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Development and Use of the Repeated Shear Test (Constant Height): An Optional Superpave Mix Design Tool

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Abstract

This study investigated rut development in asphalt pavements in order to establish the foundations for the prediction of rutting in pavement structures. This paper presents some advances in the characterization of asphalt-aggregate mixes by using finite element technology to predict permanent deformation. Although fatigue and thermal cracking may affect permanent deformation, such mechanizms are not discussed as they are considered of secondary importance.

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Modeling Permanent Deformation of Asphalt-Aggregate Mixes

1.1 Introduction

This paper presents some advances in the characterization of asphalt-aggregate mixes in light of the use of finite element technology to predict permanent deformation on pavements. Although fatigue and thermal cracking may affect permanent deformation, those mechanisms will not be considered in the material characterization section as they are considered to be of secondary importance. The objective of this study is to provide better insight into rut development, and establish the foundations for the prediction of rut evolution in pavement structures.

With increases in axle loads, load repetitions, tire pressures, and asphalt concrete thickness, a need has developed for methodology to predict rut depths in advance of construction to mitigate potential safety problems, e.g., hydroplaning. Concomitant with the development of analysis procedures which permit estimates of stresses, strains, and deformations resulting from traffic loads, pavement design systems have evolved which provide for rutting considerations. A number of these procedures include criteria for limiting values of subgrade strain to levels that preclude rutting at the pavement surface. Examples include the Shell procedure (Claessen et al., 1977); the Asphalt Institute procedure (Shook et al., 1982); and the State of Kentucky methodology (Southgate et al., 1977). Some have recommended limitations on vertical subgrade stress (rather than strain), e.g. Barksdale and Miller (1977). Others have utilized statistically-based rut depth prediction equations. For example, Saraf et al. (1976) presented such an equation that incorporated surface deflection as computed from elastic layered analysis.

Design limitations on strain or stress are based on the assumption that, if the maximum vertical compressive strain or stress at the surface of the subgrade is less than a critical value, then rutting will be limited to a tolerable level for a specified number of load applications. Unfortunately, such methodology does not necessarily preclude rutting which might occur in the asphalt-bound layer. The Shell method (Claessen et al., 1977) exemplifies a procedure which attempts to improve the above process by including additional analysis to estimate the amount of rutting occurring in the asphalt-bound layer. The procedure makes use of layered system elastic analysis and represents one of a number of such procedures termed "layer-strain predictive" methodologies. A second approach used thus far makes use of closed form viscoelastic analysis.

The layer-strain method consists of predicting rut depths using permanent deformation characteristics determined from laboratory tests together with an analysis procedure for the pavement structure using either linear or nonlinear elastic theory. The principle of this method was first proposed by Barksdale (1972) and Romain (1972). In viscoelastic closed form methodologies, moving wheel loads can be considered in conjunction with time-dependent material properties to define the state of stress and strain at particular points in the pavement structure. Material properties can be defined either in terms of models consisting of finite numbers of Maxwell and/or Kevin elements in various arrangements or in terms of generalized compliance relationships. While nonlinear viscoelastic response characteristics may provide a more realistic estimate of pavement response, the associated mathematical complexities have limited past analyses to linear characterizations (Barksdale and Leonards, 1967 and Elliot and Moavenzadeh, 1971).

One of the main reasons for the lack of advances in constitutive modeling of asphalt-aggregate mixes is closely related to available computer technology. Even if a constitutive law that accounts for all characteristics were available, until recently it would have been of no practical use for pavement analysis. This is because simulations of millions of load cycles must be performed to predict the pavement response. On the 386-based PCs, until recently the standard in the pavement industry, such an analysis would require years to complete. Recent developments in the PC and workstations have opened new avenues for engineering applications. It is expected that by 1994 these workstations will run 10,000 times faster than a 386 PC. In addition, virtual memory capabilities of these new platforms makes the difference even more pronounced. Now advances in computational capabilities and in finite element methodologies where improved elements efficiently reduce the number of operations to be executed open the door for more accurate modeling of asphalt concrete mixes. In parallel with these developments, advances in laboratory testing equipment with computerized data acquisition and control systems permits the development of complex testing protocols for implementation as standard tests.

Shell Development Corporation and SWK recently jointly developed a pavement analysis program (PACE) which uses a linear viscoelastic model consisting of two Maxwell elements in parallel. This finite element program requires input from repetitive creep tests executed at different temperatures.

The SHRP-A005 (Quarterly Reports) proposed a rate independent elastoplastic constitutive law. It requires the input from a series of tests and relies on statistical correlation with field performance for its predictive capabilities.

SHRP-A003A proposed new testing procedures (Sousa et al., 1993) and a new constitutive law for asphalt-aggregate mixes. This model consists of a nonlinear elastoplastic component (coupling between the volumetric and deviatoric responses is introduced through the elastic model) in parallel with a number of linear Maxwell elements. The A-003A model accounts for some of the main attributes of asphalt-aggregate mixtures, such as volumetric-deviatoric coupling, hardening under hydrostatic pressure, and rate dependency.

1.2 The Rutting Phenomenon

Rutting in asphalt-concrete layers develops gradually as the number of load applications increases, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification (decrease in volume and hence, increase in density) and shear deformation. Trenching studies at the AASHO Road Test (Highway Research Board 1962) and test-track studies reported by Hofstra and Klomp (1972) indicate that shear deformation rather than densification was the primary rutting mechanism. The importance of placing materials at high densities in order to minimize shear deformation was emphasized. Recent work of Eisenmann and Hilmer (1987) also concluded that rutting was mainly caused by deformation without volume change. Figure 1, reported from Eisenmann, illustrates the effect of the number of wheel passes on the surface profile of a wheel-track test slab. These data enable the measurements of the average rut depth as well as the volumes of displaced material below the tires and in the upheaval zones adjacent to them. From these observations two conclusions were drawn:

1) In the initial stage of trafficking, the increase of irreversible deformation below the tires is distinctly greater than the increase in the upheaval zones. In this initial phase, therefore, traffic compaction has an important influence on rutting.

2) After the initial stage, the volume decrement beneath the tires is approximately equal to the volume increment in the adjacent upheaval zones. This is an indication that compaction under traffic is completed for the most part and that further rutting is caused essentially by displacement with constancy of volume. This phase is considered to be representative of the deformation behavior for the greater part of the life of a pavement.

For properly compacted pavements, shear deformations, caused primarily by large shear stresses in the upper portions of the asphalt-aggregate layer(s), are dominant (Sousa et al., 1991). Repetitive loading in shear is required in order to accurately measure, in the laboratory, the influence of mixture composition on resistance to permanent deformation. Because the rate at which permanent deformation accumulates increases rapidly with higher temperatures, laboratory testing must be conducted at temperatures simulating the highest levels expected in the paving mixture in service.

In the development of the rut depth it is also necessary to recognize the evolution of the void content in a pavement section (see Figure 1 in Sousa, 1994). When air void contents drop below 2-3% the binder acts as a lubricant between the aggregates and reduces point to point contact pressures. Permanent deformation of asphalt aggregate mixes is strongly controlled by the plastic component due to the aggregate skeleton. This causes permanent deformation changes either in volume or in shear to occur mostly in hotter days or under heavy loads. Once the pavement is subjected to high stresses and high temperatures, significant permanent deformation changes will only take place under loads and temperatures of similar magnitude. When subjected to lower level loads when at lower temperatures, the mix hardly deforms.

To understand the limitations and the needs in the development of constitutive equations it is necessary to review the main attributes of asphalt-aggregate mixes that can be directly associated with permanent deformation. These characteristics are:

1. Mix behavior is rate and temperature dependent;

2. Asphalt-Aggregate mixes dilate (i.e., there is volumetric deviatoric coupling in the sense that deviatoric stresses lead to volume/pressure change, and increased hydrostatic pressure leads to deviatoric stiffening);

3. Exhibits different properties in tension and compression;

4. Cyclic loading (both mechanical and thermal) leads to crack development;

5. Residual deformations are observed at the end of loading cycles;

6. Asphalt-aggregate mix behavior is strongly dependent on air void contents;

7. Aging (i.e. loss of fluidity) can play an important role in development of permanent deformations; and

8. In some mixes moisture damage also plays an important role in development of permanent deformation.

The successful constitutive law must accommodate all of the above characteristics. An evaluation of the models discussed above reveals that none of them are able to account for all of the above characteristics. For example, linear viscoelastic models account for the rate and temperature dependency *(1)* and some linear viscoelastic models exhibit residual strains *(4)*. Linear viscoelastic models, however, fail to account for the other characteristics. In addition, experience has shown (Sousa et al.) that linear viscoelastic models that do exhibit residual strain underestimate it in the asphalt-aggregate mixes. The A005 model fails to account for *(1)* through *(4)* and *(6)* though *(8)*. In addition, the rate independent nature of the model indicates that after the first load cycle (note that in the pavement applications the loading is only in one direction) no additional residual strain will be accumulated. In reality, however, the residual strain at the end of the first cycle is only a small percentage of that at failure. The A-003A model does not account for *(3)*, *(6)* through *(8)*. Consequently it underestimates the residual strain (a property shared by the A-005 model which uses statistical analysis to correct the model's predictions). In addition, the plastic model used, while simple, is inadequate for asphalt-aggregate mixes. Finally, the nonlinear elastic model involves too many constants, which makes the model difficult to use in practice.

1.3 Asphalt Concrete Characterization

Permanent deformation of asphalt-aggregate mixes is a complex phenomenon where aggregate, asphalt and aggregate-asphalt interface properties control the overall performance. Furthermore, over time these properties change (as well as their relative contribution) until the mix reaches the end of its useful life (i.e., failure occurs due to excessive permanent deformation or crack development). In this section general trends of mix behavior are presented. Detail information about the mixes is not included because only the overall behavior is of interest for this paper, not relative performance.

1.3.1 Binder's Influence

Temperature Susceptibility and Rate of Loading

Temperature susceptibility and rate of loading effects on the mixes have been well documented, evaluated and are now quite well established (Sousa et al.,1993 and Bouldin et al.,1994) They have been generally presented in terms of master curves for dynamic modulus and phase angles. It is also well established that for small strains a thermo-rheological simple behavior can be expected.

Aging Effects

Aging of asphalt is an important aspect controlling the mix behavior over the life of a pavement structure. Two aging procedures have been developed by SHRP-A003A to capture that process. A short term oven aging procedure amounts to placing the loose mix in the oven for 4 hours at 135 ÉC. This aging procedure is supposed to simulate aging effects occurring during the compaction process in the field. It is therefore likely that a specimen prepared in this fashion might not be representative of a specimen that was 3 or 4 years in the field. A long term oven aging procedure was proposed were specimens are placed in the oven at 85 ÉC for 2, 4 or 6 days depending on the amount of aging required. The relative effects of long term and short term oven aging are quite different. Figure 2 (after AbWahab et al., 1993) presents a comparison between the master curves for an asphalt mix that was subjected to those two aging procedures. A significant stiffening in the long term aging procedure at every level of the frequency spectrum is observed. This effect strongly affects the accumulation of permanent deformation (see Figure 13 in Sousa, 1994).

Moisture Effects

Moisture effects cannot be directly associated with the binder or with the aggregate as they usually affect the interface between the asphalt and the aggregate. However because it influences the inter-aggregate bond it could be thought of as degradation of the asphalt. In modeling efforts it is proposed that moisture effects be represented by damage to the asphalt stiffness. Water can affect the bond between the binder and the aggregate and reduce the resistance to shear stresses. The relative performance in the repetitive simple shear test at constant height (RSST-CH) from specimens of the same mix of specimens subjected to moisture effects can be observed (see Figure 14 in Sousa, 1994).

1.3.2 Aggregate's Influence

Air Void Content

Air void content strongly affects permanent deformation characteristics of asphalt-aggregate mixes. Figure 3 shows not only that reduction of air void content significantly increases the number of cycles to reach a given permanent shear strain in the RSST-CH (the 0.04545 level was selected because it correlates well with 0.5 in. rut depth, Sousa, 1994), but also that its effect is binder dependent. It is also known that mixes are placed at high void contents that get reduced due to traffic densification. During that process mix properties are changing. If traffic densification causes the mix to reduce air void content to levels below 2-3%, a sharp decrease in resistance can be observed (see Figure 11 in Sousa,1994).

Stress Hardening due to Confining Pressure

Evidence of stress hardening with confining pressure can be obtained from axial creep tests executed at different confining pressures. Figure 4 diagrams the results from tests executed at 20 psi axial stress at 40 ÉC. Three confining pressures were used: 0, 15, and 30 psi. It can be observed that with the increase in confining pressure the permanent deformation was significantly reduced. This is caused by an increase of the shear moduli due to an increase in confining stress. This coupling phenomenon is considered very important as it is the primary cause of mix stability due to aggregate interlock.

Dilation

The coupling phenomenon between shear and volumetric strains can be observed in shear tests executed under constant height. The axial stress required to maintain the height constant increases with the development of shear strains (see Figure 5). This dilatency is mainly due to the aggregate particles trying to roll past each other. Dilatency can also be due to modified asphalts that exhibit rate dependent dilatency.

Dilation is an important phenomenon that accounts for the tendency of the development of confining stresses when the mix is subjected to shear strains. These confining stresses will in turn provide an increase in shear stiffness that reduces permanent deformation. Modeling efforts must account for this phenomenon if accurate predictions are to be made.

Plasticity

The most difficult properties to isolate are those directly related to the plastic behavior of the mix. These are mainly controlled by the aggregate skeleton. However, it is difficult to develop a test where they are not coupled with permanent deformation due to the viscosity of the binder. Insight into plastic properties can be obtained from repeating test sequences of different load levels or temperatures.

Evidence of plasticity using a sequence of different load levels. The RSST-CH was used in this study. The height was maintained constant with +/- 0.0005 in. while 800, 900, 1000 and 1200 cycles of a 0.1 sec haversine pulse shear stress with 4, 6, 8 and 10 psi, respectively, was applied with 0.6 sec rest periods. The tests were executed in the following order: 4, 6, 8, and 10 psi. This sequence of tests was repeated three times. The results are presented in Figure 6. It can be clearly observed that while the first test at 4 psi exhibits a relatively high rate of accumulation of permanent deformation there is no accumulation of permanent deformation under 4 psi stress after the specimen was subjected to the 6, 8, and 10 psi loading. At higher stress levels such as 8, which is only 80% of the maximum stress applied (10 psi), the rate of accumulation after the specimen was subjected to 10 psi is still quite lower than the first time the 8 psi stress level was applied. This indicates that higher stress levels permit the creation of stronger aggregate structures that the lower stress levels cannot unlock (i.e., isotropic and/or knematic hardening are present as the elastic region is expanded).

Evidence of plasticity using a sequence of different temperature levels. The RSST-CH was used in this study. The height of the specimens was maintained constant within +/-0.0005 in. While 400 cycles of a 0.1 sec haversine pulse shear stress with 10 psi magnitude was applied with 0.6 sec rest periods. Before each test, a precondition sequence of 100 cycles of 1 psi haversine shear stress was applied.

A series of 5 identical test sequences was executed on the same specimen in the following order:

1st - Test executed at 35 ÉC,

2nd - Test executed at 45ÉC,

3rd - Test executed at 35ÉC,

4th - Test executed at 45ÉC and

5th - Test executed at 35ÉC.

Before each test the specimen was maintained at the test temperature for at least 2 hours and for no more than 5 hours.

Each of the tests provided a relationship between the permanent shear strain at the end of each cycle and the number of load repetitions. The results for each of the specimens were grouped by temperature. The results obtained at 35ÉC and at 45ÉC are presented in Figure 7.

It can be noted that in the first test executed at 35ÉC the rate of accumulation of permanent deformation is higher than during the second or third tests executed at 35ÉC (after the tests executed at 45ÉC). This is particularly noticeable between 300 and 400 cycles. During the first and second 45ÉC tests the rate of accumulation of permanent deformation is approximately the same.

These results indicate that plasticity plays a very significant role on the permanent deformation phenomenon. If modeling efforts do not account for it, severe prediction errors can be expected.

Repetitive versus Creep Behavior

Axial unconfined creep and repetitive tests were executed on identical specimens at 40 C. Two stress levels were used (10 and 40 psi). Square waves repetitive loading was applied with 0.1 seconds of applied load with 0.6 seconds of rest period. Comparisons between the total time of loading and the axial strain are presented in Figure 8. If the material was linear viscoelastic then at long times of loading the two curves should be approaching each other. However it can be observed that the repetitive loading seems to change the specimen's response more than would be expected just due to the different loading rates. Furthermore for the 10 psi load level it can be observed that the specimen under creep had a tendency to stabilize the permanent deformation most likely due to locking in the aggregate structure while the repetitive loading pattern did not permit locking to occur. This is further evidence of the plastic component in asphalt-aggregate mix behavior.

1.3.3 Failure Criteria

Francken (1977) has reported that there is a threshold of states of stress beyond which specimens fail rapidly. However Sousa et al. (1991) suggests that a threshold seems to be more closely associated with strain than with stress. This research has also shown that the threshold level of strain is not a constant but influenced by both the mixture composition and the state of stress. However, for conditions investigated reasonable consistency was found in the strain level at which a variety of mixtures collapsed under creep loading. Critical (terminal) strain levels were found to be approximately 0.008 in/in for compressive creep and 0.02 in/in for shear creep.

Furthermore, tests have been executed under other conditions in which shear strains have reached 6 and 7% without failure. The failure mechanism of asphalt concrete mixtures appears to be dependent on temperature, state of stress, and strains. Therefore, a failure envelope can only be defined taking into account all those aspects.

The general aspect of the proposed failure envelope is presented in Figure 9, and is developed in the plane defined by the first invariant of the strain tensor, I_1, and the second invariant of the deviator strain tensor, $J2$. The definitions of the invariants, as well as that of the strain deviator, e, are respectively given by:

$$I_1 = \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33}$$

$$J_2 = \frac{1}{2}\left[e_{11}^2 + e_{22}^2 + e_{22}^2\right] + e_{12}^2 + e_{23}^2 + e_{31}^2$$

and

$$e_{ij} = \varepsilon_{ij} - \frac{1}{3} I_1 \delta_{ij}, \quad \delta = 1 \text{ if } i = j \text{ and } \delta_{ij} = 0 \textit{ otherwise}$$

As can be seen from the definitions, $I1$ is a measure of the volume change, while $J2$ indicates the amount of distortion. The importance of these parameters stems from the fact that they are independent of the particular coordinate system used. Consequently a universal envelope of failure can be developed that is valid, independent of the value of a specific component of the strain tensor in some coordinate system. The state of strains at a given material point at a given time is said not to have exhibited failure if the point (I_1, J_2) falls within the allowable zone.

It must be noted that the proposed failure envelope is not symmetric because it is proposed that when the volume decreases up to the point where void contents drop below 2-3% then the mixture rapidly loses shear strength as the asphalt acts as a lubricant between the aggregate particles.

Clearly the shape of this curve will depend on rate of loading and temperature. At cold temperatures and high loading rates the material failure is most likely controlled by the asphalt while at high temperatures and low loading rates the aggregate skeleton will most likely contribute to the shape of the failure envelope.

1.4 Constitutive Law and Simulations

The commonly used model to capture the above properties for asphalt is linear viscoelastic. In the present context the material could be modeled using a number of Maxwell elements to capture both the rate and temperature dependency. Furthermore, environmental effects (e.g., moisture damage, aging), can be included through the introduction of an isotropic damage model that degrades the asphalt elastic properties while keeping the characteristic times of each Maxwell element constant (note that this addition renders the model nonlinear).

The assumption that the aggregate response is rate independent is generally accepted and is well supported by experimental data (Sousa and Shewbridge, 1992). Furthermore aggregates are known to dilate. Repetitive constant height shear tests (SHRP A-003A Quarterly Reports) suggest that dilation is elastic in nature. This may explain the observation that. after an initial phase, ruts develop due to shear and without volume change. Additionally, aggregate properties were shown to depend on hydrostatic pressure and load history.

Therefore. a rate independent elastoplastic constitutive law appears to provide the best approach to model asphalt-aggregate mixtures. The dilation effect can be introduced through the elastic response. To this end, a nonlinear elastic response, emanating from a strain energy function (i.e., a hyperelastic model) that couples the volumetric and deviatoric response should be employed. The elastic model should also account for shear hardening under hydrostatic pressure and provide different elastic behavior in tension and compression.

The plastic model can be formulated in terms of the strain invariants (this amounts to the assumption that the aggregate is isotropic in the undeformed configuration), and should introduce the use of isotropic and kinematic hardening. Hardening was shown to play a crucial role in the development of permanent deformation in asphalt aggregate mixtures. The kinematic hardening can be used to account for different behavior in compression and tension, while the isotropic hardening can be used to expand the elastic region as a result of loading history. The plastic model should be decomposed into its volumetric and deviatoric components. The volumetric should account for densification as a result of loss of air void content, and the deviatoric portion should include hardening/softening due to air void content impact as shown in Figure 3 (note that this coupling is different than the dilation, which is introduced as a pure elastic behavior). The motivation for the inclusion of volumetric-deviatoric coupling effect in the plastic model is quite explicit. Consider the limit case of 0% air void content. In this case the binder will sustain the loading (akin to confined saturated soils or to consolidation in clays). Consequently, the aggregate influence is nominal, and the entire load resistance depends on the binder component in the mix.

Model

Based on the modeling efforts and validation presented by Sousa et al.(1993) it become clear that the proposed nonlinear viscoelastic model failed to provide a good representation of the asphalt-aggregate mixes' behavior under cyclic loading. Specifically, the model exhibited significant strain recovery during unloading, while test data suggested, at least for some of the mixes, significantly less recovery. Therefore the nonlinear viscoelastic constitutive law was enhanced to include a simple elastoplastic component (associative J2-plasticity with both isotropic and kinematic hardening) (Weissman and Jamjian, 1993).

It is customary in J_2-plasticity to take the hardening vector, **q**, as **q**:={a, **<O>q**} where a is the "equivalent plastic strain" that defines "isotropic hardening" and **<O>q** defines the center or "kinematic hardening" of the von Mises yield surface. The yield function, flow, and hardening rules are given by

$$f(\sigma, q) = \|\eta\| - \sqrt{\frac{2}{3}}\, K(\alpha)$$

$$\dot{\varepsilon}^p = \dot{\gamma}\frac{\eta}{\|\eta\|}$$

$$\dot{\bar{q}} = \dot{\gamma}\frac{2}{3}H'(\alpha)\frac{\eta}{\|\eta\|}$$

$$\dot{\alpha} = \dot{\gamma}\sqrt{\frac{2}{3}}$$

respectively.

In the above equations,

$$\eta := dev[\sigma] - \bar{q}, \quad tr[\bar{q}] := 0$$

and $H'(\alpha)$ and $K(\alpha)$ are the kinematic and isotropic hardening moduli, respectively. Finally, the following definition was adopted:

$H'(\alpha) := (1-\beta)H$ and $K(\alpha) := \sigma_Y + \beta H\alpha$ *with* β *within* [0,1] where σ_y, *H*, and β are material constants determined from the data.

Tests to determine material properties

The types of tests used to determine the material properties have been presented in Sousa et al. (1993) and Weissman and Jamjian (1994). They consisted of constant height shear creep, shear frequency sweeps at constant height, uniaxial strain, hydrostatic (volumetric) and repetitive simple shear at constant height. Based on the results of these tests, nonlinear elastic, viscous and plastic material properties were obtained for sixteen mixtures.

Maximum Permanent Shear Strain versus Rut Depth

The material constants obtained for the fourteen mixes were used in finite element simulations of a standard full depth, 15 in. thick, asphalt-aggregate pavement section (see Figure 10). The objectives were to establish a relationship between the maximum shear strain and the rut depth. To this end, the evolution of the rut depth with shear strain, for all fourteen mixes, is presented in Figure 11. It can be observed that a unique relationship appears to exist, for this pavement section, between maximum permanent shear strain and rut depth. That relationship is given by:

Rut Depth (in.) = Slope x Maximum Permanent Shear Strain

where:

Slope = 11

It must be noted that the material properties of the sixteen mixes vary from stiff mixes to very soft mixes. It is also interesting to note (see Figure 12) that the maximum shear strains occur below the edge of the tires (about 2 in. below the pavement surface). Figure 13 diagrams the deformed shape of a pavement section at the end of 300 cycles after the load has been removed.

It is of interest to investigate if that relationship would be affected by the thickness of the asphalt concrete layer. For selected mixes analysis were made for pavements with 4, 6, and 8 in. thick asphalt concrete layers. The subgrade modulus was changed to correspond to realistic pavement section conditions. It was further investigated if the relationship would hold true for asphalt concrete overlay placed over Portland cement concrete slabs. For selected mixes, analyses were made for overlay thicknesses of 4, 6 and 8 in. (see Figure 14). The results indicated that the slope of the relationship between maximum permanent shear strain and rut depth is thickness dependent (see Figure 15) and given by:

Slope = 0.74 * Thickness (in.)

It should be noted that this relationship was obtained for asphalt concrete over an elastic layer. It did not seem dependent on the stiffness of that elastic layer (either by modeling Portland cement concrete slabs or granular subgrade). It is suggested that thickness corrections are important when asphalt-aggregate mixes are used as overlays over Portland cement concrete but might not be applicable for analyses over aggregate layers which can also develop permanent deformation themselves. In this case an arbitrary pavement thickness might be used. Figure 12 indicates that significant reduction of shear stresses is encountered with depth. At about 15 in. within the pavement section reduced shear stresses are encountered.

These relationships seem to hold true regardless of the following:

- pavement temperature (simulated using different material properties),
- time of loading
- material properties (changing nonlinear elastic, viscous and plastic properties)
- tire pressure and load magnitude.

However they are affected by pavement thickness, relative stiffness of the pavement layers and probably by tire size and width. It should be considered that permanent deformation occurs mostly on hot days; therefore, relative stillnesses of layers associated with this condition should be used.

The establishment of these relationships is of significant importance in the development of an abridged permanent deformation procedure (Sousa and Solaimanian, 1994 and Sousa, 1994). For this purpose a flexible pavement depth of 15 in. might be used corresponding to a relationship between rut depth and maximum shear strain of 11. However if more accurate relationships are required the finite element methodology followed in this paper should be applied.

1.5 Basis for an Abridged Procedure

Although within a relatively short time accurate model predictions will be possible based on a comprehensive set of tests associated with a finite element methodology, currently that approach is not viablc for routine evaluation. Based on all the aspects of mix behavior described in this paper a procedure could be developed where a test is executed under conditions that best capture the mix behavior in the field and the results of the test are then related by statistical methods to observed field performance.

In such an approach tests should be executed in mixtures with known void content and their resistance to shear stresses evaluated by a procedure where the void content would remain unchanged throughout the test. The constant height repetitive simple shear test is ideally suited to evaluate the shear resistance of a mix at a given void content as the test is executed without volume change.

To evaluate the resistance of a mix at different void contents several specimens would be fabricated, each at a different void content, and their resistance to shear stresses evaluated. However, because mixtures placed in pavements tend to decrease air void content due to traffic densification, it is proposed that tests would only be executed at the void content corresponding to the maximum resistance.

Due to the plastic nature of the asphalt-aggregate mixes the test should be executed:
- at the temperature representative of the highest temperatures encountered in the pavement;
- at shear stresses representative of the highest applied to the pavement; and
- under repetitive conditions not only to simulate traffic but also because if creep loads were applied underestimation of the rutting propensity of a mix would occur.

Furthermore, specimens should be subjected to aging and moisture conditioning protocols corresponding to the region where the mixes will be placed. The procedures to evaluate permanent deformation of asphalt aggregate mixes presented by Sousa and Solaimanian (1994) and by Sousa (1994) obey all those criteria.

1.6 Summary and Conclusions

Permanent deformation of asphalt concrete mixes is a complex phenomenon that depends on the properties of the binder and of the aggregate structure. Modeling efforts should recognize that mixes have a rate and temperature dependent behavior, they dilate (i.e., there is volumetric deviatoric coupling in the sense that deviatoric stresses lead to volume/pressure change, and increased hydrostatic pressure leads to deviatoric stiffening), they exhibit different properties in tension and compression, under cyclic loading residual deformations are observed at the end of loading cycles, their behavior is strongly dependent on air void contents, when aged they loose fluidity which can play an important role in development of permanent deformations and in some mixes moisture damage also plays an important role in development of permanent deformation.

Modeling efforts have been made to capture most of those aspects of mix behavior and simulations of pavement sections were made with different mixes and pavement structures. Although not all aspects of asphalt mix behavior are captured by the model the most significant ones are. The analysis indicates that a relationship between rut depth and maximum shear strain exists as a function of pavement thickness, relative stiffness of the pavement layers and probably tire size (this aspect was not investigated).

Currently standard mix design cannot be based on a compressive set of tests to determine all the fundamental material properties that are related with the permanent deformation phenomenon and its performance predicted based on finite element programs. The paper suggests some of the most important aspects a test and test protocols necessary to develop an abridged procedure that could be implemented for routine use. Based on all the aspects of mix behavior described, a procedure could be developed, in which a test is executed under conditions that best capture the mix behavior in the field, and the results of the test are then related, by statistical methods, to observed field performance.

However, within a few years, given the rate at which computing speed has been growing, it can be expected that a comprehensive permanent deformation predictive capability can be implemented base on fundamental material properties such as those presented in this paper.

