Development of Criteria To Evaluate Uniaxial Creep Data and Asphalt Concrete Permanent Deformation Potential

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The uniaxial creep test is effective in identifying the sensitivity of asphalt concrete mixtures to permanent deformation or rutting. The creep test should be performed at a realistic testing temperature and at a stress level approximating field stress conditions. The creep test is shown to be sensitive to mixture variables including asphalt grade, binder content, aggregate type, air void content, temperature of testing, and testing stress state. Three parameters from the creep test are identified as effective indicators of mixture permanent deformation sensitivity: total strain at 1 hr of loading, ε_p; the slope of the steady state portion of the plot of total strain versus time of loading, m; and the creep stiffness, S_c. In addition to the creep test parameters ε_p and m, the sum of the total resilient strain from the dynamic compressive modulus test, ASTM D 3495, and ε_p should be less than one-half of the strain at failure recorded in the unconfined compression test performed in accordance with the instructions in *NCHRP Report 338*, which explains the Asphalt Aggregate Mixture Analysis System.

The process of creep in soils and other particulate media has, on occasion, been explained as a rate process. The basis of the rate process theory is that atoms, molecules, and particles participating in a time-dependent flow process are constrained from movement relative to adjacent equilibrium positions. The displacement of flow units to new positions requires the introduction of activation energy of sufficient magnitude to surmount the barrier. Mitchell (*1*) explains that the rate of shear in a particulate medium, such as soil, is influenced by a number of factors as explained by the following equation:

$$\varepsilon = 2X \frac{kT}{h} \exp\left(-\frac{\Delta F}{RT}\right) \sinh\left(\frac{f\lambda}{2kT}\right) \quad (1)$$

where

- ΔF = activation energy,
- T = absolute temperature (°K),
- k = Boltzman constant,
- h = Planck's constant,
- f = force,
- λ = distance between successive equilibrium positions,
- X = proportion of successful barrier crossings, and
- R = universal gas constant.

Equation 1 represents the direct effect of temperature on the rate of strain: as temperature increases, the rate process increases. If, in Equation 1, $(f\lambda/2kT) < 1$, the rate is directly proportional to the force, f. This is the case for an ordinary Newtonian fluid. Equation 1 is a reasonable first approximation of the rate process that explains the creep of asphalt concrete mixtures. One expects this deformation process to be a rate process.

A schematic representation of the influence of creep stress intensity on creep rate at some selected time after stress application is presented in Figure 1. At low stresses, creep rates are small and of little practical importance. The curve shape in this region is compatible with the hyperbolic sine function predicted by the rate process Equation 1. In the midrange of stresses, a nearly linear relationship is found between the log of stress rate and stress. This is also predicted by Equation 1 when the argument of the hyperbolic sine is greater than 1. At stresses approaching the strength of the material, the strain rates become very large and represent the onset of failure. From Figure 1 and Equation 1, it is apparent that the creep response of any particulate material, such as asphalt concrete, is not necessarily linear. If the stress state in the field (creep stress intensity) is one that pushes the log strain rate into the region near failure (beyond the steady state region), assumptions of linearity are not appropriate. This point is important, because in the past linear viscoelastic response of asphalt mixtures under field loading conditions has been assumed. This has partly been because such an assumption is convenient, and creep data from laboratory tests at relatively low stress levels are simply shifted to higher stress states in the field by using principles of linear viscoelastic superposition. Such an approach is clearly incorrect in the highly nonlinear region of Figure 1. The importance of selecting a realistic stress state for laboratory testing is then essential.

Another popular generalized form used to illustrate the various stages of creep is shown in Figure 2. In this figure, creep strain for a given stress level is plotted versus time and the creep strain is divided into three stages. In the first or primary stage the rate of deformation increases rapidly. In the second or "steady state" region, the deformation rate is constant, as are the angle of slope and rate of deformation. The third region is the failure stage, in which the deformation again increases rapidly.

The relationship between creep strain and logarithm of time may actually be linear, concave upward, or concave downward. A linear relationship is often assumed in engineering applications because of its simplicity in analysis. However, there is no fundamental law of behavior to dictate one form or another.

