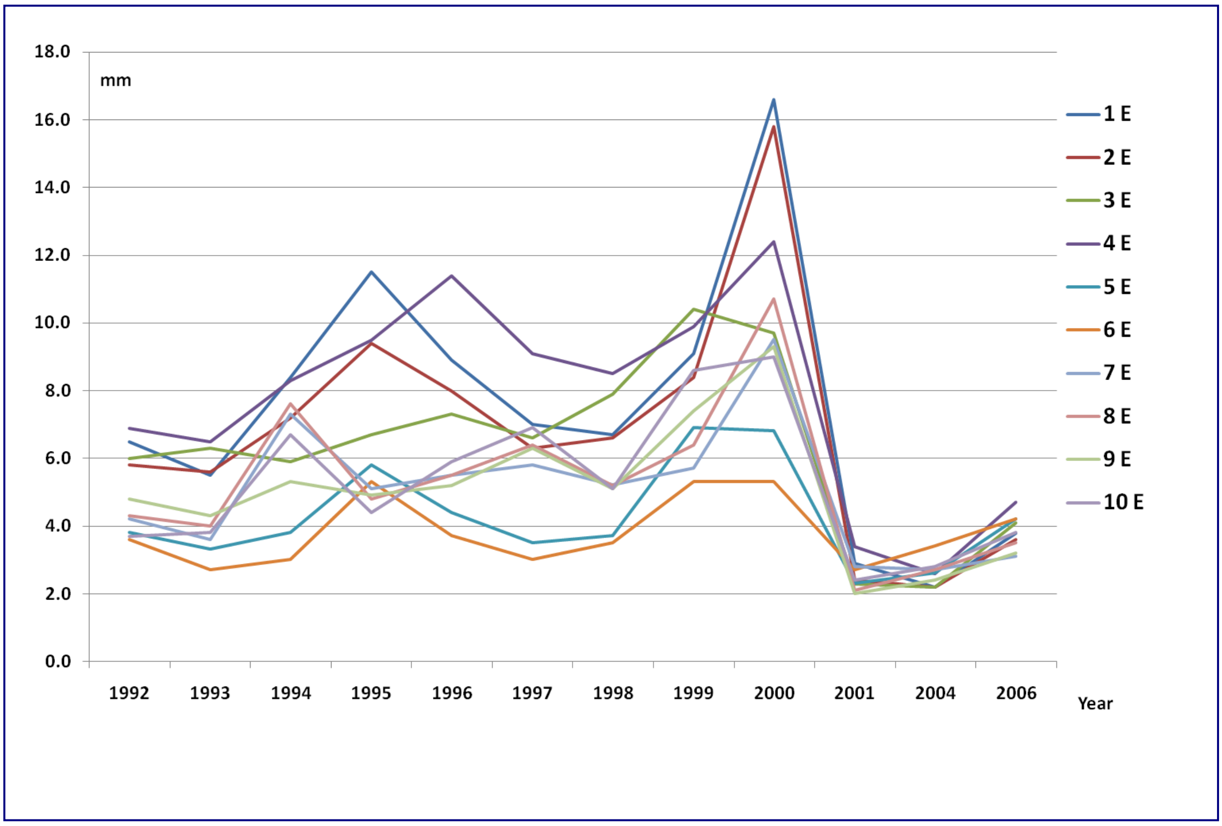

Analysis of Parameters Affecting Permanent Deformation in Road Pavement

Based on Measurement Data from LTPP-Roads

Master of Science Thesis in Geo and Water Engineering

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Cover:
Actual rutting data obtained from LTPP-road in Rv 44 at Grästorp.

Chalmers reproservice / Department of Civil and Environmental Engineering
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ABSTRACT

The complex plastic behaviour of the construction material used in the different pavement layers has been one of the main research topics in pavement engineering. The accumulation of permanent strain, permanent deformation, is affected by a mixture of material and load related factors. In this master thesis, several parameters influencing the development of permanent deformation are described and analysed on LTPP-roads in Sweden.

The analysis is performed by studying several LTPP-roads in terms of rutting and cracking. The focus of this analysis is to identify the factors that contribute to the unstable behaviour that takes place in the pavement and to investigate the correlation between these factors and the permanent deformation.

The analysis concluded that crack index has a major influence on the development of permanent deformation. Therefore, it is recommended that a future deformation prediction models should take into account the parameters such as crack index, subgrade E-modulus and water content of which this master thesis proved to be of much concern.

Keywords: Permanent deformation, Rutting, Material properties, Pavement, Shakedown theory, LTPP, Crack index, VägFEM

Analys av Parametrar som Påverkar Permanenta Deformationer i Vägöverbyggnader Baserad på Mätdata från LTPP-vägar

Examensarbete inom Geo and Water Engineering

OGUZ ACIKGÖZ & REZHIN RAUF
Institutionen för bygg- och miljöteknik
Avdelningen för geologi och geoteknik
Väg och trafik
Chalmers tekniska högskola

SAMMANFATTNING

I detta examensarbete har faktorer som påverkar utvecklingen av permanenta deformationer som uppstår i de olika lagren i en vägkonstruktion studerats i detalj. Vidare har samband mellan dessa faktorer och den permanenta deformationsutvecklingen analyserats för en rad olika test vägar, så kallade LTPP-vägar runt om i Sverige. För att erhålla en omfattande kännedom om de studerade test vägarna har data från LTPP- databasen används. Dessa har i sin tur kompletterats med grundläggande litteratur studier och studiebesök till några av dessa LTPP- vägar.

Projektet avser att identifiera de essentiella faktorerna som bidrar till det instabila beteendet som sker i vägkroppen som i sin tur kan integreras i befintliga modeller eller utgöra grunder för utveckling av nya modeller för förutsägelse av permanenta deformationer.

Från analysen har flera intressanta iakttagelser utarbetats mellan dessa faktorer och permanenta deformationen. Studierna indikerar att sprickindex har en avgörande betydelse för den permanenta deformationsutvecklingen. Därför är det rekommenderat att en ny modell för bestämning av framtida permanenta deformationer ska inkludera parametrar såsom sprickindex, undergrundsmodul och vattenhalt som berörs i detta projekt.

Nyckelord: Permanenta deformationer, Spårbildning, Material egenskaper, Vägkonstruktion, Shakedown teorin, LTPP, Sprickindex, VägFEM

Contents

Preface

This master's thesis has been carried out at the Swedish Road Administration (Vägverket) in Gothenburg for the Department of Civil and Environmental Engineering, at division of GeoEngineering. The thesis was supported by Vägverket and implemented during spring of 2010.

We would like to extend our gratitude to our supervisor Anders Huvstig at Vägverket for his extensive knowledge and support. We would also like to thank associate professor Gunnar Lannér at Road and Traffic Group at Chalmers University of Technology for his encouragement.

Finally, we would like to thank our families and all loved ones for all their moral support and happy cheering.

Göteborg June 2010

Oguz Acikgöz & Rezhin Rauf

Notations

Abbreviations

$AADT_{tot}$	Total Annual Average Daily Traffic
$AADT_L$	Annual Average Daily Traffic per lane
$AADT_{RL}$	Annual Average Daily Traffic in the right lane
AASHTO	American Association of State Highway and Transportation Officials
ABS	(Asfalt Bitumen Stenrik) Hot-mixed asphalt consisting of high aggregate content and bituminous binder
ABT	(Asfalt Betong Tät) Hot-mixed dense asphalt consisting of aggregate and bituminous binder
AG	(Asfaltgrus) Asphalt mixed gravel with low bitumen content
ATB-väg	(Allmän Teknisk Beskrivning - väg) Technical description and guidelines on construction and maintenance of roads by the Swedish Road Administration
BB	(Bergbank) Rock embankment
BBÖ	(Berg/Bitumen Överbyggnad) Hot-mixed asphalt containing crushed rock with high strength
BG	(Bitumenstabiliserad Grus) Bitumen-stabilized gravel
BS	(Bergskärning) Rock cut
BYA	(Byggnads Tekniska Anvisningar) Technical description and guidelines on construction and maintenance of roads by the Swedish Road Administration
FEM	Finite Element Method
FWD	Falling Weight Deflectometer
GBÖ	(Grus Bitumen Överbyggnad) Hot-mixed asphalt layer on unbound gravel structure.
HABS	(Hård Asfalt Bitumen Stenrik) Hard hot-mixed asphalt consisting of high aggregate content and bituminous binder
LTPP	Long Term Pavement Performance
MAB	(Medelhård Asfalt Betong) Medium-hard asphalt concrete
M-E-PDG	Mechanistic-Empirical Pavement Design Guide
NCHRP	National Cooperative Highway Research Program
RST	Road Surface Tester
Rv	(Riksväg) National road
RTL	Repeated Triaxial Load
Si	(Sprick Index) Crack Index
TPPT	The Road Structure Research Programme
VTI	National Road and Transport Research Institute

Y1B12	Surface treatment

Latin Upper case letters

A	Proportion of heavy vehicles
A	Model parameters in eq 2.10
B	Equivalent standard axles per heavy vehicle
B	Model parameters in eq 2.10
C	Model parameters in eq 2.10
D	Model parameters in eq 2.10
D_0	Deflection right beneath the load
D_r	Deflection with the distance r from the plate loading
D_{900}	Deflection 900 mm from centre of plate loading
E_0	Surface linear elastic modulus
E_v	Average modulus on equivalent depth
E_u	Subgrade E-modulus
Kr	Alligator Cracking
L_{max}	$\sqrt{p_{\max}^2 + q_{\max}^2}$
LSpr	Longitudinal Cracking
M_r	Resilient modulus
N	Number of load cycles
N_0	Reference number of load pulses (100 in this project)
N_{eq}	Total equivalent standard axles
T	Temperature
TSpr	Transverse Cracking

Latin lower case letters

a_i	Regression coefficient
a	Radius of the loading plate
a_1	Model parameters in eq 2.11
a_2	Model parameters in eq 2.11
b_1	Model parameters in eq 2.11
b_2	Model parameters in eq 2.11
c_1	Model parameters in eq 2.11
c_2	Model parameters in eq 2.11
d_1	Model parameters in eq 2.11

d_2	Model parameters in eq 2.11
f	2 for segmented plate loading
j	1, 2, 3 ... n
k	Presumed annual change of heavy traffic
m	Internal friction
m	Model parameters in eq 2.7
n	Proposed design period in years
n	Model parameters in eq 2.7
p	Mean stress
p_{max}	Maximal cyclic stresses
q	Deviator stress
q_{max}	Maximal cyclic stresses
r	Distance between the plate loading and measurement devices (geophones)
s	Internal friction angel
s	Model parameters in eq 2.7
v	Lateral contraction

Greek letters

β	Parameter for estimation of rutting
ε_0	Initial strain
ε_p	Permanent (plastic) strain
ε_r	Resilient (elastic) strain
ε_1^p	Vertical permanent deformation
ρ	Material parameters
σ_o	Contact pressure under the loading plate
σ_1	The largest vertical principal stress.
σ_2	The second largest horizontal principal stress.
σ_3	The smallest vertical principal stress.
$\sigma_{1,f}$	The largest failure principal stress
σ_d	The deviator stress

1 Introduction

Roads are one of society's most essential component. Without them, it would be very difficult to get from one place to the other in a timesaving and smooth way. Roads are therefore facing a major challenge in order to deliver these functions and consequently increase the quality of life. In order to fulfil these functions, roads must be properly designed and durable. However, there are roads built on weaker subgrade material and thus perform worse and cause losses in both serviceability and economy. Therefore, in recent decades, further demands on the design of roads have been made. Thus the construction costs shall be reduced and the miscellaneous maintenance work performed in small extent as possible. The major causes of loss in the serviceability and maintenance work is rutting and surface roughness.

1.1 Background

In the past 20 years, a series of sections in several test roads have been studied in the so called LTPP project in Sweden. The purpose of the LTPP project is to gather valuable information about these test roads in terms of rutting, surface roughness and cracking to subsequently develop a model that can predict rutting. The model will be used in road design in order to optimize the sustainability and to predict when maintenance is needed. There are available models used for this purpose since decades. However, all these models have drawbacks in predicting permanent deformation. In an attempt to find out the factors to why the results differ and to suggest improvements in existing models this master thesis was implemented.

Rutting is the greatest distress causing loss of serviceability and thus very undesirable. Rutting reduces driving comfort and worsens road safety, especially during wet conditions when the risk of hydroplaning increases significantly due to water gathered in the ruts. The reason behind the phenomenon of rutting is both material (compaction, grain shape, grain size distribution and organic contents etc.) and structural related properties (load repetitions, stresses and moisture content) in the unbound layers. These properties are in its turn affected by other additional parameters such as cracking, degree of direct sunlight, subgrade material, its E-modulus and water content. In this report the focus has mainly been on these parameters and how these affect the rutting development.

1.2 Problem Description

According to present available models that are used in prediction of permanent deformation in the unbound layer, permanent deformation development will stagnate with increasing load repetitions. Studies of roads in reality have shown faster rutting initially (pre-compaction phase), which latter gradually decreases. The increase of the permanent deformation continues in a stable and slow rate until cracks are developed. Subsequently, the rate of the rutting development drastically increases. The theories stating that the rutting should stagnate over time are only applied for stresses below a certain limit. Further studies have shown that when the stresses exceed a certain level, "Plastic Shakedown Limit", the material in the unbound layer reaches an unstable condition and plastic deformation takes place. This result in an increase of the permanent deformation, see figure 1-1.

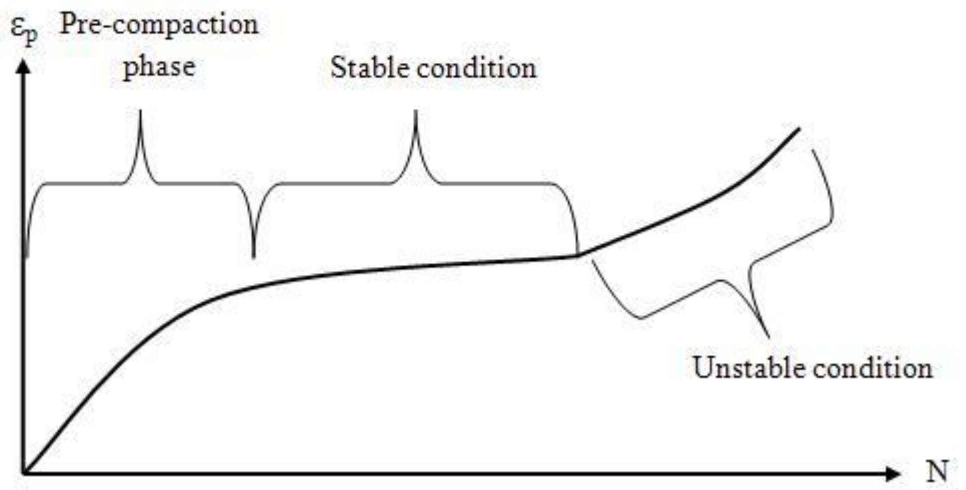


Figure 1-1 Different rutting development phases

The focus of this thesis is to investigate parameters that contribute to the unstable condition that takes place in the unbound granular material caused by increased stresses. This is the latter part of the permanent deformation development that deviates from the predictions made by the existing models, see figure 1-2.

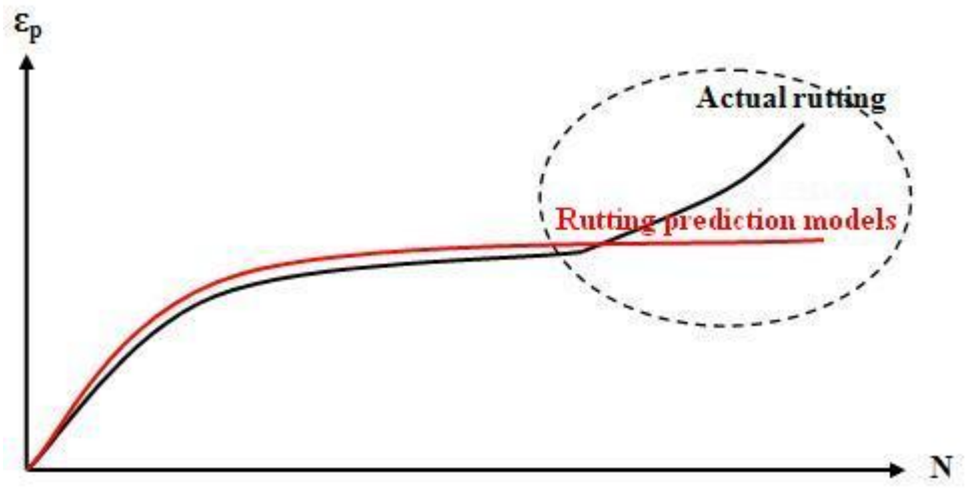


Figure 1-2 Emphasise on the circumscribed area of the rutting development

1.3 Aim

The aim is to identify parameters affecting the permanent deformation and to investigate the correlation between these parameters and the permanent deformation development by studying measurement data from LTPP-roads. This will provide basis to the development of new models and modification of parameters in existing models for better prediction of future permanent deformation.

1.4 Limitations

In order to narrow the aim and focus of this thesis some limitations have been made. These have been made due to limited time and lack of resources. These are:

- Only the permanent deformation that occurs in the pavement and the subgrade is considered.
- The IRI (International Roughness Index) is not considered in this thesis
- The permanent deformation caused by studded tyres and frozen ground is pointed out but not further analysed.
- The falling weight deflectometer (FWD) tests have been performed in only 3 seasons, spring, summer and autumn. The subgrade E-modulus that has been calculated based on these seasons is an average value.
- Among the prediction models only VägFEM has been considered in this thesis.
- Only Rv 46 at Trädet and Rv 31 at Nässjö was planned to be analysed in VägFEM.
- Test results from triaxial tests on material properties in the different layers of the pavement are not included.
- Material parameters such as mineralogy, organic content, grain shape and size distribution are mentioned but not included into the analysis.

1.5 Method

The project started with literature studies of previous research and master's theses in order to obtain a deeper understanding of this topic. Interviews with experts, Anders Huvstig and Carl-Gösta Enocksson, have been a complement to the literature study.

The measurement data from LTPP-database has been the basis to the analyses. The LTPP-database has provided essential information concerning the analysed test roads. Additional study visits to Rv 46 at Trädet, Rv 31 at Nässjö, Rv 34 at Målilla, Rv 33 at Vimmerby and Ankarsrum has been a complement to data obtained from LTPP-database.

Raw data containing rutting, crack index and subgrade E-modulus from LTPP-database was gathered in Excel sheets for each test road. The data was plotted for selected sections and analysed.

Samples of the subgrade material were taken in Rv 31 at Nässjö and Rv 46 at Trädet in order to obtain material properties by performing triaxial tests. Results from the triaxial tests were subsequently planned to be used as input values to a finite element model VägFEM. This was planned to be carried out in order to analyse parameters such as water content and subgrade E-modulus to observe the extent these affect the permanent deformation.

2 Literature Study

The literature review covers the most significant parts of the subject and is an introduction to the analysis of the project. This chapter seeks to increase the knowledge about permanent deformation and its contributing factors.

2.1 Road Pavement Design

There are different types of pavement used in road engineering (flexible, rigid and composite). The flexible pavement is the most common type of pavement used in Sweden. This type of pavement consists of material with higher quality on the top and lower quality at the bottom of the layers, since the intensity of stresses from traffic loads is higher at the road surface. In Sweden, GBÖ is the most common type of pavement that is normally used in medium-sized and major roads where the supply of aggregates is limited, see figure 2-1. The pavement consists of a thin layer of high quality asphalt (wearing course) on the top, which contributes to a more even road surface with better friction and less noise. The wearing course is placed over the base course that is either bound or unbound. The bound base is made up of bituminous binder mixed with aggregates (gravel, crushed stones, moraine and natural stone) while the unbound base layer consists of aggregates only. Grading, size and crushed surface of the aggregate are the key factors that determines the ability of the road to withstand stresses caused by traffic loads. Well-graded aggregates with larger stone size have a better resistance and distribute stresses over a larger area further down in the pavement. The base layer is placed over the subbase, which consists of moraine or a mixture of moraine and crushed stones. The material used in this layer is of a poorer quality compared to the material used in the overlying layers and thereby have a lower resistance to higher stresses. Furthermore, in order to separate the larger aggregates in the subbase from the subgrade material that consists of fine-grained material, a geo-textile is used, Kunskap Direkt (2010).

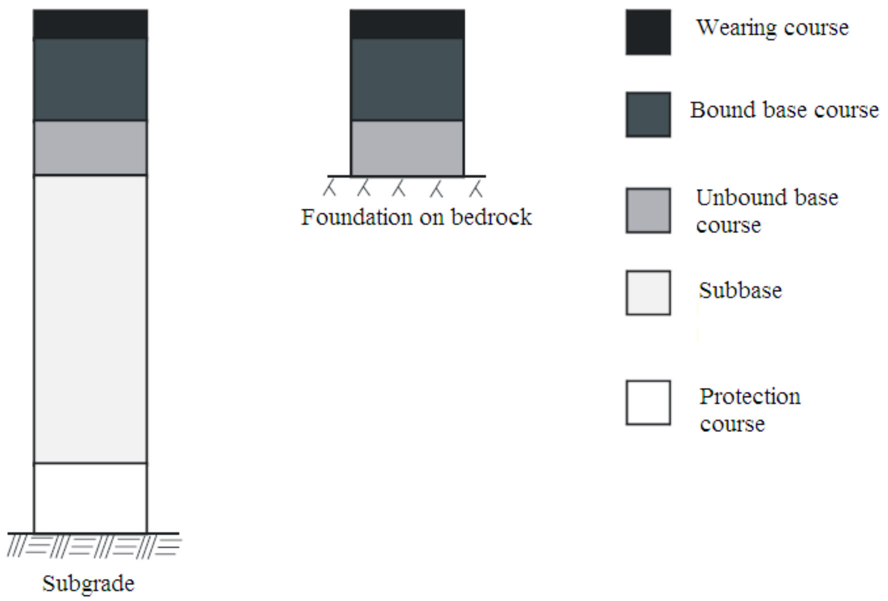


Figure 2-1 GBÖ – the most common pavement in Sweden (ATB-väg, 2005)

2.2 Standard axles

The standard axle is a fictive axle with paired wheels on each side of an axle with a total load of 100 kN, which is equally distributed between the wheels. Each wheel has a circular contact area between the tyre and the road, where each contact area are loaded with a constant pressure of 800 kPa. The centre distance between the wheels in a wheel pair is 300 mm, see figure 2.2, ATB-väg (2005).

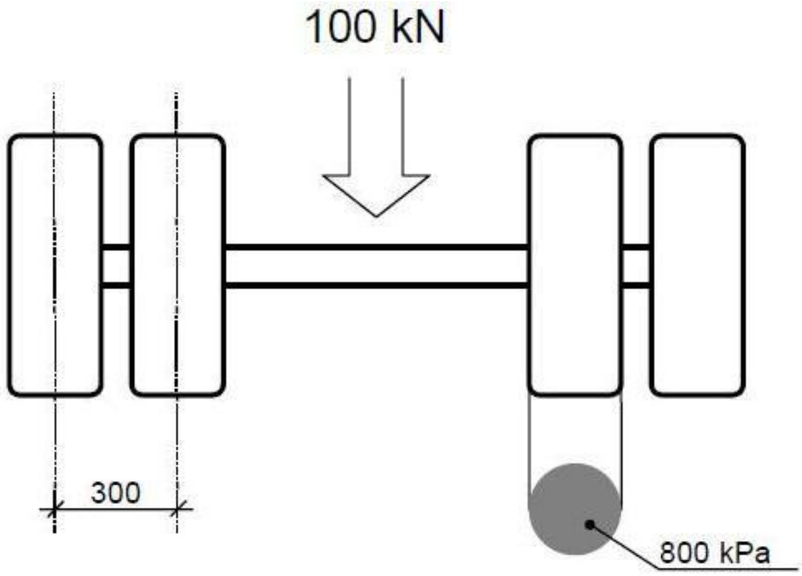


Figure 2-2 Standard Axle (ATB-väg, 2005)

The equivalent standard axle is an important parameter that is necessary to estimate in order to dimension the pavement. This is estimated from the prognosis of traffic during the proposed dimension period for the bituminous base course. The estimation is performed according to equation 2.1 below, ATB-väg (2005).

$$N_{eq} = AADT_L * 3.65 * A * B * \sum_{j=1}^{n} \left(1 + \frac{k}{100}\right)^j =$$

$$= \begin{cases} AADT_L * 3.65 * A * B * \left(1 + \frac{100}{k}\right) * \left(\left(1 + \frac{k}{100}\right)^n - 1\right) & \text{if } k \neq 0 \\ AADT_L * 3.65 * A * B * n & \text{if } k = 0 \end{cases} \quad (2.1)$$

where:

N_{eq} – total equivalent standard axles

A – proportion of heavy vehicles [%]

B – equivalent standard axles per heavy vehicle (0.33 * number of axles)

n – proposed design period in years

j – 1, 2, 3 ... n

k – presumed annual change of heavy traffic [%]

2.3 Stresses and Strains in the Pavement

2.3.1 Critical Stresses and Strains in a Pavement

Stresses and strains are developed in the road pavement as a result of repeated traffic loading. In the design of asphalt pavement, two types of strains are considered to be most critical. One is the horizontal tensile strain that appears at the bottom of the bituminous layer, between the bound and unbound layers when the tensile stress is high. This type of strain causes fatigue cracking at the road surface. The other critical strain is the vertical compressive strain that occurs on top of the subgrade, when the vertical compressive stress is high. If the quality of the subgrade is poor, the stresses that are developed causes permanent deformation or rutting on the road surface, Werkmeister (2003). These two types of strains are illustrated in the figure 2-3 below.

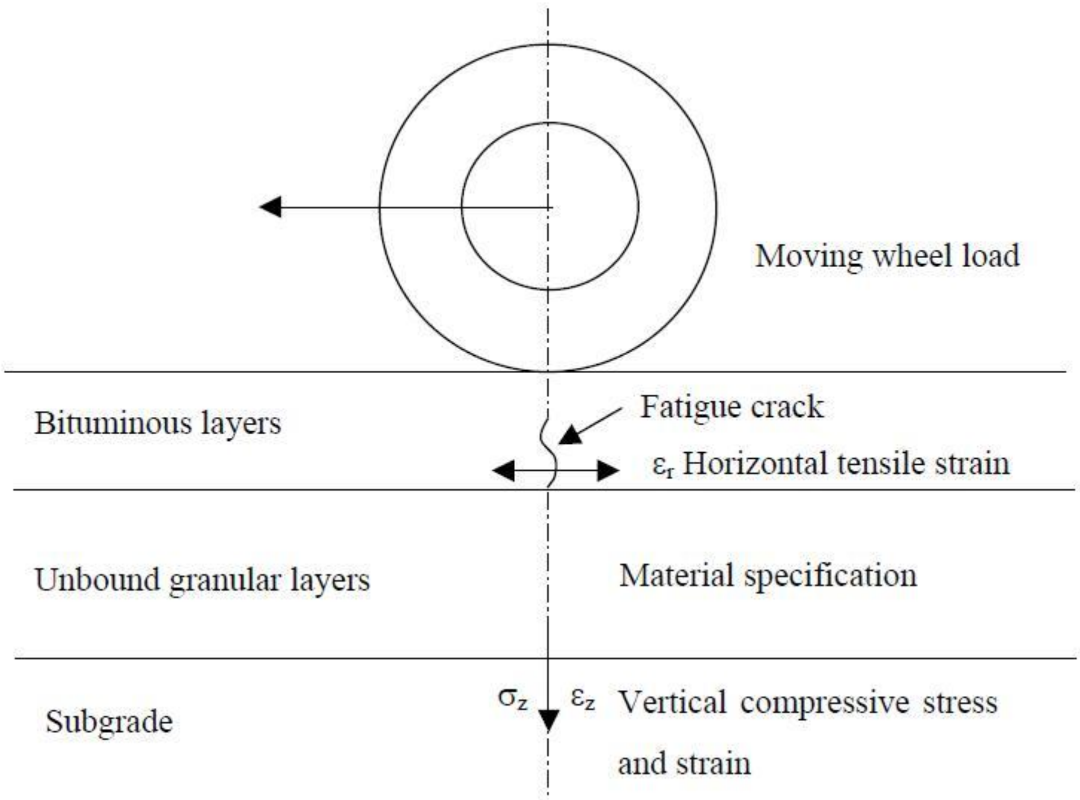


Figure 2-3 Critical stresses and strains in a three layer system (Powell et al, 1984)

2.3.2 Stresses in the Unbound Granular Layers

The stresses that occur at any given point in the unbound granular layers can be defined by its shear and normal components, see figure 2-4. The shear stresses are equaled to zero if the element is rotated to a condition where it is only affected by compression and tensile stresses on the side of the element. At this condition, the only stresses acting on the element are the principal stresses σ1, σ2 and σ3, Lekarp (1997).

Furthermore, the principal stresses σ_1 and σ_2 are the largest and the smallest stresses, respectively. In addition, σ_3 is the horizontal stress, see figure 2-4. The importance of applied stress level is strongly emphasized in the previous studies. The permanent strain is directly connected to the stress ratio consisting of deviator and confining pressure, Lekarp (1997). The deviator and mean stress are defined in equation 2.8 and 2.9 below.

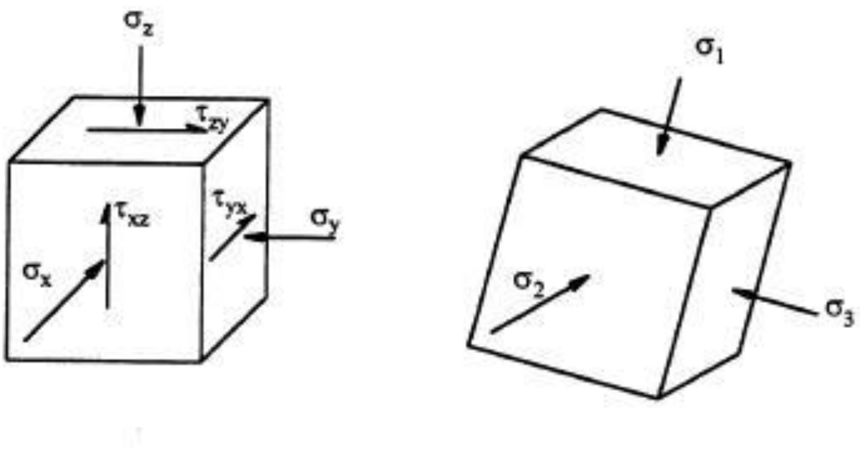


Figure 2-4 Stress components acting on a carved element (Lekarp, 1997)

2.3.3 Rotation of Principal Stresses

The permanent stain development in the unbound granular material is affected by both structural and material related factors. Therefore, it is very complicated to analyse the complex behavior of the unbound layers. Most of the researches carried out in analysis of unbound material are limited and very small knowledge is available about the actual behavior of the material under repeated traffic loading. In the laboratory analysis, triaxial tests are frequently used to simulate the stresses that are conducted to the unbound granular material under repeated vehicular loading. In this testing method, it is only possible to apply stresses in the horizontal and vertical directions. The continuous change in the direction of the principal stresses that is caused by the dynamic loading from rolling wheels is not considered. However, studies executed by Lekarp indicate that the stress reorientation in granular material under traffic loading results in larger permanent deformation than those predicted by repeated triaxial testing. Figure 2-5 below illustrates rotation of the principal stresses as a result of the variable load traffic load, Lekarp (1997).

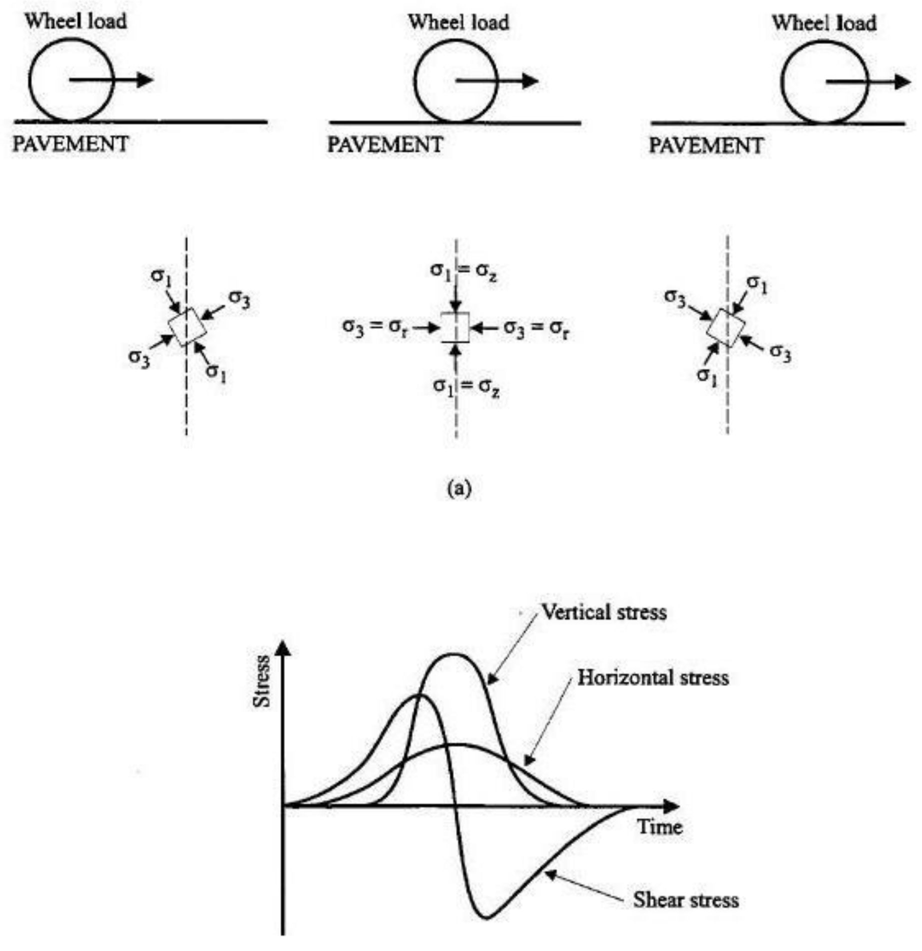


Figure 2-5 Stresses beneath a rolling wheel load (Shaw, 1980)

2.3.4 Shakedown Theory

Under repeated loading condition, the unbound granular material exhibits elastoplastic behaviour in response. Depending on the magnitude and number of load application, some of the deformation is reversible (resilient) and some irreversible (permanent). The development of permanent deformation in the pavement is highly undesirable. A higher stress level leads to increased permanent deformation and eventually results in a failure condition, collapse. For design purposes, the critical stress level between stable and unstable pavement condition has to be determined in order to prevent the uncontrolled permanent deformation. This limit is further described by the "shakedown" concept developed by Werkmeister and is termed the "shakedown limit". If the applied loads from the traffic are lower than the shakedown limit, no accumulation of plastic strain will take place in the material and the response will be completely resilient. On the other hand, if the applied loads exceed shakedown limit the material will develop permanent deformation and gradually deteriorate, Lekarp (1997). The shakedown concept is frequently used for describing the behaviour of the pavement under repeated loading condition. Depending on material response under loading, the shakedown concept is divided in four categories which are described below, Werkmeister (2003).

Phase 0. Purely elastic

Under loading and unloading condition no plastic strain is developed in the material. All deformation is completely recovered and the response is purely elastic.

Phase 1. Elastic shakedown

If the applied loading is slightly increased, after a finite number of stress/strain application the material may develop a plastic response. However, this stress level is less than required for a permanent deformation and results in a purely elastic behaviour. The material is said to have "shaken down" and the response is referred to as "elastic shakedown limit".

Phase 2. Plastic shakedown

The unbound material will develop larger permanent deformation if the stress level is further increased. The aggregate in the pavement develops plastic strain, which is much higher than the strain in phase 1. This behaviour is caused by the number and magnitude of the applied load and is referred to as "plastic shakedown". However, after a certain number of cycles, the material achieves a long-term steady state response in other words no further development of the permanent deformation. The maximum stress level at which this condition is obtained is named plastic shakedown limit.

Phase 3. Incremental collapse

In this phase, the strength capacity of the material is exceeded and the material can no longer withstand the stresses from the load, which eventually results in collapse.

The different phases are summarized and illustrated in the figure 2-6 below.

