PAPER # 111:

INCREMENTAL-RECURSIVE MODELING OF PERFORMANCE FOR CEMENT BOUND BASE LAYERS

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[1] COWI is an acronym formed by the (slightly transcribed) initials of Christian Ostenfeld and Wriborg Jønsson, who in 1930 founded the firm, now one of the largest engineering consultants in Denmark.

ABSTRACT

Pavement Design in Denmark has since 2005 been based on a three-tiered design guideline, where the 1st level is design according to a catalog, while the 2nd level is an automated mechanistic-empirical design. The most advanced 3rd level uses pavement performance simulation, based on incremental-recursive methodology, where the user can determine the most economical structure that will perform according to user-defined reliability requirements.

The recursive-incremental modeling of cement-bound materials is based on a number of full-scale instrumented pavement tests, performed in the Danish Road Testing Machine and at a field location in Sweden, where a Heavy Vehicle Simulator was used to perform fatigue loading on 3 pavement types, each of which were duplicated.

The development of pavement deterioration was monitored with Light Weight Deflectometer and Falling Weight Deflectometer, and pavement reactions throughout the experiments were measured with pressure and strain measurement devices.

The tests resulted in the formulation of a generalized deterioration model, based on maximum horizontal strain in the bottom of the cement bound layer, which approaches normal unbound materials fatigue laws for low-strength and PCC fatigue laws for high-strength materials. The model describes the deterioration of the material as a decrease in E-modulus, with an ever-diminishing loss rate. The general applicability of the model was verified against performance of heavy-trafficked road sections in Denmark that had been in service for more than two decades, where it was found that it can be used without any transfer functions. This fact is attributed to the insensitivity of cement bound materials to seasonal variations.

The model shows that a decrease from 100 % to 70 % over the first 20 years of a pavement's service life will typically be followed by a further loss of about 10 % over the following 100 years. The model has subsequently been further verified through large-scale tests, conducted in Poland under the EC-financed ECOserve project, under which a user-friendly comparative-design program has been made available on the web.

The outstanding advantage of the model is its ability to predict the future bearing capacity of the cement treated layer, thereby providing key information for the planning of staged construction and maintenance. It is intended to investigate the model's general validity under other climates by monitoring the performance of in-service roads in e.g. tropical climates, applying also other binders, such as naturally occurring pozzolanic materials.

INTRODUCTION

Contemporary pavement design concepts are moving towards methods that take into account the variability of traffic, pavement and environmental conditions and seek to present the user with the means for choosing a design reliability that matches the importance of the actual pavement in a greater road network context. The US-developed Mechanistic-Empirical Pavement Design Guide (MEPDG) is a prominent example of this trend.

Heavily trafficked road links can often be designed economically with cement stabilized base course layer, but it has long been a theoretical drawback that the recognized deterioration of this layer is not incorporated into the design analyses. An attempt to rectify this deficiency is made in the South African Mechanistic-Empirical Pavement Analysis and Design Software (mePADS), which defines 3 stages in the deterioration of the layers from "Pre-Cracked" over "Effective Fatigue Life" to "Equivalent Gravel". Only the two last phases are considered in the design analyses, and then with constant E-moduli, although the design guideline realizes that deterioration is actually a continuous process.

A proper design methodology that takes into account this deterioration could be based on incremental-recursive modeling that describes the deterioration of a layer as a function of the critical stresses and/or strains in the pavement. Such a model operates in increments of time and uses the output from one increment “recursively” as input for the next increment. Such a model would have the potential for use with variable conditions of traffic and climate and should enable a less conservative design.

The common experience with semi-rigid pavements is that they are well-suited and economically competitive on links carrying high traffic volumes. This is emphasized by the fact that countries like Germany, France and Spain have a rather large proportion (30-50 %) of semi-rigid pavements on their main road network.

Countries with generally lower traffic intensity have been more reluctant, primarily due to the well-known disadvantage of contraction cracking of the cement treated base (CTB) causing reflective cracking through the asphalt concrete surfacing. Denmark for example has less than 5 % of the total length of the main road network constructed with this pavement type.

Recent investigations by the Danish Road Directorate have revealed that a number of heavily trafficked motorways, constructed in the 1970’ies with a typical pavement structure of 20 cm CTB covered with 12 – 16 cm asphalt concrete surfacing have served the traffic for more than 20 years without wearing course replacements and without any reflective cracking.

These large discrepancies between expected and actual performance underscored the need to replace the previously used design methodology which was largely based on empirical observations with a more mechanistic approach, and preferably applicable in the new Danish pavement design guideline which at its most sophisticated level uses an incremental-recursive approach (1).

This paper deals with some of the full scale testing and research that has been carried out to achieve these goals.

THE DANISH EXPERIENCE WITH SEMI-RIGID PAVEMENTS

Design and Construction

A major part of the Danish motorway network from the 70'ies and beginning of the 80'ies were constructed with cement bound base layers.
The technology used was based on British specifications for dry lean concrete. The structural design of CTB pavements was as a consequence included in the Danish design guide for road pavements from 1984. The design chart from Road Note 29 was without modification transferred into a Danish chart, the only change being that the traffic load was stated as 10 ton rather than 8.2 ton standard axles.

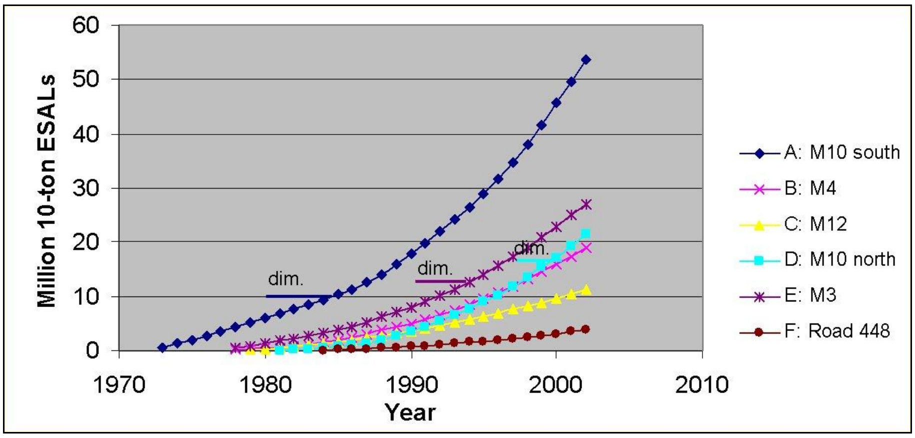
Over-performing Sections

During asphalt wearing course surveys in the mid-nineties it was realized that some pavement sections, constructed with cement bound base courses, showed no or practically no reflective cracking. The pavement sections in the Copenhagen area are shown in Figure 1.

