Material Properties of Zero-Maintenance Flexible Pavement

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This study involved models selected as suitable for the prediction of important distresses that have occurred historically in flexible pavements and the determination of material properties that will provide 20 years of satisfactory performance without maintenance and an additional 10-20 years of satisfactory performance with normal maintenance. To accomplish the overall objective, results were used from a previous study that identified the most important distresses occurring in flexible pavements, identified the material properties that affect those distresses, and selected mathematical models to predict those distresses by using the identified material properties. This paper presents the range of values selected for each of the material properties and discusses the distresses predicted by using the selected mathematical models. The distresses selected for study included fatigue cracking, rutting, and low-temperature cracking. The mathematical models selected for the analysis were VESYS for modeling fatigue cracking and rutting and the Shahin-McCullough model for low-temperature cracking. Input values were selected for each of these models, and ranges of the material properties affecting distress were established. The results for each study have been included and discussed, and practical criteria have been cited to evaluate the level of material properties required to provide zero-maintenance performance. The effects of each set of material properties used are identified, and the trade-offs of the effects of material properties on the distresses are discussed.

The Federal Highway Administration (FHWA) has for several years pursued multiple research studies aimed at producing premium pavement structures for heavily traveled highways. The objectives of these various efforts have been to develop pavement-design methodologies and to establish ranges of material properties for pavements so that they will be maintenance free for 20 years and require only routine maintenance for 10-20 years thereafter.

The research reported here had as its goal the identification of material properties that would provide optimal performance in flexible, rigid, and composite premium or zero-maintenance pavements. The portions of the project to be discussed include

1. Review of the analytical models capable of modeling distresses in flexible pavements that were selected to evaluate the effect of material properties on the production of distress,
2. Establishment of ranges of material properties and evaluation of the effects of the material properties on production of distress, and
3. Development of ranges of properties that are most suited to the production of zero-maintenance pavements.

The material properties identified as important in the study of flexible-pavement distress are discussed here and have been reported elsewhere (1).

MODEL SELECTION AND DESCRIPTION

A complete literature survey was conducted to review all analytical models available to predict pavement distress. The models reviewed included those based on linear elasticity, finite-element theory, and elastic-layer theory. These various models were classified and compared with the final model selected (1) for capability of predicting distress. The models used in the study of flexible-pavement distress were VESYS (2,3) for fatigue and rutting and the Shahin-McCullough model (4,5) for low-temperature cracking.

MODEL INPUTS

The models selected for the distress studies outlined above require the development of input values for each variable that occurs in each model. Several categories of input variable are discussed.

Common Inputs

Several categories of factors are common to all pavements and models and are independent of the pavement material properties themselves: the environment in which the pavement occurs, the traffic level to which the pavement is subjected, and the thickness of the pavement layers. Other input factors are dependent on the model used and the pavement type.

Four zones were selected as representative of the United States and are designated wet—freeze, dry—freeze, wet--no-freeze, and dry—no-freeze.

Previously reported traffic data (1,6,7) indicated that most flexible pavements sustain less than 500 000 18-kip (80-kN) equivalent single-axle loads (ESALs) per year. However, to ensure an adequate level of traffic, a design value of 1 million 18-kip ESALs was selected. Initial thicknesses for the flexible-pavement studies were developed by using SAMP6, a computer design procedure based on the procedures of the American Association of State Highway and Transportation Officials (AASHTO) (8). Inputs for SAMP6 were developed by using the traffic and environmental data associated with each zone, available structural and soil data, and 1977 material costs.

Since fatigue cracking, the most prevalent cause of failure in flexible pavements, is not considered directly by SAMP6, an elastic-layer analysis using ELSYM5 was conducted by using the initial layer thicknesses and fatigue relationships developed by Austin Research Engineers (9) and by Finn (10). This analysis indicated that only deep-strength or full-depth asphalt-concrete pavement could be expected to survive the 20 million 18-kip ESALs without fatigue damage.

By using the cost-optimization information based on serviceability, SAMP6, and these fatigue relationships, the minimum cross sections at thicknesses selected for the study of flexible pavements were 13 in (330 mm) of asphalt concrete and 8 in (203 mm) of asphalt-stabilized granular base. The base served primarily as a drainage layer and thus was open graded and asphalt treated for waterproofing, which minimized changes in modulus between seasons and provided constant support to the surface layer.

For the fatigue analysis, additional surface thickness combinations of 18, 16, and 10 in (457, 406, and 254 mm) over an 8-in base were selected.

The basic properties required for the analysis of flexible pavements were fatigue constants, stiffness, permanent deformation characteristics, and low-temperature cracking factors. Values selected for each are discussed below. VESYS IIIA was used for the first three properties; the Shahin-McCullough model was used to study low-temperature cracking.

Material Properties: VESYS IIIA Model

Fatigue Constants

By using information from various reports (11-15), we performed a regression analysis to determine the nature of and correlation coefficients for a relationship between material constants K_1 and K_2 (11). An excellent correlation coefficient and standard error were obtained for

K_2 as a function of K_1 as given below:

$$K_2 = 1.350 - 0.252 \log K_1 \quad (R = 0.95, SE = 0.29) \tag{1}$$

Since the values of K_1 and K_2 vary with temperature, they were modified to reflect the temperature range of each environmental zone by using the techniques presented by Rauhut, O'Quin, and Hudson (2).