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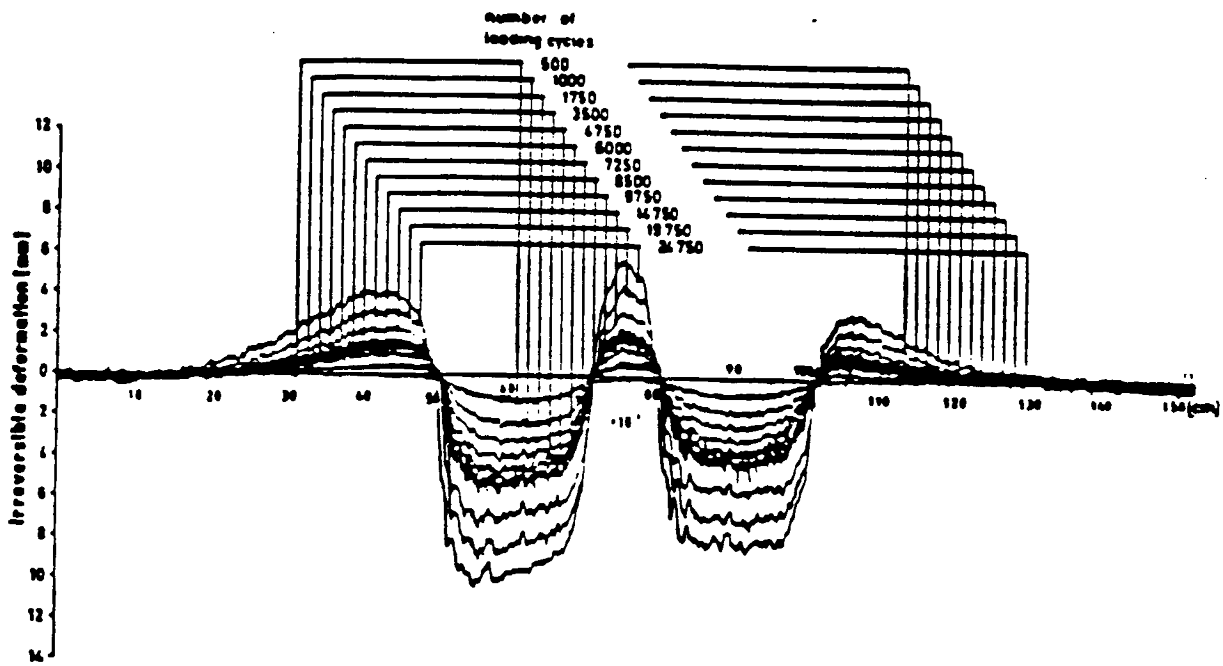

Figure 1 - Effect of number of passes on transverse surface profile (after Eisenmann and Hilmer, 1987).

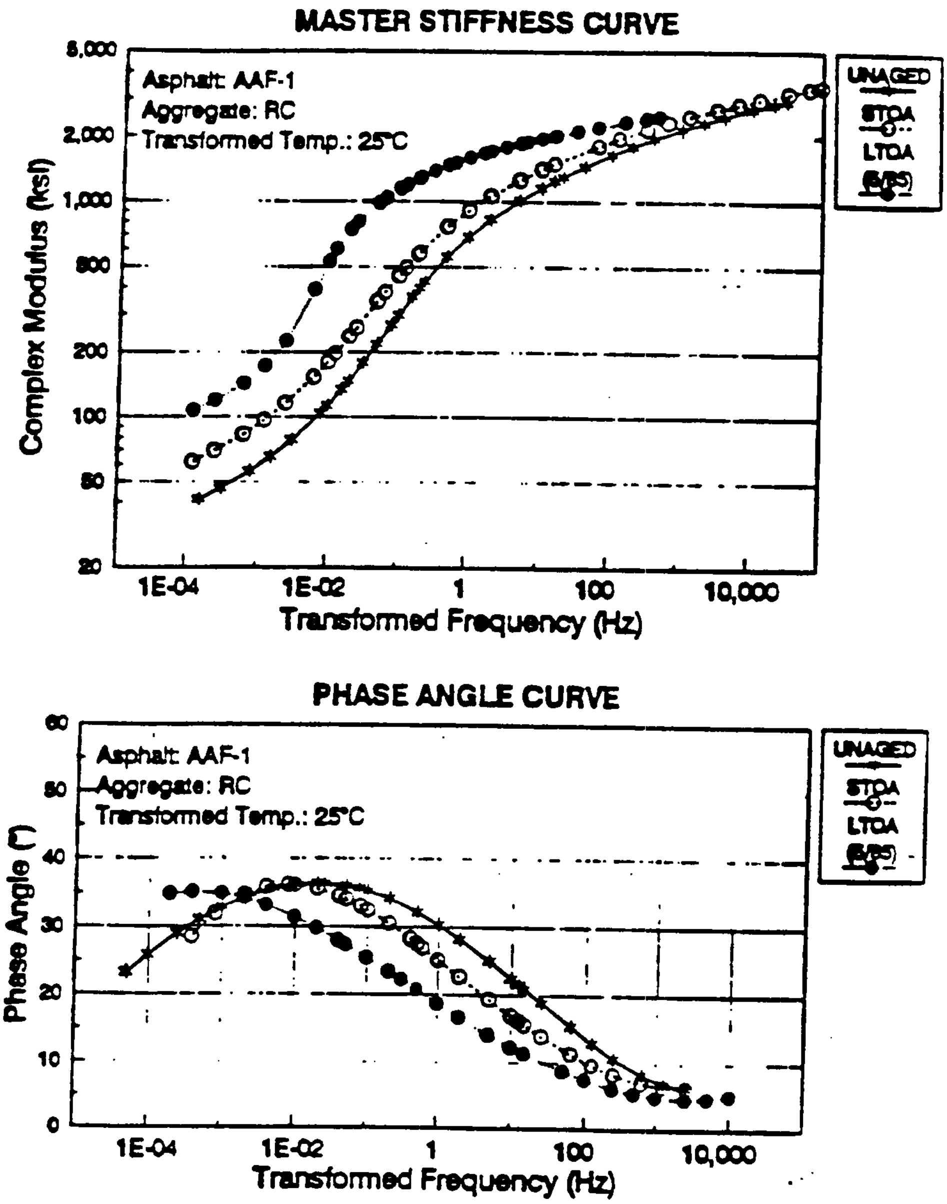

Figure 2 - Effect of aging (short term and long term) on the master curve and phase angle for an asphalt mix.

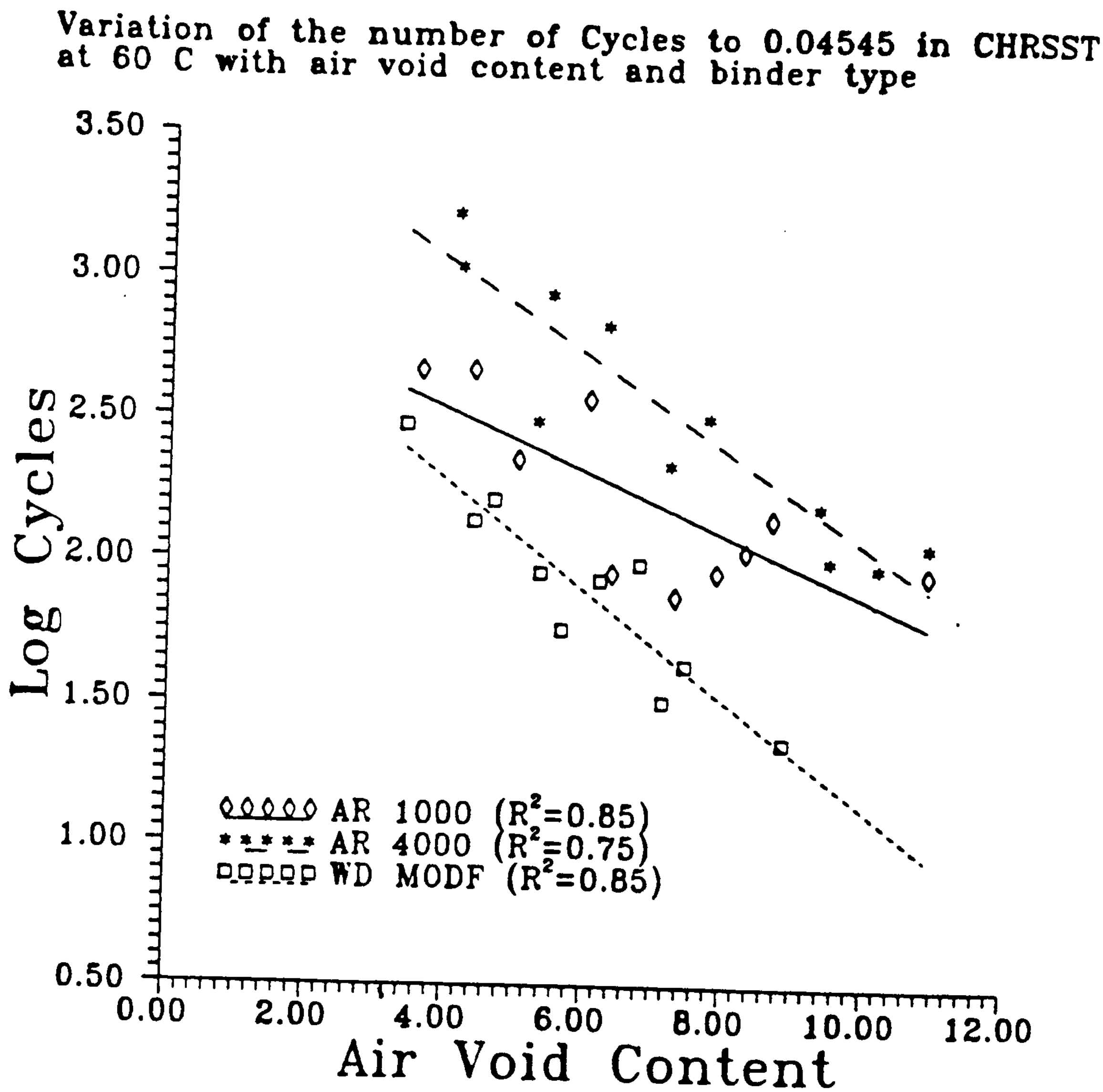

Figure 3 - Effect of air void content on the accumulation of permanent deformation for three different mixes.

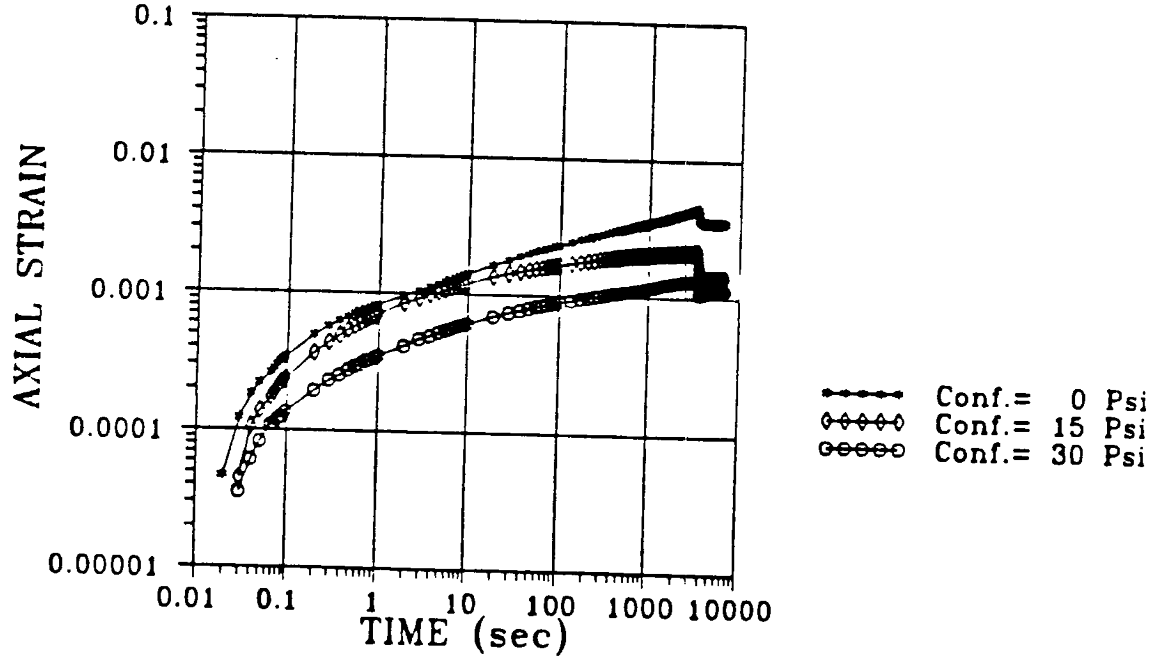

Figure 4 - Effect of confining pressure on the axial creep response of a mix at 40 C.

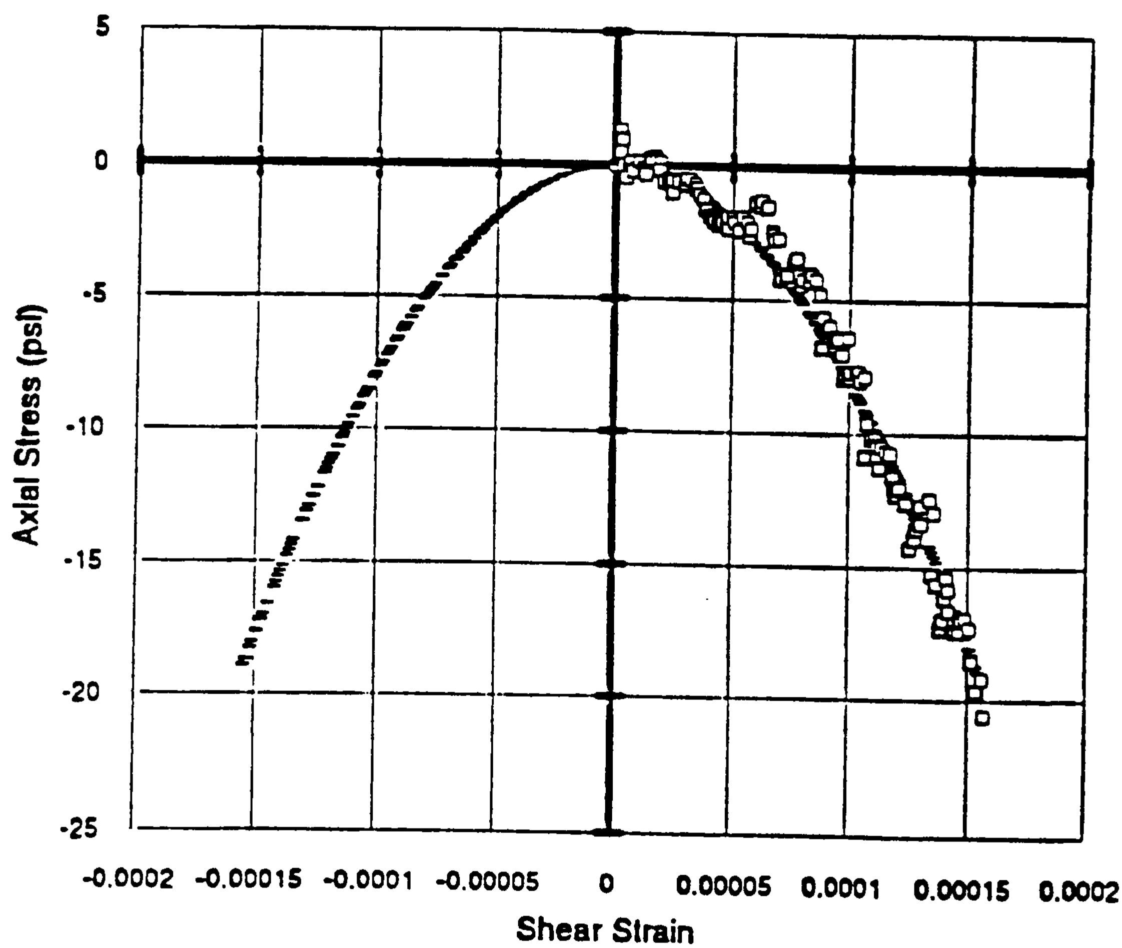

Figure 5 - Variation of the axial stress with shear strain in a constant height simple shear creep test.

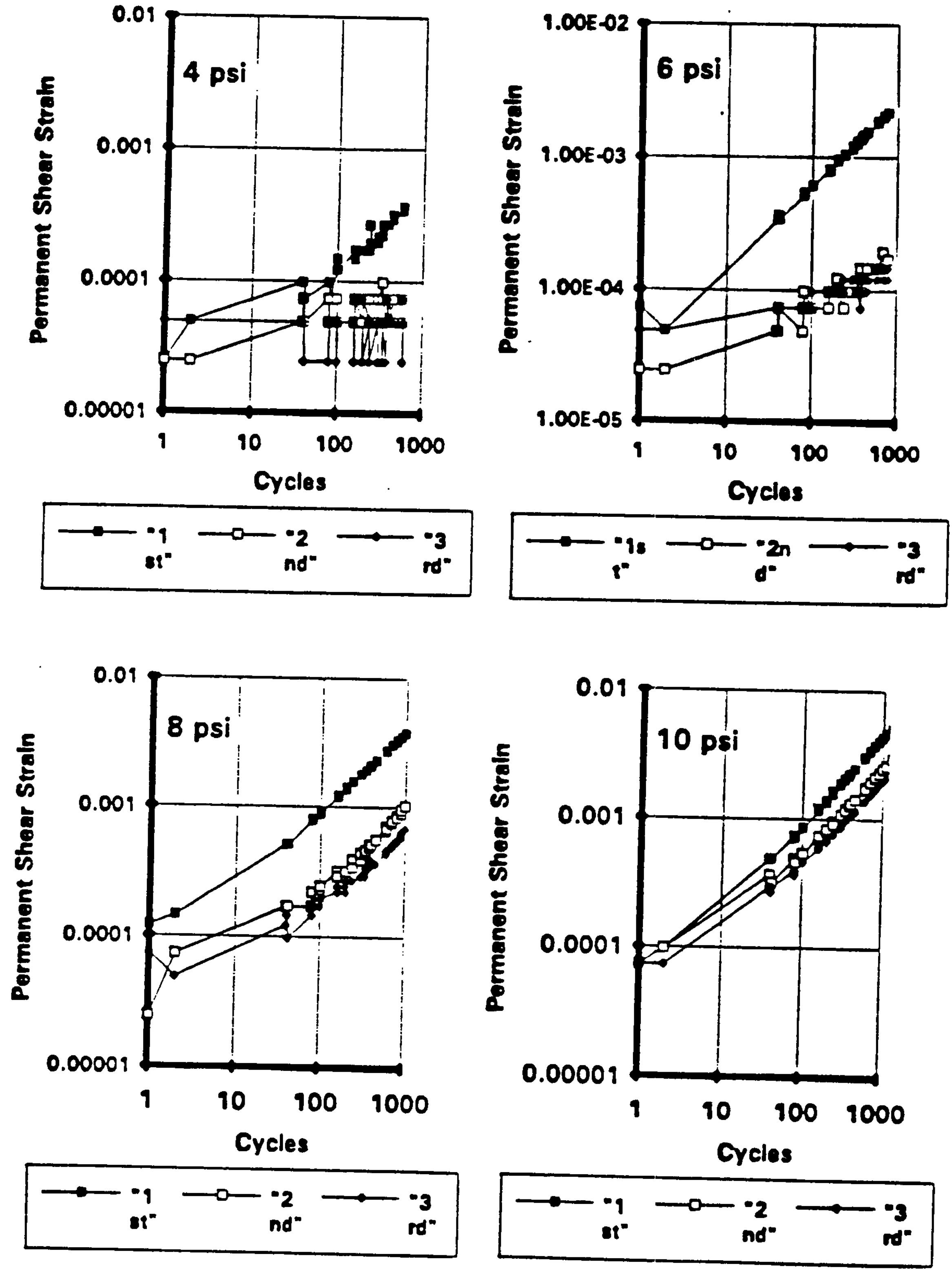

Figure 6 - Results from sequences of tests (RSST-CH) at different stress levels.

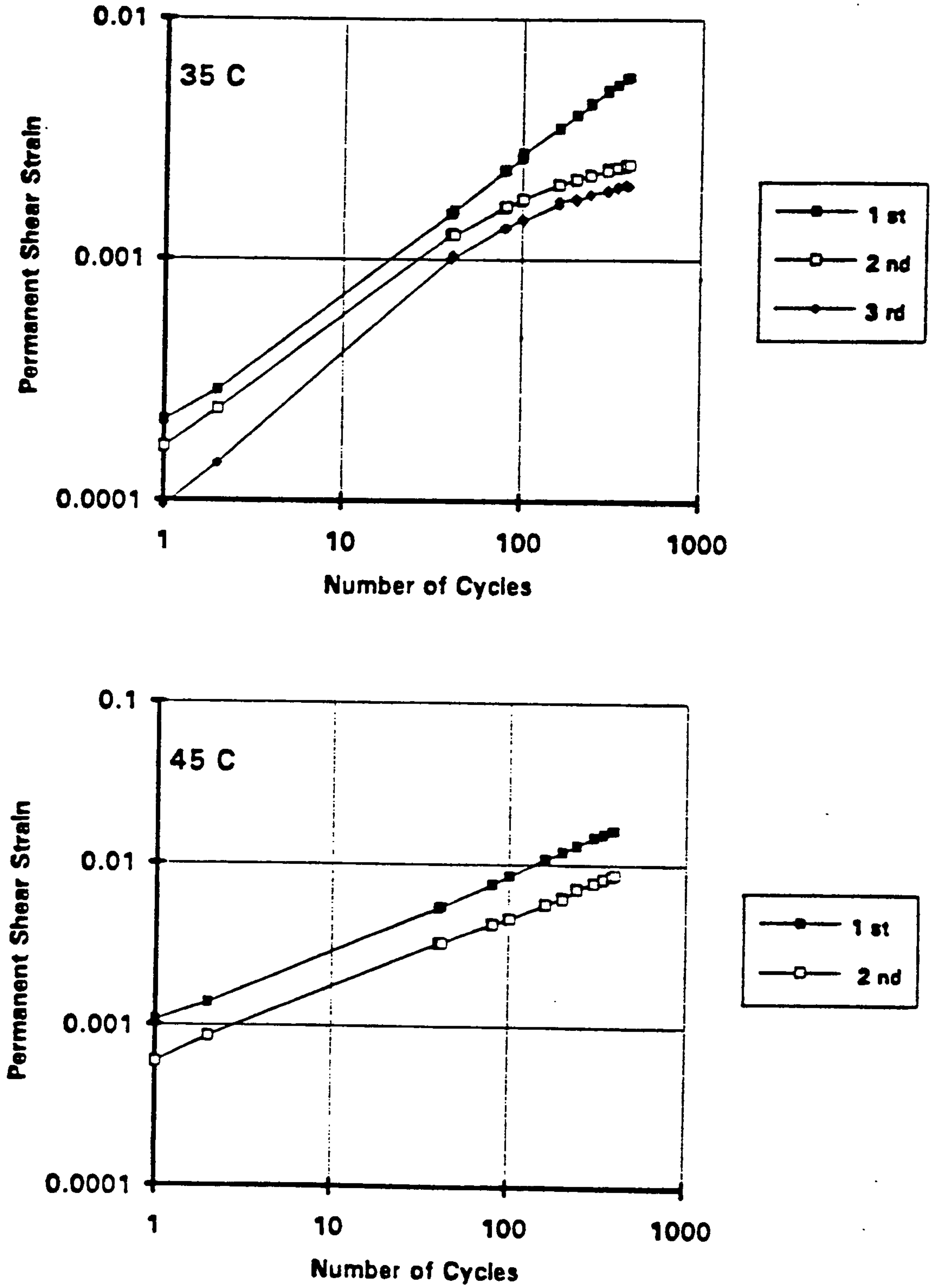

Figure 7 - Results from sequences of tests (RSST-CH) at different temperatures.

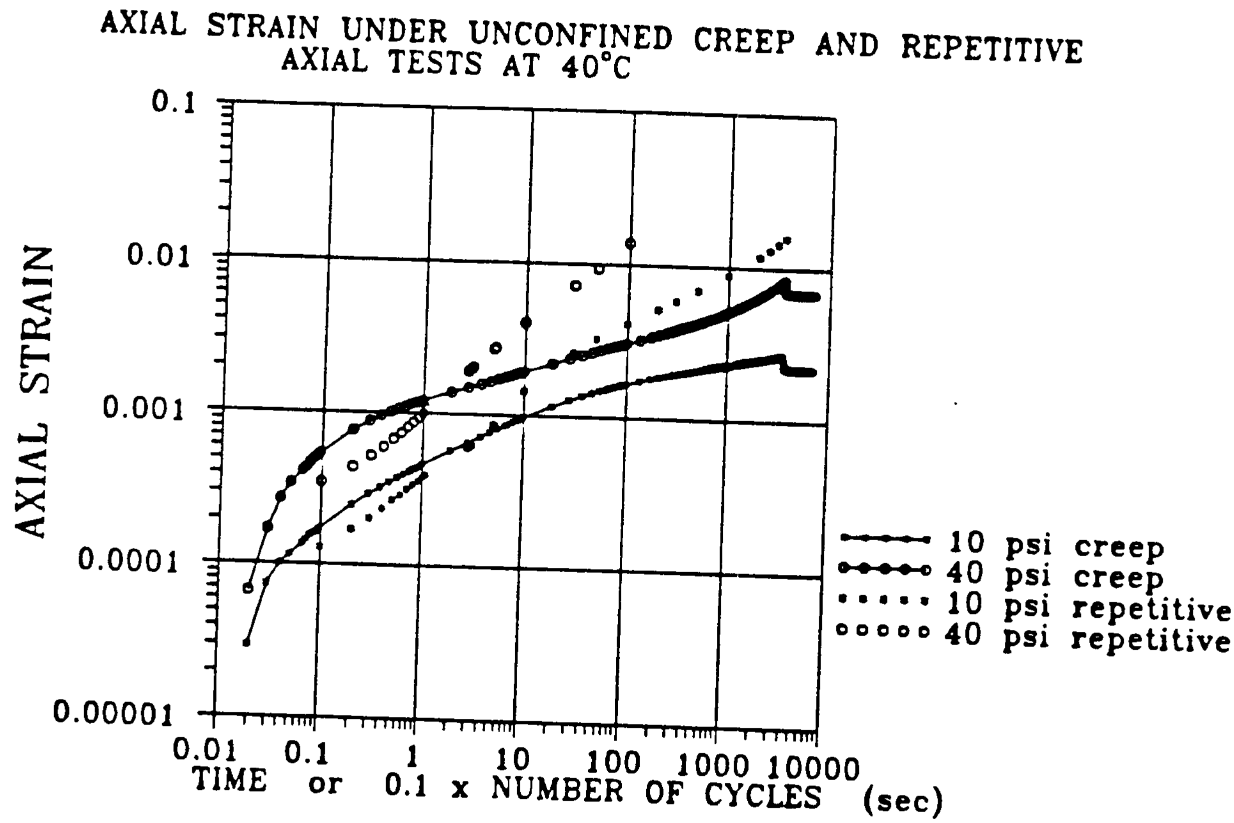

Figure 8 - Comparison between axial repetitive and axial creep behavior.

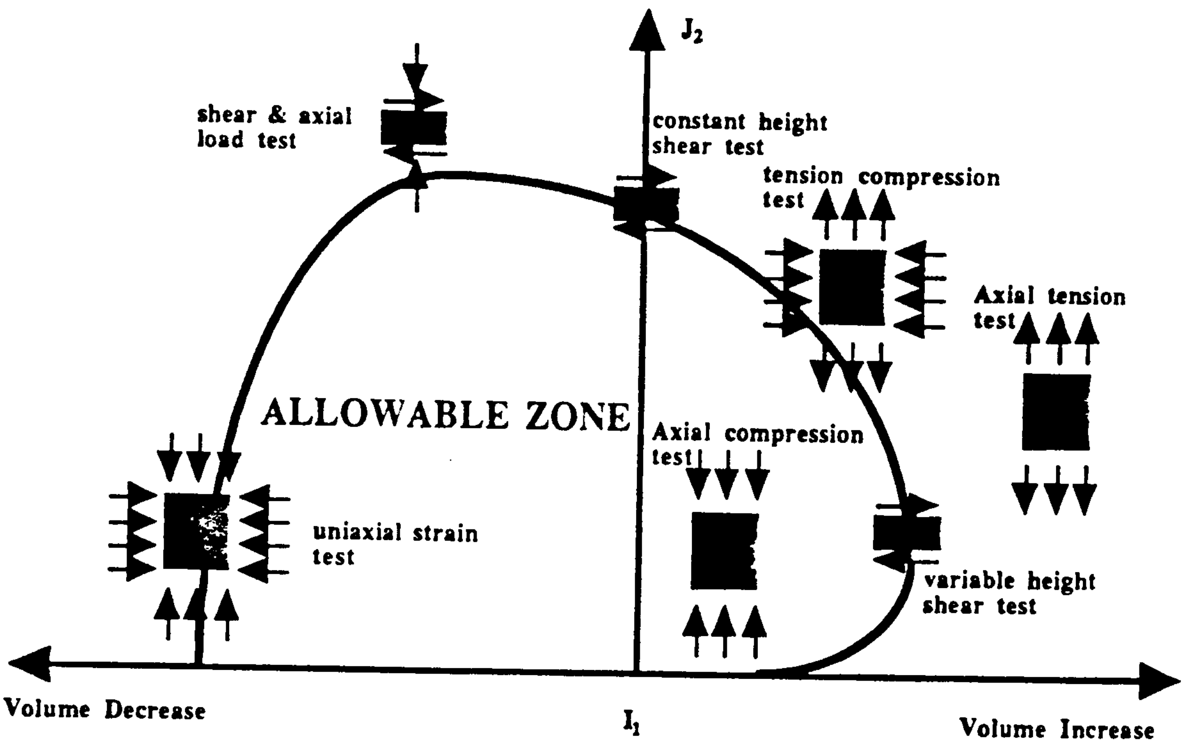

Figure 9 - Conceptual failure envelope for asphalt-aggregate mixes and associated tests required for its determination.

