.

Layer	State	*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 calibrations factors are found with best fit approach.

Layer	State	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

6 PERMANENT DEFORMATION

Listed in Tables 3 (KT model) and 4 (MEPDG model) are the parameters used in the permanent deformation predictions. The material parameters were mainly estimated from RLT tests, PL tests and FWD tests as well as compaction, saturation degree and stress state when appropriate (Wiman, 2010; Saevarsdottir & Erlingsson, 2013). In the KT model the cohesion, *c*, was reduced by 10% from "moist" to "wet" state in the crushed rock base and subbase layers and by 50% in the fine graded subgrade (Theyse, 2002; Matsushi & Matsukura, 2006). In the MEPDG model the gravimetric water content (W_c) was based on measurements and the calibration factor β_1 was adjusted to get a reasonable resemblance between the measured and calculated values.

When predicting the total deformation, the calibration factor β_1 for the subgrade had to be altered when using the MEPDG model (Table 4). It reduced with depth and the reduction was more in the "wet" state than in the "moist" state. Possible explanations are increased compaction or increased lateral pressure that increases the interlocking between the material particles with depth.

In Figure 6 the rutting profile is displayed after five different load repetitions; two in the "moist" state and three in the "wet" state. As before the calculations were performed by using the KT-model (left) and the MEPDG-model (right), the responses were gained by using both MLET and FE analyses.

