Application of SHRP Mix Performance-Based Specifications

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The Strategic Highway Research Program project, SHRP-A003A, has introduced new test methods and procedures that can be easily implemented in pavement design and in the development of performance-based specifications. The basic concepts behind the design of a high traffic, high performance pavement are presented, and the concept of the performance point as a methodology to develop site-specific contract specifications is introduced. This permits a connection between performance observations, structural pavement design, and mix design. This project is the first application of performance-based specifications based on fatigue tests and repetitive simple shear tests at constant height.

In the past few years considerable knowledge has been gained about structural and material behavior of asphalt concrete pavements. The Strategic Highway Research Program (SHRP) in the United States has provided new specifications for binders and mixtures, which permit better materials characterization. Furthermore, validation of asphalt concrete fatigue laws and experience with design of asphalt concrete pavements subjected to high traffic volumes have produced more reliable structural pavement designs.

The authors summarize a pavement design analysis for the Tagus Bridge Crossing between Lisbon and Montijo, Portugal. The project was executed by CONSULPAV for MottConsult, PONTEJO's consultant. The contractor is expected to maintain the pavement for 35 years. At the end of that period, it is expected that no major rehabilitation would be needed for another 5 years.

The pavement sections were designed based on the traffic, geotechnical, and climate data presented (*1,2*). New mechanistic design concepts and recent developments in mix characterization were applied in the project. Rehabilitation strategies proposed for the project are not part of this paper, however. This was a preliminary project. Final performance can only be predicted on the basis of laboratory fatigue, permanent deformation, water sensitivity, and aging tests performed on actual mixes. Redesign is likely to be necessary based on actual values obtained from such tests.

PROJECT DATA

Traffic

Truck traffic estimates were obtained elsewhere (*1,2*). Because no estimates were given for the 40-year life cycle, design life predictions were made by extrapolating the data based on the best fit of the data presented.

Geotechnical Information

Available data (*1*) suggest that the pavement will be resting either on clays or sands on the left bank and possibly better soils along the right bank. In the design of the flexible pavement, three cases were evaluated. The following moduli were assumed: clay, 40 MPa (5.8 ksi); sands, 60 MPa (8.7 ksi); and better soils, 80 MPa (11.6 ksi).

Weather and Temperature

Pavements will be placed between Lisbon and Montijo. The average annual precipitation in the region is about 700 mm (27.6 in.), and average annual temperature is about 17°C (63°F) (*3*). Maximum air temperatures can reach 42°C (108°F), and minimum temperatures may drop as low as −5°C (23°F). Thus, in the design, consideration should be given to water sensitivity of the materials, and materials should be selected carefully to prevent premature failure from rutting at high temperatures. Finally, temperature variations should be accounted for in the fatigue design.

Design Elements

The design approach was as follows:

1. Selection of structural cross section, using stiffness of the asphalt concrete and fatigue cracking as the criteria;
2. Verification of acceptable permanent deformation in the underlying layers;
3. Verification of acceptable permanent deformation in the asphalt concrete from shear deformation; and
4. Definition of performance-based specifications.

STRUCTURAL PAVEMENT DESIGN

The fatigue analysis system developed by SHRP-A003A researchers (*4*) recognizes that mixture performance in situ depends on critical interactions between mixture properties and in situ conditions (e.g., pavement structure, traffic loading, or environmental conditions). The analysis system thus provides not only sensitivity to mixture behavior but also sensitivity to the in situ traffic, climatic, and structural environment as well. It seeks to judge, with

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predetermined reliability, whether a mix design would perform satisfactorily in service. If it would not, the designer can opt to redesign the mix, strengthen the pavement section, or repeat the analysis using more refined measurements and estimates. The steps in the analysis system (*4*) are as follows:

1. Determine design requirements for reliability (probability of avoiding the acceptance of a deficient mixture) and performance (extent of permissible fatigue cracking);
2. Determine the expected distribution of in situ pavement temperatures;
3. Estimate design traffic demand in terms of equivalent single axle loads (ESALs);
4. Select trial mixture;
5. Prepare test specimens and condition as required;
6. Measure stiffness of trial mixture;
7. Design pavement structural section;
8. Determine design strain under standard axle load;
9. Determine the resistance of the trial mixture to fatigue (N_{supply}) in the laboratory or by regression estimate;
10. Apply a shift factor to the travel demand (ESALs) to account for differences between laboratory and in situ conditions (such as traffic wander and crack propagation) to determine N_{demand};
11. Compare traffic demand (N_{demand}), including reliability, with mixture resistance (N_{supply}); and
12. If N_{demand} exceeds N_{supply}, reanalyze current trial mixture with procedures that yield greater accuracy, or alter trial mixture or structural section and iterate.

The preceding concepts generally were followed for this project. At the preliminary design level generally accepted fatigue laws were adopted. However, mix performance varies considerably and more reliable predictions can only be made on the basis of results from fatigue tests on actual mixes. The specification section in this paper contains the actual performance specifications required for each mix.

Determination of Temperature Distribution

The new methodology proposed by SHRP-A003A (*4*) proposes limiting fatigue tests to one temperature and expressing the destructive effects of the anticipated traffic in the field as equivalent ESALs at that temperature. This is accomplished through use of temperature equivalency factors. The approach simplifies testing, which increases productivity and reduces costs.

