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Postadr.: Postboks 8142 Dep
0033 Oslo
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Tittel

Performance Prediction Models for Flexible Pavements: A State-of-the-art Report

Utarbeidet av

Rabbira Garba Saba, Anders Huvstig, Gregers Hildebrand, Even Sund, Ragnar Evensen, Haraldur Sigursteinsson, Jesper Elsander

Dato:	Saksbehandler	Prosjektnr:
2006-12-11	Rabbira Garba Saba	601703
	Kontrollert av	**Antall sider og vedlegg:**
	Joralf Aurstad	50

Sammendrag

En av aktivitetene for forprosjektet til NordFoU prosjektet - "Pavement Performance Models" var gjennomgang og ekspertvurdering av eksisterende tilstandsutviklingsmodeller. Målet med å gjennomgå modellene var å finne ut styrkene og svakhetene av eksisterende modeller for å danne et grunnlag for en mer grundig vurdering, valg og forbedring av modellene. Undersøkelsen omfattet modeller som er i bruk i de nordiske landene samt noen relevante modeller fra de andre landene (europeiske land og USA). Denne statusrapporten viser resultatet fra gjennomgangen av modellene.

Summary

One of the activities of the preparatory phase of the NordFoU project - Pavement Performance Models was to conduct a review of available performance prediction models. The purpose of the review was to find out the strengths and the weaknesses of available models in order to provide basis for more detailed evaluations, selection and improvement of models. Accordingly, models that are in use in Nordic countries as well as relevant models from other countries (European countries and USA) were reviewed based on expert evaluations of the Nordic models and available literature. This state-of-the-art report contains the result of this review.

Emneord:

Performance, models, flexible pavements, performance indicators, network level, project level

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Summary

One of the activities of the preparatory phase of the NordFoU project – Pavement Performance Models was to conduct a review of available performance prediction models. The purpose of the review was to find out the strengths and the weaknesses of available models in order to provide basis for more detailed evaluations, selection and improvement of models. Accordingly, models that are in use in Nordic countries as well as relevant models from other countries were reviewed based on expert evaluations of the Nordic models and available literature. This state-of-the-art report contains the result of this review. The following conclusions were made based on the review.

1. Performance prediction models represent a key element of road infrastructure asset management systems or pavement management systems. Thus successful implementation of these systems depends heavily on the performance prediction model used as the accuracy of the predictions determines the reasonableness of the decisions.
2. Several pavement performance prediction models have been proposed over the years. Many of these models are developed for application in a particular region or country under specific traffic and climatic conditions. Therefore they can not be directly applied in other countries or conditions.
3. Although much research has been devoted to performance modelling of pavements, a comprehensive model that can predict pavement performance accurately has yet to be developed.
4. The available models can be broadly classified into three groups; empirical, mechanistic-empirical and subjective models. Various empirical models are proposed for application at network and project levels. The mechanistic-empirical models are often developed in connection to design systems and therefore have not been widely applied in pavement management systems (PMS), but have the potential to be applied at a network level. The subjective models are mostly developed for strategic (investment) planning at the network level.
5. Almost all Nordic countries use simple performance prediction models, based on linear extrapolation of historic data, in their pavement management systems. Denmark uses a slightly different approach in which pavement roughness is predicted as a function of pavement age using non-linear models (curves). Denmark and Sweden have implemented more advanced performance prediction models in their design systems. In Sweden research is underway to further develop the performance models in connection with development of a new design method known as “active design”.
6. The simple models currently in use in PMS in Nordic countries are not suitable for prediction of pavement condition over long periods of time. Further, they can not be used for evaluation of the effect of different maintenance measures and material qualities Thus there is a need for better performance prediction models for PMS applications.
7. As the current trend is to move from purely empirical design methods to mechanistic-empirical methods, it is important to further develop performance prediction models that are suitable for these methods. Furthermore there is a need to evaluate the possibility of using the mechanistic – empirical models for prediction of the condition of the road network.

Thus, the review showed that there is a need to develop improved models for use both at the network level and the project level. In order to develop such improved models, it is

recommended to take the following steps in the NordFoU project – Pavement Performance Models.

1. Pool available data and resources. Development of performance prediction models requires large amount of data on real pavements. There are test sections or reference sections in most of the Nordic countries for which data of various level of detail are available. It is therefore important to pool these data to obtain a good basis for improvement of performance models.
2. Use the available data from test sections, heavy vehicle simulator and other sources to evaluate existing performance prediction models.
3. Select suitable models based on the evaluation and identify areas that need improvement.
4. Improve the selected models especially with regard to climatic effects and studded tire use to make it suitable for Nordic conditions.
5. Implement the improved models in each country.
6. Agree on recommended test methods so that material evaluations can be conducted using the same procedure in each country.
7. Agree on a uniform procedure for traffic data collection and processing.

1 Introduction

The accurate prediction of pavement performance is important for efficient management of road infrastructure. At the network level, pavement performance prediction is essential for rational budget and resource allocation. At programming level, pavement performance prediction is needed for adequate activity planning and project prioritization while at project level it is needed for establishing and designing the necessary corrective actions such as maintenance and rehabilitation.
Several performance prediction models have been proposed over the years. The models vary greatly in their comprehensiveness, their ability to predict performance with reasonable accuracy, and input data requirement. Most of these models are empirical and were developed for use under particular traffic and climatic conditions.Few of the models are of mechanistic – empirical type in which some of the input parameters are calculated using mechanistic models.
This report gives brief review of the existing models, particularly those models that are being used or under development in the Nordic countries. The report forms part of the preparatory work for the NordFoU project on deterioration models for flexible pavements.

1.1 The NordFoU Project – Pavement Performance Models

NordFoU is a cooperation program for Nordic countries aimed at research and development in the road sector. The program was formally established in December 2004 by the road authorities of the Nordic countries to coordinate their research and development effort.
Four different research projects were initiated at the beginning under the cooperation program, one of which deals with development of deterioration/performance models for flexible pavements and was named NordFoU project – Pavement Performance Models. The main goal of the project is to develop a practical performance model for flexible road constructions based on already existing models. The project was started at the end of 2005 and it is planned to be conducted in two stages. The first stage, which is a preparatory stage, is planned to be completed in 2006 and aims to develop project plan and prepare a state-of- the-art review of available performance prediction models. The second stage is the main project, which is planned to run from 2007 – 2009. This report reviews available pavement performance models, with emphasis on those models that are in use in the Nordic countries.

1.2 Terminology

Some terms and expressions are often interchangeably used, in some cases with slightly different meanings. It is therefore considered necessary to clarify the meaning of the key terms and expressions employed in this report. These terms are defined in the following paragraph.

Pavement: The term pavement is used in this report to mean the whole road structure with all of its layers and not just the surfacing layer.

Flexible pavement: A pavement type in which bituminous mixtures are used as surfacing materials.

Pavement performance: Pavement performance is a measure of the in-service condition of the pavement. Performance is often expressed in two ways; the first is structural performance which is expressed in terms of distresses such as cracking and the second is functional performance expressed in terms of serviceability, which in turn might be function of distresses such as rutting and roughness. The term performance in this report refers to the general condition of the pavement, including its structural and functional condition, unless otherwise specified.

Pavement deterioration: Represents a negative change in performance or condition of the pavement, i.e, an increase in distresses or decrease in serviceability.

Rutting: Surface depression in the wheel path caused by combination of deformation in the pavement layers and studded tire wear.

Roughness: Longitudinal unevenness in the wheel path.

2 Performance Prediction Models and their Use in Road Infrastructure Management Systems

Preservation of road infrastructure asset requires a systematic approach involving condition assessment and performance modelling, program optimization and development of tactical and strategic plans. A very important part of such approach is the use of pavement performance models, which allow the forward prediction of future condition based on present condition under a defined range of future loading and maintenance scenarios. The successful implementation of road asset management systems or pavement management systems (PMS) is strongly dependent upon how well future pavement condition, as predicted by the performance prediction models of the system, agrees with observed behaviour and local engineering knowledge of the road network under consideration.

Pavement materials will deteriorate under the influence of loads and climatic effects. The stresses caused by heavy loads may result in microcracking in asphalt materials and may also cause permanent deformation in pavement layers. Skid resistance will be reduced as a result of changes in surface texture due to aggregate polishing or bleeding. Frost heave may cause cracking and deformation, while spring thaw can considerably reduce the permissible stresses in the unbound materials. With time, microcracking can develop into macrocracking, allowing water to penetrate into the pavement, and so on. Ideally a pavement performance model should capture this deterioration process in a comprehensive manner (considering all influencing factors). Unfortunately, this process of material deterioration is quite complex and difficult to model.

A large number of different pavement performance (or design) models are already available but, given the same input data, they tend to produce different output (predictions). Pavement performance models should be based on fundamentally correct standard engineering principles to be reliable and acceptable. It is also important that these models are easily adjustable in accordance to available historical data and the engineer's knowledge of local materials, environmental effects, construction and maintenance practices, etc.

In spite of an enormous effort that has been made in the pavement engineering field, it still is not possible to make accurate and precise prediction of pavement life (Molenaar 2003). This is due to the fact that it is very difficult to predict many of the factors that influence the pavement performance. Unusual hot summers, cold winters, wet springs etc can not be predicted. Traffic forecasts are mostly unreliable and there is a large variation in the characteristics of pavement materials and structures. The available performance prediction models have several limitations in that most of them involve large simplifications (e.g. in material behaviour), some of them contain input factors that are difficult to quantify and most are not comprehensive enough (do not consider all influencing factors). Figure 1 illustrates the complexity of the performance prediction problem.

2.1 Measures of Pavement Performance

Pavement performance have been expressed in terms of individual pavement distress (such as rutting, cracking etc), pavement condition index, which is often a composite measure involving both the functional and structural condition, and pavement serviceability index, which includes user's evaluation of the condition of the pavement.

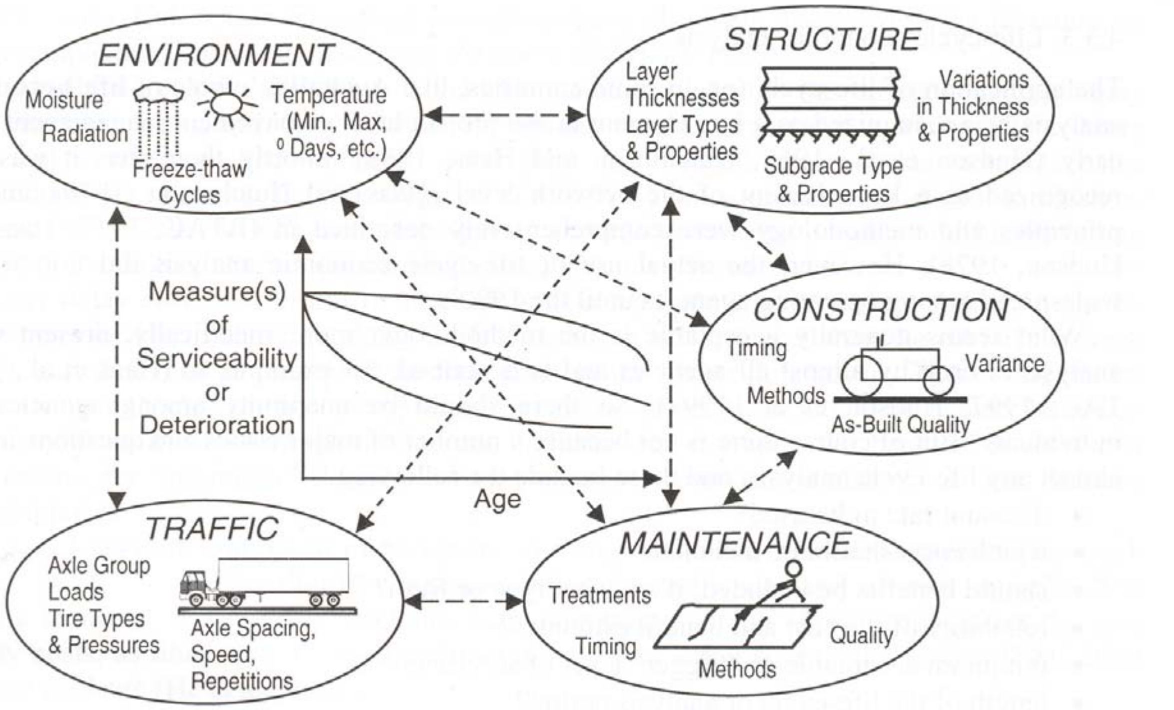

Figure 1: Factors affecting pavement performance (Haas 2003)

At the project level it might be appropriate to evaluate the distresses individually, but at the network level definition of some kind of composite measure of performance (performance indicator) is necessary. Currently, a project (COST Action 354) is under way at the European level to define performance measures and performance indicators (Litzka 2006).

2.2 Pavement Performance Prediction Models

One of the most profound challenges facing pavement managers and engineers has been the development of performance or deterioration prediction models. Several performance prediction models have been proposed over the years, some of which are simple and others more complex. Ralph Haas (2003) grouped the many performance prediction models into classes which indicate their basis as follows:

- Empirical, where certain measured or estimated variables such as deflection, accumulated traffic loads etc are related to loss of serviceability or some other measure(s) of deterioration and pavement age, usually through regression analysis.
- Mechanistic – empirical, where certain calculated responses, such as subgrade strain, pavement layer stresses and strains etc, together with other variables such as accumulated traffic loads, are related to loss of serviceability or some other measure(s) of deterioration through regression analysis or a model which is calibrated (i.e. the coefficients are determined) by regression analysis.
- Subjective, experience based where serviceability loss or other measure(s) of deterioration vs. age are estimated, for different combination of variables, using Markovian transition process models, Bayesian models etc.

2.2.1 Empirical Models

Various equations, mostly based on regression analysis, were developed for predicting pavement performance. The usefulness of these empirical equations is limited by the scope of the database that was used in their development. These kinds of regression equations are valid only under certain conditions and should not be applied when the actual conditions are different. One of the best known examples of the empirical models is the HDM – 4 developed by the World Bank.
The World Bank developed the Highway Design and Maintenance Standards Model (HDM-III) over two decades ago for use in infrastructure investment planning in developing countries. However in recent years some industrialised countries showed interest in the model and this led to further development of the model. In order to extend the scope of HDM-III and include additional capabilities such as models for traffic congestion, cold climate effects, road safety and environmental effects, the International Study of Highway Development and Management (ISOHDM) was conducted. This project produced the Highway development and Management Tool, HDM-4 (Kerali 2000). The HDM-4 has applications at the strategic, program and project levels and it includes deterioration models for various types of distresses. For instance the roughness model is described as follows:

$$\Delta RI = K_{gp}\left[\Delta RI_s + \Delta RI_c + \Delta RI_r + \Delta RI_t\right] + \Delta RI_e \qquad (1)$$

Where:

ΔRI =	total incremental change in roughness during the analysis year
K_{gp} =	calibration factor for roughness progression
ΔRI_s =	incremental change in roughness due to structural deterioration, which is a function of pavement age, number of equivalent standard axles and structural number of the pavement
ΔRI_c=	incremental change in roughness due to cracking, which is proportional to the incremental change in area of total cracking during the analysis year (% of total carriageway area)
ΔRI_r =	incremental change in roughness due to rutting, which is proportional to the incremental change in standard deviation of rut depth during the analysis year. The rut depth is the sum of four components: initial densification, structural deformation, plastic deformation, and wear from studded tyres.
ΔRI_t=	incremental change in roughness due to potholing. The potholing effect depends on the number of vehicles that actually hit the potholes, which in turn depends on the traffic volume and the freedom to manoeuvre.
ΔRI_e=	incremental change in roughness due to the environment. This component of roughness is due to factors which include temperature and moisture fluctuations and also foundation movements (e.g. subsidence)

The strength of bituminous pavements is characterized by the adjusted structural number, SNP. To take the effect of seasonal variations into account the average annual SNP is derived from SNP in dry conditions and SNP in wet conditions and the length of the dry and wet seasons. The effect of drainage on SNP is modelled through change in drainage factor, which varies from 1 (excellent) to 5 (very poor). The effect of construction quality is taken into account through a factor termed construction defects indicator.