Use of the uniaxial creep test to define the stability and rut susceptibility of asphalt concrete mixtures has long been popular because of its relative simplicity and because of the logical ties between the creep test and permanent deformation in asphalt concrete pavements. The major difficulty in devel-

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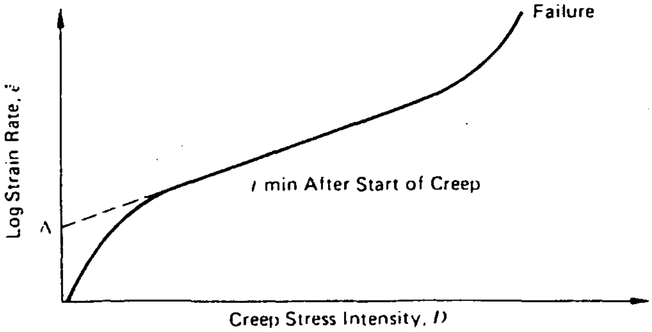

FIGURE 1 Influence of creep stress intensity on creep rate (*1*).

oping criteria associated with the creep test by which to evaluate the rutting potential of asphalt concrete mixtures is in relating these criteria to field performance. This is true for all types of laboratory testing that must be correlated with field results. However, even without the benefit of correlations between laboratory creep tests and field results, it is evident that a stable and rut-resistant mixture should not demonstrate tertiary creep if tested under stresses and at temperatures in the laboratory that simulate field conditions.

The total strain at failure after a period of loading, such as 3,600 sec, has often been used to define an acceptable mixture response in the creep test. This strain is divided into the constant stress applied to the specimen to calculate the creep modulus. This approach is used in Asphalt Aggregate Mixture Analysis System (AAMAS) to define a minimum creep modulus after 3,600 sec of loading.

It appears more proper to use only the irrecoverable strain (viscoplastic strain) in the computation of the creep modulus used to evaluate the suitability of a mix. This is because only the irrecoverable portion of the strain is important when one considers rutting potential. In actuality, the total creep modulus at the long loading time of 3,600 sec is dominated by the viscous response (irrecoverable) of the binder. The elastic portion of binder stiffness at this long loading time and the relatively high test temperatures at which the creep test is typically performed is practically nonexistent, and the viscous portion of the stiffness dominates over the delayed elastic portion. Thus, when one considers the binder only, the creep modulus calculated on the basis of total strain is essentially as appropriate as using the creep modulus based on irrecoverable modulus only.

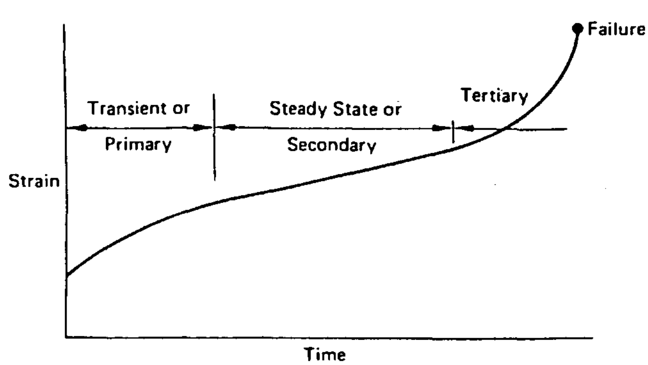

FIGURE 2 Stages of creep (*1*).

This is not necessarily the case, however, when the effects of the aggregate are considered. The resilience offered by the aggregate matrix should be considered if practicable to evaluate the effect of the aggregate matrix on the permanent deformation potential of the mixture. Probably the most direct and simple way to account for the effects of the aggregate matrix on the resilience or "recoverability" of the mixture is to perform a recovery test immediately following the creep test. This allows one to judge the effects of the resilience or recoverability of the aggregate matrix on the performance of the entire mixture.

In addition to the ultimate level of strain or the ultimate creep modulus following a given time of loading and the knowledge of what percentage of the creep is recoverable at the end of the test period, it is important to define the rate of creep. Creep tests of soils and asphalt mixtures have demonstrated that the rate of creep and the shape of the creep curve are difficult to predict. However, the following general trend is usually observed: both total strain and irrecoverable strain are functions of time of loading, temperature, stress state, mix type, and other parameters, such as the manner of loading and reloading conditions. With increasing consolidation, more asphalt cement binder is squeezed into the voids, and stress is gradually transferred to the mineral particle contacts. The strain rate decreases as this occurs. Therefore, a constant positive strain rate indicates an unstable state between the external force and the internal resistance of the material. This type of response will result in failure. During failure, the strain rate rapidly increases and the curve becomes concave upward.

DEVELOPMENT OF THE AAMAS RUTTING CHARTS

Mahboub and Little (*2*) developed a method for evaluation of creep data. This method uses the simple relationship that viscoplastic strain, ε_{vp}, occurs as a strain-hardening process according to the power law $\varepsilon_{vp} = at^b$, where a and b are regression constants and t is time. The coefficient a is dependent on stress state and mixture stiffness, whereas the exponent b represents the rate of deformation as a function of time. This value appears to fall within a tight band for good-quality asphalt mixtures when tested at a stress state that is within the linear viscoelastic region. Kinder (*3*), Perl et al (*4*), Lai and Anderson (*5*), and Mahboub and Little (*2*) have shown that b typically ranges between 0.17 and 0.25. This approach was selected for use in AAMAS [Von Quintus et al. (*6*)].