Determination of the Design Traffic

The heavy traffic was converted into ESALs using previously established conversion factors. The British conversion to 80 kN (17.9 kips) of 3.5 (based on a daily traffic of more then 3,000 vehicles) (*5*) was used to convert heavy traffic into ESALs.

It was further assumed that the critical right lane would sustain 70 percent of the traffic. The cumulative variation of ESAL was computed on the basis of these two assumptions.

Selection of Material Properties

Experience has shown (*6,7*)that the most cost effective design of flexible pavement design for high traffic pavements is generally a full-depth asphalt concrete pavement. In this type of pavement significant improvement in performance can be obtained if stiff mixes are used. On the other hand, the most significant benefit of stiff layers is reduction of vertical compressive strains in the subgrade. In general, a reduction in stiffness of the mix, for a given tensile strain, increases fatigue life. However, when stiff mixes are used in a full-depth asphalt concrete pavement, the reduction in tensile strain offsets the propensity of stiffer mixes to fatigue, while providing the benefit of reduced vertical compressive strains in the subgrade.

It is known that temperature variations differ greatly with depth. Furthermore, asphalt concrete mixes are generally susceptible to temperature. Therefore, the structural section can be designed with materials that exhibit different temperature susceptibilities, by placing them at the appropriate depth to match their advantageous properties.

Materials selected for the bottom asphalt concrete layers should be very stiff and may be temperature sensitive. The material properties for the mix (M-3) are presented in the specification section.

The material present in the top layer should have the combined properties of low temperature susceptibility and a high shear stress resistance. Shear stresses occurring under the edge of the truck tires are known to cause rutting. The development of critical shear strains is limited to the upper 10 to 12 cm (3.9 to 4.7 in.) of the pavement section. A mixture placed in this layer (M-1) should be stiff at high temperatures with good aggregate interlock and not very temperature sensitive. This can not usually be easily achieved by normal binders and it is necessary to add modifiers. To satisfy all of these requirements the mix usually has lower stiffness at low temperatures than the materials selected for the underlying layers. For the material placed between those two layers (M-2) an intermediate behavior is desirable.

Traditionally in Europe thin stone mastic asphalt (SMA) layers are used on the upper layer of a pavement section. In this project a gap-graded friction course layer (SMA) was designed to resist permanent deformation but no structural value was given for fatigue design.

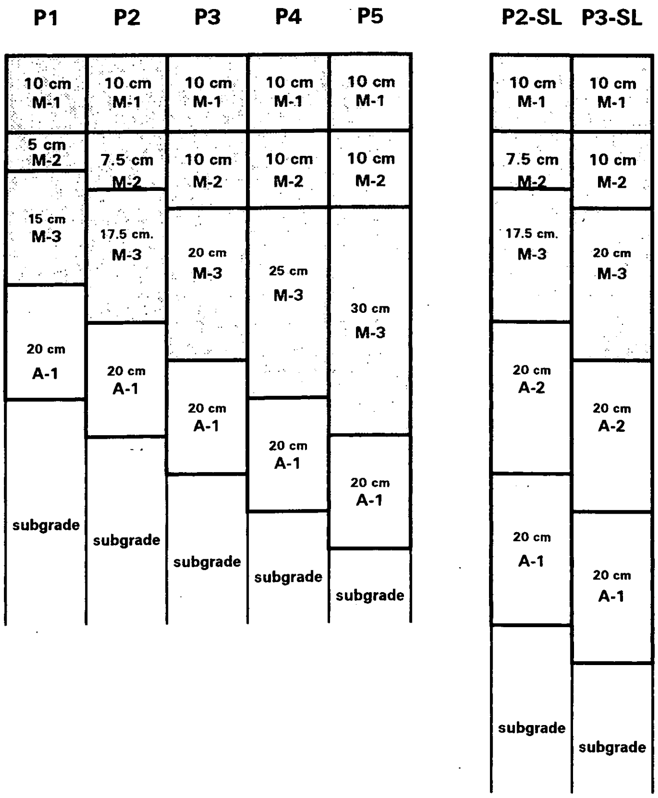
To ensure proper compaction of the mixes in situ, it is necessary to provide a stable platform. A 20-cm (8-in.) granular base was selected for placement on the subgrade. The estimated stiffness of this layer (A-1) depends on the moduli of the subgrade. It is generally accepted that the moduli are twice that of the underlying layer. A sensitivity study was made on the use of thick granular bases in an effort to investigate possible cost reductions. For this material a modulus of 180 MPa (26.1 ksi) was assumed. The additional granular base was named A-2.

Determination of Strains in Each Pavement Section

The maximum principal tensile strain at the underside of the asphalt layer usually governs the initiation of fatigue cracking in situ. Mixtures will perform adequately only if they can sustain the necessary repetitions of this strain without cracking. For mixture-analysis purposes, multilayer elastic theory provides a convenient and sufficiently accurate means for estimating the maximum in situ strain at 2°C under the axle load.

In the investigation of the optimal pavement sections several pavement cross sections were evaluated (see Table 1). The asphalt bound layers vary in thickness between 30 and 50 cm (11.8 and 19.7 in.). The performance of these full-depth sections over the three types of soils was evaluated. Two conventional sections with

TABLE 1 Schematic Representation of Structural Sections



two layers of granular bases were named P2-SL and P3-SL (see Table 1).

