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Modelling of rutting development in pavement structures

Sigurdur Erlingsson*

VTI, The Swedish Road and Transport Research Institute, SE 581 95 Linköping, Sweden.
Faculty of Civil and Environmental Engineering, University of Iceland, IS 107 Reykjavik, Iceland.

Abstract

A mechanistic empirical (M-E) approach has been developed and thereafter used to calculate the rutting performance of an arterial road in Southern Sweden. The results were then compared with measurements from the Swedish long term pavement performance (LTPP) database. The arterial road had reached the critical 15 mm rut after 18 years in operation. The M-E approach used is a two-step procedure where the response of the structure is calculated mechanistically and thereafter the performance predicted empirically based on scaling of laboratory test results. Extensive laboratory testing was carried out on samples taken from the test road. Traffic counting and Bridge Weigh-in-Motion (BWIM) data were used to determine the amount of traffic loading, and data from weather stations were used to take into account the temperature dependency of the asphalt bound layers. The analysis shows that the rutting development can be simulated adequately although the calculations show slower rate than the measurements towards the end of the simulated period. The discrepancy in the rate of rutting between the measurements and the observations that was observed after about 9 years of operation might be due to the fact that no ageing or disintegration in material characterization was incorporated in the numerical analysis but surely observed in reality.

Keywords: Performance; rutting; mechanistic empirical approach; numerical analysis; permanent deformation

1. Introduction

Pavement systems are vital elements in the infrastructure network for all societies. But they also pose significant impacts related to high material consumption, energy inputs and capital investments. An elaborate and prudent asset management system is therefore needed to enhance the sustainability of transportation infrastructure systems. One of the critical elements in any pavement asset management system is linked to a realistic life cycle assessment modelling of the pavement structure. In the past

* Corresponding author.
E-mail address: sigurdur.erlingsson@vti.se

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pavement design has relied on empirical approach that has not been able to predict performance very accurately. New mechanistic-empirical (M-E) pavement design methods are therefore being developed in different countries with the main purpose of adequately predicting pavement performance as a function of time. The study presented here has therefore focused on investigating the applicability of such a method to predict or simulate the structural degradation of typical Nordic road structures as a function of time. For this purpose, an arterial road (Rv31 Nässjö) from the Swedish long term pavement performance (LTPP) road network has been modelled to see if it was possible to predict its degradation behaviour using an M-E approach. The pavement was opened for traffic in November 1988 and has been monitored since, giving the time history of the rutting.

2. Pavement behavior modeling

An M-E design method described here is carried out in two steps. First the response of the pavement structures due to traffic loading is evaluated. Thereafter a distress prediction is carried out to evaluate how performance changes with time (Ali and Tayabji, 1998; ARA, 2004; Erlingsson, 2010a). The two steps are briefly described hereafter.

2.1. Response modelling

The elastic response of the tyre pavement interaction has been estimated by a nonlinear multi-layer elastic theory (MLET) approach giving the stresses and strains at desired locations, see Fig. 1. This is a simple, fast and reliable method to calculate the response (Huang, 2004; Erlingsson, 2007; Khazanovich and Wang, 2007).

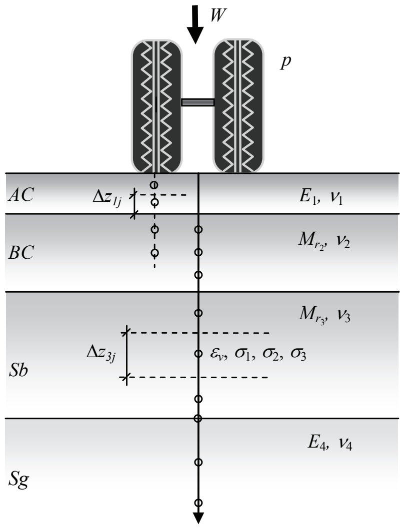


Fig. 1. The response of the pavement structure is evaluated at predefined locations.