COMPARISON OF AAMAS RUTTING CHART CRITERIA WITH CREEP MODULUS CRITERIA DEVELOPED BY OTHER AGENCIES

It is interesting to compare the creep modulus criteria of the AAMAS creep modulus charts with criteria developed by other agencies. The creep test in AAMAS is performed for 3,600 sec and includes a 3,600-sec recovery period. The 1-hr creep test period is popular, perhaps because it is long enough to be applicable to the loading conditions during which rutting occurs in the pavement yet short enough to be practicable.

The minimum required creep modulus values at 3,600 sec of loading from the creep modulus charts developed by Mahboub and Little (2) are as follows:

Pavement Category	*Minimum Creep Modulus (MPa)*
Asphalt concrete over rigid base	69 (low rut potential) 34.5 (moderate rut potential)
Full depth asphalt concrete (intermediate layers)	55.2 (low rut potential) 20.7 (moderate rut potential)
Full depth asphalt concrete (lower layers)	27.6 (low rut potential) 17.2 (moderate rut potential)
Surface asphalt concrete layers	55.2 (low rut potential) 27.6 (moderate rut potential)

Of course, all these criteria are for testing for 3,600 sec or 60 min at a test temperature of 40°C and at a stress level that approximates a realistic average vertical compressive stress within the pavement layer.

Other researchers have developed similar criteria from the creep test. Viljoen and Meadows (*7*) indicate that the minimum creep modulus to prevent rutting is 82.7 MPa after 100 min of loading at 40°C at a stress level of 207 KPa. Khedr (*8*) indicates that the minimum creep modulus after 60 min of testing at a stress level of 207 KPa at 40°C should be at least 137.9 MPa. Kronfuss et al. (*9*) suggest the following set of criteria at a stress level of 103 KPa, a test temperature of 40°C and a time of testing of 60 min:

Traffic Intensity Level	*Acceptable Creep Modulus Value of Range (MPa)*
Low	20.7 or above
Moderate	20.7 to 31.0
High	31.0 to 45.3

Kronfuss et al. (*9*) also established an upper limit of stiffness at 46.5 MPa at a temperature of 40°C. They believed that stiffnesses above this level were too high and subject to cracking due to load-induced fatigue or thermal effects. However, this upper limit was established for cooler European climates, where low-temperature fracture effects must be considered to have a different significance than in many southern parts of the United States.

One of the most comprehensive studies of the effects of mixture variables on the compressive creep characteristics of asphalt concrete mixtures is documented by Sousa et al. (*10*). The primary goal of this study was to evaluate the influence of different types of laboratory compaction equipment on selected asphalt concrete mixture properties. However, the extensive compressive creep testing performed makes this study an excellent source of creep testing information, especially for considering the influence of asphalt cement type, asphalt content, aggregate type, air void content, compaction temperature, and stress level on the results of compressive creep testing.

A careful review of these data indicates that the compressive creep test is sensitive to all mixture variables and appears to be a reasonable, reliable, and expedient routine test to evaluate the potential of various mixtures to perform satisfactorily in a pavement system.

The study by Sousa et.al. compared several compaction devices. However, only the results for specimens prepared using the gyratory compactor are discussed here. On the basis of a review of the Sousa et al. data, several parameters derived from the creep versus temperature plot when presented on a log-log scale are acceptable and reasonable parameters by which to evaluate deformation potential. These parameters include the value of the creep modulus or strain (permanent) following a specific period of loading, time to reach a critical level of strain (time of rupture), and slope of the creep curve in a designated region, such as the steady state region.

Table 1 summarizes the sensitivity of these creep test parameters from the Sousa et al. data to mixture variables. The levels of sensitivity are indicative of the applicability of the compressive creep test for prioritization of asphalt concrete mixtures. The data summarized in Table 1 indicate the following:

1. Both the slope of the steady state creep portion of the creep versus time of loading plot and the strain at a specified

TABLE 1 Comparison of Extreme Values of Slope of Steady State Creep Versus Time of Loading Plot and Permanent Strain at 1-hr Loading for Gyratory Prepared Samples [Analysis of Data from Sousa et al. (*10*)]

