Determining and Modeling Asphalt Concrete Response (ACRe)

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In road engineering research and design the principles of material mechanics have not yet become a standard tool. In this contribution a project aimed at applying these principles to asphalt concrete is presented. Attention is paid to the differences between the standard test procedures and those based on mechanics considerations, presenting examples of the disturbances that can occur to illustrate the need for mechanically sound testing. The general approach in the ACRe project is introduced, showing the material model utilised and the parameters that have to be determined. The physical meaning of the model parameters is mentioned the tests used for the model parameter determination, uniaxial tension and compression tests and four-point shear tests, are introduced. The uniaxial compression test is discussed in more detail, to illustrate the approach used in the test programme. The results from this test are also presented, as well as the way in which a general expression for the compressive strength of the material was determined on the basis of these results. Finally, an example of the application of the model for damage predictions in road engineering constructions is presented. In this example two different pavement structures are analysed, illustrating that this kind of analyses can show the different damage patterns that will occur. In current day road engineering design approaches for every structure the same location is considered normative. From observations in practise it is known that this is not the case and the differences observed in the simulations agree rather well with these observations.

Key words: **Asphalt Concrete, testing, material model, Finite Elements**

1 Introduction

Current road design methods are mostly based on experience and consider only one type of distress at the time. This results in separate criteria for, for example, fatigue cracking and rutting. This is not in accordance with the situation in a road construction where usually several types of distress occur.

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Although the existing design procedures sufficed until recently, the rapid changes in the road engineering discipline require new approaches. The incorporation of new materials, not only in the top layer but throughout the road profile, render past experience less useful. In combination with the increasing demand for maintenance free roads and the fast increases in both number and size of axle loads, this leads to a demand for design methods based on sound mechanical considerations of the material behaviour.

To enable such an approach, test methods suited to determine the material characteristics of asphalt mixes are necessary. So far, the tests performed in road engineering laboratories are usually structure-oriented tests that are kept as simple as possible so they can also be used in practice, for example for quality control and quality assessment tests (QC/QA). These considerations led to the approach for the Asphalt Concrete Response (ACRe) project, in which a material model for asphalt is being developed along with the test set-ups and procedures needed to determine the material characteristics required as input for the model. The activities with respect to the model and test procedures are strongly interrelated: on the one hand the model dictates what should be measured in a test, while on the other hand, the response observed in the tests sets the requirements for the model. As a result, model development/verification and experimental testing have been progressing in parallel throughout the project.

In this article, an overview of the approach used for the project will be presented. First of all, some considerations with respect to the testing of asphalt concrete will be mentioned on the basis of a discussion about standard road engineering test. After that, the ACRe project is discussed, presenting the material model in its current, prototype stage as well as one of the experiments. For reasons of brevity, only a single type of test, the uniaxial compression test, is discussed here. Finally, an example of the use of non-linear modelling in road engineering is presented. In the example two different road constructions are modelled, showing the predicted damage pattern due to repeated loading for both of them.

2 Testing of asphalt concrete

As stated above, the tests performed in road engineering laboratories are usually structural tests that are kept as simple as possible. These tests lead to complicated and unknown internal states of stress, which makes them difficult to interpret. The test results are either used to compare different asphalt mixes or they are related to the expected response in a road construction by means of empirical relations. Yet it is known that the processes that take place inside the road construction are anything but simple. The stresses and strains depend on the relative position of the load (which is continuously changing), the rate of loading, the temperature, the construction geometry and many more parameters. It is hard to believe that all these parameters can be taken into account with a single, simple test. Especially since, although the temperature and strain rate dependence of asphalt concrete behaviour is generally acknowledged, experiments are often performed at a single temperature or strain rate. The fact that this gives only a single characterisation and that the ranking might have been different under different conditions is disregarded.

If two or more temperatures, strain rates and states of stress must be tested to obtain reliable rankings, the question arises if it is not possible to characterise a mix once using an extensive test programme and then re-use the characteristics from there on. Eventually, this would be less expensive than re-testing a mix for each potential application. Especially since it is difficult to predict the behaviour of a road construction on the basis of simple tests. One of the reasons for this is that the response of asphaltic materials depends on the state of stress, which differs from point to point in a road construction (Figure 1a). Because of differences in subgrade, construction geometry and loads, the actual distribution of stresses will also vary from construction to construction (Section 4). Because the state of stress in many tests is complicated and often unknown, in many cases the relation between the test results and the structural response is obscure. With the increasing demand for generally applicable, performance based specifications for road constructions, the above mentioned considerations have received a great deal of attention. In the remainder of this Section, some observations on the implications of the strain rate, temperature and state of stress sensitivity for the testing of asphalt concrete are discussed.