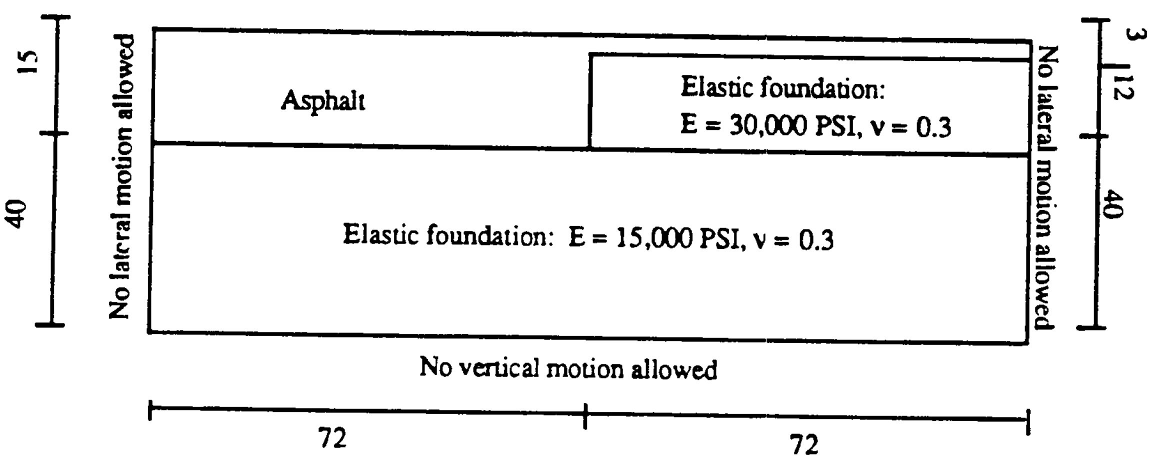

Figure 10 - Schematic view of cross section of 15 in. thick pavement.

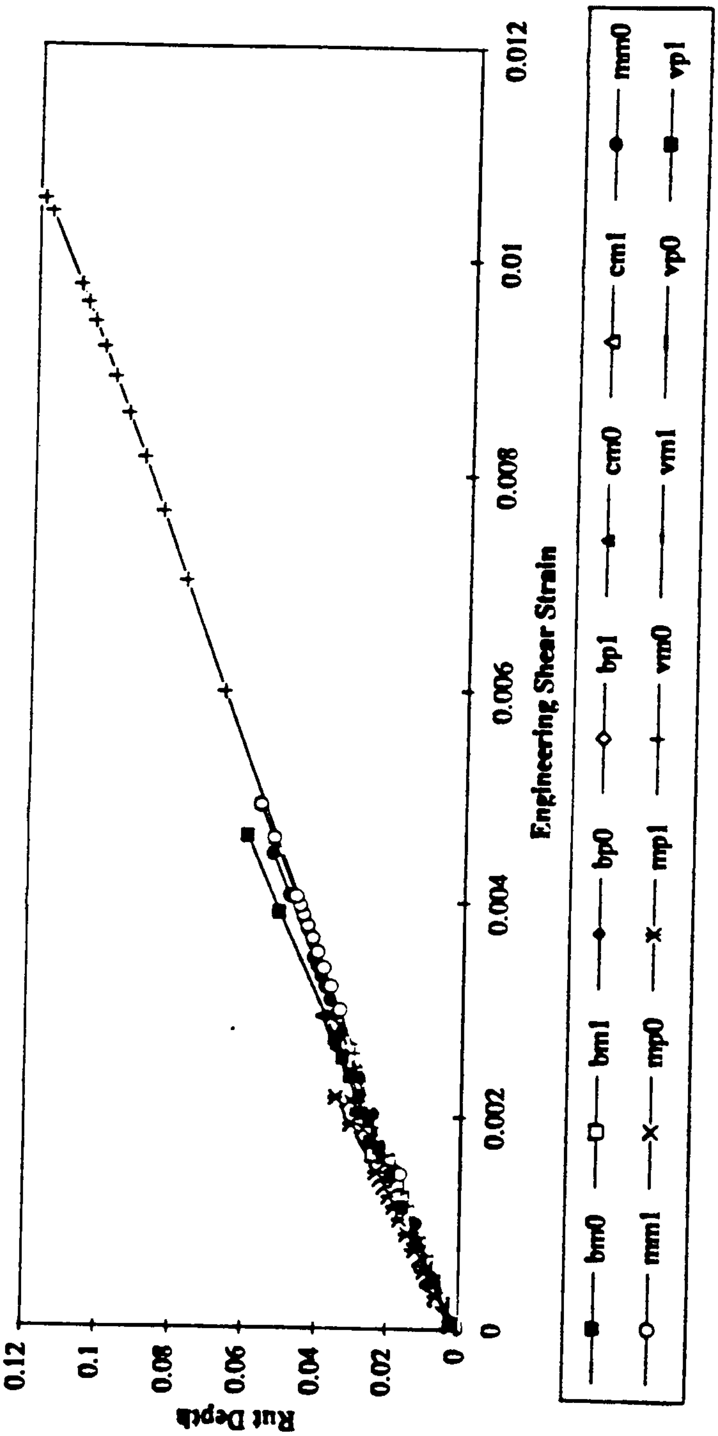

Figure 11- Variation of rut depth with permanent shear strain for 14 mixes.

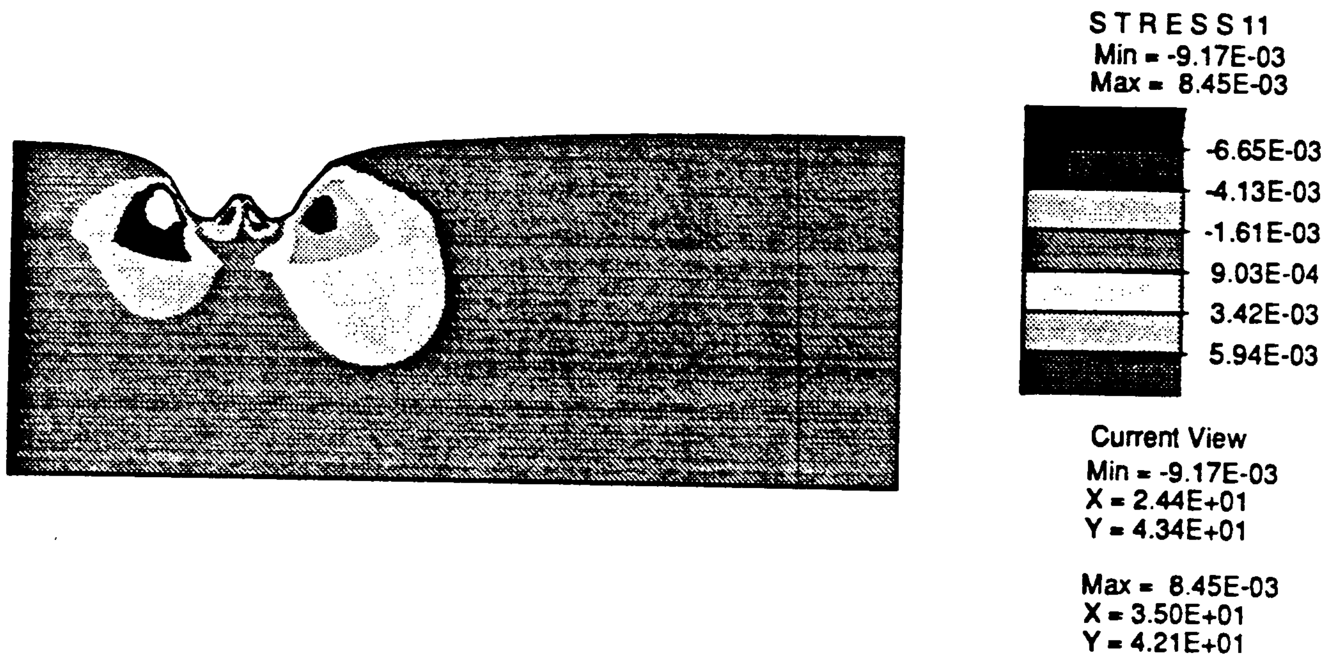

Figure 12 - Variation of shear strains within the pavement section at the beginning of the 300th cycle with the load applied. Vertical deformations amplified 50 times.

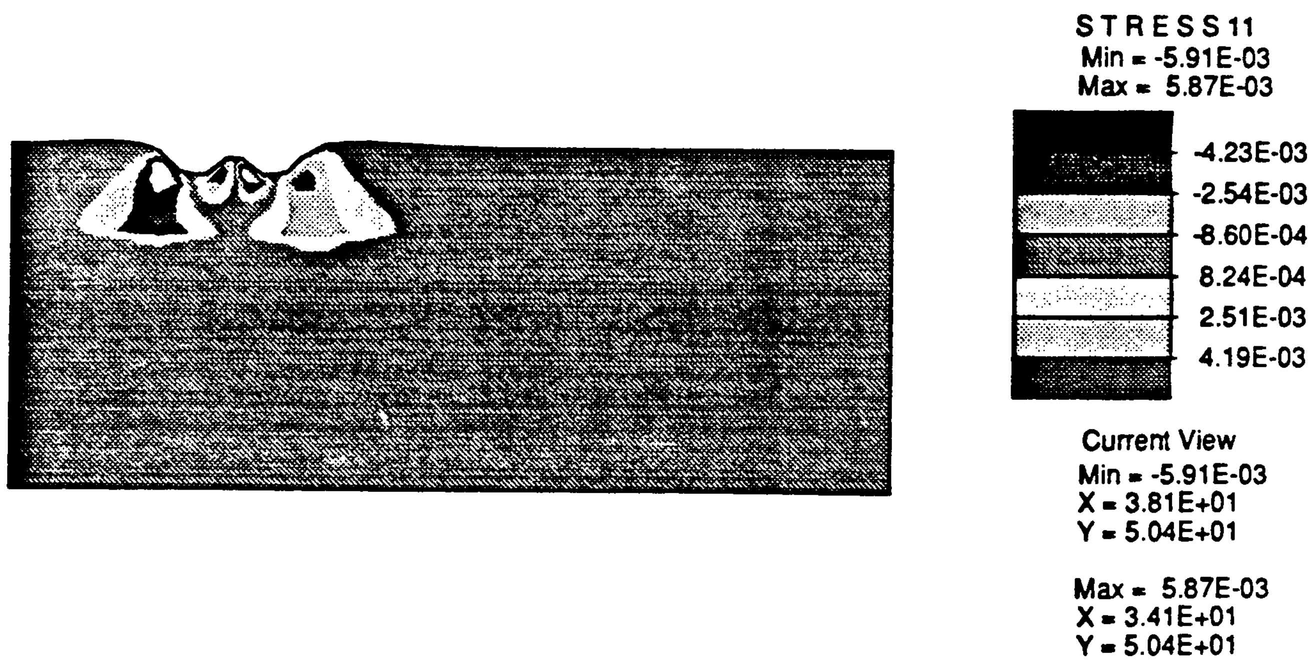

Figure 13 - Variation of residual shear strains within the pavement section at the end of the 300th cycle with the load removed. Vertical deformations amplified 50 times.

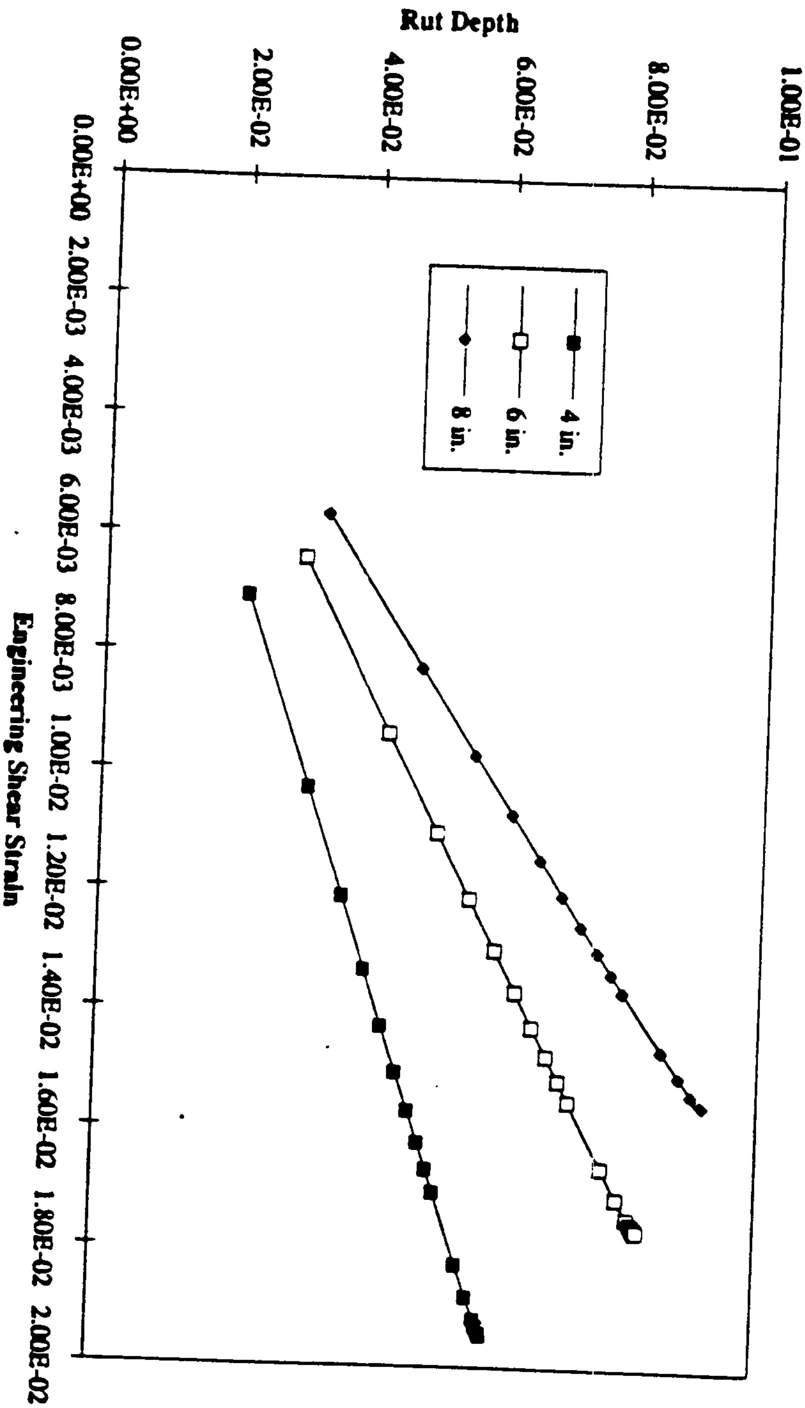

Figure 14 - Variation of rut depth with maximum shear strain for different pavement thicknesses.

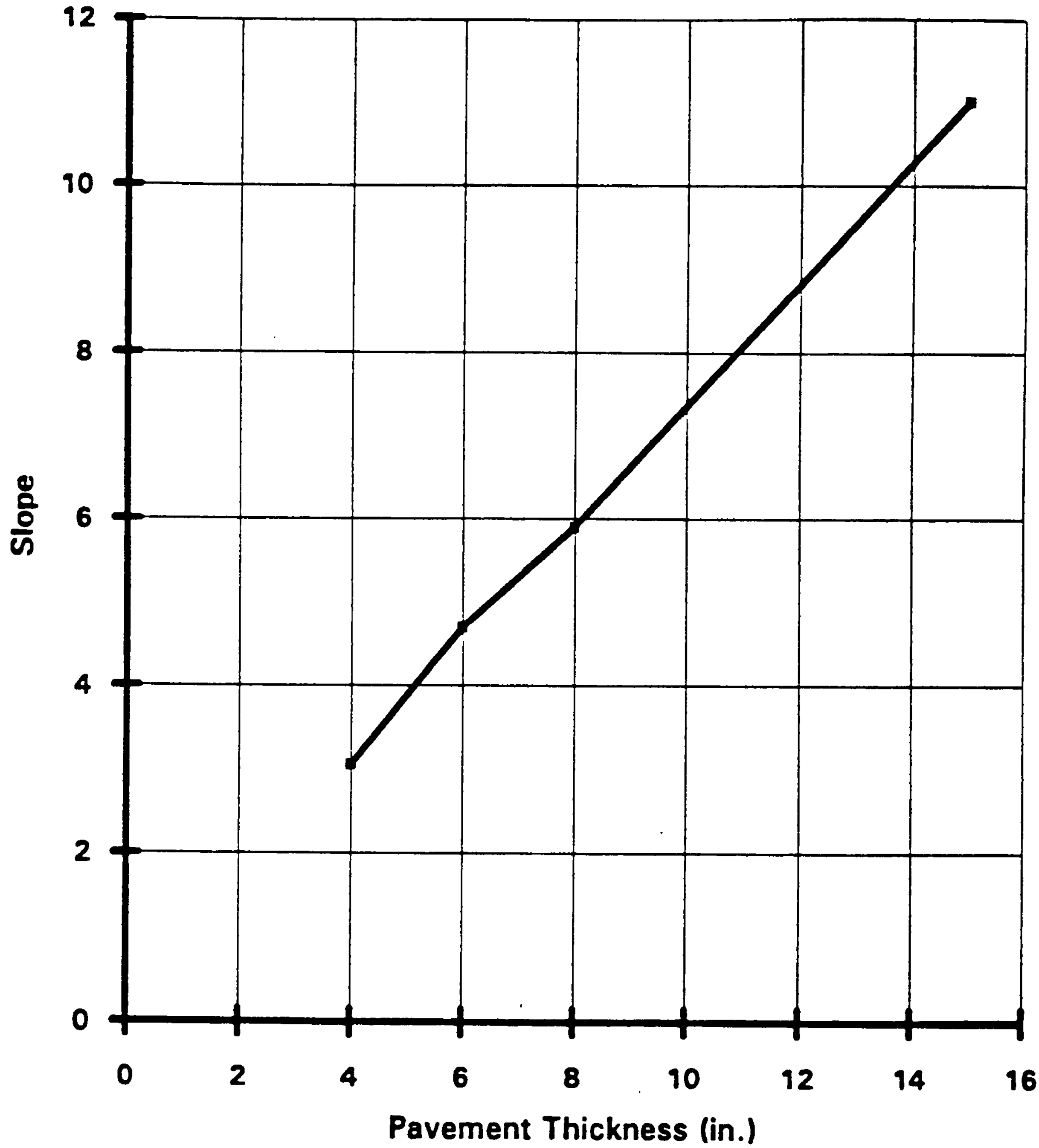

Figure 15 - Variation of the slope of the relationship between maximum permanent shear strain and rut depth with pavement thickness.

2

Abridged Procedure to Determine Permanent Deformation of Asphalt Concrete Pavements

2.1 Introduction

This paper presents a procedure that could be used for rapid evaluation and screening of asphalt aggregate mixes. It could also be adopted as a procedure to evaluate rutting propensity of a mix taking into consideration traffic level (using ESALs) and the pavement location.

The underlying assumption in this approach is the fact that permanent deformation is primarily a plastic shear flow phenomenon at constant volume, occurring near the pavement surface, caused by the shear stresses appearing below the edge of the truck tires (Sousa, Crauss and Monismith, 1991).

It is also intrinsically linked to this procedure the assumption that most of the permanent deformation occurs in hottest days and mostly due to the heaviest trucks. This assumption stems from observations in laboratory that asphalt concrete mixes exhibit strong plastic behavior described by a plasticity function that exhibits kinematic hardening. This hardening seems to be associated with the capability of the mix to develop better particle to particle contact as it develops shear strains and with the capability of the aggregate skeleton to develop dilatancy forces that in turn are capable of developing stabilizing confining stresses.

This phenomenon seems to be best captured by the Repetitive Simple Shear Test at Constant Height (RSST-CH) executed at the mean highest average 7-day maximum pavement temperature at 2 in depth. The test is executed using two actuators. One controls the magnitude of the applied shear stresses while the other insures that the specimen is tested under a strain control boundary condition by maintaining the height of the specimen constant (within an acceptable margin of error).

This procedure was developed based on data reported from road sections of the General Pavement Studies (GPS) under SHRP Long Term Pavement Performance (LTPP) and from data from Colorado sites. With a relatively low level of effort other countries and states could validate this procedure for their specific local conditions.

The major drawback of the procedure at this time is the incapability of directly incorporating the effect of tire pressure and load magnitude. This effect can only be brought into the analysis indirectly through the computation of the Equivalent Single Axle Loads (ESALs). However these equivalency factors could be accurately computed using the Permanent Deformation Model and the Finite Element Methodology proposed by A-003A (Sousa, Weissman, Sackman and Monismith 1993).

2.2 Basis for the Development of the Procedure

2.2.1 Model Analysis

A series of finite element analyses of the permanent deformation response of the pavement section shown in Figure 16 was conducted using the model proposed by Sousa, Weissman, Sackman and Monismith (1993). The model is intended to capture the macro-behavior of mixes including: 1) the shear dilatancy observed when the mix is subjected to shear strains; 2) the increase of effective shear modulus under increased confining pressure; 3) the significant variation of behavior with changes in temperature and rates of loading; and 4) the residual accumulation of permanent deformation under repetitive loading. The material properties were obtained from a series of volumetric, uniaxial, shear and frequency sweep tests. In these analyses, only the non-linear elastic and viscous properties of the mix were incorporated into the constitutive relationship. The purpose was to investigate the relationship between tire pressure, rut depth and permanent shear and axial strains.

Two stress levels for the tire loading, 200 and 500 psi, were used and were applied as a pulse loading with a duration of 0.3 sec and a time interval between pulses of 0.4 sec.. The conditions of high tire pressure and relatively long loading time were selected so that large ruts and the associated large permanent strains could be obtained within relatively few loading cycles. The magnified deformed finite element mesh is shown in Figure 17 at the end of the second load cycle for the 500 psi tire loading condition. Figures 18 and 19 show the changes in pavement profile with the load applications for the 500 psi and 200 psi conditions and for the 200 psi conditions using only the linear terms in the strain energy function (i.e. only linear viscoelastic behavior was modeled). For the 500 psi loading, it will be noted that a considerable upheaval of the pavement surface occurs between the tires. For the 200 psi, this upheaval is less pronounced (even less when the non-linear terms are ignored). The difference may be due in part to the fact that the magnitude of the elastic strain is smaller for the 200 psi loading. Sousa et al. (1993) have demonstrated that dilation exhibits a nonlinear dependence on the magnitude of the shear strain; essentially a nonlinear increase in the dilation is observed with the increase in shear strain.

Figure 20 suggests that there may be a linear relationship between rut depth and maximum permanent shear strain. (N.B. some of the nonlinearity at the larger rut depths was due to instability in the computation procedure. This was later corrected by selecting the appropriate time step). It is also important to note that the magnitude of the shear strain is larger than the magnitude of the axial strain as it can be seen in Figure 20.

After this analysis was made, it was decided to incorporate a plastic component into the permanent deformation model. With initial material characteristics for mixes containing eight SHRP asphalts and two SHRP aggregate with air void contents varying between 3% and 8%, a series of analyses were performed for a pavement structure with a shoulder. In this case a 100 psi tire pressure and a load duration of 0.01 sec and a time interval between load applications of 0.06 sec. were utilized. The variation of rut depth with shear strain is illustrated in Figure 21. A linear relationship between permanent shear strain and rut depth is present. This relationship is identical to that presented in Figure 20. These computer runs, with materials properties obtained for the 16 mixes, was executed by Symplectic Engineering Inc. The relationship that best fits all this cases, specially for rut depths above 0.02 in. is given by:

Rut Depth (in.) (r*dp*) = 11 * Maximum Permanent Shear Strain (*mpss*) *(1)*

This relationship seems to hold true regardless of:
- pavement temperature (based on simulations using different material properties),
- time of loading (based on simulations using 0.3 sec. ON and 0.4 sec. OFF , 0.1 sec. ON and 0.6 sec. OFF and 0.01 ON and 0.06 OFF)
- material properties (changing nonlinear elastic, viscous and plastic properties)
- tire pressure and load magnitude (100, 200 and 500 psi were used for a given tire size).

However equation *(1)* should be further validated for different pavement types, thicknesses and for non-uniform variation of material properties.

These results, are quite interesting, as it is to be expected that the rut depth (which is in some form the resultant of the accumulation of all permanent strains within a pavement section) be related to the maximum permanent shear strain (which is the principal strain causing rutting). The relationship should be only dependent of pavement geometry and loading geometry. Loading geometry (i.e. relationship between tire dimensions and pavement dimension) is basically the same for most pavements. Pavement geometry varies considerably. However most of the rutting develops near the pavement surface. It is expected that pavement thickness will only play a role up to some value beyond which that relationship will hold basically the same. These two points should be further investigated.

2.2.2 Field Data

Field data from the GPS sections were obtained from the SHRP A-005 project. It consisted of site number, date opened to traffic, rut depth measurement, date of rut depth measurement and ESALs up to the time of the rut depth measurement for each GPS site. Tables 1 contains a summary of relevant data. Figure 22 shows a scatter plot of rut depth versus ESAL. The rut depth was measured using either the Pasco Equipment, the Dynatest dipstick or a 4 ft. straight edge.

2.2.3 Temperature Analysis

SHRP binder/mix specifications are developed based on maximum and minimum pavement temperatures. Maximum pavement temperature was defined as the average maximum temperature for seven consecutive days. It is believed that rutting correlates better with this temperature than with mean monthly maximum or average yearly maximum pavement temperatures.

Determination of Mean Highest 7-Day Maximum Air Temperature

The sites of the General Pavement Studies cover diverse environmental conditions. The maximum pavement temperature for these sites varies within a wide range. In order to calculate the maximum pavement temperature for the GPS sites two or three nearest weather stations to each site were selected. The weather stations with more than 20 years of record were included. For each year, the average 7-day maximum temperature was calculated based on the following procedure. The maximum temperature for each day of the year is determined. Then, the maximum daily temperature for seven consecutive days are added together and the result is divided by seven. This way, an average 7-day maximum temperature is obtained. All the average 7-day maximum temperatures during the hot days are determined, and the largest number obtained this way is recorded as the highest averaged 7-day maximum temperature for that particular year. The procedure is repeated for all the years for which records are available. For example, if 30 years of data are available for one station, 30 numbers will be obtained this way. The average value of these 30 numbers will be recorded as the mean highest average 7-day maximum temperature. This last number was the value used in the calculations.

Determination of Pavement Temperature

The pavement surface temperature was then calculated using the following formula, which was developed based on the energy balance at the surface (Solaimanian and Kennedy, 1993):

$$422\ \alpha\ \tau_{\alpha}^{\ 1/\cos z} \cos z + 0.7\ \sigma\ T_a^4 - h_c\ (T_S - T_a) - 90k - \varepsilon\sigma\ Ts^4 = 0 \qquad (2)$$

Where

- z = zenith angle (approximately z = latitude-20 for May through August)
- τ_{α} = sunshine factor (0.81 for perfectly sunny conditions)
- α = solar absorptivity (default : 0.9)
- σ = Stefan-Boltzman Constant (0.1714 E-8 Btu/(hr.ft^2.R4
- h_c = surface coefficient of heat transfer (default = 3.5 Btu/(hr.ft^2.F)
- k = thermal conductivity (default: 0.8 Btu/(hr.ft^2.F)/ft)
- ε = surface emissivity (default: 0.9)
- T_a = maximum air temperature (Rankine)
- T_s = maximum pavement surface temperature (Rankine)

Once the maximum pavement temperature at the surface is found through the preceding formula, the maximum pavement temperature for any depth less than 8 inches is found through the following empirical formula (Solaimanian and Kennedy, 1993)

$$T_d = T_s\ (1 - 0.063\mathbf{d} + 0.007\mathbf{d}^2 - 0.0004\ \mathbf{d}^3) \qquad (3)$$

where **d** is the depth in inches, T_S is the maximum pavement temperature (°F) at the surface and T_d is the maximum pavement temperature (°F) at depth **d**.

It seems that most of the permanent deformation due to the shear stresses developing near the edge of the tires takes place at depths up to 4 inches. The maximum shear stress computed from non-linear visco-elastic analysis took place at about 2 inches. For this reason and also because at this depth the ranges of temperatures computed for the GPS sections fell within reasonable testing ranges, the maximum pavement temperature at 2 in depth was selected as the testing temperature for each of the GPS sections.

2.2.4 Laboratory tests

2.2.4.1 A Test Selection

Rutting (permanent deformation) in an asphalt-concrete layer is caused by a combination of densification (volume change) and shear deformations, each resulting from repetitive application of traffic loads. For properly compacted pavements, shear deformations, caused primarily by large shear stresses in the upper portions of the asphalt-aggregate layer(s), are dominant. Repetitive loading in shear is required in order to accurately measure, in the laboratory, the influence of mix composition on resistance to permanent deformation. Because the rate at which permanent deformation accumulates increases rapidly with higher temperatures, laboratory testing must be conducted at temperatures simulating the highest levels expected in the paving mix in service.

To predict permanent deformation, laboratory tests must be capable of measuring properties under states of stress that are encountered within the entire rutting zone, particularly near the pavement surface. Since there are an infinite number of states of stress it is impossible to simulate them all with a single test given the non-linear and viscous behavior of the material. For this reason several tests have been proposed to determine a constitutive law for asphalt concrete (Sousa et al. 1993). However, if a single test is to be performed to rapidly screen and evaluate the resistance of various mixes to permanent deformation, then that test should be sensitive to the most important aspects of mix behavior and executed under conditions that most significantly affect that behavior.

The repetitive simple shear test at constant height on 6 in. (0.15 m) diameter by 2 in. (0.05 m) height cylindrical specimens is proposed as an effective test to evaluate the rutting propensity of a mix. Several reasons lead to the selection of this test:

1. *Specimen Geometry*: a) A 6 in. diameter by 2 in high specimen can easily be obtained from any pavement section by coring, or from any compaction method (i.e. gyratory, rolling wheel, kneading, etc.); b) the state of stress is relatively uniform for the loads applied; c) the magnitude of loads required in testing such specimens is easily achievable by hydraulic equipment. If large stone mixes have to be studied, then 8 in. diameter by 3 in. high specimens should be used.

