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DRAINAGE ON LOW TRAFFIC VOLUME ROADS

Problem description, improvement techniques and life cycle costs

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Drainage on Low Traffic Volume Roads

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PREFACE

This Report is a final report from the Phase 2 ‘Understanding and Analysis’ section of the EU ROADEX II Project and aims to provide an insight into the state of the art of road construction and road maintenance for roads constructed across peat in the Northern Periphery of Europe.

It will concentrate particularly on the problems due to inadequate drainage and the best (current) practices for improving drainage in the four specific Partner Areas of the EU Roadex Project i.e. Troms County in Norway, the Districts of Keski-Suomi and Lapland in Finland, Region Norr of Sweden and the Highland Area of Scotland.

In addition, where considered appropriate, the report will also refer to relevant information concerning drainage and related experiences from elsewhere in the world.

A part of this report was written on the basis of a questionnaire on drainage problems in the Partner areas. The authors would like to thank Johan Ullberg, Sweden, Geoff Potter from Scotland and Taina Rantanen from Finland for answering the questionnaire and for the input to this report. A major contribution to this report was made by Sami Kari from the Technical University of Tampere who has made a comprehensive literature review on the subject.

This report was written by Geir Berntsen from the Public Roads Administration in Norway and by Timo Saarenketo from Roadscanners Oy, Finland.

Finally the author would like to thank the ROADEX II Steering Committee for its encouragement and valuable guidance in this work

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ABSTRACT

This report focuses on the problems that inadequate drainage causes for low volume traffic roads in the Northern Periphery area of Europe.

A literature review has been done on moisture content in the road structure and the relationship between moisture content and characteristics for unbound granular materials and subgrade soil. It is obvious that increased moisture content reduces the bearing capacity and changes are greatest in dense materials with a high content of fines.

Typical drainage problems in the NP-area have been addressed and proposals on how to improve the problems are made. Many of the problems are the same all over the NP-area except for in Scotland where there are problems caused by grass verges on the road shoulder.

Field observations of roads on sloping ground shows that there are big differences in rut depth and roughness on the road cut side of the road compared to the embankment side. The ground water table was much closer to the road surface in the road cut lane and therefore there was also a higher moisture content in the road structure materials and the subsoil. For 20 % of the analysed roads the rut depth in the road cut lane was 1.5 times deeper than the other lane. Just 12 % of the roads had greater rut depth in the embankment lane.

Predictions models have been used to demonstrate that the lifetime of the pavement structure (calculated as number of standard axles) will increase considerably when drainage is improved. The Swedish design system has, among others, been used to calculate some examples and by improving only the drainage the lifetime will increase by a factor of 2.2-2.6 times.

All other prediction models show the same or an even greater effect. Both field observations and prediction models show that improving the drainage will increase the pavement lifetime. On the basis of the observations and models, a table for estimating the increase in lifetime when the drainage is improved is presented.

In this paper the life cycle cost calculations, made only for one pavement life cycle, show that both drainage maintenance and improvements of the drainage system will reduce the life cycle costs. There may be no other road condition improvement measures that are more profitable that can be done on an existing road.

The conclusion is that maintaining the drainage system is cost effective and must be prioritised among other maintenance activities. The first step in strengthening a road should be to make the drainage system function properly and this should be done 1-2 years before paving.

KEY WORDS: Roadex, Drainage, Road Condition, LCA, Field Observation, Moisture, Deformation, Resilient Modulus, Drainage Improvements, Pavement Lifetime.

1 Introduction

1.1 THE PROBLEM CAUSED BY INADEQUATE DRAINAGE

Drainage in road construction is a complex topic. This report focuses on the problems that inadequate drainage causes with relation to bearing capacity and the deterioration of the road surface. Also problems due to inadequate drainage in areas outside the ditches such as erosion problems in road cuts and in outlet ditches are discussed to some extent.

Drainage of water from the road surface is also an important topic. First of all this water creates safety problems due to reduced friction on a wet pavement, but also splash and spray from the traffic is a problem. Furthermore surface water will ingress the road structure through cracks, potholes and unpaved road shoulders and then moisten unbound granular materials in the road structure.

In the first ROADEX project drainage problems were identified as being one of the worst problems shared by all of the Roadex partner Road regions. In the Roadex II project drainage problems were mapped in greater detail using a questionnaire. The task was very challenging because the problem is more complex in the cold areas since the freeze/thaw-cycles affect moisture content to a much greater extent than elsewhere.

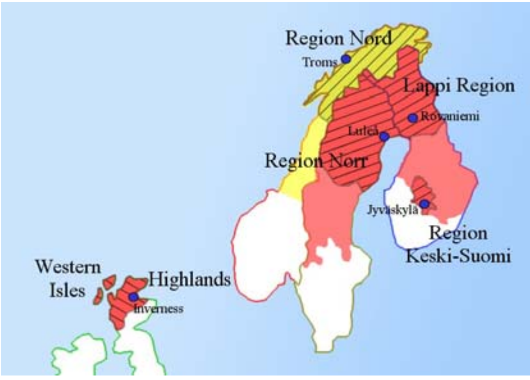


Figure 1. Partner districts in ROADEX II

It is a well-known fact that increased moisture content reduces the bearing capacity of a soil, which will increase the rate of deterioration and shorten the lifetime of the roads. In such cases, roads will need rehabilitation more often than a well drained road structure. The costs of this increase in maintenance of the road surface need to be compared with the costs of maintaining or improving the drainage.

Funding for road condition management has been decreasing in all of the countries participating in the Roadex project for several years and as a result basic ditch maintenance tasks as well as tasks related to the drainage system in general are neglected since they are low on the list of priorities. Instead of drainage maintenance the prioritised tasks have been those that are more important to the road user in the short term i.e. repaving and snow removal.

Even though inadequate drainage is a problem for the road network in the whole NP-area, only Finland has a system for monitoring the drainage of gravel roads. Norway has just started to take a photo for every 20 m of ditch. Because inadequate drainage is a problem, there is a need for a systematic approach to collecting information about drainage performance and how to handle these problems.

In this project we will describe the drainage problems and some material models that take moisture content into account. Using a questionnaire, drainage problems typical of the NP-area were mapped. Then these problems were classified and recommendations were made on how to solve them. A table listing typical problems, how to identify them, what causes the problem and how to handle it is presented in appendix 2.

The affect of improving and maintaining the drainage system is an important task, and observations and models have been used to estimate the increase in lifetime. It is also necessary to evaluate the life cycle costs because improvement/maintenance costs must be in proportion to the lifetime after the improvements.

These analyses and calculations demonstrate clearly that improving the drainage and doing drainage maintenance as required are very profitable if the deterioration of the road is related to a poorly working drainage system.

2 Impact of poor drainage on road condition

2.1 GENERAL

Moisture gets in the road structure through the normal circulation process of water. The main sources of water are precipitation, but also infiltration, absorption from the melting snow and capillary voids of soil assist in the accumulation of water in the structure.

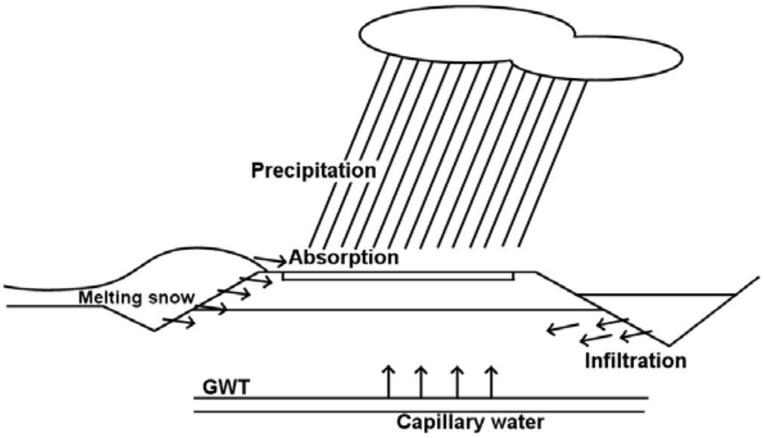


Figure 2. Main sources of the water in the road structure.

In autumn and winter precipitation, in the form of water or snow, moistens the entire road structure as well as the shoulders and ditches right to the road. Snow fills almost all of the ditches in winter at least in the colder regions. In addition, the road structure freezes and capillary forces and cryo suction raise more water to the frozen fringe. In summary, there is a large amount of excess water in the road structure in the spring.

When thawing starts in spring all of the difficulties cumulate. It is not only the thawing that is the problem, but also the direction in which it is progressing. Thawing starts from the centreline of road, because it is the only surface not covered by the snow. The road shoulders, with the exception of the shoulder on the Southern side in open areas (see Saarenketo and Aho 2005), of the structure start to thaw later in spring. The problem is that thawing forms a basin in the middle of road, where the excess water is not able to exit the road structure until the shoulders of the road have melted. This state is especially dangerous when the materials in the road structure are close to saturation.

When a material is close to the saturation level, the pore water pressure rise very easily, especially in the case of quick loading. Soil pore pressure affects the bearing capacity through the efficient normal stress, because the higher the soil pore pressure is the smaller the efficient normal stress is, which basically describes the real strength of soil. Efficient normal stress also affects the deformation and resilient properties of the soil. In case of high soil pore pressure, response of loading in the base and sub-base becomes significantly smaller, deflections at the surface become higher and the load, which moves from the surface to the subgrade, becomes larger. This causes greater strains on the structure than expected in the design and construction stages. All this leads to the fact that the operation period and service standard of the road reduce prematurely.

The amount of excess water during the spring thaw and how the pore water pressure raises under quick loading depends to a large extent on the materials. This effect will be much larger for a fine graded soil than for a coarse graded soil with an open graded grading. The resilient modulus and permanent deformation are often used as parameters for describing the bearing capacity, but the effect can also be seen in CBR-values, DCP-tests and different types of deflection measurements on the road surface.

The resilient modulus for an unbound material is mainly influenced by the stress state, dry density and moisture content. Soil suction is one of the parameters that define the stress state in unsaturated soils and also how the moisture content varies with the change in soil suction. Some researchers have the opinion that suction describes the mechanical behaviour better than moisture content.

The relation between suction and moisture content for an unsaturated soil is defined by the soil water characteristic curve (SWCC). When wetting or drying this curve does not give the correct matric suction because of hysteresis effects. An example of an SWCC is shown in figure 3. The moisture content (W) decreases when the suction increases. Increased suction increases the bulk stress as well as the resilient modulus and shear strength.

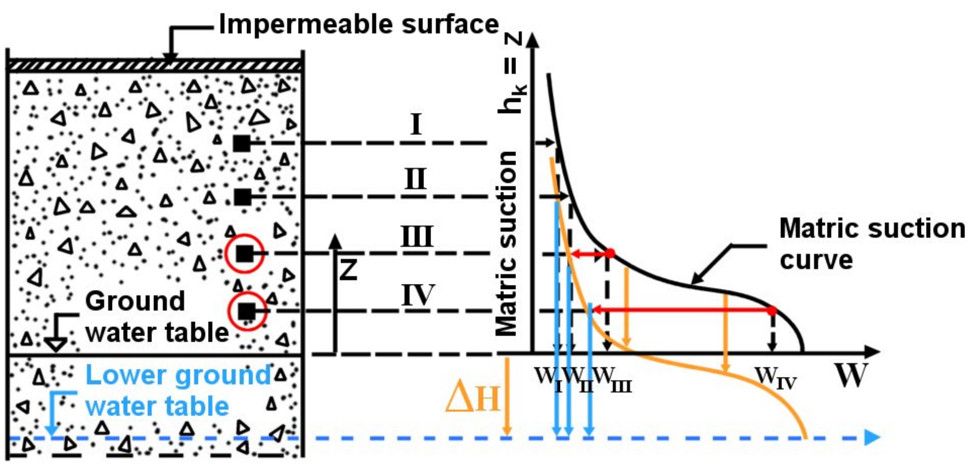


Figure 3. Soil Water Characteristic Curve – SWCC.

Using the example above it is easy to understand that lowering the ground water table will benefit the bearing capacity for the unsaturated soils. Lowering the ground water table by ΔH increases the suction and thereby the effective stress on the material. The red arrows show the changes in moisture content.

Understanding figure 3 is essential to understanding why drainage is important. The goal is to keep the ground water table as low as possible. This means that the moisture content is higher, and this will be the case when water ingresses the road structure through cracks and potholes in the pavement layer or through the unpaved road shoulders. It is therefore important to seal the cracks, patch the potholes and have a proper cross fall on the road surface so that the surface water can flow to the ditches.

2.2 ROADEX – 1

One of the key results in the state-of-the art study from the ROADEX I project was that all of the partners shared the problem of inadequate drainage and that this problem was especially bad on roads located on transversely sloping ground. Permanent deformation due to freeze-thaw cycles and bad quality road materials were also problems for all of the Roadex partners which can also be related to inadequate drainage.

The problems related to inadequate drainage in each partner district are described as follows in the ROADEX-I report:

Finland:

Insufficient drainage, especially where the road is constructed on a slope, is one of the most significant problems in road condition management in Lapland, according to interviews conducted during the study in the district office. Frost action in subgrade and poor quality base course material is also related to moisture content and therefore also to drainage. Typical damages are longitudinal cracking, uneven frost heave and shoulder deformation.

Sweden:

Typical road condition problems on secondary roads are poorly performing drainage, road structures that are too thin, poor quality base course materials (excessive fines content), frost susceptible subgrades, and peat under the road structure.

Norway:

Uneven frost heave due to frost action, shoulder deformations and thaw weakening can be experienced especially where roads are constructed on slopes so that half of the road is in the road cut, the other half resting on an embankment. Restoring insufficient drainage was previously a continuous task for maintenance, but as the funding for road maintenance has decreased, the maintenance of ditches has been left out of the routines. Today drainage is restored only as a part of a reinforcement project. The spring thaw causes a lot of problems especially on gravel roads.

Major shoulder deformation problems can be found in roads located on hill slopes and where the drainage on the upper side of the road is not functioning properly.

Scotland:

The biggest problem in The Highland Council road network is poorly performing drainage. It is the main reason for the increasing amount of severe shoulder deformation in the roads. Due to high grass verges beside the road, rainwater is ponding on the road and infiltrating the road structure and making it water and freeze/thaw susceptible. However, improving the drainage in the Highlands is much more difficult than it is in Scandinavia because the road area in Highlands is limited, in most cases, to the carriageway and narrow verge. Solving this problem somehow would however result in millions of pounds savings for the Highland Council in road network maintenance and rehabilitation costs.

3 Moisture content of road materials and subgrade soils

3.1 FORMS OF MOISTURE IN THE GROUND

Water appears in the soil in three different forms. Water in the soil is either gravitation-, capillary- or adsorption water. Gravitation water, which is part of the free water, moves in the soil voids under the force of gravity. Capillary water, also called as viscous water, is present mainly between the different soil particles. Surface tension of the soil particles is the force, which keeps the capillary water in the soil. Adsorption water consists of two layers, tightly and loosely bounded layers. Tightly bounded adsorption water is also called as hygroscopic water and the thickness of the layer is about 0,002 µm. Hygroscopic water condenses to the surface of soil particles straight from the water vapour of the air. Around the hygroscopic water is loosely bounded adsorption water layer which thickness varies from the 0,002 µm to 0,006 µm. Adsorption water is attached to the surface of the soil particles by the electrostatic force between the soil particles and water molecule (Saarenketo, 1998).

When the soil temperature drops below 0˚C, water in the ground starts to freeze. First freezes water in the biggest voids; in other words free water. Hygroscopic water freezes last or does not freeze at all. Fine graded soils have a large specific surface area and the amount of surface bound unfrozen water is therefore high below 0˚C. Features of soil that affects to the amount of unfreezing water in the ground are mineralogical properties of soil, granularity, grain size distribution, specific area of soil particles and surface tension. Anderson (1989) expressed that the amount of unfrozen water is 12 % from total volume of water at air temperature of -5˚C for the tested soil (Saarenketo, 1998). Tsytovich (1975) suggest that amount of unfrozen water at air temperature of -10˚C in Russian soils were 0.0 % for quartz sand, 3.5 % for silty sand and 15.3 % for montmorillonite clay (Saarenketo, 1998).

3.2 METHODS TO MEASURE MOISTURE CONTENT

There are two types of moisture content: gravimetric and volumetric. Gravimetric moisture content is defined as the mass of water relative to the mass of dry soil particles. The volumetric moisture content is defined as the volume of water relative to the total volume of soil. The use of volumetric water content rather than gravimetric moisture content is often more convenient because it is more directly adaptable to the computation of fluxes and water quantities added or subtracted to a soil (http://www.usyd.edu.au/su/agric/ACSS/sphysic/water.html). These two different ways to process moisture content also explains differences between different studies because gravimetric moisture content depends on the bulk density of the material and is roughly about 1.5 to 2 times smaller than volumetric moisture content.

There are many ways to measure moisture content in the laboratory, but the most commonly used are the oven-dry method and the calcium carbide CaC_2 gas pressure meter method. In the oven-dry method, a soil sample with natural moisture content is weighed and then dried in the warming cupboard. After it has been dried, the soil sample is weighed again. The amount of water contained by the soil sample is the difference between the wet and dried weight of the sample. Moisture content that is measured with oven-dry method is considered as gravimetric moisture content. Use of the calcium-carbide gas pressure method is based on the fact that water in the soil sample absorbed by the calcium-carbonate and forms acetylene gas as a product of the chemical reaction. The pressure of acetylene gas is directly proportional to the amount of acetylene and thus to the amount of water in the sample. Moisture content measured in this way is also gravimetric moisture content. (Rantamäki et al. 1997)

Currently, time domain reflectometry (TDR) is the most popular technique used to measure the moisture content of soil in field tests. Other methods that can be used to measure moisture content in the soil are capacitance-based sensors, nuclear gauges, nuclear magnetic resonance (NMR) and ground penetrating radar (GPR). The TDR technique is based on transmitting an electromagnetic pulse through the soil and recording the resulting changes in its permittivity (dielectric constant). Change in the permittivity or dielectric constant is a measurable property and is related to the volumetric water content of soil. (Svensson, 1997) When TDR is used to measure the water content of frozen soil, one of the possible sources of error is that the value of dielectric constant of frozen soil is about 4 instead of the value of 1, which is the assumption in many cases.

3.3 VARIATION OF MOISTURE CONTENT IN THE ROAD STRUCTURE

Moisture content varies in a road structure throughout the different season. In summer, the moisture content in the road structure decreases at the slow rate and starts to rise again in fall, because of precipitation. During the winter unfrozen moisture content reaches its lowest value and in the springtime when the road structure starts thawing the moisture content increases rapidly. Moisture content becomes stable again at the end of summer. Possible peaks in the moisture content diagram are usually due to rainfall or during spring thaw.

Moisture content in the base and subgrade are considered in the following paragraphs but published materials on the subject were quite small. None of the studies presented any information regarding variation of moisture content directly under the pavement.

3.3.1 Moisture in the subgrade

Seed, Chan and Lee studied the behaviour of silty clay subgrade in the laboratory already in 1962. They noticed that when the moisture content decreased from the optimum moisture content to 3 % below that, the modulus increased from approximately 34 MPa to the 69 MPa (Bayomy et al. 2002).

Jones and Witczak (1977) showed in their study which was part of San Diego County experimental base project that when moisture content of silty clay subgrade increased from approximately 11 % to approximately 20 % the resilient modulus changed from around 275 MPa to as low as 52 MPa. (Bayomy et al. 2002)

Heydinger (2003) analysed effects of variation in moisture content in subgrade soils with the help of data, which was collected by the weather station and time domain reflectometry probes (TDR) from the test road located in Delaware County, Ohio, USA. Test section, 390104, consisted of an asphalt concrete surface overlaying asphalt-treated base with no drainage. The water table for this section was very shallow varying from 0.15 m to 1.2 m below the top of subgrade. Another test section, 390204, consisted of Portland cement concrete pavement over the dense-graded aggregate base with no drainage. The water table varied from 1.8 m to 3.0 m below the top of the subgrade. In both test sites the subgrade material was clay. The average moisture content in the clay subgrade varied from the 35 % to about 40 % in both test sections.

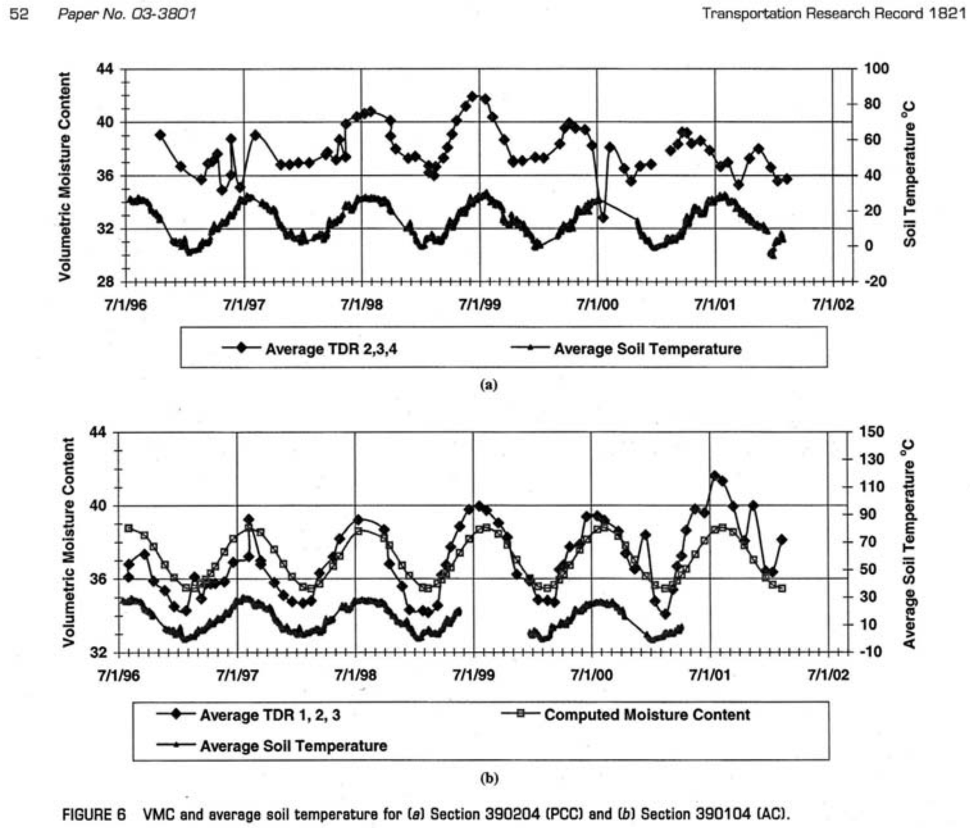


Figure 4. Volumetric moisture content and average soil temperature for (a) concrete pavement section, 390204 and (b) asphalt concrete section, 390104. (Heydinger, 2003)

Janoo and Shepherd (2000) studied the seasonal variation of moisture content with help of VITAL hydra soil probes, which measure the dielectric constant of soil, at different test sites in Montana, U.S.A. The structure of the test pavement, which was used in this study, contained 127 mm asphalt concrete surface over the 427 mm of crushed base, which included 10 to 13 % fines passing the 0,075 mm sieve. Subgrade consisted of silty or clayey gravel and sand, classified as A-2 – A-4 using the AASHTO soil classification system. Results are shown in figure 5. Average volumetric moisture content in the subgrade was during the non-frost period about 6 % and during thaw-weakening about 21 % (Janoo & Shepherd, 2000).

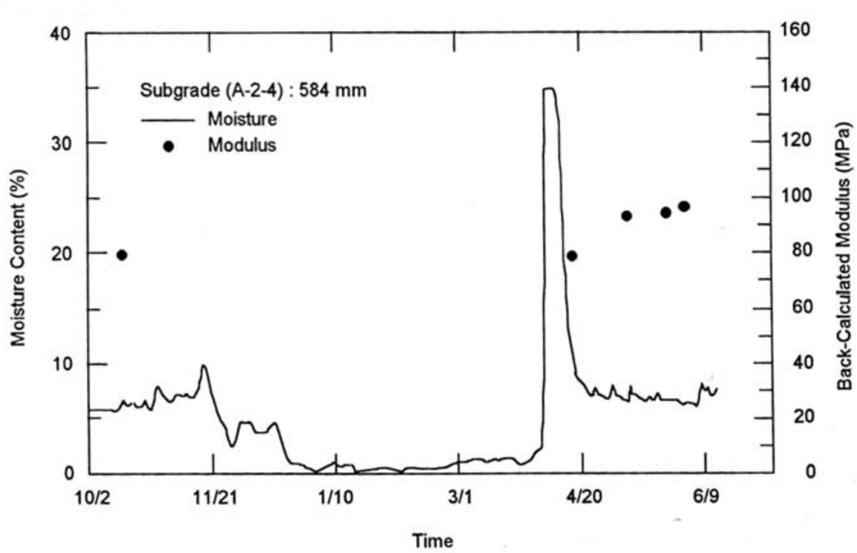


Figure 5. Back-calculated modulus and volumetric moisture content as a function of time for the subgrade. (Janoo & Shepherd, 2000)

Results from the study by Janoo and Shepherd (2000) differ quite a lot from the results of other studies. These results show that almost all the water in the road structure freezes in winter while, for example, the study of Heydinger (2003) shows that volumetric moisture content stays over the 30 %. The huge difference between the studies is partly explained by the fact that soil temperature barely goes under 0˚C in study by Heydinger. In addition, the properties of a subgrade material, such as their salt content, can affect the freezing temperature and amount of unfrozen water.

3.3.2 Moisture in the base course

Birgisson and Roberson (2000) studied the effectiveness of two typical edge-drain configurations used in rigid pavements and the influence of draining on the base moisture content. Sufficient data was collected from two test cells at the Minnesota road research project (MnROAD). To collect precipitation data they used an onsite weather station and moisture content data was collected with the help of time domain reflectometers (TDR), which were installed in the test structures. Test cell number 10 contained 240 mm of jointed plain concrete pavement over 102 mm of drained permeable aggregate stabilized base, overlying 76 mm of well-graded sandy gravel, considered as dense-graded aggregate base, with fines 1,4 % passing the 0,075 mm sieve. The subgrade material was silty clay. Test cell number 12 consisted of the same 240 mm of jointed plain concrete pavement like test cell no. 10. Under that there was a 130 mm thick well-graded sandy gravel base with fines 3 % passing the 0,075 mm sieve. Subgrade material for this test cell was also silty clay. Results from the test are shown in the figures 6, 7 and 8. The average volumetric moisture content in the base was about 22 % as shown in figure 6. The authors explained that the high moisture content in the outer wheel path is due to the fact that the pavement was wetting from the shoulder inward or the edge-drain system directed the water flow into the outer wheel path area. The average volumetric moisture content for the centreline was about 25 % as figure 7 shows. The shoulder and centreline did not seem to be affected by individual rain showers. Rainfalls significantly affected the outer wheel paths and the reason was that there was a crack between the concrete pavement and the asphalt shoulder. After the paving average moisture content dropped in the centreline from 25 % to about 15 % and for the outer wheel path, from 26 % to 22 % as seen in figure 8. This indicated that paving helped keeping water out of the pavement and also confirmed that main source of water was infiltration through the crack between the concrete pavement and the asphalt shoulder. Tests suggest that the influence of edge-drains in dense-graded bases might be very limited. In addition, the factors which the authors considered to have had influence on the performance of pavement drainage were the level of compaction around and above the drainage pipe, the layering of the pavement and the connection between the edge-drain and the shoulder. (Birgisson & Roberson, 2000)

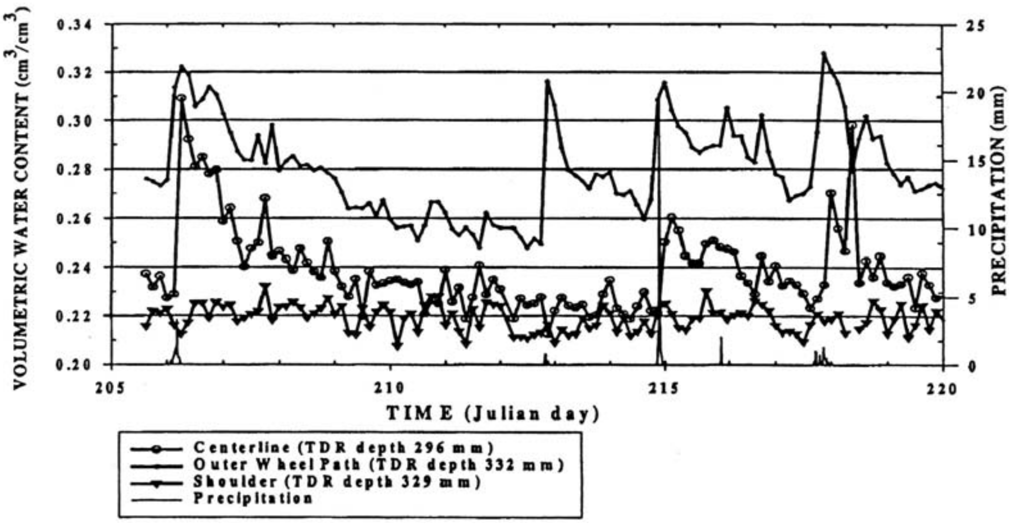


Figure 6. Pavement base layer water content measured by TDR for test cell 12. (Birgisson & Roberson, 2000)

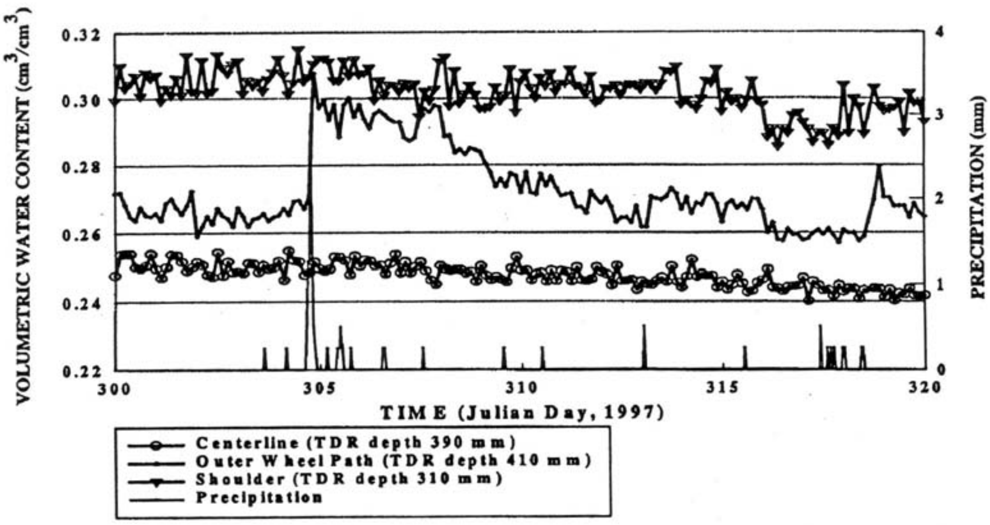


Figure 7. Pavement base layer water content, measured using TDR, from test cell 10. (Birgisson & Roberson, 2000)

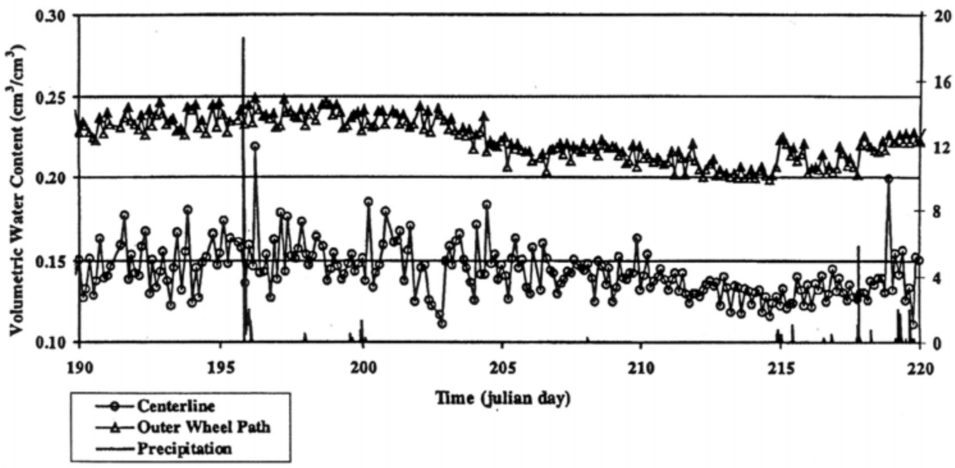


Figure 8. Effects of wedge paving on base course water contents. (Birgisson & Roberson, 2000)

Heydinger (2003) also found in his study in Ohio, USA that volumetric moisture content of dense-graded aggregate base under the concrete pavement varied from 20 % to 24 % which corresponds to a variation in saturation from 72 % to 86 %. (Heydinger, 2003)

Erlingsson et al. also measured moisture content with the help of TDR- probes on two different test roads in the Iceland. Figures 9 and 10 shows how the moisture content varies in different layers on the base at different depths during different season. One test road is located in Iceland near Vesturlandsvegur and the structure consisted of wearing course 46 mm, base course 1 34 mm (crushed gravel), base course 2 88 mm (crushed gravel), subbase 1 153 mm (crushed gravel), subbase 2 450 mm (natural gravel) and peat subgrade. The other test road is located near Thingvallavegetur, also in Iceland, and it consisted of 36 mm thick pavement, 60 mm thick base course1 made of crushed gravel, 80 mm thick base course2 made of crushed gravel, 275 mm thick subbase1 (sand) and subbase2 460 mm thick (sand). (Erlingsson et al.)

