have also been noted and ranked. To determine optimal values of each of these material properties, the effect of change in these properties on structural response must be evaluated and compared with the requirements for zero-maintenance pavements. It should be noted, however, that some important distresses cannot be evaluated by using available analytical models and must be investigated in special studies. Papers by Rauhut and others (9 and a paper elsewhere in this Record) include detailed discussions of analytical models that predict distress as a function of material properties, load, and environmental factors.

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Evaluation of Permanent Deformation in Asphalt Concrete Pavements

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Results of an evaluation of permanent deformation, or rutting, in asphalt concrete pavements subject to traffic load and environmental conditions are presented. Because previous studies have shown rutting to be a major mode of distress in various pavement systems, rutting criteria should be considered in any rational method of pavement design. For practical applications, it is often necessary to estimate the rutting expected in a pavement for a certain period or set of conditions. To accomplish this, a model of rutting estimation is necessary. An approach to rutting estimation applied successfully to subgrade soils in research at Ohio State University was used to study the variation of rutting parameters with asphalt content, temperature, and loading conditions for two mixtures that met Ohio specifications for surface-course mixtures. The data and results were obtained from uniaxial dynamic tests performed on laboratory-prepared samples and from previous research conducted at Ohio State University. The findings of the study, and comparisons with results of previous similar studies, indicate that a direct general relation exists between rutting parameters, dynamic moduli, and applied stress. The rutting parameter m was found to be almost constant within the range of study, which confirms previous results on subgrade soils that showed m to be an almost universal constant independent of soil type, stress, dynamic modulus, or saturated environmental conditions. A method for estimating permanent deformation in asphaltic concrete layers is proposed on the basis of the relation found in this study, in which the dynamic modulus was found to characterize such mix properties as density, asphalt content, gradation, air voids, and temperature with respect to rutting criteria. A general relation was also found between the rutting parameter A and the dimensionless ratio of dynamic modulus to applied stress. This relation satisfactorily describes the test results regardless of other influential factors such as type of material.

A recent survey among various state highway departments by the American Association of State Highway and Transportation Officials (AASHTO) has shown the importance of excessive permanent deformation, or rutting, as a common cause of failure in flexible pavements. Table 1 gives a summary of the most prevalent types of pavement distress reported by state highway

Table 1. Most prevalent types of pavement distress as reported by state agencies.

Type of Distress	States Reporting Distress as Most Prevalent on: Interstate Highways Number	Interstate Highways Percent	Primary Highways Number	Primary Highways Percent	Secondary Highways Number	Secondary Highways Percent	Farm-Market Highways Number	Farm-Market Highways Percent
Cracking								
Longitudinal	5	10.0	7	15.9	3	6.4	0	0.0
Alligator	1	2.0	2	4.5	5	10.6	7	25.0
Multiple	7	14.0	9	20.5	14	29.9	2	7.1
Transverse	7	14.0	7	15.9	4	8.5	2	7.1
Raveling	1	2.0	2	4.5	3	6.4	3	10.7
Rutting	14	28.0	13	29.5	6	12.8	5	17.8
Flushing	2	4.0	1	2.3	1	2.1	0	0.0
Roughness	1	2.0	1	2.3	5	10.6	0	0.0
Patching	10	20.0	1	2.3	3	6.4	5	17.9
Base failure	0	0.0	0	0.0	2	4.2	2	7.2
Corrugations	0	0.0	0	0.0	0	0.0	1	3.6
Shrinkage	2	4.0	1	2.3	1	2.1	1	3.6
Total	50	100.0	45	100.0	47	100.0	28	100.0

agencies. Under nonfailure conditions, permanent deformation reduces road serviceability and driving comfort. The hydroplaning and icing that result from accumulated water in rutting paths reduce highway safety. Therefore, any rational design scheme should include rutting as an important limiting criteria.

Several research programs at the Ohio State University have focused on establishing a comprehensive model for the estimation of permanent deformation. Majidzadeh and others have presented a mechanistic model that describes rutting phenomena in subgrade soils (1, 2) and in asphaltic concrete layers (3, 4). Research is also being conducted on unstabilized granular bases. This paper deals with the problem in asphaltic concrete.

Numerous design methods with limiting subgrade strain criteria have been proposed to control rutting in the subgrade. Discussions of various approaches to rutting prediction can be found elsewhere (1-6) and need not be repeated here. Pavements designed on the basis of limiting subgrade strain criteria have sufficient thickness to protect the subgrade soil from shear failure, but the method does not ensure that permanent deformation in the upper pavement layers will not occur. The AASHO Road Test has shown that the asphaltic concrete layer contributes anywhere from 5 to 50 percent of total pavement rutting, depending on pavement composition and season of investigation.

To investigate rutting in asphaltic concrete layers, three basic steps should be considered:

1. Determine the traffic and environmental conditions that may occur in the field.
2. Find a reliable technique of stress-strain analysis to adequately describe the distribution of stress in the pavement system.
3. Investigate material characteristics under such traffic loading and the probable intensity, function, and frequency of applied stress, and consider the role of environmental changes, which in this case are primarily those of temperature.

Step 3 is the prime concern in this study of nonfailure pavement conditions.

Laboratory simulation of field conditions is difficult but can be achieved by conducting dynamic tests under different stress levels and temperature changes. This approach, which has proved successful in several studies at the Ohio State University (1-6), was used in this study. A half-sinusoidal compressive dynamic load was applied by using 0.125-s pulses separated by 0.375-s rest periods. A hydroelectronic Material Testing System (MTS) was used to conduct the test program. Only uniaxial tests were used because previous studies on subgrade soils (6-8) and on asphaltic concrete (4) have shown that small magnitudes of confining pressure [σ_3 up to 34 kPa (5 lbf/in^2)] have no significant effect on the resilient behavior of the sample, which matches closely the behavior under three-dimensional dynamic loading with σ_3 vertical and σ_3 horizontal applied stresses.

On the basis of a linear elastic stress analysis using the Chevron program (3) and reported actual stress measurements (9), the range of applied deviatoric stresses was selected at 206.7, 275.6, 344.5, and 402.4 kPa (30, 40, 50, and 60 lbf/in^2) to simulate field conditions. A static load, approximately 4 percent of the dynamic load amplitude, was first applied to ensure complete contact between loading cell and sample to avoid impact action during dynamic testing.

Testing also has a significant effect on permanent deformation in asphalt concrete. For any given condition, the rate of deformation increases with temperature. At temperatures below a certain limit, however, the accumulation of permanent deformation is negligible. It has been reported (10) that the average temperature at which rutting occurs is 22°C (72°F). Accordingly, temperatures of 27°C, 38°C, and 49°C (80°F, 100°F, and 120°F) at each stress level and asphalt content were selected for this study to approximate temperature conditions in the field.

