UNIAXIAL EXPERIMENTS ON IDEALIZED ASPHALT MIXES

By V. S. Deshpande and D. Cebon

ABSTRACT: This paper contains details of uniaxial compression experiments on various idealized asphalt mixes with volume fractions of aggregate ranging from 40 to 85%. The compression experiments are performed over a wide range of temperatures, stresses, and strain rates. The steady-state axial behavior of the mixes has the same form as that of bitumen with the aggregate acting as a stiffener. However, the mixes with high volume fractions of aggregate dilate under a uniaxial compressive stress. The experimental results are compared with theoretical predictions of a modified version of Hashin’s composite sphere model. Good agreement is observed.

INTRODUCTION

The purpose of a pavement is to carry traffic safely, conveniently, and economically throughout its design life. There are several reasons why a pavement may cease to fulfill its function. The most common forms of structural failure are permanent deformation (rutting) and fatigue (cracking). The development of a model that explains the rutting of flexible pavements relies, at least in part, on understanding the constitutive behavior of asphalt or bituminous mixes that are used in the upper layers. Bituminous mixes are complex multiphase materials consisting of a gradation of aggregate, air voids, and bitumen. Constitutive modeling of the deformation behavior of bituminous mixes using continuum mechanics has been the focus of research among paving technologists for over 50 years. In the past, most of these approaches have been empirical. This is partly because of the complexity of the problem and partly because of limited understanding of the behavior of heterogeneous materials. Some of these models are briefly described here. [For a more detailed review, see Deshpande (1997).]

Initial attempts at modeling asphalt treated material as a soil (Nijboer 1948; Goetz and Chen 1950). These approaches assumed rate-independent plastic behavior for the mixes. However, in the 1950s a viscoelastic description of the behavior of bitumen became popular. The most common approach is that by van der Poel (1954) who introduced the use of “stiffness” to describe the mechanical behavior of pure bitumens as a function of temperature and loading time. This research resulted in the well known van der Poel nomograph (1954). van der Poel (1955) also extended the stiffness concept to experimentally map the dynamic behavior of various bituminous mixes for small strains where the linear behavior is dominant. The disadvantage of this approach is that the distinction between the elastic and inelastic part of the total deformation is ambiguous. In his extension of the stiffness concept, van der Poel (1955) assumed that the stiffness of the mix is a function only of the stiffness of the bitumen and the volume fraction of the aggregate. Subsequently, Heukelom and Herrin (1964) proposed the following relationship for predicting the stiffness of a bituminous mix:

\[ S_{\text{mix}} = S_{\text{bit}} \left(1 + \frac{2.5}{p} \frac{C_v}{1 - C_v} \right)^{\gamma} \]  

where \( S_{\text{mix}} \) and \( S_{\text{bit}} \) = stiffnesses of the mix and bitumen, respectively; and \( C_v \) = volume concentration of the aggregate defined by

\[ C_v = \frac{\text{Volume of aggregates}}{\text{Volume of (aggregates + bitumen)}} \]  

\[ p = 0.83 \log \left[ \frac{4 \times 10^6}{S_{\text{bit}}(\text{Pa})} \right] \]  

The above equations were derived from empirical fits to data from static and dynamic tests on well-compacted mixes having about 3% air voids and \( C_v \) values ranging from 0.7 to 0.9. More recently, Brown et al. (1992) modified the above equations to

\[ S_{\text{mix}} = S_{\text{bit}} \left[1 + \frac{257.5 - 2.5VMA}{p(VMA - 3)} \right]^{\gamma} \]  

where \( VMA = \) percentage of voids in mixed aggregate; and \( p = \) same function of \( S_{\text{bit}} \) [(3)]. Eq. (4) is valid for \( VMA \) values from 12 to 30% and \( S_{\text{bit}} \geq 5 \text{ MPa} \). For high values of bitumen stiffness \( (S_{\text{bit}} \approx 1 \text{ GPa}) \), the bitumen behaves as an elastic solid, and the stiffness ratio is a measure of the stiffening effect due to rigid inclusions in an elastic matrix. However, for lower values of \( S_{\text{bit}} \), the stiffness ratio becomes a function of the elastic, viscoelastic, and viscous responses of the material. Other attempts at the viscoelastic modeling of asphalt include those of Pagen (1965, 1968), Monosmith and Secor (1962), Monosmith et al. (1966), and Nair et al. (1972).

Huang (1967) showed that there were certain important limitations in using linear viscoelastic models to describe the mix behavior. In particular, he observed that dilatation occurred under applied axisymmetric compressive loading and that the deformation behavior depended on the confining pressure as well as the deviatoric stress. Because the discrepancies between linear viscoelastic models and experimental observations became more significant with increasing strains, he concluded that elastic and viscoelastic theories could only be used under transient loads of short durations and are not applicable for static wheel loads.

