Simulation of the WesTrack Experiment Using CalME

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ABSTRACT

The computer program known as CalME was developed as part of the California Department of Transportation's (Caltrans') endeavor at adopting Mechanistic-Empirical (ME) methods of pavement design. CalME has three levels of flexible pavement design; for new pavements as well as for rehabilitation. Level one is the existing empirical methods presently used by Caltrans. Level two is a "classical" ME approach, largely based on the Asphalt Institute's method, and level three is an incremental-recursive approach which allows Caltrans to validate and/or calibrate different mechanistic and empirical models.

The models of CalME have been calibrated using Heavy Vehicle Simulator (HVS) tests on new pavements as well as on overlaid pavements. This paper describes the validation and calibration done using the Federal Highway Administration's (FHWA's) project commonly referred to as WesTrack. WesTrack was subjected to frequent Falling Weight Deflectometer (FWD) testing and rutting, roughness and distresses were also recorded. Additionally, frequency sweep and resilient modulus tests were done on the asphalt materials, used in constructing WesTrack pavements, to characterize the master curves, and fatigue tests and shear tests were also conducted to determine the constants of the fatigue damage and permanent deformation models. Similarly, triaxial tests were done on the unbound materials.

For each of the 26 original sections the results of the WesTrack experiment were imported to the CalME database, and the experiment was simulated, hour by hour, using the incremental-recursive method with the model parameters derived from laboratory tests.

The measured FWD deflections were compared to the deflections calculated by CalME, to ensure that the pavement response was predicted reasonably well, for the duration of the experiment. The empirical parts of the ME models were then calibrated, so that the predicted performance would closely match the measured performance.

For prediction of asphalt fatigue a shift factor between laboratory fatigue and in-situ fatigue was determined, ranging from 5 to 15 depending on the type of asphalt mix. For permanent deformation of the asphalt layers the permanent strain determined from Repeated Simple Shear Tests – Constant Height (RSST-CH) in the laboratory should be multiplied by a factor of 80 to 90, depending on the mix type, to result in the measured permanent deformation of the asphalt, in mm. The models used for unbound materials in the HVS experiments were confirmed.

Keywords: Mechanistic-Empirical, Calibration, Cracking, Rutting

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Introduction

Performance data from the Federal Highway Administration project commonly referred to as "WesTrack" (Monismith et al. 2000, Epps et al. 2002) was used to calibrate some of the sub-models in the program CalME. CalME is a (Beta version) pavement design program developed for the California Department of Transportation (Caltrans). CalME was designed to provide a vast number of features that are essential for analyzing and designing flexible pavements (both new and rehabilitation), among which:

1. Three methods of design and analysis.
2. A GIS based input of climate zone, Weigh in Motion (WIM) data and subgrade soil type in California
3. A database library of materials with model constants for fatigue damage, shear resistance, aging etc.
4. Probabilistic analysis using Monte Carlo simulation technique
5. Accounts for the effect of pavement damage on materials strength.

The CalME software complements MEPDG (NCHRP, 2004) in a number of ways:

1. Prediction of permanent deformation in asphalt materials may be based on a shear model.
2. Models may be calibrated using HVS and test track data where pavement damage results in increasing deflections.
3. The effects of changing stiffness of the asphalt layers on the stiffness of the unbound materials may be included.
4. For rehabilitation design moduli backcalculated from FWD tests, using a companion program called CalBack, may be directly imported into CalME.

The program has three levels of design:

1. Caltrans current empirical methods, the "R-value" method for flexible structures and the "Deflection Reduction" method for rehabilitation design,
2. a "Classical" Mechanistic-Empirical design, largely based on the Asphalt Institute method, using ESALs and a weighted mean annual environmental condition, and
3. an Incremental-Recursive model based on the Mechanistic-Empirical method (IRME). In this model, the materials properties are updated in terms of damage for each time increment, using the "time hardening" approach, and used (recursively) as input to the next time increment. This approach predicts the pavement conditions at any point in time during the pavement life. The IRME mode can also be used for simulating HVS or test track experiments.

More details on the sub-models of CalME and on the calibration of these models using HVS experiments can be found in a companion paper (Ullidtz et al. 2008) and in a report prepared for Caltrans (Ullidtz et al. 2007). A flowchart is shown in Figure 1.
This paper reports on the use of the IRME mode for simulation of the WesTrack experiment.

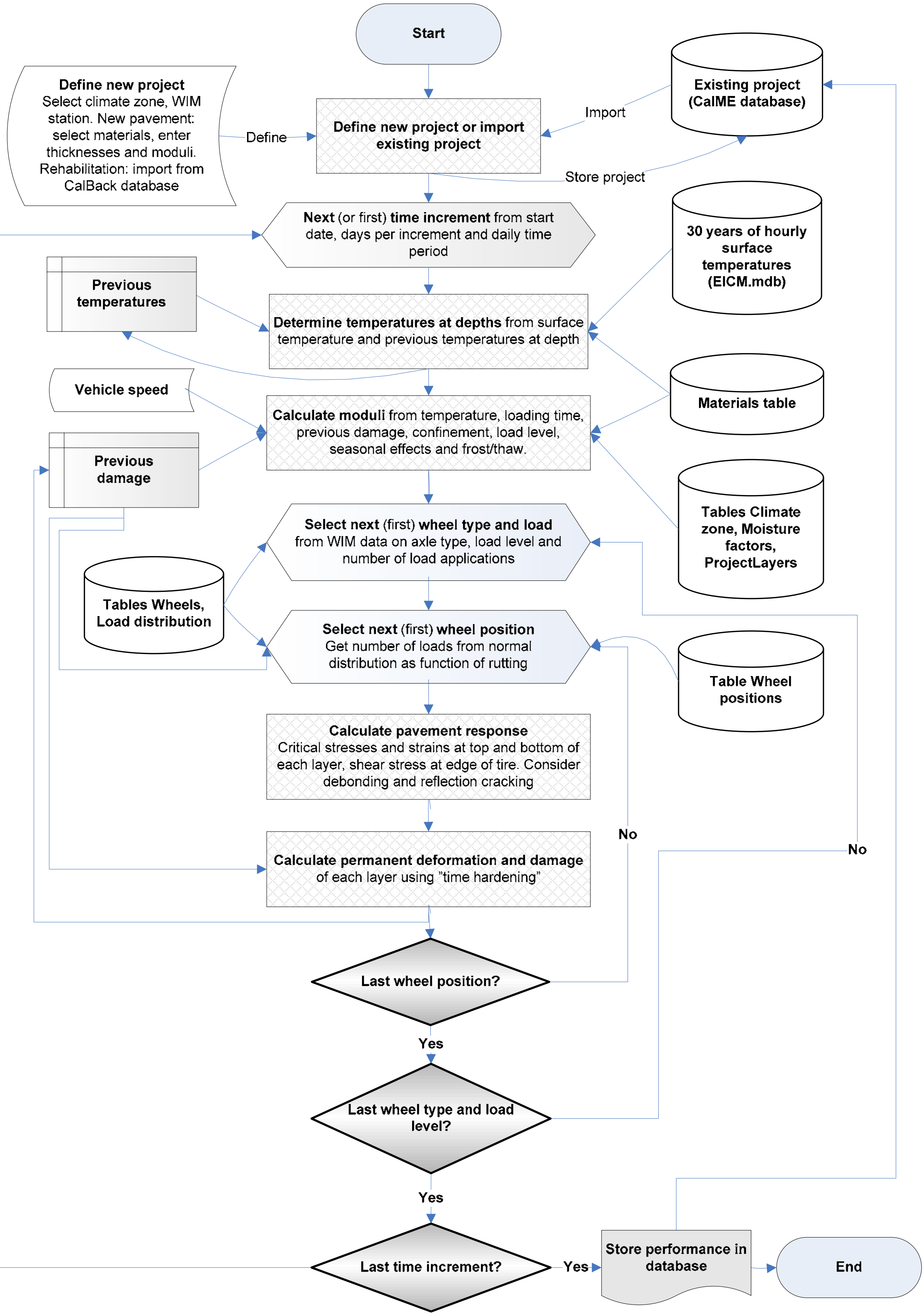

Figure 1 Flow chart for CalME

The WesTrack Experiment

The experimental test road facility, referred to as WesTrack, was constructed at the Nevada Automotive Test Center (NATC) near Fallon, Nevada (USA), under the Federal Highway Administration (FHWA) project "Accelerated Field Test of Performance-Related Specifications for Hot-Mix Asphalt Construction". The experiment had two primary objectives. The first was to continue development of performance-related specifications for hot-mix asphalt (HMA) construction by evaluating the impact of deviations in materials and construction properties from design values on pavement performance in a full-scale, accelerated field test. The second was to provide some early field verification of the Superpave® mix design procedures. Because the WesTrack site typically experiences less than 100 mm of precipitation per year and no frost penetration, it was well suited for evaluating the direct effects of deviations of materials and construction properties on performance. WesTrack was constructed as a 2.9-km oval loop incorporating twenty-six 70 m long experimental sections on the two tangents. The pavement cross sections consisted of various asphalt concrete mixes placed on a design thickness of 300 mm of aggregate base, with a thick layer of "engineered fill" below, sometimes referred to as the subgrade in this paper. The design thickness of the HMA layer (referred to as asphalt concrete [AC] in this paper) in all sections was 150 mm, placed in two 75 mm lifts. Construction was completed in October 1995; trafficking was carried out between March 1996 and February 1999. During this period, four triple-trailer combinations composed of a tandem axle, Class 8 tractor and a lead semi-trailer followed by two single-axle trailers, operated on the track at a speed of 64 km/h, providing 10.3 equivalent single-axle load (ESAL) applications per vehicle pass. The use of autonomous (driver-less) vehicle technology provided an exceptional level of operational safety and permitted loading to occur up to 22 hours per day, 7 days per week.