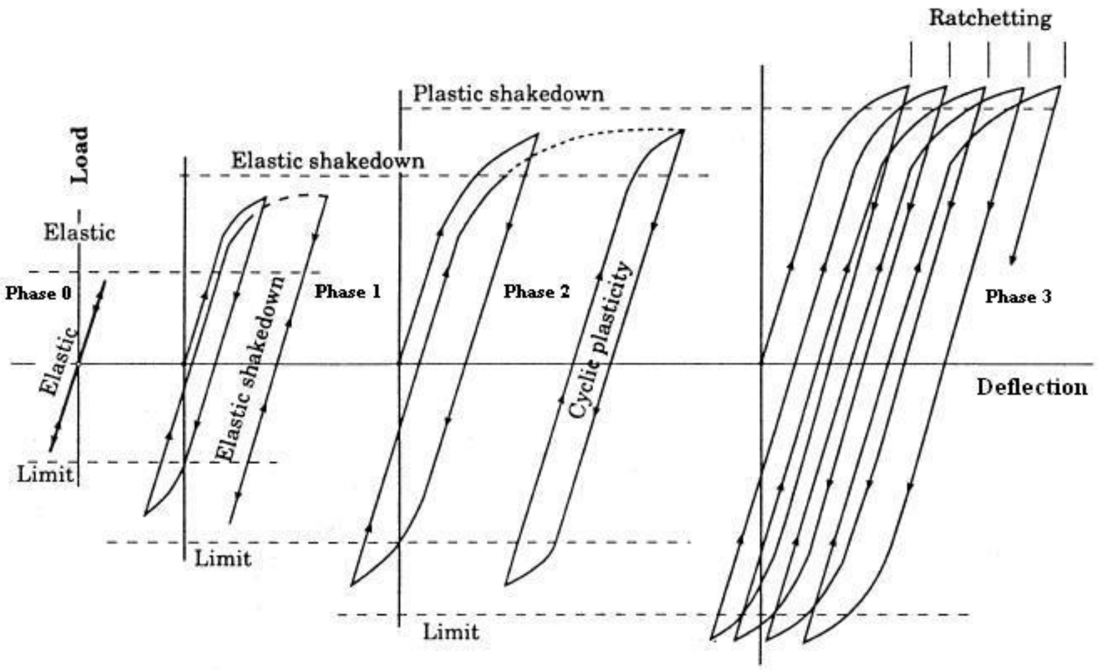


Figure 2-6 Elastic/plastic material behaviour under repeated loading (Johnson 1986)

Furthermore, figure 2-7 clearly illustrate the limit between stable and unstable behaviour of the material.

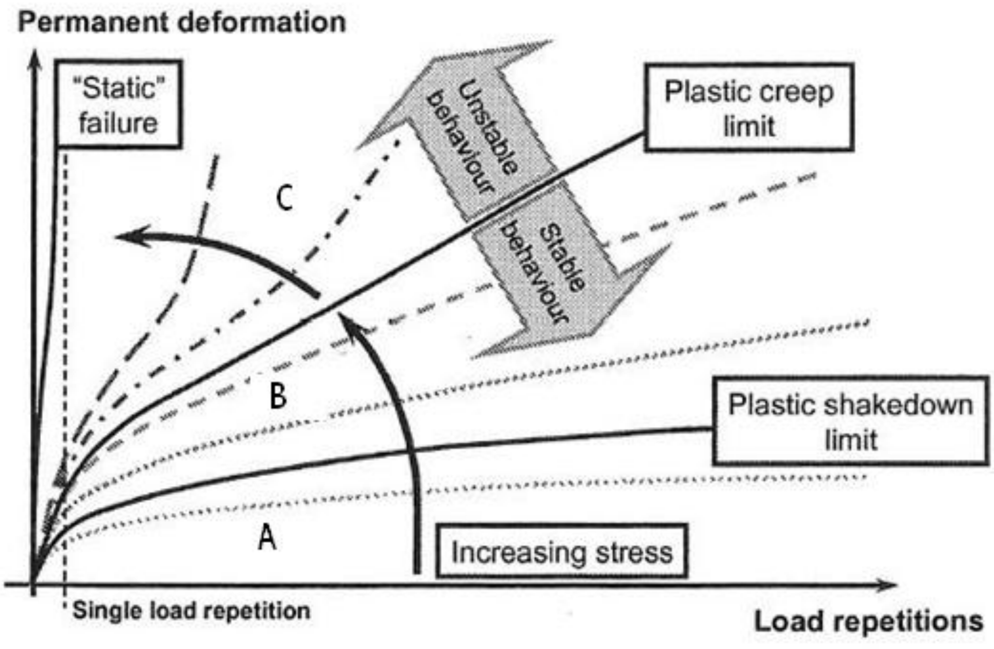


Figure 2-7 Shakedown theory applied to the permanent deformation behaviour of unbound granular material (Theyse 2007)

The observed behaviour that is illustrated in figure 2-6 does not represent the actual behaviour of the unbound granular material properly. Further studies by Werkmeister indicate that pure elastic behaviour in the material (phase 0) does not occur. Therefore, in order to make a proper prediction of the material behaviour, the permanent deformation of the unbound materials is divided into the following phases:

A: Plastic Shakedown

B: Plastic creep

C: Incremental collapse

The different phases are illustrated in figure 2-8 below and also in figure 2-7 above.

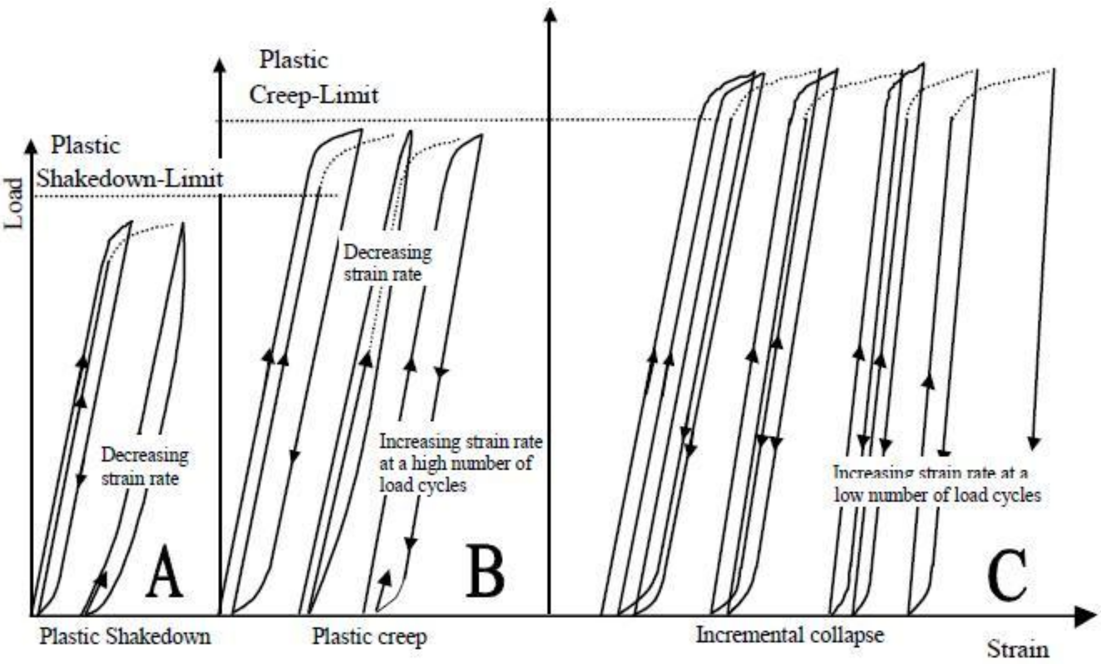


Figure 2-8 Behaviour of unbound granular material under repeated cyclic road (Werkmeister 2003)

Results from the triaxial testing indicates that range C behaviour seems equivalent to phase 3. Furthermore, Range B is initially similar to Range A with a small increase of permanent strain. The three categories are further illustrated in figure 2-7 above. Below the plastic shakedown limit (Range A) accumulation of plastic strain decreases with increasing number of load repetition until finally approaching zero. The material is reaching a stable condition and no further deformation is taking place. Above the plastic creep limit (Range C), the plastic deformation increases with number of loading cycles until the material collapses and the deformation exponentially increases. Furthermore, between these two limits the accumulation of permanent strain attains a stable condition as in Range A. However, creep deformation takes place in the material (Range B) contributing to larger permanent deformation, Korkiala-Tanttu (2009).

Moreover, the shakedown concept is a good method for describing the behaviour of the material and provides a design tool for analysis of unbound pavement bases. Therefore, it is essential to determine the shakedown limit so that the peak stresses from the induced traffic load never exceeds the value that would generate non-stabilizing behaviour. The development of excessive rutting in the granular layers can be prevented if this limit is defined, Lekarp (1997).

2.4 Permanent Deformation (Rutting)

Rutting is an undesired phenomenon in a pavement for several reasons. For the road users, it gives an increase of fuel consumption due to increased friction, increased risk of hydroplaning in wet weather conditions and increased risk of skidding when the water freezes to ice. The rut depth depends on several factors and it can vary along a road, which may cause discomfort to the road users due to unevenness, Kolisoja & Dawson (2006). Figure 2-9 illustrates rutting on a road.

Figure 2-9 Rutting in the unbound layer (Kolisoja & Dawson 2006)

Furthermore, for the road owners, rutting causes economical losses due to higher maintenance cost. According to Swedish norms a new road pavement is dimensioned for 20 years, ATB-väg (2005). However, in reality the road rarely withstand 20 years of traffic without maintenance due to rutting, Huvstig (2010).

Additionally, rutting is caused by several factors and occurs in different layers of the pavement. A rule of thumb is that the wider the ruts are the deeper in the pavement the permanent deformation occurs. The factors contributing to permanent deformation are thoroughly explained in the following chapters.

2.5 Factors Affecting Permanent Strain Response in Unbound Layers

According to previous studies the permanent deformation i.e. plastic behaviour in the granular material is affected by several factors. These factors can be both material and structural related. In the following chapter factors affecting permanent deformation are described in particular and a division has been made between material and structural factors.

2.5.1 Material Factors Affecting Permanent Strain Response

As previously mentioned, there are several of material factors that contribute to permanent deformation. Due to this fact it is extremely complex to make a prediction of rutting development. However, some of the most important material factors contributing to the development of rutting are summarized by Korkiala-Tanttu (2009) as following factors: void ratio, effective shear and mean stresses, saturation degree, grain size distribution, the level of the deformation, maximum grain size, stress history, mineralogy of the grains, secondary time factors, the structure of the soil sample and temperature. In the following subchapters some of the factors are described.

2.5.1.1 Effect of Density

One of the most important factors for the long-term behaviour of granular materials is the degree of compaction of the material, Lekarp (1997). Results from laboratory tests and studies of several test roads indicate that the resistance to permanent deformation increases with an increased compaction, see figure 2-10. Studies performed by The Road Structure Research Programme (TPPT) indicates that the rut depth is almost twice as deep for a looser pavement compared to a denser one, Korkiala-Tanttu (2009). Furthermore, the studies also indicate that the degree of compaction is particularly larger for angular aggregates (crushed coarse materials) than for natural material since the rounded natural aggregates are initially of higher relative density, Lekarp (1997). Moreover, due to lack of sustainable measuring methods, less information about the initial degree of compaction with new structure and the change caused by seasonal conditions are known, Korkiala-Tanttu (2009).

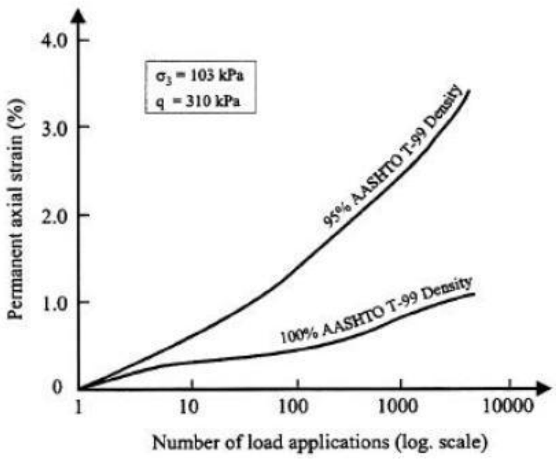


Figure 2-10 Effect of density on permanent strain (Barksdale 1972)

2.5.1.2 Effect of Stress History

The effect of stress history, i.e. the order of load application, has a significant impact on the development of permanent deformation. The permanent deformation is considerably smaller if the stress level from the load is gradually increasing compared to an instant application of the highest stress level, Lekarp (1997). For each load repetition, the underlying material gradually stiffens and the particles rearrange to a denser condition. It is proved that the development of permanent deformation is much faster in crushed gravel compared to natural stones after repeated load cycles, see figure 2-11.

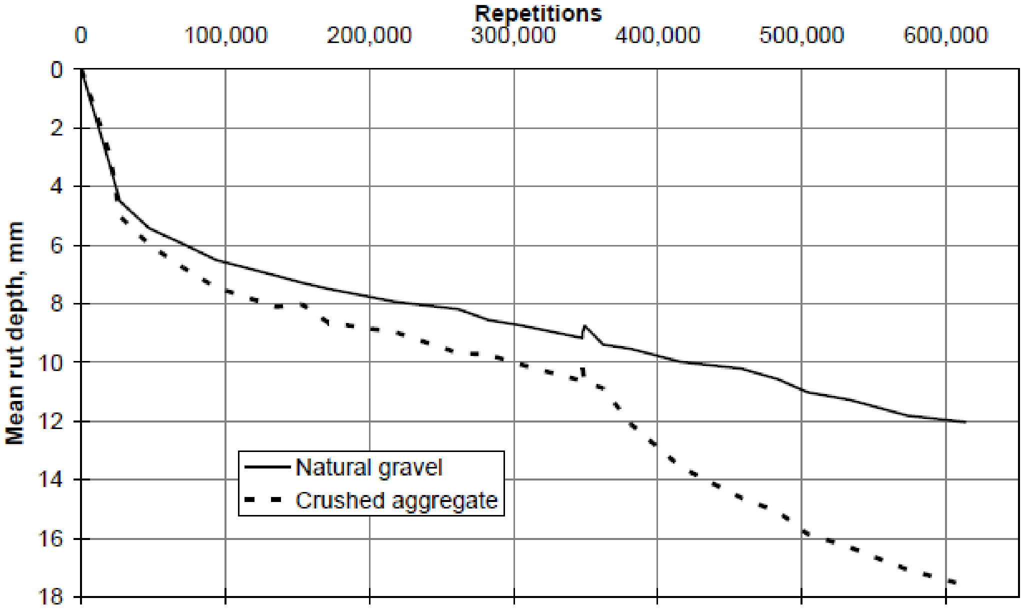


Figure 2-11 Rutting of the two material types (Odermatt et al 2004)

2.5.1.3 Effect of Grading, Fines Content and Aggregate Type

Other factors affecting the permanent deformation are the grain size distribution of the material. Materials that follow the Fuller curve, which is an optimal grain size distribution curve developed by Fuller (1907), contribute to a lower permanent deformation, Korkiala-Tanttu (2009). In addition, previous studies confirm that the resistance to permanent deformation in granular materials is reduced as the amount of fines increases, Lekarp (1997). An increase of fines content reduces the contact surface between the larger grains which in its turn leads to bad distribution of the load. Moreover, a high content of fines also makes the material more sensitive to higher water content, Korkiala-Tanttu (2009). On the other hand, the void between the particles tends to be larger if material with larger aggregates is used in the unbound layer. The water can easily drain and hence the resistance to rut increases, Kolisoja & Dawson (2004).

Furthermore, the mineralogy of the material that is used has also a major impact on the development of permanent deformation particularly in the unbound layers. The permanent deformation decreases if material with harder mineral composition is used compared to materials with softer mineralogy, Korkiala-Tanttu (2009).

2.5.1.4 Effect of Grain Shape and Surface Roughness of Grain

The grain shape and surface roughness of the grains are of great importance for the degree of compaction. Materials such as crushed gravel and other materials with similar surface characteristics need more compaction effort in comparison to cube-shaped, rounded aggregates, for instance natural river gravel. Moreover, the strain of the grains depends on the strength of the mineralogy of the grains. Flaked shaped crushed aggregates are more sensitive to rutting compared to other type of aggregates since the strength properties of the material is very poor, Lekarp (1997). Other factors that influence the quality of the aggregates are the rock source material and also the crushing process, Korkiala-Tanttu (2009).

2.5.2 Structural Factors Affecting Permanent Strain Response

The structural factors that contributes to the development of permanent deformation is summarized by Korkiala-Tanttu (2009) as follows; number of load repetitions, geometry of the pavement, initial state of the pavement layers, temperature and moisture conditions, loading factors and seasonal changes including degree of saturation. Below, a short description of the most important factors can be found.

2.5.2.1 Effect of Number of Load Cycles

The development of permanent deformation is a gradual process and every load repetition contributes to an accumulation of strain and by that an increase of the total rutting. Therefore, studies of the effect of number of load repetition are substantial for the analysis of the long-term behaviour of the material, Lekarp (1997). Previous studies have manifested that the permanent deformation increases with increased load repetition until a point where the deformation starts to decline. This stabilization of the material is only achieved if the applied stress level is low. However, if the load application is relatively large a sudden increase of the deformation can appear in the material. The particles in the material reach a point where it no longer can withstand the stresses from the load and either breaks or a rearrangement of the particles takes place, which in its turn leads to a sudden collapse, Huvstig (2010).

2.5.2.2 Effect of Stress

According to available literature, the development of permanent deformation strongly depends on the stress level. Permanent deformation increase with rising deviator stress and decreasing confining pressure, Werkmeister (2003). Furthermore, the stress ratio i.e. the relation between deviator stress and confining stress is revealed to have a great impact on the permanent deformation, see figure 2-12.

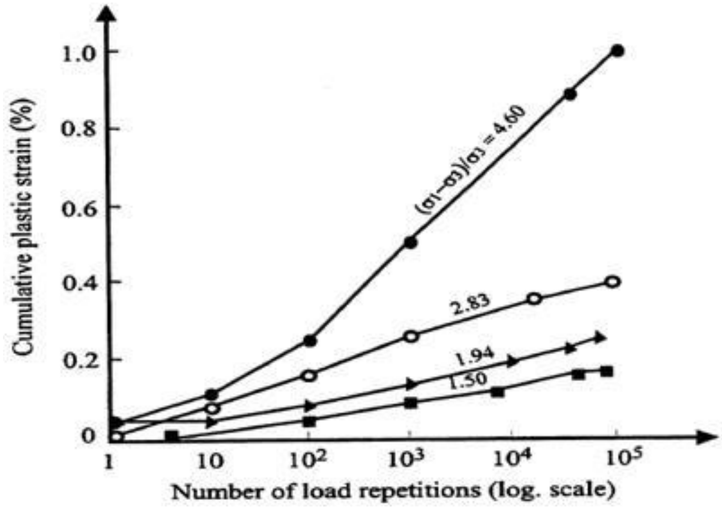


Figure 2-12 Influence of stress ratio on permanent strain (Barksdale 1972)

2.5.2.3 Effect of Moisture Content

The existent of water in the material can both have positive and negative effect on the permanent deformation. An adequate amount of water has positive effect on the strength and the stress and strain behaviour of the unbound granular material, Werkmeister (2003). On the other hand, high water content causes excessive pore pressure, which in it turns leads to a reduction in stiffness and hence increased permanent deformation, see figure 2-13.

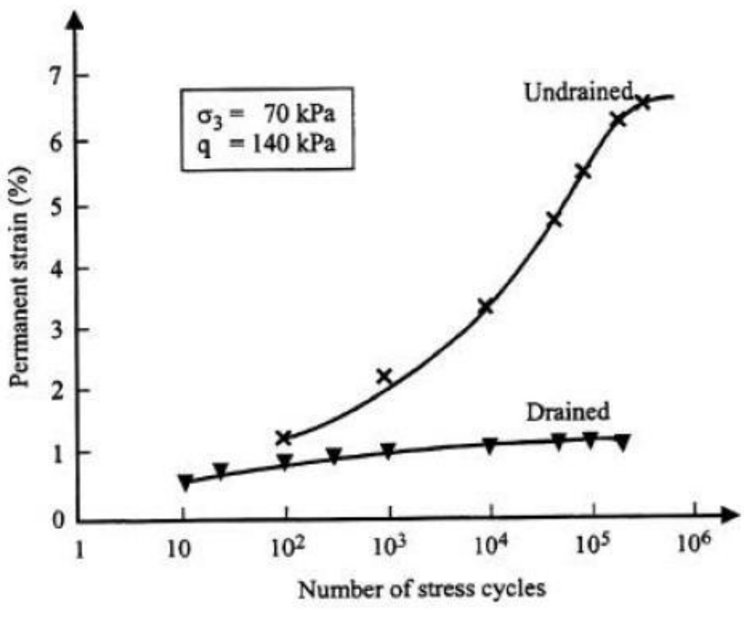


Figure 2-13 Influence of drainage on permanent deformation development (Dawson 1990)

A series of studies indicates that a rise in water content results in an increase in permanent deformation. The tests performed by Holubec (1969) shows that an increase of the water content from 3.1 per cent to 5.7 per cent results in an increase of the total axial strain by 300 per cent. Under repeated stresses from traffic load, the water between the particles become pressurised and the pore pressure counteract the stresses, which is pushing the particles together, see figure 2-14. Due to the excessive water content, the friction and contact pressure between the particles is decreased, Kolisoja & Dawson (2004). Additionally, the studies also reveals that a high degree of saturation and low permeability leads to high pore pressure and eventually decreased deformation resistance, Lekarp (1997).

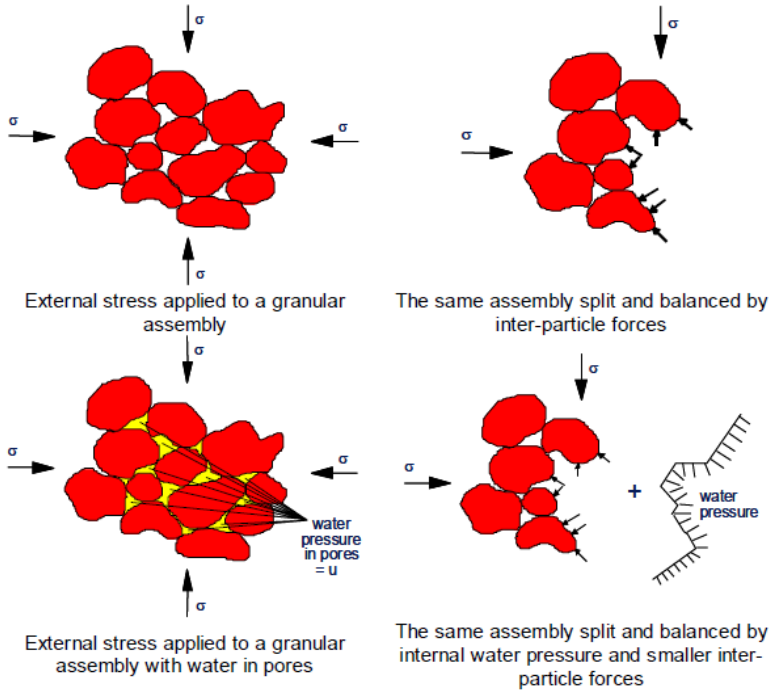


Figure 2-14 The impact of internal water content in the unbound granular material (Kolisoja & Dawson 2004)

2.6 Additional Parameters Contributing to Permanent Deformation in Road Pavement

As indicated in previous chapters, the permanent deformation is caused and worsened by several factors in the unbound layers. There are parameters that contribute to the effect of the above mentioned factors. These parameters speed up the rutting development in the roads by worsening the resistance against the stresses caused by the traffic load. Some of these are cracking, subgrade type and its E-modulus, water/moisture content in the subgrade and direct sunlight. These are thoroughly described in the following subchapters.

2.6.1 Cracking

Cracking appears in various forms and its causes can be identified. Cracks correlated to fatigue of the asphalt layer due to repeated traffic load appear in the wheel path and has the pattern of an alligator skin. The cracking begins at the bottom of the asphalt surface, where tensile stress and strain are highest under a wheel load. Furthermore, longitudinal cracks away from the wheel path occur along the joint between two lanes due to poor joint compaction during construction. It may also occur in the middle of the lane, caused by thermal stresses or settlements in the subgrade. Weaker subgrade causes cracks at the edge of a driving lane due to lack of confinement as well, Papagiannakis & Masad (2007).

Moreover, transverse cracking is non-load related and is caused by shrinkage of the asphalt surface due to low temperature or to asphalt hardening or from reflective cracks caused by cracks beneath the asphalt surface, Huang (2004). The shapes of cracks are shown in figure 2-15 below.

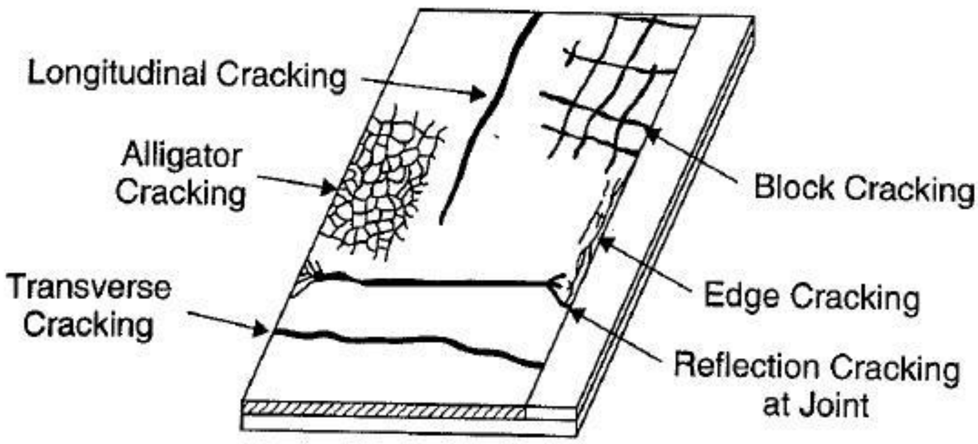


Figure 2-15 Type of cracks occurring on a road surface (Papagiannakis & Masad 2007)

The drawback with cracks is the reduction of the E-modulus of the asphalt layer, which in turn decreases the resistance against stresses caused by the traffic load. This induces higher stresses conducted to the weaker unbound layers, Huvstig (2010).

Another drawback with cracks is water penetrating through the cracks into the sub layers and accumulating in a buried rut in the subgrade and reduces the bearing capacity of the unbound layers. This causes a faster deterioration of the pavement, Kolisoja & Dawson (2006).

On the whole, it is ascertained that cracks have negative influence on the permanent deformation and should be repaired instantaneously. An indication of the extension of the cracks is the crack index, which is described in chapter 2.10.7.

2.6.2 Embankment / Cut

The effect of whether the road is constructed on embankment or cut depends on the alteration of the groundwater conditions. The groundwater level is not altered when the road is constructed on an embankment. However, it may be altered when it is constructed on a cut. When the road is constructed on a cut the groundwater table will get closer to the surface since the ground will be excavated. This will increase the water/moisture content in the unbound layers, which leads to decreased E-modulus in some instances, Huvstig (2010). The outcome was explained in chapter 2.5.2.3 above. Soil cuts are more sensitive to develop permanent deformation compared to rock cuts since soil contains smaller fractions, which retain water. This may cause problems during winter, which is explained in chapter 2.6.5 below.

2.6.3 Subgrade Type and E-modulus

Subgrade consisting of material with low E-modulus will develop higher tension in the intermediate layer between subgrade and subbase. This in its turn will result in lower mean stress and the risk of approaching the failure limit increases. During loading condition, the strength capacity of the subgrade material will be exceeded and the material eventually collapses. One explanation to this phenomenon is the soft subgrade material bends down during loading so that the particle in the bottom of the subbase separates from each other and starts to move in sideways. Eventually, this will lead to rearrangement of the particles and increased compaction of the material, Huvstig (2009). The process is illustrated in the figure 2-16 and figure 2-17 below.

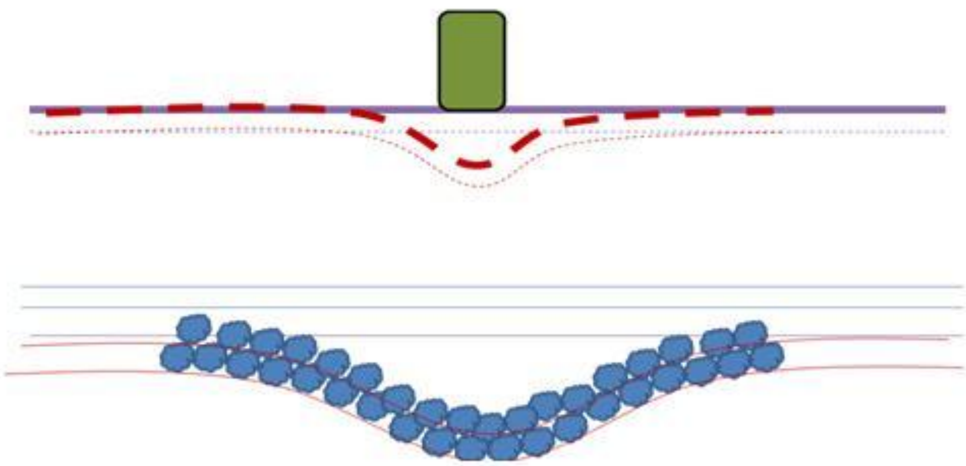


Figure 2-16 Particle behaviour in the unbound granular material under loading condition (Huvstig 2009)

This type of compaction is referred to as pre-compaction and can last in many years. Previous studies indicate that roads constructed on clay develop much larger permanent deformation compared to roads constructed on other type of subgrade material e.g. frictional material, Huvstig (2009).

Subgrade consisting of material with higher E-modulus prevents the rotation of the particles in the unbound layers and hence no particle separation takes place. Moreover, possible soil improvement methods are further described in chapter 6.

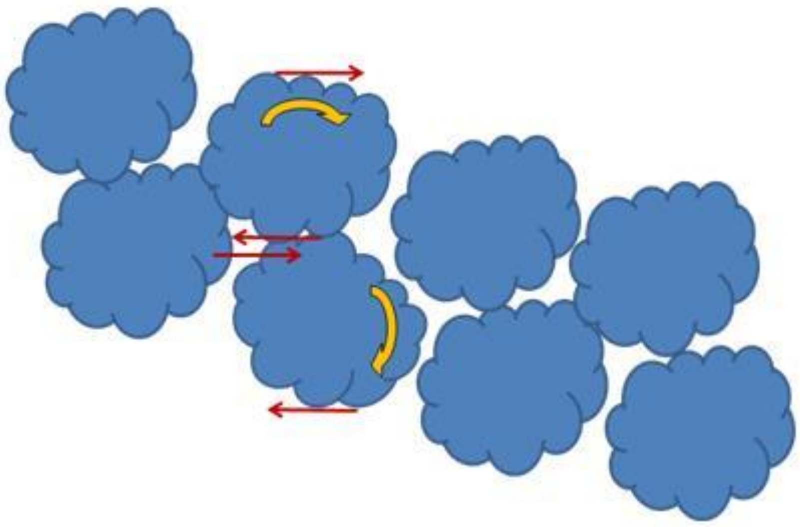


Figure 2-17 Particle rearrangement in unbound granular material on weak subgrade under loading (Huvstig 2009)

2.6.4 Water/moisture Content in Subgrade

The effect of water/moisture content in the subgrade material depends on the amount of fines in the subgrade material. High content of fines will retain the water and result in saturation of the subgrade material, which in its turn leads to larger permanent deformation, Huvstig (2010). Other effects of the water/moisture content in the subgrade is the same as for the unbound layer as explained in chapter 2.5.2.3.

2.6.5 Frozen Ground

In regions affected by frozen ground during winter periods, frost heave is occurring in connection with water or moisture. This occurs commonly in the subgrade layer. However, results from recent reports indicate that it may occur in the granular material due to absorbed water. The water may emerge in this layer due to penetration through the cracks in the surface layer or due to high groundwater level if the road is constructed on a soil cut. Additionally, if the content of fines is high in the unbound layers more water will be retained. Subsequently, during winter periods the water in the unbound layer will freeze to ice and expand. This procedure is called frost heave. Frost heave causes loosening of the unbound layers and when the frozen materials thaw during the spring compaction may be inevitable. Due to natural variation in the subgrade, the frost heave will be uneven along the pavement and result in variations in rutting as well, Kolisoja & Dawson (2006).

2.6.6 Direct sunlight

The direct sunlight makes the asphalt temperature rise, especially during summer time. This leads to a reduction of the asphalt E-modulus. Reduction of the asphalt E-modulus in its turn results in higher stresses conducted to the unbound layers. It may also lead to deformation in the asphalt layer in connection with heavy loading, Huvstig (2010).

Furthermore, direct sunlight is an element that cannot be avoided by measures. However, in some roads the effect of direct sunlight may be minor due to shading caused by surrounding terrain, which may consist of forests with tall trees or high hills.

2.6.7 Studded Tyres

Studded tyres cause abrasion of the wearing course, which eventually results in rutting. The studs wear the bitumen in the wearing course and thereby the contact between aggregates in the wearing course and the tyres increase. After repeated traffic passes the aggregates in the asphalt layer abrades and this will develop rutting in the wearing course, Huvstig (2010).

According to previous studies of a national road (Rv40), the wearing from studded tyres stood for 60 per cent of the total permanent deformation of the pavement, Nilsson & Huvstig (2009).

2.7 Methods used for Condition Assessment of Roads

There are several methods used in assessing the road condition. However, in Sweden the most common methods are Road Surface Tester and Falling Weight Deflectometer. The procedures behind these methods are described in following subchapters.

2.7.1 Road Surface Tester (RST)

The RST car is a multi-functional vehicle that is used for measuring several variables. It measures the cross profile, cross fall, longitudinal profile, texture and hilliness. The road environment can be recorded as well, Göransson (2010). In this report only the cross profile is of interest and it is measured two times with 17 lasers, where the rutting is calculated with data from 11 and 17 lasers, and two times with 19 lasers, where the rutting is calculated with data from 15 and 19 lasers. The laser placement is illustrated in figure 2-18 below. The registration is done every 10 centimetres in travelling direction and the permanent deformation is calculated according to the "thread principle (tråd principen)". The mean value is obtained for each section and driving direction, Göransson (2009).

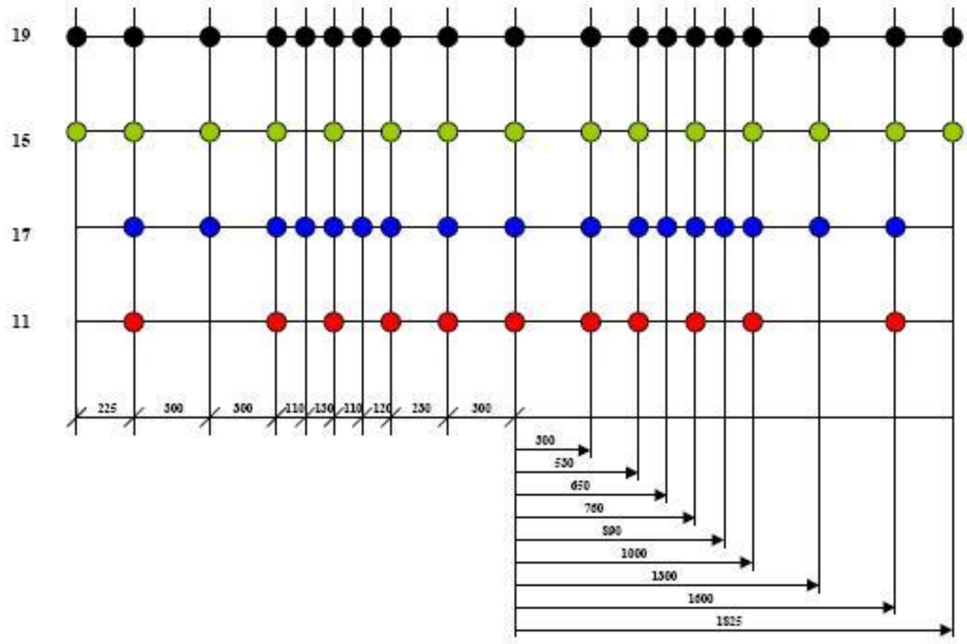


Figure 2-18 Placement for different lasers (Göransson, 2010)

Furthermore, there are some drawbacks when measuring the permanent deformation with RST. As mentioned earlier, the rut depth shall commonly increase for each year since there is wearing caused by the studded tyres. However in some measurements made in the LTPP-roads there is a variation of the rutting development, which is both positive and negative, like a sinus curve. According to Huvstig (2010) this is caused by performing the measurements on different tracks in each year and due to broader measurements when using different number of lasers in each measurement occasion. For instance, the obtained cross profile is broader when using 19 lasers compared to 11 lasers. Another drawback is that the rutting is 20 to 40 per cent larger when the measurements are performed with a beam compared to lasers, Huvstig (2010).

