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Calculation method for permanent deformation of unbound pavement materials

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Keywords permanent deformations, rutting, unbound pavement material, pavement design, stress distribution

Abstract

An analytical-mechanistic method for the calculation of permanent deformations of pavements has been developed at the Technical Research Centre of Finland (VTT) over some years by the author. The calculation method is needed in the analytical design procedure of pavements. This research concentrated on the calculation method for permanent deformations in unbound pavement materials. The calculation method was generated based on the results of full-scale accelerated pavement tests along with the complementary laboratory tests together with finite element calculations.

The objective was to develop a relatively simple material model for unbound materials, which is an analytical, nonlinear elasto-plastic model. The stress distribution studies of traffic load showed that it is very important to calculate stresses in pavements with an elasto-plastic material model to avoid false tensile stresses in unbound materials, especially when the asphalt layers are thin.

The new material deformation model can take into account the amount of the loading, the number of vehicle or wheel passes, the deformation capacity of the material and its stress state. The strains in each layer and subgrade are calculated and converted to the vertical deformations and then summed to obtain the total rutting. The method was verified against two Finnish accelerated pavement tests. The results indicated that the material model gave tolerable results for the relatively high load levels used in these Heavy Vehicle Simulator (HVS) tests as the relative error was around ± 30%. For the structures with thicker bound layers and therefore lower stress state in the unbound layers, the method gave more reliable results.

The material parameters have been defined only for the most common Finnish unbound materials in a few basic situations. The wider use of the method requires material parameter definitions for a larger range of materials. However, even in the current form the method can be applied in a relatively reliable way to compare the sensitivity of different structures against rutting.

The most important factors affecting rutting were studied to find a method to include their effect on the calculation method. These factors were loading rate, stress history, temperature and the geometry of the road embankment. The modelled examples proved that the most important factor of rate effect is the change in stress state due to the change in the resilient properties of bound layers, while the rate effects on the unbound material itself has a smaller role. The accelerated pavement test proved that rut depth depends greatly on the temperature: the rut depth grows from 10% to 15% at +10 ºC and 20 to 25% at +25 ºC compared to rut depths at +5 ºC due to the changes in the stiffness of the bound layer. The unloading-reloading cycles have only a slight effect on the permanent deformation. The introduced geometric factor describes an average, structurally independent, increase in the rate of rutting, which depends on the steepness of the side slope and on the distance to the edge of the structure.

Korkiala-Tanttu, Leena. Calculation method for permanent deformation of unbound pavement materials [Tierakenteen sitomattomien materiaalien pysyvien muodonmuutosten laskentamenetelmä]. Espoo 2008. VTT Publications 702. 92 s. + liitt. 84 s.

Avainsanat permanent deformations, rutting, unbound pavement material, pavement design, stress distribution

Tiivistelmä

Viime vuosina VTT:llä on kehitetty menetelmä tien rakennekerrosten pysyvien muodonmuutoksien laskentaan. Menetelmä on osa päällysrakennekerrosten mitoitusmenettelyä. Menetelmä kehitettiin erityisesti sitomattomien rakennekerrosten muodonmuutoksien laskemiseen ja sen kehittämisessä hyödynnettiin sekä täyden mittakaavan koetiekonetta että laboratoriokokeiden tuloksia yhdessä mallintamisen kanssa.

Tavoitteena oli kehittää sitomattomien rakennekerrosten pysyvien muodonmuutosten laskentaan suhteellisen yksinkertainen materiaalimalli, joka on analyyttinen, epälineaarinen ja elastoplastinen. Liikennekuormitusta mallinnettiin elementtimenetelmällä. Mallinnusten perusteella osoittautui, että on oleellista mallintaa tien rakennekerrosten jännitystila pysyvien muodonmuutosten laskentaa varten elasto-plastisilla menetelmillä. Mikäli käytetään puhtaita elastisia menetelmiä, sitomattomiin rakennekerroksiin muodostuu laskennallisesti – erityisesti kun asfalttikerrokset ovat ohuita – vetojännitystä, jota niissä ei todellisuudessa voi olla juuri lainkaan.

Kehitetyllä menetelmällä voidaan ottaa huomioon kuormien suuruus, ylityskertojen määrä, materiaalien muodonmuutoskapasiteetti ja jännitystila. Menetelmällä lasketaan kerroksittain muodonmuutokset, joista voidaan edelleen laskea koko kerroksen pystysuuntainen kokoonpuristuma ja koko rakenteen urautuminen. Menetelmää testattiin tuloksiin, jotka oli saatu kahdesta ohutpäällysteisestä Suomessa tehdystä täyden mittakaavan kokeesta. Testauksen perusteella osoittautui, että menetelmä antoi melko luotettavia tuloksia suhteellisen virheen ollessa ±30 %, kun kuormitustaso pysyi kohtuullisena. Menetelmä antoi tätä luotettavampia tuloksia, kun päällystekerrokset olivat paksumpia ja siten myös sitomattomien kerrosten jännitystila oli alhaisempi.

Toistaiseksi menetelmän parametrit on määritetty vain kaikkein tavallisimmille Suomessa käytetyille rakennekerrosmateriaaleille muutamissa olosuhteissa. Menetelmän laajempi soveltaminen edellyttää materiaaliparametrien määrittämistä useammille olosuhteille ja materiaaleille. Kuitenkin jo tässä muodossa menetelmää voidaan soveltaa suhteellisen luotettavasti arvioimaan eri rakenneratkaisujen urautumisherkkyyttä.

Tutkimuksessa selvitettiin myös, mitkä ovat tärkeimmät urautumiseen vaikuttavat tekijät, ja kuinka niiden vaikutus voitaisiin ottaa huomioon laskentamenetelmässä. Tutkitut tekijät olivat kuormitusnopeus, lämpötila, kuormitushistoria ja tierakenteen geometria. Esimerkkilaskelmat osoittivat, että kuormitusnopeuden vaikutusta sitomattomiin kerroksien jännitystilaan voidaan parhaiten mallintaa muuttamalla päällystekerrosten muodonmuutosmoduulia (ns. jäännösmoduulia). Sen sijaan sitomattomien kerrosten omiin ominaisuuksiin kuormitusnopeus vaikuttaa melko vähän. Täyden mittakaavan kokeiden perusteella urautuminen riippuu merkittävästi lämpötilasta: Urasyvyys kasvaa 10–15 %, kun lämpötila nousee +5 ºC:sta 10 ºC:seen ja 20–25 %, kun lämpötila on +25 ºC. Kuormitushistorialla on sen sijaan selvästi vähäisempi vaikutus. Menetelmä sisältää ns. geometriatekijän, jonka avulla voidaan arvioida tien sivuluiskan läheisyyden ja kaltevuuden keskimääräistä vaikutusta urasyvyyteen.

Preface

To reach this final goal the trip has been a long, uneven path with many potholes, cracks and the occasional rut. The trip started in 2002 in the Material and Building cluster at the Technical Research Centre of Finland. The basis for the trip was the two accelerated pavement tests (APT) done in 2000–2002. The APT tests were both financed by the Finnish Road Administration. After the APT tests the trip has progressed step by step. The first steps were financed by the Academy of Finland and the Technical Research Centre of Finland. After that the paths merged with the Deformation-project of the INFRA Technology programme, which was financed by the National Technology Agency of Finland, the Finnish Road Administration, Lemminkäinen Oyj, Skanska Tekra Oy, Finnish Road Enterprise, Lohja Rudus Oy, the City of Helsinki and the Finnish Asphalt Association. Also the Finnish Geotechnical Society has financed the research. A warm thank you goes to all the funders or funding agencies for their support!

After finally reaching the goal, I want to thank many people for their help along the way. I would like to express my sincere thanks to my supervisor, Professor Pauli Vepsäläinen from the Technical University of Helsinki, for his encouragement and advice throughout the trip. Many other people from the University have also helped me, including Professors Olli Ravaska and Terhi Pellinen. I want to also sincerely thank my co-authors Rainer Laaksonen from VTT and Andrew Dawson from the University of Nottingham. They have tirelessly guided me and given their time and advice. To all my colleagues at VTT, many thanks for the light and spirit you gave me during the dark laps of the path.

There have been two remarkable crossings on my path. The first was at the University of Nottingham in autumn 2003 when I was orientated through the swamp of stress distributions. The second was in autumn 2007, when I visited the University of Strathclyde to finalise the trip. So, my deepest warm thanks to the staff of both universities. In addition, I want to raise a thankful glass to my dear friend Minna Karstunen from the University of Strathclyde.

I am grateful to professors Hannele Zubeck from University of Alaska Anchorage and Pauli Kolisoja from Tampere University of Technology, the official reviewers of this thesis for their constructive critique.

I would not have started this trip without my parents' strong belief in me. And during the time I have rambled, there was steady and loving support from my family: Jukka, Tuomo and Kimmo. I have also got whole-hearted support and belief from other relatives and friends. Alone I would not have made it to the end. My grateful thanks!

Espoo, December 2008

Leena Korkiala-Tanttu

Contents

Appendix C: Publications III, IV of this publication are not included in the PDF version. Please order the printed version to get the complete publication (http://www.vtt.fi/publications/index.jsp)

List of publications

This thesis is based on the following publications (Appendices I–VII), which are referred to in the text by the corresponding Roman numerals:

I Korkiala-Tanttu, L. and Laaksonen, R. 2003. Permanent deformations of unbound materials of road pavement in accelerated pavement tests. Proceedings of XIIth Conference on Soil Mechanics and Geotechnical Engineering, Prague, Vol. 2, The Czech Geotechnical Society CICE, Prague. Editors: Vaníček, I., Barvínek, R., Boháč, J., Jettmar, J., Jirásko, D. and Salák, J. Pp. 665–670.

II Korkiala-Tanttu, L. and Laaksonen, R. 2004. Modelling of the stress state and deformations of APT tests. Proc. of the 2nd International Conference on Accelerated Pavement Testing, September 26.9.–29.9.2004, Minneapolis, Minnesota, CD-rom proceedings. Editor: Worel, B. 22 p. http://mnroad.dot.state.mn.us/research/MnROAD_Project/index_files/pdfs/KorkialaTanthu_L.pdf.

III Korkiala-Tanttu, L. and Dawson, A. 2008. Effects of side-slope on low-volume road pavement performance: a full-scale assessment. Canadian Geotechnical Journal, NRC Research Press, Ottawa, Vol. 45, June 2008, pp. 853–866.

IV Korkiala-Tanttu, L. and Dawson, A. 2007. Relating full-scale pavement rutting to laboratory permanent deformation testing. International Journal of Pavement Engineering, Taylor & Francis, Spon Press, Abingdon, Vol. 8. Issue 1, pp. 19–28.

V Korkiala-Tanttu, L. 2005. A new material model for permanent deformations in pavements. Proc. of the Seventh Conference on Bearing Capacity of Roads and Airfields, Trondheim 27.6.–29.6.2005, CD-rom proceedings, Publisher Ny Media AS, Oslo. Editor: Horvli, I. 10 p.

VI Korkiala-Tanttu, L. 2007. Speed and loading effects on pavement rutting. Proceedings of ICE, Geotechnical Engineering, Thomas Telford Journals, Vol. 160, July 2007, Issue GE3. Pp. 123–127.

VII Korkiala-Tanttu, L. 2008. Stress studies for permanent deformation calculations. Advances in Transportation Geotechnics, Proc. of the 1st International Conference on Transportation Geotechnics, Nottingham 25.8. –27.8.2008, CRC Press, London. Editors: Ellis, E., Yu, H.S., McDowell, G., Dawson, A. and Thom, N. Pp. 201–206.

The author's contribution

I The author has processed and analysed the test data of the HVS tests and written the publication. The co-author has done the laboratory tests and descriptions of the laboratory test procedures. The co-author has also made some comments on the text.

II All the calculations and analysis of the results have been conducted by the author. The study was part of the 'Deformation'-project, which the co-author led. The co-author has supervised the studies and commented on the written text. This conference paper has been peer-reviewed.

III The author has analysed the data, made the literature review and all calculations. The original text has been written by the author and the co-author has proposed additions and changes to the text, edited it and checked the linguistic form.

IV The author has made all calculations and analysed test and calculation results. The co-author has supervised the analysis of the results, edited the text and checked the linguistic form.

V The author has written the publication alone. This conference paper has been peer-reviewed.

VI The author has written the publication alone.

VII The author has written the publication alone. This conference paper has been peer-reviewed.

Abbreviations and symbols

Abbreviations

AADT	annual average daily traffic
AASHO	American Association of State Highway Officials (former AASHTO)
AASTHO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ALT	Accelerated Loading Test
APT	Accelerated Pavement Test
AUSTROAD	Association of Australian and New Zealand road transport and traffic authorities
CAUC	consolidated anisotropic undrained compression
CCP	constant confining pressure test
DOC	degree of compaction
DOT	Department of Transportation
EMU-coils	εmu-coil: inductive coil pair for strain (ε) measuring
FE	finite element method
Finnra	Finnish Road Administration
GEOM	geometry factor
HS	Hardening soil material model (Plaxis)
HVS	Heavy Vehicle Simulator
LCPC	Laboratoire Central des Ponts et Chaussées
LE	Linear elastic material model
LTPP	Long-Term Pavement Performance programme
LV	Low-Volume accelerated pavement test series
MC	Mohr-Coulomb's material model (Plaxis)
MMOPP	Mathematical Model of Pavement Performance
MnRoad	pavement test track in Minneapolis, Minnesota
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
OECD	Organisation for Economic Co-operation and Development
RTM	Road Testing Machine (Danish testing facility)
SAMARIS	Sustainable and Advanced Materials for Road Infrastructures project

SASW	spectral analysis of surface waves
SHRP	Strategic Highway Research Program
SO	Spring-Overload accelerated pavement test series
TPPT	Tien Päällys- ja Pohjarakenteiden Tutkimusohjelma (Road Structure Research Programme)
USCS	Unified Soil Classification System
VTI	Swedish National Road and Transport Research Institute
VTT	Valtion Teknillinen Tutkimuskeskus (Technical Research Centre of Finland)

Symbols

A	maximum value of the failure ratio R in Equations 3 and 4
B	material parameter in Equation 3
C	compaction degree and water content parameter
D	degree of compaction in Figure 12
E	Young's modulus, MPa
E_{oed}	compression modulus, MPa (Plaxis Hardening soil model)
E_{ur}	unloading/reloading modulus, MPa (Plaxis Hardening soil model)
E_1, E_2	non-linear modulus in Figure 9
E_{50}	deviatoric modulus, MPa (Plaxis Hardening soil model)
K_0	coefficient of earth pressure at rest
M_r	resilient modulus, MPa
N	steepness of the slope in Equation 11 and Figure 17
N	number of load cycles (Equations 2 and 8)
R	failure ratio (q/q_f)
T	temperature, C
X	maximum value of the failure ratio R in Publication VII
a, b	regression parameters in Equation 2
b	shear ratio parameter depending on material (Equations 8, 10)
c	material parameter (Equation 10)
c	cohesion in Equation 7, kPa
d	material parameter in Equation 10
l	distance of the loading (centre line of the wheel) from the slope crest, m

m	stiffness power parameter (Plaxis Hardening soil model)
p'	effective hydrostatic stress, kPa
p	hydrostatic stress, kPa
q	deviatoric stress, kPa
q_f	deviatoric stress in failure, kPa (Equation 5)
q_0	deviatoric stress, when p' =0 (Equation 7)
v	loading rate, km/h
α	material parameter c in Publication IV
β	material parameter d in Publication IV
γ	shear strain, -
ε_p	permanent vertical strain, -
ε^1_p	permanent axial strain in Equation 2, -
ε_s	shear strain, -
ε_v	volumetric strain, -
$\varepsilon_{z\,max}$	transient elastic displacement, -
ε_1	first principal strain in triaxial test, -
ε_1	vertical strain in triaxial test, -
$\varepsilon_{1,}\ \varepsilon_2$	strains in Figure 9, -
ν	Poisson's ratio, -
ν_{ur}	unloading-reloading Poisson's ratio, - hardening soil model
σ	normal stress, kPa
$\sigma_{z\,max}$	earth pressure, kPa
σ_1	vertical stress in triaxial test, kPa
σ_{1f}	vertical failure stress, kPa
σ_3	cell pressure in triaxial test, kPa
$\sigma'_{1,}\ \sigma'_2$	pressures in Figure 9, kPa
σ_I	the first principal stress, kPa
τ	shear stress, kPa
φ	friction angle, °
ψ	dilatation angle, °

1. Introduction

1.1 Scope of the research

Finnish pavement design is based on model structures defined by the Road Administration [Tiehallinto 2004]. Besides the standardised model structures, the design guideline allows the designer to use either the manual pavement design method according to the two layer theory of Odemark [1949] modified by FinnRa [Tielaitos 1985] or the elastic multilayer program. In the design guide the material parameters have been fixed to the empirical values and laboratory tests are seldomly used for the determination of the pavement materials. The material parameters have been presented for only the most commonly used natural or crushed granular materials. The design procedure is therefore restricted and it does not accommodate new design demands.

The demand for an analytical-mechanistic calculation method for pavement design in Finland has increased due to the changes in procurement processes. These changes enable the use of new pavement structures including for example recycled materials and optimisation of the pavement. Therefore more accurate design methods are needed to produce life-cycle performance evidence for the contractors and owners, also after construction or rehabilitation. The traditional design methods are not able to manage new innovative materials, recycled materials or reinforcements. Obviously, more efficient and sophisticated analytical-mechanistic design methods are needed. Analytical design methods also allow the studies needed in the implementation of the life cycle performance evaluations.

The Finnish design system of the life cycle based road design was introduced in the 'Road Structures Research Programme' (TPPT) [Tammirinne et al. 2002]. Figure 1 illustrates the entire design system. The life cycle performance based road design system can again been separated into smaller fragments (Figure 2), which are: fatigue, wearing and permanent deformation (rutting), and frost and settlement analysis. The research reported here concentrated on the development of a permanent deformation calculation method for unbound granular materials. The development of the accelerated pavement test (APT), also known as the accelerated loading test (ALT), has allowed a deeper understanding of the behaviour of pavements supported by laboratory tests. In the research results of an APT facility, the Heavy Vehicle Simulator HVS (also called HVS-Nordic), has been employed. The scope of the research included a literature review, experiments and their analysis and modelling together with the development of the deformation model and calculation method.

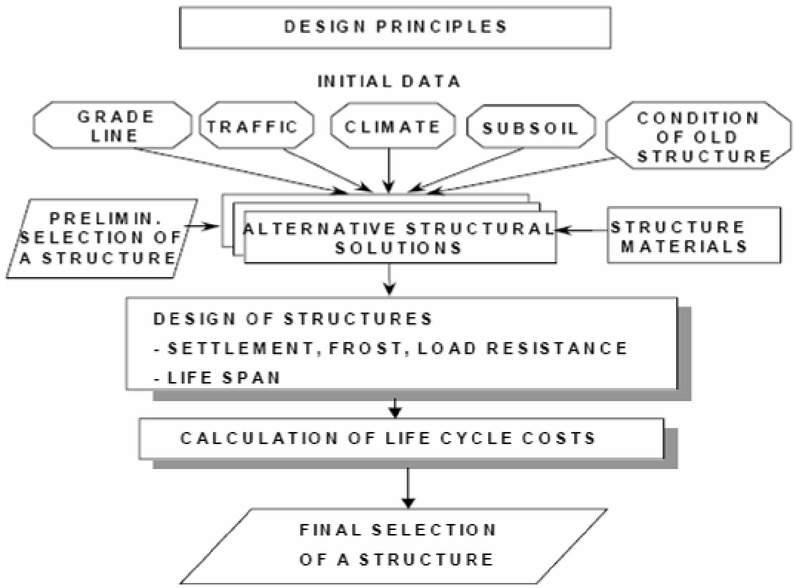


Figure 1. Flow chart of the TPPT design system [Tammirinne et al. 2002].

