Mix Rheology - a Tool for Predicting the High Temperature Performance of Hot Mix Asphalt

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Introduction

A significant amount of current work has been directed towards evaluating how the performance of bituminous mixtures is influenced by the binder properties [1,2,3,4,5]. However it is also important to relate the final mixture properties to pavement performance. The binder without question plays a prominent role but it is only one component in a very complex composite. Consequently, the long term objective should be to eliminate binder specifications, and call for certain mixture properties or performance levels. However, in order to do this it is imperative to have a set of truly performance related mixture tests. This need has become more apparent in recent years as a wider and wider set of binder modifiers have become more popular. This paper examines how binder and mixture rheology can be used as a tool for understanding the effect of modification and demonstrates this by test results carried out on a wide selection of binder modifiers.

The first comprehensive approach to develop a performance related mixture test that ties back to the rheological properties of the mixture and the binder was the static creep test [6,7]. In this procedure Van der Poel's Nomograph [8] is employed to relate the binder stiffness to the mixture stiffness. This relationship is then used to determine the viscous behavior under set traffic conditions [9] in order to predict permanent deformation in pavements [10,11]. However, these methods are limited to conventional non-modified binders and require generally empirical correlation factors to obtain reasonable predictions. In the case of highly elastic polymer modified binder systems Valkering et al. showed that static creep does not capture the true performance of the mixture with relation to permanent deformation [12]. Recent work by Bognacki et al. [13] and Bouldin et al. [14] indicate that even the relative rating obtained by static creep may not be correct. This is especially true when highly elastic soft binders are compared with inelastic stiff binders.

The reason why the static creep test fails in ranking mixtures with respect to permanent deformation is that this test does not capture elastic recoil and the time dependency of the material properties. To obtain a better understanding of the true rheological behavior of mixes it is necessary to run frequency sweeps on the material and/or to subject the material to the repetitive creep testing. The latter is especially interesting since it has been shown to correlate well with wheel tracking results.

This paper discusses the different performance related mixture tests which have been developed to predict the rut resistance and the differences in these tests are described. Also binder type and air void content are related to the properties of bituminous mixtures.

Experimental

A detailed in depth discussion of the experimentation is not given in this paper and instead reference is made to other papers which deal in detail with the experimental techniques used in this paper.

Material Preparation and Mix Testing

A detailed description of the binder preparation and rheological characterization of both straight and polymer modified bitumen are given in references [2] and [3]; the mixture preparation for the uniaxial testing and the wheel tracking in reference [15]; and the mixture preparation for the hollow cylinder and the simple shear testing device are given in reference [16]. All samples with exception of the NYNJ samples were compacted using a rolling wheel compactor so as to best simulate field compaction. The NYNJ mixes were compacted using a gyratory compactor. The air void levels of the prepared specimen were then determined using the Strategic Highway Research Program

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The oral presentation was made by Dr. Bouldin.

(SHRP) parafilm wrap technique [17].

Uniaxial creep testing was carried out in a pneumatic device, the Nottingham Asphalt Tester (see Cooper et al. [18]). The simple shear tester is described in the SHRP report [17] and the testing methodology for the hollow cylinder has been described by Sousa et al. [19]. For more details on the TRRL wheel tracking device is described in a TRRL leaflet [20]. Note : Bitumen and mixture references are given in the glossary, Appendix B.

Results and Discussion

Rheological Characterization of Mixes

Before looking at the experimental results it is important to understand the effects that occur in a bituminous pavement when it is being subjected to traffic loading. Figure 1 shows a schematic of a rut curve. These characteristic curves can be obtained by monitoring road pavements or conducting simulative wheel tracking experiments.

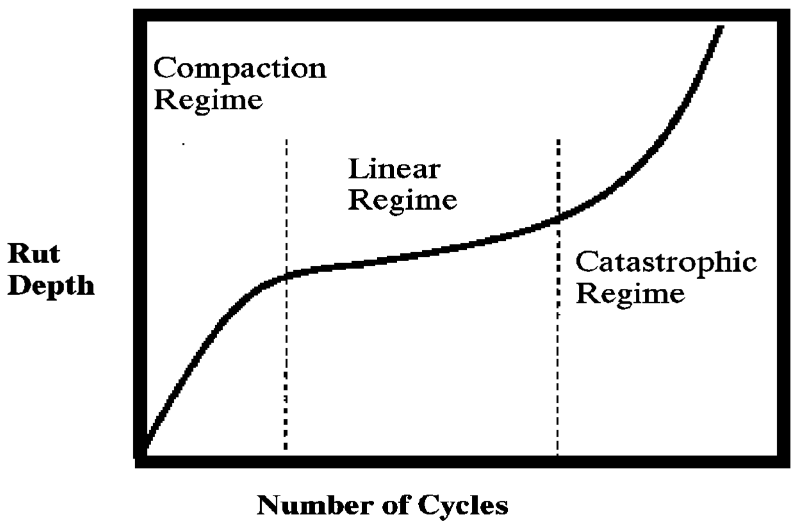

Figure 1 : Schematic Rut Curve.

In the initial range the material is considered to experience additional compaction and/or rearrangement of the aggregate skeleton. The relatively high local pressures results in the reorientation which ultimately leads to an improved aggregate interlock. Consequently, the slope reduces (or rut rate, rr, see Equation 1) as the modulus of the mixture increases (cf. Figure 1).

$$rr = \frac{d\epsilon_{acc}}{dn} \qquad \textbf{(1)}$$

In the second range, the linear range, the rate of deformation is slower. Often this range is assumed to occur after a predefined number of cycles. A common approach is then to draw a straight line from the value of strain at this point to a point consistent with the beginning of the next stage to obtain the rut rate. To determine the linear range accurately it is, however, imperative that one determines where the rut rate reaches a constant value, i.e., where the slope of the rut rate is zero (see Equation 2).

$$\frac{drr}{dn} = \frac{d^2 \varepsilon_{acc}}{dn^2} = 0 \tag{2}$$

This is illustrated in Figures 2 and 3. In some case no significant linear range occurs because the material is very unstable or the loading conditions are so severe that it reaches the tertiary range before reaching a constant slope .

The tertiary range or catastrophic range is reached when the rut rate begins to increase again. In this range we observe large scale aggregate movements which are accompanied by significant volumetric effects, i.e., the material exhibits dilatancy. This effect will be discussed later.

Before analyzing how well the different mixture tests relate to this rutting mechanism it is important to consider what kind of information can be extracted from the different types of mixture tests. Different tests can be used to obtain data relating to various objectives, as follows:

- Relative ranking of binders with respect to rutting.
- Relative ranking of mixes with respect to rutting.
- To obtain data for mechanistic pavement performance model [21,22].

To determine the relative rut performance of a mixture it is important that the test correctly defines the slope in the linear range. The definition of initial and final stages (compaction/catastrophic) appears to be less critical in this respect.

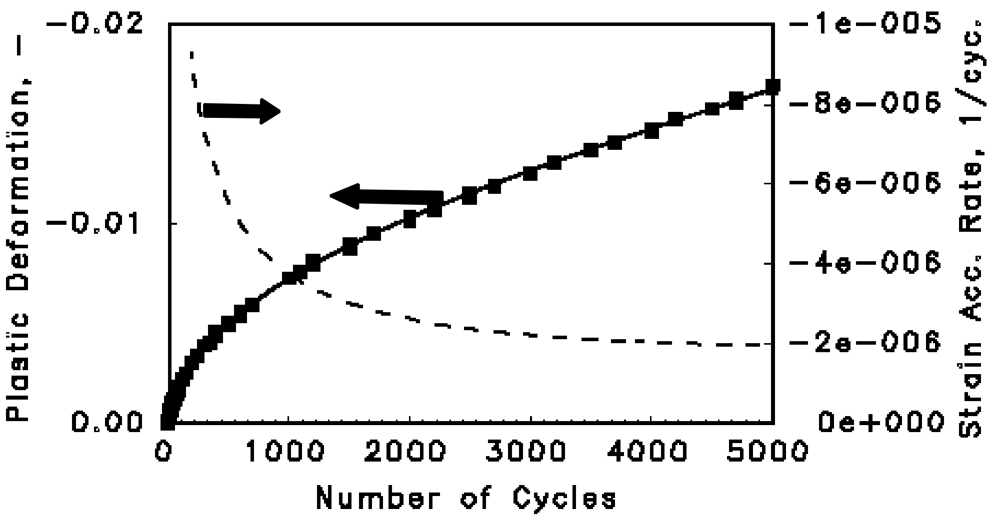

Figure 2 : Plastic Deformation and strain accumulation rate for 3% w EVA in a CV AR-4000 in the MES mixture. Test is carried out in repetitive creep at 60C.

To determine the significance of the initial compaction samples were removed from a heavily trafficked pavement (wheelpaths and lane centers). Air voids were measured and compared with the original in place air void contents. The results indicated that in this mixture the post-construction compaction was relatively low, thus suggesting that the air void levels in properly compacted specimens do not change very much. Hence, the uniaxial repetitive creep test and the simple shear test, which do not lead to any large volumetric changes, can be used to obtain mixture ranking. In addition these tests can also be used for mixture optimization and design. The results coincide with Eisenmann and Hilmer's conclusion that non-catastrophic rutting is accompanied by only small volumetric changes [23].

However, it is imperative that the testing be carried out at the correct frequency due to the time dependency of the material properties. If possible one should attempt to simulate the load profiles which occur in the field as closely as possible. Consequently, there is an advantage in moving away from the step load condition that had been traditionally used to a haversine wave form as shown in Figure 4. An additional advantages is that the initial slope of the load curve is by definition zero (see Equations 3 to 5).