Values of K_1 at temperatures other than 70°F (21°C), designated K_1 (T), were obtained by multiplying K_1 (70°F) by a normalizing value for temperature T (2).

Selected values for K_1 spanned the range of reported values, and K_2 was computed by using the equation developed earlier (11). Below are values of the K_1 and K_2 at 70°F selected for the parameter study.

Fatigue Constant	Values Selected			
log K_1	-5	-10	-15	-20
K_2	2.61	3.87	5.13	6.39

The seasonal temperatures for each environmental zone are shown below [t°F = (t°C ÷ 0.55) + 32]:

	Environmental-Zone Temperature (°F)			
Season	Wet--Freeze	Dry--Freeze	Wet--No-Freeze	Dry--No-Freeze
Winter	35	35	75	55
Spring	65	60	95	75
Summer	95	90	105	95
Fall	60	50	95	75

Stiffness

The multipliers used to establish the asphalt-concrete stiffness at each seasonal temperature were developed primarily by using results from the Asphalt Institute (16) and were checked with results from a study by Hudson and Kennedy (17). These data were plotted, and multipliers were selected for each seasonal temperature that was used in this analysis.

Permanent Deformation Parameters

The procedures for calculating permanent deformations at the pavement surface that were used in VESYS IIIA and the derivation of the terms ALPHA and GNU are discussed in detail elsewhere (2). The findings of the more-extensive testing programs conducted to evaluate ALPHA and GNU are also reported elsewhere (2,6,18). ALPHA and GNU are defined by the intercept and slope of the relationship between the logarithm of permanent strain and the logarithm of the number of load applications. ALPHA values that range from 0.7 to 0.9 and GNU values that range from 0.2 to 1.4 were selected by using previous recommendations (2).

The effects of variations in the permanent deformation characteristics of the base and subgrade materials were eliminated by using constant values for ALPHA and GNU of 0.92 and 0.25 for the base and 0.90 and 0.15 for the subgrade, respectively.

Stochastic Parameters

Because of the very heavy traffic and low mileage of zero-maintenance pavements, extraordinarily high quality control of construction was believed to be necessary. The quality-control level selected closely matched that exercised during the construction of the AASHO Road Test pavements. Rauhut has discussed the AASHO Road Test quality control and has presented typical values for the coefficients of variation for the various parameters of strength, stiffness, and permanent deformation (2). The following values were selected to be compatible with the objectives of providing zero-maintenance pavements:

Variable	Description	Input Value
VARCOEF	Coefficient of variation (C_v) in material properties for	
	Surface	0.10
	Base	0.15
	Subgrade	0.15
COEFK$_1$	C_v in K_1	0.20
COEFK$_2$	C_v in K_2	0.04
K_1K_2CORL	Correlation coefficient between K_1 and K_2	-0.9

Material Properties: Shahin-McCullough Model

The asphalt-concrete material properties required for analysis of low-temperature cracking are given below [t°F = (t°C ÷ 0.55) + 32; 1 psi = 6.89 kPa]:

	Level	
Material Property	Low	High
Original penetration at 100 g, 5 s, 77°F (mm)	120	50
Original softening point (°F)	115	125
Penetration after thin-film oven test (%)	67	70
Specific gravity	1.040	1.015
Maximum tensile strength (psi)	300	600
Thermal coefficient ($\times 10^{-6}$/°F)	11	17

Required material properties include stiffness, tensile strength, and thermal coefficient. The range of stiffness for the parameter study was determined from typical properties for AC-40 and AC-10 asphalt cements. To ensure a proper comparison of cracking and stiffness, the material properties for an asphalt cement that were used to calculate the stiffness were made comparable in terms of temperature susceptibility by producing similar penetration indices. These material properties were established by using previously determined information (19-26).

The values shown above for tensile strength represent the maximum that was expected to occur at a bitumen stiffness of approximately 10 000 psi (68 948 kPa). A range in tensile strength from 300 to 600 psi (2068 to 4136 kPa) was selected for this study. The range in thermal coefficients was derived from Simpson, Griffin, and Miles (22). The values for the thermal coefficient of asphalt concrete were held constant for this analysis even though they do vary with temperature.

Climatological data from selected cities in each environmental zone were used to develop a composite set of data representative of each zone (27). The annual air-temperature range was developed so that the lowest calculated air temperature over the analysis period was approximately the lowest 20-year extreme. The input variables that varied between the zones were from a recent report (28).

DISCUSSION OF RESULTS

Fatigue Cracking

A typical relationship between the percentage of fatigue cracking and time for the wet—freeze environmental zone is shown in Figure 1. The fatigue cracking is the predicted cumulative damage for each season of each year accumulated by using the summation-of-cycle ratios. Since the traffic was assumed to be uniform at 1 million 18-kip ESALs per year, time was used on the abscissa. These relationships show the percentage of cracking for each K_1, K_2, and thickness combination. Only values for K_1 are shown, since K_2 is a function of K_1. It is quite apparent that surface thickness is the dominant factor that influences fatigue cracking. Until the total thickness exceeded some critical value, a 20-year fatigue life could not be obtained by varying K_1 and K_2 within the range of values reported for currently available materials. For the lowest log K_1 value, which produces the longest value of fatigue life, the 13-in (305-mm) pavement experienced fatigue cracking exceeding tolerable levels in all environmental zones within 5-13 years. No set of material properties investigated met the zero-maintenance criteria for a surface thickness of 13 in over an 8-in (203-mm) base (Table 1).