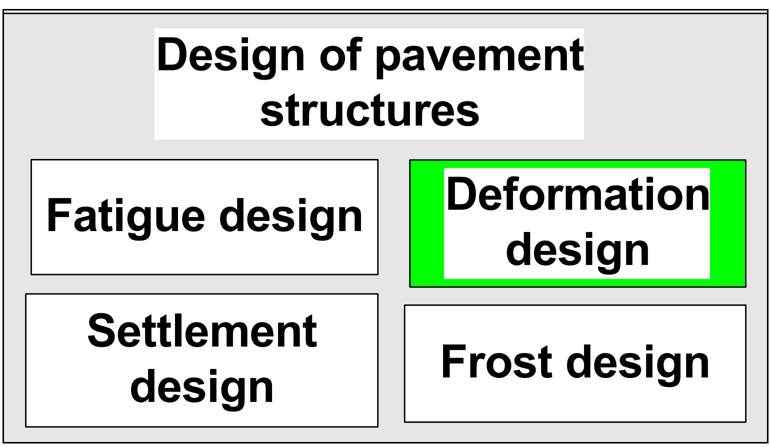


Figure 2. TPPT design system of pavement structures; design tasks.

The applied HVS tests conducted in years 2000–2001 have been specially constructed to represent Finnish low-volume road structures. For the research the most important test series have been 'Spring – Overload' [Korkiala-Tanttu et al., 2003a] and 'Low volume road research' [Korkiala-Tanttu et al. 2003b] tests. The HVS tests have been completed with simultaneous laboratory tests, such as repeated loading triaxial tests and monotonic strength tests for the unbound materials. The laboratory tests are described in more detail in the 'Deformation' project's report [Laaksonen et al. 2004]. The collected and analysed test data have been used as the backbone for the research. So far the permanent deformation material parameters have been defined for the most typical unbound Finnish road construction materials.

The developed permanent deformation calculation method can be applied for flexible pavements. And indeed, it is suitable for low-volume roads or for heavily loaded fields and port constructions. In a low-volume road or heavily loaded field, the permanent deformations of structural layers and the subgrade play a significant role in the deterioration and service life of the road. It is also possible to calculate the effect of the reinforcement with a finite element program, which includes elements for geogrids and interfaces (like Plaxis). Figure 3 illustrates the pavement structural layers that have been used in this research. The applications have mainly covered low-volume roads so there has only been one bound asphalt layer.

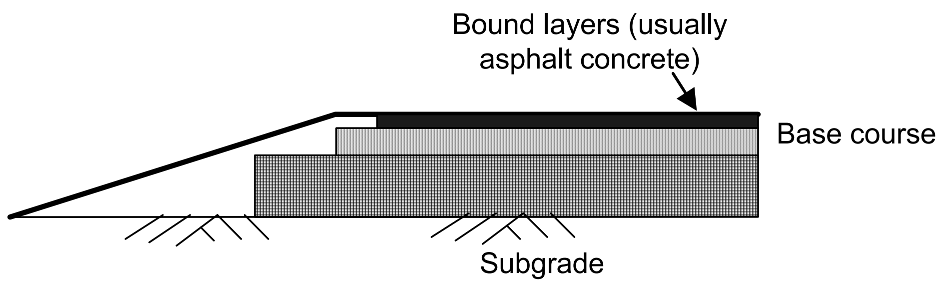


Figure 3. The terminology used for the pavement structural layers.

Figure 4 illustrates how the publications are connected to each other and how the material model development has progressed.

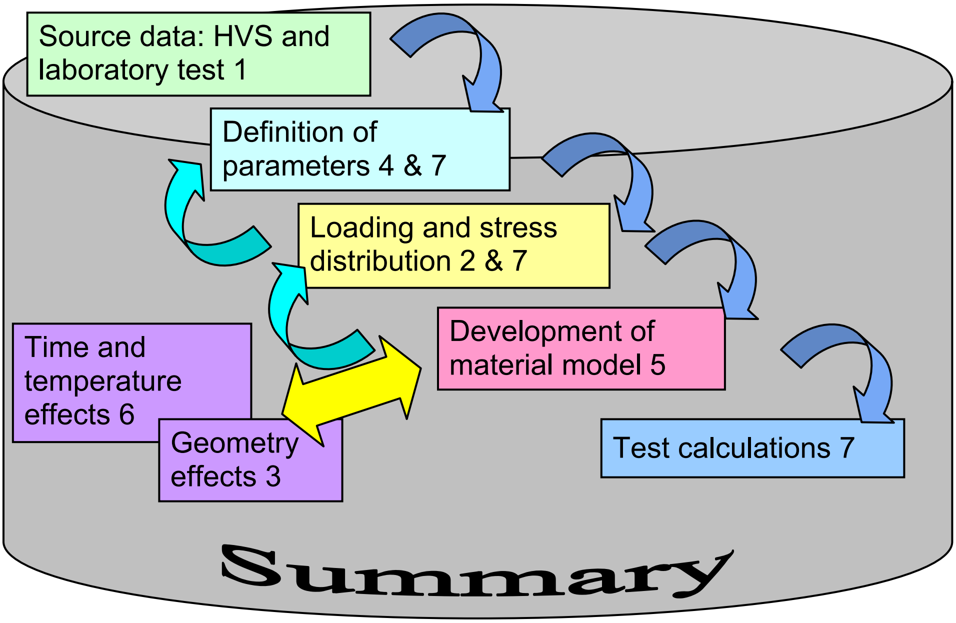


Figure 4. Progress of the material model development and the corresponding publication numbers.

1.2 Theory background

The permanent deformation of a pavement (also called rutting) has been classified into three categories in Laaksonen et al. [2004]: wearing of the asphalt layers, compaction and shear deformations. Dawson and Kolisoja [2004] have presented four different mechanisms of rutting, which have been labelled into Modes 0 to 3. The mechanisms are compaction, shear deformation, deformation of subgrade and particle damage. Both references agree that the most common type of rutting is the mixture of these basic mechanisms.

The permanent deformation is defined here to mean only the vertical permanent deformations in the unbound, granular material including also the subgrade. Even though only the vertical permanent deformations are calculated, they include also the shear strains, because the strain is defined as a function of the shear stress ratio (here failure ratio). So the permanent deformations in this work include compaction or dilatation of the unbound materials and subgrade, but not wearing or the permanent deformation in the bound surface layers.

The research approaches the problem from the geotechnical point of view, which means that in the calculations the compression stresses are positive and tensile stresses are negative. This is opposite of the Plaxis software, where the compression stress is negative and tensile stress is positive. In this thesis all presented stresses are effective stresses with the exception of the results of soil pressure cells, which measure the total stresses. The calculations have mainly been conducted in the drained conditions, because the pavements layers are usually unsaturated. However the calculations, which contain the rise of the water table level, have been conducted in the fully saturated state. Theoretically the developed material model is valid when there is no incremental collapse.

The assumptions applied in the development of the calculation method have been:

- all granular materials were treated as if they were elasto-plastic continuum materials,
- all bound materials were treated as linear elastic continuum materials,
- the stress calculations have mainly been conducted with axisymmetric studies,
- the unbound material will not have an incremental collapse (accumulation of permanent deformation after numerous loading cycles),
- tyre pressures have been treated as evenly distributed circular loadings with the same contact pressure area as for rectangular loading,
- the contact pressure areas for different tyres are based on the field studies made at VTT and they are presented in Appendices A and B,
- the seasonal changes (water content, temperature) were taken into account by changing material parameters,
- the loading rate was taken into account by changing the viscous parameters of bound material,
- the geometry of the pavement structure was taken into account by using a factor called GEOM,
- for simplicity Poisson's ratio has been chosen to be a constant value, even if the test results of for example Ekblad & Isacsson [2006] show that it is clearly stress dependent,

- the rotation of the principal axis has not been taken into account, because the stresses have been calculated under the symmetry axis, where the directions of the principal stresses concur with the symmetry axis (Figure 5),
- due to the reasons stated above the triaxial approach of the Mohr-Coulomb failure criteria has been applied,
- the maximum shear strength of unbound materials has been calculated with the Mohr-Coulomb or Hardening soil material model.

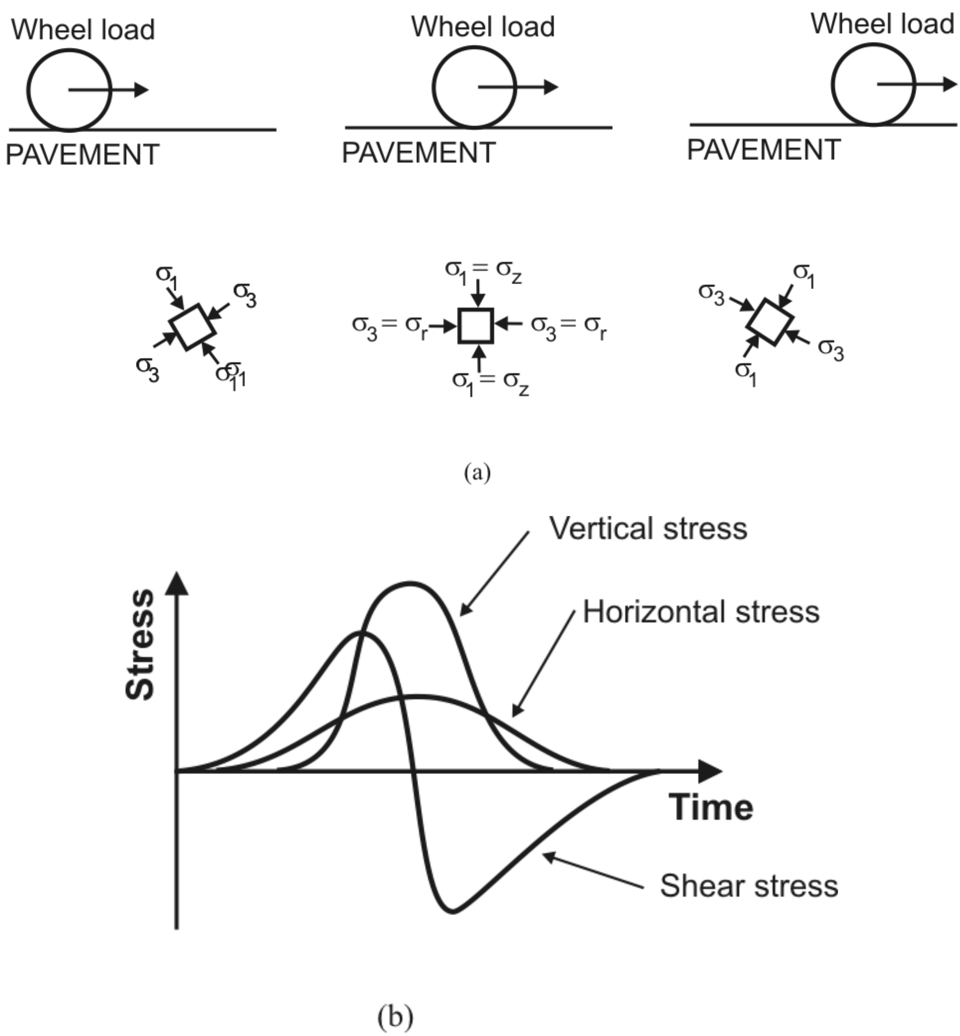


Figure 5. The rotation of the principal axis and the changes in different stresses under wheel load [e.g. Lekarp 1997].

Fredlund et al. [1996] presented a modified Mohr-Coulomb failure criterion for partly saturated materials (pavement layers are normally partly saturated), but this approach can not be applied here because the suction and partly saturated material parameters are not known (Figure 6). The suction pressure depends on the material and its water content, and it increases the strength and deformation modulus of the material. To compensate for this, higher strength parameters have been applied in the case of partly saturated materials (VII).

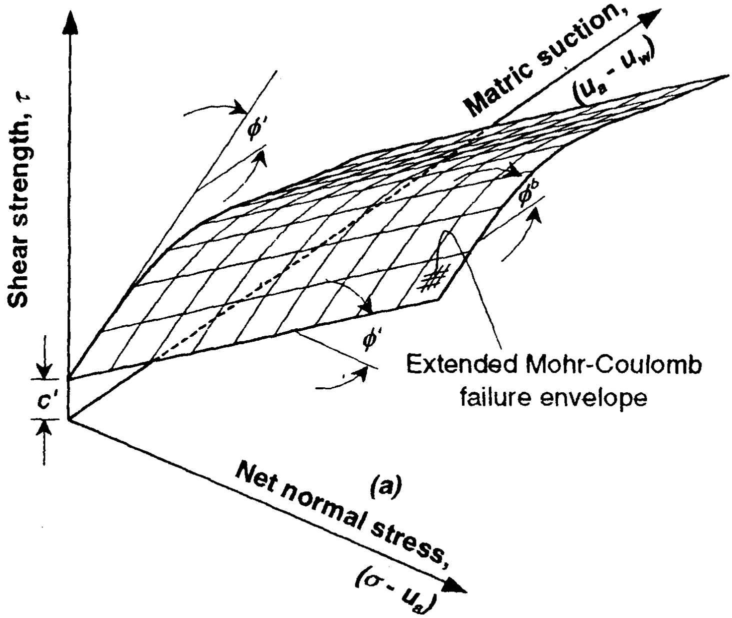


Figure 6. Extended Mohr-Coulomb failure envelope for partly saturated soils [Fredlund et al. 1996].

The deformation of the pavement material is divided into two main parts (Figure 7): elastic (or resilient) and permanent deformations. Permanent deformations are called plastic and they are irreversible. With low stress levels the deformations are mainly reversible in other words elastic, but when the stress level rises the proportion of the permanent deformation also grows together with the total growth of the deformations (Figure 8a). It is typical for the unbound granular materials that even in the low stress states they act like non-linear elasto-plastic materials and do not bear tensile stress.

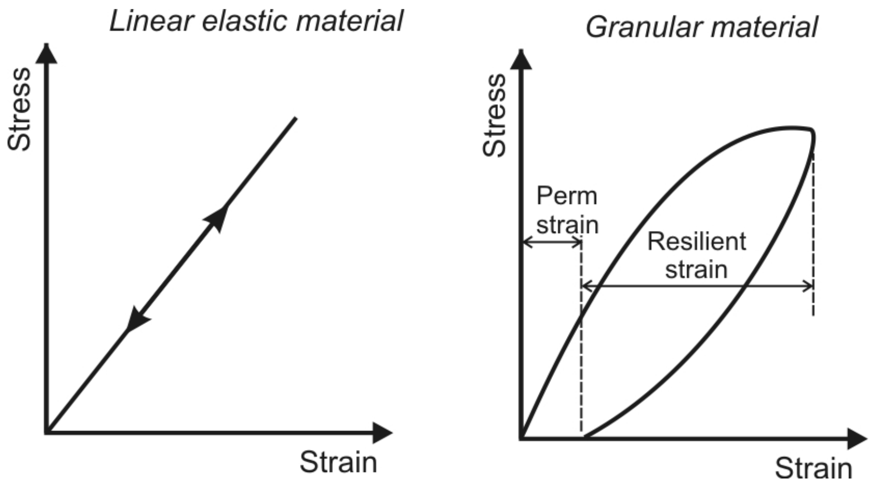


Figure 7. Deformations of linear elastic material and granular material [e.g. Lekarp 1997].

Figure 8b presents the growth of both resilient and permanent deformations under cyclic loading. The whole deformation-loading event is a combined process, where deformations are highly dependent on each other [Huurman 1997].

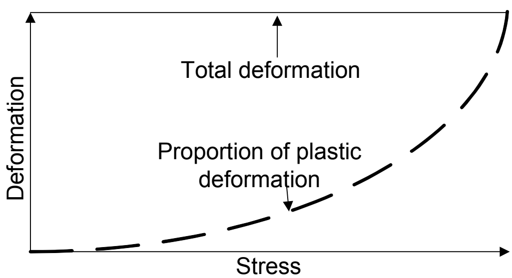


Figure 8a. Principal relation between plastic and total deformations.

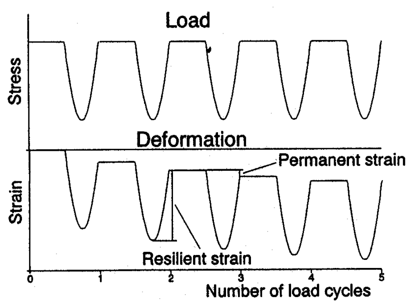


Figure 8b. Deformations of the granular material under cyclic loading [Huurman 1997].

The actual traffic loading is far from the monotonic cyclic loading and resembles more a deterministic pulse load, with varying intensity and dynamic pulses. In the calculations the loading is supposed to be permanent or dynamic. In the laboratory the traffic loading is usually simulated as a harmonic and monotonic loading generating excess pore pressure.

The strains of granular material can be either volumetric or shear strains. Volumetric strains are mainly caused by hydrostatic stress and it is either compacting or loosening. The volumetric strains are mainly occurring in the vertical direction and are concentrated under the wheel path. Shear strains are governed by the deviatoric (or shear) stress. In the pavement layers shear strains usually happen laterally, just slightly away from the centre of the wheel path. Thus, it also participates in the growth of the rut depth. The shear strain can compact, loosen or keep the material at the same volume.

For the unbound pavement materials the elastic modulus is the relation between stress and strains and it is usually referred to in pavement engineering as resilient modulus M_r. This modulus is defined with Equation 1 for the traditional triaxial test, where σ_1 is the changing vertical stress and σ_3 is the constant cell pressure. The resilient modulus is strongly dependent on the stress state (Figure 9).

$$M_r = \frac{\Delta(\sigma_1 - \sigma_3)}{\Delta\varepsilon_1} \tag{1}$$

where	M_r	is	resilient modulus (kPa)
	σ_1		vertical stress in triaxial test (kPa)
	σ_3		cell pressure (kPa)
	ε_1		vertical strain in triaxial test.

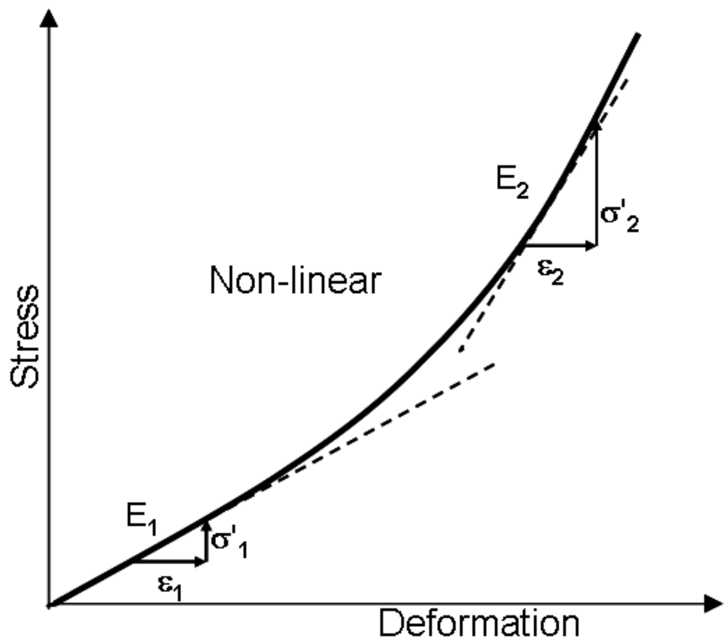


Figure 9. The stress dependency of the resilient modulus (here E_1 and E_2) [Alkio et al. 2001].

This study compares four different kinds of material models. The studied models are: linear elastic, non-linear elastic, linear elasto-plastic and non-linear elasto-plastic. Figure 10 illustrates the principles of the models. The most simple material model is the linear elastic (Hooke's law) where the needed material parameters are only Young's modulus E and Poisson's ratio υ. The non-linear elastic material model is presented in Figure 9. It is important to notice that the stress – deformation curve presented in Figures 9 and 10 present only the reversible part of the deformation, so the non-linear elastic curve has a different shape than the non-linear elasto-plastic curve. The linear elasto-plastic material model includes plastic failure criteria. The most commonly used failure criteria is Mohr-Coulomb's (MC), with the material parameters of friction angle φ and cohesion c. The non-linear elasto-plastic model combines both failure criteria and the non-linearity of the elastic behaviour. An example of the non-linear elasto-plastic material model is the Hardening soil (HS) model of the Plaxis program. Usually linear or non-linear elastic material models are implemented into multilayer calculations tools, while elasto-plastic material models usually need the finite element calculation method.

