Abridged Procedure To Determine Permanent Deformation of Asphalt Concrete Pavements

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Research was undertaken to find a quick and simple procedure for determining the permanent deformation characteristics of asphalt concrete mixes. Data on pavement structure, rutting, traffic level, and temperature were available for a number of test sections of the Strategic Highway Research Program's General Pavement Studies (GPS) and from Colorado sites. During a previous study, finite-element computations had indicated a strong linear relationship between rut depth and maximum shear strain developed in the field. Availability of field cores and data from GPS sites, along with the finite-element results, prompted a laboratory study to find a quick procedure to estimate permanent deformation of asphalt concrete pavements. Repeated simple shear tests at constant height were performed on specimens 6 in. (15 cm) in diameter by 2 in. (5 cm) high obtained from these cores. The shear stress was haversine in shape, with a peak magnitude of 10 psi (70 kPa), and was applied for 5,000 cycles at the mean highest average 7-day maximum pavement temperature and at a depth of 2 in. (5 cm). Maximum shear strain for each site was determined from the reported rut depth on the basis of the linear relationship between the two parameters. Then the number of laboratory cycles to yield this value of shear strain was determined. The number of laboratory cycles determined in this way was correlated with the traffic level (equivalent single axle loads) that had resulted in the reported rut depth. The correlation was encouraging, especially regarding pavements less than 10 years old with an R^2-value of about 0.68. For all pavements that did not exhibit excessive aging, this relationship was obtained with $R^2 = 0.80$. On the basis of this relationship, a simple procedure to evaluate the mix potential for permanent deformation is proposed.

A procedure that could be used for rapid evaluation and screening of asphalt aggregate mixes is presented. The procedure could also be adopted to evaluate the rutting propensity of a mix, taking into consideration traffic level—in terms of equivalent single axle loads (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 and caused by shear stresses appearing below the edge of truck tires (*1*).

Intrinsic to this procedure is the assumption that most permanent deformation occurs during the hottest days and as a result of the heaviest trucks. That assumption stems from laboratory observations that asphalt concrete mixes exhibit strong plastic behavior, described by a plasticity function that exhibits kinematic hardening. This hardening seems to be associated with a mix's capability 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 are in turn capable of developing stabilizing, confining stresses.

The 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 a depth of 2 in. (5 cm). The test is executed using two actuators. One controls the magnitude of the applied shear stresses, whereas the other ensures 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).

The major drawback to the procedure is its inability to incorporate directly the effects of tire pressure and load magnitude. These effects can be brought into the analysis only indirectly, through computation of ESALs. However, equivalency factors could be accurately computed using the permanent deformation model and the finite-element methodology proposed by Sousa et al. (*2*).

BASIS FOR DEVELOPMENT OF PROCEDURE

Model Analysis

A series of finite-element analyses of the permanent deformation response of a pavement section was conducted using the model proposed (*2*). The model is intended to capture the macrobehavior of mixes, including (*a*) the shear dilatancy observed when the mix is subjected to shear strains, (*b*) the increase of effective shear modulus under increased confining pressure, (*c*) the significant variation of behavior with changes in temperature and rates of loading, and (*d*) the residual accumulation of permanent deformation under repetitive loading. Material properties were obtained from a series of volumetric, uniaxial shear and frequency sweep tests. In those analyses, only the nonlinear elastic and viscous properties of the mix were incorporated into the constitutive relationship. The purpose was to investigate the relationship among tire pressure, rut depth, and permanent shear and axial strains.

Two stress levels for the tire loading, 200 psi (1400 kPa) and 5 psi (3500 kPa), were used and were applied as a pulse loading with a duration of 0.3 sec and a 0.4-sec time interval between pulses. 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 represented (Figure 1) for the end of the second load cycle for the 500 psi (3500-kPa) tire loading condition. Note that a considerable

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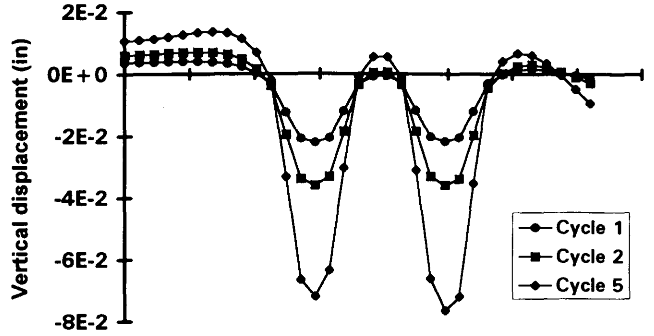

FIGURE 1 Variation of pavement profile with number of load applications; stress level, 3500 kPa; loading time, 0.3 sec; rest period, 0.4 sec.

upheaval of the pavement surface occurs between the tires. For 200 psi (1400 kPa) this upheaval is less pronounced (and even less so when nonlinear terms are ignored). Dilation exhibits a nonlinear dependence on the magnitude of the shear strain (*2*); essentially a nonlinear increase in the dilation is observed with an increase in shear strain. Figure 2 suggests that there may be a linear relationship between rut depth and maximum permanent shear strain.