At large strains, the response of bituminous mixes is nonlinear and cannot be adequately modeled by a linear viscoelastic approximation (Monosmith and Secor 1962; Pagen 1965; Monosmith et al. 1966; Pagen 1968). Lai and Anderson (1973) proposed a generalized nonlinear Kelvin model consisting of a nonlinear dashpot connected in series with a Kelvin chain. Vakili (1983) used a nonlinear viscoelastic model with a multiple integral representation for modeling tests with multiple load levels. However, due to their complexity, restriction to uniaxial behavior, and the number of constants required, these nonlinear viscoelastic models have not been applied to model the deformation behavior in realistic pavement structures.

The microstructure of bituminous mixes can be used to understand their properties, qualitatively as well as quantitatively. Notable attempts include the three-phase composite sphere
model (van der Poel 1958), the bitumen film creep model (Hills 1973), and a discrete element model (Rothenburg et al. 1992).

The deformation behavior of bituminous mixes is qualitatively well understood through the various experimental studies performed over the last 50 years. A number of important conclusions regarding the deformation properties of bituminous mixes can be drawn from the available literature (Cheung 1995; Deshpande 1997):

- Although bituminous mixes show linear viscoelastic behavior at small strains, they are, in general, nonlinear materials. The nonlinear behavior becomes more prominent at large strains.
- The deformation of bituminous mixes is loading rate and temperature dependent. This dependence is the same as that of pure bitumen.
- The deformation behavior is a function of the hydrostatic and deviatoric stress states.
- The mixes dilate even under compressive stresses.

Numerous empirical models approximating the behavior of the mixes under specific loading conditions are available. However, no single model has been developed that is capable of explaining all of the above observations in a consistent manner.

As a first step toward understanding the deformation mechanics of real mixes, the behavior of idealized mixes (with less complex microstructures) is investigated in this paper. Details of uniaxial compression experiments performed on various idealized mixes over a range of stresses, strain rates, and temperatures are presented. Preliminary conclusions about the possible mechanisms controlling the deformation of these mixes are presented. The experimental results also are compared with the predictions of a model in which the mixes are modeled as composites of a gradation of aggregate and bitumen.

**EXPERIMENTAL PROCEDURE**

The aim of this study was to understand the deformation behavior of idealized bituminous mixes. Simple compression tests were performed on these mixes. Tensile tests were not performed because other mechanisms (e.g., void nucleation and growth) become dominant.

**Mix Specification**

**Pure Bitumen**

Cheung and Cebon (1997) developed mathematical models for the deformation behavior of a 50 pen bitumen over a wide range of temperatures and strain rates. For temperatures above approximately −10°C, they found that the steady-state deformation behavior of bitumen can be described by the modified Cross model (Cheung and Cebon 1997)

\[
\frac{\sigma}{\sigma_0} = \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \left( 1 + \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^n
\]

where \( n, \sigma_0, \) and \( \dot{\varepsilon}_0 \) = material constants for bitumen. This model becomes linear (Newtonian) creep at low strain rates and power-law creep at high strain rates. They also found that the temperature dependence of bitumen was activation energy controlled (Arrhenius equation) at low temperatures \((T < 20°C)\) for their bitumen and free volume controlled (WLF equation) at higher temperatures \((T > 20°C)\)

\[
\frac{\dot{\varepsilon}_0}{\dot{\varepsilon}_0} = \exp \left( \frac{Q_c}{RT} \right), \quad T \leq T_d
\]

\[
\log \left( \frac{\dot{\varepsilon}_0}{\dot{\varepsilon}_0} \right) = -\frac{a_1(T - T_d)}{a_2 + (T - T_d)}, \quad T \geq T_d
\]

The values of the material constants \( \dot{\varepsilon}_0, \dot{\varepsilon}_0, Q_c, a_1, a_2, T_d, \sigma_0, \) and \( n \) may be found in Cheung and Cebon (1997).

The steady-state stress versus strain rate relationship for the bitumen tested by Cheung and Cebon is shown in Fig. 1. It can be seen in Fig. 1 that at 20°C the behavior of bitumen can be approximated as linear viscous below \( \sigma = 1 \times 10^5 \) Pa and power-law viscous with \( n = 2.3 \) above \( \sigma = 1 \times 10^7 \) Pa. After consideration of the behavior in shear, Cheung and Cebon (1997) concluded that in the power-law creep region, the constitutive behavior of bitumen can be approximated by

\[
\frac{\dot{\varepsilon}_0}{\dot{\varepsilon}_0} = \frac{3}{2} \left( \frac{\sigma}{\sigma_0} \right)^{n-1} \Sigma_i \sigma_i
\]
where \( \dot{\varepsilon}_0 \) = reference strain rate; \( \sigma_0 \) = reference stress; \( \sigma_e \) = von Mises equivalent stress; \( \Sigma_\varepsilon \) = deviatoric stress tensor; \( \varepsilon_{ij} \) = strain rate tensor; and \( n \) = stress sensitivity (power-law index) = \( 1/(1 - \eta) \). The bitumen tested by Cheung and Cebon was used to manufacture the idealized mixes described below.