The experimental variables were in the asphalt concrete mixes, and included 3 levels of asphalt content, three levels of air void content, and three aggregate gradations (Coarse, Fine and Fine Plus). The main performance variables were rut depth and percentage of the wheelpath area with fatigue cracking. Approximately 4.95 million ESALs were applied during the trafficking period. Several original sections failed early in the experiment; they were replaced with a mix design that duplicated the Coarse gradation mix experiment in the original construction, but changed the aggregate source. The replacement sections were constructed in June 1997 after the application of approximately 2.85 million ESALs. Only the 26 original test sections are considered in this paper. The experiment yielded clearly differentiated levels of permanent deformation and fatigue cracking among the experimental sections.

All of the initial 26 test sections used the same aggregate source and binder in the asphalt concrete. The symbols used in this paper for the three different gradations are: Fine (F), Coarse (C) and Fine Plus (P). The Fine mixes had a Superpave aggregate gradation that passed above the "Restricted Zone" in the Superpave mix design system. The Fine Plus mixes had a gradation that was slightly finer than the Fine gradation. The Coarse mixes had a gradation that passed below the Restricted Zone. For each mix type there were sections with high (H), medium (M) and low (L) asphalt content with target values of 4.7, 5.4 and 6.1 %, respectively for Fine and Fine Plus mixes and 5.0, 5.7 and 6.4 % for the Coarse mix, and with high (H), medium (M) and low (L) air voids content with target values of 4, 8 and 12 %, respectively.

In the naming system used for each section in this paper, "FML", for instance, indicates a section with a Fine (F) mix with a Medium (M) AC content and a Low (L) air voids content (a 1 or 2 following the mix name would indicate whether the section was the first or the second of replicate sections for those cells that had replicates). Measurements taken during the WesTrack experiment and used in this study included Falling Weight Deflectometer (FWD) deflections, pavement temperatures at several depths in the asphalt concrete, and pavement distress condition surveys following the LTPP protocol.

Characterization of Materials

Some of the most important models used for characterizing the materials are described in the following. Other secondary models are described elsewhere (e.g., Ullidtz et al., 2007).

Asphalt Moduli

Asphalt moduli were obtained from a number of different test methods. The largest amount of data was from backcalculation of FWD tests done during the experiment. The backcalculation of layer moduli was done using the program Elmod5 (Dynatest, 2005) with a constant non-linearity of -0.2 for the subgrade. Backcalculation of the asphalt layer moduli was done for all of the FWD test series, and for the test positions between the wheel paths as well as in the right wheel path.
Indirect tensile tests were done at University of Nevada, Reno (UNR), at "Time Zero Construction" (in September 1995), "Time Zero Traffic", "12 Months Traffic" and "Post Mortem", for some of the sections. The moduli were measured at 25 ºC with a 0.1 sec haversine load pulse, but were converted to the reference temperature of 15.4 ºC and 15 msec load pulse duration, corresponding (approximately) to the FWD load in terms of creep test loading time.
The observed increase in moduli during the experiment was sometimes very large, in some cases showing a doubling of the modulus. An increase in modulus caused by aging of the binder would be expected to be most pronounced shortly after construction, but there is no increase in modulus from "Time Zero Construction" to "Time Zero Traffic". This would indicate that most of the hardening is due to decrease in air voids caused by post compaction. Comparison of values at "12 Months Traffic" and "Post Mortem" (i.e., after trafficking had been completed) also indicates that the hardening of the asphalt occurred within the first 12 months of trafficking.
Repeated Simple Shear Tests at Constant Height (RSST-CH) were done on the original asphalt and after trafficking (post mortem). These tests confirm the large increase in modulus observed during the experiment. The ratio between the hardened shear modulus (Gpm) and the original shear modulus (Go) is shown in Table 1 for the Fine mix. The hardening is quite similar to what was found from the indirect tensile tests.

Table 1 Hardening from shear tests

Mix	Gpm/Go
FLH	2.59
FLM	1.72
FMH	2.73
FMM	1.90
FML	1.16
FHM	2.03
FHL	0.55

Frequency sweep tests on beams were carried out by University of California, Berkeley (UCB), but only for one sample of each of the Fine, Coarse and Fine Plus mixes. Shear frequency sweep data were also available for a few of the test sections, at 10 Hz and temperatures of 40, 50 and 60 ºC. Initial moduli were also derived from fatigue tests on beams. Five to six specimens were available for each test section. Figure 2 compares the asphalt moduli determined by different methods for section 18FHL, which had little hardening and no cracking during the experiment. Equation 14 from Part II Chapter 4 of the NCHRP report (Epps et al., 2002) was also used in the comparison. An asphalt content of 6.2% and an air voids content of 4.3% were used. Different sources give different values for the air voids content but 4.3% appears to be a reasonable value, for section 18, at the start of the test (dropping to about 2.1% towards the end of the experiment). The legends in Figure 1 are: "Mr HL" – initial resilient modulus from indirect tensile tests (UNR), "FS UCB" flexural frequency sweep data from UCB, "Fatigue HL" – moduli from fatigue beams (UCB), "FS FHWA" – shear frequency sweep data from FHWA, "Table 2.3" – FWD backcalculated moduli from the UCB report (Monismith et al., 2000), "Eq 14-18" – from the NCHRP report (Epps et al., 2000), with asphalt content and air voids for section 18, "FWD" – moduli backcalculated with Elmod5 and "FWD-age" the same moduli adjusted for the effects of hardening. The curve "Model" was the best estimate using the equation:

$$\log(E_i) = \delta + \frac{\alpha}{1 + \exp(\beta + \gamma \log(tr))}$$

Equation 1 Master curve format for asphalt.

where E_i is the intact (initial) modulus of the asphalt,
tr is reduced time (determined using equations given in NCHRP (2004)),
δ, α, β and γ are constants, and
log is the base 10 logarithm.

The format of Equation 1 is the format used in the MEPDG (NCHRP, 2004). Using the MEPDG procedure with the volumetric data for section 18 resulted in a very low minimum modulus (10^{δ}) of 8 MPa and a maximum modulus ($10^{\alpha+\delta}$) of more than 110,000 MPa. Both of these values are unrealistic, and the MEPDG master curve does not compare very well to the measured moduli, as seen in Figure 3.

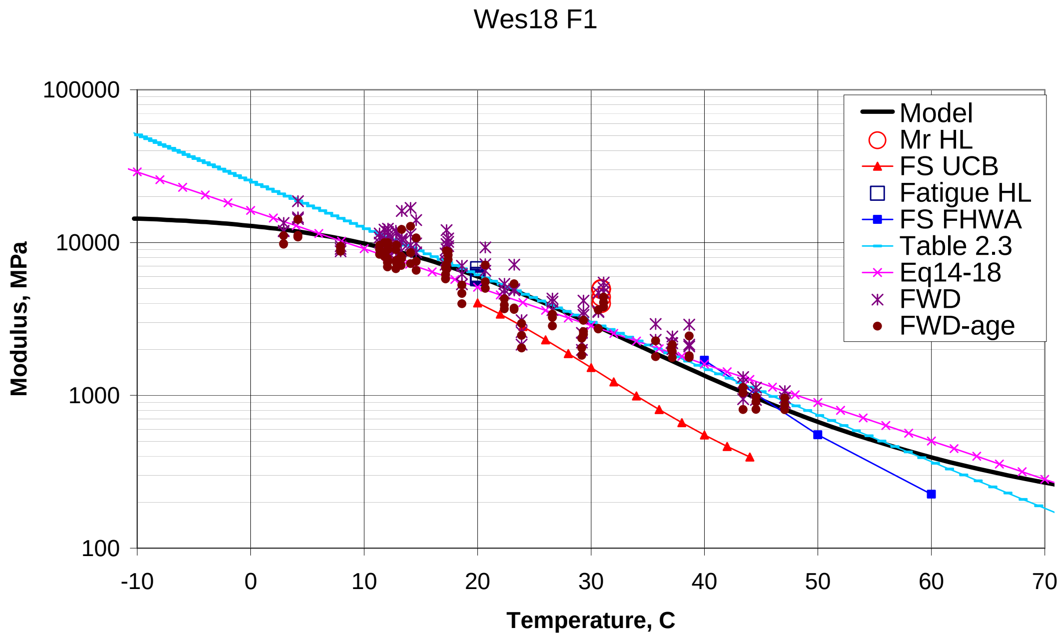

Figure 2. Asphalt modulus as a function of temperature for section 18, FLH.