For each case the maximum tensile stress and strain on the bottom of the asphalt layer and the maximum compressive vertical strain on the subgrade were determined using linear elastic analysis. Two runs were executed for a pavement section with the same binder thicknesses as P2 and P3 and after adding 20 cm of granular base. These two runs were named P3-CSL and P2-CSL. One run was executed with reduced stiffness of all the binder layers. This run was named P3-CS. The results are presented in Table 2.

Determination of the Fatigue Life of the Pavement Section

The fatigue lives (N_f) of the 33 pavement sections analyzed were computed from the critical principal tensile strain using the newly developed SHRP-A003A model, which accounts for mix void content, mix stiffness, and strain level (*8*).

$$N_f = 2.5263 * 10^5 * e^{(-0.2007 * V_0)} * (\epsilon_0)^{-3.4134} * (S_0)^{-2.1239} \quad (1)$$

where

V_0 = Void content as percent (i.e., 4, 5, or 7);
ϵ_0 = Tensile strain (in./in.);
S_0 = Mix stiffness in psi; and
e = natural logarithm base.

It is considered that this equation, which applies for stress control conditions such as those present in thick pavements, is more likely to yield accurate estimates than any other presented thus far.

Laboratory estimates (N_{supply}) can be compared with service requirements ESAL@20°c only after the application of a suitable shift factor.

$$\text{ESAL@20°C} = SF * N_{demand} \quad (2)$$

where

ESAL@20°C = design ESALs adjusted to a constant temperature of 20 °C,
SF = empirically determined shift factor, and
N_{demand} = design traffic demand (laboratory-equivalent repetitions of standard load).

SHRP-A003A (*4*) reanalyzed previously reported shift factors proposed by NCHRP (*9*) and concluded that a shift factor of 10 would likely correspond to 10 percent field fatigue cracking, whereas a shift factor of 14 would likely correspond to 45 percent field fatigue cracking. These values were used in this analysis. Table 2 presents the fatigue life expected from the pavement sections, using the assumptions presented thus far. Note that the 10 and 45 percent fatigue cracking criteria identify significantly different levels of traffic.

Figure 1 presents the variation of fatigue life for the various pavement sections. It also presents the desired design lives of 20, 30, 35, and 40 years.

The effect on fatigue life of adding an additional 20 cm of aggregate base to pavement types P2 and P3 over a clay subgrade was also evaluated. It was found that the added aggregate layer corresponds to only 2.5 cm of asphalt concrete layer. Based on these results, the use of aggregate base as a structural layer was not recommended; instead, a 20-cm layer is recommended, mainly to provide a good working platform.

By comparing P3-C with P3-CS in Table 2, it can be seen that, with softer asphalt layers, the tensile strain rose from 0.304 E-4 to 0.165 E-4, whereas the vertical compressive strain increased to 1.07E-4 from 0.735E-4. The fatigue life of the pavement section did not change, because stiffness of the mix plays an important role in the fatigue equation (softer mixes can withstand higher strains). If the critical criterion was axial compressive strain on the subgrade, then the pavement with softer asphalt would last a shorter time. It is desirable to have a stiff mix to minimize problems that might emerge from the subgrade.

Recommended Pavement Section

On the basis of these results, pavement type P3 was recommended. This pavement would probably last 20 years (given the predicted traffic pattern), at the end of which, it would only exhibit 10 percent cracking. Before cracking became too extensive, the wearing course would be removed and an overlay placed.

SUBGRADE PERMANENT DEFORMATION VERIFICATION

Permanent deformation of asphalt concrete pavements generally has two major causes

1. Excessive subgrade deformation from high stresses at the subgrade level, and
2. Plastic shear flow in the upper 10 cm (3.9 in.) of the asphalt concrete layer caused by shear stresses near the edge of the tires.

TABLE 2 Results from ELSYM for Structural Pavement Section Analysis and Corresponding Lifetime Predictions for 10 and 45 Percent Fatigue Cracking

Pavement Type	Soil		Tensile Stress (MPa)	AC Tensile Strain	Subgrade Vertical Strain	Nf 10% cracking Fatigue (5%voids) $S_F=10$	Nf 45% cracking Fatigue (4%voids) $S_F=14$
P1	A		0.67	2.37E-05	8.91E-05	1.07E+08	1.83E+08
P2	A		0.51	1.79E-05	6.69E-05	2.79E+08	4.77E+08
P3	A		0.41	1.43E-05	5.63E-05	6.00E+08	1.03E+09
P4	A		0.34	1.17E-05	4.58E-05	1.19E+09	2.04E+09
P5	A		0.29	9.76E-06	3.77E-05	2.21E+09	3.78E+09
P1	B		0.70	2.47E-05	1.02E-04	9.28E+07	1.59E+08
P2	B		0.55	1.89E-05	7.93E-05	2.31E+08	3.96E+08
P3	B		0.44	1.52E-05	6.32E-05	4.87E+08	8.33E+08
P4	B		0.39	1.24E-05	5.09E-05	9.76E+08	1.67E+09
P5	B		0.30	1.03E-05	4.18E-05	1.84E+09	3.15E+09
P1	C		0.77	2.65E-05	1.22E-04	7.30E+07	1.25E+08
P2	C		0.60	2.06E-05	9.38E-05	1.73E+08	2.95E+08
P3	C		0.48	1.65E-05	7.35E-05	3.68E+08	6.30E+08
P4	C		0.39	1.34E-05	5.91E-05	7.49E+08	1.28E+09
P5	C		0.32	1.11E-05	5.02E-05	1.42E+09	2.44E+09
P2	C	SL		1.84E-05	7.49E-05	2.54E+08	4.34E+08
P3	C	SL		1.56E-05	6.65E-05	4.46E+08	7.63E+08
P3	C	S		3.06E-05	1.07E-04	3.50E+08	5.99E+08