It was furthermore found that these sections had significantly exceeded their performance expectations in terms of 10-ton ESALs (sum of axles per day in both directions), as shown in Figure 2.

Figure 1 Location of reference motorway sections around Copenhagen with cement bound base. Blue: Reflective cracking. Green: No reflective cracking.

Figure 2 Cumulative traffic on motorway sections with indication of original design traffic (dim).



In 2003, it was decided to initiate an R&D project to improve the understanding of the fatigue process in semi-rigid pavements, and develop a deterioration model for CTB materials. For this project, 3 sections, the M10 northbound, the M10 southbound and the M3 were selected for further study. The first two sections show no reflective cracking whereas the M3 section suffers from extensive reflective cracking. These sections would be used as reference sections for calibrating the deterioration model to real field conditions.

Cores from reference pavements

From the reference sections, a number of 100-mm diameter cores were available for testing. These cores were drilled in 2001 and 2002 in connection with a general pavement analysis performed prior to the upcoming motorway widening projects in the Copenhagen area.

The number of attempted cores and number of successful cores, i.e. where the cement bound layer could be taken relatively intact from the coring, are shown in Table 1. Evidently there is a clear tendency that from the section with reflective cracking all cores are intact whereas only 2/3 of the cores from the sections that have performed better regarding reflective cracking were intact, indicating a lower strength or more cracks in the cement bound layer. Many of the cores from the M10 sections were cracked to some extent whereas all cores from M3 were as intact and coherent.

Table 1 Coring on the reference sections.

Reference pavement	Attempted cores	Successful cores	Average Compressive Strength	Average Density
A: M10 south	48	32	11.0 MPa	2.19 t/m^3
D: M10 north	43	28	20.8 MPa	2.19 t/m^3
E: M3	14	14	16.6 MPa	2.16 t/m^3

Laboratory testing on cement bound base

Eleven cores from the three sections were selected for further analysis (five from M10 north and three from each of the other two sections). As it was necessary to choose intact specimens, all of these represented the best of the cores from each section.

From the ø 100 mm cores, test specimens with a height of 100 mm were sawn, dried for a minimum of 12 hours and weighed. The density and absorption of the cores was tested by weighing over and under water after one hour of submersion. A maximum 10 mm cement mortar capping layer was applied to both ends of the specimens in order to prepare for compression testing. The compressive strength and density measurement results are also shown in Table 1.

The general strength levels are higher than anticipated, also considering the initial value from the construction of the layers in the 1970'ies, where the requirement was a 7-day compressive strength higher than 5 MPa. However, the 7-day strength values from the laboratory testing were typically 6 – 12 MPa. However, it should be noted that the strength values determined here represent the maximum obtainable for the reference sections (lump strengths). Especially for the M10 sections where there was a great variability in the coring results, some parts of the cement bound layer will definitely have a much lower strength.

FULL SCALE TESTING IN THE DANISH ROAD TESTING MACHINE 2002

The first steps in the formulation of the deterioration model for hydraulically stabilized base materials were taken as part of a full-scale test in the Road Testing Machine (RTM) at the Danish Technical University (DTU). This project, funded by the Danish Foreign Aid Agency (Danida) as part of the preparations for a highway rehabilitation project in Tanzania, tested a pavement as shown below, under 20 kN, 40 kN and 60 kN wheel loadings.

Table 2 Pavement structure in Danish Road Testing Machine trials

Layer	Thickness	Comments
Asphalt Concrete surfacing	25 mm	Dense graded, max. 8 mm, Pen. 60 bitumen
Pozzolanic Stabilized Sand	160 mm – 190 mm	6 % natural pozzolan from Tanzania, 1.5 % Hydrated Lime. Initial E-modulus 1500 MPa at 95 % Mod. Proctor
Sub base, Crushed Stone	50 mm	This layer necessary for the placement of various pavement response transducers
Clay Subgrade	-	

The rather low initial E-modulus of the pozzolan-stabilized layer was not intended, but rather a result of the low compaction effort – only a 6-ton steel/pneumatic tire roller – that was possible due to the restricted space in the RTM's climate chamber, and out of consideration for the measurement instruments (pressure cells, vertical strain cells, horizontal strain gages) that were embedded in the underlying layers.

Nonetheless, the testing provided important insight into the behavior of stabilized materials. One of the more surprising was that the material retained its solid character even when it's E-modulus had been reduced to levels approaching normal, particulate granular base course materials.

Pavement reactions were monitored throughout the test, and the stiffness of the stabilized layer assessed by means of Falling Weight Deflectometer (FWD) and Light Weight Deflectometer (LWD) measurements on the pavement surface, as reported in the Danida report (4).

The Deterioration Model

The physical-mathematical formulation

An incremental-recursive process is a “calculation rule” that determines the development of the layer’s E-modulus over a number of load repetitions as a function of the layer’s “critical reaction” to the actual load, its current E-modulus and the initial E-modulus.

In continuum damage mechanics (2), cracking originates as accumulation and growth of micro-voids and micro-cracks, developing into macro-cracks. Macro-cracks will cause a reduction of the "active" cross sectional area, for uniaxial tensile stress. The stresses must be transmitted via the remaining, intact part of the area. The "damage" may be defined as the relative amount of lost area:

$$w = \frac{A_0 - A}{A_0} = \frac{dA}{A_0}$$

where 'ω' is the damage [-],
'A_0' is the original area, and
'A' is the remaining, intact area.

The damage can also be expressed as the loss of material modulus, shown below:

$$w = \frac{E_0 - E}{E_0} = \frac{dE}{E_0}$$

where 'E_0' is the modulus of the intact material, and
'E' is the average modulus of the damaged material.

Thus 'ω' is '0' for the undamaged material ($A = A_0$ or $E = E_0$).