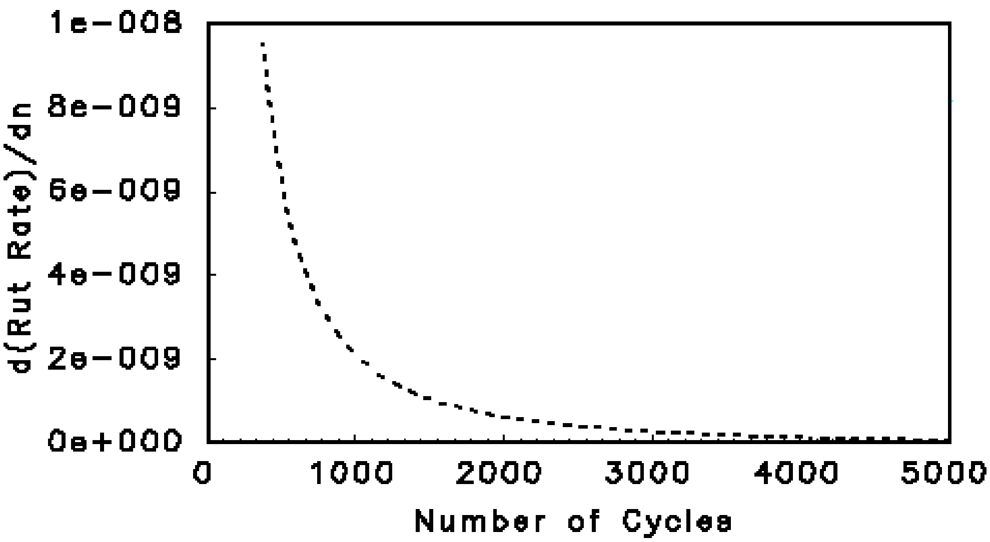

Figure 3 : **Slope of the rut rate for 3%w EVA in a CV AR-1000 in the MES mix. Test is carried out in uniaxial repetitive creep at 60C.**

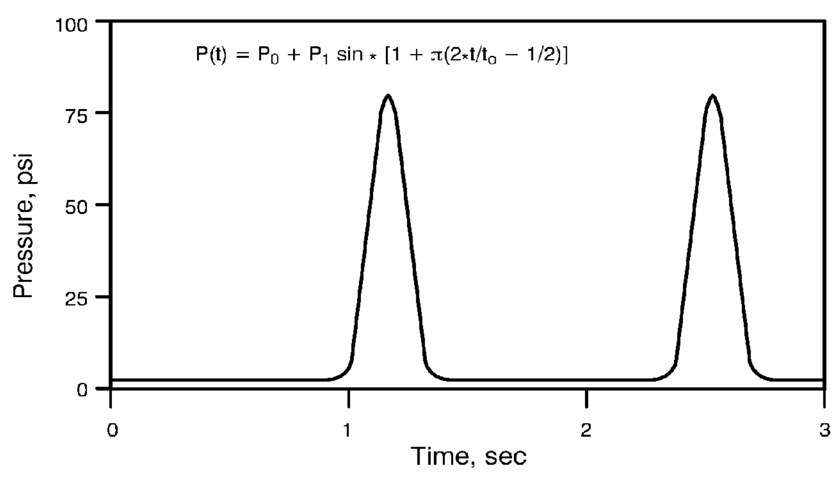

Figure 4 : **Haversine wave load application used in uniaxial repetitive creep with servo hydraulic system.**

$$p(t) = p_o + p_1 \, [1 + \sin[\pi(\frac{2t}{t_o} - ½)]] \quad \textbf{(3)}$$

$$(\frac{dp}{dt}) = \frac{2\pi p_1}{t_o} \cos[\pi(\frac{2t}{t_o} - ½)]] \quad \textbf{(4)}$$

for t = 0 dp/dt = 0

The haversine load application is very easy to control in contrast to the square wave form which has an initial slope of infinity. Square load applications lead frequently to stress overshoots or if one ramps up the load slowly a trapezoidal load curve results. The above applies only to testing with servo hydraulic systems. When working with pneumatic systems a trapezoidal shaped load pulse is obtained.

To predict rutting one approach is to run the testing at the mean highest weekly average temperature as proposed by SHRP [24]. The latter can be approximated using the SUPERPAVE™ temperature algorithm or alternatively by using more sophisticated finite element heat transfer programs such as HiRoad™ [25]. A more precise approach is to do the testing at a damage weighted temperature. This approach will be discussed in more detail in a later section. Table 1 shows how by varying the temperature and load application length the relative rut resistance can change. For example the modified binder which is very viscous but which does not significantly enhance the elastic behavior of the NYNJ mixture does even better than the SEBS modified binder at very long loading times. If, however, the loading periods are reduced so as to better simulate field loading conditions the SEB modified binder performs substantially better.

When the tests are being used to generate input data for the sophisticated finite element analysis pavement performance programs it is important that the true mechanical behavior of the mixtures is captured in the mixture test. Information is required on aspects as follows:

- Non-linear viscoelastic effects such as, for example, dilatancy.
- Plastic behavior.
- Rate dependency of the material properties.
- Stress and strain dependency of the material properties.

All this information cannot be obtained from one test such as, for example, repetitive creep. When doing the former one can map out the time and strain dependency of the material functions by using simple linear viscoelastic models.

Figure 5 shows a master curve of a mixture that was constructed using the time-temperature superposition principle [26]. As shown in Figure 6 the frequency dependency of the storage or loss modulus (Equations 6 and 7) can be closely mapped using a set of 4 to 9 relaxation times. The relaxation time spectrum shown in Figure 6 and Table 2 was obtained using a non-linear regression program developed by Baumgaertel and Winter [27].

Table 1 : Influence of loading time and temperature on strain accumulation rate.

Base Bitumen	Modifier	Mixture Design	t_o, sec	t_{rel}, sec	ϵ', μm/m/ cycle	T, C (F)
AC-40	-----	NOTT	100	300	388	53 (127)
AC-40	-----	NOTT	0.1	0.3	9.5	53 (127)
AC-40	6% SBS	NOTT	100	300	58	61 (142)
AC-40	6% SBS	NOTT	0.1	0.3	2.1	61 (142)
AC-0.5	6% SBS	NOTT	100	300	67	61 (142)
AC-0.5	6% SBS	NOTT	0.1	0.3	1.4	61 (142)
AC-20	-----	NYNJ	30	20	3930	60 (140)
AC-20	-----	NYNJ	5400	0	12,750	60 (140)
AC-20	6% SEBS	NYNJ	30	20	130	60 (140)
AC-20	6% SEBS	NYNJ	5400	0	8,820	60 (140)
AC-20	5% Modifier	NYNJ	30	20	230	60 (140)
AC-20	5% Modifier	NYNJ	5400	0	3,580	60 (140)

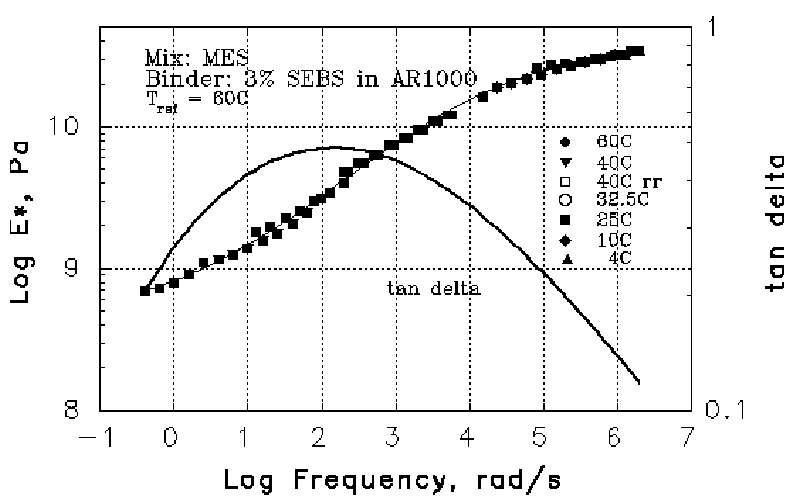

Figure 5 : Master curve of a MES mixture containing a CV AR-1000 modified with 3%w SEBS.

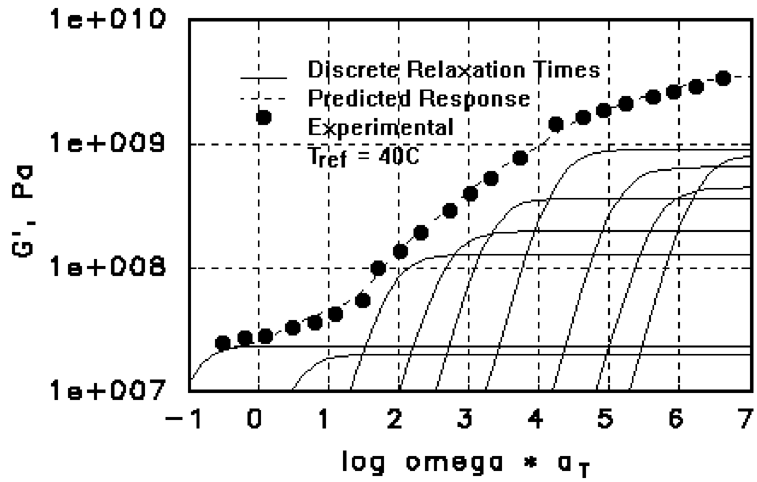

Figure 6 Frequency Dependency of the Storage or Loss Modulus

Table 2 : Moduli and relaxation times of a mixture of the SHRP MRL aggregate RH with binder AAM (air void level of 8 percent) using an n-element Maxwell model.

