ble. Transfer the mixture to a piece of aluminum foil and allow to cool at room temperature for 2 hr.

Test Procedure

1. Boiling mixture in water: Fill a 1000-mL beaker one-half full (500 cc) with distilled water and heat to boiling. Add the mixture to the boiling water. Addition of the mixture will temporarily cool the water below the boiling point. Apply heat at a rate such that the water will reboil in not less than 2 or more than 3 min after addition of the mixture. Maintain the water at a medium boil for 10 min, stirring with a glass rod three times during boiling, then remove the beaker from the heat. During and after boiling dip a paper towel into the beaker to skim any stripping asphalt from the surface of the water. Cool to room temperature, drain the water from the beaker, and empty the wet mix onto a paper towel and allow to dry.

2. Visual observation: Visually estimate the percentage of cement retained after boiling by comparing the specimen with a standard rating scale (Figure 1). A photograph should not be used. The mixture should also be examined on the following day after it has been allowed to dry because stripping of the fines is not as apparent when the mixture is still wet.

[Note: The standard rating scale (Figure 1) consists of samples that represent various degrees of stripping selected to provide examples at 10 percent intervals ranging from 0 percent to 100 percent retained asphalt cement.]

Report

The percentage of asphalt retained after boiling should be based on a comparison with the standard scale, not a photograph. Select the specimen nearest in appearance to the test specimen and report that as the test result.

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Analysis of Asphalt Concrete Test Road Sections in the Province of Quebec, Canada

JOSEPH HODE KEYSER and BYRON E. RUTH

ABSTRACT

Data were collected from test road sections in the Province of Quebec, Canada, for the purpose of evaluating the effects of materials and in situ conditions on the performance of asphalt concrete pavements. These pavements were tested in 1980 to determine rutting, ride, and deflection characteristics. In situ conditions were determined by sampling and test measurements. Asphalt concrete cores were obtained for indirect tensile strength tests and for recovery of asphalt for conventional consistency tests (penetration, viscosity, and softening point). These data were compiled along with information contained in the original construction records and pavement crack surveys. Statistical analyses were conducted and various relationships were developed that relate to factors that influence asphalt binder properties, tensile strength, and transverse cracking. The most significant findings include (a) a verification of greater age hardening when in-service asphalt concrete pavements have air voids in excess of 4.0 percent, (b) a tentative test method that incorporates the work of Goode and Lufsey to evaluate the age hardening of binders, and (c) mathematical relationships developed from statistical analyses by using recovered asphalt penetration and traffic level for the prediction of transverse cracking. Results of other analyses are presented that define those variables that have an effect on consistency parameters, mix tensile strength, rutting, and ride quality. Dynaflect deflection basins were analyzed by using an elastic layer computer program, which resulted in the development of relationships between subgrade moduli and the fifth-sensor deflection.

Data were collected from 3-km-long test road sections for evaluation of the effects materials and in situ conditions have on pavement performance. Analyses were conducted to develop relationships (a) between asphalt consistency measurements, (b) for in situ differences in asphalt and mix properties, and (c) for differences in pavement performance. Twenty-three test sections were used as the data base. These sections were located throughout the Province of Quebec, including areas in Montreal, Sherbrooke, Trois Rivieres, Rimouski, and Lake St. Jean.

The data base contained 44 variables that were initially evaluated by using a variety of statistical methods. The data included in situ measurements of rutting, cracking, pavement response (deflections), and ride (Mays meter). In 1980 core samples were obtained from the test sections to determine the properties of the asphalt mixtures and recovered bitumens. Penetration and viscosity data for the original bitumens were also included in the data base. Parameters defining environmental conditions, drainage, soil type and moisture content, layer thicknesses, and properties of foundations and pavement materials were documented. Numerical rating systems were used to quantify variables such as traffic, drainage conditions, and type of soil.

A comprehensive series of statistical methods were used to determine which variables had a high degree of interaction and to what degree of significance the variables were related to a specific material, performance, or response parameter. Analysis of variance (ANOVA) and clustering techniques were used to assess the significance of numerous variables and their interdependency. The results derived from these analyses were used as the basis for development of relationships (mathematical models) between the most promising combinations of variables.

Single independent variable and multivariable regression analyses were conducted by using these combinations. Initially, only linear-regression analyses were conducted. Consequently, few regression equations had correlation coefficients (R^2) in excess of 0.36. Subsequently, a considerable amount of effort was expended in plotting data to select potentially suitable transformations and performing additional regression analyses.

Some of the results obtained from these analyses are discussed in detail in the ensuing segments of this paper. Emphasis is placed on both the positive and negative aspects of the mathematical models. Although these models define the effects of variables, they should not be used indiscriminately. There are many pitfalls that may be encountered in using empirical models as an analytical tool or for adaptation to the design and evaluation of asphalt concrete pavement systems.

GENERAL DESCRIPTION OF TEST ROADS

In Table 1 the basic data that describe the general characteristics of the different test sections are presented. Most of the test sections were in service for 7 to 9 years under medium traffic conditions (levels 3 and 4). Freezing index values ranged between 1,500 and 3,000 degree days. Granular base thickness varied considerably, as might be expected when considering the diversity of soil types and climatic conditions.

A typical section for pavements in the Province of Quebec is shown in Figure 1.

ASPHALT CONSISTENCY RELATIONSHIPS

Initial results from the cluster analysis indicated that penetration, viscosity, and softening point for recovered asphalts were related. Various transformations were used to develop suitable mathematical models. In Table 2 the eight best regression equations, and also Equation 9, which was mathematically derived from Equations 6 and 7, are listed.

TABLE 1 Basic Data for Test Sections

Section	Age Years	Traffic Level [a]	Freezing Index, °days	Layer thickness, inches: Asphalt Conc.	Granular Base [b]	Subgrade Soil Type [c] /w%
1	9	5	1800	2.5	15	6/7
2	9	6	1933	3.3	28	5/11
3	9	4	2000	4.4	18	4/10
4	8	3.5	1800	4.4	48	1/24
5	8	5	1500	3.5	21	1/5
6	8	2	1850	4.0	41	1/6
8	7	2	2000	4.0	45	3/-
9	8	4.5	2000	3.8	30	1/31
10	7	1	2000	3.7	42	6/7
11	9	1	2000	5.5	42	3/-
12	7	1	2000	5.5	39	2/13
14	7	3	1873	3.9	39	2/11
15	7	3	1500	4.3	30	2/-
104	8	3	2800	3.0	41	1/18
107	7	3	3000	4.0	42	6/8
108	7	3.5	3000	3.0	36	2/8
109	8	4	3000	4.0	54	6/3
110	8	4	3000	3.5	42	6/8
111	7	3	2250	3.0	25	6/6
114	7	4	2250	4.0	30	4/8
116	6	3	2000	4.0	15	4/-
120	5	4	3000	3.0	24	6/4
121	6	4	3000	3.0	28	1/-

[a]Traffic ranges from high (e.g., 1, Auto routes) to low (e.g., 5, rural highway, or 6, local streets).
[b]Included all granular material above subgrade that generally contains base, subbase, and sand (frost) layers.
[c]1 = clay, 2 = clayey silt, 3 = sandy silt, 4 = silty sand, 5 = clayey sand, and 6 = sand.