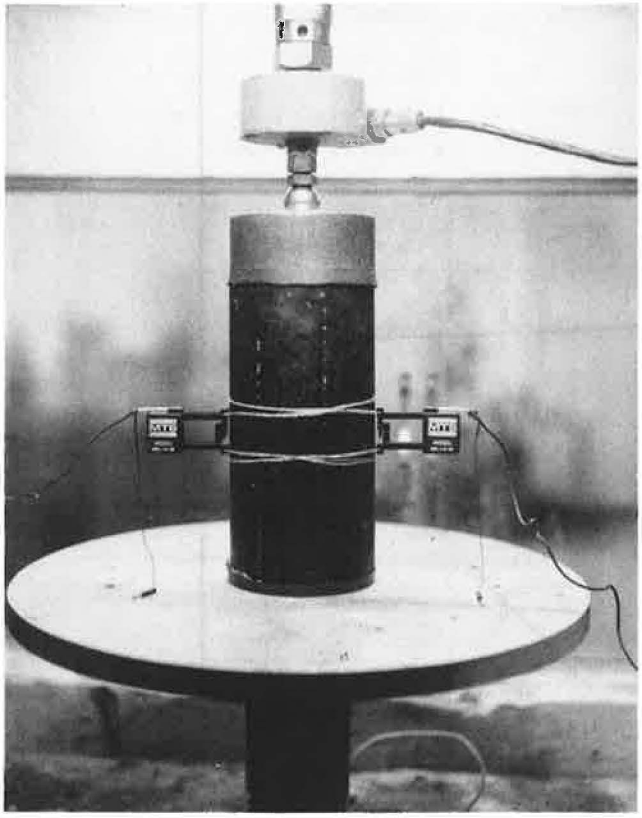
Each sample was subjected to at least 2000 load repetitions at certain stress levels and temperatures. Permanent deformations were measured by using Daytronic linear variable differential transformers mounted on the loading piston. Elastic deformations were recorded by two MTS extensometers (see Figure 1).

MIX CHARACTERISTICS AND SAMPLE PREPARATION

Two similar limestone aggregates (referred to as series 1 and 2) were used to prepare mixes that met gradation and other specification requirements of the Ohio Department of Transportation for surface-course mixtures, as given below (1 mm = 0.039 in):

U.S. Sieve Size (mm)	Percentage Passing by Weight: Specification	Mix Composition
12.7	100	100
9.5	90-100	94
4.75	45-75	55
2.36	26-58	40
0.6	8-32	15
0.3	3-22	8

Figure 1. Sample setup for testing.



U.S. Sieve Size (mm)	Percentage Passing by Weight Specification	Mix Composition
0.15	1-4	5
0.075	0-8	3

The bitumen used in preparing the mixes was 85/100 penetration AC-20. The specification limits for asphalt content were 4.5-0.5 percent. For the series 1 aggregate, only the optimum asphalt content of 6.5 percent was used; for the series 2 aggregate, three asphalt contents were used: 5.5, 6.5, and 7.5 percent.

Samples 10 cm (4 in) in diameter and 20 cm (8 in) in height were compacted statically by using a compaction pressure of 76.8 MPa (11 000 lbf/in^2), and representative samples were checked for uniformity. The general trend was for the sample to be denser at the bottom and at the middle third to have a density equal to the average overall sample density. The greatest difference in density was 80.1 kg/m^3 (5 lb/ft^3) between sample top and middle third. Sample density varied from 2336.7 kg/m^3 (145.41 lb/ft^3) at 5.5 percent asphalt content to a maximum of 2369.2 kg/m^3 (147.89 lb/ft^3) at 7.5 percent asphalt content. Air voids ranged from 4.32 to 5.11 percent at 5.5 percent asphalt content, from 2.71 to 3.27 percent at 6.5 percent asphalt content, and from 0.58 to 0.76 percent at 7.5 percent asphalt content.

ANALYSIS OF RESULTS

Dynamic Modulus |E*|

In recent research, the dynamic modulus is considered a major characteristic of highway pavement materials that reflects pavement performance under traffic load-

Figure 2. E* versus temperature at various stress levels.

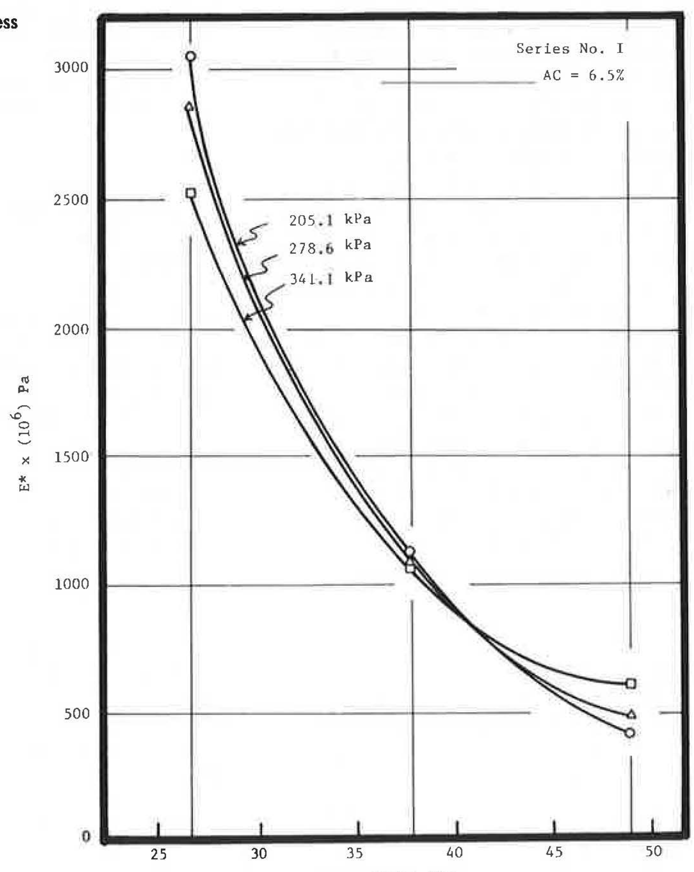
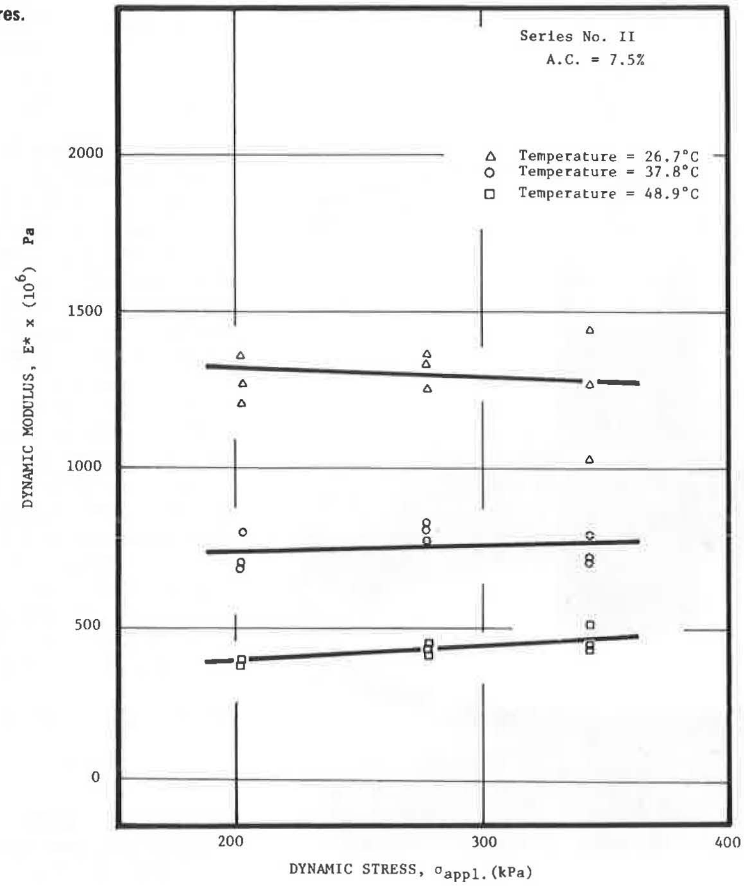


