Development of improved mechanistic deterioration models for flexible pavements

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Preface

This report contains an article written by Hans Jørgen Ertman Larsen, Danish Road Institute and Dr. Per Ullidtz, The Technical University of Denmark for the 4th International Conference on Managing Pavements held 17 - 21 May 1998 in Durban, South Africa.

Abstract

The paper describes a pilot study in Denmark with the main objective of developing improved mechanistic deterioration models for flexible pavements based on an accelerated full scale test on an instrumented pavement in the Danish Road Testing Machine. The study was the first step in “International Pavement Subgrade Performance Study” sponsored by the Federal Highway Administration (FHWA), USA. The paper describes in detail the data analysis and the resulting models for rutting, roughness, and a model for the plastic strain in the subgrade.

The reader will get an understanding of the work needed to be done to be successful in establishing mechanistic deterioration models based on Accelerated Load Testing (ALT) complemented by laboratory testing.
1. Introduction

Road pavements must be maintained in order to meet drivers’ demands for comfort and safety and to preserve road investments. Therefore models which predict the behaviour of road pavements with exposure to traffic and environmental factors over a period of time are of great interest. Typically, models are developed from mechanistic methods combined with empirical evaluation based on field, laboratory or accelerated loading tests, or from empirical methods based on field tests or accelerated loading tests. The models must be able to simulate the deterioration of a new or old pavement carrying variable traffic loads under different climatic and environmental conditions.

The work described in this paper is a part of the International Pavement Subgrade Performance Study. The “Subgrade Study” is an international road research project, which aims at improving the current mechanistic failure criterion that limits permanent deformation of the subgrade, and thereby rutting. The current subgrade failure criterion does not take soil types and moisture conditions into consideration. It is known that these factors influence the strength of the subgrade considerably, and therefore the “Subgrade Study” will investigate their inclusion in such a criterion. As the main part of the study, full-scale test sections will be constructed with five different subgrade materials. These pavements will all be instrumented, so stresses, strains, temperature and moisture contents within different pavement layers will be measured. The materials will be tested in the laboratory to develop a full understanding of the resilient and plastic behaviour of the materials. The pavement section will be trafficked by an automated loading facility and the subsequent rutting will be monitored closely and recorded.

The objectives of the International Pavement Subgrade Performance Study are:

(a) To develop an improved mechanistic subgrade failure criterion (elastic or plastic) for new and reconstructed pavements.
(b) To develop a solid basis for future research on seasonal effects of environment on pavements.
(c) To evaluate the effect of environment on resilient materials properties, especially the effect of moisture content changes over time in base course and subgrade layers, i.e., the “seasonal variability” of pavement materials.
(d) As a corollary objective, to foster international co-operation in pavement research.

Essentially, this research study is seeking to identify a superior parameter, or parameters, to replace existing vertical compressive strain criteria applied in design and analysis of subgrades.
In order to resolve a number of questions discussed by the co-operating agencies, the Danish Road Institute (DRI) in co-operation with the Department of Planning (IFP) of the Technical University of Denmark (DTU) built a full scale test pavement in the Danish Road Testing Machine to evaluate instruments and to put forward a preliminary failure criterion. The objectives of the preliminary study were to investigate instrument options, instrument installation and to study subgrade failure criteria.

In October, 1995, a test pavement was constructed and instrumented in the Danish Road Testing Machine (RTM). Instruments installed in the test pavement were Asphalt Strain Gauges, Soil Pressure Cells, Soil Deformation Transducers, emu Strain measurement system coils, Soil Moisture Probes and Agricultural Tensiometers. Accelerated loading started in December 1995, and continued until April 1996. Pavement responses under dual wheel loading and Falling Weight Deflectometer (FWD) loading were measured. Appropriate Falling Weight Deflectometer (FWD) loading levels were applied using suitable 30 cm diameter plates and geophone radii dependent upon the pavement level being tested. Pavement surface profiles and bearing capacity were regularly measured and observed, and the moisture contents and temperatures of the pavement layers were systematically monitored.

