Calculating Rutting of Some Thin Flexible Pavements from Repeated Load Triaxial Test Data

Abstract

This paper describes parts of a Nordic pavement performance prediction model study (at the project level of the NordFoU project) where a material performance model, developed at VTT research centre in Finland, has been selected as a mean of calculating the permanently accumulated (plastic) deformation (i.e. rutting) of unbound granular materials (UGMs) in flexible pavements subjected to trafficking. The paper aims to assess the suitability of this VTT model application to Swedish roads comprising thin asphalt layers over a thick UGM base. To achieve this, the VTT model has been used to calculate the deformations of two tested road sections in Sweden. These calculations have been compared with another permanent deformation model for UGM (the Gidel model) and with rutting measurements from trafficked pavements. It is shown from this study that the applied rutting prediction method with VTT model is capable of predicting the development of rutting depth despite some overestimations.

Introduction

Rutting, commonly seen as a groove or depression on the surface of flexible pavements, is the consequence of the accumulation of permanent (plastic) deformation of a pavement structure caused by repeated traffic loading. In this paper the term permanent deformation is used to describe the plastic strain summed over a particular distance, whereas the term rutting is used to describe the overall phenomenon as seen at the pavement's surface as a depression. Undeniably, rutting can also occur due to wear of the surface (a factor that is most noticeable in countries allowing the use of studded tyres in winter). This factor is not included in any of the computations in this paper, so there may be some under-estimation of total rutting by the method employed.

Rutting is undesirable for many reasons. It:

- generates steering difficulties for drivers,
- provides a potential means of water intrusion through asphalt layers into unbound pavement layers, or where rutting has also deformed the subgrade surface, it will act to keep water in the pavement at the pavement-subgrade interface, thereby leading to rapid pavement deterioration,
- provides a place where water or ice may accumulate leading to reduced skid resistance, and
- indicates that there has been shear inside the pavement structure, which will have loosened the unbound materials and, thereby, weakened the pavement.

Rutting is the most common reason for rehabilitation and maintenance measures on the road network in Scandinavian countries and the cost associated with rutting are considerable. A good permanent deformation prediction model could thus provide a better basis of maintenance investment, life cycle cost calculation and estimation of residual values for administrations (Huvstig, 2010), although rutting due to studded tyre wear would need to be incorporated as a major element on more heavily trafficked roads in countries where such tyre use in winter is significant.

The generation of rutting in unbound granular materials (UGMs) is complex. Dawson and Kolisoja (2006) concluded that there are four mechanisms of rutting which should be taken into consideration: compaction, shear (perhaps caused by direct tire-pavement interaction), weak subgrade and particle damage, depending on the road type and traffic condition. Surface rutting is one of the critical distress mode observed in flexible pavements that are largely formed of UGM. Researchers almost invariably compute rutting by summing the vertical plastic strain in all unit length of pavement structure (e.g. Allou et al., 2010; Werkmeister et al., 2001; Alabaster et al., 2002; Theyse, 2007). This means that rut depth can be predicted by summing the permanent deformation accumulated in all layers of the pavement structure including the asphalt and unbound granular layers (UGLs).

Furthermore, it has been demonstrated by accelerated pavement tests on unsealed and thinly sealed road structures (i.e. those with a double chip seal of asphalt or less) that up to 30% to 70% of the surface rutting is generated in the unbound granular layers (Arnold, 2004; Korkiala-Tanttu et al., 2003; Little, 1993; Dawson et al., 2004; Forest Enterprise, 2003). Hence the permanent deformations from UGLs could comprise the majority of the rut depth for thinly-paved or un-paved roads. Therefore, a UGM performance model will have a key role to play in rutting prediction.

The permanent deformation of UGMs is influenced by various material and structural factors (Lekarp et al., 2000). The material factors include the properties of the aggregates such as grain shape, surface roughness, maximum grain size, content of fines, grain size distribution, and degree of compaction. Structural factors include, for example, the number of load repetitions, temperature, moisture condition, geometry of the structure and stress history (Korkiala-Tanttu, 2009).

Commonly, the number of loading repetitions is one of the most important factors (Lekarp et al., 2000) with most existing rut-prediction models suggesting that the relationship of permanent deformation and number of loading repetition should follow an exponential form such as in equation (1) (Sweere, 1990):

$$\varepsilon_p = a \times N^b \tag{1}$$

where,

ε_p = permanent deformation

a, b = regression factors

N = number of load repetitions

Loading factors associated with the number of loading repetitions are also of crucial importance in modelling the traffic loading. These factors include, for example, maximum axle load, tyre pressure, rotation of principal axis, loading rate, loading history and lateral wander. The rotation of principal axis is also a very important factor because it is the main factor that differs between the real field conditions and the triaxial test conditions, and cannot be employed in the laboratory, except through highly specialized hollow cylinder or cycle shear cells which ware practically unavailable for UMGs (Korkiala-Tanttu, 2009).

Another very important factor, moisture, can alter the resistance of permanent deformation in UGMs dramatically (Wiman, 2001; Korkiala-Tanttu, 2007). The repeated traffic load may cause positive pore water pressure to increase in UGLs, which will consequently reduce the effective stress in UGLs which in turn decreases the stiffness and resistance to permanent deformation.

The degree of compaction (affecting the grain packing of UGMs) also greatly affects the permanent deformation. Resistance to permanent deformation can be much improved by increased density, especially for those UGMs that are comprised of crushed grains (Werkmeister, 2003). In addition to good compaction, good drainage is the best and most cost effective method for improving the resistance of UGMs to permanent deformations.

Shakedown theory

Shakedown theory was first developed to describe the behaviour of metal surfaces under repeated loading (Johnson, 1986) and was then adopted to depict permanent deformation behaviour of UGMs under different stress levels (Dawson and Wellner, 1999; Werkmeister et al., 2001). Werkmeister et al. (2001) have studied the relation between accumulated vertical strain and number of loading repetitions and discovered three categories of plastic deformation under different stress states. Dawson and Wellner (1999) studied the relation between vertical permanent strain rate and accumulated vertical strain and arrived at similar conclusions. These studies suggest three plastic deformation ranges (known as Ranges A, B and C – see Figure 1), dependant on the material properties of the UGM and the stress level imposed on it.