### Idealized Mixes

Five main types of mixes consisting of bitumen mixed with various volume fractions of different kinds of particulate inclusions were tested. These are listed in Table 1. Mixes D and E were low volume fraction dispersions of sand in bitumen, whereas mixes A, B, and C were fully dense mixtures [i.e., maximum aggregate volume fraction for that particular aggregate gradation; maximum random packing densities for various particle size distributions are given in German (1989)].

For a regular packing of single-sized spheres, the hexagonal or cubic close packed arrangement gives a maximum volume fraction of 74%. On the other hand the densest disordered (random) packing of single-sized spheres is known to be about 64% as for mix A (Finney 1970). The reasons for choosing these particular mixes, the mix preparation methodology, and testing procedures are detailed in the following sections.

### Mix Preparation

Previous studies have shown that the specimen preparation technique has a significant effect on all types of laboratory testing of bituminous mixes. Hills (1973) documented how the stiffness of mixes varied with different compaction techniques. Wallace and Monismith (1980) reported that a uniform density distribution in a specimen was essential to any testing program. Harvey et al. (1994) concluded that surface condition (cut as-molded) also significantly affects both air void measurements and mechanical behavior.

#### Equipment and Procedure for Specimens of Types A and B

Various compaction procedures are used in the preparation of laboratory asphalt specimens. Fwa et al. (1993) evaluated these techniques based on the density profile of the specimens. The density variation in cylindrical specimens of size 102 × 200 mm was measured continuously along each specimen using a twin-probe nuclear density gauge [see Tan and Fwa (1991) for further details]. The main conclusion from their study was that single plunger compression in multiple layers, or double plunger compression in a single layer, gave satisfactory homogenous test specimens. Consequently, it was decided to make cylindrical specimens in multiple layers using the single plunger compression technique.

A cylindrical mold was manufactured and used to cast and compact the specimens. The diameter of the cylinder was chosen so that it was at least 10 times greater than the size of the largest aggregate particles. The ratio of the height to the diameter of the specimen was chosen to be around 2 [the same as triaxial test specimens tested by Fwa et al. (1993)]. A lubricant consisting of a mixture of natural soap and glycerine was applied to the inside surface of the mold to prevent sticking of the bituminous mixture. However, the bottom surface of the plunger could not be lubricated because traces of the lubricant would be left on each compacted layer of the specimen. Consequently, the bottom surface of the plunger was coated with a thin layer of solid silicone rubber, a substance known not to bond with bitumen.

The following procedure was used to cast the specimens. A mixture consisting of the correct amounts of bitumen and aggregate was heated to melt the bitumen. The mixture was then well stirred, transferred into the mold, and compacted in a mechanical press. After the mix had cooled down to room temperature, the specimen was slowly pushed out of the bottom of the mold and stored at a subzero temperature.

The main requirements of a test specimen are (1) it should be homogeneous; and (2) the air void content should be well controlled. To decide on the optimal specimen making technique, a convergence study was conducted. The aggregate used was subspherical sand of size 300–600 µm. [Subspherical is a qualitative term used to classify sands with a low angularity (Davis 1950).] The parameters varied were the compaction technique (load, time, and type of compaction) and the mix recipe. The parameters measured to assess the suitability of the preparation technique were the air void content and density variation along the length of the specimen. Air void content was estimated by measuring the density of the specimen and comparing it to the theoretical maximum possible density for that particular mix recipe. Homogeneity was evaluated by sectioning the specimen into three equal parts along its height and comparing the density of the three parts.

It was found that as the volume fraction of sand was decreased below 64% the density variation across the height of the specimen increased (i.e., the specimen became inhomogeneous). Hammer compaction, which involved dropping a 5-kg weight through 1.5 m, and single plunger compression (in a mechanical press) with a pressure of 200 kPa resulted in a high void content. Compression in a single layer gave inhomogeneous specimens. Furthermore, it was observed that trying to increase the aggregate volume fraction above 64% by reducing the bitumen content resulted in a higher void fraction, without an increase in the overall volume fraction of the aggregate. A mix with approximately 64% by volume sand, compacted in three layers by single plunger compression, under a