The first cause can be addressed by increasing the stiffness or thickness of the pavement layers. The Shell design manual (*10*) provides guidelines for this purpose. For a 20-year design life, the limiting vertical compressive strain in the subgrade is 1.37E-4. At 20°C (67°F), the maximum vertical compressive strain for structural pavement type P3 is 7.35E-5. If a conservative value was used in the analysis corresponding to the softer binder, the vertical compressive subgrade strain would be 1.07E-04. This value is still within the limiting criteria.

On hot summer days it is likely that lower stiffness values might occur, thus increasing the vertical compressive subgrade strains. However, only a small percentage of the total truck traffic, less than 10 percent, would be present under those conditions.

MIX DESIGN

The second cause for permanent deformation is loss of shear stability of the mixture, which can only be controlled by proper mix design. Rutting in asphalt-concrete layers develops gradually with increasing load applications. It usually appears as longitudinal depressions in the wheelpaths accompanied by small upheavals at the sides. Rutting is caused by a combination of densification (decrease in volume and increase in density) and shear deformation. For properly compacted pavements, shear deformations, caused primarily by large shear stresses in the upper portions of the asphalt-aggregate layers, are dominant. Repetitive loading in shear is required 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 for the paving mixture in service. In the development of the rut depth it is also necessary to recognize the evolution of the air void content. When air void contents drop below approximately 2 to 3 percent, the binder acts as a lubricant between the aggregates and thus reduces point to point contact.

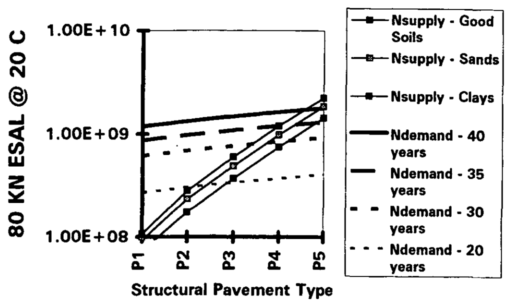


FIGURE 1 80 kN design (SHRP fatigue law, 10 percent cracking) design ESALs at 20°C critical lane.

Procedure to Evaluate the Rutting Propensity of a Mix

A procedure to estimate permanent deformation of asphalt concrete pavements was presented elsewhere (*11,12*). Figure 2 presents a nomograph of the procedure composed of four quadrants; it should be followed clockwise starting in Quadrant 1.

Quadrant 1: Esal versus Rut Depth

Step 1. Determine the number of ESALs for the design life
Step 2. Select maximum allowable rut depth

In the example, a 1,000,000 ESAL design life was selected, and the maximum acceptable rut depth was selected as 0.5 in (1.27 cm).