Crack initiation is modelled using the following equation:

$$ICA = K_{cia}\left\{CDS^2 a_0 \exp\left[a_1 SNP + a_2\left(YE4/SN^2\right)\right] + CRT\right\} \quad (2)$$

Where:

- ICA = time to crack initiation in years
- CDS = construction defects indicator for bituminous surfacing
- SNP = structural number of the pavement
- YE4 = annual number of ESALs in millions/lane
- K_{cia} = calibration factor for crack initiation
- CRT = crack retardation time due to maintenance
- a_0, a_1, a_2 are calibration parameters

The model for plastic deformation is expressed as follows:

$$\Delta RDPD = K_{rpd} CDS^3 a_0 YE4 Sh^{a1} HS^{a2} \quad (3)$$

Where:

- ΔRPDP= incremental increase in plastic deformation in analysis year, in mm
- CDS= construction defects indicator for bituminous surfacings
- Sh= Speed of heavy vehicles in km/h
- HS= total thickness of bituminous surfacing in mm
- K_{rpd}= calibration factor

Another example of empirical performance prediction model is the serviceability equation developed from the AASHO road test and used for many years in the earlier AASHTO design guides. The present serviceability index (PSI) for flexible pavements was expressed as follows (Huang 1993):

$$PSI = A_0 + A_1 \log\left(1 + SV\right) + A_2 (RD)^2 + B_1\sqrt{C + P} \quad (4)$$

Where:

- SV = mean slope variance
- RD= mean rut depth
- C= cracking (linear feet per 1000 ft^2)
- P= patching (ft^2/1000 ft^2)

A_0, A_1, A_2, and B_1 are coefficients to be determined by linear multiple regression. Equation 4 is not a performance model in itself but it is an expression of the relationship between PSI and distresses. The PSI was used in the performance equation which predicts the allowable number of axle loads to failure i.e., reduction of PSI to terminal serviceability.

Prozzi and Madanat (2004) proposed a more sophisticated statistical performance prediction model based on AASHO road test data and field data from the Minnesota Road Research Project (MnRoad). The proposed model predicts roughness based on layer thicknesses, traffic increment, and frost gradient.

The European project, COST 324 Long Term Performance of Road Pavements, reviewed performance prediction models that were in use in 11 participating countries. These countries were Austria, Belgium, Switzerland, Denmark, Spain, Finland, France, United Kingdom, Greece, Hungary, Ireland, Netherlands, Portugal, Sweden and Slovenia. Most of the countries have developed performance models for the various performance indicators such as longitudinal profile, transverse profile, surface cracking, structural cracking, structural adequacy (deflection), surface defects and skid resistance. Some of the countries use a composite index that combines the various indicators. Detailed list of all the models can be found in the final report of COST 324 (European commission, 1997). The majority of these models are empirical and are mostly based on one independent variable such as the number of repetitions of load or age. The conclusion of the project was that the existing performance prediction models are not suitable for Europe- wide application and development of new performance models was recommended.

2.2.2 Mechanistic- empirical models

In the mechanistic - empirical models, calculated response variables such as tensile strain at the bottom of asphalt layer and vertical strain at the top of subgrade are used in addition to other parameters such as traffic loading to predict performance of the pavement structure. The performance is often expressed in terms of the individual distresses such as fatigue cracking, rut depth etc. The responses, i.e., the strains and the stresses resulting from axle loading are calculated using linear elastic multilayer theory, or, in some cases, finite element method. The material properties, such as the elastic moduli for the various layer materials, are taken into account in the response calculation. The environmental effects, such as the effects of temperature and moisture, can also be taken into account through their effect on the material properties.

Performance prediction models incorporated into the 2002 mechanistic – empirical design guide, developed in USA under the National Cooperative Highway Research Program (NCHRP) 1- 37A, are typical examples of this group of models. The pavement performance measures considered in the guide include permanent deformation (rutting), fatigue cracking (both bottom-up and top-down), thermal cracking and smoothness (International roughness index, IRI). Pavement response is calculated using either the elastic multilayer theory or the finite element method.

The design procedure of the 2002 mechanistic – empirical design guide involves analysis of trial designs to ensure that the designs satisfy user defined performance requirements. The trial design is analyzed for adequacy by dividing the target design life into shorter design analysis periods or increments beginning with traffic opening month. Within each increment all the factors that affect pavement performance/damage, including traffic levels, asphalt concrete modulus, base and subbase moduli, and subgrade modulus are held constant. The critical stress and/or strain values are converted to incremental distresses. Rutting is predicted in absolute terms, i.e., the incremental rut depth calculated for each analysis period is accumulated to obtain the total rut depth. Cracking distress is predicted in terms of a damage index, which is a mechanistic parameter representing load associated damage within the pavement structure. The incremental damage is accumulated for each analysis period using Miner’s law. The cumulative damage is converted to physical cracking using calibrated models that relate the cumulative damage to observable distresses. Calibrated distress prediction models were developed using the LTPP database and other long term pavement performance data. The model equations implemented in the design guide are as follows.

The overall permanent deformation for a given season is the sum of permanent deformation for each individual layer and is mathematically expressed as:

$$RD = \sum_{i=1}^{nsublayers} \varepsilon^i_p h_i \tag{5}$$

Where:

RD	= pavement permanent deformation
Nsublayers	= number of sublayers
ε^i_p	= total plastic strain in sublayer i
h_i	= thickness of sublayer i

The relationship used to predict rutting in asphalt mixtures is based upon a field calibrated statistical analysis of laboratory repeated load permanent deformation tests. The selected laboratory model is of the form:

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 T^{a_2} N^{a_3} \tag{6}$$

Where:

ε_p	= accumulated plastic strain at N repetitions of load (in/in)
ε_r	= resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in/in)
T	= temperature (Degree F)
N	= number of load repetitions
a_1, a_2, a_3	= non-linear regression coefficients

The final laboratory expression that was initially selected for calibration had coefficients $a_1 = 10^{-3.15552}$, $a_2 = 1.734$, and $a_3 = 0.39937$. Field calibration factors β_{ri} were added to ascertain field calibration and the final asphalt rutting model has the following form:

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} a_1 T^{\beta_{r2} a_2} N^{\beta_{r3} a_3} \tag{7}$$

The model for permanent deformation in unbound granular base is expressed as follows:

$$\delta_a(N) = \beta_{GB} \left(\frac{\varepsilon_0}{\varepsilon_r} \right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h \tag{8}$$

Where:

$\delta_a(N)$	= permanent deformation for a layer/sublayer (in)
N	= number of traffic repetitions
ε_0, β, ρ	= material parameters
ε_r	= resilient strain imposed in a laboratory test to obtain the material properties listed above (in/in)
ε_v	= average vertical resilient strain in the layer/sublayer as obtained from primary response model (in/in)
h	= thickness of the layer/sublayer (in)
β_{GB}	= calibration factor

The model form given in equation 8 is also used for calculation of permanent deformation in all subgrade soils. The parameters ($\varepsilon_0/\varepsilon_r$), ρ, and β are calculated using empirical equations. It has to be noted that according to this procedure permanent deformation for the various layers are calculated separately. This means that calibration of the models requires trenching studies to obtain field data on deformation in the various layers. Available information indicates that the deformation model, particularly that for unbound layers, is being revised.

With regard to fatigue damage, the approach utilized in the design guide models both the bottom-up and top-down cracking. The approach is based on calculating the fatigue damage at the surface for the top-down cracking and at the bottom of each asphalt layer for bottom-up cracking. Estimation of fatigue damage is done according to Miner's law, which can be expressed as follows:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \qquad (9)$$

Where:

D	= damage
T	= total number of periods
n_i	= actual traffic for period i
N_i	= traffic allowed under conditions prevailing in i

The relationship used in the design guide for the prediction of the number of repetitions to fatigue cracking is expressed as follows:

$$N_f = C\beta_{f1}k_1\left(\frac{1}{\varepsilon_t}\right)^{\beta_{f2}k_2}\left(\frac{1}{E}\right)^{\beta_{f3}k_3} \qquad (10)$$

Where:

N_f	= the number of repetitions to fatigue cracking
ε_t	= tensile strain at the critical location
E	= stiffness of the material
k_1, k_2, k_3	= laboratory regression coefficients
β_{f1}, β_{f2}, β_{f3}	= calibration parameters
C	= laboratory to field adjustment factors

Transfer functions are used to calculate fatigue cracking as a percent total lane area from the fatigue damage.

The thermal cracking model implemented in the design guide is based on thermal cracking model developed under the Strategic Highway Research Program (SHRP). The model is expressed as follows:

$$C_f = \beta_1 N\left(\frac{\log C / h_{ac}}{\sigma}\right) \qquad (11)$$

Where:

C_f	= observed amount of thermal cracking (crack frequency)

β_1 = regression coefficient determined through field calibration
N(z) = standard normal distribution evaluated at z
σ = standard deviation of the log of the depth of cracks in the pavement
C = crack depth
h_{ac} = thickness of asphalt layer

The amount of crack propagation induced by a given thermal cooling cycle is predicted using the Paris law of crack propagation expressed as follows:

$$\Delta C = A \Delta K^n \tag{12}$$

Where
- ΔC = change in the crack depth due to a cooling cycle
- ΔK = change in stress intensity factor due to a cooling cycle
- A, n = fracture parameters for the asphalt mixture

The parameters A and n are calculated from creep compliance curve using the principles of visco-elasticity.

The roughness (or smoothness) of flexible pavements is dependent on other distress types such as rutting, variance of rut depth, fatigue cracking, etc. The international roughness index (IRI) is used as a measure of smoothness of flexible pavements in the design guide. The models utilized in the design guide for prediction of IRI are dependent on the base type. For unbound aggregate bases and subbases, the model expressed in equation 13 below is used.

$$IRI = IRI_0 + 0.0463\left[SF\left(e^{\frac{age}{20}} - 1\right)\right] + 0.00119(TC_L)_T + 0.1834(COV_{RD}) + 0.00384(FC)_T + 0.00736(BC)_T + 0.00115(LC_{SNWP})_{MH} \tag{13}$$

Where:
- IRI = IRI at any given time, m/km
- IRI_0 = initial IRI, m/km
- SF = site factor
- $e^{age/20} - 1$ = age term, where age is expressed in years
- COV_{RD} = coefficient of variation of the rut depths, % (assumed to be 20%)
- $(TC_L)_T$ = total length of transverse cracks (low, medium, and high severity levels), m/km
- $(FC)_T$ = fatigue cracking in wheel path, percent total area
- $(BC)_T$ = area of block cracking as a percent of total lane area (user input)
- $(LC_{SNWP})_{MH}$ = length of moderate and high severity sealed longitudinal cracks outside wheel path, m/km (user input)

The site factor is expressed as:

$$SF = \left[\frac{(R_{SD})(P_{.075}+1)(PI)}{2*10^4}\right] + \left[\frac{\ln(FI+1)(P_{02}+1)[\ln(R_m+1)]}{10}\right] \tag{14}$$

Where:
- R_{SD} = standard deviation of the monthly rainfall, mm

$P_{.075}$ = percent passing the 0.075 mm sieve
PI = percent plasticity index of the soil
FI = average annual freezing index, ^{0}C-days
P_{02} = percent passing the 0.02 mm sieve
R_m = average annual rainfall, mm

For asphalt treated bases, the IRI is expressed as follows:

$$IRI = IRI_0 + 0.0099947(age) + 0.0005183(FI) + 0.00235(FC)_T + 18.36\left[\frac{1}{(TC_S)_H}\right] + 0.9694(P)_H \tag{15}$$

Where:
$(TC_S)_H$ = average spacing of high severity transverse cracks, m (estimated from thermal cracking model)
$(P)_H$ = area of high severity patches, percent of total lane area (user input)
All other variables are as previously defined.

Another approach to the development of mechanistic-empirical performance prediction models is that pioneered by Danish researchers (Ullidtz 2002, Busch et al 2005, Hildebrand 2006). This approach involves computer simulation of pavement performance. The Mathematical Model of Pavement Performance (MMOPP) was developed based on this approach. MMOPP is capable of predicting longitudinal roughness, rutting and fatigue cracking of a pavement consisting of a bitumen or cement bound layer, a granular base and subbase layer and subgrade. To simulate gradual deterioration over time, MMOPP makes use of an incremental-recursive procedure, where the output from one time increment (one season) is used, recursively, as an input for the next time increment.
The model considers the variation of pavement layer thickness, elastic stiffness, plastic parameters and dynamic load variations along the length of the road. A pavement section is divided into short lengths of 0.3 meters in which the aforementioned parameters are varied. Pavement response is calculated using linear elastic theory. Seasonal changes are considered in MMOPP by using time increments of one season. For each section the effect of the loading is determined in terms of permanent deformation, crack initiation and fatigue induced decrease in asphalt stiffness. Roughness in terms of slope variance is calculated using the permanent deformation. This procedure is repeated with output from one step used as input in the next step for a predefined period or until a certain level of deterioration has been reached. MMOPP has been calibrated using data from AASHO road test, from full scale accelerated pavement testing and against general experience with pavements in Denmark. More detailed description of the MMOPP is given in the next chapter.

A Model similar to MOPP, referred to as Whole-life Pavement Performance Model (WLPPM) was developed in the UK (Collop and Cebon, 1995). In WLPPM a vehicle simulation is used to generate dynamic tyre forces that are a function of distance along the road. These dynamic tyre forces are then combined with appropriate pavement primary response influence functions (stress, strain and displacement) to give primary response histories at regularly spaced points along the pavement. The primary response histories are then transformed into pavement damage (fatigue and permanent deformation) using an appropriate damage model. The result is an increment of damage at each point along the pavement due to a single vehicle pass. The pavement surface profile is then updated to reflect permanent deformation damage, and the layer material parameters are changed to reflect fatigue damage.

D'Apuzzo et al (2004) developed a model for prediction of roughness progression of asphalt pavements using a modelling approach similar to that used in MMOPP and WLPPM. In this model the road length is discretized in a number of 0.3 – 0.25m long sub-sections. Different layer thicknesses and mix properties are assigned to each section by means of autoregressive time series. Further more dynamic loads are assumed to be applied to the surface at the middle point of each sub-section. Primary response due to dynamic traffic loads and the subsequent permanent deformations of pavement layers and subgrade are evaluated for each section and for each calculation step. The road pavement profile is updated using the total permanent displacement of each sub-section and this process continues until the end of the analysis period.