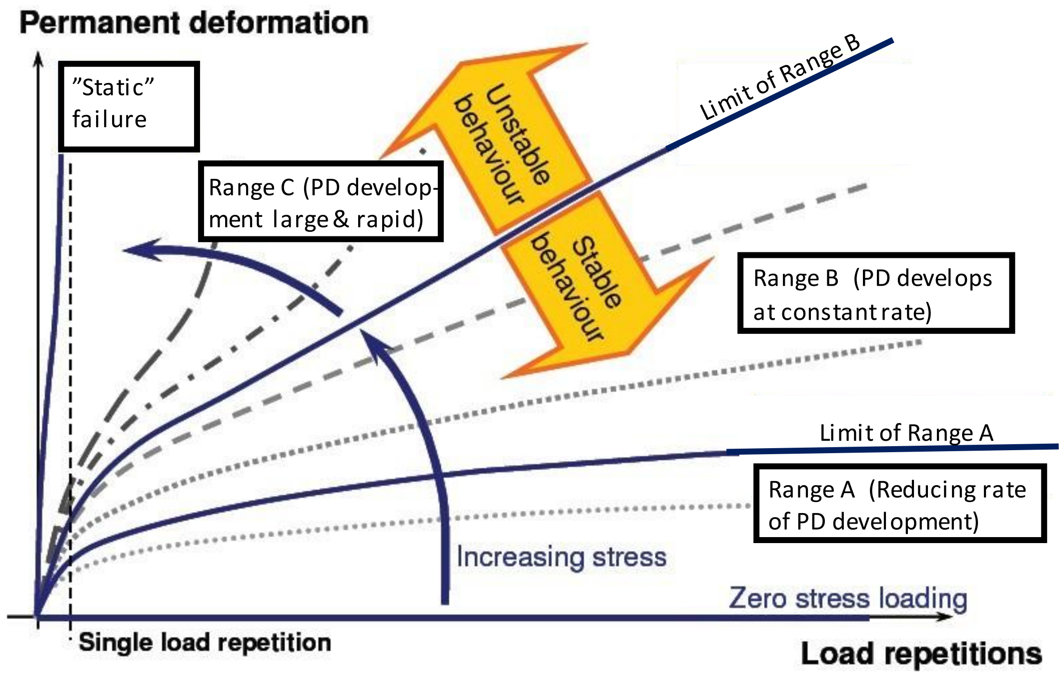

Figure 1. Shakedown behaviour by Range (adapted from Theyse (2007); PD = Permanent deformation).

Range A (Plastic Shakedown)

The UGM experiences a low level of stress which will lead to a phase of initial deformation due to a combination of imposed compaction and residual compaction by trafficking. The disordered granular material will be re-oriented and some will break due to the loose structure. In the phase of initial post-compaction, the accumulation of permanent strain is faster compared with the following phase which is characterized by an asymptotic log (ε_{1p}) versus log (N) plot. On a plot of number of loading cycles versus plastic strain diagram, plastic strain of UGMs in Range A will almost cease after a number of loading repetitions. The deformation is plastic during the first few loading cycles but subsequent deformation is resilient.

Range B (Plastic Creep)

When the imposed stress is higher, non-recoverable particle rotations and additional non-recoverable slip between particles leads to a faster accumulation of strain in Range B, compared to Range A. After an initial rapid strain accumulation, the strain does not cease but continues to accumulate at a constant rate (Werkmeister, 2003).

Range C (Incremental collapse)

When the imposed stress level is high enough, the UGM will collapse. The permanent deformation from Range C commences with a primary creep phase, during a post-compaction period, which is similar to Range A and B. However the strain accumulates much faster. Afterwards the secondary creep phase will take place, followed by non-stable tertiary creep. Range A, B and C behaviour is described in slightly different terms by Huvstig (2012). Both grain abrasion and particle crushing may occur in Range C. Werkmeister (2003) believes that grain attrition is the main reason for the collapse of UGMs, although others believe that a major reason for collapse is the volume increase caused by dilation of the material (Hoff, 1999).

In flexible pavement design and practice, Range C behaviour in UGMs should be avoided and Range A preferred. Nevertheless, in most real unbound granular layers in unsealed or thinly sealed pavements, Range A is exceeded, i.e. Range B occurs. Range B is also a practical condition for a pavement if the rate of strain accumulation and/or the traffic volumes are low. In these conditions, permanent deformation develops in an orderly manner such that many empirical-mechanistic UGM models are usable for both Range A and Range B.

Permanent deformation model

For the materials used in the current study, the permanent deformation was calculated from the output of the pavement management program VägFEM (Huvstig, 2010), described later. This comprises a finite element routine to compute stress state that models pavement response by means of a linear/nonlinear model for UGMs and a linear elastic model for asphalt concrete. In a coupled performance model, the asphalt material model is applied to all asphalt layers and the UGM model predicts permanent deformation in the granular base layer, the subbase layer and the subgrade.

NCHRP asphalt permanent deformation model

As VägFEM does not compute plastic asphalt deformations from the finite element stress analysis, the NCHRP permanent deformation model (Hugo and Epps-Martin, 2004) was used to compute the permanent deformation in the asphalt layers by correlation to the resilient behaviour as follows:

$$\varepsilon_p = \varepsilon_r \times a_1 \times N^{a_2} \times T^{a_3} \quad (2)$$

where,

ε_p = permanent strain

ε_r = resilient strain

N = number of load repetitions

T = temperature

$a_1,\ a_2,\ a_3$ = regression coefficients (1.69, 1.85, 0.275 respectively in this study, according to previous study (Huvstig, 2010))

This approach for calculating plastic asphaltic deformations takes no account of speed of traffic loading and the relationship assumed between elastic and plastic behaviour is, essentially, empirical with little fundamental basis. However, for the pavements being considered in this paper with thin asphaltic surfacing, the proportion of permanent deformation occurring in the asphalt is necessarily small. Thus, the errors introduced by virtue of these limitations are thought to be acceptably small. Seasonal variation in pavement temperature was considered by temperature distribution. The rate of permanent deformation from asphalt layers was calculated (by Eq. 2) under each temperature range and is used to derive permanent deformation throughout a year.