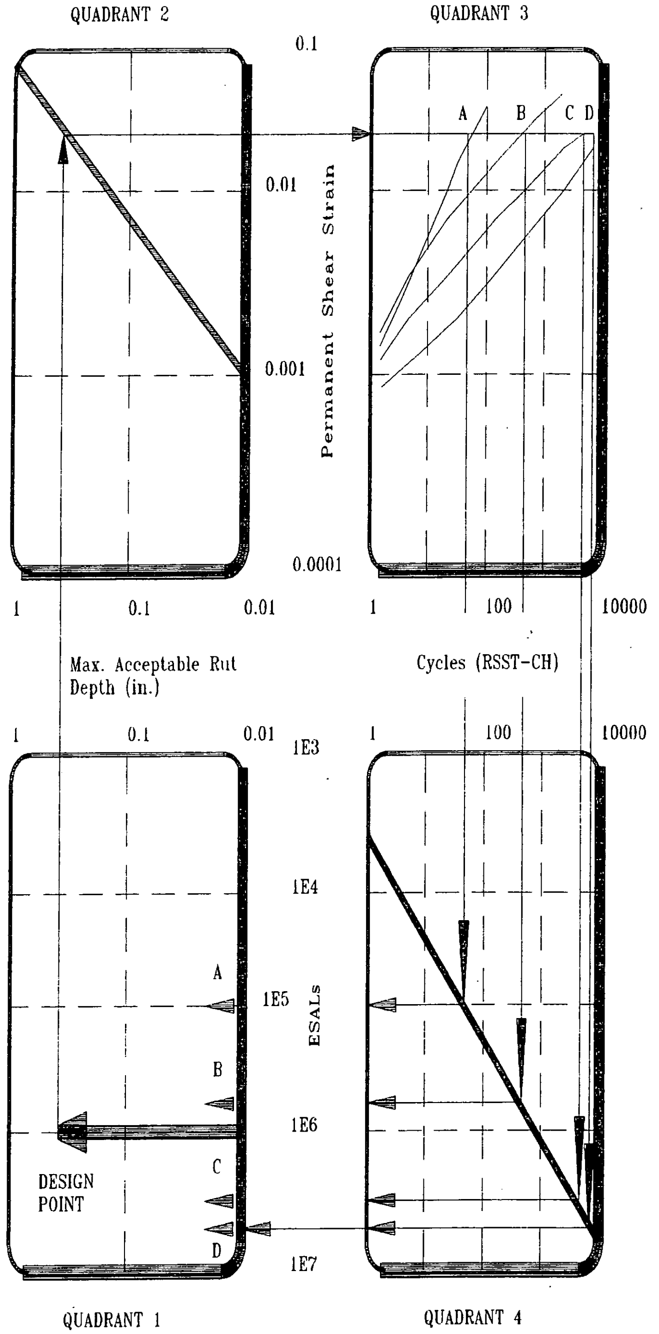


FIGURE 2 Asphalt-aggregate mix design concept (*12*).

Quadrant 2: Rut Depth versus Permanent Shear Strain

Step 3. By using the maximum allowable rut depth, the maximum allowable permanent shear strain is determined on the basis of a relationship between rut depth and maximum shear strain.

This relationship is given by

$$\text{Rut depth (cm)} = 28 * \text{maximum permanent shear strain} \qquad (3)$$

Quadrant 3: Permanent Shear Strain versus Cycles

Step 4. Determine mean highest 7-day pavement temperature on site at a depth of 5 cm (2 in.).

Step 5. Execute the repetitive simple shear test at constant height (RSST-CH) test at 69 kPa (10 psi) at that temperature.

Step 6. Determine the number of cycles in RSST-CH that yields maximum allowable shear strain given the relationship between shear strain and the number of cycles obtained from the RSST-CH.

The RSST-CH on a section 15 cm (6 in.) in diameter and 5 cm (2 in.) high was used to evaluate the rutting propensity of the mixes. Three typical graphs of permanent shear strain obtained from the simple shear test at constant height obtained are presented in Quadrant 3 (Figure 2). 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 = 1.25 cm rut depth/28; see Equation 3). It can be seen 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 maximum allowable rut depth of 1.25 cm is reached is determined by using the relationship between ESALs and RSST-CH number of cycles.

The equation is given by

$$\log(\text{cycles}) = -4.36 + 1.24 \log(\text{ESAL}) \qquad (4)$$

Equation 5 is an empirical equation relating the number of RSST-CH cycles in the laboratory to the number of ESALs in the field. This equation was obtained from SHRP general pavement section sites and has an $R^2 = 0.80$ (*11*).

With results obtained from the analysis which of the mixes would satisfy the design conditions can be identified. In the example, only Mix C and D would satisfy the requirements. Considerations should be given to reliability; adjustments might have to be made. For example, with reliability considerations, perhaps only Mix D would satisfy the requirements.

Determination of Mean Highest 7-Day Maximum Air Temperature

Maximum pavement temperature for a site usually varies within a wide range. To calculate the maximum pavement temperature for the site, data from the two or three nearest weather stations should be selected. Weather stations with more than 20 years of records should be included. For this preliminary project, only 10 years have been used. For each year, the average 7-day maximum temperature is calculated on the basis of the procedure elsewhere (*11*). For the temperatures obtained from the site at Lisbon Airport, which is very close to the location of the future bridge, that value was 31.7°C (89.1°F).

Determination of Surface Pavement Temperature

The pavement surface temperature was calculated using the procedure presented in a work by Sousa and Solaimanian (*11*). By using this iterative procedure, the surface temperature was calculated to be 56.1°C (133°F). Once the maximum pavement temperature at the surface is found through the preceding formula, the maximum pavement temperature for any depth less than 20 cm (7.8 in.) is found through an empirical equation (11). By using that equation, the following values were obtained for various depths: SMA, 2.5 cm depth, 51.9°C; M-1, 5.0 cm depth, 48.6°C; M-2, 14.0 cm depth, 41.2°C; M-3, 20.3 cm depth, 36.8°C. The temperatures in the table also correspond to the testing temperatures in the permanent shear deformation test (RSST-CH) for each of the mixtures in the pavement layers.

Most of the permanent deformation from shear stresses developing near the edge of the tires takes place at depths of less than 10 cm (3.9 in.). For this reason, special care should be given the mix design of layers SMA and M-1. The mixes should be stable at high temperatures and derive their stability from aggregate interlock.

Rutting Design Requirements

In situ aging of the mixes should also be considered; therefore, they should be tested after being exposed to an aging procedure that best corresponds to the design life that is expected.

Four levels of aging were considered for this project.