<table>
<thead>
<tr>
<th>Mix type (1)</th>
<th>Volume fraction of aggregate (%)</th>
<th>Type of aggregate (3)</th>
<th>Volume fraction of components (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AG1</td>
<td>64</td>
<td>Spherical glass beads 1.7 mm in diameter</td>
<td>64% 4% air voids</td>
</tr>
<tr>
<td>AG2</td>
<td>64</td>
<td>Spherical glass beads 1.7 mm in diameter</td>
<td>64% 20% air voids</td>
</tr>
<tr>
<td>AS1</td>
<td>64</td>
<td>Subspherical sand particles between 300 and 600 µm in size</td>
<td>64% 4% air voids</td>
</tr>
<tr>
<td>AS2</td>
<td>64</td>
<td>Subspherical sand particles between 1.18 and 2.36 mm in size</td>
<td>64% 4% air voids</td>
</tr>
<tr>
<td>B</td>
<td>75</td>
<td>Sand particles between 150 and 300 µm</td>
<td>37.5% 11%</td>
</tr>
<tr>
<td></td>
<td>Sand particles between 1.18 and 2.36 mm</td>
<td>37.5% 18%</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>85</td>
<td>Sand particles between 150 and 300 µm</td>
<td>11% 56%</td>
</tr>
<tr>
<td></td>
<td>Sand particles between 1.18 and 2.36 mm</td>
<td>18% 5% air voids</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>40</td>
<td>Sand particles between 1.18 and 2.36 mm</td>
<td>40%</td>
</tr>
<tr>
<td>E</td>
<td>52</td>
<td>Sand particles between 1.18 and 2.36 mm</td>
<td>52%</td>
</tr>
</tbody>
</table>
load of 10 MPa, gave the most homogeneous specimen, with a reasonably low void content. Consequently, this method was selected for specimen preparation. All mixes of type A were prepared by this method. Mix AG2 also was prepared by the same technique; however the bitumen content was decreased, which resulted in the high void content.

A similar study was carried out for the mix of type B. It was found that a mixture consisting of bitumen and equal quantities of sand particles between 150 and 300 μm and 1.18 and 2.36 mm, compacted in the manner described above, gave an acceptable specimen. Here again, a volume fraction of 75% corresponded to the maximum random packing density of this particular aggregate gradation (Rowe 1989).

Equipment and Procedure for Specimens of Type C

The aim here was to make specimens consisting of bitumen and 85% by volume aggregate with a reasonably low void content. To decide on the optimal aggregate design (i.e., the gradation of the aggregate) a convergence study similar to the one described above was carried out. [The aim of the convergence study was to design the aggregate such that the maximum random packing density of that gradation of aggregate corresponded to 85%. Possible aggregate gradations were taken from experimental data published in German (1989).] It was found that aggregate consisting of a volumetric mixture of 13% sand particles between 150 and 300 μm, 21% sand particles between 1.18 and 2.36 mm, and 66% rounded stones approximately 10 mm in size could be packed to 85% volume fraction.

Specimens (Ø100 × 200 mm) were made at the Department of Civil Engineering, University of Nottingham, using a roller compactor as shown schematically in Fig. 2. This simulated the compaction conditions on the road. The procedure consists of compacting the heated mixture of bitumen and aggregate in a rectangular slab under a roller. This slab was then cored horizontally (i.e., in a direction perpendicular to the compaction) to produce six cylindrical specimens per slab. Thus, these specimens were compacted in a transverse, rather than axial direction. Details of this procedure can be found in Rowe (1993).

Equipment and Procedure for Specimens of Types D and E

It was seen in the convergence study that the specimens with single-sized particles became nonhomogeneous if the volume fraction of the sand was below 64%, which corresponds to the maximum random packing density of single-sized spheres. Similarly the aggregate designs of mixes of types B and C were chosen such that 75 and 85% corresponded to the maximum random packing densities of those particular aggregate gradations, respectively. The problem with preparing a specimen with an aggregate volume fraction less than the maximum random packing density is that the sand settles and separates from the bitumen when the mixture is heated. Hence, the specimen making technique must be modified.

The same mold used for manufacturing specimens of types A and B was used to cast specimens of types D and E. Lubricant was applied to the inner surfaces of the mold to prevent sticking of the mixture. A piece of nonstick silicone paper was attached to the bottom surface of the plunger to ease removal of the specimen (these specimens were stickier and weaker than the fully dense mixes). Cold bitumen was crushed into a fine powder form and thoroughly mixed with the correct amount of sand (1.18–2.36 mm in size) and then poured into the mold. The powdered mixture was compacted in a mechanical press (at room temperature) and then heated in a furnace for about 0.5 h to 50°C, which is approximately the “softening point” of the bitumen used. This soft mixture was then further compacted at 5 MPa for about 5 min to allow the bitumen to creep, fill the air voids, and bond with the sand. The mold with its contents was then cooled down to about 0°C, and the specimen was slowly pushed out of the mold. The specimens were found to be reasonably homogeneous with negligible void content. To obtain an homogeneous specimen by this method, it is very important that the bitumen chips are smaller than the size of the sand particles.

Testing Procedure

All of the mixes were tested in uniaxial compression between two platens. Temperature control was achieved using an environmental chamber fitted to the testing machine. Testing temperatures ranged from 0 to 40°C. Data acquisition was performed using an analog to digital converter interfaced to a personal computer. Repeatability of the results was checked by performing three different tests at certain stresses and strain rates.

A lubricant consisting of a mixture of natural soap and glycerine was applied to the top and bottom surfaces of the platen to reduce friction between those surfaces and the specimen and thus to reduce bulging. The specimen was then brought to the desired temperature by keeping it inside the environmental chamber for about 40 min before starting the test. (For tests at 40°C, the specimen was maintained at the testing temperature for only 20 min to avoid excessive distortion as a result of creep induced by self-weight.)