VTT model of UGM permanent deformation

The Finnish VTT model as developed by Korkiala-Tanttu at the VTT research centre of Finland was used for predicting permanent deformation for UGMs. The VTT model takes into account the number of loading cycles, the stress state in the UGM and the material strength. The model may be expressed by the following equation (Korkiala-Tanttu, 2005):

$$\varepsilon_p = C \times (N)^b \times \frac{R}{1-R} \quad (3)$$

where,

ε_p = permanent axial strain (‰)

N = number of loading cycles

R = failure ratio

C = material parameter

b = stress state parameter

The failure ratio, R, is introduced in the model to allow for the importance of stress level relative to the shear strength of the UGMs in the development of permanent deformation. The failure ratio is calculated as:

$$R = \frac{q}{q_f} \tag{4}$$

where,

q = deviatoric stress, $q = \sigma_1 - \sigma_3$ (in triaxial test, σ_1 and σ_3 are the major and minor principal stresses)

q_f = deviatoric stress at failure (see equation (5))

Failure deviatoric stress q_f is correlated to the average principal stress by the linear equation:

$$q_f = q_0 + M \times p \tag{5}$$

where,

p = average principal stress (in triaxial test, $p = (\sigma_1 + 2\,\sigma_3)\,/\,3$)

M, q_0 = constants of the equation (the same parameters of the failure line of the material as in equation (7))

The failure deviatoric stress is obtained by a monotonically increasing static test. Tests based on different samples from the same unbound granular layer can reveal the failure envelope of the material (see Figure 2). In Figure 2, the static failure, shown by the dashed line, is obtained by regression analysis from three samples from the base layer in Trädet, which have been tested by the monotonically increasing static test. M and q_0 (in equation (5)) are the slope and intercept of the failure line.

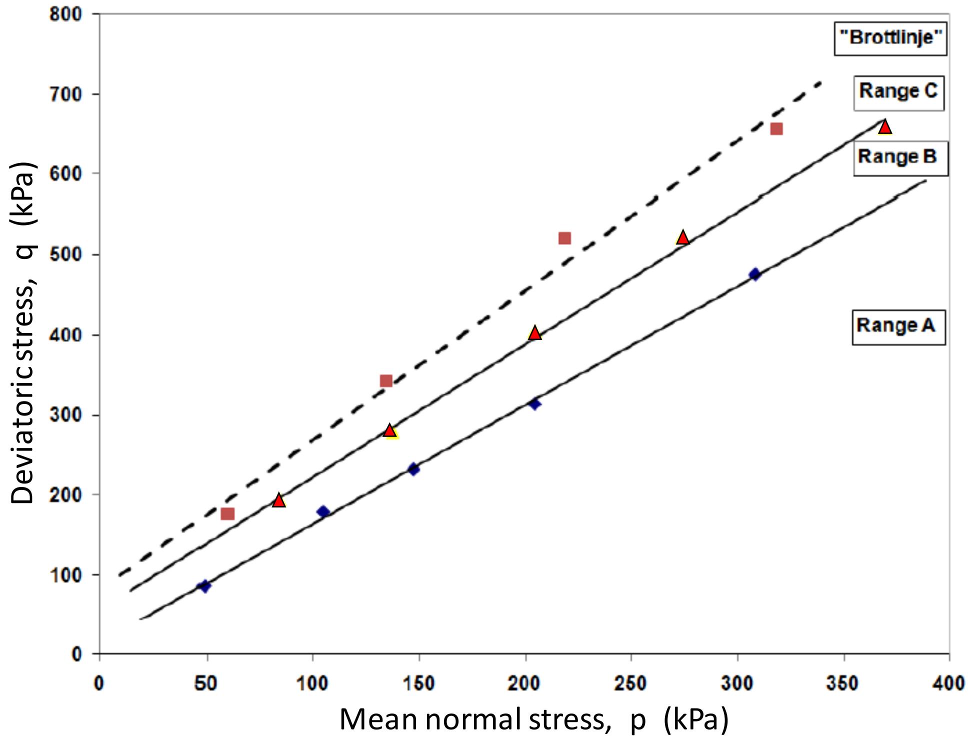

Figure 2. Failure line of base material, Trädet.

By combining equations (3), (4) and (5), the VTT model can be written as:

$$\varepsilon_p = C \times (N)^b \times \frac{1}{\frac{(M \times p + q_0)}{q} - 1} \tag{6}$$

Gidel model of UGM permanent deformation

The Gidel model (Huvstig, 2010) is another model for permanent deformation modelling of UGMs. The Gidel model is formulated as follows:

$$\varepsilon_{1p}(N) = \varepsilon_1^{p0} \times (1 - N^{-B}) \times (\frac{L_{max}}{p_a})^n \times \frac{1}{M + \frac{q_0}{p_{max}} - \frac{q_{max}}{p_{max}}} \tag{7}$$

where,

ε_{1p} = permanent axial strain

q_{max}, p_{max} = maximum values of the mean principal stress, p, and deviatoric stress, q (kPa)

p_a = reference pressure (100 kPa)

ε_1^{p0}, B, n = model constants (regression coefficient at validation stage)

L_{max} = $\sqrt{{p_{max}}^2 + {q_{max}}^2}$

Even though developed for repeated load triaxial tests, the Gidel model has been validated and integrated with the VägFEM program for a more general implementation. It is a validated tool with good rutting prediction in Sweden (Huvstig, 2010) thus it is used in this study as a base to which the VTT model can be compared.

VägFEM

VägFEM is a finite element programme developed in Sweden, which is used for calculation of permanent deformation in this study. VägFEM is a 3D programme, which is able to consider the real geometry of a road. By defining the geometry of a road, the load specification, and material properties, the stress and strain state in pavement structures can be analysed with consideration of self-weight of the pavement. Pavement responses (stresses and strains) are computed for the different layers. To achieve this, a linear elastic model can be applied for bituminous bound layer(s) and a linear elastic or nonlinear elastic model can be chosen for analysis of unbound layers. Rutting is calculated by summation of permanent deformation in all layers, with consideration of temperature and moisture variation.