As the pavement thickness was increased from 13 in to 16 in (406 mm) and 18 in (457 mm), the asphalt concrete began to perform in a manner that would satisfy zero-maintenance requirements. At a log K_1 of -10 for the 18-in surface, the fatigue cracking for all environmental zones was at an acceptable level after 20 years. However, it should be noted that, once fatigue cracking was initiated, only a few years at this very high traffic rate were required for the fatigue cracking to advance to an unacceptable level. It can be seen that zero-maintenance fatigue life could not be obtained solely by increasing the fatigue properties over a range consistent with currently available materials. Only when thicknesses were increased was it possible for several levels of fatigue properties to provide zero-maintenance service.

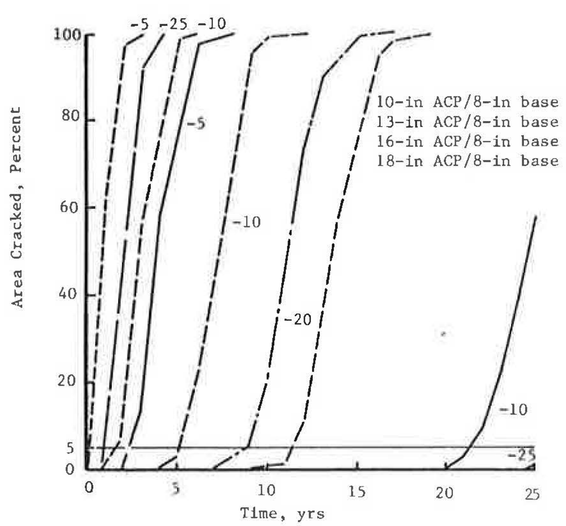
The development of fatigue cracking occurred in a similar pattern for the wet--freeze, dry--freeze, and dry--no-freeze zones. However, in the wet--no-freeze zone the time required to develop an unacceptable level of class 3 and class 4 fatigue cracking was substantially longer than it was for the other zones. This was attributed to the interaction of the strain-stiffness-fatigue relationship produced by the higher seasonal temperatures and the smaller temperature range between seasons for the wet—no-freeze zone. It should also be noted that the fatigue-life relationships are more closely spaced for the wet--no-freeze zone than for any of the other zones, which indicates that in higher-temperature zones the difference between the fatigue constants is not nearly so important as it is in the lower-temperature zones in which the range of stiffnesses during a year was larger.

Table 1 shows the fatigue life for the wet—freeze zone as a function of one of the accepted fatigue-cracking criteria. The fatigue properties significantly affected the time required to produce 5 percent fatigue cracking, but the more significant effect of thickness is evident. As can be seen in Table 1, the increase in time that was required in the wet--freeze zone to develop unacceptable cracking increased by 460 percent when log K_1 changed from -10 to -20; however, the increase in time produced by increasing the thickness from 13 in to 18 in was 960 percent when log K_1 equaled -10. Similar effects were also observed for the dry--freeze and the dry—no-freeze zones. For the wet—no-freeze zone the effect of thickness was even more dramatic than for the other zones; an increase in thickness for 13 in to either 16 in or 18 in resulted in essentially no class 3 and class 4 fatigue cracking and thus satisfied the zero-maintenance requirements.

For the 18-in pavement, a material with log K_1 less than -10 would meet the requirement for no maintenance during the first 20 years. However, to ensure that the fatigue cracking was not a problem for 25 years or more, a log K_1 equal to or less than -15 is suggested for a surface thickness of 18 in.

In developing criteria for selection of thickness requirements for zero maintenance, the engineer must evaluate the costs and uncertainties associated with the more-stringent material-property requirements as compared with the cost and uncertainty of obtaining a thicker layer and the less-stringent material-property requirements for the thicker layer. These evaluations must be performed for each proposed zero-maintenance project.

Figure 1. Fatigue in flexible pavements for various K_1, K_2 combinations and surface thicknesses: wet—freeze zone.



Note: No cracks for log K_1 = -15 and -20 for 16- and 18-in ACP over 8-in base.

Rutting

A typical relationship between cumulative permanent deformation and time for the wet—no-freeze zone and an ALPHA of 0.7 is shown in Figure 2. The relationships for the other environmental zones were similar, and summary information taken from those relationships was included in subsequent analyses.

To establish a limit for rutting, safety considerations have frequently been used. Generally, when rutting is 0.5 in (13 mm) or less, the crossfall of a premium pavement is sufficient to prevent both significant water accumulation in the wheel paths and steering problems associated with moving out of the wheel path. Therefore, any material that provided a combination of permanent deformation parameters (ALPHA and GNU) that produced a rut of 0.5 in or less after 20 million 18-kip ESALs was considered acceptable as a zero-maintenance pavement.