Figure 3. E* versus σ_{apl} at various temperatures.



ing. It is defined as the ratio between applied dynamic stress and corresponding elastic strain. A previous study (5) has shown that E* tends to decrease with the number of stress cycles N if the applied stress is higher than the endurance limit (defined as that stress level under which the sample can withstand an infinite number of cycles without failure). E* tends to increase with N when applied stress σ_{apl} is less than the endurance limit. However, in both cases E* will be almost constant after the first 1500 cycles over a wide range of stresses and temperatures (except in cases of failure, which are not the concern of this study). Therefore, E* was calculated for each sample after 2000 cycles.

Variation in temperature was found to be the factor that had the greatest effect on dynamic modulus. Figure 2 shows a typical variation of E* with temperature. An increase from 27°C to 49°C (80°F to 120°F) results in a reduction in moduli values by a factor of four to five. Applied deviatoric stress, in the ranges used, had insignificant effects on the dynamic modulus (see Figure 3). Asphalt content had a noticeable effect at lower temperatures but little effect at higher temperatures (see Figure 4).

Permanent Deformation

Figure 5 shows the typical relation between permanent strain accumulation ϵ_p/N and N on a log-log graph. This relation could be satisfactorily represented by a straight line for a nonfailure, steady-state condition, which is the normal field condition of interest in this paper.

For a failure condition, the rate of ϵ_p/N increases with N, leading to complete failure. This is the case for sample 11 in Figure 5. The concept of threshold stress, proposed by Francken (11), is that stress level under which the steady-state condition is satisfied. It depends on mixture characteristics, temperature, and stress conditions.

Such a nonfailure condition could then be represented by the following formula:

$$\epsilon_p/N = AN^{-m} \qquad (1)$$

where A and m are parameters that are dependent on mix composition, applied stress intensity and frequency, and temperature.

There were 110 samples tested for the study that did not experience progressive failure conditions and were found to satisfy the proposed equation. Ten samples followed the formula until a certain stage, after which progressive failure was observed. Similar equations have been proved true by El-Mojarrush (3) and Francken (11).

The evaluation of m and A should completely describe rutting phenomena in asphalt concrete so that we can establish a model for estimating permanent deformation in such material.

Parameter m

Parameter m was calculated for each sample as the absolute value of the slope of the straight-line relation of log (ϵ_p/N) to log N. The values of m showed no significant variation over the testing domain of various stress levels, temperatures, asphalt contents, and the two types of aggregates used. Figure 6 shows the values of m for all testing to range from 0.73 to 0.87.

Test data were divided into groups according to asphalt content, stress level, testing temperature, and/or series of aggregate. One-way analysis of variance with multiple comparisons was conducted on the means of the m values of these groups (3). Three comparative techniques were used: Tukey $\bar{B}$, Tukey HSD, and Scheffé (12). For a 95 percent confidence level, no significant difference was found among these groups. The general mean value of m is 0.78 with a standard deviation of 0.04. This trend is in excellent agreement with previous studies on subgrade soils (1, 2, 6, 7) in which m was found to be constant regardless of material type, mixture composition, stress, dynamic modulus, or saturated environmental conditions.

Parameter A

Parameter A is the permanent strain after the first cycle $[(\epsilon_p/N)_{N=1}]$. It was measured as the interception

Figure 4. E* versus asphalt content at various temperatures.

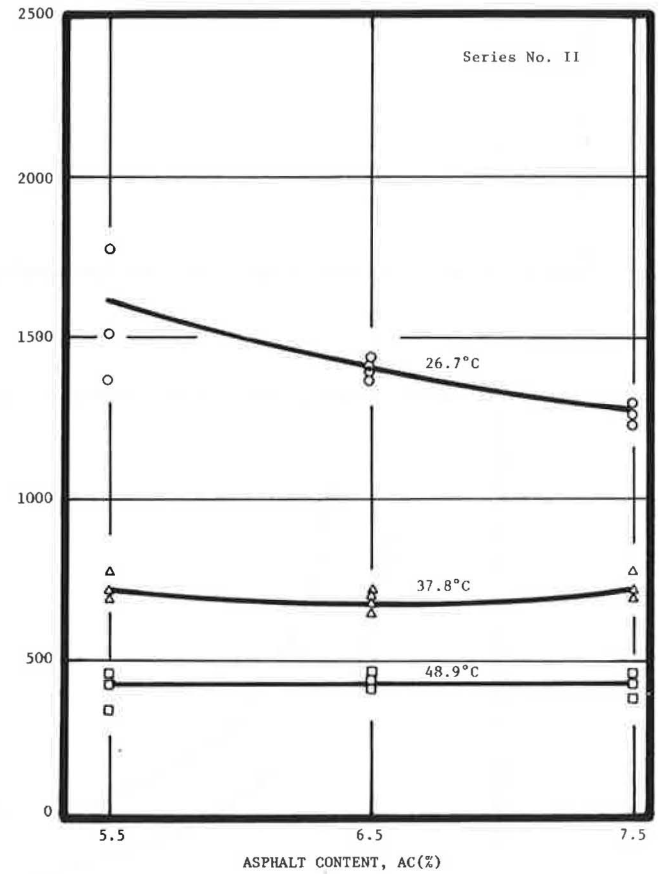


Figure 5. Typical relation between ϵ_p/N and N (continuous test).