The rate of damage is a function of actual stress or strain, and has the following format:

$$w = \left(\frac{N}{10^6}\right)^a \times \left(\frac{s \;\; or \; e}{s_{ref} \;\; or \; e_{ref}}\right)^b \times (1 - w)^g$$

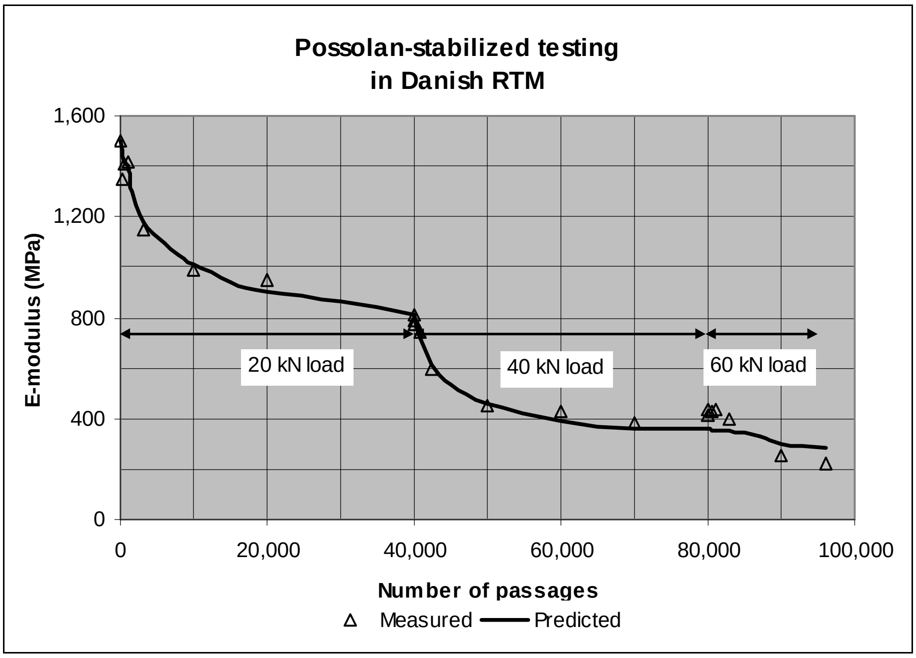
where 'N' is the number of load repetitions,
'σ' is the stress in [kPa],
'ε' is the strain in [µstrain],
'σ_{ref}' is a reference stress in [kPa],
'ε_{ref}' is a reference strain in [µstrain], and
'α', 'β' and 'γ' are dimensionless constants, [-].

On the basis of the tests, the following incremental-recursive model was developed for the E-modulus of the stabilized material:

$$w = \left(\frac{N}{10^6}\right) \times \left(\frac{e}{77 mstr}\right)^{5.6} \times (1 - w)^{10}$$

The figure below shows the development of the E-modulus of the stabilized layer, starting out at only 1,500 MPa, and deteriorating over the test period to a mere 250 MPa.

Figure 3 Measured E-moduli and deterioration model from Danish Road Testing Machine trials



HEAVY VEHICLE SIMULATOR TESTING IN SWEDEN

The project directly associated with the Danish motorway pavement experience (cf. "Over-performing Sections") was the accelerated full scale testing, performed as Heavy Vehicle Simulator (HVS) fatigue testing of 3 pavement types, divided into 6 test sections. One section of each pavement type was instrumented with transducers measuring strain in the bottom of the stabilized layers plus stresses and strains in the unbound layer immediately below the stabilized base.

Pavement characteristics were as indicated in Table 3

Table 3 Layers in test pavements

Pavement Designation	A1 & A2	B1 & B2	C1 & C2
Surfacing (30 mm)	30 mm Asphalt Concrete		
Stabilized base course (180 mm)	Well-graded gravel Max. aggregate size 16 mm, σ_C = 8 MPa	Well-graded gravel Max. aggregate size 16 mm, σ_C = 4 MPa	Well-graded sand Max. aggregate size 8 mm, σ_C = 4 MPa
Subbase (200 mm)	Sandy/silty moraine		
Subgrade	Crushed stone embankment > 2 m		

The pavements A1, B1 and C1 were instrumented along the centerline as follows:

- 6 strain gauges (SG), measuring longitudinal strain in the bottom of the stabilised base course
- 3 soil pressure cells (SPC), measuring vertical stress approx. 20 mm below the surface of the subbase

- 2 soil strain transducers, LVDT type (SST) measuring vertical strain between 20 mm and 120 mm below the surface of the subbase.

Paving
The 3 x 30 m test section was constructed on the 28 August 2003 at a motorway building site near Fagerhult, Sweden. The CTB layer was applied with an asphalt paver and compacted with an 8-ton roller. The paving width was 3.5 m and intended paving thickness 18 cm. CTB layer thicknesses were measured at the side of the layer, and were found to 17 – 18 cm at the left side of the pavement and 12.5 – 15.5 cm at the right side.After paving a bitumen emulsion was applied to prevent drying of the CTB layer.

A 30-mm asphalt wearing course was applied on the 18 September, i.e. three weeks after paving of the CTB layer.

Compaction control
After laying the cement bound base layer Nuclear Density Gauge measurements were made both before and after roller compaction. Before the first roller pass the compaction level for all three sections was 80 – 85%. After up to ten static passes the compaction level for section C was raised to around 95%. After the final 2 – 3 passes with vibration the compaction levels were as shown in Table 4

Table 4 Compaction control by nuclear density gauge and sand replacement

Section	Compaction [% standard Proctor]	Standard deviation [% standard Proctor]	Moisture content [%]
C	95.9	0.6	6.9
B	95.7	1.4	6.7
A	97.6	0.4	6.9
Compaction values measured with Nuclear Density Gauge			
	Maximum Dry Density [t/m^3]	Compaction [% standard Proctor]	Moisture content [%]
C	2.06	97.2	6.3
B	2.12	97.8	6.1
A	2.16	99.7	6.6
Compaction values measured by sand replacement			

Loading and measurement procedures
Fatigue loading on the pavements was carried out with a dual wheel, C-C distance 300 mm, tire pressure 0.8 MPa. During the 1st loading series all pavements were given 30,000 passages at a wheel load of 30 kN. A 2nd loading series was carried out on some of the pavements at 30 kN or 60 kN load, pending the outcome of the first loading series.