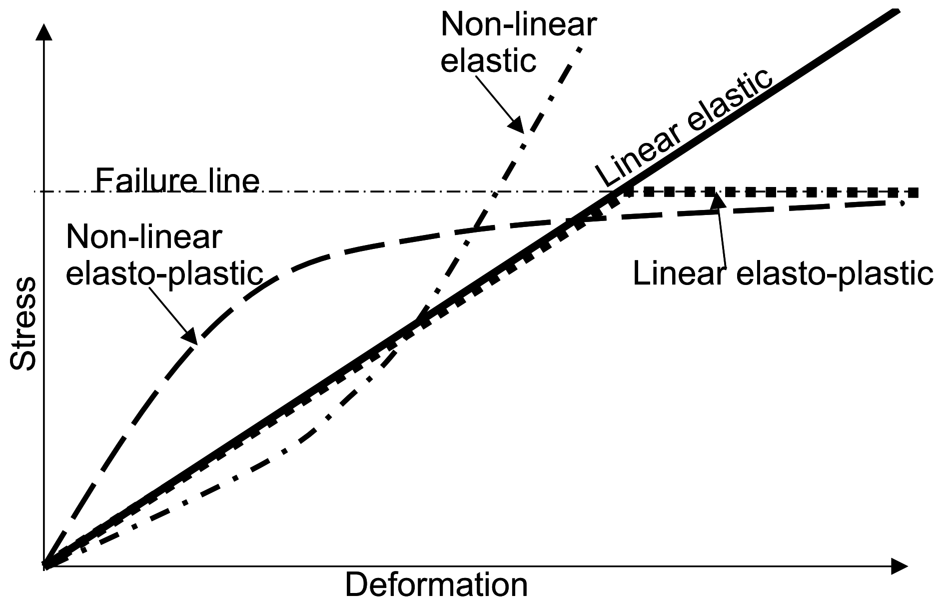


Figure 10. The principles of the material models applied.

1.3 Material factors affecting permanent deformation

The deformations of the granular material are caused by a mixture of many inducing factors. The Road Structure Research Programme (TPPT) studies by Alkio et al. [2001] have specified the following factors: void ratio, effective shear and mean stresses, saturation degree, grain size distribution, the level of the deformations, maximum grain size, stress history, mineralogy of the grains, secondary time factors, the structure of the soil sample and temperature.

Grain size distribution

One of the factors affecting permanent deformation is the grain size distribution of the material. The so called Fuller curve [Fuller and Thompson 1907] presents an optimal grain size distribution curve, where all the grains participate ideally to the distribution of the loading. Thus, the deformation sensitivity for both resilient and permanent deformations is lower than for the materials that do not follow the Fuller curve, as the studies of Kolisoja [1997] have proved. In his earlier studies [1993] Kolisoja demonstrated that in addition to grain size distribution and compaction degree, the water content together with mineralogy had affects on the deformation properties.

Maximum grain size

The maximum grain size clearly affects both resilient and permanent deformation properties – the bigger the maximum grain size, the lower the deformations [Hoff 1999]. Kolisoja [1993] has conducted quantitative studies about the relation between maximum grain size and the amount of deformations. Thus, it is difficult to define the deformation properties of a small size laboratory sample and therefore large scale laboratory tests should be preferred in the case of more coarse materials [Kolisoja 1993]. The better deformation properties are probably due to the deformation concentrations on fewer grain contacts [Kolisoja 1997].

The Swedish Road Administration has tested the deformation properties for the very coarse crushed rock base course materials with test roads and large-scale accelerated pavement tests. The maximum grain size has been varied from 90 mm and even up to 300 mm [Fredrikson and Lekarp 2004]. Figure 11 illustrates the rut depth test results of their study with the passes. As seen in Figure 11, the very coarse base course material deforms less than the finer one.

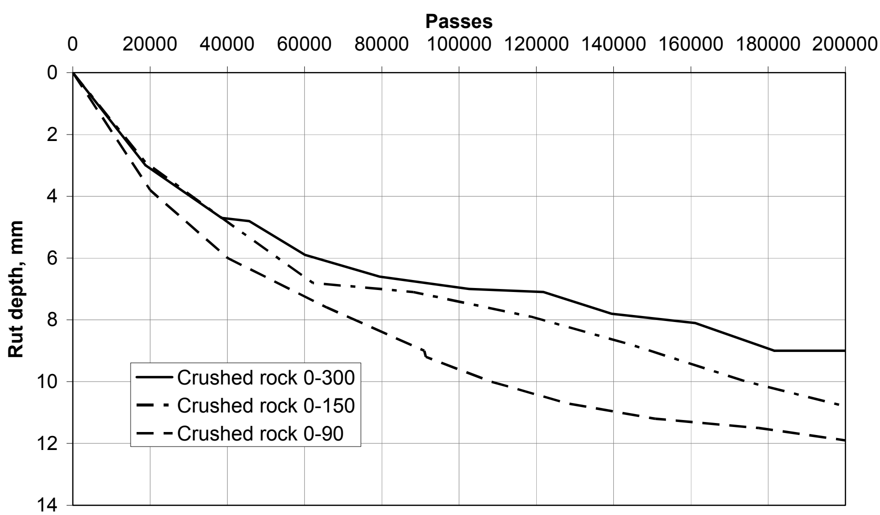


Figure 11. "Provväg E4 Markaryd" HVS test site. The rutting of different test sites with different unbound materials [Fredriksson and Lekarp 2004].

Content of fines

Recent studies of Ekblad [2004] have shown that the resilient modulus is inversely proportional to the content of fines (grain size < 0.074 mm). He proved that the rise of water content decreases more the stiffness of the materials with high contents of fines. If the content of fines is small then the bigger grains can contact each other and distribute the load, while the fines fill the empty voids between grains. As the content of fines increases, the bigger grains do not necessarily contact each other to distribute the load [Kolisoja 1997]. As a result, there is a decrease in the deformation modulus. For example, for the crushed rock of Sievi, the 10% content of fines decreased the resilient modulus with 15% compared to the 4% content of fines. The phenomenon is stronger for natural granular materials than for the crushed rock materials. Besides the content of fines, the mineralogical properties of fine particles affect the deformation properties. A high content of fines also makes materials more sensitive to the rise of the water content [Kolisoja 1993].

The degree of compaction

Besides water content and grain size distribution, one of the most important factors of permanent deformation is the degree of compaction of the material. Lekarp has demonstrated that the degree of compaction has an even stronger effect on the permanent deformations than on the resilient deformations [Lekarp et al. 2000b]. van Niekirk [2002] has addressed the fact that the degree of compaction has a more important effect on the permanent deformation than the grain size distribution.

Uthus [2007] demonstrates that the dry density, the degree of saturation and the stress level (see ch. 1.4) seem to be key parameters for determining the permanent deformation behaviour, but mineralogy, fines content and grain size distribution are also of importance.

The observations of the full-scale test roads of TPPT research have shown almost twice as deep rut depths for the looser pavements than for the dense ones [Alkio et al. 2001]. Figure 12 presents the laboratory test results of Kolisoja [1997], which clearly shows the remarkable effect of the degree of compaction on the resilient modulus.

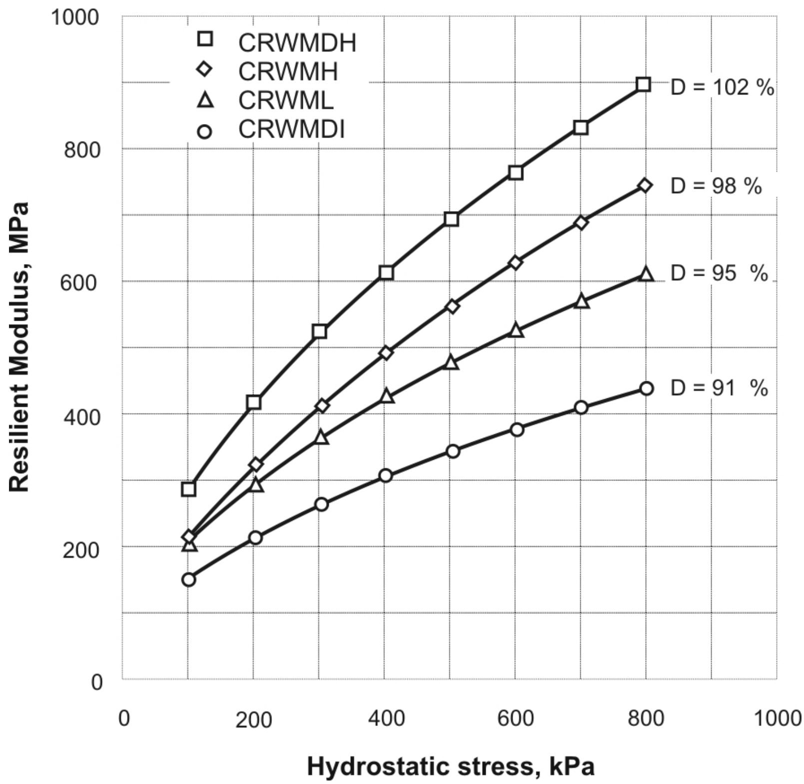


Figure 12. The effect of the degree of compaction (D) on the resilient modulus for the same granular material [Kolisoja 1997].

Odermatt et al. [2004] observed that the effect of the degree of compaction is more important for crushed coarse materials than for natural materials. The HVS-Nordic tests done in VTI (Swedish National Road and Transport Research Institute) [Odermatt et al. 2004] clearly show this, as seen in Figure 13. Both of these test structures have been compacted using the same compaction effort and the only difference was the base course material, which was crushed rock and natural gravel. The maximum grain size for both was 32 mm. The structures were loaded at the same time. Surprisingly, the natural gravel deformed slower than the crushed rock structure (Figure 13). After mineralogical and other additional studies, Odermatt et al. [2004] concluded that the crushed rock material needed more compaction effort.

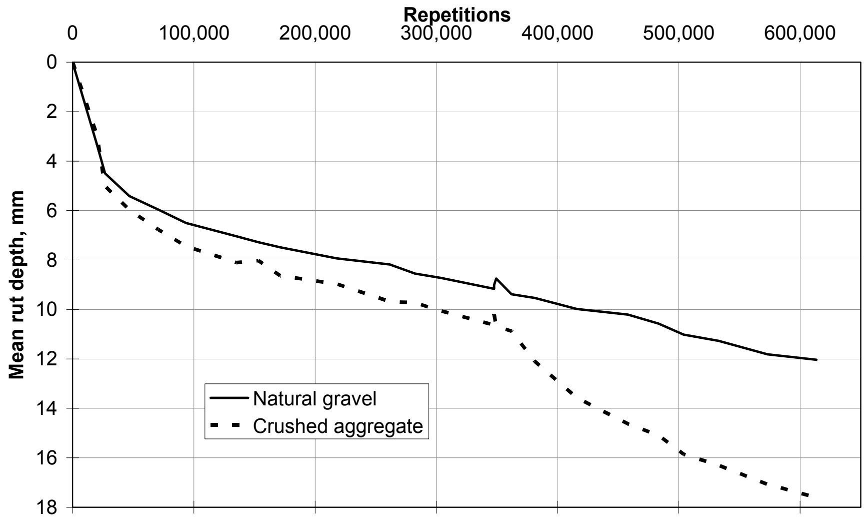


Figure 13. Rutting of the two HVS test structures [Odermatt et al. 2004].

Even though there has been much research on the effect of the degree of compaction, the less known features are the effect of the initial degree of compaction with new structures and the changes caused by seasonal conditions. The main reason for this has been the lack of suitable measuring methods.

The grain shape and the surface roughness of grains

Other affecting factors with less importance are the grain shape and the surface roughness of the grains. These factors mainly affect the compaction properties and thus the permanent deformations [Kolisoja 1993]. The strength of the grains depends on the strength of the mineralogy of the grain. In Finland natural granular materials (sand, gravels) usually have hard mineralogy. The strength of the crushed materials depends on the rock source material and possibly also on the crushing process [Kolisoja 1993]. Again the material strength affects more the permanent deformations than the resilient deformations.

The qualitative estimation of the material factors is presented in Table 1. The estimation is based on the team work of VTT's senior research scientists Rainer Laaksonen, Markku Pienimäki and the author.

Table 1. The qualitative estimation of the effect of different factors on the deformation properties of unbound materials (the positive mark stands for the lower deformations and increase of the material strength).

Factor		**Resilient modulus**	**Permanent deformations**
Grain size distribution	Smooth (Fuller curve)	++	++
	Discontinuous	+	
Maximum grain size	Big (> 90 mm)	++	++
	Normal (30–90 mm)	near to lower limit – no effect/ + (near to upper limit)	+
	Small (< 30 mm)	-	-
Content of fines	Large	-	-
	Small	+	+
Degree of compaction	Dense	+	+++
	Loose	-	---
Shape of the grains	Rounded	+	-
	Flaked	-	-?
Mineralogy	Hard	+	++
	Soft	-	-

1.4 Structural factors affecting permanent deformations

Besides the material factors, there are many other factors affecting permanent deformations. These factors include:

- number of load repetitions
- geometry of the structure (layer thickness, the inclination of the side slope, the distance of the side slope)
- initial state of the pavement layers (e.g. anisotropy)
- temperature and moisture conditions
- loading factors (maximum load, loading rate, loading history, rotation of the principal axis, lateral wander, tyre pressure, wheel type) and
- periodical behaviour (seasonal changes) including the changes in saturation degree.

In the permanent deformation calculations it is very complicated to take into account all these factors. Thus, some effects are included in the material parameters, some are excluded and some of the factors are implemented into the calculation process. The calculation method developed in this research takes into account in one way or another all of the first five points. Also the seasonal changes can be taken into account by dividing the year into shorter design periods.

Kim et al. [2007] studied the sensitivity of the input parameters of MEPDG (Mechanistic-Empirical Pavement Design Guide) software with relatively thick asphalt concrete pavements. They evaluated a total of twenty individual input parameters by studying the effect of each parameter on longitudinal, fatigue cracking, transverse cracking, rutting and roughness. They classified the sensitivity of the parameters into three categories: very sensitive, sensitive and insensitive. According to Kim et al., the only very sensitive parameter was **the annual average daily traffic (AADT),** while subbase layer thickness, the material of subbase and aggregate thermal coefficient were insensitive. The other input parameters (i.e. thickness and materials of each pavement layer, tyre pressure, traffic distribution and speed, weather) were classified as sensitive.

Several studies [i.e. Lekarp et al. 1996, Dawson & Kolisoja 2004, Laaksonen et al. 2004, Hoff 1999] have proved that **deviatoric stress** is the most dominating stress component for the permanent stresses. The deviatoric stress is discussed more in Chapter 2.

Saarenketo et al. [2002] demonstrated that the annual conditions should be divided into different periods instead of averaged annual values. Most of the AASHTO (American Association of State Highway and Transportation Officials) based design programs take into account **seasonal changes** in the unbound material parameters [Richter 2006]. The changes in material parameters have been monitored during different seasons in many different places, like in the Long-Term Pavement Performance (LTPP) programme [Henderson 2006]. The change in the resilient moduli in-situ can be measured with for example, the SASW (spectral analysis of surface waves) method, as Storme et al. [2004] have shown.

All deformations are sensitive to the changes **in water content.** Ekblad [2004] showed that resilient modulus decreases to about half when the **saturation**

degree increases from about 50% to 100%. Earlier HVS studies [Korkiala-Tanttu 2003a and 2003b] proved that the permanent deformations increase to be twice or three times greater when the groundwater table was raised 500 mm to the pavement layers.

The seasonal changes do affect for example the total water content of the layers, the saturation degree, the deformation and strength properties and the freezing-thawing cycles of the materials, and thus maybe the density. Theyse et al. [2007] have presented a yield strength model to evaluate the effect of the saturation degree on the yield strength of unbound granular material. Their approach is based on the shakedown theory deformation calculations, but it probably can be applied to other kinds of calculations as well.

The degree of compaction together with the compaction method, the stress history and the rotation of the principal axis also affect the **anisotropy** of the materials and layers. Anisotropy is caused by the preferred orientation of the aggregate, to which both the shape characteristics of the aggregate and the compaction force itself contribute. The result is that unbound pavement layers have higher stiffness in the vertical direction than in the horizontal direction. Vuong [1998] has concluded that different compaction methods end with different anisotropy and that affects more the horizontal deformations than the vertical. Also Zamhari [1998] concluded that anisotropy can have a significant role in the determination of road pavement behaviour. The assumption in this study is that anisotropy affects more the horizontal deformations than the vertical deformations. However, due to problems in implementing its effect on the calculations, it has been neglected. The results of Masad et al. [2006] showed that the anisotropic behaviour of pavement layers explains part of the shift and the calibration factors are used to relate laboratory measurements to field performance.

1.5 Literature review for the material models for permanent deformations

The first widely applied method to evaluate permanent deformations has been the implementation of the so called Fourth Power Law, introduced by AASHTO in the sixties [AASHTO 1962]. This method was based on the large empirical data of the AASHO road test sites with asphalt layers thicker than 80 mm. Many studies including the HVS Spring-Overload test (I) clearly demonstrated that the Fourth Power Law is not reliable for low-volume roads with thin asphalt layers (e.g. I).

The calculation methods for the resilient response of unbound aggregates have been presented in the state-of-the-art report of Lekarp et al. [2000a]. They have also composed a state-of-the-art report for the permanent strain response of unbound aggregates [2000b]. Werkmeister updated the review [2004]. Another state-of-the-art report of permanent deformation models is presented in NCHRP (synthesis 325) [Hugo and Epps Martin 2004].

The permanent deformation equations usually have two main components: the resulting stresses and the number of loading passes. There are also methods, where permanent deformations depend on the resilient strain instead of the resulting stresses, like the one proposed by Veverka [1979]. The Danish Energy Density Model [Zhang and Macdonald 1999] is based on the assumption that the permanent deformations are formed mainly in the subgrade. This model has been developed further to the MMOPP model (Mathematical Model of Pavement Performance) that includes two phases: primary and secondary creep [Saba et al. 2006].

Lekarp's literature review [1997] and for example Lekarp and Isacsson [1998] have proven that permanent deformations also depend on stress state, rotation of the principal axis, water content, degree of compaction, stress history, grain size distribution and mineralogy. Some of these factors are taken into account by changing material parameters, while some are more or less excluded. The calculation methods are typically a regression analysis or either laboratory or in-situ test results. This means that the calculation methods are more or less empirical models rather than pure physical or analytical models. It is typical for empirical models to be valid only in the context in which they have been created [Rodway and Wardle 1998]. This is especially true for models that describe the entire pavement structure.

As the deformations of granular material are highly stress dependent, the recent development of models has mainly been based on the shakedown concept. The shakedown concept has been fully presented in for example Werkmeister's thesis [2004]. According to the shakedown theory applied to the permanent deformation of unbound granular material, the behaviour can be divided into three categories: below plastic shakedown limit, below plastic creep limit, and below static failure (see Figure 14). Below the plastic shakedown limit the incremental plastic strain per load cycle decreases with the increasing number of load cycles until finally approaching zero. Between the plastic shakedown and the plastic creep limits, the incremental plastic strain approaches a constant value at high load repetitions. Above the plastic creep limit the incremental plastic strain increases with the number of loading cycles, resulting in the incremental collapse of the material and the exponential increase of plastic deformations [Theyse et al. 2007]. The shakedown theory has been developed from the triaxial test results.

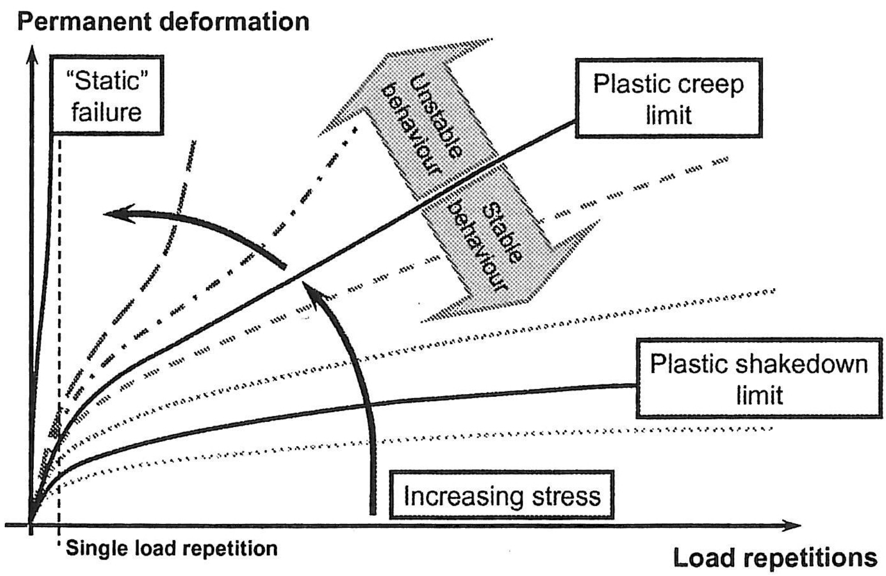


Figure 14. Shakedown Theory applied to the permanent deformation behaviour of unbound granular material [Theyse et al. 2007].