Information regarding the extent to which these last two models are validated and applied in practice is not readily available.

Mechanistic models for prediction of rutting in granular and bituminous bound materials were also developed under the European SAMARIS project, which was completed in 2006 (Hornych and El Abd 2006, Blab et al 2005). Two models were developed for permanent deformation in unbound granular materials: an empirical model and an elasto-plastic model. The elasto-plastic model was implemented in a finite-element code developed by LCPC, France. The predictions of these models were compared to measured data from LCPC's testing facility and reasonable agreement has been reported. These models are also being evaluated under the "active design" project in Sweden, the result of which would be interesting for the NordFoU project. The proposed permanent deformation model for bituminous materials was based on linear visco-elasticity.

2.2.3 Probabilistic Models

The deterioration of pavements is affected by several factors some of which are difficult to observe. Traffic load and environmental conditions change over time and are difficult to predict. This makes the performance or deterioration of pavements to vary greatly showing uncertain or random characteristics. Furthermore uncertainty can arise from the inspection or measurement process and from inability to quantify the factors that affect the deterioration process, and to model the true deterioration process of the materials. Thus pavement deterioration process shows stochastic characteristics.

Probabilistic models attempt to tackle the stochastic characteristics of the pavement deterioration process. Most of the proposed probabilistic models are based on Markov process modelling. A Markov chain is a special type discrete-time stochastic process where the state of the system (for example pavement condition) X_{t+1} at time t + 1 depends on the state of the system X_t at some previous time t but does not depend on how the state of the system X_t was obtained. In mathematical form this can be expressed as:

$$P(X_{t+1} = j | X_t = i) \tag{16}$$

Where P is the probability of the state at time t + 1 being j given that the state at time t was i, assuming that the probability is independent of time. This assumption is known as the stationary assumption and it represents a major limitation for most of the probabilistic models because it implies that the rate of deterioration of pavements is time independent. Few models use so called non-homogenous (time dependent) Markov chains to overcome this limitation (Li 2005). Some of the probabilistic models are developed based on econometric methods. More detailed review of the probabilistic models is given by Li (Li 2005).

One of the major challenges facing existing probabilistic models is the difficulty in establishing the Transition Probability Matrices (TPMs). A TPM is a square *sxs* matrix where s is the number of possible states in the system. The matrix contains the probabilities of transitioning from state i to state j, i.e, the probability of something being in one state and then changing into another state over a fixed time interval. The TPM can be established using historical data or subjective opinions of experienced engineers through individual interviews and questionnaires, which takes considerable time and expenses.

An example of such models is the Highway Investment Planning System (HIPS) used widely in Finland and Norway. These models usually group the pavements into families (group of pavement sections with similar characteristics) and as such are suited for network-level pavement management systems or strategic investment analysis for the road network. However, they are not suitable for project-level analysis.

2.3 The Need for Performance Prediction in Road Infrastructure Management Systems

The accurate prediction of pavement performance is very important for efficient management of the road infrastructure. By reducing the prediction error of pavement deterioration agencies can obtain significant budget savings through timely intervention and accurate planning (Prozzi and Madanat 2004).

Pavement performance prediction has been the key component of pavement management systems (PMS). A pavement management system is considered as a programming tool that collects and monitors information on current pavement, forecasts future conditions, and evaluates and prioritizes alternative reconstruction, rehabilitation and maintenance strategies to achieve steady state of system preservation at a predetermined level of performance. Effective implementation and utilization of pavement management systems in generating and evaluating various alternative strategies based on engineering and economic principles is largely dependent on the ability to predict the future condition of the pavement. The current trend is to integrate pavement management systems, bridge management systems, and other systems related to road infrastructure management into comprehensive road asset management systems. Asset management goes beyond the traditional management practice of examining singular systems within the road network, i.e., pavements, bridge, etc., and looks at the universal system of a network of roads and all of its components to allow comprehensive management of limited resources. Through proper asset management, governments can improve program and infrastructure quality, increase information accessibility and use, enhance and sharpen decision making, make more effective investments and decrease overall costs (OECD quoted in US Department of Transportation 1999). Figure 2 illustrates the components of a generic asset management system. It can be understood that performance modelling and performance monitoring represent key aspects of such a system.

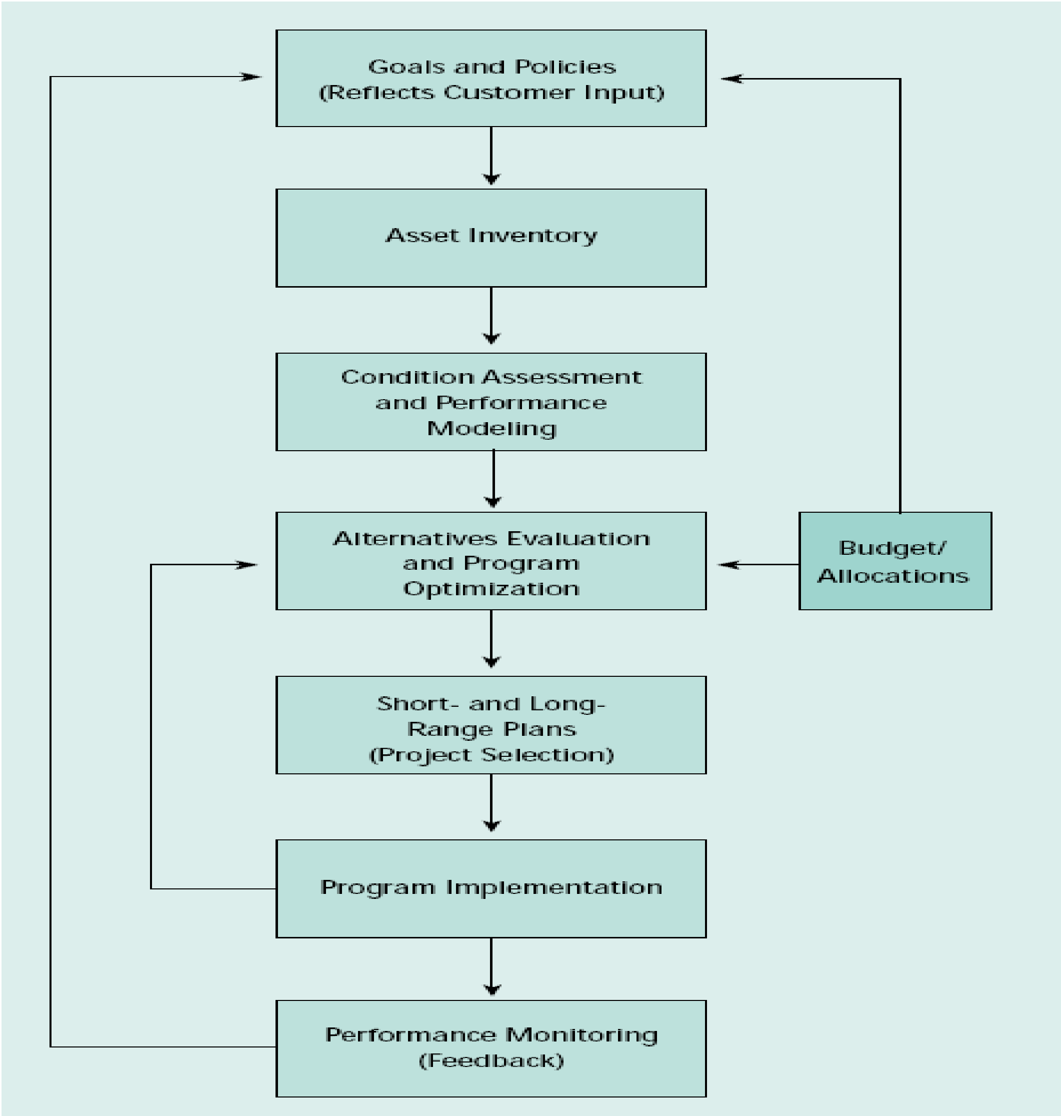

Figure 2: Generic asset management system (US Department of Transportation 1999)

Performance prediction plays a key role not only in pavement management system (or asset management system) but also in pavement structural design. Pavement design involves choice of materials and thickness for the various layers based on sound engineering and economic principles. This requires comparison of alternative materials and thicknesses, which depends heavily on the ability to predict the performance of the alternative material and thickness combinations. In the past pavement design has relied on empiricism and experience. In recent years, however, mechanistic – empirical design methods, which are based on more fundamental engineering principles, are being applied in various countries. Performance prediction models form the cornerstone of these mechanistic – empirical design procedures. Figure 3 shows the components of the mechanistic - empirical pavement design guide.

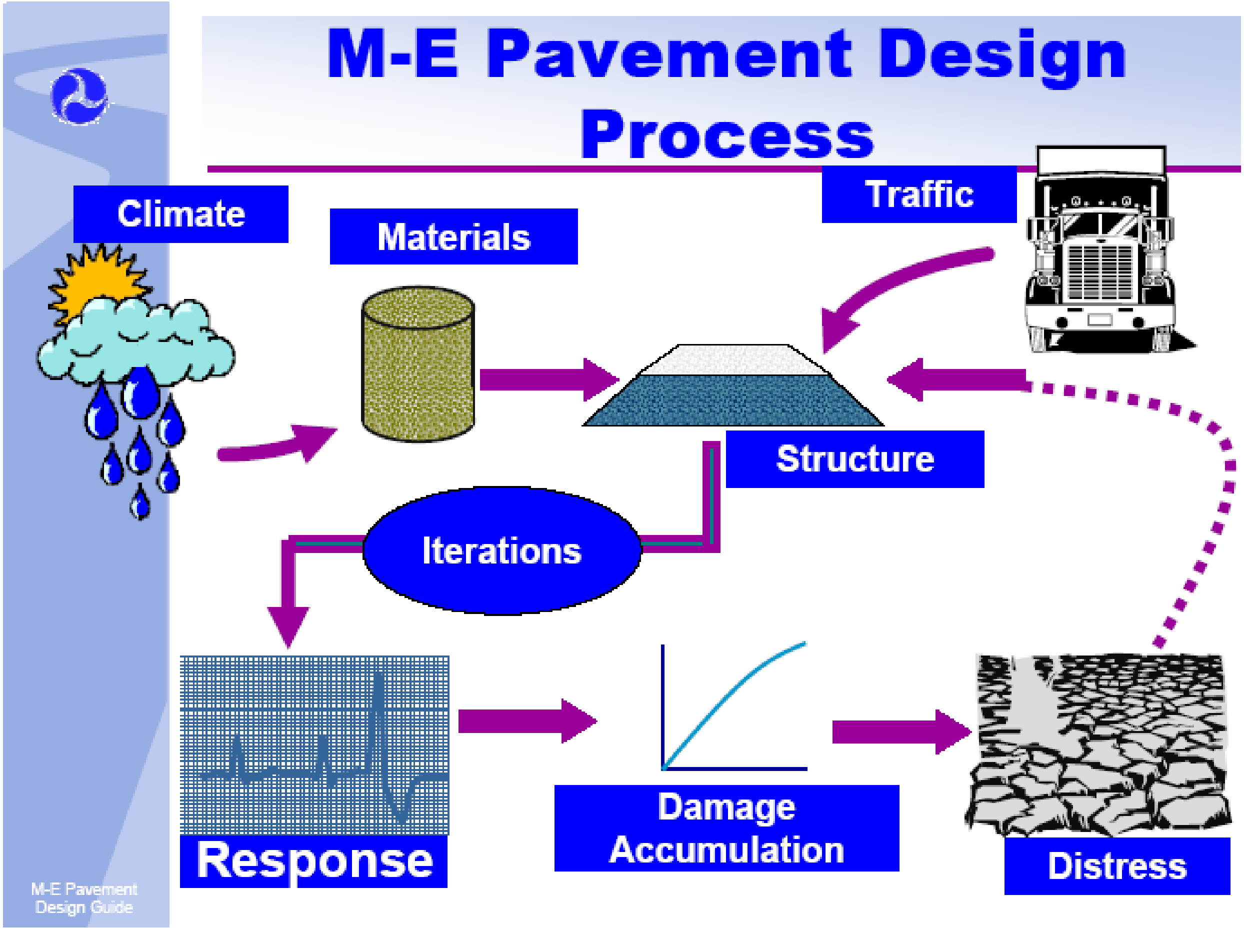

Figure 3: Mechanistic – Empirical pavement design process (M-E design guide web site)

3 State-of-the-practice: Application of Performance Prediction Models in the Nordic Countries

All Nordic countries have been using one or another form of performance prediction models. A questionnaire was sent to all Nordic countries to collect preliminary information on the use of performance prediction models towards the end of 2005. The response to the questionnaire indicated that all of the countries have implemented simple performance/deterioration prediction models, which are based on linear extrapolation of historic data in their Pavement Management Systems (PMS). However, Sweden and Denmark have been developing and implementing relatively advanced mechanistic – empirical type performance prediction models in their road design systems. Also the response from Finland indicated that a statistical deterioration model has been implemented in the Finnish pavement design system.

3.1 Response to the questionnaire

Responses of the Nordic countries to the questionnaire is summarized and shown in table 1. Only the responses to the main questions are provided in the table.

Table 1: Summary of responses to the questionnaire

Q: Do you use deterioration models? In which of the systems have you implemented the deterioration models?	
Country	**Response**
Denmark	Yes, in PMS and pavement design systems
Finland	Yes, in PMS, pavement design systems and road asset management systems
Iceland	Yes, in PMS and road asset management system (under implementation)
Norway	Yes, in PMS and road asset management system (under implementation)
Sweden	Yes, in PMS and pavement design systems

Table 1 cont.

Q: Describe the deterioration models you use briefly	
Denmark	Statistical deterioration models for condition index (used for local roads) and IRI and bearing capacity (for state and county roads) are implemented in PMS. Mathematical model of pavement performance (a simulation model) is implemented in pavement design system.
Finland	At network level, a probabilistic model for rutting, IRI, sum of defects, and bearing capacity is used. At the program level simple extrapolation is used for rutting, IRI and sum of defects based on the last measurement.
Iceland	RoSy PM system based on visual inspection.
Norway	A simple linear extrapolation based on registered data is used in PMS. Performance models of USA's MEPDG were recently calibrated for Norwegian conditions.
Sweden	In PMS, simple statistical model is applied. In the current design system for flexible pavements modified Kingham's criteria (fatigue damage) is used. A new system known as Active Design, which involves on the site calculation of future rutting in bound and unbound material is under implementation on five road building projects in western Sweden. The deterioration models used in this system come from USA's new design guide, Dresden technical university (Germany) and LCPC (France).
Q: Do the deterioration models you use consider effect of climate change on pavement performance? How?	
Denmark	Yes, the variation of layer moduli is described through seasonal factors and for asphalt materials the damage rate is determined as a function of temperature.
Finland	Yes, the empirical statistical model includes both traffic loads and climate, but it is impossible to separate them.
Iceland	No
Norway	Yes, MEPDG (USA's), which is being implemented in road asset management system, has a climate model.
Sweden	Yes, we have a frost heave calculation model based on temperature data. The model uses thermodynamics.