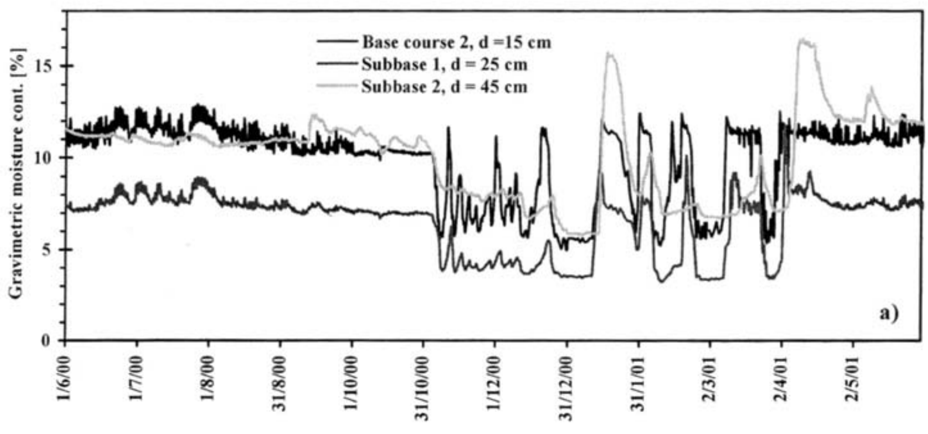


Figure 9. Gravimetric moisture content at three different depths in the road structure at Vesturlandsvegur, Iceland. (Erlingsson et al.)

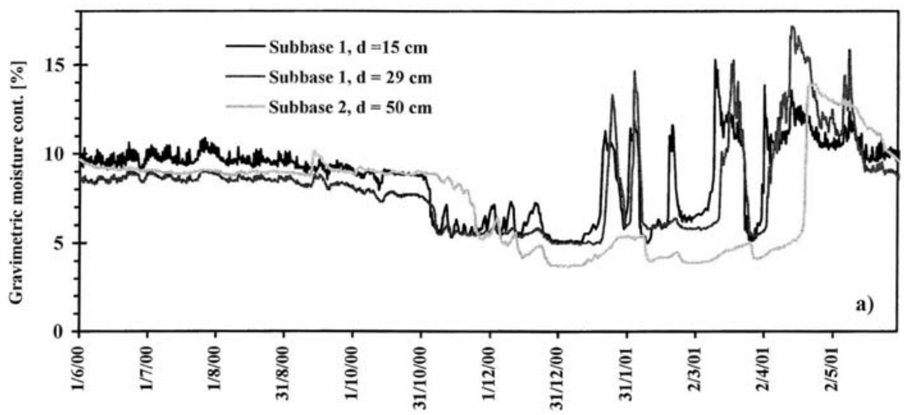


Figure 10. Gravimetric moisture content at three different depths in the road structure at Thingvallavegur, Iceland. (Erlingsson et al.)

Figures 9 and 10 presents unfrozen moisture content in road structure and not the actual moisture content, and that is the reason why moisture content seems to be so low in the winter time. As the figures show during the summer months and autumn moisture content decreased at a slow rate but some fluctuation occurred due to rainfall. The fluctuation was greatest close to the surface and decreases deeper in the ground. Late summer values are close to the natural moisture content of the material. Moisture content dropped rapidly when the freezing period started. During the wintertime a short period of thawing can been seen as a sharp and high peak in the moisture content. Permanent thawing started in late March or early April, and in that period the moisture content of different layers were frequently higher than the optimum moisture content of the material. In both cases, the average gravimetric moisture content varied from 8 % to 10% at late summer or autumn. During the spring thaw moisture content was a little bit higher than other times. (Erlingsson et al.)

Janoo and Shepherd (2000) also studied the seasonal variation of volumetric moisture content in the base course at test sites in Montana, USA. The structure of the test pavement, where the moisture of the base layer was measured, contained 76 mm asphalt concrete surface over the base and subgrade which was a silty soil, classified as A-4 by AASHTO soil classification system. Results are shown in the figure 11. Moisture content during the winter is about 10 % and about 27 % in the summer. (Janoo & Shepherd, 2000)

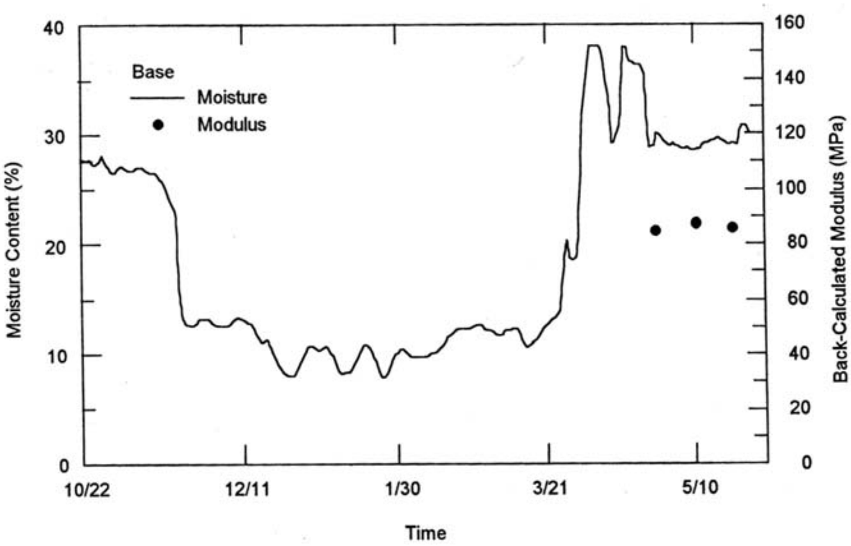


Figure 11. Back-calculated base modulus and volumetric moisture content as a function of time. (Janoo & Shepherd, 2000)

Bayomy et al. (2002) studied variation of moisture content in different seasons and different subgrade materials in various locations in the U.S.A. The surface type for all the test sections was flexible except test section 48-4143 which was a rigid surface type. Results from the study are shown in figures 12 and 13. Moisture content in the figures is gravimetric moisture content. (Bayomy et al. 2002)

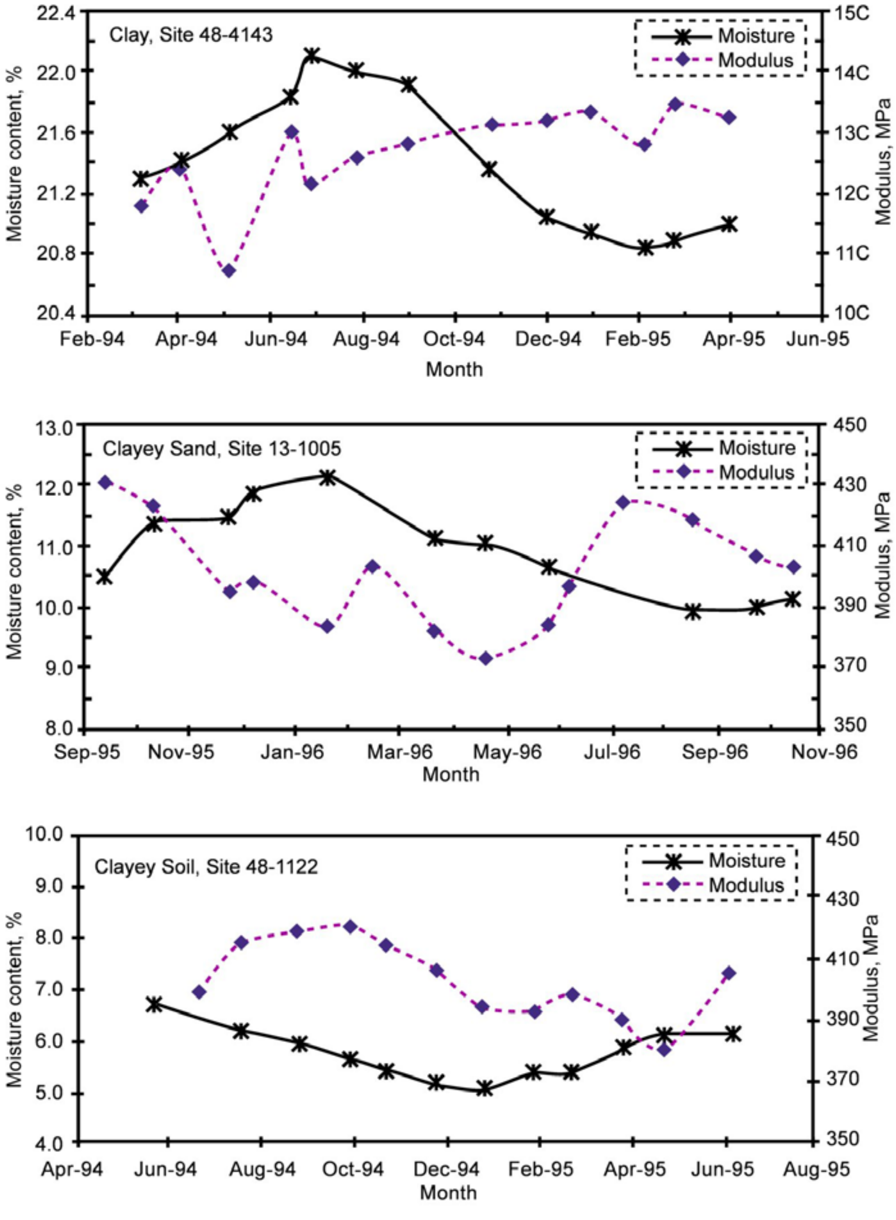


Figure 12. Variation of resilient modulus and gravimetric moisture content with time for various soil types. (Bayomy et al. 2002)

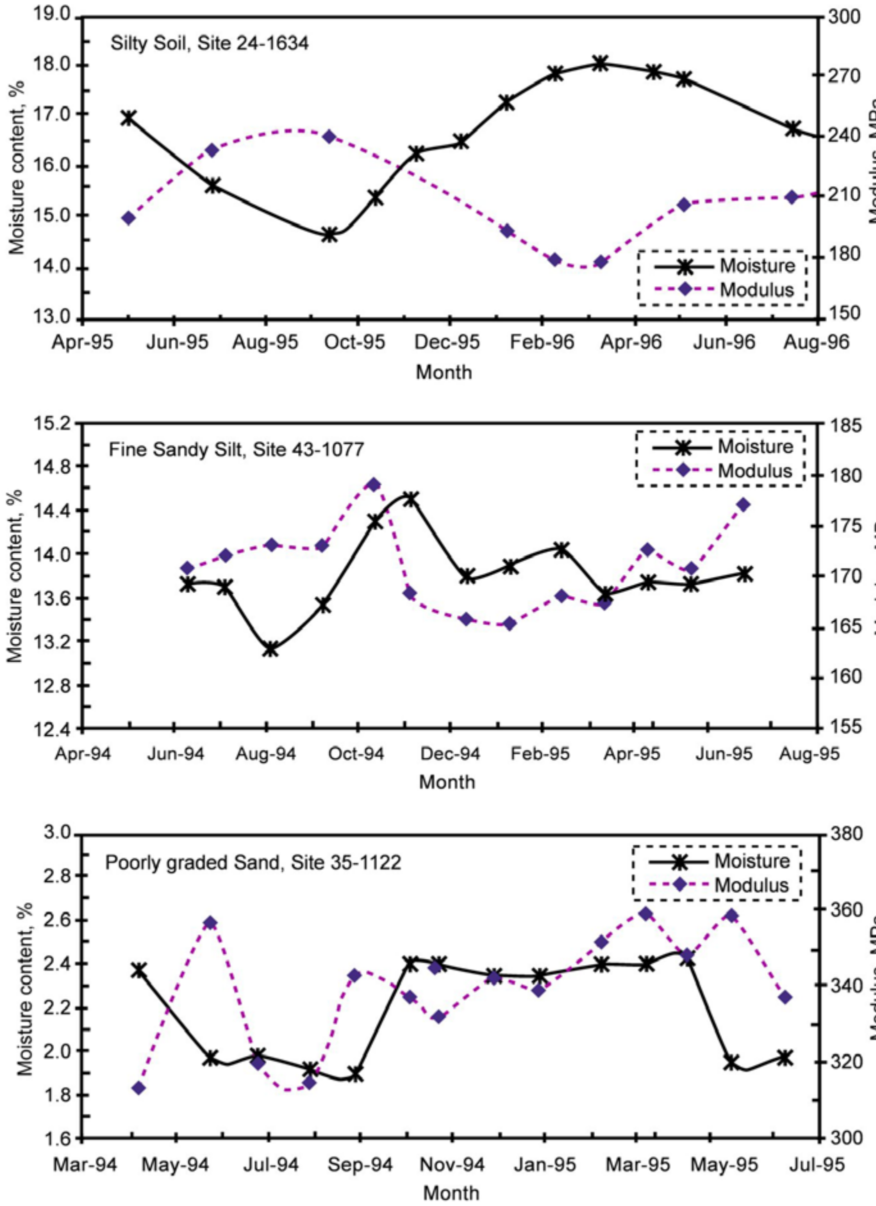


Figure 13. Variation of resilient modulus and gravimetric moisture content with time for various soil types. (Bayomy, 2002)

3.4 EFFECT OF LEVEL FOR GROUND WATER TABLE

McGaw (1972) has investigated the effect that the level of the ground water table and frost penetration velocity (rp) has on the frost heave development. Two fine graded soils were investigated and the figures below show that these two parameters have a significant effect on the frost heave ratio. In all frost penetration velocities the frost heave ratio decreases when the depth to the ground water table is getting bigger.

The frost heave is caused by water formed in ice lenses when the frost is penetrating through the frost susceptible materials. A large frost heave ratio indicates a large quantity of excess water in the soil and this will cause problems during thawing. It is obvious that lowering the ground water table reduces the rate of deterioration of the road during the spring thaw.

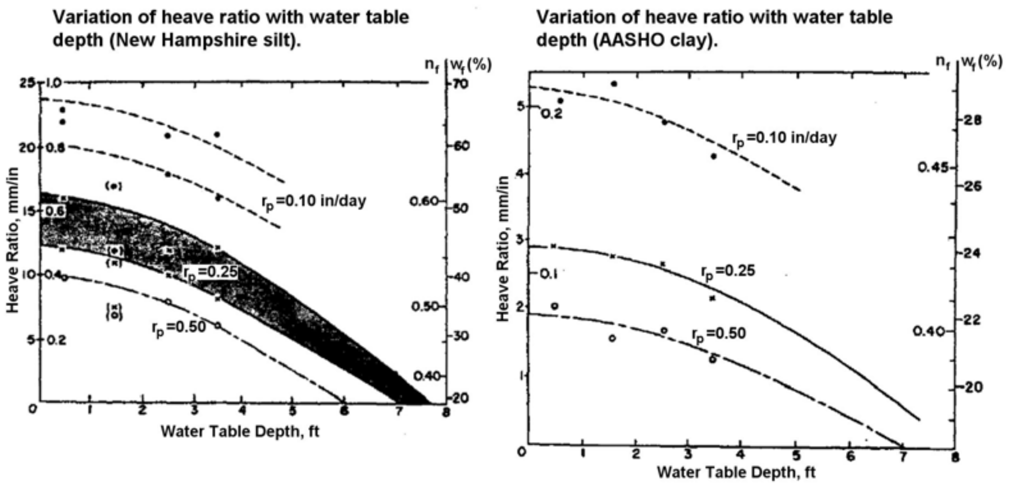


Figure 14. Frost heave ratio as a function of ground water table depth and frost penetration velocity.

P. M. Noss (1978) investigated, in his doctorate study, the interdependency of suction and moisture content. There are many parameters affecting this relation as, for example, if the material is wetting or drying.

The suction/moisture relationship was tested for 5 different gravel samples in the laboratory. The grain size distributions are shown in figure 15 and are typical material used as unbound subbase. The materials range from not frost susceptible to low frost susceptibility.

Figure 16 shows the soil water characteristic curve for these materials and the laboratory findings correspondences to the result from the field measurements. The materials with the least fines have less moisture content than the others when the suction increases. Just above the water table, the moisture content is almost the same for all materials.

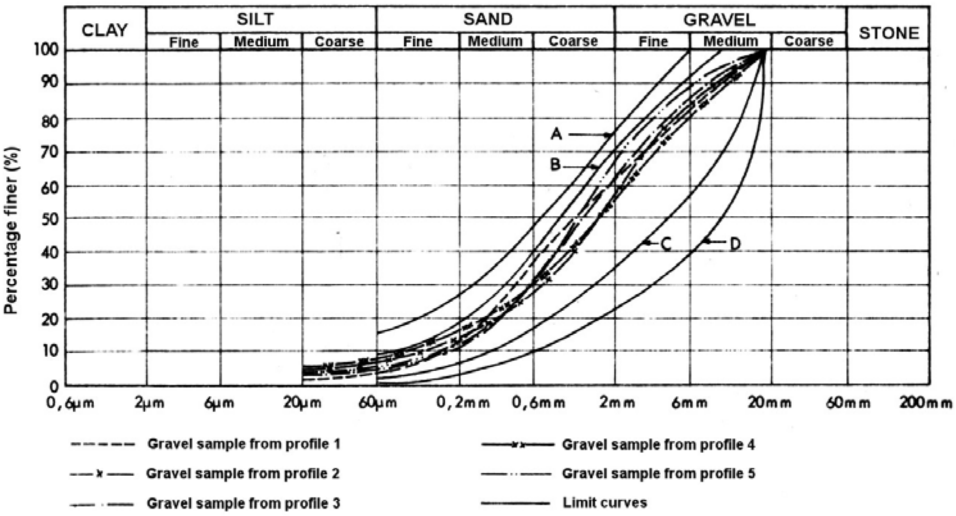


Figure 15. Grading for the tested materials. Noss (1978).

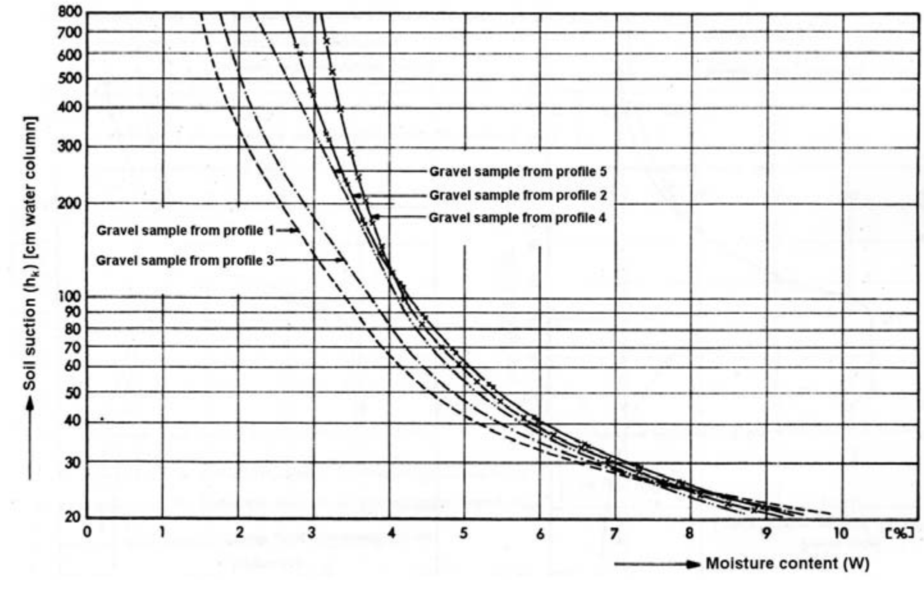


Figure 16. Soil water characteristic curve (SWCC) for granular material in figure 3.12 Noss (1978).

4 Moisture content and material properties

4.1 EFFECT OF MOISTURE ON MATERIAL PROPERTIES

Traffic loads causes stresses and strains on the road structure. Interdependency between the stresses and strains on the road construction materials are complex. The majority of the strain is recoverable or resilient and a part is permanent. This permanent strain cause permanent deformation to the road surface. The extent of the permanent strain is a function of the resilient strain.

Due to their magnitudes, the resilient strain and deformation is easier to measure than the permanent strain. The resilient modulus is the stress divided by resilient (or elastic) strain and is often referred to as elastic modulus.

The resilient modulus of a material depends on factors such as stress state, density, suction and moisture content. For frost susceptible materials the thawing period will be critical because the material has low density caused by the ice lenses and the moisture content and pore pressure is unfavourable. This problem is not a drainage problem, but an effective drainage system may reduce the spring thaw problem. Still, in the following review, spring thaw condition has been described.

The stresses and strains in and under the road structure are used to calculate the fatigue of the pavement. To calculate this deterioration a resilient modulus of materials is needed along with their dependency of moisture content. The simplest way to describe interdependency is to assume that the behaviour of the material is linear and elastic. It is also possible to use other models.

In order to calculate the deformation in a material at a specific stress level, the relationship between stresses and strains (both elastic and permanent strains) should be known. The simplest way to describe relationship is to assume that behaviour of the material is linear and elastic. Under a single load, most of the deformation caused by this load is elastic and resilient but a part is also permanent. Under repeated loads the amount of permanent deformation increases until it reaches the point where the amount of permanent deformations increases at very slow rate, but at high stress levels the deformations may start to increase again. Moisture affects both the resilient and permanent deformation properties of soil.

4.2 RESILIENT DEFORMATION PROPERTIES

Resilient modulus describes the interdependency of stress and resilient (or elastic) strain and is needed to calculate the affect a load has on a road structure.

As such, investigations of the resilient modulus of unbound materials in a road structure under different moisture content have been an area of interest in so many laboratory and field tests. The falling weight deflectometer (FWD) is mostly used to assess the stiffness of a pavement. Modulus of the pavement layers can be calculated by using FWD data and different backcalculating methods. Zhang and Macdonald showed that the resilient modulus depends on the backcalculating method used, for example for the base, which was, in this case, graded crushed natural aggregate and aggregate size ranging from 0-32 mm, the resilient modulus was two times larger using FEM on 5-layer system (finite element method) than using MET on the 3-layer system (method of equivalent thicknesses). In some studies instead of FWD is used seismic pavement analyzer (SPA), developed by University of Texas, USA. It shows similar trends to that of FWD trends, but it gives greater values than those measured by FWD (Zaugloul et al. 2002). Seismic modulus is low-strain linear elastic modulus (Zaugloul et al. 2002), when resilient modulus is non-linear elastic modulus. Behaviour of both seismic and resilient modulus under loading depends on the material.

Following is a review of the studies, which have investigated the behaviour of resilient modulus in different circumstances. Both laboratory and field tests are considered.

4.2.1 Laboratory tests

Johnson et al. (1978) conducted field plate bearing tests and laboratory resilient modulus tests on silty soils during freeze-thaw cycling. Tests showed that during the critical thaw period the modulus of a material could be as low as 2 MPa, increasing to 100 MPa and higher when the material had fully recovered. However, immediately after thawing the material was too loose to be tested without prior consolidation, thus it was not possible to test the minimum resilient modulus. Minimum resilient modulus was about 1-2 % of the fully recovered modulus immediately after thawing, the modulus had reached only 5-10% of the recovered value after 20 days and only 12-20 % after 60 days of thawing (Simonsen & Isacsson, 1999).

Cole et al. (1986) investigated the resilient modulus of various fine and coarse-grained soils during a freeze-thaw cycle. The authors observed that the resilient modulus was highly dependent on the soil state, decreasing at least two orders of magnitudes upon thawing. The resilient modulus increased by a factor of about 2 as the material recovered from the effect of a freeze-thaw cycle (Simonsen & Isacsson, 1999).

Berg et al. (1996) also examined various fine and coarse-grained soils during a freeze-thaw cycle. The observations were similar than those of Cole et al. All materials exhibited an increase of two or three orders in resilient modulus at temperatures below -2°C. The authors also observed that the moisture level greatly influenced the unfrozen modulus, but to a different degree, depending on the material. When unfrozen, the resilient modulus of all materials was stress dependent and also increased as the degree of saturation decreased (Simonsen & Isacsson, 1999).

Simonsen and Janoo (1999) also conducted resilient modulus laboratory tests during full freeze-thaw cycling. Various coarse and fine-grained subgrade soils were continuously tested at selected temperatures from room temperature to -10°C and back to room temperature. Soils were tested inside a triaxial cell, thereby eliminating external disturbances during handling. The results indicated that all soils exhibited a substantially decreased resilient modulus after a freeze-thaw. The authors observed a decrease of about 60 % in resilient modulus for clay soils and a decrease of approximately 25 % in resilient modulus for coarse gravelly sand. A review of the literature on general aspects of the resilient behaviour of unbound materials is found in the work of Lekarp et al. (1999) (Simonsen & Isacsson, 1999).

Yuan and Nazarian (2002) investigated the modulus-moisture relationship under constant compaction with the help of a free-free resonant column test. The authors noticed that modulus-moisture relationship under a constant stress level exhibits two patterns depending on the material. For the fine-grained material the relationship resembles the typical moisture-density curve. The maximum modulus occurs at a moisture content of about 13 % which is less than the optimum moisture content which is about 18 % for that type of material. For moisture contents greater than the value of peak modulus, the resilient modulus decreases with an increase of moisture. Clean, coarse-grained material, with optimum moisture content of about 8 %, modulus-moisture relationship under a constant stress level demonstrates a different trend than fine-grained material. The modulus increases with a decrease in moisture until a point about 3 %. Below that moisture content, the specimens are so fragile that they could not stand without cracking. In that stage, their measured moduli are quite low. (Yuan & Nazarian, 2002).

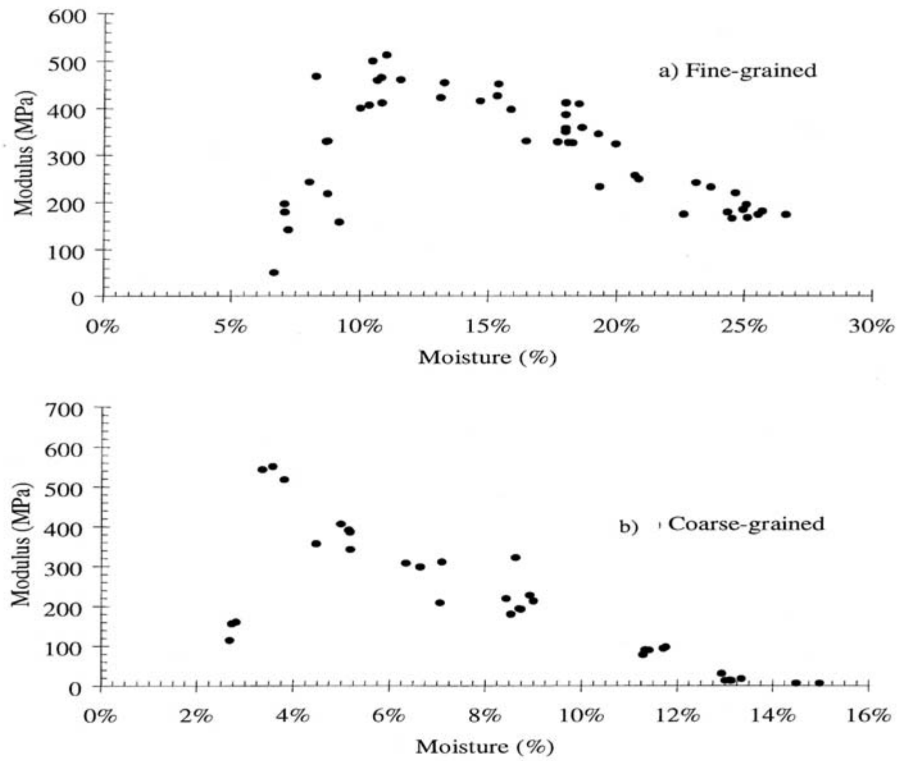


Figure 17. Variation in seismic modulus with moisture content under constant compactive effort for fine-grained (a) and coarse-grained (b) material. (Yuan & Nazarian,2002).

In summary, the results of the study of Yuan and Nazarian (2002) show that under constant stress level, the maximum modulus is obtained at a moisture content lower than the optimum. Difference between the optimum moisture content and moisture content at which maximum modulus occurs, depends on the fines content of the mixture. For clean sands, the modulus increases until the specimens become too fragile. (Yuan & Nazarian, 2002).

Yuan and Nazarian (2002) tested variations of the modulus-moisture relationship with the help of a large- scale test model. The composition of the material that was used in the base was approximately 44 % gravel, 48 % sand and 8 % fines. Maximum grain size was 20 mm and the initial volumetric moisture content was about 6.5 %. Subgrade material was practically uniform fine sand with about 3 % fines and its initial moisture content was about 12 %. The base material absorbed water during the test to a level of about 9 %, so the overall volumetric moisture content was about 15.5 %. Resilient modulus started to change 1.5 days after they had started adding water. The reduction in the water absorption rate coincided with a decrease in the rate of reduction in modulus. After three weeks, the resilient modulus was about 200 MPa, which is approximately 1/6 from the initial modulus. During the drying period, the resilient modulus increased. After 10 weeks the resilient modulus of the base material was about 1.5 times greater than the initial modulus. For the subgrade material the maximum drop of modulus was only about 30 % when it was about 85 % for the base material. This was expected due to a lack of fines in the subgrade material. After the completion of the drying cycle, the resilient modulus of the subgrade material had recovered to the value of 1.5 times the modulus at the start of the experiment. This study demonstrated the importance of the material type, especially fine content, to the moisture susceptibility of a material. This experiment also presents a great example why more accurate means of measuring the moisture content in the field should be implemented so that the results from the laboratory and the field can be compared. (Yuan & Nazarian, 2002).

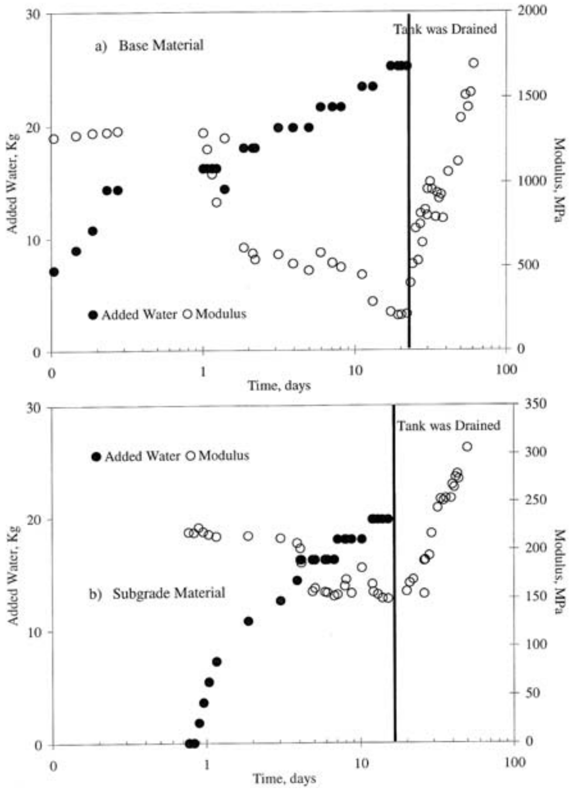


Figure 18. Variation in the modulus as a function of moisture changes in base material (a) and for subgrade material (b). (Yuan & Nazarian, 2002).

In the soak test experiment of Yuan and Nazarian (2002), it was observed that the higher the fine content was in the specimen, the greater were the changes in moisture-modulus trends during the drying and wetting cycles. If the fines content of a material increases the variations in the modulus values and dielectric values also increase. For almost all materials, except clean coarse-grained materials, different patterns between drying and wetting cycles will be exhibited. The moisture-modulus curve, shown in figure 19a, may be used to adjust the seasonal variation in the modulus of the base or the subgrade material by simply measuring the change in the moisture content of the material at regular intervals. The test material was typical base material used in El Paso, Texas, USA. (Yuan & Nazarian, 2002).

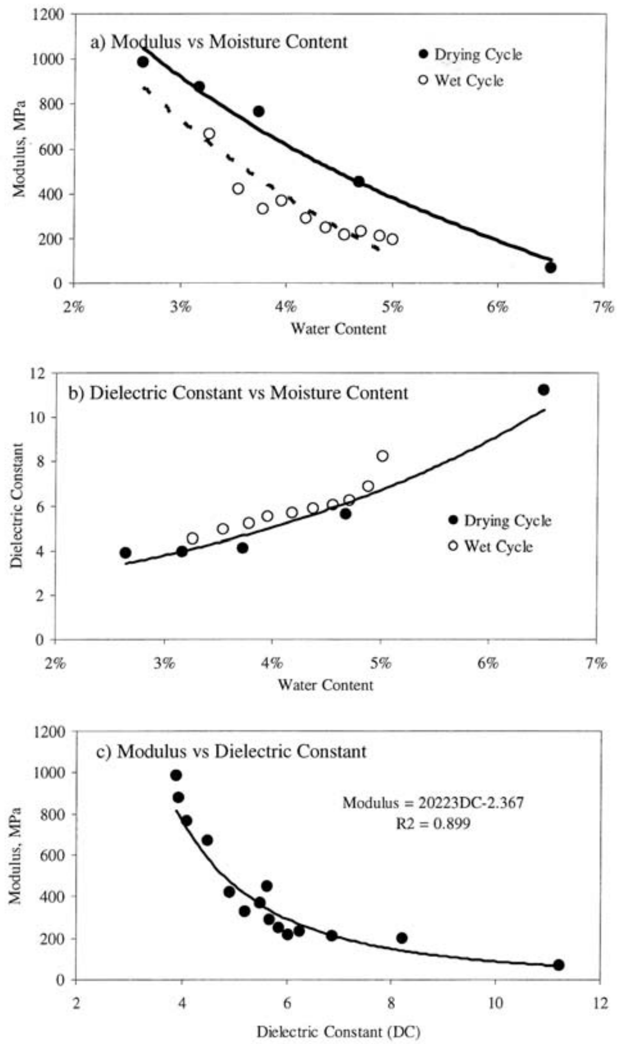


Figure 19. Variation in modulus and dielectric constant demonstrating moisture content during soak test. Modulus in the figure is seismic modulus. (Yuan & Nazarian, 2002).