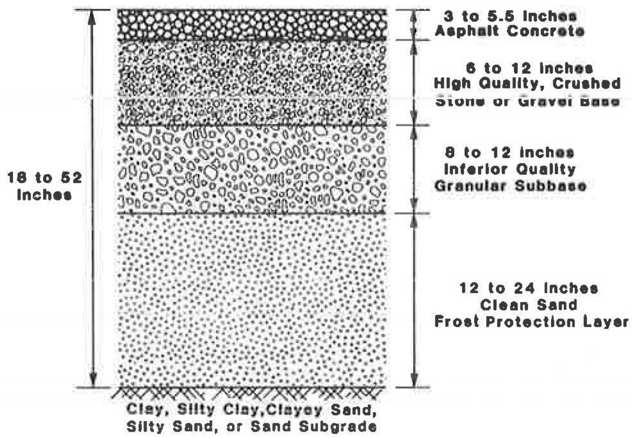

FIGURE 1 Typical pavement sections in the Province of Quebec.

Figures 2-4 illustrate the data and the developed viscosity-penetration, softening point-penetration, and viscosity-softening point relationships, respectively. It is obvious that excellent correlations were obtained between these variables.

The results suggest there is little need for more than one consistency measurement, provided the developed relationships apply to original as well as to recovered asphalts throughout the diverse range of bitumens used by the paving industry. However, it is a known fact that penetration-viscosity relationships for original asphalts can be considerably different. This difference has been used in Canada as an indicator of temperature susceptibility and to establish asphalt specifications to minimize thermal cracking at low temperatures (1,2).

Data generated and reported by Lefebvre (3) were analyzed to establish a softening point-penetration relationship for a variety of original asphalts with penetrations up to 265. Regression analysis of 24 different asphalts yielded the following equation with an $R^2 = 0.77$:

$$SP = 95.026(450/Pen)^{0.1232}$$

Comparison of this equation with Equation 7 indicates little difference between the constants and coefficients. Plots were prepared that demonstrated satisfactory agreement, except that the Lefebvre data tended to give lower softening point values when the penetration was 80 or less. Although these results are not conclusive, they imply that parameters generated by using both penetration and softening point, such as penetration index (PI), are of questionable value.

The Quebec Ministry of Transport specifications for asphalt cements include a requirement for retained penetration using the residue from the thin-film oven test. Figure 5 shows the comparison between original and recovered penetration with respect to the percent retained penetration specifications. Seven of the 15 test road binders

TABLE 2 Relationships Between Consistency Parameters

Eqn. No.	Regression Equations for Different Consistency Parameters[a]	R^2
1(a)	$\text{Log(PEN)} = 2.947 - 734.0\text{E-}7(SP)^2$	0.947
	or:	
1(b)	$(SP)^2 = 38969 - 12896\ \text{Log(PEN)}$	
2(a)	$\text{Log(PEN)} = -0.96543 + 345.48(\frac{1}{SP})$	0.958
	or:	
2(b)	$\frac{1}{SP} = 0.00299 + 0.00278\ \text{Log(PEN)}$	
3(a)	$\text{Log(PEN)} = 4.0438 - 0.8692\ \text{Log(VIS)}$	0.671
	or:	
3(b)	$\text{Log(VIS)} = 4.0355 - 0.77183\ \text{Log(PEN)}$	
4(a)	$\frac{1}{PEN} = 0.001122 + 378.0\text{E-}7(VIS)$	0.815
	or:	
4(b)	$VIS = 94.282 + 21602(\frac{1}{PEN})$	
5(a)	$VIS = -2544.7 + 23.828(SP)$	0.848
	or:	
5(b)	$SP = 110.89 + 0.03559(VIS)$	
6	$VIS = 13.053(\frac{6000}{PEN})^{0.7835}$	0.940
7	$SP - 91.812(\frac{450}{PEN})^{0.1579}$	0.956
8	$VIS = 105.66(\frac{SP}{91.812})^{4.7843}$	0.937
9	$VIS = 99.337\ (\frac{SP}{91.812})^{4.9621}$ (derived from 6&7)	

Note: Presented in this table are mathematical models developed by statistical regression analyses to define the relationships between penetration at 25°C, viscosity at 275°F, and the ring and ball softening point temperature for bitumens recovered from cores in 1980.

[a]PEN = penetration at 25°C (77°F); VIS = viscosity, centistokes at 135°C (275°F); and SP = softening point temperature (°F).

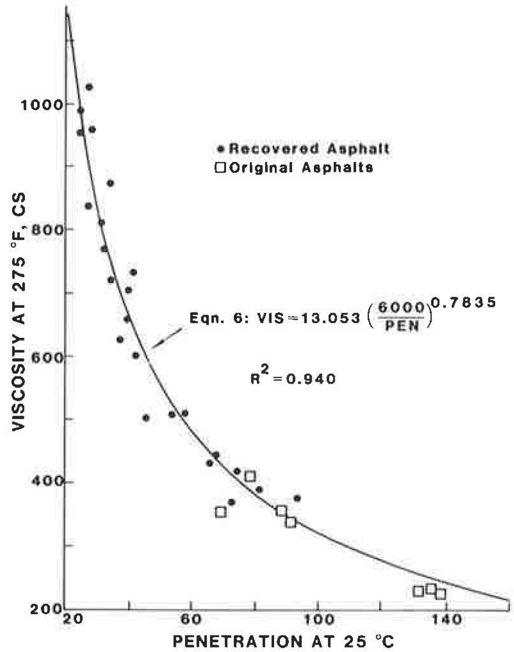

FIGURE 2 Viscosity-penetration relationship.