Table 1 cont.

Q: If stresses and strains are used as input parameters in your deterioration models, which methods are used to calculate the stresses and strains?	
Denmark	The Method of Equivalent Thicknesses is used in PMS. In MMOPP, the Method of equivalent thicknesses as well as elastic multi-layer theory is used.
Finland	The deterioration models use other parameters.
Iceland	FWD measurements are used.
Norway	Elastic multi-layer theory is implemented in MEPDG (which is being calibrated).
Sweden	Linear elastic multi-layer theory and finite element method.
Q: Is there any ongoing research and development work in your country with the aim of developing deterioration models for flexible pavements?	
Denmark	Yes
Finland	No
Iceland	No
Norway	No
Sweden	Yes
Q: Do you have test sections that are built to study pavement deterioration and whose conditions are regularly observed? To what extent were the SHRP/LTPP sections monitored in your country?	
Denmark	No, but 7 SHRP/LTPP sections monitored from 1993 – 2002.
Finland	Yes, but with no detailed observation. SHRP/LTPP monitored until 1999.
Iceland	No, No SHRP/LTPP sections
Norway	No, Some monitoring of SHRP/LTPP sections, data has yet to be found.
Sweden	Yes, 350 sections of various age + 5 sections built in connection with active design. SHRP/LTPP sections were observed to a large extent.

3.2 Evaluation of Models Currently Used in the Nordic Countries

Under the NordFoU project, all countries were requested to objectively evaluate the models and methods they are using to predict the performance/deterioration of pavements. The objective of the evaluation was to find out the strengths and weaknesses of existing models in order to determine the areas that need improvement. This section presents results of these evaluations for those countries that conducted it.

3.2.1 Denmark

Denmark uses deterioration models both in pavement management systems (PMS) and in pavement design system. The following review of the Danish models is provided by Gregers Hildebrand.
The VEJOPS pavement management system is used by the Danish Road Directorate for planning and optimisation of pavement maintenance at municipal roads. VEJOPS applies a rating called Condition index to describe the condition of a pavement section. The condition index is determined by the expression:
:

$$ConditionIndex = \sum \frac{Extent(\%)}{100}.severity.factor \qquad (17)$$

Where the extent of a certain type of deficiency is given in percent of area or length, severity expresses the severity of the specific deficiency, and factor is a calibration constant.

Advantages of this model are:

- The model is simple and can be implemented in any PMS that is based on visual condition survey
- Model calculations can be conducted by hand or spreadsheet
- Simple input data
- Allows local calibration

The disadvantages of this model are:

- The model is primarily adopted to the visual survey form of the Danish road directorate
- Difficult to directly apply to other conditions (climate, road materials, loadings)

Evenness
In the Belman pavement management system used for the major Danish roads, two different models are used for predicting evenness: one for increasing evenness and the other for decreasing, evenness.

The model for decreasing evenness describes how the evenness worsens year after year when no maintenance action like new wearing course or strengthening is invoked. The decreasing evenness is usually attributed to influence from climate and traffic loading. The model is illustrated in figure 4 (Title: Decreasing evenness; x-axis: age of pavement; y-axis: IRI).

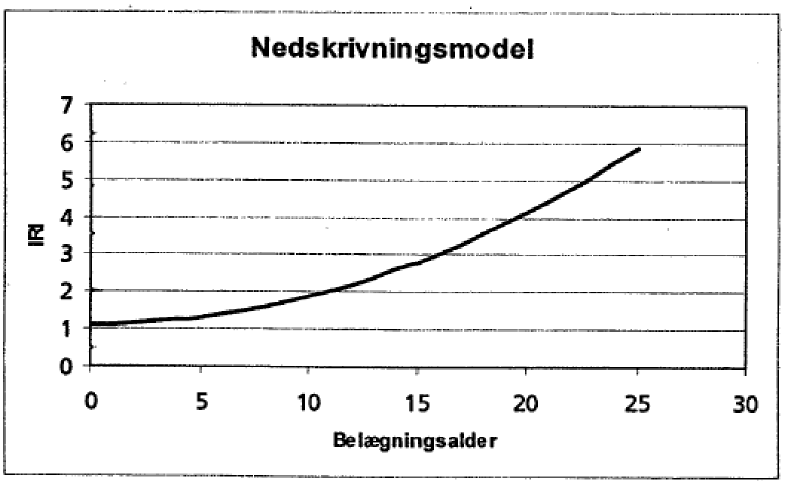

Figure 4: Illustration of the model for decreasing evenness

The advantages of this model are:
- Very simple
- Easy to implement
- Intuitively, the model appears to be correct

The disadvantages of the model are:
- Purely empirical and does not explain the relationship to parameters such as traffic, climate, and road materials, which are decisive for IRI development
- The model is applicable only for the road conditions for which it is calibrated (big Danish roads)

If a road section is improved with a new wearing course or a strengthening layer the evenness will be improved and IRI decreases. The evenness of the improved pavement depends on the evenness of the old pavement: the better the foundation (i.e., more even) the better the evenness of the new pavement. The model for increasing evenness is illustrated in figure 5 (Title: Improving evenness; x-axis: evenness of existing pavement in IRI; y-axis: evenness of new pavement in IRI):

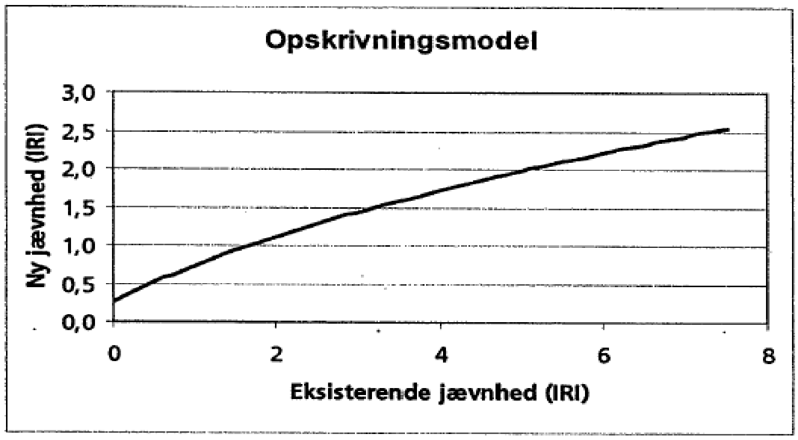

Figure 5: Illustration of the model for increasing evenness

The advantage of this model is that it is simple and easy to implement.
The disadvantages are:

- Difficult to evaluate intuitively the correctness of the model
- Purely empirical and does not explain the relationship to parameters such as traffic, climate, and road materials, which are decisive for IRI development
- The model is calibrated only against Danish major roads

With regard to bearing capacity, every road section in the Belman pavement management system has a structural lifetime (in years) and a strengthening need (in mm asphalt).These key numbers are determined based on falling weight deflectometer testing in the field. Structural lifetime and strengthening need are both used to describe the development of pavement bearing capacity.

Decreasing bearing capacity is forecast using a model in which the increasing strengthening need is determined as a function of structural lifetime and traffic load. Improving bearing capacity is forecast using a pavement catalogue which provides expected lifetimes for different types of new wearing courses and strengthening solutions.

The advantages of the bearing capacity prediction models are:

- Simple models, which are intuitively correct
- Easy to implement
- The model for decreasing bearing capacity is based on physical measurements and analytical-empirical calculations

The disadvantages of the models are:

- The model for decreasing bearing capacity requires falling weight deflectometer measurements as an input
- The correctness of the pavement catalogue and whether it is representative of pavements in practice is questionable
- Difficult to apply the model in other countries/conditions
- There is also a question on whether a given pavement actually carries the traffic given by the catalogue

Denmark has also implemented a model in its design system known as Mathematical Model of Pavement Performance (MMOPP). In this system structural deterioration of the asphalt layer (cracking) is given by the following model:

$$E_{after} = E_{before}\left(1-0{,}5\left(\frac{\varepsilon_{calc}}{\varepsilon_{allowble,1million}(VB/10\%)}\right)^{n}\frac{dN}{K_{temperature}CP_{factor}}\right) \tag{18}$$

Where:

ε_{calc}	= calculated strain at bottom of the asphalt layer
$\varepsilon_{allowble,\ 1\ milliom}$ passes	= allowable strain at bottom of the asphalt layer at 10^6 load
VB	= binder content (in volume percent)
n	= exponent for the fatigue model – Kirk's exponent of 5,62 is applied
dN	= number of passes in the period

$K_{temperature}$	= temperature correction, which makes the material less susceptible to cracking at high temperature
CP_{factor}	= calibration constant
E_{after}	= Young's modulus for the deteriorated asphalt layer
E_{before}	= Young's modulus for perfectly new material

The advantages of this model are:

- Fundamentally (physically) based model, which is relatively simple to implement
- Relatively simple to determine the necessary input parameters

The disadvantage of this model is related to the question of how to determine the factors K and CP.

Permanent deformation is modelled as a two phase process in MMOPP. The following model is used for phase 1 (primary creep, decreasing strain rate).

$$\varepsilon_p = AN^B\left(\frac{\sigma_1}{\sigma'}\right)^C \qquad \text{(for } \varepsilon_p < \varepsilon_0\text{)} \qquad (19)$$

For phase 2 (secondary creep, constant strain rate), the model is expressed as follows:

$$\varepsilon_p = \varepsilon_0 + (N - N_0)A^{\frac{1}{B}}B\varepsilon_0^{1-\frac{1}{B}}\left(\frac{\sigma_1}{\sigma'}\right)^{\frac{C}{B}} \qquad \text{(for } \varepsilon_p > \varepsilon_0\text{)} \qquad (20)$$

in which:

$$N_0 = \varepsilon_0^{\frac{1}{B}}A^{\frac{-1}{B}}\left(\frac{\sigma_1}{\sigma'}\right)^{\frac{-C}{B}} \qquad (21)$$

Where:

εp	= the plastic strain
N	= the number of load repetitions
σ1	= the major principal (vertical) stress
σ'	= the reference stress (atmospheric pressure, 0,1Mpa)
A,B,C	= calibration constants

The advantages of this model are:

- Physically based model, which is developed based on, among others, test in the Danish road test machine
- Calibrated and verified against Danish data
- The model can be calibrated for other conditions/ countries
- The same general expression is used for all materials
- The model can be implemented in an incremental-recursive procedure

The disadvantage of the model is that it requires data to calibrate A, B, and C.

3.2.2 Iceland

The following brief review of the Icelandic practice regarding the use of pavement management systems and deterioration models was provided by Haraldur Sigursteinsson.

The use of deterioration models in Iceland is relatively new and is currently being implemented on trial basis. The deterioration of roads has for many years been evaluated by visual inspection and manual registration of pavement distress and failures, from which the roads have been rated. At the project level various calculation programs are used. The Bisar program is used for calculating the stresses and strains in the pavement and MN/Pave, program from Minnesota DOT is under consideration for use in evaluating expected deterioration in connection with extreme change in traffic loading.

RoSy PM-system

The Pavement Management Unit is now working on implementation of PMS and road asset management system using the RoSy PM program, which is mainly based on visual inspection. Each year the PM unit collects condition data on all paved roads which consist of 4.173 km of state roads. Information stored in the system includes basic information (road names and numbers), geometric data (width, length), traffic data, and condition data (structural and functional conditions). This data is used in the RoSy to monitor the condition of the roads and identify pavements that need maintenance or rehabilitation immediately and in near future. On the basis of this data the optimum maintenance solution can be calculated.

The structural condition of the road network is evaluated by FWD measurements. ICERA is operating one FWD and yearly measurements are conducted on a network of about 1000 km. The aim is to measure all new roads one year after construction and all roads should be measured at least every sixth year but more frequently if deterioration is registered after visual inspection.

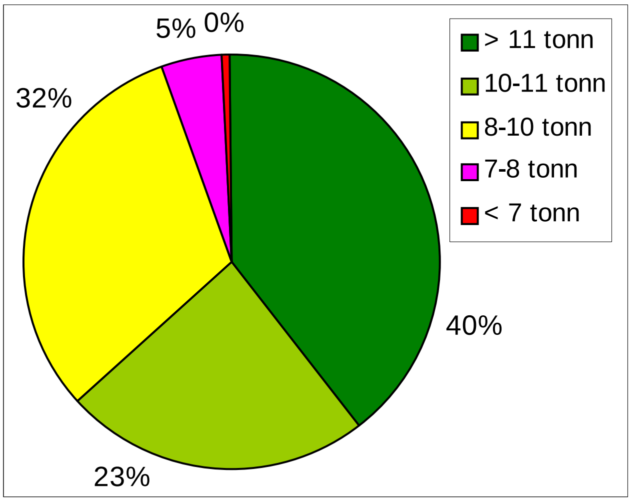

Figure 6: Asphalt pavements divided into bearing capacity groups based on measurements on 3220 km roads in the years 2000 – 2005

The result is normally presented as bearing capacity in tons calculated using program from KUAB based on constant derived empirically in Norway. The program produces values for the bearing capacity of road subsections which will result in a standard amount of deterioration. This method uses the D0 and D20 seismometer values. It uses the following formula to calculate a type of elasticity modulus:

$$E = \frac{K1xP}{\sqrt{(D0x(D0 - D20))}} \tag{22}$$

K1 is a constant that has been derived empirically to give the same value as the standard E modulus in a standard case.

With a given heavy traffic density and a given E modulus, the road will deteriorate to a certain degree in a unit period of time. This calculation compares the calculated elasticity with a reference value (normally this value is 200MPa), and the actual heavy traffic volume with the reference volume (50 per day) to calculate what the axle load should be to produce the same amount of road deterioration as in the standard case.
This axle load is referred to as the bearing capacity of the road (BE_i) calculated by following formula:

$$BE_i = 11x[\frac{(\frac{110xP}{\sqrt{D0x(D0 - D20)}})}{200}]^{0.6} x(\frac{50}{ADTT})^{0.072} \tag{23}$$

Where P is the tire pressure and ADTT is the actual heavy traffic volume (annual daily heavy vehicle traffic). The constants in equation 22 were calculated by the Norwegians.

Thus in a given case where there are 100 heavy vehicles a day and the program calculates a bearing capacity of 6 tons, to keep the deterioration of the road to the standard amount, the axle load should be restricted to 6 tons.