Two types of tests were conducted. In the “constant strain rate” tests, a constant axial displacement rate was applied to the specimen, whereas in the “constant stress” creep tests, a constant axial force was applied (rapidly) to the specimen and held.

For all specimens other than those of type C, the axial strains (i.e., load line displacement), radial strains, and axial loads were measured. Only axial strains and axial loads were measured for the specimens of type C because the 38-mm radial strain transducer was too small to fit the 100-mm-diameter type C specimen. The radial strains were measured using a “hall effect” radial strain transducer. A schematic of the transducer is shown in Fig. 3. Details of the working principle of this type of transducer may be found in Clayton et al. (1989). The transducer was fitted onto the 38-mm specimens by sticking the two diametrically opposite radius pads onto the specimen with Loctite. Because the distance between the hinge and the hall effect sensor was twice the distance between the hinge and the center of the pads, the displacement across the hall effect sensor was twice the change in diameter of the specimen. All signals were recorded simultaneously by the data-logging system.
EXPERIMENTAL RESULTS

The mixes with 64% by volume inclusions were tested over the entire range of test conditions (i.e., constant stress and constant strain rate tests, at various temperatures and strain rates). Tests on the other mixes were performed only at a single temperature. Results are described in detail for the 64% mixes and then compared with the results for the other mixes.

Mixes with 64% Aggregate (Mix Type A)

Constant strain rate tests were performed at 20 and 40°C. Typical data are shown in Fig. 4, where the nominal stress is plotted against the nominal axial strain for various applied axial strain rates at 20°C. It can be seen that at a particular strain rate, the stress increases until the strain reaches approximately 4%. Beyond 4%, the stress starts to decrease. Following Ward (1983) and Cheung (1995) the “steady-state creep stress” is defined as the maximum observed stress. It can be seen in Fig. 4 that at a constant temperature the value of the steady-state creep stress increases with increasing applied strain rate.

Constant stress creep tests also were performed on specimens at temperatures ranging from 0 to 40°C. A typical result is shown in Fig. 5, where the axial creep strain is plotted as a function of the time elapsed after application of the load. The creep curve is divided into three regions: primary creep \( t < 100 \text{ s} \) where the strain rate decreases, secondary creep \( 100 \text{ s} < t < 250 \text{ s} \) where the strain rate remains constant, and tertiary creep \( t > 250 \text{ s} \) where the strain rate increases. The “steady-state creep strain rate” is defined as the secondary strain rate of the material.

The constant strain rate test and the constant stress creep test are complimentary experimental tools for investigating the behavior of bituminous mixes. At higher temperatures, where small stresses will result in high strain rates, it is more convenient to perform constant strain rate tests in which the strain rates can be easily prescribed. At lower temperatures where the strain rates are much lower, it becomes necessary to perform constant stress tests so that low strain rates can be obtained in response to a prescribed load. It was expected from tests on pure bitumen (Cheung 1995), and observed in the tests described here, that results from both types of tests agree at intermediate levels of temperature and strain rate.

Steady-state axial creep behavior observed from the two sets of tests on mix AS1 is shown in Fig. 6. Here the steady-state axial strain rate is plotted against the axial stress for temperatures ranging from 0 to 40°C. The complementary nature of the constant strain rate tests and the creep tests can clearly be seen in this figure, where results from both types of tests are seen to agree well at 20°C and intermediate levels of strain rate.

Also shown in Fig. 6 is the curve obtained by Cheung and Cebon (1997) for bitumen at 20°C, plotted using the modified Cross model [(5)]. It can be seen that the curve for the mix at 20°C has the same shape as that for pure bitumen at 20°C over five decades of strain rate and three decades of stress. The main difference is that the steady-state strain rate of the mix is less than that of pure bitumen at the same stress level (i.e., the mix is “stiffer” than pure bitumen). Consequently, the model proposed by Cheung and Cebon (1997) for pure bitumen [(5)] can be modified by replacing \( \dot{\varepsilon}/\dot{\varepsilon}_0 \) with \( S\dot{\varepsilon}/\dot{\varepsilon}_0 \) and used to describe the uniaxial steady-state creep behavior of the mixes

\[
\frac{\sigma}{\sigma_0} = \frac{S\dot{\varepsilon}}{\dot{\varepsilon}_0} \left( \frac{1}{1 + \left( \frac{S\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^n} \right)
\]

where \( S \) = stiffening factor shown in Fig. 6. The constants \( n \) and \( \sigma_0 \) determine the form of the curve, and \( \dot{\varepsilon}_0 \) is the reference strain rate. It can be seen in Fig. 6 that, for mix AS1 at 20°C, \( S \) takes a value of about 1,000. Thus, for a particular stress level, mix AS1 has a steady-state strain rate about 1/1,000 the amount of pure bitumen.