Elastic moduli were calculated from the testing of samples abstracted from pavements, allowing for temperature and vehicle speed. These were used in the VägFEM program so as to calculate the elastic strain in the road structure for different temperatures at different seasons. The traffic had also been measured for the same periods and this data was also used in VägFEM.

Laboratory tests and calibration of VTT model

Samples were taken from the pavements' unbound layers in two Swedish roads. The chosen roads are Rv 31 in Nässjö and Rv 46 in Trädet. The structure of the roads is similar: a wearing course over two asphalt layers and unbound layers including base, subbase and subgrade. Table 1 (a & b) provides the sections from the two roads:

Table 1a. Layer information of Rv 31, Nässjö (Huvstig, 2010).

Layer	Thickness mm	Material	Year
Wearing course	24	Hot remixing plus 60ABS16 (new wearing course)	2007-08-10
Second asphalt layer	35	80MABT16 (wearing course)	1989-07-01
First asphalt layer	50	110AG (Bituminous bound layer)	1988-11-01
Base	115	Gravel base material: A:BYA 6: 03	1988
Subbase	500	Gravel of class A:BYA 6:03	1987-88
Subgrade		Silty moraine, class 6 (frost class 3)	

Table 1b. Layer information of Rv 46, Trädet (Huvstig, 2010).

Layer	Thickness mm	Material	Year
Wearing course	0	Y1B16 (surface treatment)	1988
Second asphalt layer	12	Maju 30 MABT12 (Adjustment layer)	1987
First asphalt layer	70	165AG (Bituminous bound base)	1986
Base	125	Gravel base material: BYA 6:06	1986
Sub base	410	Gravel of class A: BYA 6:03	1986
Subgrade		Friction mateiral, class 1 (Frost class 1)	

Note: code refers to BYA (1984). For instance, the subbase materials from the two roads belong to material group A, which consists primarily of gravel, sandy gravel and sandy moraine. For material group A, specific sieve requirement need to be fulfilled (see BYA, 1984).

The road Rv 31 has been open to traffic since 1988. Although the traffic between 1988 and 1989 contributed to the deformation of the road, as a second asphalt layer was added in 1989, the previous rutting was erased. Thus rutting is considered to start from 1989. The analysis ends before 2004 when the last rutting measurement was recorded. The analysis period for Rv 46 Trädet was chosen in the same manner. Rutting was measured by laser profiling. The rutting measured by the laser equipment is usually smaller than measurement using a straight-edge, which represents a more realistic value.

According to the Swedish LTPP (Long Term Pavement Performance) database, the AADT was 314 standard axles (10 tonne) per day for road Rv 31, measured in 1996. The annual traffic growth rate is 1.2% as compound growth. The AADT for Rv 46 Trädet was 193 standard axles per day, measured in 1998 and the annual traffic growth rate is 1.3%. Figure 3 presents the traffic condition (in cumulative ESALs) for both roads.

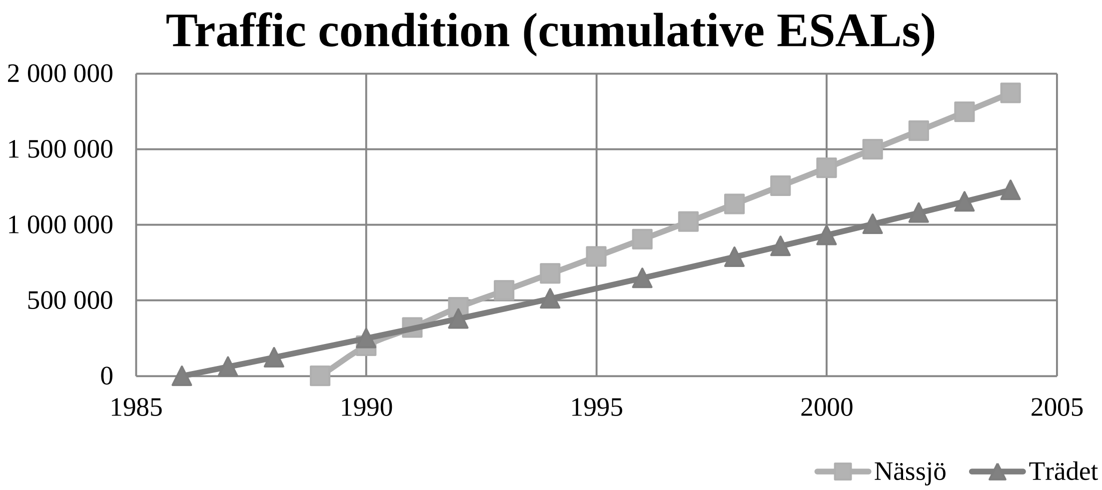

Figure 3. Traffic condition for the two roads.

Samples were taken from many layers of the two roads for laboratory tests. In Nässjö, samples were taken from all the granular layers, including the unbound base, subbase and subgrade. In Trädet, the subbase material is assumed to be the same as in Nässjö in the same layers because material of the same class was used. The conditions of samples including density and moisture are listed in the table below:

Table 2. Conditions of some samples.

Road section	Layer	Dry density ($g/\,cm^3$)	Moisture (%)
Rv 31 Nässjö	Unbound base	2.25	1.85
	Subbase	2.25	4.00
	Subgrade	2.02	8.00
Rv 46 Trädet	Unbound base	2.42	1.80

	Subgrade	1.90	7.00

Triaxial tests were performed on the specimens for both the unbound base and subgrade soils (which are made from all samples according to the European Standard (CEN, 2004)) to study the permanent deformation behaviour and static failure properties so that the material models could be validated/calibrated. Two types of triaxial tests are included in the study: repeated load triaxial (RLT) test and static failure test.

The RLT test is performed to study both the resilient and permanent deformation behaviour of UGMs and subgrade soils by imposing a large number of loading repetitions on a prepared cylindrical specimen according to the European Standard (CEN, 2004). The loadings are imposed in the form of a confining pressure (σ_3) and a deviatoric pressure (q), using a constant confining pressure method (high pressure level) as in Table 3. The frequency of the loading pulse was maintained between 0.2 Hz and 10 Hz. In the RLT test, deformation of the specimen is measured by transducers and recorded so that the results can be used for shakedown range analysis and validation/calibration of the permanent deformation models.