Variable	Slope of Steady State Creep Curve			Permanent Strain at One-Hour Loading, %		
Considered	Maximum	Minimum	Absolute	Maximum	Minimum	Absolute
AC Grade	0.41	0.32	0.09	0.43 *(48.3)	0.20 *(103.4)	0.23
AC Content	0.35	0.30	0.05	0.30 *(62.7)	0.20 *(103.4)	0.10
Aggregate Type	0.45	0.25	0.20	1.20 *(17.2)	0.20 *(103.4)	1.00
Air Void Content	0.50	0.30	0.20	3.00 *(6.9)	0.30 *(69.0)	2.70
Temperature	0.40	0.30	0.10	0.50 *(6.9)	0.15 *(138.0)	0.35
Stress Level	0.40	0.30	0.10	0.50 (41.4)	0.09 *(227.5)	0.41

* Values in parentheses are creep modulus in units of MPa at one-hour loading for 207 KPa constant stress level at a test temperature of 40°C.

time of loading are sensitive to changes in mixture variables, and thus a relationship between slope of the steady state creep curve and strain at a specified time of loading (or the associated creep modulus) must exist.

2. The most influential mixture variables in terms of their effect on the slope of the steady state creep curve and on the permanent strain at 1-hr loading are as follows, in order of influence (most influential to least influential): air void content of the mixture, aggregate type, stress level, temperature, asphalt cement grade, and asphalt cement content.

3. Values of creep modulus were calculated for the permanent strains measured at 1 hr of loading under the constant stress level of 207 KPa used in testing. A substantial difference in creep modulus exists when one compares the levels of each variable. The creep modulus for the maximum level of each variable (most deleterious level) ranges from 62.7 MPa to 6.9 MPa, whereas the range for the minimum level of each variable (least deleterious) is from 227.5 MPa psi to 69 MPa.

Analysis of these data suggests that a creep modulus greater than 69 MPa after a specified period of loading, representative of field conditions, is indicative of very low sensitivity to rutting. On the other hand, critical creep moduli, representing moderate to high levels of sensitivity to rutting, are in the range of 41.4 to 6.9 MPa. It is more difficult to assign a critical value of rate of creep that differentiates mixtures on the basis of rutting susceptibility.

DEVELOPMENT OF TEXAS TRANSPORTATION INSTITUTE RUTTING EVALUATION PROCEDURE

Establishment of Critical Values of Slope of Creep Strain Versus Time of Loading Curve and Strain at 1-hr Loading

An extended evaluation of more than 100 mixtures was conducted to ascertain the critical values of slope of the steady state portion of the creep versus time of loading curve and the permanent strain after a 1-hr loading period. The variables addressed in the parametric study included the following: asphalt content, asphalt type and grade, aggregate type and gradation, temperature, air voids, polymer modification, and stress level.

These studies consisted of a partial factorial experiment with the following factors and levels of factors:

- Aggregate type—100 percent crushed limestone (CLS), 90 percent CLS + 10 percent natural sand (field sand), 80 percent CLS + 20 percent natural sand, and 60 percent CLS + 40 percent natural sand;
- Asphalt—AC-20, AC-10, and AC-20 + polymer modification at three levels (4.3, 5.0, and 6.0 polyolefin); and
- Asphalt content—optimum, optimum − 0.8 percent, optimum + 0.4 percent, and optimum + 0.8 percent.

Creep curves from a number of miscellaneous mixtures (not a part of the factorial study) will be used to supplement findings from the factorial study.

Tables 2, 3, and 4 present selected, representative data from the parametric study. Table 3 is a summary of uniaxial compression data from a parametric study in which aggregate in a 100 percent crushed limestone mixture was replaced in 10 percent increments with natural field sand of a rounded to subrounded nature. From this information the following conclusions are drawn:

1. A relationship between slope of the steady state creep versus time of loading curve and time to tertiary creep exists. The relationship is capricious and often poorly defined. However, it is apparent that guidelines can be developed by which to prioritize the potential of mixtures to deform permanently or to broadly categorize rutting potential in a mixture design/analysis system.

2. The slopes of the creep curves in Table 2 progress in a logical manner. Slope increases with the increase in natural,

TABLE 2 Comparisons of Steady State Slopes Before Tertiary Creep in Unconfined Mixtures (Testing Performed at 40°C)

Mixture Identification	Slope of Steady State Creep Prior to Tertiary Creep	Time to Tertiary Creep, Sec	Strain at 3,600 seconds, percent
100% Crushed Stone, σ_3=0 KPa, σ_1 = 414 KPa, 3.2% Air Voids	0.23	7,000	0.55
100% Crushed Stone, σ_3=0 KPa, σ_1 = 414 KPa, 6.3% Air Voids	0.42	1,700	>3.0
10% Natural Sand, σ_3=0 KPa, σ_1 = 414 KPa, 3.6% Air Voids	0.32	3,200	0.9
10% Natural Sand σ_3=0 KPa, σ_1 = 414 KPa, 5.9% Air Voids	0.54	800	Failed
20% Natural Sand, σ_3=0 KPa σ_1 = 414 KPa, 3.3% Air Voids	0.42	2,800	2.0
20% Natural Sand, σ_3 = 0 KPa, σ_1 = 414 KPa, 5.2% Air Voids	0.34	400	Failed