2. *Rotation of Principal Axes*: It is the simplest test that permits controlled rotation of principal axes of strain and stress which are important in studying rutting.

3. *Repetitively Applied Loads*: Studies have indicated that to capture the rutting phenomena, application of repetitive loads is required given the viscous nature of the binder (mixes behave differently at different loading rates) and also the granular nature of the aggregate (aggregates behave differently under static and repetitive loads).

4. *Dilation*: One of the most important aspects controlling the stability of a mix is dilation. Under shear strains densely compacted mixes tend to dilate (just like dense sands). If dilation is constrained (as it is in the pavement to some degree by the adjacent material) then confining stresses are generated. It is in part due to the development of these confining stresses that a mix derives its stability against shear strains. Mixes with little tendency to dilate will have a higher propensity for rutting. In the constant height simple shear test the development of axial stresses

is fully dependent on the dilatancy characteristics of a mix. As permanent shear strains increase the mix will develop (due to dilation) more or less axial stresses depending on the aggregate type, structure, texture and void level. The axial stresses developed this way tend to stabilize the mix. In this test configuration a mix will resist permanent deformation either by relying on high binder stiffness to minimize shear strains or by aggregate structure stability imparted by the development of axial stresses due to dilation. These two mechanisms are the most important ones that provide resistance to permanent deformation in a mix (mixes with some modified binders can have additional dilation forces caused by modifier dilatancy resulting from shear strain rates). A well compacted mix with a good granular aggregate will develop high axial forces at very small shear strain levels. Poorly compacted mixes can also generate similar levels of axial stresses but they will require much higher shear strains.

Stiffer binders help in resisting permanent deformation as the magnitude of the shear strains is reduced under each load application. The rate of accumulation of permanent deformation is strongly related to the magnitude of the shear strains. Therefore, a stiffer asphalt will improve rutting resistance as it minimizes shear strains in the aggregate skeleton.

In the constant height simple shear test these two mechanisms are free to fully develop their relative contribution to the resistance of permanent deformation as there are not constrained by imposed axial or confining stresses. This might be one of the reasons why this test is so discriminating for asphalt-aggregate mixes.

Recognizing that pavement rutting is predominantly a shear flow phenomenon, it seems reasonable to impose shear stresses and allow the material to develop its own axial stress (representing to some extent the in-situ conditions where dilation is restrained by adjacent material).

2.2.4.2 Test Procedure

The execution of a repetitive simple shear test at constant height required the design and fabrication of a totally new equipment. Taking into consideration that this test would be executed on a routine basis efforts were made to insure the easiest possible interface with the user.

The testing system fabricated by Cox & Sons, Inc. Colfax, California, as been presented by Sousa, Tayebali et al. 1993. It basically consists of two orthogonal tables which are mounted on bearings. The tables are connected to two hydraulic actuators which are controlled using servo-valves under feedback closed-loop digital algorithms. To insure that the shear and axial forces are transmitted to the specimen, aluminum caps are glued to the parallel faces of the specimen. A gluing device was also developed by Cox and Sons, Inc. to insure that the caps faces are glued parallel. New hydraulic clamps insure an easy interface with the user by eliminating the need to use tools to fasten the specimens to the moving tables.

This equipment can accommodate several specimen sizes but for permanent deformation evaluation the recommended specimen size for shear testing is a cylinder with 6 in. diameter by 2 in height. If large stone mixes are to be tested the recommended specimen size is 8 in. diameter by 3 in. height.

To execute a repetitive simple shear tests at constant height the vertical actuator maintains the height of the specimen constant using as feedback the output of an LVDT measuring the relative displacement between the specimen caps. The horizontal actuator under control by the shear load cell applies haversine loads corresponding to a 10 psi shear stress magnitude with a 0.1 sec loading time and 0.6 sec rest period.

Experience with a wide range of mixes tested at different temperatures and stress levels demonstrated that the 10 psi shear stress magnitude was a reasonable level at which good mixes would exhibit some permanent deformation while poor mixes would not fail excessively fast. Finite element computations have shown that critical shear stress levels in the field might be around 20 to 25 psi. Associated with these shear stresses confining pressures of about 30 psi and axial stresses of about 80 psi were also found. However, no lateral confinement is applied during this laboratory test.

Tests were executed until 5% shear strain was reached or up to 5000 cycles. Prior to testing specimens were conditioned with 100 cycles of 1 psi haversine loading with a 0.1 sec. loading and 0.6 sec. rest period. The purpose of this preconditioning was instrumentation setup. Tests can be executed at any temperature. For this study the test temperature varied according to the geographic location of the pavement site.

Specimen Preparation

Cores were obtained from various GPS sites to cover a wide range of environmental conditions. A total of 40 sites were included in this study. Two inch thick specimens were cut out of these field cores with a double-blade saw. Efforts were made to cut specimens from the area between one to three inches below the surface. Then, the specific gravities of the specimens were determined using parafilm. The specimens were let to dry before being glued to the caps. A DEVCON 5 minute plastic steel putty was used to glue the specimens to the caps, which was let to cure for several hours before testing.

Each specimen was placed in an oven having the same temperature as the mean highest 7-day maximum pavement temperature (at 2 in. depth) for at least two hours (but no more than four hours) before being tested.

Given that the specimens had slightly different diameters, the shear load required to yield a 10 psi shear stress level for each specimen was calculated based on the area of the specimen.

2.2.4.4 Test Results

A Constant Height Repetitive Simple Shear Test was performed on one specimen from each GPS site. The tests were performed at 10 psi stress amplitude (with 0.1 second loading time and 0.6 second rest period) and at 7-day maximum pavement temperature encountered at the 2-inch depth. Figure 23 exhibits a typical graph of the permanent shear strain versus number of cycles obtained from the tests. It is apparent that some mixes really deform faster than others and that not only do they have different slopes but also different intercepts.

2.2.5 Analysis

Based on equation (*1*) the maximum shear strain correspondent to the measured rut depth for each of the GPS sites was computed. This value was related to ESALs (see Figure 24). Typical results from repetitive shear tests on GPS specimens (as shown in Figure 23) were used to determine the number of shear cycles required to reach the level of maximum shear strain calculated from equation (1). This process relates the number of cycles in the RSST-CH to reach the same magnitude of permanent shear strain as in the field caused by the ESALs.

Table 1 contains the results from all the tests. The right most column contains information about the rejection criteria used for the data (in some cases specimens would have been rejected based on other factors such as test executed at the wrong temperature or LVDTS that got loose during testing). Results from specimens with air voids less than 1.5% and more then 8.0% were eliminated. Specimens with voids less than 1.5% are overcompacted and not representative of the conditions prevailing during most of the life of the pavement. Specimens with void content above 8% are likely to densify before entering into the plastic shear flow stage. From all the data three of the points were removed as outliers.

The scatter plot of number of cycles in the test versus ESALs for all the data (without the outliers) is presented in Figure 25.

Recognizing the possibility of two populations (LINE A and LINE B), a closer investigation of the age of the pavements was made. Table 2 contains two sets of data represented in Figure 25. It is noticeable that two trends can be observed in the data; one obtained from specimens tested after being aged in the field for an average of 16 years, an another obtained from specimens aged in the field for an average of only 8 years. It should be noted that sites in LINE B have a maximum pavement temperature at 2 in. depth higher (in average) than those from LINE A. The average air void content is similar for both populations.

As it can be expected specimens with aged asphalt perform relatively better in the RSST-CH test. These results were obtained from specimens from out-of the wheel path field cores that have been subjected to aging and to limited traffic. However, since the specimens are from 2 in. depth, the magnitude of aging is lower than what would occur at the surface.

It was also noticed that most of the variability in the data came from the specimens of sites with ages above 10 years. This is to be expected as ESAL prediction, aging and traffic are all factors that can cause data variability. An investigation of the relationship between cycles in RSST-CH and ESALs was made for pavements less than 10 years old (see Figure 26). The following relationship was obtained with an R^2=0.68:

$$\log(\mathbf{Cycles}) = -4.09 + 1.204 \log(\mathbf{ESAL}) \qquad (4)$$

We can speculate that mixes with high air voids, from older pavements exposed to highest temperatures will probably age more. Therefore we can consider the product (age x voids x temperature/10) as a variable that if high indicates high likelihood of having a more aged mix then if the product is low. The last two columns of Table 2 contain the values obtained for this product. The very last column presents the values for sites more than 10 years old.

It can be observed that the average product (age x voids x temperture/10) for LINE A is almost half of that product for LINE B (for all sites). The average product for pavements more than 10 years old in LINE A (517) is lower than the correspondent product in LINE B (891) and is close to the average for all points in LINE A (427). This provides a rational to justify that specimens in LINE A more than 10 years old are not really as aged as the specimens belonging to LINE B.

Using all the data except those points from LINE A and discarding points from site 53071 as outliers it can be observed that a very clear trend with very little data variability exists (see Figure 27). The equation of the best fit is given by:

$$\log(\mathbf{Cycles}) = -4.36 + 1.240 \log(\mathbf{ESAL}) \qquad (5)$$

This relationship was obtained with an R^2=0.80.

The best fit lines obtained from this two criteria is presented in Figure 28. Based on this results it is suggested that equation (5) be used in the development of an abridged procedure to evaluate permanent deformation for asphalt concrete pavements.

This value is indicative of a very good correlation specially if it is taken into consideration that:

- In some cases the rut might also be due to densification, subgrade effect, pavement surface irregularities, etc.
- The RSST-CH was executed with specimens that, although not in the wheel path, have been already subjected to traffic to various degrees. Their behavior might be different from those obtained form newly compacted mixes.
- The calculation of the maximum pavement temperature at 2 in. depth is just an estimate of the real temperature.
- The testing rate is 0.1 sec loading with 0.6 unloading while in the pavement the rate is closer to 0.02 sec loading with almost random spacing.
- ESALs were not actually measured but were extrapolated based on DOT data.

Figure 29 shows the ranges in rut depth and ESALs used in this relationship. Overall it should be recognized that the relationship is indicative of a very strong relationship between the proposed test procedure and the rutting behavior in the field.

2.3 Abridged procedure

Based on the results presented an abridge procedure to determine rutting propensity of a mix could be developed. The following steps should be taken:

1. Determine Number of ESAL for design life. Corrections should be made to account for reliability factors in the procedure and in the tests.
2. Select maximum allowable rut depth (*mrd*).
3. Determine 7-Day Maximum Pavement Temperature at the Site at 2 in Depth.
4. Execute the RSST-CH test @ 10 psi at that temperature.
5. Using equation (1), relating the rut depth with the maximum permanent shear strain in a pavement section, determine the maximum allowable permanent shear strain (*mpss*).
6. Using the results obtained from the RSST-CH, determine the number of cycles needed to reach the maximum allowable permanent shear strain.
7. The number of ESALs that can be carried by that mix in the pavement before the maximum allowable rut depth is reached is determined using the relationship between ESALs and number of cycles in RSST-CH. The relationship derived from equation *(5)* is:

$$\mathbf{ESAL}_{mrd} = 10 \wedge ((4.36 + \log(\mathbf{N}_{mpss}))/1.24) \qquad (6)$$

where:

$\mathbf{ESAL}_{mrd}$-Number of Equivalent Single Axle Loads to develop maximum allowable rut depth (*mrd*)

$\mathbf{N}_{mpss}$- Number of Cycles in RSST-CH (@ 10 psi @ 7-Day max. pav temp@0.1 ON, 0.6 OFF) to reach the maximum permanent shear strain (*mpss*) correspondent to the maximum allowable rut depth (*mrd*)

Although quite simple this procedure could be successful because:

1. It takes into consideration some of the most important aspects of asphalt-aggregate mix behavior and the rutting phenomena such as:

Shear stresses - The test measures the resistance to shear stresses, which is in line with observations from the wheel test tracks and the field where the rutting was primarily attributed to shear flow under constant volume caused by the shear stresses near the edge of the tires.

Dilation - Asphalt-aggregate mixes dilate under shear strains and the added stability that can be derived from the aggregate structure when dilation is constrained can be captured by the constant height simple shear test.

Temperature susceptibility of the binder- Binders are temperature susceptible. The relative effect of their stiffness and their viscosity are recognized by executing the test at the 7-day maximum pavement temperature at 2 in depth where most of the permanent shear strains occur.

Plasticity of the aggregate structure - One of the components of the permanent deformation of asphalt-aggregate mixes is the plastic behavior. Executing the test under repetitive loads at high shear stress levels takes into consideration the fact that the deformation patterns of the aggregate structure are more sensitive to repetitive loads than to creep loading. Furthermore it is recognized that, after high stress levels have been applied, the incremental permanent deformation due to lower stress levels is almost negligible.

Rate of loading - the repetitive test is executed with a pulse load of 0.1 sec. Although this rate of loading is smaller than that representative of most of the traffic it is sufficiently fast to be within the some order of magnitude.

2. The test itself is easy to execute:
   - it lasts only up to one hour,
   - it is easy to obtain specimens in the laboratory,
   - it can be executed on cores from the field,
   - it does not require confining pressures; therefore, it simplifies the requirements for the shear machine,
   - it can be executed either in the field or in the laboratory by technicians that can easily be trained.

3. Data analysis and interpretation is straight forward and easily comprehended by Highway engineers:
   - all concepts are intuitively simple
   - all analysis can be done on any portable computer

4. The test and procedure can be used for quality control.

5. This procedure was obtained relating laboratory results to field performance. Therefore, it already contains any inherent shift factors. Its simplicity will permit different states and countries to adopt and adjust it to their local conditions.

2.4 Other considerations

To implement this procedure, a few factors should be considered:

1. Evaluation of tire pressure effects on the rate of accumulation of permanent deformation can only be done by computing ESALs for the axles with different tire pressures. This could be achieved using, for instance, a permanent deformation model as presented by Sousa et al. (1993).

2. Aging and water sensitivity should be addressed and incorporated in the procedure. The mix should be submitted to short term aging (representative of field mixing and placement process) and to water sensitivity conditioning before being subjected to the RSST-CH. This would represent the most severe conditions encountered in the field. Executing the water conditioning procedure might weaken the asphalt-aggregate interface and reduce the resistance to shear deformation. Long term aging should not be executed as it would stiffen the asphalt binder, and therefore, provide improved performance. Furthermore the correlation presented was obtained for pavements less than 10 years old.

3. The assumption of uniformly distributed ESALs (inherent in the procedure) over the year, and over the day could be improved. This might be achieved also by taking advantage of a comprehensive finite element model for permanent deformation which could also take into consideration the relative contribution for

permanent deformation of the ESALs applied at different temperatures. It is likely that most permanent deformation occurs from traffic passing when the pavement temperature is within 5 C of the maximum pavement temperature at 2 in depth.

The procedure could be further improved if the test was executed at the mean highest 7-day maximum pavement temperature corrected to compensate for the rate of loading effect. Normal traffic, traveling at 55 MPH, applies pulse loads with a duration of about 0.015 seconds at 2 in depth. The tests in laboratory are executed with a 0.1 sec loading pulse. Taking advantage of time-temperature superposition decreasing the temperature would simulate the faster rate of loading encountered in the field. This might provide a more accurate balance between permanent deformation due to the viscous behavior of a binder and the plastic component due to the changes in the magnitude of the shear strains. The exact amount of temperature shift could be given by temperature shift factors obtained by the shear frequency sweep results executed at different temperatures. The testing temperature would be further adjusted to take into consideration the field rate of loading (mixes to be placed in up-hill pavement sections with slower traffic could be tested at higher temperatures than mixes to be placed in level sections).

It should be further recognized that in the constant height repetitive simple shear test the specimen hardly changes volume. Therefore the tests should be executed on a mix with air void contents representative of those predominant over the life of the pavement (Sousa, 1994).

To implement the procedure based on these findings it must be recognized that equation *(1)* might not be valid in all cases. However, this assumption can be easily demonstrated and validated by executing the types of analyses presented in section II-1 for a series of pavement configurations. It is likely that a family of curves could be developed for different pavement thickness.

It should also be recognized that there is an inherent variability in any test procedure. Therefore, reliability considerations should be incorporated in the procedure.

2.5 Summary

The foundations for the development of an abridged procedure to determine the permanent deformation potential of an asphalt-aggregate mix has been presented. In this procedure, it is recognized that the asphalt-aggregate mixes exhibits nonlinear elastic, viscous and plastic behavior. The nonlinear behavior such as dilation and stress hardening is mostly influenced by the aggregate skeleton. A finite element model which takes into account these aspects of mix behavior was used to establish a relationship between rut depth and maximum permanent shear strain in a pavement section. This relationship seems to be independent of a wide range of input variables and material properties. However, it is likely dependent on pavement structure for thin pavement sections.

The constant height repetitive simple shear test (RSST-CH) was used as the accelerated laboratory test for evaluating the rutting propensity of the mix. This test was executed at the critical pavement temperature at 2 in. depth. For the purpose of this analysis the critical pavement temperature was defined as the 7-day maximum pavement temperature at 2 in. depth. This depth was selected because computations have shown that the maximum shear stresses, causing the permanent deformation in the pavements, are encountered at 2 in. depth near the edge of the tires.

This procedure was derived from data obtained from forty GPS sections around the North-American continent. It is mainly based on the execution of the RSST-CH and at the mean highest 7-day maximum pavement temperature encountered at 2 in. depth.

The fundamental link between the laboratory tests and the field performance was derived from determining a relationship between the number of cycles in the RSST-CH to reach a given permanent shear strain and the number of ESALs to cause the same permanent shear strain in the pavement section. For pavements less that did not exhibit significant aging that relationship was obtained with an R^2 of 0.80.

Specimens should be compacted in laboratory to air voids contents expected in the field with a compaction procedure that simulates the aggregate structure caused by traffic. It is suggested that the RSST-CH test be performed on specimens compacted in laboratory to about 3 to 4% void content. Recognizing that this procedure was developed based on cores from the field and not from laboratory compacted specimens efforts should be made to determine that effect. Then, as the rut depth measurements from the site become available, the results can be compared and the existing relationship can be either verified or improved.

2.6 Chapter References

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Sousa, J.B., A. Tayebali, J. Harvey, P. Hendricks and C. Monismith, "Sensitivity of Strategic Highway Research Program A-003A Testing Equipment to Mix Design Parameters for Permanent Deformation and Fatigue", Transportation Research Record no.1384, pp 69-79, 1993.

Sousa, J.B, "Permanent Deformation of Asphalt-Aggregate mixes" paper accepted for publication at the AAPT annual meeting 1994.

GPS SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE Years	ESAL YEARS	Rut Depth (in)	Shear Strain	Number Cycles RSST-CH	REG. CRITERIA
21001	GX21-1	AK	90	6.1	7	399844	0.18	0.0164	87	
21004	GX1-1	AK	90	3.9	13	1791505	0.33	0.0300	3419	
41007	GX41-1	AZ	138	2.8	11	21365008	0.41	0.0373	576	out
41021	GX-19	AZ	135	1.0	11	11549655	0.52	0.0473	26876	void
					12	12633956	0.53	0.0482	28553	void
41025	GX22-1			0.0	13	13651008	0.16	0.0145	286	void
41036	GX8-1	AZ	138	6.6	6	4322385	0.14	0.0127	11523	
					7	4769968	0.14	0.0127	11523	
53071	GX64-1	AR	125	3.9	1	637500	0.14	0.0127	6872	
					2	1275000	0.16	0.0145	9694	
					3	1657500	0.16	0.0145	10585	
68153	GX51-1	CA	120	3.4	12	614903	0.16	0.0145	376	
68156	GX26-1	CA	120	6.3	15	820162	0.14	0.0127	30063	
82008	GX10-1	CO	125	1.5	17	1225650	0.42	0.0382	19963	
					18	1283072	0.46	0.0418	28155	
131031	GX33-1	GA	128		9	227047	0.28	0.0255	4763	
					10	256209	0.28	0.0255	4763	
161020	GX61-1	ID	120	6.0	3	142749	0.14	0.0127	44	
					4	178200	0.15	0.0136	52	
171003	GX32-1	IL	125	3.5	3	139986	0.12	0.0109	185	
					4	179982	0.17	0.0155	395	
					5	216645	0.15	0.0136	299	
201009	GX29-1	KS	130	7.9	4	284935	0.20	0.0182	451	
					5	404268	0.23	0.0209	582	
211014	GX14-1	KY	120	4.1	5	1418454	0.18	0.0164	3089	
					6	2051845	0.19	0.0173	3827	
231012	GX44-1	ME	110	2.1	4	980000	0.23	0.0209	756	
					5	1190000	0.25	0.0227	931	
261012	GX3-1	MI	115	6.5	9	714861	0.23	0.0209	803	
					10	802748	0.26	0.0236	1031	
271019	GX11-1	MN	115	9.0	9	435438	0.22	0.0200	83390	void
					10	472975	0.23	0.0209	9324	void
341030	GX23-1	NJ	120	0.5	18	1115000	0.56	0.0509	14355	void
					19	1160000	0.68	0.0618	23103	void
341031	GX31-1	NJ	120	1.0	16	5075000	0.37	0.0336	486	void
					17	5325000	0.38	0.0345	512	void
351022	GX62-1	NM	120	5.2	6	724306	0.15	0.0136	1304	
401015	GX43-1	OK	130	2.1	13	955031	0.23	0.0209	2016	
					14	1040193	0.24	0.0218	2692	
404164	GX35-1	OK	130	4.0	14	633750	0.15	0.0136	19076	
					15	686250	0.14	0.0127	15860	

Table 1 - Summary of test conditions and results

GPS SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE Years	ESAL	Rut Depth (in)	Shear Strain	Number Cycles CHRSST	REG CRITE
479025	GX30-1	TN	125	8.1	9	233159	0.14	0.0127	19804	voic
					11	288553	0.18	0.0164	37120	voic
481039	GX71-1	TX	130	3.9	7	1637481	0.16	0.0145	1105	
					8	1993484	0.23	0.0209	2170	
481047	GX18-1	TX	130	2.5	18	5500000	0.20	0.0182	1625	out
481048	GX42-1	TX	130	1.1	15	786000	0.20	0.0182	251336	voic
					17	856600	0.16	0.0145	131878	void
481069	GX81-1	TX	130	2.3	13	2573568	0.34	0.0309	8205	
					14	2751168	0.32	0.0291	7185	
481077	GX15-1	TX	130	1.8	7	1394648	0.38	0.0345	3039	
811805	GX65-1	CAN	110	4.3	9	1190182	0.25	0.0227	133	out
851801	GX4-1	CAN	90	4.1	5	1183357	0.21	0.0191	2668	
892011	GX63-1	CAN	115	4.8	10	853376	0.17	0.0155	813	
					11	933380	0.20	0.0182	1258	

COLOR ADO SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE Years	ESAL	Rut Depth (in)	Shear Strain	Number Cycles CHRSST	REG. CRITEF
14	14 H	CO	124	4.6	23	3282000	0.80	0.0727	166839	
29	29 I	CO	120	7.1	9	5002000	0.30	0.0273	4442	
30	30 H	CO	120	6.6	9	4622000	0.60	0.0545	3122	
13	13 A	CO	124	7.5	6	1257000	0.10	0.0091	1030	
13	13 B	CO	124	7.1	6	1257000	0.10	0.0091	675	

Table 1 - Summary of test conditions and results (cont.)

LINE A SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE YEARS	Temp * Voids* Age	
21001	GX21-1	AK	90	6.10	7	384	
21004	GX1-1	AK	90	3.90	13	456	456
41036	GX8-1	AZ	138	6.60	7	638	
53071	GX64-1	AR	125	3.90	3	146	
68153	GX51-1	CA	120	3.40	12	490	490
161020	GX61-1	ID	120	6.00	4	288	
171003	GX32-1	IL	125	3.50	5	219	
201009	GX29-1	KS	130	7.90	5	514	
211014	GX14-1	KY	120	4.10	6	295	
231012	GX44-1	ME	110	2.10	5	116	
261012	GX3-1	MI	115	6.50	10	748	748
351022	GX62-1	NM	120	5.20	6	374	
401015	GX43-1	OK	130	2.10	14	382	382
481039	GX71-1	TX	130	3.90	8	406	
481069	GX81-1	TX	130	2.30	14	419	419
481077	GX15-1	TX	130	1.80	7	164	
851801	GX4-1	CAN	90	4.10	5	185	
892011	GX63-1	CAN	115	4.80	11	607	607
29	29 I	CO	120	7.10	9	767	
30	30 H	CO	120	6.60	9	713	
13	13 A	CO	124	7.50	6	558	
13	13 B	CO	124	7.10	6	528	
	LINE A	AVERAGE	119	4.84	8	427	517

LINE B SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE YEARS	Temp * Voids* Age	
131031	GX33-1	GA	128		10		
404164	GX35-1	OK	130	4.00	15	780	780
68156	GX26-1	CA	120	6.30	15	1134	1134
82008	GX10-1	CO	125	1.50	18	338	338
14	14 H	CO	124	4.60	23	1312	1312
	LINE B	AVERAGE	125	4.10	16	891	891

Table 2 - Summary of results for LINE A and B.

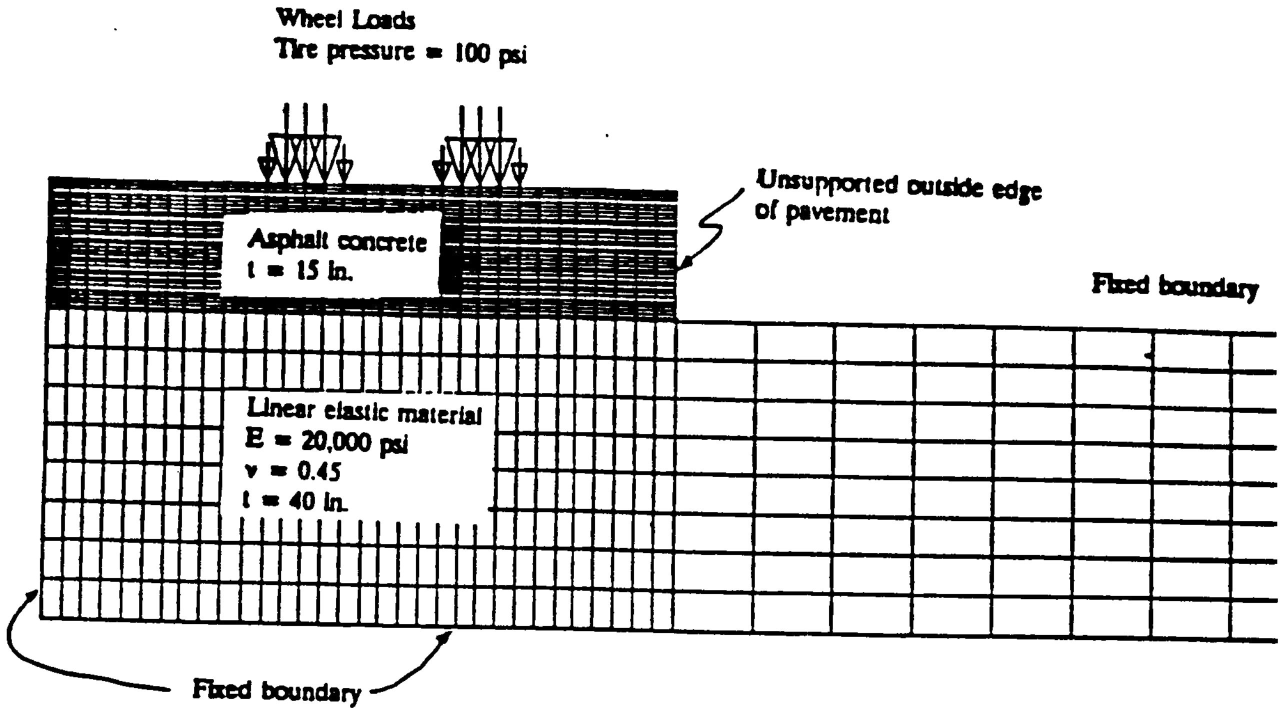

Figure 16 - Pavement Cross-Section represented by 2-dimensional finite-element mesh.

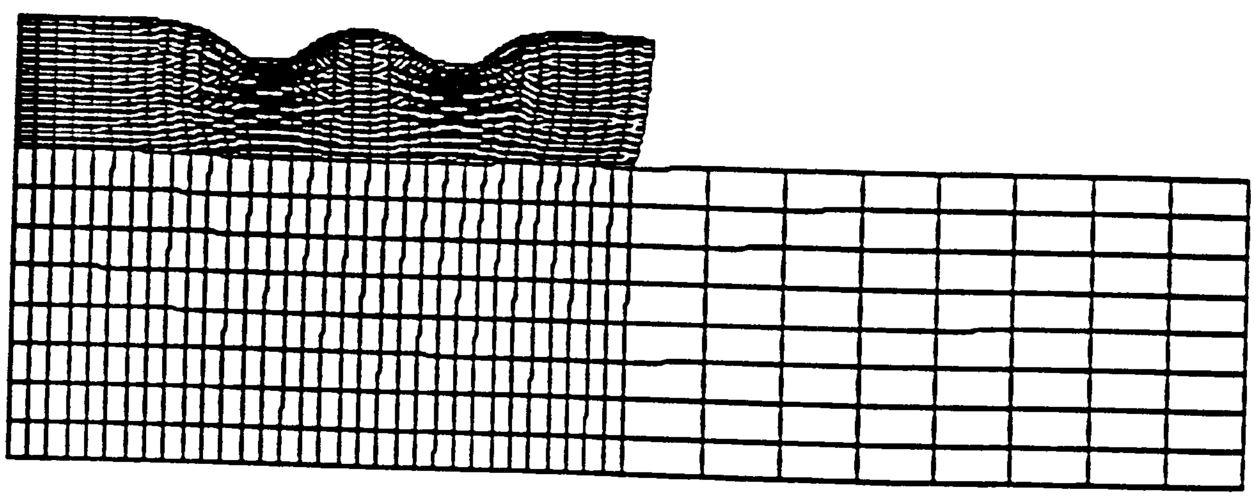

Figure 17 - Deformed finite-element mesh, second loading cycle.