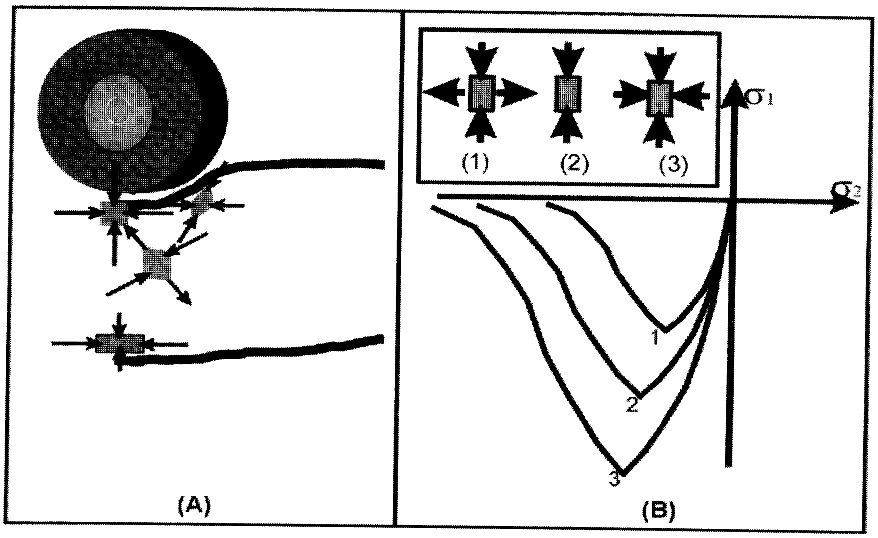

Figure 1: (a) states of stress in a road vary in time and from place to place, (b) the response of asphalt concrete is different for different states of stress.

Although the temperature and strain rate sensitivity of asphalt concrete is generally recognised, the way in which these aspects are taken into account in experiments is not always adequate. For example, the strain rate sensitivity is often considered to be a frequency influence, and tests are performed using sinusoidal deformation signals at a given frequency. Since the deformation rate is the derivative of the deformation with respect to time, a sinusoidal deformation represents a multitude of deformation rates (a cosine signal). Not only the frequency, but also the amplitude of the deformation rate is influenced by the frequency of the deformation signal. This is illustrated in Figure 2, where the tick lines represent two deformation signals with different frequencies and equal amplitude. The deformation rates are shown as thin lines and it can be seen that the deformation rate of the deformation signal with the lower frequency does not only have a lower frequency, but also a smaller amplitude. This is because the amplitude of the deformation rate is proportional to the frequency of the deformation signal, Equation (1).

$$u(t) = A\sin(2\pi ft) \;\; ; \;\; \dot{u} = \frac{du}{dt} = 2A\pi f\cos(2\pi ft) \tag{1}$$

Where: $u(t)$ = deformation as a function of time

A = amplitude of the deformation signal

f = frequency

t = time

If a material is sensitive to the rate of deformation, naturally the latter range (which includes considerably higher rates) will lead to a stronger and stiffer response. Although for a practical test it may appear advantageous to test a range of strain rates in a single test, for material characterisation this is not the case. Since the material is strain rate sensitive, the material characteristics should be determined for a single strain rate at the time. After determining the characteristics for several, individual strain rates, they can be expressed as a function of the strain rate. In this way, a general relation is obtained.

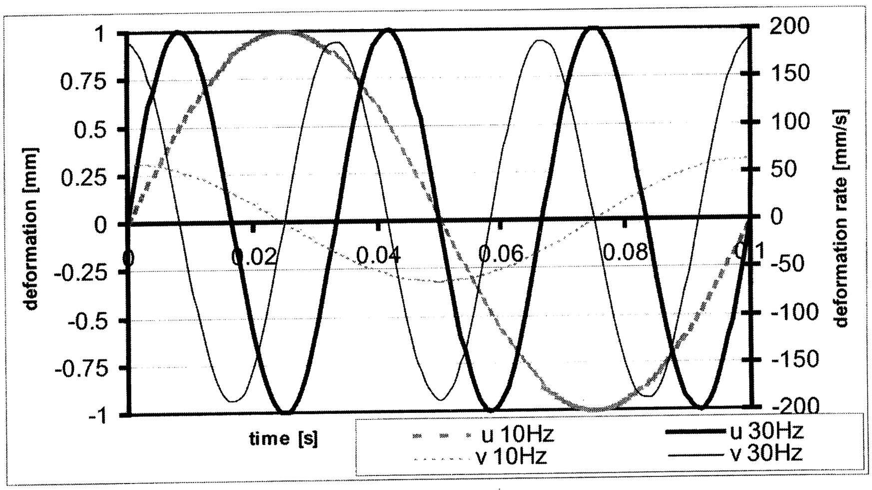

Figure 2: Relation between the frequency of the deformation signal and the deformation rate (deformation: ±1 mm and 10 and 30 Hz, respectively).

Despite the temperature dependence, tests are often performed without actual temperature control. The specimens are stored in temperature controlled storage prior to testing, however the actual test is often performed using set-ups without facilities to maintain a specific temperature, assuming that the temperature does not change significantly during a test. This raises the question how much the specimen temperature can change (and how long that takes) before it influences the results. To get an indication of this, a series of uniaxial compression test with (ACRe test programme) and without temperature control (Arif) are compared (Arif 1999, Erkens et al. 1998b en 2000). The test, including placement of the specimen and instrumentation, lasted a couple of minutes (2 to 5, depending on the deformation rate). In both cases, the specimens were 50 mm in diameter and 100 mm high and the material was a dense asphalt concrete with bitumen 45/60 and a maximum aggregate size of 5 mm (DAC 0/5). The average results at each test condition are presented in Table 1. The room temperature at the time of the test without temperature control was approximately 20°C, from Table 1 it can be seen that for specimens with a temperature above the room temperature the strength was overestimated ($f_{c,Arif} > f_{c,ACRe}$), while for specimen temperatures below that of the room the strength was underestimated ($f_{c,Arif} < f_{c,ACRe}$). This is due to cooling down in the former case and heating up of the specimen in the latter situation. Obviously, the combination of these temperature differences and the test duration did result in an influence of the lack of temperature control.