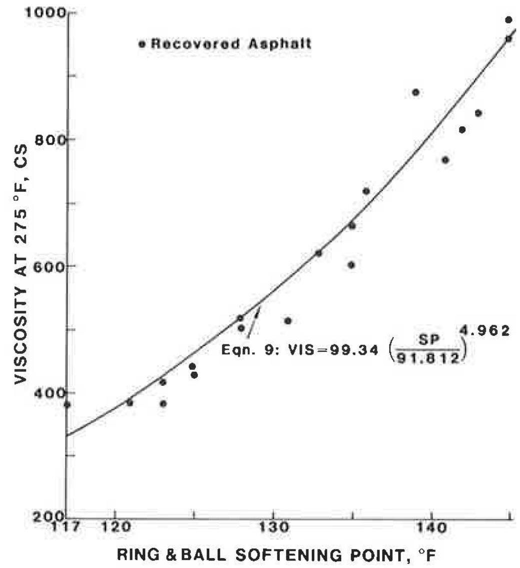

FIGURE 4 Viscosity-ring and ball softening point relationship.

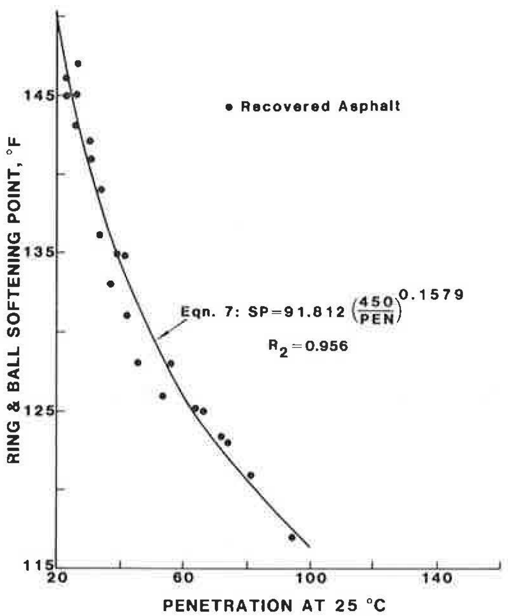

FIGURE 3 Ring and ball softening point-penetration relationship.

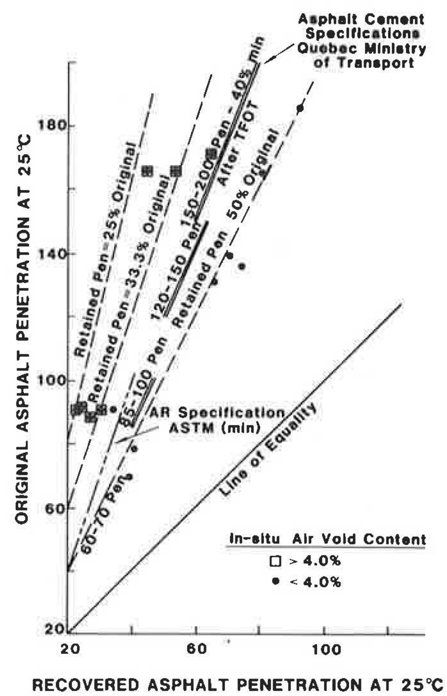

FIGURE 5 Retained penetration comparison.

had a retained penetration greater than 49 percent, which is considerably greater than the current Ministry of Transport specifications. The remaining eight binders had a lower retained penetration than either the ASTM asphalt residue (AR) or the Ministry of Transport specifications.

Although these specifications do not apply to recovered asphalts, it is an indication that excessive hardening has occurred. The exact cause of the high degree of age hardening was not identified, although it was observed that the in-service air void content was greater than 4.0 percent in seven of the test sections. These sections had the lowest retained penetration, which suggests that inadequate compaction was achieved during construction, the bitumen was damaged because of overheating, or the composition of the bitumen promoted rapid age hardening.

RELATIONSHIPS BETWEEN ASPHALT CONSISTENCY AND MIX PARAMETERS

A review of the results from clustering and initial regression analyses indicated that (a) asphalt consistency values were affected by the air void content of the mix and (b) both parameters influenced the tensile strength of cores obtained from the test section. Additional statistical analyses were performed, which resulted in the development of the regression equations given in Table 3.

TABLE 3 Relationship for Air Void Effects and Tensile Strength

Eqn. No.	Regression Equations for Different Parameters[a]		R^2
9	$VIS = 531.74\ (VV)^{0.2904}$	(sections 1 to 15)	0.762
10	$VIS = 295.87\ (VV)^{0.3295}$	(sections 104 to 121)	0.692
11	$PEN = \frac{50.43}{(VV)^{0.3237}}$	(sections 1 to 15)	0.714
12	$PEN = \frac{144.34}{(VV)^{0.6384}}$	(sections 104 to 121)	0.910
13	$\Delta VIS = 203.61\ (VV)^{0.5716}$	(sections 1 to 15)	0.692
14	$\Delta VIS = 84.37\ (VV)^{0.7068}$	(sections 104 to 121)	0.707
15	$\Delta PEN = 35.30\ (VV)^{0.300}$	(sections 1 to 15)	0.488
16	$\Delta PEN = 55.46\ (VV)^{0.357}$	(sections 104 to 121)	0.255
17	$ITS = 134.01 + 0.2461\ (VIS) - 15.4203\ (VV)$		0.567
18	$\text{Log } ITS = 0.6694 + 0.6531 \text{ Log } (VIS) - 0.2479 \text{ Log } (VV)$		0.567
19	$\text{Log } ITS = 3.31015 - 0.48905 \text{ Log } (PEN) - 0.2842 \text{ Log } (VV)$		0.471
	or: $ITS = \frac{2042.44}{(PEN)^{0.48905}\ (VV)^{0.2842}}$		

Note: Presented in this table are mathematical models developed by statistical regression analyses to define the relationships between tensile strength, air void content, and asphalt consistency parameters from cores in 1980.
[a]VIS = viscosity @ 135° C, cs; PEN = penetration @ 25° C; VV = percent air void content; ITS = indirect tensile strength @ 0° C (32° F); 0.05 in./min rate of loading.