Werkmeister's thesis [2004] included many laboratory test series with repeated loading tests. The test results of the resulting permanent deformation accumulated with repeated loading, which was described and compared with the types of responses usually predicted by the shakedown approach. It was concluded that the method of description could give a powerful material assessment (ranking) and provided a pavement design tool for the analysis of unbound pavement bases.

Most of the existing models suggest that the relation of the permanent deformation and number of passes is an exponential function. Sweere [1990] presented one of the most simple and widely used equations (2). This equation has then been modified to different kinds of equations, like Huurman's [1997], (IV). Also the development of the deformation model reported here is based on a similar approach (V).

$$\varepsilon_p^1 = a \cdot N^b \tag{2}$$

where ε^1_p is permanent axial strain
a, b regression parameters
N number of load cycles.

1.6 Deformation calculation methods and software

Lately there has been much research going on to develop analytical calculation methods and software for the design of pavement structure. Most of the methods have been developed for fatigue design of pavement layers and they are mainly based on Burmeister's elastic multilayer theory [1945]. For example the program Finnish APAS [2004] together with NOAH [NOAH web-site] and VEROAD [Eckmann 1998] belong to this category. Usually the stress state under traffic loading has been calculated with linear or non-linear elastic material models.

The Nordic countries (Sweden, Norway, Denmark and Iceland) are currently cooperating in the development of the pavement design program called VagFEM [Huvstig et al. 2008]. The VagFEM has an existing interface for the finite element code ABAQUS with a linear or nonlinear elastic resilient modulus for unbound materials, and a linear (possibly also viscous) elastic resilient modulus for bituminous bound material. VagFEM is a 3D Finite Element Model that simulates the real geometry of the road. For the prediction of rutting due to permanent deformations in bituminous bound and unbound materials, models developed through the extensive research projects SAMARIS and the U.S. Design Guide, plus the Danish design program MMOPP, have been included as separate Excel programs in VagFEM [Saba et al. 2006]. The VagFEM program is expected to be ready by the end of year 2009.

VEROAD (Visco-Elastic Road Analysis Delft) was developed at Delft University of Technology in the Netherlands and is maintained by Netherlands Pavement Consultants. The VEROAD program is a linear viscoelastic multilayer program that also has the usual linear elastic material models [Eckmann 1998]. VEROAD was planned to be used mainly as a design program for bound pavement layers, so it can not calculate rutting of the subgrade soil.

The NOAH commercial software [NOAH web-site] (developed by Nynas in Belgium) is based on automated computation loops that allow it to investigate the effect of varying design parameters. The software incorporates a database on material properties and typical pavement structures, loading and environmental conditions. NOAH is a powerful multi-layer stress – strain calculation program (based on elastic layer theory) with some specific features such as anisotropy, variable boundary conditions, unlimited number of loadings and layers. Like VEROAD, NOAH is a design program mainly for bound pavement layers' fatigue and it can not calculate permanent deformations.

In many cases the permanent deformation calculations have been based on the empirical 'Fourth Power Law' [Dawson et al. 2004]. The development of finite element methods together with the development of material models have made it possible to also create more sophisticated permanent deformation calculation methods. Most of the permanent deformation calculation methods, so far, have been prototypes and have not been implemented into pavement design programs, like Huurman's proposed method [1997]. In most cases the development of mechanical calculation methods has been conducted in either an accelerated pavement testing facility and/or during a large laboratory test programme.

One of the most widely used pavement design programs, CIRCLY, is the method developed in Australia. It is based on the Austroad Design Guide [1992]. It has some extra tools for high duty pavements (HIpave) and for airfields (APSDS) [CIRCLY web-site]. The last version of the program is from the year 2004. CIRCLY is flexible and transparent. It has a great variety of loading wheels and loads, the properties of layers and layer thickness can be easily changed and it accounts for lateral wander. The program is based on the multilayered linear elastic theory. In CIRCLY the permanent deformations are calculated only in the subgrade. The calculation is based on an empirical equation, called 'subgrade failure criteria' that is related to the vertical strain at the subgrade [Wardle and Rodway 1998].

There are many different pavement design programs in the United States of America. Most of them are based on the AASHTO Pavement Design Guides [1972, 1985, 1993] and are all based on empirical equations developed using the 1950s AASHO road test data. In 2004 AASHTO introduced the Mechanistic – Empirical Pavement Design Guide (MEPDG) and the accompanying software [Masada et al. 2004]. The guide uses mechanistic-empirical numerical models to analyse input data for traffic, climate, materials and the proposed structure and to estimate damage accumulation over the service life. The predicted distresses are longitudinal, transverse and alligator cracking, rutting and roughness [Kim et al. 2007].

The freeware program MnPAVE combines known empirical relationships with a representation of the physics and mechanics behind flexible pavement behaviour [MnPAVE web-site]. MnPAVE utilises “WESLEA” to perform a linear elastic analysis of a multilayer pavement structure. WESLEA is the U.S. Army Corps of Engineers Waterways Experiment Station, Layered Elastic Analysis method [van Cauwelaert et al. 1989]. Thus WESLEA is the analytic engine that calculates the stresses, strains, and displacements in MnPAVE. Unfortunately it is not possible to calculate rutting with MnPAVE.

IlliPave has been developed from a finite element program made by E. L. Wilson in 1965 [reported by Thompson 1984]. The adoption for use in pavement analysis has mainly been done by the University of Illinois, hence the name IlliPave. The program is a non-linear finite element program based on the design Procedure AASHTO Guide 1985 with stress dependent models to describe the resilient modulus. Failure criterion based on the Mohr-Coulomb theory is used for granular materials and fine-grained subgrade soils to modify calculated stresses so that they do not exceed the strength of the material [Aksnes 2002]. Tseng and Lytton [1989] have developed a rutting model into a modified IlliPave that is based on the resilient strains.

ARKPave is an academic finite element program tool, which can calculate rutting in unbound materials [Qiu et al. 1999]. The material model used for rutting is based on how the subgrade soils contribute to surface rutting and fatigue cracking, the two major distress modes in flexible pavements.

MichPave is another finite element program developed for pavement analysis at Michigan State University in 1989 [Harichandran et al. 1989]. As in IlliPave, stress dependent material models can be used, and the correction of stresses follows the same principles based on the Mohr-Coulomb theory. The main difference between IlliPave and MichPave is that in MichPave the material below a lower boundary is considered to be linear elastic and with infinite extension both in the vertical and horizontal directions [Aksnes 2002]. Another academic finite element calculation tool for flexible pavement rutting has been developed by Fang et al. [2004] that has been based on creep models.

The CalMe program is a draft program. Its development has been funded by the California Department of Transportation (Caltrans). CalMe also includes the possibility to calculate rutting of unbound layers as a function of the vertical resilient strain and modulus of the material. The design models of CalMe have been calibrated to date with Heavy Vehicle Simulator data and WesTrack data. Additional calibration is currently underway using field data from California, and materials have been obtained from MnROAD and the NCAT test track to be tested and used for calibration of the models [Ullidtz et al. 2006].

PerRoad is a flexible mechanistic based perpetual pavement design and analysis software program [PerRoad web-site]. PerRoad is a commercial, relatively cheap software that has been developed for the Asphalt Pavement Alliance and is again based on the AASTHO Design Guide. It includes seasonal changes, and also the number of load cycles for failure caused by rutting can be calculated. The number of passes depends on the vertical strain on the upper part of the subgrade [Newcomb 2004].

The VESYS model and finite element design software is a well developed probabilistic and mechanistic flexible pavement analysis computer program series and it is connected to AutoCAD [VESYS web-site]. The commercial VESYS series are based on the elastic model of layered homogeneous material in half infinite space with some viscoelastic-plastic theory applications. This system uses a mechanistic multilayer elastic probabilistic primary response model that can be used to analyse the primary response; it can also calculate distress of pavements, in terms of stress, strain, deflection, rutting, roughness and cracking damage. The pavement models used in VESYS have been calibrated by using the Full-Scale Pavement Test data, such as AASHO road test data, the Federal Highway Administration's Pavement Testing Facility test data, OECD DIVINE project test data, Texas Mobile Load Simulator test data, Louisiana DOT's APT test data and Ohio SHRP test pavement data.

So far there has not been a separate, available finite element program for pavement calculations developed in Europe. Yet, some prototype applications have been integrated into the finite element program CESAR-LCPC [Elabd et al. 2004]. The CESAR method is a simplified one for the modelling of permanent deformations in order to estimate the rutting of low traffic flexible pavements. The method is based on a recently developed elastoplastic model of predicting permanent deformations and consists of integrating the permanent strains along the vertical direction in order to obtain the vertical displacements in the structure (i.e. rutting of the layer). The CESAR program is used to determine the stress state in layers. For unbound layers a nonlinear elastic model is applied, and for asphalt layers a viscoelastic model.

Werkmeister [2004] has also integrated the non-linear elastic DRESDEN material model, which is based on the shakedown theory, to the finite element program FENLAP to calculate permanent deformations in unbound layers.

As described in this thesis, the objective of the 'Deformation' project in 2003–2004 was to develop a technology for controlling, measuring and designing against rutting for Finnish conditions, which would cover the most important factors that affect rutting, for example: periods, lateral wander and a more sophisticated stress calculation method. In the project a rutting calculation tool called "Pavedef" was developed. In the prototype tool, the year can be divided into seasons with different lengths and different material parameters for each pavement layer, the stress distribution is then calculated for each period and they are summed to get the total rut depth [Laaksonen et al. 2004]. The developed permanent calculation method was planned to be part of the Pavedef tool, but due to the strict timetable of the 'Deformation' project, it could not be implemented. Therefore a more simple rutting calculation model and the stress calculation with Burmeister's multilayer theory were implemented into it.

In summary of the available permanent deformation calculation tools or software (Tables 2a and 2b), it can be said that there are some commercial or freeware software that can evaluate the rutting process with simple methods. More sophisticated, finite element based methods have been implemented into academic applications, but usually these applications are not publicly available.

Table 2a. The features and limitations of analytical (multilayer) flexible pavement design analysis methods and software programs.

Type of analysis/software	**Features**	**Limitations**
BISAR (developed by Shell)	– linear elastic multilayer program for pavement design – widely used	– stress calculation based on elastic multilayer theory – excludes rut calculation
KENLAYER	– non-linear elastic and viscoelastic multilayer program for pavement design – widely used	– stress calculation based on elastic multilayer theory – rut calculation of subgrade
APAS + Pavedef (developed in Finland)	– non-linear elastic multilayer program for the design of pavement layers – Pavedef includes seasonal changes and lateral wander	– stress calculation based on elastic multilayer theory – Pavedef is a non-commercial prototype
VEROAD (developed in Delft)	– commercial linear elastic and viscoelastic multilayer program for the design of bound pavement layers	– stress calculation based on elastic multilayer theory – excludes rut calculation
NOAH (developed in Belgium)	– commercial linear elastic and viscoelastic multilayer program – anisotropy	– stress calculation based on elastic multilayer theory – excludes rut calculation
HUURMAN (developed in the Netherlands)	– finite element with non-linear plastic material model – lateral wander	– non commercial prototype – developed mainly for concrete block pavement

Table 2a. Continued. The features and limitations of analytical (multilayer) flexible pavement design analysis methods and software programs.

Type of analysis/software	**Features**	**Limitations**
CIRCLY (developed in Australia)	– linear elastic multilayer calculation tool for the design of pavement layers – lateral wander – heavy duty pavements and airfields	– stress calculation based on linear elastic multilayer theory – permanent deformations only for subgrade by an empirical equation
MEPDG software + JULEA for flexible pavement analysis (developed by AASHTO)	– analysis of transversal, longitudinal, alligator cracking, rutting and roughness – lateral wander – seasonal changes	– stress calculation based on linear elastic multilayer theory – for the analysis of pavement not for design
CalMe developed in California Department of Transportation (Caltrans)*	– non-linear elastic multilayer theory by MnLayer tool – probabilistic analysis – rutting of unbound layers – - design models calibrated with APT results	– draft program – stress calculation based on elastic multilayer theory
PerRoad (developed to the Asphalt Pavement Alliance)*	– commercial program – linear elastic multilayer theory – seasonal changes – rutting criteria (vertical strain on the subgrade)	– stress calculation based on elastic multilayer theory – no rutting of unbound pavement layers
VESYS (manufacturer Mentor Graphics, works with AUTOCAD)*	– commercial plastic and viscoelastic multilayer design – probabilistic analysis – rutting of unbound layers – parameters calibrated with APT results	– stress calculation based on elastic multilayer theory

* based on the AASTHO Design Guide

Table 2b. The features and limitations of finite element flexible pavement design analysis methods and software programs.

Type of analysis/software	**Features**	**Limitations**
HUURMAN (developed in the Netherlands)	– finite element with non-linear plastic material model – lateral wander	– non commercial prototype – developed mainly for concrete block pavement
VÄGFEM (developed in the Nordic countries, development in progress)	– finite element with linear or non-linear plastic material model (ABAQUS)	– 3D takes into account the real geometry of the road – permanent deformations calculated in separate Excel tool
IlliPave (developed in the University of Illinois, based on AASHO Pavement Design Guides)*	– non-linear finite element program – failure criterion Mohr-Coulomb	– non-commercial tool – basic version excludes rut calculation
ARKPave (developed in the University of Arkansas)*	– non-linear elastic finite element program – rutting of the unbound materials	– non-commercial tool
MichPave (developed by Michigan Department of Transportation)*	– commercial program – non-linear elastic finite element program – Mohr-Coulomb theory – rut calculation	– stress calculation based on elastic theory
CESAR-LCPC	– CESAR is a commercial non-linear elastic finite element program – elastoplastic rut calculation	– CESAR-LCPC is a non-commercial prototype – stress calculation based on elastic theory
DRESDEN + FENLAP (developed in Germany)	– non-linear elastic DRESDEN material model based on shakedown theory – pavement responses finite element program FENLAP	– non-commercial prototype – stress calculation based on elastic theory

* based on the AASTHO Design Guide

2. Permanent deformation calculation method

A new material model developed for unbound materials by the author is an analytical, relatively simple, nonlinear elasto-plastic model. The deformation model's equation is founded on the theory of static loading, which is extended to dynamic loading cases. The equation is relatively simple and it binds permanent deformations to the most important factors. The material deformation model can take into account the number of passes, the capacity of the material and its stress state. The deformations in each layer are calculated separately and then added together to obtain the total rutting on the surface of the structure.

Equation 2 shows that permanent deformations will accumulate according to a simple power function. All HVS tests together with triaxial tests have shown that Equation 2 is valid for pavement materials and for the total rutting depths in the surface of the pavement (V). Yet, some triaxial tests have shown that even at lower deviatoric stress states, permanent deformations begin to accumulate after numerous loading cycles, i.e. as reported in Kolisoja 1997 and Werkmeister 2004. This kind of incremental collapse can not be described by means of Equation 2. After all, most of the permanent deformation models are based on this kind of power function. Equation 2 is also applied within the deformation equation presented by the author later in this work.

Besides the number of loadings, permanent deformations depend on the shear strains of the material. In the developed equation the yielding and shear strains are described through the failure ratio R. This means that the deformations are larger when the failure ratio is close to failure. The failure ratio R in this case is defined as the ratio between deviatoric stress and deviatoric stress at failure (q/q_f). Many studies [i.e. Lekarp et al. 1996, Dawson & Kolisoja 2004, Laaksonen et al. 2004, Hoff 1999] have proven that deviatoric stress is the most dominating stress component for the permanent stresses. Besides this, it is relatively easy to calculate from the normally used stress calculations (V). An analogical approach has been presented by Brown and Selig [1991]. Figure 15 shows how the vertical strains in triaxial tests depend on the failure ratio R. It is notable that the vertical strains for different materials do not depend so much on the material, but on the failure ratio. The degree of compaction for the materials in Figure 15 varied from 95% to 100% and the water content from 4% to 8%.

Many different function types were attempted, but the hyperbolic function proved to work the best. Hence, the hypothesis was that the hyperbolic constitutive equation of Kondner and Zelasko [1963] could be used to describe also the dependency of stresses and deformations of unbound granular materials in cyclic tests (Equation 3). The shear strain γ can be approximated to be the permanent deformation ε_p. If the shear ratio τ/σ can be approximated to be the shear stress ratio in failure R (q/q_f), then the permanent deformation can be described by Equation 4. Parameter A is the maximum possible ratio for R, which theoretically is one. Equations 5–7 express the definition of the deviatoric stress in failure in the case where the first principal stress concurs with the vertical axis (accordingly right under the centre of the loading).

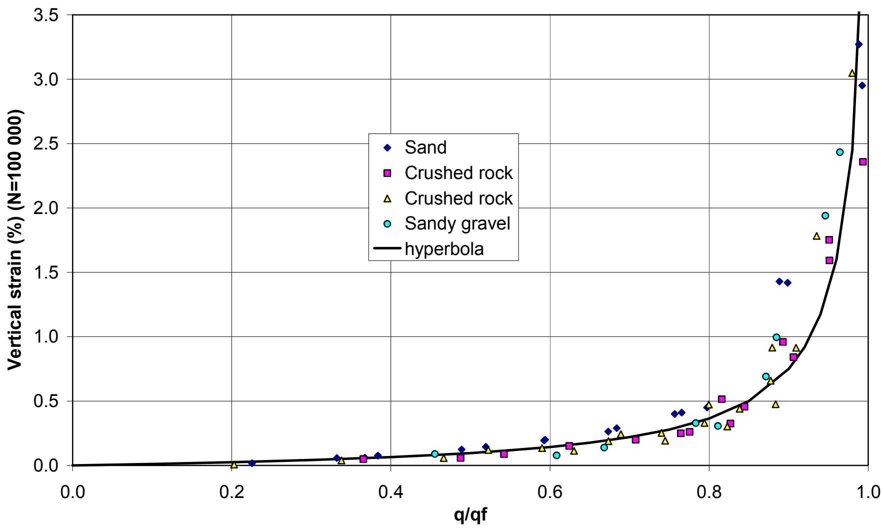


Figure 15. The failure ratios versus vertical permanent strains in VTT's cyclic triaxial tests (Publication V; stress responses calculated with MC model).

$$\frac{\tau}{\sigma} = \frac{A\gamma}{B+\gamma} \tag{3}$$

if τ/σ is approxirated to be $\frac{q}{q_f} = R$

and γ is approximated to be ε_p =>

$$\varepsilon_p = B \cdot \left(\frac{R}{A - R} \right) \tag{4}$$

where	τ	is shear stress, kPa
	σ	normal stress, kPa
	B	material parameter, -
	γ	shear strain, -
	ε_p	permanent strain, -
	p	hydrostatic stress, kPa
	q	deviatoric stress, kPa
	q_0	deviatoric stress, when p' =0 (equation 7)
	q_f	deviatoric stress in failure, kPa
	R	failure ratio (q/q_f)
	A	maximum value of the failure ratio R (theoretically 1)
	φ	friction angle, °
	c	cohesion, kPa.

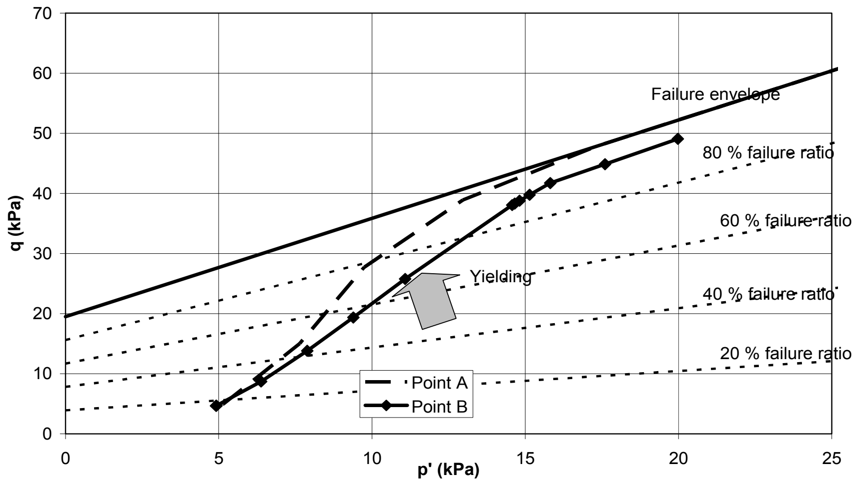
$$q_f = q_0 + M \cdot p \tag{5}$$

$$M = \frac{6 \cdot \sin\phi}{3 - \sin\phi} \tag{6}$$

$$q_0 = \frac{c \cdot 6 \cdot \cos\phi}{3 - \sin\phi} \tag{7}$$

Equation 4 expresses the acceleration of the growth of permanent strains when the failure envelope is approached. Figure 16 illustrates the situation in p'-q space, where p' is effective hydrostatic pressure. To define the deviatoric stress in failure, the strength properties of the materials should be measured with, e.g. triaxial tests.