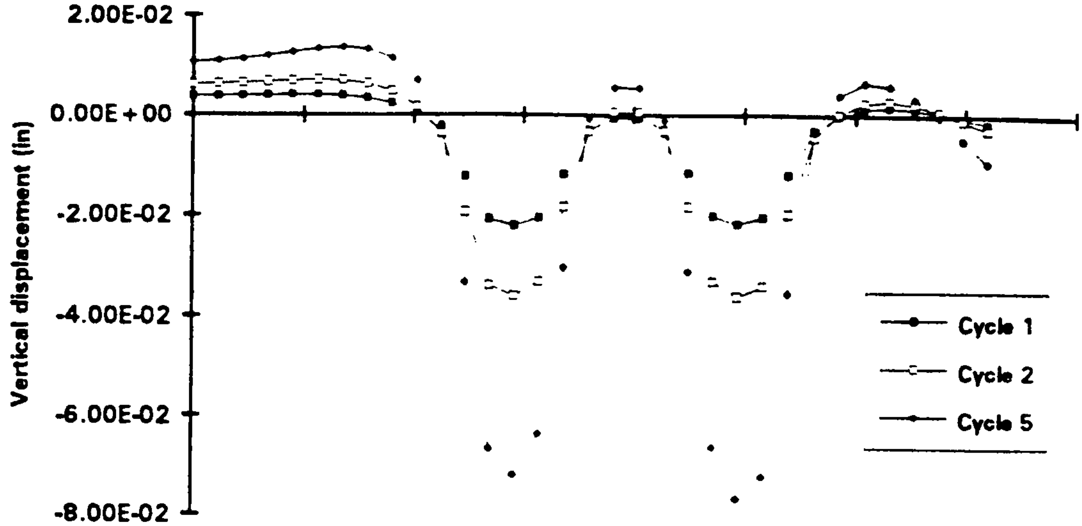

Figure 18 - Variation of pavement profile with the number of load applications. Stress level 500 psi, loading time 0.3 sec, rest period 0.4 seconds.

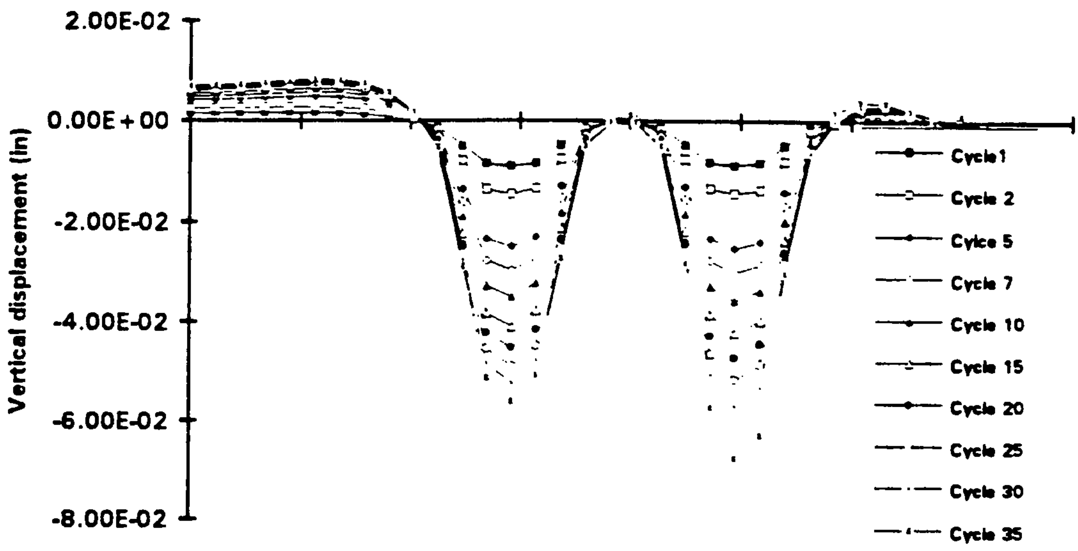

Figure 19 - Variation of pavement profile with the number of load applications. Stress level 200 psi, loading time 0.3 sec. rest period 0.4 seconds.

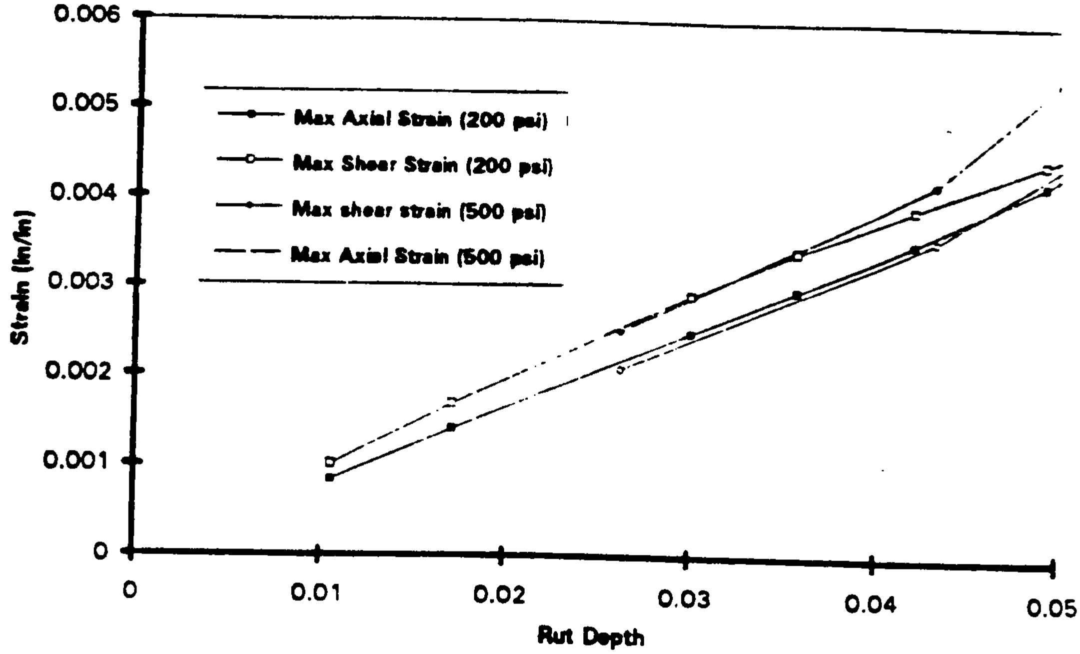

Figure 20 - Comparison of relationships between rut depth and strain for 200 psi and 500 psi stress levels.

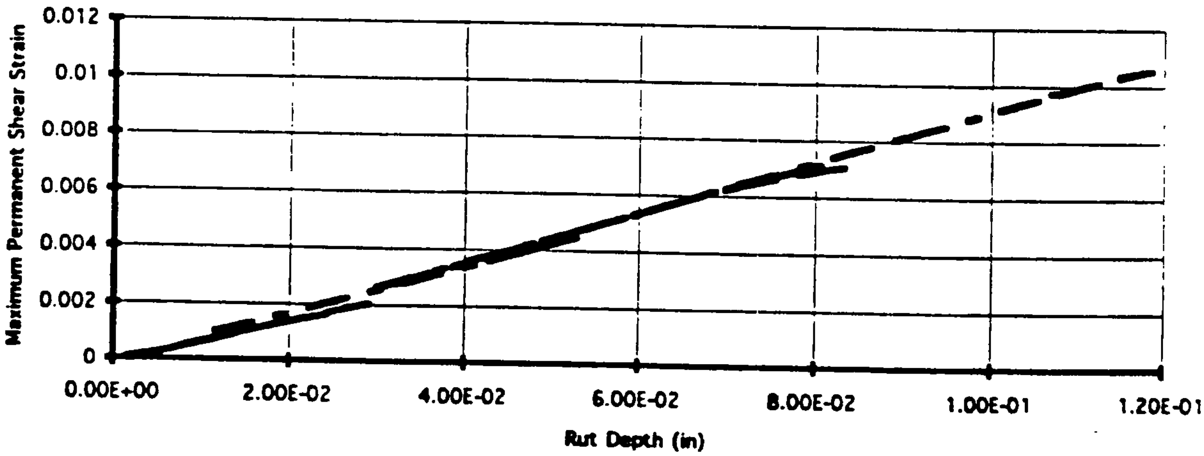

Figure 21 - Variation of the rut depth with the Maximum Permanent Shear Strain for 16 Mixes

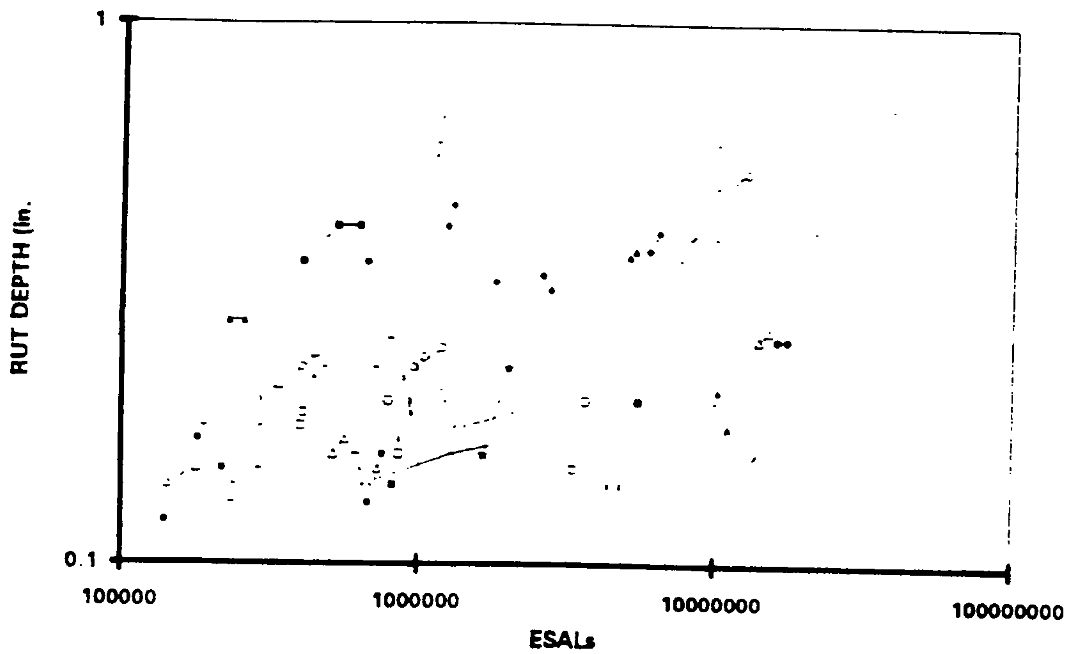

Figure 22 - Variation of Rut Depth with ESALs for the GPS sections.

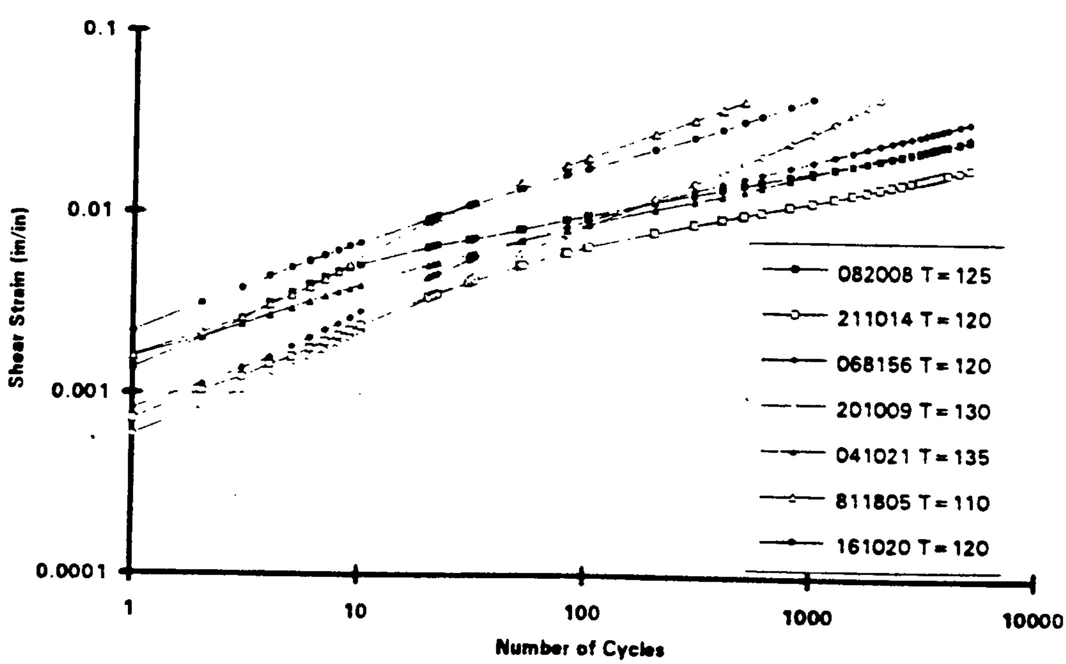

Figure 23 - Variation of the permanent shear strain in RSST-CH with the number of load repetitions for some GPS sites.

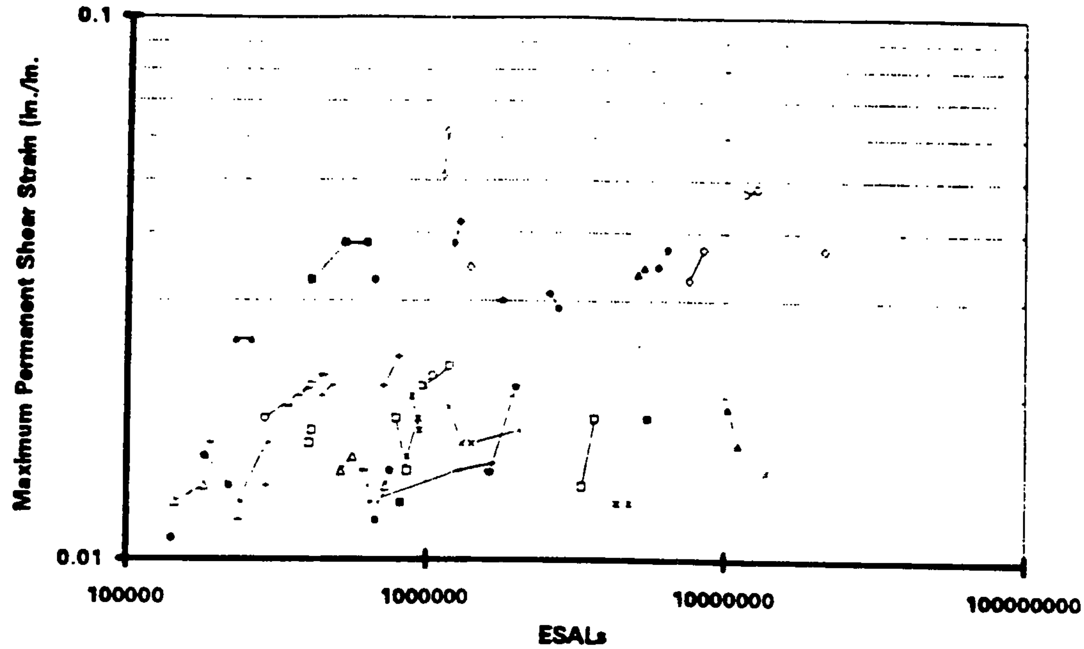

Figure 24 - Variation of the maximum shear strain with ESALs for the GPS sections including some possible judgment about quality of mixes and specification limits.

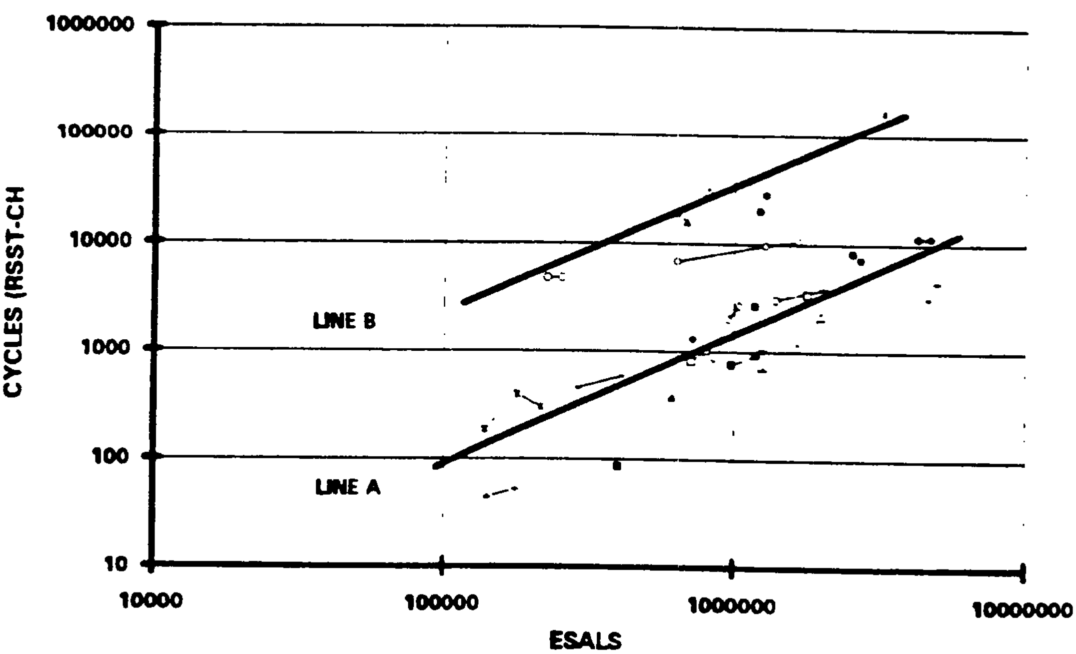

Figure 25 - Relationship between number of cycles in RSST-CH and ESALs to reach the same shear strain level. LINE A and B indicate two possible populations.

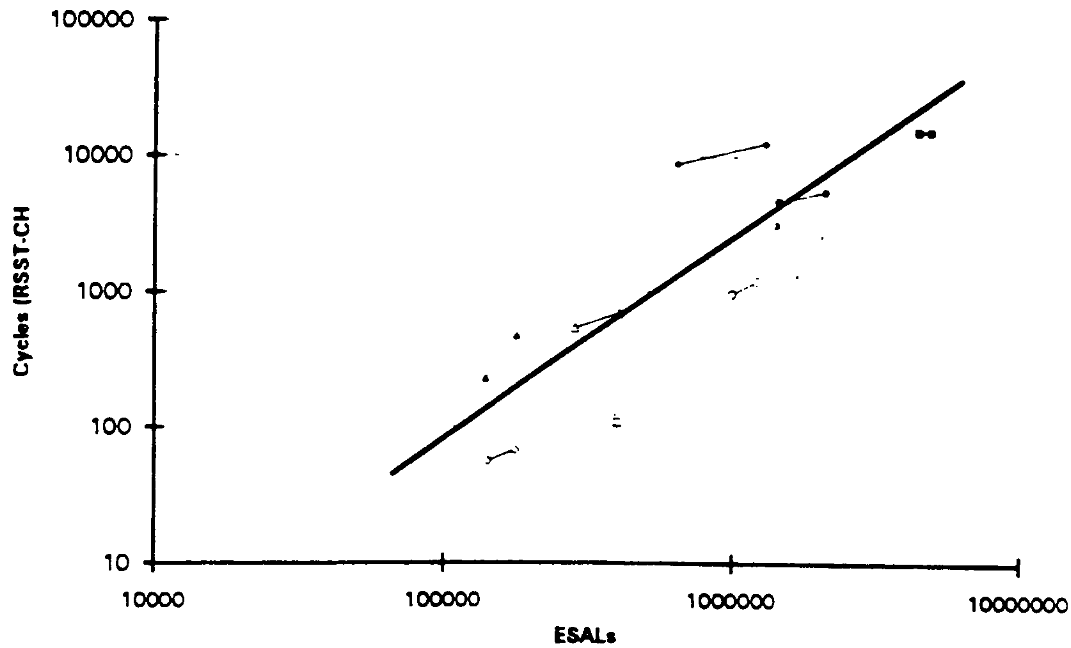

Figure 26 - Variation of the number of cycles to the field shear strain in RSST-CH with ESALs for the sections less than 10 years old.

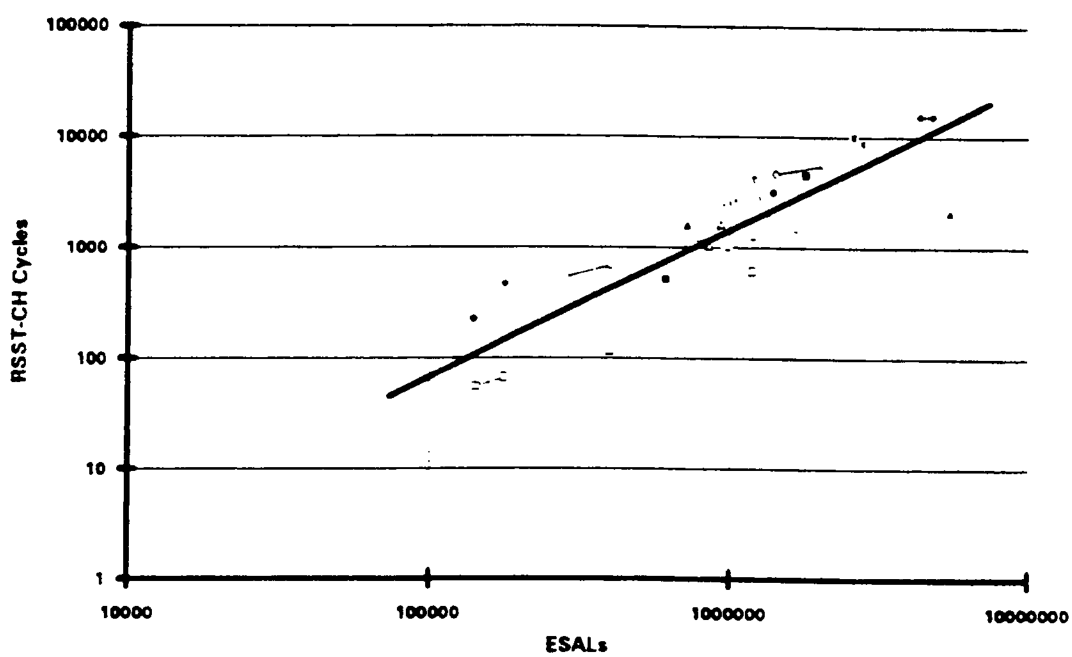

Figure 27 - Variation of the number of cycles to the field shear strain in RSST-CH with ESALs for the sections less that did not exibit significant aging.

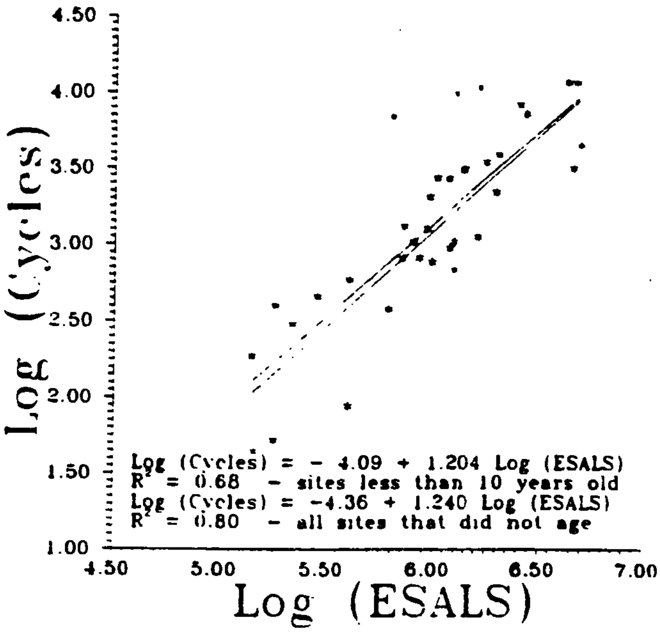

Figure 28 - Variation of the number of cycles to the field shear strain in RSST-CH with ESALs for the sections less that did not exibit significant aging and those less than 10 year old.

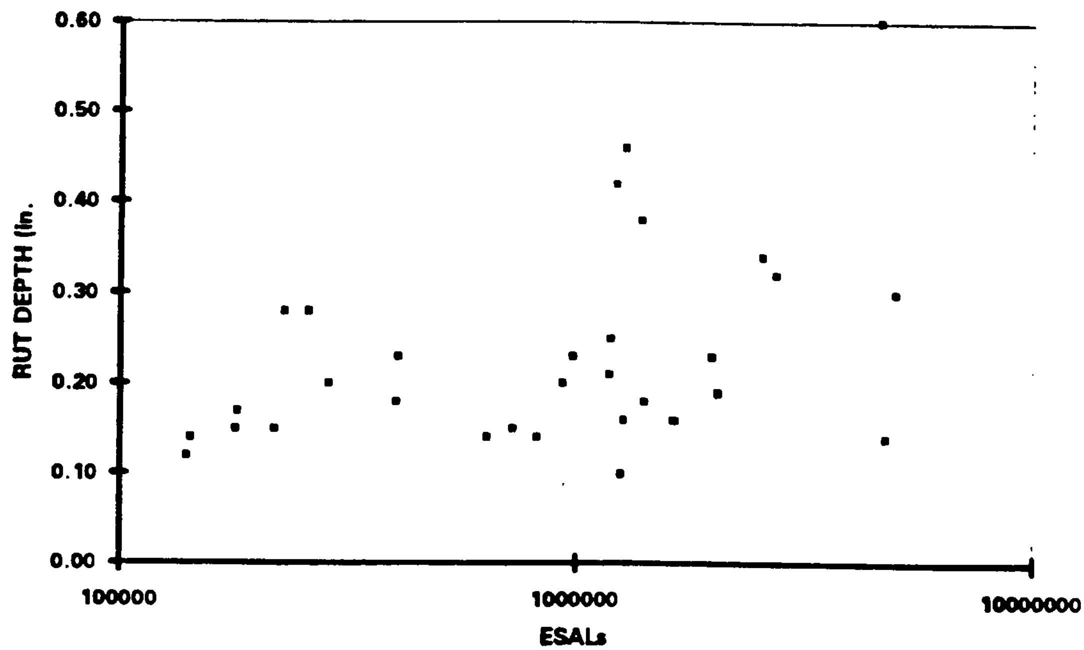

Figure 29 - Range of rut depths and ESALs used in the development of the relationship.

3

Asphalt-Aggregate Mix Design Using The Repetitive Simple Shear Test (Constant Height)

3.1 Introduction

One of the goals of the Strategic Highway Research Program (SHRP) effort was the development of a mix design procedure. Several levels of complexity can be incorporated into a procedure. The adoption of one procedure over another is most likely tied to its cost and reliability, taking into consideration the magnitude of the construction project. This paper presents a procedure that could be used for mix design.

The underlying assumption of this approach is that permanent deformation, in well constructed pavements, is primarily a plastic, strain hardening, shear flow phenomenon at constant volume, occurring near the pavement surface, caused by the shear stresses occurring below the edge of the truck tires.

Also, intrinsically linked to this procedure is the assumption that most of the permanent deformation occurs in the hottest days due to the heaviest trucks. This assumption stems from observations in the laboratory that asphalt-concrete mixtures exhibit strong plastic behavior described by a plasticity function that exhibits kinematic hardening. This hardening seems to be associated with the capability of the mixture to develop better particle-to-particle contact as it develops shear strains, and with the capability of the aggregate skeleton to develop dilatancy forces that in turn are capable of developing stabilizing confining stresses.

This phenomenon seems to be best captured by the Repetitive Simple Shear Test at Constant Height (RSST-CH) executed at the mean highest 7-day maximum pavement temperature at 2 in. depth. One of the advantages of this test is that it does not cause any change in volume in the specimen during testing. This is particularly important as a mix resistance to shear deformation should be measured with a test that does not cause any change in volume (densification or dilation).

This procedure was developed based on data reported from road sections of the General Pavement Studies (GPS) under SHRP Long Term Pavement Performance (LTPP). With a relatively low level of effort other countries and states could validate this procedure for their specific local conditions. This procedure could be improved when ESALs can be determined as a function of the tire pressures using, for instance, the model presented by Sousa , Weissman et at. (1993); currently tire pressures of 100 psi are assumed.

3.2 Rutting in Dense Graded Mixes

3.2.1 The Phenomena

Rutting in asphalt-concrete layers develops gradually with increasing number of load applications, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides (Sousa and Weissman, 1994). In the development of the rut depth, it is also necessary to recognize the evolution of the void content in a pavement section.

Air-void content can be used as a measure of the degree of compaction of a mix. For all the GPS sites, air-void measurements were made on pavement cores along with measurements of rut depth. Figure 30 illustrates the variation of rut depth with air-void content. It can be noted that with the exception of two sites, larger rut depth was obtained for sites where air void content had dropped to below 3%. When the air-void content drops below 2-3% the binder acts as a lubricant between the aggregates and reduces point-to-point contact pressures. Without the aggregate skeleton resisting the shear stresses which appear near the edge of the tires, the mix rapidly develops large permanent shear strains which, in turn, cause the development of the rut. During the majority of the pavement life, as traffic densifies the mix, it steadily develops better aggregate interlock and resistance to shear stresses. Only when the reduction of the air-void content causes the binder to prevent point-to-point contact in the aggregate does the mix lose stability.

Rutting can be considered as two components that occur simultaneously with different degrees of importance: 1) reduction of volume, densification, better particle-to-particle contact up to the point where pore pressures in the binder prevent aggregate-to-aggregate contact 2) continuous development of shear strains which exhibit a strain hardening aspect not only due to dilation effects but also due to mix densification.