G_i, Pa	λ_i, sec
$8.3 \bullet 10^8$	5.9•10-7
$4.5 \bullet 10^8$	2.4•10-6
$6.5 \bullet 10^8$	8.4•10-6
$9 \bullet 10^8$	6.2•10-5
$3.7 \bullet 10^8$	5.1•10-4
$2 \bullet 10^8$	2.3•10-3
$1.2 \bullet 10^8$	0.014
$2 \bullet 10^7$	0.35
$2.4 \bullet 10^7$	9.8

$$E_{\omega}^{/}=\sum \frac{E_i.\lambda^2.\omega^2}{1+\lambda^2.\omega^2} \quad \textbf{(6)}$$

For normal pavements where only a limited time domain needs to be covered, 2 to 4 relaxation times will provide a good approximation of the material properties. This is shown in Figure 8 where we fitted both the deformation during one load pulse and the subsequent relaxation time using a two element Maxwell model which is shown in Figure 7 [28].

$$E_{\omega}^{//}=\sum \frac{E_i.\lambda.\omega}{1+\lambda^2.\omega^2} \quad \textbf{(7)}$$

The elastic and viscous components of the Maxwell elements (Es and Vs) can be determined by numerical analysis of the creep pulses and/or the frequency sweeps. However, the stiffness and viscosity of the mixture is not only temperature but also strain dependent, Equations 8 and 9. This stems from the fact that the material strain hardens as shown in Figure 9.

$$E = f(\omega, T, J_2) \tag{8}$$

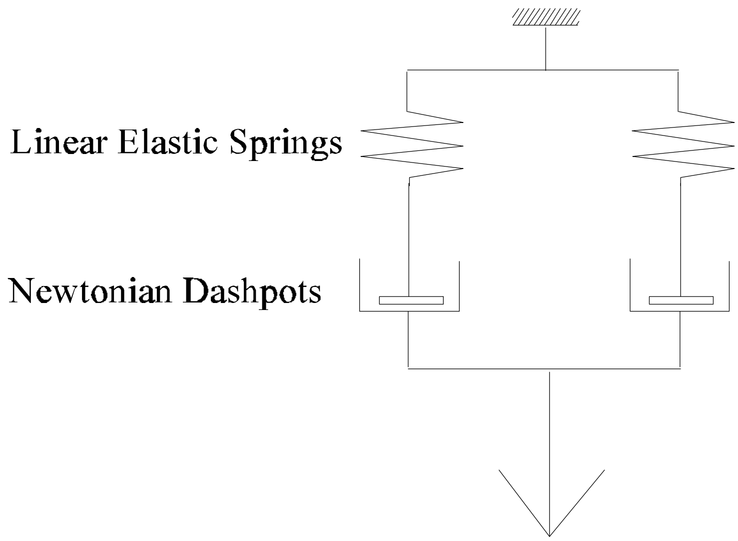

Figure 7 : Two element Maxwell model.

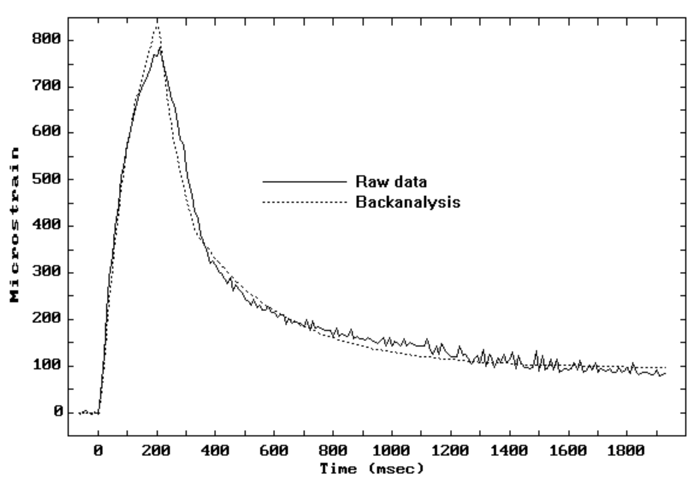

Figure 8 : Measured versus strain using a two element Maxwell Model

$$V = f(\omega, T, J_2) \tag{9}$$

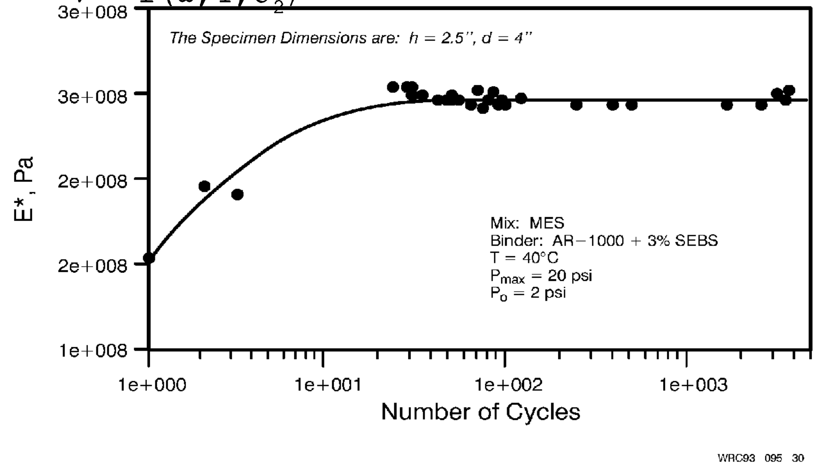

Figure 9 : Effect of strain hardening on mixture stiffness.

Consequently, a single curve does not describe a material adequately but instead a set of curves is needed. This effect becomes more pronounced at high temperatures where J_2, a measure of shear strain, can reach large values. Therefore one cannot measure only one frequency sweep to rheologically characterize a mix. The frequency sweeps shown here were all obtained by preconditioning the sample. In this preconditioning the sample was subjected to a series of loads until the stiffness modulus tapers off as shown in Figure 9. It is furthermore assumed that the material properties are a function of the second deviatoric strain invariant, J_2, and not of the actual accumulated strain.

As previously mentioned dilatancy is important to describe large strain behavior of bituminous mixtures. Earlier work by Sousa et al. had indicated that volume increases may occur [29]. A measure for dilatant behavior is the Poisson's ratio[18], ν, which is the defined as a function of the ratio between the shear and tensile/compressive stiffness.

$$\nu = \frac{E}{2G} - 1 \tag{10}$$

One would expect large scale aggregate movements to occur with soft (low moduli) mixes and/or with mixes approaching catastrophic failure. These dilatant mixes should exhibit comparatively large radial strains, i.e., the shear modulus should be significantly smaller than the tensile modulus.(For incompressible materials one obtains a Poisson's ratio of 0.5 and hence the tensile/compressive modulus is three times as large as the shear modulus.) This issue is very important because most current analysis models assume a Poisson's ratio of 0.35 for bituminous materials.

The hollow cylinder provides a unique tool to determine the importance of dilatancy because the equipment can measure the extensional and the shear modulus directly on the same specimen. The experimental results shown in Figure 9 indicate a clear dependency of the Poisson's ratio on the stiffness of the mix. The Poisson's ratio appears to follow one relationship and is apparently not influenced by the type of binder. At high stiffness values the Poisson ratio is approximately 0.3 while at low stiffness values Poisson's ratios up to 7 were measured. This

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[5]The term Poisson's ratio has been traditionally used with isotropic linear elastic material behavior. The materials considered in this paper clearly do not meet this requirement. For this reason many authors often use the expression strain ratio or talk about volume change. The term Poisson's ratio is used in this paper since many technologists are familiar with the definition.

implies that soft materials will undergo significant volumetric changes during loading. A master curve is shown in Figure 10 that can be used to predict ν as a function of the tensile/compressive stiffness.

Influence of Binder on Mixture Performance

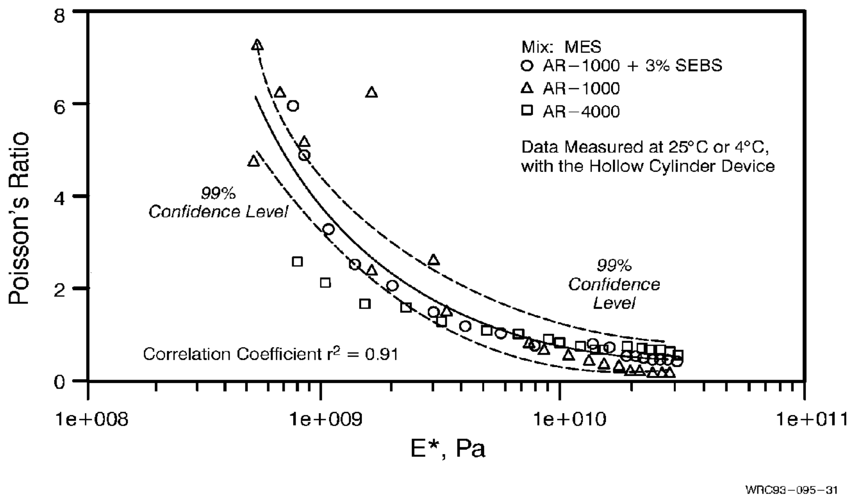

Figure 10 : Plot of Poisson's ratio versus tensile stiffness of various different binders in the mixture MES.

In the most recent years there has been some discussion on the issue how the binder rheology influences the high temperature performance of bituminous materials. Goodrich [30] had argued that there was little effect of the binder on the high temperature properties while Valkering et al. [12] and Bouldin et al. [31] had seen significant improvements in the rut resistance when using stiff or highly elastic binders in wheel tracking devices. The SHRP researchers had also based their specification on the presumption that in fact the binder plays an eminent role in preventing rutting [32]. They proposed using the inverse shear loss compliance, 1/J", (see Equation 11) as a measure for the ability of the binder to resist permanent deformation. This is a novel and intriguing approach as it captures both the stiffness and elastic behavior of the binder in one parameter via the complex modulus, G^*, and the phase angle, δ. It also accounts for the rate dependency (or loading time dependency) of the material functions because the moduli are a function of the angular velocity, ω.

$$1/J''_{\omega} = \frac{(G^*_{\omega})^2}{G''_{\omega}} = \frac{G^*_{\omega}}{\sin\delta_{\omega}} \tag{11}$$

In the case of an ideally elastic material the phase angle is 0° and hence 1/J" is infinite. In this case no permanent deformation should occur. Consequently, an ideally viscous material the phase angle is 90° and 1/J" is proportional to the viscosity of the binder as shown in Equation 15. In fact most straight unmodified binders behave purely viscous at elevated temperatures (T > 60C) as shown in Table 3. The larger 1/J" the better the binder should be in mitigating permanent deformation, the rating is directly proportional to 1/J".

$$\textit{Ideally Elastic Material}: \delta = 0° \quad 1/J'' = \infty \tag{12}$$

$$\textit{Ideally Viscous Material}: \delta = 90° \quad 1/J''_{\omega} = G^*_{\omega} = \eta_{\omega}\omega \tag{13}$$

$$rr = f\left[\frac{1}{J''_{\omega}}\right] \qquad \textbf{(14)}$$

Table 3 also clearly illustrates the effect that polymer modification has on 1/J". In the two cases shown here we find roughly a sixfold increase of the inverse shear loss compliance after addition of 4%w SBS. These mixes are in fact equivalent to the AC-40 shown in Table 3 and are expected to clearly outperform the straight AC at temperatures exceeding 60C because of the reduced temperature susceptibility of the PMBs. The PMBs perform obviously significantly better than the straight AC-40 at low and intermediate temperatures.