Table 3. Stress level of RLT test (CEN, 2004).

Sequence 1			Sequence 2			Sequence 3			Sequence 4			Sequence 5		
Confining stress, σ_3 kPa	Deviator stress, σ_d kPa		Confining stress, σ_3 kPa	Deviator stress, σ_d kPa		Confining stress, σ_3 kPa	Deviator stress, σ_d kPa		Confining stress, σ_3 kPa	Deviator stress, σ_d kPa		Confining stress, σ_3 kPa	Deviator stress, σ_d kPa	
constant	min	max	constant	min	max	constant	min	max	constant	min	max	constant	min	max
20	0	50	45	0	100	70	0	120	100	0	200	150	0	200
20	0	80	45	0	180	70	0	240	100	0	300	150	0	300
20	0	110	45	0	240	70	0	320	100	0	400	150	0	400
20	0	140	45	0	300	70	0	400	100	0	500	150	0	500
20	0	170	45	0	360	70	0	480	100	0	600	150	0	600
20	0	200	45	0	420	70	0	560						

Note: the above table uses the CEN nomenclature for deviatoric stress, σ_d whereas the symbol q is used elsewhere in this paper.

Shakedown range analysis is based on the permanent deformation behaviour of each specimen under each level of deviator stress in each sequence, which generally corresponds to approximately 10,000 load repetitions. Werkmeister (2003) suggested a method to determine the shakedown range of UGMs using the RLT test result which has been written into the European Standard (CEN, 2004):

Range A: $$\varepsilon_{3000} - \varepsilon_{5000} < 0.045 * 10^{-3} \quad (8)$$

Range B: $$0.045 * 10^{-3} < \varepsilon_{3000} - \varepsilon_{5000} < 0.4 * 10^{-3} \quad (9)$$

Range C: $$\varepsilon_{3000} - \varepsilon_{5000} > 0.4 * 10^{-3} \quad (10)$$

Where ε_{3000} (10^{-3}) and ε_{5000} (10^{-3}) are the plastic strains at 3000 and 5000 load cycles, respectively, in the RLT test.

Most of the empirical-mechanistic permanent deformation models (e.g. equations (1), (3), (7) and many others not mentioned in this paper) have great difficulty or are unable to predict permanent deformation under high stress, i.e. Range C behaviour. Consequently, material responses that indicate Range C behaviour should be excluded from the model calibration.

The definition of the material properties for the VTT model was done from the RLT tests. Regression factors for the VTT model were obtained from the best-fit curve.

After the RLT test, a monotonically increasing static failure test was performed on the same specimen until failure of the material. Obviously, the specimen's behaviour will be influenced by the stress history of the RLT test; however, experience suggests that this effect is negligible (SAMARIS, 2006). The static test applies deviatoric stress at a strain rate of 1%/min. Tests were performed at four different confining stress levels (10, 20, 40 and 80 kPa).

Result and discussion

The results were interpreted in three steps: shakedown range analysis, model validation/calibration and permanent deformation calculation.

Shakedown range analysis evaluates the shakedown condition of the investigated UGMs, based on the RLT tests. The aim of this analysis is to study the shakedown behaviour of the UGM in the investigated road sections. Stress paths under "Range C" will be excluded from unbound material model calibration because of the inability of most of the models to reproduce "Range C" behaviour.

Table 4. An example of shakedown range evaluation (result for sample 2, subgrade, Nässjö).

Step	Number of loading cycles	Confining pressure (kPa)	Deviatoric pressure (kPa)	Shakedown range
0	10000	20	50	A
1	10000	20	80	A
2	10000	20	110	B
3	10000	20	140	B
4	10000	20	170	B
5	10000	20	200	C

Table 4 summarises behaviour of one set of the RLT tests on UGM with a constant confining pressure of 20 kPa. In accordance with Sequence 1 of Table 3, the deviatoric pressure was kept constant in each step but stepped up to a higher pressure level in the following step. This type of shakedown range analysis was performed for all samples at the initial sequence (not necessarily the Sequence 1 of Table 3) but data indicating Range C behaviour was excluded from use in model validation/calibration.

Model validation/calibration Parameters b and C (see equation (6)) were defined for the VTT model for both UGM and subgrade by a regression technique. From guessed initial values, the sum of the squares of the difference between the measured, and VTT model estimates of, strain was minimized by iteration to obtain the most appropriate values of parameters b and C.

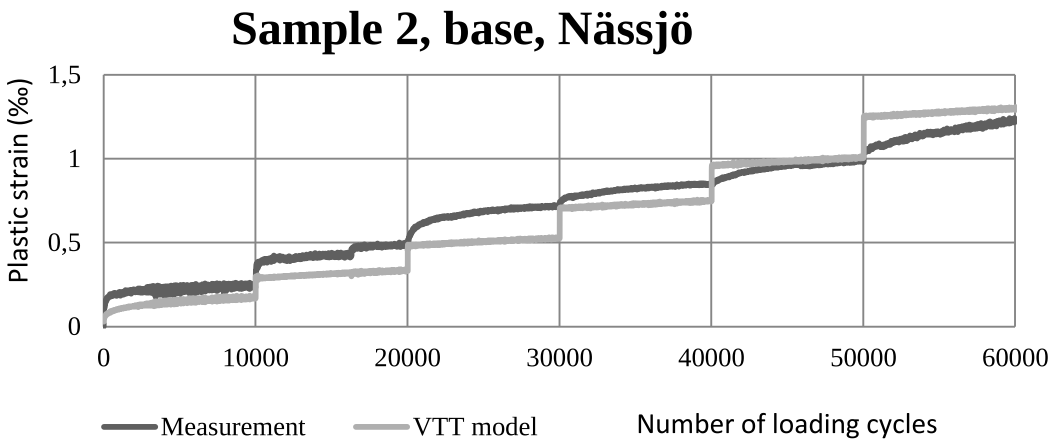

Figure 4. An example of regression factor calibration (Sample 2, base, Nässjö).