3.2.2 Rutting Evaluation

To predict permanent deformation, laboratory tests must be capable of measuring properties under states of stress that are encountered within the entire rutting zone, particularly near the pavement surface. Since there are an infinite number of states of stress, it is impossible to simulate them all with a single test, especially given the non-linear and viscous behavior of the material. For this reason several tests have been proposed to determine a constitutive law for asphalt concrete that can be implemented as part of a finite element computer program to solve any boundary value problem (Sousa, Weissman et al., 1993).

Mix design could be done by executing the battery of tests presented in Sousa, Weissman et al. 1993 (i.e. uniaxial strain, simple shear, volumetric and frequency sweep tests), determining the fundamental material properties of the mixes with different asphalt contents and different void contents, performing a finite element simulation (Figure 31 and 32) and determining the rut depth as a function of the number of cycles. For each of the mixes consideration of fatigue life and development of permanent deformation would be made and the optimal asphalt content would be selected.

Currently that procedure is too time consuming and expensive to be used in a day-to-day operation and it still does not address the change in properties due to changes in void contents, aging and moisture damage. The need to obtain a rapid, yet reliable, measure of mix propensity for rutting has prompted researchers to adopt abridged procedures based on a single test (executed under particular conditions) and develop correlations with observed field performance (i.e. the Marshall and Hveem methods).

However, if a single test is to be performed to rapidly screen and evaluate the resistance of various mixtures to permanent deformation then that test should be sensitive to the most important aspects of mix behavior and should be executed under conditions that most significantly affect that behavior.

Based on the dual nature of mix behavior (densification and plastic shear flow at constant volume), two types of test procedures can be conceived: 1) test procedures where volumetric and shear components can occur simultaneously; 2) and test procedures where shear components are predominant and volumetric changes are minimized.

Wheel track tests, the Marshall procedure and the Hveem procedure fall in the first category. In each of these procedures the mix can develop volumetric and shear components (i.e., it can densify or dilate and it can shear). The constant height simple shear tests fall in the second category. A very accurate procedure based on wheel track tests could be developed (depending on the relative size of the wheel and the slab). However it would not be very practical to produce several large slabs at different asphalt contents, execute the tests and relate them to field performance. There are two possible avenues available for the development of an abridged procedure based on the results of a single test: 1. Volumetric/shear procedure and 2. Shear procedure without volume reduction.

VOLUMETRIC/SHEAR PROCEDURE - In this procedure one would develop a test where repeated loads (axial and/or shear) would be applied in such a manner that a net reduction of volume would take place and the change in the air-void content (at a given temperature, rate of loading, etc.) would be monitored. The number of load applications required to reach 3% void content (for instance) would be determined. A correlation between that number and field traffic would provide the basis for a regression function.

This procedure poses a few technical problems because the change in voids has to be monitored while the mixture densifies under repetitive axial/shear stresses. It should be noted that in the field the process of densification is accomplished by kneading action due to the tire pressure. The test procedure should provide for a densification process that creates an aggregate structure identical to that created in the field.

It is imperative to recognize that neither the unconfined axial test under repetitive loading nor the unconfined biaxial repetitive shear test, both proposed candidate tests for mix analysis, satisfy any of the above conditions. In the axial test the specimen volume actually **increases** as permanent deformation occurs. The rapid failure observed at about 1% axial strain must not be related or associated with the fact that a mixture fails in the field after reaching 3% air voids. **These are two totally different mechanisms**. This can be clearly observed in Figures 33 through 36. Axial creep and repetitive tests were executed at 40 C without confining pressures. It can be observed that the I_1 invariant (or volumetric strain = axial strain + 2* radial strain) was always negative near failure. In the axial strain versus number of cycles it can be observed that the rapid increase in axial strain occurred when the volumetric strain was negative. This indicates **expansion** of the specimens and **not** reduction of volume (densification). These data demonstrate that in the unconfined axial tests the specimen fails with an increase in volume.

The tertiary creep, observed in the unconfined axial creep tests, leading to accelerated failure is due to cracks in the specimen whereas the rapid failure, observed in the pavement after reaching air void contents below 3%, is due to a loss of bearing capacity in the aggregate skeleton. The reduction of air voids, due to traffic densification, causes the asphalt to fill most of the voids. At this stage the asphalt actually acts as a lubricant between the aggregate particles as it reduces particle-to-particle contact. This could be compared with the development of pore pressure in liquefaction of sands.

This indicates that the unconfined axial test cannot be used to simulate the mechanism of failure observed in the field where the mixture actually fails with plastic shear flow after a decrease in volume takes place. The best simulation of this phenomenon would be a wheel track test or a uniaxial strain test where densification of the specimen would occur by applying confining pressures and repetitive shear, and axial loads. If a wheel test track is developed for this purpose it should permit the unconstrained development of the shear strains. (The LCPC device does not satisfy this criterion as the wheel is almost the same size of the slab which is constrained laterally.)

A wheel track developed for this purpose would require testing specimens of significant size not suitable for routine testing. A uniaxial strain test would require evaluation of shear resistance as the voids decrease. This is a viable alternative that should be pursued.

If a uniaxial strain test is developed then the confining pressure would be controlled by the perimeter measuring device in order to insure that no radial deformation, hence no increase in volume, would take place (basically the Hveem procedure). However, in the process of densifying the specimen in such a test, shear stresses need to be applied to determine if the specimen had lost its stability. (By just looking at a volumetric component one would not be able to determine loss of shear resistance.) On the other hand in the Marshall procedure the shape of the loading heads constrain the specimen causing some densification in one direction while the specimen exhibits cracks (volume increase) in other directions. The state of stress in the Marshall test is too complex and the relative effects of densification, dilation and shear resistance are difficult to evaluate. The failure is too rapid and the loading is not repetitive.

The most promising test to be used in a volumetric/shear procedure is the uniaxial strain test combined with the simple shear test. In such a procedure all mixes (with different asphalt contents) would be compacted to initial in-situ air-void contents and the procedure would simultaneously densify and shear the specimen in a manner similar to that encountered in the field. The aggregate structure created during the test densification process should be similar to that caused by traffic densification in the field. During the duration of this research it was not possible to develop such a procedure.

SHEAR PROCEDURE WITHOUT REDUCTION IN VOLUME - In this procedure, tests would be executed on dense graded mixes with known void contents and their resistance to shear stresses would be evaluated by a procedure where the void content would remain unchanged throughout the test. The constant height, repetitive, simple shear test is ideally suited to evaluate the shear resistance of a mix at a given void content as the test is executed without change in volume.

To evaluate the resistance of a mix at different void contents, several specimens would be fabricated, each at a different void content, and their resistance to shear stresses evaluated. However, because mixtures placed in pavements tend to decrease in air-void content due to traffic densification, it is proposed that tests only be executed at the air-void content corresponding to the maximum resistance.

Field data (see Figure 30) have shown that mixtures tend to become unstable when void content falls below the 3% level obtained using the parafilm technique (Harvey, et al., 1993). It is therefore reasonable to execute the simple shear test in specimens with void contents around 3% where the mix is most resistant to shear deformation. This value seems to be reasonable for dense-graded mixes. Other mix types may have other levels of critical air-void content.

This procedure assumes that a mixture in the field would eventually densify to 3% air-void content, given sufficient traffic and sufficiently high pavement temperatures. This procedure would provide the answer to the following question: *Once a mixture reaches 3% air-void content how many repetitions of the shear forces imposed by the traffic will it take* *before developing excessive shear deformation*? In this procedure specimens would be compacted to about 3% air void content and shear tests at constant volume would be performed.

The repetitive simple shear test at constant height performed on a 6 in. (0.15 m) diameter by 2 in. (0.05 m) height cylindrical specimen is proposed as the test to evaluate the rutting propensity of a mixture. To test specimens with larger particle sizes an 8 in. diameter by 3 in. high specimen is recommended.

3.3 The simple shear test

3.3.1 Test Selection

Several reasons lead to the selection of the simple shear test at constant height:

1. *No Change in Volume:* during the test there is almost no change in volume. The height of the specimen is maintained constant under feedback closed loop control. The top and the bottom of the specimens are glued preventing lateral movements. The test is executed under conditions close to shear flow with no change in volume.

2. *Specimen Geometry*: a) A 6 in. diameter by 2 in. high specimen can easily be obtained from any pavement section by coring, or from any compaction method proposed by SHRP (i.e. gyratory or rolling wheel compaction); b) The state of stress is relatively uniform for the loads applied; c) The magnitude of loads required in the test of such specimens are easily attained by hydraulic equipment. If large stone mixes have to be studied then 8 in. diameter by 3 in. high specimens should be used.

3. *Rotation of Principal Axes*: it is the simplest test that permits controlled rotation of the principal axes of strain and stress which is important in studying rutting.

4. *Repetitive Applied Loads*: studies have indicated that to capture the rutting phenomena, application of repetitive loads is required given the viscous nature of the binder (mixtures behave differently at different loading rates) and the granular nature of the aggregate (aggregates behave differently under static and repetitive loads).

5. *Dilation*: one of the most important aspects controlling the stability of a mix is dilation. Under shear strains, densely compacted mixtures tend to dilate (just like dense sands). If dilation is constrained (as it is in the pavement to some degree by the adjacent material), then confining stresses are generated. It is in part due to the development of these confining stresses that a mix derives its stability against shear strains. Mixes with little tendency to dilate will have a higher propensity for rutting. In the constant height simple shear test the development of axial stresses is fully

dependent on the dilatency characteristics of a mixture. As permanent shear strains increase, the mix will develop (due to dilation) more or less axial stresses, depending on the aggregate type, structure, texture and void level. In this test configuration, a mix will resist permanent deformation either by relying on high binder stiffness to minimize shear strains or by aggregate structure stability imparted by the development of axial stresses due to dilation. These two mechanisms are the most important ones that provide resistance to permanent deformation in a mix (mixtures with some modified binders can have additional dilation forces caused by modifier dilatancy resulting from shear strain rates).

The axial stresses act as a confining pressure and tend to stabilize the mixture. A well compacted mixture with a good granular aggregate will develop high axial forces at very small shear strain levels. Poorly compacted mixtures can also generate similar levels of axial stresses but they will require much higher shear strains.

Stiffer binders help in resisting permanent deformation as the magnitude of the shear strains is reduced under each load application. The rate of accumulation of permanent deformation is strongly related to the magnitude of the shear strains. Therefore a stiffer asphalt will improve rutting resistance as it minimizes shear strains in the aggregate skeleton.

In the constant height simple shear test these two mechanisms are free to fully develop their relative contribution to the resistance of permanent deformation as they are not constrained by imposed axial or confining stresses. This might be one of the reasons why this test is so discriminating for asphalt-aggregate mixes.

Recognizing that pavement rutting is predominantly a shear flow phenomenon it seems reasonable to impose shear stresses and allow the material to develop its own axial stress (representing to some extent the insitu conditions where dilation is restrained by adjacent material).

3.3.2 Equipment Description

The execution of a repetitive simple shear test at constant height required the design and fabrication of a totally new equipment. Taking into consideration that this test would be executed on a routine basis efforts were made to insure the easiest possible interface with the user.

The testing system used for permanent deformation was developed by Cox and Sons and has been presented by Sousa, Tayebali et al., 1993. It basically consists of two orthogonal tables which are mounted on bearings. The tables are connected to two hydraulic actuators which are controlled using servo-valves under feedback closed-loop digital algorithms. To insure that the shear and axial forces are transmitted to the specimen, aluminum caps are glued to the parallel faces of the specimen. A gluing device was developed by James Cox and Sons, Inc. to insure that the caps faces are glued parallel. New hydraulic clamps insure an easy interface with the user by eliminating the need to use tools to fasten the specimens to the moving tables.

3.3.3 Test Procedure

To execute a repetitive simple shear tests at constant height (RSST-CH) the vertical actuator maintains the height of the specimen using as feedback the output of an LVDT measuring the relative displacement between the specimen caps. The horizontal actuator under control by the shear load cell applies haversine loads corresponding to a 10 psi shear stress magnitude with a 0.1 sec loading time and 0.6 sec rest period. Specimen shear deformation is measured with an LVDT mounted directly on the specimen thus away from the end effects and away from the deformation of the glue.

Experience with a wide range of mixtures tested at different temperatures and stress levels demonstrated that the 10 psi shear stress magnitude was a reasonable level at which good mixtures would exhibit some permanent deformation while poor mixtures would not fail excessively fast. Finite element computations have shown that critical shear stress levels in the field might be around 20 to 25 psi. Associated with these shear stresses confining pressures of about 30 psi and axial stresses of about 80 psi were also found. However, no lateral confinement is applied during this laboratory test because the axial strain control boundary provides for partial development of axial confining stresses and application of confining stresses would make the test more complex and less easy to execute. Furthermore cost of testing equipment would increase considerably if confining stresses were required making it less interesting as a field quality control tool.

Tests were executed until 5% shear strain was reached or up to 5000 cycles. The 5% shear strain level was selected because finite element studies (see Sousa and Weissman, 1994) have shown that at that level of permanent shear strain rut depths of about 0.5 in. can be expected. Prior to testing specimens were conditioned with 100 cycles of 1 psi haversine loading with a 0.1 sec. loading and 0.6 sec. rest period. The major purpose of this preconditioning is to set up the instrumentation. Tests can be executed at any temperature. For this study the test temperature varied according to the geographic location of the GPS site.

3.4 The procedure

A procedure to estimate the permanent deformation of asphalt concrete pavement based on the constant height repetitive simple shear test (RSST-CH) was presented by Sousa and Solaimanian (1994). Figure 37 diagrams a nomograph of the procedure. It is composed of four quadrants and it should be followed clockwise starting in quadrant 1.

QUADRANT 1 - *ESALs versus Rut Depth*

Step 1. Determine number of ESALs for design life

Step 2. Select maximum allowable rut depth

In the example, a 1,000,000 ESAL design life was selected. The maximum acceptable rut depth was selected to be 0.5 in.

QUADRANT 2 - *Rut Depth versus Permanent Shear Strain.*
Step 3. Using the maximum allowable rut depth the maximum allowable permanent shear strain is determined based on a relationship between rut depth and maximum shear strain.

A series of finite element analyses of the permanent deformation response of the pavement sections (similar to those presented in Figures 31 and 32) was conducted using material properties obtained from a series of volumetric, uniaxial, shear and frequency sweep tests. In the analyses, non-linear elastic, viscous and plastic properties of the mix were incorporated into a constitutive relationship (Sousa, Weissman, et al. 1993). It was observed that the variation of the maximum shear strain in the pavement and the rut depth is linear. The variation of rut depth with maximum permanent shear strain is illustrated in Quadrant 2 in log-log scale. This relationship was obtained for several loading and material conditions:

Rut Depth (in.) = 11 * Maximum Permanent Shear Strain *(1)*

This relationship seems to hold true regardless of the following:
- pavement temperature (simulated using different material properties),
- time of loading (simulated using 0.3 ON and 0.4 OFF and 0.1 ON and 0.6 OFF)
- material properties (changing nonlinear elastic, viscous and plastic properties)
- tire pressure and load magnitude (100, 200 and 500 psi were used for a given tire size).

However it should be validated for different pavement types, thicknesses and for non-uniform variation of material properties.

QUADRANT 3 - *Permanent Shear Strain versus Cycles*

Step 4. Determine Mean Highest 7-Day Pavement Temperature at the Site at 2 in Pavement Depth

Step 5. Execute RSST-CH test @ 10 psi at that temperature

Step 6- The number of cycles in RSST-CH is determined to yield maximum allowable shear strain based on the relationship between shear strain and number of cycles obtained *from RSST-CH.*

SHRP binder/mixture specifications are developed based on maximum and minimum pavement temperatures. Maximum pavement temperature was defined as the average maximum temperature for seven consecutive days. It is believed that rutting correlates better with this temperature than with mean monthly maximum or average yearly maximum pavement temperatures.

The sites of the General Pavement Studies (GPS), used to develop the relationship, cover diverse environmental conditions. The maximum pavement temperature for these sites varies within a wide range. In order to calculate the maximum pavement temperature for the GPS sites two or three nearest weather stations to each site were selected. The average maximum air temperature for seven consecutive days (giving largest value) was determined for these stations based on the years for which weather records were available. This value was used as the maximum air temperature at the site.

Once the maximum pavement temperature at the surface was found the maximum pavement temperature for any depth less than 8 inches is found through an empirical formula (Solaimanian and Kennedy, 1993).

It seems that most of the permanent deformation due to the shear stresses developing near the edge of the tires takes place at depths up to 4 inches (Sousa, Crauss and Monismith 1991). The maximum shear stress computed from non-linear visco-elastic analysis took place at about 2 inches. For this reason and also because at this depth the ranges of temperatures computed for the GPS sections fell within reasonable testing ranges, the maximum pavement temperature at a 2 in. depth was selected as the testing temperature for each of the GPS sections.

Four typical graphs of the Permanent Shear Strain obtained from the Simple Shear Test at Constant Height executed at 10 psi stress amplitude (with 0.1 sec loading time and with 0.6 sec. rest period) and at the mean highest 7-day maximum weekly pavement temperature encountered at 2 in. depth (RSST-CH@10 psi @0.10 N, 0.6 OFF@Mean High. 7-Day Max Pav Temp @ 2 in. Depth) versus number of cycles obtained for some of the mixes are presented in Quadrant 3. These relationships were obtained for dense graded mixes at different asphalt contents (mix A with higher asphalt content then mix D)

This graph permits the determination of the number of cycles in RSST-CH required to reach a given permanent shear strain level (in this case shear strain= 0.04545 = 0.5/11). We can see that mix D performs better than mix A.

QUADRANT 4 - *Cycles (RSST-CH) versus ESAL*

Step 7- The number of ESALs that can be carried by that mix before the desired rut depth (0.5 in.) is reached is determined using the relationship between ESALs and RSST-CH *number of cycles.*

An investigation of the relationship between cycles in RSST-CH and ESAL was made for pavements less than 10 years old (Sousa and Solaimanian 1994). The equation of the best fit is given by:

$$\log(\mathbf{Cycles}) = -4.36 + 1.24 \log(\mathbf{ESAL}) \qquad (2)$$

This relationship was obtained with an $R^2 = 0.80$. This value is indicative of a very good correlation specially if the following is taken into consideration:

-Although the GPS sections where selected based on distresses primarily due to the asphalt concrete layers In some cases the rut might have been due to densification, subgrade effect, pavement surface irregularities, etc.

- The RSST-CH was executed with specimens which had already been subjected to traffic in various degrees. Their behavior is different from those obtained from newly paved mixtures.

- The calculation of the maximum pavement temperature at 2 in. depth is just an estimate of the real temperature.

- The testing rate is 0.1 sec loading with 0.6 unloading while in the pavement the rate is closer to 0.02 sec loading with almost random spacing.

- ESALs were not actually measured but were extrapolated based on DOT data.

To fully take advantage of these results consideration should be given to the fact that in a mixture design tests will be executed under unaged or aged conditions and no traffic conditions. These results were obtained for out-of-the-wheel-path field cores that have been subjected to aging but limited traffic.

Equation (2) is basically an equation of shift factors relating the number of cycles in laboratory of the RSST-CH with the number of ESALs in the field.

QUADRANT 1 - *ESAL versus Rut Depth*

With the results obtained from the analysis we can identify which of the mixes would satisfy the design conditions. In the example only mixes C and D satisfy the requirements. Considerations should be given to reliability and adjustments may be required. Depending on the level of reliability mix D might be the only one to satisfy the requirements.

Although quite simple, this procedure is robust as it addresses a few important aspects of the mix behavior and is also of practical implementation.

3.5 The mix design

A diagram of the proposed mix design approach is presented in Figure 38. To facilitate the explanation of the procedure assume that one wants to design a pavement in a location where the mean highest 7-day maximum pavement temperature at 2 in. depth is 50 C. Further assume that one wants to insure that the pavement would be designed to carry 1,500,000 ESALs in the first 5 years. Design criteria would impose a 0.5 in. rut depth as the maximum acceptable (a 0.375 in. rut depth limit, for instance, could also be adopted).

In this case instead of only executing tests at about 3.2 % air void content where maximum resistance to shear stresses is expected for this dense graded mix, as suggested in the considerations regarding the Shear Procedure Without Reduction in Volume, specimens were compacted also to different air void contents. The purpose of this experiment design was also to illustrate the strong effect of air void content in mix performance.

1. Prepare Specimens:

A series of specimens would be prepared at different air-void and asphalt contents. Given that the shear test at constant height is particularly sensitive to aggregate structure (Sousa et al., 1990, Harvey and Monismith, 1993, and Harvey, Sousa and Monismith 1994) careful selection of compaction method and compaction temperature should be made. At this stage selection of binder type, modifiers, gradations and aggregate type should also be made. In this example Watsonville Aggregate and Valley Asphalt were selected.

2. Condition Specimens:

After specimen fabrication selection of conditioning procedure should take place. This selection should be site specific. Moisture conditioning, freeze-thaw and aging conditioning protocols should be selected.

3. Execute Test:

Before executing the test, selection of test temperature must be made. Test temperature should be representative of the highest pavement temperatures encountered in the field. Reliability considerations should be incorporated in the temperature selection. In this example the specimens were tested at the maximum 7-day pavement temperature at 2 in. depth, under Constant Height Repetitive Simple Shear Test at 10 psi (0.1 ON, 0.6 OFF). The results are presented in Table 1.

4. Determine ESAL to design rut depth:

Based on the procedure presented in section IV, the results from these calculations are presented in the last column of Table 1 and Figure 39. It can be observed that changing the asphalt content between 4.5% and 6% and air voids content between 2.8% and 5.5% varies ESAL from 245780 to 2.6 million. In this design a design rut depth of 0.5 in. was selected.

Figure 40 shows three views of the variation of the number of cycles (RSST-CH) to reach 0.04545 (= 0.5 / 11 from equation *(1)*) permanent shear strain versus void and asphalt contents. The relative influence of asphalt and void content on the performance of the mixture can clearly be observed. Figures 12 shows the variation of the ESAL to reach 0.5 in. with air voids and asphalt contents based on equation *(2)*.

Reliability considerations should be incorporated as presented by Sousa et all (1993) and Sousa, Harvey, Bouldin and Azevedo, 1994.

5. Select Asphalt Content:

Only a limited number of mixes can sustain the selected 1,500,000 design ESALs before reaching the rut depth of 0.5 in. (see Table 1) and Figure 41. The results suggest that there are a few combinations of asphalt content and air-void content that will probably satisfy the initial condition. However, to minimize compaction effort it is beneficial that the combination with the highest possible void would be selected. This would insure less compactive effort during field compaction and faster improvement in performance with traffic densification. The selected value would be asphalt content of 4.9% and traffic compaction would insure that the required resistance would occur when air-void content is less than 5.1 % but more than 2.9% (see Figure 41). If fatigue were of concern (after executing the proper tests) one might select an asphalt content of 5.4% but one would be forced to realize that adequate performance would only occur when traffic compacts the mix in the field to an air void content between 4.2% and 3.0% (see Figure 41).

With reliability considerations this might not be an acceptable mixture. A good compromise, for instance, might be selection of an asphalt content of 5.1 %.

It is interesting to note that the first choice of asphalt content (no fatigue considerations) would probably be obtained by the Hveem method. This agreement in one condition which experience has demonstrated good behavior in the field reinforces the reliability of the proposed methodology. One should not however assume that the Hveem would suffice, as the new methodology is capable of reliable predictions in conditions well above those limiting the Hveem methodology.

This example illustrates graphically how this abridged procedure (based on the GPS data) can be used for mix design providing the engineer with tools to make rational decisions.

Design Considerations

In the initial stages of the process of densification the mixture will exhibit increasingly better resistance to permanent deformation. This can be attributed to better particle-to-particle contact that in turn, promotes higher dilation forces (under shear strains) which cause the mix to become more stable. This stability is lost when the densification process gets to a point where the binder has no other place to go and starts acting as a "lubricant" between the aggregate particles. This can clearly be seen in Figure 39 by following the lines (at a given asphalt content) from a high void content to a low void content. A significant drop in resistance can be observed around 3%. Note that this laboratory observation is consistent with the field data presented in Figure 30. This similarity is also an indication that the RSST-CH is capturing the most important aspects of the rutting phenomenon.

Following the 4.5% asphalt content line (the highest line) in Figure 40b from high void content to low void content, it can be observed that as the mixture densifies, its resistance to permanent deformation increases considerably. If enough energy is provided the mixture will eventually densify past the 3% void content level and reduces its resistance to permanent deformation.

A different behavior can be noted by observing the performance of a 6% asphalt content mixture (the lowest line). Following its behavior from high void content to low void content it can observed that the increase in resistance is not very pronounced. Therefore, a steady stream of trucks could rapidly cause it to reach 3% void level.

Recognizing that a mixture can densify under traffic it should be designed so that when it reaches the void content at which it exhibits its best performance it will be able to take the design traffic. This concept is only valid if the laboratory-compaction method used in the process of preparing mixtures at different void contents produces aggregate structures similar to those that are obtained from traffic densification.

From the mix design experiment it can be observed that mixtures compacted at 5 % voids hardly resist any permanent deformation (very few ESALs). However it is known that similar mixtures are placed in California with 5-7% void level and they exhibit good performance. It is therefore reasonable to assume that they have densified with traffic and they resist well because they have reached void content levels between 3 and 4%. This emphasizes the need to compact the mixtures in the laboratory to certain void content levels and not to arbitrary compaction energies or "standard compactions", with a compaction procedure that creates aggregate structure similar to those obtained in the field after traffic densification.

Instead of evaluating 16 mixes to determine the behavior of the mix in a matrix of air-void and asphalt contents one could only evaluate 4 mixes (with two replicates) at air-void content of 3.2% (+/-0.2%). We would obtain just a slice through the surface (near the point of highest resistance) similar to the highest line in Figure 40 c. If during laboratory compaction specimen air void content is not exactly 3.2% the expected performance at 3.2% could be derived (interpolated or extrapolated) assuming a log(ESAL)-linear(VOIDS) relationship at a given asphalt content from performance of specimens at other air void content. In this analysis it must be recognized that shear resistance increases with decreasing air void content up to a maximum below which shear resistance decreases rapidly with air void content.

Based on these eight tests alone selection of the asphalt content could be made. Considering that the mix should contain as much asphalt as possible to improve fatigue and raveling behavior, the highest asphalt content that would satisfy the rutting criteria should be selected taking into account reliability considerations (Sousa et al., 1993 and Sousa, Harvey, Bouldin and Azevedo, 1994). This methodology could be used to determine pay factors for contractors that exceed the required asphalt content.

Aging Considerations

One of the factors affecting rutting is aging. As the mix ages the binder stiffens, the elastic strains decrease and the permanent deformation accumulated at each load application decreases. The relationship presented in QUADRANT 4 was obtained from relating the results of the RSST-CH on pavement cores three to 14 years old with the pavement performance. If a procedure is to be developed based on these findings then a correlation between laboratory-fabricated specimens and field performance has to be established.

The specimens tested in the mix design example were prepared in the laboratory and only went through the short-term oven aging procedure (4 hours at 135 C on the loose mix). This aging procedure is supposed to simulate aging effects occurring during the mixing process in the plant. It is therefore likely that a specimen prepared in this fashion might not be representative of a specimen that was 3 or 4 years in the field. The long-term oven aging might be a more appropriate procedure. The relative effects of long term and short term are quite different (see Figure 42) This will significantly affect the number of ESALs predicted by the procedure and emphasizes the need to establish a criteria of laboratory specimen fabrication that will produce specimens with characteristics similar to those encountered in the field. Before this procedure can be used to its fullest extent, research has to determine the best test protocol in laboratory to account for aging effects.

Water Sensitivity Considerations

Water can affect the bond between the binder and the aggregate and reduce the resistance to shear stresses. Figure 43 diagrams the relative performance in the RSST-CH from two pairs of specimens of the same mix; one (WET1 and WET2) was subjected to the water sensitivity procedure proposed by SHRP-A003A using the ECS and the other was not (DRY1 and DRY 2). These results suggest that the selection of the asphalt content on a mix design can be affected by water sensitivity considerations. Furthermore the relationship presented in equation *(2)* was developed from field specimens that were exposed to water to different degrees.

Other Considerations

The proposed mix design procedure can be implemented with or without considerations for aging and water sensitivity. Figure 44 diagrams an example of a conceptual design of the various degrees of complexity that can be incorporated in the procedure.

In the simplest approach the tests would be conducted on specimens subjected only to short-term oven aging (STOA). In this case, one should recognize that the mix in the field will probably age and the results would be representative of the first year of traffic. Therefore, design life Level A should correspond to the first few years of traffic and an asphalt content of 4.8% might be selected. If in this first year of traffic water sensitivity was a concern then the procedure could be used on specimens subjected to the water condition protocol appropriate for the site location.

If the 10-year design life was, for instance, design life Level B, one could select a lower asphalt content, say 4.6 %, without doing any more tests. This would be a conservative approach. One could also consider that aging would take place (without actually doing the long-term aging procedure) and select the 4.8% asphalt content anyway. It must be noted that these numbers are used only as an example. The magnitude of savings in binder, or extra performance in fatigue life, can vary significantly from mix to mix.

However, if the 10-year design life is to reach Level C, there is no guarantee that a mix just subjected to STOA would be able to sustain that traffic level. In this case, one might consider actually executing the long-term oven aging procedure (LTOA) to simulate field aging and then test the mix. It can be seen in the example that in this case with 4.6 % asphalt content the Level C design level would be reached.

If water sensitivity and long-term aging were of concern, then both protocols could be followed and the asphalt content selected on that basis. This mix design concept is based on the assumption that LTOA and water sensitivity protocols are site specific (i.e., on climates that never freeze, it should not be necessary to execute the freeze cycles of the ECS protocol).