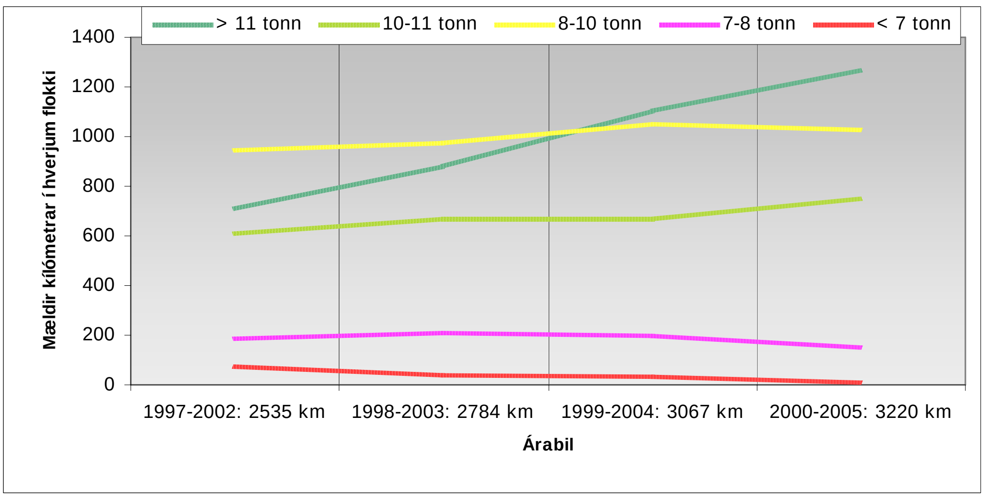

Figure 7: Change in measured km of roads in each bearing capacity groups.

The FWD results have been used to evaluate the change in bearing capacity of the road network as well as evaluating the need for maintenance and strengthening of specific roads.

IRI Evaluation

Evaluation of roads roughness by measuring IRI value is now under implementation. These measurements started in 2005 and it is assumed that the whole road network can be measured every 4th year. Figure 8 shows results from roughness measurements in 2005.

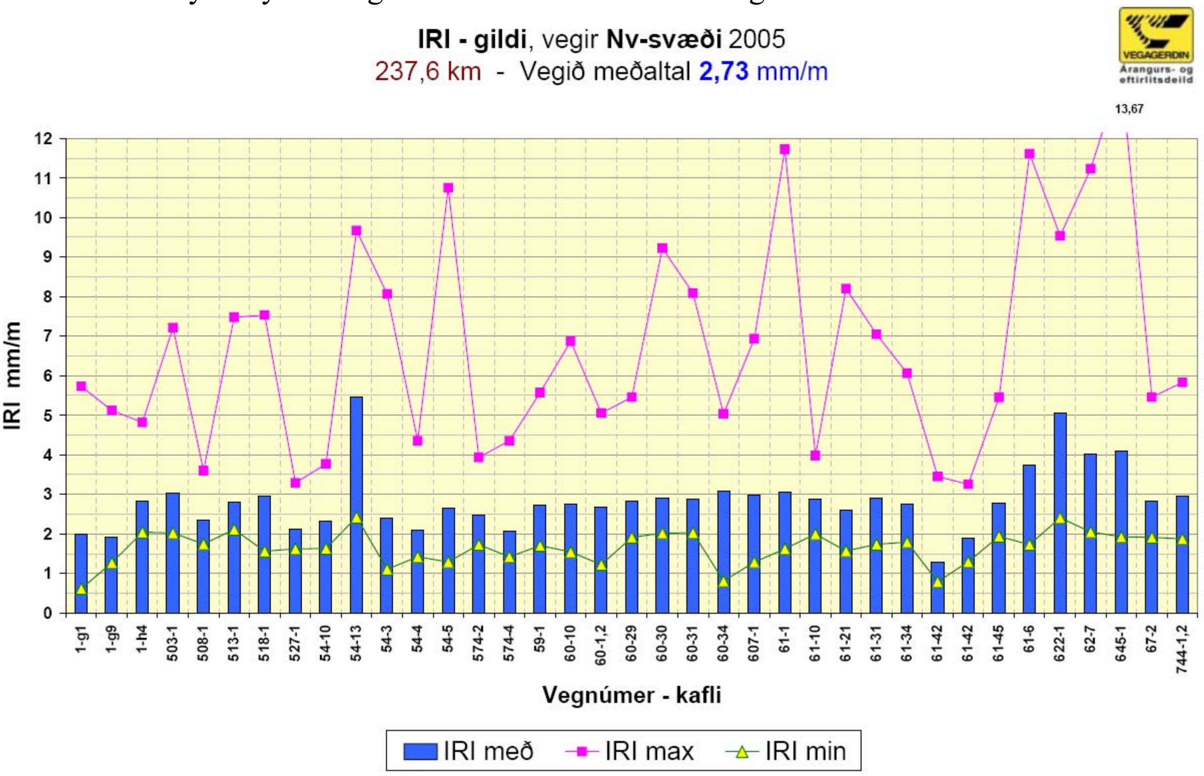

Figure 8: Results from measurements on NV-Iceland in 2005

3.2.3 Norway

In Norway simple linear extrapolation is used to predict future evenness (IRI) and rutting in PMS. However, in recent years, the Highway Investment Planning System (HIPS) have been used for purpose of network-level strategic planning in the Norwegian PMS. In addition, an attempt has been made to calibrate the performance models of the new American mechanistic – empirical design guide (MEPDG) for Norwegian conditions.

Short reviews of the Norwegian PMS and the calibration of MEPDG are given in the following paragraphs. Review of PMS was provided by Even Sund while that of MEPDG was provided by Ragnar Evensen.

The Norwegian Maintenance Standard ("Håndbok 111") includes the following pavement condition parameters:

- Rutting (mm)

- Roughness – IRI (mm/m)
- Friction (coefficient of friction on wet surface)
- Cracking (extent, both width and length)
- Cross-fall (%)
- Potholes
- Longitudinal edges
- Local unevenness / settlement including those related to spring thaw
- Height difference between gravel shoulders and pavement

For rutting and roughness the maintenance standard sets both target values at the network level and maintenance triggering values at project level. These are also the only two parameters for which future development is predicted.

The Norwegian Public Road Administration (NPRA) has fully implemented pavement management system for use at the project/project-selection level. The Norwegian PMS utilizes all available data from the Road Data Bank (RDB) and presents them to the users in a way that makes it easier to make the correct decisions when producing project level plans. The Norwegian PMS is used primarily by regional and district pavement engineers to plan pavement works at the project and project selection levels for a 6-year period. The main focus is on making and adjusting plans for the next 1-2 years so that budget constraints are met. The condition parameters included in the system are rutting, roughness (IRI) and cross-fall. PMS uses simple prediction models based on historically measured data for each road section for rutting and roughness (IRI) to estimate when the maintenance triggering values in the maintenance standard will be reached. An overview of the Norwegian project-level PMS is shown in figure 9.

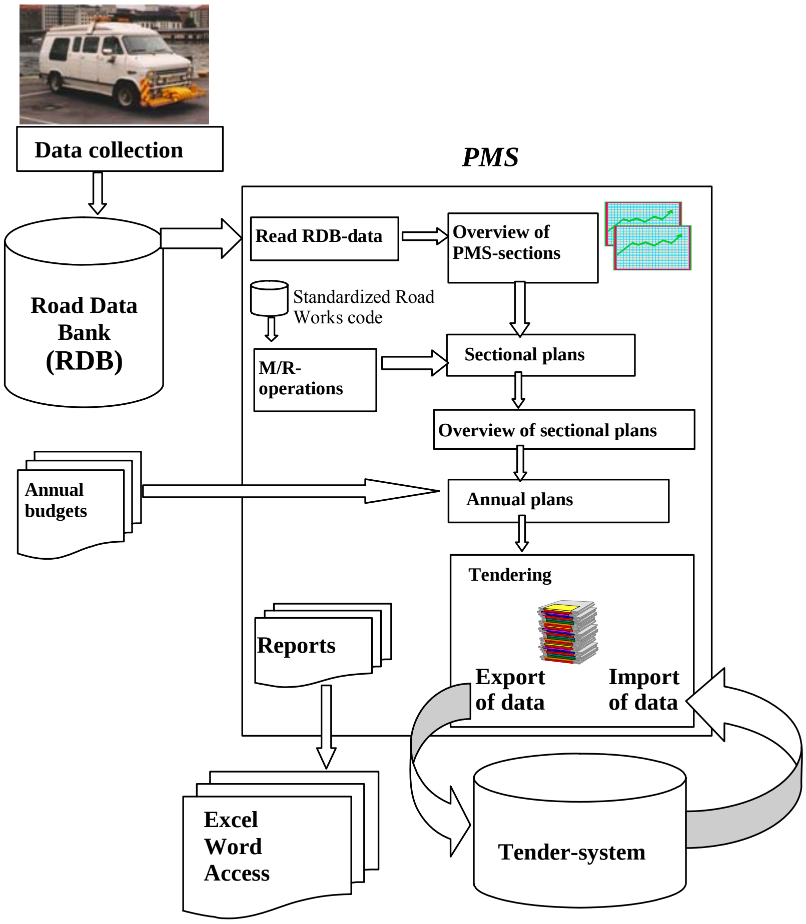

Figure 9: Overview of the Norwegian PMS at the project level

Data is collected on most of the national and county roads every year. The whole road network is divided into homogeneous PMS-sections. These can vary in length from a few hundred meters to several kilometres. Figure 10 shows a sample screenshot from the Norwegian PMS where the historical and predicted future development of rutting and roughness (90-percentile values) are shown, while figure 11 shows the detailed condition (rutting, roughness and cross-fall) along every 20 meters of the section.

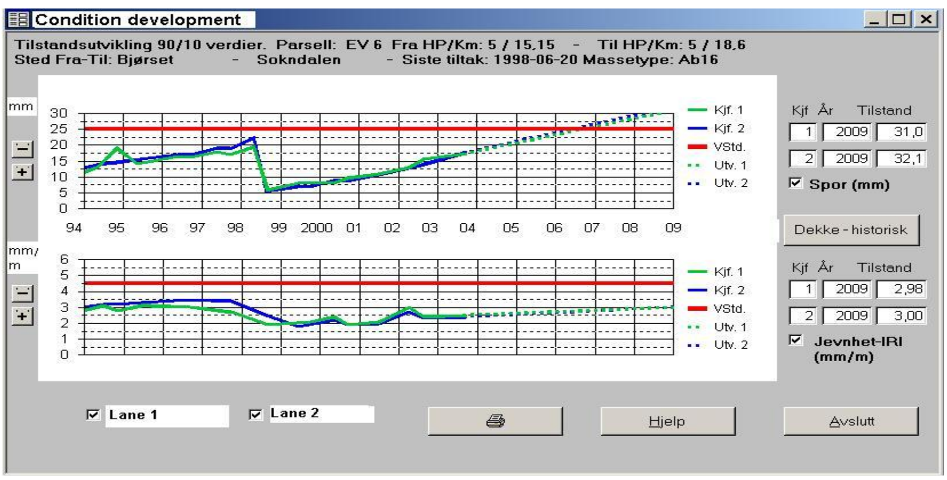

Figure 10: Historical and predicted future condition development for rutting (top) and IRI (bottom) for a specific road section

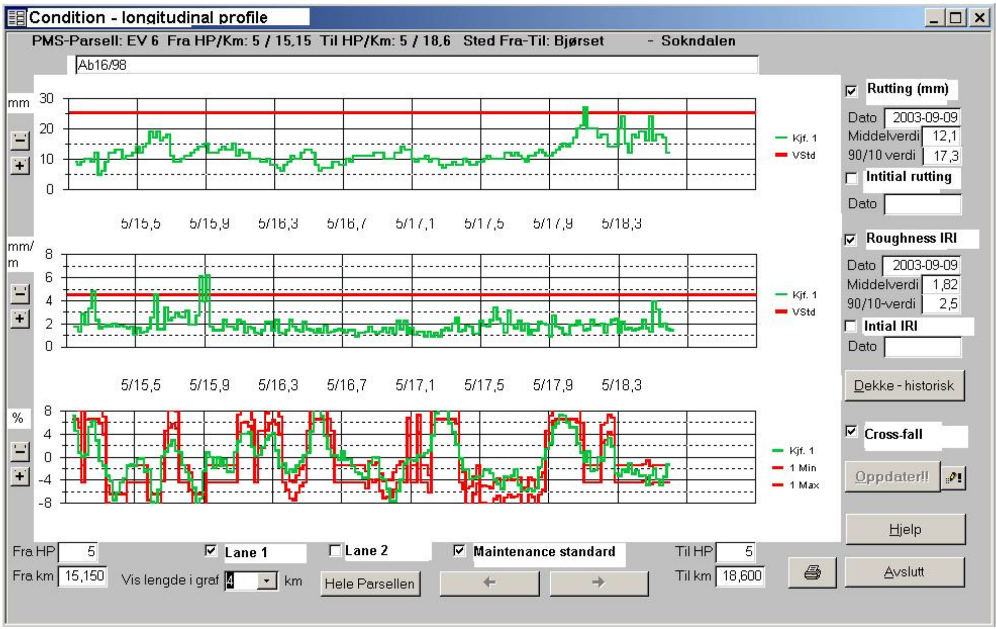

Figure 11: Detailed condition along a specific road section; rutting (top), IRI (middle) and cross-fall (bottom)

In calculating the predicted future development of rutting and roughness, a simple linear extrapolation of historical data is used. Only the 90-percentile values for roughness (IRI) and rutting are predicted. The main steps in this extrapolation are:

1. If there exists two measurements of pavement condition that are at least one year apart after the last registered maintenance action (that covered at least ⅓ of the section length), the first and last measurement points are used to calculate the straight line extrapolation.
2. If the previous step (1.) could not be used, the condition development between the two last registered maintenance actions (that covered at least ⅓ of the section length) are used based on the same principle as described under 1.
3. If neither 1. or 2. can be used the average of all positive (> 0) contributions to the condition parameter values are used to estimate future linear development.

If there exists no historical data, average values dependant on traffic-levels are used (see table 2). The predicted annual increase is limited by the maximum and minimum values defined for different levels of traffic (AADT), as given in table 2.

This model was not the result of an extensive research effort, but the result of a pragmatic attitude of the NPRA with regard to the use of the PMS.

Table 2: Maximum, minimum and average values for annual increase in roughness and rutting used in the Norwegian PMS

	AADT	< 1000	1000 – 3000	3001 –5000	5001 –15000	> 15000
Roughness (IRI mm/m per year)	Max	0.4	0.35	0.30	0.30	0.30
	Average	0.2	0.15	0.10	0.10	0.10
	Min	0.1	0.05	0.05	0.05	0.05
Rutting (mm/year)	Max	1.0	2.0	4.0	6.0	20.0
	Average	0.5	1.0	2.0	3.3	5.3
	Min	0.25	0.5	1.0	2.0	2.5

The model used in the Norwegian PMS is not mechanistic and does not involve any advanced regression, but is a very simple model whose main aim is to give information about which sections will become critical in the next 1 – 3 years. It is thought, although not scientifically investigated, that this simple model is adequate in fulfilling its intended purpose.

There is a need to analyse if the current very simple condition prediction models in the Norwegian PMS are adequate and if they should be modified. If they have to be modified, the following are some of the alternatives that may be implemented in the future PMS:

1. Use of a standard straight line regression based on all available data after the last major maintenance action.
2. Use of another form of regression curve to better take into account non-linearity in condition development, e.g. accelerated initial rutting.
3. Some kind of mechanistic model based on primary response calculation, material models, in-situ material parameters, traffic and climate. This approach would constitute a substantial change in the current PMS-program. This would also call for significant increase in the data needs and involve a significant research effort before it can be implemented.