Overall regression factor was calculated based on all test samples (see Table 5a) from the same layer of the road to yield the regression factors for that layer. To exclude the effect of stress history, only the initial sequence was considered in the calculation. The pressure level used in the initial sequence (see Table 5b) on various samples can be found in Table 3. Figure 4 presents results from a sample material, showing the strains measured in the laboratory tests and the strain that would be predicted by the best-fit parameters b and C for the same material. A similar approach was also adopted to fit the Gidel model (equation (7)) to the same laboratory data. All validated factors are listed below:

Table 5a. Validated factors for the VTT model and the Gidel model.

	Layer	The VTT model		The Gidel model		
		C	b	e_{1p}	B	n
Nässjö	base	0.038	0.218	0.800	0.080	0.190
	subbase	0.052	0.200	2.700	0.018	1.080
	subgrade	0.117	0.200	83.429	0.001	1.832
Trädet	base	0.038	0.200	0.520	0.057	0.100
	subgrade	0.038	0.340	53.089	0.007	0.569

Table 5b. Pressure sequence (Table 3) for model validation.

Layer	Nässjö	Trädet

	Sample	Initial sequence	Sample	Initial sequence
Base	1	1	1	1
	2	1	2	2
	3	2	3	3
	4	3		
Subbase	1	1		
	2	1		
	3	2		
Subgrade	1	1	1	1
	2	1	2	1
	3	1	3	1

Note: in the calculation, regression factors may yield negative parameter values. To avoid this, the factors are recalculated, and constrained to yield values in the ranges. The range is $0.038 < C < 0.12$ and $0.2 < b < 0.4$ for the VTT model; and $0 < B < 0.1$ and $0 < n < 2$ for the Gidel model. The range for the VTT model was based on experience from a series of laboratory tests with different samples ranging from sand to crushed rock (Korkiala-Tanttu, 2005) which covers the samples in this study.

Permanent deformation calculation was performed by running the finite element program VägFEM loaded with either the VTT or the Gidel model to evaluate rut depth. The calculated rut depth was reduced by 30% to allow the effect of lateral wheel wander as recommended by the NCHRP synthesis 325 report (Hugo and Epps-Martin, 2004). The calculated rutting at two different roads sections is shown in Figures 5 (a & b) with the in-situ rutting as measured by laser equipment.

It should be mentioned that the rutting immediately after the pavement construction should be zero, both from measurement or calculations, before any traffic loading commences. The reason why the measurement curves do not start from zero is because rutting measurement was not available until many years after construction. So the measurement curve started at the year when first rutting measurement was made.

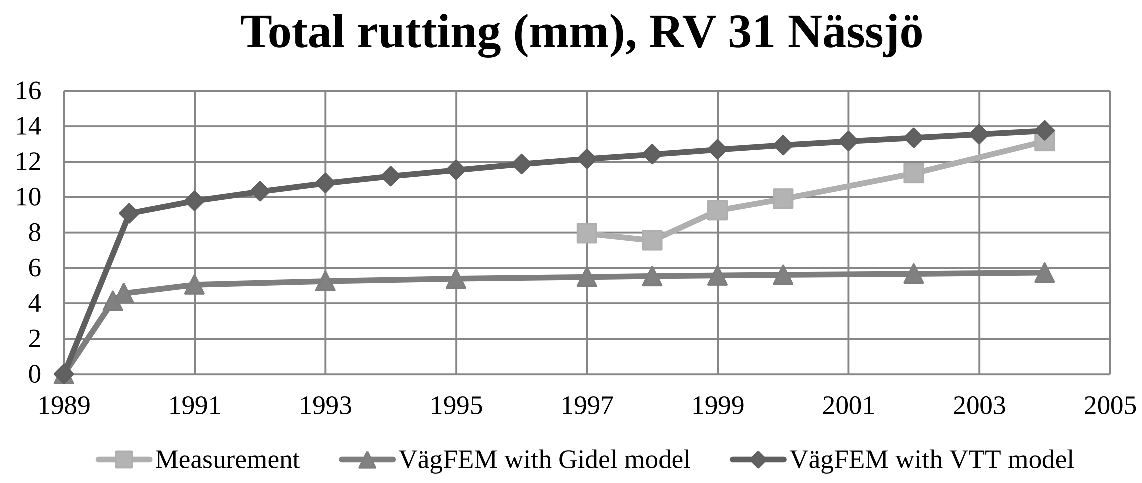

Figure 5a. Comparison between rutting measurement and model prediction for Nässjö.

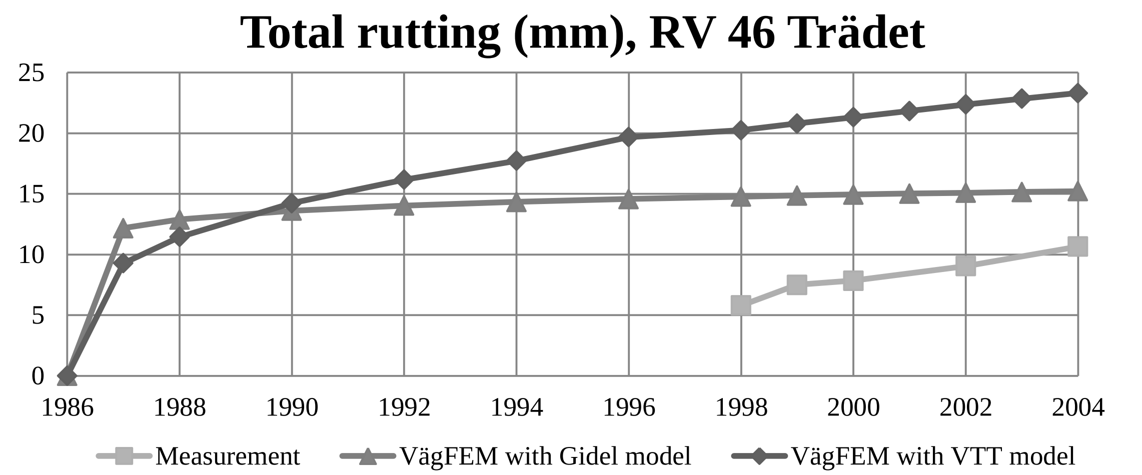

Figure 5b. Comparison between rutting measurement and model prediction for Trädet.