Table 3 : Inverse shear loss compliance and complex modulus of various polymer modified and unmodified binders at elevated temperatures.

Binder	G^* @ 60C and 1 rad/sec, Pa	1/J"@ 60C and 1 rad/sec, Pa
ET AC-5	63	63
ET AC-40	568	568
BOS AC-7	94	94
ET AC-5 + 4%w SBS	445	550
BOS AC-7 + 4%w SBS	686	735

In order to determine if the basic assumptions made by Anderson et al. [32] were reasonable a series of both straight unmodified and polymer modified binders rheologically were characterized. Wheel tracking tests were performed on mixes containing these binders. The value of 1/J" was then determined at a frequency which was commensurate to the rate of deformation in the wheel tracker. This is as previously mentioned extremely important because of the time dependency of the material functions. If the two tests are run at completely different time scales no satisfactory correlation can be expected. Figure 10 shows how well the rut rate in the linear range of the rut curve correlates with 1/J" of the binder for the Mesquite Nevada mixture described in Reference 31. The rut rate versus 1/J" relationship can be described using a simple exponential approach.

$$rr \propto \left[\frac{1}{J''_{\omega}}\right]^{\alpha} \qquad \alpha < 0 \qquad \textbf{(15)}$$

The better the aggregate the smaller the exponent α and the smaller the potential improvement in rut resistance that can be achieved using high performance binder systems [33].

The very same kind of relationship as previously mentioned can be obtained from repetitive creep experiments because the rut rate should be proportional to the strain accumulation rate. In this case one determines the strain accumulation rate, ϵ'_{acc}, in the linear range and plots it versus 1/J" as shown for the very same mixes in Figure 11.

$$\epsilon'_{acc} \propto \left[\frac{1}{J''_{\omega}}\right]^{\beta} \qquad \beta < 0 \qquad \textbf{(16)}$$

$$\epsilon'_{acc} = \frac{d\epsilon_{acc}}{dn} \quad where \quad \frac{d^2\epsilon_{acc}}{dn^2} = 0 \qquad \textbf{(17)}$$

The results are also shown in Table 4. For obvious reasons there is no one relationship for all materials because the aggregate structure plays an important role. Hence, any attempt to obtain one master curve from mixes with different binder content, air void content, gradation and aggregate type is flawed and will not lead to satisfactory results [33].

On the other hand it is interesting to note how poorly traditional mixture tests do in predicting rut resistance. This is especially true for polymer modified systems as shown in Figure 12. The Marshall stability of a series of MES mixes are plotted versus their respective measured rut rates. In these systems there is essentially no correlation between the rut rate and the Marshall stability ($r^2 = 0.08$). The reason for this is that the Marshall test cannot capture the true viscoelastic behavior of the mixes and thus will grossly under-predict the relative performance of PMBs.

Influence of Air Void Content on Mixture Performance

Another very important parameter that influences the mixture performance is the air void content. Recent work by Sousa [34] and Brown [35] illustrates the propensity of over compacted mixes to exhibit permanent deformation. However, probably the most significant problem in the field is that mixes are under compacted, i.e. that the in-place air void content is too high. Repetitive creep is an excellent tool to determine the effect that air void content has on permanent deformation. Figure 13 shows how the strain accumulation rate may increase by orders of magnitude when density is not achieved. In this case (OR mix) the actual Marshall mixture design called for an air void content of 4.4 percent which performs satisfactorily (cf. Figure 14 and Table 5). However, after constructing and coring the pavement the actual in place air void levels were found to range between 8 percent and 13 percent. Within a period of weeks the sections containing high air void levels failed and required repaving even though the base bitumen was a high performance binder. This illustrates that even highly elastic rut resistant binders will fail when not properly compacted. This is also seen with the Mesquite Nevada mixture design (MES) which tends to be significantly more rut susceptible at high air void levels (cf. Table 5). The number of load applications required to obtain 4.55 percent permanent strain in shear increases by a factor 16 when decreasing the air void level from approximately 8 to 3.3 percent as shown in Figure 15.

Determining the Appropriate Test Conditions

As previously mentioned, it is extremely important to run the testing at the appropriate test conditions. The most important test parameters are temperature and loading rate. (If one is trying to determine the stress dependency of the material properties then the state of stress becomes a third important variable.)

When one is running the testing in order to obtain data for a finite element program, the testing procedure is relatively straightforward. All one needs to do is to run the testing within the temperature envelope that the pavement temperature program has determined for the location in which one is interested. Figure 16 shows the temperature depth profile envelope obtained for a typical site in Northern England using the HiRoad™ pavement program. The temperature range at which the testing should be carried out should be sufficient to enable determination of the material properties over the entire temperature range that occurs within a certain layer. At low temperatures no additional testing needs to be carried out once the pseudo-glassy modulus has been reached (because the modulus does not change with decreasing temperature). It is interesting to note that, as one would expect the highest level of binder performance is required in the top 200mm of the pavement. In the example shown here (cf. Figure 16) the top asphaltic lift has to perform over a temperature range of approximately 50C whereas at a 200-mm depth the range decreases to approximately 25C.

Table 4. Influence of Inverse Shear Loss Compliance on Rut Rate and Strain Accumulation Rate of Mixes Using the Mixture Design and Aggregate Described in Reference 31

Acronym	Base Bitumen	Modifier	1/J", Pa @ 60C and 1 rad/sec	rr, mm/1000 cycle[1]	ϵ', in/in/1000 cycle[2]	Marshall Stability, lb (Newtons)
AR-4000	AR-4000	-----	245	0.11	----	2,340 ± 210
AR-1000	AR-1000	-----	75	0.32	0.074	1,970 ± 390
PMB A	AR-1000	SBS, 3%w	244	0.11	0.0042	2,040 ± 190
PMB C	AR-1000	SEBS, 2%w	156	0.15	----	----
PMB D	AR-1000	SEBS, 3%w	387	0.075	0.0027	1,730
PMB F	AR-1000	SBR, 2.5%w	153	0.16	----	
PMB H	AR-1000	EVA, 3%w	102	0.185	0.029	1,610 ± 50

[1]Measured in TRRL Wheel tracker as described in Reference 31.
[2]Measured using a uniaxial pneumatic system
Loading conditions: t_l=0.2 sec; t_{rel}=1.8 sec; p_o=0.3 psi; p_{max}=29psi
Waveform: Trapezoidal

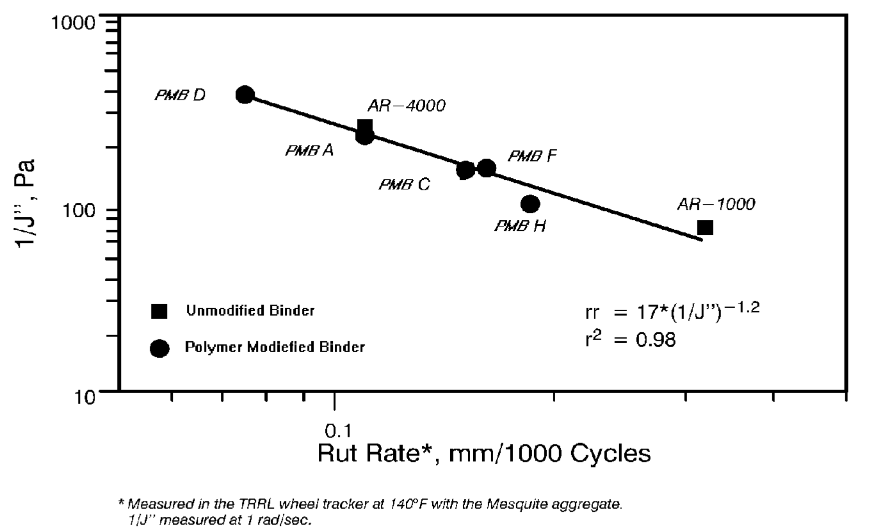

Figure 11 : Rut rate of various binders in the MES mixture versus inverse shear loss compliance measured at 1 rad/sec and 60C.

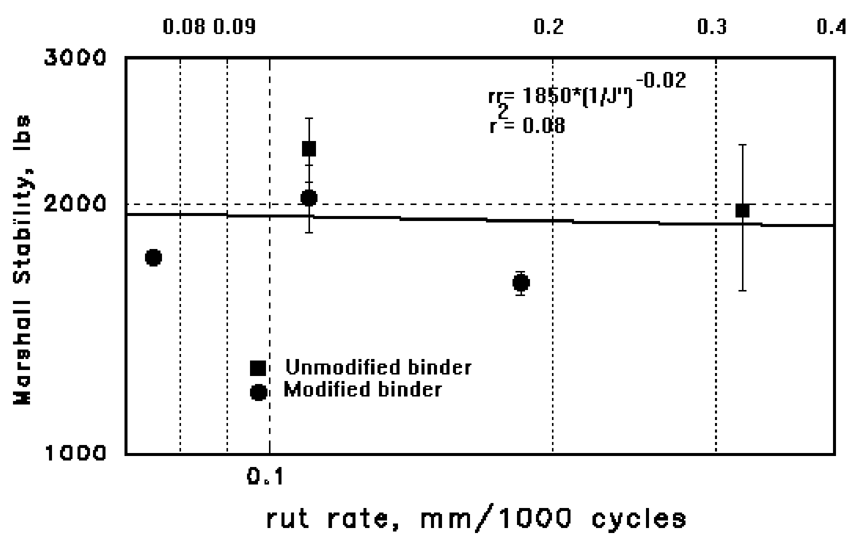

Figure 12 : Marshall stability of various binders in the MES mixture versus the rut rate at 60C.