Saarenketo et al. (2001) investigated changes in resilient modulus in different kinds of crushed aggregates with varying fine content. Materials were tested in three different stages, first when the sample was dry then after absorbing water and finally after a freeze-thaw cycle. Tests were performed with help of a triaxial test. Results from the test are shown in the figure 20.

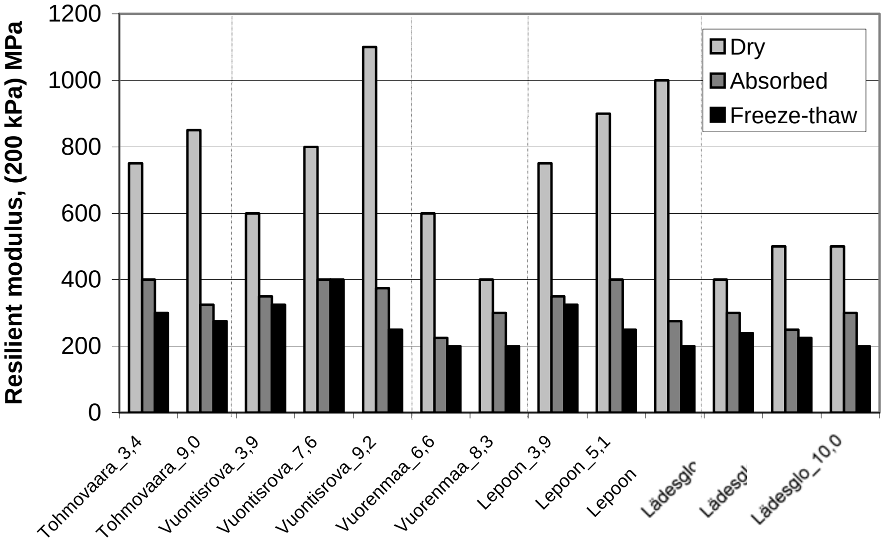


Figure 20. Variation in the resilient modulus of different kinds of crushed stone with varying fine content. Tests were performed at three different stages. (Saarenketo et al. 2001).

As seen in figure 20, if the fines content of dry crushed stone increases, then the resilient modulus increases as well with the exception of crushed stone from Vuorenmaa. When the sample has absorbed water and gone through a freeze-thaw cycle the resilient modulus decreases as the fine content of the sample increases. (Saarenketo et al. 2001).

4.2.2 Field tests

Jong et al. (1998) investigated resilient modulus with different moisture content in different field sites located in Wisconsin, U.S.A. The Unity test section consisted of a clay subgrade, a 230 mm thick gravel base and a 310 mm thick asphalt concrete while the Westby test section included clay subgrade, a gravel base varying from 250 mm to 460 mm and a 4.4 mm thick asphalt concrete surface. The FWD data was collected from several different stations at field sites and because the collected data did not differ significantly from station to station, thus average values could be used to represent the entire site. Results are shown in figures 21, 22 and 23. (Jong et al. 1998).

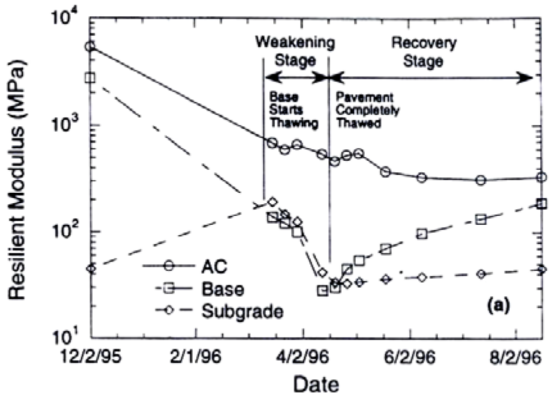


Figure 21. Resilient modulus from the Unity site in 95-96. (Jong et al 1998.)

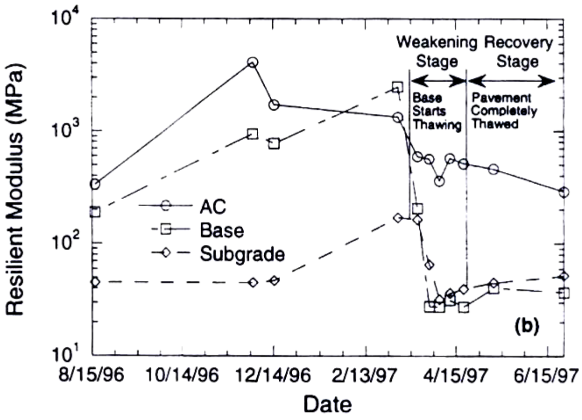


Figure 22. Resilient modulus from the Unity site. (Jong et al. 1998)

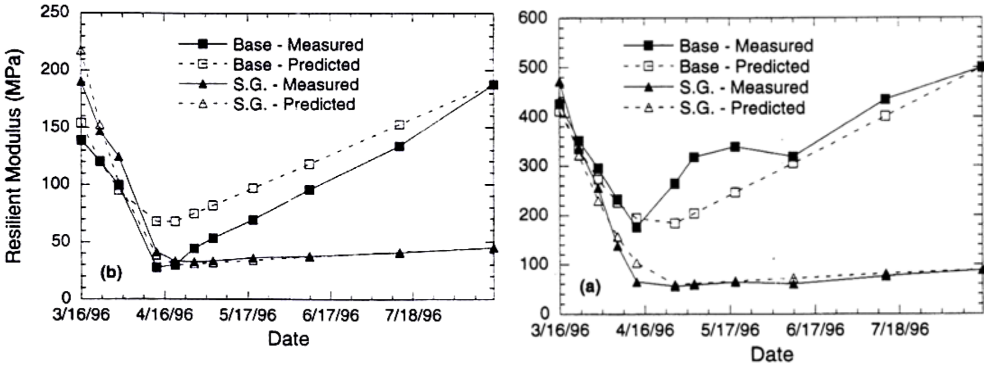


Figure 23. Resilient modulus results from the Unity site in 96-97. (Jong et al. 1998)

The main conclusions which Jong et al. (1998) made from their study were that typical resilient modulus for frozen gravel base was about 12 times higher than their values when not frozen and for the frozen clay subgrade about 4 times higher than non frozen values. Furthermore the authors noticed that when thawing was completed, the base and the subgrade modulus typically were 35 percent and 65 percent, respectively, of the prefreezing values. Similar results (reduction in the pavement modulus) have been reported by Mahoney et al. (1984) in their field study in the state of Washington, U.S.A. The resilient modulus recovered to the prefreezing values about 4 months after the thawing had completed (Jong et al. 1998).

Similar results to Jong et al. (1998) have also been found by Erlingsson et al. in their test road study in Iceland. The structure of the test road consisted of a wearing course 46 mm, base course 1 34 mm (USCS: GP-GM), base course 2 88 mm (USCS: GW-GM), subbase 1 153 mm (USCS: GW-GM), subbase 2 450 mm (USCS: SW-SM), 250 mm high embankment and subgrade (peat). The results from their tests are shown in the figure 24. (Erlingsson et al.)

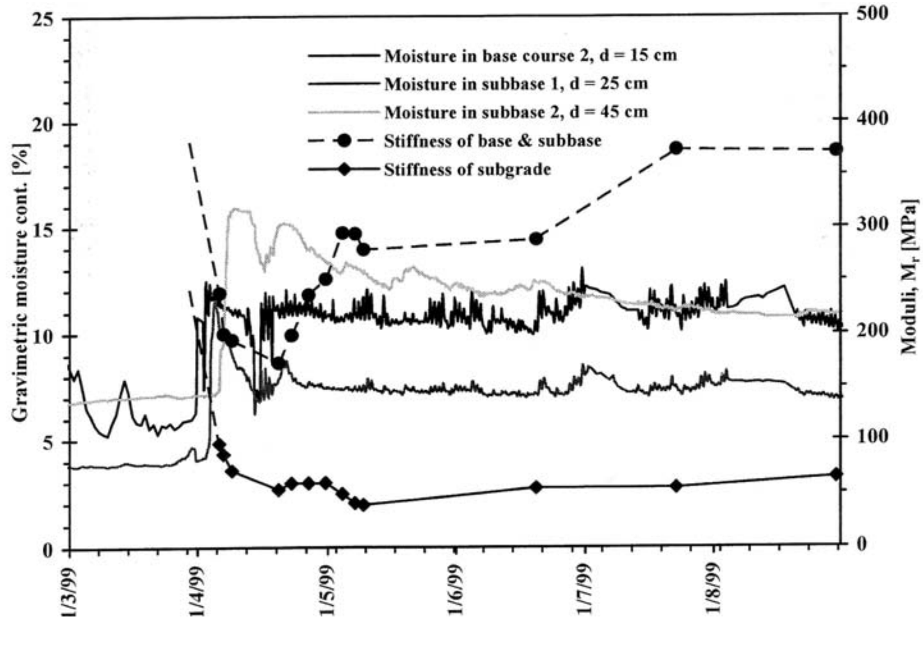


Figure 24. Resilient modulus and gravimetric moisture content from Vesturlandsvegur in Iceland. (Erlingsson et al.)

During the spring thaw period the resilient modulus was at the minimum about 173 MPa for base and subbase and about 39 MPa for the subgrade. In the late summer the resilient modulus reached the highest values, which was 374 MPa for the base and subbase and 65 MPa for the subgrade. It can been seen from the results that resilient modulus for the base and subbase was about 55 percent smaller in springtime than in late summer, and for subgrade the same decrease was about 40 %. The relationship between the moisture content and resilient moduli is also presented in figure 24. When the moisture content reached its highest value, the resilient modulus was lowest. In addition, in late summer, when moisture content decreased and was near to optimal moisture content then the resilient modulus reached its highest value (Erlingsson et al.).

Zaugloul et al. (2002) also found in their study in New Jersey in U.S.A. that reduction in the pavement modulus occurs in spring. In their study of 24 different test sections, the change in the pavement modulus was about 35 percent on average while the same range was only about 20 percent for subgrade modulus on average. Subgrade type was mainly clayey or silty sand. The authors also noticed that the reduction in the pavement modulus happens earlier than that of the subgrade modulus. In addition, they noticed that fluctuation of the ground water table has no significant impact on the pavement or subgrade modulus. (Zaugloul et al. 2002)

Janoo and Barna performed their FWD tests on full scale test sections during the thawing period at the Frost Effects Research Facility at the Cold Regions Research and Engineering Laboratory, Hanover, NH, U.S.A. The test section, which was used in this test, consisted of asphalt concrete pavement 76 mm, base course 226 mm (classified GP-GM using USCS and did not have more than 6 % fines), test subgrade 1,2 m and original subgrade 1,5 m. Both subgrades, test and original, were made from the same material (classified as silty sand and had approximately 30 % fines but the test subgrade had a moisture content of 15 % and the subgrade had only 9 %. From the in-situ stress-strain measurements, which were performed by the authors, it was observed that the reduction in the base modulus during the critical thawing period was about 64 %. The thaw-weakening period for the base was three weeks. The subgrade modulus was reduced considerably to 3-6 % of its pre-freeze values. The subgrade thaw-weakening period varied. At the top of the test subgrade the stiffness did not recovered for 88 days. The subgrade recovered to about 80-85 % of the pre-freeze values after 39 days. From the FWD tests it was noticed that base course stiffness was reduced by 29 % of its pre-freeze value during thawing period. Based on the FWD data, the thaw-weakening period for the base was two weeks. For the subgrade the reduction factor was 44 % and the thaw-weakening period was approximately 15 days. The current COE criterion for modulus reduction recommends a reduction of 70 % for the classification of this test soil. The experiment states that the current criterion significantly overestimates the stiffness of the base and subgrade soil during the thaw period. (Janoo & Barna)

Table 1. Reduction in subgrade stiffness during the critical period. (Janoo & Barna)

Layer	Depth (mm)	Subgrade Modulus		Reduction Factor
		Pre-freeze	Minimum	
Base		284	182	64%
	From FWD	59	17	29%
Subgrade	453 (top of subgrade)	420	24	6%
	762	271	9	3%
	1067	227	6	3%
	From FWD	263	115	44%

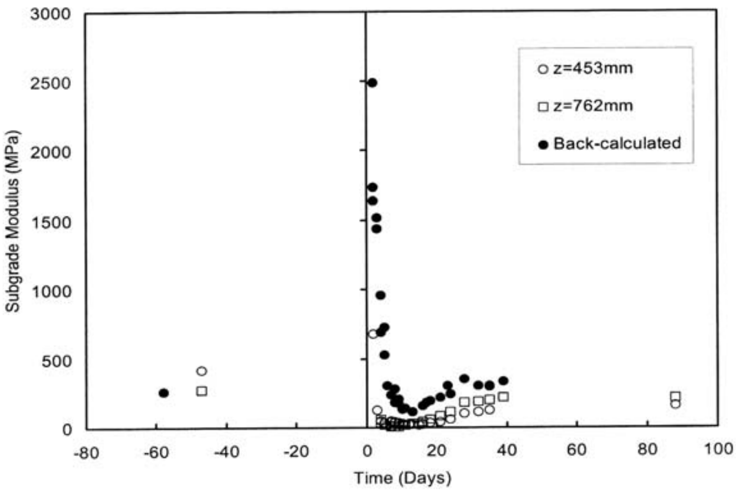


Figure 25. Change in the subgrade modulus during the thaw period. (Janoo & Barna)

Many of the studies suggest reduction factors to the resilient modulus especially during the thaw-weakening period. Ehrola (1996) wrote in his textbook that when moisture content increases 10 % the modulus of elasticity decreases 20 MPa for silty subgrades. Drumm et al. noticed in their study that a change of 1.5 % in volumetric moisture content at fine-grained soils would result in a change in the degree of saturation by 4.75 % and determined that the resilient modulus would decrease by almost a factor of two, from 130 to 70 MPa, for the same increase of saturation (Heydinger, 2003). Heydinger (2003) also stated that the resilient modulus can vary by a factor >2 for changes in saturation of 10 % to 15 %. Janoo and Berg found that clay subgrade was weakened by a factor ranging from 2 to 2,2 when it was subjected to a freeze-thaw (Janoo & Berg, 1990). Table 2 presents reduction factors which the U.S. Army Corps of Engineers has suggested for the frost-susceptible soils.

Table 2. The U.S. Army Corps of Engineers modulus reduction factors for frost-susceptible soils. (Janoo & Barna)

Frost Group	Soil description	Percentage finer than 0.02 mm (by weight)	Typical Soil Types (USCS)	Reduction Factors for Thaw Periods (% of non-frost modulus)
NFS*	(a) Gravel, crushed stone, crushed rock	0 - 1.5	GW, GP	100
	(b) Sands	0 - 3	SW, SP	
PFS**	(a) Gravel, crushed stone, crushed rock	1.5 - 3	GW, GP	90
	(b) Sands	3 - 10	SW, SP	
S1	Gravelly Soils	3 - 6	GW, GP, GW-GM, GP-GM	75
S2	Sandy Soils	3 - 6	SW, SP, SW-SM, SP-SM	70
F1	Gravelly Soils	6 - 10	GM, GW-GM, GP-GM	60
F2	(a) Gravelly Soils	10 - 20	GM, GW-GM, GP-GM	50
	(b) Sands	6 - 15	SM, SW-SM, SP-SM	
F3	(a) Gravelly Soils	> 20	GM, GC	30
	(b) Sands, except very fine silty sands	> 15	SM, SC	
	(c) Clays, PI > 12	----------	CL, CH	
F4	(a) Silts	----------	ML, MH	30
	(b) Very fine silty sands	> 15	SM	
	(c) Clays, PI < 12	----------	CL, CL - ML	
	(d) Varved clays and other fine-grained, banded sediments	----------	CL, ML and SM, CL, CH and ML, CL, CH, ML and SM	

* Non frost susceptible
** Possibly frost susceptible

Bayomy et al. (2002) also studied variation in the resilient modulus and moisture for different subgrade materials during different seasons in various locations in the U.S.A. Test section 48-4143 had a rigid surface and the other test sections had a flexible surface. Results from the study are shown in figures 30 and 31. Moisture content in the figures is gravimetric moisture content. (Bayomy et al. 2002)

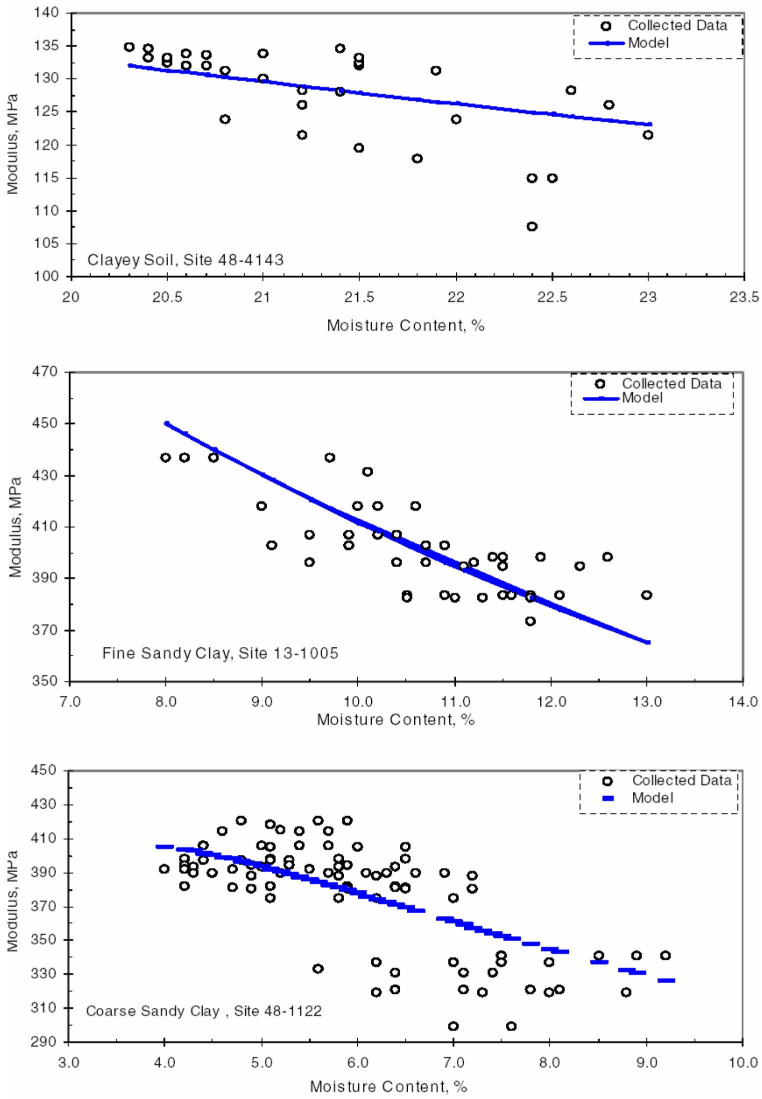


Figure 26. Modulus- moisture relationships for plastic soils. (Bayomy et al. 2002)

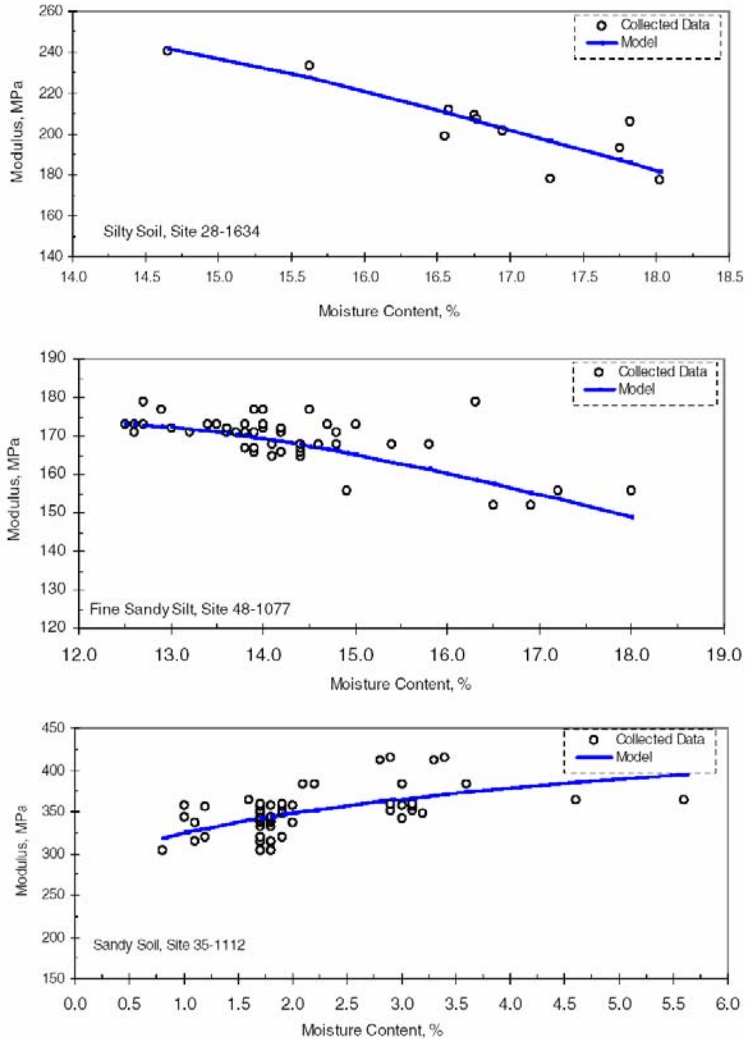


Figure 27. Modulus- moisture relationships for non-plastic soils. (Bayomy et al. 2002)

Lary and Mahoney (1984) have done investigation on deterioration for 4 test sections in the U.S.A. The resilient modulus was calculated as a function of moisture content, dry density and stress state, and the regression equation was generated through laboratory analyses. These regression equations can be expressed in the following form:

$$\log M_r = k_1 + k_2 \cdot \log \theta_1 + k_3 \cdot w + k_4 \cdot \gamma_d$$

k_1, k_2, k_3 and k_4 are constant parameters for each soil
w - moisture content
γ_d - dry density
θ_1- sum of principal stresses (1. stress invariant)

If all parameters are constant except the moisture content the equation can be written like:

$$M_r = K \cdot 10^{k3 \cdot w} \quad \text{where K is a constant.}$$

If the regression equation is transformed to SI-units, k3 for base material varies between -0.0124 and -0.0324. For the subgrade the parameter varies from -0.0122 to – 0.0554. This is shown in figure 30 for base materials and the worst moist susceptibility subgrade in the equations in table 2. (Ref. Berntsen 1993).

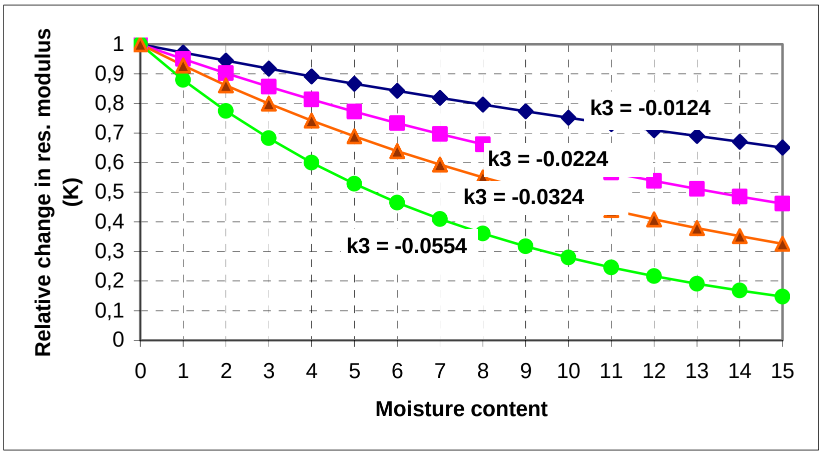


Figure 30. Relative change in resilient modulus. (Berntsen G. 1993)

For the most water susceptible material a change of 1 % in the moisture content will reduce the resilient modulus by 7.2% and the materials that are least water susceptible the modulus will change by 2.8 %.

If a base gravel material has a resilient modulus equal to 150 MPa, a 1 % change in moisture content will change the resilient modulus by 4.2 to 10.8 MPa.

4.3 PERMANENT DEFORMATION PROPERTIES

Permanent deformations in the road structure are those, which can be seen from the top of the road structure in the form of rutting. These are the deformations to which a normal motorist pays his attention. Permanent deformations include rutting and vertical imbalance of the road structure. Rutting includes plastic deformation in bound layers, compaction from traffic loads, wearing of the pavement and permanent deformations from the base, sub-base and subgrade. Lampinen (1993) studied wearing caused by studded tyres in Finland and noticed that studs caused 70 % from the rutting (Ehrola, 1996). However since early 1990´s, wearing caused by the studded tyre has markedly decreased due to developments in studs, tyres and pavement. This leads to the fact that the relative proportion of permanent deformation in the base and subgrade as a cause of rutting has increased. Also wearing due to studded tires is only a problem on roads where the traffic volumes are greater then 2500-3000 AADT and in the ROADEX-II project, the main focus is on low volume traffic roads.

The following sections present a review of the literature concerned with permanent deformation of the road structure. Because the investigation of permanent deformation is rather laborious and complicated the number of studies found that investigated the matter was quite small.

4.3.1 Laboratory tests

Zhang and Macdonald (2002) conducted tests to measure permanent deformations in the road structure with help of a road testing machine in Denmark. The structure of the experimental pavement, which was used in the tests, consisted of 84 mm thick asphalt, a 140 mm thick base, made from graded crushed natural aggregate, a subgrade 1376 mm thick, made from clayey silty sand, and at the bottom a 181 mm thick drainage layer resting on a 250 mm thick concrete slab. After two freezing and thawing periods and a total of 4800 load repetitions applied during the both cycles, the IRI increased from 1.8 m/km, which was measured after 160 000 load repetitions, to 4.36 m/km. At the same time the average rut depth also increased from 10 mm to 22 mm. Also during this test period a soil deformation transducer measured plastic strains at the top of subgrade and this showed that 15 mm of the total 22 mm rut depth occurred in the subgrade. That is about 60 % of total permanent deformation measured at the pavement surface. On the other hand, the Finnish TPPT-Project has obtained quite different results. In that project, researchers noticed that 80 % of the permanent deformation happens in the unbound base course, 10-13 % in the subbase and only 7-10 % in the subgrade. The most important individual factor that should be measured in the subgrade during the thawing period are the pore water pressures. Zhang and Macdonald (2002) found that 60-75 % of the total increase in plastic strains in the subgrade happens during the thawing period. They also stated that rate of pavement deterioration caused by thaw weakening of unbound granular materials and subgrades in the road pavements during the thaw weakening periods is orders of magnitude greater in frost affected regions than in the more warmer regions, above 0°C, under the same load conditions (Zhang & Macdonald)

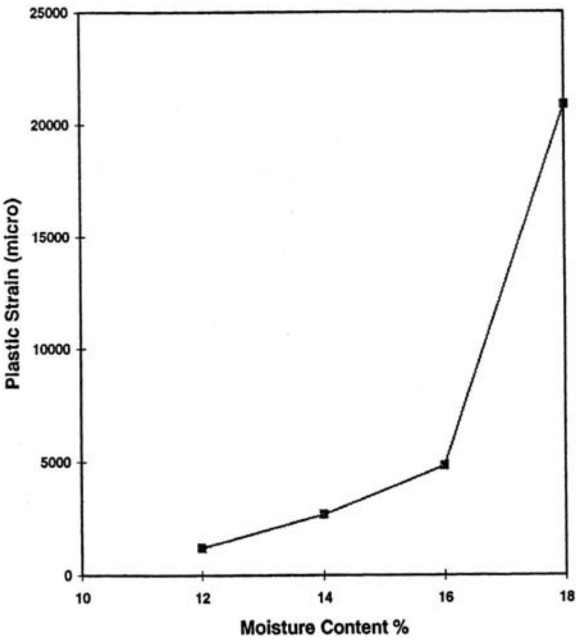


Figure 31. Effect of moisture content on plastic strain of silty clay. Vertical stress in 150 kPa. (Behzadi & Yandell, 1996)

Behzadi and Yandell (1996) investigated resilient and permanent deformation properties of silty clays, which is normal subgrade material in Australia. The tests performed were triaxial tests with repeated loading. Silty clay was tested under different moisture content. Results are presented in figure 33. (Behzadi & Yandell, 1996)

As seen from figure 33, the permanent strain at 10 000 cycles increases from 1200 µstrain, measured with a moisture content of 12 %, to about 20 900 µstrain with a moisture content of 18 %, which is almost 18 times larger than the plastic strain at start of the test. At the same time, saturation of silty clay rose from 55 % to 82 %. (Behzadi & Yandell, 1996)

In the EU-supported RTD project on Construction with Unbound Road Aggregate in Europe (COURAGE) the seasonal variability effects on granular materials within constructed pavement was investigated. 11 test sections in 5 countries were used.

The permanent deformation was tested in laboratory for different moisture content and dry density. The moisture content is given as a percentage of optimum moisture content (OMC) and the dry density as a ratio of maximum dry density (DDR).

The permanent strain model used in this work is:

$$\varepsilon_{1p}(N) = A_1 \cdot \left[1 - \left(\frac{N}{100}\right)^{-B}\right]$$

where:

A_1 - “strain rate-type” parameter
B - parameter
ε_{p1} axial permanent strain with removal of first 100 cycles
N - number of cycles

The results from the tests demonstrate that the effect of moisture on permanent deformation is huge. Figure 32 gives examples on how the permanent strain parameter A1 varies when the RMC (Relative Moisture Content) increases. The right diagram shows the strain rate ($d\varepsilon_p/dN$) as a function of RMC.

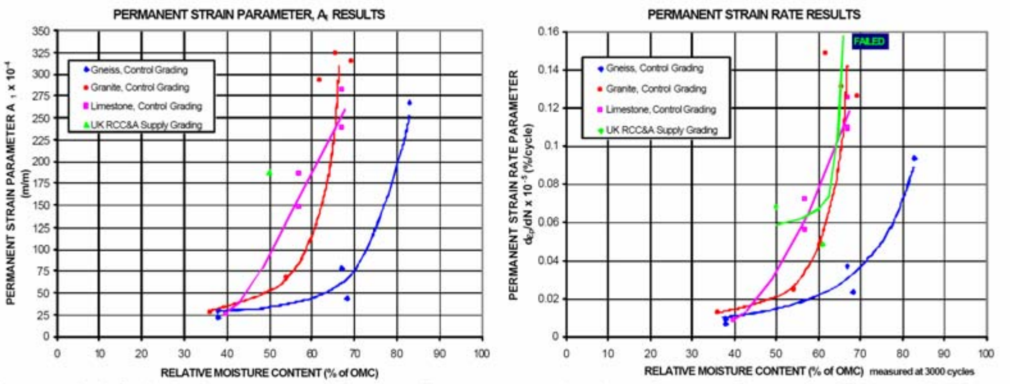
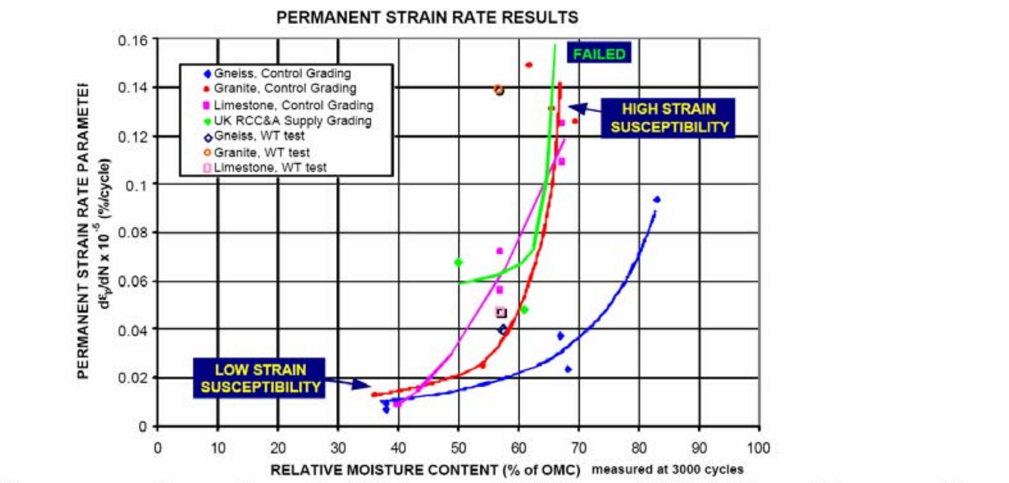


FIGURE 6.16 & 6.17: PERMANENT STRAIN PARAMETER A_1 AND STRAIN RATE $d\varepsilon_p/dN$ VS RMC AT DDR=97%, CONTROL GRADING

Figure 32.

All materials undergo rapid strain increases once a moisture content that is approximately 60 % of the optimum is reached. Two of the materials, recycled concrete and asphalt (RCC&A) and the granitic material completely failed at 67 % of the optimum moisture content. The tests were performed using repeated load test (RLT) with a relative high rate of strain.

The tests were also performed using a wheel tracking test and even though the stress level was lower, the result was much the same as in the RLT test. Figure 33 demonstrate this.



PERMANENT STRAIN RATE $d\varepsilon_p/dN$ AGAINST RMC FROM RLT & WHEEL TRACKING TESTS

Figure 33. Results from COURAGE

Some of the conclusions presented in the Courage final report were:

- In all the European countries in which pavements were monitored for seasonal moisture changes, it was observed that the variation in all structural layers followed clearly defined seasonal variations, with the moisture content being the highest in autumn and spring
- Considerable seasonal moisture variations can occur in the pavement structure from country to country, primarily due to material type, temperature and rain/snow fall.
- Pavements founded on embankments are more likely to perform better than pavements through cuttings. In pavement structures through cuttings the moisture content was found to be slightly higher, but the variation was lower, compared to a pavement structure found on an embankment. For structures through cuttings, water has the opportunity to flow into the pavement layers particularly if deep side drains are not installed.
- In pavement structures located in road cuttings moisture content was found to be slightly higher, but the variation was lower, compared to pavement structures built on an embankment. Water can flow into the pavement layers In road structures located in road cuttings, particularly if deep side drains are not installed.
- The moisture in the pavement structure is very dependent on the:
  - precipitation levels
  - the integrity of the sealed surface
  - the final preparation applied to the shoulders of the pavement (sealed or unsealed and seal width, partial or full)
  - level of the pavement (raised pavement or pavement in cutting)
  - ability of the pavement to self drain (permeability)
  - adequacy of the pavement's drainage system
- Increased moisture in the pavement structure has a detrimental influence on the bearing capacity of the pavement. This is strongly linked to the reduced levels of resilient modulus and higher susceptibility to permanent deformations of UGM when their moisture content increases. FWD surveys showed that in Northern countries,

increases in moisture content of the granular layers were accompanied by increased pavement deflections and a reduction in strength which was most noticeable after the spring thaw.