Table 1. Time required (in years) for 5 percent fatigue cracking to occur.

	13-in Thickness				16-in Thickness		18-in Thickness			
	Log K_1				Log K_1		Log K_1			
Zone	-5	-10	-15	-20	-10	-15	-5	-10	-15	-20
Wet—freeze	0.2	2.0	5.2	11.2	8.0	25[a]	22[a]	21.2[a]	25[a]	25[a]
Dry—freeze	0.1	1.4	4.7	11.5	7.5	25[a]	2.2	20.7[a]	25[a]	25[a]
Wet—no-freeze	6.7	7.4	7.4	8.7	25[a]	25[a]	25[a]	25[a]	25[a]	25[a]
Dry—no-freeze	0.5	1.3	2.5	4.8	9.1	24.7	5.1	23.7[a]	25[a]	25[a]

[a]Satisfies zero-maintenance requirements.

As shown in Figure 2, most of the rutting occurs early in the service life of the pavement; at least 50 percent occurred during the first five years. After five years, most combinations of ALPHA and GNU sustained very little additional cumulative permanent deformation.

The time required for a 0.5-in rut to develop was determined for each ALPHA value and each zone; the results are shown in Table 2. Analysis of these results showed that a GNU level of 1.4 was unsatisfactory for ALPHA values less than 0.8. Since most reported ALPHA values are less than 0.9, GNUs of 1.4 or greater are not expected to satisfy the zero-maintenance requirements for rutting. However, for ALPHAs as high as 0.9, all values of GNU met the rutting criterion.

Figure 2. Rutting versus time for ALPHA = 0.7, all GNUs, and 13-in surface: wet—freeze zone.

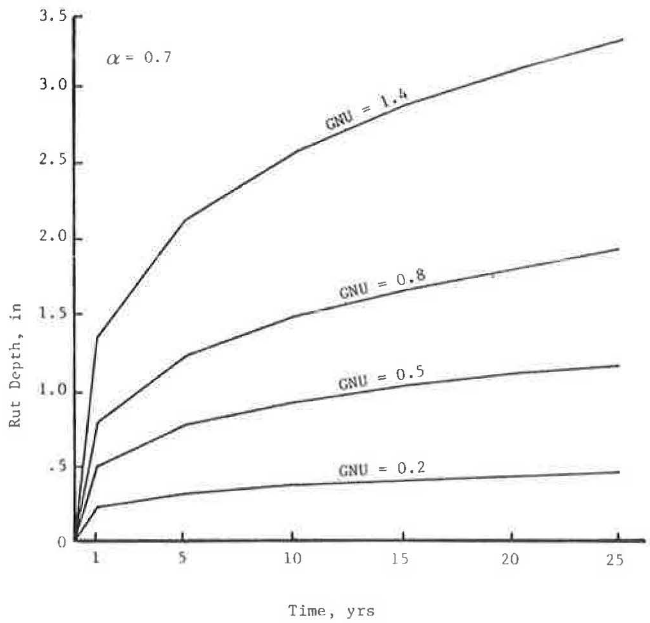


Table 2. Zones in which acceptable rutting performance occurred for both 13- and 18-in surfaces.

Value of GNU	Value of ALPHA: 0.7	0.8	0.9
0.2	WF, DF, W-NF, D-NF[a]	WF, DF, W-NF, D-NF[a]	WF, DF, W-NF, D-NF[a]
0.5	–	WF, DF, W-NF, D-NF[a]	WF, DF, W-NF, D-NF[a]
0.8	–	WF, DF, D-NF[a]	WF, DF, W-NF, D-NF[a]
1.4	–	–	WF, DF, W-NF, D-NF[a]

Note: WF = wet—freeze zone; DF = dry—freeze zone; W-NF = wet—no-freeze zone; D-NF = dry—no-freeze zone.

[a]Zero-maintenance requirements are satisfied for each environmental zone listed.

The form of the relationship between GNU and time to a 0.5-in rut depth was the same for the wet—freeze, dry—freeze, and dry—no-freeze zones. The interactive effects of temperature and stiffness produced a characteristically different response curve for the wet--no-freeze zone. Again, with the higher temperatures, lower stiffnesses, and increased deflections, the cumulative permanent deformations were more critical in that zone than in any of the other environmental zones.

Table 2 also shows the material combinations of ALPHA and GNU that produced acceptable rutting performance by using the previously cited criteria. The only combination that did not meet the criteria was for ALPHA of 0.8 and GNU of 0.8 in the wet—no-freeze environmental zone. This wet--no-freeze zone was typically very warm and was the only zone where rutting rather than fatigue cracking was the most prevalent distress.

Low-Temperature Cracking

A summary of the results of the low-temperature cracking analysis is presented in Table 3. For each combination of inputs the tabulated values represent the calculated amount of low-temperature cracking.

The AC-40 mixtures experienced greater cracking than did the AC-10 mixtures. These results imply that low-temperature cracking increases as the bitumen stiffness increases in all environmental zones, even though almost no cracking occurred in the no-freeze zones.