Asphalt Consistency

Equations 9 and 10 (Table 3) indicate that viscosity at 275°F (135°C) would increase between 20 to 60 centistokes for each 1.0 percent increase in air void content for pavements that are about 7 to 9 years old. A similar effect was obtained from Equations 11 and 12, which predict a decrease in penetration of between 1 to 20 for each 1.0 percent increase in air void content. Figures 6 and 7 show these relationships and the plotted data for viscosity and penetration of the Abson recovered asphalts versus the in-service air void contents. The trends illustrated in these figures suggest that the 275°F (135°C) viscosity and the 77°F (25°C) penetration for recovered asphalts are primarily dependent on the grade of the original asphalt cement and the air void content. Obviously, high air void contents after 7 to 8 years of traffic were the result of inadequate compaction and insufficient densification due to traffic. Four of the 23 test sections had air void contents in excess of 6.4 percent. These sections were constructed with 60-100 penetration asphalt cements and were among the nine sections exhibiting the most cracking.

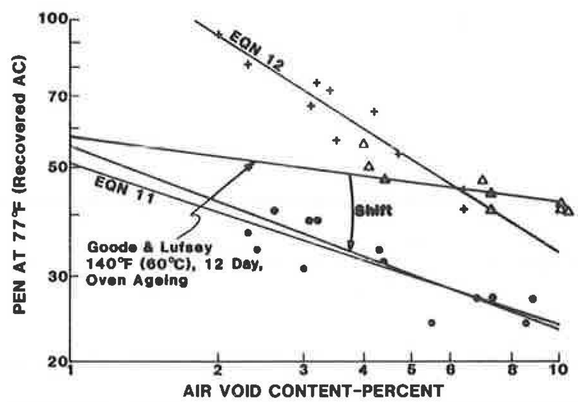

FIGURE 7 Recovered asphalt penetration-air void content relationship.

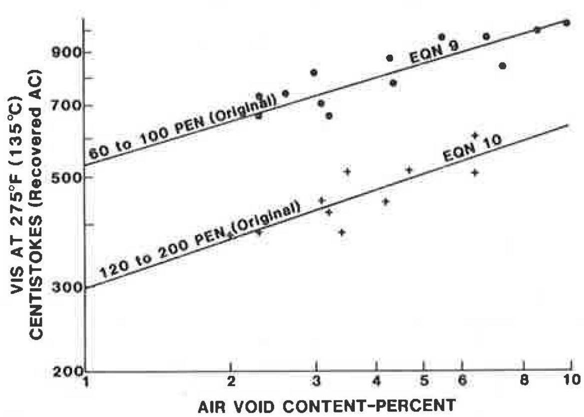

FIGURE 6 Recovered asphalt viscosity-air void content relationship.

Relationships were developed for the actual change from original to recovered consistency values with respect to in-service air void content. Figure 8 shows the data and resulting trend from regression Equations 13 and 14. Regression analyses were also performed to evaluate the change in penetration as a function of air void content. Equations 15 and 16 define the change in penetration. Correlation coefficients for these equations were extremely low. However, the exponents in both equations are almost identical. This suggests that the effect of air void content on hardening is logarithmically linear for any grade of asphalt cement, and the constant in the

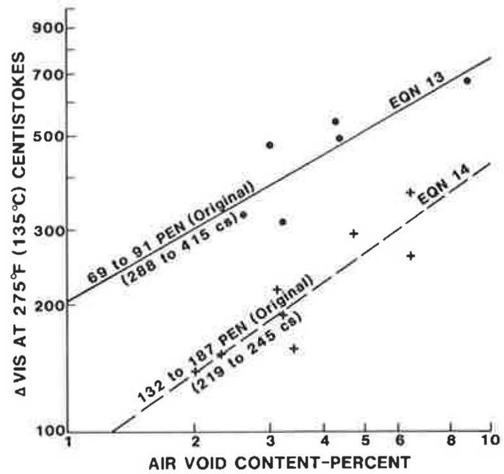

FIGURE 8 Effect of in-service air void content on asphalt hardening.

equation denotes the shift factor to accommodate different grades.

Air void and penetration data from an asphalt hardening study by Goode and Lufsey (4) were used to predict the oven age hardening effects. Their aging procedure used mixtures prepared with a 91 penetration asphalt cement at different air void contents and processed for 12 and 63 days in a 140°F (60°C) oven. The penetration was obtained on asphalts recovered from specimens with and without oven aging. The average penetration of the nonaged specimens was 65.4. The difference between this value and the 12-day oven-cured specimens was used to establish the following equation:

$$\Delta PEN = 9.66\ (VV)^{0.372}$$

where

ΔPEN = 65.4 - PEN at 12 days,
VV = air void content (%), and
R^2 = 0.704.

By using the following expression,

$$RPEN = 65.4 - S(\Delta PEN)$$

where RPEN is the penetration of asphalt recovered from test road samples, and S is the shift factor to predict penetration of approximately 8-year-old pavements with 85-100 penetration grade asphalt.

The shift factor obtained to predict the in-service penetration from the 12-day oven-aging data was 1.85. The shifted relationship is illustrated in Figure 7. The relatively good approximation provided by this procedure suggests that the methods used by Goode and Lufsey (4) could be used to estimate the age hardening of asphalts and the resulting indirect tensile strength. This, of course, depends on the amount of hardening that occurs in mixing either in the laboratory or at the plant, as well as the difference in climatic effects and composition of the asphalt. Additional experimentation will be required to determine if this procedure is valid or can be modified to accommodate different grades of asphalt cement and the effects of time exposure and hardening rate.

Mix Tensile Strength

Indirect tensile tests were conducted on cores at 0°C (32°F) by using a 0.05 in./min (1.27 mm/min) rate of loading. The influence of asphalt viscosity and air void content on the tensile strength of these cores is defined in Table 3 by Equations 17 and 18. Figure 9 illustrates the relationship provided by Equation 17. The relatively low R^2 values for both equations appeared to be caused primarily by excessive variation in tensile strength or air void content values or both.

According to Equation 17, an increase in viscosity of 100 centistokes would increase the tensile strength at 0°C by 25 psi. Similarly, a 1.0 percent increase in air void content would reduce the tensile strength by 15 psi. This reduction is approximately equivalent to a 5 to 10 percent reduction in tensile strength. Other limited indirect tensile strength data from cores provided a 12 percent change per percent voids for air void contents between 5.5 and 8.7 percent. Results by other investigators indicate that the dynamic modulus of asphalt concrete pavements is reduced by about 11.5 percent for each 1.0 percent increase in air void content (5).