2.7.2 Falling Weight Deflectometer (FWD)

The FWD is a non-destructive mechanical device used for evaluating the pavement structural condition where the bearing capacity and the expected life span are determined. From the device a weight is dropped down on a damped spring system at different heights and the deflection at various points is measured by several geophones that are placed on different distances from the point of impact, see figure 2-19. The distance from the plate loading determines which layer that is affected by the falling weight, the farther away from the loading centre the less deflection will be on the surface layers. At the loading centre, the deflection is affected by the entire pavement while farther away from the load only the subbase and subgrade material are affected, Vägverket metodbeskrivning 112 (VVMB:112) (1998).

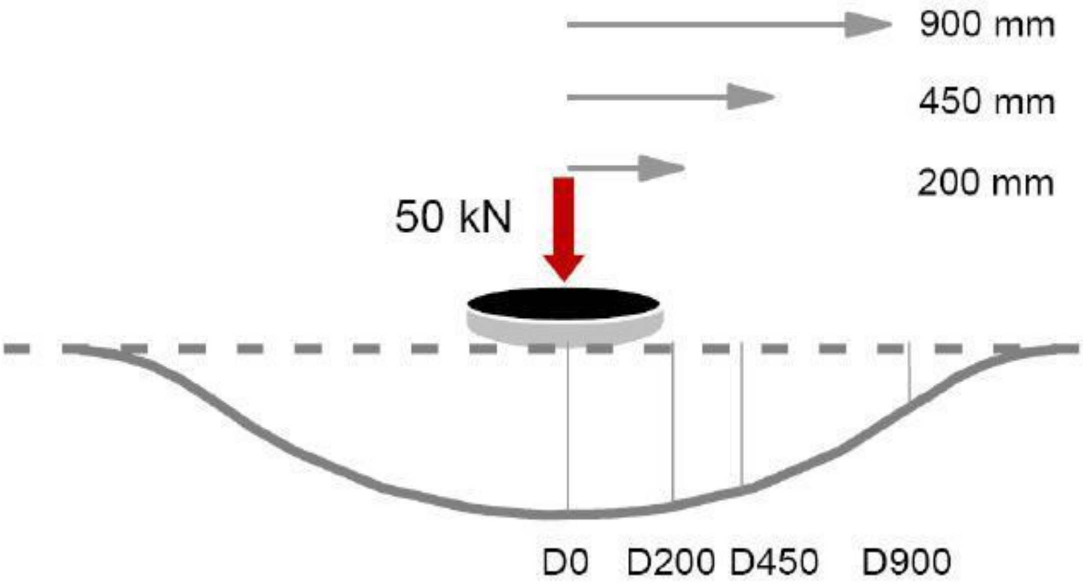


Figure 2-19 Deflections measured at different distances (VVMB 112:1998)

The obtained measurements from the geophones can later be used to determine the E-modulus of the different pavement layers through a process called back calculation. The calculation can only be performed if the sub layers are assumed to be homogeneous, isotropic and linear elastic. The deflection that is developed is directly connected to the stiffness of the underlying layers and the thickness of it. The calculations are performed by equation 2.2, 2.3 and 2.4 for surface modulus, medium modulus and subgrade modulus, respectively, VVMB 112:1998.

$$E_0 = \frac{1000 * f * (1 - v^2) * \sigma_0 * a}{D_0} \quad (2.2)$$

where:

E_0 – surface linear elastic modulus [MPa]

D_0 – deflection right under the load [µm]

f – 2 (for segmented plate loading)

σ_0 – contact pressure under the loading plate [kN]

v – lateral contraction [-]

a – radius of the loading plate [mm]

$$E_v = \frac{1000*\sigma_0*a^2*(1-v^2)}{D_r*r} \qquad (2.3)$$

where:

E_v − Average modulus on equivalent depth [MPa]

r − Distance between the plate loading and measurement devices (geophones) [mm]

D_r − Deflection with the distance r from the plate loading [µm]

$$E_u = \frac{52000}{D_{900}^{1.5}} \qquad (2.4)$$

where:

E_u − Subgrade E-modulus [MPa]

D_{900} − Deflection 900mm from centre of plate loading [µm]

In this thesis equation 2.4 has formed the basis for calculation of the subgrade E-modulus.

2.8 Method used for Material Property Assessment

In order to predict the actual behaviour of the unbound granular material under loading conditions, several testing methods are available. The idea behind these existing testing devices is to simulate the stresses occurring in the field beneath the traffic load. In this report only repeated load triaxial test has been considered.

Repeated Load Triaxial Test (RLT)

The RLT is the optimal method that is used to simulate the natural stresses occurring in the unbound granular layers. This testing device is used in laboratory, where it allows testing of material of different material properties, moisture content, density and other important parameters that are necessary for the creation of optimal testing conditions. The procedure of RLT is to apply cyclic stresses, pulses similar to those occurring in the actual pavement, to a sample of 150 mm in diameter and 300 mm height. The stresses applied are both vertical stresses, deviator stress at the top and bottom of the sample, and radial stresses applied through confining fluid or air pressure. Subsequently, the axial and radial strain that is developed due to the applied stresses is measured and the nonlinear stress-strain characteristic of granular materials is determined. Furthermore, the obtained result can be used to determine the permanent deformation in the tested sample. The RLT is very advantageous since it is inexpensive and time saving, Lekarp (1997) and Werkmeister (2003). In figure 2-20 below, a triaxial device is illustrated.

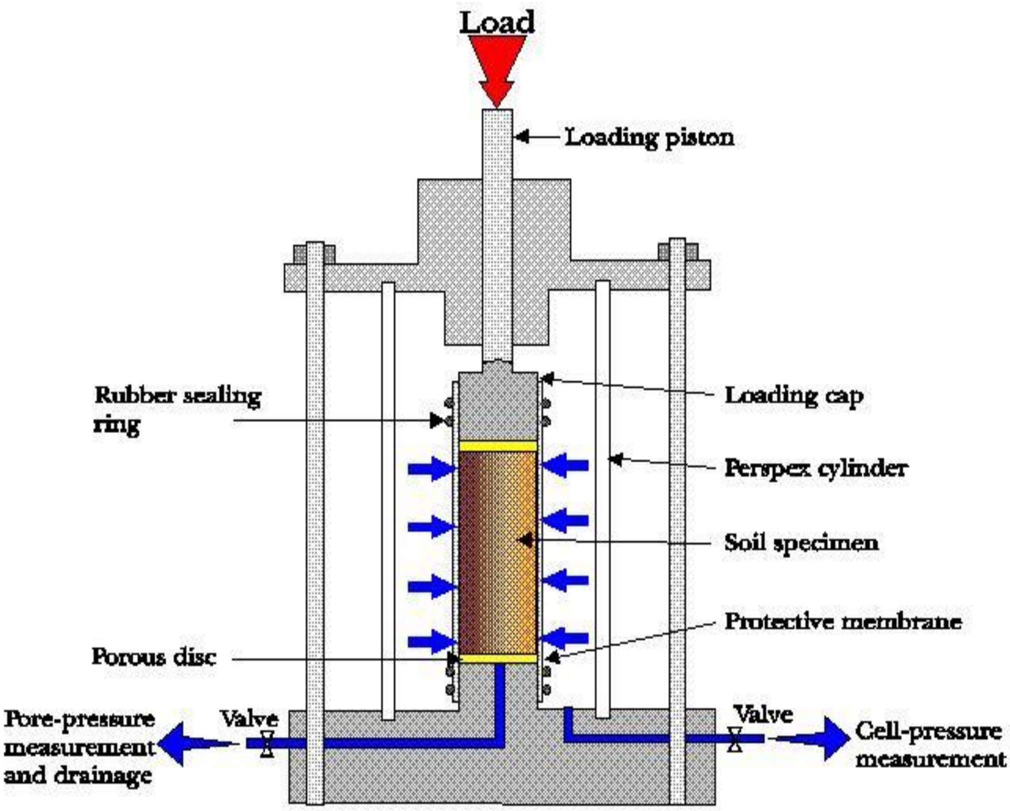


Figure 2-20 Triaxial testing apparatus (University of Bristol 2010)

2.9 Existing Rutting Prediction Models

There are numerous computer models for modelling the permanent deformation. The aim of these models is to make a good prediction of the actual rut development. In this report only VägFEM has been used. Following subchapters describes VägFEM and the models included in the VägFEM software.

VägFEM

The models available today for calculating permanent deformation require that the E-modulus for the different pavement layers is constant. The fact is that this is not the absolute truth since the stress and strain relationship in the unbound and subgrade material is nonlinear. Therefore, a better representative model called VägFEM was developed to increase the understanding of how the material in the different layer behaves in reality, Nilsson & Huvstig (2009).

VägFEM is 3D Finite Element Model that simulates the real geometry of the road and takes into account the actual material properties in different layers in the road pavement. This pavement design program is developed from a cooperation research made by the Nordic countries (Sweden, Norway, Denmark and Iceland). The program uses two material models to describe the mechanical behaviour, one linear elastic or nonlinear elastic modulus for the unbound materials and one linear elastic resilient modulus for bituminous bound material, Korkiala-Tanttu (2009). Before the calculations can be performed, parameters like number of load repetition, placement of the load, tyre pressure, road geometry, type of material used and its layer thickness and E-modulus has to be defined, see appendix III. Furthermore, the type of model, linear elastic or nonlinear elastic that will be used for different layers has to be defined as well. After specifying these parameters the input file is send to an ABAQUS server, where the calculations are initiated. The results from the calculations are sent back in a PDF file and an extensive Excel file with all information about strains and stresses in the different pavement layers. Subsequently, the results can be used in a separate excel program for prediction of the permanent deformations. The excel program integrate the deformation in each layer over the layer thickness to estimate a total layer deformation. There are a number of available material models in the Excel program that is used to evaluate road permanent deformation, Nilsson & Huvstig (2009). The models are described in the following subchapters.

NCHRP 1-37A

NCHRP 1-37A is a mechanistic-empirical model used in pavement modelling for prediction of permanent deformation. The software used in this model is called M-E-PDG (Mechanistic-Empirical Pavement Design Guide) and was developed by the American Association of State Highways and Transportations Officials (AASHTO). The expression used in this model is given in equation 2.5 and 2.6 below, which estimates the permanent strain in the asphalt layer and in the subbase/subgrade, respectively.

$$\varepsilon_p = \varepsilon_r * a_1 * N^{a_2} * T^{a_1} \quad (2.5)$$

where:

ε_p – permanent strain in asphalt layer

ε_r – resilient strain

T – temperature

a_i – regression coefficients

N – number of load pulses

$$\varepsilon_p = \left(\frac{\varepsilon_0}{\varepsilon_r}\right) * e^{-\left(\frac{p}{N}\right)^{\beta}} \quad (2.6)$$

where:

ε_p – permanent strain in subbase/granular and subgrade

ε_0, ε_r, ρ and β are material parameters that are evaluated by using different formulas depending on water content.

Gidel Model

The Gidel model is a superposition method developed at LCPC (Laboratoire Cental des Ponts et Chaussées) France and is used for analysis and prediction of permanent deformation in unbound granular material. This empirical model describes the variation of permanent strain as a function of number of load cycle and maximum loaded stress, Hornych (2001). The Gidel model consists of following equation:

$$\varepsilon_1^p(N) \times (10^{-4}) = \varepsilon_{10}^p \times \left(1 - \left(\frac{N}{N_0}\right)^{-B}\right) \times \left(\frac{L_{max}}{p_a}\right)^n \times \frac{p_{max}}{m * p_{max} + s - q_{max}} \quad (2.7)$$

where:

ε_1^p – vertical permanent strain

p_a – 100 [kPa]

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

N – number of load pulses

q_{max} – maximal cyclic stresses [kPa]

p_{max} – maximal cyclic stresses [kPa]

ε_1^p, m, s, n – model parameters

N_0 – reference number of load pulses (100 in this project)

In order to make a complete analysis the mean stress p and the deviatory stress q must be defined, see equation 2.8 and 2.9. Additionally, the model parameters m and s represent the failure lines in the p/q plane and is determined by the triaxial tests. These are geotechnical parameters and stands for the internal friction and the internal cohesion in the material. If the internal friction angel is equaled to 45 degrees, which is the case for crushed aggregates with good quality, then m will be equal to 2.1.

Moreover, several test results indicate that the internal friction (s) is between 20-30 kPa, Huvstig (2010).

Mean stress: $p = (\sigma_1+2\sigma_3)/3$ (2.8)

Deviator stress: $q = \sigma_1-\sigma_3$ (2.9)

Dresden model

The plastic Dresden model is developed at Dresden Technical University by Sabine Werkmeister and is based on the existing Huurman model. Instead of using the failure ratio $\sigma_1/\sigma_{1,f}$ Werkmeister has developed a stress dependency of parameters to principal stresses σ1 and σ3 Korkiala-Tanttu (2009). The Dresden model consists of following equation:

$$\varepsilon_p\,(N) = A \times \left(\frac{N}{1000}\right)^B + C \times \left(e^{D\frac{N}{1000}} - 1\right) \tag{2.10}$$

where:

$\varepsilon_p\,(N)$ – permanent strain

N – the number of load cycles

A, B, C, D – model parameters

The developed empirical model is a function of number of load cycles and the main stresses (σ_1 and σ_3). The first part of the model describes a linear increase of the permanent strain as a function of number of load cycles when plotting the function on a log (ε_p) – log(N), see the green curve illustrated in figure 2-21. During this condition the material behaves stable and can be referred to as range A. In the case of stable behavior, parameters C and D is equal to zero. However, at higher stress level the first part in the formula cannot describe the unstable material behavior alone. Therefore, a completing term is added in the formula to obtain a more realistic description of the unstable condition that occurs during higher stress level. The development of permanent deformation at this stage is rather exponential than linear see figure 2-21.

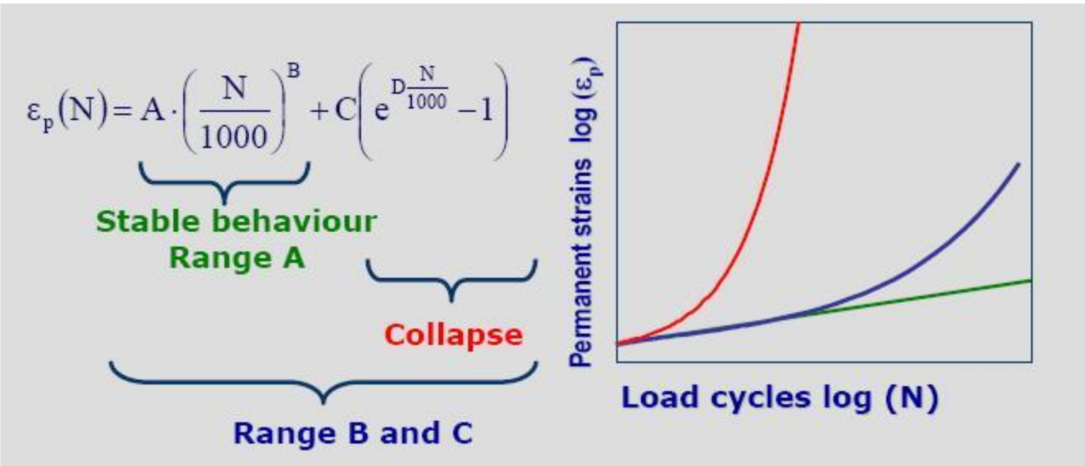


Figure 2-21 Permanent behaviour in different ranges (Huvstig 2010)

The material parameters used in the model is described in following equation.

$$A = a_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{a_2} \; ; \; B = b_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{b_2}$$

$$C = c_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{c_2} \; ; \; D = d_1 * \left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{d_2} \tag{2.11}$$

where:

a_1, c_1 − model parameter [%]

$a_2, b_1, b_2, c_2, d_1, d_2$ − model parameter [−]

σ_1 − major principal stress [kPa]

$\sigma_{1,f}$ − major failure principal stress [kPa]

2.10 LTPP-database

LTPP stands for Long Term Pavement Performance, which is a database built up of high performance data describing the current state of the road, its strength or nominal construction, traffic load and prevailing climate for various test roads scattered around in Sweden. VTI, the National Road and Transport Research Institute, has developed this database commissioned by Vägverket, National Road Administration in Sweden. The collection of data started in 1984, however it was compiled into a database in 2002 and available at the homepage of Vägverket. The database is annually updated with new data, Göransson (2009).

Purpose of LTPP-database

The primary purpose of this research project is to develop a state changing and degradation model by collecting condition data for various LTPP-roads. This developed model will later be used in maintenance planning by predicting the change of the condition in time and determining the most appropriate type of maintenance and when it should be performed, Göransson (2009).

Type of Investigated Roads

The type of road pavement investigated in this database is either GBÖ or BBÖ. Normally, an object consists of 8-13 sections, where each section is 100 meters and the investigation comprises the lanes in both directions. However, only one direction is considered in highways. The reason to why a relatively large number of sections are chosen in each object is to monitor the change of state of a single road pavement under various surrounding conditions such as embankment/cut, forest/open terrain and so on, Göransson (2009).

Measurement of Traffic Load

Data on number of traffic, type of vehicles and number of standard axles are gathered from measurements performed by Vägverket or by VTI. The measurements are normally performed every fourth year on each object. Most of the objects have $AADT_{tot}$ less than 6000 since there is a need of maintenance every 3-5 years in the heavy trafficked road due to wearing caused by studded tyres. Vehicles that have a total weight exceeding 3.5 ton are considered as heavy vehicles. The proportion of heavy vehicles is given in percentage and is usually between 10-14 per cent of the total traffic on most of the objects, Göransson (2009). The number of heavy vehicles times a factor of 0.33 in its turn gives the standard axles, N100 that passes the road per direction and day, Göransson (2010). The number of standard axles is calculated according to equation 2.1 in chapter 2.2.

Measurement of Rutting

Measurements on permanent deformation are performed with RST, Road Surface Tester and by ocular investigation every second year, normally during the summer period. As mentioned in chapter 2.7.1 the mean value is obtained for each section and driving direction. In the evaluation later, the results are compared and the accuracy controlled. If the spread is too large new measurements are performed until the result is acceptable. The deepest ruts, for the same number of lasers, are saved in the database, Göransson (2009).

FWD Measurement

The measurements are performed on the right wheel path in five sections per stretch. It is performed either in the autumn after maintenance or, as previously done, in the spring before maintenance, and in the summer as well. The deflection is registered within different ranges, as mentioned in chapter 2.7.2. In addition, the air temperature, the surface temperature and the temperature under the surface are registered, Göransson (2009).

Ocular Inspection

The condition of each section is thoroughly evaluated through survey on site once a year. The surveyor registers every defect/damage, which are evaluated in a severity scale of 1-3. Common types of registered defects/damage are longitudinal/transversal cracking, alligator cracking, bleeding, lane/shoulder separation and so on, Göransson (2009).

Calculation of Crack Index

The damage in this context are load commanded and stored in the database with data that describes the type of crack, the severity of damage, the side position and where the crack begins and ends for each survey. Cracks that are shorter than 1 meter are registered as 1 meter. Furthermore, the cracks that are on the wheel path and just outside the wheel path are considered. The crack index is calculated according to following formula:

$$\text{Crack index (Si)} = 2 * Kr\ (m) + LSpr\ (m) + TSpr\ (pcs) \tag{2.12}$$

where:

$$Kr\ (m) = Kr_{low}\ (m) + 1.5 * Kr_{medium}\ (m) + 2 * Kr_{severe}\ (m) \tag{2.13}$$

$$LSpr\ (m) = LSpr_{low}\ (m) + 1.5 * LSpr_{medium}\ (m) + 2 * LSpr_{severe}\ (m) \tag{2.14}$$

$$TSpr\ (pcs) = TSpr_{low}\ (pcs) + 1.5 * TSpr_{medium}\ (pcs) + 2 * TSpr_{severe}\ (pcs) \tag{2.15}$$

where:

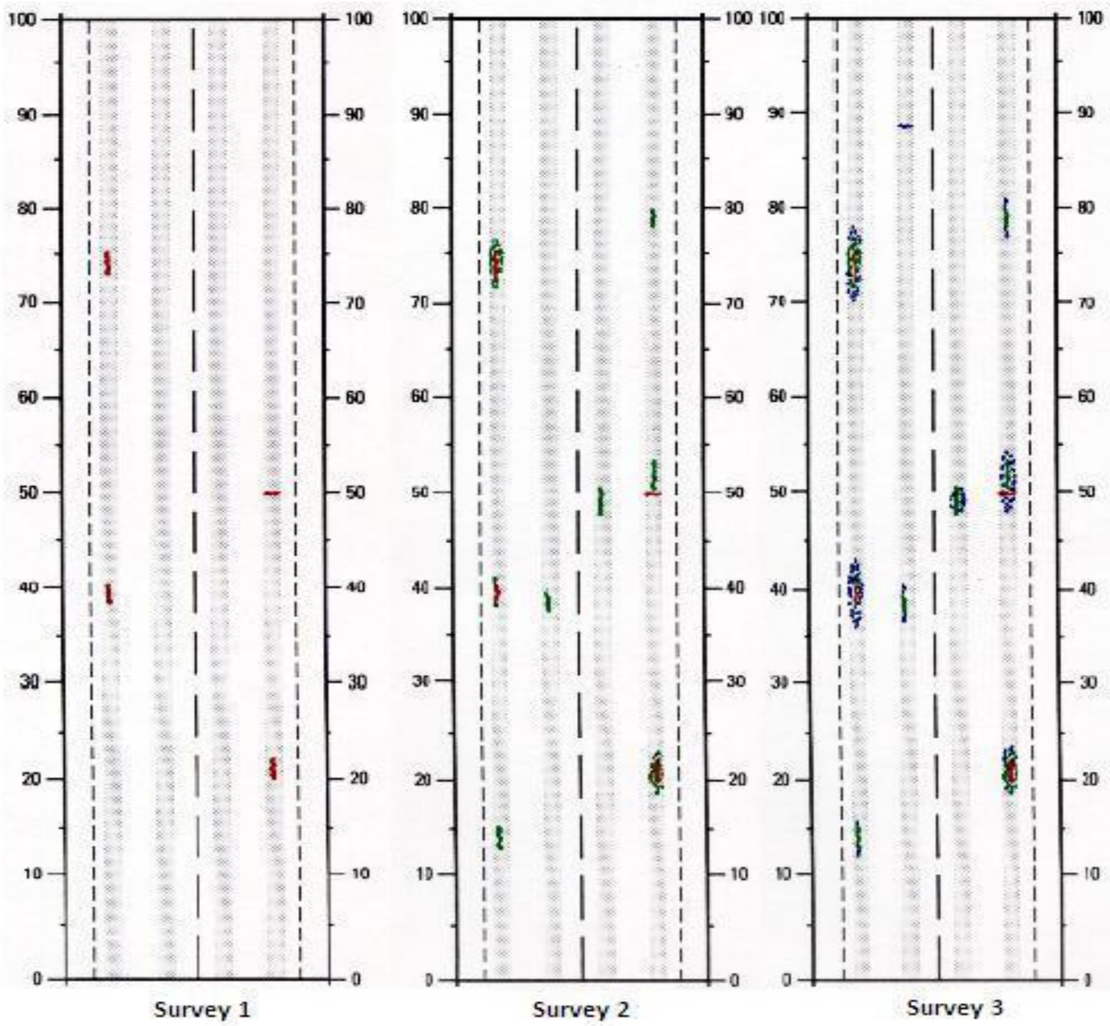
Kr – Alligator cracking

LSpr – Longitudinal cracking

TSpr – Transverse cracking

Each crack length is multiplied with a factor 1 if the severity is low, factor 1.5 if the severity is medium and factor 2 if it is severe. As equation 2.12 indicates in the total crack index, the length of the alligator cracking is multiplied with a factor of 2. Different combinations were tested for the including coefficients, however these coefficient i.e. equation 2.13, 2.14 and 2.15 were finally chosen to assure continuous development of the crack index, Göransson (2009).

Figure 2-22 shows an example of the size and development of the crack index.



	Crack index, Si = 7	Crack index, Si = 46	Crack index, Si = 103
1.	$LSpr_{low}$, length 2 m	Kr_{low} 6m	Kr_{medium}, 8 m
2.	$LSpr_{low}$,, 1m	$LSpr_{severe}$, 3 m	Kr_{low}, 7 m
3.		$LSpr_{medium}$, 2 m	$LSpr_{severe}$,3 m
4.			$TSpr_{low}$, 1 pc
5.		$LSpr_{low}$,2 m	$LSpr_{medium}$,4 m
6.		$LSpr_{low}$,3 m	Kr_{low}, 3 m
7.		$LSpr_{low}$,2 m	$LSpr_{severe}$,6 m
8.		$LSpr_{medium}$,3 m	Kr_{medium}, 6 m
9.	$TSpr_{low}$= 1 pc	$TSpr_{medium}$, 1 pc	
10.	$LSpr_{medium}$, 2 m	Kr_{medium}, 4 m	Kr_{severe}, 5 m

Figure 2-22 Example of size and development of the crack index between different surveys (LTPP-manual 2009)

3 Description and Analysis of the LTPP-roads

The following subchapters contain description and analysis of the 9 LTPP-roads that have been studied for the past 20 years. These test roads have been studied in terms of the year it was opened to traffic, latest maintenance performed, annual average daily traffic, per cent of heavy vehicle, increase of traffic in percentage, type of pavement, whether it is constructed on an embankment or a cut, type of subgrade material, material used in the unbound layers and their thicknesses and topography. The data were collected in the LTPP-database.

The analyses focus on 3 or 4 sections in each test road. These sections are thoroughly analysed using factors such as rut depth, type of subgrade material and its E-modulus, crack index, surrounding terrain in terms of direct sunlight and whether the road is constructed on an embankment or a cut. The subgrade E-modulus was calculated according to equation 2.4 in chapter 2.7.2 by data from FWD in the LTPP-database. The surrounding terrain was described in LTPP-database, however by field visits at some roads that were located in forestial or hilly terrains the extent of the direct sunlight could be observed.

3.1 Rv 53 Nyköping

Rv 53 is a national road in the province Södermanland in the south-eastern part of Sweden. The part of the road included in LTPP is situated close to Nyköping and consists of 10 sections. The road was opened to traffic in 1987 and the latest maintenance was performed in 1993. The analysis is performed with data from 1993 to 2006.

Furthermore, the test road consists of two 3.75 m wide driving lanes. It is located in a hilly cultural landscape and the road is constructed on embankments. The pavement type is GBÖ 70 in all sections and the unbound layers consist of same material and thickness in all sections as well, as shown in the table 3-1 below.

Table 3-1 Pavement in RV 53 at Nyköping

Layer	Thickness [mm]	Material	Year
Wearing course	28	100MABT16	1993-08-15
First asphalt layer	40	70HABS16 Ort Funkt	1987-07-01
Base	115	Gravel base material: BYA	1987
Sub base	500	Material class A	1986-87
Subgrade		Silt	

The subgrade consists of silt or silt together with other soil type all sections. Sections 1, 6 and 7 have been selected for further analysis. Section 1 consists of silt together with silty clay, section 6 consists of only silt while section 7 consists of silt together with silty sand.

Moreover, the $AADT_L$ was determined to be 1227 vehicles in 2006 with an annual increase of 3.25 per cent. The proportion of heavy vehicles corresponded to 7.32 per cent.

Observation of data

In the traffic lane towards north, the rutting development is increasing linearly with a slow rate in the various sections, see figure 3-1. The crack index, on the other hand, initiates to increase in 1998 in the various sections. The largest rate is observed in section 7, see figure 3-2. In 2002, the crack index reaches a high value in section 7, which causes a rapid increase of the rutting development.

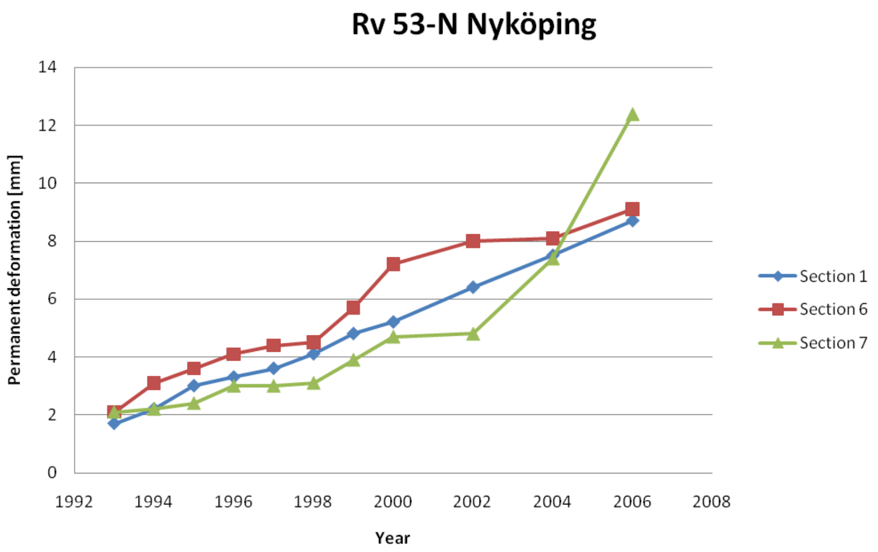


Figure 3-1 Rutting in the traffic lane towards north in Rv 53 at Nyköping

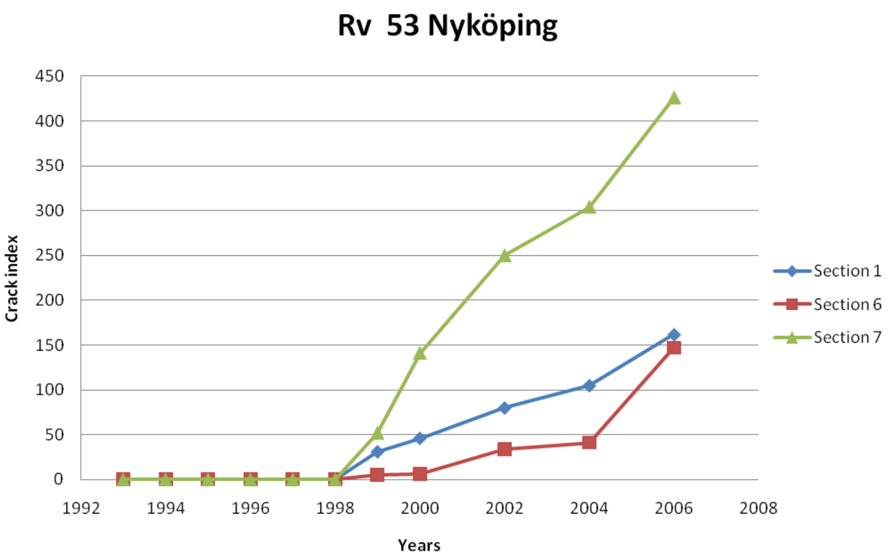


Figure 3-2 Crack index in the traffic lanes in Rv 53 at Nyköping

Furthermore, the subgrade E-modulus is approximately similar in the various sections and therefore excluded from the analysis.

Unlike the northern direction, the deepest ruts in the traffic lane towards south are observed in section 1, as shown in figure 3-3. The permanent deformation curve shows the same pattern as in the traffic lane towards north with a linear increase in each section. However, section 7 does not show the same pattern, with a rapid increase as its adjacent traffic lane towards north.

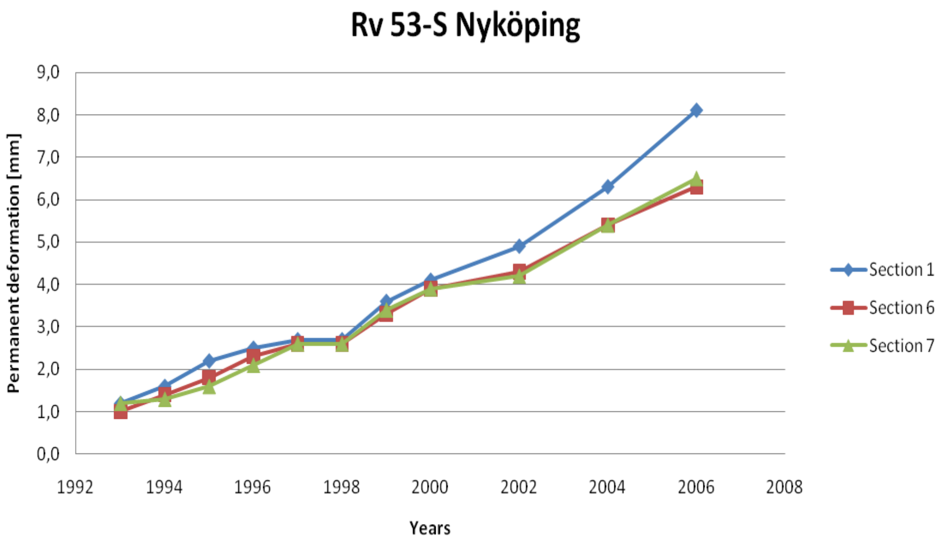


Figure 3-3 Rutting in the traffic lane towards south in Rv 53 at Nyköping

Analysis

It is observed that the rutting development is linear in the analysed sections, except in section 7 towards north. In this section the rutting development is linearly increasing until 2002. Subsequently, there is an acceleration of the rutting, which is strongly believed to be correlated to the crack index that reaches a high value at this period. As it was stated in chapter 2.6.1 cracking leads to strength reduction in the asphalt layer, which in its turn causes higher stresses conducted to the unbound layers. This in combination with weak subgrade causes rearrangement of the particles and increased compaction of the unbound material, which eventually leads to increase of the permanent deformation. The fundamentals to this phenomenon are explained in chapter 2.6.3 above.

However, section 7 towards south does not show the same pattern. This is believed to be connected to fewer cracks in this lane compared to the lane towards north. Another factor that should not be excluded is the effect of direct sunlight, however since there was no field visit at this site it is impossible to make a conclusion in this respect.

Moreover, the rutting development is linearly increasing in the other analysed sections as mentioned above. This is believed to be correlated to the subgrade material, which mostly consists of silt. Silt and materials that consist of smaller fractions are more susceptible to water content compared to materials consisting of larger fractions. This is believed to have a vast effect on the permanent deformation development. The effect of this is continuous rutting development without stagnation or acceleration.

3.2 Rv 31 Nässjö

Rv 31 is a national road in the province Småland in the south-eastern part of Sweden. The part of the road included in LTPP is situated close to Nässjö and consists of 11 sections. The road was opened to traffic in 1988 and the latest maintenance was performed in 2007. The analysis was made with data collected from 1992 to 2007.

Moreover, the test road consists of two 3.75m wide driving lanes and is located in forestial land. Sections 1, 6 and 11 were selected to be analysed and of these sections, sections 1 and 11 are constructed on embankment while section 6 is constructed on soil cut. The pavement is GBÖ 70 in all sections and the unbound layers consist of same material and thickness in all sections as well, as shown in the table 3-2 below.

Table 3-2 Pavement in RV 31 at Nässjö

Layer	**Thickness [mm]**	**Material**	**Year**
Wearing course	24	Remixing Plus 60ABS16	2007-08-10
Second asphalt layer	35	80MABT16	1989-07-01
First asphalt layer	50	110AG	1988-11-01
Base	115	Gravel base	1988
Sub base	500	Gravel material: BYA 345:03	1987-88
Subgrade		Fine-sandy moraine	

The $AADT_L$ was measured to be 1616 vehicles in 2007 of which 14.59 per cent was considered as heavy vehicles. The annual increase was registered to 1.2 per cent.

Observation of Data

The permanent deformation for both directions is plotted against time in figure 3-4 and 3-5 below.