In the response analysis the stiffness modulus stress dependency of the unbound aggregates has been assumed to follow the relationship (Yoder and Witczak, 1975; Uzan, 1985; ARA, 2004):

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

where the mean stress p is a defined as $p = (\sigma_1 + \sigma_2 + \sigma_3)/3$ with σ_1, σ_2 and σ_3 as principal stresses, k_1 and k_2 are experimentally determined material constants and p_a is a reference stress taken as 100 kPa.

2.2. Rutting modeling

The pavement performance modelling concentrates on the rutting development. In the approach each layer j is divided into i sub-layers and the rut manifested on the surface is calculated by accumulation of the permanent deformation $\hat{\delta}_p$ in each layer with depth according to:

$$\hat{\delta}_p(t) = \sum_{i=1}^{n}\sum_{j=1}^{m} \hat{\varepsilon}_{p_{ij}}(t) \cdot \Delta z_{ij} \tag{2}$$

where $\hat{\varepsilon}_{p_{ij}}$ and Δz_{ij} are the average accumulated plastic strain and thickness of the ith sub-layer of layer j, respectively, n is the total number of layers and m is the total number of sub-layers of each layer. The time t is then a function of the number of applied load repetitions. The bound layers, unbound aggregate layers (base course and subbase) and the subgrade were all modelled using different approaches.

For the asphalt concrete layers, a model dependent on temperature and number of load cycles is used where the accumulated strain is given as (ARA, 2004; Salama et al., 2007; Hu et al., 2011):

$$\frac{\hat{\varepsilon}_p(N)}{\Delta\varepsilon_r} = a_1 \cdot T^{a_2} \cdot N^{a_3} \tag{3}$$

where N is here the accumulated number of standard axles, $\Delta\varepsilon_r$ is resilient strain induced during the load cycle, T is the temperature in degrees centigrade, and a_1, a_2 and a_3 are material parameters determined in the laboratory. For the unbound aggregate layers (base course and subbase) the model used is given as (Korkiala-Tanttu, 2005):

$$\hat{\varepsilon}_p(N) = C \cdot N^b \cdot \frac{R}{A - R} \tag{4}$$

where C and b are material parameters, A is a parameter independent of the material (A = 1.05) and R is the deviatoric stress ratio defined as:

$$R = \frac{q_{\max}}{q_f} = \frac{q_{\max}}{s + mp_{\max}} \tag{5}$$

where the deviator stress q is defined as $q = (\sigma_1 - \sigma_3)$ and $p_{\max}$ and $q_{\max}$ represents the peak values of the mean normal and deviatoric stress during the load cycle, respectively. Further are m and s defined from the static Mohr Coulomb failure envelope of the material.

The development of the permanent deformation in the subgrade is assumed to follow the Tseng and Lytton (1989) model given as:

$$\frac{\hat{\varepsilon}_p(N)}{\Delta\varepsilon_r} = \varepsilon_0 \cdot e^{-\left(\frac{\rho}{N}\right)^{\beta}} \tag{6}$$

where ε_0, ρ and β are material dependent regression parameters.

3. The pavement structure

Fig. 2 shows the pavement structure from the Swedish long term pavement performance (LTPP) database used to test the procedure. The road was opened for traffic in November 1988 and during the first eight months the asphalt concrete (AC) consisted of a normal 50 mm thick road base mix with largest aggregate size 22 mm. In July 1989 an additional 35 mm of wearing course with largest aggregate size 16 mm was placed on top of it. The base course consisted of crushed natural gravel was 115 mm in thickness and the subbase was 0/100 mm natural material, 500 mm in thickness. The subgrade consisted of organic glacial till.