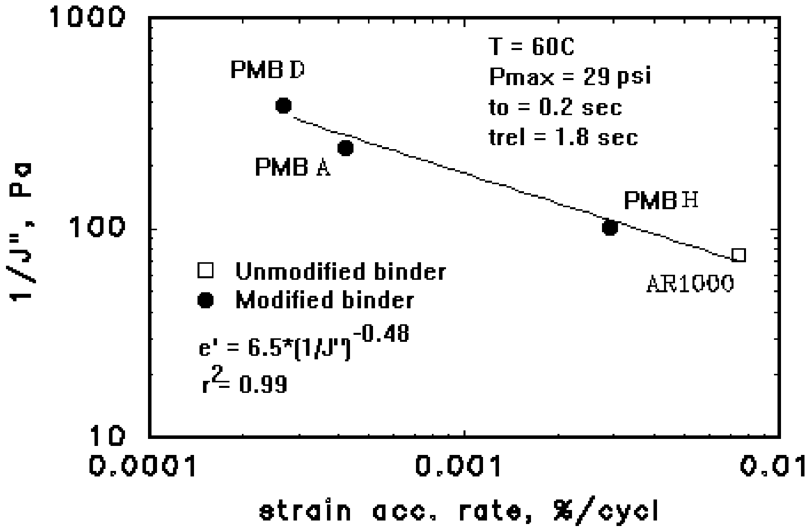

Figure 13 : Strain accumulation rate of various binders in the MES mixture versus inverse shear loss compliance.

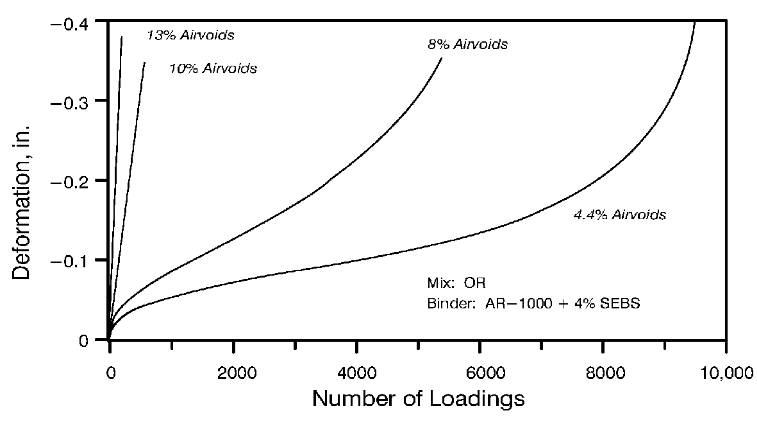

Figure 14 : Uniaxial repetitive creep experiments at 40C of the OR mixture with an CV AR-1000 containing 4%w SEBS at different air void levels.

Table 5 : Influence of air void level on rut resistance as measured in repetitive creep.

Air Voids, Percent	Mixture Design	ϵ', μin/in/ 1000 cycle	# cycles where ϵ_{acc}= 4.55 percent
4.4	OR[1]	10	----
8	OR	40	----
10	OR	4900	----
13	OR	11700	----
3.3	MES[2]	----	1642
3.4	MES	----	1071
3.9	MES	----	305
4.1	MES	----	859
4.6	MES	----	668
5.6	MES	----	216
5.6	MES	----	318
6.7	MES	----	156
7.7	MES	----	96
8.1	MES	----	101
8.9	MES	----	114

[1]Binder: California Valley AR-1000 with 4%w SEBS.
Equipment: Uniaxial servohydraulic.
Loading Conditions: t_o=0.365 sec; t_{rel}=1 sec; p_o=2psi; p_{max}=20psi
Waveform: Haversine
Temperature: 60C

[2]Binder: California Valley AR-4000
Equipment: Simple shear servohydraulic system
Loading Conditions: t_o=0.1 sec; t_{rel}=0.6 sec; p_o=0psi; p_{max}=10psi; constant height
Waveform: Haversine
Temperature: 60C

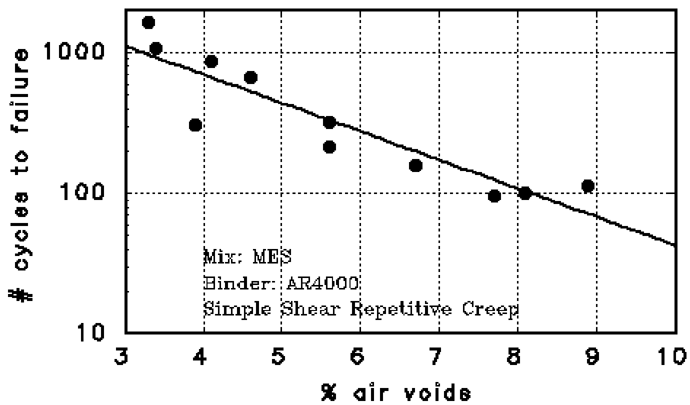

Figure 15 : Number of cycles to failure (accumulation of 4.55 percent strain) of a CV AR-4000 as a function of air void content in the MES mixture using the simple shear test device at 60C.

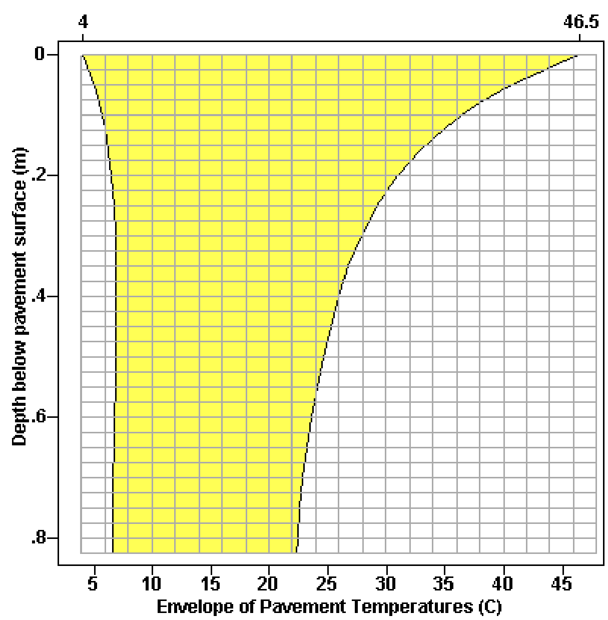

Figure 16 : Predicted pavement temperature envelope for Northern England using HiRoad™, case based upon 50 percent cold events and 95 percent hot events covered.

one should use a damage weighted temperature, $\bar{T}_r$.

$$\bar{T}_r = \frac{\int_{Jan}^{Dec}\int_0^{24h}\int_0^{J_2^{max}} \check{N}(t) \cdot D(T, J_2) \cdot T \, dJ_2 \, dt \, dm}{\int_{Jan}^{Dec}\int_0^{24h}\int_0^{J_2^{max}} \check{N}(t) \cdot D(T, J_2) \, dJ_2 \, dt \, dm} \qquad \textbf{(18)}$$

Where the normalized traffic density is defined in Equation 19.

$$\check{N}(t) = \frac{N(t)}{\int_0^{24h} N(t)\, dt} \qquad \textbf{(19)}$$

For standard testing it is obviously impossible to solve Equation 19 in each case. Appendix B gives an example case which clearly illustrates that with regard to rutting using the highest expected pavement temperature is a good, conservative approach. The reasons for this are:

- The highest portion of the traffic occurs during the day.
- The damage functions increase exponentially with temperature.
- Traffic induced strain hardening of the mixture reduces the rut susceptibility at lower temperatures.

However, pavement analysis programs should have the capability to determine a damage weighted temperature for the performance predictions.

Conclusions

The results clearly show the importance of using performance related mixture tests such as repetitive creep for predicting the performance of bituminous mixtures particularly when polymer modifiers are used. The reasons for this are:

- The test addresses the strain hardening of the mix.
- The test accounts for the ability of the material to exhibit elastic recoil.
- The test accounts for the time dependency of the material functions.
- The test can be used to determine the effect of dilatancy by measuring the Poisson's ratio of the material, i.e., the radial versus the axial strains.
- The test can be used to predict the response of the pavement using sophisticated viscoelastic models.

The repetitive creep experiments clearly demonstrate that the binder has a pronounced influence on the ability of the mixture to resist permanent deformation . For the modifier systems studied here it is found that highly elastic polymer modified binders with large values of 1/J" appear to perform the best. As expected, air voids have a pronounced effect on the rut resistance of the material. The improvements from using high performance binders can be completely lost when the in-place mixture has not been compacted to proper densification. With regard to the test conditions the work discussed above shows that it is not only important to run the testing at realistic loading times but also to carry out the test at the highest expected pavement temperature to prevent premature rutting of the mix.

Acknowledgements

The authors would like to thank J.B. Sousa, R.C. Elliott, J.S. Kendrick, D. Le and J.W.C. Moehring for their assistance with the materials testing work, M.J. Sharrock for his assistance with aspects of the analysis and Mr C.J. Heath for reviewing the text.

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Appendix A - Glossary

English Symbols

BOS	Boscan crude	
CV	California crude	
D	Damage function	
Ď	Normalized reference damage ($Ď_{Tref}$ = 1)	
E	Tensile modulus or modulus of Maxwell spring	Pa or psi
E_i	Modulus associated with i-th spring	
E_ω	Tensile modulus at a angular velocity ω	Pa or psi
E_ω'	Storage tensile modulus at a angular velocity ω	Pa or psi
E_ω''	Loss tensile modulus at a angular velocity ω	Pa or psi
ET	East Texas crude	
EVA	Ethylene-vinyl-acetate copolymer	
G	Shear modulus	Pa or psi
G_ω^*	Complex shear modulus at a angular velocity ω	Pa or psi
J_2	Second deviatoric strain invariant	
MES	Mesquite aggregate and mixture design	
m	Month	
NOTT	Nottingham aggregate and mix design.	
NYNJ	New York and New Jersey Port Authority aggregate and mixture design	
N	Traffic density	
Ň	Normalized traffic density	
n	Cycle	
OR	Oregon aggregate and mixture design	
PMB	Polymer modified binder	
p(t)	Pressure (force per unit area) at a time t	Pa or psi
p_o	Static pressure	Pa or psi
p_1	Pressure term in equation 3	Pa or psi
p_{max}	Maximum pressure applied to specimen	Pa or psi
rr	Rut rate	m/m/cycle
SBS	Styrene-butadiene-styrene block copolymer	
SEBS	Styrene-ethylene-butylene-styrene block copolymer	
SHRP	Strategic Highway Research Program	
T	Temperature	C or F
$\bar{T}_r$	Permanent deformation weighted average temp.	C or F
t	Time	sec
t_o	Loading period	sec
TRRL	Transportation Research Laboratory	
t_{rel}	Relaxation period	sec
V	Viscosity of dashpot	Pa.s