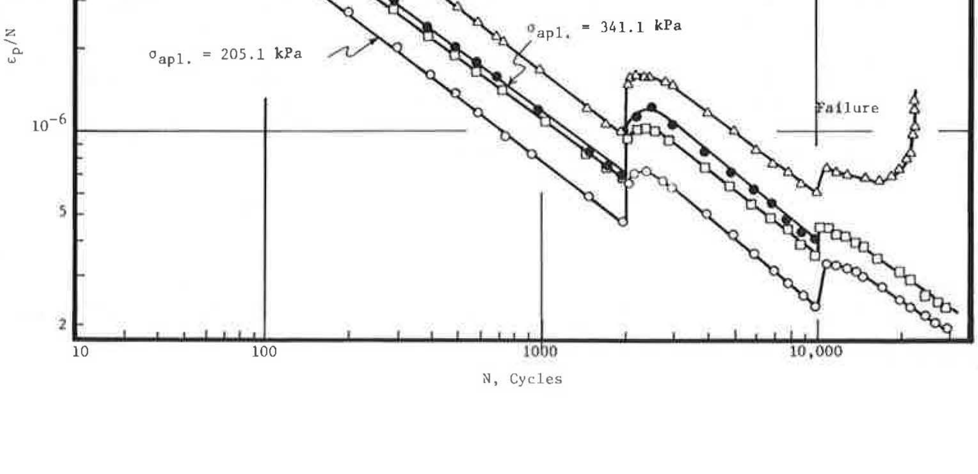


Figure 6. E* versus m for types 1 and 2 aggregate.

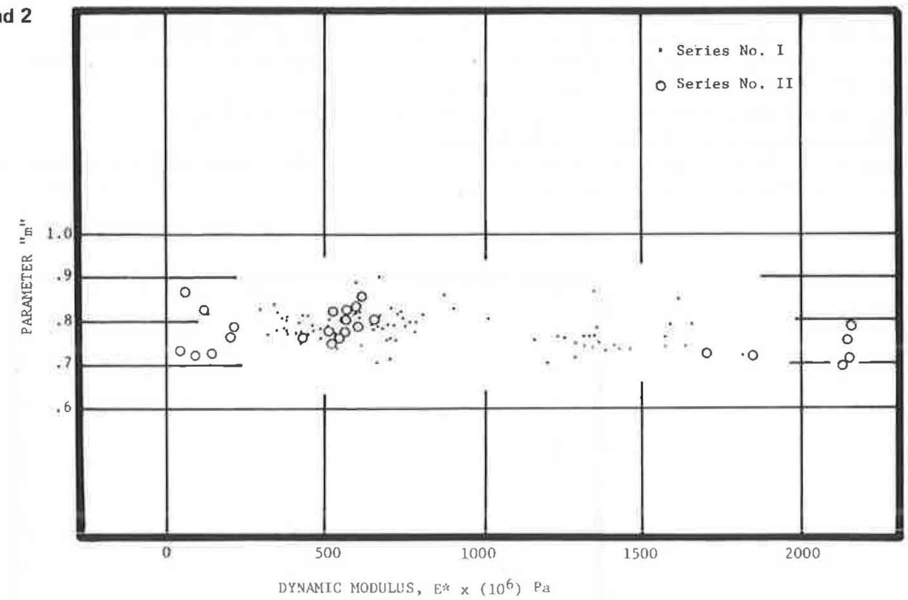
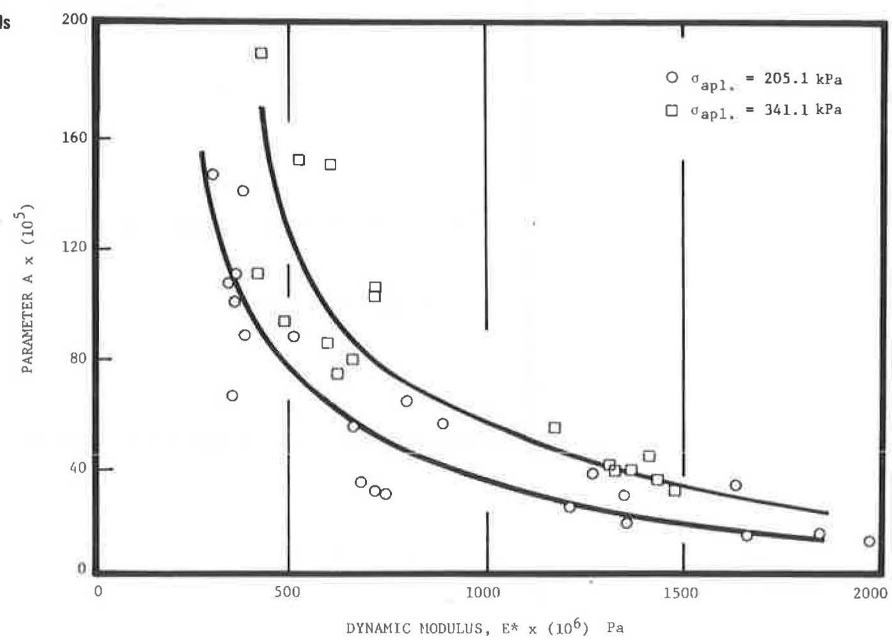


Figure 7. E* versus A for various stress levels (series 2).



of the straight line of log (ϵ_p/N) versus log N with the axis of N = 1. In general, A increases with applied dynamic stress and testing temperature.

Recognizing the fact that E* is a basic material mechanical characteristic, we plotted the values of A versus the corresponding values of E* for each stress level. Figure 7 shows typical curves for one case. From these curves, it can be deduced that a power relation may exist between A and E* (see Figure 8). The relation could be represented by a straight line with an adequate correlation coefficient. Mathematically, this relation is

$$A = K(|E^*|)^{-S} \tag{2}$$

where K and S are constants that are dependent on applied dynamic stress. A similar relation has been proposed by Majidzadeh and others (1) for fine-grained subgrade soils.

Furthermore, Figures 9-11 show the relation between log A and log (E^*/σ_{apl}) for data obtained from samples using series 1 and 2 aggregate, respectively, for all the various conditions of stress levels, asphalt contents, and temperatures. It can be seen that a straight line can represent the relation with a high correlation coefficient. Figure 12 shows the same relation—and the same trend in the relation—for all samples tested in this study. Accordingly, the parameter is correlated with the dimensionless ratio E^*/σ_{apl} by the following formula:

$$A = J(E^*\sigma_{apl})^{-S} \tag{3}$$

where J and S are constants believed to be universal for asphalt concrete. However, studies of other mixtures are required to investigate this point.