Table 1. The influence of a constant specimen temperature during the test on the observed uniaxial compressive strength.

v [mm/s]	T [°C]	$f_{c,ACRe}$ [N/mm²]	$f_{c,Arif}$ [N/mm²]	$\Delta = f_{c,ACRe} - f_{c,Arif}$ [N/mm²]	$\Delta\% = \frac{f_{c,ACRe} - f_{c,Arif}}{\lvert f_{c,ACRe}\rvert} X 100\%$
0.1	30	-1.9	-2.0	+0.1	+5%
0.85*)	30	-3.1	-4.0	+0.9	+29%
0.1	15	-5.7	-3.8	-1.9	-33%
0.85*)	15	-10.9	-8.0	-2.9	-27%

Since for most tests the specimen will be exposed to room temperature for at least two minutes, it appears that testing of asphalt concrete without temperature control should be prevented.

Besides the temperature and strain rate sensitivity, asphalt concrete response also depends on the state of stress. Although it is generally known that the response of asphalt concrete in tension differs from that in compression and that the bearing capacity increases with increasing confinement, the consequences of this stress dependent behaviour are often neglected. That the response is state of stress dependent implies that it will be different for different states of stress. This explains why it is so difficult to relate test results to construction behaviour. The stresses in a road construction vary enormously and damage will occur at the position where the state of stress that is most damaging to that mix will occur, while a test only submits the mixes to a single combination of states of stress. Furthermore, most "simple" tests have complicated non-uniform internal stress distributions and that results in non-uniform behaviour. As a result the response that is observed in, for example, Marshall, bending and indirect tensile tests is the result of the cumulative response to a (particular) combination of stresses. This makes it hard to interpret the results, since this response will occur only for that particular combination of stress states. For this reason it is nearly impossible to generalise the response or to compare it to tests on other mixes or other set-ups.

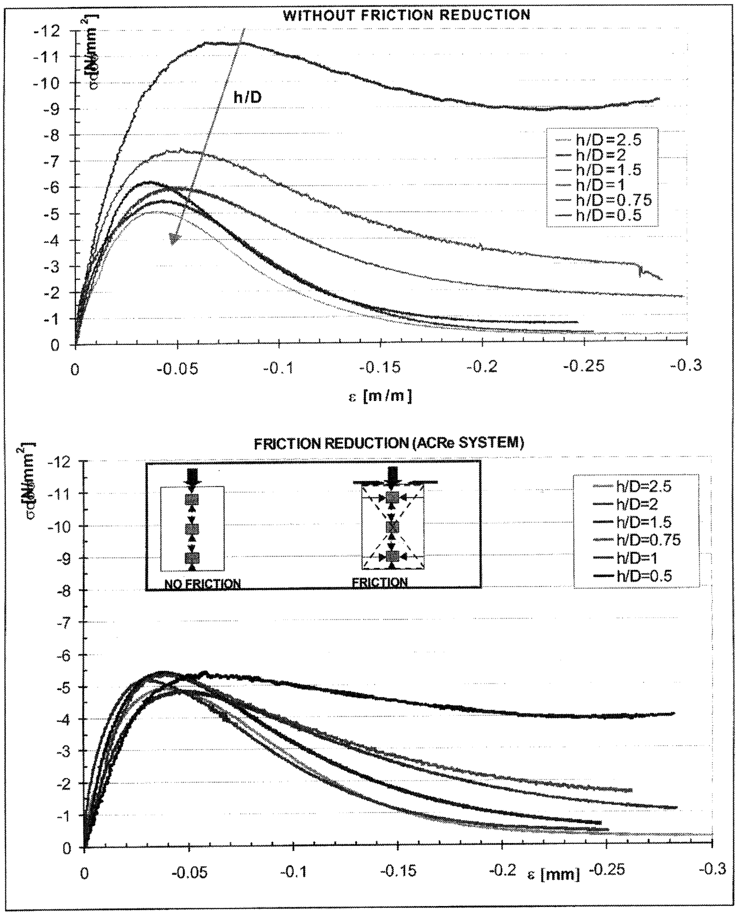

Figure 3: An example of the state of stress sensitivity of asphalt concrete (tests for T=30°C and strain rate = 0.05 s^{-1})

That the effect of the state of stress can be considerable is illustrated by Figure 3. In this graph the average stress strain curves of uniaxial compression tests on specimens with different height to diameter (*h/D*) ratios are shown. For each ratio a number of tests with and without reduction of the friction between specimen and loading plates was performed. From Figure 3 it can be seen that, especially for smaller *h/D* ratios, the effect of friction (which leads to a triaxial compression state of stress near the loading plates) is considerable. If this influence were neglected, the compressive strength would be considerably over-estimated. For more information on the friction reduction system, the interested reader is referred to Erkens et al. (1998).