It seems that, with the VTT model, VägFEM generally had a high prediction of rutting for the two sites. Prediction by VägFEM with Gidel model revealed that most of rutting occurred within the first year of trafficking. However the development of rutting is predicted to be minor after that, which does not agree with the rate of development of rutting by the measurement.

VägFEM with the VTT model exhibited a relatively good prediction for Nässjö (Figure 5a), judging from the available rutting measurement, despite a decreasing overestimation until 2004. With the Gidel model, VägFEM seems to have underestimation and the prediction of rate of development of rutting was poor as discussed.

For the Trädet site, Figure 5b reveals overestimated rutting by VägFEM with either the VTT model or the Gidel model. Again, prediction of VägFEM with the Gidel model showed that most of the rutting (approximately 86%) occurred within the first two years. Despite the overestimation, VägFEM with the VTT model had a good prediction of the rate of development of rutting.

This data and mismatch of the modelled response gives some further support to the two-part approach first proposed by Alabaster et al. (2002) in which an early, so-called, ‘compaction’ phase is followed by a ‘wear’ phase. Werkmeister (2003) noted a similar division between early and later accumulation of plastic deformation although she used different terms for these. From this understanding it would be concluded that the VägFEM/VTT combination gave good replication of the component termed ‘wear’ by Alabaster et al. (Werkmeister’s ‘post-compaction’ or ‘Phase 1’), but not of the phase they termed ‘compaction’ (Werkmeister’s ‘Phase 2’). Note, Alabaster et al.’s ‘wear’ component is not the same as the wear contribution to rutting as described at the beginning of the Introduction of this paper, although it may include that contribution as a component.

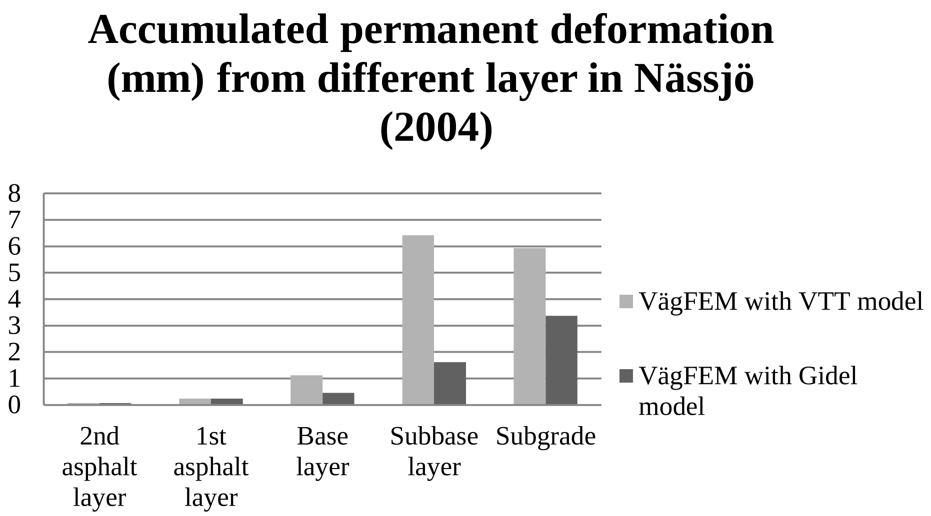

Figure 6. Prediction of permanent deformation in different layers, Nässjö.

Figure 6 shows the permanent deformation from different layers in Nässjö at the end of 2008. The permanent deformation prediction from the VTT model is higher than that from the Gidel model in all unbound layers and subgrade. The difference is especially notable for the subbase layer, presumably because of the high failure ratio (see equation (4)).

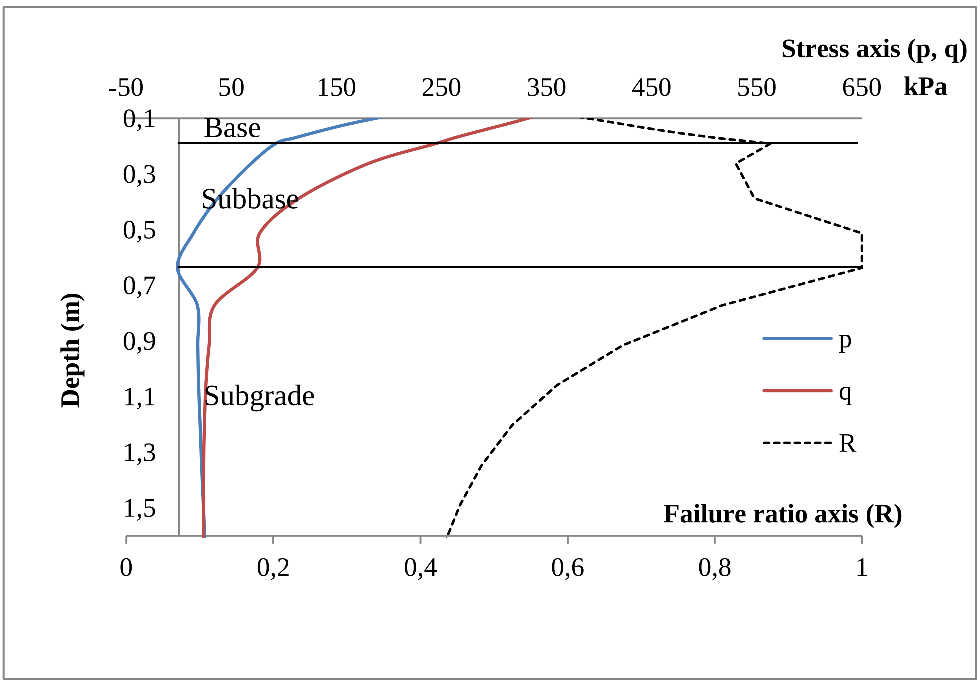

Figure 7. Stress and failure ratio profile calculated from VägFEM result, Nässjö (at 23 °C).