There is also a question if the models should be developed to be probabilistic in stead of deterministic, to better take into account and quantify the uncertainties involved. This question applies regardless of the type of prediction model (simple extrapolation, straight line regression or mechanistic).
In addition to the project level system described above, analyses have been carried out at the network level that involved the use of a Finnish system called HIPS (Highway Investment Planning System).This system is based on modelling pavement condition development using Markov-chains and mathematical optimisation techniques to find the optimal condition distribution (rutting and roughness) taking into account user costs. These analyses were carried out in the period 1998 – 2004. There may be a need to develop these network level models further or, alternatively, modify the PMS so that it can fulfil the needs for strategic analysis also at the network level.

Calibration of MEPDG for Norwegian Conditions

As part of the Norwegian ”Vegkapital” project, the Mechanistic-Empirical Pavement Design Guide, MEPDG, has been tested for performance prediction of the national road network in Norway. All the tests have been made with version 0.7 of the programme, available June 2004. A version 0.9 will be available June 2006.

A realistic test of the MEPDG requires input values of the design parameters that are typical for Norwegian conditions. These parameters include climate, traffic and materials in the subgrade and pavement layers.

Climatic data files (icm-files) have been generated for three metrological weather stations in Norway, i.e. Stavanger, Lillehammer and Tromsø. For Stavanger and Tromsø the files contain data from daily observations over five years (1.1.2001 – 31.12.2004), for Lillehammer the observation period is 2 ½ years (1.5.2002 – 31.12.2004). The climatic data comprises maximum and minimum daily temperatures, daily averages of wind speed, precipitation and percentage of sunshine. The depth to the ground water table has been manually set to 6 ft for all the observation stations.

The MEPDG requires detailed information on the traffic. This includes vehicle class and axle load distributions as well as several other types of data. The user can select from a list of default values. It is, however, assumed that the data is representative of the traffic in the USA, which is quite different from the traffic on Norwegian roads. In Norway the data available on vehicle class distribution etc. is limited. By courtesy of the Swedish Road Administration, comprehensive BWIM-data (Bridge-Weigh-in-Motion) from several locations in Sweden was made available to the project.

In Norway heavy vehicles are defined as vehicles with an allowable total weight of 3,5 tonnes or more. The BWIM-data includes no information on the allowable total weight of the vehicles. It was therefore decided to restrict the analysis to vehicles with a total weight of 3,0 tonnes or more. In total, data from 8 locations in Sweden were analysed with respect to:

- Vehicle class distribution according to the FHWA vehicle classification.
- Average number of single, dual and triple axles for each vehicle class.
- Axle load distribution for single, dual and triple axles for each vehicle class.
- Hour by hour distribution of the heavy vehicles.

The traffic on Norwegian roads is split into short and long vehicles, with a recorded total length of 5,5 meter as the separation length. In earlier analyses of Norwegian WIM-data, it was found that the number of heavy vehicles (defined as vehicles with a measured total weight of 3,0 tonnes or more) could be estimated by multiplying the number of long vehicles by a factor of 0,8.

It is assumed that the heavy vehicles on Norwegian roads are quite similar to those on Swedish roads. There is, however, one important difference. The allowable total weight of vehicles is 50 tonnes in Norway and 60 tonnes in Sweden. The distributions of the total weight of the different vehicle classes, indicate that this difference is vital for only one of the vehicle classes (semi or full trailer vehicles with six axles or more). For this class it is assumed that the difference in allowable total weight has an influence on both the average number of axles per vehicle as well as the axle load distributions.

As the MEPDG was to be applied on the Norwegian national road network, the material properties have to be related to a rather coarse classification of the materials in the pavement and in the subgrade. For the materials in the base and the subbase of the road the classification is limited to whether the material is sand/gravel or crushed rock, and to the degree of frost

susceptibility of fine-grained materials. For the subgrade materials the information is restricted to a classification of the materials into 6 classes with respect to the assumed bearing capacity of the material. The MEPDG allows the user to apply either the AASHTO or the Unified Soil Classification System (USCS) for the materials.
The performance predictions of the national road network in Norway are based on the USCS with respect to

- plasticity index
- maximum dry unit weight
- specific gravity of solids
- hydraulic conductivity
- grain size distribution

The default values are adjusted according to Norwegian experience, whereas for the other parameters the default values were applied.

The performance prediction is limited to longitudinal roughness (IRI) and rut depth. With respect to bottom up and surface down cracking there is almost no information available for roads in Norway. Accordingly, for these parameters there are no data to compare with the predicted performance. For transverse low temperature cracking preliminary studies indicated that extended low temperature cracking is predicted for the Lillehammer climate, no low temperature cracking is predicted for the Stavanger and Tromsø climate. With respect to rut depth and longitudinal roughness comprehensive data are available covering all national and county roads measured yearly.

The increase in rut depth predicted with MEPDG is related to permanent deformations in the asphalt layers, in the granular layers of the pavement and in the subgrade. The measured rut depth is the sum of ruts caused by permanent deformations and ruts caused by the wear from studded tyres. The rut dept caused by the wear from studded tyres is estimated by combining data on the traffic with studded tyres, the wear resistance of the asphalt surface courses, and the average lane widths.

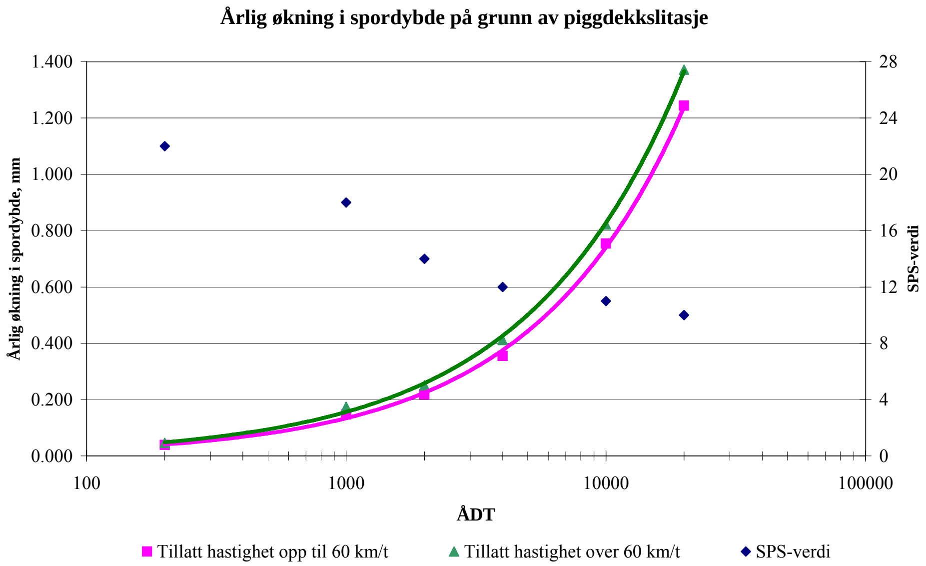

Figure 12: Annual increase in rut depth due to studded tyre wear

In figure 12 the estimated yearly increase in rut depth caused by studded tyre wear is presented. The graph also includes a presentation of the asphalt's SPS-values. The SPS is an expression of the estimated wear resistance of the asphalt.
Preliminary studies indicated that the average predicted increase in rut depth corresponds well with Norwegian experience. However, the predicted increase in IRI is rather insensitive to variations in the pavement structures. This is not in agreement with Norwegian experience. The predicted increase in IRI is also much more homogeneous than what is observed for roads in Norway.

The MEPDG offers several calibration possibilities for the various performance prediction models. The level of detail in the calibration must, however, correspond with the availablity of data. For a simple calibration seven reference sections of road projects were established, i.e. four in Akershus, one in Oppland and two in Sør-Trøndelag county. The reference sections are minimum 500 m long, are relatively homogeneous and are constructed 5 – 15 years ago. Another requirement for the selection of reference sections was that data of good quality on layer thicknesses and material properties were available from the construction reports, so that no further investigations were required.

With respect to rut depth as predicted by the MEPDG, it was observed that the initial rut depths were unrealistically large for all the reference sections. It was therefore decided that the calibration should be based on the yearly-predicted increase from 5 to 20 years after the road was opened to traffic. For the calculations it was further assumed that the road foundation was constructed three years before the road was opened to traffic.

The limitations above made it necessary to rely on rut depths measured in the general survey of the national roads for the calibration. It was not possible to increase the accuracy of the rut depth measurements by multiple runs as a part of the calibration preparation. The results of the calibration are presented in figure 13. For rut depth, it was found that the calibration factor should depend on the traffic volume, and vary from 5 to 9.

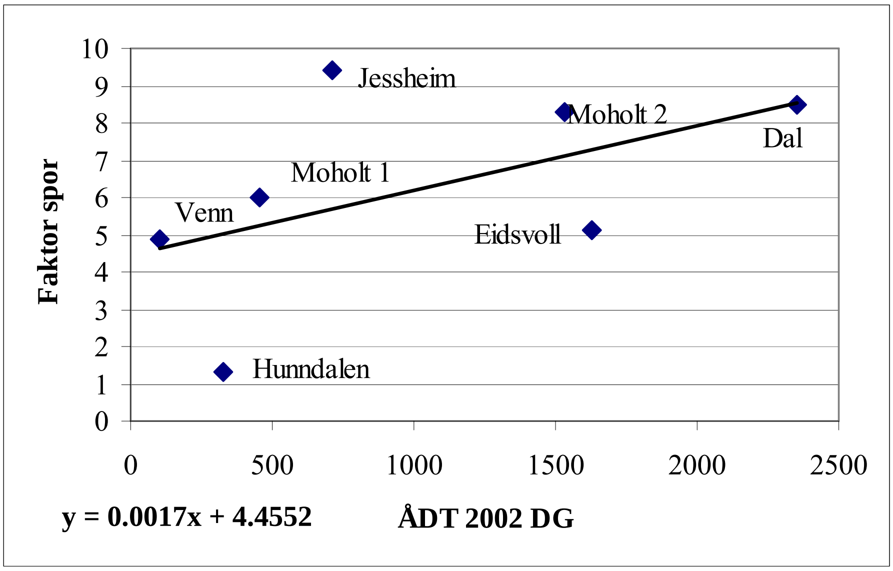

Figure 13: Calibration factor for rut depth

The reference sections that differ most from the line of calibration, Jessheim and Hunndalen, are the sections for which the measured rut depths show the largest variations from one year to another.

The calibration of IRI is presented in figure 14. If the section Venn is ignored, a calibration factor of 0,9 is the best guess. It is assumed to be independent of AADT.

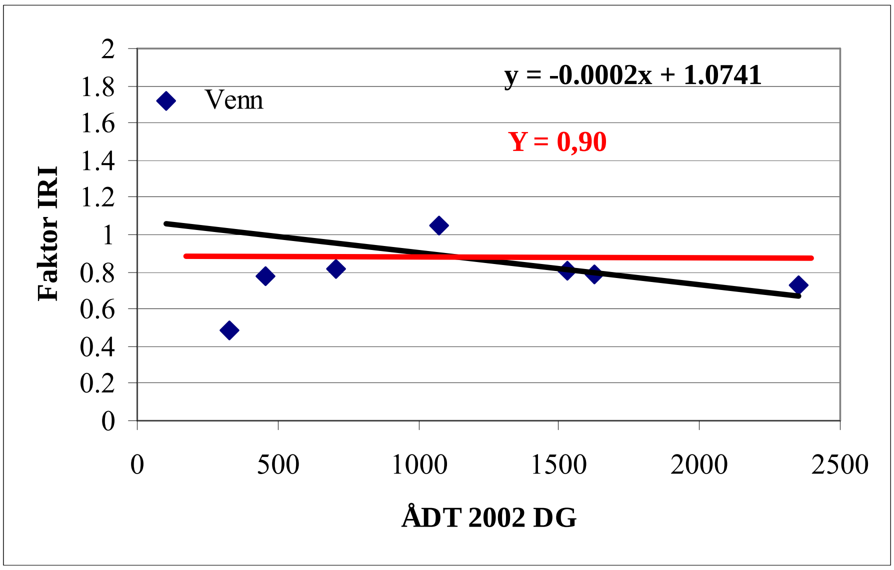

Figure 14: Calibration factor for IRI

For reasons presented above, it was later decided not to use results from the MEPDG for the prediction of IRI for the national road network.

Based on the available information in the Road Data Bank the national road network has been split into a large number of typical sections depending on:

- AADT

- The subgrade materials (6 strength classes)
- Whether the materials in the pavement are frost susceptible or not

For each combination the following data for the national roads were calculated:
- Average pavement thickness
- Average thickness of the asphalt
- Total length of roads

In Norway rut depths and IRI have been recorded every year since 1989 as part of the Norwegian Pavement Management System. However, the measuring principle was changed in 1997 for IRI and in 1999 for rut depth. All analyses of the recorded pavement performance have therefore been restricted to measurements later than 1997 and 1999 for IRI and rut depth respectively.

The average annual increase in predicted rut depth as a function of the AADT is compared with the recorded increase in figure 15. In this graph the predicted increase is the sum of estimated wear for studded tyres and rut depth caused by permanent deformations, predicted by the use of the MEPDG.

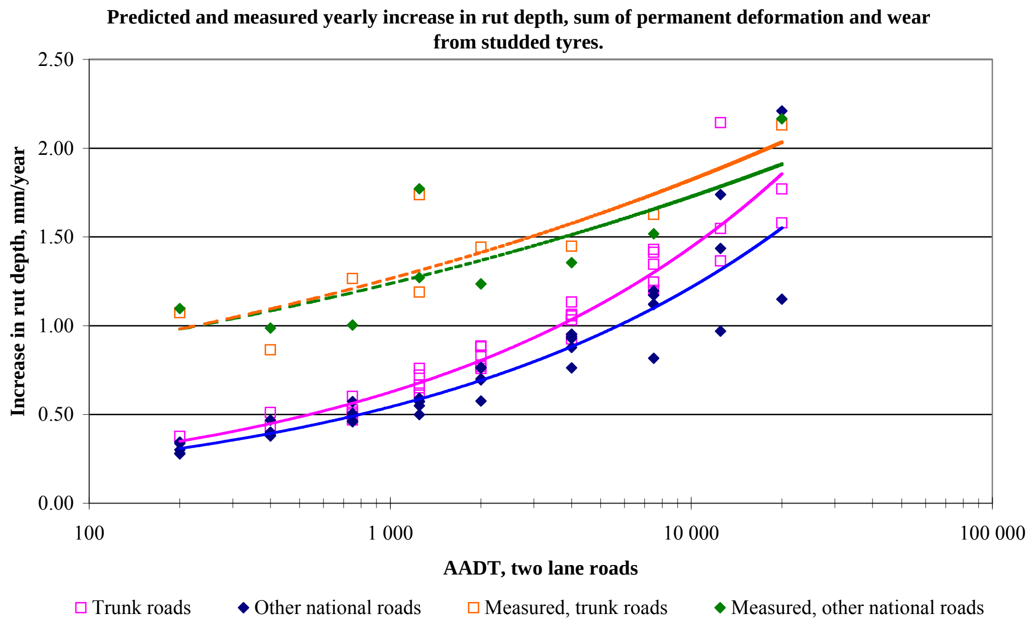

Figure 15: Predicted and measured yearly increase in rut depth

The objective of the study was to analyse the effect of changes in the triggering levels for IRI and rut depth with respect to road maintenance actions, hence the service lives of the asphalt surface layers. Such changes have thereby an effect on the road maintenance costs as well as the road user costs.