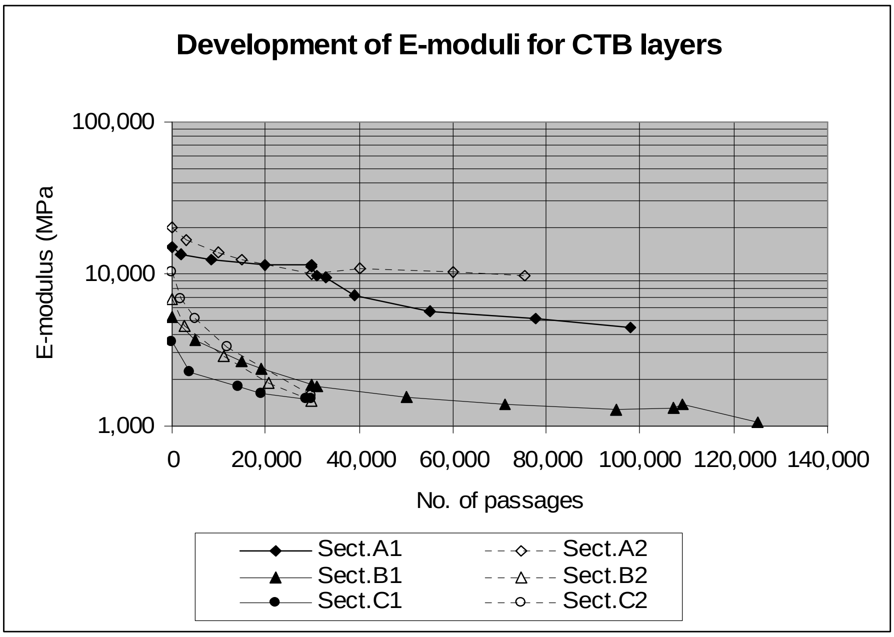
FWD measurements at 50 kN was carried out before loading commenced (23 October) and after 1st and 2nd loading series (21 November and 11 December) in 11 positions on each pavement (corresponding to the instrument positions on the instrumented pavements).

LWD (Light Weight Deflectometer) measurements at a load level of 10 kN were carried out with a Prima 100 LWD at the same times as the FWD measurements plus at variable intervals during fatigue loading, shortest intervals in the beginning where changes were expected to occur most rapidly.

Instrument recordings were taken roughly at the same time as the LWD measurements, with the wheel positioned centrally and with one tire over the instrument line, plus at 50 mm intervals moving from one side to the other.

The deterioration of the stabilized layers can be determined through combining the FWD and LWD measurements. The LWD measurements were used to interpolate between the E-moduli determined for the CTB layer from the FWD measurements, assuming unchanged elastic properties of the other layers in the pavements, resulting in the values shown in Figure 4.

Figure 4 E-moduli of CTB layers determined from FWD and LWD measurements



Incremental-recursive Model
The deterioration was then modeled according to the principles, developed in the previously described project in the Danish Road Testing Machine.

For the actual stabilized materials is chosen the longitudinal strain in the bottom of the layer as “critical reaction”. This value is chosen because:

- It will under normal passage conditions be the largest strain in the layer.
- It is fairly constant over the entire width of a dual wheel.
- It is relatively insensitive to variations in the E-modulus of the layer below.

The chosen model hereby becomes:

$$w = \left(\frac{N}{10^6}\right)^a \times \left(\frac{e_{horizontal}}{e_{ref}}\right)^b \times (1-w)^g$$

Determining the model constants
The calculation method chosen was the simplest possible, namely Boussinesq theory combined with the Method of Equivalent Thicknesses (MET), with input E-moduli as indicated above. Other parameters applied were:

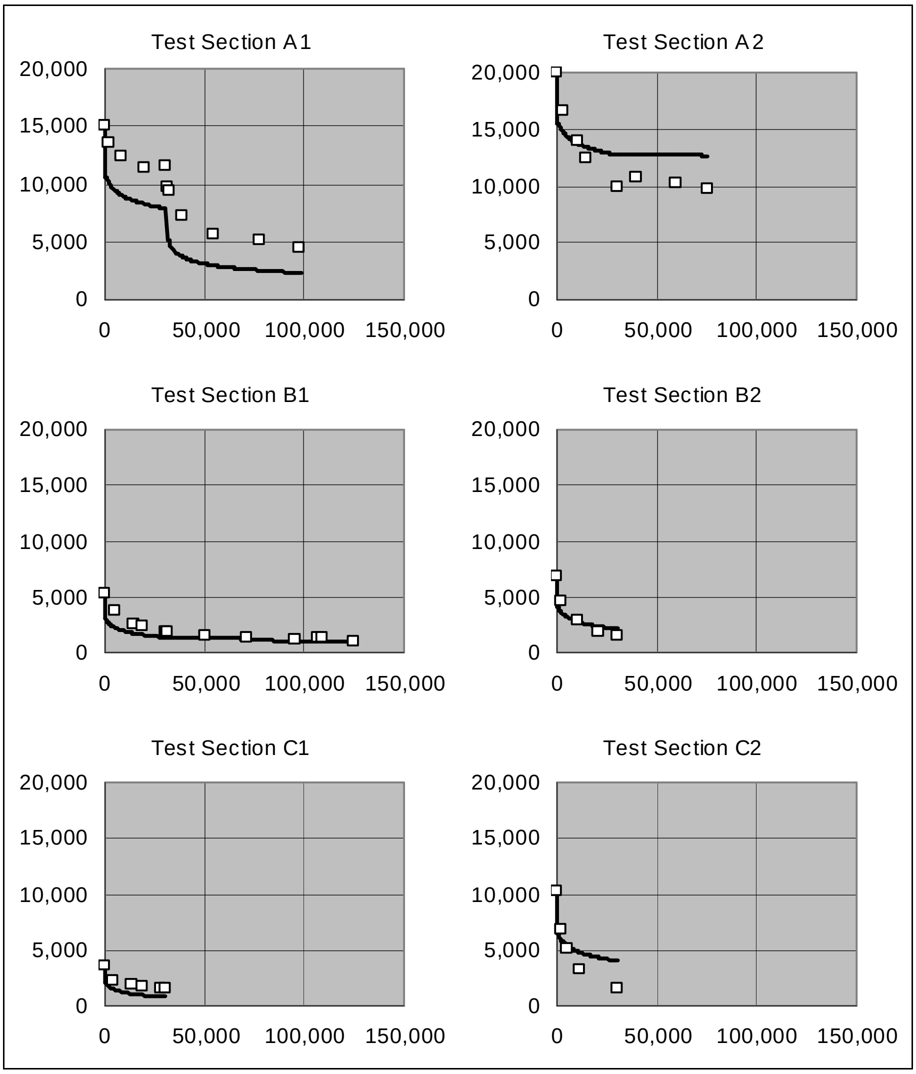
Asphalt Thickness:	30 mm
Stabilized Base Thickness:	180 mm
Poisson's Ratio:	0.35
MET Correction Factor:	0.9

The determination of the model constants was carried out as a stepwise process, using an Excel spreadsheet.