3 The ACRe project

In order to arrive at a better understanding of the damage mechanisms in asphalt concrete it is necessary to establish more fundamental knowledge about the material properties and response. In that response, the above mentioned influences must be considered. Furthermore, it must be realised that to truly investigate the material behaviour, structural influences on the observed response should be prevented. Tests should be designed with their potential problems in mind, trying to establish uniform internal states of stress.

At this moment the Finite Element Method is used in many related engineering material fields to study the influence of the material properties on the structural response. This numerical method offers the possibility to actually look inside the structure in order to witness the damage mechanisms that occur, which will generate new insight in those mechanisms. The understanding of these mechanisms will enable the improvement of design methods, which will lead to better roads and a reduction in maintenance.

A pre-requisite for the use of the Finite Element Method is the availability of material models that can describe the triaxial behaviour in both the linear and non-linear range. For Asphalt Concrete such a model is not available at present. For this reason, a project was started at the Delft University of Technology to develop and implement a three-dimensional non-linear material model for asphalt concrete. The model will incorporate all aspects of asphalt material behaviour, elasticity, visco-plasticity and cracking and it will be implemented in the Finite Element Package CAPA-3D (Scarpas, 1992). The model that was selected after an extensive literature survey is the model proposed by Desai (Desai 1986).

3.1 *The material model*

The ACRe-model uses the flow surface proposed by Desai (1986) in combination with a set of constitutive relations developed to facilitate the description of asphalt concrete response. In broad outlines, the model works as shown in Figure 4. This figure compares the one- and two-dimensional situation. Originally, the behaviour is linearly elastic, represented by the straight line until point 1 in the left-hand diagram. From point 1, which corresponds to ellipse 1 in the 2D case, the response is non-linear but the load carrying capacity can still increase. This response is commonly known as hardening and in the 1D case it looks as a curve with diminishing slope (between points 1 and 3). In the 2D case this phase of response corresponds to a series of successive, in size increasing, ellipses (1 to 3). The strength (the maximum, point 3) in the 1D situation corresponds to the largest ellipse in the 2D case. After this point the strength decreases (softening), which is illustrated by the descending branch in the 1D diagram (between point 3 and 4). In the 2D case this again corresponds to series of successive ellipses, this time diminishing in size. A material model consists of the shape and size of the flow surface (the ellipse), the relations which describe the changes in shape and size (the successive ellipses) and the relations between stresses and strains inside the ellipse at each stage (within the first ellipse this would be Hooke's law).

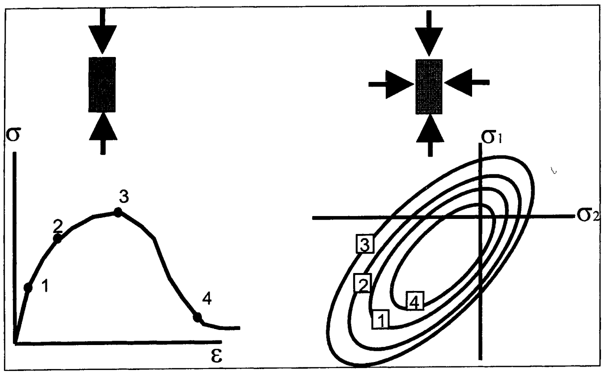

Figure 4: The principle of a material model for the 1- and 2-dimensional case. The numbered points in the 1D diagram correspond to the ellipses in the 2D case.

In the elastic region the constitutive relation is:

$$\underline{\sigma} = \mathbf{D}\varepsilon \tag{2}$$

In which σ is the stress vector, the strain vector and **D** the elasticity matrix. Equation is the three dimensional form of Hooke's law. As long as the state of stress is such that the material is not damaged, this relation describes the deformations. Once the material response becomes inelastic, the material exhibits remaining deformations. This damage can be cracking and / or plastic deformations, based on the constitutive relations that are used. Consequently, the model is capable of predicting and describing combinations of failure mechanisms, such as rutting and cracking. In the inelastic region, the strains are decomposed into several components:

$$\varepsilon = \varepsilon_e + \varepsilon_i = \varepsilon_e + \varepsilon_p + \varepsilon_{cr} \tag{3}$$

Where ε_e is the elastic strain and ε_i the inelastic strain, which consists of a cracking (ε_{cr}) and a plastic component (ε_p). Since the stresses in a material are related to the elastic strains, the constitutive relation in the inelastic region becomes:

$$\underline{\sigma} = \mathbf{D}\underline{\varepsilon}_e = \mathbf{D}(\underline{\varepsilon} - \underline{\varepsilon}_p - \underline{\varepsilon}_{cr}) \tag{4}$$

Whether the material is damaged by a specific state of stress depends on the flow surface:

$$f = \frac{J_2}{p_a^2} - \frac{\left[-\alpha\left(\frac{(I_1 - R)}{p_a^2}\right)^n + \gamma\left(\frac{(I_1 - R)}{p_a^2}\right)^2\right]}{\sqrt{1 - \beta\cos(3\theta)}} = 0 \tag{5}$$