1. Short-term oven aging, 4 hr at 135°C (loose mix). The design ESALs expected during the first year should be used in the analysis. Predicted rut depth should not be more than 0.6 cm (0.2 in.).
2. Long-term oven aging, 2 days at 85°C (compacted specimens). The design ESALs expected during the first 3 years should be used in the analysis. The predicted rut depth should not exceed 1.0 cm (0.4 in.).
3. Long-term oven aging, 4 days at 85°C (compacted specimens). The design ESAL expected during the first 6 years should be used in the analysis. The predicted rut depth should not exceed 1.20 cm (0.5 in.).
4. Long-term oven aging 6 days at 85°C (compacted specimens). The design ESAL expected during the first 10 years should be used in the analysis. The predicted rut depth should not exceed 1.25 cm (0.55 in.).

It is known that as a mix ages it stiffens, thereby providing better resistance to permanent shear deformation. It is therefore important to take the age mix into account in the test protocol. Table 3 presents a worksheet for the determination of the parameters for the testing procedures.

Maximum design rut depth at the end of 10 years is 12.5 mm (0.5 in.) This is an acceptable value for pavements with a transverse slope of 2.5 percent. At that time, the first layer should be removed and replaced therefore eliminating any rut.

In Table 3, Column B contains the design ESALs at 80 kN. Column C shows the equivalent laboratory aging level, and Column D contains the cumulative number of ESAL until the corresponding aging level. Column E was determined using Equation 4, and Column F is a design requirement proposed by the authors that can be changed to fit any requirement. Column G was determined using Equation 3.

RSST-CH performance points, reported in Columns E and G for mixes SMA, M-1, M-2, and M-3—at the maximum pavement temperatures encountered at each depth—are presented in the following section.

SPECIFICATIONS

Specifications have been prepared for a preliminary design stage only and provide a general indication of the requirements needed to execute the project. During the execution design stage, more discriminating, accurate, and exact statements must be made. Although several specifications may be presented, only those related to the performance of the mixes in terms of stiffness, fatigue, and permanent deformation (with and without the effect of aging and water sensitivity) are crucial to the behavior of pavement. Penalties and bonuses might be awarded to the contractor on the basis of these values. Gradations and aggregate types and asphalt contents and types can vary as long as performance specifications are satisfied. If any of the performance specifications are not met by a mix, the mix has to be rejected or a new section redesigned to accommodate the new specification if possible. During the execution project level, testing of actual asphalt concrete mixtures is performed.

Specifications have been developed on the basis of new concepts of mix and pavement design. The study is a departure from the Marshall and Hveem empirical methodologies of mix design in that it stresses the determination of fundamental material properties known to affect pavement performance. Fatigue, permanent defor-

TABLE 3 Determination of Permanent Deformation Performance Points for Mix Design

A	B	C	D	E	F	G
Year	LANE (DESIGN) T=(ESAL/2)*0.7	Aging Proced.	Cumm. ESALS to Aging Level	Equivalent Cycles RSST-CH	Total Rut Depth (cm) Acceptable in period	Equivalent Permanent Shear Strain RSST-CH
1	2.84E+06	STOA	2.84E+06	**4,389**	0.60	**2.14E-02**
2	5.92E+06	2 Day				
3	9.26E+06	LTOA	1.80E+07	**43,362**	1.00	**3.57E-02**
4	1.29E+07					
5	1.69E+07	4 Days				
6	2.12E+07	LTOA	6.89E+07	**228,898**	1.20	**4.29E-02**
7	2.59E+07					
8	3.10E+07	6 Days				
9	3.65E+07	LTOA				
10	4.25E+07		2.05E+08	**883,134**	1.25	**4.46E-02**

mation, and thermal cracking (although the last one was not considered for this project) are the primary causes of distress in asphalt concrete pavements. These properties are affected by aging and moisture. Protocols have been developed to condition the mixes before testing to determine their fundamental properties. The project specifications require fatigue life and RSST-CH values for conditioned specimens above the minimal 85 percent found for the unconditioned specimens. The moisture conditioning is referred to as "ECFat" and "ECShear" for fatigue beams and RSST-CH specimens, respectively. Moisture conditioning for fatigue and permanent deformation is performed by vacuum saturation of the specimens, followed by immersion for three cycles of 6 hr at 60°C. Between cycles they remain immersed for at least 4 hr at 25°C (75°F).

Specifications for Determination of Fatigue Properties

1. All fatigue testing will be performed at 20°C (67°F), and according to the SHRP A-003A beam fatigue protocol. For each material the following fatigue beam testing program will be performed to determine acceptance.

–The design strain level is the maximum tensile strain at the bottom of the asphalt concrete layer. At this strain level the mix is expected to last N_{demand} cycles. For each material, the mix must reach the required number of repetitions at a given maximum strain. The procedure for determining whether the required number of repetitions is reached is detailed in items immediately following. A reliability factor of 90 percent is applied to the project fatigue life.

–Beam specimens will be fabricated at 4 percent air-void content (with parafilm method), ±0.5 percent air-voids. Beams will be prepared by using rolling wheel compaction and will be mixed and compacted at the specified construction temperatures.

–Two tests, one at 300 microstrain and the other at 700 microstrain, will be conducted to provide a preliminary fatigue curve (relation of $N_f = k_1 * \text{strain}^{k_2}$). From this curve, two strain levels will be selected for further testing, those that result in N_f equal to 50,000 and 500,000 repetitions. The approximate testing time is 24 hr.