The first step is determination of the constants α, β, γ and ε_{REF} separately for the 6 test sections to obtain the closest possible fit for each section. Having done this, it became apparent that α and ε_{REF} could be fixed as constants, whereas β and γ could be described as functions of the initial E-modulus of the stabilized layer. The final step was an optimization of the combined fit for all test sections, varying the 6 constants slightly. The fit obtained was a central fit, so that 50% of the points fall below and 50% above the regression line.

The resultant correspondence between measured and model values is shown in Figure 5

Figure 5 *Deterioration predictions against measured values. X-axis indicates No. of passages, while Y-axis shows measured E-moduli (points) and incremental-recursive predictions (curves)*



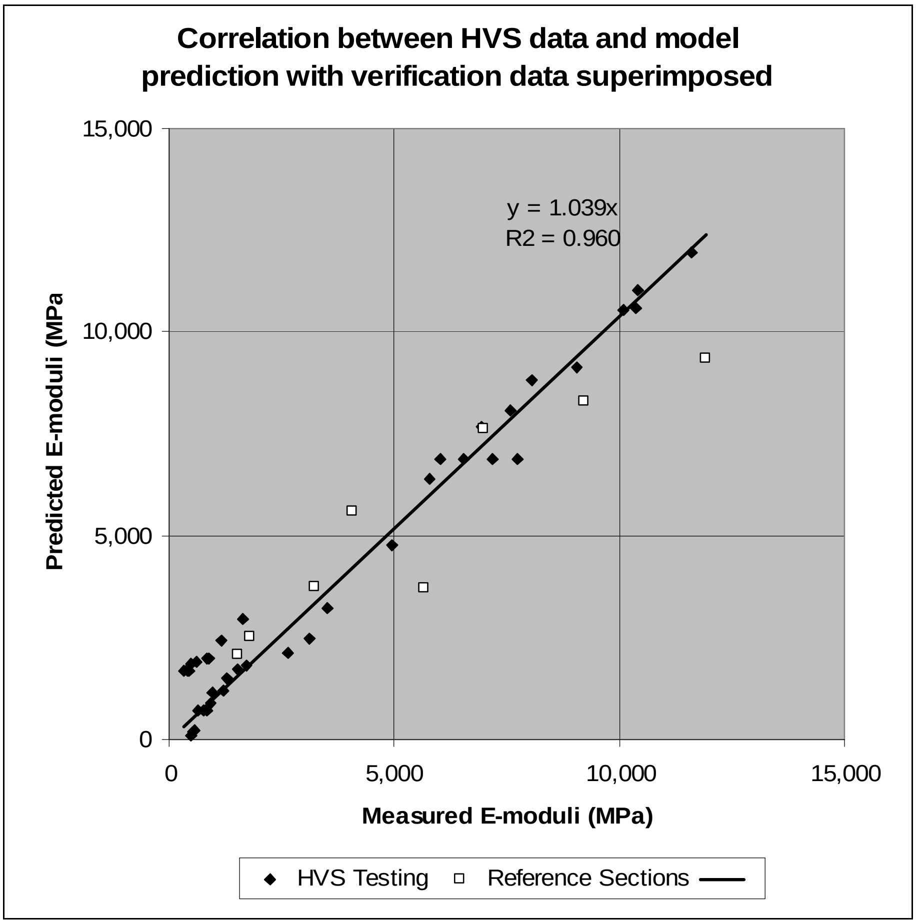
Verification against in-service sections

The final step in the process was validation of the model against actual motorway data, which were available for 4 sections on the motorway net around Copenhagen (Figure 1) as shown in Table 5. The data from this table is plotted together with the model data in Figure 6. The verification data plot on both sides of the model regression line, indicating that there is no basis for determining a shift factor from accelerated testing to actual performance.

Table 5 Verification Data

Section	E-modulus	Cumulated 10-ton ESALs
A, RHS, Middle Lane, 2002	9,207 MPa	2.7×10^6
A, RHS, Heavy Lane, 2002	4,062 MPa	2.7×10^7
A, LHS, Middle Lane, 2002	3,231 MPa	2.7×10^6
A, LHS, Heavy Lane, 2002	1,533 MPa	2.7×10^7
D, RHS, Heavy Lane, 1990	11,939 MPa	1.8×10^6
D, RHS, Heavy Lane, 1998	6,977 MPa	6.7×10^6
D, LHS, Heavy Lane, 1990	5,673 MPa	1.8×10^6
D, LHS, Heavy Lane, 1998	1,806 MPa	6.7×10^6

Figure 6 Measured vs. model E-moduli for CTB



The final incremental-recursive model then becomes

$$w = \left(\frac{N}{10^6}\right)^a \times \left(\frac{e}{e_{REF}}\right)^b \times (1-w)^g$$

where

N is the number of load repetitions,

ε is the maximum horizontal strain in the bottom of the stabilized layer

and the constants are as follows:

α = 0.25

β = 0.25 + 0.90 × ($E_{INITIAL}$/10,000 MPa)

γ = 0.05 + 0.90 × ($E_{INITIAL}$/10,000 MPa)

ε_{REF} = 45 µstr

For deterministic design purposes (see"practical applications"), a central model is not always satisfactory, since some measure of safety is normally required. Normal Danish design models are based on 25% percentiles of E-moduli for the pavement layers, i.e. 75% of the initial E-moduli will be above the design values. If a similar line of reasoning is applied to the incremental-recursive model, it should predict values, where only 25% of the measurements from the HVS sections fall below the prediction. This objective can be achieved by reducing the α –value to 0.19.

FULL SCALE TESTING IN THE POLISH "STEND" FACILITY

Pavement Preparation

During the years 2002-2006 a group of more than 50 companies and organizations cooperated in the European Thematic Network ECOserve. The network worked on improving technologies within the cement concrete based sector of the Road Construction Industry, with the aim of reducing CO2 emissions and other pollution effects, and at the same time improving resource utilization and advancing the knowledge on design and construction of pavements with cement bound base courses.

A key element in the pavement design work was a full-scale verification of the performance model for cement bound base course materials. The testing was conducted in the "STEND" testing rig at the Institute of Roads and Bridges (IBDiM) in Zmigrôd, Poland. The STEND system can apply loads of up to 1,000 kN through two 400-mm stroke servos (5). The servos can operate alone or linked, and the load pulse can be programmed to a variety of shapes, for the actual project a half-sine with a duration of 1 second was chosen, the rest period between two pulses also being 1 second.