Results showed that cracking increased as tensile strength decreased. The model predicted significantly lower cracking as the mixture strength increased from 300 to 600 psi (2068-4136 kPa). In general, changes in strength had a greater effect on cracking than did changes in asphalt type. However, it should be pointed out that significant changes in strength are usually coupled with changes in asphalt type, so that the effects of these two factors cannot be easily separated.

Other results showed that cracking decreased as the thermal coefficient was reduced from 17×10^{-6}/°F (31×10^{-6}/°C) to 11×10^{-6}/°F (20×10^{-6}/°C). The effects of changes in thermal coefficient on cracking are comparable to those of strength but controllable only through aggregate selection.

The asphalt cement that worked best to prevent low-temperature cracking was one that had low stiffness at low temperatures. For a particular grade of asphalt

Table 3. Predicted low-temperature cracking.

Type	Tensile Strength of Mixture (psi)	Thermal Coefficient ($\times 10^{-6}$/°F)	Cracking (ft/1000 ft^2): Wet—Freeze Zone	Dry—Freeze Zone	Wet—No-Freeze Zone	Dry—No-Freeze Zone
AC-10[a]	300	11	15[b]	67	0.2[b]	0.1[b]
		17	63	123	2.9[b]	1.5[b]
	600	11	0.3[b]	4.78[b]	0[b]	0[b]
		17	3.9[b]	33	0.0[b]	0.0[b]
AC-40[c]	300	11	37	138	1.0[b]	0.6[b]
		17	98	167	11[b]	7.0[b]
	600	11	1.3[b]	45	0.0[b]	0.0[b]
		17	14[b]	107	0.2[b]	0.1[b]

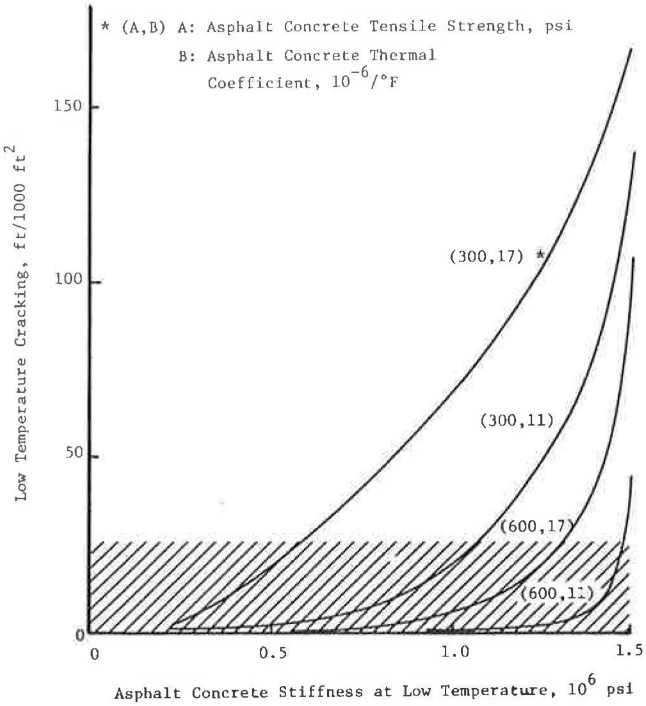
Note: 1 psi = 6.89 kPa; t°F = (t°C ÷ 0.55) + 32; 1 ft = 0.3 m.

[a]Properties of AC-10 are penetration value, 120; original penetration temperature, 77°F; ring-and-ball penetration temperature, 115°F; penetration after thin-film oven test, 67 percent; specific gravity, 1.040.

[b]Satisfies zero-maintenance requirements.

[c]Properties of AC-40 are pentration value, 50; original penetration temperature, 77°F; ring-and-ball penetration after thin-film oven test, 70 percent; specific gravity, 1.015.

Figure 3. Low-temperature cracking versus asphalt concrete stiffness.



cement, low stiffness at low temperatures can be obtained by means of a high penetration index. The penetration index is a measure of asphalt consistency developed by Shell researchers that is based on measurements of penetration at 77°F and the ring-and-ball softening point.

Figure 3 was prepared for use in selecting material-property levels to minimize low-temperature cracking; the shaded area satisfies zero-maintenance requirements, as in Haas (29). At a mixture stiffness of 500 000 psi (3 447 000 kPa), the predicted cracking was greater than 10 ft/1000 ft² (33 m/1000 m²) for a maximum tensile strength of 300 psi and a thermal coefficient of 17 x 10^{-6}/°F. For these properties, the predicted cracking was approximately 20 ft/1000 ft² (66 m/1000 m²). As can be seen in Figure 3, thermal cracking at a stiffness of 500 000 psi may be eliminated either by decreasing the thermal coefficient or by increasing the tensile strength. After extensive observations in Canada, McLeod (30) recommended a limiting stiffness of 500 000 psi to prevent low-temperature cracking for an expected low temperature of -40°F (-40°C).