TABLE 3 Comparison of Steady State Creep Slopes and Permanent Strain at 1-hr Time of Creep Loading (Testing Performed at 40°C)

Mixture Identification	σ_3, KPa	Air Voids, percent	Slope of Steady State Creep Curve	Permanent Strain at One-Hour Loading, percent
100% Crushed Stone	103	4.0	0.17	0.36
	207	4.2	0.15	0.28
10% Natural Sand	103	4.5	0.22	0.56
	207	3.9	0.10	0.40
20% Natural Sand	103	4.0	0.25	0.68
	207	3.6	0.18	0.48

rounded sand content of the mixture. This can also be said of the time to tertiary creep (defined as a rapid upward slope change) as the time to rupture is progressively smaller with an increase in the percentage of natural field sand.

3. On the basis of these data, it is apparent that a log-log slope of the creep versus time of loading curve of less than 0.25 is indicative of a mixture that will not become unstable (reach tertiary creep) within the testing period of 3,600 sec.

4. The trends demonstrated in Table 3 substantiate the findings of other researchers, such as Sousa et al. (*10*).

Table 3 summarizes the data for the same crushed limestone mixtures with replacement of varying percentages of the aggregate portion with natural field sand but with confining pressures of either 103 or 207 KPa. These data demonstrate that the application of confinement substantially and pre-

TABLE 4 Comparison of Uniaxial Creep (σ_1 = 414 kPa) Data from 10 Selected Mixtures (Each Value Is Average of Data Points) (Testing Performed at 40°C)

Aggregate	Binder	Air Voids, percent	Slope of Steady State Creep Curve	Time to Tertiary Creep, seconds
100% Crushed	AC-10	3.2	0.23	7,000
	AC-10+ 4.3% LDPE*	3.7	0.17	> 10,000
	AC-10+ 6.0% LDPE	3.4	0.15	> 10,000
90% Crushed 10% Natural Sand	AC-10	3.8	0.32	3,200
	AC-10+ 5% LDPE	4.2	0.25	6,000
80% Crushed 20% Natural Sand	AC-10	3.3	0.42	2,800
	AC-10+ 5% LDPE	3.4	0.22	5,500
100% Rounded River Gravel (RG)	AC-20	4.2	0.40	2,000
80% RG+ 20% Crushed	AC-20	4.4	0.30	3,000
80% RG 20% Crushed	AC-20 +5% LDPE	3.9	0.24	5,900
100% Crushed Granite	AC-20	5.1	0.30	4,000
	AC-20 +5% LDPE	5.0	0.17	20,000
Stone Mastic Mixture (SMA)	AC-30	3.0	0.35	2,000
0.3% Fiber (Georgia Granite)	AC-30 +5% LDPE	3.0	0.20	> 3,600
SMA with Crushed Gravel (Colorado) 0.3% Fiber	AC-10	2.8	0.29	3,600
	AC-10+ LDPE	3.0	0.20	> 3,600

* Polyolefin - Low Density Polyethylene (LDPE).

dictably reduces the slope of the steady state portion of the creep curve and reduces the magnitude of permanent strain at the 1-hr loading time. Note in Table 3 that as the confining pressure is increased from 103 to 207 KPa, in each case, the slope is reduced and the strain at 1-hr loading is significantly reduced. This points out the influence of state of stress on the results of the creep test and the importance of trying to mimic the state of stress induced in the actual pavement as closely as possible during laboratory creep testing for mixture design/analysis.

Data in Tables 2 and 3 demonstrate the presence of the tertiary creep region at a strain of approximately 0.8 to 1.0 percent for all unconfined mixtures. The time at which this tertiary creep region begins is obviously dependent on mixture variables. The tertiary creep region is reached in uniaxial creep testing for all mixtures except the low air void, 100 percent crushed mixture. However, upon the application of confinement, the tertiary creep region is not reached within the 1-hr loading period.

Table 4 presents a summary of extended mixture creep data for mixtures with other aggregate types and with different binders, including polymer-modified binders. These data further substantiate the information presented in Table 3 for a wide variety of mixtures.

Results summarized in Tables 2 through 4 and results from previous research were used to establish criteria for evaluation of creep test data as a diagnostic test. Test criteria will be discussed at the end of this section.