The test pavements were built side by side on a foundation of clayey gravel. The material had a maximum dry density of 2.034 t/m^3 (Modified Proctor) at optimum moisture content of 7.4 %. The 1-m thick foundation was built in 4 lifts, obtaining compaction degrees between 95 % and 97 %.

Two types of Cement Treated Bases (CTB) were prepared, a sandy (A) and a more gravely material (B), (Figure 2, Tables 1 and 2) of maximum densities 2.17 g/cm^3 and 2.29 g/cm^3, respectively.

Figure 7. Gradations, CTB aggregates

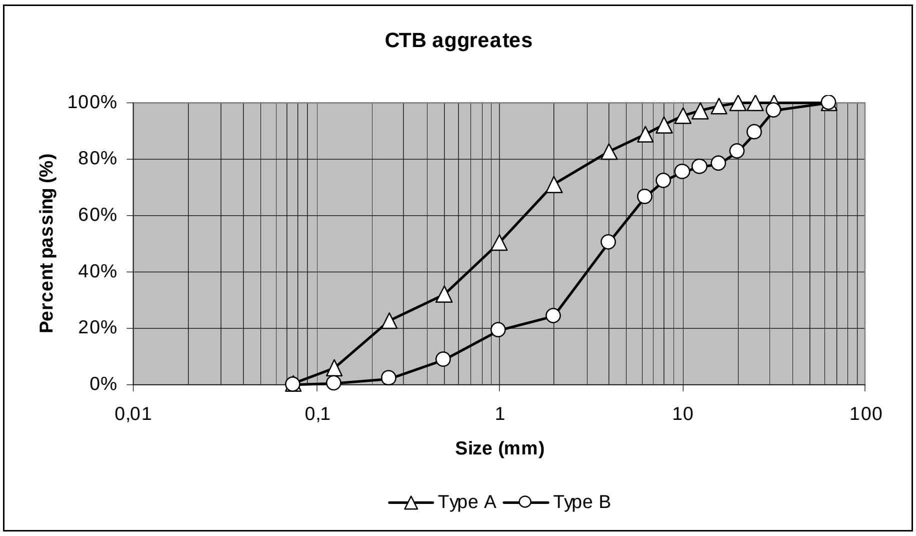


Table 6. Compressive strength testing

	Mix Type A (5.3 % cement)		Mix Type B (4.4 % cement)	
	Average	Standard Deviation	Average	Standard Deviation
	MPa	MPa	MPa	MPa
Cores 7-day	6.68	0.20	4.97	1.36
Cores 14-day	6.44	0.08	6.61	0.77
Cores 28-day	8.34	1.42	6.72	0.11
Proctor Cylinders 28-day	10.22	1.77	13.18	0.66

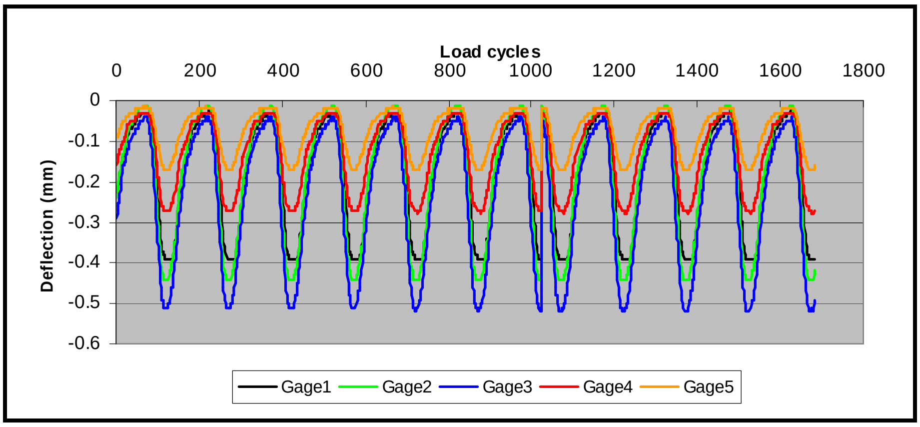
The surfacing was a 35-mm pre-fabricated asphalt wearing course, supplied by one of the Cluster member companies, Dura Vermeer, Netherlands.

After a 1 month hardening period the pavements were fatigue tested at 4 loading positions with pulse loadings at 150-kN and 250-kN levels.

Surface deflections from the 5 gages (3 at a distance of 235 mm and 2 at a distance of 450 mm from the load centre) were recorded as time series, with approximately 220 points per second. A typical output history plot is shown in Figure 8. The apparent discontinuity at 1050 load cycles is caused by the fact that readings were recorded in sequences of 1024 points, which were then stored before the next reading could be initiated. The data were organized in spreadsheets with semi-automatic detection routines for successive maxi-mum and minimum deflection readings, calculating from these the actual displacement caused by the load. For each recording number (N-value) a total of 10 to 30 displacement values were recorded and combined into an average deflection for each gage.

On each of the two pavement types were conducted fatigue loadings at two positions, termed "Inner" and "Outer", indicating whether they were closer or farther away from the middle of the combined sections. The pavement slabs were termed "Pavement A" and "Pavement B", corresponding to the CTB materials.

Figure 8 Typical time history recording



Deflection analysis

Deflection analysis, based on the recursive-incremental model was done with a spreadsheet program using Boussinesq theory, modeling the pavement as a 3-layer system:

- 35 mm asphalt, E-modulus 1,000 MPa
- 180 mm CTB, E-modulus variable
- Subgrade E-modulus variable in range 350 - 450 MPa

Analysis of the measured deflections was then carried out in the following manner:

- For each of the series (Pavement A - Inner position, Pavement B - Inner position etc.), the deflections were analyzed simultaneously, varying the subgrade and CTB parameters until satisfactory agreement between measured and calculated deflections was obtained.
- In one analysis - same pavement type, inner and outer positions) - identical modeling for the CTB deterioration was applied.
- Same model constants, except initial E-modulus, was used for one pavement (A or B) in both 235-mm and 450-mm analyses.

Examples of these analyses and the predicted deterioration are shown graphically below.