After analysis, the authors decided to incorporate a plastic component into the permanent deformation model (*3*). With initial material characteristics for mixes containing eight Strategic Highway Research Program (SHRP) asphalts and two SHRP aggregate with air void contents varying between 3 and 8 percent, a series of analyses were performed for a pavement structure with a shoulder. In this case, a 10 psi (70-kPa) tire pressure and a load duration of 0.01 sec and a time interval between load applications of 0.06 sec were used. The computer runs, which included material properties obtained for the 16 mixes, yielded a relationship that best fits all the cases, especially for rut depths above 0.02 in. (0.05 cm). It is given by

$$\text{Rut depth (in.)}\ (rdp) = 11 * \begin{array}{c}\text{maximum permanent}\\ \text{shear strain}\ (mpss)\end{array} \qquad (1)$$

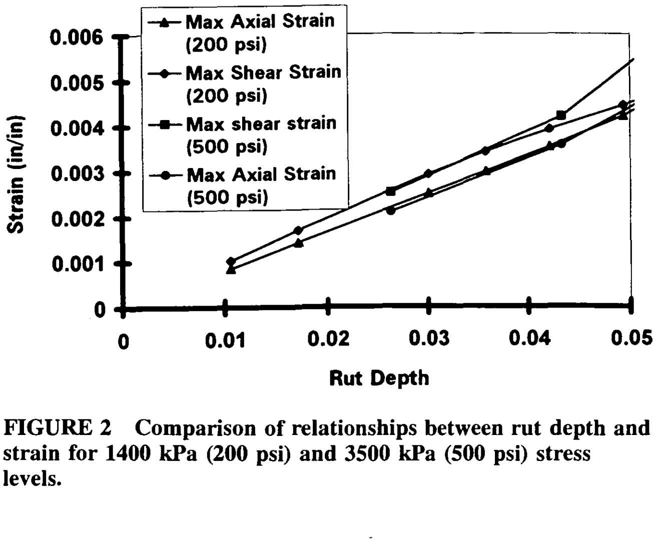

FIGURE 2 Comparison of relationships between rut depth and strain for 1400 kPa (200 psi) and 3500 kPa (500 psi) stress levels.

The 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); and
- Tire pressure and load magnitude [100, 200, and 500 psi (700, 1400, and 3500 kPa) were used for a given tire size].

However Equation 1 should be validated for different pavement types and thicknesses, and for nonuniform variation of material properties. The relationship should be dependent on pavement geometry and loading geometry. Loading geometry (i.e., the relationship between tire dimensions and pavement dimension) is basically the same for most pavements. Pavement geometry, on the other hand, varies considerably. 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 the relationship will hold. However, these two considerations should be investigated further.

Field Data

Field data from SHRP's General Pavement Studies (GPS) were obtained from the SHRP A-005 project. Data consisted of the site number, date section was opened to traffic, rut depth measurement, date of rut depth measurement, and equivalent single axle loads (ESALs) (up to the time of the rut depth measurement) for each GPS site. Table 1 contains a summary of relevant data. Figure 3 shows a scatter plot of rut depth versus ESALs. The rut depth was measured using Pasco equipment, a Dynatest dipstick, or a 72-in. (1.2-m) straightedge.

Temperature Analysis

SHRP binder and mix specifications are developed on the basis of maximum and minimum pavement temperatures. Maximum pavement temperature is defined as the average maximum temperature for 7 consecutive days. It is believed that rutting correlates better with this temperature than with either the mean monthly maximum or the average yearly maximum pavement temperature.