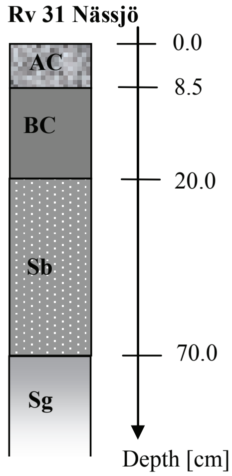
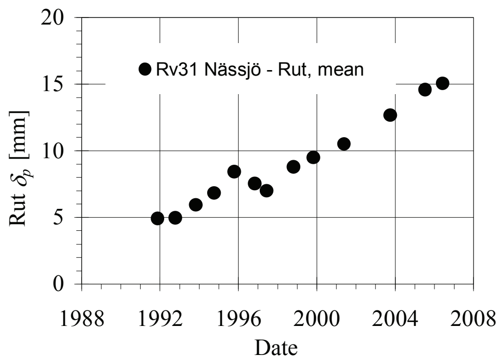


Fig. 2. Cross section of the pavement structure at Rv31 Nässjö. AC = asphalt concrete, BC = Base course, Sb = subbase and Sg = subgrade

Information about the rut depth δ_p, and visual cracking of the surface expressed as a crack index I_{cr} has been collected regularly as a part of the Swedish LTPP programme and stored in a database, see Fig. 3.



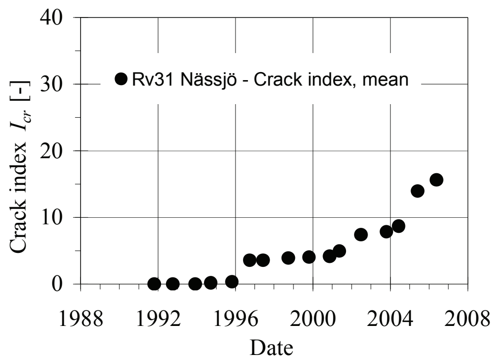


Fig. 3. Cross section of the pavement structure at Rv31 Nässjö

As can been seen after 18 years of operation has the average maximum rut δ_p reached 15 mm and the crack index I_{cr} was 16. The Swedish road administration reference limit values for δ_p and I_{cr} are 15 mm and 190 respectively. Therefore it is clear here that the critical distress mode is the rutting criteria; thus the analysis here concentrated on simulating the rutting development of the pavement structure.

4. Traffic loading and climate data

The traffic volume during the operation time has been estimated based on traffic counting as well as from Bridge Weigh-In-Motion (BWIM) measurements, respectively. During a traffic counting measurement, made in 2002, the average annual daily traffic (AADT) was 3044 where heavy vehicles consisted of 14.6% ($AADT_{hv}$ = 444). Data from the Swedish Bridge Weigh-In-Motion (BWIM) system at the nearby station Forserum has further been analysed. Data was collected during a one week period in 2005 and again in 2009. Based on this it was concluded that the equal single axle loads (ESALs) were N_{100} = 220 per day and lane at the time the road was opened for traffic with a yearly increase of 1.2% where N_{100} represents the 100 kN dual wheel standard axle with 800 kPa tyre pressure.

Hourly temperature data for the eight-year period between 2001 and 2008 has been analysed from measurements at the LTPP site, including both the air temperature as well as measurements from a sensor in the pavement close to the surface (Fig. 4). The temperature data from the sensor located in the pavement has further been used to divide the year into time periods where the asphalt concrete stiffness was considered constant.