The pilot study in Denmark has since been followed by a second study of the same subgrade material in the RTM, and the US Army’s Cold Regions Research Engineering Laboratory (CRREL) has constructed the first set of instrumented pavements and conducted accelerated load testing with the new Heavy Vehicle Simulator (HVS) purchased from Dynatest (US), manufactured in South Africa. The developed deterioration models will be verified through the Minnesota Road Research Project (Mn/ROAD), USA, programme and the Road Structures Research Programme (TPPT), Technical Research Centre (VTT), Finland. The FHWA expects that the International Pavement Subgrade Performance Study will provide an important input to the ongoing work of revising the AASHTO Design Manual (1993).

The structure in this paper is as follows. After the Introduction, three main sections give a short introduction to the Danish Road Testing Machine, the test pavement structure and materials, and the instruments in the Test Pavement. Section 5 gives an understanding of classical mechanistic design methods versus deterioration models. Accelerated Load Testing (ALT) used for developing mechanistic-empirical models is the subject of Section 6 in which the reader will also find a detailed description of the developed models for rutting, roughness and for the plastic strain in the subgrade.

Most pavement deterioration prediction models have been established using mechanistic-empirical approaches based on extensive empirical data from roads, or accelerated loading tests, complemented by laboratory testing, in which deterioration models are developed by statistical regression of time-series data collected in studies of experimental or in-service pavements. Through this paper, the reader will get an understanding of the work conducted in Denmark, which is an example of
development of mechanistic deterioration models based on accelerated load testing. A recommendation to the reader based on experience gained from the present work is to develop general forms of deterioration models which may be modified to suit variability of materials characteristics and behaviour, and that deterioration prediction models should be calibrated with field data before being applied.

To the authors knowledge there are currently no pavement deterioration models available which are sufficiently developed and validated to be used in practice for all road and traffic conditions, climates and environments existing throughout the world. However, pavement deterioration models, generally suited to temperate and low temperature climates and to road and traffic conditions existing in more industrialised countries, have yet to be fully developed and validated.
2. The Danish Road Testing Machine

This chapter briefly describes the Danish Road Testing Machine. It is owned by the Danish Road Institute and the Danish Technical University in Lyngby. It has been in operation for the last 25 years in co-operation with the Department of Planning at the DTU.

2.1 Concrete Pit and Climate Chamber
The Danish Road Testing Machine (RTM) is an indoor full-scale accelerated load testing facility, with a width of 2.5 m (8.2 feet) and a length of 27 m (89 feet). The central 9 m (30 feet) is the actual test section, which is 2 m (6.6 feet) deep. A plan and section of the RTM are shown in Figure 1.

The RTM is enclosed in a climate chamber, 4 m (13 feet) wide and 3.8 m (12.5 feet) in height. Heating and cooling equipment make it possible to maintain a temperature range of -20°C (-4 F) to + 40°C (104 F). The ground water level is automatically controlled and may be raised or lowered as desired.

2.2 RTM Load Cart
The wheel load is hydraulically applied by a single or a dual wheel. The maximum dual wheel load is 65 kN (14.6 kips) and the maximum velocity is approximately 30 km/h (16 mph). 10 000 load repetitions at this load may be applied during one 24-hour day. This corresponds to approximately 70 000 passages of a standard 80 kN (18 kips) axle load. The lateral position of the wheel can be automatically changed during testing to give a desired transverse wheel load distribution (wander).

In the current project a dual wheel load was applied. The speed of the dual wheel load was approximately 20 km/h (12.4 mph). The speed of transverse movement (wander) of the dual wheel load was set at approximately two tenths of one percent of the driving speed. The width of the transverse movement (wander) was about 0.90 m.
THE DANISH ROAD INSTITUTE
ROAD TESTING MACHINE (RTM)
Climate chamber - 27.5m long, 4m wide and 3.8m high

Figure 1. The Danish linear track Road Testing Machine.
3. Structure and materials of the test pavement

This chapter briefly describes the materials used to construct the test pavement.