Determination of Mean Highest 7-Day Maximum Air Temperature

GPS sites represent diverse environmental conditions; maximum pavement temperature for these sites varies widely. In order to calculate the maximum pavement temperature for the GPS sites, two or three weather stations close to each site were selected; the weather stations with more than 20 years of records were included. For each year the average 7-day maximum temperature was calculated. First, the maximum temperature for each day of the year was determined; then the maximum daily temperatures for 7 consecutive days were added together and the result divided by 7 to obtain an average 7-day maximum temperature. All the

TABLE 1 Summary of Test Conditions and Results

GPS SITES	Spec. name	ST	Max Pav Temp (C)	Voids Cont. (%)	AGE Years	ESAL YEARS	Rut Depth (mm)	Shear Strain	Number Cycles RSST-CH	REG. CRITERIA
21001	**GX21-1**	**AK**	32.2	6.1	7	399844	4.6	0.0164	87	
21004	**GX1-1**	**AK**	32.2	3.9	13	1791505	8.4	0.0300	3419	
41007	**GX41-1**	**AZ**	58.9	2.8	11	21365008	10.4	0.0373	576	out
41021	**GX-19**	**AZ**	57.2	1.0	11	11549655	13.2	0.0473	26876	void
					12	12633956	13.5	0.0482	28553	void
41025	**GX22-1**			0.0	13	13651008	4.1	0.0145	286	void
41036	**GX8-1**	**AZ**	58.9	6.6	6	4322385	3.6	0.0127	11523	
					7	4769968	3.6	0.0127	11523	
53071	**GX64-1**	**AR**	51.7	3.9	1	637500	3.6	0.0127	6872	
					2	1275000	4.1	0.0145	9694	
					3	1657500	4.1	0.0145	10585	
68153	**GX51-1**	**CA**	48.9	3.4	12	614903	4.1	0.0145	376	
68156	**GX26-1**	**CA**	48.9	6.3	15	820162	3.6	0.0127	30063	
82008	**GX10-1**	**CO**	51.7	1.5	17	1225650	10.7	0.0382	19963	
					18	1283072	11.7	0.0418	28155	
131031	**GX33-1**	**GA**	53.3		9	227047	7.1	0.0255	4763	
					10	256209	7.1	0.0255	4763	
161020	**GX61-1**	**ID**	48.9	6.0	3	142749	3.6	0.0127	44	
					4	178200	3.8	0.0136	52	
171003	**GX32-1**	**IL**	51.7	3.5	3	139986	3.0	0.0109	185	
					4	179982	4.3	0.0155	395	
					5	216645	3.8	0.0136	299	
201009	**GX29-1**	**KS**	54.4	7.9	4	284935	5.1	0.0182	451	
					5	404268	5.8	0.0209	582	
211014	**GX14-1**	**KY**	48.9	4.1	5	1418454	4.6	0.0164	3089	
					6	2051845	4.8	0.0173	3827	
231012	**GX44-1**	**ME**	43.3	2.1	4	980000	5.8	0.0209	756	
					5	1190000	6.4	0.0227	931	
261012	**GX3-1**	**MI**	46.1	6.5	9	714861	5.8	0.0209	803	
					10	802748	6.6	0.0236	1031	
271019	**GX11-1**	**MN**	46.1	9.0	9	435438	5.6	0.0200	83390	void
					10	472975	5.8	0.0209	9324	void
341030	**GX23-1**	**NJ**	48.9	0.5	18	1115000	14.2	0.0509	14355	void
					19	1160000	17.3	0.0618	23103	void
341031	**GX31-1**	**NJ**	48.9	1.0	16	5075000	9.4	0.0336	486	void
					17	5325000	9.7	0.0345	512	void
351022	**GX62-1**	**NM**	48.9	5.2	6	724306	3.8	0.0136	1304	
401015	**GX43-1**	**OK**	54.4	2.1	13	955031	5.8	0.0209	2016	
					14	1040193	6.1	0.0218	2692	
404164	**GX35-1**	**OK**	54.4	4.0	14	633750	3.8	0.0136	19076	
					15	686250	3.6	0.0127	15860	

(*continued on next page*)

average 7-day maximum temperatures during the hot days are determined, and the largest number obtained this way is recorded as the highest average 7-day maximum temperature for that particular year. The procedure is repeated for all years for which records are available. For example, if 30 years of data are available for one station, 30 numbers will be obtained in this way. The average value of these 30 numbers will be recorded as the mean highest average 7-day maximum temperature. This number was the value used in the calculations.