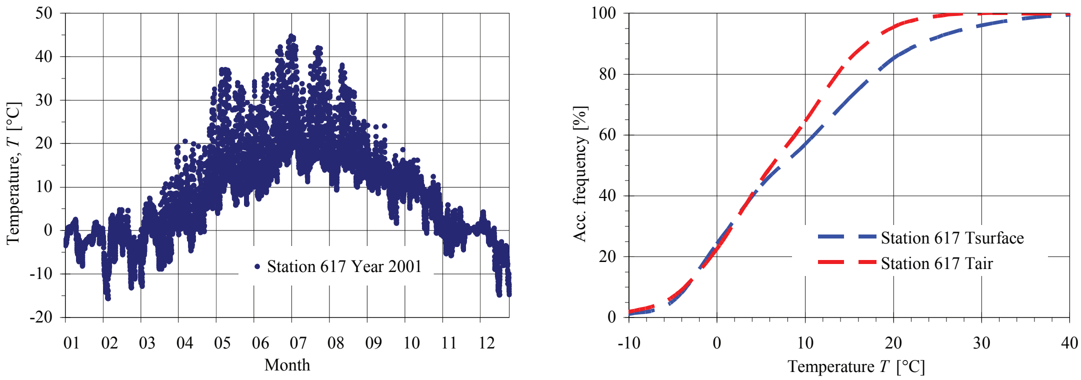


Fig. 4. Temperature data from the LTPP arterial road section Rv31 Nässjö (station 617). a) hourly temperature measurements at the pavement surface for 2001. b) normalized accumulated temperature for both the air and the surface of the pavement structure for 2001 – 2008

Based on Figure 4b) the frequency distribution of the hourly temperatures in the pavement layer for the eight year time period was estimated. The results are given in Table 1.

Table 1. Average annual frequency distribution of hourly temperatures for the sensor located within the pavement structure at the LTPP road Rv 31 Nässjö

Interval [°C]	$T \leq 4.9$	$5.0 \leq T \leq 14.9$	$15.0 \leq T \leq 24.9$	$25.0 \leq T \leq 34.9$	$35.0 \leq T$
Temperature [°C]	0	10	20	30	40
Frequency [%]	43.5	28.5	20.1	6.2	1.7

In the analysis the time step was set to one year and the year was thereafter divided into temperature periods according to Table 1 where the asphalt concrete had a constant stiffness within each period and the frequency was used to estimate the yearly amount of traffic that was applied during each period. The response was then calculated for each temperature and the performance thereafter accumulated, revealing the performance history.

5. Material properties

Samples were taken from the LTPP test road structure and tested in the laboratory. Field testing has further been carried out. The relevant test results are briefly described here.

5.1. Asphalt concrete

The AC layer consisted of a 35 mm wearing course over a 50 mm road base mix. The binder penetration of both layers was pen 160/220. Based on IDT (indirect tensile) tests from cores, 150 mm in diameter, the master curves from the cores were estimated. The two AC layers revealed very similar properties and were therefore treated for simplicity as one single layer with identical material properties.

Based on the master curves the resilient modulus of the AC layer has been estimated to be close to 6,500 MPa at 10°C. The Poisson's ratio has further been estimated to be close to 0.35. The stiffness of the asphalt concrete E at a specific temperature T was corrected according to the simple relationship:

$$E_T = E_{T_{ref}} \cdot e^{-b\cdot\left(T-T_{ref}\right)} \tag{7}$$

where T_{ref} is the reference temperature in °C, here taken as 10°C and b is a material constant. The material parameters for the performance modelling of the permanent deformation according to equation (3) have been estimated from a dynamic triaxial testing carried out at three different temperatures (Nilsson and Huvstig, 2009).