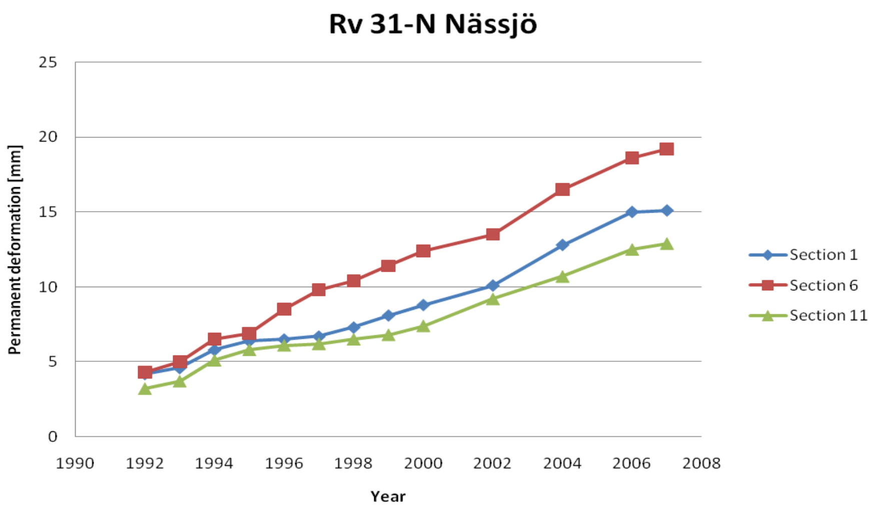


Figure 3-4 Rutting in the traffic lane towards north in Rv 31 at Nässjö

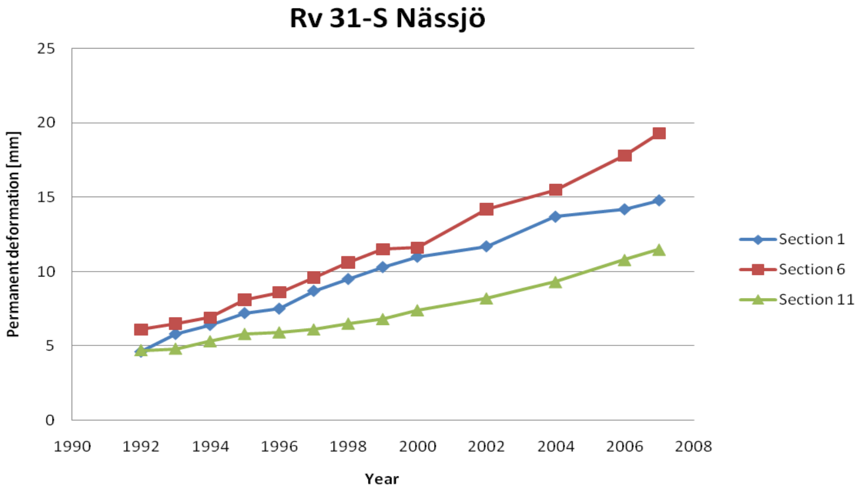


Figure 3-5 Rutting in the traffic lane towards south in Rv 31 at Nässjö

From the figures above, it is observed that the analysed sections have the same shape on the deformation curve, a linear increase. The largest permanent deformation is observed in section 6. The subgrade E-modulus is lowest in section 6 as well, see diagram 3-6.

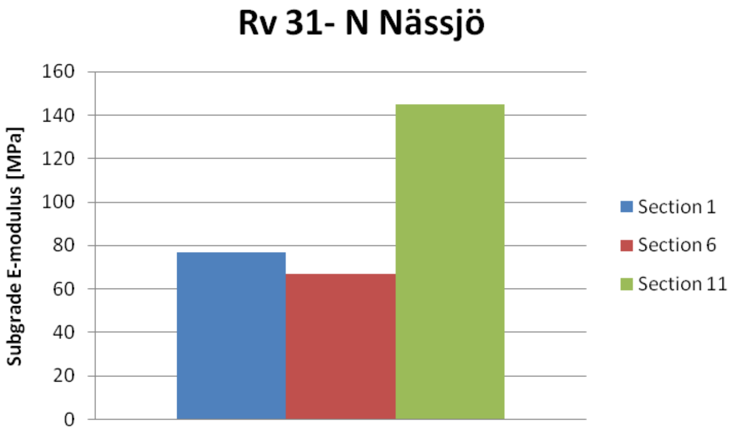


Figure 3-6 Subgrade E-modulus in the traffic lane towards north in Rv 31 at Nässjö

The figure 3-6 above indicates that section 11 has twice as large subgrade E-modulus as section 6. Unlike the traffic lane towards north, the subgrade E-modulus is different in the southern direction, as shown in figure 3-7 below. The E-modulus is higher in sections 1 and 6, while it is lower in section 11. Since it is an average value it is believed that local variations may occur.

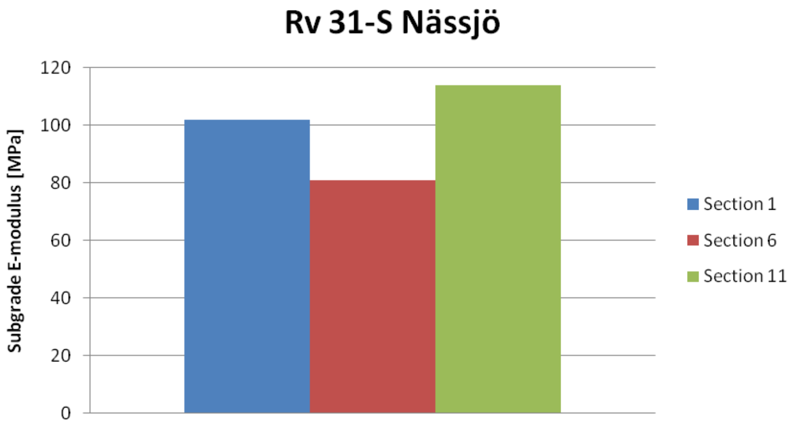


Figure 3-7 Subgrade E-modulus in the traffic lane towards south in Rv 31 at Nässjö

Furthermore, there is no crack index of larger significance in the sections and it is therefore neglected from the analysis.

Analysis

The rutting development is linearly increasing in each analysed section with different inclination. The steepest inclination occurs in section 6 for both the northern and southern direction. The linear increase of the permanent deformation is believed to be caused by the subgrade material that consists of fine-sandy moraine, where the fine-sand is susceptible against water content and the effect of water content is explained in chapter 2.5.2.3. Moreover, the permanent deformation is continuous due to weaker subgrade material as well, where the effect of this matter is explained in chapter 2.6.3.

Furthermore, section 6 is constructed on a soil cut, which implies that the groundwater level will be closer to the surface. This in its turn increases the water content both in the subgrade and the unbound layers. Additionally, this is believed to be one of the reasons to the larger permanent deformation occurring in section 6. Another reason to the larger permanent deformation is the low subgrade E-modulus in this section compared to sections 1 and 11.

3.3 Rv 33 Vimmerby

Rv 33 is a national road in the province Småland in the south-eastern part of Sweden. The part of the road included in LTPP is situated close to Vimmerby and contains 12 sections. The road was opened to traffic in 1980 and the latest maintenance was performed in 1988. The analysis is made with data from 1988 to 2006.

Furthermore, the test road consists of two 3.5m wide traffic lanes and is located in a hilly forestial terrain, where both cuts and embankment occurs in each section. The pavement is GBÖ 50 for all sections and the unbound layers consist of same material and thickness in all sections as well, as shown in the table 3-3 below.

Table 3-3 Pavement in RV 33 at Vimmerby

Layer	**Thickness [mm]**	**Material**	**Year**
Wearing course	0	Y1B12 (Surface treatment)	1988-07-01
First asphalt layer	75	110BG+60MABT12	1980-07-01
Base	80	Gravel base	-
Sub base	350	Gravel material: BYA 345:03	-
Subgrade		Varying, see table 3-4	

Moreover, sections 1, 7, 8 and 12 have been selected to be further analysed and the subgrade material in these sections and their E-modulus are shown in the table 3-4 below.

Table 3-4 The type of subgrade material in the various sections and their E-modulus in RV 33 at Vimmerby

Section	**Type of subgrade material**	**E-modulus [MPa]**	
		East	*West*
1	Fine-sandy moraine/sandy moraine partly on bedrock	250	552
7	Fine sand partly on bedrock/gravelly sand	120	128
8	Gravelly sand/sandy gravelly moraine/fine sand	123	117
12	Sandy gravelly moraine on bedrock	711	746

The $AADT_L$ was measured to be 1273 vehicles in 2006 with an annual increase of 4.6 per cent. The amount of heavy vehicles was 11.25 per cent.

Observation of Data

The permanent deformation plotted against time in the traffic lane towards east is illustrated in figure 3-8 below.

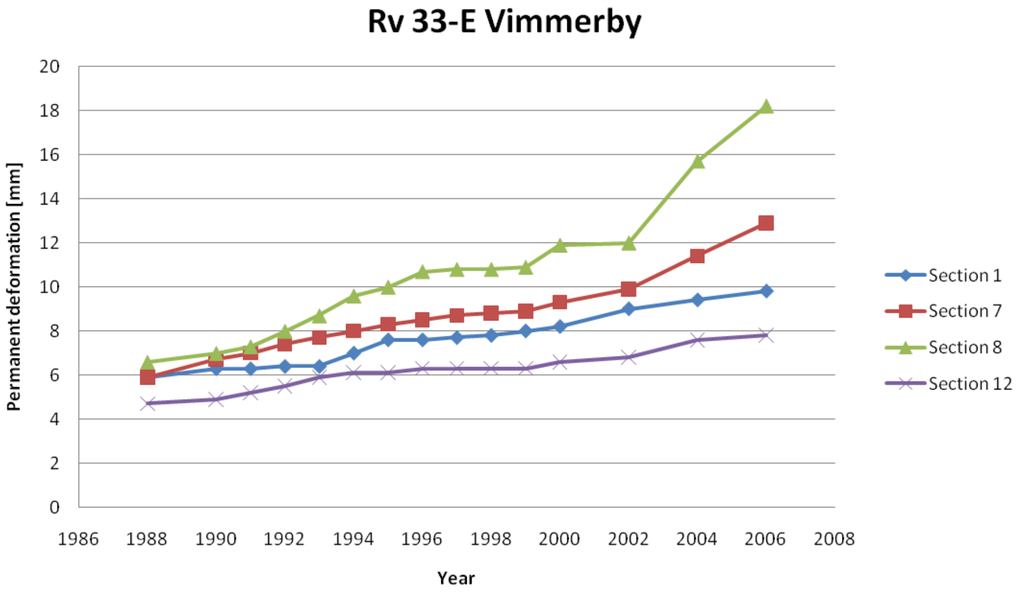


Figure 3-8 Rutting in the traffic lane towards east in Rv 33 at Vimmerby

The initial permanent deformation in 1988 is within a range between 4 and 7 mm. The development varies in the various sections and the largest development occurs in section 8 and section 7, respectively. This may be correlated to the crack index, see figure 3-9, where these two sections develop larger crack index compared to the other two sections.

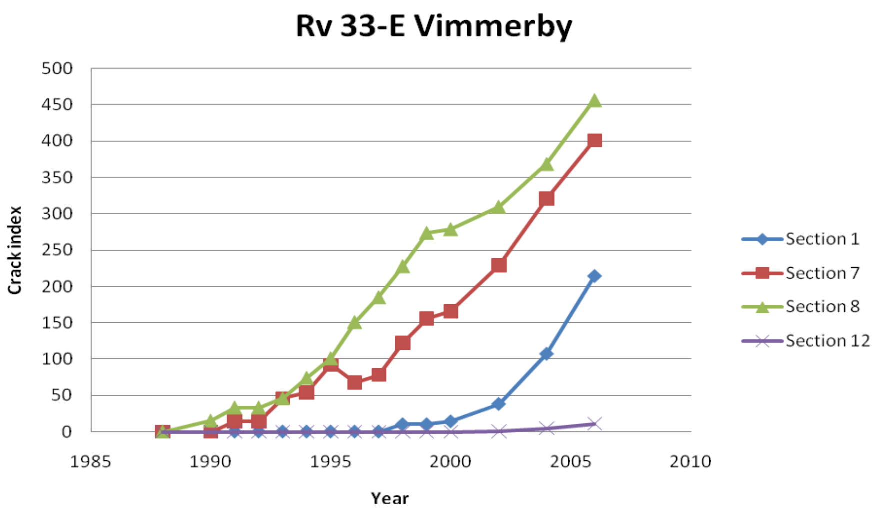


Figure 3-9 Crack index in the traffic lane in Rv 33 at Vimmerby

Furthermore, the subgrade E-modulus is low in sections 8 and 7, see figure 3-10. The E-modulus is visibly largest in section 12, where the subgrade material is situated directly on bedrock. In addition, there is no larger variation in the subgrade E-modulus between the different traffic lanes except for the E-modulus in section 1, which is twice as large in eastern direction compared to western direction, see table 3.4 above.

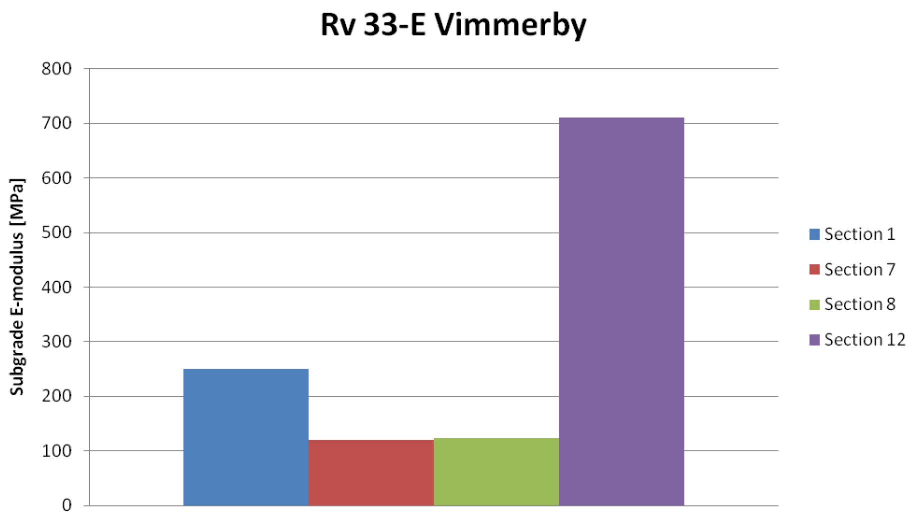


Figure 3-10 Subgrade E-modulus in the traffic lane towards east in Rv 33 at Vimmerby

On the other hand, the permanent deformation that is developed in the traffic lane towards west is totally dissimilar to its adjacent traffic lane towards east, as illustrated in figure 3-11.

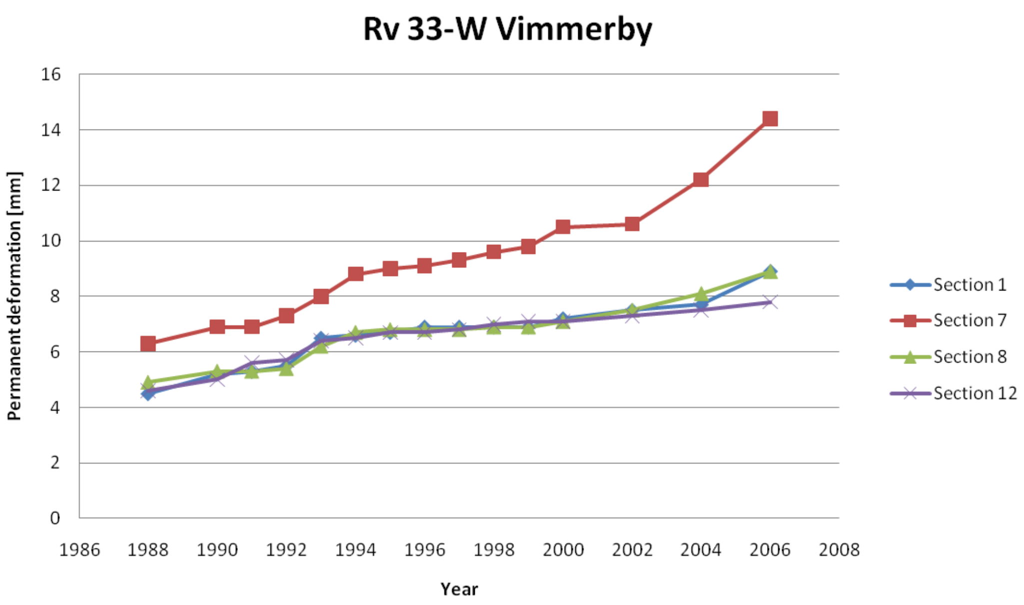


Figure 3-11 Rutting in the traffic lane towards west in Rv 33 at Vimmerby

Figure 3-11 indicates a larger permanent deformation in section 7 throughout the years compared to other sections, which have equal rutting development over the years. The shape of the permanent deformation curve in section 7 is similar to the curve of section 8 in the eastern section.

Analysis

In this LTPP-road the rutting development is dissimilar in both directions. In the traffic lane towards east section 8 has developed the largest permanent deformation, while in the traffic lane towards west, in its turn, section 7 also has developed the largest permanent deformation. The rutting development in section 8 towards east is slowly increasing until 1998, when it accelerates. This is believed to be correlated to the simultaneous increase of the crack index at the same period. It is noticeable that the increase occurs when the crack index passes 250. Section 8 is closely followed by section 7, where the rutting is increasing slowly until 2002 when it suddenly increases. This increase is also correlated to the crack index and occurs when the crack index passes 250. Furthermore, it is believed that section 8 has larger permanent deformation due to the subgrade material that has a larger proportion of fine-sand compared to section 7.

Furthermore, sections 1 and 12 have s smaller permanent deformation compared to the other sections. This is believed to be linked to the subgrade material, which consists of bedrock in these sections and the fewer cracks compared to the other sections as well. The subgrade E-modulus is highest in section 12 where smallest permanent deformation occurs.

Considering the traffic lane towards west, the permanent deformation is largest in section 7 as mentioned above. Section 8, on the other hand, has considerably lower rutting development compared to section 7 and its adjacent section towards east. This is believed to be correlated to the direct sunlight that is shaded by the surrounding forests consisting of tall trees in this section and thus results in much smaller permanent deformation. However, it is further believed that measurement errors together with fewer cracks in this traffic lane may have affected the outcome as well.

Additionally, section 7 has the same pattern in the rutting development as in section 7 towards east with acceleration in 2002 and slow increase before that year.

Eventually, the rutting development and the rut depth in sections 1 and 12 are similar to its opposite in the eastern direction.

3.4 Rv 33 Ankarsrum

The LTPP-road in Ankarsrum consists of 10 sections and is located in the same province as the road in Vimmerby. The road was opened to traffic in 1979 and the latest maintenance was performed in 1990. The data collected from LTPP-database was measured between 1990 and 2006. The test road consists of two 3.5m wide traffic lanes, which are located in a hilly terrain.

Furthermore, sections 1, 4 and 10 have been selected to be thoroughly analysed and the various sections are constructed on embankments. The pavement is BB for all sections and the unbound layers consist of same material and thickness in all sections as well. The subgrade material and the E-modulus in these sections are shown in the table 3-5 below.

Table 3-5 The type of subgrade material in the various sections and their E-modulus in RV 33 at Ankarsrum

Sections	**Type of subgrade material**	**E-modulus [MPa]**	
		East	*West*
1	Stony clayey moraine	99	108
4	Stony fine-sandy moraine	353	276
10	Clay / moraine	136	116

The $AADT_L$ consisted of 1014 vehicles in 2006 of which 14.41 per cent was determined as heavy vehicles. The annual increase of the vehicles was 1.7 per cent.

Observation of Data

Section 10 has developed the largest rutting in the traffic lane towards east and the rutting development increases linearly in each section, see figure 3-12.

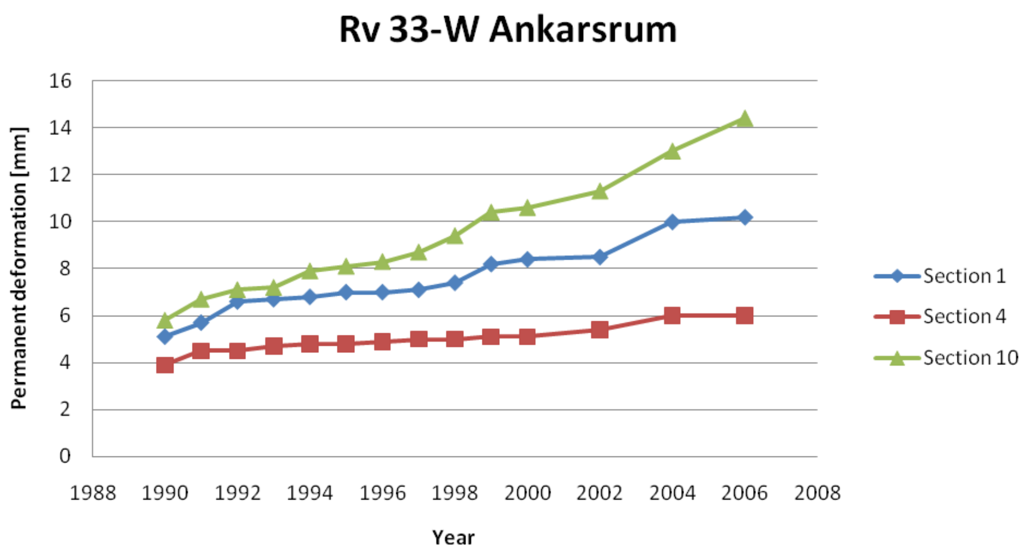


Figure 3-12 Rutting in the traffic lane towards west in Rv 33 at Ankarsrum

The rutting development is dissimilar in the traffic lane towards east. However the largest permanent deformation is occurring in sections 10, see figure 3-13.

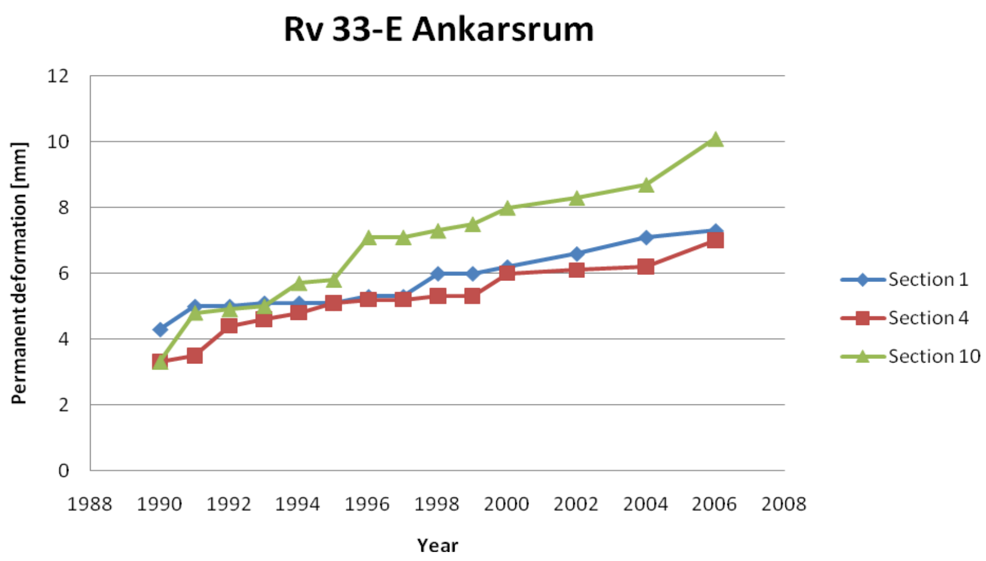


Figure 3-13 Rutting in the traffic lane towards east in Rv 33 at Ankarsrum

The development of the permanent deformation is correlated to the subgrade E-modulus in these sections, which are shown in figure 3-14.

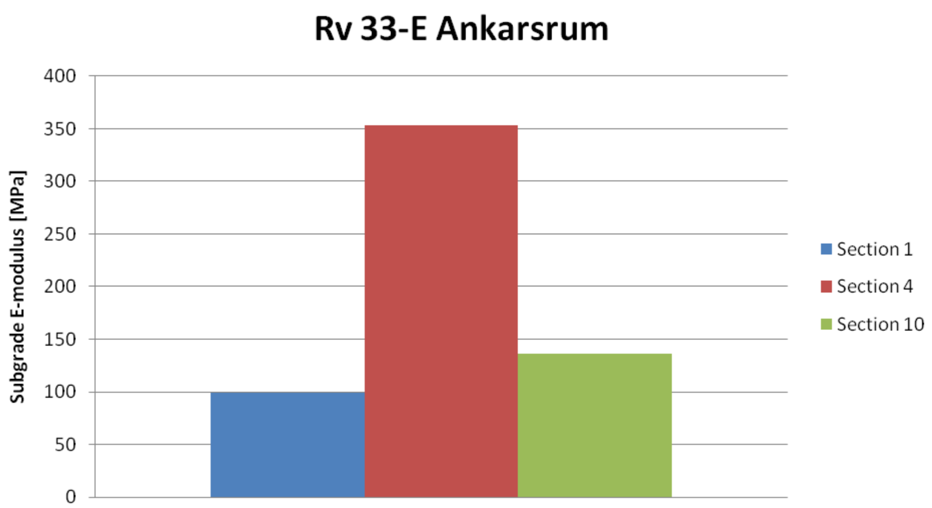


Figure 3-14 Subgrade E-modulus in the traffic lane towards west in Rv 33 at Ankarsrum

Furthermore, the subgrade E-modulus is similar in the traffic lane towards west, see table 3-5 above.

Additionally, there is no crack index of larger significance in the analysed sections and it is therefore neglected from the analysis.

Analysis

In the traffic lane towards west, the rutting plotted against time indicates a linear development in each section with different inclination. The difference in the inclinations is believed to be correlated to the subgrade material and its E-modulus. Despite lower E-modulus in section 1 compared to section 10, section 10 has developed larger rut depth. This is caused by the subgrade material, which consists of clay in this section. Clay is the most susceptible subgrade material to water content and one of the weakest subgrade materials. The effect of the water content is described in chapter 2.5.2.3 and the subgrade materials in chapter 2.6.3.

Furthermore, the linearly increase in section 1 is believed to be influenced by the clayey part of the subgrade material. However, it shows the same pattern as in section 10 with a lower inclination on the rutting development.

In addition, the permanent deformation in section 4 is much lower compared to the other sections, which is correlated to the subgrade material that consists of stony fine-sandy moraine and has a high E-modulus. The rutting development has stagnated and a very low increase is observed in the plots.

On the other hand, the traffic lane towards east has developed less permanent deformation, which is believed to be caused by less heavy axle loads in this direction. Additionally, there are some differences in the shape of the deformation curve for this direction compared to the western direction. The rutting development in section 10 is equal to its adjacent in the western direction. However, sections 1 and 4 have the same pattern in rutting development, which is strongly connected to measurement errors since data in section 4 was modified to create a more realistic rutting development.

Moreover, the crack index and the surrounding terrain were neglected in the analysis. The crack index was neglected due to very low values and the surrounding terrain due to observation made in the field visit that indicated similar environmental conditions in each analysed section.

3.5 Rv 34 Målilla

Rv 34 is a national road in the province Småland in the south-eastern part of Sweden. The part of the road included in LTPP is situated close to Målilla and divided into 10 equally sized sections. The road was opened to traffic in 1987 and the latest maintenance was performed in 2002. The analysis is made from data between 1992 and 2002.

Furthermore, the test road consists of two 3.75 m wide driving lanes, which are located in a mixed terrain, where both cuts and embankment occurs. The sections selected for further analysis consists of section 1, 6 and 9. Sections 1 and 9 are constructed on soil cuts, while section 6 is constructed on an embankment. The pavement is GBÖ 60 for all sections and the unbound layers consist of same material and thickness in all sections as well, as shown in the table 3-6 below.

Table 3-6 Pavement in RV 34 at Målilla

Layer	**Thickness [mm]**	**Material**	**Year**
Wearing course	28	70HABS16, Heating	2002-07-01
First asphalt layer	75	110AG+60MAB	1987-10-13
Base	125	Gravel base material: BYA	-
Sub base	400	Gravel and Sand: BYA	-
Subgrade		Sand	

The $AADT_L$ consisted of 1383 vehicles 2002 with an annual increase of 0.4 per cent. The proportion of heavy vehicles was 16.81 per cent.

Observation of Data

In the traffic lane towards north the permanent deformation development is largest in section 6, which is closely followed by section 1, as shown in figure 3-15. Section 9 has approximately 4 mm smaller deformation compared to sections 6 and 1.

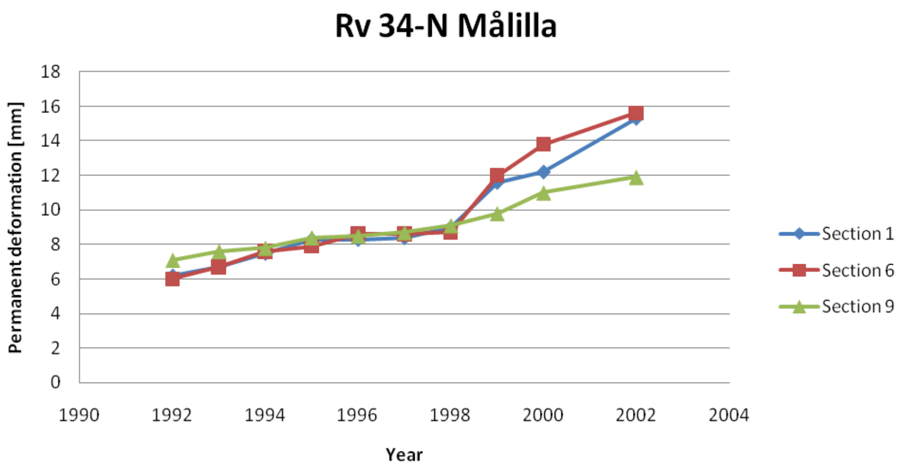


Figure 3-15 Rutting in the traffic lane towards north in Rv 34 at Målilla

In the opposite direction, the southern direction, the largest rutting is measured in section 9. Noticeable in this direction is that, section 1 has considerably smaller rutting compared to its adjacent in the opposite direction. The rutting is smaller for sections 1 and 6, while it has increased for section 9, as shown in figure 3-16.

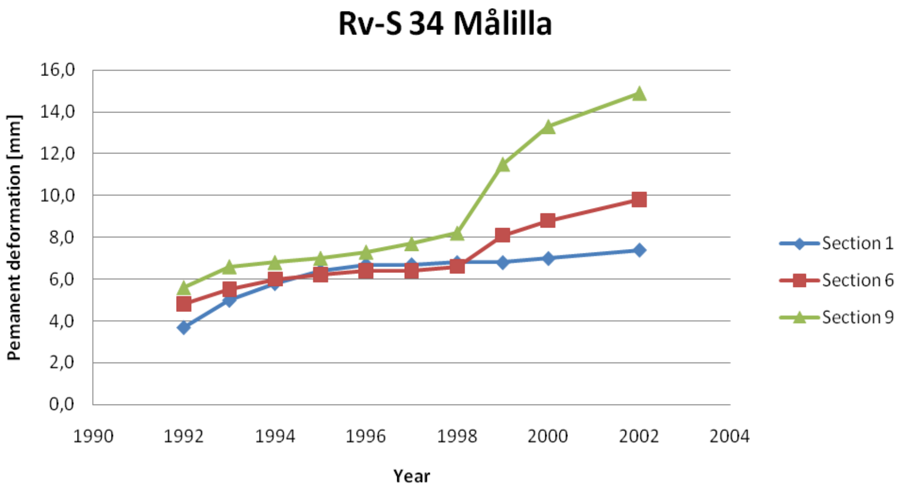


Figure 3-16 Rutting in the traffic lane towards south in Rv 34 at Målilla

The crack index is similar for the various sections, see figure 3-17. It is steadily increasing in each section over the years.

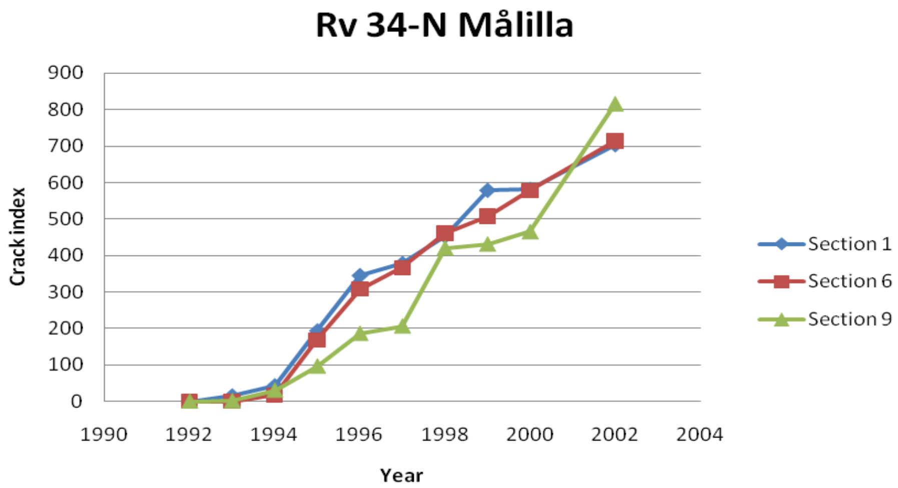


Figure 3-17 Crack index in the traffic lane in Rv 34 at Målilla

The subgrade E-modulus is about the same in the selected sections since the subgrade material consists of purely sand in each section. Therefore, it is neglected in the analysis.

Analysis

The rutting development stagnates in each analysed section between 1992 and 1998 to latter increase rapidly from 1998 to 2002. The period between 1992 and 1998 can be considered as the compaction phase. The rapid increase that latter occurs in 1998 is believed to be correlated to the crack index, which reaches a high value at the same year in each section.

In the traffic lane towards north, the largest increase and thereby the largest permanent deformation occurs in section 6, closely followed by section 1. The increase in section 9 is smaller than the increase in these sections. Furthermore, it is remarkable that section 6 has larger permanent deformation than sections 1 and 9, since sections 1 and 9 are constructed on a soil cut. As previously mentioned, the groundwater level is closer to the surface in a soil cut due to excavation of the ground, which affects the water content in both subgrade and in the unbound material of the pavement and causes larger permanent deformations. However, factors such as measurement errors and fewer cracks in this traffic lane in section 6 may have affected the outcome of the rutting in this section. Moreover, the effect of the surroundings is neglected since the field visit at the road resulted in similar conditions in this traffic lane.

In the traffic lane towards south, on the other hand, section 9 has the largest permanent deformation. The increase is less in section 6, whereas no increase is registered for section 1. This rutting pattern is more reasonable since section 9 is constructed on cut, which affects the water content of the subgrade and the unbound materials as mentioned above. The rutting development in section 1 may be correlated to the surroundings, which consists of tall trees that shade against direct sunlight during midday and prevents reduction of E-modulus for the bound layers in some extent. It may also be caused by fewer cracks in this traffic lane compared to its adjacent in the other direction as well.

Eventually, this road shows that the cracks should be noticed per lane and section rather than per section. The analysis with regard to cracks would be more simple and correct.

3.6 Rv 44 Grästorp

Rv 44 is a national road in the province Västra Götaland in the south-western part of Sweden. The part of the road included in LTPP is situated close to Grästorp and consists of 10 sections. The road was opened to traffic in 1981 and the latest maintenance was performed in 2003. The analysis is made for a period from 1992 to 2001.

Furthermore, the test road consists of two 3.5 m wide driving lanes, which are located in an agricultural landscape with both cuts and embankments. Sections 1, 6 and 9 were chosen to be thoroughly analysed and of these sections, section 1 is constructed on a soil cut and section 6 on a rock cut, while section 9 is constructed on an embankment. The pavement is GBÖ 70 for sections 1 and 9, while it is BS for section 6.

Moreover, the unbound layers consist of same material with different thicknesses. The base consists of gravel with a thickness of 186 mm for sections 1 and 9, while it is 86 mm for section 6. The subbase in its turn is made up of gravel and sand with a thickness of 450 mm for sections 1 and 9, whilst section 6 has no subbase. This is due to the subgrade material, which is bedrock in section 6 while it is silt and clay for the other sections.

The $AADT_L$ was measured to be 1557 vehicles in 2001 with an annual increase of 4.6 per cent. Heavy vehicles were considered to be 16.73 per cent.

Observation of Data

From the observation of data, the largest permanent deformation in the traffic lane towards east is observed in section 1, see figure 3-18. The deformation is 3 times larger compared to section 6 and almost 2 times larger than section 9.