Determination of Pavement Temperature

Pavement surface temperature was calculated using the following formula, which was developed on the basis of energy balance at the surface (*4*) (it needs to be solved interatively):

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

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·R^4)],
h_c = surface coefficient of heat transfer [default = 3.5 Btu/(hr·ft^2·F)],

TABLE 1 (*continued*)

GPS SITES	Specimen name	ST	Max Pav Temp (C)	Voids Content (%)	AGE Years	ESAL	Rut Depth (mm)	Shear Strain	Number Cycles CHRSST	REG. CRITERIA
479025	**GX30-1**	**TN**	51.7	8.1	9	233159	3.6	0.0127	19804	void
					11	288553	4.6	0.0164	37120	void
481039	**GX71-1**	**TX**	54.4	3.9	7	1637481	4.1	0.0145	1105	
					8	1993484	5.8	0.0209	2170	
481047	**GX18-1**	**TX**	54.4	2.5	18	5500000	5.1	0.0182	1625	out
481048	**GX42-1**	**TX**	54.4	1.1	15	786000	5.1	0.0182	251336	void
					17	856600	4.1	0.0145	131878	void
481069	**GX81-1**	**TX**	54.4	2.3	13	2573568	8.6	0.0309	8205	
					14	2751168	8.1	0.0291	7185	
481077	**GX15-1**	**TX**	54.4	1.8	7	1394648	9.7	0.0345	3039	
811805	**GX65-1**	**CAN**	43.3	4.3	9	1190182	6.4	0.0227	133	out
851801	**GX4-1**	**CAN**	32.2	4.1	5	1183357	5.3	0.0191	2668	
892011	**GX63-1**	**CAN**	46.1	4.8	10	853376	4.3	0.0155	813	
					11	933380	5.1	0.0182	1258	

COLORADO SITES	Specimen name	ST	Max Pav Temp	Voids Content (%)	AGE Years	ESAL	Rut Depth (mm)	Shear Strain	Number Cycles CHRSST	REG. CRITERIA
14	**14 H**	**CO**	51.1	4.6	23	3282000	20.3	0.0727	166839	
29	**29 I**	**CO**	48.9	7.1	9	5002000	7.6	0.0273	4442	
30	**30 H**	**CO**	48.9	6.6	9	4622000	15.2	0.0545	3122	
13	**13 A**	**CO**	51.1	7.5	6	1257000	2.5	0.0091	1030	
13	**13 B**	**CO**	51.1	7.1	6	1257000	2.5	0.0091	675	

k = thermal conductivity [default: 0.8 Btu/(hr·ft^2·F)/ft],
ϵ = surface emissivity (default: 0.9),
T_a = maximum air temperature (Rankine), and
T_s = maximum pavement surface temperature (Rankine).

Once the maximum pavement temperature at the surface is found using the preceding iterative procedure, the maximum pavement temperature for any depth of less than 8 in. (20 cm) is found through the following empirical formula (*4*):

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

where

d = depth in inches,
T_s = maximum pavement temperature (°F) at surface, and
T_d = maximum pavement temperature (°F) at depth d.

It seems that most permanent deformation from shear stresses developing near the edge of the tires takes place at depths up to 4 in. (10 cm). Maximum shear stress computed from nonlinear visco-elastic analysis took place at about 2 in. (5 cm). 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 depth of 2 in. (5 cm) was selected as the testing temperature for each of the GPS sections.

Laboratory Tests

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, are dominant. Repetitive loading in shear is required in order to accurately measure the influence of mix composition on resistance to permanent deformation in the laboratory. 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 able to measure properties under states of stress that are encountered

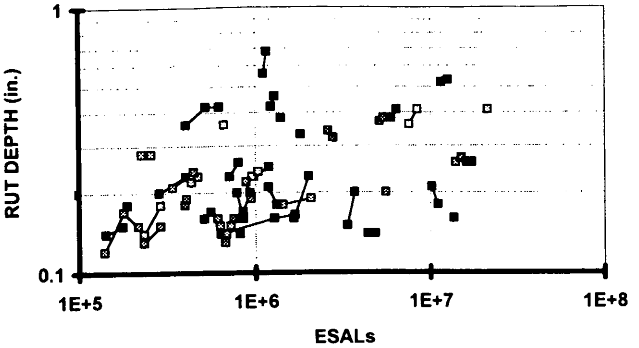

FIGURE 3 Variation of rut depth with ESALs for GPS sections.

within the entire rutting zone, particularly near the pavement surface. Because there are an infinite number of states of stress, it is impossible to simulate them all with a single test, given the nonlinear and viscous behavior of the material. For this reason, several tests have been proposed to determine a constitutive law for asphalt concrete (*2*). However, if a single test is to be performed to rapidly screen and evaluate the resistance of various mixes to permanent deformation, that test should be sensitive to the most important aspects of mix behavior and executed under conditions that most significantly affect that behavior (*3*).