Figure 9. Deflection match as 235-mm distance

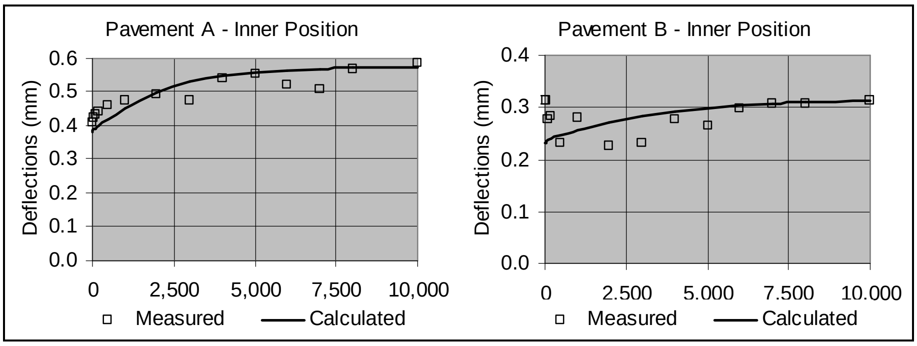
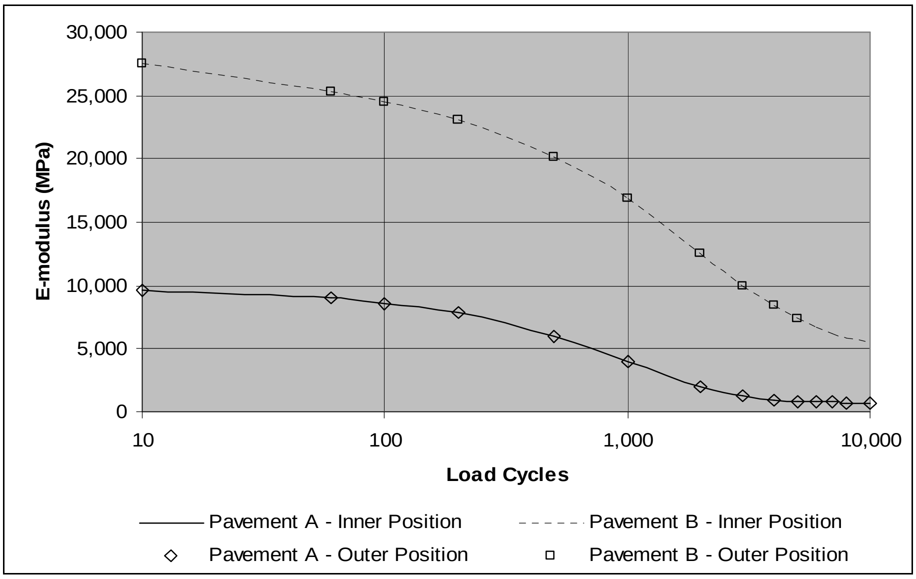


Figure 10. Deterioration development at 235-mm distance



The tests confirmed that the material deterioration was in agreement with the deterioration model, and that the rate of deterioration was higher for the low-quality, sandy Type A material than for the high-quality gravely Type B material. The initial E-modulus of the Type A material was found to be lower or equal to the initial E-modulus of the Type B material, although the Type A material has the highest 28-day compressive strength. This was also in agreement with the expected difference between sandy and gravely CTB materials.

DETERMINISTIC DESIGN CRITERION

Analysis of Incremental-recursive Modeling

An incremental-recursive model is not a practical tool in mechanistic pavement design. This requires a traditional deterministic design criterion of the form:

$$\varepsilon_{PERMISSIBLE} \quad = \quad A \times (N/10^6)^{-1/B} \times (E_{INITIAL}/E_{REF})^C$$

Using the incremental-recursive model with different pavement structures and load combinations, it is possible to predict deterioration histories from different initial tensile strain conditions as shown in Figure 11.

Based on these curves, it is possible to develop traditional deterministic design criteria that relate initial strain to allowable number of load repetitions. Figure 12 shows such criteria, based on terminal E–moduli of 33%, 25% and 20% of the initial value.

Similar criteria can be developed for other initial E-moduli, leading to generalized criterion equations, where the -1/B term approaches that of typical unbound criteria for very low E-moduli, and gets close to the values normally associated with Portland Cement Concrete for high E-moduli.

The 25 µstr curve in Figure 11 also demonstrates that when it takes 800,000 passages to cause a 30 % loss in E-modulus, the next 10 % loss will not occur until after a total of 6 million passages, i.e. if the first 30 % loss comes over 20 years, theoretically it will take another 130 years for the E-modulus to decrease another 10 %. Caution and differences in the detailed parameters for various material types dictates that this prediction should be reduced to 100 years for the next 10 %.

Figure 11 Deterioration development from different initial strain conditions

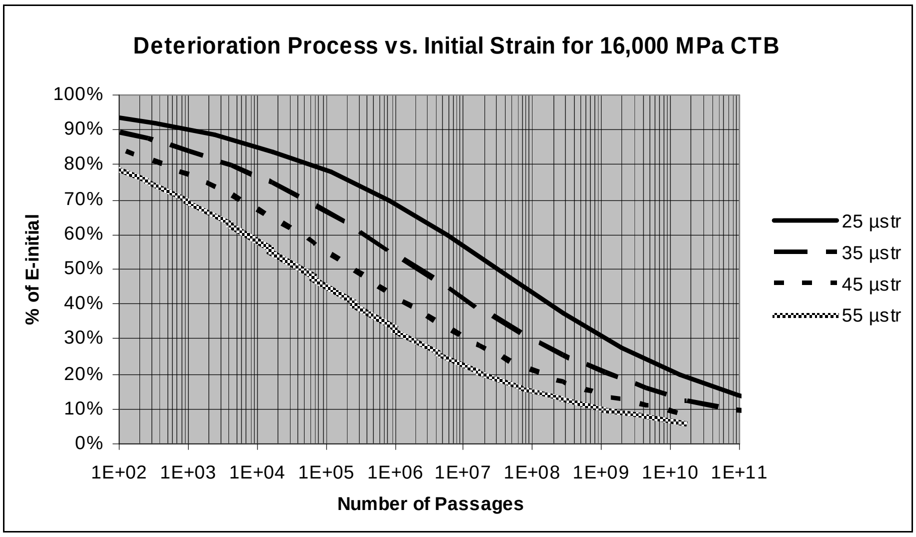
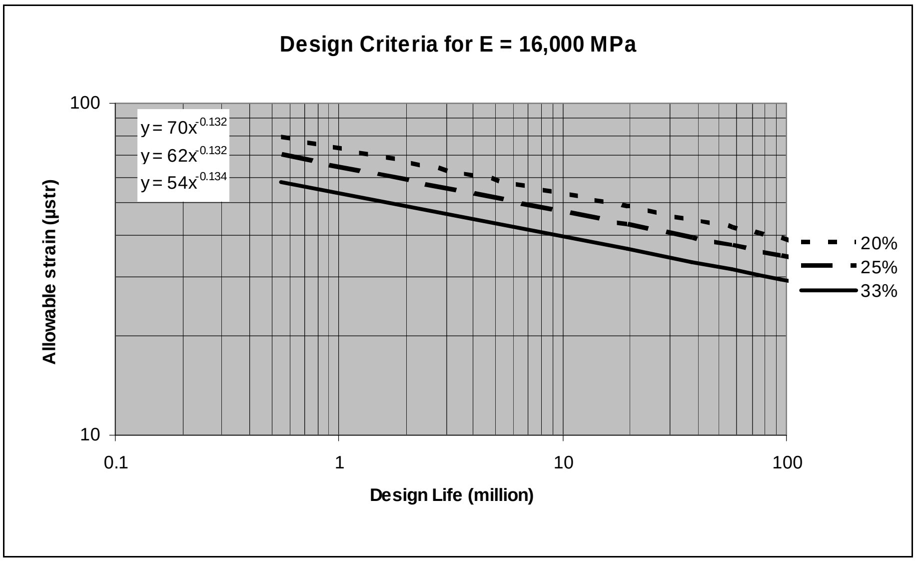