As a result of the differences presented above it was decided to use the recorded annual changes in IRI and rut depth and not the performance predicted by the use of version 0.7 of the MEPDG.

Most Norwegian roads are rather inhomogeneous with respect to the subgrade materials. There are frequent changes from rock to moraine and soft clays. For this reason, and others, it is probably a great challenge to make an international system for IRI predictions applicable to Norwegian conditions.

With respect to rut depth, which is commonly the triggering parameter for maintenance actions, it is more likely that future versions of the MEPDG will be calibrated for reliable performance predictions of Norwegian roads.

It is a widely accepted fact that the models for permanent deformations in granular materials in version 0.7 of the MEPDG, need further development. At the present time it is not known whether version 0.9, released in July 2006, includes a revised version of the models for permanent deformations.

3.2.4 Sweden

Sweden has implemented deterioration/performance models both in its pavement design system and pavement management system. The models implemented in the PMS are simple linear extrapolation of historical data. The current Swedish pavement design system is an analytical method based on linear-elastic theory. The design is conducted using a computer program known as PMS Objekt 2000. The following brief review of the design system is based on an article written and published by Tomas Winnerholt (Winnerholt, 2002).

PMS Objekt 2000 can be used for design of new construction as well as maintenance and rehabilitation. The traffic load is expressed in terms of equivalent single axle load for each design section. The program contains a database of “standard pavements” depending on subgrade material. The user chooses the appropriate “standard pavement” and then modifies it to meet the specific bearing capacity requirements placed on the section.

Linear elastic model is used to calculate the strains at two levels in the construction; at the boundary between the bound and unbound materials and at the top of subgrade. The boundary between the bound and unbound materials is analyzed for horizontal tensile strains. At the subgrade level the vertical compressive strains are calculated. The calculation is performed for various climatic periods, typically: winter, thawing, summer and autumn. The materials have different e-moduli during these periods. An equation is then used to transform the strains into standard axles. The equation used for bituminous materials is a modified Kingham’s equation. The allowable number of axles is then calculated and this has to be greater than the equivalent number of standard axles to meet the bearing capacity demand.

The design procedure also includes calculation of frost heave using a frost heave calculation model. The model simulates how the freezing process propagates through the pavement and how much frost heave it produces on its way. The model needs data on actual surface temperature in the winter time.

The following are the strengths and weaknesses of the current design system according to Jesper Elsander (e-mail communications with Jesper Elsander, June 2006).

The strengths of the system are:

- It gives economically acceptable road structure in the whole country.

- It is validated against cracking on real pavement and gives reasonable results.
- It is built on a proven technique.
- Realistic temperature data is used for the frost design.

The weaknesses of the system are:

- It designs only to withstand fatigue and does not consider rutting.
- It does not allow other materials than the standard material (particularly for bitumen bound layers).
- No realistic value for stiffness modulus (just values adopted in the model).

Sweden is also developing a new design method, which aims to improve road design through "active design" on site and also predict future rutting. The following paragraphs give a brief description of the new design method as provided by Anders Huvstig.
Active design involves changing the design of the pavement structure based on measurements conducted on actual pavement materials on site. The prediction of future rutting is based on the program VägFEM for calculation of elastic response and four Excel programs, based on new research results from USA, France and Germany.

During 2003 and 2004, SRA produced a new user-friendly finite element program, "VagFEM", which is based on ABAQUS and was designed by the consultant Volvo IT. It is a 3D FEM program in which it is possible to describe the real geometry of the road. It also makes it possible to do calculations with non-linear elasticity for the unbound materials, and with viscosity for the bituminous bound materials. The model used today for the description of the non-linearity is: $M = k1\theta^{k2}$. Burger's model is used for describing the viscosity of the bituminous bound layers.
One important aspect of the FEM program usable for road design, especially for active design on construction sites, is the calculation time. "VagFEM" is simplified in some respects, resulting in calculation times of less than 20 minutes. The input data can be quickly entered into a high speed computer, which produces results in a PDF file. The results are delivered as diagrams and values for stress and strain (see next section, Description of "VagFEM").
In "VagFEM", a special part of the program can also simulate the plate loading test on unbound material. It has proved possible to analyze the whole curve from the plate loading test in order to decide the values of k1 and k2. This can be done by running the program three or four times, until the curve from the program is similar to the curve from the test on the material in situ.
The next step in the development of "VagFEM" is to deliver the output data in the form of a digital file with a view to these data, denoting stress and strain at certain levels in the road structure, becoming direct input in four new Excel programs for calculating the permanent deformations (in the first version, rutting). These programs will use some theories from the USA's Design Guide. They will also use two different material models for unbound materials developed by LCPC (France) and at Dresden (Germany)/Delft (Netherlands) in certain developed Excel programs - for comparison with the results on actual roads.

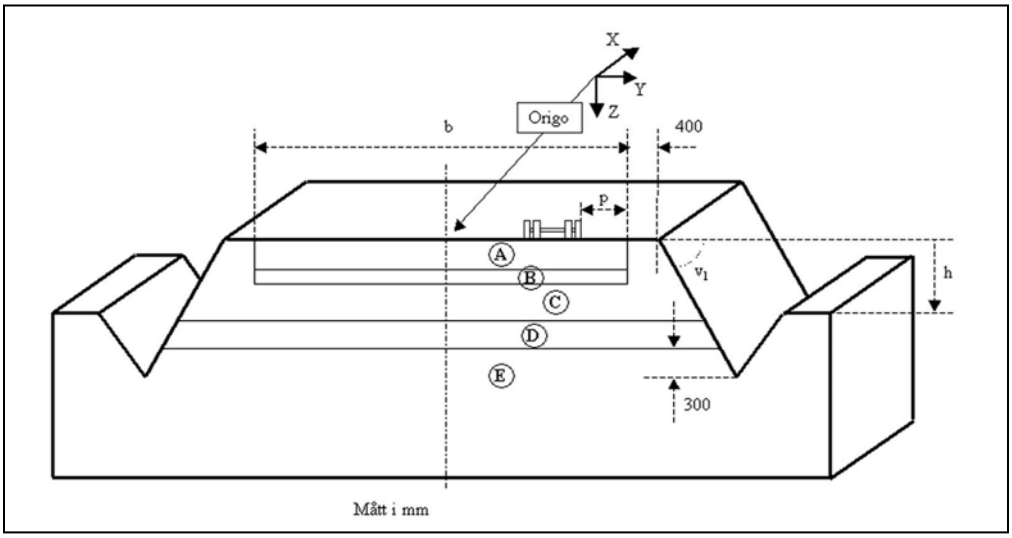

Figure 16: Geometry of the model

“VagFEM” may be briefly described as follows. The input data comprises road geometry, thickness of layers (see figure 16), position of loading, elasticity modulus (and viscosity) for the bituminous bound layers and elastic modulus or resilient modulus for the unbound layers. The weight of the road material is included in the model.
The output data comprise diagrams of deformation, stresses and strains in different parts of the road structure.

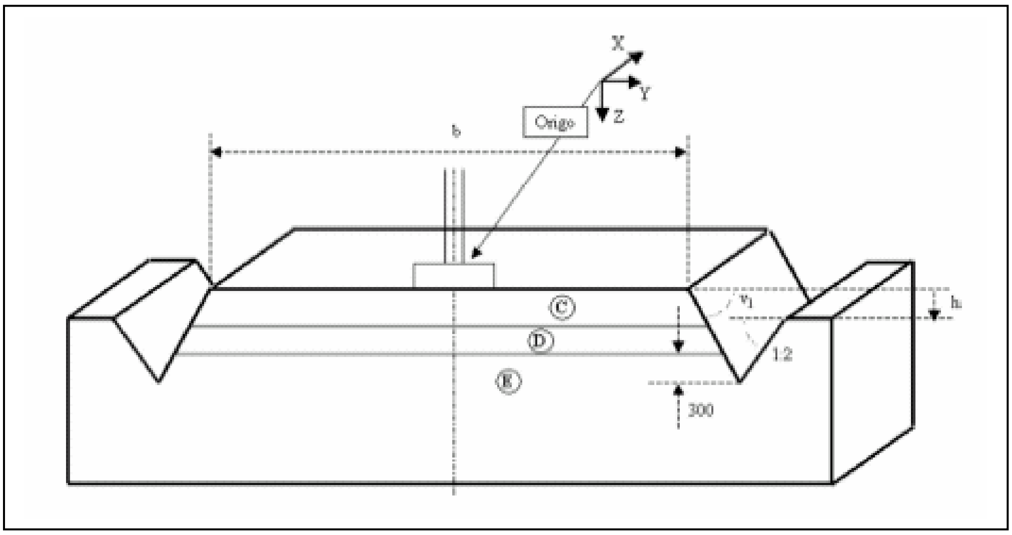

Figure 17: Model geometry for plate loading

”VägFEM” also includes a module for calculating step by step the deflection during plate loading tests, see figure 17. Its purpose is to make it possible to calculate the resilient modulus for the unbound layers. To this end, the resilient modulus of the subgrade needs to be ascertained, which involves plate loading tests at the top of the subgrade and at the top of the base layer. The calculated values of the resilient modulus for these layers become input data for calculating the next layer.
The output data from the calculations in this module comprise the deflection beside the plate (figure 18) and a diagram for the various load steps under the plate (figure 19).

One possibility is to compare results from FWD with the curve in figure 18 in order to estimate the dynamic resilient modulus of the unbound layers.

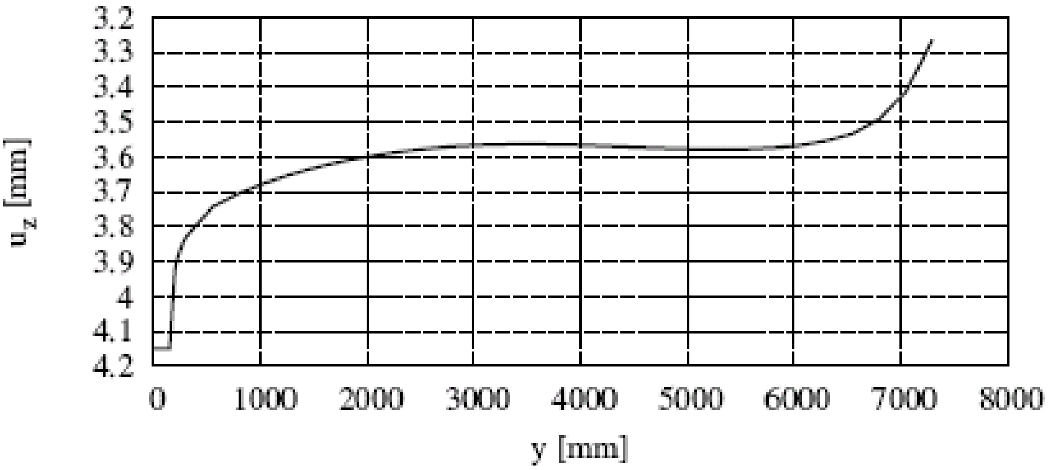

Figure 18: Simulated deflection besides the plate

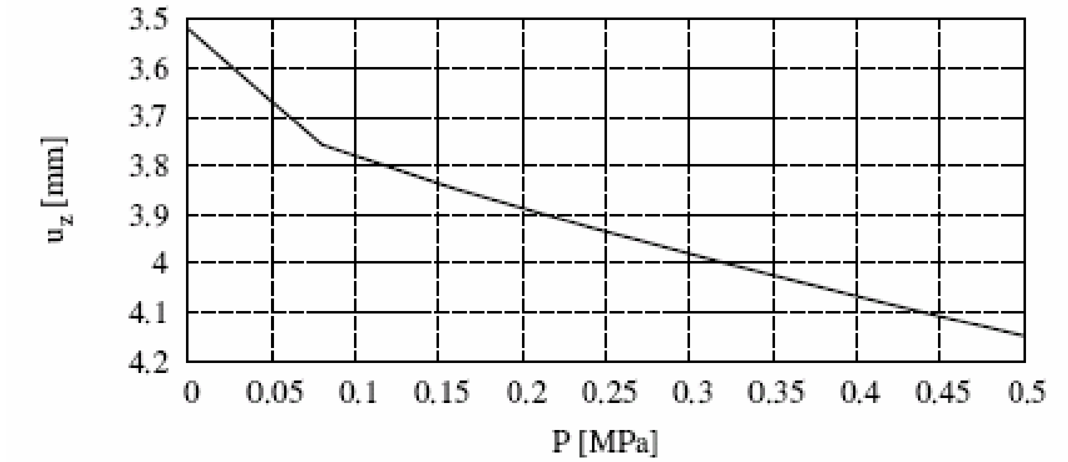

Figure 19: Simulated deflection under the plate

In order to develop the method of calculating rutting, the following models has been chosen and developed as Excel programmes. For bituminous bound layers, the following model is chosen from the Design Guide (USA).

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 \cdot N^{a_2} \cdot T^{a_3}$$

ε_p	**– Accumulated plastic strain at N repetitions of load**
ε_r	**– Resilient strain of the asphalt material (from FWD-data)**
N	**– Number of load repetitions**
T	**– Temperature (10°C)**
a_i	**– Non-linear regression coefficients (from NCHRP 1-37A)**

For the unbound material, the following models have been chosen:

Unbound base layers

$$\log\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = 0.80978 - 0.06626 \cdot W_c - 0.003077 \cdot \sigma_\theta + 0.000003 \cdot E_r$$

$$\log(\beta) = -0.9190 + 0.03105 \cdot W_c + 0.001806 \cdot \sigma_\theta - 0.0000015 \cdot E_r$$

$$\log(\rho) = -1.78667 + 1.45062 \cdot W_c + 0.0003784 \cdot \sigma_\theta^2 - 0.002074 \cdot W_c^2 \sigma_\theta - 0.0000105 \cdot E_r$$

Subgrade

$$\log\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = -1.69867 + 0.09121 \cdot W_c - 0.11921 \cdot \sigma_d + 0.91219 \cdot \log E_r$$

$$\log(\beta) = -0.9730 - 0.0000278 \cdot W_c^2 \sigma_d + 0.017165 \cdot \sigma_d - 0.0000338 \cdot W_c^2 \sigma_\theta$$

$$\log(\rho) = 11.009 + 0.00068 \cdot W_c^2 \sigma_d - 0.40260 \cdot \sigma_d + 0.0000545 \cdot W_c^2 \sigma_\theta$$

The problem with the models in the USA's Design Guide is that they do not take the material characteristics from the triaxial tests into consideration

The LCPC model.

$$\varepsilon_1^p(N) = \varepsilon_1^{p0}[1 - N]^{-B} \cdot \left(\frac{L_{\max}}{p_a}\right)^n \cdot \frac{1}{m + \frac{s}{p_{\max}} - \frac{q_{\max}}{p_{\max}}}$$

The Huurman / Dresden model.

$$\varepsilon_p(\mathrm{N}) = \mathrm{A} \cdot \left(\frac{N}{1000}\right)^B + C\left(e^{D\frac{N}{1000}} - 1\right)$$

Four road projects are under construction or are to be built according to the new method in western Sweden, and are to be monitored.