The repetitive simple shear test at constant height on cylindrical specimens 6 in. (15 cm) in diameter by 2 in. (5 cm) high is proposed as an effective test to evaluate the rutting propensity of a mix.

Test Procedure

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

The testing system fabricated by Cox and Sons, Inc. of Colfax, California, was presented by Sousa et al. (*5*). The system consists of two orthogonal tables that are mounted on bearings. The tables are connected to two hydraulic actuators that are controlled using servovalves under feedback, closed-loop digital algorithms. To ensure that 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 Cox and Sons, Inc. so that the parallel faces of the caps could be glued. Hydraulic clamps made the equipment easy to use by eliminating the need to use tools to fasten the specimens to the moving tables.

The equipment can accommodate several specimen sizes, but for permanent-deformation evaluation, the recommended specimen size for shear testing is a cylinder 6 in. (15 cm) in diameter and 2 in. (5 cm) high. If large stone mixes are to be tested, the recommended specimen size is 8 in. (20 cm) in diameter by 3 in. (7.5 cm) high.

In executing a repetitive simple shear test at constant height, the vertical actuator maintains the height of the specimen constant by using as feedback the output of a linear variable differential transformer (LVDT) measuring the relative displacement between the specimen caps. The horizontal actuator under the control of the shear load cell applies haversine loads corresponding to a 10-psi (70-kPa) shear stress magnitude with a 0.1-sec loading time and a 0.6-sec rest period. Experience with a wide range of mixes tested at different temperatures and stress levels demonstrates that the 10-psi (70-kPa) shear stress magnitude is a reasonable level at which good mixes exhibit some permanent deformation whereas poor mixes do not fail much too quickly. Finite-element computations indicate that critical shear stress levels in the field might be between 20 psi (140 kPa) to 25 psi (75 kPa). Associated with these shear stresses, confining pressures of about 30 psi (210 kPa) and axial stresses of about 80 psi (560 kPa) were also found. However, no lateral confinement is applied during the laboratory test.

Tests were executed until 5 percent shear strain was reached or there had been up to 5,000 cycles. Before testing, specimens were conditioned with 100 cycles of 1 psi (7-kPa) haversine loading with a 0.1-sec loading and 0.6-sec rest period. The preconditioning was done for the 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 a total of 40 GPS sites to cover a wide range of environmental conditions. Specimens 2 in. thick were cut out of selected field cores with a double-blade saw. Efforts were made to cut specimens from an area 1 to 3 in. below the surface. Specific gravities of the specimens were determined using parafilm. Specimens were allowed to dry before being glued to the caps, and a DEVCON 5-min plastic steel putty was used to glue them, which was allowed to cure 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 a depth of 2 in. (5 cm)] for at least 2 hr (but no more than 4 hr) before being tested.

Given that the specimens had slightly different diameters, the shear load required to yield a 10 psi (70-kPa) shear stress level for each specimen was calculated on the basis of the area of the specimen.

Test Results

An RSST-CH was performed on one specimen from each GPS site. Each test was performed at 10 psi (70-kPa) stress amplitude (with a 0.1-sec loading time and a 0.6-sec rest period) and at 7-day maximum pavement temperature encountered at the 2-in. (5-cm) depth. Figure 4 exhibits a typical graph of the permanent shear strain versus number of cycles obtained from the tests. It is apparent that some mixes deform faster than others; not only do they have different slopes but different intercepts also.

Analysis

Based on Equation 1, the maximum shear strain corresponding to the measured rut depth for each of the GPS sites was computed. Typical results from repetitive shear tests on GPS specimens (Figure 4) were used to determine the number of shear cycles required to reach the level of maximum shear strain calculated using Equa-

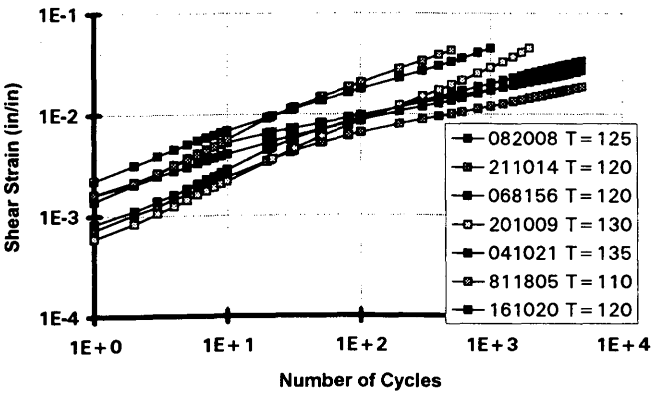

FIGURE 4 Variation of permanent shear strain in RSST-CH, with number of load repetitions for some GPS sites; temperatures in degrees Fahrenheit.