The substantial difference between test temperatures for viscosity and indirect tensile tests may have a significant effect on the interpretation of these results. The viscosity and shear susceptibility of these recovered asphalts at lower temperatures (e.g., 0°C) could possibly provide more realistic and meaningful relationships. Unfortunately, only penetration values were available for analysis

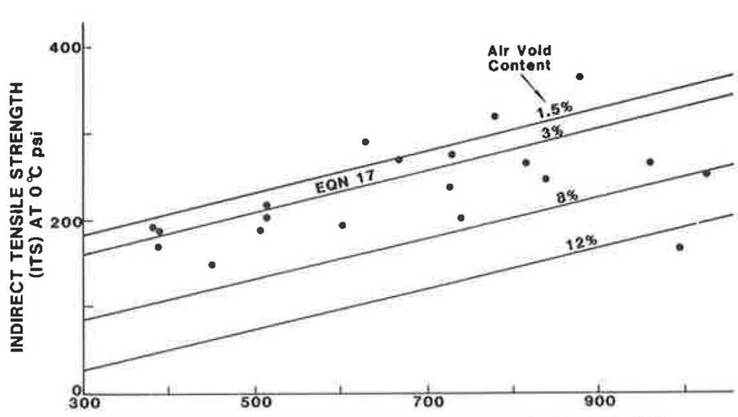

FIGURE 9 Effect of viscosity and air void content on indirect tensile strength.

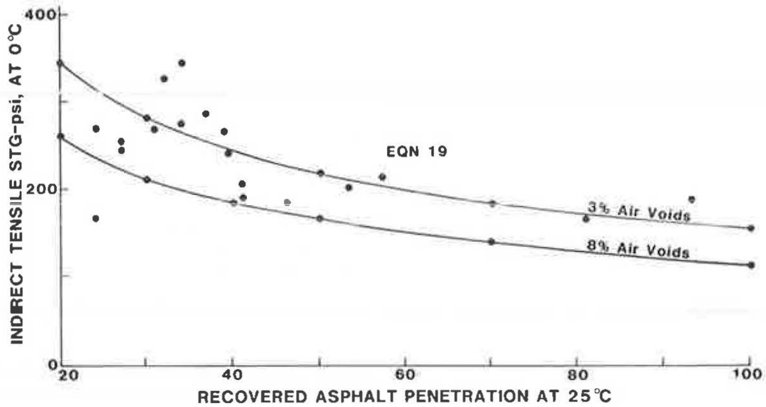

FIGURE 10 Effect of penetration and air void content on indirect tensile strength.

at these lower temperatures. These values and corresponding air void content values were used in a regression analysis that yielded Equation 19 and is shown with the plotted data in Figure 10. Although the correlation coefficient is low, it is obvious that the effect of air voids on tensile strength is almost identical for both Equations 18 and 19.

The Schweyer constant stress rheometer was used to obtain low temperature viscosity data for asphalts recovered from five test sections. The regression equation developed by using the 0°C viscosities is

$$ITS = 0.6164\ (VC)^{0.2815} - 15.42\ (VV)$$

where

ITS = indirect tensile strength at 0°C (psi),
VC = constant power viscosity (J = 100 w/m^3) at 0°C (Pa.s),
VV = air void content (%), and
R^2 = 0.851.

This equation predicts the indirect tensile strength within about 10 percent of the measured values. In this equation, a 1.0 percent change in air void content would decrease the indirect tensile strength 6 to 8 percent within the 180- to 280-psi range.

The results of analyses for consistency and tensile strength indicate the need to attain a reasonably low air void content in the compacted mix that will minimize age hardening and the potential for low-temperature cracking. In general, pavements with air voids in excess of 4 or 5 percent after 1 or 2 years of service are likely to harden sufficiently to prevent any further traffic densification. This in turn can promote additional hardening of asphalt binders and eventually result in cracking.

MIX PARAMETERS RELATED TO PAVEMENT CRACKING

Longitudinal pavement cracking was classified according to the lineal feet of simple or multiple and polygonal longitudinal cracks in the 3-km-long test sections. The amount of longitudinal cracking was insignificant and generally less than 100 lineal feet. Only four pavement sections exhibited longitudinal cracking in excess of 50 lineal feet. Two of these sections had extremely low penetration and the other two had exceptionally high pavement deflection values.

Transverse cracks were classified as the number of simple or multiple and polygonal transverse cracks in a test section. Also, the severity of transverse cracking was rated according to the class of cracking (class 1, 2, or 3). Except for sections 5 and 15, the amount of class 1 cracking exceeded 58 percent of the total transverse cracking. Twelve of the 23 test sections had less than 35 transverse cracks (1.2 cracks per 100 m). Cracking in the other sections increased to a maximum of 176 transverse cracks (5.9 cracks per 100 m) for section 10.

Efforts to develop correlations between longitudinal cracking and other parameters were unsuccessful primarily because of the lack of longitudinal cracking. Analysis of the transverse cracking data resulted in the development of single and multiple variable relationships. Some of the regression equations that were obtained from analyses of the transverse cracking data are given in Table 4. Equations 20-22 have extremely low correlation coefficients, but suggest that penetration, traffic, and air void content affect the degree of pavement cracking.

When plotted, transverse cracking and penetration data indicated that the data should be segregated according to traffic levels 3 to 6 (medium to light) and 1 to 3 (heavy to medium). The results of regression analyses for each category of traffic yielded excellent results. Figure 11 shows the trends for regression Equations 23 and 24 and the corresponding data points.

Sections 1 and 6 were excluded from the analysis because they did not conform to the trends of other test sections. The thickness of the asphalt concrete in section 1 was only 2.5 in. (64 mm), which was less than any of the other pavements. Section 6 had extremely low Dynaflect deflections and an extremely high subgrade modulus, which apparently produced considerably less cracking, even though the recovered asphalt penetration was 31.