4.3.2 Field tests

In field study of 120 pavements in Alaska, more than 200 parameters and their influence on road performance were studied by McHattie et al. 1980; Esch et al. 1981; Esch and McHattie 1982, 1983. Studies suggested that the proportion of fines (particles smaller than 0.075 mm) in the base and sub-base was the most important parameter influencing cracking and bearing capacity during thawing. (Simonsen & Isacsson, 1999)

Janoo and Barna conducted their tests on full scale test sections during the thawing period at the Cold Regions Research and Engineering Laboratory, Hanover, NH, USA. Strains were measured with strain gauges and permanent deformations using a laser profilometer. They noticed in their tests that critical strains during thaw-weakening in the base for the crushed ledge were 3-5 times larger than pre-freeze values. In the subgrade, which was made from silty sand, critical strains were over 10 times its pre-freeze values. Permanent deformations at the end of thawing in the base and subgrade were 12 and 55 mm. (Janoo & Barna)

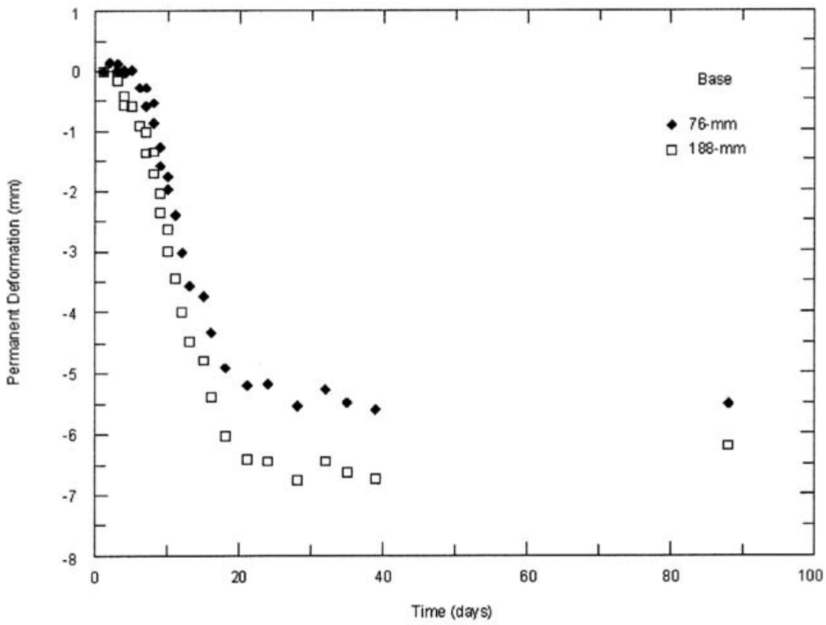


Figure 34. Permanent deformation in the base during thawing period. (Janoo & Barna)

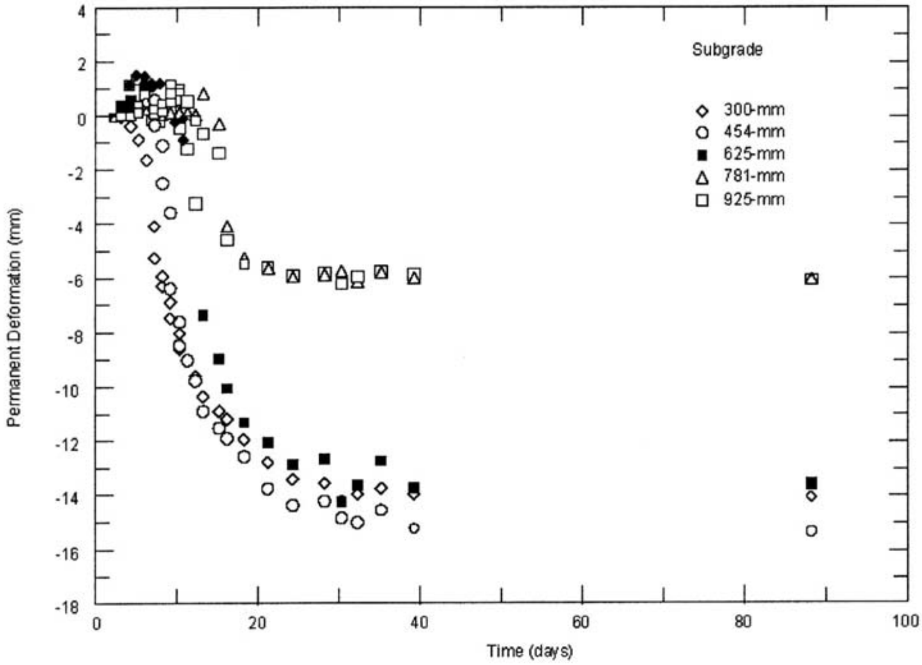


Figure 35. Permanent deformation in the subgrade during thawing. (Janoo and Barna)

5 Drainage problems classification and their solution

5.1 GENERAL

The typical drainage problems in the NP area were mapped using a questionnaire. Even though the ground conditions, landscape and climate varies a lot throughout the NP-area, the drainage problems are basically the same. The exception being Scotland where there are some special problems related to the use of grass verges. The problems are grouped in three main categories as shown in figure 36.

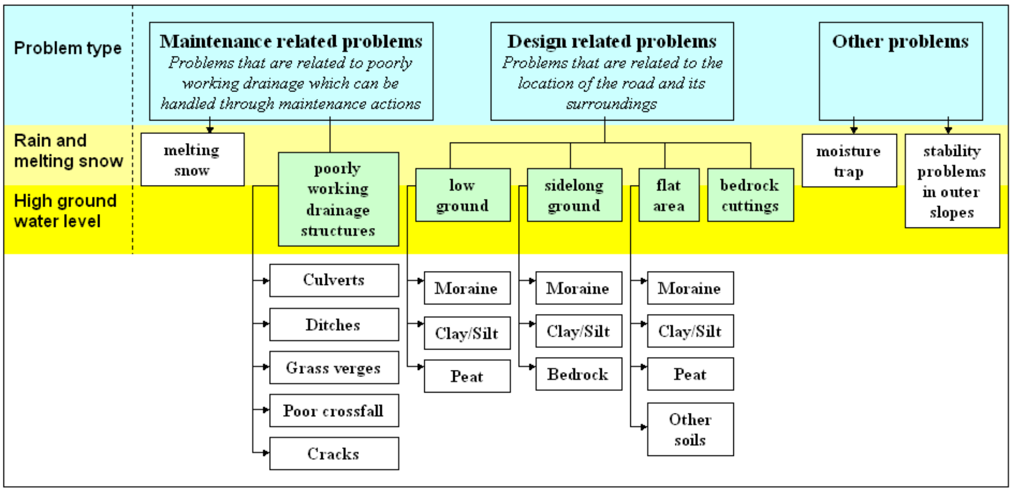


Figure 36. Category of drainage problems.

In the following sections the recorded problems are described by:

- problem description
- how to recognize the problem
- what causes the problem
- estimates for how to improve the drainage

5.2 MAINTENANCE RELATED PROBLEMS

5.2.1 Problems caused by melting snow

During the winter, frost can penetrate roads to a depth of more than 3 m depending on climatic conditions and the subsoil. The snow behaves as insulation, and the snow banks will cause more shallow frost penetration at the edge than in the middle of the roadway.

When frost susceptible soil freezes excess water is transported to the frost fringe and forms ice lenses that cause frost heaves. For this to happen there must be water available.

During thawing periods there can be a lot of water from melted snow and possibly even from spring rains on the road surface and in the ditches. In areas that have a low annual average temperature, thawing starts mainly from the top and proceeds downwards and a small amount of the thawing starts from the bottom of the frozen soil and proceeds upwards. In warmer areas with a higher average annual temperature the temperature in the ground (2 meter below the surface) is also higher. In these areas, a greater part of the thawing starts from the bottom and proceeds upwards, as such, the excess water will not be blocked by a nearly impermeable frozen soil. The frozen soils are almost impermeable in comparison to non-frozen soils. The melt water and rainwater do not drain because the ditches are filled with snow and, as such, are not functioning. Excess water from the ice lenses, in that case, has only one path through which to drain that being upwards through the road structure, which together with the surface water will cause an excess pore water pressure. This together with the high moisture content will reduce bearing capacity during these periods.

In Scotland and in coastal areas in Norway there are several periods when the temperature is above 0°C during the winter. These freeze-thaw periods act in the same manner as described above although the thawing periods are shorter.

The only direction the excess water from the ice lenses can flow is upwards through the road structure which in turn will result in an excess of pore pressure. This together with the high moisture content will reduce the bearing capacity during these periods.

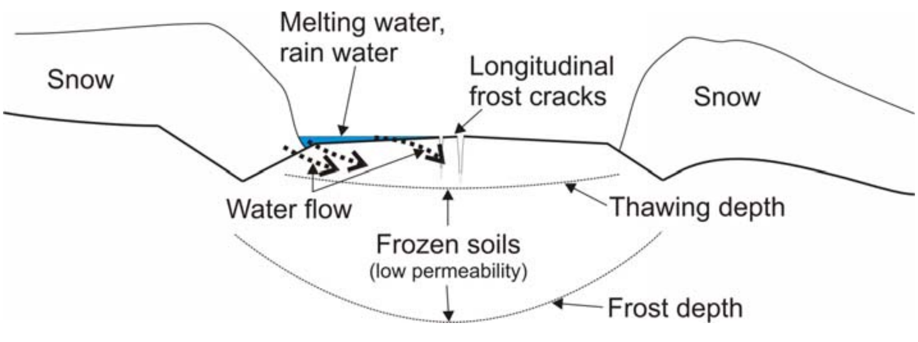


Figure 37. Drainage problem during spring thaw.

Figure 38 illustrates the changes in pore pressure during the thawing period. There will be a temporary ground water table above the thawing front that changes the stress state radically because of the changes in the pore pressure.

This will reduce the resilient modulus for water susceptible materials above the thawing front as described in the previous literature review in this report.

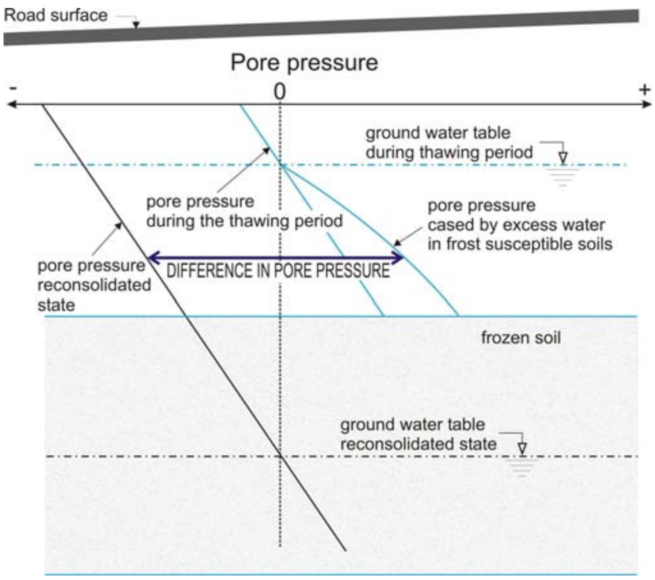


Figure 38. Pore pressure in the pavement structure during spring thaw.

Improvement techniques – suggestions:

- Snow can be cleared from the ditches during the thawing periods to remove the surface water. See figure 39.

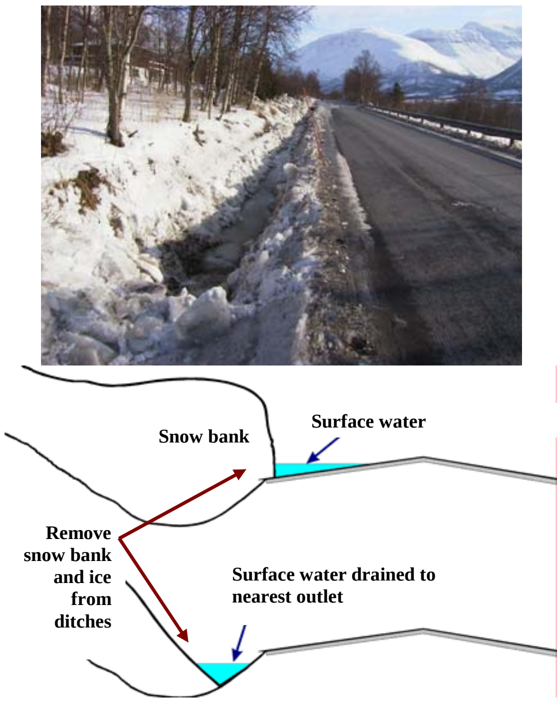


Figure 39. Removal of ice and snow from the ditch.

- Use of deep drainage
- Frost insulation (expensive).
- Raise the carriageway and make wider and deeper ditches.
- Use of non water- or frost susceptible material in the road structure.
- When designing a road structure, the bearing capacity of the subgrade soils during the critical spring thaw period must be considered.

5.2.2 Poorly working drainage structures

When there is a lack of funding for road maintenance, the elements that are most important for road users in the short term are often prioritised and measures that are important for the road condition over the long term are postponed. The consequences of which are that the drainage is not maintained as it should be and the moisture causes increased deterioration because of the reduced bearing capacity of the materials, differential frost heave and erosion problems.

5.2.2.1 Culverts

Culverts along the road

There are culverts under the road transversely and alongside the road under private accesses and road intersections.

The culverts along the road direct guide the water in the ditches under road intersections and private accesses to the nearest outlet. These culverts normally have a smaller diameter than the culverts under the road transversely and the water also has a lower stream velocity. The culverts are often clogged and, as such, will elevate the ground water table in the road structure. These small diameter and clogged culverts can cause water flow to the road and severe erosion problems (see figure 40).

Figure 40. Clogged culvert causing erosion problems after heavy rains (photo Veikko Puranen).

The culverts in the private accesses are placed at the bottom of the ditch, which is shallow compared to the culvert across the road. Because of the low flow velocity, fines are easily deposited in the culvert and reduce the effective drainage area. Shallow placement, low flow velocity of the water and a limited drainage area make the culvert susceptible to frost. Ice can clog the culverts and the problem can be severe during the spring thaw when the snow melts and the need for well working culvert is urgent (figure 41)

The length of the culverts depends on the width of the access and is often large where there are shops, petrol stations and other businesses. These are more exposed to frost and ice and more difficult to clear of fine soils and rubbish.

Ice clogging the culvert.

Improvements:

- Clearing of the culvert
- Steam the culvert to remove the ice blockage
- Solar panel and heater cable
- For difficult problems there may be a need to replace the drainage system with deep drainage and an outlet basin with a sand trap.

Transverse culverts

Transverse culverts are the culverts that drain water through the road.

Clogging of the culvert and/or the inlet

Figure 42. Clogging of culvert.

If the stream velocity is lower in the culvert than upstream sand and gravel will be deposited in the culvert. As such, an important maintenance operation is to clear the culvert when the amount of deposited material has reached a predetermined level. If this is neglected the culvert will not have sufficient capacity to drain the water and the water will flow across the road surface and into the road structure. This is a traffic safety problem and may also cause erosion with the consequence that the road is washed away as can be seen from the picture in figure 42.

The inlet of a culvert may also be clogged by branches, mud, gravel, rubbish and other things. The consequence is the same as described above.

Improvements:

- Inspect and clear both the inlet and the culvert when necessary.
- Reconstruct the inlet.

Defective culverts

Due to frost heaves, settlements or faulty construction, the culverts crack and deteriorate. The consequence is that water will flow uncontrolled and may cause erosion and raise the ground water table. When extreme water flows occur, the result may be that the road structure will be washed away and the road must be consequently closed.

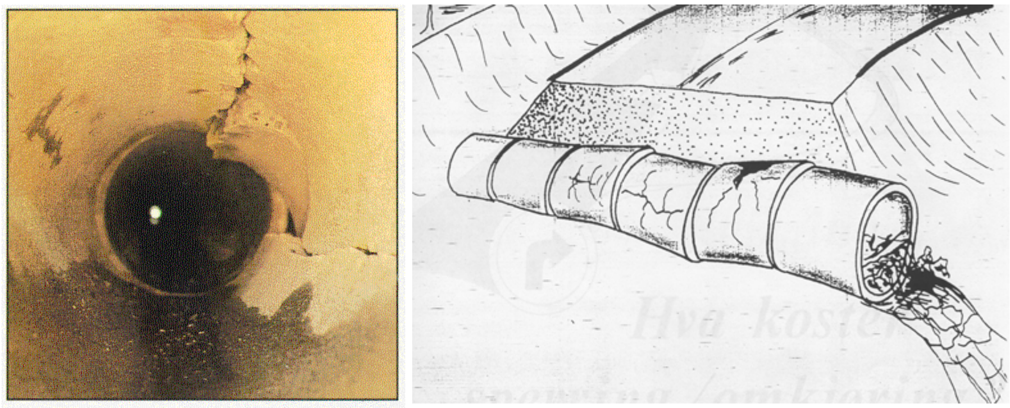


Figure 43. Cracked culverts and the possible consequences.

Improvements:

- Exchange the culvert and make a sufficient bed. Frost free foundation.
- Reline the culvert using a PEH pipe inside the old culvert and then inject concrete between the two pipes.

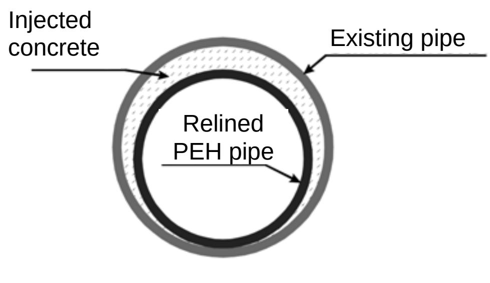


Figure 43. Relining to improve defect culverts.

Jäässä ollut rumpu ei vetänyt tippakaan

Sulavesi söi tiehen ammottavan montun

Ice clogging the culvert

Transverse culverts are also exposed to frost problems. If ice clogs the culvert the water will flow across the road. This is mainly a problem during early spring and in the winter during mild weather periods with large rainfalls.

The road structure is normally frozen in this period. Erosion can become a problem, but not as large as later when the ground and road structure has thawed.

Improvements:

- Reconstruct the culvert
- Use of steam to open the ice clog
- Solar panel and heater cable

Figure 44. Steaming a clogged culvert.

Figure 45. Examples of ice clogging transverse culvert

5.2.2.2 Ditches

There are different kinds of ditches, but on low volume traffic roads the most common are open ditches that have the function to drain both surface water and water from the road structure. In Sweden the depth of the ditch must be 30 cm below the road structure and in Norway 35 cm on a newly constructed road.

The main problem is the lack of ditch clearing. The guidelines tell that when the ditches are filled to a certain level the ditch must be cleared, but due to insufficient allocations this operation is given a lower priority than maintenance of the road surface.

Mud and vegetation will, after some years, fill up the bottom of the ditch and the drainage capacity will be reduced. The consequence is that the ground water table rises in the road structure and the bearing capacity is reduced. The effect is discussed in chapter 3.

Figure 46. Clogging of ditches in a silty area. The shot was taken during first spring after ditch cleaning.

In areas where the subsoil is fine graded, the need for ditch clearing is greater than it would be normally because these soils are easily eroded. The stability of ditch slope can also be a difficult problem especially with silty subgrade soils as can be seen in figure 48. Fine graded soils also have low bearing capacity at high moisture content and it is therefore necessary to have a well working drainage system.

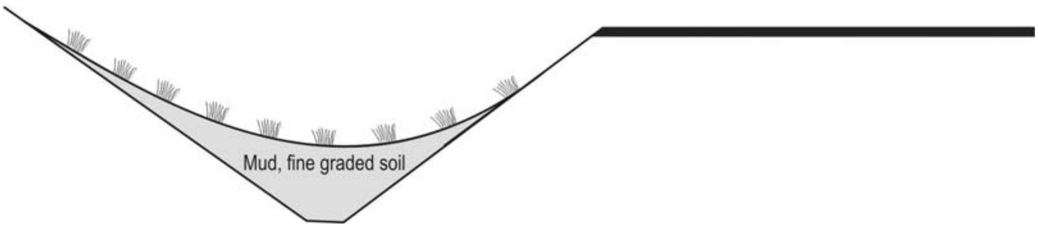


Figure 47. Clogging of ditches.

Improvements:

- Clearing the ditches often enough
- Exchange the material in the outer slope of the ditch using coarse aggregate and geotextile as illustrated in figure 48.

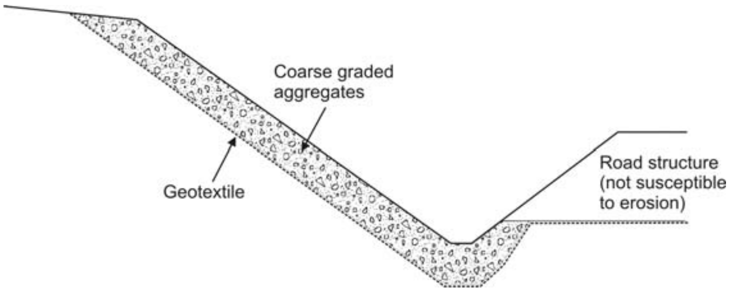


Figure 48. Protection of outer slope of a ditch.

- One solution to this ditch filling problem is shown in figure 49. After clearing the ditch, a coarse and open graded material (e.g. 30-80 mm) is added to the bottom of the ditch to a thickness of about 0.5 m. The coarse aggregate has to be wrapped in geotextile to avoid having fine particles clog the coarse material and thereby lower the drainage capacity.

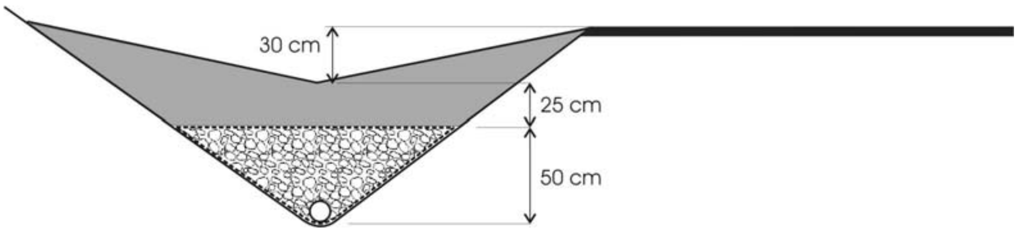


Figure 49. Cord of coarse graded material in a geotextile.

It's also possible to use a drainpipe in bottom of the drain cord to increase the flow capacity.

5.2.2.3 Grass Verges

High grass verges are a problem mainly in Scotland on the older narrow road network. The landowner's use of the area around the road limits drainage possibilities and prevents water from flowing away from the roadway. The problem can be seen in figure 50.

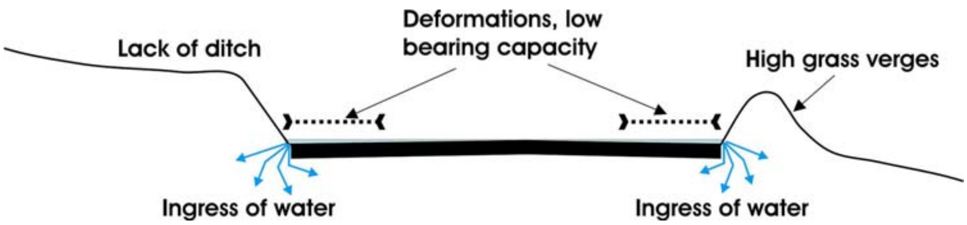


Figure 50. Water ingress on roads without ditches.

The surface water is then forced to drain through the pavement structure instead of draining to normal ditches. The results are reduced bearing capacity and deformations.

Figure 51 and 52. Drainage problems on roads without ditches.

The same kind of problem can be found in other NP countries when the turf on the road shoulder edge grows higher than the asphalt surface as illustrated in figure 53. The turf will prevent the surface water from flowing into the ditch and must therefore be removed. In Norway this turf must be removed if it is more than 3 cm above the surface. This problem is, first of all, related to a lack of maintenance.

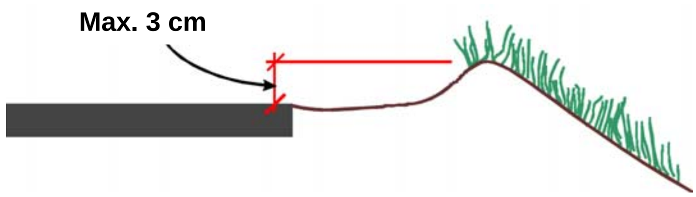


Figure 53. Turf on the edge of the roadway.

Improvements.

- Remove the verges and make ditches to drain surface water and pavement structure
- Deep drainage (subdrain)
- Remove the turf
- Edge drainage as shown in the figure 54 and 55

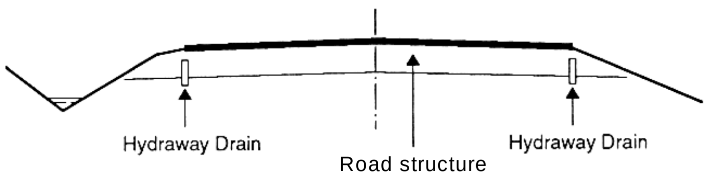


Figure 54. Hydraway drain.

It is important to get the surface water flowing into the deep drainage or the edge drain.

Figure 55. Installation of Hydra Drain

5.2.2.4 Poor cross fall

The purpose of cross fall on roads is to:

- drain the surface water
- create friction and driving comfort

How fast water drains from the road surface is, among other factors, dependant on the effectiveness of the cross fall. This depends on the transverse cross fall and the slope of the road as shown in figure 56. In Norway in order for cross fall to be considered effective it must be at least 1 %.

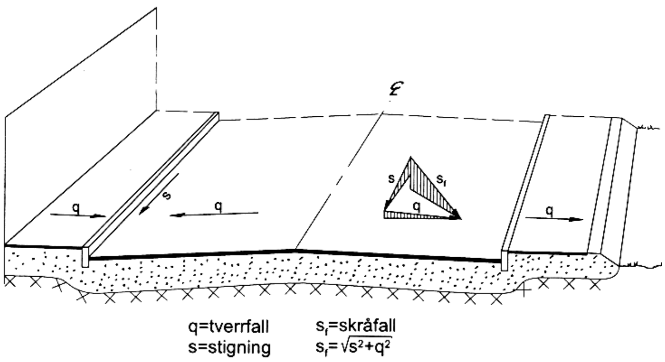


Figure 56. Cross fall and slope.

Water on the road surface is mainly a traffic safety problem. A wet surface reduces friction which leads to longer braking distance. Surface water will also freeze during the night at those times of the year with frost nights and temperatures above freezing during the day. The roads become very slippery if this happens and the change in friction may come as a surprise to those who are driving.

Ruts and an uneven road surface prevent the surface water from draining. Surface water will infiltrate the road structure, the amount of which depends on cracks, potholes and the permeability of the pavement.

When rehabilitating or resurfacing it is important to maintain the proper cross fall on the road. In areas where the road is horizontal there will be problems achieving sufficient cross fall in reverse clothoid. The solution is shown in figure 57 and describes a “movable ridge”.

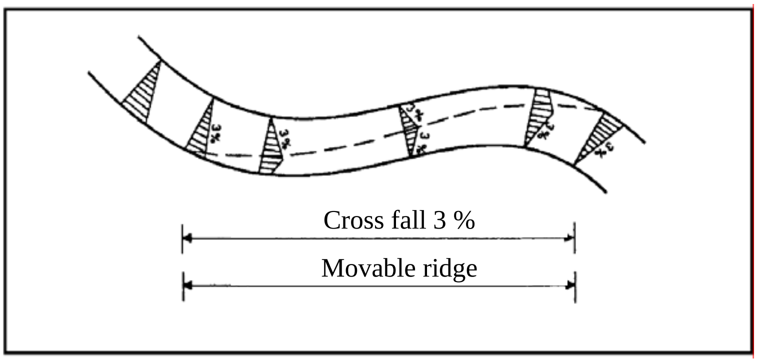


Figure 57. “Movable ridge”.

5.2.2.5 Cracks and potholes

The low traffic volumes roads in Sweden, Finland and Norway have a very thin asphalt surface layer and crushed gravel is often used in the base course. The stiffness of the gravel course is low compared to other base materials and this can cause large horizontal stresses in the asphalt surface layer. If the bitumen is stiff this will cause alligator and longitudinal cracking. Cracking caused by frost heave, weak edges and heavy wheel loads is also common.

The surface is often uneven on these roads and water is concentrated in ruts and other recesses. If the surface is cracked, the water will penetrate into the gravel course and reduce the bearing capacity for the material. This will accelerate the deterioration of the road.

HDM-4 from the World Bank has a model for deterioration, as a function of moisture content, which demonstrates that cracking and potholes contribute to a large extent. The model is presented in chapter 6.

To handle this problem, flexible materials must be used in the surface layer and, as such, soft bitumen is recommended. It is also important to seal the surface so that the water will only flow into the ditches and not into the road structure.

Figure 58. Cracks and potholes.

During the spring thaw period microcracking may also have a positive effect. If the materials in the road structure have excess water, it can drain or evaporate from these materials during the spring thaw (see Saaranketo and Aho 2005). Having the possibility to drain excess water through the surface layer instead of the ditches will lead to faster drying of the materials.

This effect could also be used when resurfacing a road where the conditions are as described above. Instead of using dense graded asphalt that is almost impermeable, open graded asphalt with a void content 18-24 % can use be used. The layer thickness has to be at least 6 cm.

This porous asphalt will drain the excess water, but it will also lead water to the gravel in the base course.

For these types of roads the base course gravel is stabilized using foamed bitumen or emulsified bitumen. In Norway foamed bitumen is used when the content of fines (>0.075 mm) is more than 7-8 %. This will make the base material non-water susceptible and it will have great resistance against permanent deformation during the spring thaw period.

Figure 59 demonstrates how a stabilized base course made of gravel can affect the rutting of a road.

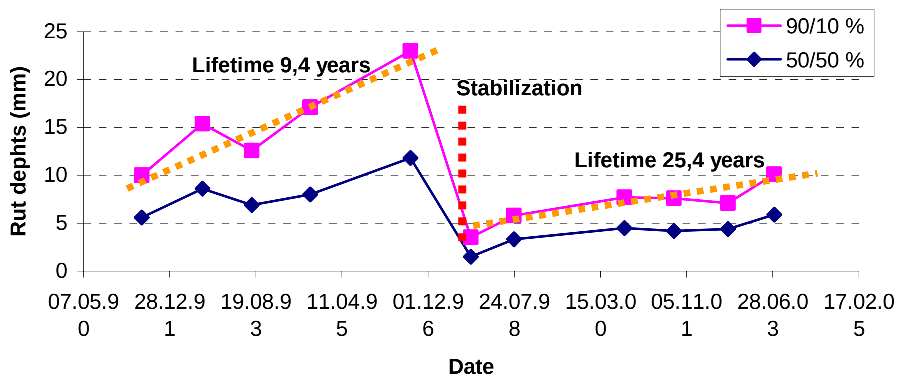


Figure 59. Example of the effect of stabilizing a gravel base course on the lifetime of a road. Where the predicted lifetime for the road prior to stabilization was 9.4 years and then following stabilization it increased to approximately 25 years.

Other products and techniques for treating unbound materials are presented in the ROADEX-II project report written by Kolisoja and Vuorimies (2005).

5.3 DESIGN RELATED PROBLEMS

5.3.1 General

Even if the drainage system is constructed according to the guidelines, the system can still be insufficient. The guidelines cannot address all the possibilities, but if it is evident that the problems are caused by moisture in the road structure there is a need to improve the existing drainage system.

The following sections describe a few examples of where a road was built according to design standards, but where drainage improvements were still required.

These examples and analysis are based on comparing two lanes of the road with different drainage conditions. Norway records the rut depth of all paved roads, every year in both directions, and has good statistics on these kinds of problems.

5.3.2 Sloping ground

In the greater part of the NP-area, the roads are constructed on sloping ground where one half of the road is situated in a cutting and the other half of the road is situated on an embankment as shown in figure 60.

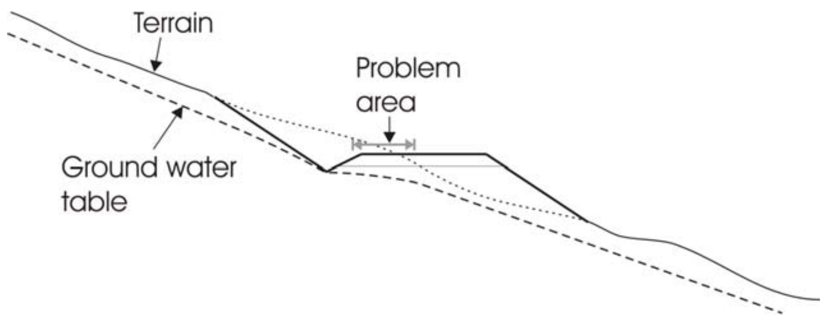


Figure 60. Drainage problem on sloping ground.

In these cases, the ground water table will normally be nearer to the road surface (and, as such, to the wheel load) on the road cut side. The moisture content is a function of the distance from ground water table. When the ground water table rises the moisture content will increase according to the matric suction curve for the materials in the road structure.