The bituminous mix is essentially a composite consisting of rigid aggregate in a bitumen matrix. Consequently, the temperature dependence of the mix is expected to be governed by the bitumen phase and independent of the aggregate. For the range of temperatures investigated, the temperature dependence of the mix was found to follow (6) and (7). Exactly the same temperature dependence was observed for mixes AG1, AG2, and AS2. However, it can be seen in Fig. 6 that at low
strain rates at 40°C, deviation from the expected behavior was observed. This is discussed later in the paper.

The constant strain rate and creep behavior of mixes AG1, AS1, and AS2 were very similar, and all these mixes showed the same steady-state behavior (i.e., the same stiffening factor). However, mix AG2, which contained 20% air voids, had a steady-state strain rate approximately 1/350 the amount of pure bitumen for the same applied stress (i.e., $S \approx 350$).

For the cylindrical specimens the volumetric strain $H$ is given by

$$H = 2E_{11} + E_{33}$$

(10)

where $E_{11}$ and $E_{33}$ = radial and axial strains, respectively. The distortional strain $E$ is given by

$$E = E_{33} - \frac{2}{3} (E_{33} - E_{11})$$

(11)

The relationship between the distortional strains and volumetric strains for every test conducted on mix AS1 is shown in Fig. 7. Ignoring the behavior at very small strains (<0.1%), the relationship is linear and can be expressed as

$$H = s|E|$$

(12)

where $s$ = dilation gradient or slope of the line in Fig. 7. The measured values of $s$ varied randomly between 0.76 and 0.85 with an average value of approximately 0.8. This variation can be accounted for by considering the uncertainty in the diameter of the specimen, nonuniform deformation of the specimen (bulging), and the accuracy of the transducer. Furthermore, $s$ was seen to be independent of temperature, stress, strain rate, or type of test (i.e., constant strain rate or creep test). It should be noted that the normal sign convention is that dilation is positive; hence the absolute value of the distortional strain is taken in (12).

The relationship between the volumetric and distortional strains for all mixes of type A (i.e., AG1, AG2, AS1, and AS2) was found to be independent of stress or strain rate and given by (12) with $s \approx 0.8$ (Table 2).

**Mixes of Types B and C**

It was seen in the previous section that the temperature dependence of the mix was governed by the bitumen and not dependent on the type of aggregate. The temperature dependence of the bitumen is due to the activation energy or free volume in the bitumen microstructure [(6) and (7)]. All of the aggregates considered here were large compared to bitumen molecules and do not interact with the microstructure of the bitumen. Therefore, the addition of the aggregate is not expected to change the temperature dependence of the bitumen, and the behavior of these mixes at different temperatures can be estimated from the temperature dependence of bitumen [(6) and (7)].

The constant strain rate and creep behavior of these mixes was seen to be qualitatively similar to the type A mixes. The main difference between the three sets of mixes (i.e., A, B, and C) is that the axial strain at which steady-state was reached

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**TABLE 2. Dilation Gradient $s$ for Various Mixes Tested**

<table>
<thead>
<tr>
<th>Mix (1)</th>
<th>Dilation gradient $s$ (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.7–0.85</td>
</tr>
<tr>
<td>B</td>
<td>0.6–0.7</td>
</tr>
<tr>
<td>C</td>
<td>Not measured</td>
</tr>
<tr>
<td>D</td>
<td>$\approx$0</td>
</tr>
<tr>
<td>E</td>
<td>$\approx$0</td>
</tr>
</tbody>
</table>
decreased from approximately 4% in the 64% mixes to 3% in the 75% mixes and about to 2.5% in the 85% mixes.

Fig. 8 includes a summary of the steady-state axial creep behavior of the 75% and 85% mixes. The axial steady-state creep behavior of these mixes also is described by (9) with $S$ decreased from approximately 4% in the 64% mixes to 3% in the 75% mixes and about to 2.5% in the 85% mixes.

The radial strains were measured for the 75% mixes. These specimens were also observed to dilate under compressive loading. The relationship between the volumetric strain and the distortional strain was found to be independent of stress or strain rate and given by (12) with $s = 0.6–0.7$ (Table 2).

**Mixes of Types D and E**

Constant strain rate tests were performed on mixes consisting of bitumen with low volume fractions of aggregate [40% (mix D) and 52% (mix E) by volume]. The tests on the 40% dispersions were performed at 0°C because the mix was too soft and deformed under its self-weight at 20°C.

The constant strain rate curves were qualitatively similar to those of the other mixes with the only difference being that the maximum stress was reached at approximately 6 and 13% nominal axial strains for the 52 and 40% mixes, respectively.

Fig. 8 also shows the steady-state axial creep behavior of the 40 and 52% mixes. [The results shown in Fig. 8 for the 40% mix have been corrected to a temperature of 20°C using (6).] It can be seen in this figure that the steady-state axial creep behavior of these mixes can be described by (9) with $S \approx 30$ and 100 for the 40 and 52% mixes, respectively. The stiffening factor $S = 30$ for the 40% mix was obtained at 0°C. However, as seen from the tests on the 64% mixes at various temperatures, this factor is insensitive to the testing temperature, and it is expected that it would be the same at 20°C.