The permanent deformation in the first loading cycle ("a" in Equation 2) can be described with the aid of shear strain ratio (Equation 4). When the permanent shear strain component (Equation 9) is combined together with the cyclic loading function, a new permanent deformation equation, which calculates the vertical permanent strain, is introduced as Equation 8 (V and VII).



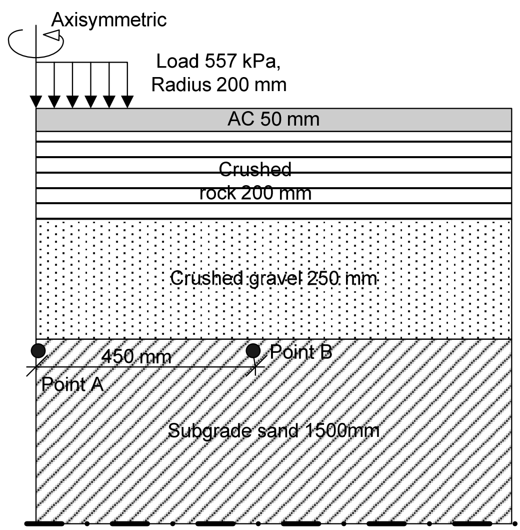


Figure 16. The contours of failure ratios for sand (cohesion 10 kPa and friction angle 40°) and two stress paths in p'-q space when the wheel load increases from 0 kPa to 557 kPa (V).

$$\varepsilon_p = C \cdot N^b \cdot \frac{R}{1-R} \tag{8}$$

$$a = C \cdot \frac{R}{1-R} \tag{9}$$

where ε_p is permanent vertical strain
R deviatoric stress ratio (q/q_f)
b shear ratio parameter depending on the material and
C material parameter depending on the compaction and saturation degree.

The calculation process integrates the stress calculations with finite element calculations together with analytical permanent deformation calculations. The determination of the parameters b and C is presented in Chapter 3. The material parameter C describes the amount of permanent deformation in different materials and it depends on various factors. The most important factors are the material, its degree of compaction (DOC) and the saturation degree. The value of the parameter C is stress dependent and thus the values for parameter C can not be directly compared with each other. Material parameter b depends on the strength properties of the material and the state of the material (DOC and saturation degree) and emphasises the effect of shearing to the permanent deformations.

Parameter A from Equation 4 has been substituted with the value of 1 in Equation 8. This theoretical value of 1 can be used if the pavement response calculations (stress state) are done with a non-linear elasto-plastic material model (like the hardening soil model). But if a linear elasto-plastic material model (like Mohr-Coulomb) is applied, the expression (R/(1 - R)) can increase to indefinite values as R approaches one. Thus the denominator in Equation 8 should be in the form (A - R), where A is about 1.05. Parameter A has also been called X in Publication VII.

To get the total deformations, the vertical deformations in each layer are calculated with Equation 8 and then changed to vertical displacement per layer by multiplying it with the thickness. The vertical compressions are then added together to obtain the total rutting in the surface of the structure.

3. Test structures and material parameters

The HVS test structures called Spring-Overload (SO) and Low-Volume (LV) have been presented in Publications I, II, III and IV. The more detailed descriptions are found in test reports Korkiala-Tanttu et al. [2003a] and [2003b].

The material parameters for the permanent deformation calculations were defined with cyclic triaxial and monotonic strength tests. The laboratory tests were part of the HVS projects and some of them were involved in the 'Deformation'-project [Laaksonen et al. 2004]. For the validation of the material equation and parameters, other triaxial test results have also been used, like the repeated loading tests (RLT) developed by Werkmeister [2004]. Unfortunately there were so few HVS test results, that the test results could only be used as validation data instead of using them in the development. The HVS results also showed that full-scale tests are needed to get a better understanding of the scaling (IV and V).

Figure 17 illustrates the parts of the modelled structures and the used material parameters for each layer for the hardening soil model. The corresponding parameters for the Mohr-Coulomb material model for unbound layers are simpler: Young's modulus E, Poisson's ratio υ, friction angle φ, dilatation angle ψ and cohesion c. The linear elastic material model is applied for the bound surface layer (asphalt concrete). Yet, for each loading case the corresponding material parameters are defined depending on the prevailing temperature and loading rate. This is a simplified method to take the viscoelastic character of the bound layers into account. A similar approach has been used also in some other calculation methods, for example in APAS [APAS 2004] and Pavedef [Laaksonen et al. 2004].

Because the developed permanent deformation equation has been directed towards the unbound layers, the permanent deformations of bound layers have been neglected. The permanent deformations of asphalt for APT tests in relatively cold conditions (+10 °C) and with thin bound layers have been small compared to the deformations in unbound layers, which justifies this omission.

The tested materials were typical Finnish unbound pavement materials and subgrade soils: crushed rock, crushed gravel, gravel, fine sand and lean clay. The material parameters from the different publications (II, III, IV, V and VII) have been collected in Tables 3–4. Parameters c and d have been called in Publication IV α and β, respectively. Table 3 presents the results of modified Proctor tests (SFS-EN 13286-2) for the defined materials. Table 4 presents the defined strength parameters of the material and the water contents and density of the samples. The strength parameters were defined with the monotonic strength test.

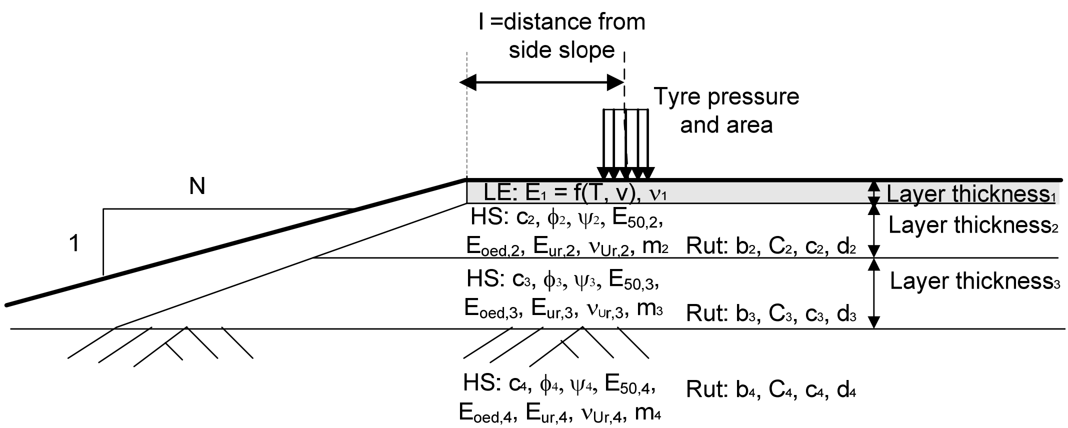


Figure 17. Symbols for material parameters.

Table 3. The results of modified Proctor tests (SFS-EN 13286-2).

Material (Finnish classification system)	**Material (USCS classification)**	**Maximum dry unit weight/maximum dry density (Mg/m^3/kN/m^3)**	**Optimum water content (%)**
Fine Sand	SC = Clayey or Silty sand	1.91/18.7	10.4
Crushed gravel	Crushed gravel	2.35/23.1	5.6
Crushed rock	Crushed rock	2.14/21.0	5.0
Sandy gravel	SW = well graded, coarse sand	2.17/21.26	6.5
Lean clay (in situ values)	CL = medium plastic inorganic clay	14.4 kN/m^3	32.0

Table 4. Strength parameters for different materials defined in the monotonic strength test.

Material	Dry unit weight /Dry density (Mg/m^3)/ (kN/m^3)	Water content (%)	Degree of compaction (% of modified Proctor)	Angle of friction (°)	Cohesion (kPa)
Sand	1.81/17.8	8.4	94.5	35.5	12.9
Crushed gravel	2.22/21.8	2.9	94.5	44.7	35.6
Crushed rock	2.14/21.0	2.9	99.6	43.1	43.0
Sandy gravel	2.0/19.6	5.8	92.2	41.2	8.7
Clay (CAUC)	18.2	32.4	-	29.6	12.9

The material parameters used in the calculations for stress distribution were modified from the laboratory and HVS defined parameters. In the first stage the Mohr-Coulomb material model was tested to calculate stresses and deformations of the Spring-Overload and Low-volume test structures. These material parameters are presented in Tables 5 and 6 (II). The resilient moduli of Tables 5 and 6 are determined both from back-calculations and laboratory tests. The dilatation angle has been defined from the friction angle according to the guidelines of the Plaxis manual [Brinkgreve 2002].

Table 5. Spring Overload: Input parameters for the Mohr-Coulomb material model.

Material	Asphalt (linear elastic)	Base course crushed rock	Subbase Crushed gravel	Subgrade Sand
Thickness, mm	50	200	250	1500
Young's modulus, E MPa	5400	300–220–190	140–90	75
Poisson's ratio, ν	0.3	0.35	0.35	0.35
Unit weight, kN/m^3	24	21.2	22.0	18.0
Cohesion, c kPa	-	30	20	8
Friction angle, φ °	-	43	45	36
Dilatation angle, ψ °	-	13	15	6
K_0	1	0.32	0.30	0.42

- not determined

Table 6. Low-volume: Input Parameters for Mohr-Coulomb material model.

Material	**Asphalt (linear elastic)**	**Base crushed rock 1**	**Base crushed rock 2**	**Subbase Gravel**	**Subgrade Clay**	**Bottom Sand**
Thickness, mm	50	200	200	200	1350	500
Young's modulus, E MPa	5 400	320–250	150–110	70	10–8	75
Poisson's ratio, ν	0.3	0.35	0.35	0.35	0.35	0.35
Unit weight, kN/m^3	25	21.2	20.5	20	18	18
Cohesion, c kPa	-	25	15	9	10	12
Friction angle, φ°	-	40	40	36	25	36
Dilatation angle, ψ °	-	10	10	6	0	6
K_0	1	0.32	0.32	0.4	0.8	0.42

- not determined

Further modelling was done with the Plaxis program's hardening soil model (HS). This approach has been applied in the calculations of the Spring-Overload test in Publication VII. These parameters are presented in Table 7. Table 7 also includes the hardening soil parameters for the Low-volume tests to give a whole parameter description.

The material parameters presented in Tables 5 to 7 have been defined to the applied stress states and the degree of compaction. It is also important to notice that some of the applied strength and deformation parameters differ remarkably from the laboratory defined parameters. For example, the strength parameters of the hardening soil model have been increased to better model the unsaturated behaviour of unbound materials. This is described in more detail in Publication VII. The deformation moduli for hardening soil model were determined from the triaxial tests with back-calculations of Plaxis. The back-calculated E_{ur} moduli have relatively high values compared to the E_{50} – about three times larger. In the final calculations the parameters were adjusted to better represent the circumstances in the real structure (water content and degree of compaction). In this fitting, it was attempted to retain the ratios between parameters.

Table 7. Strength and deformation parameters of the hardening soil model for SO and LV test structures (VII) when reference stress is 100 kPa and m is 0.5.

Material	**Material model**	**DOC (%)/ w (%)**	**Friction φ ° /dilatation ψ angle °**	**Cohesion, c kN/m²**	**Unloading/ reloading modulus, E_{ur} MPa**	**Compression modulus, E_{oed} MPa**	**Deviatoric modulus, E_{50} MPa**
Asphalt concrete	LE*	-	-	-	-	-	5400*
SO Crushed rock	HS†	95.8/4.6	55/25	20	750	173	250
LV Crushed rock	HS†	89/4.1	55/25	20	750	190	230
SO Sandy gravel	HS†	98.1/7.3	58/38	20	900	210	330
LV Gravel	HS†	92/8.0	48/18	15	210	71	70
SO Sand (dry)	HS†	101.4/9.9	40/10	15	420	110	120
LV Clay (wet)	HS†	-	25/0	12	30	12.5	10
SO & LV Sand (moist)	HS†	-	36/6	8	420	95	100

*linear elastic Young's modulus, Poisson's ratio is 0.3
†hardening soil

Table 8 illustrates the shear ratio parameter b determined from laboratory tests and Emu-coil measurements (IV) and Tables 9 and 10 describe the defined material parameters of the permanent deformation calculation method. Shear ratio parameter b has in the later publications been determined with the help of parameters c and d. Equation 10 (V) presents this simple linear connection. The values of material parameters c and d are mainly based on the triaxial test results. The triaxial tests have been repeated loading tests following VTT's in-house test protocol for constant confining pressure (CCP) test condition (VII).

$$b = d \cdot \left(\frac{q}{q_f} \right) + c \tag{10}$$

where b is shear ratio parameter depending on the material
q deviatoric stress, kPa

q_f	deviatoric stress in failure, kPa (defined in V)
c	material parameter (constant for shear ratio line)
d	material parameter (slope for shear ratio line).

The material parameter C for the permanent deformation method has been defined from the triaxial laboratory tests (Figure 15). The loading frequency in the triaxial tests was 5 Hz, which compares to the loading rate of 11 km/h. Its values have also been fitted to match stress responses calculated with the MC model. The problem with the parameter definition was that the amount of full-scale tests was only two. Because in the axisymmetric 2D case the HS stress responses and especially the shear strength ratio R are smaller than the MC's, parameter C needs redefining. Otherwise the method will give far too low of deformation values. Table 9 presents the material parameters used in the permanent deformation method for the MC model and Table 10 for the HS model. The material parameter C has been estimated to have about 2 to 4 times larger values for the HS model than for the MC model. The value of parameter C has been defined from the Finnish accelerated pavement tests [Korkiala-Tanttu et al. 2003a & 2003b]. The definition of parameter b for sand is presented in Figure 18.

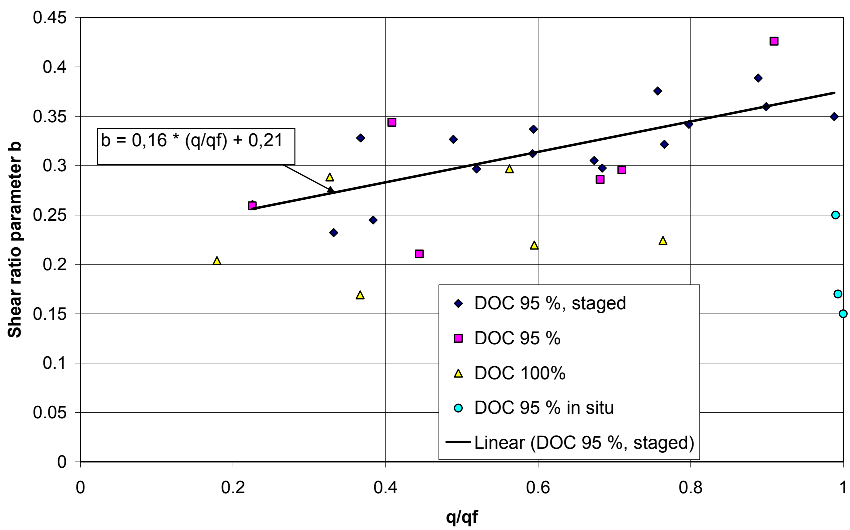


Figure 18. Definition of shear ratio parameter b for sand (IV).

Table 8. Shear ratio parameter b determined from laboratory tests and Emu-coil measurements (IV).

Stress state parameter *b*	**Crushed rock, σ_3 = 60 kPa**	**Crushed rock, σ_3 = 25 kPa**	**Crushed rock, σ_3 = 16 kPa**	**Crushed gravel**	**Sandy gravel**	**Sand**
In laboratory	0.20	0.18	0.18	0.28	0.13–0.48	0.23–0.39
In situ	0.28–0.4	0.3–0.38	0.4	0.34–0.38	0.25–0.45	0.11–0.38

Table 9. The parameters for permanent strain calculations for the Mohr-Coulomb (MC) model (VII).

Material	**Parameter d**	**Parameter c**	**C (%)**	**DOC (%)**	**w (%)**
HVS: Sand	0.16	0.21	0.0038 (±0.001)	95	8
HVS: Sandy gravel	0.18	0.15	0.0049 (±0.003)	97	5…7
HVS: Sandy gravel	0.18	0.15	0.0021 (±0.001)	100	5…7
HVS: Crushed rock	0.18	0.05	0.012 (±0.004)	97	4…5

Table 10. The parameters for permanent strain calculations for the Hardening Soil (HS) model (VII).

Material	**Parameter d**	**Parameter c**	**C (%)**	**DOC (%)**	**w (%)**
HVS: Sand	0.16	0.21	0.016 (±0.004)	95	8
HVS: Sand	0.16	0.21	0.035 (±0.01)	95	saturated
HVS: Sandy gravel	0.18	0.15	0.02 (±0.01)	97	5…7
HVS: Sandy gravel	0.18	0.15	0.008 (±0.003)	100	5…7
HVS: Crushed rock	0.18	0.05	0.048 (±0.016)	97	4…5

4. Modelling of stress states

The stress distribution studies of traffic load have shown that it was very important to calculate stresses in pavements with an elasto-plastic material model to avoid tensile stresses in unbound materials (II). The chosen material model drastically affects stress distribution, and also to some extent resilient deformations. It seems obvious that the sensitivity of the materials for permanent deformations can been evaluated from the modelled stresses. Therefore in the calculation of permanent deformations, it is important to model stress distribution with a ‘better’ model than a conventional linear elastic material model. By using a linear elastic material model there is a high risk that there will be tensile stresses in the unbound pavement layers. The risk is emphasised in pavement structures that are thinly paved or totally unpaved. These tensions will cause unrealistic stress concentrations with misleading information about permanent deformation sensitivity.

The objective of the stress analysis in Publication VII was to compare the stress responses of unbound pavement materials and the subgrade analysed with 2D and 3D models to give more confidence to the developed calculation method. The stress analysis included three different calculation cases: the most common 2D axisymmetric, 2D plane strain (long continuous line loading) and true 3D cases. All the calculations were conducted with the Plaxis code; 2D cases with Plaxis version 8.6 and 3D cases with Plaxis 3D version 2 [Brinkgreve and Broere 2006]. The HVS test set-up for the Spring-Overload test (SO) was chosen as a test structure (Figure 19). The wheel load is a dual wheel type.

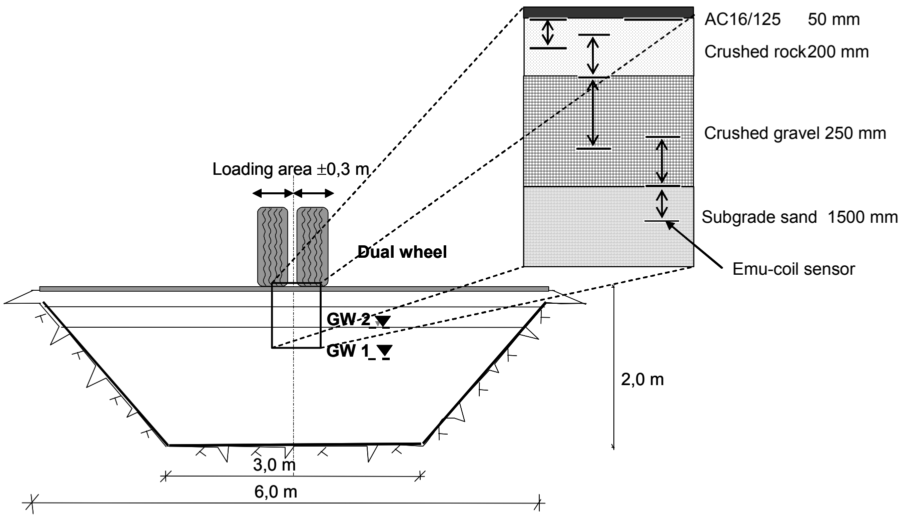


Figure 19. Cross section of the Spring-Overload test.