Determination of Appropriate Testing Temperature

A testing temperature of 40°C was selected for creep testing because of the history of use of this temperature for creep testing, because of the selection of this test temperature by AAMAS, and because the use of this testing temperature makes sense when one considers temperature profiles in pavements under climatic conditions in the southern United States.

In Texas, for most traffic profiles, the 40°C test temperature as an approximation of a nominal high pavement temperature is acceptable. A complete evaluation of the justification of this test temperature is given elsewhere (*11*).

Determination of Appropriate Level of Axial Stress in Unconfined Compression Creep Test

The literature is filled with creep test data where low stress states are applied in the laboratory creep test. For example, in the VESYS procedure (*12*) a uniaxial stress of 138 KPa or less is recommended. If the strain during a preconditioning period exceeds 0.0635 mm of strain (0.25 percent), the stress level is to be reduced until the strain falls below the 0.0635-mm level. This often results in a uniaxial stress of as low as 34.5 KPa. The major reason for maintaining a stress level within these bounds is to stay within the linear viscoelastic region so that the pavement can be analyzed using linear viscoelastic theory. However, stress levels between 34.5 and 138 KPa are usually much too low to simulate field stress conditions. This is not a problem for linear viscoelastic theory because the difference between the stress induced in the pavement and the test in the laboratory can be easily handled by linear viscoelastic superposition. However, the asphalt concrete does not respond in a linear viscoelastic manner up to the point of failure. The response is often highly nonlinear and, therefore, a laboratory mixture evaluation test must account for this nonlinearity by testing at the appropriate stress level.

Mahboub and Little (*2*) developed *Z* factors similar to those developed by Shell researchers that demonstrate that vertical compressive stress within the asphalt concrete pavements layer generally range between 65 and 86 percent of the average contact stress between the tire and the pavement surface. Since today's truck tires are often inflated to as high as 1034 KPa, this can mean average vertical compressive stresses of as much as 5 times those prescribed by methods such as VESYS (*13*).

Von Quintus et al. (*6*) used linear elastic theory to calculate the distribution of vertical compressive stresses within the asphalt concrete layer. In an example showing how to use the AAMAS method, they suggested using 448.2 KPa in the uniaxial static creep test (40°C) to simulate the stress in an asphalt concrete surface layer (full depth) subjected to a tire pressure of 896.4 KPa, which varied from 792.9 KPa at the top of the layer to 138 KPa at the bottom. The 448.2 KPa compressive stress was used because it is the point at which the horizontal stresses are approximately zero. This would represent a critical stress condition for the uniaxial test.

Roberts et al. (*14*) demonstrated that the conditions of stress under actual loading may be much more severe than is demonstrated by layered elastic approximation, because layered elastic approximation does not account for the nonuniformity of loading across the tire carcass or the horizontal shearing stresses induced by braking or cornering.

To more realistically evaluate the stress level to which the uniaxial creep sample should be subjected, various pavements' structural sections were modeled with the modified ILLI-PAVE structural computer program. Realistic tire contact pressures, stress distributions, and shearing stresses were introduced for each specific condition. From the calculated stress conditions, octahedral normal and shear stresses were calculated and contours of equal normal stress and shear stresses were plotted using a computer graphic program.

The Mohr-Coulomb failure theory is a realistic simple approximation of the failure stress level in an asphalt mixture at high pavement temperatures:

$$\tau_{(\mathrm{oct})\mathrm{failure}} = c + \sigma_{(\mathrm{oct})\mathrm{normal}} \tan \phi \qquad (2)$$

where c is the cohesive strength and $\sigma_{(\mathrm{oct})_n} \tan \phi$ is the strength mobilized through frictional interaction among the aggregate particles. On the basis of the Mohr-Coulomb failure law, it is apparent that the critical stress state within the asphalt concrete pavement layer should exist where the stress state is such that the frictional component of shear strength has the least potential to develop and where the induced shear stress is greatest. This would occur where the ratio of octahedral normal stress to octahedral shear stress (NTSR) is a minimum, based on this hypothesis. Contours of NTSR for six important pavement structural and environmental conditions were developed.