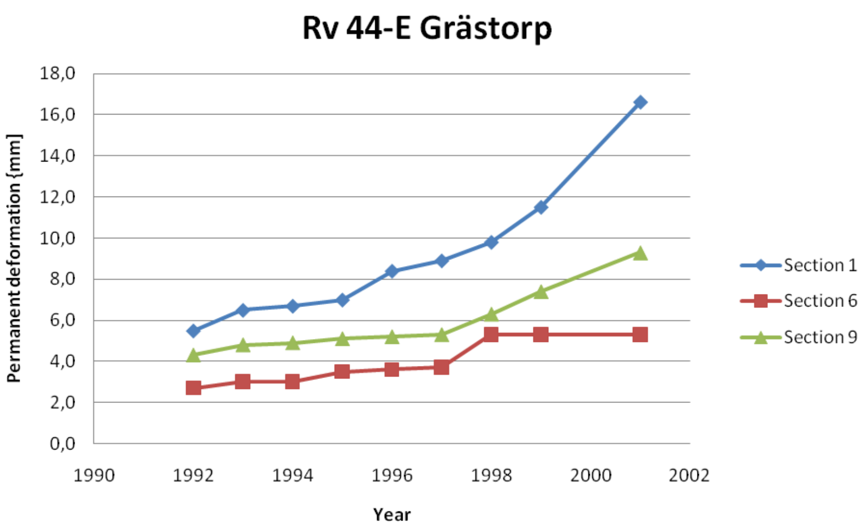


Figure 3-18 Rutting in the traffic lane towards east in Rv 44 at Grästorp

The permanent deformation development in the opposite traffic lane is dissimilar and the largest deformation in this direction is observed in section 9. The rutting is larger in section 9 in this direction compared to the other direction, while it is smaller in section 1. Moreover, the rutting is similar in section 6, see figure 3-19.

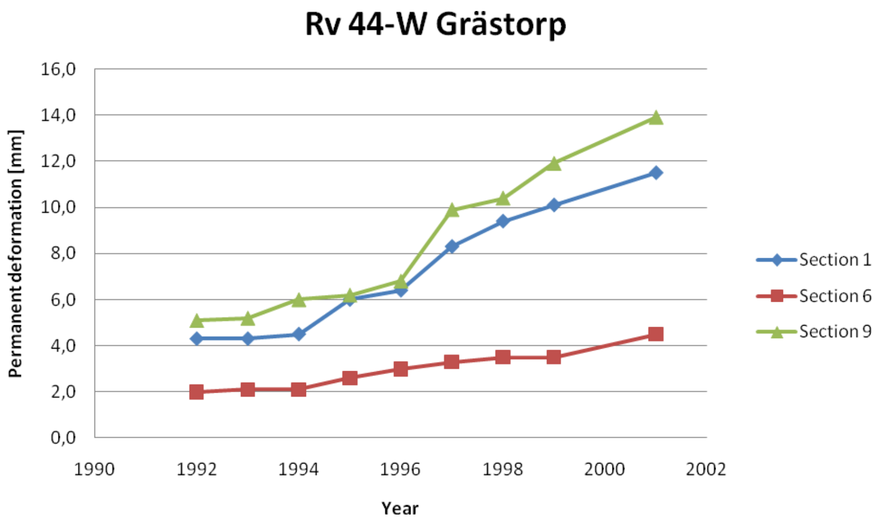


Figure 3-19 Rutting in the traffic lane towards west in Rv 44 at Grästorp

The crack index is largest in section 9 followed by section 1. There are significantly less cracks in section 6 compared to these sections, see figure 3-20.

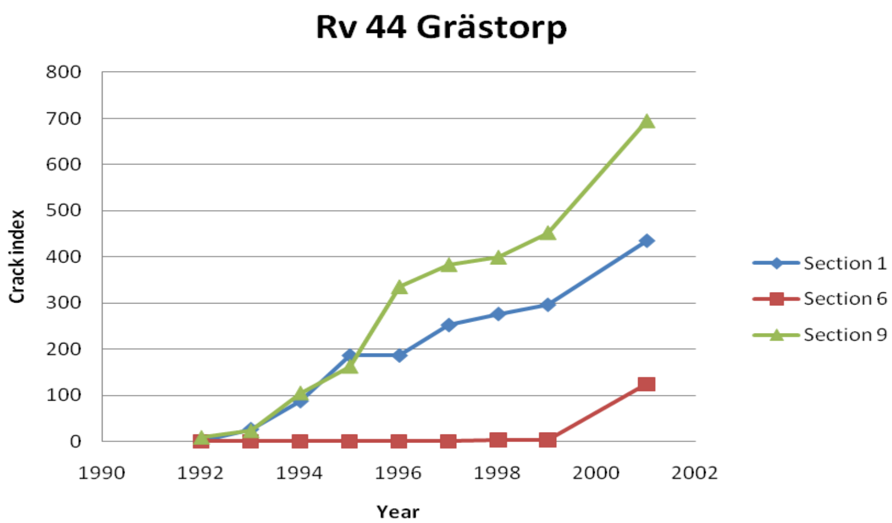


Figure 3-20 Crack index in the analysed sections in Rv 44 at Grästorp

The subgrade E-modulus, in its turn, is largest in section 6, which is constructed on a rock cut as mentioned above. The other two sections have 25 times smaller E-modulus than section 6, as shown in figure 3-21.

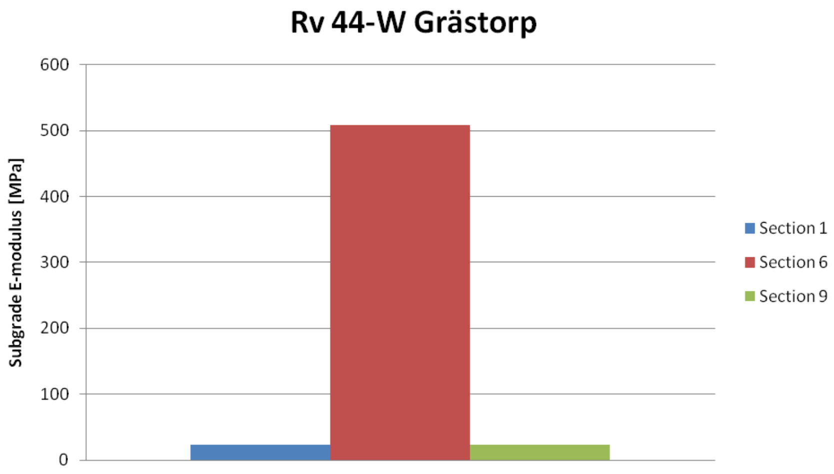


Figure 3-21 Subgrade E-modulus in the traffic lane towards west in Rv 44 at Grästorp

Moreover, the subgrade E-modulus is approximately equal in the traffic lane towards east, why only the western direction is illustrated.

Analysis

In the traffic lane towards east, the rutting development is slowly increasing until 1995, when it starts to increase more rapidly in sections 1 and 9. This is connected to the crack index that simultaneously increases to a significant level at the same period as well as the subgrade material that consists of soft silt or clay. The permanent deformation is larger in section 1 over the years since it is constructed on a soil cut. The effect of this is described in chapter 2.6.2.

Moreover, the permanent deformation in section 6 increases only 2 mm during a decade. This is linked to the subgrade material that is made up of bedrock and has a high E-modulus. In addition, the low crack index in this section should not be disregarded as well.

On the other hand, the state of the adjacent traffic lane towards west is similar with a visible increase that occurs in 1995. However, section 9 has developed larger rutting compared to section 6 in this direction. This is believed to be caused either by fewer cracks in this direction in section 6 compared to the opposite direction or by more cracks in section 9 in this direction compared to the other direction. Another crucial element that should be considered is the surrounding terrain since the shading of the direct sunlight affects the rutting development, see chapter 2.6.6 for further description. Moreover, since there was no field visit to this road, the analysis in these respects is based on probable theories. The analysis in this respect would be more reasonable if the crack index was stated per lane and section. Additionally, the data derived from LTPP-database illustrated an unreasonable rutting development and had to be modified to show a more realistic permanent deformation curve. Therefore, the measurement errors should not be disregarded as well.

Eventually, section 6 has the same development as its adjacent traffic lane.

3.7 Rv 46 Trädet

Rv 46 is a national road in the province Västra Götaland in the south-western part of Sweden. The part of the road included in LTPP is situated close to Trädet and consists of 9 sections. The road was opened to traffic in 1986 and the latest surface treatment was performed 1988. The measurement data were collected between 1992 and 2006.

Furthermore, the test road consists of two 3.75m wide traffic lanes and is located in an agricultural landscape. The sections selected for further analysis were sections 1, 6 and 9. In addition, sections 1 and 9 are constructed on embankments, whilst section 6 is constructed on a soil cut. The pavement is GBÖ 70 for section 1 and GBÖ 50 for section 6 and 9.

Moreover, the unbound layers consist of same material with different thicknesses in the subbase. The base consists of gravel with a thickness of 125 mm for all sections, while the thickness of the subbase is 500 mm for section 1 and 300 mm for sections 6 and 9. The subgrade material varies for the sections, as shown in table 3-7 below.

Table 3-7 The type of subgrade material in the various sections and their E-modulus in RV 46 at Trädet

Sections	**Type of subgrade material**	**E-modulus [MPa]**	
		North	*South*
1	Peat on fine-sand material	67	57
6	Sandy gravel	79	65
9	Fine-sandy sandy gravel	126	148

The $AADT_L$ was measured to be 1068 vehicles in 2006 with an annual increase of 1.3 per cent, where 14.41 per cent was registered as heavy vehicles.

Observation of Data

The deepest ruts for the northern direction is observed in section 9, see figure 3-22. Additionally, the rutting in section 9 is twice the rutting in sections 1 and 6 in 2006.

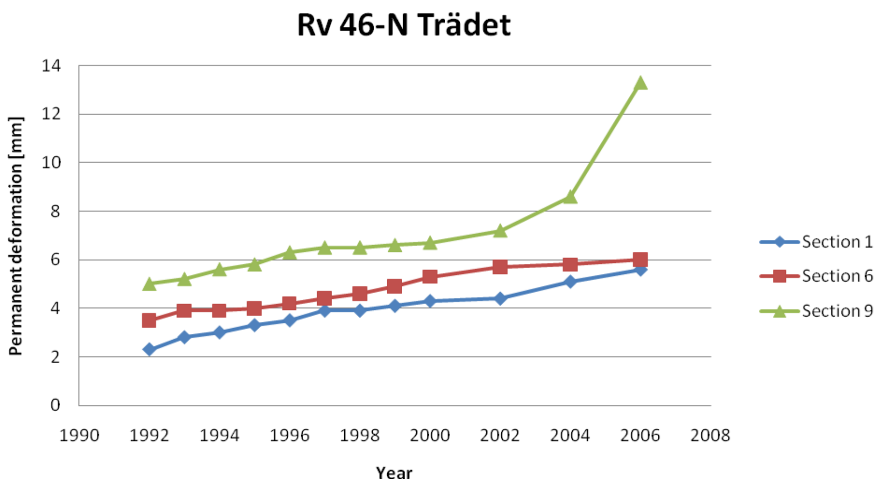


Figure 3-22 Rutting in the traffic lane towards north in Rv 46 at Trädet

The permanent deformation on the opposite direction is larger in each section. The major difference is observed in section 6, which has a considerable larger deformation compared to the northern direction, see figure 3-23.

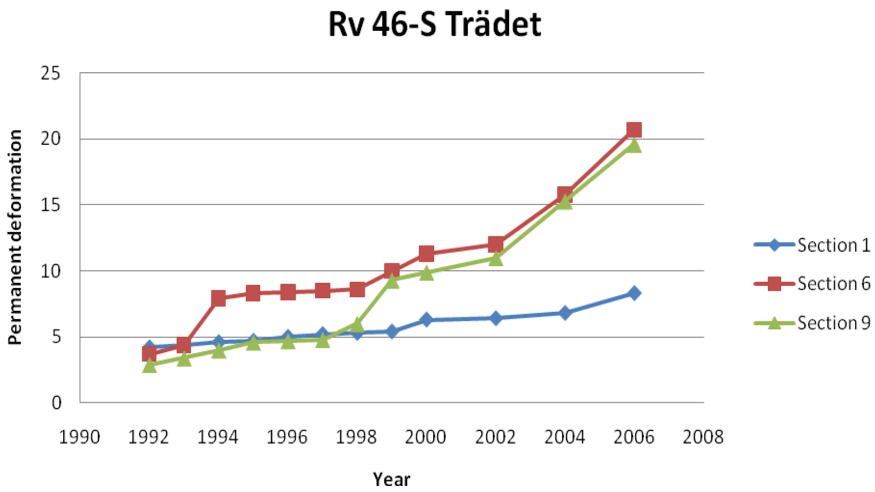


Figure 3-23 Rutting in the traffic lane towards south in Rv 46 at Trädet

Furthermore, the crack index has same pattern for section 6 and 9, where section 9 comes out best in the end. In section 1, in its turn, only few cracks are noticed, see figure 3-24.

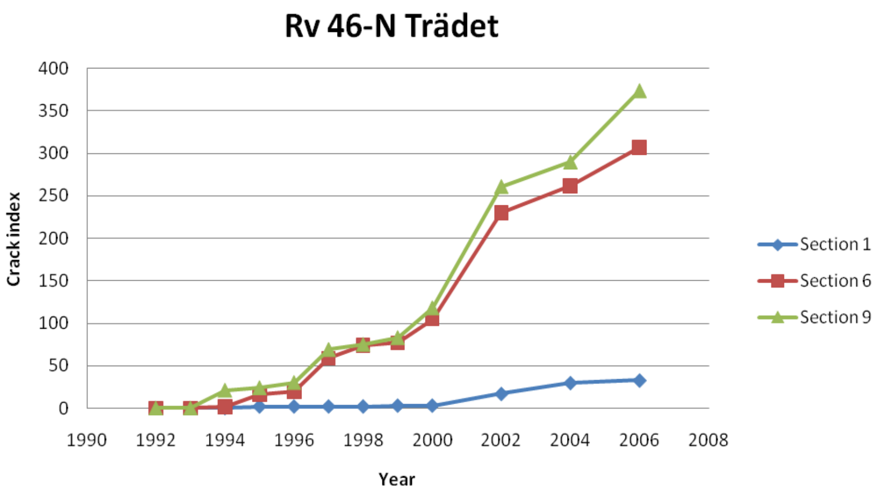


Figure 3-24 Crack index in the traffic lane in Rv 46 at Trädet

Concerning the subgrade E-modulus, section 9 has larger E-modulus compared to the other analysed sections. The subgrade E-modulus in the various sections is shown in figure 3-25 below.

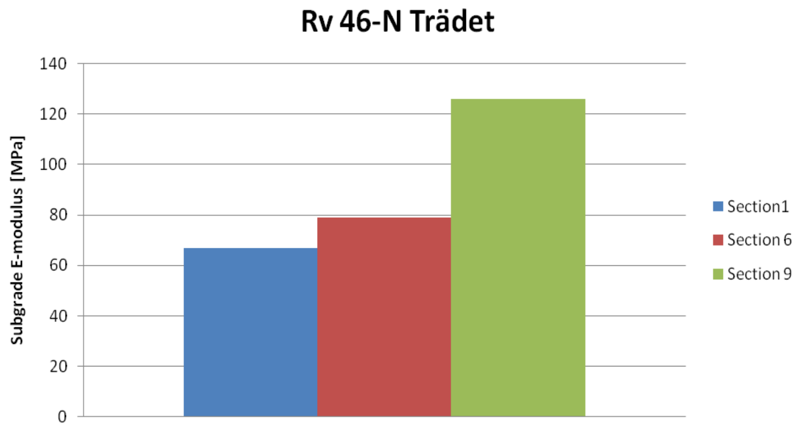


Figure 3-25 Subgrade E-modulus in the traffic lane towards north in Rv 46 at Trädet

The subgrade E-modulus in the opposite direction is larger for section 9, while it is little smaller for sections 6 and 1, see figure 3-26.

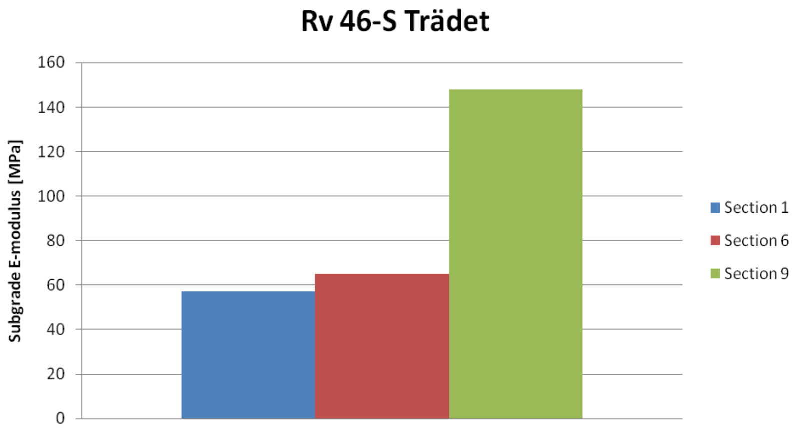


Figure 3-26 Subgrade E-modulus in the traffic lane towards west in Rv 46 at Trädet

Analysis

In the traffic lane towards north, the initial rutting is different for each section. However, the rutting development has the same trend of slowly increase until 2002, when the rutting in section 9 increases rapidly. Noticeable is the crack index that passes 250 at the same period. The development in section 6 is remarkably not following the same trend as section 9 even though the crack index is similar. Section 6 has also lower E-modulus compared to section 9 and the section is constructed on soil cut as well, while section 9 is constructed on embankment. One reason to why section 9 has larger permanent deformations may be linked to the subgrade material, which contains fine-sand. The effect of subgrade material containing smaller fractions is described in chapter 2.6.3 above.

Furthermore, the rutting development in section 6 can be connected to errors in measurement data, since the data was corrected to illustrate a more realistic rutting development. Another probable factor may be fewer cracks in this traffic lane.

Additionally, the rut depth in section 1 is low compared to the other sections. This is due to thicker asphalt layer and subbase in this section compared to sections 6 and 9. Another reason is the low crack index value.

On the other hand, the rutting development is larger in the traffic lane towards south. This is believed to be connected to heavier axle loads in this direction compared to the other direction. The rutting development pattern is similar to the other direction. The permanent deformation is slowly increasing until 2002, when it accelerates. This is believed to be connected to the crack index in this direction as well. Unlike the northern direction, the deepest ruts are observed in section 6. This may be connected to more cracks in this traffic direction compared to the other direction. Also, the surrounding environment may be a contributing factor since there is a slope in this section from west to east. The excavation made in the traffic lane towards south is deeper compared to north, which implies higher pore water pressure in the lane towards south. The high pore water pressure reduces the effective stress, which leads to reduction in the E-modulus, which may cause larger permanent deformation. The circumstance is illustrated in figure 3-27 below.

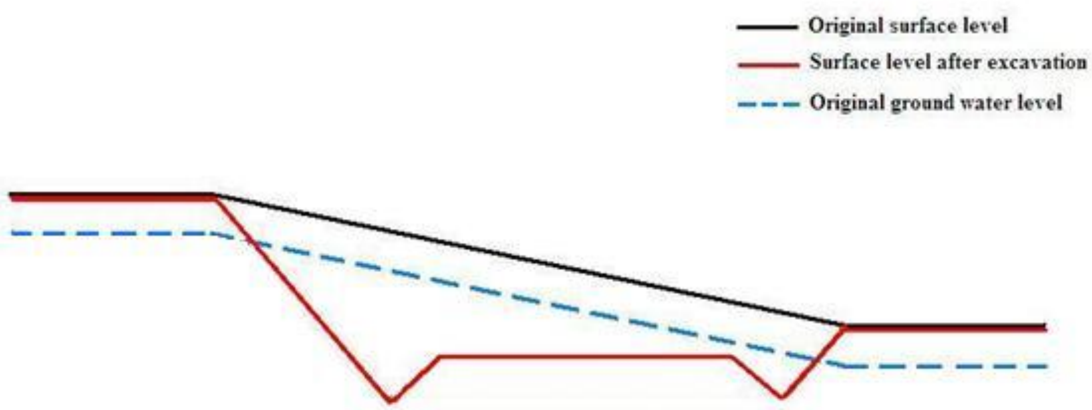


Figure 3-27 Section 6 in Rv 46 at Trädet seen in cross section from south

In addition, the permanent deformation in section 9 follows the same trend as the permanent deformation development in section 6. However, the permanent deformation is slightly smaller in section 9 compared to section 6.

Moreover, the permanent deformation curve in section 1 is similar to its adjacent in the traffic lane towards north.

3.8 E6 Frillesås

E6 is a Swedish motorway and the part of the road included in LTPP is situated close to Frillesås in the province Halland in south-western part of Sweden. The road, consisting of 10 sections, was opened to traffic in 1979 and the latest maintenance was performed in 1999.

Furthermore, the road consists of two 3.5 m wide traffic lanes in each direction. It is located in a coastal landscape with both cuts and embankments. The analysis has been performed in sections 1, 4 and 9, where section 1 is constructed on a rock cut, while sections 4 and 9 are constructed on embankments. The pavement is BS for the section constructed on rock cut and BB for those constructed on embankments. The base is included in the pavement while the subbase material consists of bituminous bound crushed bedrock for all sections with a thickness of 150 mm. The subgrade material in the various sections and their E-modulus are shown in table 3-8 below.

Table 3-8 The type of subgrade material in the various sections and their E-modulus in E6 at Frillesås

Sections	**Type of subgrade material**	**E-modulus [MPa]**
1	Bedrock	268
4	Sandy fine-sand	130
9	Clay	100

The $AADT_{tot}$ was measured to be 19294 vehicles in 2006 with an annual increase of 5.85 per cent. Heavy vehicles stood for 16.81 per cent of $AADT_{tot}$ and according to Enocksson (2010), 95 per cent of the heavy traffic is concentrated on the right lane. However, in the calculations it was assumed to be 100 per cent, which implied in an AADT of 11034 vehicles in the right lane.

Observation of Data

The measurements in E6 Frillesås were only performed in one direction, towards north. Therefore, only the traffic lane towards north is considered. Furthermore, the rutting was largest in section 9 closely followed by sections 4 and 1. The deformations in the analysed sections are similar and linearly increasing, as it is illustrated in figure 3-28.

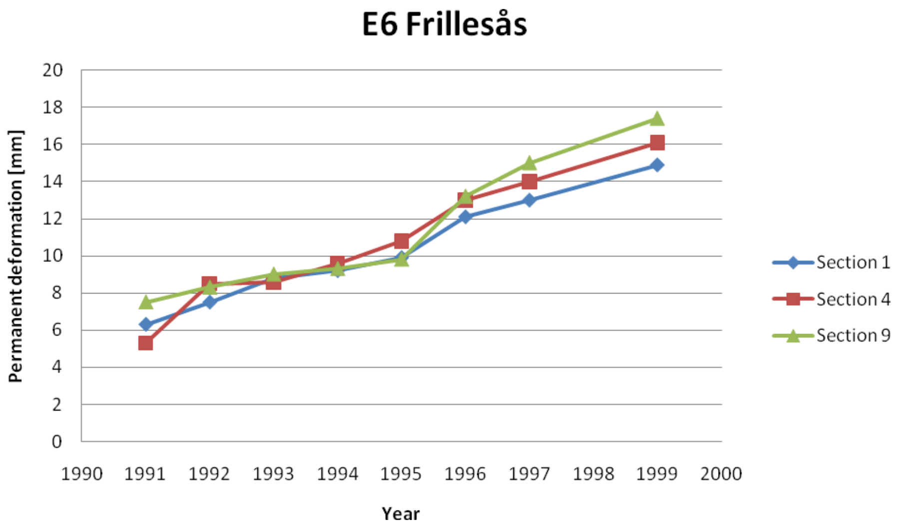


Figure 3-28 Rutting in the traffic lane towards north in E6 at Frillesås

The subgrade E-modulus is largest in section 1, which is constructed on bedrock. The part of the test road consisting of clay has the lowest E-modulus, which is section 9. Section 4, which consists of sandy fine-sand, has larger E-modulus than section 9 and lower than section 1, see figure 3-29.

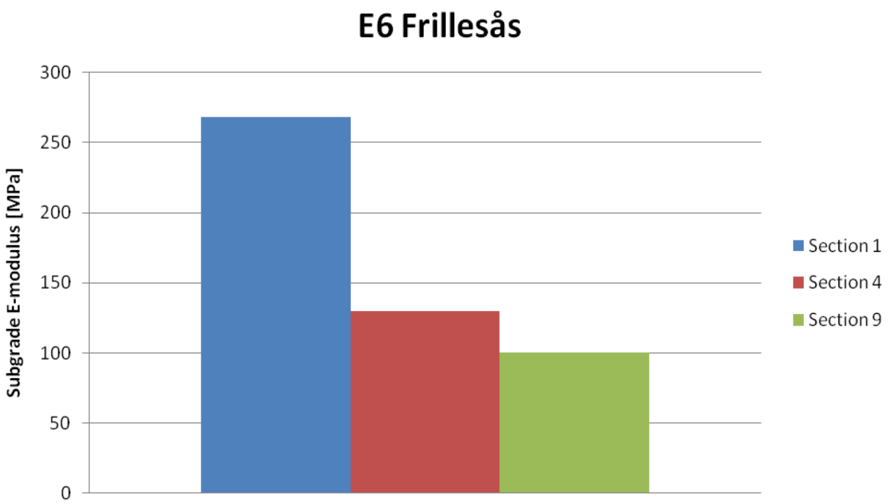


Figure 3-29 Subgrade E-modulus in the analysed section in E6 at Frillesås

The crack index is not of significance in the analysed sections and therefore neglected in the analysis.

Analysis

The shape of the deformation curve that occurs when plotting the rutting against time is uncertain, since there are several corrections made in the rutting measurement data. However, it is concluded that the permanent deformation is linearly increasing with different inclination in each analysed section. The steepness of the inclination depends on the subgrade material and its E-modulus. The largest permanent deformation occurs in section 9, where the subgrade material consists of clay and has lower E-modulus compared to the other sections.

Furthermore, section 9 is followed by section 4, where the subgrade material consists of sandy fine-sand. Section 1 has smaller permanent deformation since it is constructed on a rock cut.

In addition, section 1 has large permanent deformation even though it is constructed on rock cut. However, since it is a cut the groundwater table is closer to the road surface in this section compared to the other sections and this may affect the water content in the unbound material, which results in larger permanent deformation.

Moreover, it is believed that there are some errors in the FWD-data since the difference between the subgrade E-modulus is only marginal for the section constructed on clay and for the section constructed on bedrock. Bedrock is a more solid material compared to clay and therefore should have a much higher E-modulus.

3.9 E6 Tvååker

This LTPP-road is situated close to Tvååker in the province Halland in south-western part of Sweden, close to the other test road in Frillesås, and consists of 10 sections. The road was opened to traffic 1988 and the latest maintenance was performed in 1991 for section 1 to 4 and in 2001 for section 5 to 10. The analysis is made with data from 1992 to 2004.

Furthermore, the test road consists of two 3.75 m wide traffic lanes in each direction. The road is located in a coastal landscape with only soil cuts, except in section 1 where the road is levelled with the terrain. The pavement is GBÖ 90 in all sections. Both the base and subbase consists of the same material and the same thickness, which are 120 mm for the base and 600 mm for the subbase. Moreover, the subgrade material consists of clay with different E-modulus for the various sections.

The $AADT_{tot}$ was measured to be 18746 vehicles in 2004 with an annual increase of 7.35 per cent. The proportion of heavy vehicles is 16.81 per cent and the $AADT_{RL}$ was 10949 for the same assumptions as in previous chapter 3.8.

Observation of Data

The measurement in this LTPP-road was also only performed in the traffic lanes towards north. The sections are divided into two parts. Part one includes section 1 to 4 and part two includes section 5 to 10. This is due to the maintenance performed in 2001 in part two.

In the first part, sections 1 and 4 were selected to be analysed in detail. Between these two sections, section 4 has the largest permanent deformation, see figure 3-30.

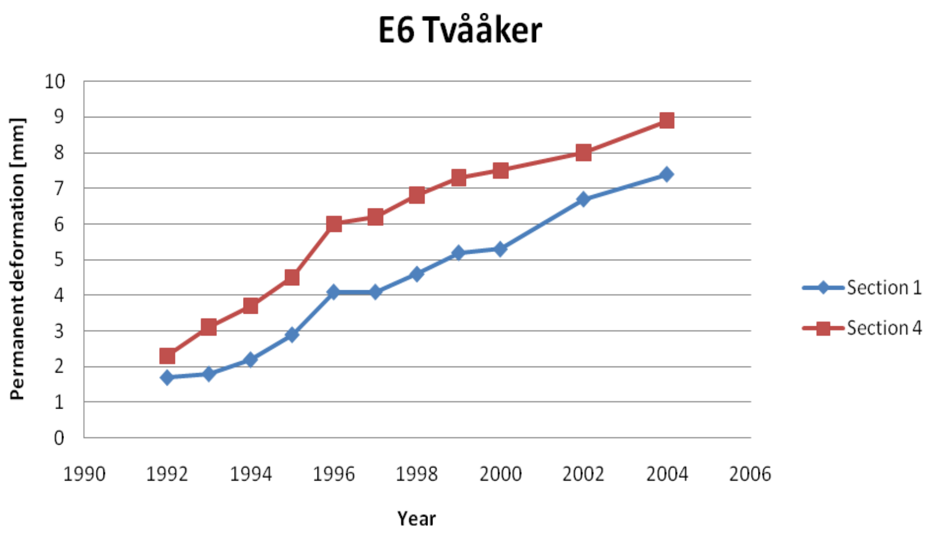


Figure 3-30 Rutting in section 1 and 4 in the traffic lane towards north in E6 at Tvååker

The subgrade E-modulus was lower in section 4 compared to section 1, as shown in figure 3-31.

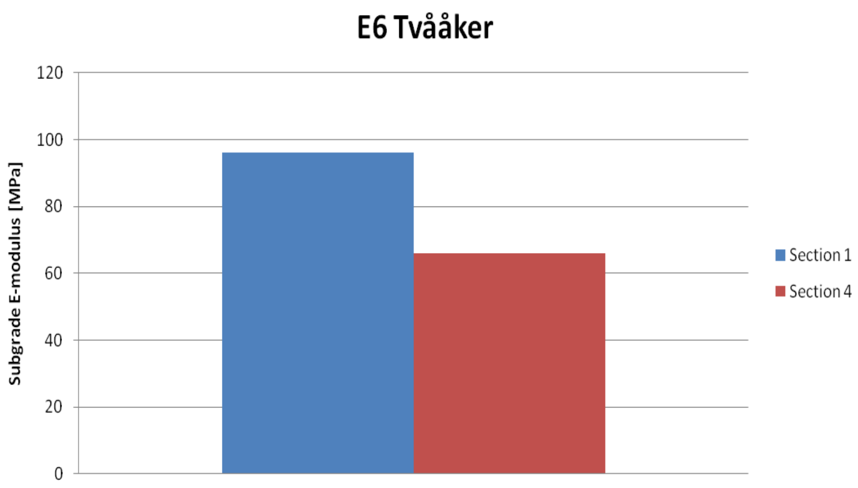


Figure 3-31 Subgrade E-modulus in section 1 and 4 in E6 at Frillesås

Considering the second part, sections 5 and 10 were chosen for further analysis. As illustrated in figure 3-32, section 5 has developed the largest permanent deformation.

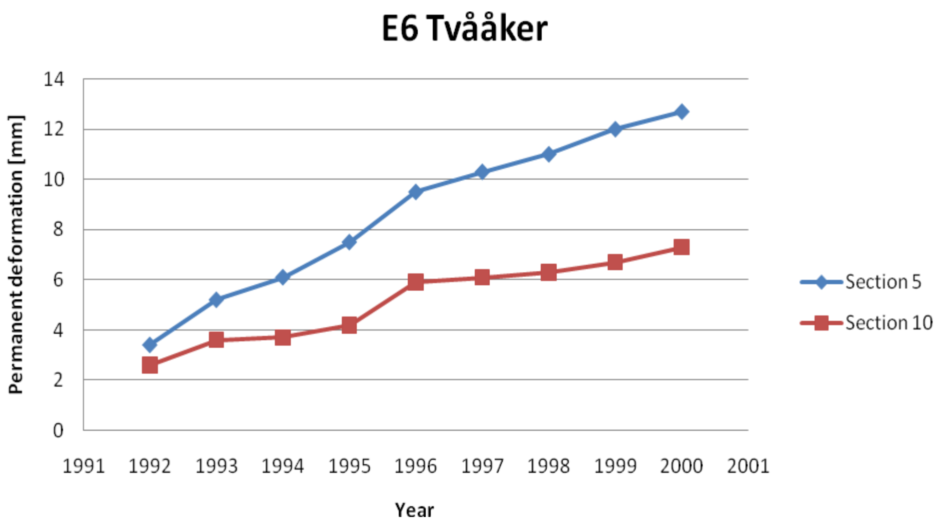


Figure 3-32 Rutting in section 5 and 10 in the traffic lane towards north in E6 at Tvååker

On the other hand, section 5 has slightly lower subgrade E-modulus compared to section 10, see figure 3-33.

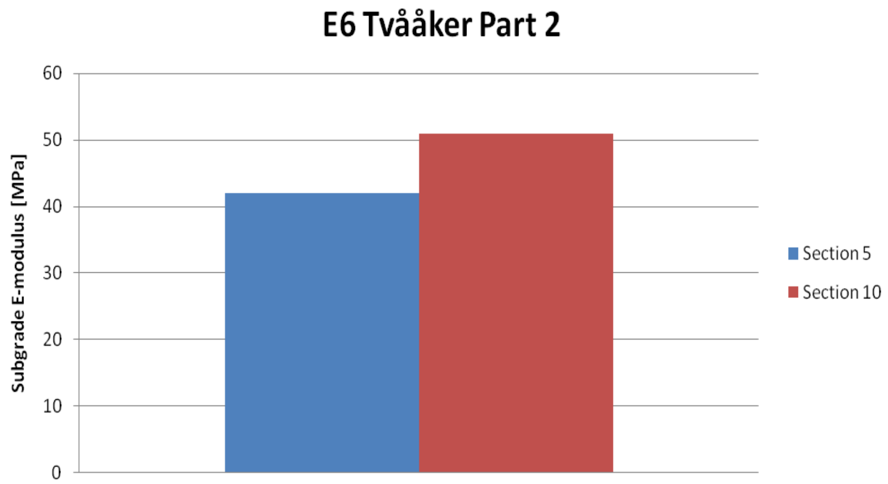


Figure 3-33 Subgrade E-modulus in section 5 and 10 in E6 at Frillesås

The crack index was neglected since insignificant cracks were registered in the measurements for the various sections.

Analysis

From the observation made in the first part, it can be concluded that the permanent deformation is highly influenced by the subgrade E-modulus. The rutting development is linearly increasing in both sections with different inclination depending on the subgrade E-modulus. Section 4 has lower subgrade E-modulus compared to section 1, which results in larger permanent deformation in section 4. Additionally, another contributing factor is believed to be the higher water content in the unbound material, since section 4 is constructed on soil cut while section 1 is levelled with the surrounding terrain.

On the other hand, the inclination of the permanent deformation curve, in the second part, is two times larger in section 5 compared to section 10. The permanent deformation is also linearly increasing with different inclination in these sections as in the previous ones. The steepness of the inclination of the deformation curves is believed to be connected to the subgrade E-modulus. The subgrade E-modulus is lower in section 5 compared to section 10 and therefore steeper.

Moreover, another contributing factor may be the water content. However, there was no field visit at this road thus the effect of this is uncertain.

4 Results from the Analyses

4.1 Crack index

From the analyses it can be concluded that there is an obvious correlation between the permanent deformation and the crack index. The permanent deformation increases drastically if the crack index passes 250. This is observed in Rv 53 at Nyköping, Rv 33 at Vimmerby, Rv 34 at Målilla, Rv 46 at Trädet and Rv 44 at Grästorp.

In order to find out the percentage of additional rutting that is caused by cracks, the rutting is plotted against time. The rutting development is assumed to follow the same development pattern if the crack index does not reach a crack index value of 250. Therefore, a black line is drawn until the end of the measurements, see figure 4-1. Subsequently, by dividing the actual rutting (red line) with the rutting observed at the end of the “black line” that date, a percentage is obtained.