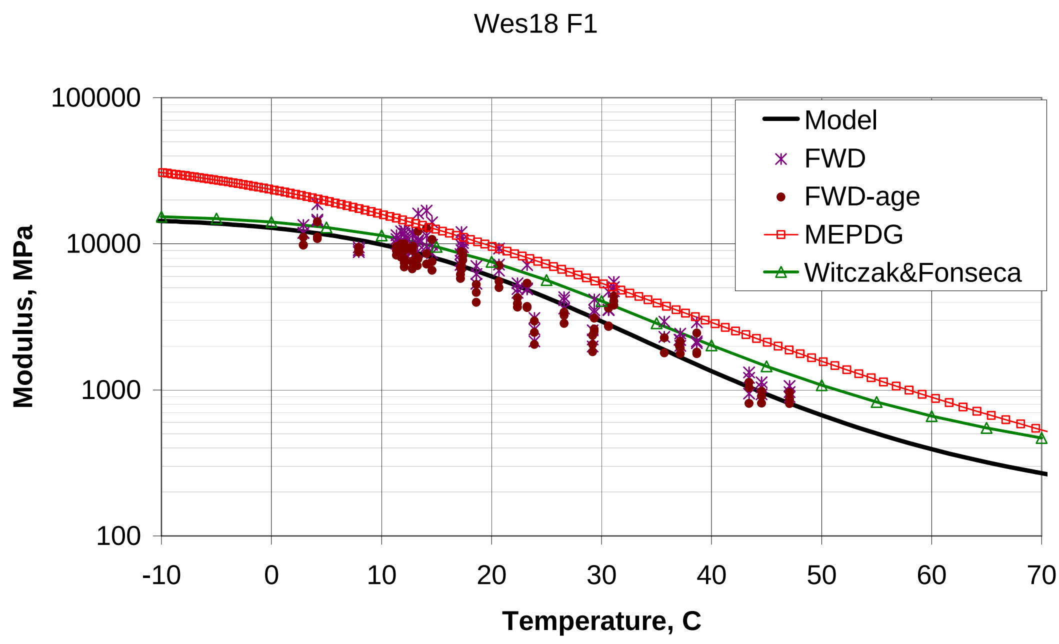

Figure 3. Section 18 Best fit "Model" master curve compared to master curve estimated from volumetric data following MEPDG.

An older version of the MEPDG master curve was given by Witczak & Fonseca (1996). This version is also shown in Figure 3 and fits the measured data better than the master curve estimated from volumetric data following the MEPDG procedure.

Unbound Layer Moduli

Triaxial tests were available for the aggregate base and for some of the "engineered fill" lifts, but only for some of the test sections. For the aggregate base, the triaxial modulus was primarily a function of the bulk stress, $\theta = \sigma_1 + \sigma_2 + \sigma_3$, with the shear stress (or deviator stress) having very little effect on the modulus. The modulus could be calculated from:

$$Eab = 206\,MPa \times \left(\frac{\theta}{0.1\,MPa}\right)^{0.64}$$

Equation 2. Modulus of Aggregate Base from triaxial tests.

where θ is in MPa. The agreement between the modulus measured in triaxial tests and the modulus calculated from Equation 2 is shown in Figure 4.

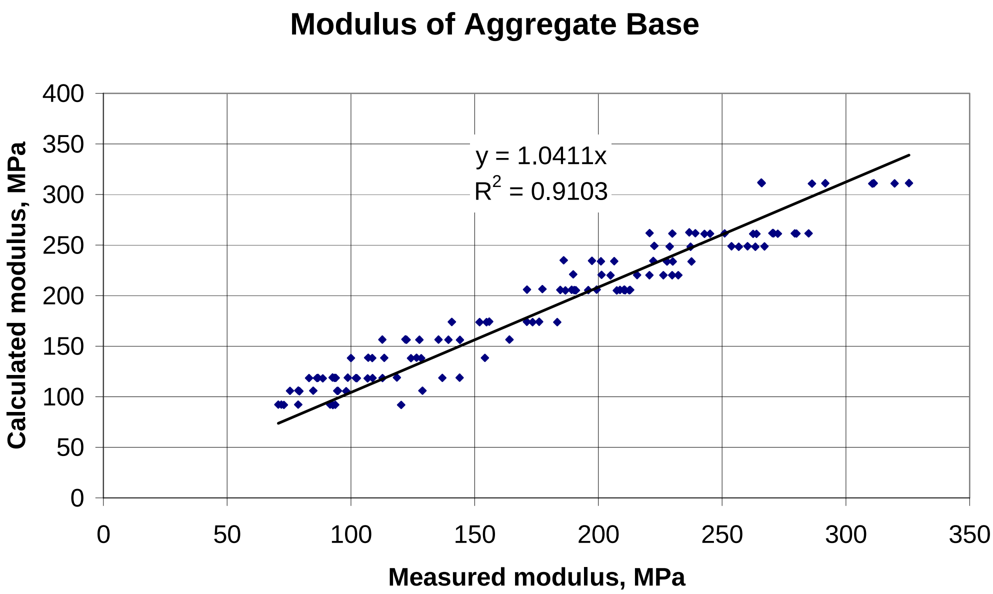

Figure 4. Moduli calculated from Equation 1 versus moduli from triaxial tests.

Triaxial tests on the engineered fill showed a large variation, with moduli ranging from 20 MPa to 170 MPa.

Analyses of moduli backcalculated from FWD tests showed that the moduli of the unbound layers were also functions of the stiffness of the pavement layers above the layer considered. An example from the first FWD test series on all of the WesTrack test sections is shown in Figure 5.

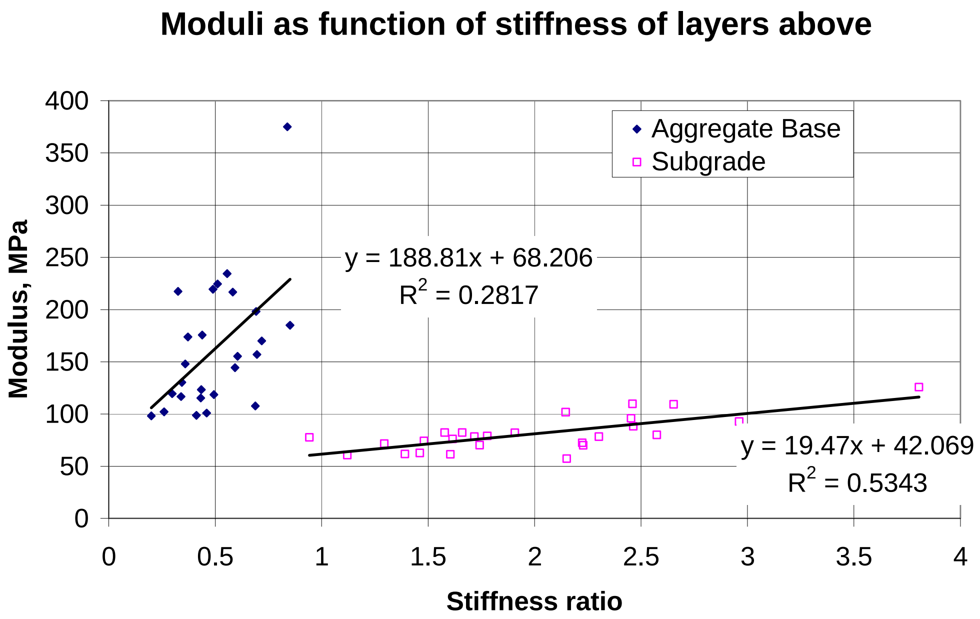

Figure 5 Correlation between moduli of unbound layers and stiffness ratio of layers above the layer considered ($S/3500^3$ in Equation 3), all sections.

The regression equations in Figure 5 result in the relationships:

$$E_{AB} = 257\,MPa \times \left(1 - \left[1 - \frac{S}{3500^3}\right] \times 0.73\right),\ R^2 = 0.28,\ SEE = 52\,MPa$$

$$E_{SG} = 62\,MPa \times \left(1 - \left[1 - \frac{S}{3500^3}\right] \times 0.32\right),\ R^2 = 0.53,\ SEE = 11\,MPa$$

$$S = \left(\sum_{1}^{n-1} h_i \times \sqrt[3]{E_i}\right)^3$$

Equation 3. Influence of confinement on stiffness of unbound layers.

where E_{AB} is the modulus of the aggregate base,
E_{SG} is the modulus of the subgrade,
h_i is the thickness of layer i, in mm,
E_i is the modulus of layer i, in MPa, and
n is the number of the layer considered.

Similar relationships were derived for the aggregate base of the individual test sections for the calibration of the incremental-recursive models of CalME.
The bulk stress was calculated using CalME for section 18 at a depth of 50 mm below the top of the Aggregate Base (AB) and the same depth below the top of the Subgrade (SG, Engineered Fill). Figure 6 shows the calculated bulk stress for AB and SG for the duration of the WesTrack experiment.