It is clear that, in experimentally establishing the

Figure 8. General relation between E* and A for series 2 (all asphalt contents).

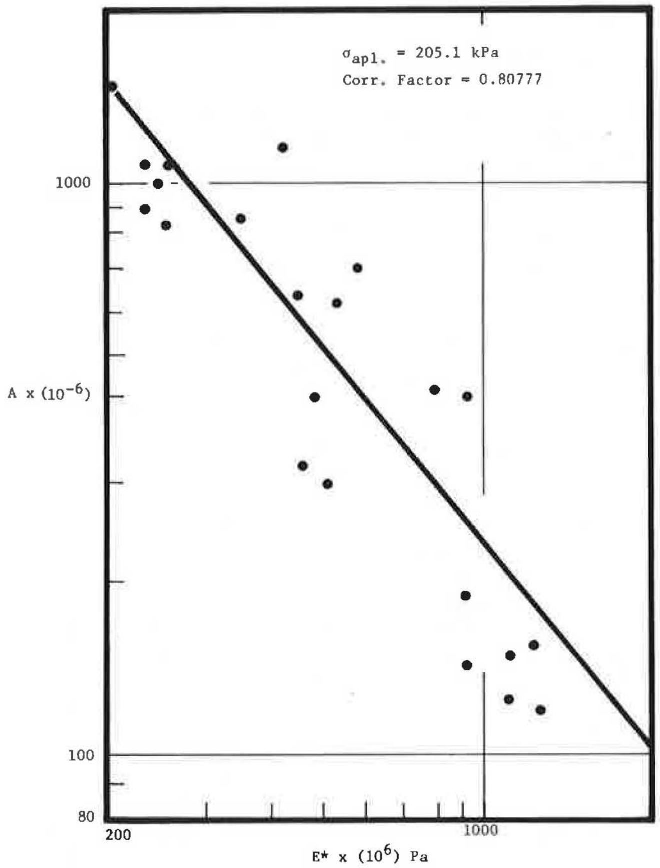


Figure 9. E^*/σ_{apl} versus A for series 1 (initial tests).

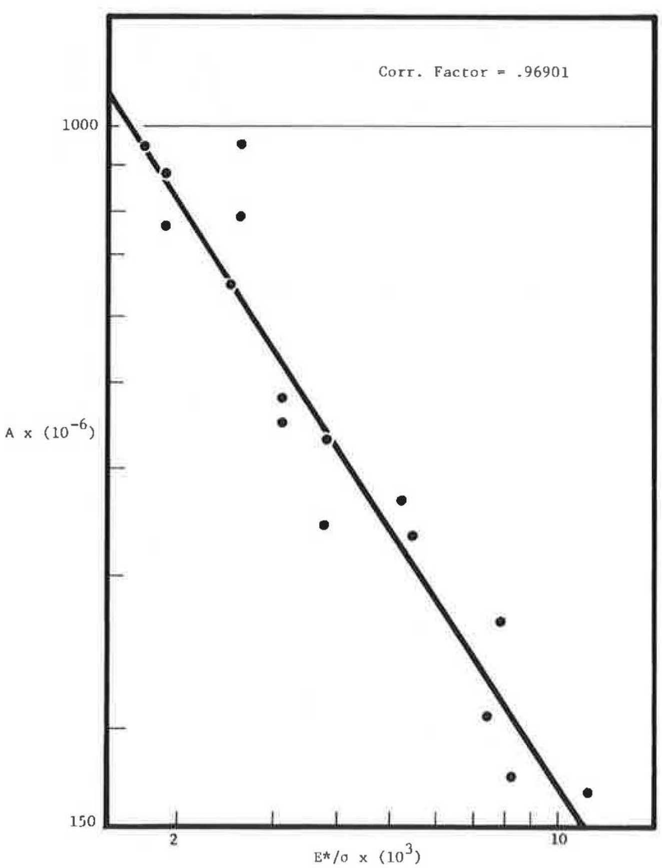


Figure 10. E^*/σ_{apl} versus A for series 1 (continuous tests).

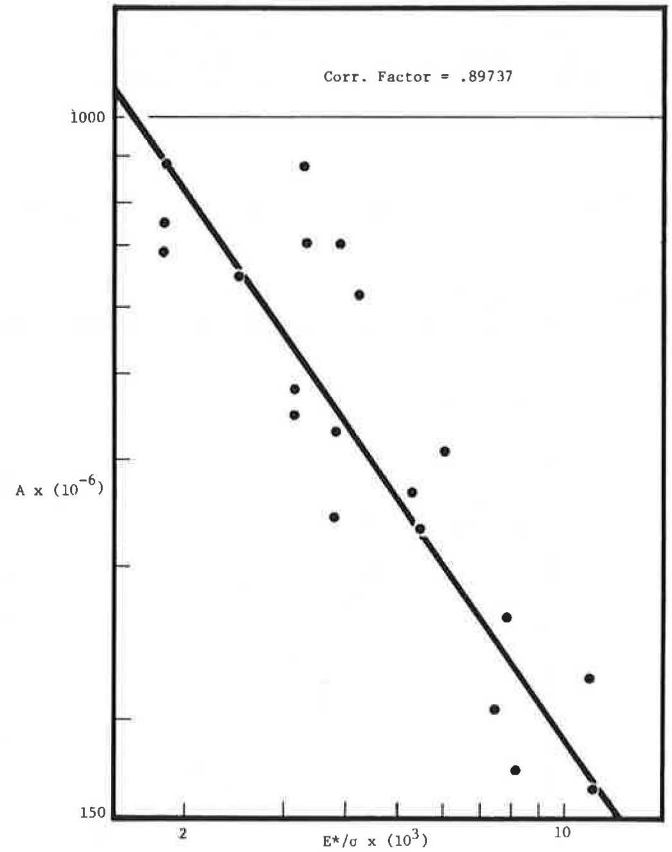
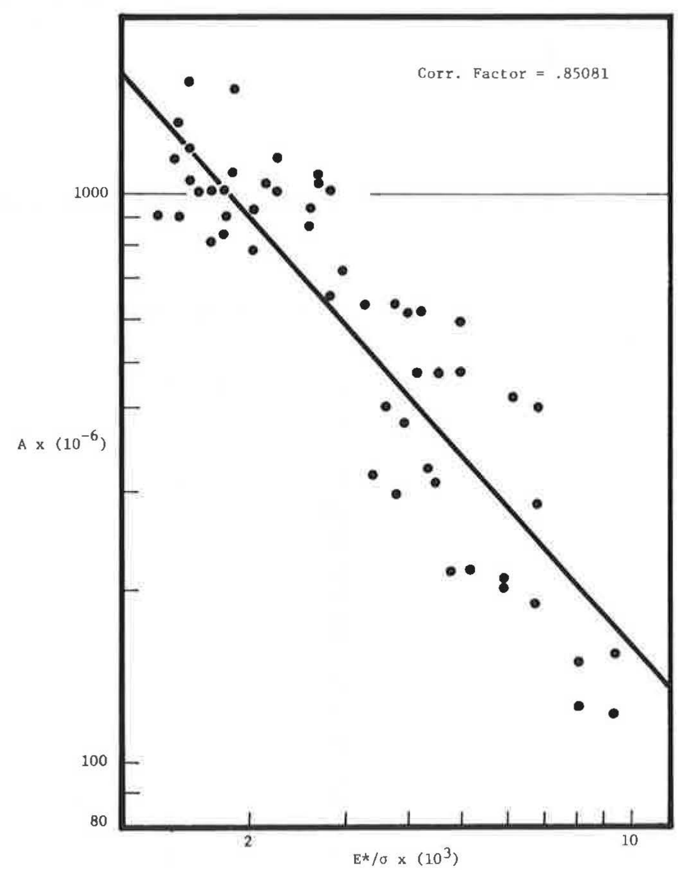