Due to the nature and background of the Shahin-McCullough model, the study has several imposed limitations. The most severe was the restriction on the length of the analysis period. The asphalt-cement aging model was a regression model based on data for asphalt cements that had been aged for 10 years or less. No basis exists for extrapolating the results to longer periods of aging. Thus, a 20- or 30-year analysis could not be directly simulated to demonstrate zero-maintenance pavement performance for the material properties under investigation. However, the material properties that minimize low-temperature cracking over a 10-year period were assumed to be applicable for the zero-maintenance time period.

The results show that the amount of cracking predicted was a function of the lowest input temperature. The daily temperature drops for both the wet and dry environmental zones were approximately the same. The greatest amount of cracking was predicted for the wet--freeze zone, in which the low air temperature was -30°F (-34°C). The least amount of cracking was calculated for the dry--no-freeze zone, in which the low air temperature was 8°F (-13°C).

The ideal asphaltic concrete for prevention of low-temperature cracking was determined to be one that has a minimum thermal coefficient of contraction, a high tensile strength, and a minimum stiffness at the expected low temperature. Currently available materials are generally unable to provide this set of properties and at the same time meet the other distress requirements.

It is difficult to infer the low-temperature performance of conventional asphaltic paving materials for zero-maintenance pavement criteria. Although conservative low temperatures were used, the effects of aging were not considered over the required 20-year maintenance-free period. If additional aging is assumed to have no effect, the results obtained in this analysis can be applied to longer analysis periods.

SUMMARY

Permanent Deformation

Permanent deformation parameters used in VESYS were not related to any other material properties, and the values suggested in Table 2 should be used for the environmental zones indicated. No significant interaction was observed between pavement thickness and predicted rutting for the permanent deformation parameters included in this analysis. However, if the full-depth asphalt-concrete thickness exceeds 18 in (457 mm), additional rutting calculations should be performed to ensure that excessive rutting is not predicted. To establish the rutting potential of an asphalt-concrete mixture that has been proposed for use in a zero-maintenance pavement, laboratory testing should be performed to determine the values of ALPHA and GNU for each mixture; Kenis (3) has proposed testing procedures for measuring these values. VESYS IIIA showed that rutting was a very significant problem for the wet--no-freeze zone but that it was not a significant problem for the other zones, especially the freeze zones.

Fatigue Cracking

Zero-maintenance fatigue cracking criteria were satisfied by several of the material-property combinations included in the study. Adequate fatigue life could not be achieved by using material properties that were included in this study for the 13-in (330-mm) surface over an 8-in (203-mm) base for any environmental zone. Even when the log K_1 was increased to -25, which corresponds to a material similar to an excellent sulfur asphalt, the improvement in material property was not sufficient to overcome the effects of strain in the thinner 10-in (250-mm) and 13-in pavements. However, when the thickness was increased to 16 in (406 mm), the better material-property values began to produce performance compatible with zero-maintenance requirements, as shown in Figure 1 and as summarized in Table 1. As the pavement thickness was increased to 18 in, more than 75 percent of the material combinations met the zero-maintenance fatigue criterion.

Low-Temperature Cracking

As can be observed in Table 3, low-temperature cracking for the no-freeze zones was minimal. It should be noted that the Shahin-McCullough model does predict cracking for the combinations of high thermal coefficient and low tensile strength of the mixture; the predicted crack spacing is about 80 ft (24 m). In the freeze zones, the model results indicate that the high tensile strength and low thermal coefficient would be satisfactory for an AC-10 but that a crack spacing of about 20 ft (6 m) would result for the AC-40 from the same mixture characteristics. For the freeze zones, the AC-10 with high tensile strength and low thermal coefficient served best to minimize or to prevent formation of low-temperature cracking.

Techniques considered to prevent low-temperature cracking and to keep fatigue life high and rutting low work at cross-purposes. To minimize low-temperature cracking, the stiffness should be low at the design low temperature; thus a low-viscosity asphalt is required. A low-viscosity asphalt, however, also produces a low stiffness during the warmer seasons; the result is that the stresses and strains in the pavement structure increase and produce a reduction in fatigue life and an increase in rutting. Increasing stiffness at higher temperatures to enhance resistance to fatigue damage also increases the stiffness at low temperature. It should be noted that reasonable stiffnesses at higher temperatures can be obtained by using the high-tensile-strength materials.

McLeod has suggested values of stiffness versus temperature that should eliminate low-temperature cracking (30). These stiffnesses are for loading times of 20 000 s but can be shifted to other loading times and temperatures by using techniques described in the literature (30).

Haas (29) has suggested that a maximum cracking index of 12.5 for primary roads will ensure adequate performance. This cracking-index value transforms to a transverse crack spacing of approximately 30 ft (9 m). Haas also indicated that results from the Ste. Anne road test showed that increasing the thickness of the asphalt-concrete layer from 4 in (102 mm) to 10 in (254 mm) produced a 50 percent reduction in the cracking frequency. As a result, a suggested criterion for low-temperature cracking of 30 ft/1000 ft^2 (98 m/1000 m^2) was included in Figure 3 and adopted for use in this analysis.

One other consideration in relation to low-temperature cracking is that the type of subgrade influences the effect of the low-temperature cracking on performance and maintenance. Studies by Haas have shown that if the subgrade has a very low swell potential the low-temperature cracking will have little effect on performance and subsequent maintenance. However, if the subgrade soil is a clay, the designer should carefully consider paving materials that will prevent formation of low-temperature cracking. Water infiltration through the low-temperature cracks will produce subsequent movements in the subgrade soil either through swelling or frost action and reduce the maintenance-free life of the pavement structure.