–A minimum of four fatigue tests is required (two tests at each of the two strain levels). A least-squares regression fatigue curve must be fitted through the test points. More than four beams can be used to reduce the required number of fatigue load repetitions (N_r). The following equation (*4*) is used to calculate N_r:

$$N_r = N_{demand} * \text{EXP}(Z \times SDn) \quad (5)$$

where

EXP = natural log base, *e;*
Z = coefficient that varies with the confidence levels (Z = 0.84 for 80 percent, 1.28 for 90 percent, 1.64 for 95 percent), and
SDn = standard deviation for the number of beam tests at two strain levels (*4*) and is given by
= SQRT $[0.006903 * (N_{supply})^{0.2988}]$ for 1 replicate per strain level,
= SQRT $[0.009653 * (N_{supply})^{0.2455}]$ for 2 replicate per strain level,
= SQRT $[0.013450 * (N_{supply})^{0.2086}]$ for 3 replicate per strain level, and
= SQRT $[0.017640 * (N_{supply})^{0.1817}]$ for 4 replicate per strain level.

The following two examples illustrate the options that exist for testing to determine acceptance of the material, assuming $N_{demand} = 3.38 * 10^7$ repetitions.

Option 1. Four tests must be performed, two at each strain level. A least-squares regression fatigue curve fitted through the four points plus the two preliminary points must result in a calculated N_r of at least $8.4 * 10^7$ cycles at the design strain [$(N_r = 3.38 * 10^7 *$ EXP $(1.28 * 0.707)$]. The approximate testing time is 48 hr.

Option 2. Eight tests must be performed, four at each strain level. Following the procedure in Option 1, the calculated N_f must be at least $7.7 * 10^7$ cycles at the design strain [$N_r = 3.38 * 10^7 * EXP\ (1.28 * 0.645)$]. The approximate testing time is four days.

Figure 3 illustrates the specification concept and identifies a performance point in fatigue corresponding to the design level

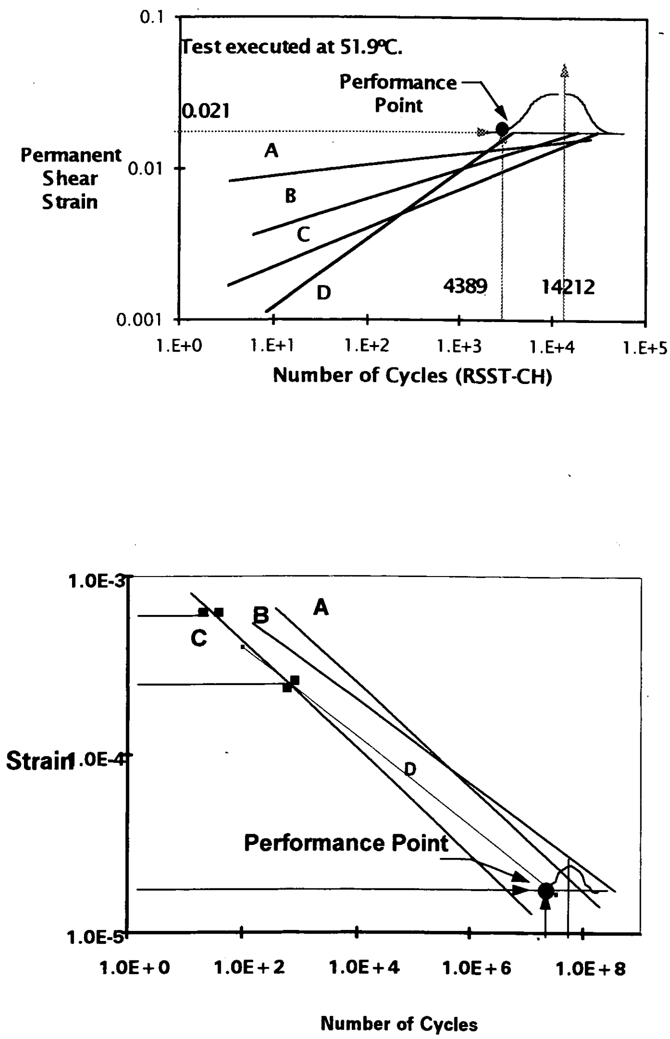


FIGURE 3 Illustration of performance point concept for permanent deformation and fatigue.

computed by ELSYM. Mix A and B would satisfy the requirements, whereas Mix C and D would be rejected (or the pavement section designed).

On the basis of these concepts, fatigue performance points for the mixes are presented in Table 4. Maximum tensile strain is encountered at the bottom of the M3 layer, although it was expected that the top mix, M1, should obey the same criteria, because fatigue cracks can be initiated on the top of the asphalt concrete layer. Given the reduced stiffness of the top mix, it is likely that in some cases aging might harden the mix and therefore reduce its fatigue characteristics. For this reason, fatigue tests after long-term oven aging were recommended .

M3 fatigue resistance basically will control the fatigue life of the pavement. As moisture can affect fatigue performance, a fatigue-life requirement after moisture conditioning is recommended.

For the M2 mix, no fatigue requirement is recommended because tensile strain levels are likely to be very low (except during crack propagation).

During fatigue testing, measurements of stiffness can be made. Table 4 presents the stiffness performance points for the mixes. Requirements after moisture conditioning are recommended for critical mixes.