Multivariable regression analyses were performed in an attempt to improve the correlation by using combinations of those parameters considered to have a significant effect on pavement cracking. Equation 25 defines transverse cracking (TC) on the basis of penetration and traffic. This equation format plus the interaction of the thickness and traffic gave Equation 26 (see Table 4). Although the correlation is mediocre ($R^2 \approx 0.68$), the effects of penetration, thickness, and traffic appear to be generally reasonable. Figure 12 illustrates the effects of these variables. If traffic is lighter than levels 1 to 3, the amount of cracking is minimal, particularly if the in situ binder penetration is greater than 40, as shown in Figure 11. Increasing pavement thickness from 3 to 6 in. (76 to 154 mm) when the penetration is greater than 40 reduces the amount of

TABLE 4 Relationships for Transverse Cracking

Eqn. No.	Regression Equations for Different Parameters[a]	Std. Error	R^2
	Single Variable Correlations:		
20	$TC = 25.326 + 0.396(\frac{120}{PEN})^{3.3}$	37.8	0.311
21	$TC = -131.251 + 247.84(TRA)^{-0.3}$	33.8	0.449
22	$TC = 4.8471 + 10.035\ (VV)$	40.4	0.213
23	$TC = 5.15 + 0.235(\frac{120}{PEN})^{3.3}$	----	0.933
	(Light to medium traffic, Levels 6 to 3)		
24	$TC = 21.58(\frac{120}{PEN})^{1.206}$	----	0.855
	(Medium to heavy traffic, Levels 3 to 1)		
	Multiple Variable Correlations		
25	$TC = -129.37 + 0.323(\frac{120}{PEN})^{3.3} + 218.57(TRA)^{-0.3}$	27.6	0.650
26	$TC = -239.91 + 0.321(\frac{120}{PEN})^{3.3} + 407.54(TRA)^{-0.3}$ $- \frac{17.05\ THICK}{(TRA)^{-0.3}}$	27.2	0.679
27	$TC = -97.619 + 0.0682(VIS) + 263.96(TRA)^{-0.3} - 0.0228(FI)$ $- 0.0426\ (THICK)(ITS)$	27.9	0.682

Note: Presented in this table are mathematical models developed by statistical regression analyses to define the relationships between different parameters and transverse cracking.

[a]TC = total simple, multiple, and transverse cracks; PEN = penetration @ 25°C; TRA = traffic (1 = heavy, 6 = light); THICK = thickness of asphalt concrete pavement (in.); FI = freezing index (° days); ITS = indirect tensile strength at 0°C.

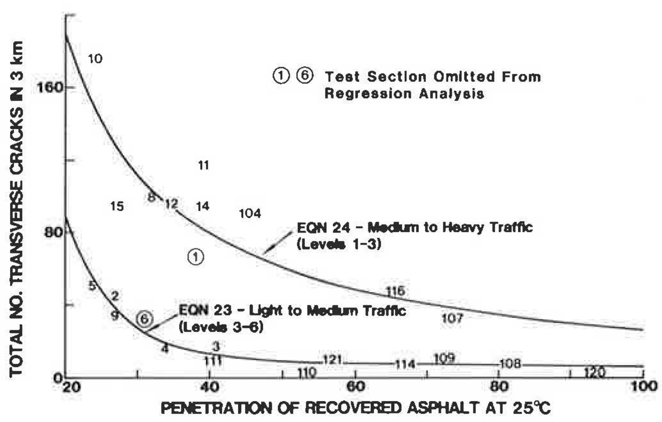

FIGURE 11 Effect of binder penetration and traffic on transverse cracking.

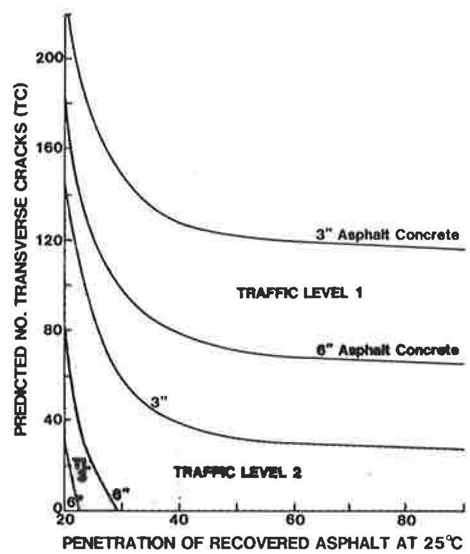

FIGURE 12 Effect of variables on transverse cracking.

cracking by about 40 percent for heavy traffic conditions (level 1). Furthermore, these results indicate that 3 in. of asphalt concrete with a binder penetration of 50 to 60 is essentially equivalent to 6 in. of asphalt pavement with a 25 penetration asphalt binder.

Asphalt viscosity, traffic, freezing index, thickness, and indirect tensile strength parameters were evaluated to determine their effects on transverse cracking. The results of these analyses did not indicate any improvement over those expressions containing penetration as the asphalt consistency measurement. Equation 27 defines the best multiple variable correlation obtained using the viscosity. This equation indicates that cracking increases with viscosity and traffic, decreases with an increase in freezing index, and decreases with increasing thickness and tensile strength. The effect of these variables on cracking is logical, but the effects of variables other than viscosity and traffic are relatively insignificant.

The most important aspects of the transverse cracking analyses are the effects of traffic level and asphalt binder consistency. It is apparent that traffic contributes to the severity of transverse thermal cracking. Equally important is the extremely significant effect of penetration, as shown in Figure 11, which suggests that pavement cracking would be substantially reduced if in situ penetration was never less than about 40.

PREDICTION OF TRANSVERSE CRACKING

Two models were selected for evaluation and use in the prediction of transverse cracking and cracking temperatures. The Haas model (6) is intended for use in calculating a cracking index. The Asphalt Institute procedure (7) determines a critical temperature that may be compared with the minimum temperature in the winter to evaluate the potential for thermal cracking. The following discussion presents the results and demonstrates that neither method appears suitable for transverse cracking predictions for highways in Quebec.

A method for designing asphalt pavements to minimize low-temperature shrinkage cracking was proposed by Haas (6). This method was empirically derived by using stiffness of the asphalt concrete pavement, a winter design temperature, and type of soil to predict a cracking index (I). The cracking index is the number of transverse cracks per 500 ft of two-lane highway pavement. Transverse cracks that do not extend across one lane are not counted.

A computer program was written to facilitate computation of the cracking index. Data for the recovered asphalt were used instead of data for the original asphalt because of incomplete data. This produced computed stiffness and cracking index values much greater than those that would have been obtained by using original asphalt properties. The effect of age was evaluated by using the difference between cracking index values for in-service age in years and zero years ($I_n - I_o$). This assumes that the in-service asphalt stiffness for input into the model does not change over the age of the pavement.

The results of cracking index computations for the different test sections are given in Table 5. The predicted amount of cracking ranged from 394 to 847 transverse cracks in a 3-km section as compared with 1 to 176 in the test road sections. Figure 13 shows that the amount of predicted cracking is not related to the number of simple transverse cracks observed in 1980. It is obvious that the cracking index is not suitable for Quebec conditions. The extremely high degree of predicted cracking may be due to using the recovered asphalt properties. However, even the soft residues (e.g., 93 penetration) gave exceedingly high values of cracking (e.g., 394 per 3 km).