Greek Symbols

α	Exponent in equation 14	
β	Exponent in equation 15	
ϵ	Strain	m/m
ϵ_{acc}	Accumulated strain	m/m
ϵ'_{acc}	Strain accumulation rate [$d\epsilon_{acc}/dn$]	μm/m/cycle
κ	Constant in Equation 21	
λ	Relaxation time	sec
η	Viscosity	Pa.sec
τ	Heating rate	C/h
ν	Poisson's ratio	
ω	Angular velocity	rad/sec

Subscripts

acc	accumulated
i	i-th relaxation time or i-th Maxwell element
ω	at a angular velocity ω

Appendix B

The Effect of Temperature on the Damage Function

To solve Equation B-1 we assume that the traffic is evenly distributed over the day and that at night the traffic density is close to zero. Furthermore we assume an Arrhenius-like temperature dependency of the mix and assume that the pavement exhibits a linear increase of the temperature over the course of the test period.

$$\bar{T}_r = \frac{\int_{Jan}^{Dec}\int_0^{24h}\int_0^{J_2^{\max}} \check{N}(t) \cdot D(T, J_2) \cdot T \; dJ_2 \; dt \; dm}{\int_{Jan}^{Dec}\int_0^{24h}\int_0^{J_2^{\max}} \check{N}(t) \cdot D(T, J_2) \; dJ_2 \; dt \; dm} \tag{B-1}$$

$$\check{D}(T) = e^{(T-T_{ref})\bullet\kappa} \tag{B-2}$$

$$\check{D} = 1 \quad when \quad T = T_{ref} \tag{B-3}$$

$$T(t) = T_{ref} - \tau \bullet t \tag{B-4}$$

In this case Equation B-1 can be simplified.

If one sets $\kappa = 0.16$ and $\tau = 5C/h$ one obtains for a test day with a maximum pavement temperature of 47.5C a rutting damage weighted temperature of 41.25C. This emphasizes how heavily high temperatures need to be weighted.

$$\bar{T}_r = \frac{\int_{t_{\min}}^{t_{\max}} e^{(T-T_{ref})\bullet\kappa} \bullet T \; dt}{\int_{t_{\min}}^{t_{\max}} e^{(T-T_{ref})\bullet\kappa} \; dt} \tag{B-5}$$

$$= \frac{\int_{t_{\min}}^{t_{\max}} e^{t\bullet\tau\bullet\kappa} \; (t\bullet\tau + T_{ref}) \; dt}{\int_{t_{\min}}^{t_{\max}} e^{t\bullet\tau\bullet\kappa} \; dt} \tag{B-6}$$

$$= \frac{\frac{T_{ref}}{\kappa \bullet \tau} \bullet [e^{t\bullet\tau\bullet\kappa}]\Big|_{t_{\min}}^{t_{\max}} + \frac{1}{\kappa^2\bullet\tau} \bullet [e^{t\bullet\tau\bullet\kappa} \bullet (\kappa\bullet\tau - 1)]\Big|_{t_{\min}}^{t_{\max}}}{\frac{1}{\tau \bullet \kappa} [e^{t\bullet\tau\bullet\kappa}]\Big|_{t_{\min}}^{t_{\max}}} \tag{B-7}$$

$$= T_{ref} + [\tau\bullet t - \frac{1}{\kappa}] = T_{\max} - \frac{1}{\kappa} \tag{B-8}$$

Discussions

MR. RICHARD DAVIS: (Prepared Discussion) This is a fine paper on a very timely subject - the permanent deformation of hot mix asphalt pavement(HMA). Permanent deformation or rutting occurs in two major but very different ways. One is through structural weakness far down in the pavement or in the subgrade and the other is through plastic deformation in the top four or five inches of the HMA. The first type is due mostly to load while the second is due to stress or tire pressure. It is interesting that most state highway departments have legal limits for load, but few if any have any restriction on tire pressure. My discussion like the paper is addressed to the second type of rutting.

Figure 1 of this paper recalls a very similar figure from a paper that I presented almost nine years ago (1), Figure A of this discussion. This figure shows increasing resistance to deformation or work hardening as energy is supplied to the mix as compaction. Figure 1 of the paper also shows a linear regime of deformation and then what seems to be inevitable catastrophic failure. But this catastrophic failure is not inevitable as shown in Figure B of this discussion which is also from my paper (1). All that is necessary is to increase the bearing capacity or the elastic limit to the point where they are greater than the stresses being applied to the pavement. The major point of my discussion is that it is unnecessary for the mix to enter the catastrophic phase and that all designers of HMA should learn how to avoid this region.

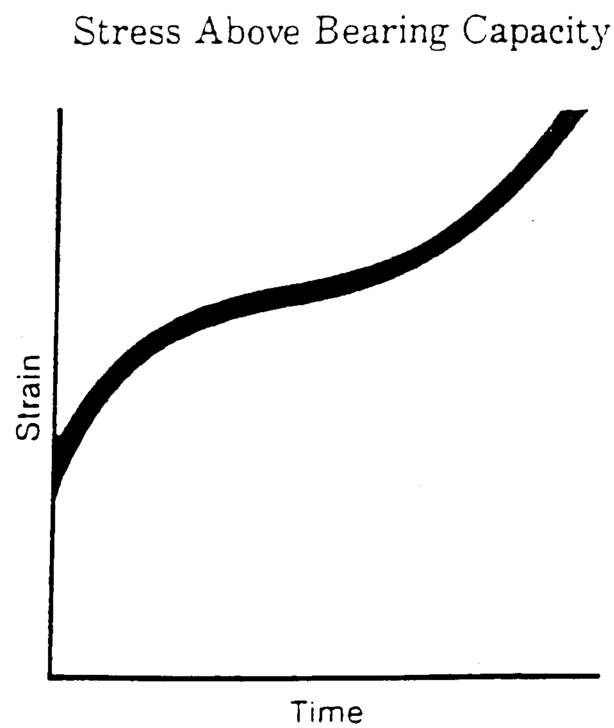

Figure A.

Many people are losing confidence in the ability of HMA to carry heavy truck traffic at higher pavement temperatures and this threatens not only the use of HMA for heavy truck traffic at these temperatures but also for many other conditions and uses. Many designers of HMA are trying to extend the linear regime in Figure 1 of the paper by not fully compacting the pavement, leaving it with 10 to 12 percent air voids. This leaves the pavement

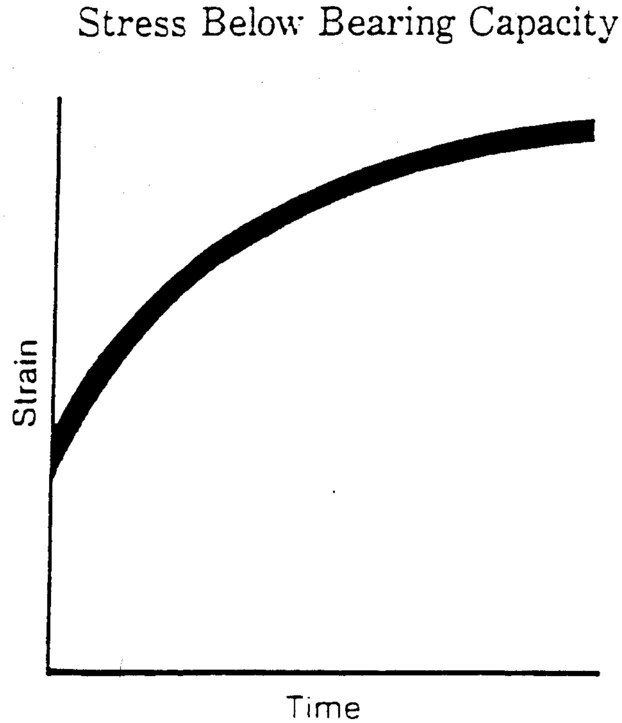

Figure B

open to damage by moisture, oxidation, and ravelling which greatly reduces its durability and can be almost as much a threat to the future of HMA as rutting. Your discusser considers rutting to be the greatest threat to the use of HMA at the present time but also believes that both these threats can be removed through proper design which can result in increased bearing capacity, low air voids, and greater durability. Those people who live in regions where the pavement temperatures do not reach the 130F to 150F range may find it difficult to get very excited about rutting, but the loss of confidence in the hotter regions can spread to the cooler regions even though there may not be a sound basis for it.

At this point a little history might help to clarify the problem of rutting. The inability of HMA to resist permanent deformation at high temperatures has always been its "Achilles Heel". The early rock asphalt mixtures were composed of fine aggregate and it was noted early that the adjustment of asphalt content to precise levels was required if rutting was to be reduced. It was because of deformation problems with sheet asphalt pavements that F. J. Warren patented an asphalt pavement with stone in it. Warren felt that his patent covered all pavements in which any of the aggregate was retained on a 10 mesh sieve. The Warren Brothers Company brought suit in the Federal Court in Topeka, Kansas against a number of parties in that area for infringing its patent. But after considering the high cost of such suits, George C. Warren decided to accept an agreement by the defendants to restrict their use of aggregates to those that would pass a 1/2-in. (13-mm) sieve. This was the origin of the term "Topeka Mix" from which evolved conventional HMA .

George C. Warren explained to the Board of Directors of the Warren Brothers Company that he really had not given up anything because a mixture with stone no larger than a 1/2 in. (13mm) could never support the heavy traffic for which the Warren pavement was designed. Before breaking into laughter the reader should realize that heavy traffic of that day consisted of heavy wagons with narrow steel rimmed wheels and trucks with solid rubber tires. Little did Mr. Warren foresee the balloon or pneumatic truck tire which was developed in the late 1920s and which made the use of HMA feasible under truck traffic.

One of the stated purposes of the balloon tire was to make it possible for a truck to drive over ordinary asphalt pavement in the hottest weather without noticeable deformation. After some research it was decided that 60 psi (0.41 MPa) was the highest pressure that could be tolerated at high temperatures. We see that at the time of the

AASHO Test Road a tire pressure of 70 psi (0.48 MPa) was adopted. Had pressures of 150 psi (1.03 MPa) been used the results of the test road for asphalt pavement would have been completely different.