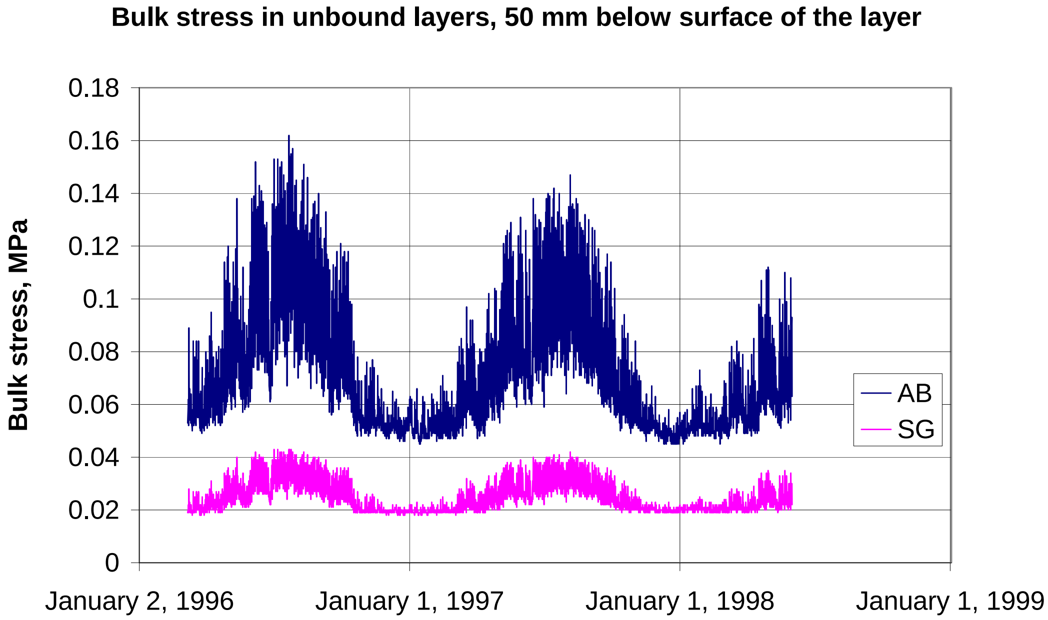

Figure 6. Bulk stress in Aggregate Base and in Subgrade, 50 mm below the surface of the layers, section 18

Figure 7 shows the modulus of the AB, as calculated from the bulk stress using Equation 2 for triaxial tests and as determined from backcalculation of FWD data. During cold periods where the asphalt layer is stiff, the bulk stress in the AB is low, resulting in a low triaxial modulus. The opposite is true for the FWD moduli.

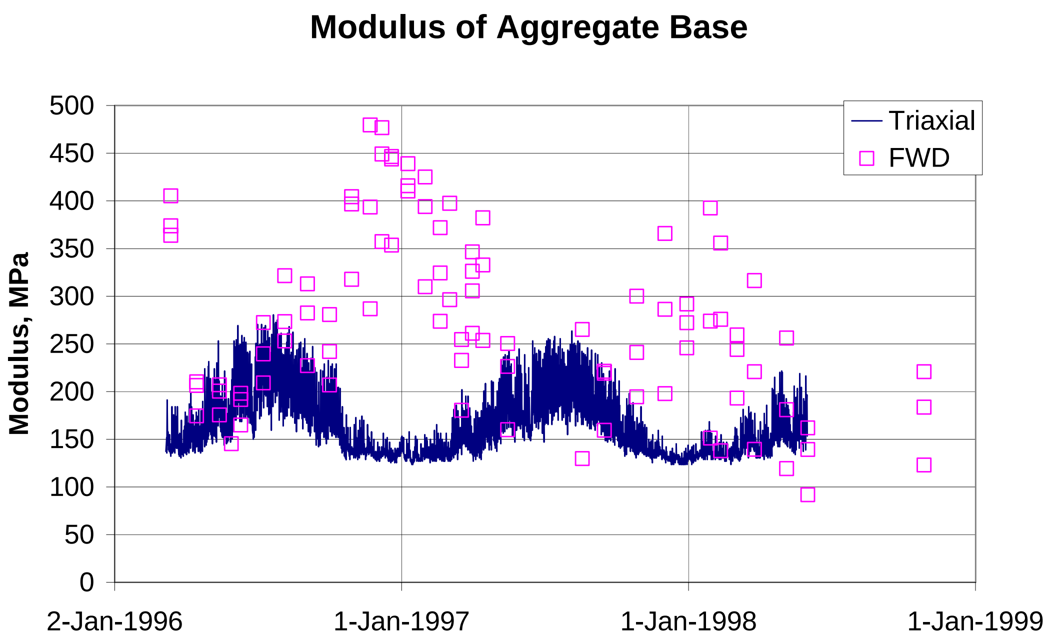

Figure 7 AB moduli at test section 18, from triaxial tests and FWD.

Figure 8 compares the moduli calculated by CalME, using the stiffness function given in Equation 3, to the moduli backcalculated from FWD testing. Inspection of Figures 6 and 7 reveals that the use of the stiffness function (Equation 3) results in a much better agreement with moduli backcalculated from FWD testing than use of the bulk stress relationship (Equation 2) derived from triaxial testing.

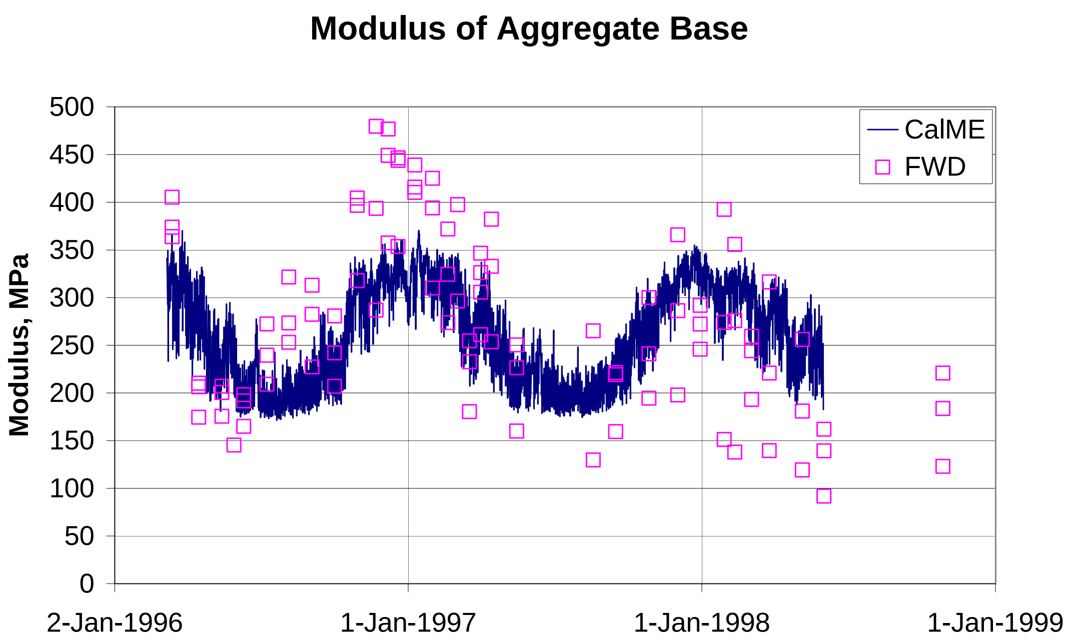

Figure 8 AB moduli calculated by CalME compared to FWD determined moduli.

Figure 9 shows the moduli of different lifts of the Engineering Fill, as determined from triaxial testing at a bulk stress of 30 – 40 kPa, which is on the high side of the actual bulk stress. The moduli from the first FWD tests in March 1996 are shown as a comparison. The average modulus from triaxial testing is 112 MPa, and 81 MPa from FWD tests. Multiplying the FWD derived subgrade modulus by a factor of 0.35, as recommended by the MEPDG (NCHRP, 2004), would clearly not be appropriate.

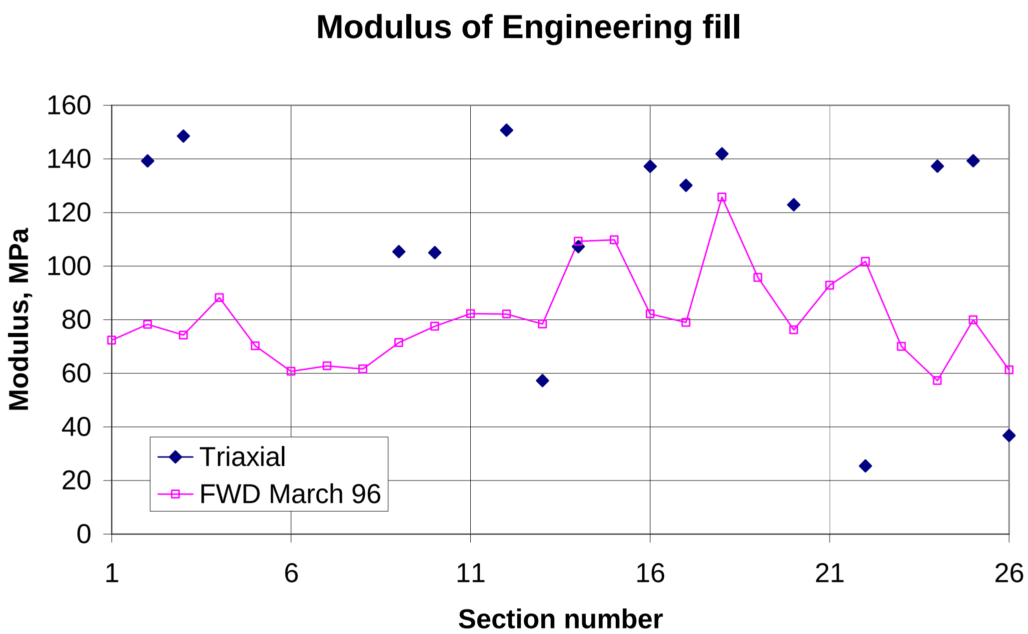

Figure 9. Modulus of Subgrade from triaxial tests and from FWD backcalculation.