For:

$f < 0 \Rightarrow$ *elastic behaviour, the material is not damaged,*

$f = 0 \Rightarrow$ *inelastic behaviour, the material gets damaged.* (6)

With: J_2 is the second deviatoric stress invariant

I_1 is the first stress invariant

θ is Lode's angle

p_a is the atmospheric pressure

α, β, γ, n and **R** are the model parameters

The stress invariants describe the state of stress that is evaluated and the model parameters control the size, shape and position of the flow surface. Since the flow surface controls what states of stress damage the material, the model parameters actually control the response to the state of stress that is applied. Because the material response for asphalt concrete depends on the temperature and loading rate, the model parameters can be expressed as a function of these influences. The effect of the individual model parameters is illustrated in Figure 5.

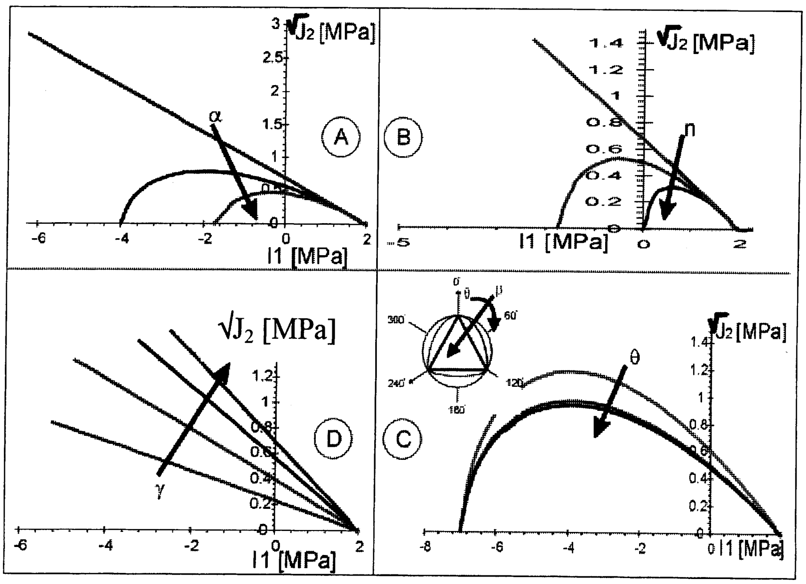

Figure 5: Influence of the model parameters on the size, shape and position of the flow surface in the I_1- $\sqrt{J}_2$ surface.

In Figure 5, (A) shows the influence of α, the hardening parameter, which controls the size. For $\alpha=0$, the surface becomes a straight line. Figure 5 (B) shows the effect of n, which is related to the onset of dilatation, Figure 5 (C) shows how β controls the shape of the cross-section on the π-plane, which varies from circular to triangular. Finally in Figure 5 (D) the way in which γ determines the ultimate slope of the surface is illustrated.

The fifth model parameter, R, is the triaxial tensile strength. This parameter determines the position of the flow surface, since R is the intercept with the positive I_1-axis. For $R=0$, the material is cohesionless and the surface passes through the origin, while for higher R values the intercept shifts to the right. In all cases shown in Figure 5 R is set to 2 MPa.

For a given asphalt mix each set of conditions (temperature and strain rate) corresponds to a set of model parameters. These original parameter values describe the elastic (damage initiation) limit for those conditions. The material degradation is incorporated in the model by changing some of the parameters as a function of the inelastic strains. The hardening (an increase of the area within the flow surface, Figure 4) is described by decreasing $\propto$ from its original value to zero. From Figure 5a it can be seen that this leads to an open surface (straight line). Softening is modelled by decreasing the slope of this "ultimate response" line via a reduction of the original γ value.

More detailed descriptions of the model at different stages throughout the project can be found in other publications (Scarpas et al. 1997, Scarpas et al. 1998a and b and Erkens et al. 2000).

3.2 *Tests that provide the model parameters*

The 3D material model described in the previous section provides a relation between stress and strain for any state of stress. Of course it is impossible to test all those states of stress, since the possibilities are infinite. Instead, a limited number of stress conditions is used in combination with a (mathematical) model that matches the expected material behaviour, based on the states of stress that are tested, the model parameters are determined. This means that the results from those test conditions are generalised to a full 3D surface on the basis of the characteristics of the model that you chose. In the ACRe project the material model is being developed along with an experimental programme that results in the required model parameters. The test programme involves among others uniaxial tension and uniaxial compression tests. The results from these two tests suffice for a first estimate of the model parameters (the first estimate of the 3D surface can be found from those test results).