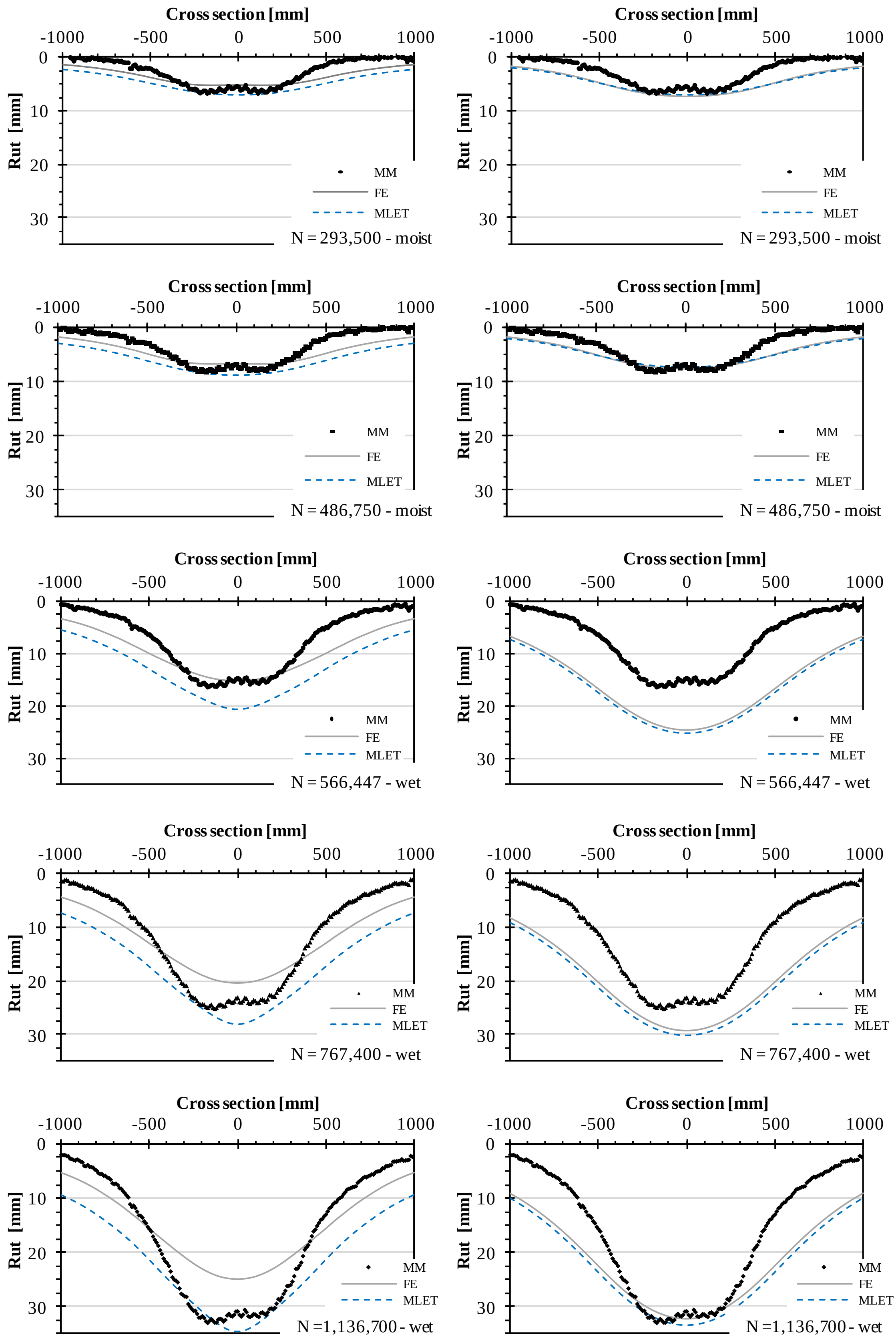


Figure 6 Cross section of the rutting profile after different numbers of load repetitions, two in the "moist" state and three in the "wet" state. The calculated values were obtained by using the KT model (left) and the MEPDG model (right). The response values were obtained from two different analyses, MLET and FE.

For both methods (KT and MEPDG) the rutting profile had reasonable correlation with the measurements in the “moist” state while more variations were observed in the “wet” state. The calculated rutting profile did not have the same shape as the measured one, but the centre values were similar.

The KT model shows a significant difference when using stress values from MLET and FE analyses despite MLET and FE giving similar response results (section 4). The permanent deformation was smaller when using FE stress response compared to the MLET one but the shape of the curves were similar. The change in permanent deformation were bigger than changes in R, q, and q_f indicating some sensitivity in the model.

When using the MEPDG model, similar results were gained when using the vertical strain from MLET and FE analyses as the strain is a linear factor in the model. The method does not resemble well the amount of rutting observed in the middle of the “wet” state but reaches similar values in the end.

7 CONCLUSIONS

In this work, the response and permanent deformation of a flexible pavement tested with an HVS machine was analysed. The structure was studied in “moist” and “wet” states before and after raising the groundwater table. The raised water level had a significant effect on the structural behaviour, increasing the water content in the unbound material layers causing a reduction in the resilient stiffness and increasing the permanent deformation.

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. Generally good agreements were found between the measured responses and calculated values using both FE and MLET with no significant difference between them.

The accumulation of permanent deformation of the unbound layers was modelled using two simple work hardening material models, one stress dependent, KT, and one strain dependent, MEPDG. The responses used in the permanent deformation prediction were gained from both MLET and FE analyses. When using the MEPDG model similar results were gained when using MLET and FE responses. Despite the MLET and FE analyses providing similar stress responses, the amount of permanent deformation from the KT model varied, indicating some sensitivity in the model.

The calculated rutting profile for KT and MEPDG did not reach the same shape as the measured one, but central values were similar. The MEPDG model does not resemble well the amount of rutting in the initial stages of the “wet” state whilst the KT model had a reasonably good correlation with the measurements after various numbers of load repetitions when using the MLET responses. Both these models are semi-empirical and more analysis and calibration is required to improve their accuracy for various load and environmental situations

8 ACKNOWLEDGEMENTS

The work described in this paper was sponsored by the Icelandic Road Administration (ICERA), The University of Iceland Research Fund, Ludvig Storr's Culture and Development Fund and the Icelandic Research Fund (grant no. 141210-051). Data were used from the Swedish National Road and Transport Research Institute (VTI), and the testing was performed in collaboration with The Swedish Transport Administration (TRV).