Fatigue Damage of Asphalt

The model for damaged asphalt concrete modulus used in CalME follows the relationship previously given in Equation 1:

$$\log(E) = \delta + \frac{\alpha \times (1-\omega)}{1 + \exp(\beta + \gamma \log(tr))}$$

Equation 4. Modulus of damaged asphalt concrete.

where the damage, ω, is calculated from:

$$\omega = A \times MN^{\alpha} \times \left(\frac{\mu\varepsilon}{200\ \mu strain}\right)^{\beta} \times \left(\frac{E}{3000\,MPa}\right)^{\gamma} \times \left(\frac{E_i}{3000\,MPa}\right)^{\delta}$$

Equation 5: Damage as a function of load applications, strain, modulus and temperature.

where MN is the number of load applications in millions,
$\mu\varepsilon$ is the tensile strain at the bottom of the asphalt layer,
E is the modulus,
E_i is the modulus of the intact material,
tr is reduced time, and
A, α, β, γ, and δ are constants (not related to the constants of Equation 4).

The constants of Equation 5 were determined from four point bending beam tests, at constant strain at a temperature of 20 ˚C. The value of γ was fixed at $\beta/2$, making the damage a function of strain energy. The parameter δ was based on the parameter for initial asphalt moduli in the Asphalt Institute criterion for asphalt fatigue. With this criterion the damage will be proportional to the initial modulus raised to $-\alpha$ times -0.854 ($= 0.854\times\alpha$). This results in positive values of δ, between 0.3 and 0.5, where the results of fatigue testing at different temperatures indicated a negative value of -1.9. Using the value from fatigue testing done at temperatures different from 20 ˚C resulted in damage being predicted in the warm periods, where the observed damage occurred mostly during relatively cold periods. The reason for this difference between the laboratory and the in situ damage is not known.

Permanent Deformation of Asphalt

The permanent deformation of the asphalt is partly due to a decrease in air voids caused by post compaction and partly to shear deformation. The average decrease in air voids over the first 12 months of the experiment, ΔAV, for the top and bottom lifts combined based on measurements from cores, was found to be:

$$\Delta AV = 0.36 \times AV_{original},\ R^2 = 0.64$$

Equation 6. Average decrease in air voids.

This decrease was used in the simulations with CalME. It was assumed to occur over the first 60 days with traffic loading and was added to the shear deformation.
The majority of the permanent deformation of the asphalt was due to shear deformation. A shear-based approach, developed by Deacon et al. (2002), was used. The permanent, or inelastic shear strain, γ^i, is determined from RSST-CH tests as a function of the shear stress, τ, the elastic shear strain, γ^e, and the number of load repetitions. The best fitting relationship for the materials used was found to be a power function as follows:

$$\gamma^i = A \times MN^{\alpha} \times \exp\left(\beta \times \tau / \tau_{ref}\right) \times \gamma^e$$

Equation 7. Power function for permanent shear strain.

where γ^e is the elastic shear strain,
τ is the shear stress,
MN is the number of load repetitions in millions,
τ_{ref} is a reference shear stress (0.1 MPa), and
A, α, and β are constants determined from the RSST-CH (constants are not related to those in Equation 5)

The constants should be determined at an air voids content corresponding to post compaction. The permanent deformation of the asphalt is calculated from:

$$dp = K \times \sum h_i \times \gamma_i^i$$

Equation 8: Calculation of permanent deformation.

where K is a calibration factor (determined from HVS testing or actual field asphalt rutting data),
h_i is the thickness of layer i, and
γ^i_i is the inelastic (permanent) shear strain in layer i calculated from Equation 7

The summation is done for the top 100 mm of the asphalt where the majority of the permanent deformation in the asphalt has been observed to occur.

The permanent deformations of the aggregate base and of the subgrade were calculated using the model described in Ullidtz et al. (2007). The contributions to the permanent deformation from these layers were small and in good agreement with the observations from the WesTrack experiment.

Simulation of the WesTrack experiment using CalME

The results from WesTrack were imported to the CalME database and the experiment was simulated, section by section, using a time increment of one hour, and the measured temperatures and load applications during each hour. It is very important that the pavement primary responses (stresses, strains and displacements) are predicted reasonably well for the duration of the experiment. If the responses are not correctly predicted it will not be possible to calibrate the empirical relationships.
To predict the pavement responses for the duration of the experiment, it is necessary to consider any fatigue damage that develops during the testing, as this will influence

those responses. Getting a good match between measured and calculated responses, therefore, implies an adequate calibration of the fatigue damage functions.

Pavement Response

The only measured response available from WesTrack was the FWD deflections. Figure 10 shows the FWD deflection at the center of the loading plate in the wheel path (position F3) obtained for Section 18 at various times during the WesTrack experiment. The deflections correspond to a peak load of approximately 40 kN (the actual load level was used both for measured and calculated values). The legends marked “M” are the measured values. The measured points are connected by fully drawn lines. The corresponding calculated deflections have legend “C” and the points are connected by dotted lines. Figure 10 shows results from four FWD positions. “35_1” in the header of the figure indicates a test point between station 30 m and 39 m.

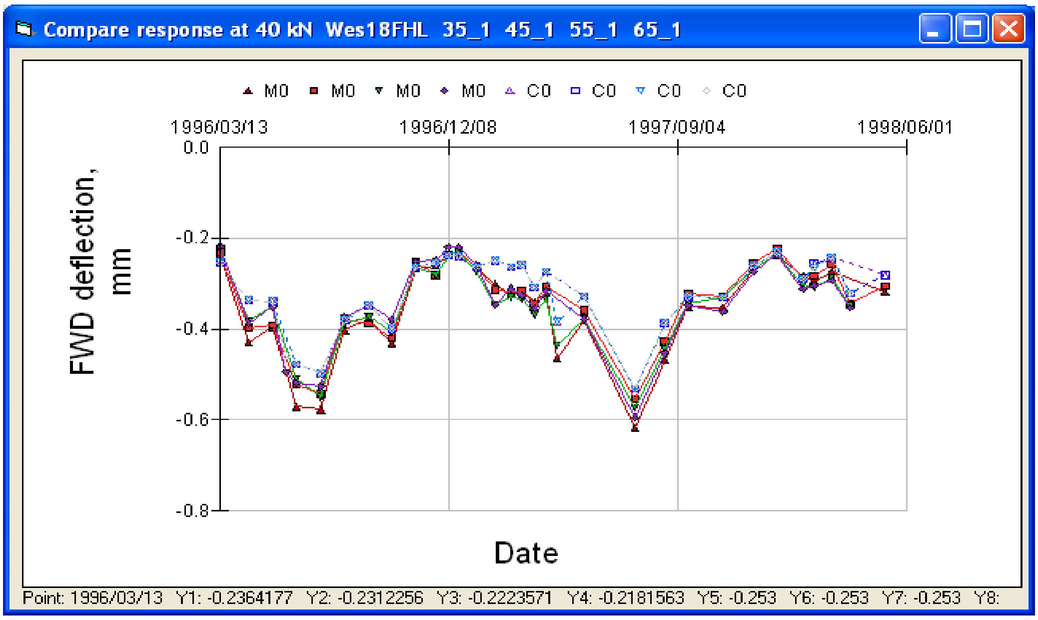

Figure 10. FWD deflections at section 18 (in wheel path, geophone under the loading plate).

The agreement between the measured and the calculated deflections is seen to be very good in this case. The mean difference between the four measured deflections in Figure 10 is 2 µm (10^{-6} m) and the Root Mean Square (RMS) difference is 55 µm. The mean difference between measured and calculated deflections is 3 µm and the RMS is 33 µm, so the scatter in the measured deflections is as large as the difference between the measured and calculated values.

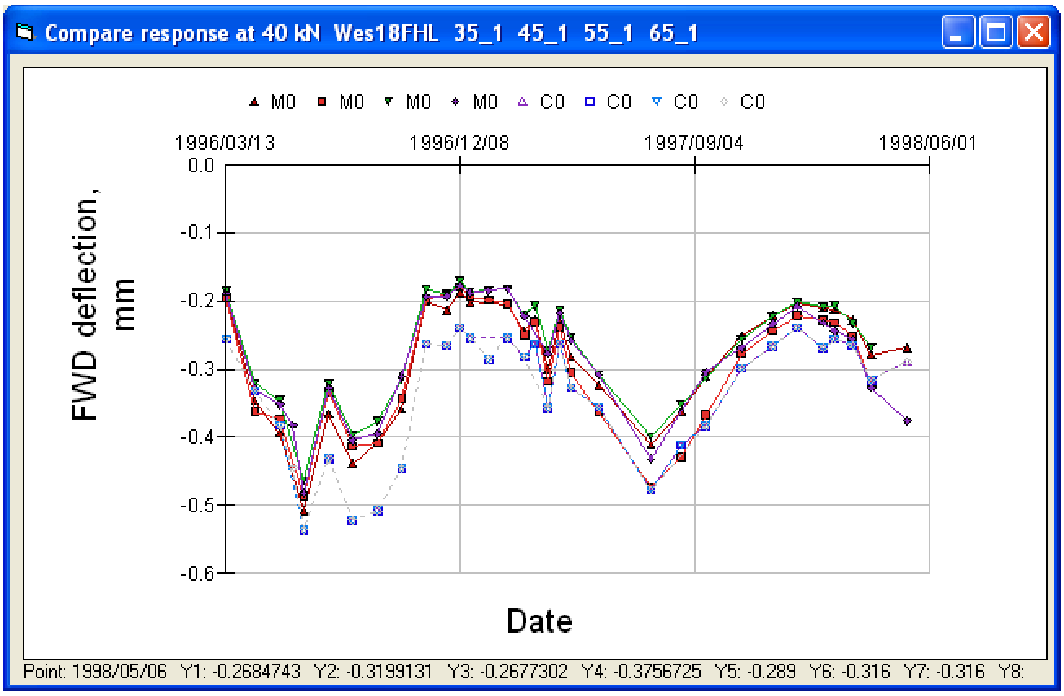

Figure 11. FWD deflections at section 18 (between wheel paths, geophone center of load plate).