A material model basically predicts the strain that results from a given state of stress. That prediction is based on the actually measured response to a limited number of states of stress, in tests. In order to get a reliable model, a sufficient number of stress conditions, preferably far apart in the 3D-stress space is needed Ideally, to determine the model parameters for a triaxiality, temperature and strain rate dependent material one would like to use triaxial tests at different temperatures and strain rates. On the other hand, knowledge on the uniaxial behaviour is very useful to get an impression of the capacity that is needed in triaxial testing. Furthermore, uniaxial tests are special cases of triaxial testing, since they can be considered triaxial tests with zero confinement. This means that they do provide information for the 3D material model. Since conventional triaxial test equipment originates from soil mechanics, it is not capable of applying tension-compression types of loading. These tension-compression states of stress are of particular interest, since they cause a lot of damage in asphalt pavements (Figure 1). For this reason, it was decided to perform the multiaxial tests in the ACRe project not by means of a triaxial cell but with a four-point shear set-up that was developed during a previous project. This consideration, in combination with the fact that uniaxial compression and tension tests already provide a great deal of information on the model parameters led to a test programme that included uniaxial compression and tension tests and well as multiaxial tests.

3.2.1 *The uniaxial compression test*

The set-up was built in a 3D-space frame that was connected to a concrete block foundation. This heavy construction was necessary, since the actuator mounted in the set-up had a capacity of 150 kN. Such a large actuator was necessary to perform tests at low temperatures and high loading rates. Inside the space frame, a pedestal supported the bottom loading plate. The top plate was rigidly connected to the actuator. Massive guidance bars were connected to the bottom plate and passed through linear bearings in the top plate. These guidance bars prevented rotation of the actuator and, thus, the top plate during tests at low temperatures and high loading rates. They also ensured that the loading plates remained parallel throughout the test. An insulated temperature cabinet was placed around the loading plates. In this cabinet, the temperature could vary between –5 °C and 33 °C. The temperature was controlled via two temperature sensors, one connected to the top plate and one connected to the bottom plate. These sensors and one additional temperature sensor that registered the air temperature were monitored throughout the test.

The constant deformation rate that was applied to the specimen was controlled through a set of three displacement transducers, which were placed between the loading plates, at 120° intervals. These displacement transducers were also used to register the axial deformation of the specimen. Additional information on the deformations in the linear range of response was obtained from an additional set of three displacement transducers with a smaller range. A similar approach was used for the radial deformations, which were registered by means of a string that was wrapped around the specimen and passed over two potentiometers. Additional information on the small deformations that occurred at the beginning of the test was obtained from an MTS circumferential kit.

The applied load was registered by a LeBow loadcell, which could be calibrated for and used in four different ranges: 20, 50, 100 and 150 kN, depending on the test conditions and the expected response. This was necessary because the compressive strength (f_c) varied widely over the test conditions. These covererd three temperatures (0, 15 and 30°C) and four strain rates (0.1×10^{-2}, 1×10^{-2}, 5×10^{-2} and 10×10^{-2} s^{-1}). The average results from the tests are presented in Figure 7, Figure 8 and Figure 9.

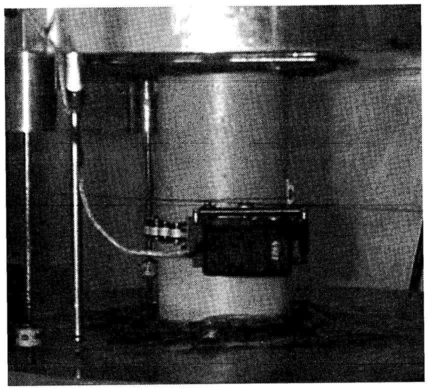

Figure 6: Close-up of the specimen in the compression set-up, showing the guidance bars and the instrumentation that registered the axial and radial deformations.

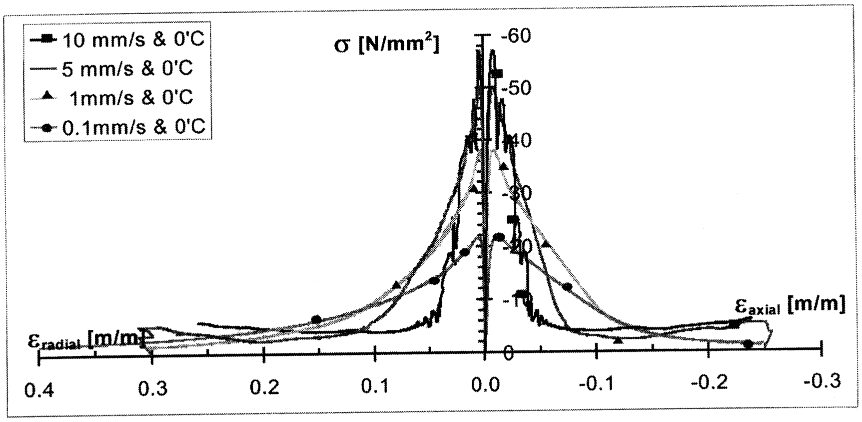

Figure 7: Average test results for 0°C.