The Asphalt Institute has compiled the results of various researchers into design procedures to minimize low-temperature transverse cracking (7). Critical temperatures (T_c) for cracking were computed for each section by using these procedures. The difference between T_c and the minimum winter temperature (T_{min}) was used to compare the amount of transverse cracking in the test sections. The data in Table 6 present values of T_c, T_{min}, and $T_c - T_{min}$. Figure 14 illustrates that there was

TABLE 5 Cracking Index Values (Haas method)

Section No.	Io @ 0 yrs[a]	I_n @ Age in yrs[a]	19.69 (I_n-I_o) No. Cracks in 3km	No. of Transverse Cracks: simple	polygonal & multiple	Total
1	1016	1054	748	38	25	63
2	1870	1913	847	34	11	45
3	825	861	704	7	11	18
4	749	784	689	9	9	18
5	999	1037	748	27	23	50
6	750	785	689	11	21	32
8	1471	1450	650	17	83	100
9	1321	1360	768	32	3	35
10	2303	2337	669	107	69	176
11	894	933	768	91	4	95
12	923	953	591	28	59	87
14	684	712	551	64	30	94
15	934	965	610	59	38	97
104	1626	1665	768	49	42	91
107	1251	1281	591	23	9	32
108	931	960	571	3	5	8
109	1757	1794	729	4	6	10
110	2391	2430	768	1	0	1
111	1910	1942	630	7	4	11
114	851	879	551	7	1	8
116	825	850	473	35	12	47
120	916	936	394	1	1	2
121	956	983	532	3	7	10
$\bar{x}$			654			49.1
Std. Dev.			111			44.6

[a]Predicted number of transverse cracks in 500 ft.

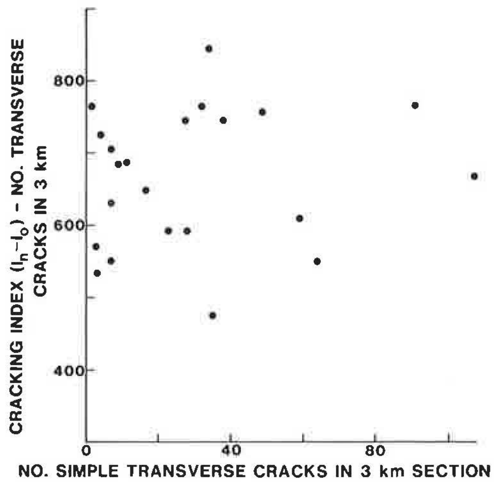

FIGURE 13 Comparison of predicted to measured transverse cracks.

TABLE 6 Predicted Cracking Temperatures (Asphalt Institute method)

Section	Predicted Cracking Temp. (T_C), °C	Winter minimum Temp., °C	T_C - T_{min}
1	-33	-31.7	-1.3
2	-32	-35.0	+3.0
3	-33	-30.0	-3.0
4	-34	-31.7	-2.3
5	-35	-27.8	-7.2
6	-37.5	-37.2	0
8	-37	-37.2	0
9	-32	-30.0	-2.0
11	-38	-31.7	-6.3
12	-37	-31.7	-5.3
14	-33	-32.8	0
15	-40	-30	-10.0
104	-39	-36.7	-2.3
107	-47	-39.4	-7.6
108	-41	-39.4	-1.6
109	-43	-39.4	-3.6
110	-39	-43.9	+4.9
111	-39	-37.2	-1.8
114	-44	-27.2	-16.8
116	-39	-27.2	-11.8
120	-40	-39.4	-0.6
121	-41	-39.4	-1.6

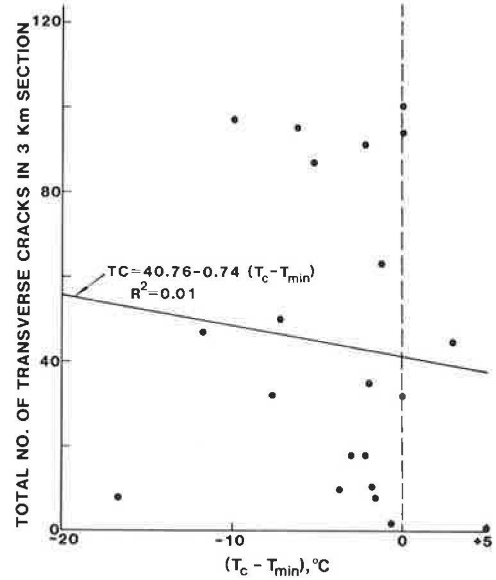

FIGURE 14 Comparison of transverse cracking to predicted differential cracking temperature.

essentially no correlation between the number of transverse cracks and the predicted differential cracking temperature. These poor results may be due to insufficient range in data, inadequate methodology for computation of critical temperatures, or poor definition of minimum temperatures over the time period that the pavements have been in service.

EVALUATION OF RIDE QUALITY

The Mays meter is used extensively for evaluation of the ride quality of pavements. Measurements were obtained on the test road sections during the summer and winter. Inspection of the Mays meter and transverse cracking values indicated that ride quality was not generally degraded by cracking. Little difference was observed between summer and winter measurements.

Regression analyses were performed to evaluate variables that had a significant effect on ride quality. Only three of the various combinations of variables were sufficiently significant to warrant further investigation. These three regression equations are given in Table 7.

The equations indicate that ride quality decreases as the soil becomes more sandy and increases with freezing index, percent fill, depth of drainage ditches, and the amount of transverse cracking. Close inspection of plots of these parameters versus Mays meter values indicated that the bias created by the limited range and interaction of variables produced a meaningless relationship. Only the percent fill has any basis for rational interpretation when considering trends of the plotted data. Improved ride in fill sections may be the result of better quality control and drainage conditions than encountered in cut sections. The indication that better riding quality occurs with greater cracking is probably related to the design and quality control on major highway pavements that are subjected to heavy traffic. However, in some instances traffic can produce definite improvement in ride over that obtained immediately after construction.

RUT DEPTH OBSERVATIONS

Attempts to develop rational relationships to predict rut depth from the available data were totally unsuccessful. This may be partly caused by the insufficient range in rutting depth values. Only 5 of the 23 test sections exhibited rutting in excess of 0.5 in., and 10 sections had nonmeasurable, insignificant rutting. Therefore, it can be concluded that rutting is not a problem when similar asphalt mixtures and pavement designs are used for new construction in comparable climatic regions.