The problem is demonstrated in the picture below.

Figure 61. Deformation on the road cut side of the road.

This problem is normally found areas with moraine and sand / silty materials. Where the subgrade soil is clay or peat, the terrain is normally flat.

Example 1. Rv-858, Troms County, Norway

The first example where this problem can be observed, is national road RV-858 in Troms County, Norway

The rut depth is shown in figure 62. This is a typical example of a road on sloping ground. Rut depth is plotted for every 20 meter, but these lines are difficult to read so the average of one measurement in centre and 5 measurements before and 5 after this measurement are plotted with thicker lines (moving average of 11 points).

The blue line represents the road cut lane and the red line the embankment lane. The average rut depth for the road cut side is 12.9 mm and for the embankment side 7.9 mm. The road was repaved in 1991-93 and is a road with relatively good bearing capacity.

This effect can also be seen from the rut area that is shown in figure 5.30.

After repaving there is normally 3-4 mm of rutting caused by the stresses from the construction traffic. The road cut side has developed about 1 mm rut each year while the rut depth on the other side has only increased approximately 0.4-0.5 mm each year. The consequence is that the road cut side triggers the need for rehabilitation many years earlier than the well-drained embankment side. The lifetime ratio (drained lane/undrained lane) is more then 2.

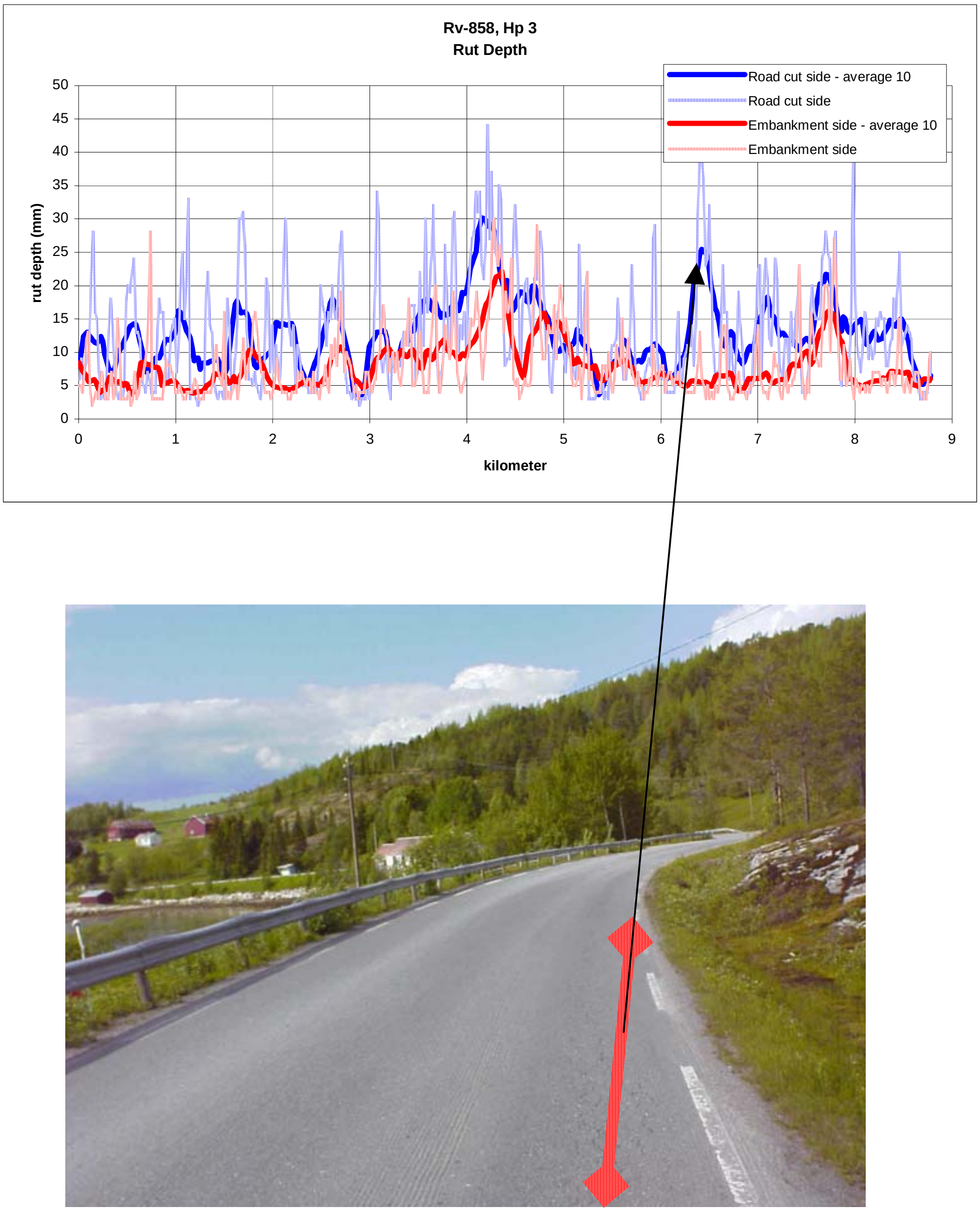


Figure 62. Rut depths on both sides of the road. Example from Rv-858, Norway.

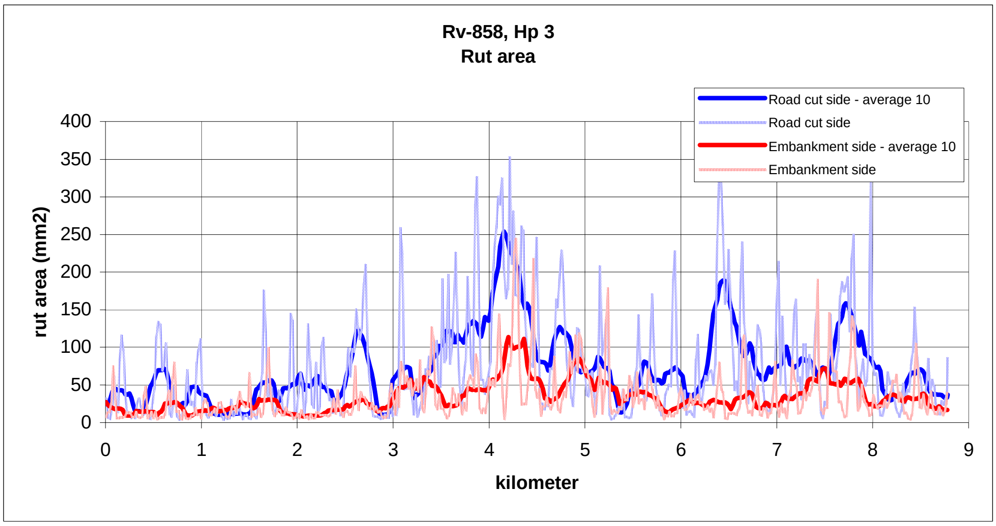


Figure 63. Rut area for both sides of the road. Example from Rv-858, Norway.

Example 2. Rv 861, Troms County, Norway.

The next example is from the road Rv-861 in Senja in Norway. The photo and the diagram in figure 64 illustrate the problem. This road was repaved in 1991 and the average rut depth in the right lane of the road was 21.6 mm while in the other lane it was 12.7 mm. In this section, 10 % of the right lane had more than 33.9 mm ruts and the corresponding value in the left lane was only 23.4 mm.

Figure 64. Example of road Rv-861 in Senja Norway about deformation on upper side of the road.

In Norway, the functional design lifetime, with respect to rutting, is calculated as the time from when the surface was paved until 10 % of the road section has ruts greater than 25 mm. For roughness there is a similar way of calculating the functional lifetime, but the limit value depends on AADT, type of road etc. In this case, a maximum of 10 % of the road cannot have an IRI value higher than 5.60 mm/m. This section had an average value in the road cut lane at 3.34 mm/m and 10 % is worse than 5.50 mm/m. For the other side the corresponding values were 2.15 and 3.60 mm/m. The ruts were the factor triggering a requirement to repave this section.

The upper lane develops, on average, 3.1 mm of rutting every year and 2.0 mm on the other side if assuming that 3 mm of rutting occurred just after the last repaving. It has also been assumed that the increase in rut depth is linear with time. The well-drained lane will then have a functional lifetime of 11 years and the other side only 7.1 years and this produces a ratio of 1.55.

Example 3. HW 21 (E8), Kilpisjärvi, Finland

An example from Finland with the same problem is presented from HW 21 near to Kilpisjärvi on (E8) not too far from the Norwegian border. The map in figure 65 shows that the terrain is sloping ground.

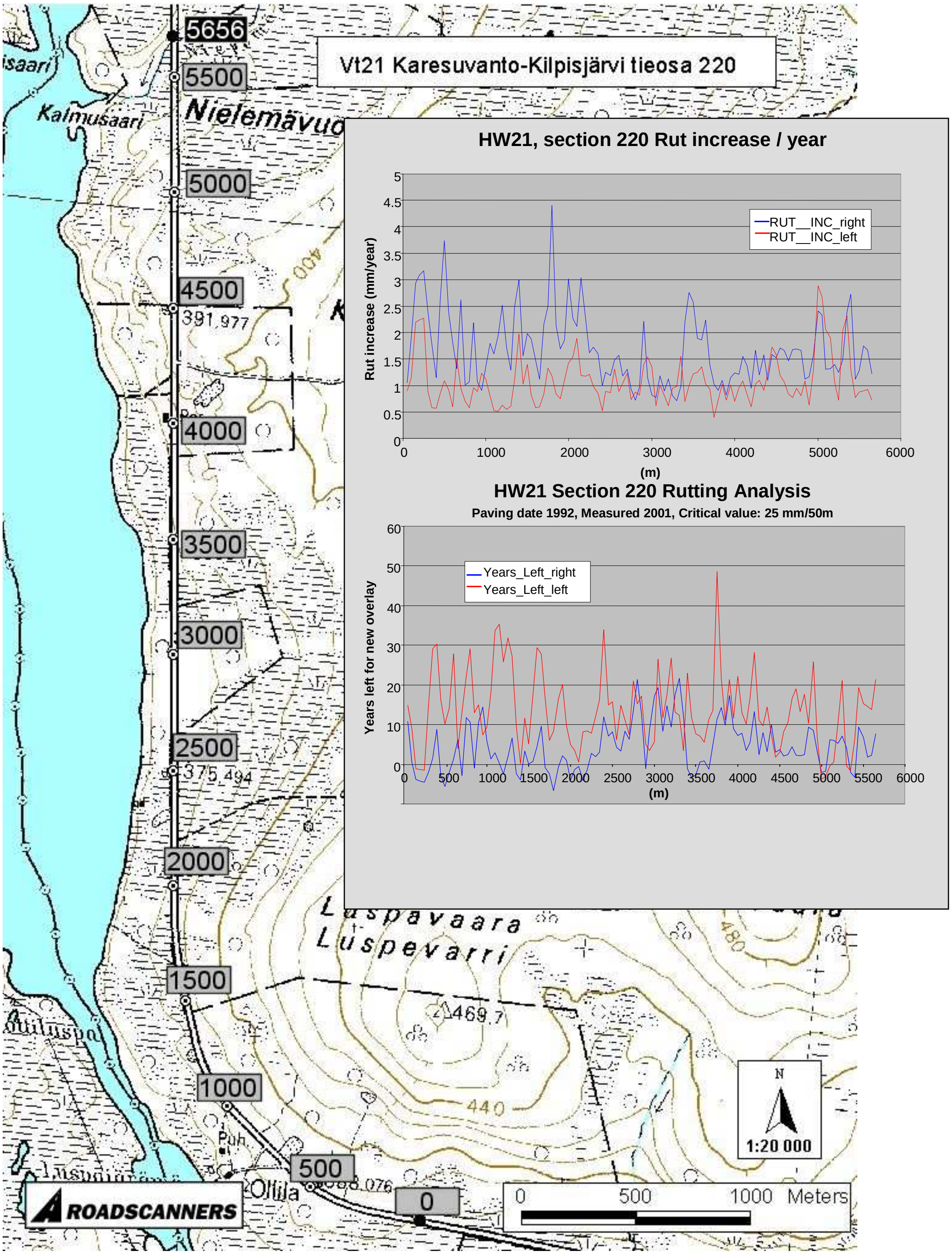


Figure 65. Road section on HW 21 – Kilpisjärvi, Finland, presented with average rut increase/ year and remaining life time.

The graphs in figure 65 show the average rut depth progression / year for both lanes and in section 0-2500, where the ground slope is the steepest; there is clearly a difference in rut development. The road was paved in 1992 and the rut depth was recorded in 2001. The remaining lifetime for the lanes is also illustrated in figure 65.

If only section 0-2500 is examined the annual increase in rut depth is 2.0 mm in the right lane and only 1 mm in the left lane. This indicates that the lifetime is twice that in the drained lane if rut depth development is linear.

The rut depth measurement was done when the pavement was 9.5 years old and the calculated remaining lifetime for the right lane was 1.8 years and for the left lane 15.0 years. In other words the lifetime for the left lane was 24.5 years and 11.3 for the right lane. This is a ratio of 2.17.
Increased deterioration through road cuttings

One of the key findings of the COURAGE final report (see chapter 4.3.1) was that in pavement structures through cuttings the moisture content was found to be slightly higher than that of a pavement structure situated on an embankment. It is the same that we have on sloping ground, but here we have one lane situated in a cutting and one lane on an embankment.

Figure 5.34 shows the results of an analysis of 155 km of low volume national roads in Norway (Rv-87, Rv-947 and Rv-865). The roads cross section is recorded for every 500 m and the average rut depth for a section of 100 m on each side of where the recorded cross section is used as rut depth in the diagram. The average rut depth increase (mm/year) is determined for sections where there are ditches on both sides, where there is only ditch on one side and where the road is located on an embankment (no ditches) (see figure 66).

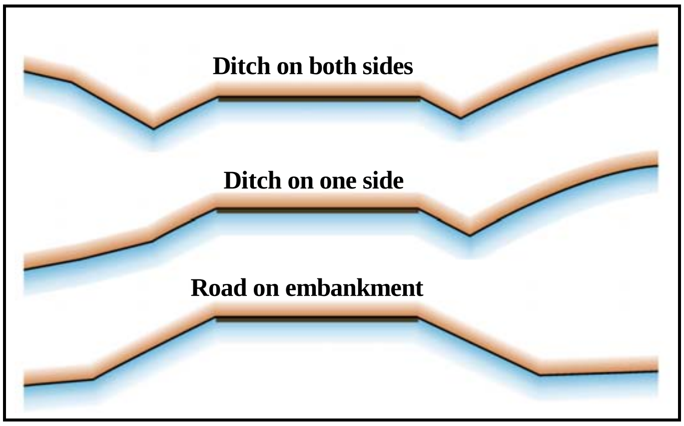


Figure 66. Typical cross sections used in the analysis presented in figure 67.

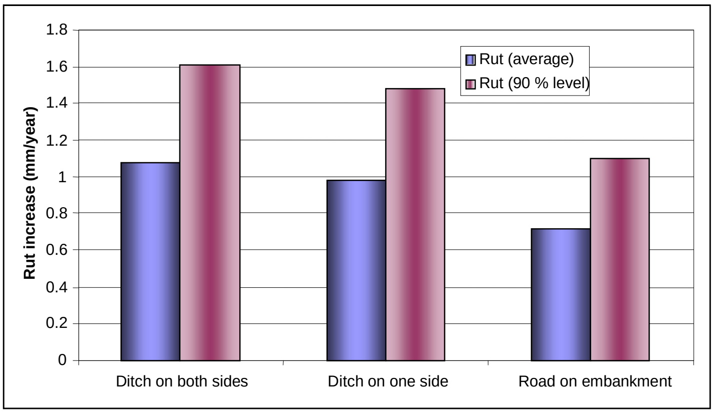


Figure 67. Rut depth progress depending on cross section.

In Norway, the 90 % statistical level of the of the rut depth (among other factors) are used as a damage trigger factor for deciding when to repave a road section.

If the initial rut depth right after repaving was 3 mm and maximum allowed rut depth is 25 mm, a road situated on embankment will have a lifetime equal to 20 years. A road located in a cutting will have a lifetime equal to 13.7 years.

The drainage is, of course, not the only reason for this difference; just one of the factors. The soils beneath the foundation level are, very often, not the same in road cuts and embankments, and particularly not if the subgrade soil is difficult to handle with the construction machines. Such materials are fine graded soils like silt and clay.

Bedrock blocking the ground water flow

On sloping ground the ground water will flow under the road. If there is bedrock or impermeable materials near the road area, these objects may block or concentrate the ground water to places where the potential for developing frost heaves, spring thaw softening and reduced bearing capacity due to high moisture content is high.

This effect can be seen in figure 68 where the spring thaw problems were mapped for several years and where the materials in the ground were surveyed using ground penetrating radar. The survey showed that these spots had more spring thaw problems than elsewhere.

Figure 69 shows back-calculated values for the E-modulus from a site where the bedrock blocks the ground water flow. It can be seen clearly that the modulus values of the pavement are significantly lower than for the rest of the road section. The reason is probably that micro cracks have already developed in the asphalt due to a weaker road structure. The materials in the road structure for the problematic spots are the same as for the rest of the road and the changes in bearing capacity have to be caused by different moisture content.

From figure 70 it can be observed that the rut depth is greater where the ground water flow is blocked by bedrock. The same goes for the IRI-value.

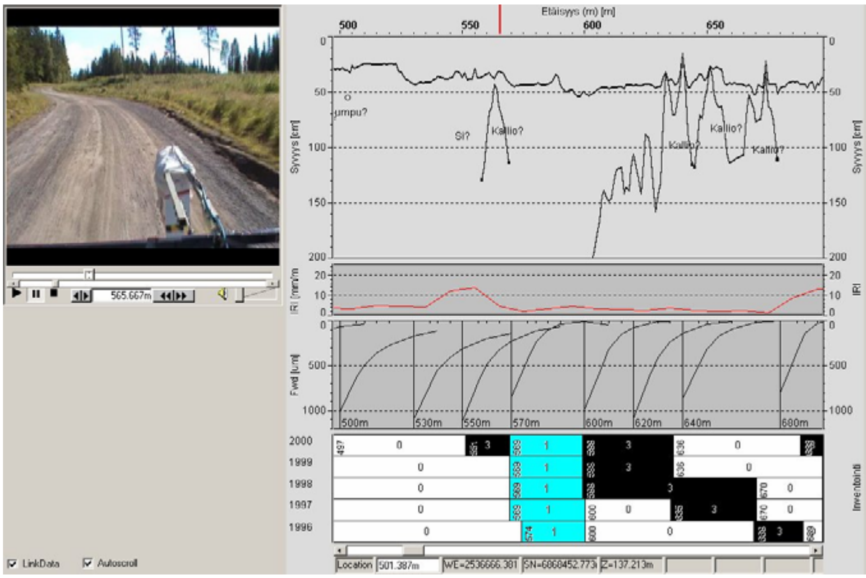


Figure 68. Severe spring thaw problems on sloping ground with bedrock blocking ground water flow: the most severe, class 1 (blue bars), problems are located between the bedrock peaks Road 3424, Kuorevesi, Finland.

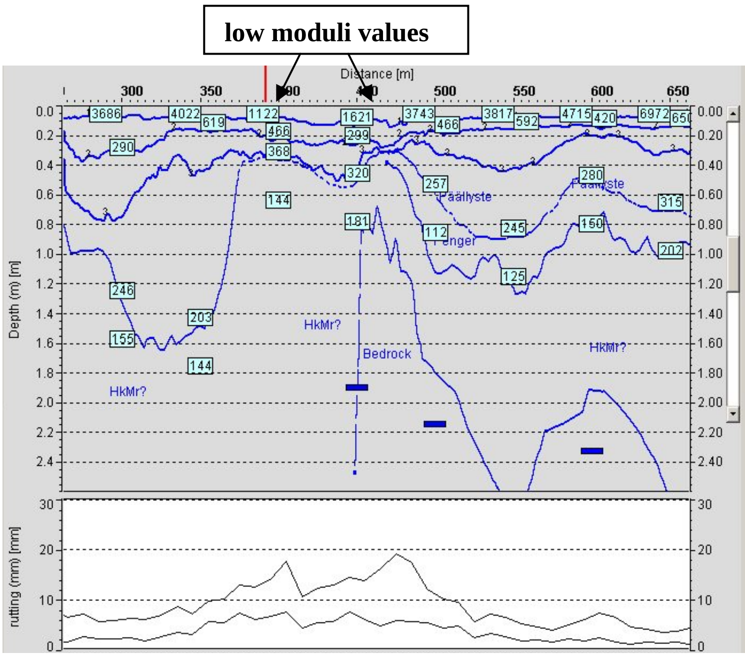


Figure 69. Low asphalt moduli values caused by bedrock blocking ground water flow, HW 21 Finland.

Improvements:

- Increase the road structure thickness on the road cut side.
- Subdrain to lower the ground water table on the road cut side (figure 5.37)

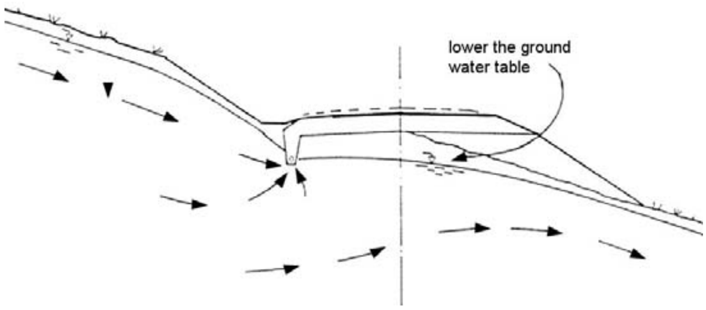


Figure 70. Drain system in sloping ground.

- Edge drain
- Remove bedrock/impermeable materials that block the ground water flow
- Frost insulation (expensive)
- The use of additional culverts in places where the water is being blocked

5.3.3 Drainage problems on "low ground"

In areas of low ground where the natural drainage system for surface water is not present then the water has to infiltrate the subgrade soil. When the ground is frozen or after period of heavy rainfall or snow melt, the water will have accumulated on the surface and will cause problems for the road as shown in the picture in figure 72.

In addition to causing trouble for traffic, the raised ground water table will increase the deterioration of the road. On gravel roads this can soften the road structure and the road surface to the extent that the road becomes impassable.

The example in figure 73 shows survey results done on a gravel road during spring thaw in a section with this low lying problem.

Figure 72. Drainage problem on low laying ground, Rd 19778, Kemijärvi, Finland.

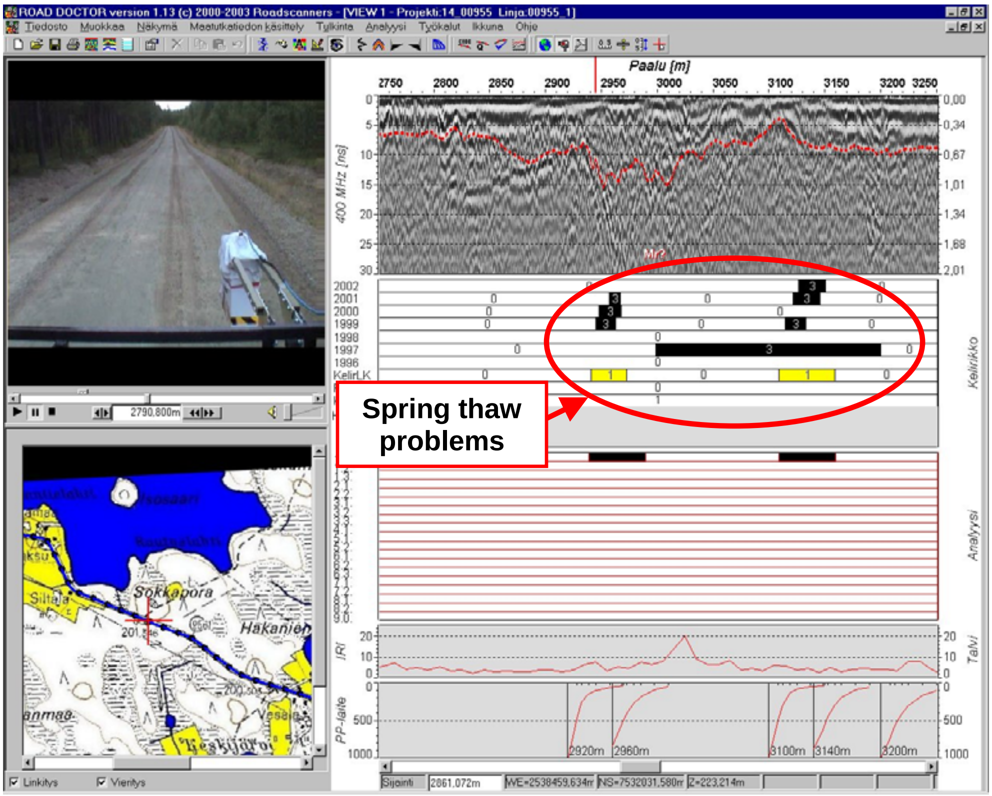


Figure 73. Spring thaw problems in a moist and low lying moraine "valley".

Improvements

Moraine

If the materials in the subgrade are moraine it is possible to make infiltration wells or infiltration ditches. The dimensions depend on the permeability of the moraine and how much water accumulates in the problem area.

The problem can also be solved by raising the grade line of carriageway.

Clay/silt or peat

When the subgrade materials are clay, silt or peat it is not possible for water to infiltrate the ground. Peat is already saturated and clay and silt has low permeability and will not accumulate any water.

The best way to solve the problem is to raise the grade line of the carriageway. How much, depends on the water table how severe is the problem. The difference in elevation (height) between the road surface and the water table depends on which materials are used in the construction. If dense graded gravel is used the difference should be at least 50-60 cm. If coarse, well-drained materials are used, the difference should be at least 30-40 cm.

When raising the grade line in low lying valleys with weak subgrade soils, such as peat and gytja, the stability and differential settlement risks should be always evaluated.

5.3.4 Drainage problems on flat area

When a road crosses flat area there will be problems similar to when it is located on low-lying ground. When there is a long distance to the natural drainage system, it is difficult, as such, to get rid of the water. This problem is most apparent during the spring thaw when the ground is still frozen and there is a lot of water from melted snow and rainwater. The water has not infiltrated the subsoil and creates large local dams that raise the ground water table and may cause problems for the traffic flow.

During periods of heavy rainfall the subsoil may have problems draining the surface water. The extent of the problem depends on the amount of water and the permeability of the subsoil. In any case, the ground water table will rise the consequences of which are described in the previous section.

Improvements

Without regard to the kind of subsoil, raising the carriageway grade line is one way to handle this problem. It is sensible to use coarse graded materials that are not susceptible to water.

Drainage (outlet) ditches are also a solution, but due to the terrain these will be long, deep and expensive, and this is the main reason why they have not already been built.

Moraine

If the subsoil is moraine, it is possible to use infiltration ditches or infiltration wells. These will help infiltration in periods when the ground is frozen or when the surface cannot drain the water down to the ground water table

Clay/silt and peat

When the subsoil is peat or soil with low permeability, it is not possible to use infiltration. If making drainage ditches is not possible, the only way to handle this problem is to raise the carriageway grade line. The subsoil materials are susceptible to settlements and increased load may cause uneven road surface.

5.3.5 Drainage problems where the road is constructed in bedrock cuttings

In cuttings through bedrock there can be frost heave problems and also bearing capacity problems if the cutting does not permit water to drain.

When water is not drained from the road structure it can lead to a reduced bearing capacity. During the frost season, ice lenses form on top of the bedrock which causes uneven bumps in the road surface. Depressions in the bedrock surface collect water and if there are frost susceptible materials in the road structure segregation ice will form.

Bedrock may also block the water under the road and to road structures from road side and bedrock/boulder dams the water flow on a long distance.

Improvements:

- Blast the bedrock to a depth of 1-2 m below the top of the formation. This will create cracks in the bedrock and the water will be able to drain from the road structure.
- Ditches/deep drainage that prevent the water from entering the road structure
- Remove basins in the bedrock that are collecting water.
- Frost insulation.

5.4 OTHER PROBLEMS

5.4.1 Moisture trap

In several roads that were reinforced during the seventies and the eighties, the normal method was to put gravel layers directly on top of the old bound surface, to form a sandwich construction (figure 74).

Water that penetrates the asphalt surface, unpaved road shoulder and from the ditches (snow melt periods) will be trapped between these two bound layers. The moisture content will increase more than it would for a normal road structure and will be moist for a longer time due to the lack of drainage possibilities. If the moisture content is close to saturation level a dynamic load causes high hydraulic pressure and this breaks the pavement above.

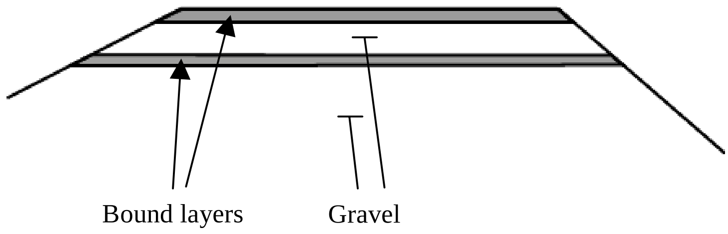


Figure 74. Saturated layers due to bound layers in the construction.

Improvements:

- Mill through the structure to crush the lower bound layers. Experiences in Finland and Sweden have shown that old pavement should be always be broken if its' distance from the pavement surface is less than 40 cm.
- Add foamed or emulsified bitumen when milling to stabilise the materials

5.4.2 Stability problems in the outer slope

This is a very common problem in road cuts. The types of damage related to this problem are:

- erosion caused by surface water
- erosion caused by ground water
- surface slides

The problems are illustrated in figure 75 and 76.

Materials from the damaged slopes flow into the ditches and block the water flow and cause the ground water level to rise. The problem is worst where there is fine graded sand and silt, and the ground water flow is high.

This problem can be solved partly by improving the drainage outside the road structure. If the situation is not too complicated, it is only necessary to establish vegetation on the outer slope or dig a back drain ditch above the upper shoulder of the outer slope to take control of the surface water and lower the ground water table. Water from these ditches must be directed to the natural drainage system.

Figure 75.. Typical road damage due to stability problem of outer slope.

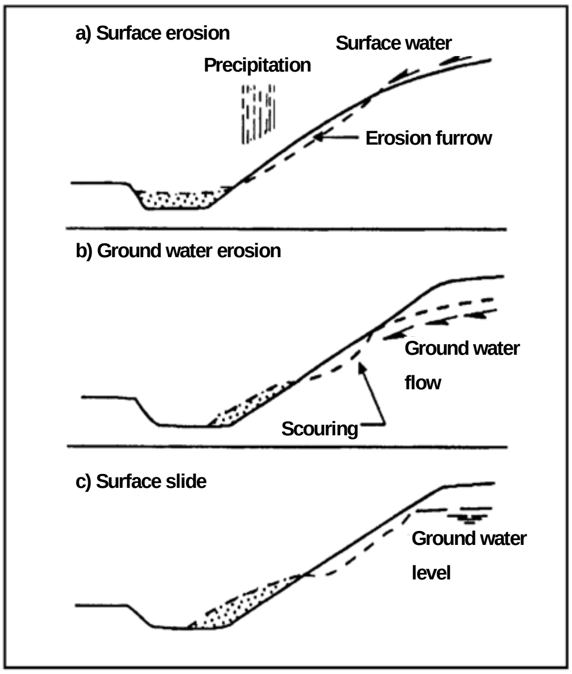


Figure 76. Stability problems in the outer slope.

If the conditions are more complicated a temporary protection of the slope surface may be needed until the vegetation is established. There are several kinds of fabrics that can be used. There may also be a need to install drains in the slope as shown in figure 77.

Figure 77. Example of making road cut slopes more stable.

If the conditions are very complicated all of the measures mentioned above will be needed. In addition, it will be necessary to cover the slope surface with coarse graded gravel or macadam. A geotextile must be used between the subsoil and the coarse material on top to separate the materials.

A new technique has just been developed in Norway for making surface drains in the road cuts. The material that is used is recycled glass that has been through a process where the glass is transformed to foam. This foamed glass is delivered as a granular material with maximum grain size 64 mm. The density is very low (300 kg/m3) and in road constructions the material is used as low weight fills and frost insulation.

When using the foamed glass in drainage ditches the glass is wrapped in geotextile and formed into a sausage shape. This sausage is easy to handle and can be installed by only one or two men. This is a great advantage because the soils where there is a need for this kind of solution, the materials are fine graded, water susceptible and unstable, and they are not easy for a large construction machine to handle.

6 Observations of road deterioration and prediction models

6.1 GENERAL

The literature review in chapter 3 and 4 demonstrates that the interdependencies of moisture content in unbound material and its material properties are very complex. The number of parameters is enormous and calculating the deterioration of a road section as a function of drainage quality has to be simplified.

There are some models that calculate or at least take into account the effects of drainage. A few of them are described in the following sections. Examples are used to demonstrate the effect of drainage improvements.

Several design guidelines also take into account the moisture condition in the road structure and the subsoil. Some of these models are presented.

Statistical evaluations of observed deteriorations of roads are also presented in the following sections.

6.2 EFFECTS OF IMPROVING ROAD STRUCTURE DRAINAGE

Almost every researcher mentions in their studies that drainage is the most important thing when considering long-term serviceability of the road structure. However, there have been only a few studies that have investigated to what extent good drainage really affects the lifetime of a road structure.

Long et al. (1996) have studied the differences between drained and undrained test sections in California, U.S.A. Only the base courses of the drained and undrained tests sections were different. The base course of the drained test section consisted of a 76 mm thick bitumen treated permeable material over a 183 mm thick aggregate base while the undrained test section consisted of a 274 mm thick aggregate base. The whole test road structure consisted of a 137 mm thick asphalt concrete surface over a 274 mm thick base course overlying a subbase, of which the thickness varied. Results from the study are presented in figures 6.1 and 6.2, which presents how many load repetitions remain before a failure in different cases using different fatigue prediction models. In calculations aggregate base and subbase were combined and modelled as one layer, because the calculation program, ELSYM5, could only model five layers (Long et al. 1996).