In the Marshal mix design procedure mixes with increasing asphalt contents are compacted with the same energy (50 or 75 blows) yielding specimens with decreasing air void contents. These specimens are then tested and the correspondent stability values plotted against asphalt content normally exhibit a maximum value If Figures 40 or 41 are sliced through a plain corresponding to mixes prepared with identical compaction energies (see Sousa, Tayebali, et al. 1993) reveals a curve (see Figure 45) which also contains a peak (in the figure the peak would occur at 4.1% void content and 5.0% asphalt content). Clearly the position of the peak varies with the location of the slice, which is to say, with the selected compaction effort. Considering the incresing volumes of traffic, increasing loads and tire pressures it is difficult to select a compaction effort which might be representative of that imposed by the future traffic. For this reason it is proposed that mixes be compacted in laboratory to void contents at which they exhibit their best performance (or expected in the field after traffic densification).

3.6 Summary

A mix design procedure to estimate the rutting resistance of dense-graded asphalt concrete mixes has been presented. It is based on a relationship between rut depth and maximum shear strain developed from a non-linear elastic, visco-plastic constitutive equation to describe the behavior of the asphalt concrete incorporated into a finite element program. This relationship seems to be independent of material properties (it might be dependent on pavement structure for thin pavement sections).

This procedure was derived from data obtained from 40 GPS sections around the North-American continent. It is mainly based on the execution of a repetitive simple shear test at 10 psi under constant height conditions and at the mean highest 7-day maximum pavement temperature encountered at 2 in. depth. This depth was selected because computations have shown that the maximum shear stresses, causing the permanent deformation in the pavements, are encountered at 2 in. depth.

The fundamental tie between the laboratory tests and the field was derived from determining a relationship between the number of load applications in the laboratory test and the number of ESALs in the field that would cause the same maximum shear strain, in the laboratory specimen and in the pavement section, respectively. For pavements that did not exhibit significant aging the relationship was obtained with an R^2 of 0.80.

For dense graded mix design, it is proposed that mixes with different asphalt contents be compacted to 3.2% (+/- 0.2%) void content (or to the air void content expected after traffic densification) using a procedure that creates an aggregate structure similar to that created by traffic densification. It is possible that mixes might have to be aged and/or moisture conditioned to simulate conditions occurring 2 to 3 years after construction. Using the procedure presented herein, the expected number of ESALs to reach a given rut depth can be determined for each asphalt content. Recognizing that higher asphalt contents promote better fatigue life and minimize raveling potential, the highest asphalt content that satisfies the required design life (adjusted for reliability considerations) should be selected.

3.7 Chapter References

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Sousa, J. B. and Weissman, S. L. "Modeling Permanent Deformation of Asphalt-Aggregate Mixes," Paper accepted for publication at the AAPT 1994 annual meeting.

Table 3 - Typical summary of results from a mix design experiment on a Watsonville aggregate with Valley Asphalt.

	AC Content	VOIDS Content	Axial Stress	Shear Stress	# Cyles to 3.0%	# Cylces to 4.0%	# Cylces to 4.545%	# Cylces to 5.0%	ESALS to 0.5 in
MA1J22.DAT	4.5	3.2	7.02	10.17	882	2510	3997	5598	2634835
MA2J42.DAT	4.9	3.0	8.11	10.14	530	1666	2794	3979	1973985
MB1J22.DAT	4.5	4.8	9.05	10.23	634	1730	2687	3798	1912791
MB3J42.DAT	4.9	4.9	8.78	10.14	616	1545	2255	3067	1660655
MA1J12.DAT	4.5	2.8	7.61	10.26	455	1260	1954	2722	1479451
MA3J22.DAT	5.5	3.3	8.45	10.11	637	1344	1851	2366	1416231
MB3J32.DAT	5.5	4.2	7.85	10.27	534	1136	1519	1960	1207540
MA3J32.DAT	5.5	3.7	6.84	10.04	461	1039	1481	1936	1183119
MA3J12.DAT	5.5	3.7	4.97	10.41	366	931	1392	1900	1125439
MA2J32.DAT	4.9	2.8	5.16	10.18	338	806	1197	1595	996467
MB1J12.DAT	4.5	5.3	5.65	10.25	233	729	1164	1872	974253
MA4J32.DAT	6.0	2.5	6.01	10.36	160	393	585	829	559379
MA4J12.DAT	6.0	1.8	8.06	10.36	110	253	356	505	374757
MA4J22.DAT	6.0	3.0	8.51	10.35	93	219	334	468	355966
MB3J22.DAT	5.5	5.5	7.33	10.30	106	230	320	422	343883
MB3J12.DAT	5.5	5.5	7.56	10.47	70	154	211	279	245782

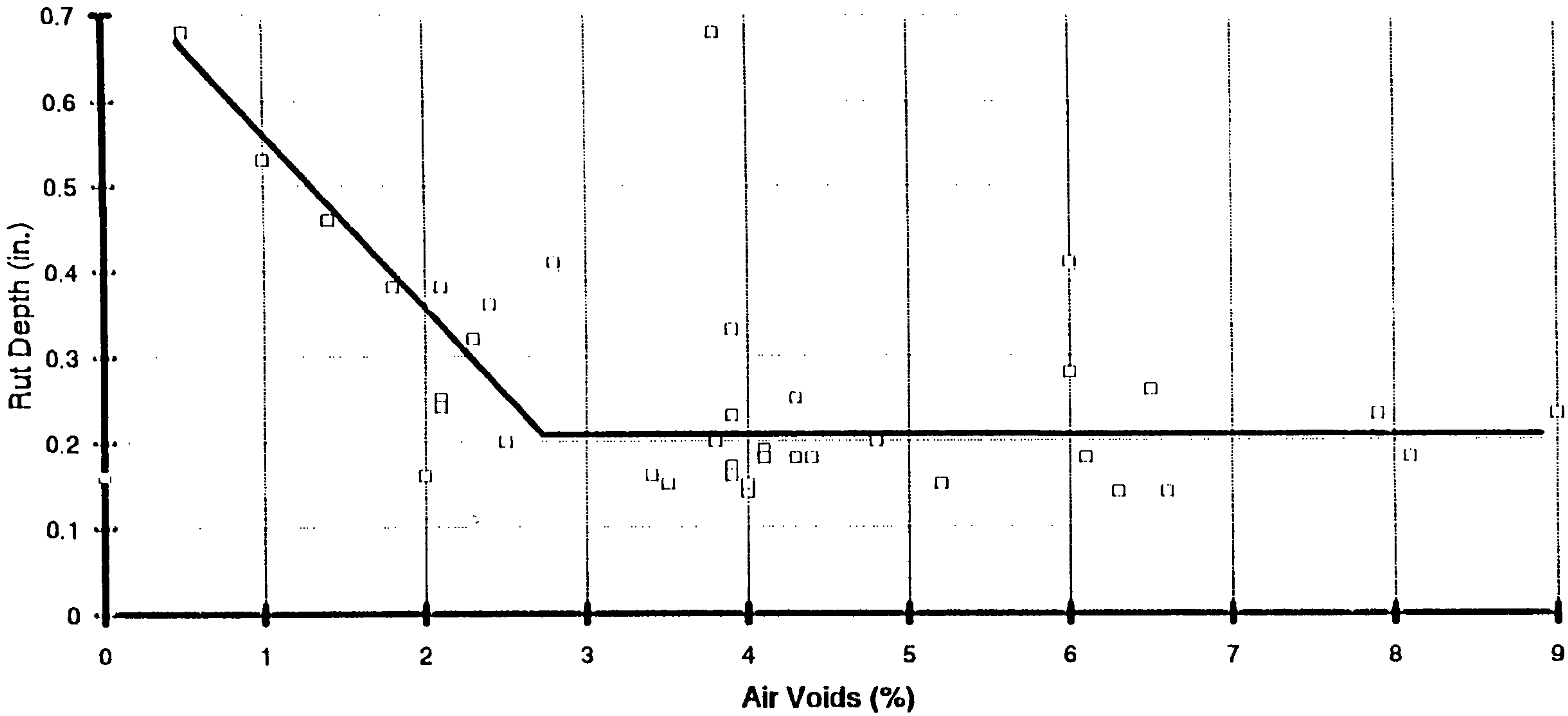

Figure 30 - Variation of rut depth with air voids for GPS pavement sections.

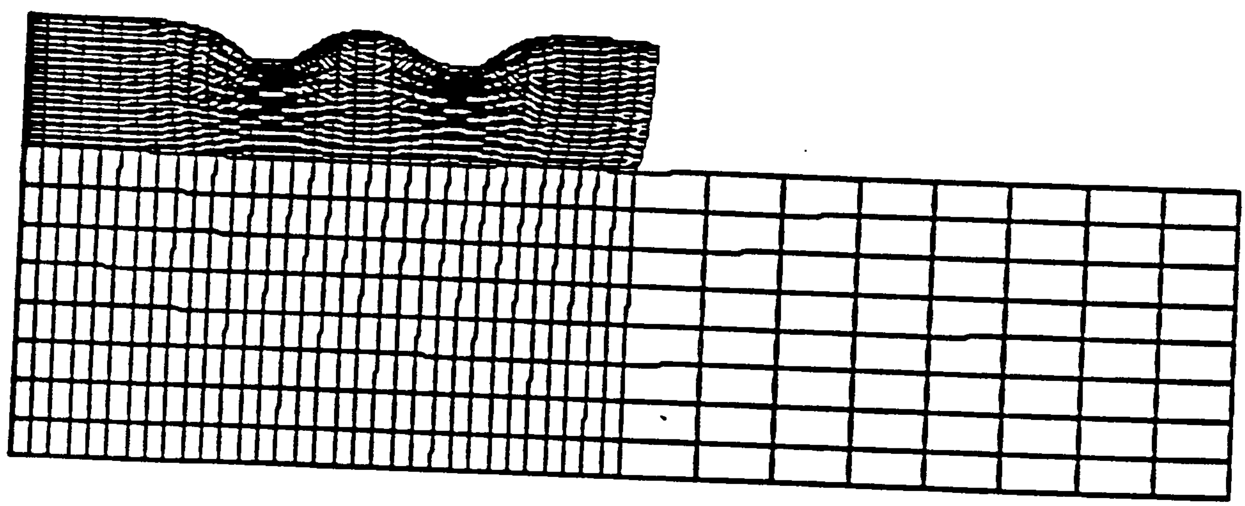

Figure 31 - Deformed finite-element mesh, second loading cycle.

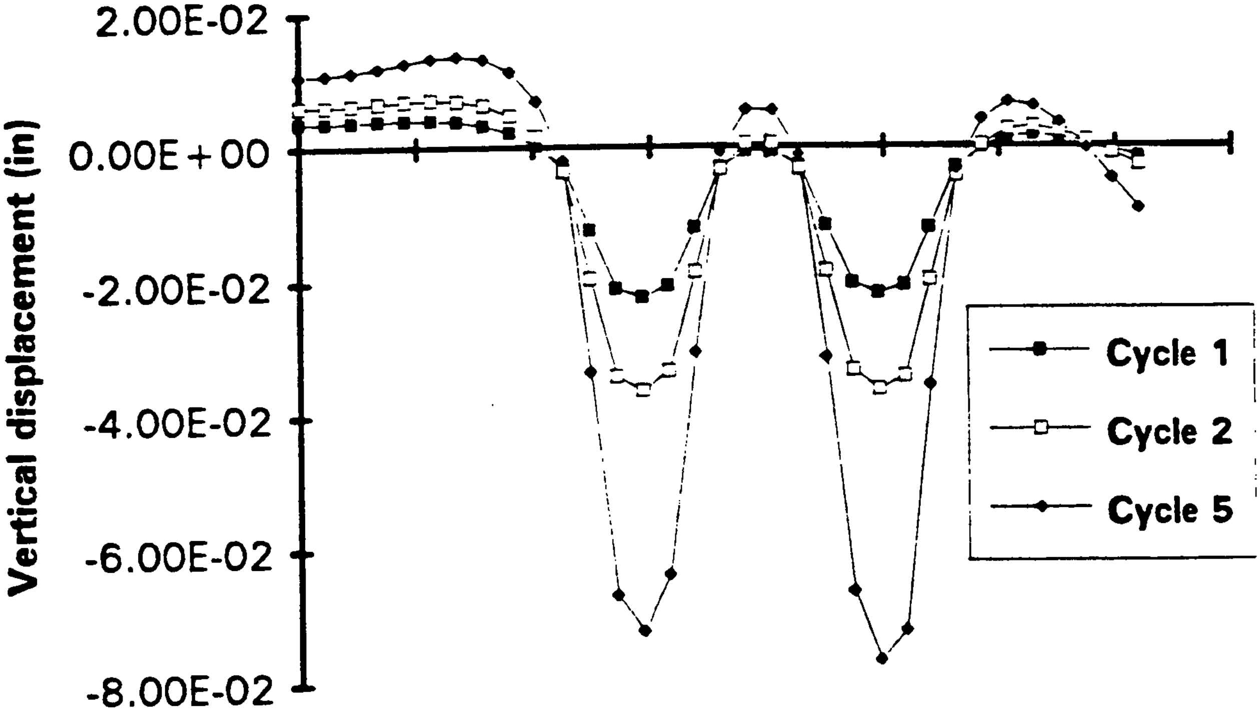

Figure 32 Variation of pavement profile with the number of load applications, stress level 500 psi, loading time 0.3 sec, rest period 0.4 seconds.

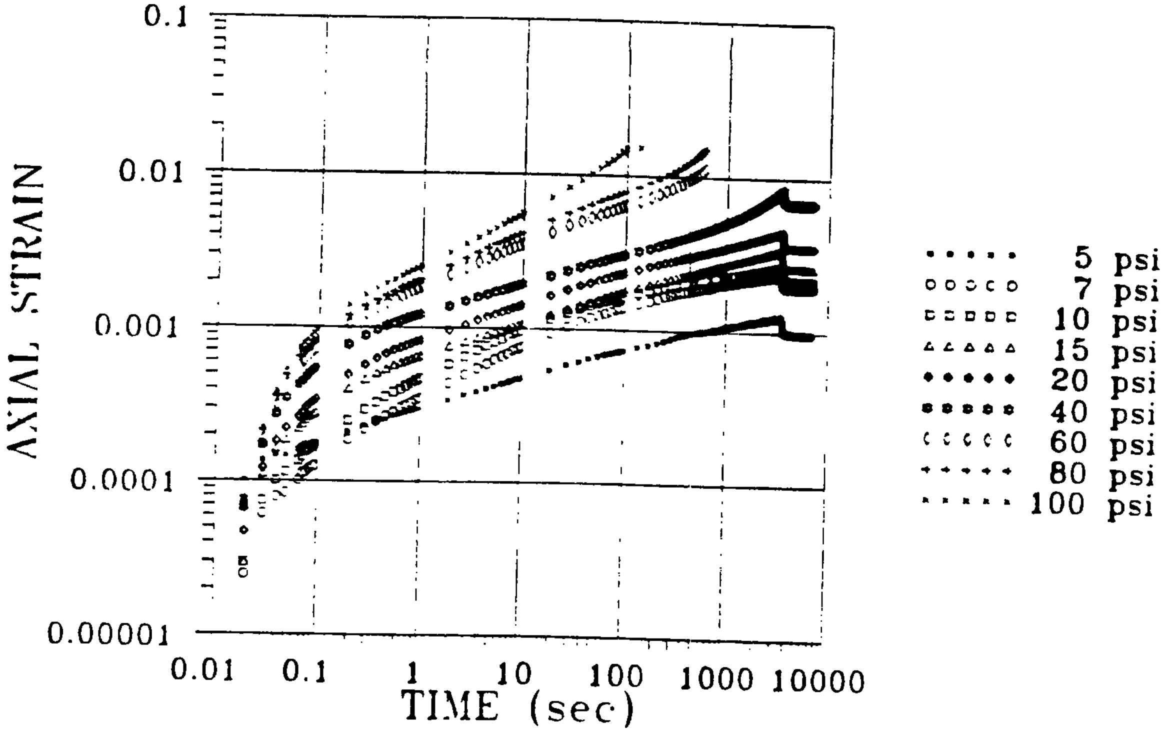

Figure 33 - Variation of the axial strain with time and stress level for an asphalt concrete mix tested under unconfined axial creep test at 40 C.

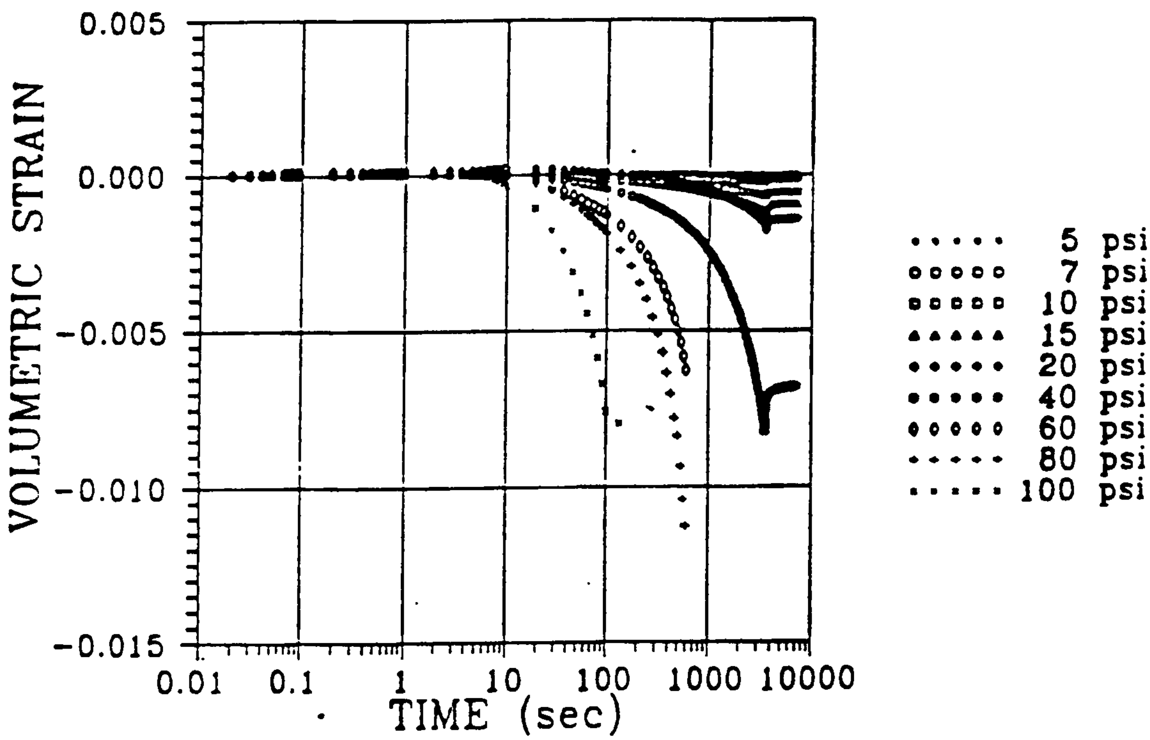

Figure 34 - Variation of the volumetric strain with time and stress level for an asphalt concrete mix tested under unconfined axial creep test at 40 C (negative corresponds to dilation).

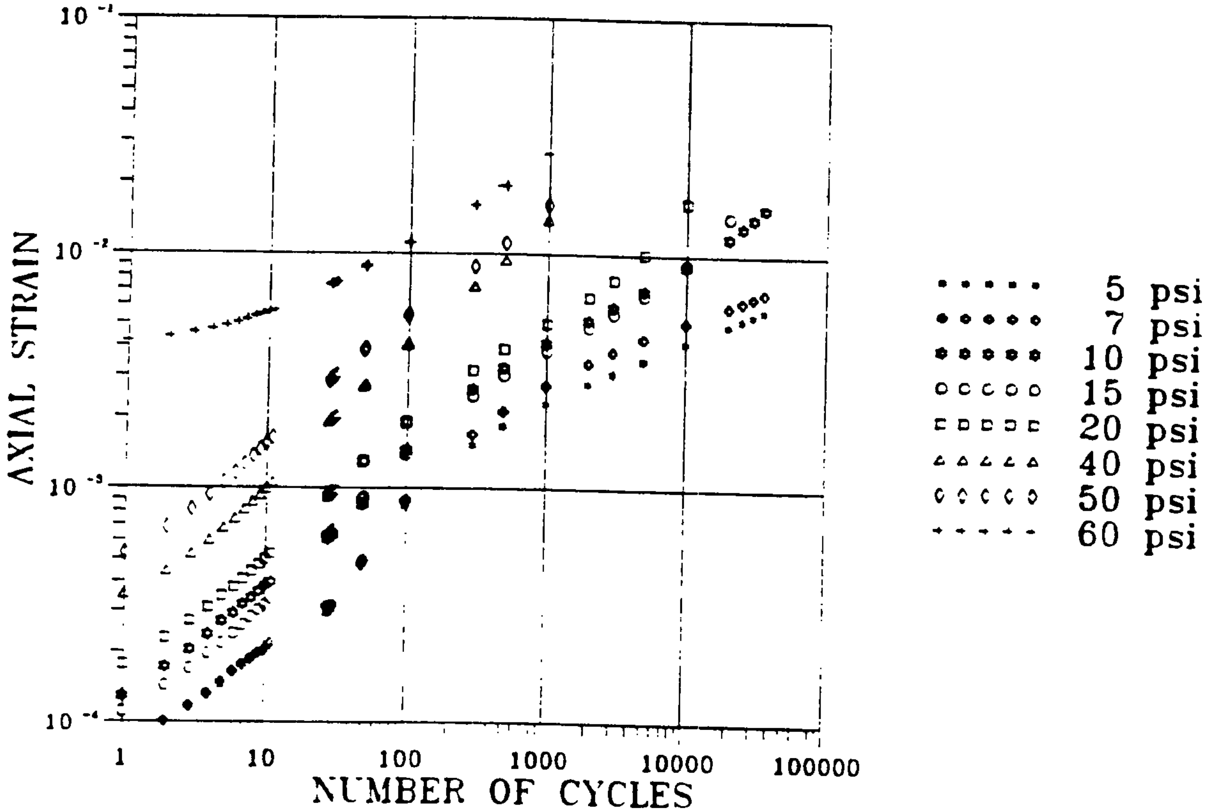

Figure 35 - Variation of the axial strain with time and stress level for an asphalt concrete mix tested under unconfined repetitive axial test at 40 C .

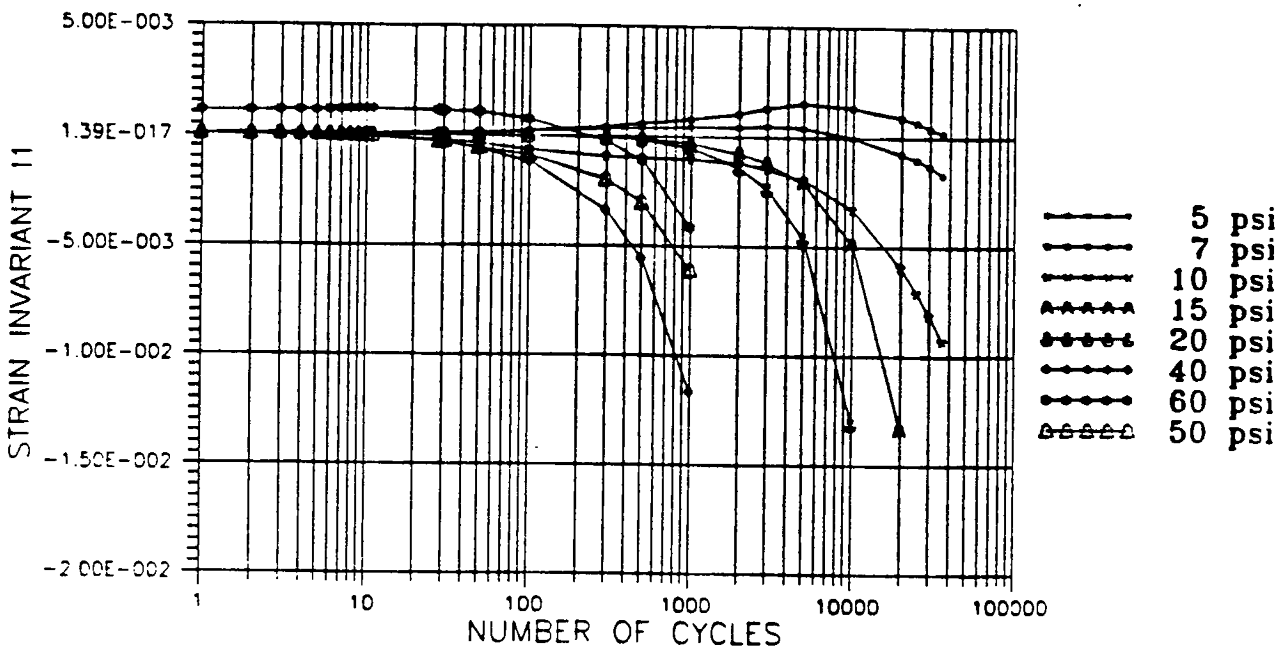

Figure 36 . Variation of the strain invariant I_1 (volumetric strain) with time and stress level for an asphalt concrete mix tested under unconfined repetitive axial test at 40 C .

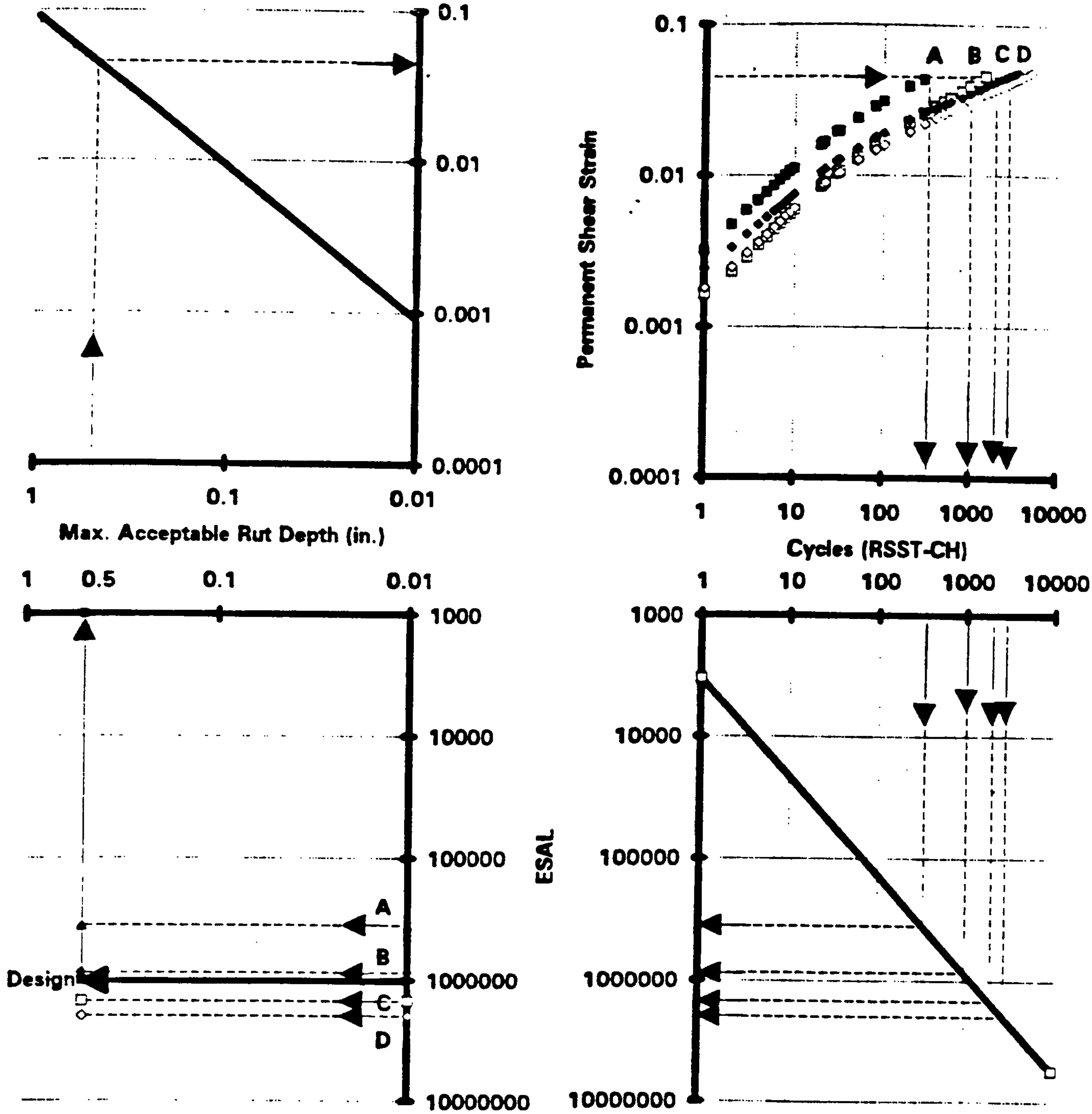

Figure 37 - Schematic diagram of the abridged procedure for permanent deformation.