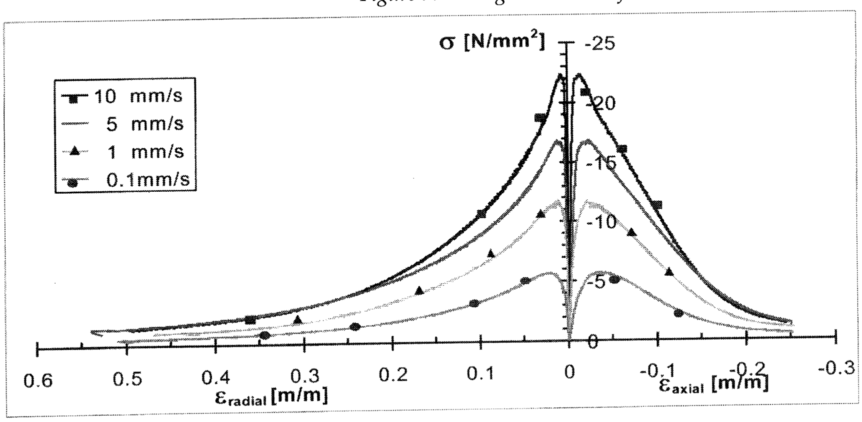

Figure 8: Average test results for 15°C.

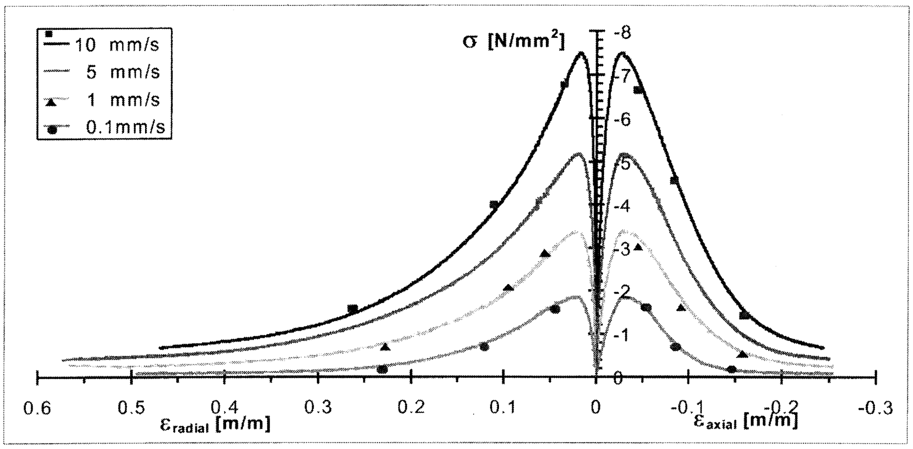

Figure 9: Average test results for 30°C.

Based on the test results an expression for the compressive strength as a function of strain rate and temperature was developed. Such a relation is necessary to generalise the results for use in a model that can be used to describe the response for conditions that were not tested. Before determining the relation some expectations about the general trend were formulated, which are shown underneath:

- **v=0:** $\mathbf{f_c=0}$
- **v→∝:** the strength will not increase indefinitely, but reach a limit value: $\mathbf{f_c=C(T)}$
- **T→-∝:** for extremely low temperatures, asphalt will exhibit glass-like, linear-elastic behaviour until sudden, brittle fracture occurs: $\mathbf{f_c=C^*}$
- **T →∝:** for extremely high temperatures (approximately 160°C) the bitumen will become a fluid: $\mathbf{f_c=0}$

These considerations, which are discussed in more detail in Erkens et al. 2000, led to the expression shown in Equation (6).

$$f_c = -108\left(1 - \frac{1}{1 + \left[\dot{\varepsilon} * e^{\left(-86.3 + \frac{24260}{T}\right)}\right]^{0.32}}\right) \tag{6}$$

With:

f_c = compressive strength in N/mm²
$\dot{\varepsilon}$ = strain rate in s^{-1}
T = temperature in Kelvin
R^2 = 0.99

The average compressive strengths obtained from both test programme and the values predicted by Equation (6), as well as the differences between the two, are shown in Table 2.

Temperature [°C]	Strain rate [s^{-1}]	f_c tests [N/mm²]	f_c Eq. [N/mm²]	Δf_c [N/mm²]
0	0.1 ×10^{-2}	-21.5	-21.5	0
0	1 ×10^{-2}	-38	-37	+1
0	5 ×10^{-2}	-49.5	-50.3	-0.8
0	10 ×10^{-2}	-56.5	-56.3	-0.2
15	0.1 ×10^{-2}	-5.7	-5.8	-0.1
15	1 ×10^{-2}	-11.5	-11.4	+0.1
15	5 ×10^{-2}	-17	-17.9	-0.9
15	10 ×10^{-2}	-22.3	-21.4	+0.7
30	0.1 ×10^{-2}	-1.9	-1.6	+0.3
30	1 ×10^{-2}	-3.5	-3.3	+0.2
30	5 ×10^{-2}	-5	-5.4	-0.4
30	10 ×10^{-2}	-7.5	-6.6	+1.1
Sum of Squares		*3766.64*	*4.57*	

Table 2: Measured and predicted average compressive strengths .

A graphical representation of the test results and the strengths predicted by Equation (6) are shown in Figure 10. The markers represent the individual test results and the lines show the predicted values.