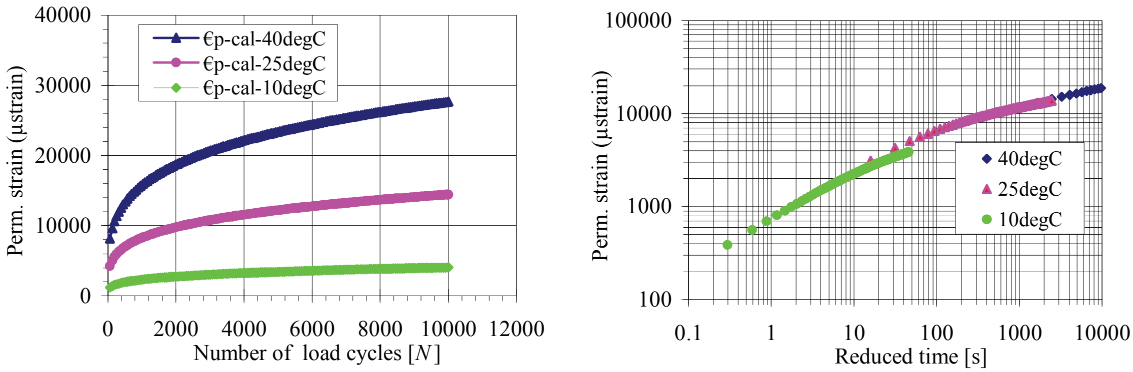


Fig. 5. Accumulated vertical permanent strain in an asphalt concrete sample (AG22) in a triaxial test carried out at three different temperatures, a) as a function of number of load cycles and b) as a function of reduced time

The material parameters for the AC layer used in the analysis are given in Table 2 where ν is the Poisson's ratio.

Table 2. Material parameters for the asphalt concrete

$E_{T_{ref}}$	ν	b	$\frac{a_1}{\Delta\varepsilon_r^{lab}}$	a_2	a_3
[MPa]	[-]	[-]	[-]	[-]	[-]

6,500	0.35	0.065	0.17	1.85	0.27

5.2. Base course and subbase material

The base course (BC) material consisted of crushed natural 0/32 mm aggregates. It has been analysed in a repeated load triaxial (RLT) test, 150 mm in diameter and 300 mm in height. Both the resilient properties as well as permanent deformation properties have been evaluated. Fig. 6 shows the results of the stiffness testing (Hoff, 2010). The subbase (Sb) is a 0/100 mm natural material. It has been tested in the same way as the base course. The material properties for both the BC and Sb layers used in the analysis are given in Table 3.

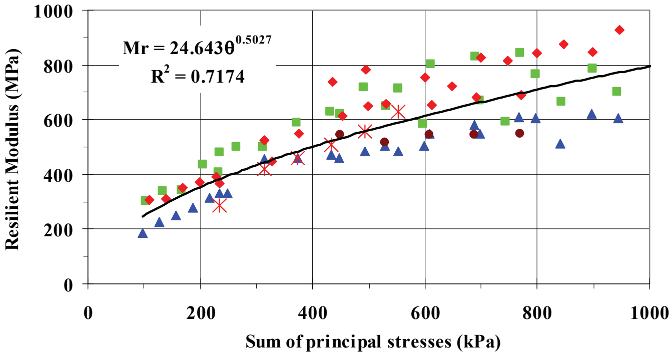


Fig. 6. Resilient stiffness of the base course material plotted as a function of the sum of the principal stresses.

Table 3. Material parameters for the base course and the subbase layer

Layer	k_1 [-]	k_2 [-]	ν [-]	s [kPa]	ϕ [°]	C [-]	b [-]	A [-]
BC	2460	0.5	0.35	138.1	2.06	$1.5\cdot10^{-5}$	0.3	1.05
Sb	946.4	0.4	0.35	166.3	1.20	$1.0\cdot10^{-4}$	0.3	1.05

5.3. Subgrade

The stiffness characteristic of the subgrade (Sg) has been evaluated based on falling weight deflectometer (FWD) measurements carried out regularly during the operation time of the LTPP section. The permanent deformation characteristics of the subgrade have been estimated from previous accelerated pavement testing (APT) using a Heavy Vehicle Simulator (HVS) where similar subgrade material was used. For further details see Erlingsson (2007) and (2010b) and Wiman and Erlingsson (2008). The material parameters used here for the subgrade are given in Table 4.

Table 4. Material parameters for the subgrade layer

M_r [MPa]	ν [-]	$\frac{\varepsilon_0}{\Delta\varepsilon_r^{lab}}$ [-]	ρ [-]	β [-]
100	0.35	20	10,000	0.2

6. Permanent deformation prediction

Using the above described procedure and the material parameters in Tables 2 - 4 the rutting has been calculated for the LTPP test road. The contribution of each layer is given in Figure 7.