Moreover, the largest response against cracks in the surface layer is observed in section 7 in Nyköping. The permanent deformation increased 107 per cent between 2002 and 2006, when the crack index passed 250, see figure 4-1 and 4-2. This corresponds to an annual increase of 27 per cent, which is an extreme case since other roads has not reached high values as in Nyköping.

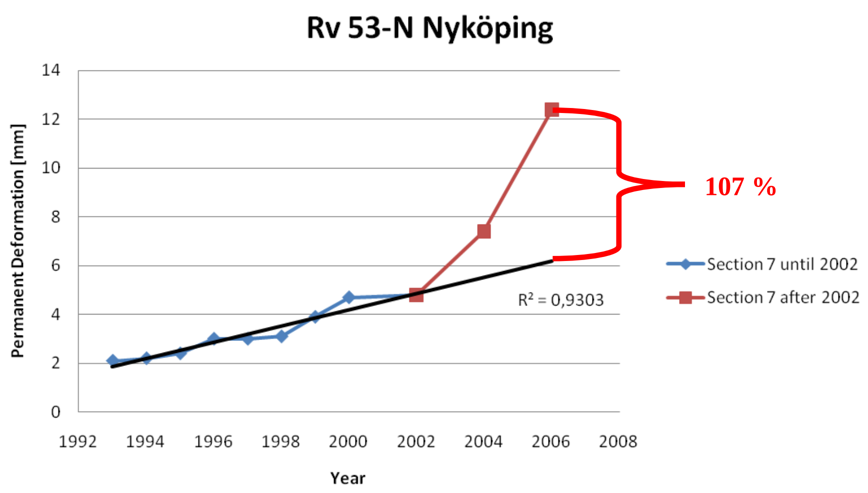


Figure 4-1 Rutting development in section 7 towards north in Rv 53 at Nyköping

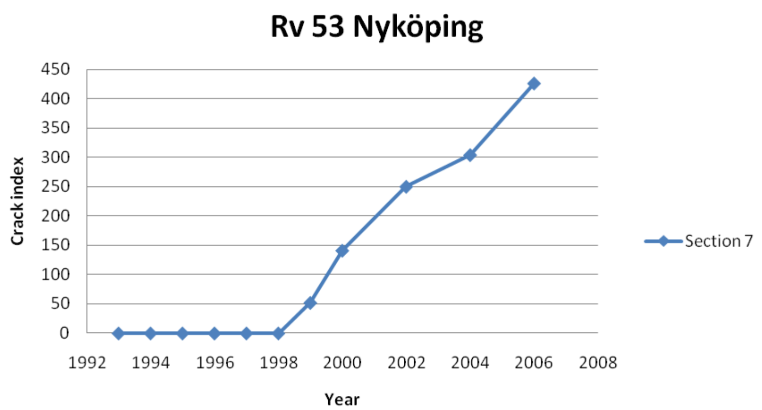


Figure 4-2 Crack index in section 7 in Rv 53 at Nyköping

In addition, roads with subgrade material consisting of silt, clay or fine-sand the annual increase of the permanent deformation was between 10 and 20 per cent when the crack index passed 250. This occurred in Rv 46 at Trädet and Rv 44 at Grästorp. Section 1 towards east in Rv 44 at Grästorp is illustrated in figure 4-3 below as an example of this. Notice the crack index 1998 in figure 4-4 as well.

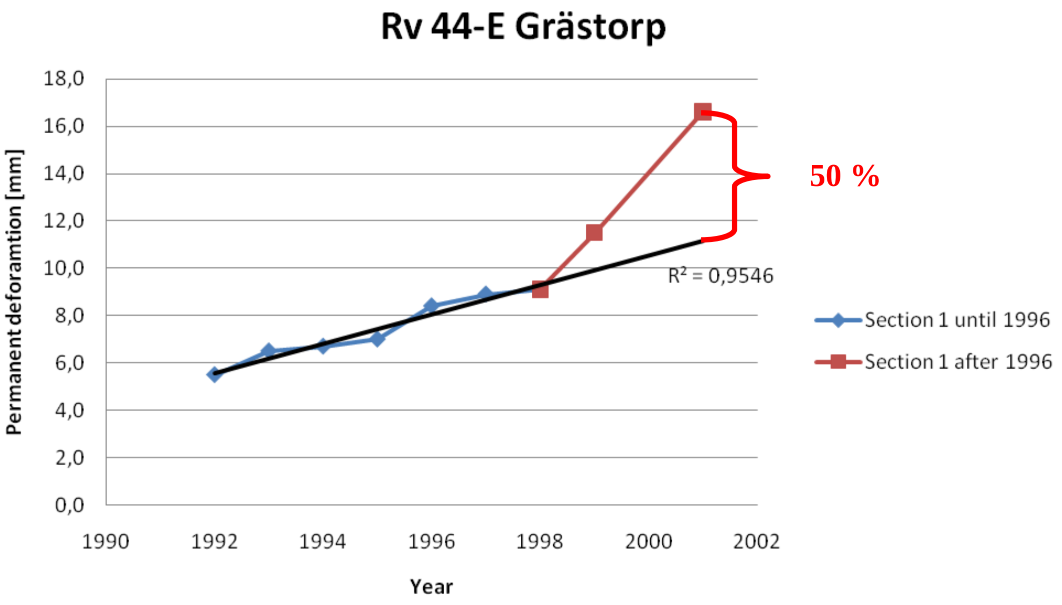


Figure 4-3 Rutting development in section 1 towards east in Rv 44 at Grästorp

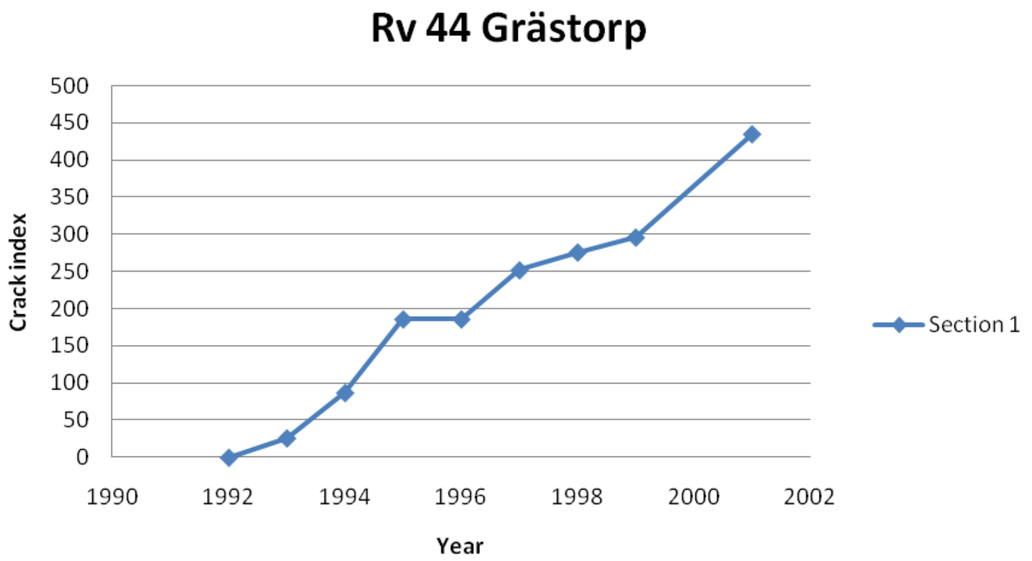


Figure 4-4 Crack index in section 1 in Rv 44 at Grästorp

Moreover, in roads constructed on subgrade consisting of granular material with larger fractions such as sand, gravel or moraine, the annual increase of the permanent deformation was smaller, between 5 and 10 per cent, when the crack index passed 250. This occurred in RV 34 at Målilla, Rv 46 at Trädet and Rv 33 at Vimmerby. Section 7 towards west in Rv 33 at Vimmerby is an example of this occurrence, which is illustrated in figure 4-5 below and the crack index is illustrated in figure 4-6.

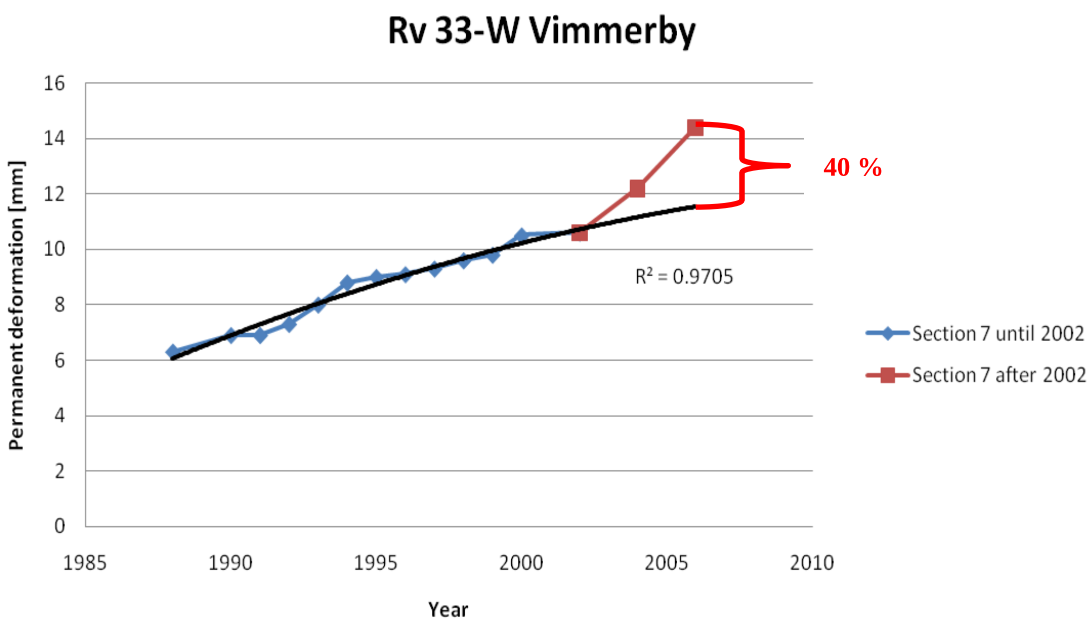


Figure 4-5 Rutting development in section 7 towards west in Rv 33 at Vimmerby

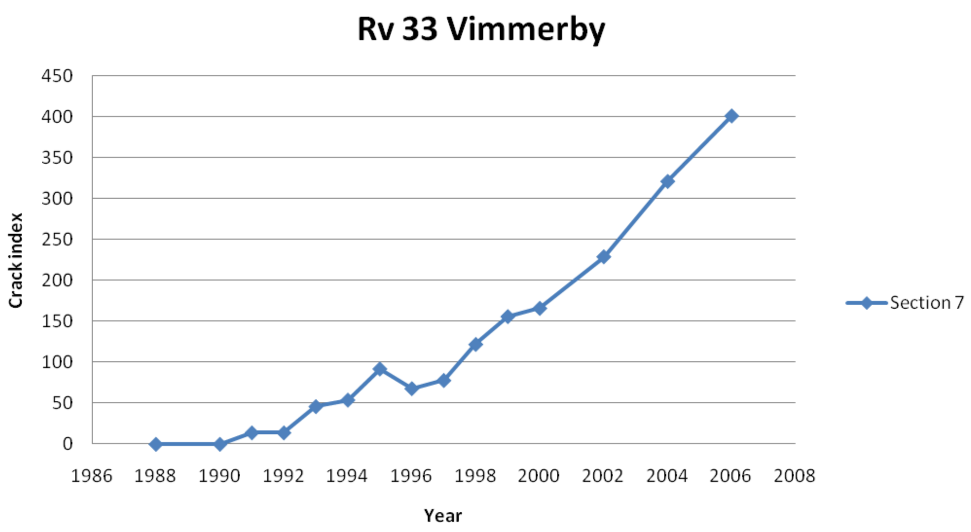


Figure 4-6 Crack index in section 7 Rv 33 at Vimmerby

Eventually, in roads with low or zero crack index the permanent deformation has not showed the same pattern, a sudden increase. Therefore, it can be underlined that there is an apparent link between the sudden increase of the permanent deformation and crack index especially when the crack index exceeds 250.

Table 4-1 below illustrates road sections where the crack index has an influence on the rapid increase of the rutting development after reaching a crack index 250. See appendix II for the figures illustrating the correlations in the various sections.

Table 4-1 The correlation between the crack index and permanent deformation in various sections

Road	Section	Correlation	Subgrade type	Annual rutting increase [Si > 250]
Rv 53-N	7	Good	Silt/silty sand	27 %
Rv 53-S	7	None	Silt/silty sand	-
Rv 33-E	7	Good	Fine-sand partly on bedrock/ gravelly sand	6 %
	8	Good	Gravelly sand/sandy gravelly moraine/fine-sand	10 %
Rv 33-W	7	Good	Fine-sand partly on bedrock/ gravelly sand	9 %
	8	None	Gravelly sand/sandy gravelly moraine/fine-sand	-
Rv 34-N	1	Good	Sand	9 %
	6	Good	Sand	10 %
	9	Fair	Sand	5 %
Rv34-S	1	None	Sand	-
	6	Good	Sand	6 %
	9	Good	Sand	13 %
Rv 44-E	1	Good	Silt/clay	13 %
	9	Good	Silt/clay	12 %
Rv 44-W	1	Fair	Silt/clay	7 %
	9	Good	Silt/clay	14 %
Rv 46-N	6	None	Sandy gravel	-
	9	Good	Fine-sandy sandy gravel	17 %
Rv 46-S	6	Good	Sandy gravel	7 %
	9	Good	Fine-sandy sandy gravel	10 %

4.2 Embankment / Cut

The analyses made in the LTPP-roads shows that the part of the roads constructed on soil cuts has larger permanent deformation compared to the parts constructed on embankments, only if the roads are constructed on homogeneous subgrade material and has a crack index less than 250. This circumstance is observed at Rv 31 at Nässjö, where section 6 is constructed on soil cut and has larger rutting compared to sections constructed on embankments. However, this theory cannot be applied in roads with crack index above 250. Despite that the sections are constructed on soil cuts the permanent deformation is smaller compared to sections constructed on embankments with larger crack index value. This indicates that the influence of crack index is more crucial compared to whether the road is constructed on an embankment or a cut. This is observed in the analysed sections in Rv 34 at Målilla, Rv 44 at Grästorp and Rv 46 at Trädet.

In addition, sections constructed on rock cuts have smaller permanent deformation in general. This is probably correlated to their high E-modulus.

4.3 Subgrade type and E-modulus

Roads constructed on subgrade material consisting of clay, silt or fine-sand the analyses show a linearly increasing rutting development when plotted against time. There is no stagnation nor substantial increase of the rutting observed in these plots. The inclination of the rutting development curve is based on the subgrade E-modulus. The lower E-modulus the steeper inclination in the rutting curves. This occurs in some of the sections in Rv 53 at Nyköping, Rv 31 at Nässjö, Rv 33 at Ankarsrum, E6 at Frillesås and E6 at Tvååker. Figure 4-7 below illustrates the occurrence in Rv 31 at Nässjö where the subgrade material consists of fine-sandy moraine and the rutting development is almost linearly over the years. The R^2-value is more than 95 per cent in each section. The subgrade E-modulus for these sections is shown in figure 4-8 below.

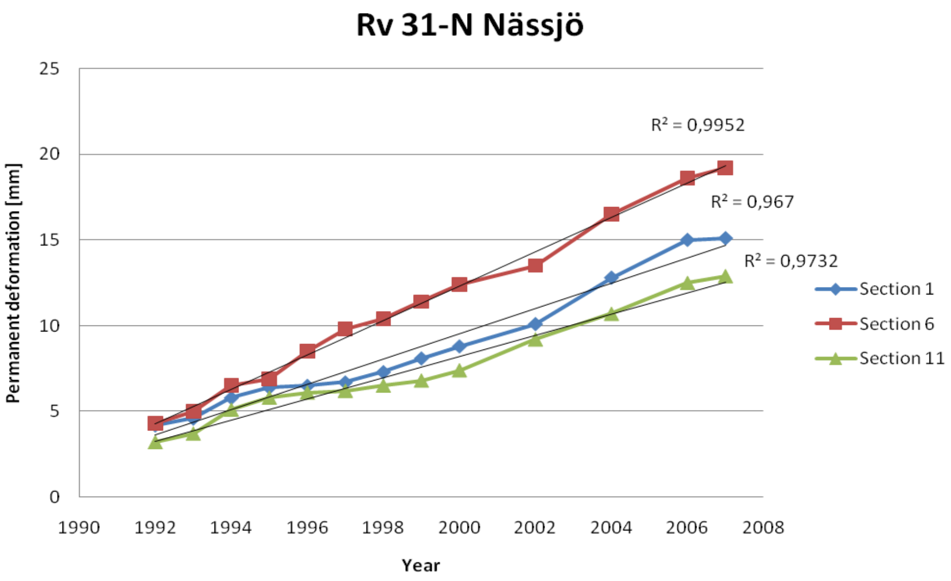


Figure 4-7 Rutting in the traffic lane towards north in Rv 31 at Nässjö

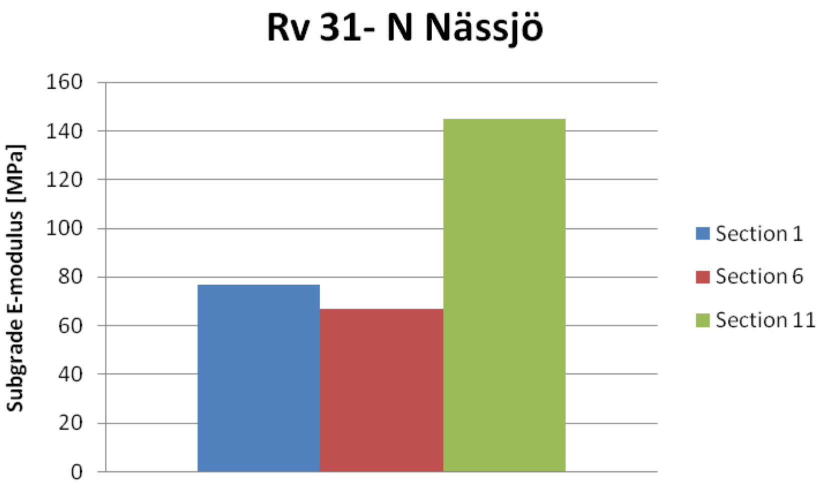


Figure 4-8 Subgrade E-modulus in the traffic lane towards north in Rv 31 at Nässjö

Furthermore, in roads constructed on subgrade consisting of material with larger fractions such as sand, stone, moraine and bedrock the rutting development is dissimilar. Unlike the permanent deformation development caused by the smaller fractions, the development stagnates in the sections consisting of these materials as long as it is zero or low crack index. This is observed in some sections in Rv 44 at Grästorp, Rv 33 at Vimmerby and Rv 33 at Ankarsrum. Figure 4-9 illustrates the rutting development in Rv 44 at Grästorp, where the subgrade material is made up of bedrock in section 6 and has considerably higher E-modulus compared to the other sections.

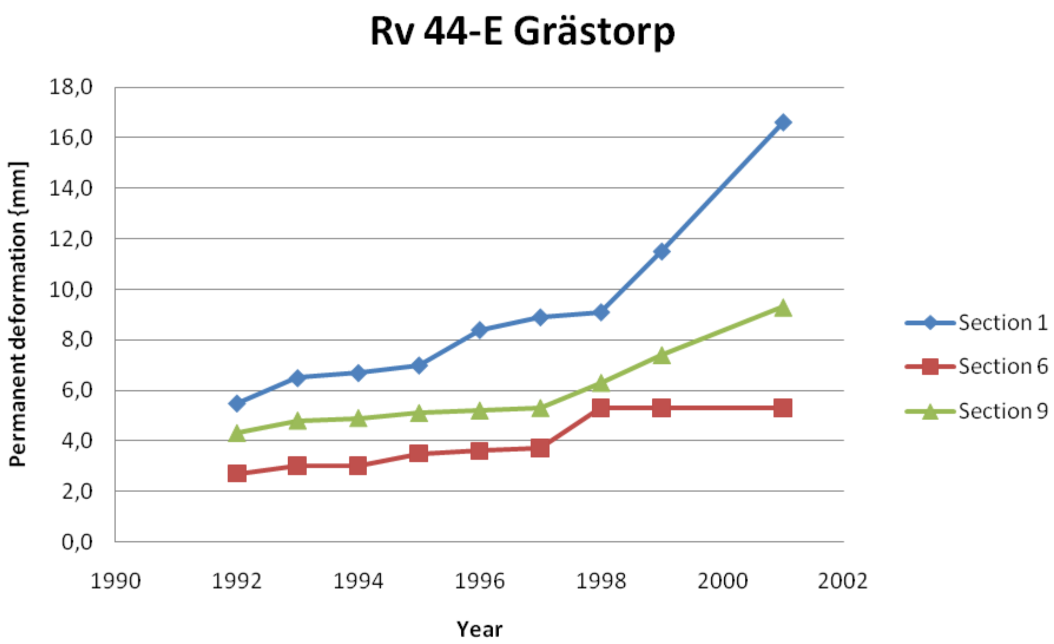


Figure 4-9 Rutting in the traffic lane towards east in Rv 44 at Grästorp

Moreover, in Rv 33 at Ankarsrum it is observed that the rutting is more dependent on the subgrade material than the subgrade E-modulus, see figure 4-10 and 4-11. Section 1 consists of stony clayey moraine and has lower E-modulus compared to section 10 that consists partly of clay and partly of moraine. Probably, the effect of pure clay is larger on the permanent deformation, which results in larger permanent deformation.

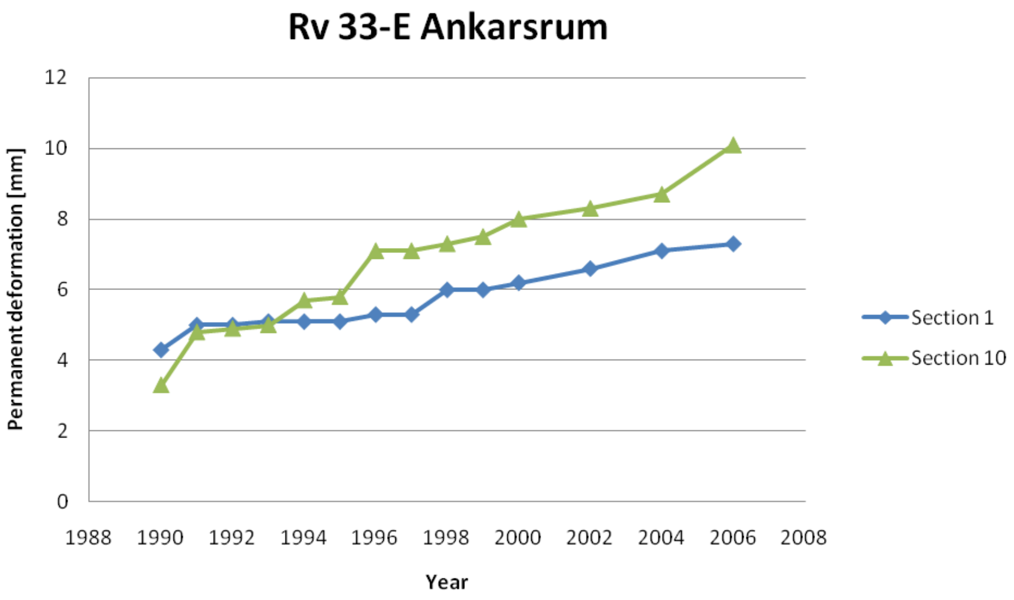


Figure 4-10 Rutting in the traffic lane towards west in Rv 33 at Ankarsrum

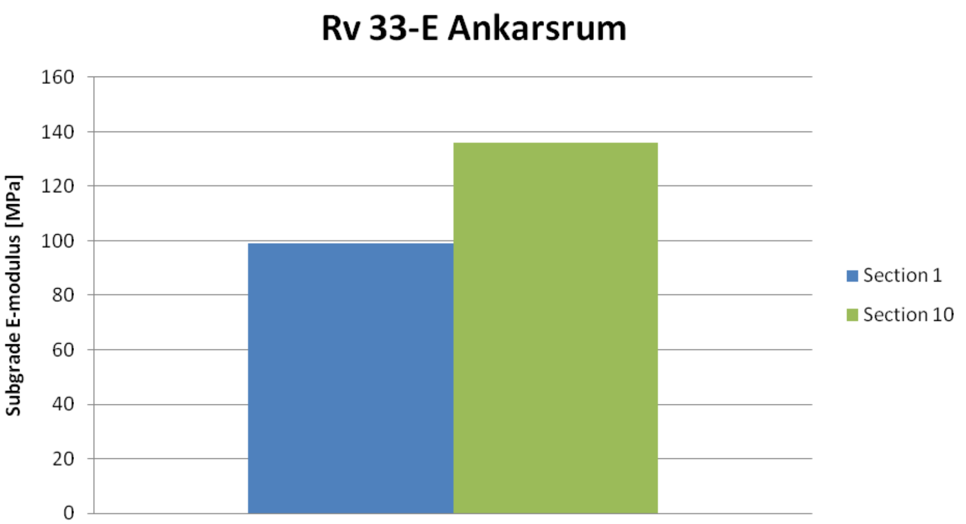


Figure 4-11 Subgrade E-modulus in the traffic lane towards west in Rv 33 at Ankarsrum

4.4 Direct sunlight

The effect of direct sunlight is similar in Rv 31 at Nässjö, Rv 33 at Ankarsrum and Rv 46 at Trädet since there is no shading on these sections. On the other hand, in Rv 33 at Vimmerby and Rv 34 at Målilla the circumstances are different due to the surrounding terrain consisting of tall trees in some sections. This is particularly observed in section 8 at Rv 33 and section 1 in Rv 34 where the trees shade the road. Therefore, the rutting was substantially lower in these sections. The effect of direct sunlight in section 8 at Rv 33 at Vimmerby is illustrated, see figure 4-12.

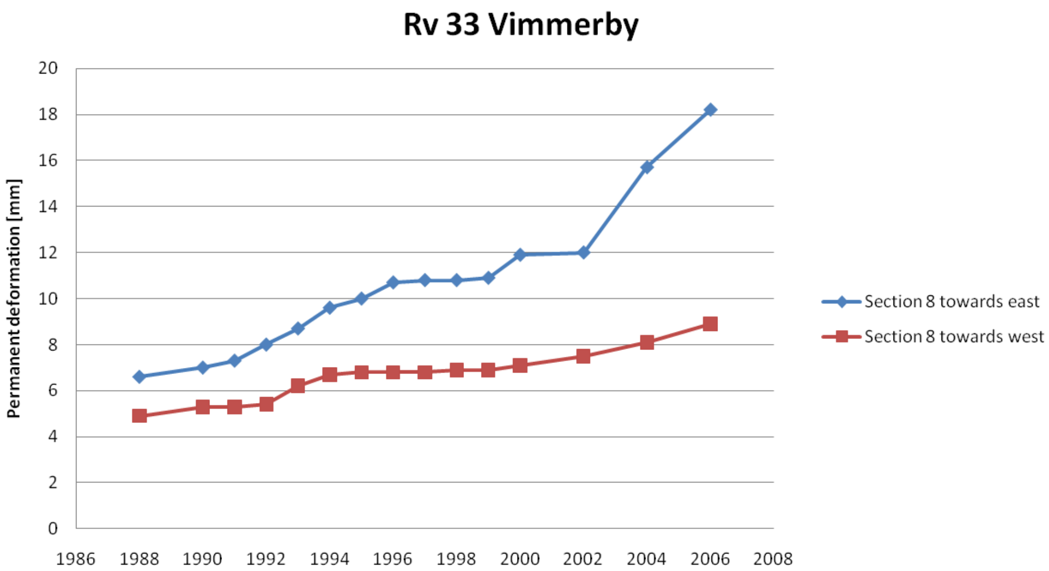


Figure 4-12 Rutting in the traffic lanes in Rv 33 at Vimmerby

Furthermore, it is believed that the rutting in section 8 towards east is affected by other factors beyond direct sunlight as well. These factors determine the extent of the permanent deformations as it is explained in chapter 3.3.

4.5 Measurement errors

As previously mentioned there is several measurement errors registered in the raw data obtained from the LTPP-database. In some sections the rutting development was undulating when the rutting was plotted against time, see appendix I. Logically, this is unreasonable since the permanent deformation should increase each year due to wearing caused by studded tyres. However, the obtained rutting data was modified to show a more realistic development. This may have affected the shape of the deformation curve in some sections, which gave a different shape of the deformation curve than the actual one. Therefore, it is believed that the measurement errors have a major influence on the accuracy of analyses.

Furthermore, another source of error is the FWD measurements. The data obtained from FWD measurements, which has been used to calculate the subgrade E-modulus, is believed to contain errors. The reason to this is the marginal differences in subgrade E-modulus in some sections, which consists of different subgrade material type such as bedrock and clay. Bedrock is a more solid material compared to clay and therefore should have a much higher E-modulus.

5 VägFEM analysis

This chapter was planned to contain the results obtained from the VägFEM analyses. Unfortunately, due to problems in the installation of this program at the Swedish Road Administration the analyses could not be carried out. However, samples on the subgrade material were taken from the LTPP-roads in Rv31 at Nässjö and in Rv 46 at Trädet to be analysed in triaxial tests. These samples were tested in different conditions, wet and dry. Results obtained from the triaxial tests would subsequently be used as input values in VägFEM, together with the previous triaxial test results carried out on the unbound materials in each road. VägFEM would provide the principal stresses that the material in the different layers was exposed to. These stresses would be plotted in the Theyse's "Range graph" in order to determine in what range the stresses would end up in. Furthermore, the effect of the different conditions with emphases on the water content in the subgrade and its E-modulus would be analysed and the correlation between these parameters and the permanent deformation would be drawn.

6 Possible Improvement Techniques

There are several improvement techniques available in order to minimize the rutting development in the road pavement. These techniques are preferred according to the occurrence of various soil types in the area, available knowledge and experience from previous road pavements. In Sweden, knowledge about improvement techniques is very limited and the use of these methods are restricted compared to other countries e.g. Germany. Some of the possible improvement techniques are briefly described in the following subchapters.

6.1 Soil Improvement Technique

Soil improvement is used to reduce the deterioration process that implies rotation of the particles and particle separation in the road pavement especially in the unbound layers. Since this layer does not have any tensile strength, it is essential that the support in the subgrade is sufficient so that the rutting development can be minimized. Previous studies and experience indicate that soil improvement contributes to an increased life span of the road pavement and decreased maintenance cost. The main advantages of soil improvement can be seen below.

- Increased strength and durability
- Decreased negative effects of water such as frost and expansion of the soils
- Improvement of workability of the soil
- Decreased overall stress-level and a better distribution of the load.
- Decreased layer thickness
- Increased life span and decreased maintenance cost.

Despite all the advantages of soil improvement, this method is rarely used in Sweden. The major reasons, except the high costs of executing this method, are restricted knowledge and lack of equipment that can manage the thick layers of weak subgrade material such as clay. This method has only been applied in some test sections at various locations to analyse whether the method reduces the rutting or not. The process of soil stabilization and the advantages of this method are further described in the master thesis written by Eriksson & Svensson (2009).

6.2 Higher Aggregate Quality

The capacity of the road to withstand deterioration is affected by several factors. Properties such as grain size, grain shape and grain mineralogy of the material in the unbound layers determines the strength capacity of the road pavement. Therefore, in order to reduce the rutting, it is important that aggregates with high quality are used in all layers of the road pavement. Furthermore, it is recommended that crushed gravel and other materials with similar surface characteristics are used. This type of material is easier to compact compared to rounded aggregates and the post- compaction can be reduced. Moreover, a higher content of fine and organic matter makes the material in the unbound layers more sensitive to high water content and subsequently result in larger permanent deformation. Consequently, it is recommended that granular material with small fine content and organic matter is used in order to drain the water away through the layers, Lekarp (1997).

6.3 Modified Asphalt Mixture

The type of asphalt mixture and material composition in the wearing course strongly affect the size of the permanent deformation. When the aggregates in the asphalt mixture are covered by bitumen, an almost impermeable layer is created. This impermeable layer in its turn prevents water to drain into the road pavement and as a consequence the negative effect of the water can be minimized. The bitumen in the wearing course is highly temperature dependent. Therefore, it is essential that appropriate asphalt mixture is used based on the climate situation in the area. It is common that special additives e.g. polymers are mixed with the asphalt to obtain a desired asphalt mixture. Previous studies made on asphalt material indicate that addition of polymers in the asphalt is more preferable than regular asphalt mixture. The polymer-based asphalt can withstand tensile stresses and strain much better compared to regular asphalt which in its turn develop smaller permanent deformation, Nilsson & Huvstig (2009).

6.4 Thicker Layers

The stresses developed due to traffic loading must be distributed over the road pavement so that less stresses are conducted down to the subgrade. The E-modulus is highest at the top of the pavement, in the wearing course, and much lower in the unbound layers further down in the pavement. The subgrade has considerably the lowest E-modulus in the pavement. Therefore, it is important that the thickness of the pavement layers is dimensioned according to the traffic load and subgrade condition. Thicker pavement layers can be used if the road will be constructed on subgrade consisting of poor material e.g. clay. Previous studies made by Huvstig indicate that thicker layers increases the initial construction cost. However, it reduces the long term costs due to less maintenance costs.

6.5 Higher Compaction Level

It is well known from previous studies that the level of compaction has a significant importance for the long term behaviour of the granular material in the road pavement. According to Huvstig a higher compaction level drastically decreases the post-compaction of the granular material that occurs due to repeated traffic loading. The importance of compaction is further described in chapter 2.5.1.1.

6.6 Use of Intermediate Layer

According to experience an intermediate layer with higher E-modulus than the subgrade but lower E-modulus than the overlaying unbound layers can be used to reduce the tensile stresses that appear in the unbound layers especially in the subgrade, Huvstig (2009). This layer is usually consisting of sand. This process is illustrated in the figures 6-1 and 6-2 below.