Figure 11 shows the deflections measured between the wheel paths, compared with the calculated deflections based on the damaged asphalt in the wheel path (calculated deflections are identical to the calculated deflections of Figure 10). The figure clearly shows that the asphalt in the wheel paths did suffer some damage, resulting in larger deflections, even though no visible fatigue cracking was recorded on this section. Figure 10 shows that the deflections calculated by CalME, using the master curve for the asphalt material and the models for determining the stiffness of the unbound materials, and considering the effects of fatigue damage, are in good agreement with the measured deflections. This is a good indication that the response model is functioning correctly, and that the predicted stresses and strains will also be reasonably correct, so that they can be used for calibrating the empirical models dealt with in the following sections.

Fatigue Damage

Section 18 had no visible cracking during the experiment, but still suffered some fatigue damage, as indicated by both the FWD backcalculated moduli and the simulated damage (parameter ω in Equation 5) shown (on the left axis) in Figure 12. Both are from the right wheel path (there were no FWD tests done in the left wheel path). The damage from FWD backcalculated moduli was calculated on the assumption that any difference between the backcalculated modulus and the modulus from the master curve, adjusted for temperature and hardening, was due to damage. The cracking in the two wheel paths is shown at the axis to the right (in this case there was no cracking, as mentioned above).

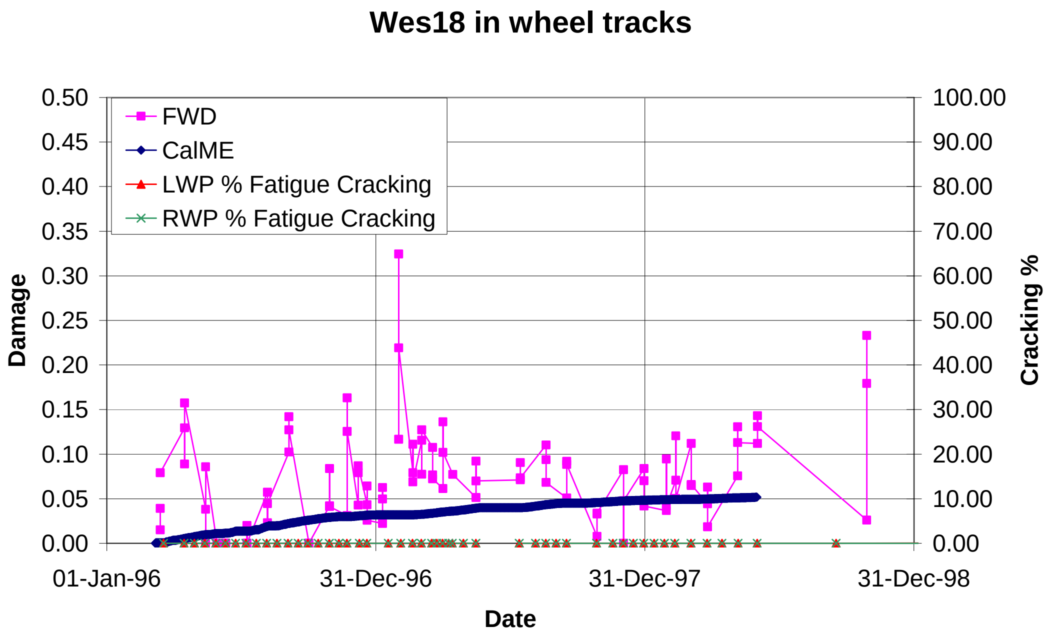

Figure 12. Damage in right wheel path of Section 18 (LWP left, RWP right wheel path).

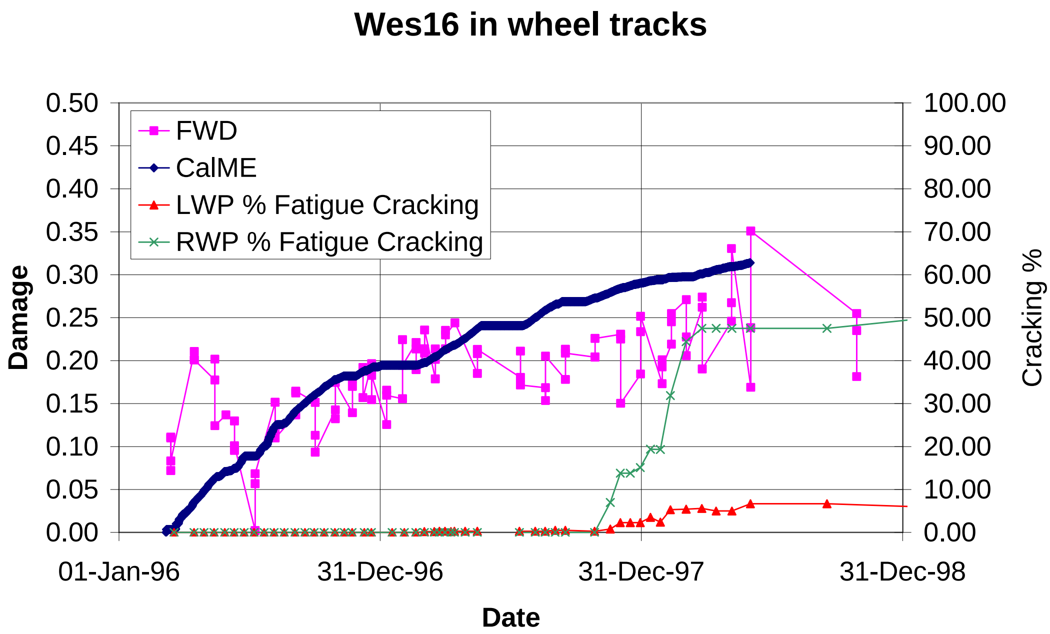

Figure 13. Damage in right wheel path of Section 16.

The damage predicted for Section 16, which had low binder content and high air voids, is shown in Figure 13. This section had a few percent cracking in the left wheel path (LWP) and about 50% in the right wheel path (RWP), at the end of the experiment.

Permanent Deformation

Figure 14 shows the down rut in the right wheel path of Section 18. Again, measured values are connected by a fully drawn line whereas the values calculated by CalME are connected by a dotted line.

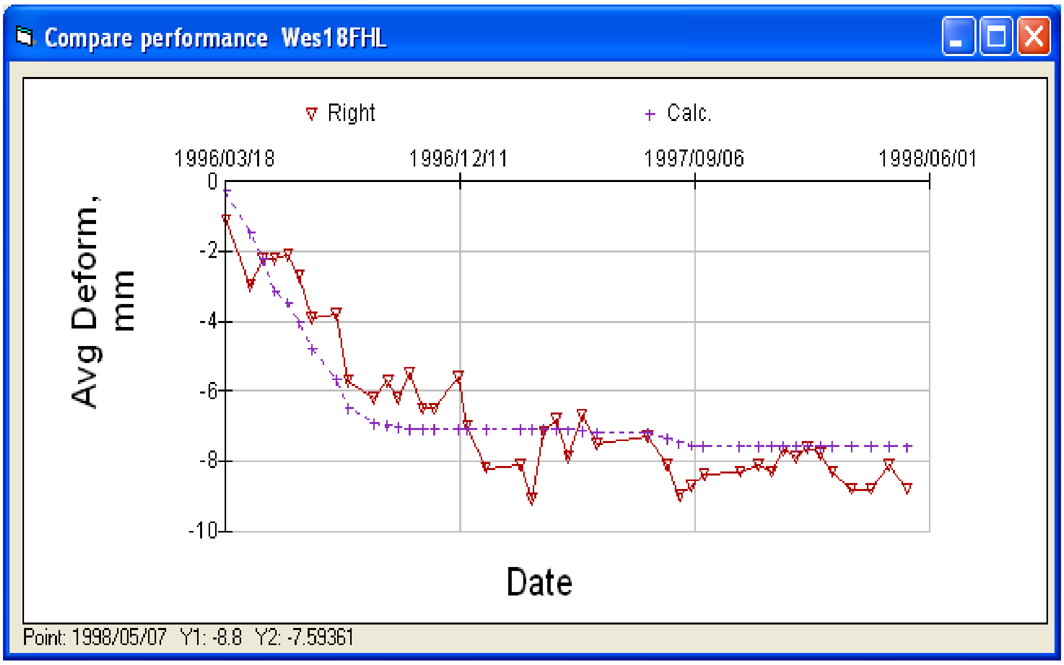

Figure 14. Down rut in right wheel path at Section 18.

In Figure 14 the mean difference between the measured down rut depth and the calculated permanent deformation is 0.9 mm and the RMS is 1.3 mm. Figure 15 shows the simulation results compared to the measured maximum rut depths (the distance from the highest peak of the rut to the bottom of the rut) in the right and left wheel paths. The mean difference between rutting in the left and the right wheel path in Figure 15 is 2.7 mm and the RMS is 4.0 mm.
The maximum rut depth in the right wheel path, shown in Figure 15, is not much different from the down rut shown in Figure 14, whereas the rutting in the left wheel path is larger.