The applied model for unbound materials in Publications II and VII was Mohr-Coulomb (MC), linear elastic and linear elastic with a tension-cut-off property. In Publications V and VII the hardening soil model (HS), which is a non-linear elasto-plastic material model with Mohr-Coulomb failure criterion, was also applied. The HS model also includes the tension-cut-off property. This property means that in the stress state calculations no tension stresses are allowed. A more detailed description of the material model is presented in Plaxis's manual [Brinkgreve 2002].

The results of the stress analysis are presented in Figures 20 to 22. Figure 20 presents the calculated deviatoric stress responses with 2D and 3D models. The stress components have been calculated under the centre line of loading and the wheel load has been 50 kN. Figure 21 illustrates the vertical stress responses with 2D and 3D. Due to the stress sign rules of Plaxis compression, the compressive stresses have negative values. The measured earth pressure at the top of the subgrade sand is presented in Figure 21.

The stress comparisons (Figure 20) clearly show that the deviatoric stresses with the HS material model give smaller deviatoric stresses in both 2D and 3D cases than the MC model. The calculated deviatoric stresses in 3D for the MC and HS models were close to each other. The average relative difference between 3D

MC and HS calculated stresses was 11% and it varied between -1 to 21%. For 2D the difference was smaller: on average MC defined stresses were about 4% greater than HS defined stresses.

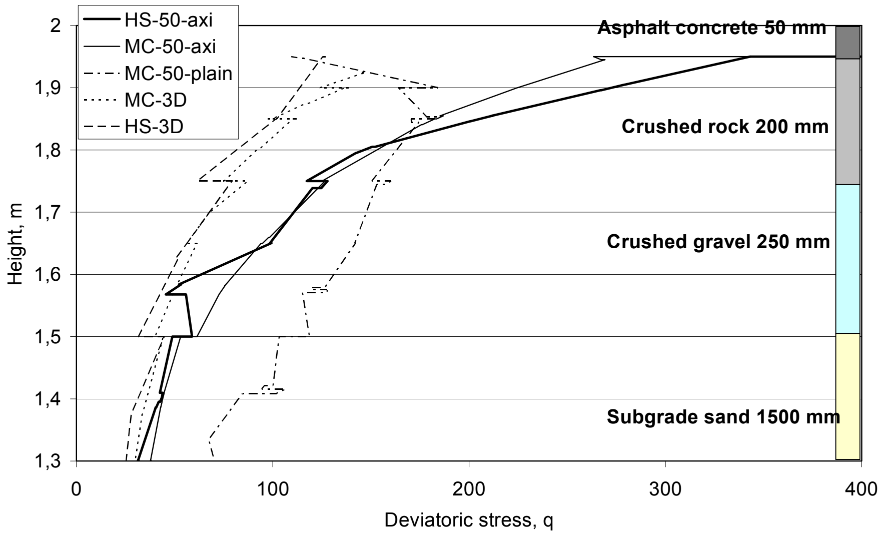


Figure 20. Comparison of the deviatoric stresses in the centre line of loading (HS = Hardening soil, MC = Mohr-Coulomb) Spring-Overload test (VII).

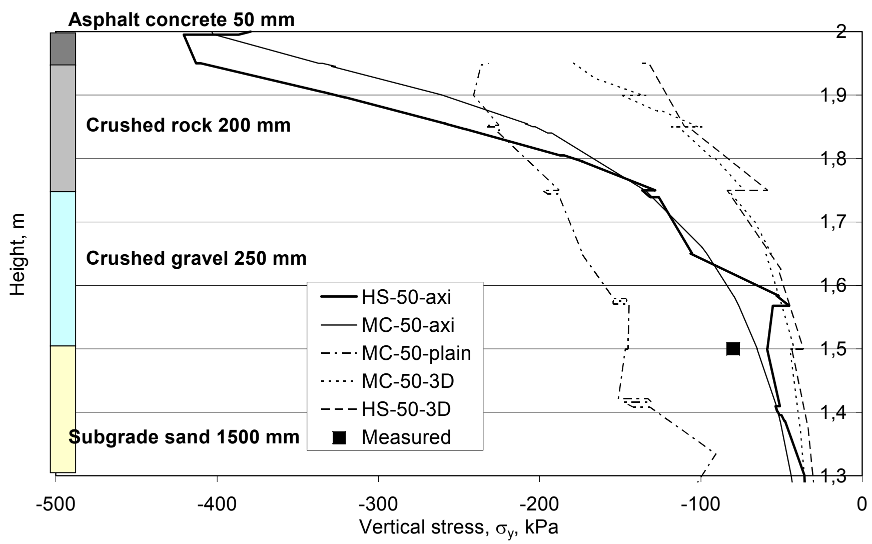


Figure 21. Comparison of the vertical stresses in the centre line of loading Spring-Overload test (VII).

At the height of about 1.5 m the deviatoric stresses for 3D and 2D axisymmetric cases approached each other and the differences were less than 10 kPa. The differences between plain strain and axisymmetric modelling were the largest in the lower part of the structure.

The vertical stress component (Figure 21) shows the same phenomenon as the deviatoric stress comparison: stresses calculated with the MC and HS models are relatively close to each other. The 2D and 3D stresses separate from each other in the upper part of the pavement (to the height of about 1.5 m). Again the vertical stresses in the plain strain case decrease very slowly downwards.

Figures 22a and 22b presents the calculated failure ratios for the SO and LV tests. The failure ratio equation is only valid under the centre of loading, where loading is axisymmetric and the angle of the major principal stress (σ_I) concurs with the vertical axis. Therefore, it can not be applied in the 3D results of two concurrent wheel loads, because the first principal axis differs from the vertical axis even under the centre line of one wheel load.

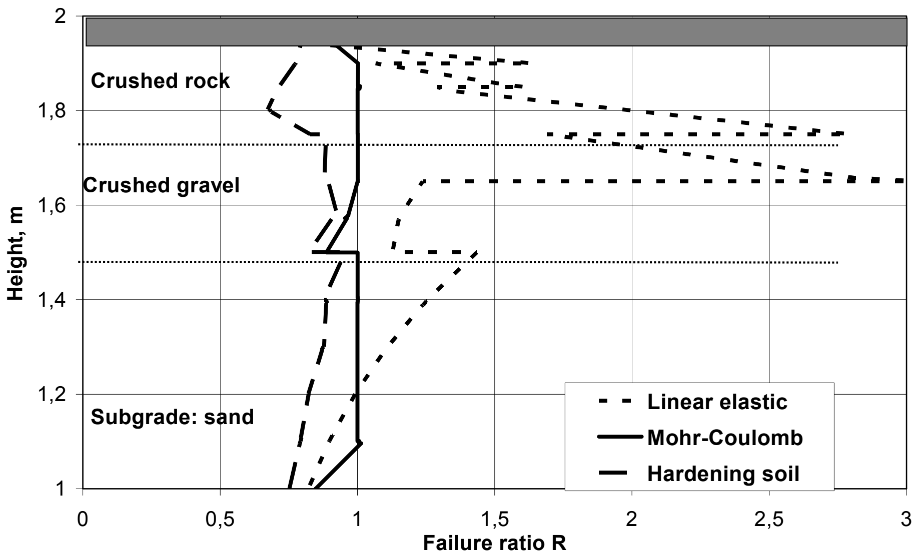


Figure 22a. Comparison of the failure ratio in the centre line of loading Spring-Overload test, when loading is 70 kN (VII).

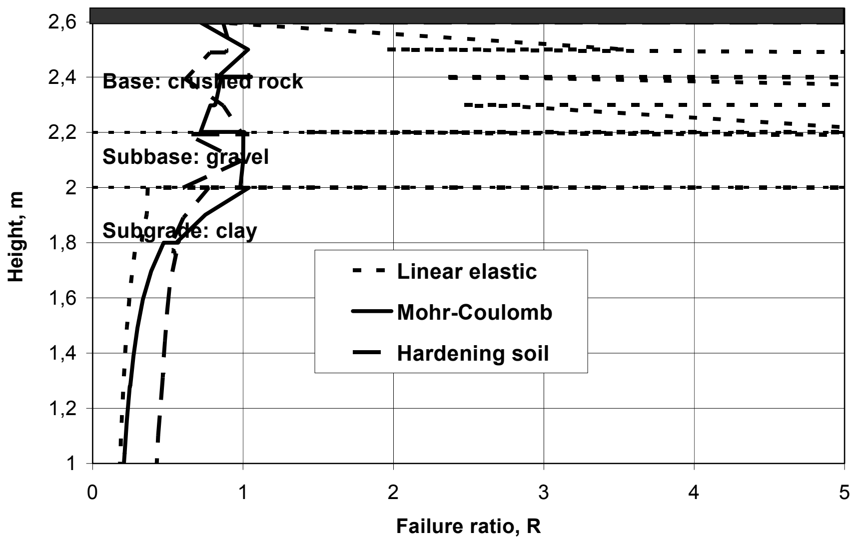


Figure 22b. Comparison of the failure ratio in the centre line of loading Low-Volume test, when loading is 50 kN (III).

The surprising distribution of permanent deformations in the Spring-Overload (SO) and Low Volume (LV) tests demonstrated (Figure 31) that it is not easy to predict where permanent deformations in the pavements occur. Because of the similarities of the loading conditions, it was obvious that the explanation for the permanent strain distribution could only be the relations between stresses and stiffness (material properties and their thickness) (III). Figures 22a and 22b clearly show that linear elastic models concentrate the stresses to the upper layers. For the MC model the failure ratio is near to one even in the upper part of the subgrade, especially for the SO structure (down to the depth of about 900 mm). The HS model gives failure ratios that resemble the strains (Figure 31).

The axisymmetric analysis simplifies the geometry more in the upper part of the structure than in the lower part, because of the changes in the shape of the contact area. This fact has even more effect on dual wheel loading than for the single wheel case, which resembles more of the presumed circular loading area. Figures 23 and 24 compare the real contact areas of dual or single tyre to the equal circular loading area of Plaxis calculations with the same contact pressures. The contact pressures and areas were measured in connection with the earlier HVS tests in 2000 and they have been collected to Appendices A and B for both dual wheel and single wheel of HVS-Nordic, respectively. In the stress calculations of Plaxis the loading area radius has to be kept constant for different load levels, so the applied contact pressures are not exactly the same as in Appendix B.

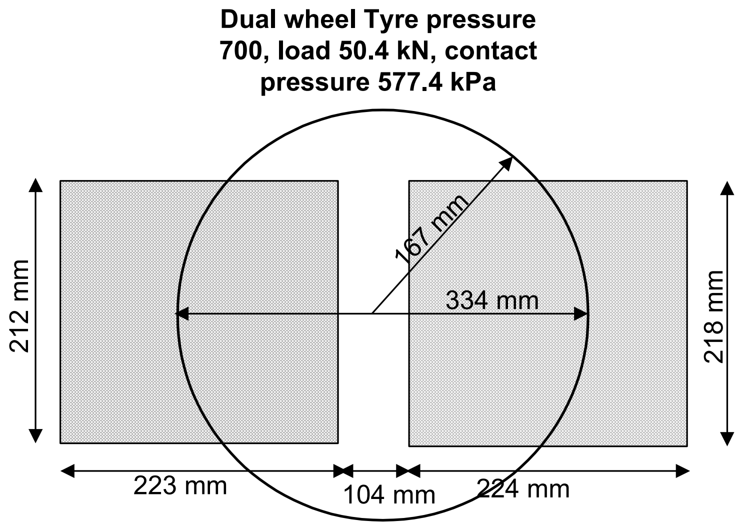


Figure 23. Contact area of dual tyres compared to the loading area of Plaxis calculations.

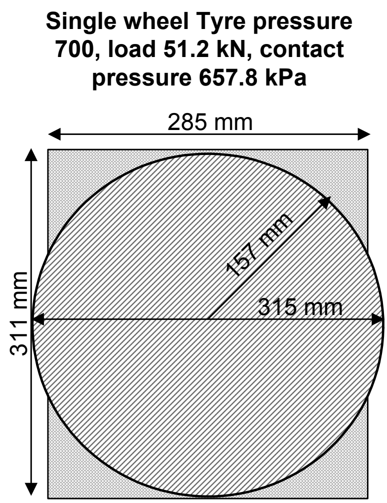


Figure 24. Contact area of a single tyre compared to the loading area of Plaxis calculations.

5. Special factors affecting permanent deformations

5.1 Pavement edge effects

A geometric factor has been introduced to describe the effect of the side slope on the rate of rutting development (III). The geometric factor GEOM (Equation 11) describes an average, structurally independent term that increases the rate of rutting. The GEOM factor depends on the steepness of thee slope and on the distance from the edge of the construction. The GEOM factor is only needed in the vicinity of the side slope. If the distance of the wheel path is more than 2.5 metres from the side slope, its effect can be neglected. The GEOM factor was developed (III) as a pragmatic approach, recognising that it can only be tentatively proposed given the limited verification.

$$GEOM = 1 + \frac{0.86}{l^{1.454} \cdot 2.7^{\frac{N}{3}}} \tag{11}$$

where N is the horizontal steepness of the slope, if the vertical measure is one; unit-less

l is the distance of the loading (centre line of the wheel) from the slope crest; m.

The validity of the empirically developed GEOM factor was tested by comparing the values obtained from in-situ measurements, back-calculated from in-situ loading tests and from laboratory tests, which differed appreciably (III). The Orkanger [Aksnes 2002] study results fit well with the data collected in this study, tending to confirm the findings concerning the interaction of permanent deformation in the vicinity of the side slope. The GEOM factor gives the best estimation of measured response, while the wedge method (III) overestimates the effect giving very small rut depth ratios and the FE calculations, based on resilient response, significantly underestimate the effect. The GEOM factor gives an acceptable approximation of the increase in rutting due to a proximal side slope, especially for less steep slopes. It can be used as a part of the permanent deformation calculations until better approaches are available.

5.2 Loading rate effects

The rate of loading can affect unbound layers in two ways:

1) the changes in stress, which stem from changes in the bituminous overlay's modulus (thermo-viscoplastic material) and
2) the effects of loading rate itself on the plastic response of unbound material.

The results of Publication VI demonstrate that the effect of loading rate is very sensitive to changes in temperature and also to the structure. The loading rate effect of visco-plastic bituminous materials is so clearly proven [e.g. Laaksonen et al. 2004] that it should be taken into account when, for example, the results of slower accelerated loading tests are adjusted to a higher loading rate. One simple way to do this is by calculating the stress state at both loading rates and by estimating the effect on rutting through the changes in the stress levels.

5.3 Loading history effects

The VTT triaxial test results prove that the permanent deformation of granular materials is dependent on the stress history (VI). Yet, the test results show that the superposition of individually calculated permanent deformations at separate loading levels can be used in calculation procedures relatively reliably. The calculation will probably underestimate permanent deformation when the number of loading cycles is low and overestimate when the number of cycles is high.

5.4 Temperature effects

Publication VI shows that the effects of temperature can be considered by using a visco-elastic material model for asphalt layers in the modelling. This can be done by determining a temperature corresponding resilient modulus for the asphalt materials.

6. Comparison of calculated and measured permanent strains

The permanent strains have been calculated for the Spring Overload (SO) (VII) and Low Volume (LV) APT tests. The LV results are only presented in this thesis. The SO test structure is presented in detail in Publication VII. The stress responses have been produced from the 2D axisymmetric modelling calculations. The calculated strains have been compared with the measured permanent strains for each layer. The permanent strains have been measured with the Emu-coil (or also called the "ε-mu coil") system [Korkiala-Tanttu et al. 2003a and 2003b]. The Emu-coil consists of two coils, the distance of which is measured as a function of changes in magnetic flux density [Janoo et al. 1999]. The 2D axisymmetric stress responses were chosen because failure ratio R can not be determined from the 3D results. Both the Mohr-Coulomb (MC) and Hardening Soil (HS) models were applied to evaluate the differences between them. Figure 25 illustrates the calculation results of the SO structure at the loading of 50 kN and Figure 26 at the loading of 70 kN.

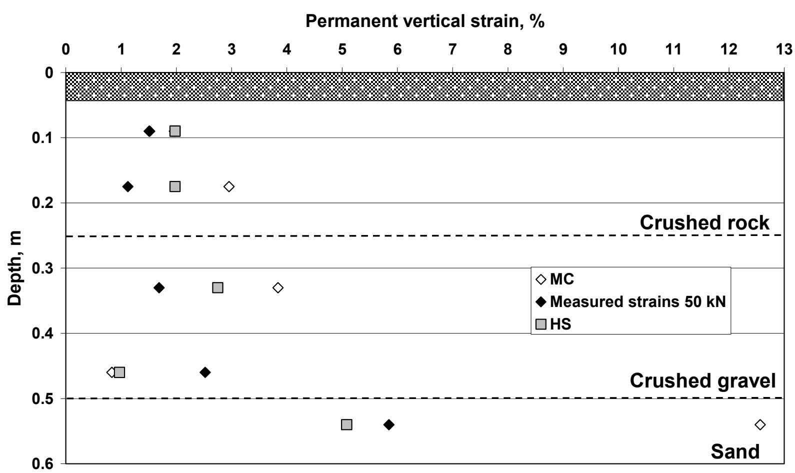


Figure 25. Calculated and measured vertical strains in the centre line of SO structure with 50 kN loading (VII).

The calculation approaches underestimate the permanent strains for the high loads and overestimate it for the lower loads for SO structures. The measured and calculated strains together with the relative errors are presented in Table 11.

Negative values present overestimation while positive values present underestimation. As the stress studies showed (Figure 20), in the 2D axisymmetric case the deviatoric stresses are nearly three to four times great in the upper 250 mm layer compared to the real 3D deviatoric stresses. When the HS model is used, the high stress concentrations in the base layer are redistributed into a larger area. Therefore the failure ratios are relatively near to each other and the method is not very sensitive in the base course of the SO structure.

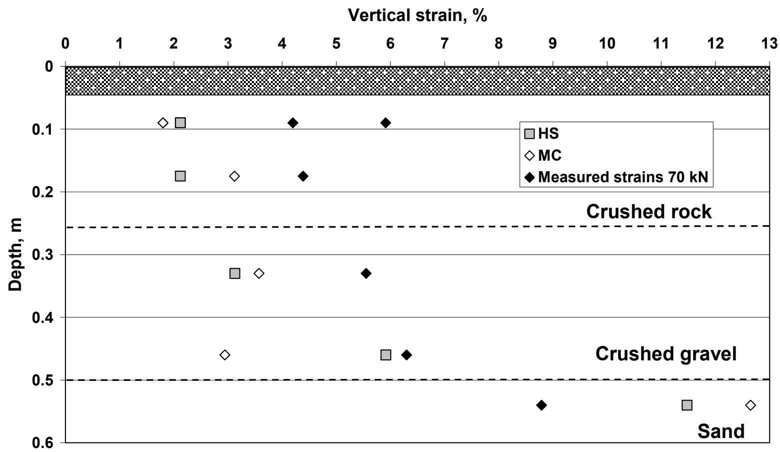


Figure 26. Calculated and measured vertical strains in the centre line of SO structure with 70 kN loading (VII).

Table 11. The measured and calculated strains and relative errors for SO structure for 50 kN and 70 kN loads.

Layer	Measured strain (%) 50 / 70 kN	MC		HS	
		Calculated strain (%) 50 / 70 kN	Relative error (%) 50 / 70 kN	Calculated strain (%) 50 / 70 kN	Relative error (%) 50 / 70 kN
Crushed rock upper*	1.5 / 5.1	1.8 / 2.0	-30 / +63	2.0 / 2.1	-31 / +62
Crushed rock lower	1.1 / 4.4	3.0 / 3.1	-163 / +29	2.0 / 2.1	-76 / +52
Crushed gravel upper	1.7 / 5.6	3.8 / 3.6	-128 / +70	2.7 / 3.1	-63 / +44
Crushed gravel lower	1.7 / 6.3	0.8 / 2.9	+67 / +53	1.0 / 5.9	+61 / +6
Sand	5.8 / 8.8	12.6 / 12.7	-115 / -44	5.1 / 11.5	+13 / -31
Average error			-66 / +33		-21 / +31

**measured values are averages of two measurements*

Figure 27 illustrates the calculated and measured vertical strains of the LV test structure for a 30 kN load and Figure 28 for a 40 kN load. Table 12 presents the measured and calculated values together with the relative error.

Table 12. The measured and calculated strains and relative errors for the LV structure.