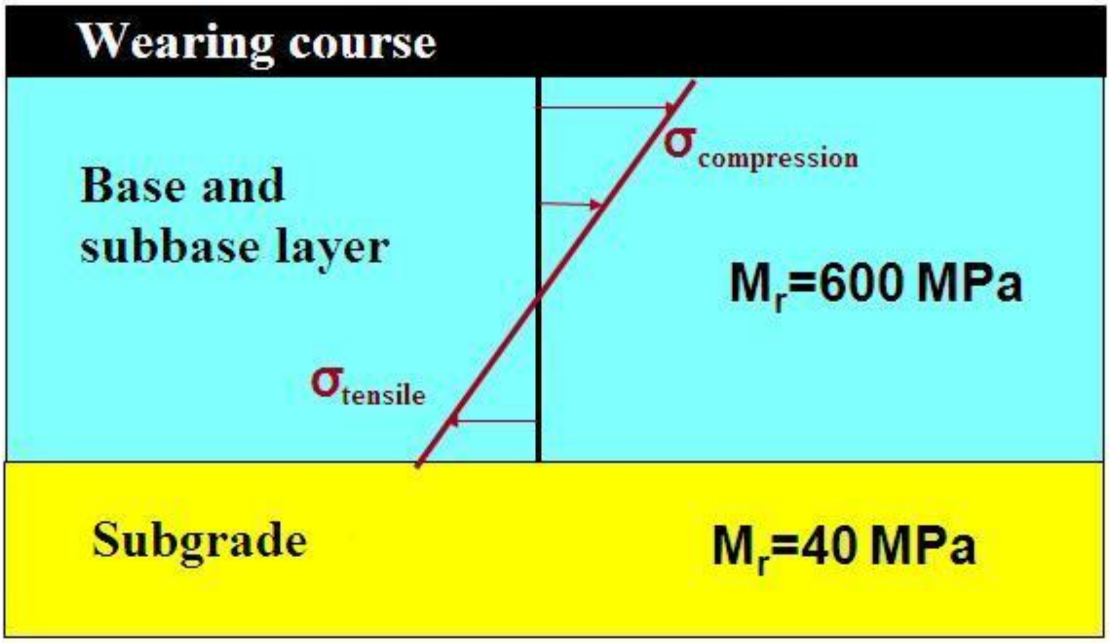


Figure 6-1 Stresses occurring in the pavement construction (Huvstig 2009)

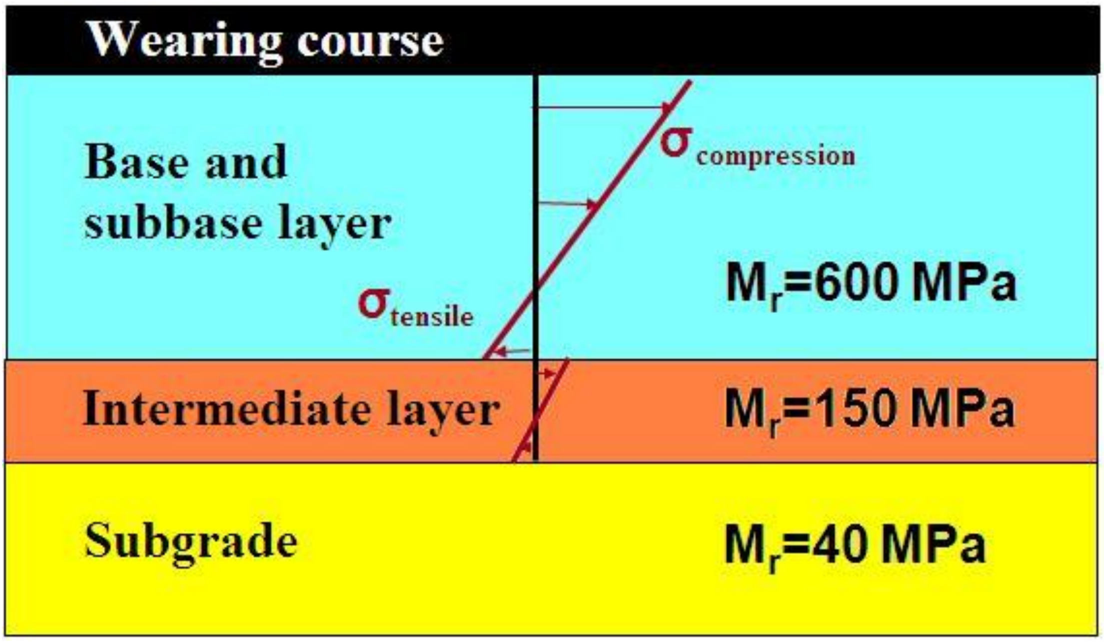


Figure 6-2 Stresses occurring in pavement construction after applying the intermediate layer (Huvstig 2009)

7 Discussion

The results from the analyses contain some uncertainties. This is mainly caused by the errors in the rutting measurements performed by the RST vehicle, which has lead to corrections in the rutting raw data. According to the RST measurements the rutting development decreases during some years in some sections, which is unreasonable since there is some rutting caused by studded tyres each year. This has affected the results in the analyses due to corrections that had to be made in order to create more realistic rutting development in some sections. The outcome of the corrections is that the rutting development curve has been affected, which causes uncertainty in the analyses. Therefore, it is believed that the measurement errors in the rutting data have a major influence on the accuracy of the results obtained from the analyses.

Furthermore, the subgrade E-modulus that is calculated based on FWD data obtained from the LTPP-database is believed to contain some errors as well. In some sections the difference between the subgrade E-modulus is just fairly, even if the subgrade material type differs. Bedrock is a more solid material compared to clay but the difference in subgrade E-modulus is marginal in some roads, for instance in E6 at Frillesås between sections constructed on these materials. This is irrelevant and believed to be caused by the measurement errors.

Additionally, a detail worth to mention is that the subgrade E-modulus calculated in this project is an average value of FWD measurements performed in different seasons; spring, summer and autumn. This may also have affected the calculated subgrade E-modulus as well.

8 Conclusions

The purpose of this thesis was to identify parameters affecting the permanent deformation and to investigate the correlation between these parameters and the permanent deformation. This project would provide basis to the development of new models for better rutting prediction in future.

From the analyses made it is ascertained that the major influence on the escalation of the permanent deformation development was caused by cracking. The crack index is the indicator of cracking and it is obvious that the permanent deformation escalates when the crack index exceeds a value of 250. This value is considered as the limit, where the permanent deformation starts to increase continuously. The effect of the crack index is highly dependent on the subgrade material. In roads constructed on subgrade material consisting of clay, silt and fine-sand the effect is larger and the increase of the permanent deformation development per year between 10 and 20 per cent in the analysed LTPP-roads. On the other hand, in roads constructed on materials with larger fractions such as sand, gravel and moraine the effect is smaller and the increase of the permanent deformation development per year between 5 and 10 per cent.

Furthermore, the analyses of the LTPP-roads indicate that the subgrade material determines the shape of the permanent deformation development when it is plotted against time. It is clearly visible that on the analyses carried out in roads constructed on subgrade material consisting of clay, silt and fine-sand the permanent deformation development is linearly increasing over the years. On the other hand, roads constructed on subgrade material consisting of sand, gravel, moraine and bedrock the permanent deformation development stagnates over the years. Concerning the subgrade E-modulus, correlations have been found in roads that are entirely constructed on homogeneous subgrade material. In sections where the subgrade E-modulus is lower the permanent deformation is larger.

In addition, the direct sunlight has also a crucial role on the permanent deformation development. The effect of direct sunlight is only observed in two road sections in which the sunlight is shaded and causes considerably smaller permanent deformation. This may have occurred in other roads as well but due to limited information of roads that have not been visited it cannot be proved.

Another studied parameter of importance is the water content in the subgrade material. The importance of this parameter is observed in roads that are constructed on same subgrade material but in different formation type, on cuts, where the groundwater level is closer to the road surface. The roads constructed on cuts have larger permanent deformation development compared to roads constructed on embankments. However, this is only occurring for roads consisting of homogenous subgrade material and a crack index lower than 250. The effect of the water content could not been proved since the software, VägFEM, that were supposed to be used in this purpose was out of service.

Eventually, the parameters that are mentioned above should be included in both new and existing rutting prediction models to achieve a better prediction of permanent deformation development.

9 Recommendations

In order to evaluate the effect of the parameters that contributes to permanent deformation development more accurately, further analysis has to be done. In the measurement data obtained from the LTPP-database the reliability was insufficient due to unreasonable rutting development. The obtained rutting data was modified to show a more realistic development. Therefore, in order to increase the reliability of future rutting measurements it is recommended to use manual rut measurement instead of RST. Previous investigation made by Anders Huvstig (2009) indicates that measurements performed with a beam (manual measurement) shows approximately 40 per cent larger permanent deformation than the measurements performed by RST.

In order to identify the causes behind the permanent deformation the entire pavement should be considered. Deeper analysis should be carried out in the different pavement layers as well as in the subgrade material. The first step in this context is to make triaxial tests on both subgrade and unbound materials in each test road. This should be performed to increase the knowledge and understanding of how these materials behave in different conditions, mainly in wet and dry conditions. The reason to why these tests should be performed in each road is that all test roads are made up of different materials with different properties and therefore behaves dissimilar in varying conditions. The emphasis is made on the water content.

The second step is to perform tests on the asphalt to analyse the effect and behaviour in different conditions. Beyond these tests, field investigation can be done during hot summer days to visually observe the behaviour of the asphalt in both direct sunlight and in shade, which is linked to the temperature in the asphalt.

Further analysis of the causes of cracking is recommended since it has a crucial impact on rutting development as underlined in this thesis. It is recommended that the crack index is calculated per lane and section instead of per section. In addition, it is also recommended that studies have to be made on how to improve the asphalt mixture to increase the resistance against stresses and thereby prevent cracking.

Furthermore, there is a need of further researches of frozen ground and thawing and how these affect the rutting development. This has to be analysed in order to understand the behaviour of the pavement during these conditions.

It is further recommended that the knowledge about the different improvement techniques on weak subgrade such as clay and silt should be improved. This can be done by applying these techniques in new roads that would be constructed on weak subgrade materials to monitor the result.

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Appendices

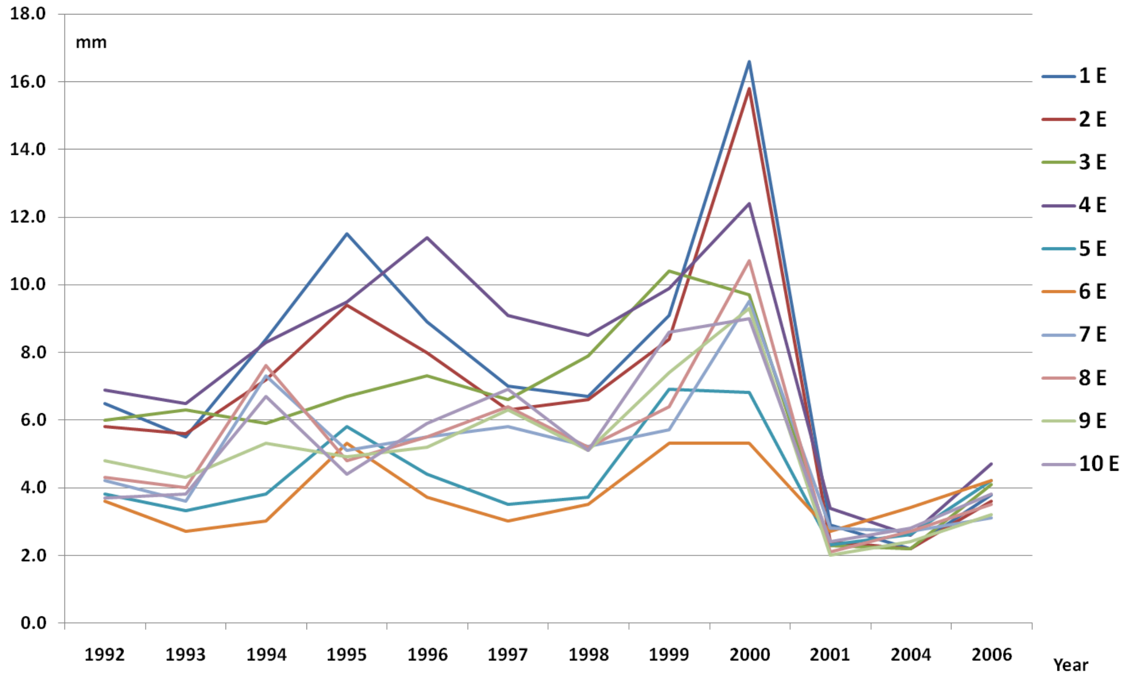
I. Example of Raw Data on an LTPP-road

II. Results from the Crack Index Analysis

III. VägFEM

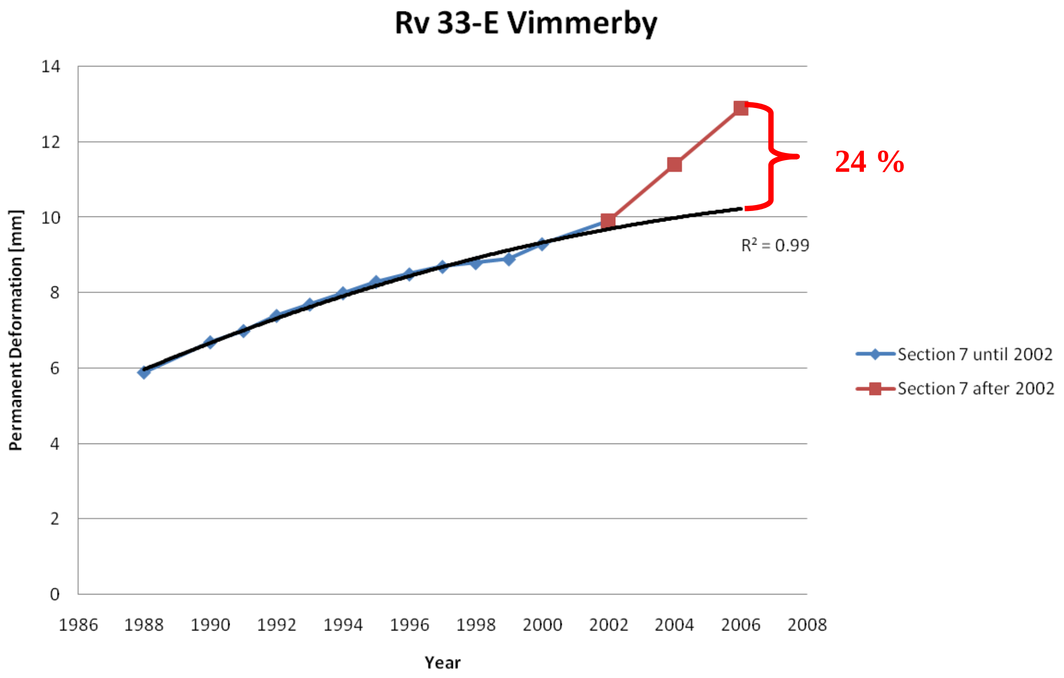
I. Example of Raw Data on an LTPP-road

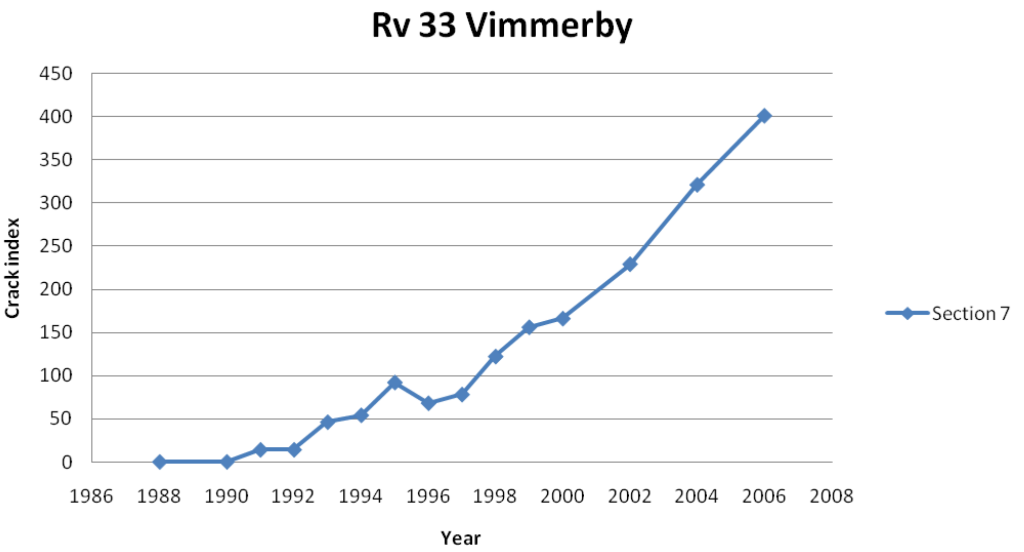
An example from raw data on rutting from LTPP-database measured with RST in each section of the LTPP-road in Rv 44 at Grästorp towards east.



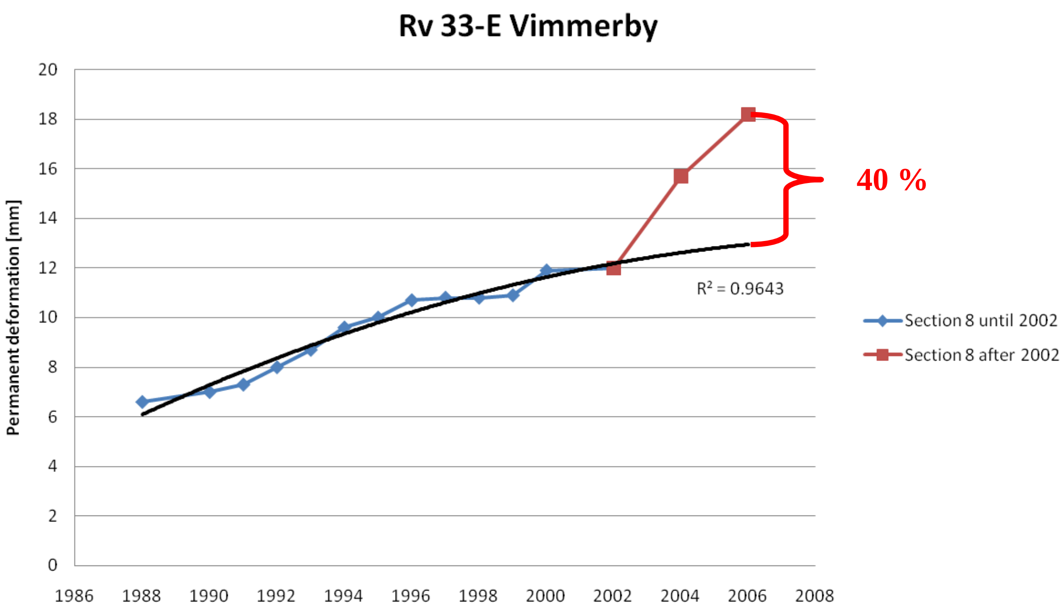
II. Results from Crack Index Analysis

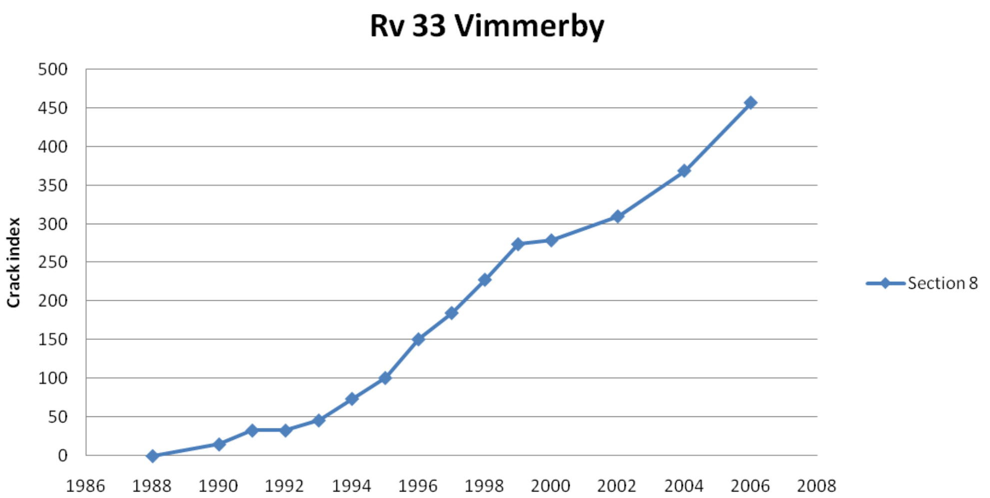
Results from the crack index analysis at Rv 33 towards east in section 7.



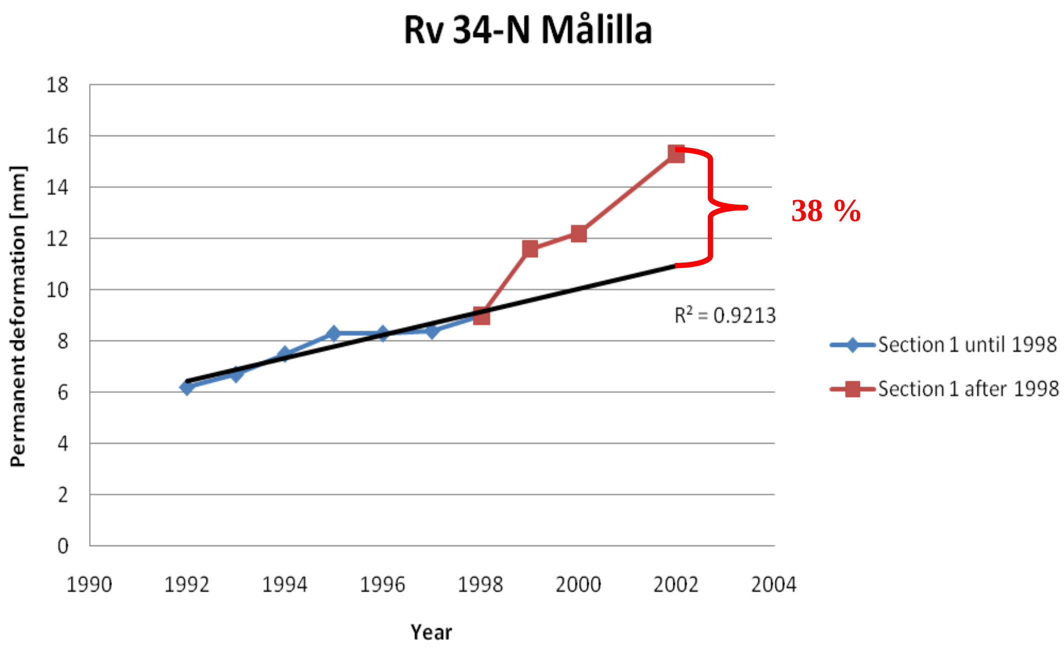


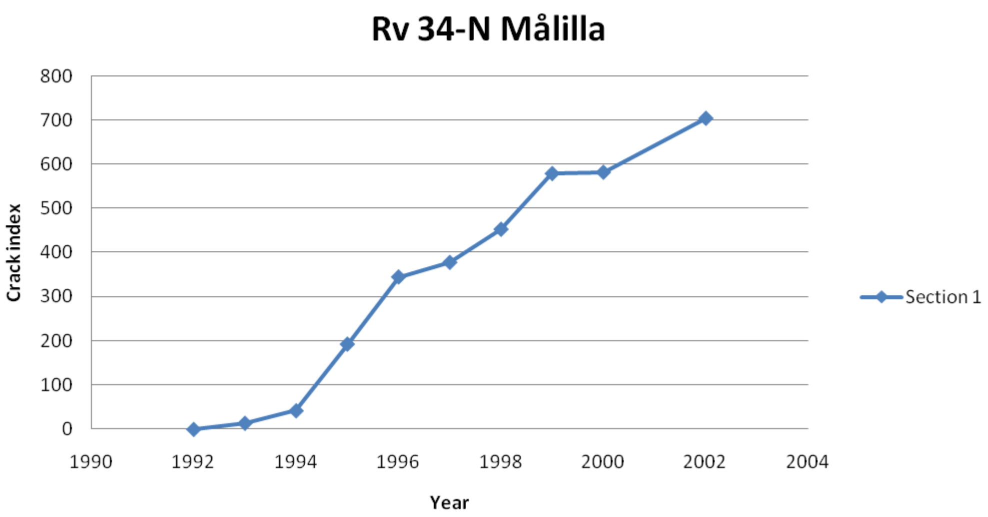
Results from the crack index analysis at Rv 33 towards east in section 8.



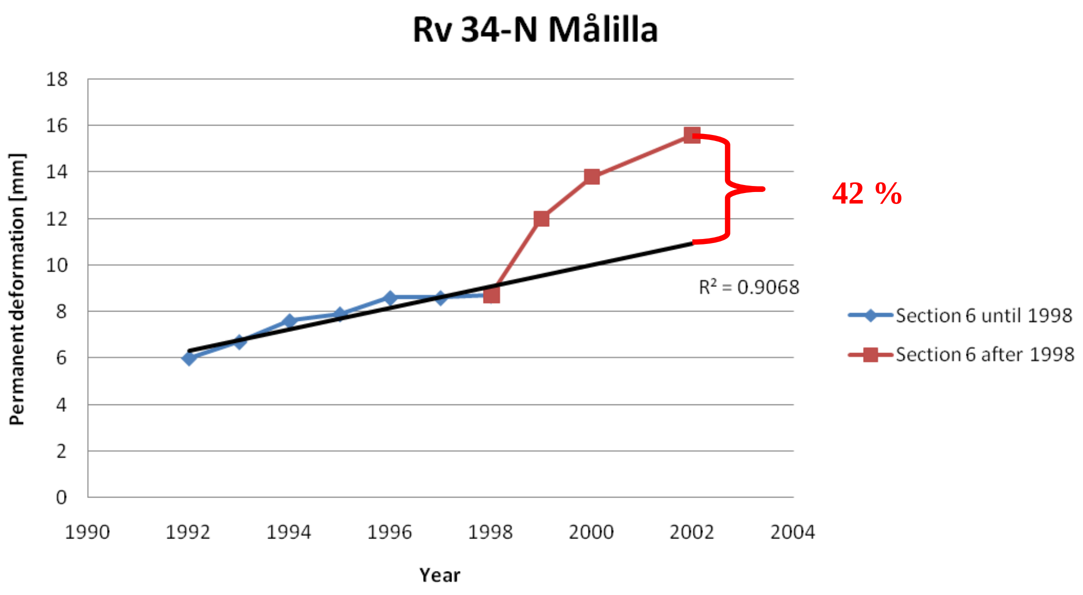


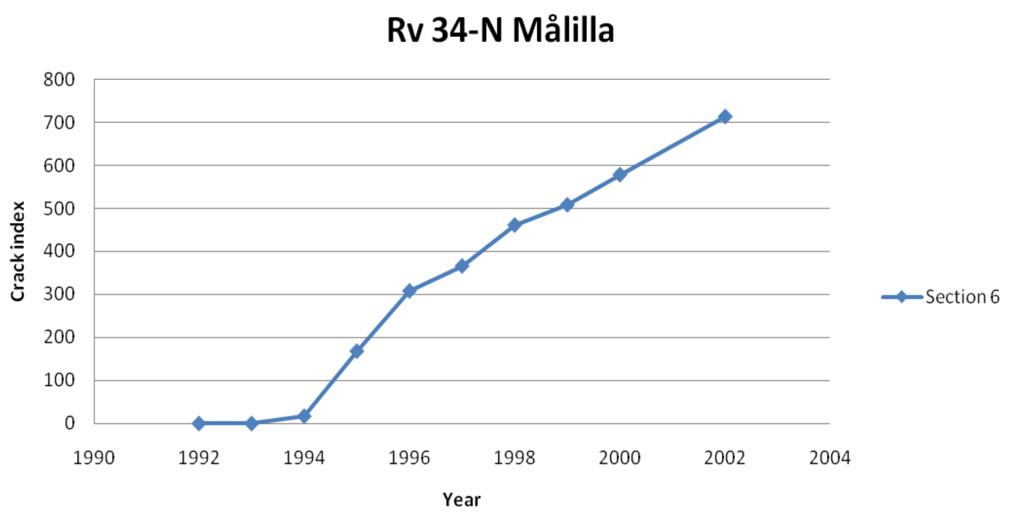
Results from the crack index analysis at Rv 34 towards north in section 1.



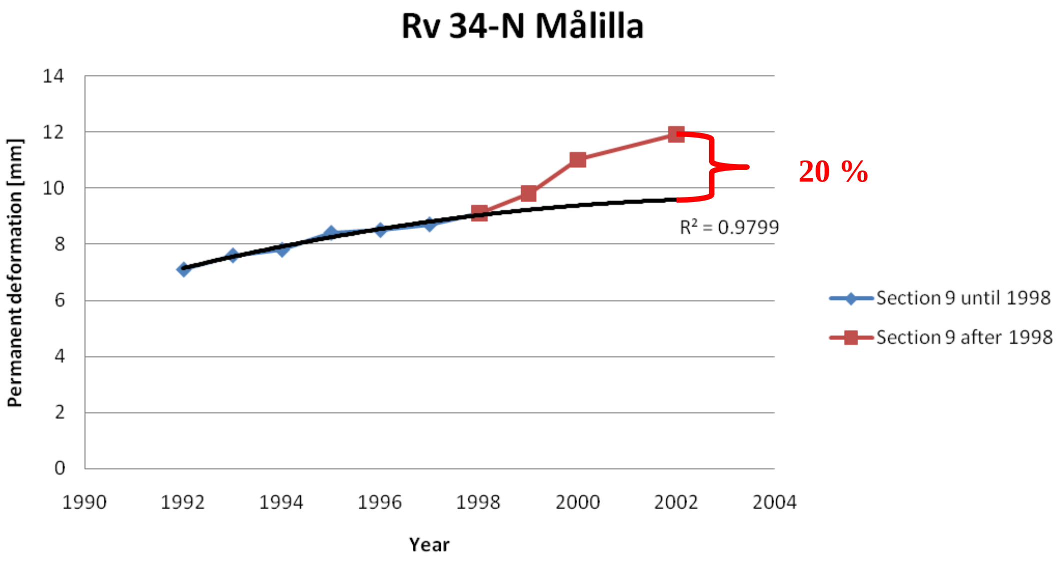


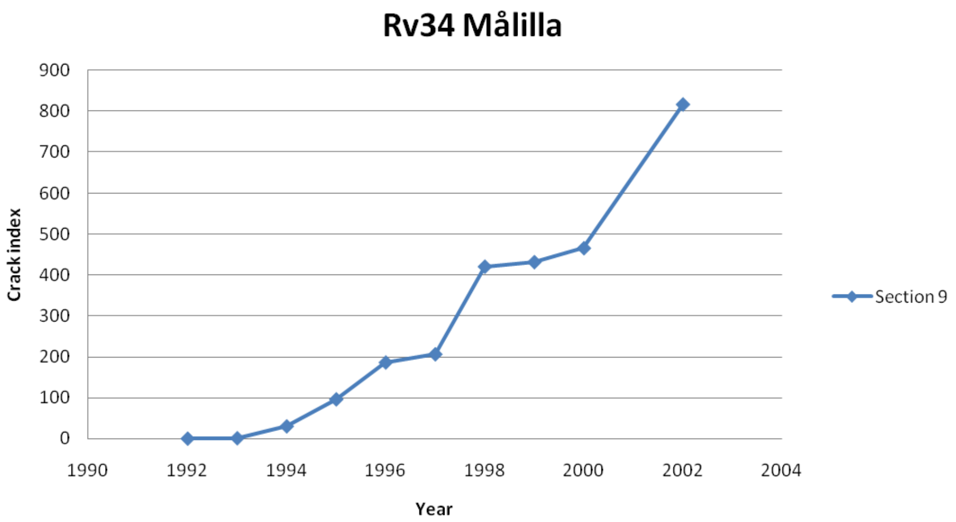
Results from the crack index analysis at Rv 34 towards north in section 6.



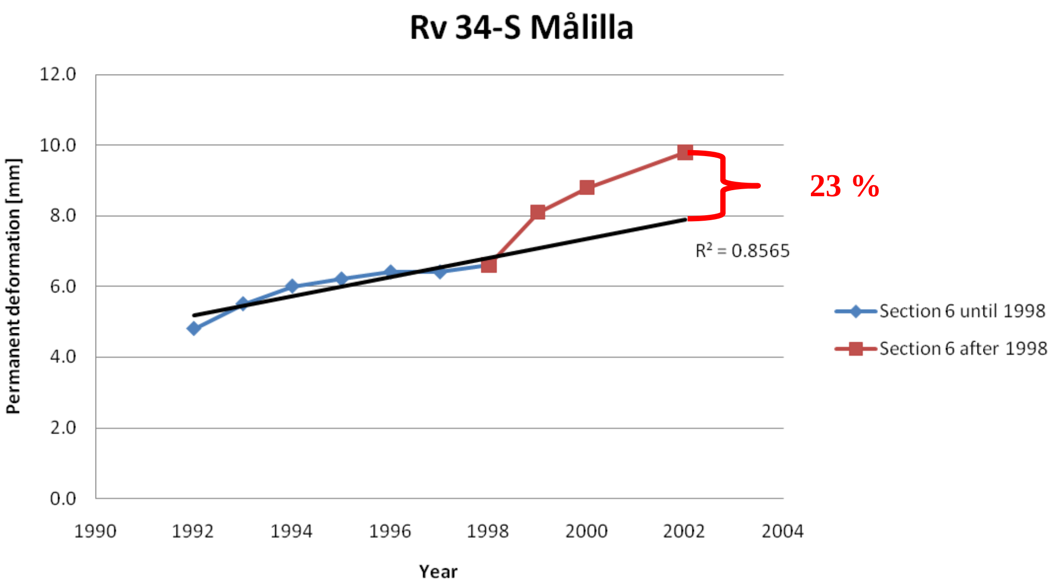


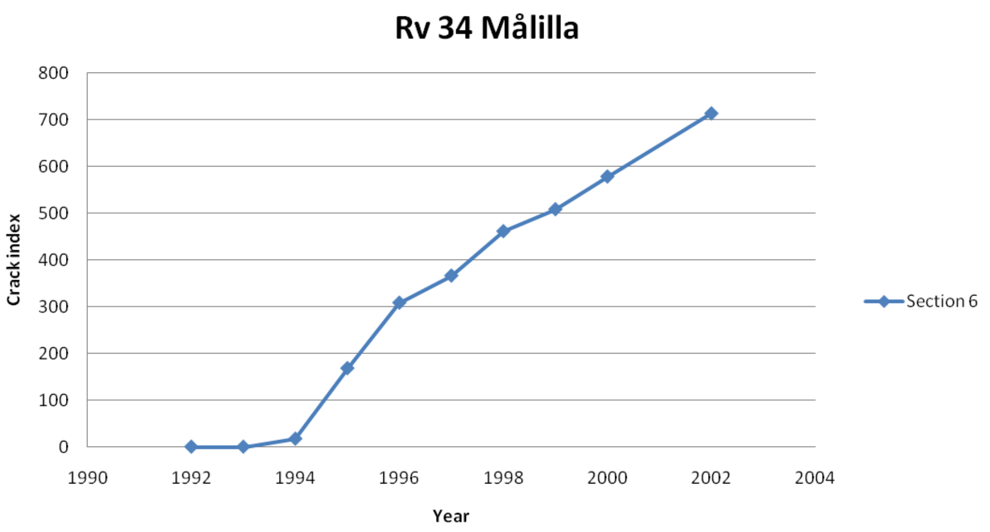
Results from the crack index analysis at Rv 34 towards north in section 9.



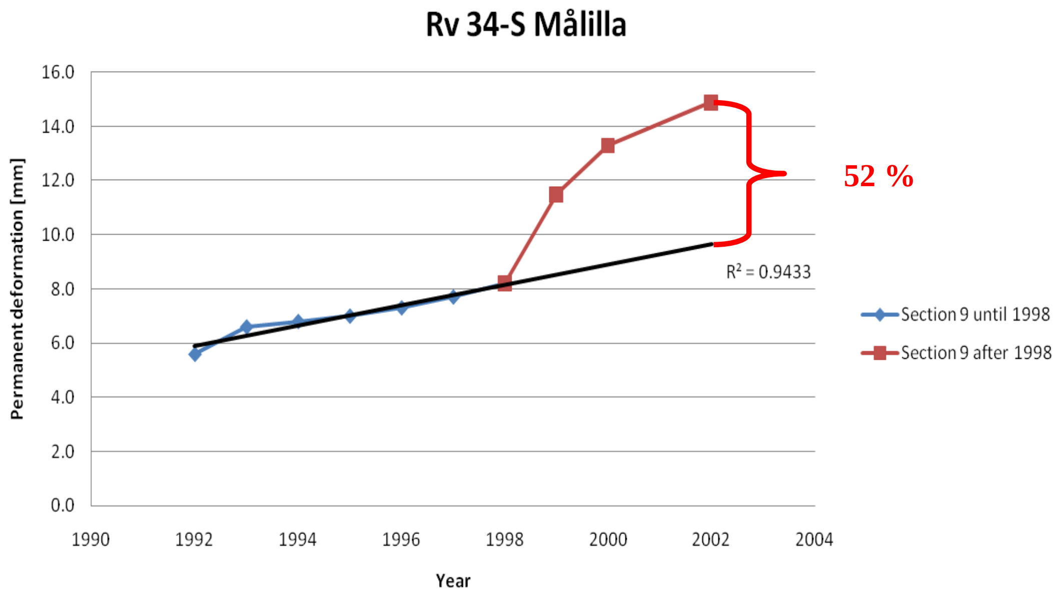


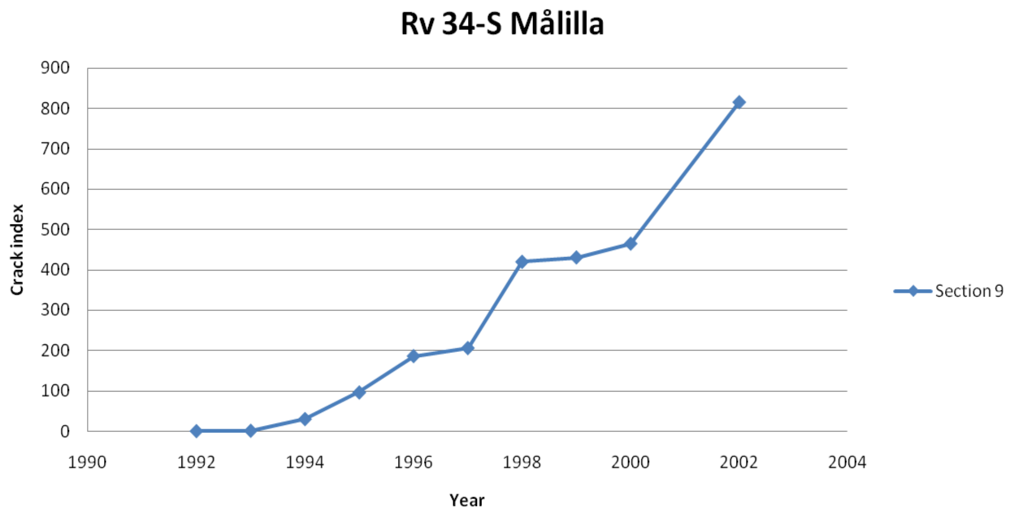
Results from the crack index analysis at Rv 34 towards south in section 6.