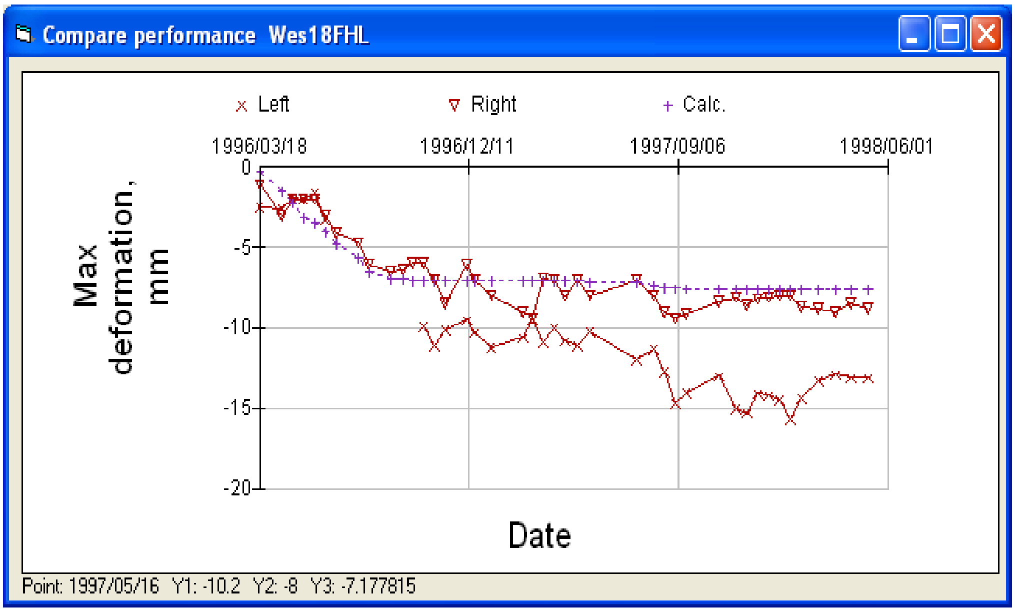

Figure 15. **Maximum rutting in left and right wheel paths at Section 18.**

Summary of Analyses and Conclusions

Deflection Response

The agreement between the measured and calculated response during the duration of the WesTrack trafficking on each section, in terms of the deflection under the load plate of the FWD, was very good in most cases. The measured deflections will be a function of the actual asphalt temperature during the FWD tests, the contact between the FWD loading plate and the pavement surface, as well as of the position of the test. The calculated deflection is a function of the following factors which are considered in CalME:

- the estimated asphalt temperature during the FWD test,
- the asphalt modulus versus reduced time relationship,
- the moduli of the unbound materials (aggregate base and subgrade),
- the hardening of the asphalt material as a function of post compaction and ageing, and
- the damage to the asphalt caused by fatigue.

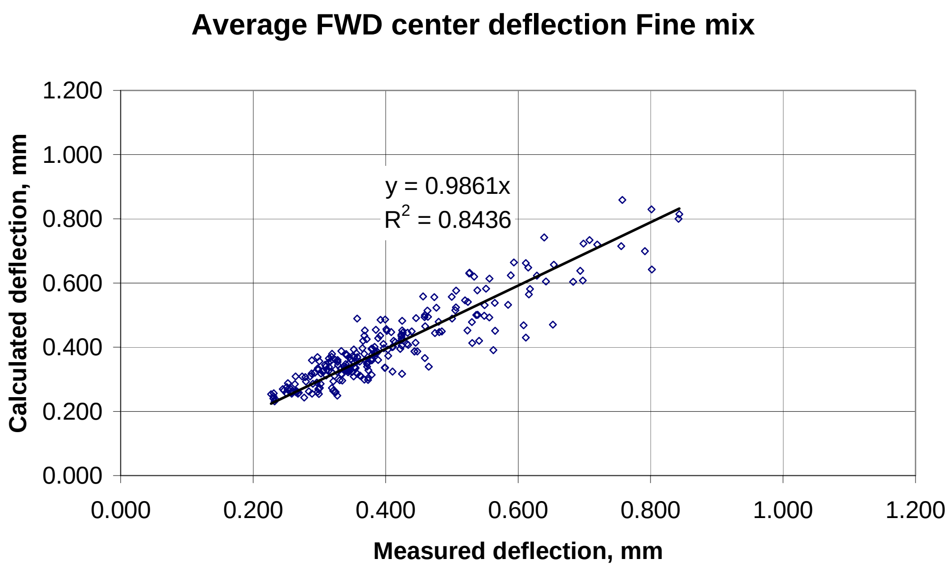

Figure 16. Measured and calculated center deflections on Fine mix sections

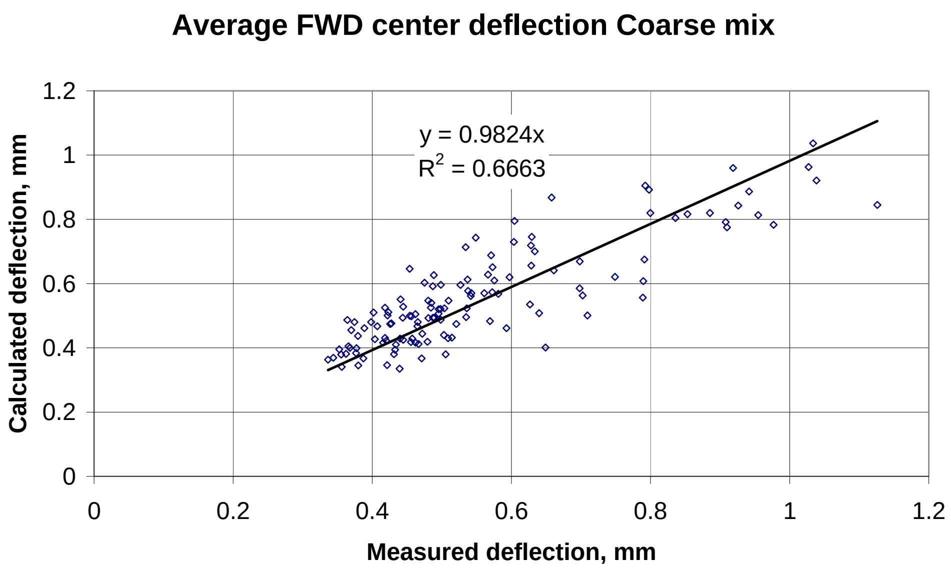

Figure 17. Measured and calculated center deflections on Coarse mix sections.

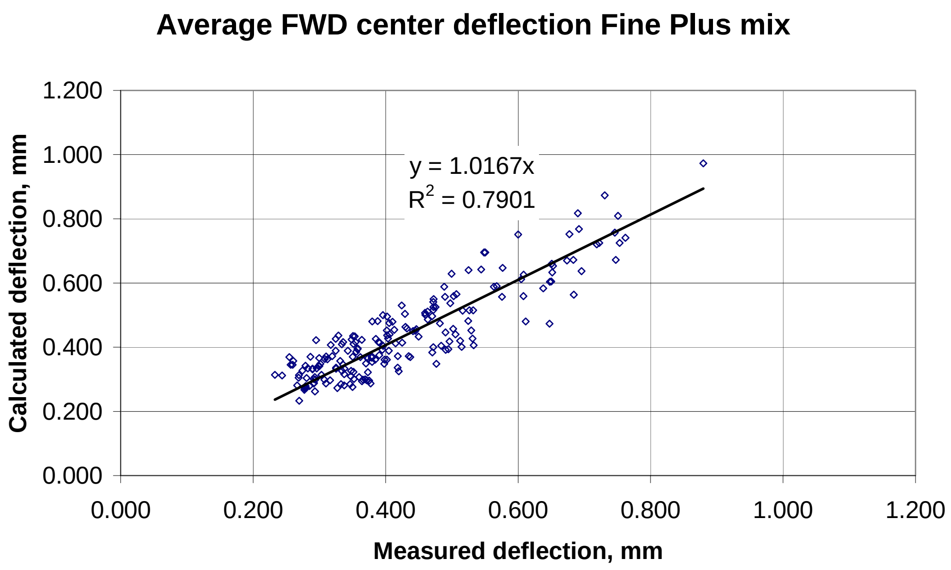

Figure 18. Measured and calculated center deflections on Fine Plus mix sections.

Figure 16 to Figure 18 show the average deflection at the center of the FWD loading plate, calculated for each section and each monitoring session. Only tests done at stations of 30 m or higher were used, in order to avoid the transition sections.
The standard error of estimate is 50 μm for the Fine mix sections, 103 μm for the Coarse mix sections, and 60 μm for the Fine Plus mix sections. For the Fine mix and the Fine Plus mix these values are similar to the standard deviations of the measured values for a single monitoring session on one test section. In other words, the difference between measured and calculated values is similar to the scatter in the measured values. For the Coarse mix the standard error of estimate is somewhat higher.

Relation of Cracking to Damage

Figure 19 compares the damage, ω, predicted by CalME based on the laboratory fatigue data, to the damage estimated from the FWD tests in the right wheel path. As explained earlier, the FWD backcalculated asphalt moduli were corrected for the effects of (estimated) temperature and hardening due to ageing and decrease in air voids content. The difference between the adjusted modulus and the modulus calculated from the modulus versus reduced time model was then assumed to be due to damage.
On average, the Fine, Coarse and Fine Plus mixes all have less damage predicted by CalME than estimated from the FWD. The Coarse mix shows the largest difference, with the FWD estimated damage being 2.2 times that of the CalME predicted damage, even though the shift factor, between laboratory and in situ damage, used for the Coarse mix was 5, compared to 15 for the other two mixes. For the Fine and the Fine Plus mixes the ratio is 1.3 and 2.0, respectively. This indicates that the shift factors are too large, and that they are a function of mix type.