tion 1. The process relates the number of cycles in the RSST-CH to reach the same magnitude of permanent shear strain as is caused by the ESALs in the field. Table 1 contains the results from all the tests. The last column contains information about the rejection criteria used for the data (in some cases, specimens would have been rejected on the basis of other factors, such as a test executed at the wrong temperature or LVDTs that got loose during testing). Results from specimens with air voids less than 1.5 percent and more than 8.0 percent also were eliminated. Specimens with voids less than 1.5 percent are overcompacted and not representative of the conditions prevailing during most of the life of the pavement. Specimens with void content above 8 percent are likely to densify before entering into the plastic shear flow stage. Out of all the data, three 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 5. In recognition of 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 5. Note 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 and another obtained from specimens aged in the field for an average of only 8 years. Sites in Line B have a maximum pavement temperature at 2 in. (5 cm) depth higher (on average) than those from Line A. The average air void content is similar for both populations.

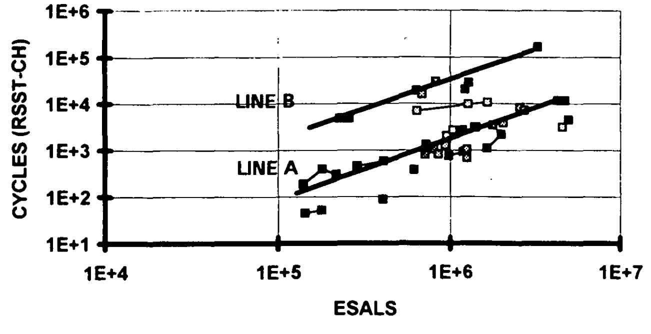

FIGURE 5 Relationship between number of cycles in RSST-CH and ESALs to reach the same shear strain level; Lines A and B indicate two possible populations.

Specimens with aged asphalt perform relatively better in the RSST-CH. Results were obtained from specimens taken from out-of-the-wheel-path field cores that had been subjected to aging and to limited traffic. However, because the specimens were obtained at a 2 in. (5 cm) depth, the magnitude of aging is less than what would occur at the surface.

Most variability in the data came from specimens taken from sites aged 10 or more years. This is to be expected, because 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 that were less than 10 years old. The following relationship was obtained with an $R^2 = 0.68$:

$$\log(\text{cycles}) = -4.09 + 1.204 \log(\text{ESAL}) \qquad (4)$$

One expects that mixes with high air voids from older pavements exposed to higher temperatures have probably aged more. Therefore, the product (age × voids × temperature/10) represents a variable, and a high value indicates the greater likelihood of having a more aged mix than if the value were low. The last two columns of Table 2 contain product values. The last column presents the values for sites more than 10 years old.

Observe that the average product (age × voids × temperature/10) for Line A is almost half that of the average 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 corresponding average product in Line B (891) and is close to the average for all points in Line A (427). That observation provides a rationale to justify that specimens in Line A more than 10 years old are not really as aged as the specimens belonging in 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 6). The equation of the best fit is given by

$$\log(\text{cycles}) = -4.36 + 1.240 \log(\text{ESAL}) \qquad (5)$$

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

The best-fit lines obtained from the two criteria are presented in Figure 7. On the basis of these results, Equation 5 might be used to develop an abridged procedure to evaluate permanent deformation for asphalt concrete pavements. The product value is indicative of a very good correlation, especially if the following is considered:

- The rut might also be related to densification, subgrade effect, or pavement surface irregularities in some cases;
- The RSST-CH was executed with specimens that, although not in the wheel path, already had been subjected to traffic to various degrees and whose behavior might be different from specimens obtained from newly compacted mixes;
- The calculation of the maximum pavement temperature at a depth of 0.05 m is just an estimate of the real temperature;
- The testing rate is a 0.1-sec loading and 0.6-sec unloading, whereas in the pavement, the rate is closer to a 0.02-sec loading and almost random spacing; and
- The ESALs were not actually measured but were extrapolated from U.S. Department of Transportation data.