Radial strains were measured for both the 40 and 52% mixes. It was seen that $H = 0$ for both these mixes (i.e., they showed nondilational behavior) unlike the fully dense mixes.

**DEVIAITION AT HIGH TEMPERATURES AND LOW STRAIN RATES**

Although the uniaxial behavior of the mixes can be described by the modified Cross model over most regimes tested, deviations from this behavior at high temperatures were observed. It can be seen in Fig. 6 that at low stress levels the behavior of the mix AS1 at 40°C (the encircled region) is unlike that observed at the same stress levels at 20°C. The stress-strain curve of a constant strain rate test for a low applied strain rate at 40°C is shown as curve $Q$ in Fig. 9. It can be seen that this stress-strain curve is different from a higher strain rate, stress-strain curve at 40°C (curve $P$ in Fig. 9). The maximum stress (which for all of the other cases was defined as the steady-state stress) occurs at a strain of around 10% for low strain rates ($Q$), whereas it occurs around 3–4% for constant strain rate tests performed at lower temperatures or at higher strain rates at 40°C ($P$). There is also a dramatic drop in the stress after the maximum stress is reached in curve $Q$, relative to the other tests in which the stress drops gradually. The sudden drop in stress is thought to be associated with some kind of internal failure. This contrasts with the viscoplastic yielding, which is thought to cause the gradual drop in the higher strain rate tests.

It can be seen from the encircled region in Fig. 6 that the maximum stress in the constant strain rate tests does not change significantly with applied strain rate in that region (i.e., the points lie approximately on a vertical line). This may imply that the bituminous mix behaves similar to a “soil” in that region, with a maximum stress corresponding to the failure stress of the soil and independent of the strain rate. It also can be seen that the creep tests do not agree with the constant strain rate tests in this region. Further tests (possibly triaxial tests at high temperatures) need to be performed before any conclusions can be drawn about the mechanisms governing the deformation of the bituminous mixes in this regime.

**DISCUSSION**

The major effect of adding rigid subspherical aggregate to bitumen is “stiffening” the bitumen without changing the form of the uniaxial steady-state stress versus strain rate relationship, or the temperature dependence. This stiffening effect was found to be dependent only on the volume fraction of voids and rigid inclusions. The increase of the stiffening factor $S$ with volume fraction of the aggregate is shown in Fig. 10. The stiffening effect was observed to be independent of the particle size for the aggregates used (mixes AG1, AS1, and AS2 gave...
the same results) and shape (mixes with subspherical sand particles and glass spheres gave the same results). However, this conclusion cannot necessarily be extended to mixes with aggregate of high angularity.

Although the uniaxial behavior of the bituminous mixes has the same form as that of pure bitumen, the overall constitutive relationship of these mixes changes considerably at high volume fractions of the aggregate. Bitumen for most practical purposes is an incompressible viscous solid. On the other hand, the fully dense bituminous mixes (i.e., types A and B) dilate under compressive loading. The dilation was observed to be independent of the stress or strain rate and varied linearly with the distortional strain. Also, the dependence of the volumetric strain on the distortional strain was found to change from the 64% mix to the 75% mix.

The low volume fraction mixes (i.e., the 40 and 52% dispersions) did not undergo a volume change under uniaxial compression (see Table 2 for a summary of dilation gradients of various mixes), and their overall constitutive relationship may be expected to have a similar form to that of pure bitumen. All of these facts suggest that the dilation is governed by the internal constraints imposed by the high volume fraction of rigid aggregate and is not dependent on the properties of the bitumen.

Another effect of the increase in volume fraction of the rigid aggregate is that the macroscopic strain, at which the steady state is reached, decreases. The variation of the nominal axial strain to reach a steady state with the volume fraction of aggregate is shown in Fig. 11 (the error bars represent the scatter in the experimental data). The steady-state stress for pure bitumen was achieved at about 20% nominal strain (Cheung and Cebon 1997), whereas for the mix with 85% by volume aggregate, the strain was about 2.5%; the overall behavior of the mix is governed by the bitumen films separating the inclusions. The local strain in these films is much higher than the overall macroscopic strain and increases with a decrease in the film thickness. The average film thickness in turn decreases with an increase in volume fraction of the aggregate. Thus these films (and hence the mixes) reach a steady state at a low overall macroscopic strain.

The rigid inclusions in the bitumen act as stress concentrators. Hence, even if the macroscopic stress is \(<1 \times 10^5\) Pa the local stresses are much higher. Therefore, in most cases, locally the bitumen is expected to always behave as a nonlinear viscous solid. This indicates that, while analyzing these high volume fraction mixes, the bitumen should be modeled as a nonlinear viscous material.