It could be observed from analysis (Figure 7) that the failure ratio is high in the subbase layer, which indicates that the stress at that depth is sufficiently close to the static failure stress of the material. When the R approaches 1, the value of 1 – R (the denominator of the VTT formula, see equation (6)) will approach zero. Thus the VTT formula would then predict exaggerated strain values, which will thereby cause an overestimation of the rut depth. Nonetheless, this high failure ratio state is unlikely to occur in the real granular materials because the aggregates can rearrange themselves by plastic deformation under higher stresses, resulting in better distribution of stress to bring down the stress level. Unfortunately, VägFEM is incapable of modelling these effects.

Thus, to use the VTT model when the failure ratio equals 1, the denominator of the VTT equation is set to be 1.05 – R to avoid an unrealistic prediction, as is also recommended by the author (Korkiala-Tanttu, 2009). Even so, overestimation of deformation under high failure ratio cannot be avoided. For instance, it is clearly seen in Figure 7 that R is high in the subbase layer, and the overestimation of permanent deformation by VägFEM with the VTT model is highest in this layer.

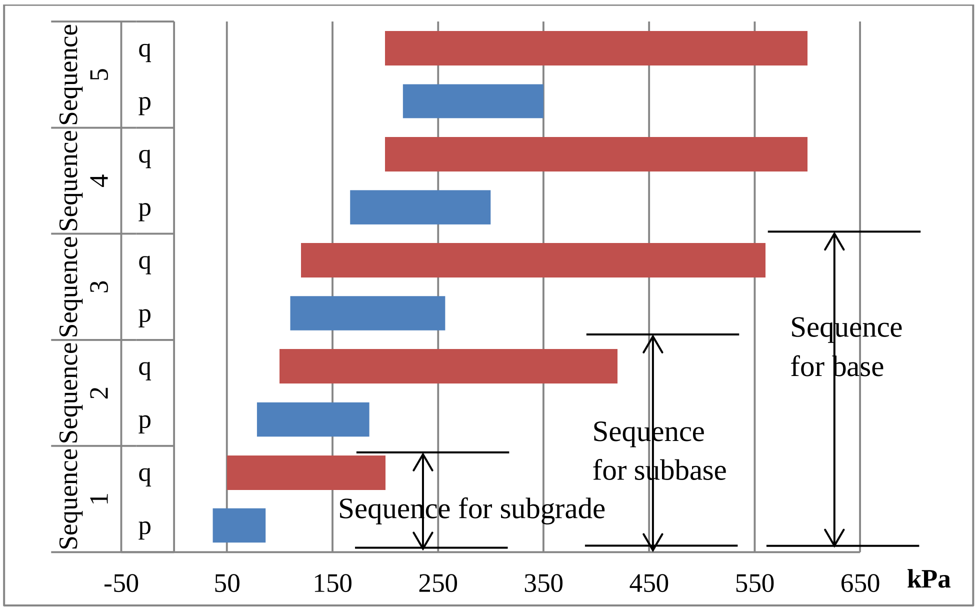

Figure 8. Stress state in triaxial testing.

Figure 8 shows the p and p stress level adopted in all five sequences of European standard triaxial testing (high pressure level) calculated according to the confining and deviatoric stress for each sequence in Table 3. It can be seen that the stress range adopted in model calibration (see Table 5b) for different layers approximately covers the anticipated in-situ stress state (seen in Figure 7). In general, the average stress in the base material is greater than in the subbase and much greater than that in the subgrade. Thus for subgrade material in the analysed roads, Sequence 1 was considered to be representative for the in-situ stress. Sequence 2 was added in the validation for the subbase material and Sequence 2 & 3 for the base material (see Table 3 & 5b). Thus this data demonstrates that the stresses computed using the VägFEM programme match those assumed for the calibration of the permanent deformation models that have been used to obtain Figures 5a & 5b.

From all the prediction results, it can be seen that the validated VTT model had fair prediction of the rutting development for the two Swedish roads. The possible difference in straight edge and laser measurement, described earlier, may offer a small correction to this overestimation, but it is not sufficient to explain it.

It should be mentioned that the Gidel model calculates permanent deformations only in Range A from VägFEM and that the program limits the value of $(m + s/p_{max} - q_{max}/p_{max})$ to 1. The reason for this is that the permanent deformation becomes unrealistic high when the stress level approaches the failure line. In this way the Gidel model calculates only the 'compaction' phase (Range A) of the permanent deformation, and neglects the 'wear' phase (surely the basis of Range B), which takes place with about the same deformation every year (i.e. in linear proportion to the amount of heavy traffic). See also the NordFoU, report on calibration (Huvstig, 2010).

Conclusion

The main effort of this study has been to identify the usefulness and limitations of the two UGM permanent deformation models. On the basis of the study described, the following conclusions can be drawn:

- Despite an overestimation of the magnitude of the rutting, the prediction from the VTT model is reasonable at describing the rate of development of rutting, but probably not the initial accumulation.

- The VTT model tends to give an overestimation of rutting for thin asphalt flexible pavements, suggesting that it requires adjustment as failure stress is approached in the unbound granular layer(s) of the pavement – a situation that may occur in pavements with a thin asphalt surface or unsealed pavements.

- The 'compaction – wear' approach suggested by Alabaster et al. (2002) provides a useful concept for understanding rut development. The VTT model is much better at replicating the 'wear' phase than the 'compaction' phase as defined by Alabaster et al. The Gidel model may be more suitable for predicting the 'compaction' phase.

- Both the VTT model and Gidel model predicts high percentage of rut depth in the first one or two years after trafficking. The prediction of development of rutting after one or two years is minor by Gidel model. Associated with this observation it may be concluded that the Gidel model is better at predicting the deformation in the first 'compaction' phase than in the second 'wear' phase.

- When the stress level in a pavement's UGM approaches failure, plastic deformation prediction becomes challenging.

This paper has illustrated the difficulties of obtaining controlled in-situ data from "the thinly sealed" pavements that can be used to validate idealised permanent deformation models. Specific, controlled test data might be more able to achieve this. Considering the data from an alternative perspective, the results should caution users of idealised models from expectations that they will be able to adequately capture all the variability and uncertainties of rut development in real pavements.

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