TABLE 2 Summary of Test Conditions and Results for Lines A and B

LINE A SITES	Specimen name	ST	Max Pav Temp (F)	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

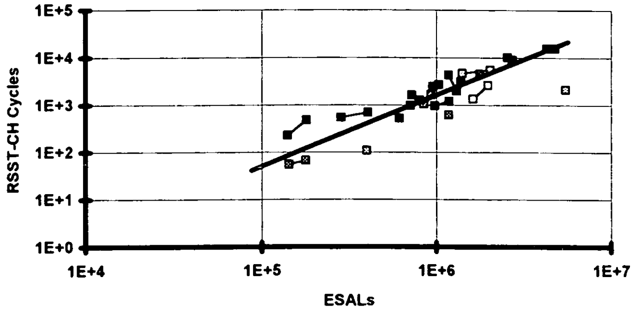

FIGURE 6 Variation of number of cycles to field shear strain in RSST-CH with ESALs for sections that did not exhibit significant aging.

Overall the relationship between the proposed test procedure and the rutting behavior in the field is a strong one.

ABRIDGED PROCEDURE

On the basis of test results, an abridged procedure to determine the rutting propensity of a mix can be developed by following these steps:

1. Determine the number of ESALs 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*).

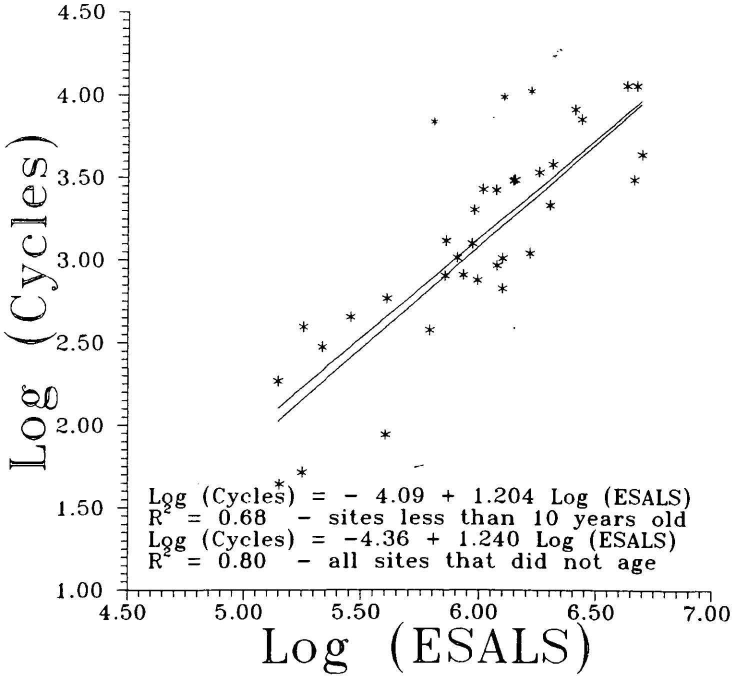

FIGURE 7 Variation of number of cycles to field shear strain in RSST-CH with ESALs for sections that did not exhibit significant aging and those less than 10 years old.

3. Determine 7-Day maximum pavement temperature at a site at a depth of 0.05 m.
4. Execute the RSST-CH at 70 kPa at that temperature.
5. Using Equation 1, relate the rut depth to the maximum permanent shear strain in a pavement section and 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. Determine the number of ESALs that can be carried by that mix in the pavement before the maximum allowable rut depth is reached by using the relationship between ESALs and number of cycles in RSST-CH. The relationship derived from Equation 5 is as follows:

$$\mathrm{ESAL}_{mrd} = 10 \ \hat{} \ [(4.36 + \log (N_{mpss})]/1.24) \qquad (6)$$

where ESAL_{mrd} equals the number of equivalent single axle loads to develop maximum allowable rut depth (*mrd*), and N_{mpss} is the number of cycles in RSST-CH to reach the maximum permanent shear strain (*mpss*) correspondent to the maximum allowable rut depth (*mrd*). (Testing conditions included 70 kPa, a 7-day maximum pavement temperature, 0.1 sec on, and 0.6 sec off.)