3.1 Design of the Test Pavement

The test pavement was carefully designed to produce a certain amount of rutting within a short time frame. Some properties of the subgrade and base course materials are tabulated and shown in Table 1 and Figure 2. The design is based on the following considerations:

- The applied dual wheel load was planned to be at two levels: 35 kN with 100 000 load repetitions and 50 kN with 25 000 load repetitions. It was estimated that each load level and corresponding load repetitions would result in a rut depth of approximately 25 mm.
- The pavement structure was set at 75 mm for the asphalt concrete, 150 mm for the granular base course, 1300 mm for the subgrade and 200 mm for the drainage layer.
- The subgrade modulus was assumed to have a value of 50 MPa, based on the laboratory test data, and the ratio of the base course modulus to the subgrade modulus initially adopted was approximately 3. The asphalt modulus was assumed to have a value of approximately 3000 MPa.
- It was intended to generate rutting in the RTM test pavement without generating significant cracking or roughness in the asphalt concrete surface.
- With the assumed pavement structure, the assumed elastic moduli of the pavement layers and the planned load levels, the expected vertical strain at the top of the subgrade, and the expected horizontal strain at the bottom of the asphalt were calculated, using the theory of elasticity. The expected vertical strain at the top of the subgrade was initially determined to be about 1100 microstrain with a dual wheel load of 35 kN, and about 1500 microstrain with a dual wheel load 50 kN. The expected horizontal strain at the bottom of the asphalt was estimated be in the range of 400-450 microstrain with a dual wheel load of 35 kN, and approximately 500 microstrain with a dual wheel load 50 kN.
- The existing surface and subgrade failure criteria were utilised to obtain the permissible vertical strain at the top of the subgrade and the permissible horizontal strain at the bottom of the asphalt.
- After construction, a FWD test was carried out on the asphalt surface. Using the actual layer thicknesses and the layer moduli backcalculated from the FWD data, load levels were recomputed.
Percent Passing #200 Sieve % | 44.1  
---|---  
Specific Gravity | 2.67  
Proctor Moisture-Density Test |  
Optimum Moisture Content % | 9.3  
Maximum Density kg/m³ | 2040  
Atterberg Limits |  
Liquid Limit % | 23.2  
Plastic Limit % | 13.0  
Plasticity Index % | 10.2  
AASHTO Classification | A-4(2)  
(borderline with A-6(2)) |  
USCS classification | SC

<table>
<thead>
<tr>
<th>3.2 The Test Pavement Materials</th>
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<tbody>
<tr>
<td>3.2.1 Drainage Layer</td>
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<tr>
<td>A porous granular material of coarse natural gravel is used. The grain sizes are in the range of 16 mm to 22 mm. No laboratory tests have been carried out on the drainage layer or the geotextile covering it.</td>
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<tr>
<td>3.2.2 Subgrade</td>
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| The subgrade soil was a Danish “moraine clay,” which comes from a sand and gravel quarry near Holbaek, west of Roskilde. The laboratory test data are shown in Table 1 and Figure 2.  

The subgrade soil used in the test pavement can be described as being a “moraine clay”. About 50% of the particles are in the sand size range. The second largest component (30%) is silt. Nearly 15% of the material is in the clay size range (finer than 2 microns), and the plasticity and dry strength are rather low.  

The California Bearing Ratio (CBR) was determined in the laboratory on unsoaked specimens using a standard Proctor compaction effort. The test results are shown in Figure 2. The CBR of this “moraine clay” is highly sensitive to the compaction moisture content. For a change in the moisture content from 7.5% to 11%, the CBR reduces from 25% to 5%; the CBR reduces further to 2% at 13% moisture content.  |
The 1300 mm subgrade was constructed in nine layers, each approximately 150 mm thick. Each layer was dried and compacted until satisfactory densities and moisture contents were measured by a Troxler nuclear moisture-density gauge. The average compaction density for the subgrade layers varies between 90.7% and 97.1% of maximum dry density, and the average moisture content varies between 10.2% and 12.2%. Most of the variation occurred between the layers.