Figure 12 Fatigue criteria for $E_{INITIAL}$ = 16,000 MPa



PRACTICAL APPLICATIONS

Danish Design Guidelines

The Danish pavement design guidelines consist of a printed report with a pavement catalogue plus a computer program, capable of providing traditional mechanistic-empirical design of all common pavement types, utilizing standardized materials. As for CTB materials, the following selection of materials is available:

Table 7 Stabilized materials in Danish pavement design guideline

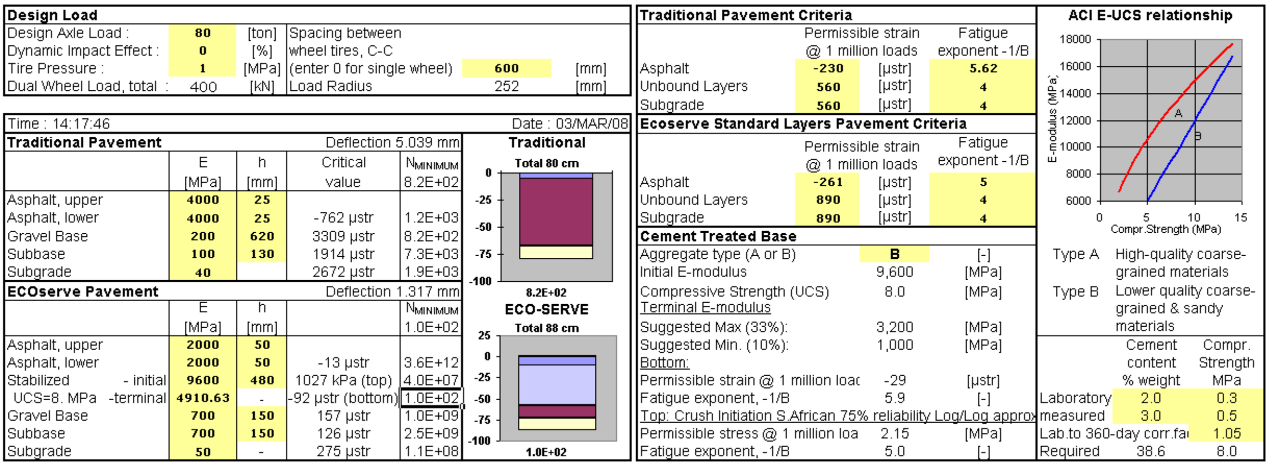
Stabilized Material	Compressive Strength, f_C	$E_{INITIAL}$ (MPa)	$E_{TERMINAL}$ (MPa)	Criterion Equation (N= Number of 10-ton standard axles)
Gravel	10 MPa	15,000	2,000	$\varepsilon_h = -0.000090 \times (N/10^6)^{-0.125}$
Gravel	8 MPa	13,000	2,000	$\varepsilon_h = -0.000075 \times (N/10^6)^{-0.139}$
Sand	6 MPa	7,000	1,000	$\varepsilon_h = -0.000070 \times (N/10^6)^{-0.213}$

The program also includes an incremental-recursive simulation component, but the stabilized materials have so far not been implemented there.

The ECOserve Pavement Design Program

As part of the thematic network ECOserve (cf. Full scale testing in the polish "stend" facility), a generalized pavement design spreadsheet program was developed, incorporating the generalized criterion equations. The program allows the user to design a traditional flexible design specifying traffic in terms of single- or dual wheel, and selecting appropriate fatigue criteria. He can then compare this to designs for a wide range of well-graded or uniformly graded stabilized materials, selecting the initial and terminal condition (E-modulus) of the stabilized material. Well-documented relationships between E-moduli and compressive strength (6) then help the user to set up specifications for the stabilized material. If tests relating cement content and strength are available, the user can also calculate the required amount of cement. The figure below shows the user interface of the program.

Figure 13 ECOserve pavement design program



CONCLUSIONS

The three projects described above have each contributed significantly to a better understanding of the macro-behavior of cement- or similarly stabilized materials.

It is, however, obvious that the deterioration models are derived on the basis of a very limited data material for accelerated pavement testing, and that the verification against in-service pavements also could be better. This could also improve the reliability considerations, which so far have only been based on theoretical analyses.

On the other hand, the fact that the models lead to criteria that conforms to PCC criteria for high E-moduli, and unbound materials criteria for low E-moduli, lends significant credibility to the analyses. This is finally enhanced by the circumstance that the criteria actually lead to pavement designs that are in agreement with empirical expectations.

With specific emphasis on Accelerated Pavement Testing, it may be stated that:

- Even projects, where then prepared pavements have properties that are far from those desired during the planning phase, can yield meaningful results.
- When testing stabilized materials, it is important to take the time factor into consideration - pavements, left unloaded over a period - a phenomenon that seldom occurs for in-service pavements, can regenerate considerably, which makes analysis difficult.
- Test pavements should be duplicated
- Testing of full-scale pavement structures in stationary rigs (fixed load position) can only confirm the general tendencies of fatigue laws, but are not useful for more sophisticated analyses, since the fixed loading position leads to different deterioration rates according to the distance from the load center.

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