Figure 11. E^*/σ_{apl} versus A for series 2 (all asphalt contents).

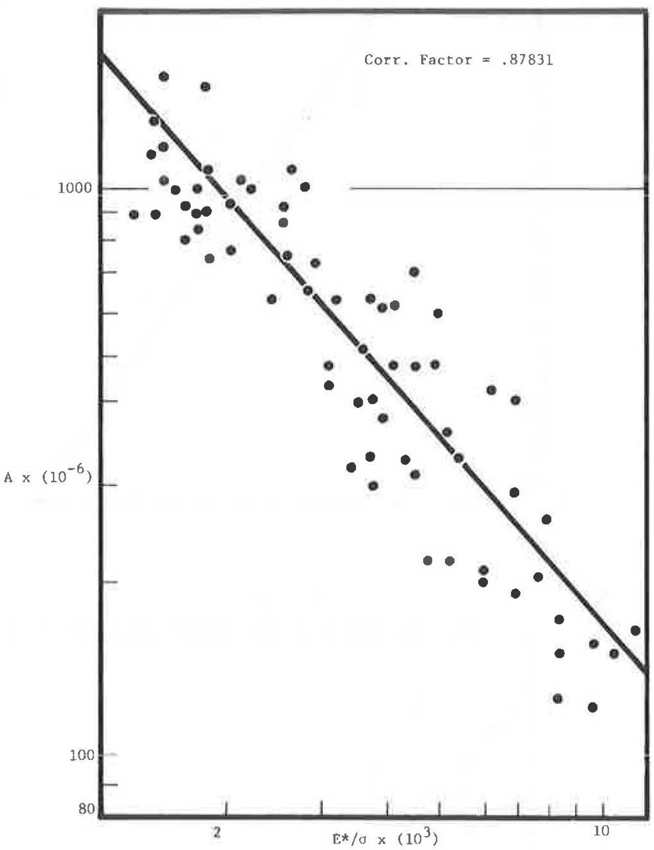


constants J and S for any mix, A could be determined in correspondence with measured values of E* and applied dynamic stress. This means that Equations 1 and 3 describe the phenomena of permanent deformation in asphaltic concrete for any specific condition as follows:

$$\epsilon_p/N = J(E^*/\sigma_{apl})^S N^{-m} \tag{4}$$

Equation 4 has contributed to a substantial reduction of the experimental work that should be performed to evaluate rutting in asphaltic concrete under different conditions.

Figure 12. E^*/σ_{apl} versus A for all samples tested.



Temperature Variation

As mentioned above, Equation 4 describes rutting in an individual environmental and stress condition. But what about the prediction of rutting under a series of different conditions? The main influencing factor in this case is variation in temperature.

The 30 samples prepared by using series 1 aggregate were subjected to continuous testing under a series of temperature changes. Each sample was tested under constant dynamic and static stress levels while the temperature was changed, after a specific number of loading cycles, from 27°C to 38°C to 49°C (80°F to 100°F to 120°F) or from 38°C to 49°C (100°F to 120°F). Figures 13 and 14 show typical results of these tests.

Some samples experienced failure when tested continuously at a high stress level and 49°C temperature. Those samples were not included in the analysis (see sample 7 in Figure 13). In the nonfailure cases, when the temperature was increased at a certain point the samples showed a sudden increase in rate of deformation and followed a different dynamic creep curve (Figure 13). This is also shown by the vertical jump in the rate of deformation of the curves for ϵ_p/N versus N at the points of temperature changes (see Figure 14). After the first few hundred cycles at the new higher temperature, the creep curve becomes more steady and follows Equation 1 with a higher value of A. Apparently, these few hundred cycles are necessary to rearrange the sample structure in its strongest form to compensate for the new temperature level. It also goes through a diverse stage in which E* decreases in value.

Parameters m and A were analyzed for these continuity tests by following the same pattern used for initial testing. First, m in Equation 1 falls in the same range as that discussed at the beginning of this paper. A was calculated by using the accumulative values of permanent deformation over the whole testing sequence before each temperature state. Equation 3 was also found to hold for these conditions with a correlation coefficient of 0.897.

Figure 13. Typical dynamic creep curves.