TABLE 7 Relationship Between Mays Meter (ride quality) and Other Parameters

Equation No.	Regression Equations for Different Parameters [a]	R^2
28	MMW = 3.093 - 0.0049 (FI) - 0.0041 (PF) - 0.0862 (DDD) - 0.0046 (TC)	0.510
29	MMW = 3.131 - 0.0059 (FI) - 0.0044 (PF) - 0.0910 (DDD) - 0.0047 (TC) + 0.063 (TYSOL)	0.585
30	MMS = 2.541 - 0.0037 (FI) - 0.0029 (PF) - 0.075 (DDD) - 0.0043 (TC) + 0.079 (TYSOL)	0.608

[a]MMW = Mays meter (winter), MMS = Mays meter (summer), FI = freezing index, degreee-days), PF = percent fill, DDD = depth of drainage ditches (ft), TC = total number of transverse cracks, TYSOL = type of soil: clay = 1.0, clayey silt = 2.0, sandy silt = 3.0, silty sand = 4.0, clayey sand = 5.0, and sand = 6.0.

DYNAFLECT DEFLECTIONS AND SUBGRADE MODULI

Dynaflect deflections were used to estimate the moduli of the pavement and subgrade. These subgrade modulus values were plotted versus the Dynaflect fifth-sensor (S-5) deflections, as shown in Figure 15. These data were evaluated by using regression analysis, and the results were compared to the relationship established by Majidzadeh (8). Figure 15 shows that the estimated moduli are less than those obtained by Majidzadeh for equivalent deflections. The slope of the line established by the regression equation indicates a greater difference between the Majidzadeh and estimated modulus values at low deflections (high modulus). This may be attributed to the differences in pavement systems or to calibration and accuracy of the Dynaflect deflections.

An analysis was performed to tune the layer moduli and deflection basins for five of the test sections that were selected to encompass a large range in recovered penetration and Dynaflect deflections values. Low-temperature viscosities were determined for recovered asphalts by using the Schweyer constant stress rheometer. The dynamic modulus of the asphalt concrete pavement was obtained by using these viscosities and a correlation between viscosity and dynamic modulus (9). The computed asphalt concrete moduli and the estimated base and subgrade moduli with their appropriate Poisson's ratio were used in a multilayer elastic stress analysis program. The predicted deflection basins were compared with Dynaflect measured basins. The base and subgrade moduli were adjusted slightly until the predicted deflection response was essentially equal to the mean measured response.

The subgrade moduli obtained from this analysis were plotted against the predicted deflection (S-5), as illustrated in Figure 15. The slope of the line is parallel to the Majidzadeh relationship and suggests that this trend may provide an excellent relationship for the prediction of subgrade moduli. Regression analysis of these data yielded the following equation:

$$E_s = 5.3966(S_5)^{-1.0006}$$

where

E_s = subgrade modulus (psi),
S_5 = Dynaflect fifth-sensor deflection (in.), and
R^2 = 0.997.

Other analyses were conducted in an attempt to relate deflections to transverse cracking. In general, the deflection data were stratified according to the hardness of the binder (penetration). Therefore, no improvement was obtained over the previously presented transverse cracking predictions. There was a definite indication that pavement stresses were related to the degree of cracking.

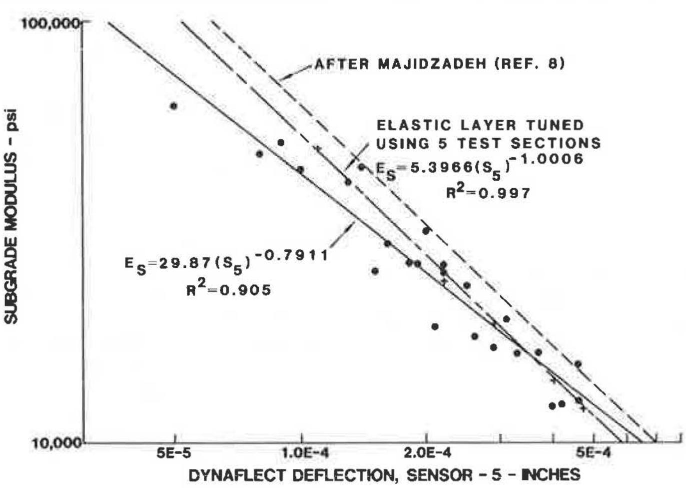

FIGURE 15 Subgrade modulus-Dynaflect deflection relationships.

However, this is also masked by the thermal stresses that were not evaluated because pavement cooling curves were not available.

SUMMARY

The results of analyses performed on data collected from 23 test road sections in the Province of Quebec, Canada, provided information on the effect that certain parameters have on the properties and performance of asphalt concrete. The most significant finding was that consistency of recovered asphalts and traffic level defined to a high degree of significance the amount of transverse cracking on 6- to 9-year-old pavements.

The effects of air void content on asphalt hardening and indirect tensile strength at 0°C (32°F) were identified. Relationships were developed by using the results of the Goode and Lufsey (4) investigations, which compared voids and penetration by using the 60°C (140°F) oven aging test. A tentative procedure for prediction of in-service hardening of bitumens has been suggested for use in determining the effects of air void content and age on the indirect tensile strength and consistency (penetration or viscosity) of the asphalt binder.

Relationships developed between penetration, viscosity, and ring and ball softening point of recovered asphalts demonstrate that these consistency measurements are related to such a high degree of significance that only one of these consistency parameters is needed to characterize the asphalt. However, viscosity-penetration relationships for original asphalts are not uniquely related and form the basis for selection of asphalts to minimize low-temperature cracking potential.

Several existing models were used to predict transverse cracking and cracking temperature. Analysis of these values and the actual number of observed transverse cracks provided absolutely no correlation between predicted and actual cracking. The use of two categories of traffic level and the penetration (25°C) of recovered asphalt provided an excellent relationship with the number of transverse cracks.

The depth of rutting was insignificant, except for five sections that had ruts in excess of 0.5 in. No relationships were found to define the amount of rutting.

Analysis of ride quality using the Mays meter indicated that fill sections provide a better ride than do cut sections. The amount of transverse cracking had little effect on ride quality. Other variables were considered in the analysis, but their effect on ride quality was marginal or not significant.

A relationship was developed between subgrade moduli and Dynaflect fifth-sensor deflections. Data from five sections that had substantially different characteristics were used and adjusted in a multilayer elastic stress analysis program until the predicted deflection basin conformed to the mean or typical Dynaflect basin. The developed relationship compares favorably with the data generated to estimate subgrade moduli for the other test sections.

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