*Figure 2. Sieve curves, Proctor compaction curves, and unsoaked CBR-moisture content relationship for base course and subgrade materials.*
3.2.3 Base Course
The base course granular material was selected from a commercial quarry near Roskilde. The granular material was a non-plastic crushed gravel with a maximum grain size of 32 mm, and an unsoaked CBR of >100% at the optimum moisture content of 7.1%. The grain size distribution, moisture-density relationship and CBR test results are shown in Figure 2. The base course was constructed in one layer to an average compaction density of 95.1% of maximum dry density and an average moisture content of 4.9%. The general compaction requirement for base course material in Danish road construction is achievement of 95% (average for 5 Samples) of the maximum dry density for the material obtained using the current Danish vibration compaction test method (DRI).

3.2.4 Asphalt Concrete
The asphalt concrete was a densely graded asphalt concrete designed by the contractor for a thin bituminous surfacing layer, 75 mm thick. After compaction, the layer had achieved 97.8% of Marshall density. Repeated Load Indirect Tensile Tests have been carried out using two types of equipment: an MTS unit and a Nottingham Asphalt Test (NAT) unit. The E-modulus determined by the MTS unit was approximately 3000 - 5000 MPa dependent on temperature (25 and 28°C) and load pulse duration (31 - 65 milliseconds). The NAT unit gave an average indirect tensile stiffness modulus of 1316 MPa at 25°C.
4. Instruments in the test pavements

This chapter describes the instruments installed in the RTM test pavement.

4.1 Soil Pressure Cells
Stresses in the unbound materials due to the applied dynamic loading were measured by means of Soil Pressure Cells (SPCs). The cells are made from Titanium using double diaphragm pressure cells. The surface of the cell is finished with a coating of epoxy and sand. The space between the two membranes is filled by incompressible oil (hydraulic liquid), which transfers the stress applied to the outer membrane to the inner. The ratio of the diameters of the two membranes are designed so that only very small deformations of the surface membrane are needed for measurement. The diameter of the surface membrane is 66 mm. According to the expected range of stress values at different depths in the structure, the thickness of the instrumented membrane is designed for specific pressure ranges.

4.2 Soil Deformation Transducers
Soil Deformation Transducers (SDT) were used to measure dynamic strains and permanent deformations in the subgrade. The transducers were placed to measure horizontal as well as vertical strain. The principle of the SDT is that when there is a deformation in a soil, the change in distance between two plates is recorded by a Linear Variable Differential Transformer (LVDT) and transmitted to the data acquisition system.

4.3 Other Instruments
Plastic and resilient strains in unbound materials were also measured by the Strain Measuring system (emu). The emu is a device for measuring the strains or displacements taking place between pairs of inductive coils. The emu system did not seem to produce results with a satisfactory high degree of precision due to noise and to a signal caused by the moving metal frame of the loading cart.

Horizontal strains at the bottom of the bituminous layer were measured by Asphalt Strain Gauges (ASG).

VITEL Soil Moisture Probes (SMPs) were used to measure the volumetric moisture content and temperature of the soil.

Agricultural Tensiometers, which consist of a porous ceramic cup and a mercury manometer, were also installed in the subgrade material to measure the soil pore pressures.
4.4 Instrument Calibration
All the installed instruments were carefully calibrated in the laboratory. The presence of a soil pressure cell may change the stresses in the soil and often the signal of the gauges will depend on the soil stiffness and the loading history. The calibration of the stress cells documented that the signal was independent of soil stiffness and of loading history. A different make of pressure cells was rejected for this study because it did not live up to this requirement.

4.5 Instrumentation of the Pavement
All instruments measuring stresses and strains, except one SPC, were located in test area A (the southern end of the test section) along two parallel lines. SPCs, SDTs and ASGs were located in these two longitudinal lines, 170 mm (6.7 in) on either side of the RTM centreline. The centre-to-centre distance between the dual tyres of the loading cart is 340 mm (13.4 in). The reason for locating the instruments directly under the centreline of the tyres is, of course, to measure the maximum responses. The main portion of the deformation was expected in the top of the subgrade, therefore the SPCs and the SDTs were installed in the three upper subgrade layers. The ASGs were placed at the interface of the base course and the asphalt concrete. In general, two instruments were used to measure each direction of stress or strain in each layer where the respective instrument was installed. This was done to obtain repetitive measurements and thereby also to provide a check on the instruments.