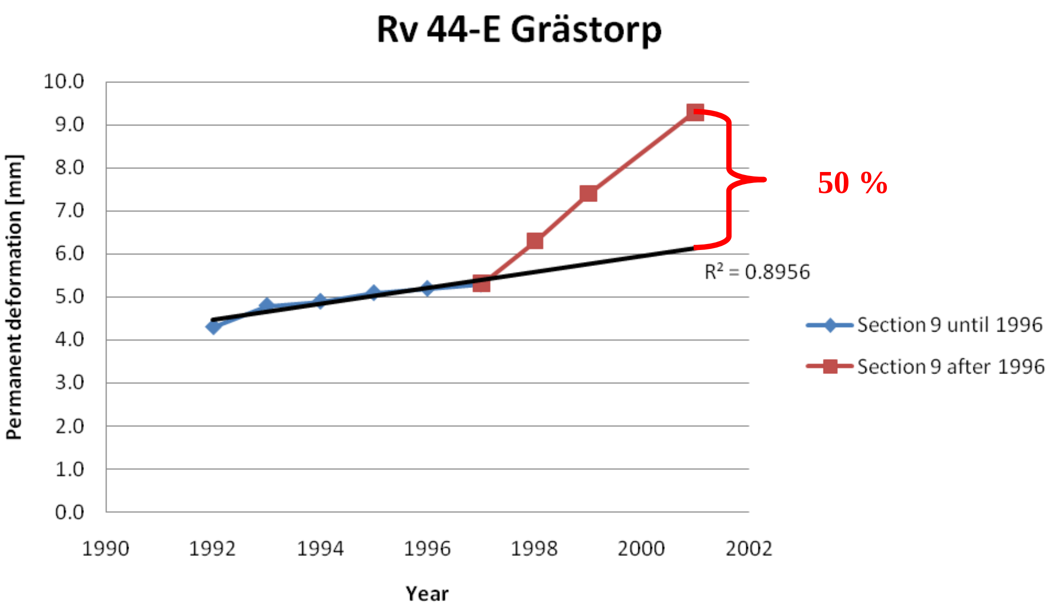


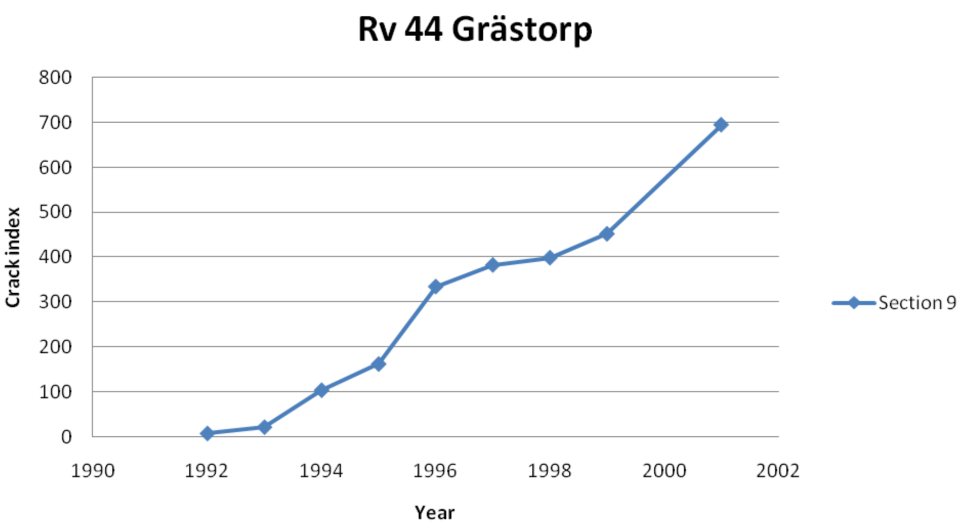
Results from the crack index analysis at Rv 34 towards south in section 9.



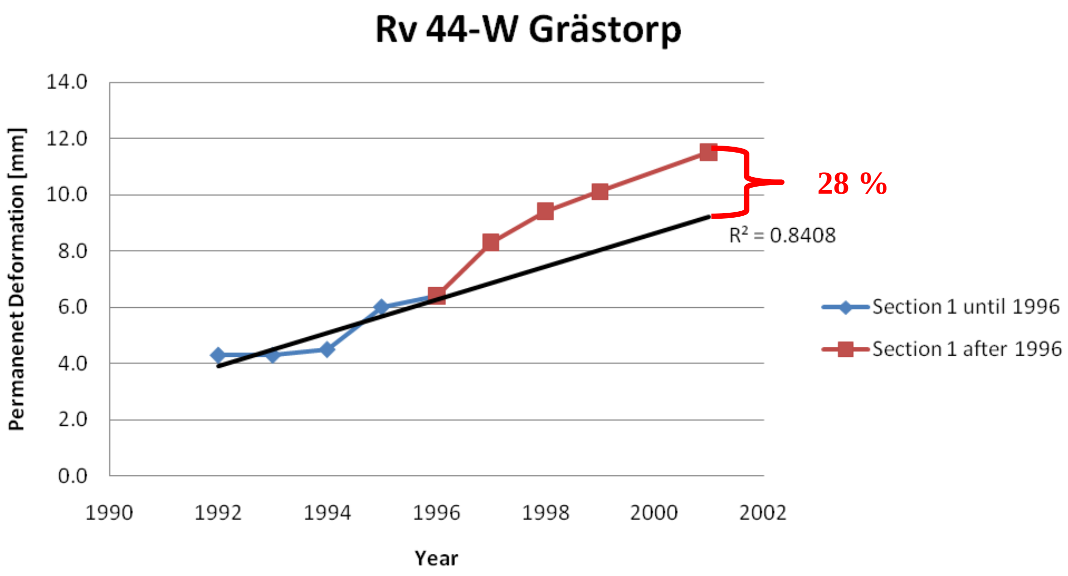


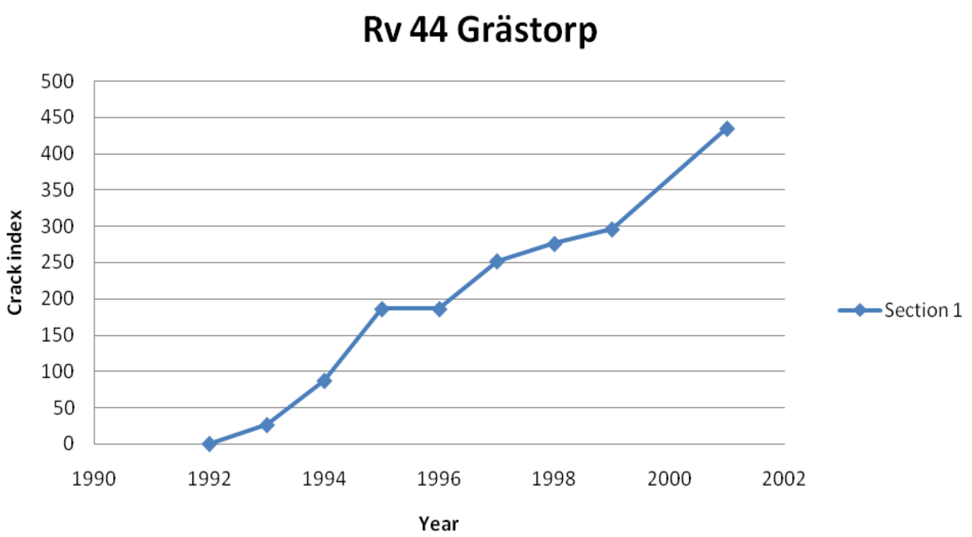
Results from the crack index analysis at Rv 44 towards east in section 9.



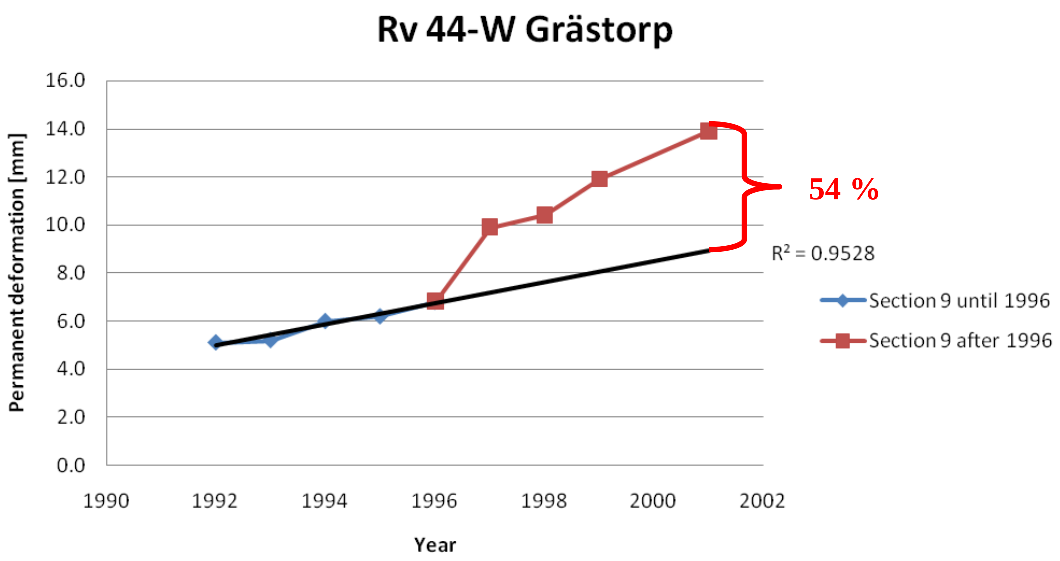


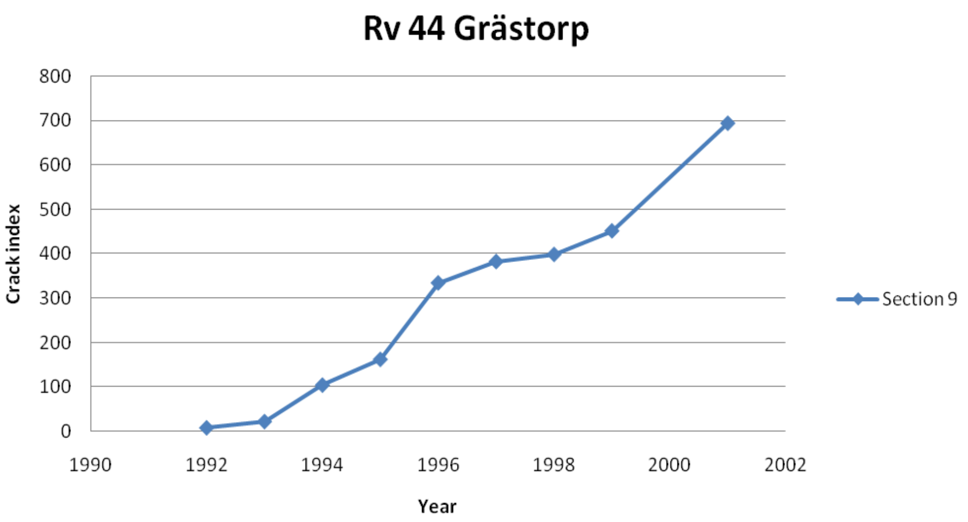
Results from the crack index analysis at Rv 44 towards west in section 1.



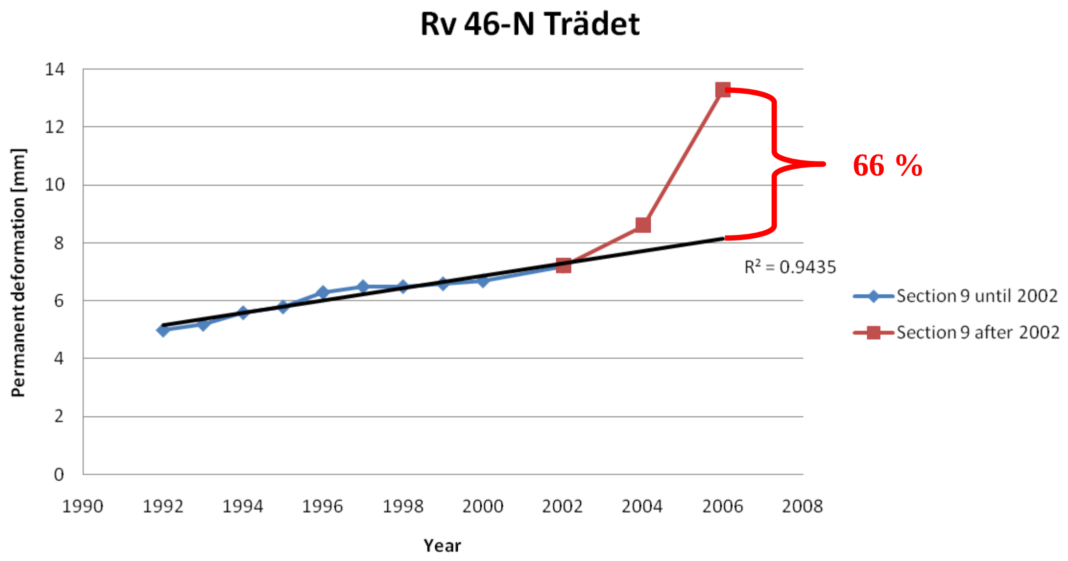


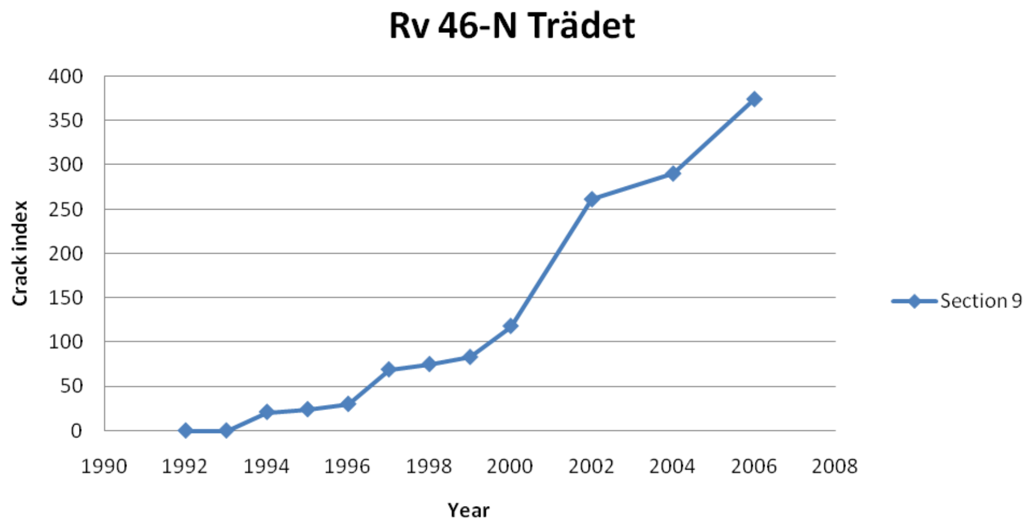
Results from the crack index analysis at Rv 44 towards west in section 9.



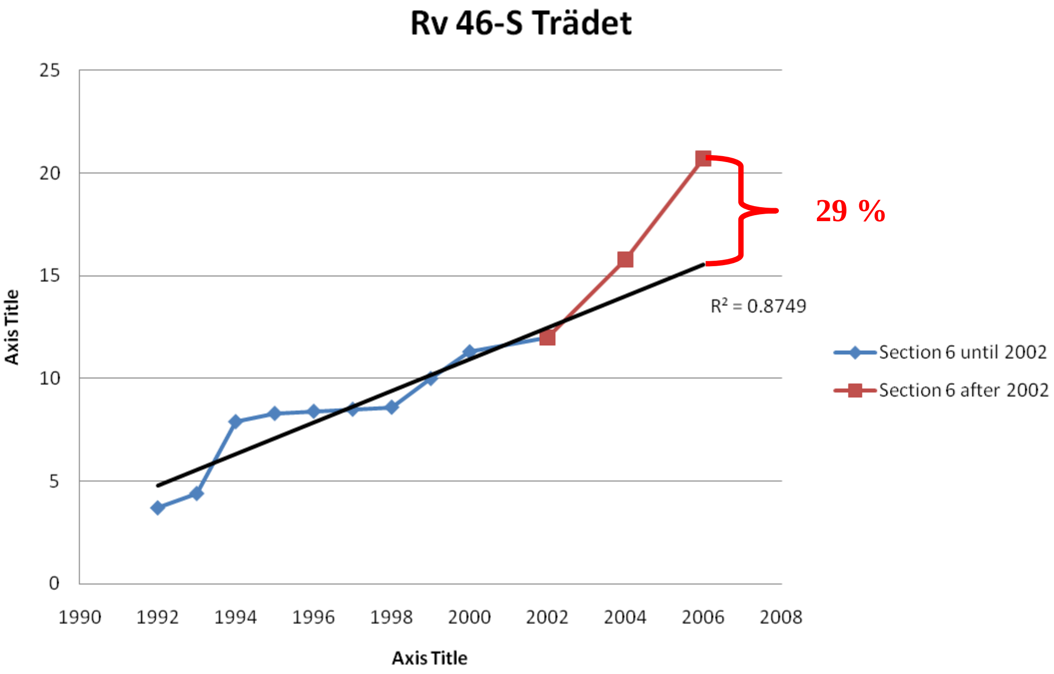


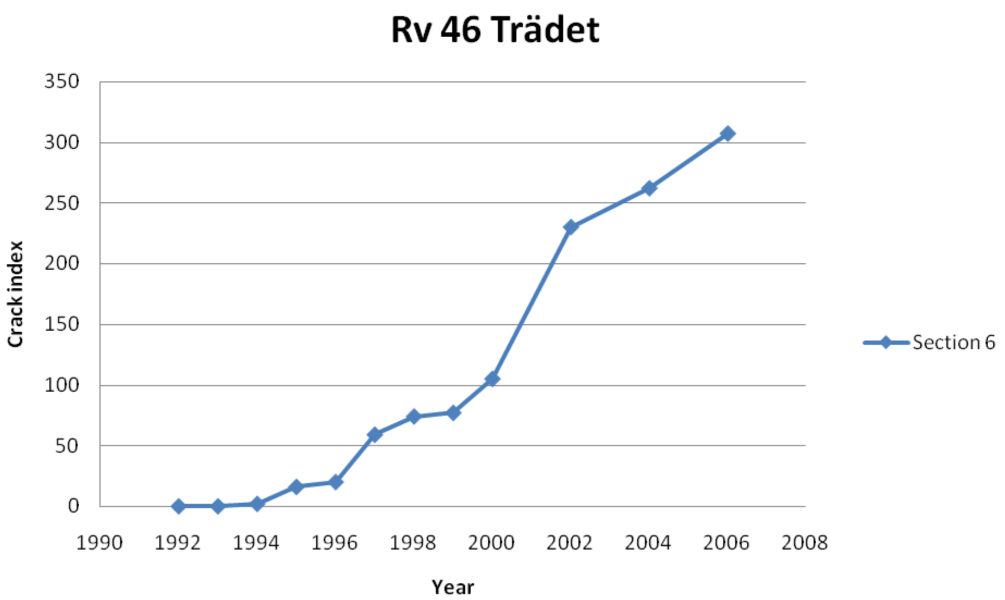
Results from the crack index analysis at Rv 46 towards north in section 9.



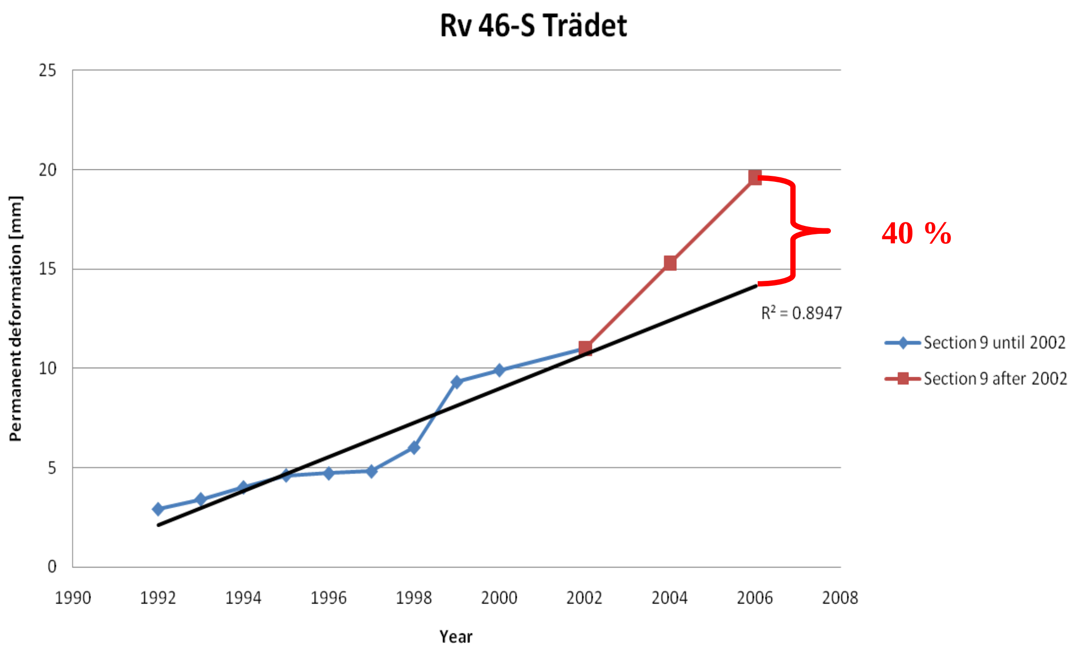


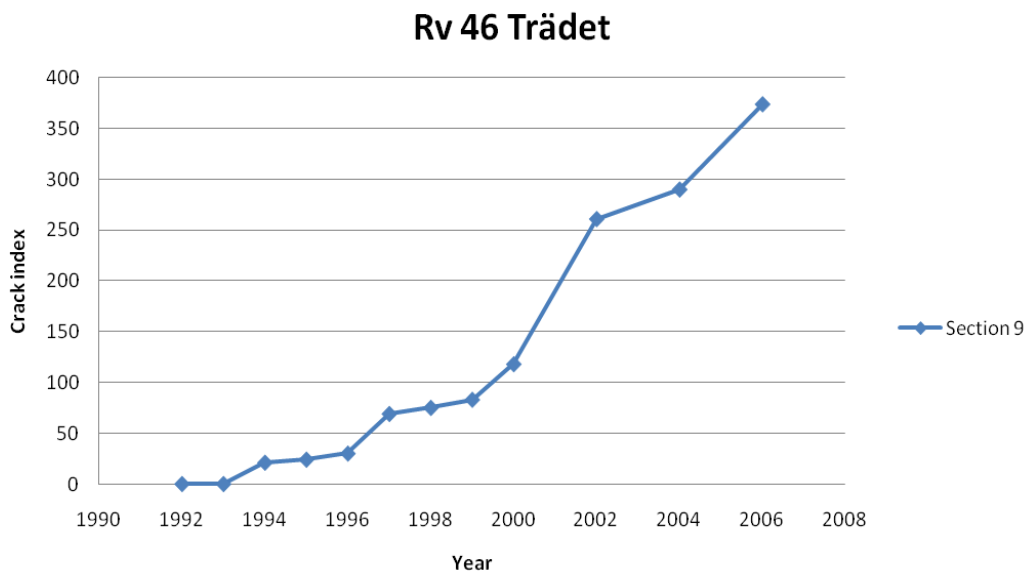
Results from the crack index analysis at Rv 44 towards south in section 6.





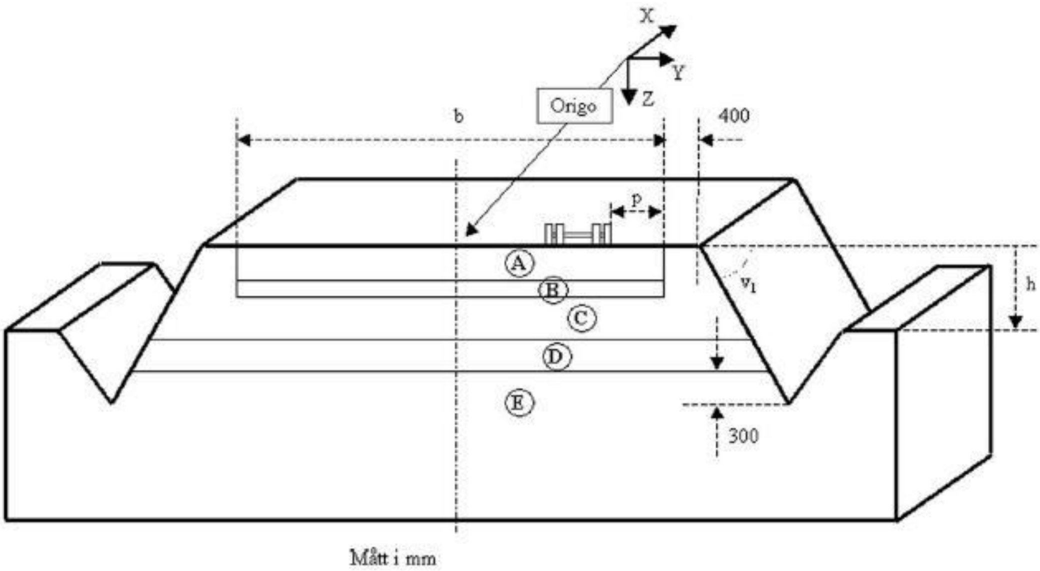
Results from the crack index analysis at Rv 44 towards south in section 9.

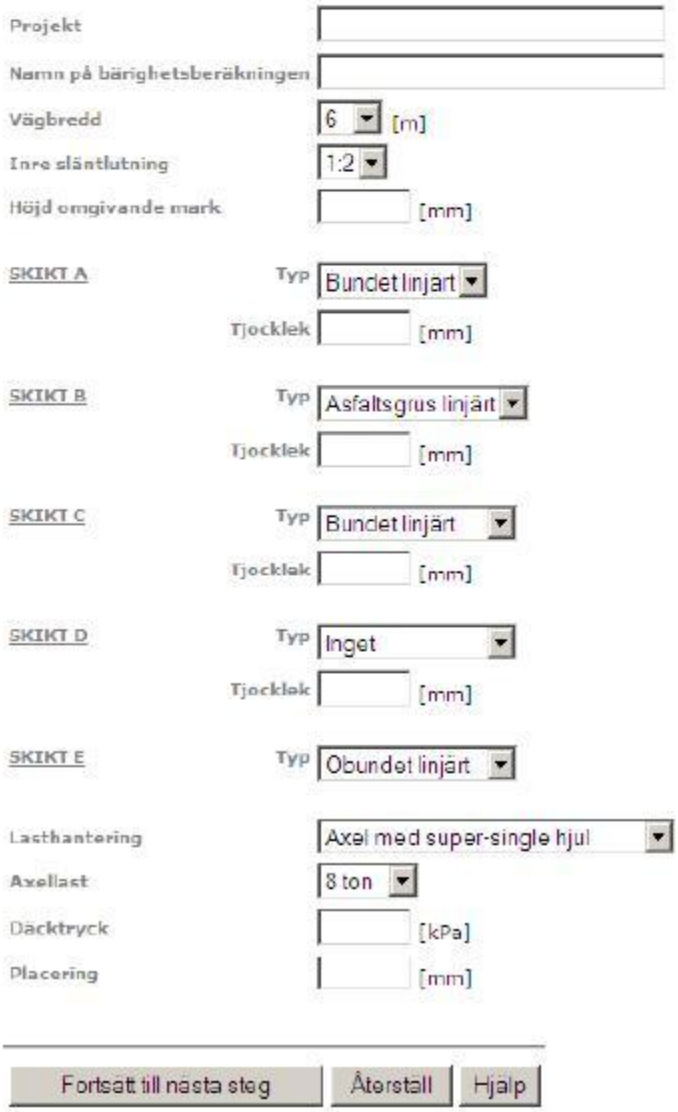




III. VägFEM

Input page of VägFEM





Projekt		
Namn på bärighetsberäkningen		
Vägbredd	6	[m]
Inre släntlutning	1:2	
Höjd omgivande mark		[mm]
SKIKT A	Typ	Bundet linjärt
	Tjocklek	[mm]
SKIKT B	Typ	Asfaltsgrus linjärt
	Tjocklek	[mm]
SKIKT C	Typ	Bundet linjärt
	Tjocklek	[mm]
SKIKT D	Typ	Inget
	Tjocklek	[mm]
SKIKT E	Typ	Obundet linjärt
Lasthantering	Axel med super-single hjul	
Axellast	8 ton	
Däcktryck		[kPa]
Placering		[mm]

Fortsätt till nästa steg | Återställ | Hjälp

Input to VägFEM_PD

Result specification Input maximum number of load repetitions and the number of intermediate values.

Number of load repititions, N	1000000
Number of load repetition values for result plots	20
Load repetition stepping type (linear or exponential)	exponential

Load FE   Evaluate   Change Mat

Layer material model specification

layer A	Material	NCHRP 1-37A, asphalt concrete		
Material parameters			a_1	1,691
			a_2	0,273
			a_3	1,849
Temperature			T	20

layer B	Material	NCHRP 1-37A, asphalt concrete		
Material parameters			a_1	1,691
			a_2	0,273
			a_3	1,849
Temperature			T	20

layer C	Material	Gidel model		
Material parameters			ε_1^{p0}	832,578349
			B	0,00366381
			n	3,91087723
			m	1,6
			s	26,2

layer D	Material	Gidel model		
Material parameters			ε_1^{p0}	832,578349
			B	0,00366381
			n	3,91087723
			m	1,6
			s	26,2

layer E	Material	NCHRP 1-37A, subgrade		
Water content			W_c	45

Output from VägFEM_PD, principal stresses

resfil	yes	company	VV/Chalmers	bername	E6 Dingle 80MPa	projekt	Manhattan Project	user	Michael B A Svensson	bertyp
Geometry	edgh	1	edgw	3	b	13	h	1,45		
Layers										
layer A	lin el	t[m]	0,04	E[Pa]	4500000000	rho[kg/m^3]	2400	nu[-]	0,35	
layer B	lin el	t[m]	0,13	E[Pa]	3500000000	rho[kg/m^3]	2000	nu[-]	0,35	
layer C	Uzan	t[m]	0,28	pa[Pa]	2100000000	pb[-]	0,88	rho[kg/m^3]	2400	nu[-]
layer D	Uzan	t[m]	1	pa[Pa]	2100000000	pb[-]	0,88	rho[kg/m^3]	2000	nu[-]
layer E	lin el	t[m]	1,55	E[Pa]	80000000	rho[kg/m^3]	2000	nu[-]	0,35	
Load										
axt	0	axh	0,051145833	axf	98200	axpos	0,5	axp	800000	
Results, extrapolated to nodes within layers.										
Centre values	y	3,65								

node	z-coord[m]	S11[Pa]	S22[Pa]	S33[Pa]	S12[Pa]	S13[Pa]	S23[Pa]	S1[Pa]	S2[Pa]	S3[Pa]
layer A										
343	5,51E-16	-1546100	-1131700	-830600	224,6	3719	1049	-830677,0077	-1131703,546	-1546119,446
13848	0,01	-1181600	-934680	-781130	89,98	1435	9004	-780598,6633	-935206,1612	-1181605,155
2470	0,02	-883980	-769100	-760070	2,373	-141,7	17699	-746319,1011	-782850,7323	-883980,1666
13895	0,03	-616510	-616650	-717510	-77,54	-9132	24265	-610309,4315	-616580,1301	-723780,4384
351	0,04	-409950	-502740	-715750	-99	-9186	29517	-409663,5685	-498745,4608	-720030,9508
layer B										
351	0,04	-317280	-389460	-553790	-77	-7145	40210	-317036,9775	-380193,6456	-563297,3769
37627	0,105	182030	59967	-317310	-79,42	3710	31408	182057,7603	62563,38667	-319934,147
374	0,17	828400	613370	7023	160,5	-7621	16397	828470,7043	613813,0787	6509,216947
layer C										
374	0,17	-20118	-24975	-71635	3,543	-177,1	6411	-20117,30693	-24110,27137	-72500,4217
14038	0,2167	-19644	-24277	-70930	2,288	-24,23	7085	-19643,98825	-23224,76327	-71982,24848
2494	0,2633	-15431	-19866	-61931	-1,216	8,437	7530	-15430,99846	-18558,69348	-63238,30806
14001	0,31	-15563	-19526	-61370	-0,9654	30,51	7649	-15562,97333	-18171,6233	-62724,40337
2491	0,3567	-13169	-16610	-55830	0,5999	-22,61	7656	-13168,98223	-15168,48785	-57271,52992
13960	0,4033	-13104	-16188	-54644	-0,017382	-4,317	7513	-13103,99918	-14772,32938	-56059,67143
366	0,45	-11559	-14327	-50669	1,837	-25,61	7307	-11558,97704	-12912,87251	-52083,15045
layer D										
366	0,45	-10694	-13487	-49211	2,137	-29,82	7270	-10693,96833	-12064,19639	-50633,83327
36781	0,575	-11395	-13459	-47251	-1,686	22,24	6588	-11394,97946	-12220,0532	-48489,96734
6259	0,7	-9207	-10743	-40077	1,752	-22,4	5871	-9206,968749	-9611,612907	-41208,41834
36767	0,825	-9970	-11301	-39635	-0,3528	0,9827	5248	-9969,999888	-10366,39513	-40769,60498
6256	0,95	-8403	-9707	-36269	0,5968	-3,908	4652	-8402,999446	-8915,82658	-37060,17397
36739	1,075	-8060	-9582	-36661	-0,3521	3,158	4022	-8059,999642	-8997,246253	-37245,7541
6253	1,2	-5451	-7455	-34830	0,8815	-2,619	3336	-5450,999557	-7054,329548	-35230,6709
36711	1,325	-3216	-6049	-36583	-0,577	-0,3893	2300	-3215,999859	-5876,722661	-36755,27748
822	1,45	2379	-1765	-36245	0,9032	0,1591	1056	2379,000201	-1732,688948	-36277,31125
layer E										
822	1,45	-16355	-16583	-37266	0,1508	0,020095	961	-16354,99987	-16538,44489	-37310,55523
20821	1,579	-17757	-17929	-39316	0,025762	0,3829	818,5	-17756,99998	-17897,72102	-39347,279
3731	1,708	-19128	-19251	-41354	-0,010702	-0,4265	689,2	-19127,99999	-19229,53072	-41375,4693
20794	1,837	-20564	-20649	-43550	-0,018556	0,053442	567,3	-20564	-20634,95555	-43564,04445
3728	1,967	-21990	-22046	-45741	0,017427	-0,1233	455,3	-21989,99999	-22037,25463	-45749,74537
20767	2,096	-23457	-23494	-48007	-0,0008788	0,073891	356,3	-23457	-23488,82222	-48012,17778
3725	2,225	-24939	-24964	-50287	-0,00047648	-0,1113	265,5	-24939	-24961,21666	-50289,78334
20740	2,354	-26448	-26468	-52589	4,77E-05	0,039344	196,6	-26448	-26466,52037	-52590,47963
3722	2,483	-27994	-28014	-54919	-0,0024519	-0,063167	135,6	-27994	-28013,3166	-54919,6834
20713	2,612	-29559	-29581	-57227	0,0015182	0,024968	108	-29559	-29580,5781	-57227,4219
3719	2,742	-31189	-31211	-59581	-0,023274	-0,087528	89,11	-31188,99997	-31210,72013	-59581,27989
20683	2,871	-32833	-32850	-61866	0,021305	0,064292	118,5	-32832,99997	-32849,51609	-61866,48394
816	3	-34575	-34575	-64211	-2,57E-08	2,32E-08	159,5	-34574,1416	-34575	-64211,8584