Layer	Measured strain (%) 30 / 40 kN	MC		HS	
		Calculated strain (%) 30 / 40 kN	Relative error (%) 30 / 40 kN	Calculated strain (%) 30 / 40 kN	Relative error (%) 30 / 40 kN
Crushed rock	0.7 / 2.2	0.7 / 1.3	-2 / +40	1.3 / 1.4	-91 / +38
Gravel	0.6 / 0.9	0.5 / 0.8	+7 / +9	0.6 / 1.1	+0 / -33
Clay up	0.3 / 0.2	0.2 / 0.3	+37 / -67	0.3 / 0.3	-4 / -95
Average error			+14 / -6		-32 / -30

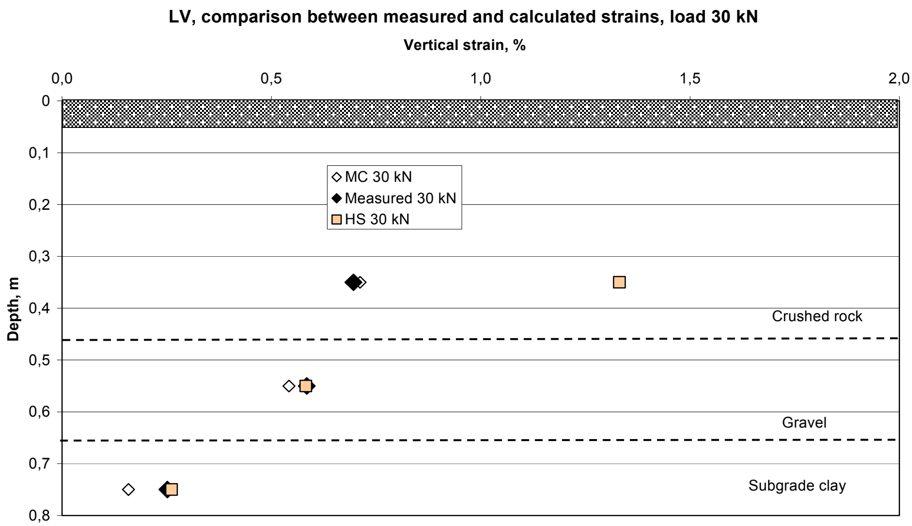


Figure 27. Calculated and measured vertical strains in the centre line of LV structure with 30 kN loading.

For the LV test structure the phenomenon seems to be quite similar, showing a trend where the underestimation is growing for the higher loading levels. In the LV test structure it is obvious that the permanent deformation parameters could not be defined reliably for the relatively loose (degree of compaction only 89%) part of the base layer, because in such cases the calculation method failed to estimate the permanent strains. The strains are very small in the clay layer, so the relative errors give very high values, even though the differences are quite small.

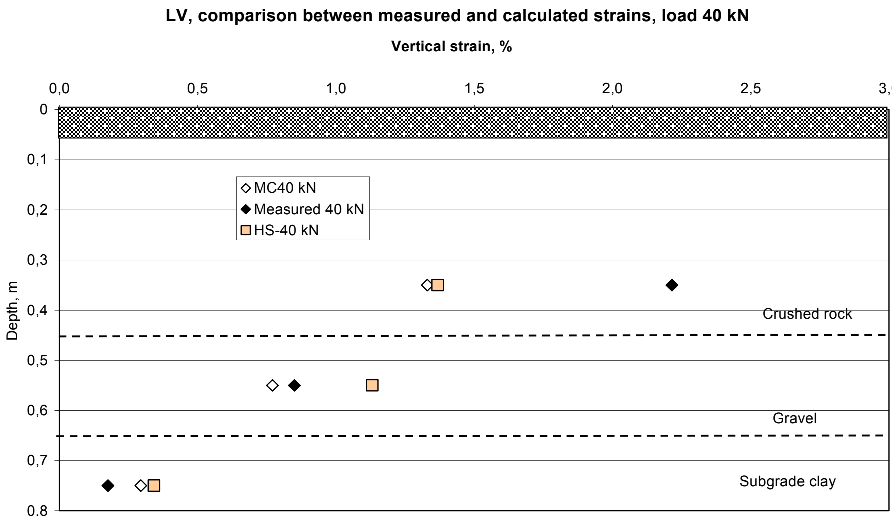


Figure 28. Calculated and measured vertical strains in the centre line of the LV structure with 40 kN loading.

In general the variation of the results is greater for the MC model than for the HS model especially when the loads are higher. The problem with the high load level is that when the maximum shear strength ratio of 1 for the MC model has been reached, the permanent deformations will be dependent on the load level.

The measurement of strains also includes uncertainties. The measured error of Emu-Coil pairs, according to Janoo et al. studies [1999], was within ± 1 mm, which is also at the threshold limit of the ability to detect permanent deformations. This error corresponds to the %-unit error of ± 0.5% to 1.25% depending on the distance of the coils, which means that the LV's calculated strains for 30 kN and 40 kN are within or near to this error range. But for the SO structure, where short distance Emu-coils were mainly used, this error range was exceeded in many cases.

7. The process description of the developed calculation method

The calculation process of the permanent deformations for one loading level and one period is presented in Figure 29. If the effect of seasonal changes is needed, it is possible to repeat each calculation case by changing the material parameters for each seasonal period. Then the number of loadings has to be re-evaluated for each period separately. The test calculations with the Pavedef program [Laaksonen et al. 2004] have shown that most of the rutting can happen during one period, typically under hot summer days, when the high temperature of the asphalt remarkably decreases its stiffness and the unbound layers below it are susceptible to much larger stresses than during other periods. For the low-volume roads, where the asphalt is thinner, the rutting can be concentrated to the thaw – weakening period. The test calculations with Pavedef also confirmed the assumption that loading case studies can usually be concentrated to the heaviest loadings cases to reduce calculation time.

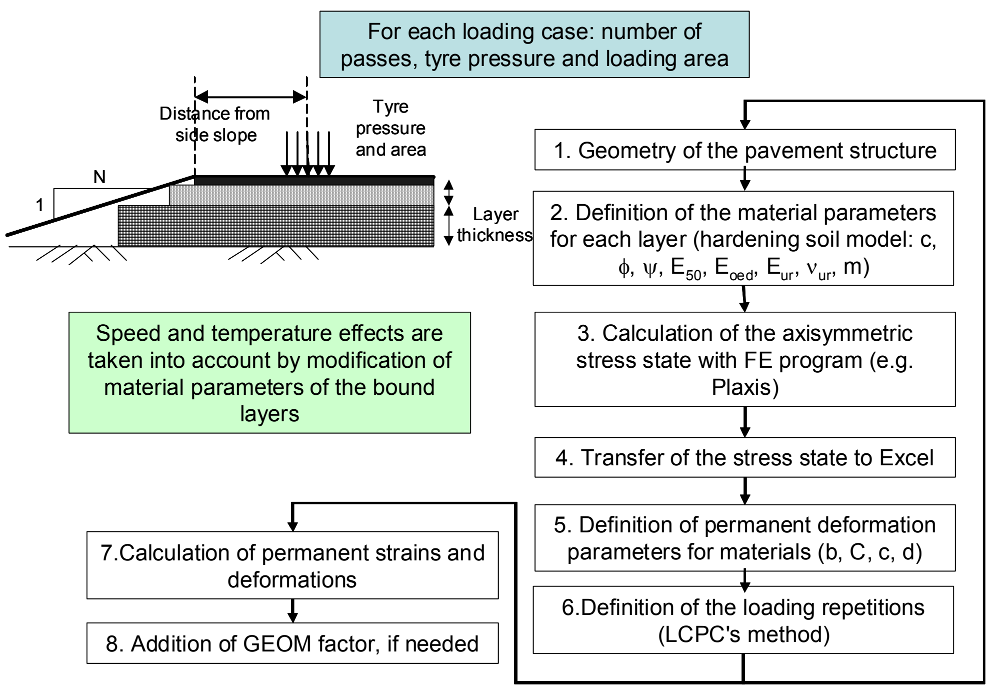


Figure 29. The calculation process of permanent deformations with the developed material model.

The rate of loading, temperature and seasonal changes are all taken into account through the changes in material parameters (steps 2 and 5). The layer thickness of the pavement is taken into account in the first step.

After the calculation steps 3 and 4, the number of loading repetitions in each loading level can be defined by applying the superposition method developed at LCPC France [Gidel et al. 2001]. In the LCPC's method the successive loading stage curve has been separated into reference curves for each loading stage. By using the curve-fitting method the tests have shown that about 10% of the passes of previous loading levels correspond to the 'compaction' effect of those previous loadings. So if more than one loading level is used the number of passes should be increased by about 10%. By using this method each loading level can be treated separately.

Figure 30 illustrates the principle of the method for the Danish ALT test RTM2 sand [Danish Road Institute 1997a and 1997b]. After that the strains are calculated from Equation 8. The strains are multiplied with the thickness of the layer to get the deformations and then summed to get the total rut depth.

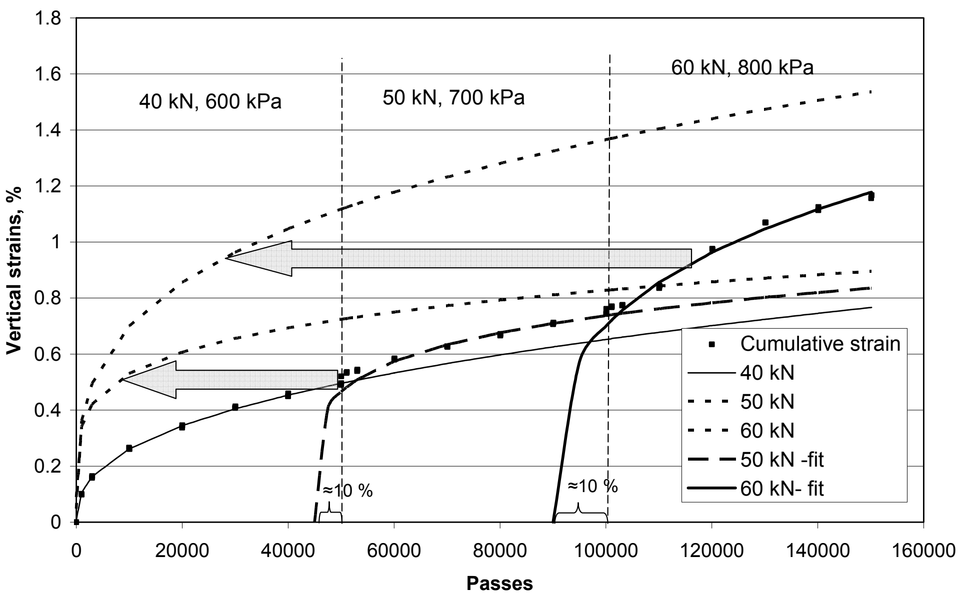


Figure 30. The separation of the successive loading-deformation curve into different loading level reference curves for the Danish RTM2 test sand.

If the wheel path is near the pavement edge (≤ 2.5 m), the effect of the slope can be taken into account by using the GEOM factor. This can be done in the last step 8 by multiplying the total rut depth with the GEOM factor.

An example calculation is presented in Table 13 for a LV structure with the load levels of 30 kN and 40 kN, using the HS model and without the GEOM factor, so step 8 is excluded. The contact pressure is 424 kPa (30 kN) and 565 kPa (40 kN), when the loading area radius is 150 mm. The thickness of the asphalt concrete is 50 mm, base course (CR = crushed rock) 400 mm (lower part 200 mm) and subbase (G = gravel) 200 mm. From the subgrade clay, only the upper 200 mm is included. The material parameters of the HS model for stress calculations were presented in Table 7.

Table 13. An example calculation of the permanent deformations (LVstructure).

Step	**Description**	**CR low**	**G**	**Clay**
3 / 4	Average deviatoric average stress, kPa, 30 kN/40 kN	63/77	26/32	16/19
	Average deviatoric stress in failure, kPa, 30 kN / 40 kN	79/100	34/38	33/34
	Failure ratio R, average 30 kN/40 kN	0.80 / 0.80	0.74 / 0.85	0.49/ 0.55
5	C (%), c and d (Table 10)	0.64, 0.05 0.18	0.02, 0.15 0.18	0.05% 0.21*
	b 30 kN/40 kN	0.22/ 0.22	0.29/ 0.31	0.21*
6	N(30 kN)= 3 600, N(40 kN) = 5 400 + (3 600 *10%) = 5 760			
7	Permanent strain, % 30 kN/40 kN	1.33/ 1.37	0.58 /1.22	0.26/ 0.34
7	Permanent deformation, mm 30 kN/40 kN	2.6/2.7	1.2/2.4	0.5/0.7

*deformation parameters C, c and d for clay have been defined from triaxial tests

The developed calculation method has not yet been implemented into any program. The Finnish prototype for rutting calculations Pavedef could be a suitable foundation for the program implementation as it was planned to be. But since the use of the stress calculation tool Pavedef is based on non-linear elastic materials, the developed method can not be implemented into it without also changing the stress calculation sub-routine. In spite of this, Pavedef has many other positive features: it is based on periodical design and it can also take into account lateral wander, which is not possible for the developed method.

8. Discussion

8.1 The background for the selection of the material equation development

There are some permanent deformation calculation methods that mainly assume the subgrade deforms and then there are the opposing methods that argue the base course is deforming the most. The vertical strains measured with EMU-coils from the Finnish HVS test structures showed that the deformations can develop in different pavement layers in pavements with thin bound layers. How much different parts of the pavement generate permanent deformations depends on the material parameters and on the structure (e.g. stiffness and strength – stress relationships). In the LV test the base course deformed most and in the SO test the subgrade deformed the most (Figure 31). So, from the surface of a moderately rutted road it is impossible to know where in the structure rutting has really occurred. The only possible method is to analyse the permanent deformations in different pavement layers.

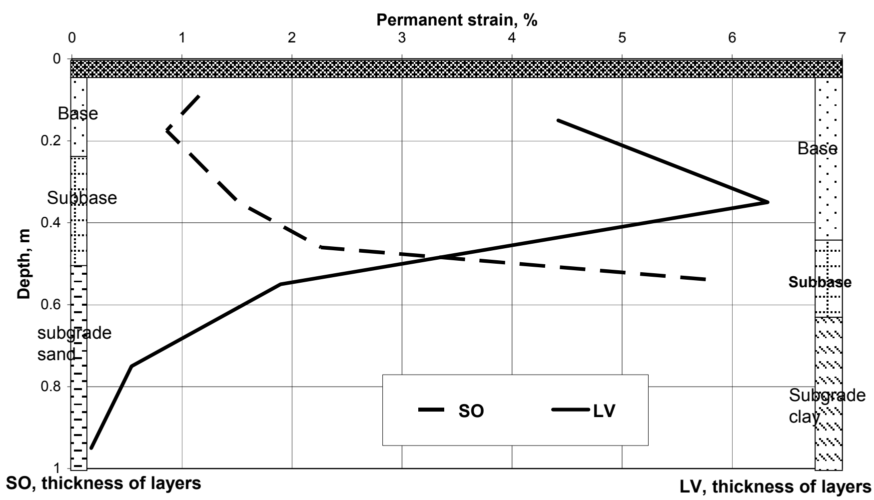


Figure 31. The measured permanent strains in SO and LV structures (I).

Some permanent deformation calculation methods are based on the magnitude of resilient strain, like Veverka [1979] and Zhang and Macdonald [2000]. The problem with this approach is that the relationship of resilient and permanent deformations is highly non-linear. Moreover, many studies have shown that there is a limit value of traffic load after which the permanent deformations will grow quickly. This limit value can be called the shakedown plastic limit or yield stress. Yield stress can be clearly seen when both resilient and permanent deformations are compared with each other (Figure 32). The amount of yield stress depends on many factors, the most important of which are material, water content, stress state and density. In the 'Steep slope' test the yield stress was not exceeded so the permanent deformations were quite moderate (I). The 'Steep slope' test was a HVS test that was conducted on the LV test structure after rehabilitation of the initial structure [Korkiala-Tanttu et al. 2003c].

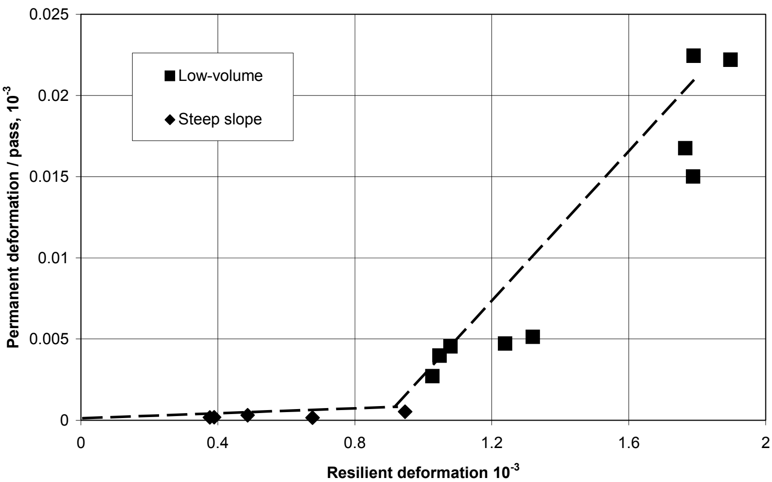


Figure 32. The ratio of resilient and permanent deformations in the 'Low-volume' and the 'Steep slope' tests in the base course's crushed rock (I).

The basic hypothesis in this study is that deformations grow fastest when the stress state approaches failure. There are many different possibilities to define the relationship between stress state and corresponding failure stress. Huurman [1997] treated the stress dependency of the parameters by binding them to the failure proportion of the major principal stress $\sigma_1/\sigma_{1,f}$. Huurman also added a second term to his equations, which are similar to Sweere's equation (1), to describe the incremental failure of the material (see shakedown theory in Chapter 2). Werkmeister [Werkmeister 2004] has developed Huurman's equation further, but instead of using failure ratio $\sigma_1/\sigma_{1,f}$, she has developed a stress dependency of parameters to principal stresses σ_1 and σ_3. The model is called the

DRESDEN-Model. The reason for the declining failure ratio $\sigma_1/\sigma_{1,f}$ for the DRESDEN-Model was that the definition of the failure parameters for unbound granular materials was considered to be tricky.

Both Huurman [1997] and Werkmeister [2004] have based their deformation equations on triaxial test results and shakedown theory. In the axisymmetric triaxial test it is easy to define the first (σ_1) and third (σ_3) principal stresses. The development of the permanent deformation equation in this study is based mainly on the results of full scale APT results, where the definition of the three principal stresses is much more complicated. Thus, the deviatoric stress q and the corresponding deviatoric stress in failure q_f were decided to represent the failure ratio. Added to this is the fact that the failure ratio $\sigma_1/\sigma_{1,f}$ does not represent directly the failure ratio, which was proven to be the most important variable. The failure ratio in this study was calculated directly under the loading, where the principal stresses are supposed to concur with normal stress.

The initial assumption of the permanent deformation equation development was also that the stress level should be taken into account in the equation. Figure 33 illustrates the values of parameter C for sandy gravel as a function of effective hydrostatic pressure p'. Most of the test results have been derived from triaxial tests and the results include only a couple of HVS test results. Surprisingly the value of parameter C, which describes the amount of permanent deformation in the particular stress level, seems to be independent or only slightly dependent on the stress level p'. The same kind of behaviour was also detected for sand and crushed rock. While there seemed to be no strong connection, the stress level factor was not included in the deformation equation. This behaviour might be particularly true for the triaxial tests, when there is no rotation of the principal axis. The calculations with real HVS structures LV and SO verified that the prevailing stress level should be included in the calculation equation.

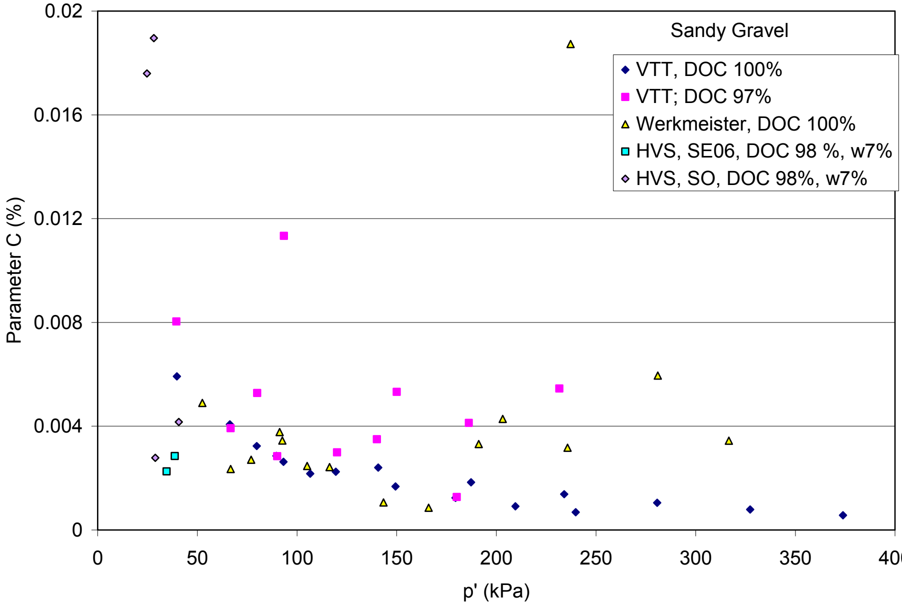


Figure 33. Parameter C and hydrostatic stress p' for sandy gravel (V).