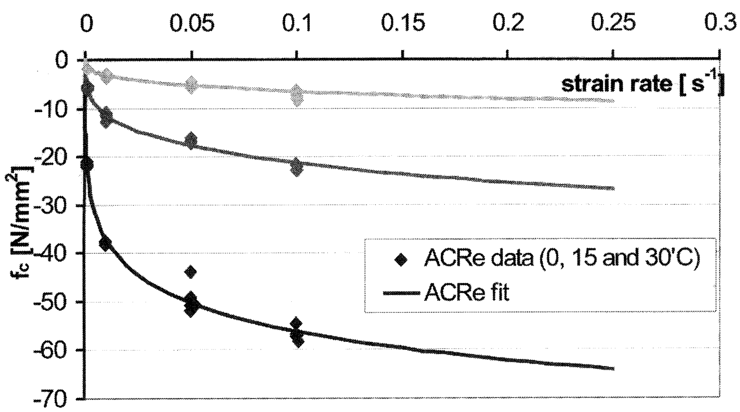

Figure 10: The individual data (markers) and predicted values (lines).

Already at the beginning of this paper it was mentioned that the model parameters are determined on the basis of test results. Of course, the parameters can be determined most accurately when all test results are available. But it would be impractical to wait with the parameter determination until the end of the project. For modelling and model calibration reasons, preliminary values for the model parameters are determined at every stage of the project. In this case, it means that the results from the uniaxial compression tests described above and a series of preliminary uniaxial tension tests were used. The parameters were used in the simulations presented in the next section.

4 An example application of the ACRe model for Asphalt Concrete

Since the influences of strain rate, temperature and state of stress are incorporated in the model, the behaviour corresponding to the conditions in any point of a construction can be determined automatically. As a result, the "weak spots" in any combination of geometry, material and loading will show up in an analysis, because they will be the first to show damage.

For example, if the layer thickness changes, the stress distributions also change. To investigate this effect a dynamic 3D finite element simulation using the ACRe-model in its current, prototype formulation, was run using the package CAPA-3D (Scarpas, 1992). The load was simulated by a circular evenly distributed, standard axle (p=0.707 N/mm^2) which "patted" the pavement at a single position (like a repeated falling weight load). Damage was defined as the square root of the sum of the squares of the permanent strain components (Equation (7)). Since the test programme is still underway, the model parameters are based on incomplete data and should be considered as preliminary values. For this reason, the actual values are not of real interest in this simulation but the overall pattern is, since the effect of changes in the thickness and such can become more or less pronounced with changes in the model parameters, but the effect itself remains. This effect is shown in Figure 11, for half the cross section at the centre of the load.

$$\varepsilon_{eq} = \sqrt{\varepsilon \cdot \varepsilon^T} = \sqrt{\sum \varepsilon_{ij}^2} \quad ; \quad i, j = x, y, z \tag{7}$$

As can be seen, the damage in the thinner, more flexible construction starts at the bottom while that for the thicker and stiffer construction initiates at the top. This seems logical, since thin constructions are sensitive to bending, while thicker constructions are more sensitive to shear. In the first case, damage would be expected to initiate at the bottom, where bending tensile stresses occur, while in the second case damage is expected higher up in the construction. This effect is also known from practise, since thicker (high way) construction in many countries (i.e. Great Britain, the Netherlands and South Africa) do not show the classical fatigue crack pattern (bottom up), but rather damage that initiates at the top (Groenendijk, 1998).

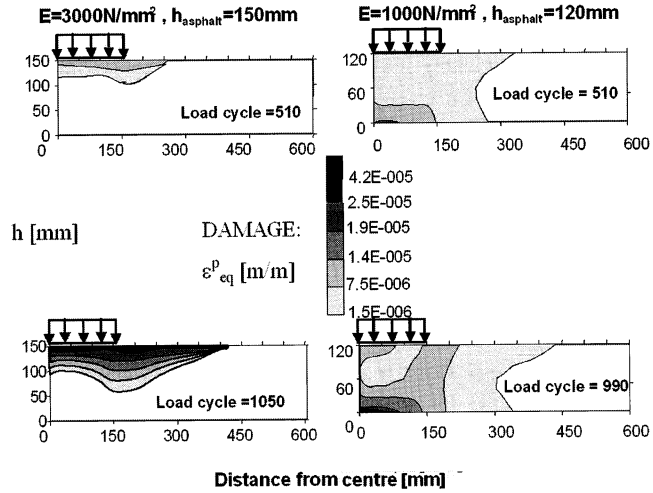

Figure 11: Prediction of damage patterns for two different pavement structures using a non-linear material model.

5 Conclusions

Typical road engineering tests are aimed at providing an indication of structural parameters, while being simple to perform. As such, they can not be used for material parameter determination, because this requires sophisticated testing with the aim of establishing a simple and reproducible internal state of stress. For asphalt concrete this means that a test should not only result in a simple state of stress, but it should also impose a single strain rate and temperature. By varying only one of these parameters at the time, the material properties can be expressed as functions of temperature, strain rate and state of stress. The approach is illustrated by the general expression derived for the compressive strength on the basis of the tests results presented in Section 3.2.1. This will allow the analyses of more complicated stress states, such as exists in may standard tests and road constructions, by means of the Finite Element Method.

If all the important variables that influence the response are incorporated in a material model, it is possible to predict the "weak spots" for each combination of the construction geometry, material and load. This allows the detection of different damage mechanisms for different combinations of geometry and material and this information can be used to develop and enhance design procedures. As such it allows more realistic design approaches than the current ones, where failure is supposed to occur at the same position regardless of the materials and construction geometry.

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