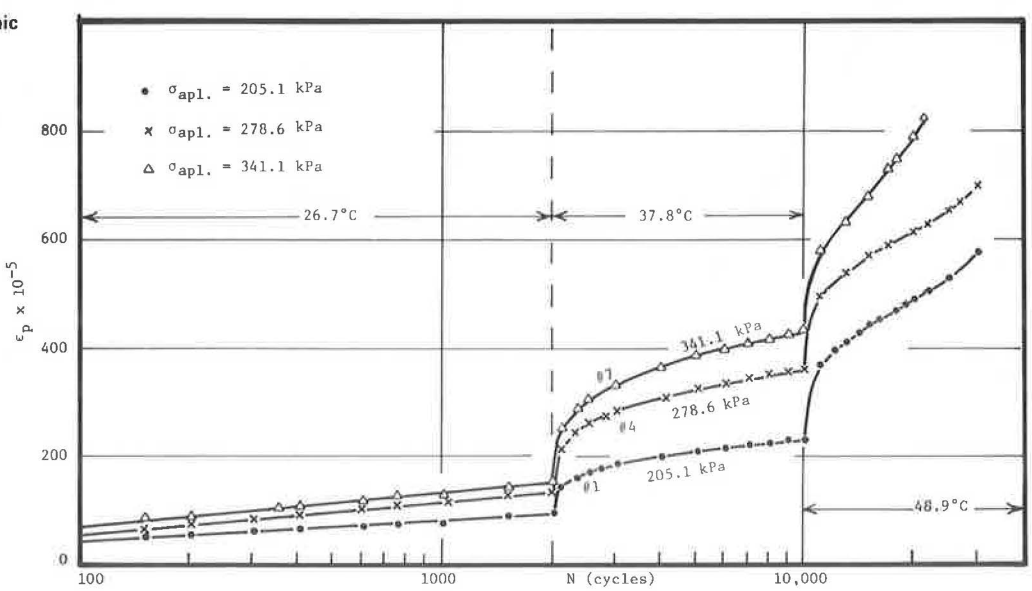
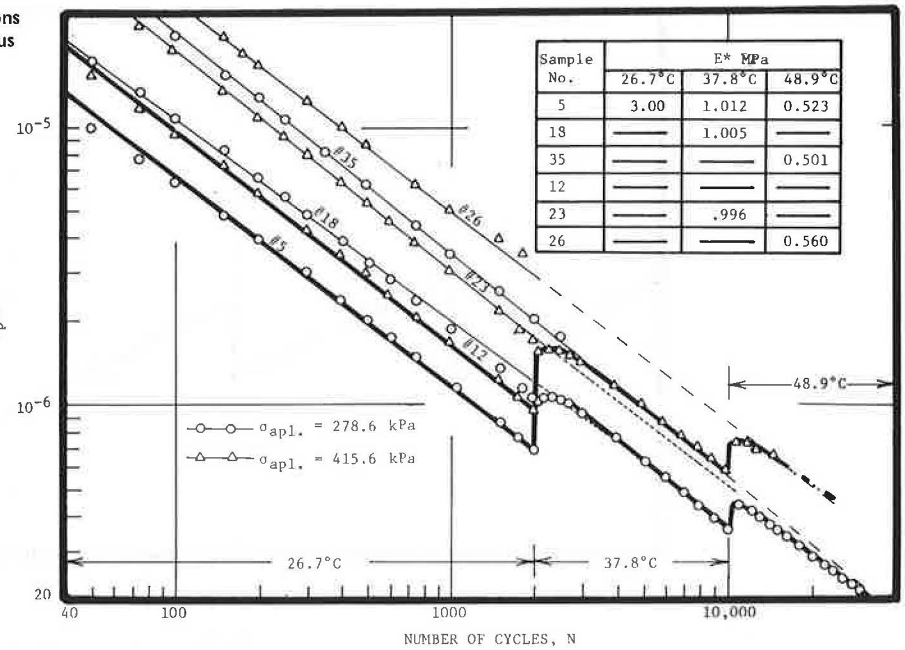


Figure 14. Effect of variations in temperature on ϵ_p/N versus N.



Identical samples, which had been tested initially at 38°C or 49°C (100°F or 120°F), were introduced for comparison. The idea was to compare the dynamic creep curves of the samples tested, either initially or in a continuity test, under the same stress level and temperature and, if possible, the same E*. To perform such a comparison, the results of such samples were plotted on log (ϵ_p/N) versus log N graphs (Figure 14). The dynamic modulus was considered a basic mechanical property according to which samples were judged and determined to be identical.

In Figure 14, sample 5 is compared with sample 18 at 38°C and with sample 35 at 49°C. At 38°C, both lines that represent the data of samples 5 and 18 are close. At 49°C, sample 5 has a higher dynamic modulus than sample 35, and sample 5 thus shows a higher resistance to rutting. That is expected even if both samples have been tested for rutting under the same conditions. The same analysis can be applied to samples 12, 23, and 26 in Figure 14.

In other words, Figure 14 shows that, if a sample that was previously tested at a lower temperature is retested at a higher temperature, the resulting dynamic creep curve could be represented by that of an identical sample initially tested under the same condition, provided the stress level is kept constant in all cases. Although this conclusion was not completely satisfied in all cases, the relation was generally acceptable. The observations are encouraging for further investigation into this approach.

MODEL FOR RUTTING PREDICTION

The findings of this study can be put to use in the following procedures for rutting evaluation:

1. Divide the time period over which rutting is to be evaluated into seasons. Exclude those seasons in which the temperature is lower than 22°C (72°F) (10). Find the traffic and the average corresponding temperature for each season. Investigate variation in temperature along the asphalt depth.
2. Conduct an analysis of stress distribution in the pavement under the equivalent traffic load by using an appropriate technique (several techniques are proposed by the literature, i. e., linear or nonlinear elastic, elastoplastic, and viscoelastic).
3. Divide the asphaltic concrete layer(s) within the pavement system into imaginary sublayers. The number of these sublayers depends on total thickness and stress distribution.
4. Perform dynamic tests on representative samples. Apply dynamic stress levels in the ranges obtained in step 2, and use testing temperatures found in step 1. Measure elastic and plastic deformations. The measured values of E* should be fed back into step 1 to check the assumed values.
5. From step 4, find the constants J and S of Equation 4 to establish A versus E* and σ_{apl}.
6. Calculate the permanent strain for each sublayer (ϵ_{pr}) as in the following example: Assume four consecutive environmental changes to be considered. E_i*, A_i, N_i, and M_i are the corresponding variables at the i th condition (see Figure 15). At the end of the first stage (condition a) and according to Equation 1, the value of log (ϵ_p/N) can be written as

$$\log(\epsilon_p/N)_1 = \log A_1 - m_1 \log N_1 \quad (5)$$

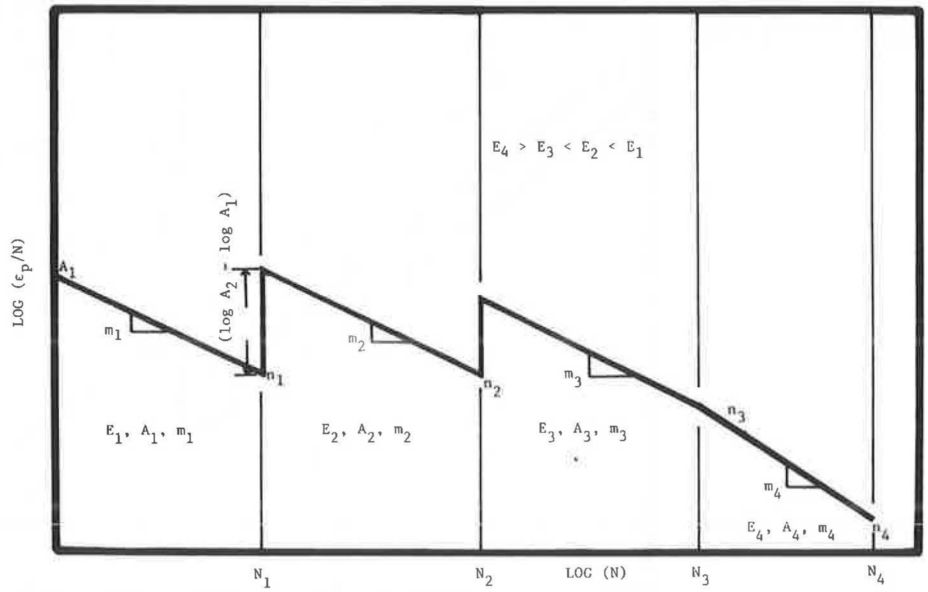
During the second stage (condition b), the pavement undergoes an increase in temperature. Assume incremental change in environmental conditions, i.e., a sudden change in the pavement characteristics caused by this temperature change at (n_1 + 1) cycles. $E_2^* < E_1^*$; consequently $A_2 > A_1$, and permanent deformation will increase at a higher rate. Assume that log (ϵ_p/N) will correspondingly increase suddenly at (N_1 + 1) cycles by the value (log A_2 - log A_1) (Figure 15). Then, at the end of this stage log (ϵ_p/N) is