8.2 Stress studies

The stress studies (II and VII) proved the importance of the use of the more complicated material model rather than the linear elastic model in the permanent deformation calculations. If the HS and MC material models are compared, it was quite natural that the non-linear, hyperbolic elasto-plastic HS model gave smaller deviatoric and vertical stresses in both the 2D and 3D modellings (Figures 20 and 21). The differences in stresses were bigger in the upper part of the structure, where the different contact pressure and shape of the loading have more effect. At the depth of 500 mm the deviatoric stresses for 3D and 2D axisymmetric cases approached each other and the differences were less than 10 kPa. The differences between plain strain and axisymmetric modelling were the largest in the lower part of the structure. These results were all as expected. In the plain strain case the deviatoric stresses decreased surprisingly slowly downwards. Thus, deformations can easily be overestimated if plain strain modelling is used. After all, the plain strain modelling can only be recommended for use when the shape of the pavement is analysed (like shoulder and side slope steepness).

The calculated deviatoric stresses in 3D for the MC and HS models were close to each other. The average relative difference between 3D MC and HS calculated stresses was 11% and it varied between -1 to 21%. For 2D the difference was smaller: on average MC defined stresses were about 4% bigger than HS defined stresses.

The vertical stress component shows the same phenomenon as the deviatoric stress comparison: stresses calculated with MC and HS models are relatively close to each other. The 2D and 3D stresses separate from each other in the upper part of the pavement (to the depth of about 500 mm). The 3D stress calculation is supposed to give more reliable results, because the load distribution can be modelled in a more realistic way. It is also probable that with a single wheel load the difference between 3D and 2D is smaller in the upper part of the pavement.

The HS and MC stress responses were reasonably close to each other, but the failure ratios separated more clearly. On average the failure ratio R for the HS model varied from 0.7 to 0.84 for the LV structure and from 0.79 to 0.85 for the SO structure, while it was around 0.95 to 1 for the MC model. This is why the stress calculations for the permanent deformations are recommended to be done with the HS model. If a suitable non-linear elasto-plastic model is not available, it is also possible to use the MC type model. In such case the denominator in Equation 8 should be in the form (X-R), where X is about 1.05. This correction must be made, otherwise the deformations become infinite.

The calculated resilient deformations, which fitted considerably well with the measured ones, were not very sensitive to the used material model (II and III) unlike the permanent deformations. The reason for this is probably that the resilient deformations are governed by major principal stress and the permanent deformations are governed by the deviatoric stress and failure ratio.

8.3 Material parameters

The test calculations (IV and V) have shown that the material equation is quite sensitive to the changes in parameter b. On the other hand the value of material parameter C depends very much on the degree of compaction and on water content. For calculation purposes the value of parameter C is more important, because its value is easier to modify. Also definition of the material strength parameters is important, because the failure ratio has a great effect on the total deformations. Because the pavement materials are usually well compacted and their saturation degree should be small, the friction angle and cohesion can have quite high values in the pavement layers. The friction angle can be 10° to 15° higher than normal values in geotechnical applications.

The material parameter C for the permanent deformation method has been defined mainly from the triaxial laboratory tests. Due to the fact that it depends on the failure ratio R, it has to be defined separately for MC and HS material models. The material parameter C has been estimated to have about 2 to 4 times larger values for the HS model than for the MC model.

8.4 Permanent deformations

The calculation approaches underestimate the permanent strains for the high load levels and overestimate it for the lower load levels. The average relative errors for MC calculations (Tables 11 and 12) varied from +33% to -66% (negative values for overestimation), and for HS calculations from +31% to -21%. The stress studies showed (Figure 21) that in the 2D axisymmetric case the deviatoric stresses are clearly overestimated in the upper part of the structure down to about 400 mm depth. That is the main reason why the permanent deformations with different load levels are near to each other in the base layer.

The later studies of the author [Korkiala-Tanttu 2009] have shown that the calculation method can work much better than in these two cases. The calculation method was verified with two other APT test structures from Denmark and Sweden. In these cases the relative error of deformations was from +7% to -17% with the HS material model. The main reason for the better calculation results was that in these two cases the total rut depths were smaller (around 10 to 13 mm) compared to the SO and LV structures where the rut depths varied from 20 to 60 mm with much lower load repetitions. The stress state and especially the failure ratios in both the LV and SO structures were much higher – with R around 0.85, while it was around 0.73 for the Danish and Swedish verification structures. In the two verification cases the thickness of the bound layers was clearly larger, from 84 mm to 100 mm compared to abound 50 mm for the SO and LC structures. From this it can be concluded that the calculation method gives more reliable results when the structure has thicker bound layers and the stress state in unbound material is lower.

In general the underestimation is greater for the MC model than for the HS model. Besides the problems of the failure ratio to be nearly one, another reason for the underestimation is that the method does not take into account the rotation of the principal axis. For example the studies of Joer et al. [1998] and Kim & Tutumluer [2006] have proven that the rotation of the principal axis has a significant effect on the permanent deformations. In future research an implementation of the hydrostatic pressure component into Equation 8 might improve the underestimation feature of the model.

Also the strain measurement includes uncertainties. The calculated error was between or near the error range of Emu-coils for the LV structure for loads 30 kN and 40 kN. But for the SO structure, where short distance Emu-coils were mainly used, this error range was exceeded in many cases.

8.5 Application area of the method

The developed calculation method is a simplified, averaging approach to evaluate the permanent deformations in unbound granular layers. Due to the simplifications and basic assumptions it has some limitations and a source of errors.

The biggest limitation is that the material parameters have been defined only for basic Finnish pavement materials. The definition is mainly based on the empirical data and laboratory tests are seldomly made because they are considered to be too expensive. Therefore there is very little if any parameter information for the use of new or recycled materials. If a periodical calculation procedure is used, the seasonal changes of material parameters should also be known in order to find out the dependency on, for example, the water content and degree of compaction. And to get a larger application area for the method, a larger set of materials should be tested to define their material parameters. An interesting trial to define more material parameters could be the analysis of the many well documented accelerated loading tests, for example from Sweden or the USA.

In real pavement structures, the material parameters even in one layer vary in large scale due to the compaction degree, water content, changes in material composition and so on. Because averaged material parameters are used, the method calculates only the averaged deformations in unbound layers.

The modelling of stress states also includes many simplifications, like the axisymmetric approach and the assumption of the validation of the Mohr-Coulomb failure criteria. Also the method is not capable of coping with an incremental collapse in the structure.

The geometry of the structure is taken into account by the GEOM factor, which is a pragmatic and useful approach. Further validation and theoretical development is warranted to enhance its accuracy and robustness.

9. Concluding remarks

The objective was to develop a material model for unbound materials, which is an analytical, nonlinear elasto-plastic model. The developed calculation method is based on the theory of static loading, which is expanded to dynamic loading cases. The material equation is relatively simple and it binds permanent deformations to the most important governing factors. To get the right scaling for the rutting calculations, both laboratory and in-situ tests were used.

In the calculation of permanent deformations, it is important to model stress distribution with a better model than a conventional linear elastic material model. By using a linear elastic material model there is a high risk that there will be false tensile stresses in the unbound pavement layers. The risk is emphasised in pavement structures which are thinly paved or totally unpaved. These tensions will cause unrealistic stress concentrations with misleading information about permanent deformation sensitivity. The best method to model unbound materials proved to be the non-linear plastic material modelling (HS), which gives more reliable results than the MC model.

The 2D axisymmetric modelling proved to be accurate enough to be used in the stress calculations, because it gives quite reasonable stress distributions in the lower part of the pavement structure. In the upper part of the pavement the stresses are overestimated, especially for the dual wheel load. Because the basic assumption has been that there is no rotation of the principal axis, the maximum deviatoric stress calculation method is not valid in real 3D conditions.

The test calculations for the HVS test structures indicated that the material model gave tolerable results for the relatively high load levels, as the relative error was around ± 30%. The later verification calculations of two APT tests from Denmark and Sweden showed much better performance and the relative error of deformations varied from + 7% to - 17%. From these cases it can be concluded that the calculation method gives more reliable results when the structure has thicker bound layers and therefore the stress state in unbound material is somewhat lower, even if the wheel load is about the same. For extremely high load levels, the method underestimates the deformations. The method needs more development so that it can be better implemented also at higher load levels.

The most important factors affection the results in addition to the material properties of the unbound materials were studied to find a method to implement their effects on the calculation method. The temperature of the bound materials together with the location of the side-slope proved to be the most important factors.

Currently the material parameters of only the most common road construction materials for the calculation method exist. Thus the implementation of the calculation method needs more research for the definition of a wider set of material parameters and conditions (i.e. varying compaction degrees and water contents). However, even in the current form, the method can be applied in a relatively reliable way to compare the sensitivity of different structures against rutting. The method is well-suited for the comparison of different pavement structures and their rutting sensitivity. If the calculation method could be implemented into a permanent deformation program, its applicability could be enlarged remarkably.

For the implementation, the non-linear elasto-plastic material model (HS) is recommended to be used in the stress calculations to give more reliable results than the MC model. If the MC model is used, parameter A in Equation 8 should be about 1.05. This correction has to be made, otherwise the deformations will be infinite. Also the material parameter C should be chosen according to the used material model.

The 2D axisymmetric modelling is recommended to be used in the stress analysis, because in this equation form the maximum deviatoric stress calculation method is not valid in real 3D conditions.

It is also recommended that the loading rate and temperature are taken into account in the definition of the material parameters of bound layers. The geometry of the slope can be taken into account by using the GEOM factor, which depends on the steepness of the side slope and on the distance to the edge of the construction, but not on the load.

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Appendix C: Publications III, IV of this publication are not included in the PDF version. Please order the printed version to get the complete publication (http://www.vtt.fi/publications/index.jsp)

Appendix A: Measured contact areas for dual wheels of HVS-Nordic

HVS-Nordic dual wheel

The contact area with varying wheel load and tyre pressure

The contact area has been measured with planimeter from the paper.

Spring-overload test		Tyre 1			Tyre 2							
Tyre pressure	Wheel load	Width	Length	Measured area	Width	Length	Measured area	Distance between tyres	Measured total area	Contact pressure	Average width	Average length
kPa	kN	cm	cm	cm^2	cm	cm	cm^2	cm	m^2	kPa	cm	cm
500	30.2	22.1	19.2	386.5	22.2	19.9	381	10.5	0.07675	393.5	22.15	19.55
500	40.1	22.5	22.5	458.5	22.3	22.5	443	10.3	0.09015	444.8	22.4	22.5
500	50.59	22.7	25.3	530	22.6	25.4	536.7	10.2	0.10667	474.3	22.65	25.35
500	60.88	22.6	27.7	581.1	22.6	27.6	595.9	10.2	0.1177	517.2	22.6	27.65
500	70.97	22.6	29.8	637.7	22.6	29.9	644.8	10.3	0.12825	553.4	22.6	29.85
500	81.9	22.7	34.8	730.9	22.7	34.4	714.8	101.1	0.14457	566.5	22.7	34.6
600	30.33	22.1	18.9	347.3	22.3	20.1	389.5	10.4	0.07368	411.6	22.2	19.5
600	40.47	22.2	20.5	392.5	22.4	21	423.5	10.25	0.0816	496.0	22.3	20.75
600	50.92	22.6	22.3	460	22.7	23.3	509.4	10.1	0.09694	525.3	22.65	22.8
600	60.15	22.6	25.2	523.5	22.7	26.2	553.7	10.2	0.10772	558.4	22.65	25.7
600	71.05	22.7	27.6	574.7	22.8	28.3	602.1	10.1	0.11768	603.8	22.75	27.95
600	82	22.8	30.9	644	22.7	32.4	678.3	10.1	0.13223	620.1	22.75	31.65
700	30.25	21.4	17.1	309.5	21.6	17	317.6	11	0.06271	482.4	21.5	17.05
700	40.59	22	19.3	377.5	22	19.4	386	10.6	0.07635	531.6	22	19.35
700	50.41	22.3	21.2	429.5	22.4	21.8	443.6	10.4	0.08731	577.4	22.35	21.5
700	61.19	22.7	23.9	503.8	22.7	23.6	509.3	10.3	0.10131	604.0	22.7	23.75
700	70.94	22.7	26.3	556.5	22.7	26	568.9	10.2	0.11254	630.4	22.7	26.15
700	81.2	23	29	630.1	22.9	29.4	637.3	10.1	0.12674	640.7	22.95	29.2
800	30.26	21.2	16.2	290.4	21	17.2	303.6	11.5	0.0594	509.4	21.1	16.7
800	40.15	22.2	18.8	363.4	22.3	18.7	374.2	10.3	0.07376	544.3	22.25	18.75
800	50.93	22.5	21.1	419.4	22.6	21	430.2	10.2	0.08496	599.5	22.55	21.05
800	60.15	22.7	22.7	477.2	22.9	22.5	484.2	10.1	0.09614	625.7	22.8	22.6
800	70.56	22.8	24.3	520	22.6	24.6	529.8	10.1	0.10498	672.1	22.7	24.45
800	80	22.9	27.7	601.4	22.9	28.3	605.8	10	0.12072	662.7	22.9	28
* calculated average value for 850 kPa												
850	70.82	22.85	24.05	517.0	22.75	24.4	528.3	10.1	0.10452	677.6	22.80	24.23
900	30.26	21.1	16.6	296.4	21.6	17.3	308.7	11.1	0.06051	500.1	21.35	16.95
900	40.3	22.3	19	367	22	19.1	367.6	10.5	0.07346	548.6	22.15	19.05
900	50.73	22.7	20.4	421.1	22.6	21.5	440.3	10.1	0.08614	588.9	22.65	20.95
900	60.86	22.8	22.3	468.4	22.7	23.9	488.9	10.1	0.09573	635.7	22.75	23.1
900	71.08	22.9	23.8	513.9	22.9	24.2	526.7	10.1	0.10406	683.1	22.9	24
900	80	23	26.5	572.8	23.1	27.1	593.2	10	0.1166	686.1	23.05	26.8

Appendix B: Measured contact areas for single wheel of HVS-Nordic

HVS-Nordic Super Single wheel						
The contact area with varying wheel load and tyre pressure						
The contact area has been measured with planimeter from the paper.						
Low-volume road test						
Tyre pressure	Wheel load	Width	Length	Measured area	Contact pressure	Measured area
kPa	kN	cm	cm	cm^2	kPa	m^2
500	31.01	24.40	30.90	652.40	475.32	0.06524
500	40.34	29.60	31.00	814.40	495.33	0.08144
500	50.79	34.50	31.20	941.40	539.52	0.09414
500	61.16	37.20	31.40	1064.30	574.65	0.10643
500	70.90	38.00	31.40	1141.10	621.33	0.11411
600	31.00	23.00	30.60	594.90	521.10	0.05949
600	40.44	27.50	31.00	730.60	553.52	0.07306
600	51.01	32.30	31.20	855.70	596.12	0.08557
600	61.19	34.00	31.40	954.30	641.20	0.09543
600	71.31	36.90	31.60	1073.40	664.34	0.10734
600	80.00	38.80	31.70	1171.60	682.83	0.11716
700	31.01	21.70	30.00	543.40	570.67	0.05434
700	39.90	25.40	30.90	667.40	597.84	0.06674
700	51.17	28.50	31.10	778.20	657.54	0.07782
700	60.94	31.80	31.30	883.30	689.91	0.08833
700	71.15	33.70	31.40	977.50	727.88	0.09775
700	81.80	36.30	31.60	1081.10	756.64	0.10811
800	31.01	20.70	29.30	520.30	596.00	0.05203
800	40.08	23.80	29.60	625.90	640.36	0.06259
800	49.99	25.40	31.00	707.60	706.47	0.07076
800	60.85	28.70	31.40	825.10	737.49	0.08251
800	70.91	30.00	31.40	889.10	797.55	0.08891
800	82.10	34.00	31.60	1019.50	805.30	0.10195
900	30.98	20.30	30.30	511.20	606.03	0.05112
900	39.88	22.10	30.60	581.20	686.17	0.05812
900	50.79	25.10	31.00	715.30	710.05	0.07153
900	60.90	28.00	31.20	809.20	752.60	0.08092
900	71.12	29.00	31.20	869.20	818.22	0.08692
900	82.00	32.00	31.60	990.60	827.78	0.09906

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Calculation method for permanent deformation of unbound pavement materials

Abstract

An analytical–mechanistic method for the calculation of permanent deformations in pavements has been developed at VTT. The calculation method is needed in the analytical design procedure of pavements. The research concentrated on the calculation method for permanent deformations in unbound pavement materials. The calculation method was generated based on the test results of accelerated pavement and laboratory tests.

The stress state of the pavement was modelled with finite element methods to be able to use non-linear elasto-plastic models. The deformations were calculated from the number of passes and the material parameters were defined from the saturation and compaction degrees along with loading rate, temperature and so on. The deformations in each layer were calculated and then summed to obtain the total rutting. The method was tested against two Finnish accelerated pavement tests. The results indicated that the material model gave only tolerable results for the extremely high load levels as the relative error was around ± 30%. For the thicker bound layers and lower stresses, the method gave better results. The method is also useful for the comparison of the sensitivity of different structures against rutting. The material parameters have so far been defined only for the most common materials.

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Tierakenteen sitomattomien materiaalien pysyvien muodonmuutosten laskentamenetelmä

Tiivistelmä

Tutkimuksessa on kehitetty analyyttis-mekaaninen menetelmä tien sitomattomien rakennekerrosten pysyvien muodonmuutosten laskemista varten. Menetelmää tarvitaan päällysrakenteiden mitoittamisessa. Menetelmän kehittäminen perustui täyden mittakaavan koetiekone- sekä laboratoriokoetuloksiin. Menetelmässä rakenteen jännitystila mallinnetaan elementtimenetelmällä, jolloin voidaan soveltaa epälineaarisia elasto-plastisia materiaalimalleja. Muodonmuutokset lasketaan jännitystilan ja ylitysten määrän funktiona. Parametrien määrittämisessä otetaan huomioon kuormitusnopeus, lämpötila sekä saturaatio- ja tiivistysaste. Rakennekerrosten ja pohjamaan muodonmuutokset lasketaan erikseen ja ne summataan kokonaisuran määrittämiseksi.

Menetelmää testattiin täydenmittakaavan koetiekonekokeiden tuloksiin. Menetelmä antoi näissä kohteissa melko luotettavia tuloksia suhteellisen virheen ollessa noin ±30%. Paksummilla päällysteillä menetelmä toimi luotettavammin. Toistaiseksi menetelmän materiaaliparametrit on määritetty vain kaikkein tavallisimmille materiaaleille. Tästä huolimatta menetelmä soveltuu jo nykyisellään rakennevaihtoehtojen deformaatioherkkyyden arviointiin.

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An analytical-mechanistic method for the calculation of permanent deformations in unbound pavement layers and subgrade has been developed at VTT. The calculation method is needed in the analytical design procedure of pavements. The calculation method was generated based on the test results of accelerated pavement tests along with the complementary laboratory tests made in Finland.

The developed calculation method for unbound materials was based on an analytical, nonlinear elasto-plastic model. The stress state of the structure was modelled using finite element method and non-linear elasto-plastic material model. The deformations were then calculated for each layer from the number of passes, the bearing capacity of the material and its stress state. The saturation and compaction degrees were taken into account by varying material parameters. So far only the basic material parameters are known, thus more material studies are needed.

The method was tested against two Finnish accelerated pavement tests. The results indicated that the material model gave tolerable results for the high load levels. For the lower load levels the method gave more reliable results. The method can also be applied to the comparison of the sensitivity of different structures against rutting.

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