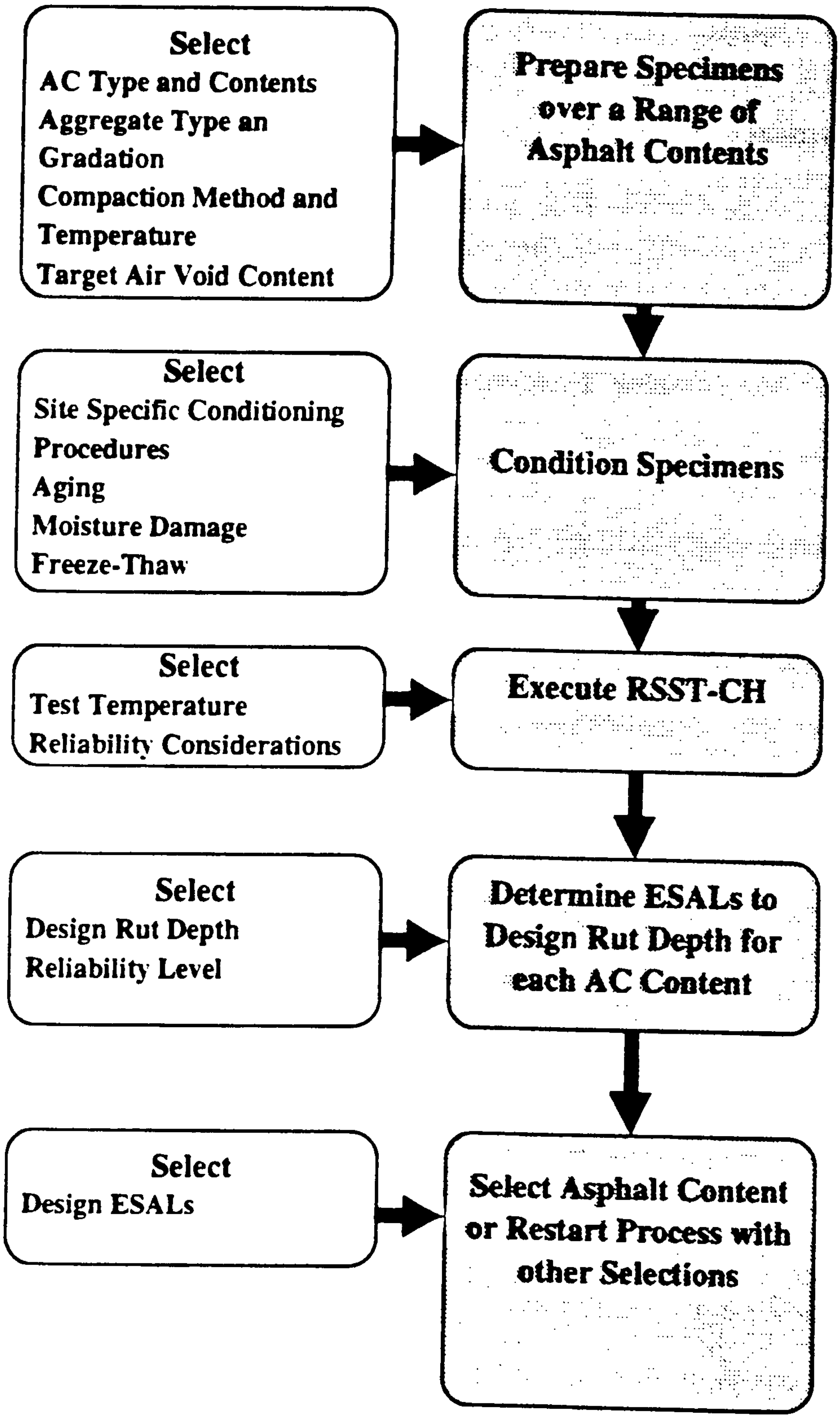

Figure 38 - Permanent deformation mix design flow chart

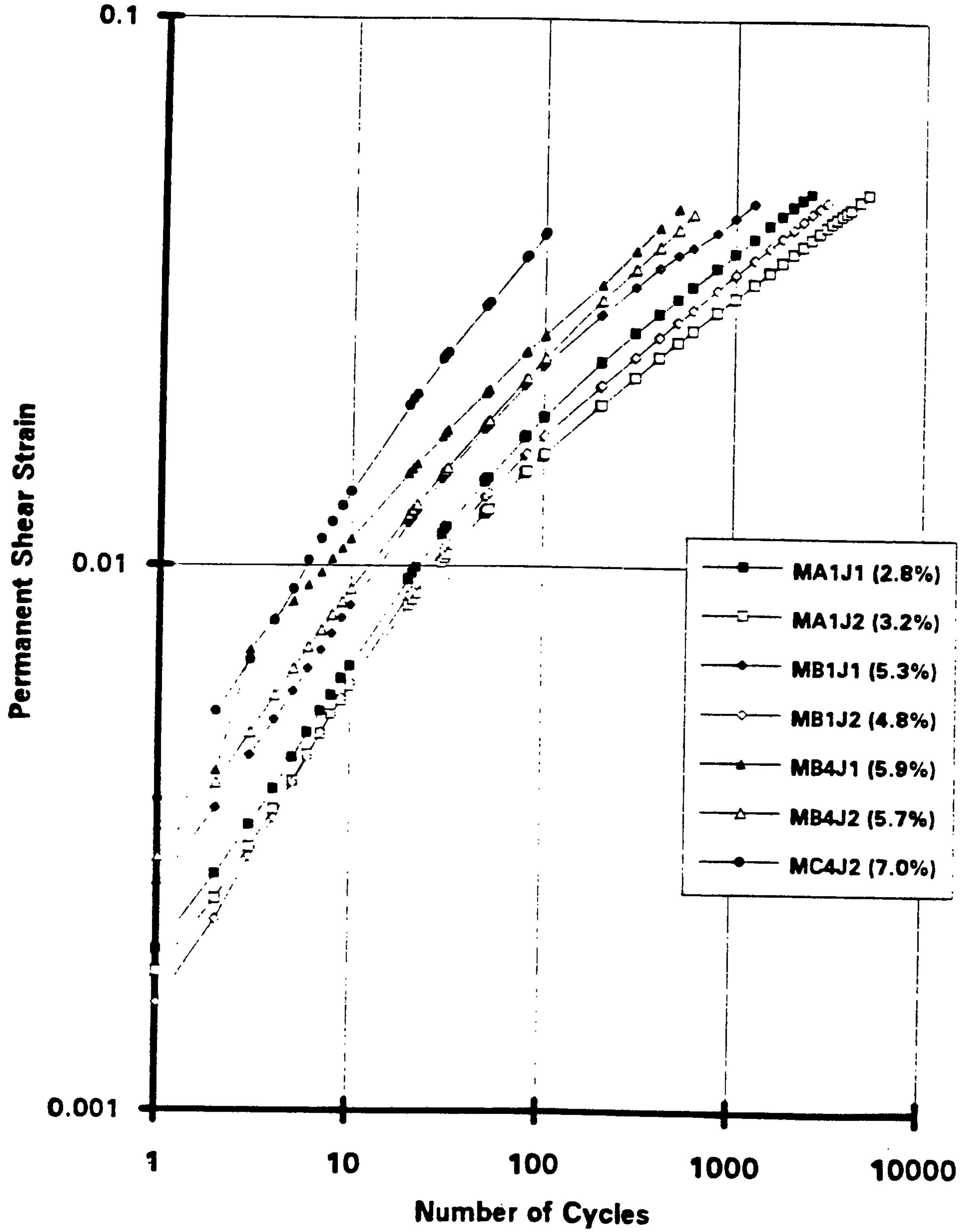

Figure 39 - Variation of the permanent shear strain in the RSST-CH at 50 C and 10 psi for mixes in Table 1 (values in parenthesis indicate air void content).

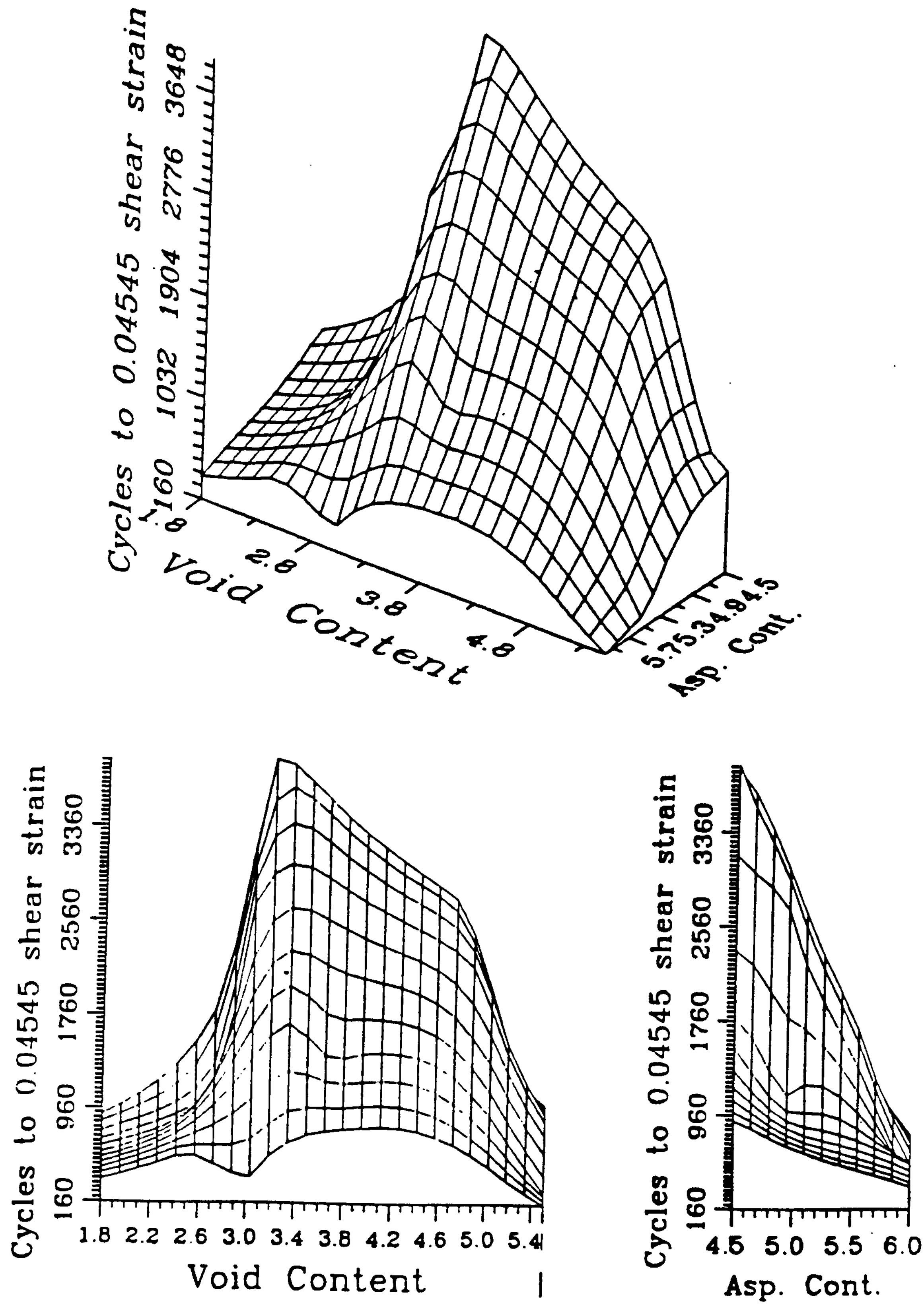

Figure 40 - Effect of asphalt and void content on the number of cycles to reach 0.04545 permanent shear strain.

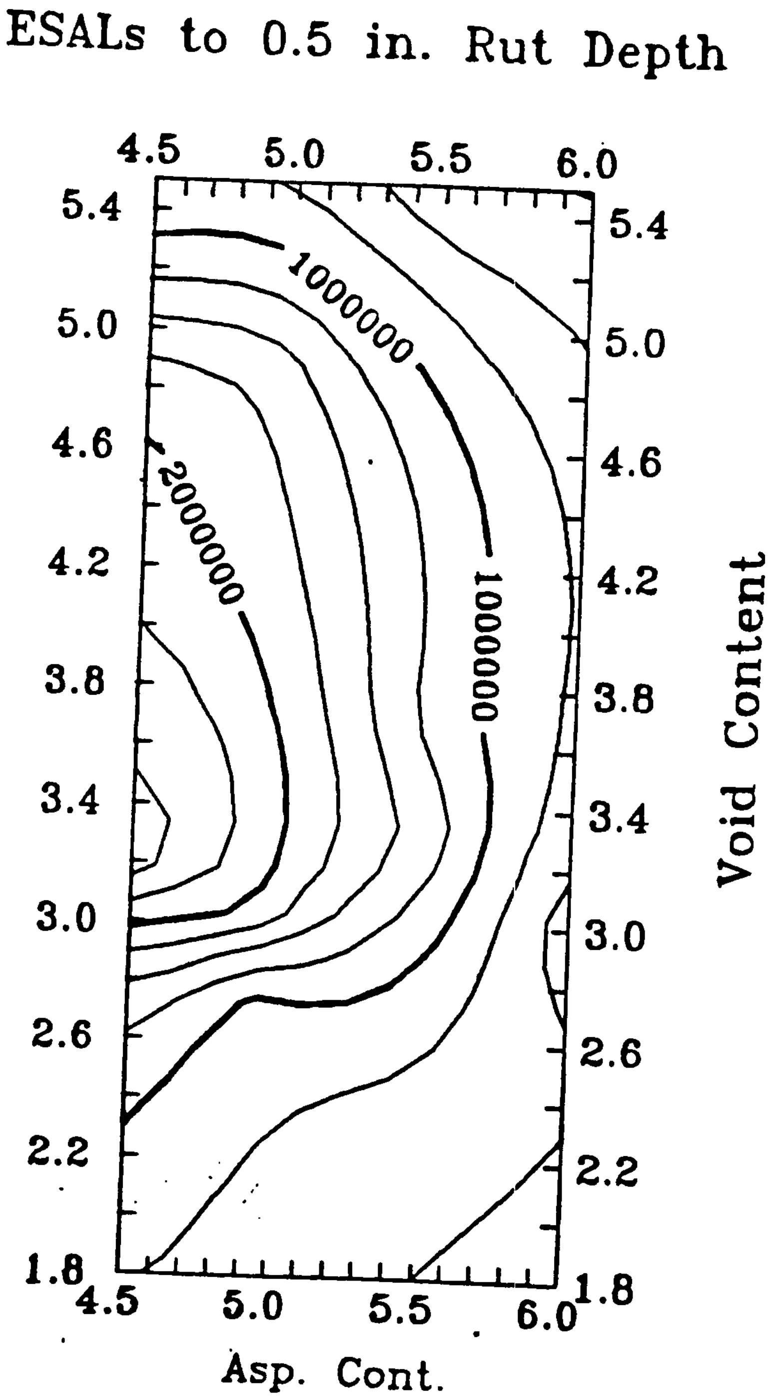

Figure 41 - Effect of asphalt and air void content in the number of ESALs to reach 0.5 in rut depth.

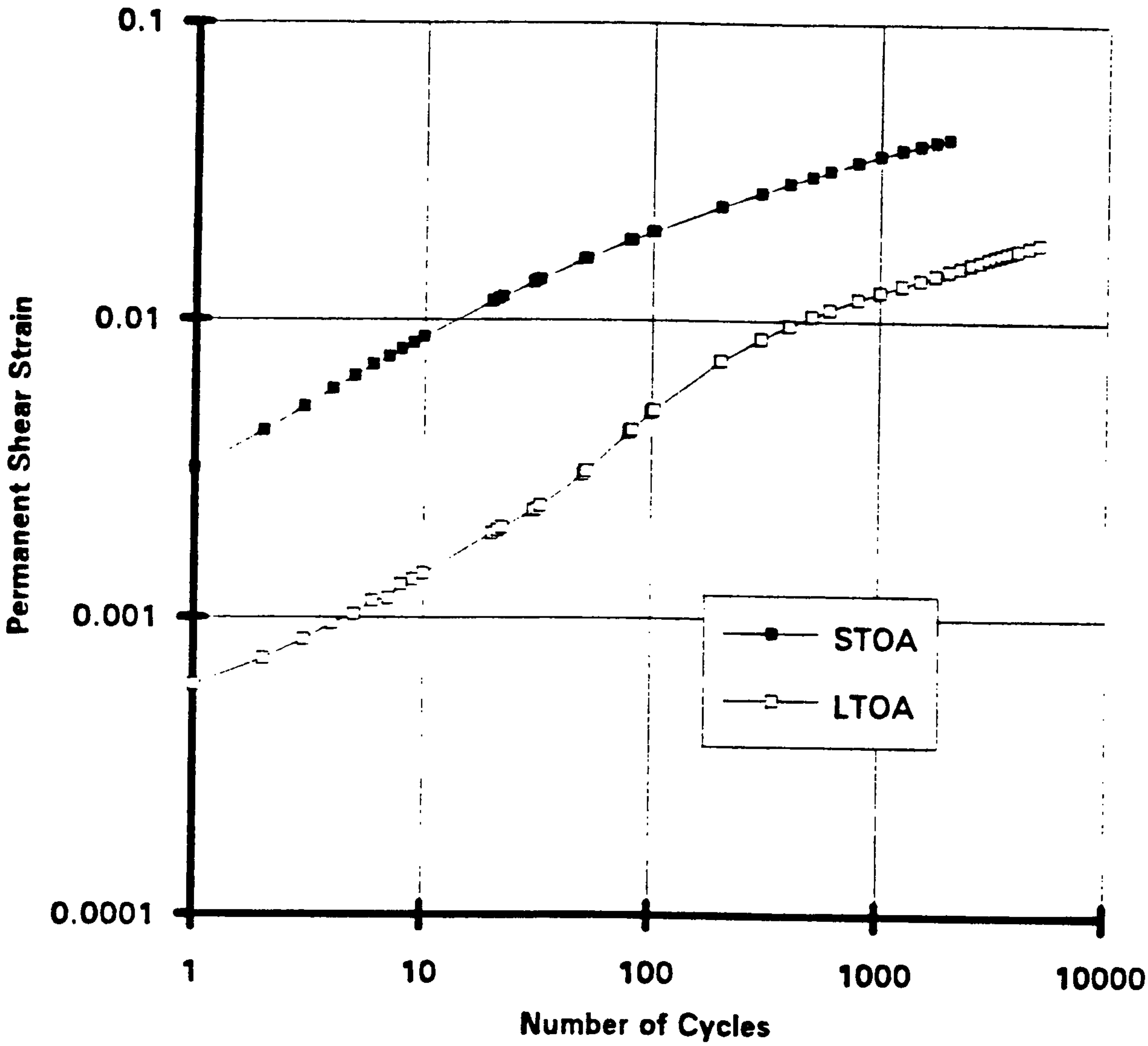

Figure 42 - Variation of permanent shear strain in RSST-CH on specimens of the same mix subjected to short-term and long-term aging.

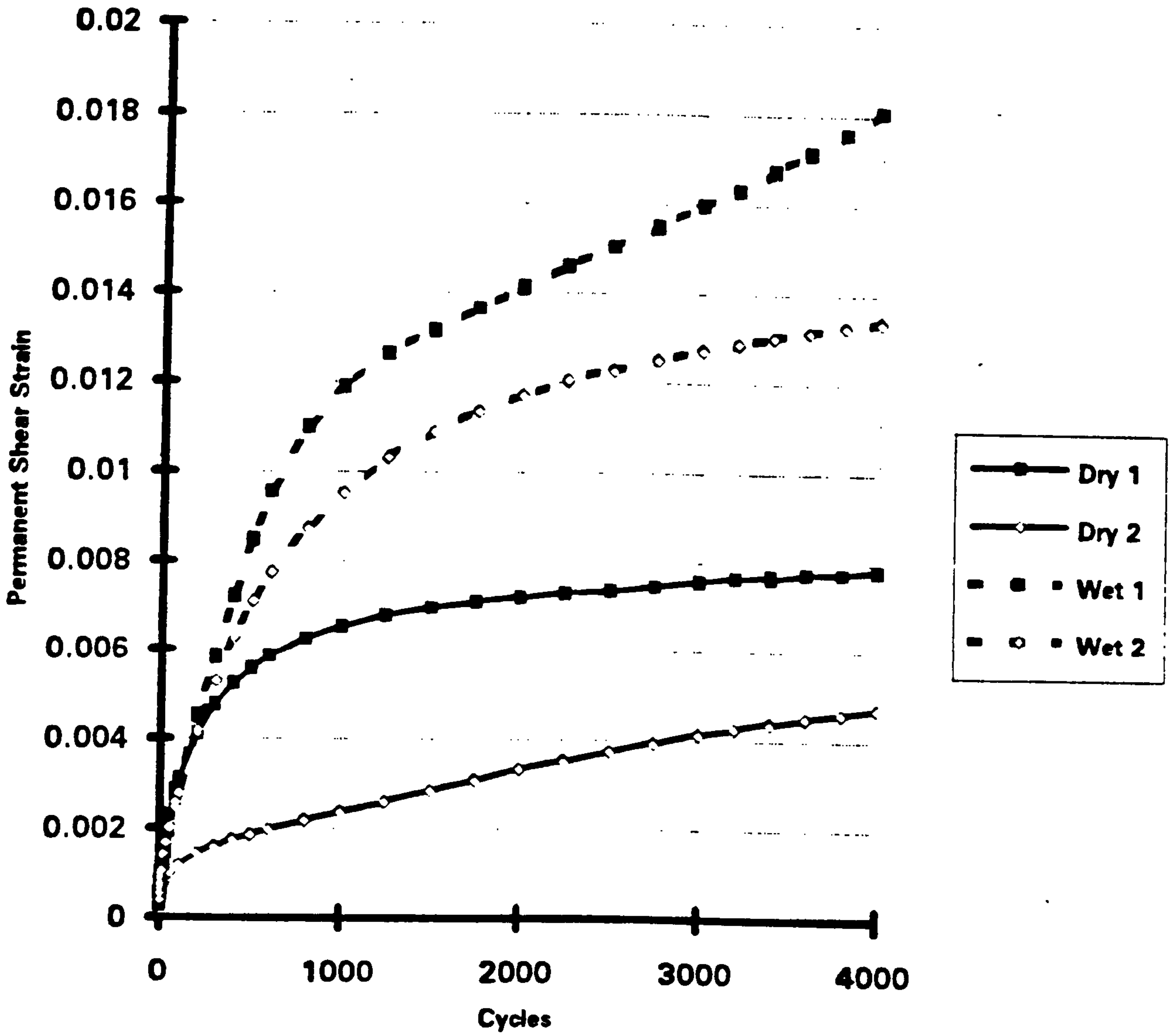

Figure 43 - Comparison of the variation of permanent shear strain in RSST-CH on specimens of the same mix subjected (WET) and not (DRY) to the ECS water conditioning protocol.

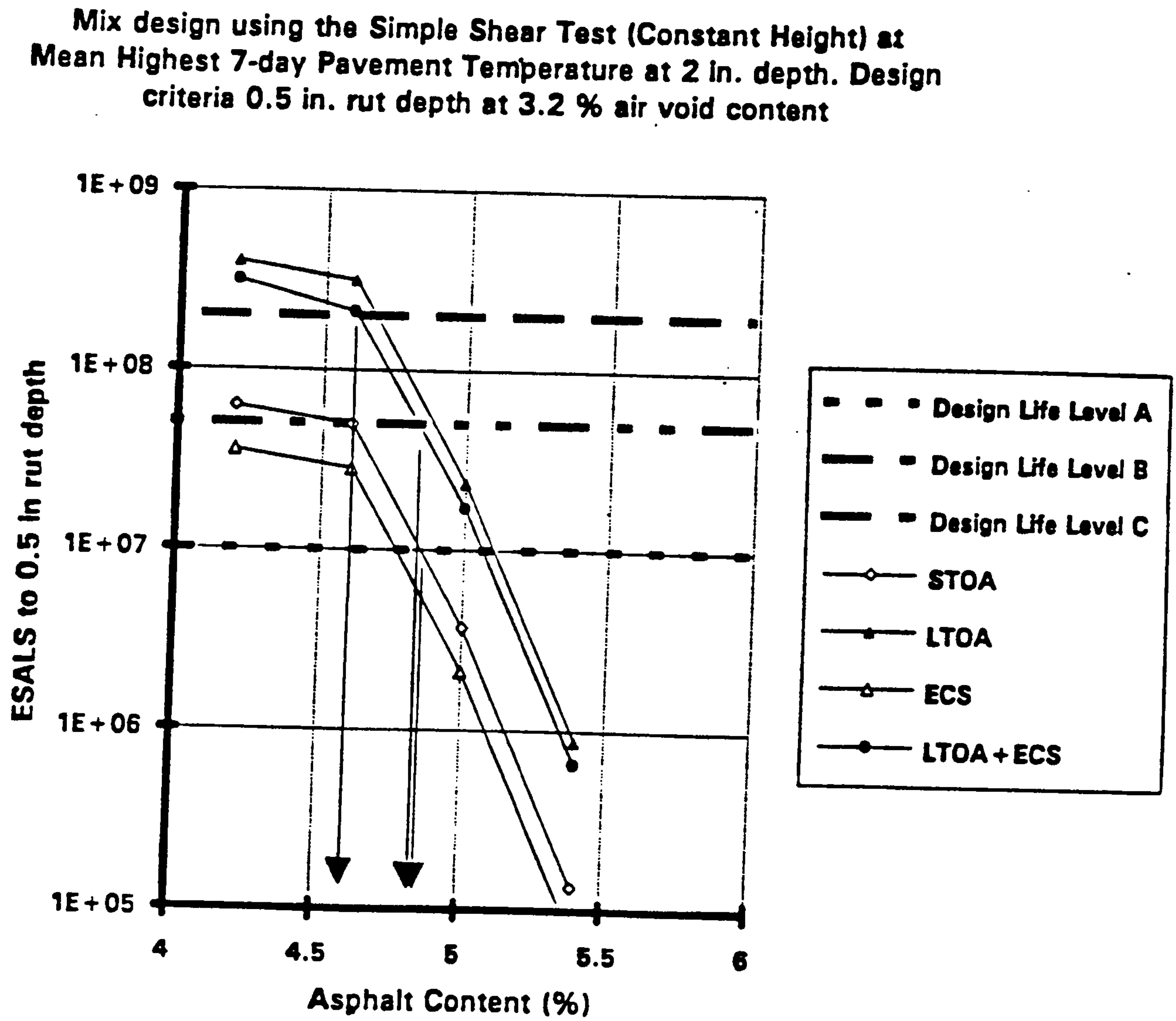

Figure 44 - Conceptual mix design graph for a mix using the proposed procedure.

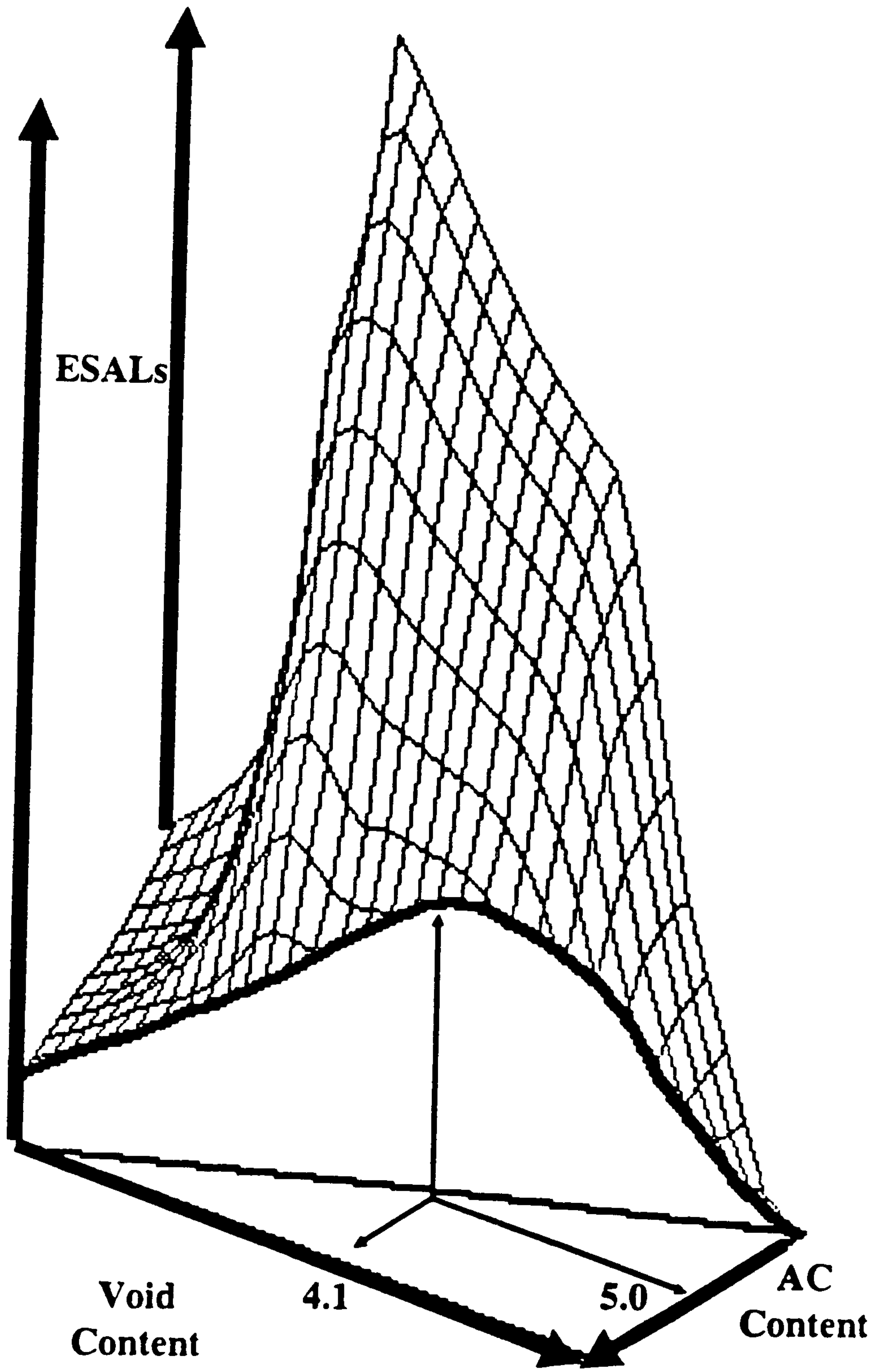

Figure 45 - Cross section of the ESALs/Void Content/AC content surface through a plain of similar compaction effort.

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