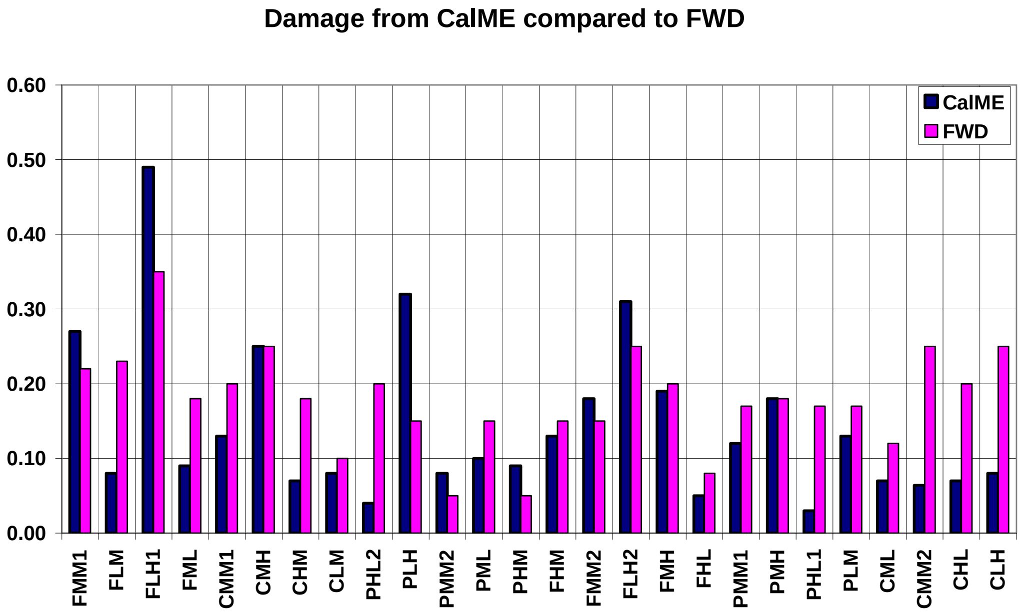

Figure 19. Damage predicted by CalME compared to damage estimated from FWD tests.

An approximate relationship between damage at crack initiation and thickness of the asphalt layers was determined from HVS testing (Ullidtz et al., 2008):

$$\omega_{initiation} = \frac{1}{1+\left(\frac{h_{AC}}{250\,mm}\right)^{-2}}$$

Equation 9. S-shaped curve for damage at crack initiation as a function of AC thickness

where $\omega_{initiation}$ is the damage corresponding to crack initiation, and
h_{AC} is the combined thickness of the asphalt layers.

Using this equation on the WesTrack experiment would result in a damage of 0.26 at crack initiation. This value appears to be quite reasonable for the Fine mix and the Fine Plus mix. However, for the Coarse mix, the calculated damage was much lower at crack initiation and would correspond better to Equation 10 with the AC thickness raised to -5, rather than to -2, i.e.:

$$\omega_{initiation,\,coarse\,mix} = \frac{1}{1+\left(\frac{h_{AC}}{250\,mm}\right)^{-5}}$$

Equation 10. Crack initiation for coarse mix

The cracking (in percent) can be modeled as a function of the calculated damage using an equation of the format:

$$Cr\% = \frac{1}{1+\left(\frac{\omega}{\omega_o}\right)^{\alpha}} \times 100$$

Equation 11 Cracking in percent as a function of damage.

where $Cr\%$ is the cracking in percent,
ω is the calculated damage obtained from Equation 5, and
ω_o, and α are constants.

In Figure 20 the fully drawn curves were calculated from Equation 11 on the assumptions that crack initiation would correspond to 5% cracking and that α was -8.

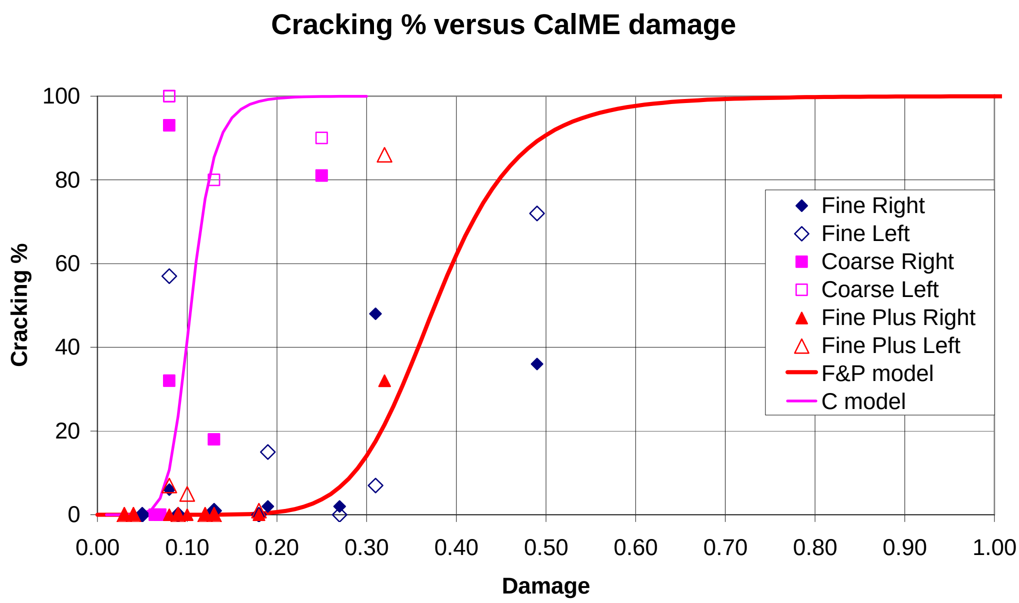

Figure 20. Cracking models compared to terminal cracking at WesTrack.

One of the Fine mix sections, 02FLM, had surprisingly good fatigue performance in the laboratory tests, considering that the AC content was low. The calculated damage is, therefore, low even though the section had a considerable amount of cracking in the left wheel path and some cracking in the right wheel path. With the large difference between the performance in the right and left wheel paths there is a possibility that the laboratory fatigue specimens were obtained from material that was not totally representative of the section. The calculated damage may also have been underestimated at some of the sections with poor fatigue performance, because the first series of FWD tests were carried out when the sections had already had a traffic load corresponding to 4500 ESALs. Any damage caused by these loads is not included in the calculated damage.

Permanent Deformation

Figure 21 shows the final permanent deformation calculated by CalME (for the right wheel path) and the maximum rutting recorded for the right and left wheel paths, which is not necessarily at the end of the experiment. The average values are: CalME

15.9 mm, 15.8 mm measured at right wheel path, and 18.2 mm measured at left wheel path.

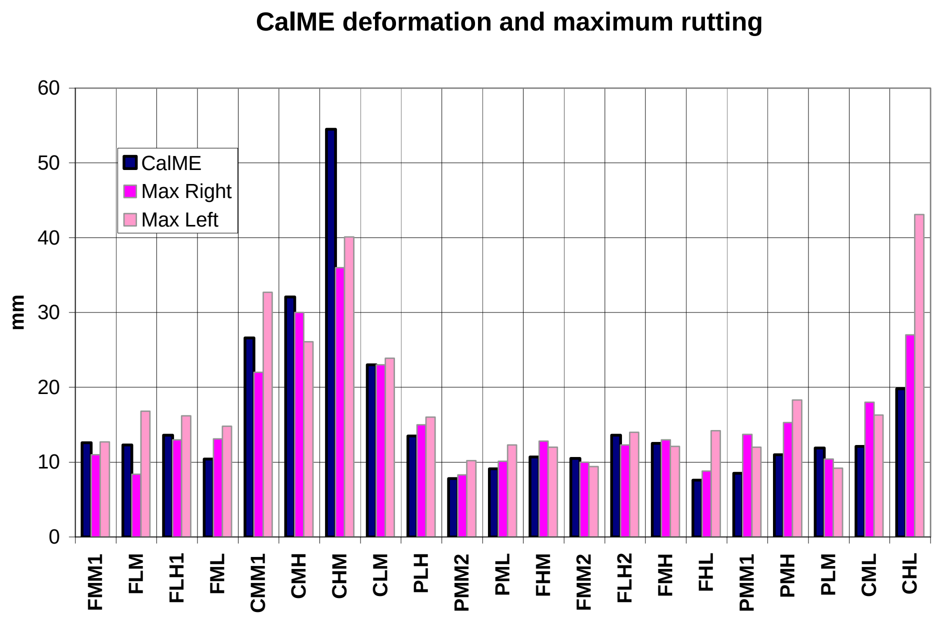

Figure 21. CalME predicted permanent deformation and rutting in right and left wheel paths.

The main conclusions from the simulation of the WesTrack experiment with CalME are:

- The pavement response, in terms of resilient deflections, was predicted quite well, with the difference between measured and calculated values being similar to the scatter of the measured values, in most cases.
- The damage predicted by CalME, using laboratory fatigue tests, was somewhat lower than the damage estimated from the FWD tests. This indicates that the shift factors used in CalME should be reduced. The effect of temperature on damage needs further study.
- The permanent deformation predicted by CalME from RSST-CH testing in the laboratory was close to the measured rut depths.

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