The triaxial loading conditions are found from the NTSR relationship using the following simple relationships derived from Mohr-Coulomb failure theory:

$$\sigma_1 \text{ (axial stress)} = \sigma_{\text{oct critical}} + \frac{\sqrt{2}}{2}\tau_{\text{oct critical}} \tag{3}$$

$$\sigma_3 \text{ (confining stress)} = \sigma_{\text{oct critical}} - \frac{\sqrt{2}}{2}\tau_{\text{oct critical}} \tag{4}$$

where σ_{oct} is the normal octahedral stress on the critical plane and τ_{oct} is the octahedral shear stress induced by the load in the field on the critical plane. σ_1 and σ_3 are determined from the critical NTSR and the value of the normal stress at the point of critical NTSR for a specific pavement category. It is then a simple matter to approximate σ_1 where $\sigma_3 = 0$ based on typical values of c and ϕ and the Mohr-Coulomb equation. The σ_1 value for uniaxial loading is an averaged value for approximate c and ϕ values for typical asphalt concrete mixtures.

In most overlay cases, the uniaxial stress should be between 275.8 and 551.7 KPa. For most designs a uniaxial stress of between 344.8 and 413.7 KPa is appropriate.

Criteria for the Evaluation of Uniaxial Compressive Creep Test Data

Tables 5 and 6 present the summary of criteria suggested for use in the evaluation of compressive creep test data. Table 5 presents the characteristics or parameter values of the compressive creep curve required to provide rut-resistant mixtures. These values were developed from a review and study of the data presented in Tables 2 through 4 and of the data from other researchers.

The basis for the development of these criteria is the understanding of the nature of the creep response as explained earlier in this section. Most important, instability in the creep curve occurs at strains above approximately 0.203 mm/mm. Thus the general approach was to ensure that for a loading condition representing field conditions as closely as possible, the strain at a time of loading representing the traffic intensity and accumulation in the field does not exceed 0.203 mm/mm. The traffic intensity in these tables is defined by the number of standard axle equivalents to which the pavement will be subjected. This calculation was based on the assumption that a wheel with 690 KPa contact pressure moving at 96.5 km/hr applies a haversine-type stress function to an element of pavement over a period of approximately 0.01 sec. Thus, a 3,600-sec creep period is representative of approximately 360,000 applications of a standard axle equivalent (ESAL). For traffic intensities greater than 360,000, the total strain at 3,600 sec was calculated that would result in a total strain at the end of the appropriate period that does not exceed approximately 0.203 mm/mm. This calculation was made taking into account the slope of the steady state creep curve of various mixtures.

The approximation of performance of mixtures that are to perform in high traffic areas (greater than about 360,000 ESALs) is difficult because of the often erratic changes in the nature and slope of the creep curve between strains of about 0.5 and 1 percent. Hence, a better way is to test the specimen in creep for the appropriate period of creep loading.

Footnote 2 in Table 5 identifies an additional criterion that must be met. This criterion is based on the AAMAS procedure [Von Quintus et al. (*6*)], which suggests that the permanent strain at the end of a 1-hr period of creep loading should be compared with the trace of the stress versus strain results of the unconfined compression test. Accordingly, the sum of the permanent strain, ε_p, at the end of the 3,600-sec loading period of the creep test and the total resilient strain, ε_{rt}, measured during the uniaxial resilient modulus test, should not exceed approximately 50 percent of the strain determined during the unconfined compression test, ε_{qu}, ASTM T 167. The total strain recovered, ε_{rt}, is measured at a loading frequency of 1 cps (0.1-sec load duration and 0.9-sec rest period). In equation form this reads as follows:

$$\varepsilon_p + \varepsilon_{rt} < 0.5\varepsilon_{qu} \tag{5}$$

TABLE 5 Strain at 1-hr Creep Loading and Slope of Steady State Creep Curve Required To Reduce Rutting Potential to Very Low Level

Total Strain at One-Hour of Loading, %	Slope of Steady State Creep Curve					
	< 0.17	< 0.20	< 0.25	< 0.30	< 0.35	< 0.40
< 0.25	IV[2]	IV[2]	IV[2]	IV[2]	IV[2]	III
< 0.40	IV[2]	IV[2]	IV[2]	III[2]	III[2]	III[2]
< 0.50	IV[2]	IV[2]	III[2]	III[2]	III[2]	II
< 0.80	III[2]	III[2]	II	II	II	II
< 1.0	I	I	I	I	I[1]	
< 1.2	I[1]	I[1]	I[1]			

Notes:
I - Low traffic intensity: < 10^5 ESALs
II - Moderate traffic intensity: Between 10^5 and 5 x 10^5 ESALs
III - Heavy traffic intensity: Between 5 x 10^5 and 10^6 ESALs
IV - Very heavy traffic intensity: >10^6 ESALs

1. Must also have $\epsilon_p < 0.8\%$ at 1,800 seconds of creep loading
2. Should also meet the following criterion: $\epsilon_{rt} + \epsilon_p < 0.5\ \epsilon_{qu}$