**COMPARISON OF EXPERIMENTAL RESULTS WITH COMPOSITE MODELS**

The idealized mixes can be modeled as three phase composites, consisting of bitumen, rigid aggregate, and air voids.
In this section, predictions of one such model, the Hashin composite sphere model (Hashin 1962), are compared with experimental observations. The idealized mixes consist of aggregate particles embedded in a bitumen matrix. A model morphology for this type of composite was constructed by Hashin (1962). It consists of a gradation of spherical particles embedded in a matrix [see Hashin (1962) for details]. Hashin derived upper and lower bounds on the shear and bulk moduli of a linear particulate composite using this morphology. Suquet (1993) proposed an approximate method to transform these and other bounds to the case of a composite consisting of rigid inclusions in a nonlinear viscous matrix. Deshpande and Cebon (1999) checked the accuracy of this transformation numerically and found that it was acceptable for low values of the power-law exponent $n$. Because the bitumen used in this study has a low $n = 2.3$, Suquet’s transformation is used here.

If the idealized mixes are modeled as three-phase composites consisting of rigid inclusions, voids, and bitumen in the appropriate proportions, all composite analyses including the composite sphere model will predict a reduction in volume over the range of volume fractions tested, the lower composite sphere upper bound agrees well with experimental results. The idealized mixes are considered here to be twophase composites consisting of rigid inclusions and bitumen (as a first approximation, the volume fraction of the voids is small enough to ignore). Furthermore, the observed dilation of the fully dense mixtures is ignored. Thus, this comparison gives meaningful insights into the applicability of this model for the low volume fraction dispersions (i.e., the 40 and 52% mixes). For the fully dense mixes this comparison is incomplete and should be viewed with caution.

The composite sphere model will predict that the two-phase composite (bitumen + aggregate) will have a constitutive law of the form

$$
\frac{E_{ij}}{E_{ij}^U} = \frac{3}{2} \left( \frac{\sigma_i}{\sigma_0} \right)^{n-1} \left( \frac{\sigma_j}{\sigma_0} \right)^{n-1} S_f
$$

(13)

where $S$ = stiffening factor; and $E_{ij}$ = strain rate of the mix. Using Suquet’s transformation (1993) the composite sphere upper bound specifies

$$
S = (1 - f)^{n-1/2} \left[ 1 + f \left( \frac{2}{5} (1 - f) - \frac{f(1 - f)^{2/n}}{10} \left( \frac{f(1 - f)^{2/n}}{21} + \frac{10}{21} \right) \right) \right]^{-1/2 (n+1/2)}
$$

(14)

where $f$ = volume fraction of aggregate. The lower bound is given by

$$
S = (1 - f)^{n-1/2} \left[ 1 + \frac{5f}{2(1 - f)} \right]^{n+1/2}
$$

(15)

A comparison between the stiffening factor predicted by the composite sphere model and the experimental observations from uniaxial compression tests on bitumen reinforced by various volume fractions of rigid inclusions is shown in Fig. 10. It can be seen in Figure 10 that, although the nonlinear composite sphere upper bound agrees well with experimental results over the range of volume fractions tested, the lower bound underpredicts the stiffening of the aggregate substantially. Thus, even though the composite sphere upper bound shows promising results, the following points need to be noted:

- The low volume fraction dispersions (i.e., not the fully dense mixes) were observed to deform at constant volume (s = 0). The composite sphere model upper bound that predicts this incompressible behavior as well as the correct stiffening effect is thus a good model for bitumen reinforced by a low volume fraction of rigid particulate inclusions.

Comparison of the predictions of the composite sphere model with data from tests on real asphalt mixes can only be made if the steady-state constitutive behavior of the bitumen used in the mixes is known [i.e., the constants in (8)]. The writers have not found any such data in the literature. Such comparisons would be very useful and are suggested as a topic of further study.

CONCLUSIONS

- The uniaxial steady-state stress versus strain rate relationship of the idealized asphalt mixtures has the same form as that of bitumen, with the aggregate acting as a stiffener. The temperature dependence of the mixes is the same as that of pure bitumen.

- The behavior of the idealized mixes is not strongly dependent on the size or shape of the particles. This conclusion may not be extendable to aggregate with a high angularity.

- The stiffening factor $S$ increases with the volume fraction of the aggregate and is approximately the same in the linear and nonlinear viscous regimes of behavior.

- The steady-state deformation behavior of the mixes is affected by the volume fraction of voids. An increase in the void content results in a reduction in the stiffening factor.

- The overall strain to attain a steady-state decreases with an increase in the volume fraction of the aggregate.

- The steady-state deformation behavior of the mixes shows a distinct change in mechanism at higher temperatures (40°C) and lower stress levels.

- The mixes with a high volume fraction of aggregate dilate under uniaxial compressive loading. The volumetric strain varies linearly with distortional strain and is independent of the strain rate, stress, or temperature. However, it changes from the 64% mix to the 75% mix. The low volume fraction mixes (the 40 and 52% mixes) deform at constant volume under uniaxial compression.

- Predictions of the stiffening factor $S$, using a nonlinear Hashin’s composite sphere upper bound, agree well with experimental measurements for a variety of idealized asphalt specimens with volume fractions of aggregate from 40 to 85%. However, the model does not include the mechanism that causes the fully dense idealized asphalt mixes to dilate under compressive loading.

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APPENDIX. REFERENCES