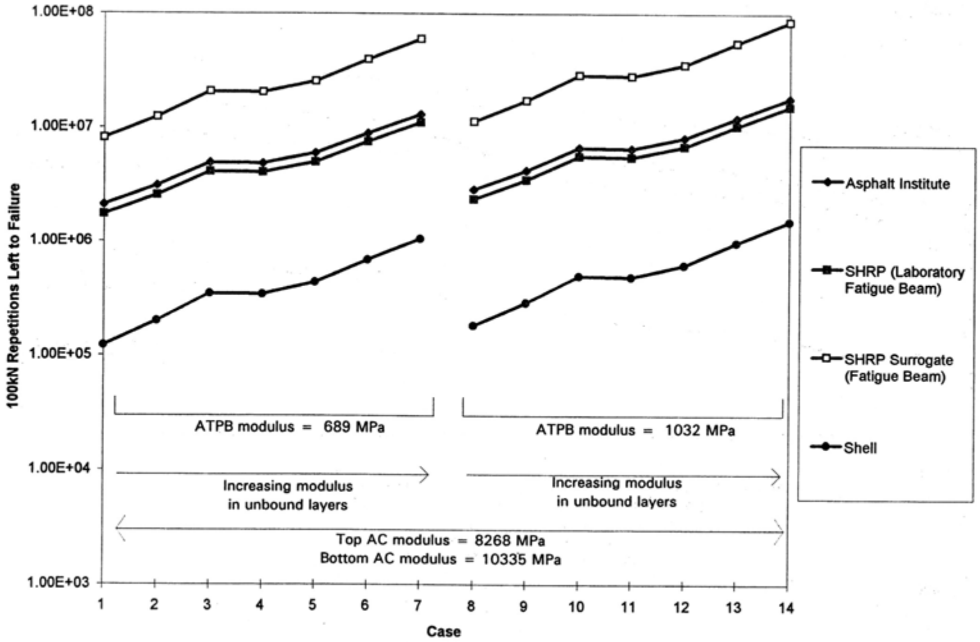


Figure 78. Remaining number of 100-kN load repetitions before fatigue failure for the drained section with subbase thickness 218 mm. (Long et al. 1996).

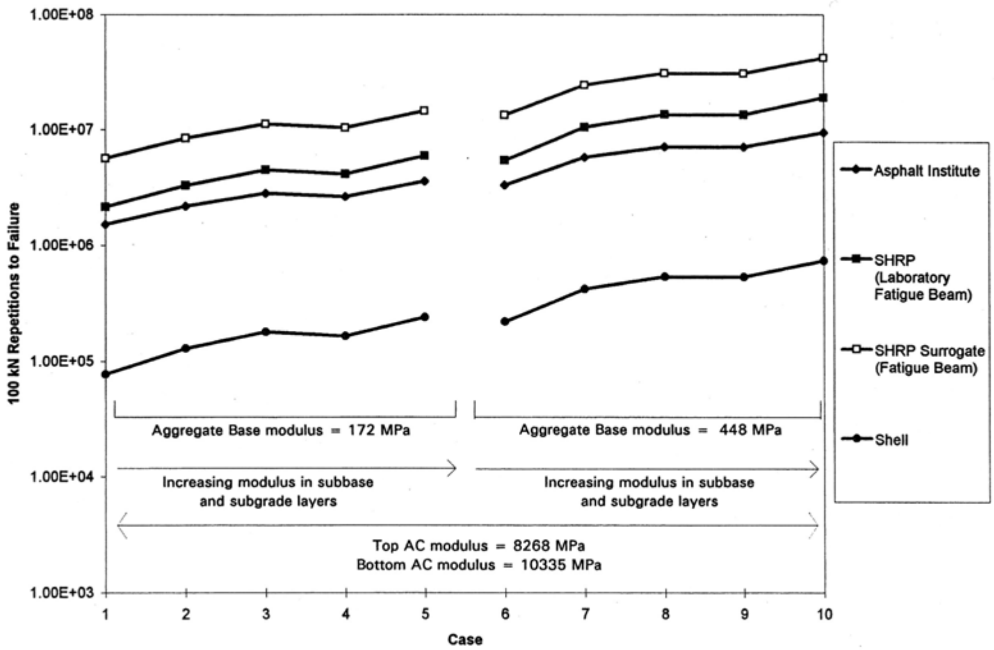


Figure 79. Remaining number of 100 –kN load repetitions before fatigue failure for the undrained section with subbase thickness 218 mm. (Long et al. 1996).

When comparing the results from study of Long et al. (1996), it is obvious that drainage improves the service life of a road structure. (Long et al. 1996) The difference in repetitions before failure between the undrained section with a base modulus of 448 MPa (figure 79, curves at right hand side) and the drained section with a base modulus of 689 MPa (figure 78, curves at left hand side) is about 25 %, in other words the drained section has a 25 % longer service life than the undrained section. However, the drained and undrained sections of this study are not totally comparable because the drainage arrangement is not normally as such. In this particular case the drainage will improve the stiffness of the structure, which is not the case when using normal lateral drain arrangements.

A recent study of Werkmeister et al. (2002) suggests that the influence of a small change in moisture content (1%) has a significant effect on the deformation properties of the unbound granular materials and the shakedown limits (see also Dawson and Kolisoja 2005). The increase of stresses from increasing the moisture content from 4 to 5% is quite small, but the shakedown limits decrease considerably. Carmichael and Stewart (1985) stated that a one percent increase in moisture content causes a decrease of 4.3 MPa in the resilient modulus for a granular material (Bayomy et al. 2002). Bayomy et al. (2002) suggest that the decrease in the resilient modulus of well-graded coarse material that contains a small amount of non-plastic fines is about 3.4 MPa when the moisture content increases by one percent. The increase in resilient modulus is about 3.8 MPa for sands and gravels containing substantial amounts of plastic fines.

The University of Oulu has done some research on the effect of lowering the ground water table for a gravel road and the results are illustrated in figure 80.

Drainage improvement reduced first year spring thaw problems by 53 %, but after the fourth year the percentage was only 24 %. Using a linear model, the effect would have been ignored in 8 year after improvement.

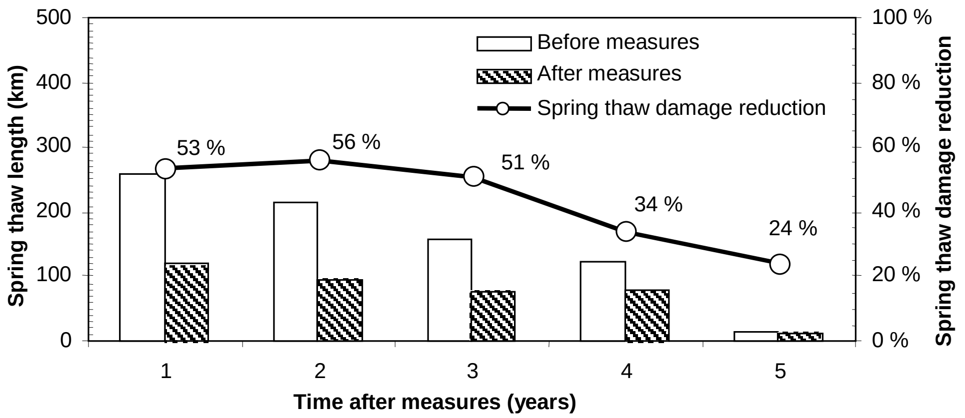


Figure 80. Effect of drainage on gravel roads.

The number of studies that contain any comparison between the performance of drained and undrained sections made from the same material, is quite limited. However, a good drainage system and adequate water permeability of the unbound granular layers is very important and unbound granular materials used in pavement construction should be not too sensitive to water, because high moisture content of the material may indicate a high risk for rutting (Werkmeister et al. 2002).

6.3 USE OF DATA FROM A ROAD ON SLOPING GROUND

6.3.1 Observations from 184 km of county roads in Troms county

Chapter 5.3 described the problem of differences in the rate of deterioration in the two lanes on sloping ground. Three examples show that the ratio of lifetime for the drained lane and the undrained lane is 1.57-2.17. These are just examples to describe the problem. In the following sections approximately 185 km of county roads have been analysed.

The difference in lifetime for the two traffic lanes is caused by the difference (mainly) in drainage condition. The ground water table is nearer to the traffic load on the road cut side and will cause deeper rutting. The difference in lifetime is a measure of the effect of lowering the ground water table.

The variations are large and a statistical approach is needed, but that has not been done in this report.

In Norway, rut depths are measured every year on both sides of all paved national and county roads. The roads are divided into sections where the condition is almost the same. These sections are used in the Norwegian PM system. For each section, the statistical distribution for the rutting is calculated and the rut value describing the level where 90 % of the road section has lesser ruts is decided. The rut value is used as a damage trigger factor and when this exceeds 25 mm the road surface should be improved.

Data for four county roads are shown in appendix 1. The diagrams present the relationship between rut depth on the road cut lane and the other lane, and the same for the roughness index. Road 1, Fv-53, is 35.2 km. For more than half (52 %) of the road the ratio between rut depth in the drained and undrained lane is more then 1.47.

Road 2, Fv-141, is 46.8 km. 10.5 km (22.5 %) like the previous one, which is more than 1.5.

Road 3, Fv-263, is about 38.4 km and both the IRI and rut depth is higher in the road cut lane and the ratio is lager then 1 for almost the entire section. As can be seen this ratio is higher for the rutting than for the roughness, and the highest ratio for the rutting is 3.6. For many of the sections the ratio is about 2.

Road 4, Fv-293, is 62.5 km. The ratio is lower than that for Fv-263, but the values for rutting and roughness are in the same order of magnitude. For several sections this ratio is about 1.5 and if we take into account the initial values immediately after the last repaving, the difference in lifetime is about 40 %.

The diagram in figure 81 shows the cumulative distribution for the ratio of rut depth in both lanes for 184 km of road. Only 12 % has a greater rut depth on the drained side of the road, 19.5 % has a rut depth ratio greater than 1.5 and for the rest (68.5 %) this ratio is between 1.0 and 1.5.

The ratio of the IRI-value for the two lanes is not at the same magnitude as for the rut depths, but it is clear that the progression of the IRI is worse for the undrained lane. Road 3 and 4 in appendix 1 illustrate this. For Road 4 the IRI value is, on average, 1.28 times larger in the undrained lane than in the other lane.

The drainage condition and the subgrade soil vary throughout this 184 km of road network. The maintenance of the drainage system for the county roads in Troms is normally not a prioritised work task (as can be seen in the pictures from the road sections in Appendix 1) and the worst parts have no ditches at all or very shallow ditches.

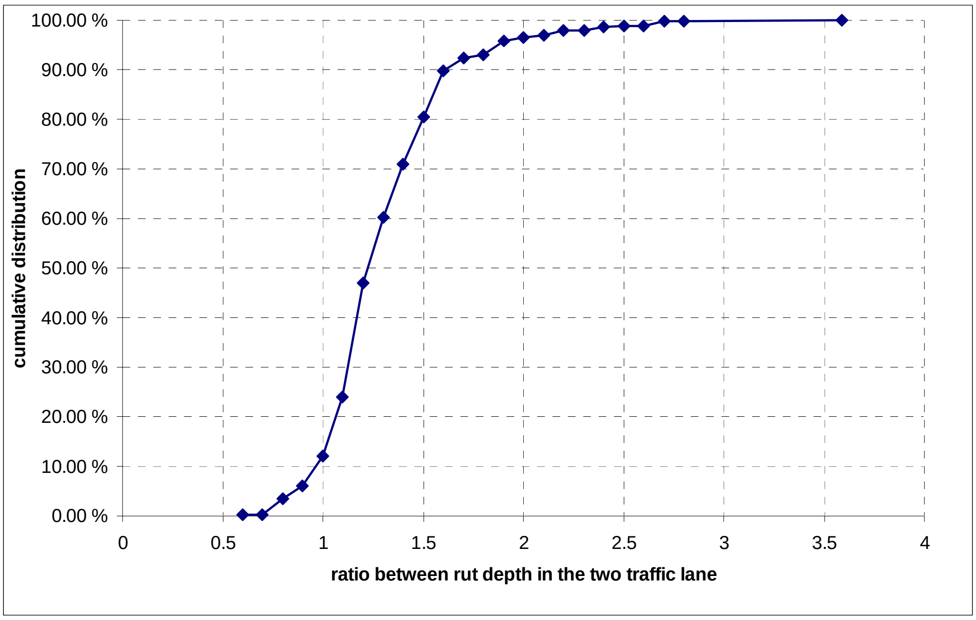


Figure 81. Cumulative distribution of the ratio for rut depth in the drained and the undrained lane.

In the COURAGE final report (see chapter 4.3.1) one of the key findings was that in pavement structures through cuttings the moisture content was found to be slightly higher when compared to that of a pavement structure situated on an embankment. It is the same when the ground is sloping, but in this case one lane is located in a cutting and the other lane on an embankment.

6.3.2 Lifetime compared to recorded rut depth

The rut depth development is, in practice, linear and has an initial value of 3-4 mm after repaving. The lifetime, according to rut depth, is then:

$$\frac{Maximum\ rut\ depth\ permitted - initial\ rut\ depth}{(recorded\ rut\ depth - initial\ rut\ depth) \big/ year\ since\ last\ repaving}$$

The change in lifetime, expressed as a ratio, is then:

$$\frac{(recorded\ rut\ depth - initial\ rut\ depth)_{drained}}{(recorded\ rut\ depth - initial\ rut\ depth)_{undrained}}$$

On new pavement and pavement with small ruts the initial rut depth will have a larger impact on the calculated rut depth ratio. The differences will be overshadowed by the initial value. In the figure below, the ratio for the lifetime and the ratio for rut depth in both traffic lanes are shown for a selected road in appendix 1. As can be seen the lifetime ratio is a bit higher than the rut depth ratio.

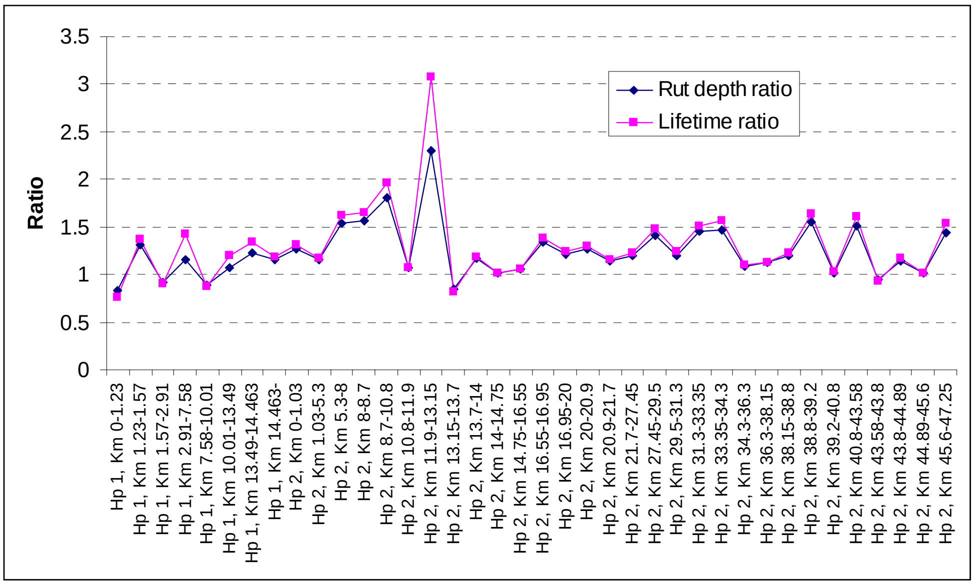


Figure 82. Difference in rut depth and lifetime ratio for both traffic lanes.

The alignment is also an important parameter affecting where deformations develop. This is illustrated in figure 83. It is the inner lane in curves where the road cuts are largest where the rutting problems are most significant.

It's not only the drainage condition that can cause differences in rutting between the drained and undrained lanes. The materials and compaction of the material can be different and it is also possible that the subgrade soil varies in the cross section. Last but not least, the traffic load will also have a different impact on rut formation in the inner and outer lanes of the curves in a road cut. The wheel loads are more concentrated on the inner lane and will have a greater impact on the deformations.

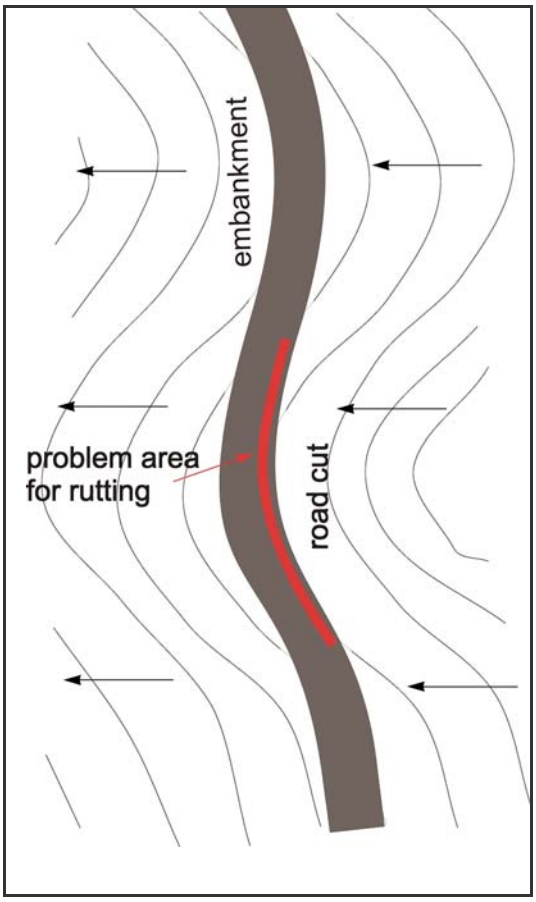


Figure 83. Location for rut deformation.

6.4 MODELS IN GUIDELINES

Models that describe the variations in the resilient modulus as a function of the moisture content can be used together with models for moisture content in the pavement structure to calculate stresses and strains. Then other deterioration models can be added and the calculations can be carried out for every change in moisture content throughout the road's lifetime. (Mechanical approach.)

6.4.1 Swedish design guide

In Sweden road administration has started to use an analytical design system and a computer program called PMS Objekt. This can be downloaded from the Swedish Road Administrations WEB-site.

The design guideline (ATB väg 2003) uses the resilient modulus as input parameters and the calculations are done for 6 seasons. The resilient modulus changes from one season to the next in almost all of the layers in the road structure. The system considers that the moisture content varies throughout the year.

For materials that meet the requirements in the design guide, the resilient modulus does not depend on how well the drainage is working. For other materials the modulus depends on the drainage classified into 3 categories.

These classes are:

Drainage class 1 – appropriate drained

- Sporadic occurrence of plants that attract moisture in the zone 0.5 m below the road boarder
- Stagnant water with a surface higher than 0.8 m under the road border no longer than 1 week continuously

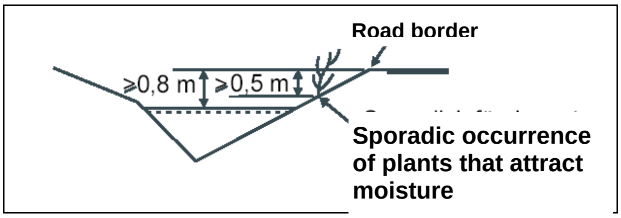


Figure 84. Drainage class 1.

Drainage class 2 – unsure drained

- Occurrence, but not generally, of plants that attract moisture in the zone 0.5 m below the road boarder
- Stagnant water with a surface higher than 0.8 m under the road border no longer than 1 month continuously

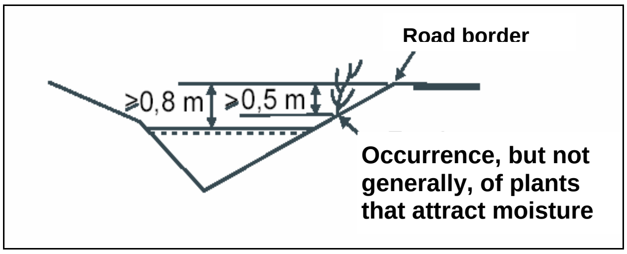


Figure 85. Drainage class 2.

Drainage class 3 – poor drained

- The conditions for drainage class 2 are not fulfilled.

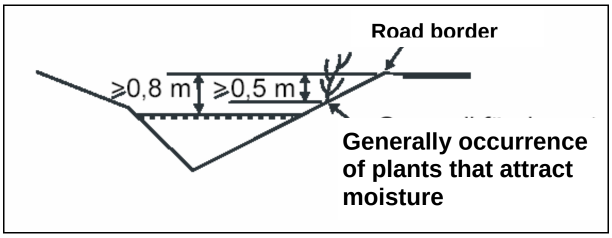


Figure 86. Drainage class 3.

The variations of the modulus for the subbase layer are shown in table 3.

Table 3. Resilient modulus for subbase.

	Resilient modulus MPa					
	Winter	Thawing periods in winter	Spring thaw	Late spring	Summer	Autumn
Drainage class 1	1000	1000	70	85	100	100
Drainage class 2	1000	1000	70	85	85	85
Drainage class 3	1000	1000	70	70	70	70

The materials are also separated into new and old subbase. For new uncrushed subbase materials the resilient modulus is reduced from 240 MPa to 160 MPa when the drainage class is changed from class 1 to class 3.

For older subbase materials the differences are more dependent on drainage conditions. The modulus decreases by 1/3, as can be seen in figure 87, when the drainage condition changes from class 1 to class 3. The subgrade is classified into groups according to their bearing capacity. The diagrams in figure 88-90 illustrate how the resilient modulus changes for the subgrade when the drainage conditions are in a different class.

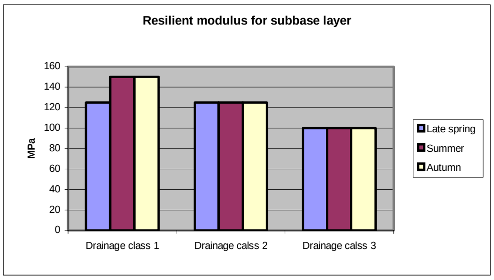


Figure 87. Res. modulus for subbase layer

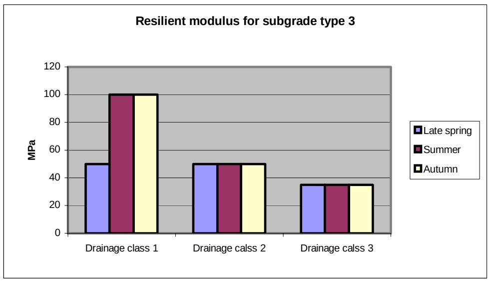


Figure 88. Res. mod. for subgrade type 3 (moraine)

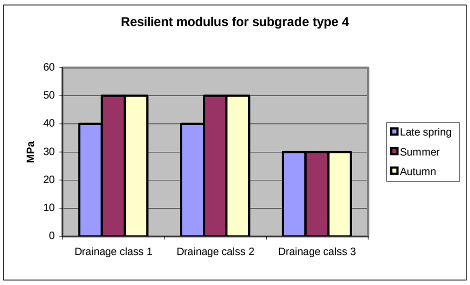


Figure 89. Res. mod. for subgrade type 4 (clay)

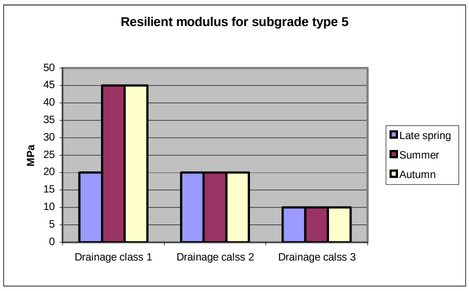


Figure 90. Res. mod. for subgrade type 5 (silt)

The E-modulus has been decreased in the latest version of the design guide compared to previous versions.

From these values it is possible to calculate the effect of improving the drainage class of a road section to a better class by using the calculated stresses and strains in suitable deterioration models.

PMS Object uses a linear elastic model that takes into account the drainage class. The system calculates number of standard axles until the maximum permanent deformation and fatigue cracking permitted occurs.

To demonstrate the effect of different drainage classes, two examples are calculated. For the first example the following road structure is used:

- 40 mm wearing course of bitumen + 70 mm asphalt as strengthening
- 120 mm bituminous macadam
- 800 mm old sub-base gravel material
- silt in the subgrade

Figure 91 shows the affect drainage has on deformation and fatigue cracking. If the drainage is improved from class 3 to class 1 the lifetime will increased by a factor of nearly 2.2.

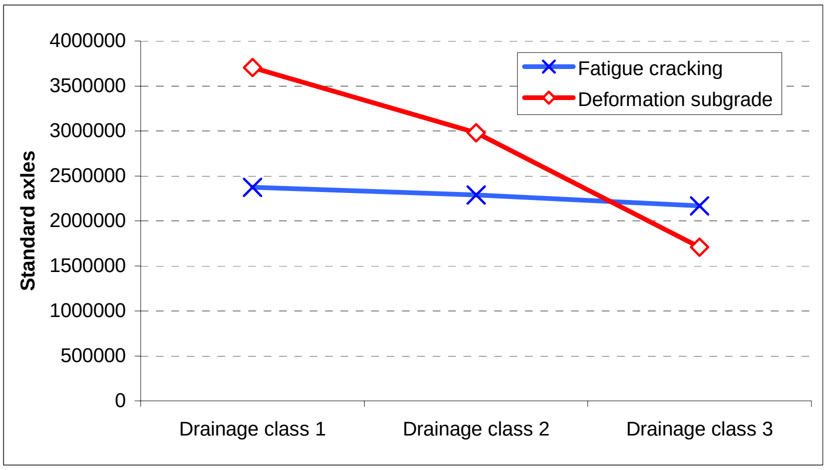


Figure 91. Effect of drainage - example 1.

As can been seen, the lifetime related to deformation, is more than doubled. The lifetime related to fatigue cracking has only increased by 10 %.

The next example has the following road structure:

- 40 mm wearing course of bitumen
- 120 mm bituminous macadam
- 500 mm old sub-base gravel material
- moraine (according to subsoil type 3 in the Swedish design guide - ATB väg)

For this road structure improving the drainage was the only strengthening measure. Figure 92 demonstrates that the lifetime related to deformation, increased by a ratio of 2.6, but for fatigue cracking the lifetime only increased by 4 %.

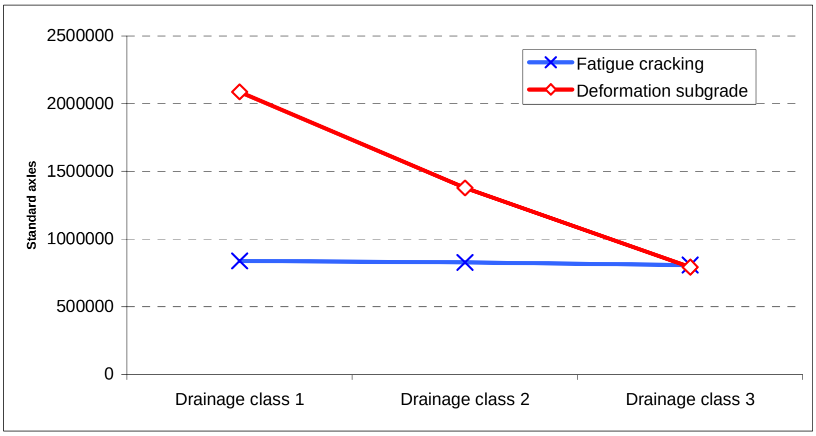


Figure 92. Effect of drainage - example 2.

If the drainage quality is improved from class 3 to class 2 only (as can be the case when just clearing the ditches), the lifetime will increase by a factor equal to 1.75 related to deformation, for both examples.

6.4.2 Aashto design guide

The most comprehensive model today is perhaps in the new AASTHO-design guide from 2002, but the design system is not ready for use. The previous version of the design guide used structural numbers and layer coefficients. The guidelines makes use of correction factors for the layer coefficients depending on the quality of the drainage and the percentage of time that the pavement structure is exposed to moisture levels approaching saturation. The correction values are presented in table 4.

Table 4.

Table 2.4 Recommended m_i values for modifying structural layer coefficients of untreated base and sub-base materials in flexible pavements.

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

The layer coefficients are a function of the elastic modulus and the equation has the form $a = k \cdot \sqrt[3]{E}$ where "a" is the layer coefficient, "k" is a constant and "E" is the elastic modulus.

If "a" changes with a factor equal to "y", then "E" will change:

$$y = \frac{a_1}{a_2} = \frac{k \cdot \sqrt[3]{E_1}}{k \cdot \sqrt[3]{E_2}} = \sqrt[3]{\frac{E_1}{E_2}}$$

$$E_1 = y^3 \cdot E_2$$

If y = m_i in table 4, the changes in E-modulus depending on the drainage condition can be calculated. If a typical subbase has an elastic modulus equal 170 MPa the table above will look like table 5 when using E-modulus.

Table 5. E-modulus for subbase materials depending on drainage condition - AASTHO Guide.

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less then 1 %	1-5 %	5-25 %	Greater then 25 %
Excellent	466 - 418	418 - 373	373 - 293	293
Good	418 - 332	332 - 259	259 - 170	170
Fair	332 - 259	259 - 170	170 - 87	87
Poor	228 - 179	179 - 87	87 - 37	37
Very poor	179 - 146	146 - 72	72 - 11	11

It is obvious that the stiffness of a road structure will change enormously depending on the drainage quality and this factor is more important than how often the materials are exposed to high moisture content. These values are not the values just when the materials are moist, but a design value that describes the effective layer stiffness throughout the design period.

The changes in elastic modulus are much greater than for the Swedish design guide, but knowing exactly where to place the different “drainage classes” into AASTHO’s “Quality of drainage” is not clear. There is no doubt that the change in lifetime is larger using the AASTHO numbers when the drainage quality is changed from “very poor” to “excellent” than using the Swedish deterioration models for rutting/roughness and fatigue cracking.

6.4.3 HDM-4

In HDM-4 from the World Bank, there are models for predicting road deterioration. These models do not use a mechanistic approach as the models above. For flexible pavements HDM-4 include 8 models for predicting the rate of deterioration:

Surface deterioration:

- cracking
- ravelling (disintegration of the surface course)
- potholes
- edge damage

Deformation:

- Rutting
- Roughness

Surface texture:

- Depth of texture
- Friction

The models are integrated so it is difficult to focus on the effect of moisture content only.

The models for roughness and rutting have considered the effects of the infiltration of water into the pavement structure caused by cracks and potholes in the surface layer.

The rutting model is a function (among other parameters) of the mean monthly precipitation and the total area of cracking as a percentage of the total carriageway area. The different phases in the model can be seen in figure 93 and describes that the quality of the material in the road structure has a significant effect on the deformations. (Ref: Kannemeyer, Louw.)

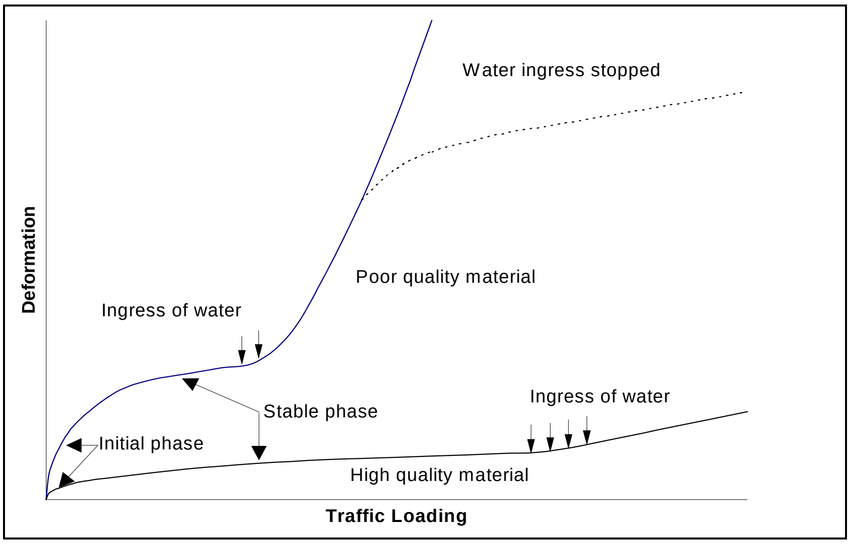


Figure 93. HDM-4 – deformation (Kannemeyer, Louw.).

HDM-4 has also models that predict the percentage of areas that are cracked and have potholes for each year.

The roughness increase in one year is calculated by HDM-4 as a sum of:

- a structural deformation contribution (Structural number, axle loadings, environmental conditions)
- rutting contribution (Structural number, axle loadings, thickness of asphalt surface)
- cracking contribution (change in total cracking consisting of structural, wide and thermal)
- potholes contribution (number of potholes)

The environmental condition for the structural deformation contribution reflects greater the increase in structurally based cracking in wetter climate conditions.

$$\Delta IRI = K_{gp} \cdot (\Delta IRI_{Structural} + \Delta IRI_{Crack} + \Delta IRI_{Rut} + \Delta IRI_{Pothole}) + \Delta IRI_{Environment}$$

The influence on IRI caused by the rutting, ΔIRI_{Rut} , is a function of the standard deviation of the rut depth. K_{gp} is a calibration factor for the equation.

In some of the models Structural Number (SN) are used. For a pavement structure the SN is the sum of the layer coefficients multiplied by the layer thickness of all layers above the subgrade. The strength of the subgrade contributes to the SN and the contribution depends on the pavement total thickness and the CBR-value for the subgrade.

HDM-4 normally use only two seasons in the model; one dry and one moist season, but this does not prevent the use of extra seasons, for example spring thaw.