CONCLUSIONS

It has been demonstrated that, within the limits of the models selected, zero-maintenance pavements can be constructed; however, these pavements are very thick. In addition, the trade-offs of effects of properties on the various distresses must be carefully considered in order to provide pavements that will serve traffic for 20 years with no structural maintenance.

Combinations of material properties and necessary structural characteristics have been summarized. It should be noted that special emphasis must be given to provision of quality control and adequate thickness if conventional materials are used in the designs.

There is a need for additional materials research to develop, refine, and adapt new materials that can produce thinner surfacing layers for flexible pavements to provide zero-maintenance performance. The reductions in layer thickness can be used to offset the cost of producing these superior materials.

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REFERENCES

1. J.B. Rauhut, F.L. Roberts, and T.W. Kennedy. Models and Significant Material Properties for Predicting Distresses in Zero-Maintenance Pavements. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-78-84, Sept. 1978.
2. J.B. Rauhut, J.C. O'Quin, and W.R. Hudson. Sensitivity Analysis of FHWA Structural Model VESYS II. Federal Highway Administration, U.S. Department of Transportation, Rept. FHWA-RD-76-24, March 1976.
3. W.J. Kenis. Predicted Design Procedures—A Design Method for Flexible Pavements Using the VESYS Structural Subsystem. Proc., 4th International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, Aug. 1977.
4. M.Y. Shahin. Prediction of Low-Temperature and Thermal Fatigue Cracking of Bituminous Pavements. Univ. of Texas, Austin, Ph.D. dissertation, Aug. 1972.
5. M.Y. Shahin. Design System for Minimizing Asphalt Concrete Thermal Cracking. Proc., 4th International Conference on Structural Design of Asphalt Pavements. Univ. of Michigan, Ann Arbor, Aug. 1977.
6. J.B. Rauhut and P.R. Jordahl. Effects on Flexible Highways of Increased Legal Vehicle Weights Using VESYS IIM. Federal Highway Administration, U.S. Department of Transportation, Final Rept. FHWA-RD-77-134, Jan. 1978.
7. M.I. Darter and E.J. Barenberg. Zero-Maintenance Pavement Requirements and Capabilities of Conventional Pavement Systems. Federal Highway Administration, U.S. Department of Transportation, Interim Rept. FHWA-RD-76-105, April 1976.
8. R.L. Lytton, W.F. McFarland, and D.L. Schaefer. Flexible Pavement Design and Management: Systems Approach Implementation. NCHRP, Rept. 160, 1975.
9. H.J. Treybig, B.F. McCullough, P. Smith, and H. Von Quintus. Overlay Design and Reflection Cracking Analysis for Rigid Pavements: Volume 1--Development of New Design Criteria. Federal Highway Administration, U.S. Department of Transportation, Final Rept. FHWA-RD-77-66, Jan. 1978.
10. F.N. Finn, C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah. Development of Pavement Structural Subsystems. NCHRP, Project 1-10B, Final Rept., Feb. 1977.
11. A.S. Adelimila and T.W. Kennedy. Fatigue and Resilient Characteristics of Asphalt Mixtures by Repeated-Load Indirect Tensile Test. Center for Highway Research, Univ. of Texas, Austin, CFHR Res. Rept. 183-5, Aug. 1975.
12. C.L. Monismith, J.A. Epps, D.A. Kasianchuk, and D.B. McLean. Asphalt Mixture Behavior in Repeated Flexure. Office of Research Services, Univ. of California, Berkeley, Rept. TE-70-5, Dec. 1970.
13. B.F. Kallas and V.P. Puzinauskas. Flexural Fatigue Tests on Asphalt Paving Mixtures. ASTM Special Tech. Publ. 508, 1972, pp. 47-65.
14. P.S. Pell and K.E. Cooper. The Effect of Testing and Mix Variables on the Fatigue Performance of Bituminous Materials. Proc., AAPT, Vol. 44, 1975.
15. D. Navarro and T.W. Kennedy. Fatigue and Repeated-Load Elastic Characteristics in In-Service Asphalt-Treated Materials. Center for Highway Research, Univ. of Texas, Austin, CFHR Res. Rept. 183-2, Jan. 1975.
16. M.W. Witczak. Development of Regression Models for Asphalt Concrete Modulus for Use in MS-1 Study. Asphalt Institute, College Park, MD, Jan. 1978.
17. W.R. Hudson and T.W. Kennedy. An Indirect Tensile Test for Stabilized Materials. Center for Highway