In the middle 1970s this discusser began to see pavements fail in a new way. This is in the period that people began to speak of the "goodies" being removed from the asphalt. Knowing enough about the manufacture of asphalt to know that the "goodies" had never been there in the first place, other explanations were sought. There were those who said these failures were due to "stripping", but this was hard to believe since they most often occurred during long hot dry spells. Also why should a pavement which had shown no problems with moisture for twenty years suddenly begin to fail. In searching for another explanation it was learned that many truckers had increased their tire pressures to the 125 to 150 psi (0.86 to 1.03 MPa) range. Knowing of the results of the tests in the 1920s, it was immediately recognized as a manifestation of HMA's "Achilles Heel" and a most serious problem unless tire pressures could be lowered. It is the discusser's belief that the rutting problem would disappear if tire pressures were lowered to 60 psi (0.41 MPa) today.

After determining that there was little chance of lowering tire pressures, the management of Koppers Co. was approached (for whom your discusser worked at the time) and it was pointed out that while the rutting problem was a great challenge it was also a great opportunity. The easy solution was to put a small amount of something in the mix that would increase its elastic limit to the point that the higher tire pressures could be borne without permanent deformation. Koppers, having marketed a wide variety of polymers, instituted a study in this area. A formulation was found that was technologically successful in coping with very heavy truck traffic even in the hottest weather but a cost analysis convinced the discusser that it was too expensive to ever be commercially competitive.

Since the problem still existed and Koppers owned a large number of hot plants, a limited study was undertaken to find another way of increasing the bearing capacity or elastic limit of hot mix asphalt. The paper (1) summarizes the results of this study. In order to make some of these concepts more understandable, the reader is referred to Figure C. This figure shows a point load being applied to an elastic half-space. A point load having no area applies an infinite stress at the point of loading which would plasticize any solid. The plasticized volume would be determined by the yield strength or elastic limit of the solid in the same way that it is in the pavement. In the case of the pavement the plasticized volume is determined by the level of stress being applied to the pavement by truck tires and the elastic limit of the pavement. If the applied stress is below the elastic limit of the pavement at its highest temperature the pavement is not plasticized and there is no rutting.

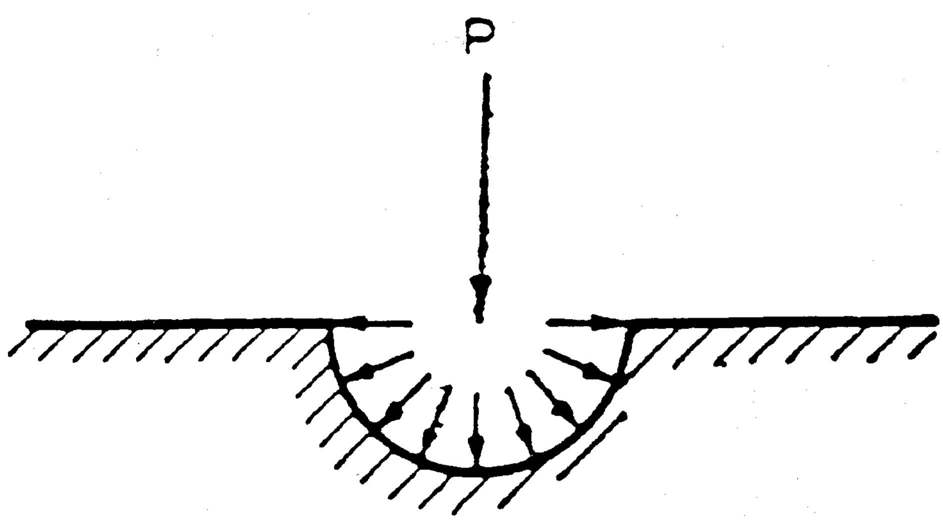

Figure C.

Figure D is the first figure from my college textbook on strength of materials. It shows the relation between stress and strain for a solid. The straight line from the origin to the proportional limit depicts the region of stress where the deformation is elastic and fully recoverable. At stresses in excess of the proportional limit the solid flows and the deformation is not fully recoverable. This is what happens in our rutting pavements. An obvious solution to this problem is to increase the stress at which flow begins. The elastic limit of an asphalt pavement can be increased through increasing the size of the stone. It would make this discussion far too long to explain the mechanism of this increase here, but a full explanation can be found in my book (2). This condition is shown in Figure B of this discussion. Figure 1 of the paper which prompted this discussion shows that the applied stresses exceed the elastic limit of the pavement and the pavement is failing at various rates throughout the linear and catastrophic regimes. Most engineers would prefer to keep their designs in the linear elastic region if they could and this is possible through an increase in the elastic limit of the HMA pavement.

Figure E shows how the vertical stress decreases with depth on the axis of a semi-infinite elastic medium due to a circular uniform stress. It is clear that the stress decreases rapidly with depth. This means that material with a high elastic limit is only required at or near the surface of the pavement where the stress is highest. Your discusser has been told many times that

large stone deep down in the base of the pavement has not reduced rutting of the second kind. This is not surprising since it can not increase the elastic limit at the surface of the pavement where the stresses are highest. Material with an elastic limit high enough to cope with the high stresses at the surface must be where these high stresses are in order to reduce rutting. Large stone can be used as base material for cost reduction where it is plentiful and cheap, and/or because of reduced binder requirements with large stone. It also increases the resistance to stress wherever it is used, but it does not help with the rutting problem of the second kind when it is deep in the pavement and the high stresses are at the top of the pavement.

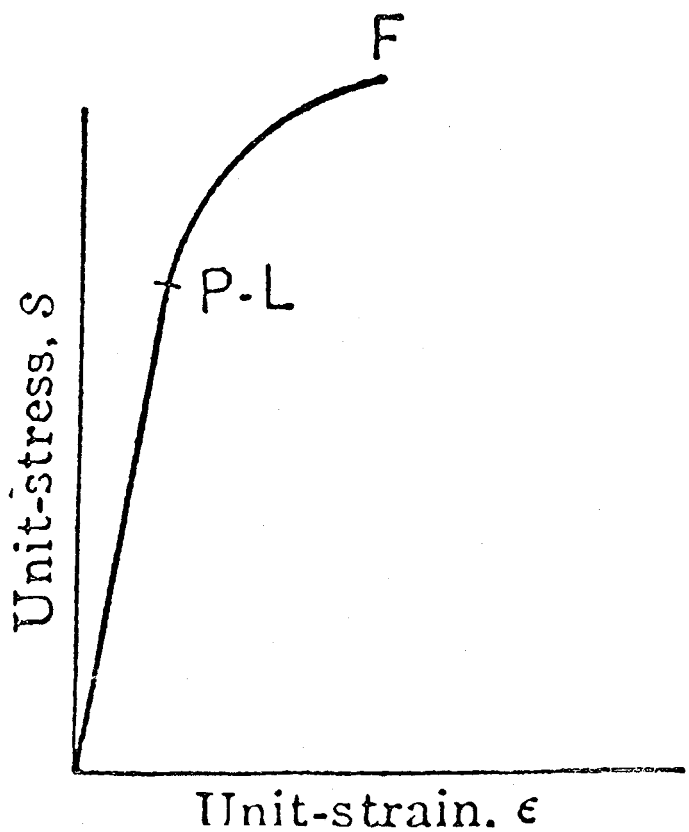

Figure D.

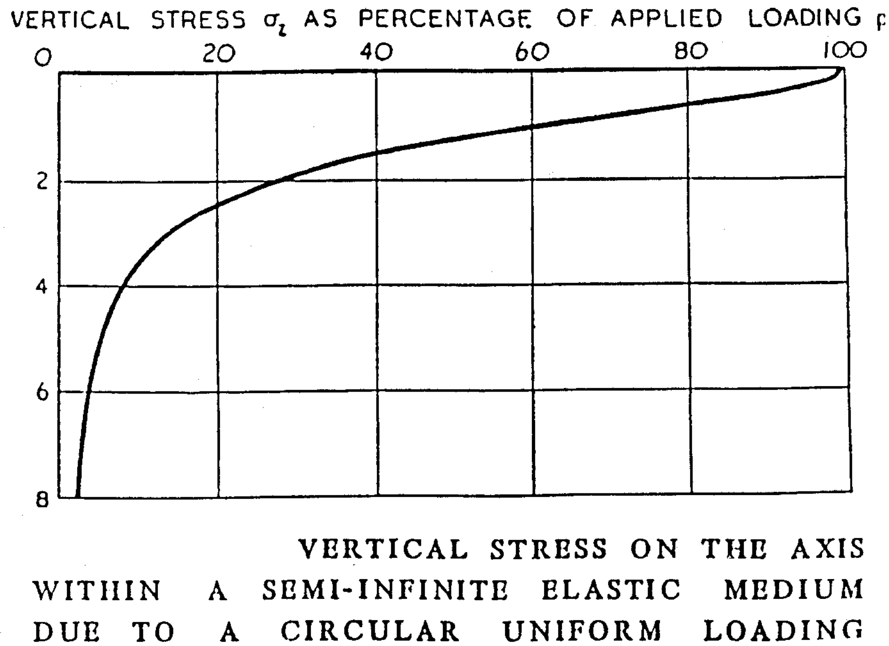

Figure E.

Figure F shows how shearing stress both increases and decreases with depth on the axis of a semi-infinite elastic medium due to a uniform circular stress. The highest shear stresses occur at the surface at the edge of the loaded area. Again it is important that the hot mix asphalt at the surface have the greatest resistance to stress. Large stone can furnish the needed resistance.

After recommending large stone as an answer to the rutting problem, it would seem wrong to close this critique without some explanation of what is meant by the term large stone. In an effort to keep this discussion to a reasonable length only a short outline of the role of large stone will be given here. Those who desire more information are referred to the paper (1) and if they want considerably more information on both the analysis and the mechanism of large stone mixtures, the book (2) is recommended.

The term large stone as used here refers to stone that is large in relation to the layer under design. The layer thickness must be great enough to reduce stresses to the point where the underlying pavement or subgrade can support them. Figures E and F show that stress is reduced rapidly with depth in an elastic medium. This means that a three inch layer of dense graded mixture with nominally 1 1/2 in. (39mm) top size stone would, in most instances, reduce the transmitted stresses to the point that a conventional HMA would be able to cope with them. So where a greater thickness is necessary for structural reasons, conventional HMA usually would provide the necessary resistance to stress underneath a large stone surface.