The drainage conditions are used for calculating an Adjusted Structural Number and they use the same models as presented in the AASTHO guidelines.

The models for ΔIRI_{Crack}, ΔIRI_{Rut} and $\Delta IRI_{Pothole}$ all depend on rainfall. The $\Delta IRI_{Environment}$ model is a function of a moisture index and temperature classification and is not at all dependent on the drainage conditions.

The HDM-4 models are complex and they impede with each other. They handle the effect of the infiltration of water through a cracked surface and it is possible to adjust the models for local conditions. The intention is that the model can be used all over the world; also in the NP area.

6.4.4 FHWA, LTPP-PROGRAMME IN SHRP

In the SHRP programme in the U.S.A., 223 road sections in four climate zones with asphalt surface and granular base have been investigated to model the development of IRI. The results from the studies are presented in the publication “Investigation of Development of Pavement Roughness”, FHWA-RD-97-147.

The models are based on multiple regressions of the condition development from a large range of potential parameters and have through regression analyses quantified the importance of moisture content.

The diagram below shows some examples of IRI value as a function of pavement age and moisture content in the subgrade. The diagrams illustrate examples for three different conditions in two climate zones. All examples are roads with an asphalt surface and a granular base layer. The regions are:

- Region Dry, freeze
- Region Wet, no freeze
- Region Wet, freeze, subgrade - fine content (material < 0,075 mm) > 50 %

The diagrams were made in a project studying the socio-economic effect of drainage in Norway and typical Norwegian conditions were used in the model. An average annual traffic of 1000 vehicles was also applied.

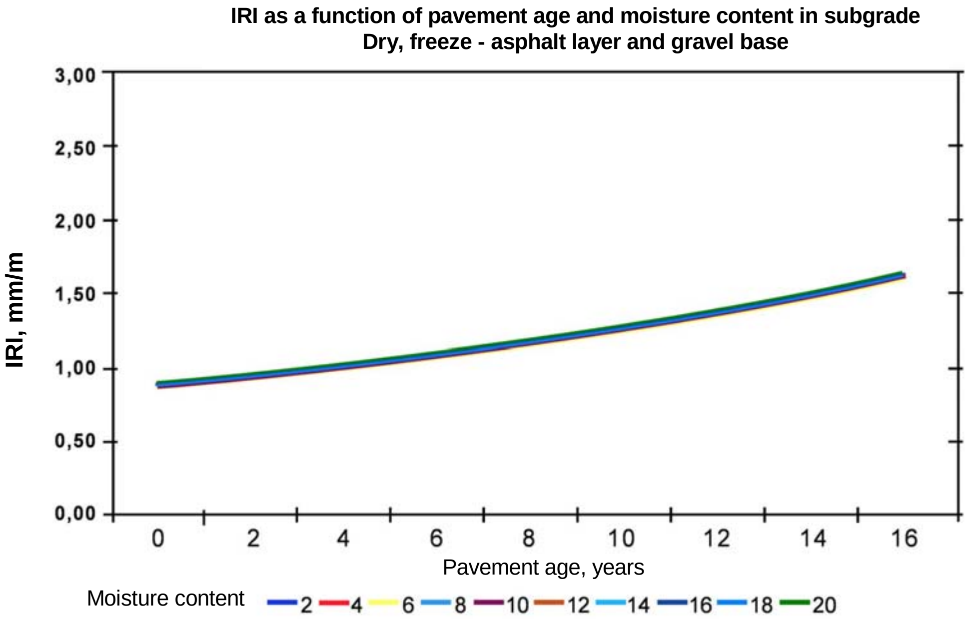


Figure 94. Example 1 on IRI value development on dry freeze conditions.

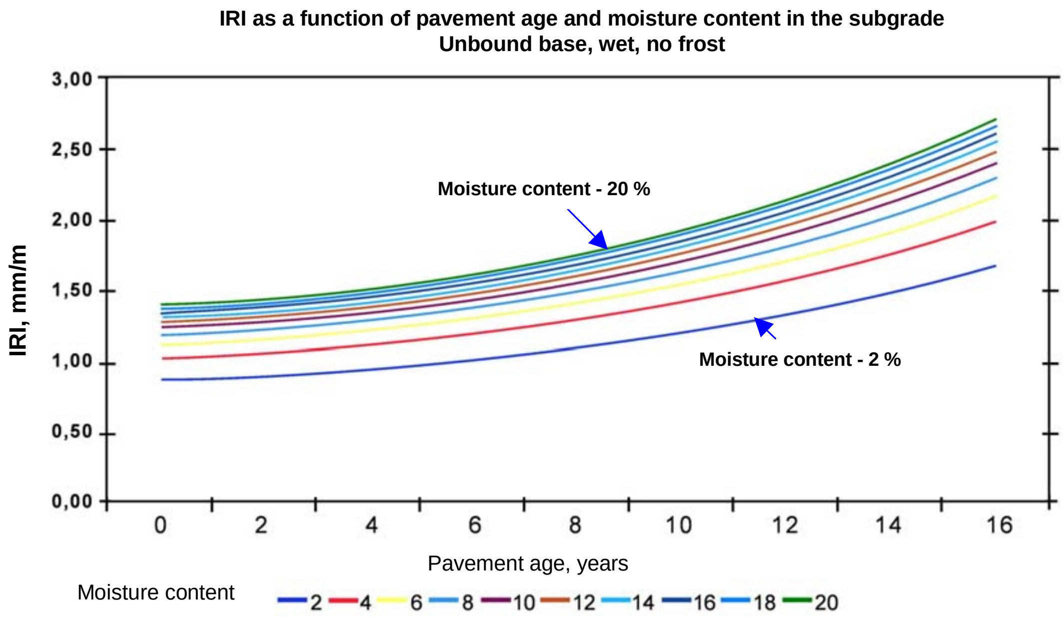


Figure 95. Example 2 of IRI value development on wet no frost conditions.

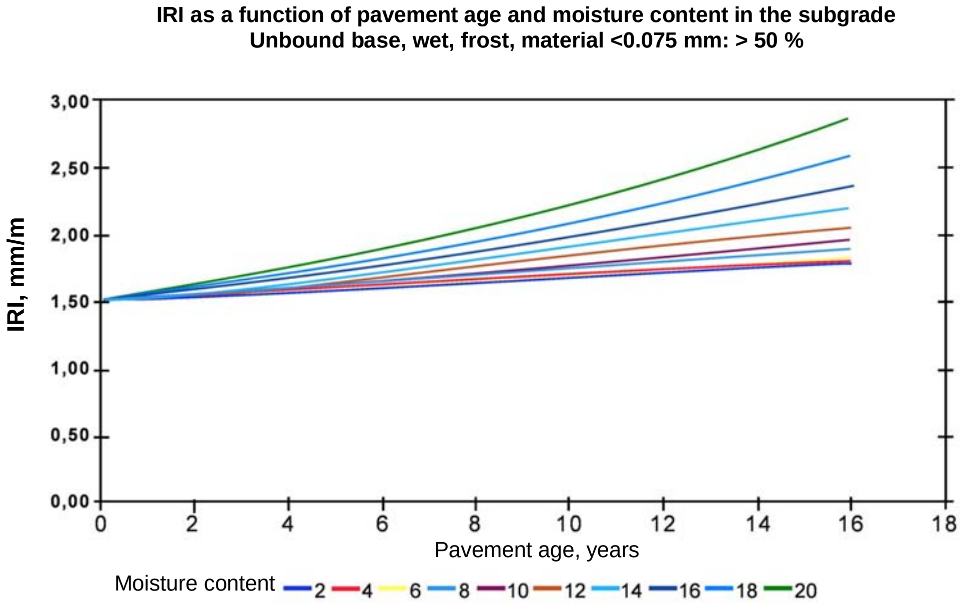


Figure 96. Example 3 of IRI value development on wet and frost conditions.

The first figure (94), dry and freeze, the moisture content has no affect on the development of IRI even thought the road structures are exposed for freeze/thaw cycles. It is mainly the age of the pavement that counts.

From figure 95, wet and no freeze, and figure 96, wet and freeze with the largest content of fines, are the only ones that show a clear relationship between moisture content in the subgrade and the increase of IRI value.

Because the model uses many parameters it is possible to make the wrong conclusions. The other parameters can overshadow the moisture parameter (as for example the precipitation). There is a need for further research.

The model uses the “Integrated Climatic Model” (ICM), which will be used in the AASHTO 2002 Design Guide. The ICM calculates the temperature in the pavement structure, moisture content and probably also the frost heave from climatic data collected from metrological stations.

6.5 CALCULATION OF THE AFFECT OF DRAINAGE ON DEFORMATION IN THE SUBGRADE

The relationship between elastic strain on top of the subgrade and the number of load repetitions before failure is often used as a design criterion in different road structure design guidelines. The relationship can be expressed by the equation:

$$N = a \cdot \left(\frac{1}{\varepsilon_t}\right)^b$$

a and b are constants
N - number of axle loads
ε_t - vertical elastic strain on top of the subgrade

When the ground water table is changed the resilient modulus for the material in the road structure also changes. This changes the vertical strain on top of the subgrade and the number of load repetitions until failure will also change.

In this research the influence of drainage is of great interest and this can be expressed as the relation between N_{draind} and $N_{undraind}$. The equation is:

$$\frac{N_{undraind}}{N_{draind}} = \left(\frac{a \cdot \left(\frac{1}{\varepsilon_{t-undraind}} \right)^b}{a \cdot \left(\frac{1}{\varepsilon_{t-draind}} \right)^b} \right) = \left(\frac{\varepsilon_{t-undraind}}{\varepsilon_{t-draind}} \right)^{-b}$$

The value of “b” varies mainly depending on the subsoil. For silt and clay the value used in different models is b=4, but for gravel and sand this value varies more. (The Swedish model PMS Object use b=4).

Example 1 – Use of linear elasticity and E-modulus from AASTHO design guide

In the first example, the E-modulus from Table 6.3 (AASTHO-guide) with “Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation” equal to 1 %, 5 % and 25 % has been used to calculate the strain on the subgrade. The road structure, in this example, is a 3 cm asphalt layer, 20 cm of crushed rock as base, 60 cm of sand as subbase and soft subsoil (10 MPa).

The strains on top of the subgrade are calculated using linear elasticity (the NOAH program from Nynäs) and only the modulus for the subbase layer has been varied in the computations. Using the equations above the relative changes in the number of standard axles to failure is shown in figure 97.

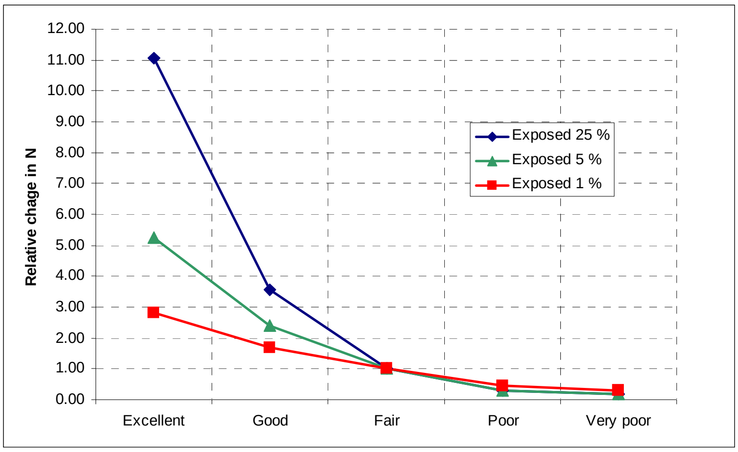


Figure 97. Change in N using AASTHO.

The changes are compared to "Fair" drainage quality. The next figure (98) shows the same, but with a scale that illustrates the worst drainage condition.

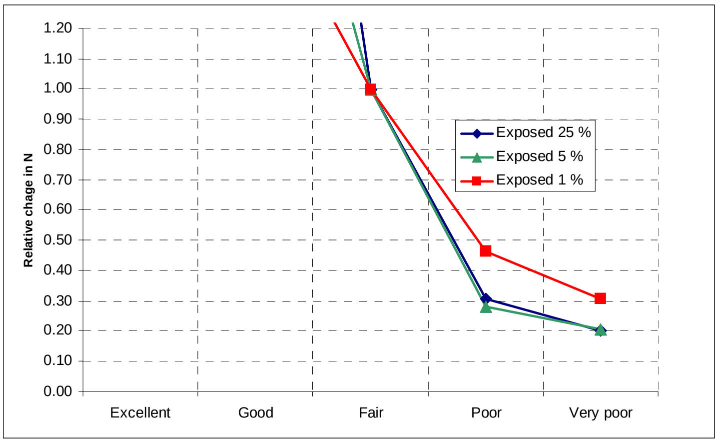


Figure 98. Change in N using AASTHO focusing the area "Very poor" to "fair".

In this example the drainage condition will affect the lifetime extremely. When improving the quality from "fair" to "excellent" the lifetime change are dependent on the time the materials are exposed to high moisture content, but not when improving from "poor" or "very poor" to "fair". If we focus on the last improvement, the lifetime will increase by a factor of about 5 when improving the drainage from quality "very poor" to "fair" and 3.4 when improving from "poor" to "fair". By improving from "very poor" to "excellent" the increase in lifetime will be enormous and does not seem to be realistic.

Only the E-modulus for the subbase has been varied in this example and in addition there will be a contribution from the subgrade and base layer as the moisture content is dependent on the pore suction.

Example 2

The second example demonstrates how models presented in the literature review can be used to calculate the affect of drainage.

The model presented by Lary and Mahoney (1984)(see chapter 4.2) is used as a model for the moisture- modulus relationship. The model can be expressed by the equation:

$E = K \cdot E_0^{k3}$

K and k3 are parameters and are presented in figure 32 and E_0 = E-modulus when w=0 %. To decide the elastic modulus in this example the k3-factor is chosen to be -0.0324.

The relation between soil suction and moisture content (soil water characteristic curve (SWCC)) is presented by Noss (chapter 3.4) and the values from figure 16 have been used to estimate the moisture content for the subbase.

Two road structures have been analysed, one drained and one undrained. The structure is the same as in the previous example and for the drainage condition the ground water table are given as being 20 cm below the foundation level. For the undrained condition the ground water table is given as being 20 cm above the bottom of the road structure. The subgrade soil is assumed to be silt according to the Swedish design guide. (E=10 MPa). The subbase is divided into three layers with different suction levels and moisture contents and also by different elastic modulus.

The road structure and the two drainage conditions are illustrated in figure 99 and 100.

Drained construction.
Ground water table 20 cm below formation level.

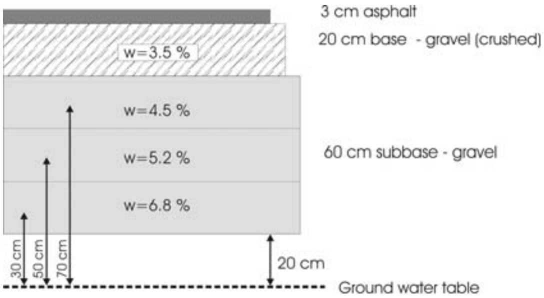


Figure 99. Road structure and ground water table - drain condition.

Undrained construction.
Ground water table 20 cm above formation level

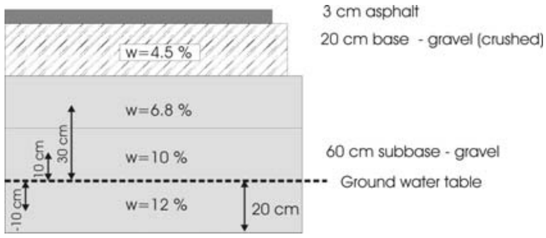


Figure 100. Road structure and ground water table - undrain condition.

The vertical strain on top of the formation level is calculated using the computer program NOAH form Nynäs. NOAH uses linear elasticity in the calculations.

The diagram in figure 101 shows the strains on top of the formation level.

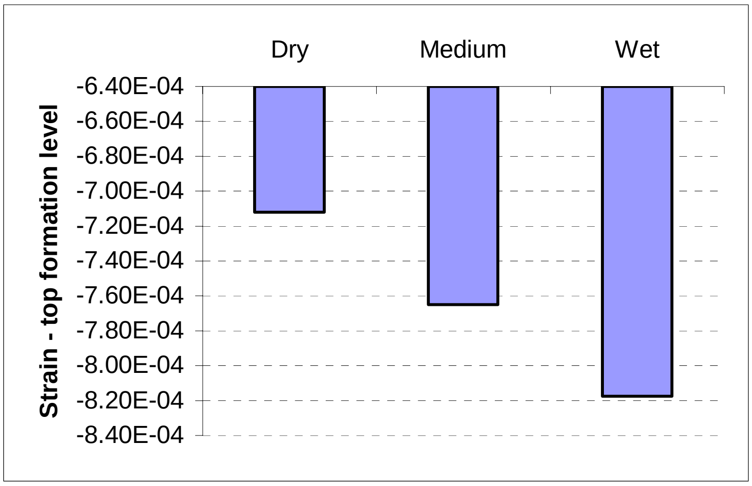


Figure 101. Strain on top of the foundation.

If the equation for relative change in N is used, bringing the drainage condition from wet to dry will increase the lifetime by a factor 1.74.

$$\frac{N_{drained}}{N_{undrained}} = \left(\frac{\varepsilon_{t-drained}}{\varepsilon_{t-undrained}}\right)^{-4} = \left(\frac{-7.12\cdot 10^{-4}}{-8.17\cdot 10^{-4}}\right)^{-4} = 1.74$$

This change in lifetime is only caused by the changes in the subbase materials. In addition there will also be a contribution from changes in E-modulus for the subgrade and the base layer and this will (can) increase this ratio considerably.

The increase in lifetime can also be achieved by increasing the subbase thickness. Figure 102 shows the elastic strain on top of the foundation for the undrained conditions when the subbase layer thickness alters.

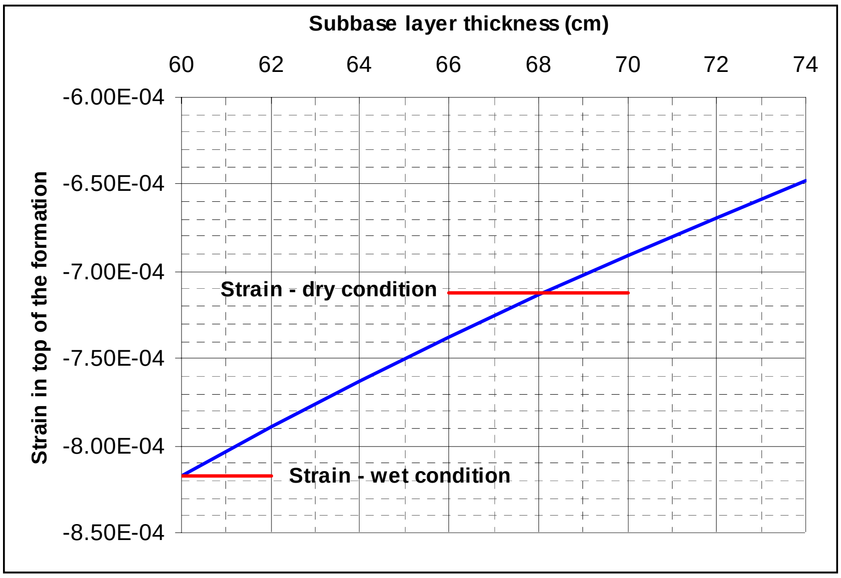


Figure 102. Comparison of the effect of drainage and increasing subbase layer thickness.

An increase in the subbase layer of 8 cm will have the same effect as lowering the ground water table 40 cm from 20 cm above the foundation level to20 cm below. This of course cannot be done on an old road, but it is important to be aware when designing a road structure and drainage system. There are places where it is difficult to provide sufficient drainage and the best solution can be increasing the road structure thickness.

6.6 SUMMARY - PREDICTION MODELS AND OBSERVED DRAINAGE EFFECTS

Typical for the NP-area is that there are frost actions that are followed by spring thaw with excess moisture content. The drainage condition will also affect how this excess moisture influences the road deterioration. The models that are developed for areas without frost and much lower frost index cannot be used.

Rut deformation on sloping ground shows that there are differences between the drained and undrained traffic lane. The ratio between the rut depth on the road cut lane and the other lane varies from 1 (equal rut depth) up to 3.6. Approximately 20 % of the roads investigated have a ratio larger than 1.5. This ratio will of course depend on the drainage condition and the subgrade soil. The distribution of the rut depth ratio is shown in figure 81.

The IRI ratio shows that the deterioration is different in the two traffic lanes. It is more difficult to measure the IRI and there are some variations that complicate use of this parameter in comparisons.

The lifetime of roads on embankments for 155 km national roads is, on average, 20 years and in road cuttings 13.7 years. The drainage quality is different, but the change in lifetime is not only a result of the different moisture contents. Different soils (and the best soils available) are often used in embankments and this can contribute to an increase in the lifetime.

When comparing the results from the Swedish design guide with the observed deterioration of a road cut lane and an embankment lane on sloping ground, the results are surprisingly similar. The examples used in the Swedish design system, shows that improving the drainage system will increase the lifetime by a factor of 2.2-2.6 for the chosen conditions. The subsoil in this example was moraine, but if silt had been used the ratio would have been larger.

The examples using the models from Lary and Mahoney (1984) and Noss (1978) show a ratio of changed lifetime for drained / undrained condition equal to 1.74 and in these cases only changes in the subbase were taken into account. This ratio would have increased if the effect of the changed moisture content in the subgrade and base layer had been added then the result would have been at same level as in the Swedish design guide and the observations that have been made done on sloping ground.

The models from SHRP are not easy to understand. It’s clear that also these models give increased deterioration when the moisture content increases and that the differences are larger when there is frost. If we compare the IRI value in figure 96 for moisture content 10 % after 16 years, we will have the same value when the moisture content are 20 % after 6,8 years.

The effect of drainage using the AASTHO guidelines shows a much larger increase in lifetime if the drainage quality is improved from “very poor” to just “fair”. The lifetime multiplies 5 times in the examples.

7 Effect of drainage on road life cycle costs

7.1 GENERAL

The field observations and predictions models demonstrate that drainage has an effect on the road deterioration, but it is difficult to tell exactly how large this effect is. This depends on many parameters as, for example, material in the subsoil and road structure, precipitation, cracks and potholes in the asphalt layer, terrain, frost and thawing condition, maintenance and how the drainage system was designed.

The effects of drainage will change over the road network and it is impossible to record all the parameters that are needed. Therefore we must use simplified models.

7.2 EFFECT OF DRAINAGE – GROUPED

In order to evaluate the life cycle costs for drainage systems in different conditions, it is necessary to make some assumptions and simplifications. It is impossible to use an analytical approach where it is necessary to know the material type in the road structure and the subgrade soil, the cross section of the road and the ditch and the climate. This may be possible in the future when additional data collecting, interpretation and analysis methods have been developed.

One simplification is to categorize the problems areas in groups where the condition is similar and the effect of improvement is the same. From the models described previously in the report and the observation that were made on deterioration on sloping ground, the factor for extended lifetime is estimated in table 6.

These estimates will of course depend on the extent to which improvements are made and the numbers in the table show what is possible to achieve.

Table 6. Changes in lifetime when improving the drainage system.

Drainage condition	Drainage classes 1)	Factor - change in lifetime by improving the drainage system
Group 1 Drainage system does not work at all (or drainage system does not exist). Water susceptible soil in road structure and subgrade. Very high ground water table. Low ground and rocks blocking the ground water flow. Often local spots.	>3	> 2,5
Group 2 Drainage system does not work at all and the soil in road structure and subgrade are less water susceptible then in group 1. Drainage system is working badly because of lack of maintenance (ditches and culverts not cleared) and water susceptible soil in road structure and subgrade.	3	2-2,5
Group 3 Drainage system is working badly because of lack of maintenance. (Ditches and culvert not cleared.) The soil in road structure and subgrade are less water susceptible.	2	1,5-2
Group 4 Drainage system is working unsatisfactory because of lack of maintenance or the maintenance guidelines are not sufficient.	1-2	1-1,5

1) Comparison to the drainage classes in the Swedish design guide.

To decide the effect for each drainage task one of the models presented in chapter 6 or an equal model must be used.

7.3 COST FOR DRAINAGE MAINTENANCE AND IMPROVEMENTS

Knowing the costs for drainage maintenance and improvements is essential in order to be able to calculate the effect on life cycle costs. The costs will vary between the countries in the NP-area and also within each country.

Since the lifetime in this report is defined as the time between two repavings, it is practical to use drainage costs as a percentage of the repaving costs.

Ditches

Normally the cost of maintaining the drainage system is much less than resurfacing and in Norway resurfacing on a low traffic volume road will cost 8-10 times more than clearing the ditches and culverts. (Ditch clearing costs 10-12 % of the repaving costs.) (New pavement: 32-37 €/meter. Ditch clearing: 3.7-4.5 €/meter). In Finland, where the LC analysis was also done, the prices were lower but the ratio between ditch cleaning and new pavement was roughly the same.

A ditch may not function even though it is shaped according to the design guidelines. In such cases the drainage has to be improved by increasing the depth of the ditch or by using deep drainage. Deepening an open ditch may require the acquisition of land and other complications due to elements near the road area. These improvements are more expensive and the costs depend on the nature of the problem. Normally, drainage improvements should be done 1 -2 years before repaving. The cost of installing deep drainage ranges from 30 -50% of the cost of repaving depending on the type of subgrade soil.

A comparison between the costs of repaving of 5 or 6 meter wide road and costs of ditch clearing was done. Normally there are only short parts of a long section that have deteriorated due to inadequate drainage, but these short parts are the reason why the whole section has to be repaved as illustrated in figure 103. Looking at an entire section, the relative costs will still be reduced and making drainage improvements will be even more cost effective.

Raising the carriage way grade line

To raise the carriage way will require a new road structure. It is possible to recycle the old base and pavement and use this as a part of the new base, but still the costs for this improvement will be high.

New subbase, recycling the old base and pavement and widening the road will have a cost of 0.8-1.5 times the costs of a new pavement. However it should be kept in mind that the benefits of raising the grade line will generally last much longer than one pavement life cycle and raising the grade line also reduces winter maintenance costs.

Other improvements

The cost for other improvements like removing rocks that are blocking the ground water flow, improving culverts, improving outlets and inlets are difficult to estimate. These problems are often single spots that are one of the reasons why a whole section has to be repaved.

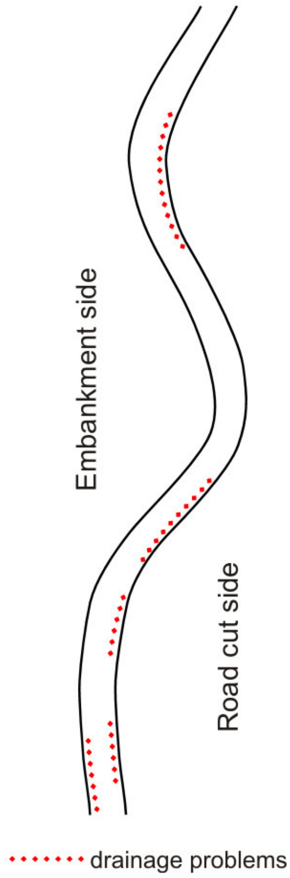


Figure 103. Local drainage problems cause need for rehabilitation of a longer section.

7.4 COST COMPARED TO CHANGE IN LIFETIME AND LIFE CYCLE COST

The goal by improving the drainage system is to reduce the deterioration caused by inadequate drainage to such an extent that the lifetime is not below what is normal. This means that the focus has to be made on single spots and shorter sections where the problems are evident.

On sloping ground, where one lane is drained, the goal is to reduce the variation of the curve in the diagram, figure 104, and to adjust the median to be equal to 1. The goal is to have the same condition in both lanes, but this may result in comprehensive drainage improvements and a difference must be accepted.

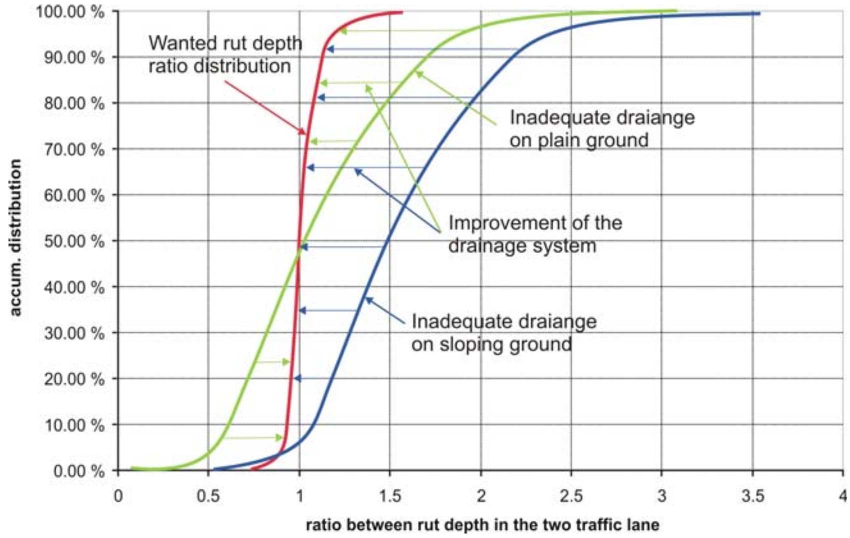


Figure 104. Goal for improving the deterioration on different lanes.

One other solution to this problem is to increase the road structure thickness on the road cut side of the road and maintain a normal drainage. As demonstrated in example 2 in chapter 6.5, an increase of the subbase layer thickness by 8 cm had the same effect as lowering the ground water table by 55 cm. When designing a road structure on sloping ground the subbase should be about 10 cm thicker in the road cut lane then on the embankment lane. (5 cm reduction on the embankment that is added to the road cut side.)

In general life cycle cost can be expressed by the equation:

$$LCC = PV_0 = B_0 + V_0 - R_0 + T_0 + M_0 \text{ where:}$$

LCC = PV_0 = life cycle cost: present value of all cost between year 0 and *N* with year 0 as basis of comparison

B_0 = construction cost (in year 0)

V_0 = present value of all maintenance costs in the period

R_0 = present value of a possible remaining value of the construction and maintenance costs at the end of the period

T_0 = present value of possible excess costs for the road users in the period

M_0 = present value of possible environment costs in the period

Year 0 = basis of comparison

PV_0 = present value of all repaving cost between year 0 and *N* with year 0 as basis of comparison

The present values are calculated by the equation: $PV_0 = \sum_{n=0}^{N} \frac{K_n}{(1+r)^n}$

N = number of years in the period for the analysis

K_n = costs in year n

r = discount rate

In this analysis the focus has been on the repaving cost, cost for drainage improvements and maintenance cost for the drainage system. The road user costs and the environment costs have not been considered. The period of the analysis is chosen to be 50 years. Figure 105 shows how the calculation of the present value is done with and without drainage improvement. (Red colour indicates drained condition).

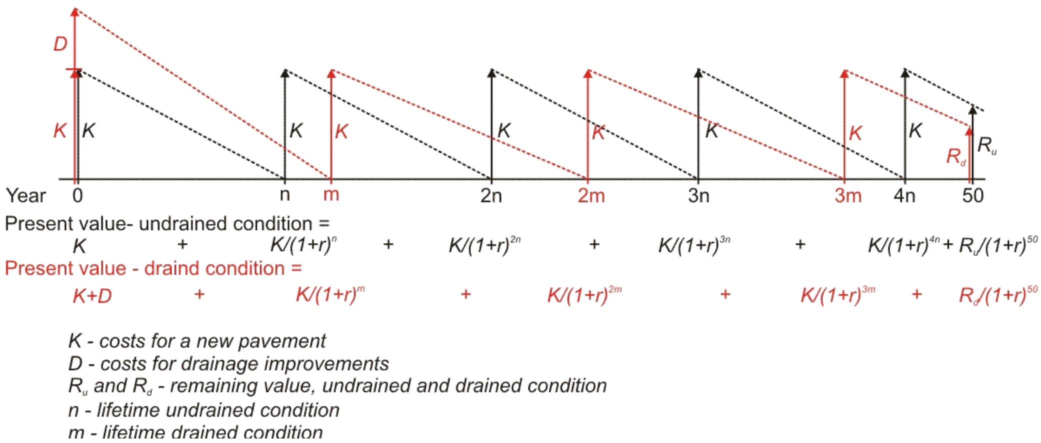


Figure 105. Calculation of present value.

The diagram in figure 106 shows the present value of repaving for a period of 50 years. Repaving is done in cycles depending on the lifetime. The repaving costs are set equal to 100 for each meter road. (Or 100 %. All other costs are given as a percentage of the repaving cost.).

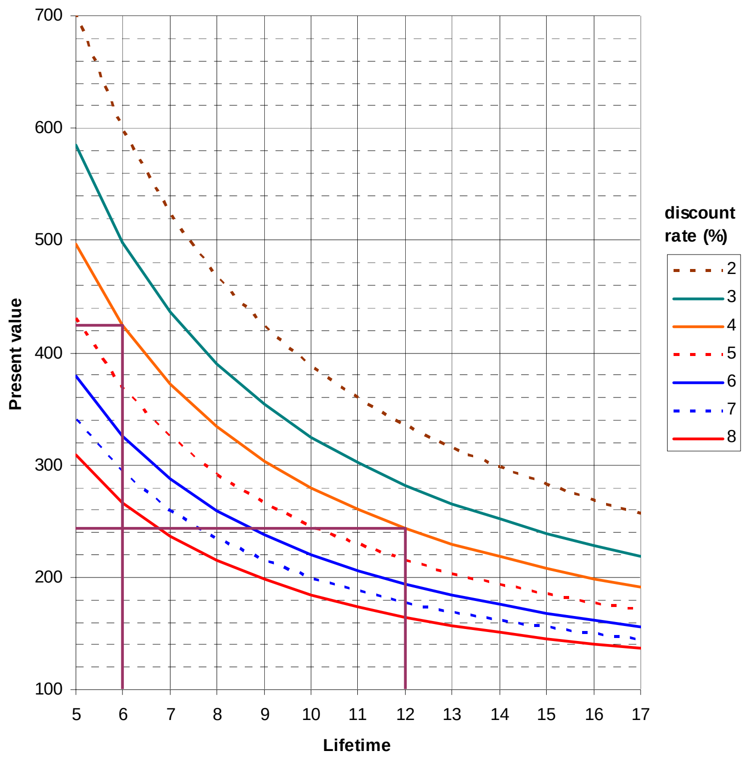


Figure 106. Present value of repaving costs as a function of lifetime and discount rate.