$$\log(\epsilon_p/N)_2 = \log(\epsilon_p/N)_1 + \log A_2 - \log A_1 - m_2 (\log N_2 - \log N_1) \quad (6)$$

In the third stage (condition c), the temperature gets higher; $E_3^* < E_2^*$ and $A_3 > A_2$. Under the same assumption as that given for condition b above, ϵ_p/N at the end of this stage is

$$\log(\epsilon_p/N)_3 = \log(\epsilon_p/N)_2 + \log A_3 - \log A_2 - m_3 (\log N_3 - \log N_2) \quad (7)$$

Figure 15. Prediction of accumulation of permanent deformation.



Finally, the improving stage occurs where the temperature is lower than that of condition c. The pavement passes through a hardening phase where $E_4^* > E_3^*$. Since permanent deformation is irrecoverable, no jump is expected in log (ϵ_p/N) at N_3 cycles. Hence, the expression for log (ϵ_p/N) at the end of this stage is

$$\log(\epsilon_p/N)_4 = \log(\epsilon_p/N)_3 - m_4(\log N_4 - \log N_3) \quad (8)$$

Substituting Equations 5, 6, and 7 into Equation 8, we get

$$\log(\epsilon_p/N_4) = \log[A_3 N_1^{-(m_1-m_2)} N_2^{-(m_2-m_3)} N_3^{-(m_3-m_4)} N_4^{-m_4}]$$

or

$$(\epsilon_p/N)_4 = A_3 N_4^{-m_4} \prod_{i=1}^{3} N_i^{-(m_i-m_i+1)} \quad (9)$$

If m is constant, Equation 9 takes the following form:

$$(\epsilon_p/N)_4 = A_3 N_4^{-m} \quad (10)$$

In general, if we have j environmental changes and if A_{max} is the maximum value of A for the weakest condition of the pavement during these j changes, at the end of the j th stage, ϵ_p/N is expressed as

$$(\epsilon_p/N)_j = A_{max} N_j^{-m_j} \prod_{i=1}^{j-1} N_i^{(-m_i+m_i+1)} \quad (11)$$

For $m_1 = m_2 = \ldots = m_i = m$, Equation 11 can be written as

$$(\epsilon_p/N)_j = A_{max} N_j^{-m} \quad (12)$$

Obtaining ϵ_p for each sublayer for different seasons by using step 5, the total contribution of the asphalt concrete layer(s) to total pavement rutting is

$$\gamma_p \text{ total} = \sum_{i=1}^{n} \epsilon_{pi} h_i \quad (13)$$

where

n = number of sublayers,
ϵ_{pi} = permanent strain of the ith sublayer, and
h_i = thickness of the ith sublayer.

CONCLUSIONS

Based on the results on this study, a complete methodology for rutting evaluation has been proposed. The following conclusions can be drawn:

1. Equation 1 represents the dynamic creep curves for asphalt concrete.
2. Parameter m in Equation 1 is constant in the range of study.
3. The dynamic modulus characterizes the material properties; i.e., it reflects the effects of asphalt content, density, mix gradation, air voids, and temperature as far as permanent deformation criteria are concerned.
4. A general relation was found between A and the dimensionless ratio E^*/σ_{apl} (Equation 4). This relation was found to describe test results regardless of other influencing factors, including type of material. This may lead to the projection of the constants J and S in Equation 4 to be universal for asphalt concrete mixes.

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Subjective and Mechanical Estimations of Pavement Serviceability for Rural-Urban Road Networks

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To manage paved road networks efficiently, highway agencies require comprehensive, periodic inventory or pavement evaluation data. These data should be reliable, quickly and easily acquired, and manageable. The results of a study that involved subjective and mechanical measurements of both the roughness and structural adequacy of a combined rural-urban road network are presented. A new high-speed, computerized unit was used to make the mechanical measurements of roughness, and panel ratings were used for the subjective measurements. Structural adequacy was both rated by a panel and mechanically measured with a Dynaflect. Very good correlations were obtained between panel ratings and measured roughness for both urban and rural sections. The relations obtained and the resource and time requirements involved demonstrate that, by using such methods, highway agencies can quickly and efficiently inventory the serviceability of paved road networks. Ratings of structural adequacy did not correlate with measured deflection. The results suggest that, whereas surface distress or condition can be measured by panel ratings, structural adequacy can only be measured by mechanical means.

All highway agencies conduct inventory measurements and needs studies of some sort on their road networks based on various measures of functional, structural, and serviceability adequacy. Candidate projects are identified from these studies, and priorities are determined for investments in both new construction and rehabilitation to the limit of the available budget. Figure 1 shows the major elements of such a system of inventory needs priority within the framework of an overall system of road management.

Periodic inventory or evaluation measurements provide the basis for the identification of needs and all subsequent management activities. In other words, this is the basic management information.

Much of the inventory information currently collected by highway agencies is a combination of subjective and objective measurements. Ideally, such information would be subject to the following criteria:

1. It would be capable of being collected quickly, easily, and efficiently; be as objectively based as possible; be reliable and repeatable; and relate directly to the structural, functional, and serviceability indicators;
2. It would cover or represent the entire network and include sufficient frequency on the more deteriorated sections so that needs could be identified soon enough in advance to allow for proper programming; and
3. The information would itself be manageable.

In the light of these considerations, the Waterloo region of Ontario initiated a pavement evaluation study in 1978 as a part of their 1979 needs study update (1, 2). This regional municipality is about half urban and half rural and has about 1935 km of arterial roads and streets. A key requirement for the needs study update was the correlation of certain subjective pavement ratings with actual physical field measurements. The purpose of this paper is to describe the following specific results of the study:

1. The use of a new high-speed unit for obtaining roughness and other road data on an automated, mass-inventory basis;
2. The correlation of measured roughness with