TABLE 6 Creep Stiffness Criteria at 1-hr Creep Loading

Level of Rut Resistance	Traffic Intensity Level	Required Minimum Creep Stiffness, MPa, for Test Constant Stress Level of:		
		207 KPa	345 KPa	483 KPa
Highly Rut Resistant	IV	103.4	120.7	155.1
	III	48.3	69.0	96.5
	II	34.5	44.8	60.3
	I	20.7	27.6	41.4
Moderately Rut Resistant	IV	51.7	69.0	96.5
	III	34.5	50.0	69.0
	II	24.1	41.4	51.7
	I	17.2	20.7	27.6

This relationship attempts to ensure that the permanent strain developed in the creep test is limited so that strain softening does not develop in the mixture. Strain softening was generally thought to occur at approximately one-half the value of the strain at peak load during the unconfined compressive test. Extensive testing by Little and Youssef (*11*) shows the consistency of strain softening (nonlinearity) beginning at approximately $0.5\varepsilon_{qu}$.

It is suggested at this point that the sum of the total resilient strain, ε_{rt}, and the permanent strain from the creep test, ε_p, be limited to $0.5\varepsilon_{qu}$. This specification should be a part of the criteria for creep evaluation. This evaluation is then a practical substitute for the resilient recovery factor until more complete and specific testing is performed.

Three parameters are used to evaluate the creep data: slope of the steady state creep curve, strain at 1-hr of loading, and the sum of the total resilient strain and total strain at the end of 1 hr of creep loading at 40°C under realistic loading conditions.

Table 6 summarizes the concomitant creep stiffness values calculated from data in Table 5. These values are presented because they are popularly used guidelines for rut sensitivity.

GUIDELINES FOR EVALUATION OF RUTTING POTENTIAL BASED ON CREEP DATES

The rutting potential of asphalt concrete mixture in this procedure is based on the 1-hr creep test at a test temperature of 40°C. The following steps are required for evaluation of rutting potential:

1. Determine the traffic intensity of the roadway where the mixture is to be used. Traffic intensity is defined as the number of ESALs predicted during the 180 hottest days of the year. This is a conservative approach. Determine the pavement structure where the mixture is to be used.
2. Enter the appropriate table for the pavement structure in question [see Little and Youssef (*11*)] and determine the uniaxial stress level to be applied during the 1-hr creep test at 40°C.
3. Perform the creep test and record creep data in accordance with Figure 14 of *NCHRP Report 338*.
4. Obtain a continuous readout over the 3,600-sec test period (at least one data point every 100 sec) and plot the creep data on an arithmetic scale. The purpose is to identify tertiary creep if it exists during the 1-hr creep loading period.
5. Calculate the steady state portion of the creep curve between approximately 1,000 and 3,600 sec.
6. Enter Table 5 with the slope, *m*, and the strain at 1-hr loading, ε_p, and determine for which levels of traffic the mixture is acceptable. If the mixture is not acceptable for the traffic level intended, alter the mixture through changes in the aggregate gradation, mineral aggregate selection, binder selection, or binder modification.
7. From results of the resilient modulus test performed before the 1-hr creep test (ASTM D 3497 and Paragraph 2.9 of *NCHRP Report 338* (*6*) and from the uniaxial compressive creep test (AASHTO T 167 and *NCHRP Report 338*, Paragraph 2.9), ensure that the requirement of Equation 5 is met.
8. As a verification of the rutting potential, the amount of rutting can be approximated by the procedure discussed in Paragraph 4.5.2 of *NCHRP Report 338*.

CONCLUSIONS

Uniaxial creep test data can be used to evaluate the permanent deformation potential of asphalt concrete mixtures when the laboratory creep testing is performed in such a manner as to simulate realistic field stress states.

The creep test parameters that have been shown to rationally relate to permanent deformation potential are strain at 1-hr loading and concomitant creep stiffness at 1 hr and the log-log slope of the steady state portion of the creep strain versus time of loading plot. In addition, the total resilient strain, ε_{rt}, from ASTM D 3497, strain at failure from compressive strength testing (AASHTO T 167), ε_{qu}, and creep strain at 1-hr loading ε_p, should be used to ensure that

$$\varepsilon_p < 0.5\varepsilon_{qu} + \varepsilon_{rt}$$

It is realized that cyclic testing is more realistic and better predicts permanent deformation damage potential. However, if cyclic testing equipment, procedures, or expertise are not available, uniaxial creep testing can be effectively used to prioritize permanent deformation potential among various mixtures subjected to specific climatic, pavement structural, and loading conditions. Although ASTM D 3497 is used if equipment is available to compute ε_{rt}, this determination requires relatively few loading cycles, whereas 10,000 cycles are more normally required in permanent deformation testing.

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