Specifications for Determination of Permanent Shear Deformation Properties

All permanent shear deformation testing will be performed at the permanent shear deformation design temperature and according to the SHRP A-003A testing protocol. RSST-CH tests will be executed at 69 kPa (10 psi) shear stress (0.1 sec loading with a 0.6 sec rest period).

Cylindrical specimens, 15 cm in diameter by 5 cm high, will be fabricated to a target 3.2 percent air-void content (with parafilm method), ± 0.4 percent air voids. If air-void contents of the specimens are not exactly 3.2 percent, it is acceptable to interpolate or extrapolate the results from specimens with void contents between 2.9 and 3.8 percent with a linear relationship between log of cycles to the required strain with a linear variation of the voids. Specimens will be prepared using rolling wheel compaction and will be mixed and compacted at the specified construction temperatures.

Two RSST-CH tests carried out for at least 4,000 repetitions will be conducted at the specified testing temperature to provide a preliminary rutting curve relation of [Log (permanent shear strain) = A + B * log (cycles)]. A least-squares regression rutting curve must be fitted through the test points for data that begin after 1,000 cycles. This relationship can be extrapolated to any strain level. The approximate testing time is 2 hr.

The number of cycles N_{demand} to reach the required permanent shear strain has to be adjusted to account for reliability based on the variance of the test. The required number (N_r) of cycles is calculated by

$$N_r = N_{demand} * \mathrm{EXP}\ (Z \times SDn \qquad (6)$$

where

EXP = natural log base *e*;

TABLE 4 Permanent Deformation, Fatigue, and Stiffness Performance Points for Mixes

MIX	CONDITIONING PROCEDURE	TESTING TEMPERATURE (°C)	PERMANENT SHEAR STRAIN	N (demand) cycles RSST-CH(1)
SMA	STOA	51.9	0.0214	4389
	2 days LTOA	51.9	0.0357	43362
	4 days LTOA	51.9	0.0429	228898
	6 days LTOA	51.9	0.0446	883134
	STOA+ECShear	51.9	0.0214	3730 (2)
M1	STOA	48.6	0.0214	4389
	2 days LTOA	48.6	0.0357	43362
	4 days LTOA	48.6	0.0429	228898
	6 days LTOA	48.6	0.0446	883134
	STOA+ECShear	48.6	0.0214	3730 (2)
M2	STOA	41.2	0.0446	883134
M3	STOA	36.8	0.0446	883134

1- From Table 3
2- From Table 3 requiring 0.85% resistance after moisture conditioning

			FATIGUE		STIFFNESS	
MIX	CONDITIONING PROCEDURE	TESTING TEMPERATURE (°C)	DSL-TENSILE STRAIN	N(demand)(1)	TENSILE STRAIN	STIFFNESS GPa
M1	STOA	20	1.65E-06	3.38 E7	700E-06	>8
	STOA + ECFat	20	1.65E-06	2.87 E7	700E-06	>7
	4 LTOA	20	1.65E-06	2.87 E7		
M2	STOA	20	Not Required	Not Required	700E-06	>10
M3	STOA	20	1.65E-06	3.38 E7	700E-06	>20
	STOA + ECFat	20	1.65E-06	2.87 E7(2)	700E-06	>16

1 . Demand traffic dividing by 10 (shift factor)
2 . Accepting a reduction of 85% in fatigue life due to moisture effects

Z = coefficient that varies with the confidence levels (Z = 0.84 for 80 percent, 1.28 for 90 percent, 1.64 for 95 percent); and

SDn = standard deviation for the RSST-CH for two replicates (= 0.918).

Example: If the N_{demand} is equivalent to 4389 RSST-CH cycles, then the test results should be higher than

$[N_r = 4389 * \mathrm{EXP}\ (1.28 * 0.918) = 14212$

The test should be executed at the required testing temperature within ± 0.5°C. Figure 3 illustrates the concept of the performance point in permanent deformation for the SMA mix. Note that either Mix A, B, or C would satisfy the criteria, but Mix D would not.

On the basis of these testing procedures, the performance points for permanent deformation of all mixes are presented in Table 4. Only SMA and M1 mixes have performance requirements after aging and moisture conditioning. If the mix satisfies the N_{demand}, the 6 day long-term aging requirement with short-term oven aging only, then there is no need to execute the 2-4- or 6-day oven-aging process and corresponding testing. Although mixes M2 and M3 will not be subjected to the same shear stress levels in the pavement as those imposed on mixes SMA and M1, the same testing procedure (RSST-CH) was recommended, but at lower testing temperatures.

SUMMARY

The authors present new concepts for asphalt-aggregate mix evaluation based on the findings of the SHRP-A003A project. With new methodologies, it is now possible to tie together pavement performance, structural pavement design, and mix design. The key that links all these concepts together is performance-related mix specifications.

A new pavement was designed in Portugal by CONSULPAV for the new crossing of the Tagus river between Lisbon and Montijo. Asphalt-aggregate mixes were specified based on the results from four-point bending beam fatigue tests, flexural stiffness, and RSST-CH. The specifications constitute a departure from Marshall and Hveem mix design concepts and permit definition of performance points for mixes that are truly site specific.

The concept of performance point has been introduced to facilitate development of specifications. This powerful new approach can be easily implemented at any project level.

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