The discount rate varies and the diagram gives curves for 2 – 8 %. This diagram can be used to evaluate the life cycle costs. If the costs of the drainage maintenance/improvements are less then the changes in present value, the improvement will reduce the life cycle costs. In other words, when this condition is fulfilled drainage improvement will be profitable:

$PV_{0\text{-}drained}$ + *maintenance/improvement costs* < $PV_{0\text{-}undrained}$

If, for example, a road with drainage problem and a lifetime equal to 6 years the present value is 498 with discount rate at 4 %. (498 means that the present value is 498 % of the costs for a new pavement. If this cost is 35 €/meter then the present value is 35*4,98 = 174.3 €/meter). If an improvement that increases the lifetime by a factor of 2 (lifetime equal 12 years) is made, the present value of repaving is 282 (or 35*2.82 = 98.7 €/meter). To calculate the life cycle cost the cost for the drainage improvement has to be added to this value. If the sum is less then 498 the improvement will lower the life cycle costs. As long as the drainage improvements cost less then 216 % of a new pavement (that is 35*2.16 = 75 €/meter), the improvements will be profitable. The calculation assumes that the improvement is done once and is performed in year 0.

Drainage maintenance, as for example ditch clearing and clearing culverts, inlets and outlets, have to be done regularly and the present value for these costs must be determined. Figure 107 shows the present value depending on the cycle for ditch clearing and the discount rate for two different ditch cleaning costs.

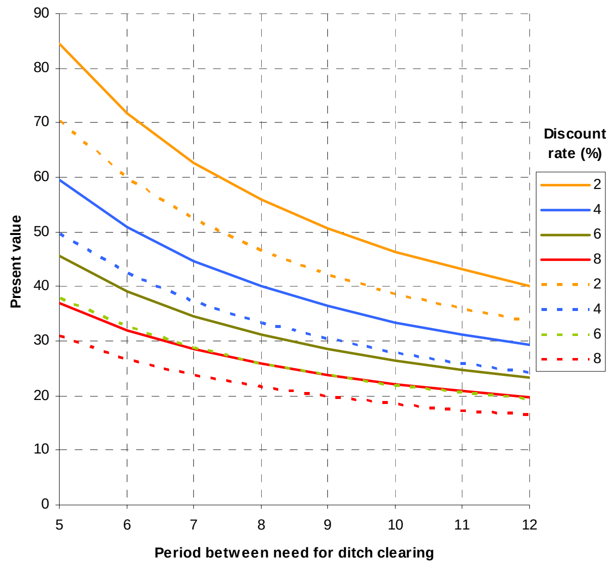


Figure 107. Present value for repeated ditch clearing.

If the ditch clearing has to be done every 7th year the present value is 44.7 % when the cost is 12 % of the repaving costs and discount rate is 4 %.

Figure 89 and 90 shows examples where the Swedish design guide are used to calculate changes in lifetime as a function of drainage classes. If we anticipate the ditch clearing just changes the drainage quality from class 3 to class 2 the increase in lifetime is about 1.75 times. If the lifetime for a road section is 8 years then a ditch clearing will extend the lifetime to 14 years. If the example above is used (7 years interval between two ditch cleaning) the LCC is (50 years comparison period):

Without ditch cleaning:	Present value of repaving: 333.5 % of a new pavement
With ditch cleaning:	Present value of repaving + present value of ditch clearing = 218.4 + 44.7 = 263.1 % of a new pavement.

In this example the increase in lifetime does not have to be more then two years to be profitable. The present value, if the lifetime is 10 years, is 278.9 and if the ditch clearing cost is added the life cycle cost is 278.9 +44.7=323.6. This is less than present value without drainage.

If we use discount rate 8 % the present value without drainage is 215.1. If we only increase the lifetime by two years by clearing the ditch every 7th year, the present value of repaving and ditch clearing is 212.9. This is still profitable, but not as profitable as whit discount rate equal 4 %.

7.5 HOW OFTEN DRAINAGE CAN BE PROFITABLY IMPROVED?

An interesting question is how often drainage improvement measures could be taken while still keeping the life cycle costs profitable. As an example, LCA calculations were made using a drainage improvement cost of 4.100 €/km and pavement replacement cost 35.000 €/km (ratio 0.117). The results of this analysis can be seen in figure 108. The figure shows that by improving/maintaining the drainage it is possible double the lifetime (from 10 to 20 years) and that drainage maintenance can be done every second year and this will still be profitable even though the discount rate is as high as 8 % (used in Norway). If the increase in lifetime is only by 50 % (from 10 to 15 years) and the discount rate is only 4 % (used in Finland), drainage maintenance can still be done every third year. Normally there is no need for doing drainage maintenance more often than this. This example calculation also did not take into account increases in other maintenance costs due poorly working drainage and the benefits of keeping drainage in good condition should be calculated for longer than one pavement life cycle because frost fatigue due to high moisture content will affect to the long term performance of the road structures.

The calculations also showed that it is always worth evaluating the use of more expensive drainage improvement solutions than only ditch cleaning. For instance, if pavement lifetime can be doubled and discount rate is 4 % drainage improvement can cost 8400 €/km and it can be still renewed every 5 years and the life cycle costs are still less than without drainage renovation.

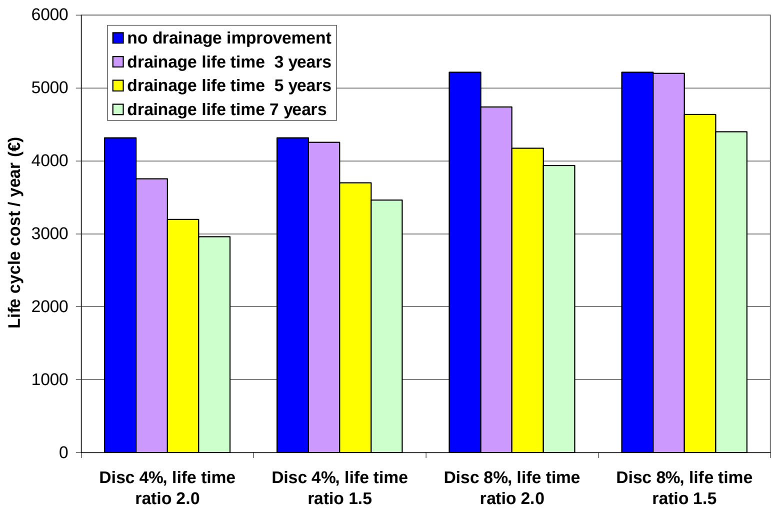


Figure 108. Example of life cycle cost analysis results showing the benefits of drainage improvements. In these costs, pavement rehabilitation costs have been calculated at 35.000 €/km and drainage improvement costs 4100 €/km. Results are presented using two lifetime ratios of 2.0 (10 to 20 years) and 1.5 (10 to 15 years and using to discount rates (4 % and 8%).

In addition, as already stated in chapter 7.3 (see also figure 103) the work will be even more beneficial because normally drainage improvement should only be done on certain parts of the road.

8 Summary and recommendations

This report describes the affect inadequate drainage has on road deterioration and typical drainage problems in the NP-area and makes suggestions on how to solve the problems. Observations of deterioration caused by a lack of drainage have been presented well as different models that can be used to predict the deterioration. LCA for drainage improvements and maintenance has also been discussed.

The report has several parts. The first part is a literature review concerning moisture content in road structures and subgrade soils and the affect moisture has on the characteristics of unbound road building material. All the research results of the subject shows clearly that the bearing capacity of unbound granular materials (E-modulus, deformation properties) are affected by changes in the moisture content. For coarse graded and open soils this effect is less significant. But with dense graded materials and materials with a high content of fines the characteristics can change considerably.

The second part of report classifies the typical drainage problems in the NP-area and also describes how to recognize these problems. The biggest problems are in road sections located on sloping hills. This publication also gives different proposals for improving the problems presented herein. A summary of the classification method and a proposal for improvement methods and structures is presented in appendix 2.

The third part presents modeling and theoretical calculations concerning the effect of drainage on the pavement lifetime. This research was based on the Swedish design guide and the ASSTHO design guide. The results of these calculations showed that working drainage will increase the pavement lifetime depending on models and structures used by a factor of 1.7 - 2.6.

In the fourth part, these theoretical calculations were verified through field observations from Norway and Finland. The observations from individual test cases gave lifetime a ratio of 1.6 – 2.2 for road lanes located on poorly drained and well drained sites. In a network level investigation of 184 km of road in Norway, 19.5 % of the road network had a pavement lifetime ratio greater than 1.5. When normally trigger values for rehabilitation measures or repaving are based on the fact that 15 – 20 % of rutting values are higher than the trigger values these calculations show that poorly working drainage is the key parameter governing the timing for a new pavement.

In the fifth part of this report, life cycle cost analyses were done in order to demonstrate the economical benefits of drainage improvements. These LCA results show that drainage improvement is almost always economic. If pavement lifetime can be doubled it is economical to clean the ditches even every second year if needed. Also more expensive solutions than just ditch cleaning should always be considered. When starting to improve the condition of problem roads drainage improvement measures should be done 1-2 years before the repaving in such a way that the moisture has time to drain from the materials.

This Roadex drainage report clearly shows that maintaining and improving the drainage system is perhaps the most cost effective measure on roads where inadequate drainage is the main cause of deterioration. And as the previous work done in the ROADEX project has shown, poorly working drainage is one of the biggest problems in the NP area.

Authors also suggest that inadequate drainage does, in this case, not only mean lack of maintenance but also that the drainage system was not designed according to the guidelines. The existing guidelines may be insufficient and further improvement of these guidelines should be considered.

Figure 109. Improving the drainage and keeping it in good condition is the most profitable in road located on sloping hills.

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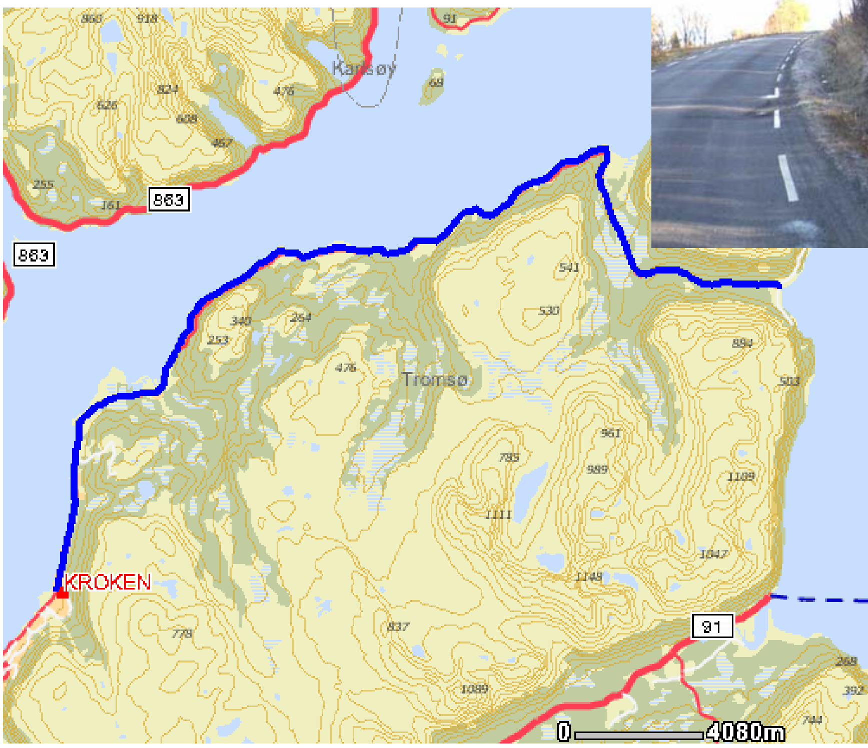
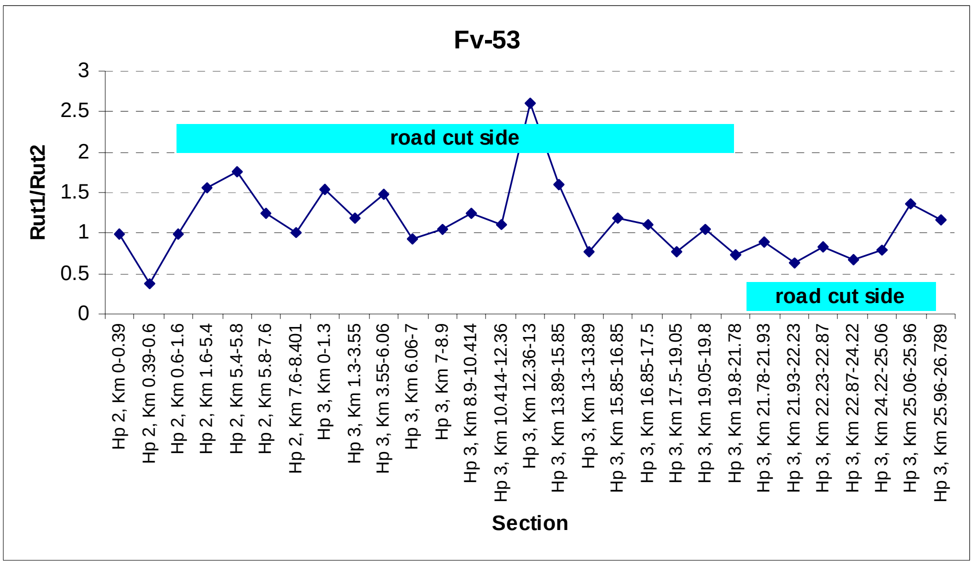
Appendix 1 – Investigation of 184 km county roads on sloping ground

The next pages show 4 maps and statistics for 4 county roads in Troms county. These roads are:

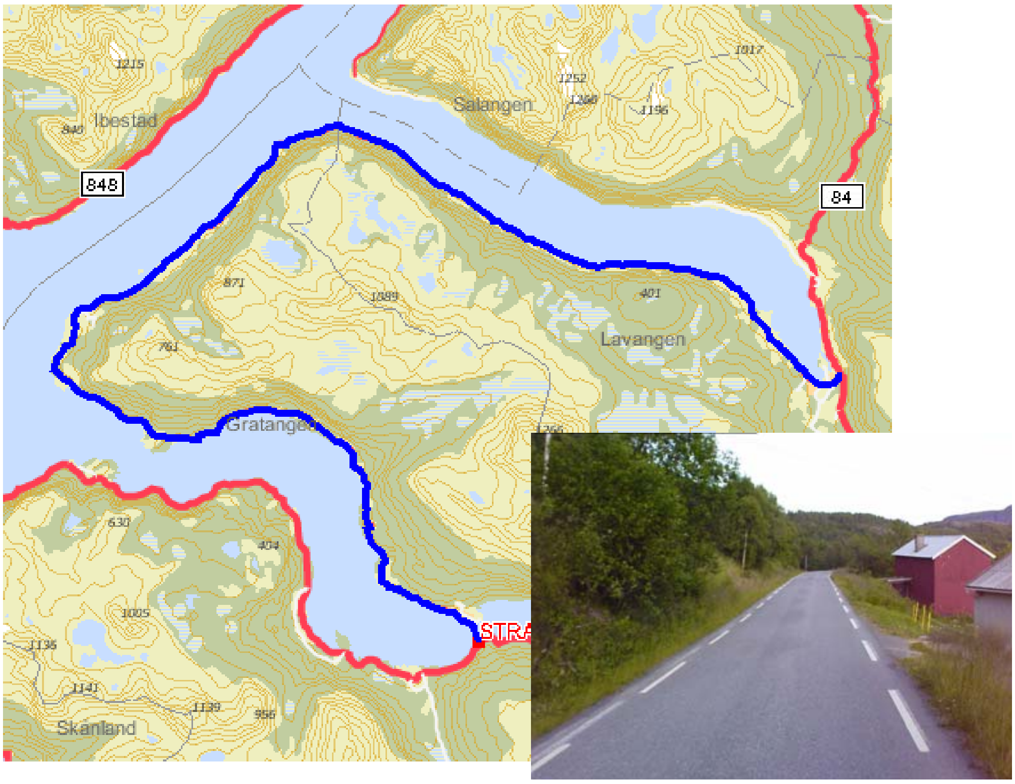
- Road 1, County Road FV-53
- Road 2, County Road FV-141
- Road 3, County Road FV-263
- Road 4, County Road FV-293

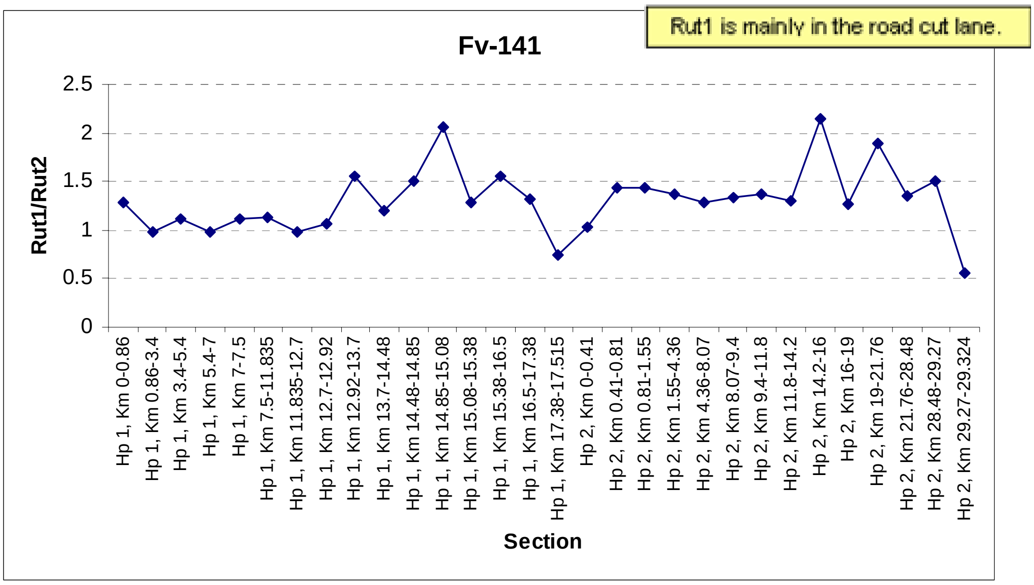
Road 1, County Road FV-53



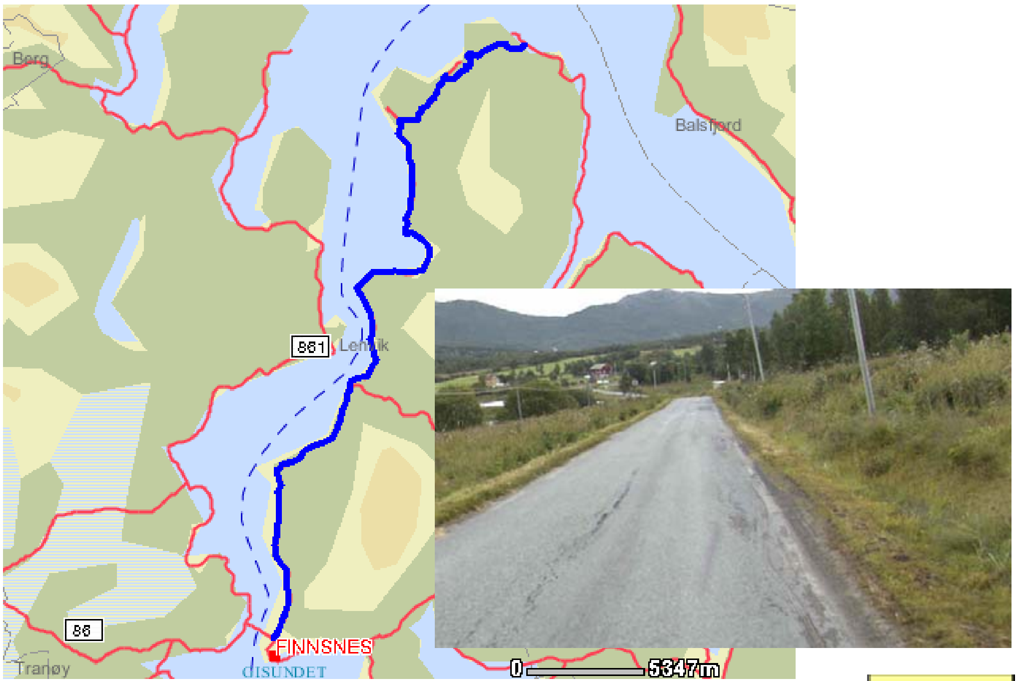


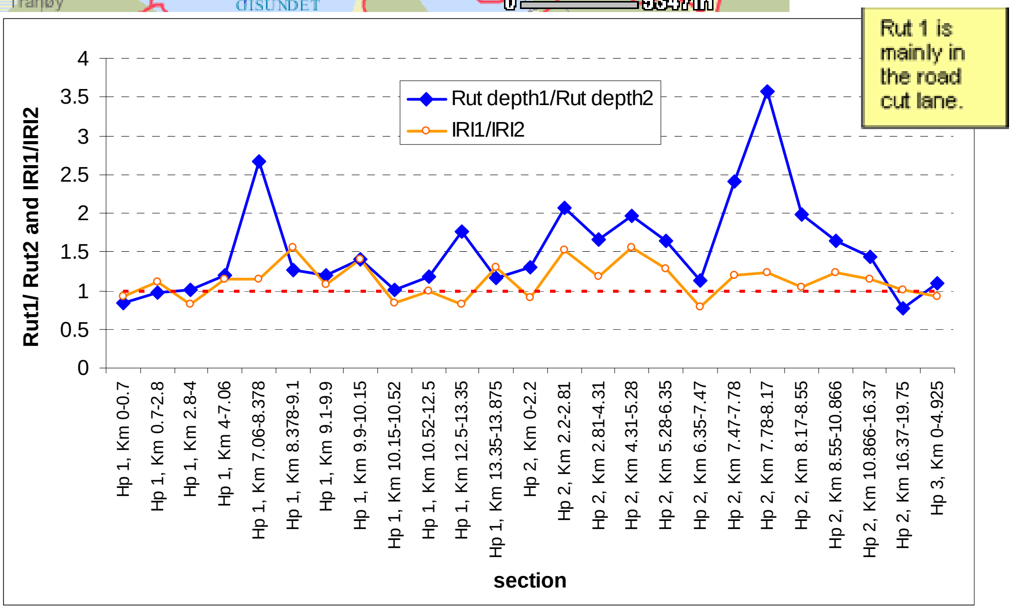
Road 2, County Road FV-141



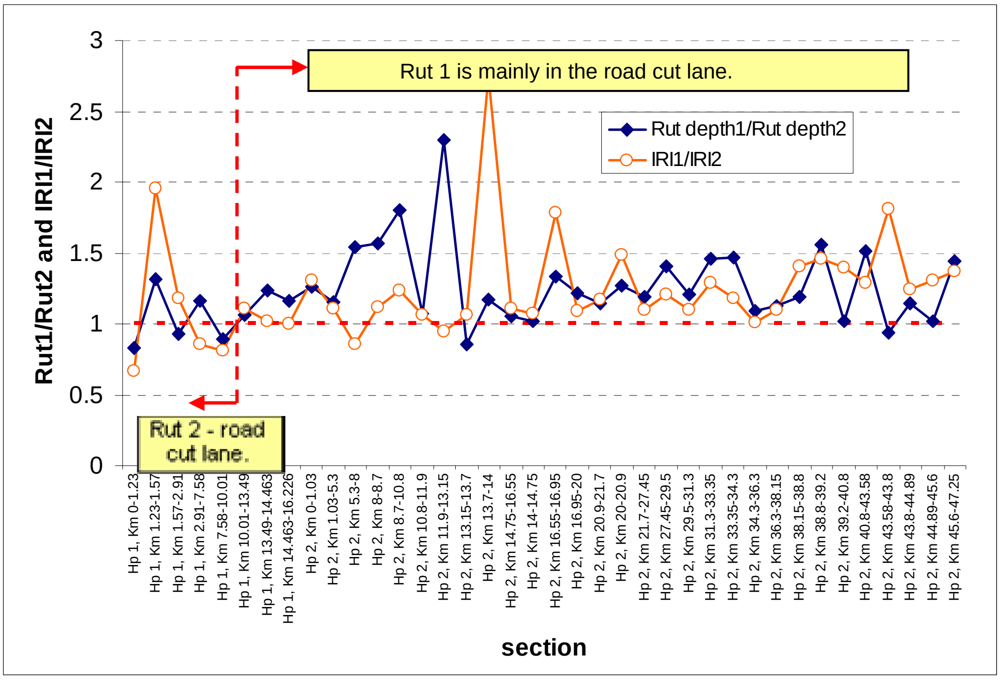


Road 3, County Road FV-263





Road 4, County Road FV-293



Appendix 2 – Table for recognising drainage problems and proposals for solutions

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
Maintenance related problems	Water saturation of road structures during the spring thaw period and freeze-thaw cycle periods in mild winters.	*Bearing capacity problems*. Paved roads: Rutting, cracking and deformations. Gravel roads: Plastic deformation problems during the spring thaw period. In severe cases road can be nearly impassable.	Frost susceptible subgrade soils or unbound road materials that form ice lenses during the frost period resulting in excess pore water during the spring thaw and as a result low bearing capacity. Melted snow and surface water penetrates the road structure from the ditches, road shoulders and from cracks in the road surface.	▪ Clear ice and snow from the ditches to allow the surface water to flow into the drainage system. ▪ Deep drainage ▪ Replace frost susceptible materials with coarse aggregates. ▪ Frost insulation. ▪ Base course stabilization ▪ Strengthening of road structures against spring thaw weakening. ▪ Proper cross fall
	Ditch clogged, i.e. ditches do not stay open.	*In general this problem is related to bearing capacity problems, caused by a lack of ditch clearing.* Mud and fine graded soils are filling the ditch and requiring a lot of work to keep it open. Erosion of outer slope of the ditch.	Too steeply sloped ditches related to the type of subgrade soil. Lack of erosion protection.	▪ Clearing the ditch more often ▪ "Piping the ditch" (subsurface drain) ▪ Ditch trenches filled with coarse material. ▪ Erosion protection. Coarse material, vegetation or different kinds of fabric in the ditch slope.

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
(continuation) Maintenance related problems	Culvert defects.	Visual inspection reveals the structural condition of the culvert. The road is deteriorated (settlements, unevenness) near the culvert. Often frost heaves. Holes in road surface.	Settlements, clogged inlet, frost heave movements, too small a culvert, improper construction of culvert and/or inlet	▪ Replace the culvert with a bigger culvert ▪ Clear the inlet/outlet ▪ Reline PEH-pipe (see ch. 5.2.2.1, fig. 43)
	Blocked culvert inlet.	Rubbish, branches, turf, mud are blocking the inlet. Problems especially after heavy rains when a great amount of surface water needs to be drained away.	The inlet may be designed incorrectly. Culvert diameter is too small. The area upstream from the culvert is eroded and the materials are deposited in the inlet.	▪ Clear the inlet. ▪ Reconstruct the inlet. ▪ Replace with a bigger culvert
	Ice clogging the culverts	Ice clogs the culvert and water will flow across the road during mild weather in winter and during snow melting period in spring. Pooling in the upper ditch.	The frost accesses the culvert either through penetrating from above or through the pipe itself. Low water flow velocity. Drainage area is reduced due to lack of clearing.	▪ Clearing the culvert of sand, gravel etc. will reduce the problem ▪ Steam to melt the ice ▪ Reconstruct the culvert (lower it if possible) and the outlet and inlet. ▪ Solar panel or wind mill that power a heater cable
	Turf on the road shoulder	Turf on the shoulder grows and blocks surface water from flowing off of the road Traffic safety problem (pooling) in addition to deterioration of the road.	Vegetation that grows on the road shoulder and the inner slope of the ditch will form turf that grows larger every year.	▪ Remove the turf

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
Design related problems	Grass verges	The pavement is deteriorated on the edge of carriageway and mainly at the lowest points where water remains on the surface during rainfall.	Some roads have grass verges instead of ditches. The surface water is prevented from leaving the pavement and will ingress the road structure.	▪ Remove the verges and make ditches ▪ Deep drainage ▪ Edge drainage ▪ Surface water must be able to flow away as soon as possible.
	Drainage problems for a road located in low lying ground (bottom of a small valley)	The road floods when the snow melts and during heavy rainfalls. Permanent deformation problems in these sections. Differential frost heave problems	Due to topography problems it is not possible to redirect the water flow away from the vicinity of the road. Ground water table too close to the road structures	*Moraine* ▪ Infiltration wells/ditch ▪ Raise the grade line using coarse graded materials *Clay/silt or peat* ▪ Raise the grade line using coarse grade materials ▪ Infiltration structures do not work!
	Inadequate drainage in side sloping ground.	Rut deformation in upper wheel track in upper lane related to sloping ground.	High ground water table in the road cut lane. Too weak a road structure. Upper ditches are not cleared.	▪ Improve the drainage system by clearing the ditches. ▪ Deep drainage to lower the ground water table. ▪ Reinforcement of the road structure on the road cut side. ▪ Steel reinforcement.

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
Design related problems (continuation)	Drainage problems where bedrock surface is close to the road structures.	Water is not draining from the road structure and this leads to reduced bearing capacity. During the frost season ice forms on top of the bedrock blocking water flow and this causes uneven bumps to form in the road surface.	Bedrock blocks the water from flowing under the road. Water reaches frost susceptible material on top of the bedrock. Frost front reaches bedrock and starts to block ground water.	▪ Blast the bedrock to a depth of 1-2 m below the foundation level ▪ Soil replacement down to bedrock level using coarse aggregates ▪ Blast the bedrock under upper ditch ▪ Make deep drainage in upper slope to prevent water from ingressing the road structure ▪ Use many culverts ▪ Frost insulation. ▪ Remove bedrock/boulders that block the water flow.
	Drainage problems on flat ground.	The ditches or even the road floods during the period when snow melt or heavy rainfalls. Permanent deformation problems especially on road shoulders	Due to the flat terrain it is difficult to drain the water away from the vicinity of the road. High ground water table causes high moisture content in the road structure.	*Moraine:* ▪ Raise the grade line ▪ Replace road materials with materials not susceptible to water and frost. ▪ Stabilize water susceptible materials. ▪ Infiltration wells or ditch ▪ Long drainage ditches or deep drainage. *Clay/silt or peat* ▪ Infiltration is not possible ▪ Raise the carriage way grade line - (be aware of possible settlements) ▪ Long drainage ditch (surface or subdrain)

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
Other problems	Saturated layers due to bound layers in the construction. (Moisture trap)	Fast rutting and formation of alligator cracking in pavement after paving. Water squeezes out of cracked pavement during spring thaw and after rainfall.	Old and impermeable pavement is left below the unbound base course closer than 40 cm to the new pavement bottom. Water will be trapped between these pavement layers and material becomes saturated. Dynamic loads cause hydrostatic pressures that breaks the pavement.	▪ Check for existence of old pavement in the unbound layers with ground penetrating radar for instance ▪ Break the old pavement below or in the base if it is closer than 40 cm ▪ Mill through all the bound layers down to the bottom of the old pavement and mix these together with the gravel layer. Add bitumen to stabilise the material when milling.
	Erosion and surface slides in road cut slopes	Materials from the surface of outer slope are eroding and being carried down, blocking the ditch and raising the ground water level.	Too steep a slope. High ground water table and/or high ground water flow. Erosion susceptible materials in the slope	▪ Surface drains ▪ Ditch above the road cut slope to decrease the ground water table. ▪ Add Vegetation ▪ Cover the slope surface with coarse graded gravel or macadam. Geotextile between the subsoil and the coarse material.

Category	Problem description	How to recognise the problem	What causes the problem	Drainage solutions Proposal
	Drainage problems on gravel roads (drainage of the road structure of a gravel road)	The road surface loses its strength becomes plastic and the road is nearly impassable during the spring thaw period.	Caused by frost susceptible materials in the road structure. This is not a drainage problem, but drainage might reduce the problem.	▪ Clean the ditches often enough ▪ Deep drainage to lower the ground water table. ▪ Replace some of the road structure with coarse graded materials. ▪ Increase the road structure thickness by adding a new gravel base course and surface layer. ▪ Ensure that the road surface has a proper cross fall

Roadex Publications

ROADEX II

- ROADEX II - Focusing on Low Volume Roads in the Northern Periphery DVD
- User Perspective to ROADEX II Test Areas' Road Network Service Level
- Permanent deformation
- New material treatment techniques
- Managing spring thaw weakening on low volume roads
- Socio-economic impacts of road conditions on low volume roads
- Dealing with bearing capacity problems on low volume roads constructed on peat
- Drainage on low traffic volume roads
- Environmental guidelines
- Environmental guidelines, pocket book
- Road management policies for low volume roads – some proposals
- Structural Innovations
- Monitoring, communication and information systems & tools for focusing actions

ROADEX I

- Roadex Multi-media CD-ROM
- Road Condition Management of Low Traffic Volume Roads in the Northern Periphery
- Winter Maintenance Practice in the Northern Periphery,
- Generation of 'Snow Smoke' behind Heavy Vehicles

ROADEX II
NORTHERN PERIPHERY

Roadex II Lead Partner: The Highland Council, Transport, Environmental & Community Service, HQ, Glenurquhart Road, Inverness IV3 5NX Scotland.