The implementation of this new method included careful planning and provision of training for personnel in the use of mechanistic pavement design method. It also involved a close co-operation between client and contractors, in which benefits (or savings) were shared equally between the client and contractors, in order to create a "Win – win" situation. The experience gained so far indicates that this method results in improved quality of the road and a saving of up to 5 % of the pavement cost.

Results from earlier projects with better compaction of the unbound base layers also indicate that the progress of rutting and unevenness will get an essential reduction, which means a better future performance.

4 Implementation Issues

Implementation of performance prediction models requires a significant effort and an extensive data. The amount of data required would depend on the type of model. Simple empirical models can be developed and implemented using data that is usually collected as part of a pavement management system. However, mechanistic-empirical models require more detailed material data, which is not usually available in the pavement management databases. In the following sections the data required for the implementation of performance models will be briefly discussed.

4.1 Implementation of Mechanistic – Empirical Models

Implementation of existing mechanistic–empirical models requires extensive calibration effort. The success of the calibration depends on the availability of data on material, climate, traffic and field performance data for in-service pavements. The climate data can often be obtained from metrological agencies and traffic and performance data are often collected as parts of monitoring activities within the framework of pavement management systems. Therefore, it is the material data collection that requires additional effort in the implementation process. The primary characteristics (mechanistic properties) that are used to evaluate the performance of pavement materials under various loading and environmental conditions in most mechanistic-empirical models are the resilient modulus (E) and Poisson's ratio (μ). Input requirements vary from model to model. For example, for MEPDG the primary material property input requirements are as shown in table 3 (Olidis and Hein, 2004).

To obtain the primary input parameters one needs to conduct laboratory testing. Determination of some of the parameters such as the dynamic modulus, Poisson's ratio, and creep compliance requires use of repeated load triaxial test, which is considered to be expensive and time consuming. However, some improvements that can potentially make the triaxial testing fast and simple have been reported in the literature. Recognizing the difficulty and cost of providing complete and detailed material information for all applications, most of the mechanistic-empirical methods use a hierarchical approach, which allows the user to select the input parameters based on the importance of the project. For example, MEPDG uses the following three levels:

- Level 1 – requires detailed testing of specific materials to be used in the project and would typically be used for a research test section or a very high volume road.
- Level 2 – is intended for routine use and complex testing such as modulus determination is not required. Resilient modulus values are determined through correlations with other more standard testing procedures such as California Bearing Ratio (CBR), aggregate gradation, plasticity index and moisture content.
- Level 3 - typical material property default values are used. This level has the lowest level of accuracy and it is intended for use for low volume roads.

Table 3: Required material information in MEPDG

Material	Required information
Hot Mix Asphalt / Recycled Asphalt	• Dynamic modulus • Poisson's ratio • Tensile strength • Coefficient of thermal expansion • Creep compliance • Thermal conductivities • Asphalt binder stiffness • Aggregate properties
Unbound material / Subgrade	• Seasonally adjusted resilient modulus • Poisson's ratio • Unit weight • Gradation • Hydraulic conductivity • Optimum moisture content • Plasticity index • Coefficient of lateral pressure

Metha et al (2006) proposed a conceptual framework for developing asphalt mix catalog for implementation of MEPDG. The framework includes:

- Preliminary evaluation to determine if the agency should dedicate resources for collecting project specific (level 1) input. The preliminary evaluation includes selecting existing pavement sections of different pavement stiffness and layer thickness. The performances of these pavements should be evaluated using default values (level 3) input. If the predicted performance does not correlate reasonably well with the measured performance, then the agency should take measures to collect data for some site specific material properties as input in the MEPDG.
- Division of the region/country into several geographical areas based on the initial evaluation.
- Selection of monitoring and sampling sites of sections which are scheduled for paving.
- Selection of mixtures from each of the sampling sites.
- Laboratory testing and analysis
- Development of database of mixture properties.

Figure 20 illustrates the proposed conceptual framework.

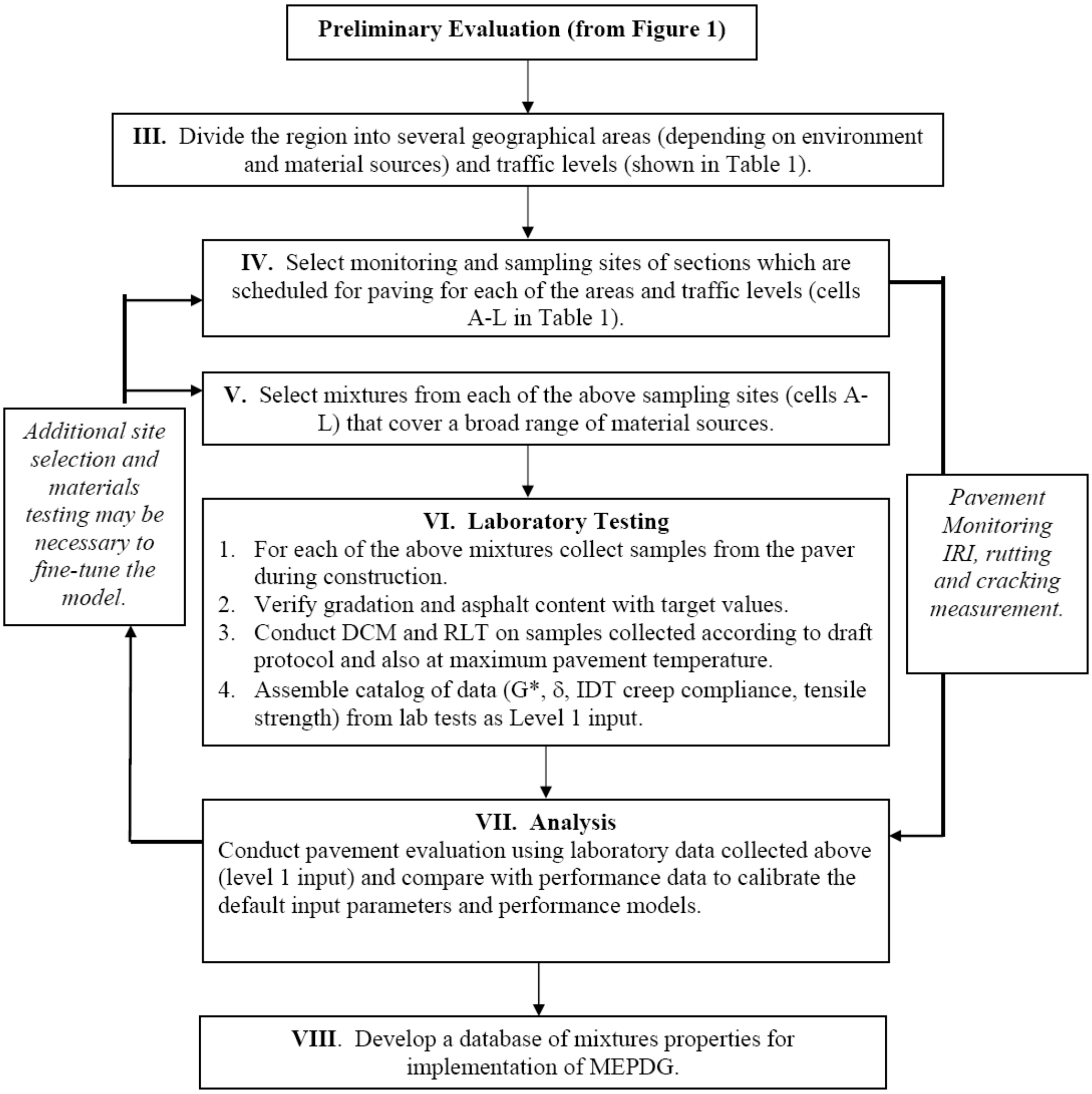

Figure 20: Conceptual framework for establishment of database of mixture properties (Mehta et al, 2006).

4.2 Implementation of Statistical (purely empirical) Models

As mentioned before the comprehensiveness of the statistical models vary greatly and so does the data need for implementation of these models. Some of these models predict performance as a function of traffic or age of pavement, in which case the need for data collection is reduced very much. Other more comprehensive models, such as the World Bank's HDM-4 require extensive information on the road. It has been proposed that calibration of HDM-4 can be conducted at the following three levels (Paterson 2001):

Level 1: Desk based study and minor field work.
Level 2: Based on field survey data of sample of pavement sections.
Level 3: Based on long-term pavement performance data to reconfigure predictive relationships.

5 Conclusions and Recommendations

Based on the review of literature on existing pavement performance prediction models the following conclusions are made:

1. Performance prediction models represent a key element of road infrastructure asset management systems or pavement management systems. Thus successful implementation of these systems depends heavily on the performance prediction model used as the accuracy of the predictions determines the reasonableness of the decisions.
2. Several pavement performance prediction models have been proposed over the years. Many of these models are developed for application in a particular region or country under specific traffic and climatic conditions. Therefore they can not be directly applied in other countries or conditions.
3. Although much research has been devoted to performance modelling of pavements, a comprehensive model that can predict pavement performance accurately has yet to be developed.
4. The available models can be broadly classified into three groups; empirical, mechanistic-empirical and subjective models. Various empirical models are proposed for application at network and project levels. The mechanistic-empirical models are often developed in connection to design systems and therefore have not been widely applied in pavement management systems (PMS), but have the potential to be applied at a network level. The subjective models are mostly developed for strategic (investment) planning at the network level.
5. Almost all Nordic countries use simple performance prediction models, based on linear extrapolation of historic data, in their pavement management systems. Denmark uses a slightly different approach in which pavement roughness is predicted as a function of pavement age using non-linear models (curves). Denmark and Sweden have implemented more advanced performance prediction models in their design systems. In Sweden research is underway to further develop the performance models in connection with development of a new design method known as “active design”.
6. The simple models currently in use in PMS in Nordic countries are not suitable for prediction of pavement condition over long periods of time. Further, they can not be used for evaluation of the effect of different maintenance measures and material qualities Thus there is a need for better performance prediction models for PMS applications.
7. As the current trend is to move from purely empirical design methods to mechanistic-empirical methods, it is important to further develop performance prediction models that are suitable for these methods. Furthermore there is a need to evaluate the possibility of using the mechanistic – empirical models for prediction of the condition of the road network.

In order to develop and implement improved pavement performance prediction models in the Nordic countries it is recommended to take the following steps in the main NordFoU project:

1. Pool available data and resources. Development of performance prediction models requires large amount of data on real pavements. There are test sections or reference sections in most of the Nordic countries for which data of various level of detail are

available. It is therefore important to pool these data to obtain a good basis for improvement of performance models.

2. Use the available data from test sections, heavy vehicle simulator and other sources to evaluate existing performance prediction models.
3. Select suitable models based on the evaluation and identify areas that need improvement.
4. Improve the selected models especially with regard to climatic effects and studded tire use to make it suitable for Nordic conditions.
5. Implement the improved models in each country.
6. Agree on recommended test methods so that material evaluations can be conducted using the same procedure in each country.
7. Agree on a uniform procedure for traffic data collection and processing.

List of References

1. Molenaar, A.A.A., 2003. *Pavement performance evaluation and rehabilitation design*, key note paper, Proceedings of MAIREPAV' 03, Guimaraes, Portugal.
2. Haas, Ralph, 2003. *Good technical foundations are essential for successful pavement management*, key note paper, proceedings of MAIREPAV' 03, Guimaraes, Portugal
3. Kerali, Henry G. R., 2000. *Overview of HDM-4*, The World Road Association (PIARC).
4. Yang H. Huang, 1993. *Pavement analysis and design*, Prentice Hall.
5. Prozzi, J. A. and Madanat, S.M., 2004. *Development of pavement performance models by combining experimental and field data*, Journal of Infrastructural Systems, Vol. 10. no. 1.
6. European Commission, Director General for transport, 1997. *COST 324: Long term performance of road pavements*, final report of the action.
7. Litzka, Johann, 2006. *Performance indicators for road pavements*, COST Action 354, Proceedings of the 1st TRA conference, Gothenburg, Sweden.
8. National Cooperative Highway Research Program (NCHRP) 1-37A, 2004. *Guide for mechanistic-empirical design of new and rehabilitated pavements*, USA.
9. Ullidtz, Per, 2002. *Analytical tools for design of flexible pavements*, key note address, Proceedings of the International Conference on Asphalt Pavements, Copenhagen, Denmark.
10. Busch C., Hildebrand, G. and Ullidtz, Per, 2005. *Pavement design by means of performance simulation*, TRB annual meeting CD-ROM.
11. Hildebrand, G., Busch C., Ullidtz, Per, 2006. *New Danish pavement design guide using simulation to obtain optimum results*, Proceedings of the 1st TRA conference, Gothenburg, Sweden.
12. Collop, A.C. and Cebon, D., 1995. *A model of whole life flexible pavement performance*, Proceedings of the institution of mechanical engineers, part C: Journal of Mechanical engineering sciences, vol 209, no.6.
13. D'Apuzzo,M., Nicolosi, V., and Mattarocci, M., 2004. *Predicting roughness progression of asphalt pavements by empirical-mechanistic model*, obtained from WWW.Siiv2004.uniti.it.
14. Hornych, P. and El Abd, A., 2006, *Development and validation of a method of prediction of structural rutting of unbound pavement layers*, SAMARIS final report D27.
15. Blab R., Kappl, K., Lackner, R., and Aigner, L., 2005. *Permanent deformation of bituminous bound materials in flexible pavements: Evaluation of test methods and prediction models*, SAMARIS final report, D28.
16. Zhen, L., 2005. *A probabilistic and adoptive approach to modelling performance of pavement infrastructure*, Ph.D dissertation, the University of Texas at Austin
17. US Department of Transportation, 1999. *Asset management primer*.
18. Huvstig, A., 2006. *The development of a new road design method in Swe*den, Proceedings of the TRB annual meeting.
19. Winnerholt, T., 2002. *A new approach to pavement design in Sweden: New Swedish road design manual*, Proceedings of the BCRA conference, Lisbon, Portugal.
20. Elsander, J. E-mail communications.
21. Olidis, C. and Hein, D., 2004. *Guide for the Mechanistic-empirical design of new and rehabilitated pavement structures: Material characterization, is your agency ready*? Paper presented at the 2004 annual conference of the Transportation Association of Canada.

22. Metha Y., Myers, L., Price, S. and D'Angelo, C. P., 2006. *Conceptual framework for material characterization in mechanistic-empirical flexible pavement design*, CD-proceedings of the 10th international conference on asphalt pavements, Quebec, Canada.
23. Paterson, W.D., 2001. *Road deterioration, works effect and calibration*, World Bank, HDM-4 Tutorials.

Statens vegvesen

Statens vegvesen Vegdirektoratet
Postboks 8142 Dep
N - 0033 Oslo

Tlf. (+47 915) 02030
E-post: publvd@vegvesen.no

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