Research, Univ. of Texas, Austin, CFHR Res. Rept. 98-1, Jan. 1968.
18. J.F. Shook and B.F. Kallas. Factors Influencing Dynamic Modulus of Asphalt Concrete. Proc., AAPT, Vol. 38, 1969.
19. W.H. Gotolski, S.K. Ciesielski, and L.N. Heagy. Progress Report on Changing Asphalt Properties of In-Service Pavements in Pennsylvania. Proc., AAPT, Vol. 33, Feb. 1964, pp. 285-319.
20. W.J. Kenis. Progress Report on Changes in Asphaltic Concrete in Service. HRB, Bull. 333, 1962, pp. 39-65.
21. W.J. Liddle, G.M. Jones, and D.E. Peterson. Use of Synthetic Rubber-in-Asphalt Pavement to Determine Mixture Behavior and Pavement Performance. Materials and Tests Division, Utah State Highway Department, Interim Rept. Utah Project HPR 1 (8), BPR Studies 10 and 14; State Studies 903 and 910, Dec. 1979.
22. W.E. Simpson, R.L. Griffin, and T.K. Miles. Correlation of the Microfilm Durability Test with Field Hardening Observed in the Zaca-Wigmore Experiment Project. Symposium on Paving Materials, ASTM Special Tech. Publ. 277, 1959, pp. 52-63.
23. J. Skog. Results of Cooperative Test Series on Asphalts from the Zaca-Wigmore Experimental Project. Symposium on Paving Materials, ASTM Special Tech. Publ. 277, 1959, pp. 46-51.
24. E. Zube and J. Skog. Final Report on the Zaca-Wigmore Asphalt Test Road. Materials and Research Department, Division of Highways, California Department of Transportation, Sacramento, 1959.
25. L.W. Corbett and R.E. Merz. Asphalt Binder Hardening in the Michigan Test Road After 18 Years of Service. TRB, Transportation Research Record 544, 1975, pp. 27-34.
26. R.J. Schmidt. Use of ASTM Tests to Predict Low-Temperature Stiffness of Asphalt Mixes. TRB, Transportation Research Record 544, 1975, pp. 46-55.
27. Local Climatological Data: Annual Summaries for 1977. National Oceanic and Atmospheric Administration, Rockville, MD, 1978.
28. F.L. Roberts, T.W. Kennedy, and G.E. Elkins. Material Properties to Minimize Distress in Zero-Maintenance Pavements. Federal Highway Administration, U.S. Department of Transportation, Draft Final Rept., Aug. 1979.
29. R.C.G. Haas. A Method for Designing Asphalt Pavements to Minimize Low-Temperature Shrinkage Cracking. Asphalt Institute, College Park, MD, Res. Rept. 73-1, Jan. 1973.
30. N.W. McLeod. Prepared Discussion on Ste. Anne Test Road. Proc., Canadian Technical Asphalt Association, 1969.

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Distress Behavior of Flexible Pavements That Contain Stabilized Base Courses

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The distress behavior of full-scale experimental pavements is analyzed and discussed. The pavements contained five different base-course materials, namely, bituminous concrete, aggregate cement, aggregate-lime-pozzolan, aggregate bituminous, and crushed stone. Three types of aggregate—limestone, slag, and gravel—were used in the aggregate-cement base. Distress behavior discussed includes rutting, surface roughness, and cracking. Distress behavior observed is related to the pavement response, which was analyzed by using the BISAR computer program. The critical responses analyzed are maximum tensile strain at the bottom of the base course and maximum compressive strain at the top of the subgrade. Various equations relating distress and response are established that permit prediction of the amount of rutting, roughness, and cracking, and allowable subgrade compressive strains to limit different distress modes within specified levels are also established. Field distress data are also related to the present-serviceability-index (PSI) values of each test pavement. From these relationships, various levels of each mode of distress manifestation are established for each level of PSI drop. Results obtained from this study may be useful in selecting allowable distress levels and allowable subgrade compressive strain for pavement design and can also be helpful in developing the relationship between distress and performance.

Distress is related to structural defects that result from a variety of traffic- and environment-related causes; it may affect pavement performance directly or indirectly. In general, distress takes one of three forms—fracture, distortion, or disintegration. The distress mechanisms that contribute most significantly to a reduction in pavement serviceability, according to Finn (1), are fatigue cracking, rutting and slope variance, and cracking caused by shrinkage or by changes in temperature and subgrade moisture.

To design a pavement structure that will be maintenance-free within a design period of normally 20 years requires a thorough understanding of distress behavior and an ability to predict the degree of various distress manifestations. To date, a number of mathematical distress models have been developed for flexible pavements. An extensive list of these models is given by Rauhut, Roberts, and Kennedy (2). Some models capable of predicting rutting, fatigue cracking, and low-temperature cracking are VESYS A (3), PDMAP (4), the Shell method (5), and WATMODE (6). Some of these models were developed from statistical analysis of field data and thus require modification for use under other loading, environmental, and material conditions. Others were developed with assumptions to simplify loading and structure conditions and require field data for calibration and validation.

This paper discusses the distress behavior of several types of pavements studied at the Pennsylvania Transportation Research Facility, and various equations that relate distress and response variables are developed. These equations permit prediction of the degree of different distress manifestations and also provide data useful for relating distress and performance.

EXPERIMENTAL PAVEMENTS

The Pennsylvania Transportation Research Facility is a 1.6-km (1-mile), one-lane, 3.7-m (12-ft) wide test road that was constructed in the summer of 1972. The original facility consisted of 17 sections; each section contained either different base-course materials with the same layer thicknesses or one type of base-course material with