OTHER CONSIDERATIONS

To implement this procedure, a few factors should be considered (*6*):

- Evaluation of tire pressure effects on the rate of accumulation of permanent deformation can be done only 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. (*2*).
- Aging and water sensitivity should both be addressed and incorporated into the procedure. The mix should be subjected to short-term aging that is representative of the field mixing and placement process and to water sensitivity conditioning before being subjected to the RSST-CH. Such treatment 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 carried out; it would stiffen the asphalt binder and therefore provide improved performance. Furthermore, the correlation presented was obtained for pavements less than 10 years old.
- The assumption of uniformly distributed ESALs (inherent in the procedure) during the year and during 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 account the relative contribution to 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 a 2-in. (5-cm) depth.

The procedure could be improved if the test were executed at the mean highest 7-day maximum pavement temperature, corrected to compensate for the rate of loading effect. Normal traffic, traveling at 90 km/hr, applies pulse loads with a duration of about 0.015 sec at 2-in. (5-cm) depth. Laboratory tests are executed with a 0.1-sec loading pulse. Taking advantage of time-temperature superposition, decreasing the temperature simulates the faster rate of loading encountered in the field. That might provide a more accurate balance between permanent deformation from the viscous behavior of a binder and the plastic component because of 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 adjusted further to account for the field rate of loading (mixes placed in up-hill pavement sections with slower traffic could be tested at higher temperatures than mixes placed in level sections).

In addition, when an RSST-CH is done, a specimen hardly changes volume. Therefore, tests should be executed on a mix with air void contents representative of those that predominate during the life of a pavement (*6*).

To implement the procedure on the basis of these findings, it must be recognized that Equation 1 might not be valid in all cases. However, the assumption can be demonstrated and validated by executing analyses the authors presented earlier for a series of pavement configurations. It is likely that a family of curves could be developed for different pavement thicknesses. Also, there is inherent variability in any test procedure; therefore, reliability considerations should be incorporated into the procedure (*7*).

SUMMARY

Foundations for the development of an abridged procedure to determine the permanent deformation potential of an asphalt aggregate mix have been presented. Asphalt aggregate mixes exhibit nonlinear, elastic, viscous, and plastic behavior. Nonlinear behavior, such as dilation or stress hardening, is mostly influenced by the aggregate skeleton. A finite-element model that takes these aspects of mix behavior into account was used to establish a relationship between rut depth and maximum permanent shear strain in pavement sections. The relationship seems to be independent of a wide range of input variables and material properties; however, it is probably dependent upon pavement structure for thin pavement sections.

RSST-CH was used as the accelerated laboratory test for evaluating rutting propensity of a mix. The test was executed at the critical pavement temperature at a depth of 2 in. (5 cm). For this analysis, critical pavement temperature was defined as the 7-day maximum pavement temperature at a 2-in. (5-cm) depth. This depth was selected because computations have shown that maximum shear stresses, those causing the permanent deformation in the pavements, are encountered 2 in. (5 cm) beneath the surface near the edge of the tire tracks. The procedure was derived from data obtained from 40 GPS sections around the North American continent. The procedure is mainly based on the execution of the RSST-CH at the mean highest 7-day maximum pavement temperature encountered at a 2-in. (5-cm) depth. The fundamental link between the laboratory tests and the field performance was derived from the 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 that did not exhibit significant aging, that relationship was obtained with an R^2 of 0.80. Specimens should be compacted in the 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 be performed on specimens compacted in the laboratory to about 3 to 4 percent voids content. If this procedure is to be used in a mix design framework, efforts should be made to age and moisture condition laboratory-prepared specimens to be representative of the conditions expected in the field. As more rut-depth measurements from sites become available, and predictions are made on the basis of the proposed procedure, results can be compared, and the existing relationship can be either verified or improved.

ACKNOWLEDGMENTS

The work reported was conducted as part of SHRP Project A-003A. The project, Performance Related Testing and Measuring of Asphalt Aggregate Interactions and Mixes, is being conducted by the Institute of Transportation Studies, University of California, Berkeley. Carl L. Monismith is principal investigator and Jorge B. Sousa, assistant research engineer. Carl Monismith's unwavering challenges to this procedure are especially appreciated as they strengthen the foundations where it rests. The authors express their appreciation for the collaboration and support of John Deacon in the conceptual development of some aspects of the abridged procedure and to Shmuel Weissman of Symplectic Engineering Inc. for the development of the modeling capabilities that permitted the determination of the relationship between rut depth and maximum shear strain. Michell Jamjim ran the finite-element program and John Harvey coordinated some aspects of the specimen preparation. Timothy Aschenbrener from the Colorado Department of Transportation provided the data and cores for the Colorado Sites. His efforts in testing those cores are appreciated.

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