The elastic limit of an asphalt pavement is increased by many factors, such as the volume concentration of the aggregate, the size of the largest stone, the tensile strength of the binder, the frictional properties and shape of the particles of the aggregate, and the stiffness of the filler-binder paste. The importance of the volume concentration of aggregate and the frictional properties and shape of the particles are widely appreciated and practiced. The effect of the stiffness of the filler-binder paste can be significant, but this discusser is hesitant to propose it as a solution to the rutting problem, because it would present so many difficult plant control and laydown problems. Increasing the tensile strength of the binder was the primary research objective of the discusser for many years. It was recognized that it could be an excellent answer to many HMA problems. This research resulted in technologically successful binders due to both processing changes in the manufacture of the asphalt and through alteration by the addition of various substances, but none was cost or commercially successful. This is why increased stone size was

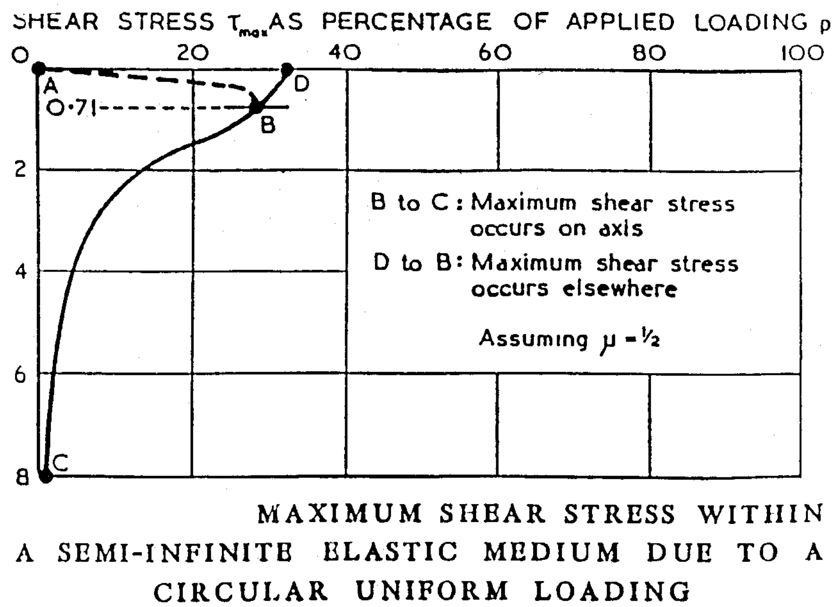

Figure F.

recommended nine years ago as a solution to the rutting problem. Even though there are parts of this country where local supplies of large stone are limited or non-existent, most of the country has large stone at competitive prices. For those places that don't, a careful analysis of costs for alternative methods is in order. Most of the country with little change in present practice should find it relatively easy to solve their rutting problems with a change to larger stone in their surface mixes.

Some contractors who have had little experience in handling large stone are concerned that it will be difficult to handle and therefore, would reduce productivity. In most instances they find, after a little experience, that the use of large stone is not as difficult as they had feared and in some instances their productivity is actually increased. Productivity is very important since the cost and competitiveness of the HMA is strongly related to it. Large stone pavements do require increased vigilance against segregation, but this is not really new since all graded aggregates require such vigilance. For those for which segregation is too big a problem, one size, large size stone can be used. One size stone can not segregate, and the large stone still increases the bearing capacity of the pavement. The top size of the stone may have to be increased above that for dense graded stone to achieve the desired level of bearing capacity and the one size stone mix may be more subject to oxidation and stripping problems, but it also offers a good solution to rutting problems.

HMA with increased resistance to deformation due to stone structure is less temperature susceptible than conventional HMA. This is one reason that it retains its resistance at high temperatures. This means that higher stress must be applied to properly compact the pavement. Not more rollers, not more rolling time, not higher temperature for the mix, but higher stress. The rolling can be accomplished in the same length of time with the same amount of rolling if the proper stress level is used. Considerably higher stress is required to compact dense large stone mixes to a surface rugosity of fine surface mixtures and this is not possible with one size, large stone mixtures, but the desired surface rugosity can be obtained as it was under the Warren patents by a thin layer of fine aggregate mixture.

In summary, the following conclusions are offered.

- Rutting threatens the future use of HMA for heavy truck traffic and by extension for other applications.
- The rutting problem is predictable and a solution to the problem is indicated by an examination of asphalt pavement history.

- Large stone in HMA is recommended as the simplest and most cost effective method of eliminating rutting.
- Higher stress rolling is required to compact large stone mixtures.

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MR. ROBERT DUNNING: As we have been saying, Dick Davis is right. To corroborate what Dick has said, in Las Vegas my son was a project manager on a project where they had rutting during construction. By changing the gradation he was able to stop rutting. Clark County has changed their specifications so that coarse gradations can be used. If I am allowed to have control of the gradation, I can stop rutting. I think the direction you are going is excellent, especially I think your measure of dilatancy is very important. However, I am a bit offended with the implication that over the last 20 years I have not known how to do mix designs. Because I think I do. I can use a Marshall and use Hveem equipment to design a mix that works. We have many mixes we have designed that have worked. We can get good performance mixes and can make sure the film thickness is correct. I do think the people in this room know how to make mixes if we are allowed to have proper control over the materials.

DR. MARK BOULDIN: We all probably have a good intuitive feel of how to make mixes. I'm sure an old timer like you can probably dream up some pretty good mixes. There a lot of people here who know intuitively the right things to do to make a proper mix. However, what we need is to go one step beyond that. We need to be able to quantify performance. This is especially important when "peddlers" come to your door telling you they have a material "a" that we do certain things. For example, in California we are already seeing that for certain ground tire rubbers the thickness design is being cut down to 50 percent. We need performance measures and should not be doing this arbitrarily. SHRP provides us the tools to get away from the gut feeling and to make road building a true engineering science.

MR. DUNNING: I'm not disagreeing with that. I'm not disagreeing with the direction you are going. In fact, I am very excited about the way you are going, especially with the constant height shear test. I did think you were a little strong on those two methods because we can use those if that is all we have to use. If I have control of the aggregate I don't have to worry about rutting any more because I can stop it.

I have two other things. First, when you get into the polymers you do have the first and second normal stress differences. I would like you to mention how that might have an impact on the dilatancy. The second, there is one thing that is missed in most design systems. There is a bias between the properties in the mix design and the properties of the field-produced materials. This is especially critical as we establish new specifications. We must find a way to handle that bias otherwise the contractors are going to be penalized 50 to 100 percent on jobs.

DR. BOULDIN: I would like to thank you for the last two questions. Bob, you are absolutely correct. Polymer modified asphalts have, what we call, normal stresses. If you would do, for example, a simple shear test in the DSR which is now part of the SHRP specifications you would see that there is not only a torque in the shear direction but we are actually having a normal stress perpendicular to that which would be trying to push the platens apart. It can be a very significant effect because they are elastic. Jorge Sousa did some work on a few of these mixes. We did not publish it here, but on those Mesquite mixes he did look at how that impacted their performance. Our conclusion was, although it was preliminary data, the higher elastic and higher than normal stresses were in the binder the higher the normal stresses were in the mix. What are normal stresses in mix? That is what we call dilation. And that is essentially the confining pressure that the material can build up. The higher the confining pressure of the material, the better the rut resistance of the mix. So you are very right. With a simple shear tester we have a unique tool in that we not only measure shear resistance but we also can measure the perpendicular force. By keeping it a constant height we make it more rut resistant. That is why I think it is so important that we use a simple shear tester versus uniaxial.

To your question with regard to field quality control, I agree with you 100 percent. In fact Jorge and I are working on that. I think it is imperative that all the testing we have been speaking about today needs to be brought to the field. It does not help us just to go out and make beautiful mix designs that we cannot verify in the field. If I just go out in the field and then fall back on the Marshall we have done ourselves no favor. What we need to do is be able to verify mix performance, using, for example, simple shear testing, fatigue testing, indirect tension, TSRST, etc., on field specimens.

MR. KAI TAM: Could you comment on how you determined the rate of rutting based on your experience? People working on the wheel tracking machine recognize that there are three phases in the rutting process. The first one is the consolidation, the second is the reorientation of particles, and the third is the plastic flow or creep rutting. Could you comment on that?

DR. BOULDIN: What we did is a very simple thing. If you look at the curves Dick Davis showed, there was actually a fourth range that we call the catastrophic range. In this range the mix undergoes particle movement. Unfortunately most people only take two points where they think it looks linear. To get the slope we took the raw data, applied a smoothing function to get rid of any spikes. Then we differentiated to obtain the slope. If you take the first differential you get the slope , i.e. strain accumulation rate, which one can plot versus the number of cycles. Where that slope starts tapering off you get a constant slope which is our definition of linear range. If you take the second derivative, that one should be 0 where the slope is constant. So if you look in the paper I have a figure where you can see that the slope starts coming down to a constant value. It starts very high in the initial and then comes down. Where the second differential passes through is where we took the rut rates. The same applies to uniaxial or simple shear testing.

MR. GEOFF ROWE: When Dick was talking about elastic versus viscoelastic, one of the principle reasons we went for viscoelastic was so that we could model the elastic recovery. We considered that was the real behavior which we couldn't model with either the elastic or viscous methods that we had. In terms of Marshall we see this ranking of materials as a big problem. In the UK they typically have a Marshall specification which is lower for modified materials than it is for conventional materials. So you have a situation with two specifications. This would occur whether you are getting a Marshall property just from any compaction method, Marshall, Hveem or gyratory. We really need to do a torture test which ranks the material more accurately. Something like simple shear or wheel tracking (which is probably going to be the way the UK goes) in terms of assessing the mixture after you have done the design. What needs to be in all levels of traffic to make sure that you have the right level of stability with the unaged material. I wanted to emphasize again that you do need to test the samples from the field. I think an emphasis needs to be placed on getting the cores from the field so they are compacted in the same state to that which traffic is going to subject them. You actually are comparing like with like when field cores are tested. That is a point I wanted to emphasize.

MR. JOHN MCRAE: We can build rut free pavements.