The emu-coils were installed in three stacks through layer 7 to layer 1 of the subgrade. One stack was installed at the centre of the test section. The coils of this central stack were cast in a sand cement concrete and were used for reference strain measurements. The other two stacks were at the centre of area A and area B, respectively. The Agricultural Tensiometers and the SMPs were installed both in area A and B at different depths in the test pavement.

4.6 Profiling and Structural Evaluation of the Pavement
During construction, the surface levels of each layer in the pavement were measured using the Profilometer. This consists of a wheeled T-frame which runs on the same rails as the loading cart. The Profilometer is a distance measuring and datalogging instrument, which is used to define the surface profiles of each pavement layer. It is equipped to measure distances in the transverse and longitudinal horizontal axes to an accuracy of +/- 0.6 mm.

The water content and the compaction in the pavement layers were measured under the construction using a Troxler type 3411-B nuclear moisture-density gauge.

The pavement responses (stresses, strains and deflections) were also measured under FWD loading. The tests were carried out during construction to check whether the instruments worked properly during and after the accelerated loading test to study the structural condition of the test pavement. The findings concerning FWD tests and a detailed description of the data collection from all the instruments have been previously published (Ertman and Ullidtz, 1997) and will not be dealt with here.
5. Classical mechanistic-empirical design methods versus deterioration models

A recent survey by the Transportation Research Laboratory (TRL) for COST 333 showed that out of the 17 European countries taking part in the survey, 13 were using a mechanistic-empirical (analytical-empirical) approach, at least as a background, for the design of new pavements. None of these methods, however, are proper deterioration models, in the sense that they can be used directly to predict the gradual deterioration of pavements, with time and loading. The design methods are used to calculate the permissible number of loads, to cause a certain amount of either cracking or of rutting and roughness of the pavement.

As illustrated in Figure 3 the classical design method will predict the number of loads, Np, to cause the initial condition, Ci, to deteriorate to the terminal condition, Ct, but it will not give any indication of the condition between these two points. It should also be noticed that the permissible number of loads is determined from the critical stresses or strains corresponding to the initial structural condition.

In a pavement management system the condition of the pavement must be known at any point in time, and in addition it should be possible to predict future condition from the actual condition at any point time, partly because the initial condition is not always well known but mainly because of the complexity of the deterioration process that cannot be taken into consideration if only the initial condition is used. A structural deterioration will change the rate of decrease of the functional condition, and vice versa if a deteriorated functional condition leads to increased dynamic loading. This process is confounded by the effects of climatic variations and ageing.

Figure 3. Classical design of new pavements.
To develop incremental-recursive deterioration models, both Accelerated Loading Tests (ALT) and real-time loading tests (RLT) are needed. ALTs can be used to develop models for closely controlled conditions of materials, traffic loading and climate, over a reasonably short span of time, but these models then need to be verified for the much more complex conditions of actual pavements.
6. Preliminary pavement subgrade performance study

If an Accelerated Loading Test is used for developing mechanistic-empirical models, instrumentation becomes essential. Stresses, strains and deflections are calculated using a theoretical model, like the theory of elasticity, which is based on assumptions that are seldom correct for pavement materials or structures. The actual pavement response may, therefore, not correspond to the theoretical response. But the correct pavement response is needed in order to relate the ALT results to laboratory tests on the one hand and to real time loading on the other. Some of the problems of instrumentation and pavement response have been previously described (Ertman and Ullidtz, 1997) and will not be dealt with here. An important finding needs, however, to be mentioned here, because it is of importance to the deterioration models described in the following. When comparing measured stresses and strains (resilient values) with linear elastic theory, horizontal strains in the asphalt and vertical stress on the subgrade matched reasonably well, whereas vertical strains did not. The subgrade material used in the test (clayey till, “moraine deposit”) had pronounced non-linear elastic characteristics. Measured vertical strains in the subgrade were found to be 2 to 3 times larger than strains calculated using linear elastic theory. The models described in the following are based on the measured strains and cannot be directly compared to existing models based on linear elastic theory.