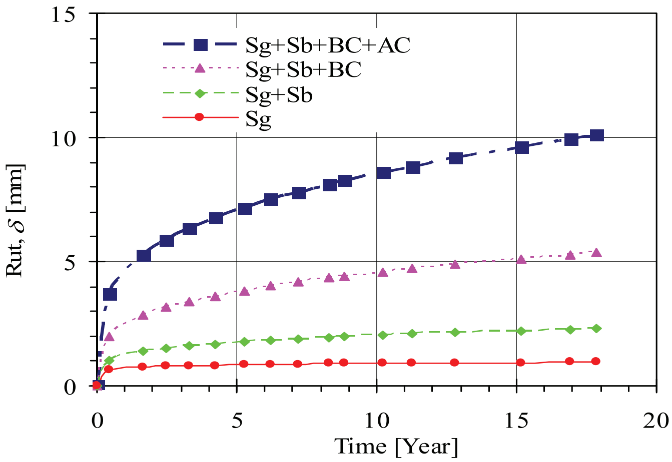


Fig. 7. Accumulation of permanent deformation of each layer for the LTPP road as a function of time

As stated earlier, the pavement structure was built in two steps, first with a 50 mm road base mix and then, after it had been open for traffic for eight months, a final 35 mm thick wearing course was added. All the virgin rutting during this initial eight month period, where the rate of accumulation of the permanent deformation is fast, was therefore adjusted to zero at the time the wearing course was placed. The development of plastic deformation was much milder thereafter as most of the post compaction in the pavement structure had already taken place, although a new 35 mm wearing course is virginally deformed. Taking this into account the measured and calculated ruts are shown in Fig. 8.

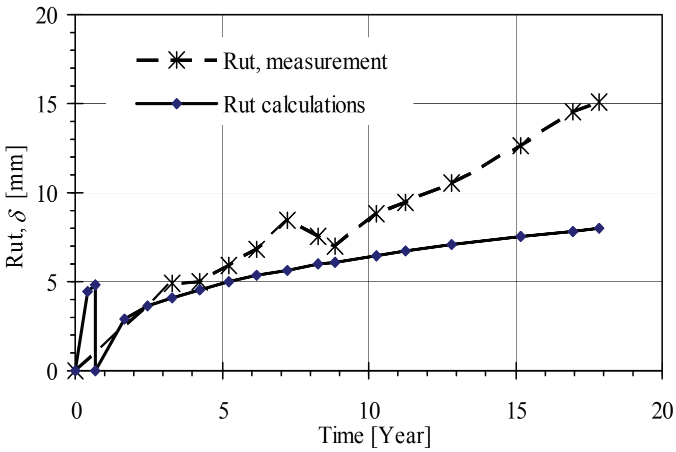


Fig. 8. Comparison of calculated with measured development of rutting as a function of time for the LTPP road Rv 31 Nässjö. The calculated values have been adjusted to represent the procedure of placing the wearing course eight months after the road base mix was placed

The predicted rutting seems to have followed the observation quite well during the first 8 – 9 years but thereafter the model indicates a milder rate of rutting development compared to the measurements. Wear due to studded tyres were not modelled here although they are frequently used in Sweden during the winter months. This might explain at least partly the fact that the modelled rut lies below the measured rut in Fig. 8.

7. Conclusion

Performance prediction was carried out for one LTPP test road section in southern Sweden. The procedure used was based on a two-step M-E approach, that is the response of the pavement structure was calculated mechanistically and thereafter the performance was predicted based on an empirical evaluation of laboratory tests that were scaled to present the actual field conditions. The major findings from this study were:

- The M-E approach can be used to predict the rutting development of typical Swedish roads.
- The simple performance models used here seems to predict adequately the expected permanent deformation of each layer at least for an 8 - 9 years time period.
- Performance calculations for longer periods need to be adjusted to include changes in material properties due to ageing of material, seasonal variation as well as changes in the structural integrity (cracking) of the structure.
- As studded tyres are frequently used in the Nordic countries wear due to studs should be added to the total rut to improve the prediction of the total rut.

8. Acknowledgements

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