9 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.

Ahmed, A. W. & Erlingsson, S. (2014). Evaluation of permanent deformation model for asphalt concrete mixtures using wheel tracking and Heavy Vehicle Simulator tests. In *Compendium of Papers of the 93rd Annual Meeting of the Transportation Research Board*, Transportation Research Board of the National Academies, Washington, D.C., January, 2014.

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.

Helwany, S., Dyer, J. & Leidy, J. (1998). Finite-element analyses of flexible pavements. *Journal of Transportation Engineering*. 124, 491-499.

Hornych, P., Balay, J. M., Mauduit, C. & Bodin, D. (2013). Evaluation of the concept of equivalent temperature for pavement design. *Proceedings of the 9th International Conference on the Bearing Capacity of Roads, Railways and Airfields (BCRRA 2013), 25-27 June 2013*. Trondheim, Norway.

Huang, Y. H. (2004). *Pavement Analysis and Design*. 2nd ed. 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.

Korkiala-Tanttu, L. (2009) Verification of rutting calculation for unbound road materials. *Proc. of the Institution of Civil Engineers, Transport 162*. 162/TR2, 107 – 114.

Lekarp, F., Isacsson, U. & Dawson, A. (2000a). State of the art. I: Resilient response of unbound aggregates. *Journal of Transportation Engineering, ASCE*. 126/1, 66-75.

Lekarp, F., Isacsson, U. & Dawson, A. (2000b). State of the art. II: permanent strain response of unbound aggregates. *Journal of Transportation Engineering, ASCE*. 126/1, 76-83.

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.

Loulizi, A., Al-Qadi, I. L. & Elseifi, M. (2006). Difference between in situ flexible pavement measured and calculated stresses and strains. *Journal of Transportation Engineering*. 132, 574-579.

Lytton, R.L., Uzan J., Fernando, E. G., Roque, R., Hiltumen, D. & Stoffels, S. M. (1993). Development and validation of performance prediction models and specifications for asphalt binders and paving mixes. *The Strategic Highway Research Program Project Rep. No. SHRP-A-357*.

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.

May, R. W. & Witczak, M. W. (1981). Effective granular modulus to model pavement response. *Transportational Research Record 810*. Washington D.C.: National Research Council, 1-9.

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.

Rahman, S. & Erlingsson, S. (2013). Evaluation of permanent deformation characteristics of unbound granular materials from multi-stage repeated load triaxial test. In *Compendium of Papers of the 92nd Annual Meeting of the Transportation Research Board*, Transportation Research Board of the National Academies, Washington, D.C., January, 2014.

Saevarsdottir, Th. & Erlingsson, S. (2014). Modelling of responses and rutting profile of a flexible pavement structure in an HVS test. Road Materials and Pavement Design. doi: 10.1080/14680629.2014.939698

Saevarsdottir, Th., Erlingsson, S. & Carlsson, H. (2014). Instrumentation and performance modelling of Heavy Vehicle Simulator tests. International Journal of Pavement Engineering. doi: 10.1080/10298436.2014.972957

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

Schanz, T., Vermeer, P. A. & Bonnier, P. G. (1999). The hardening soil model: Formulation and verification. In Ronald B.J. Brinkgreve (ed.) *Beyond 2000 in Computational Geotechnics- 10 Years of PLAXIS*. A. A. Balkema, Rotterdam, The Netherlands.

Schwartz, C. W. (2002). Effect of stress-dependent base layer on the superposition of flexible pavement solutions. *The International Journal of Geomechanics*. 2/3, 331-352. doi: 10.1061/(ASCE)1532-3641(2002)2:3(331).

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 Center.

Uzan, J. (1985). Characterization of granular materials. *Transportational Research Record 1022, TRB*. Washington, D.C.: National Research Council, 52-59.

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