In the preliminary test two dual tyre load levels were applied: 20 kN (tyre pressure 500 kPa) and 40 kN (tyre pressure 600 kPa) for 50,000 repetitions each. During the testing a slight decrease in asphalt modulus, from about 3200 MPa to 2500 MPa, was observed from FWD testing. The moduli of basecourse and subgrade remained almost constant during the test. Poisson’s ratio for the subgrade ranged from 0.2 to 0.4.

The permanent or plastic strains were recorded at frequent intervals during the testing. Figure 5 shows the vertical plastic strains in layers 1, 2 and 3 of the subgrade as a function of the number of load applications.

TZ in the legend indicates vertical strain, the first number is the layer number, and the rest is an indication of position. Each layer had a thickness of approximately 150 mm and the strain cell measured over the lower 100 mm of the layer. It may be noticed that the largest vertical strain is recorded in layer 2 of the subgrade, showing the large variability of the clayey till subgrade.

It may also be noticed that for the full length of the test the plastic strains are in the phase of decreasing strain rate. Within this phase it could be assumed that the vertical plastic strain, \( \mu e_p \) (in \( \mu \)strain), can be written as a function of the number of load repetitions, \( N \), and of the vertical resilient strain, \( \mu e_r \), in the following format:
\[ \mu e_p = A \times N^\alpha \times \mu e_z^\beta \]  
(Eq. 1)

Figure 5. Vertical plastic strain in the subgrade.

where \( A, \alpha \) and \( \beta \) are constants. This relationship would be in agreement with the classical design criteria. With a known resilient strain, the permissible number of loads (to cause a certain amount of plastic strain) could be calculated.

For use in an incremental-recursive procedure the following format would be more practical:

\[ d\mu e_p = \alpha \times \mu e_p \left( \frac{A \times \mu e_z^\beta}{\mu e_p} \right)^{1/\alpha} \times dN \]  
(Eq. 2)

where \( d\mu e_p \) is the increase in plastic strain caused by the loads \( dN \).

Using this incremental equation with the measured vertical resilient strains as \( \mu e_z \) the solid lines in Figure 6 are obtained with the following constants:

\( A = 0.0121, \alpha = 0.22 \) and \( \beta = 1.37 \).

In Figure 6 the mean value of the measured strain in layer \( i \) is indicated by \( M_i \) and the corresponding calculated value by \( C_i \). The agreement is far from perfect, but could
serve as a first approximation.
An approximate value for the permanent deformation at a depth $z$, may be obtained by integrating the plastic strains from $z$ to $\infty$ (Ullidtz, 1987):

$$
    d_p = \frac{\varepsilon_p}{(2\beta - 1)} \times z
$$

(Eq. 3)

![Graph showing measured and calculated vertical plastic strains.](image)

To get the permanent deformation at the top of the subgrade Odemark’s equivalent depth should be entered for $z$.

Similar relationships were derived for International Roughness Index, IRI m/km, and for rut depth, RD mm. These relationships are less useful than the relationship for the plastic strain, because they depend on the full pavement structure, rather than on a single material. With a different pavement structure on the same subgrade material the relationship might be different. The relationships are given below and the calculated values based on each of the two measured vertical strains in layer 1 of the subgrade are compared to the measured IRI and RD in Figures 7 and 8.

$$
    IRI = 0.49 \times 10^{-6} \times N^{0.23} \times \mu \varepsilon_z^{1.532}
$$

(Eq. 4)

and

$$
    RD = 1.44 \times 10^{-6} \times N^{0.23} \times \mu \varepsilon_z^{1.536}
$$

(Eq. 5)

Both of these equations were obtained using an incremental format as given by equation 2 (Eq. 2).
It is interesting to note that the value of $\alpha$ is practically the same for all three relationships, even though they were derived from three rather different measures of deterioration, i.e. from plastic strain in the subgrade, or from roughness and rutting at the pavement surface.

The damage of a load would be proportional to the load raised to $B/\alpha$, or to a power of 6.23, 6.66 and 6.67 for equations (Eq. 1), (Eq. 4) and (Eq. 5) respectively.

![Figure 7. Measured and predicted roughness.](image1)

![Figure 8. Measured and predicted rut depth.](image2)
Acknowledgement

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