PAVEMENT SURFACE CHARACTERISTICS AND MATERIALS

C. M. Hayden, *editor*

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PAVEMENT SURFACE CHARACTERISTICS AND MATERIALS

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NOTE

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Foreword

The symposium on Pavement Surface Characteristics and Materials was held in Orlando, Fla., on 11 Dec. 1980. ASTM Committees E-17 on Traveled Surface Characteristics and D-4 on Road and Paving Materials sponsored the symposium. C. M. Hayden of the Federal Highway Administration served as symposium chairman and edited this publication.

Related ASTM Publications

Asphalt Pavement Construction: New Materials and Techniques, STP 724 (1980), 04-724000-08

Quality Assurance in Pavement Construction, STP 709 (1980), 04-709000-08

Recycling of Bituminous Pavements, STP 662 (1978), 04-662000-08

Walkway Surfaces: Measurement of Slip Resistance, STP 649 (1978), 04-649000-47

Low-Temperature Properties of Bituminous Materials and Compacted Bituminous Paving Mixtures, STP 628 (1977), 04-628000-08

A Note of Appreciation to Reviewers

This publication is made possible by the authors and, also, the unheralded efforts of the reviewers. This body of technical experts whose dedication, sacrifice of time and effort, and collective wisdom in reviewing the papers must be acknowledged. The quality level of ASTM publications is a direct function of their respected opinions. On behalf of ASTM we acknowledge with appreciation their contribution.

ASTM Committee on Publications

Contents

Introduction

Road surface characteristics and paving materials have been a serious concern since ancient times. Prior to the 20th century the emphasis was on providing an all-weather surface which offered minimal rolling resistance. Any concern for friction was likely to be aimed at providing enough traction for a draft animal to pull a load without falling down. With the advent of motor vehicles and aircraft using various types of man-made pavements, engineers became concerned with the friction available for stopping and cornering.

As early as 1924, T. R. Agg of Iowa State University, followed by Ralph Moyer, began testing road pavements for wet weather friction. Agg's and Moyer's work paralleled by Stinson and Roberts' work at Ohio State University laid a foundation on which we are still building. It is interesting to note that Professor Moyer's recommendations at the Highway Research Board Meeting in 1933 are reasonably close to some of the recommendations in papers in this symposium.

The depression and war years seemed to divert our attention so much that in the 1950's we had to relearn Moyer's experience. In 1958, the Virginia Department of Highways sponsored the First International Skid Prevention Conference and a correlation study of known methods of measuring pavement friction at Charlottesville, Va. ASTM recognized the growing interest in skid resistance by organizing Committee E-17 on Skid Resistance.

The new E-17 committee guided the Virginia Department of Highways in a second correlation study at Tappahannock, Va. Results of this study led ASTM to adopt two methods: the ASTM Method for Measuring Surface Frictional Properties Using the British Pendulum Tester [E 303-74(1978)], and the ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79).

Following adoption of ASTM Method E 274-79, the Florida State Road Department and other organizations constructed skid testers to meet the new method's specification.

In 1967, the Florida State Road Department and the U.S. Bureau of Public Roads, with the guidance of ASTM Committee E-17, organized another correlation study to examine the degree of standardization.

The correlation studies in Charlottesville, Tappahannock, and Florida showed a disturbingly poor correlation of the quantitative results from skid testers. As a result, Pennsylvania State University was sponsored by the Transportation Research Board, American Association of State Highway and Transportation Officials, and Federal Highway Administration (FHWA) to determine the reasons for poor correlations among locked-wheel skid testers and to propose improvements. The Penn State Correlation Study in 1972 provided the foundation for the study conclusions reported in the National Cooperative Highway Research Program (NCHRP) Report 151.

In the mid-1960's the U. S. Congress recognized a need for more attention to safety. Legislation requiring state safety programs and federal appropriations resulted in a broad range of safety programs. Included was funding for a large amount of research and development in wet weather safety by the FHWA and state highway departments. Much of the research in skid resistance in the 1970's was sponsored in part with federal money.

In 1972, FHWA began the Field Test and Calibration Centers to provide a complete calibration and evaluation service for skid measurement systems. The aim was to improve and standardize skid test equipment and operation and to provide a reference standard for measurements.

In 1977, a Second International Skid Prevention Conference was held in Columbus, Ohio, to exchange information on wet weather skidding. Emphasis was placed on application of the massive amount of research which had been completed. The interaction of the driver, the vehicle, and the pavement surface was of prime concern. The conference showed that we had learned a lot about wet weather safety, but it also showed that we need to learn a lot more.

Significant wet weather safety research and development work has been completed in recent years. These developments are being applied in measuring pavement surface characteristics, constructing better pavements and analyzing suspected hazardous locations. The papers presented here discuss pavement texture measurement, relation of different texture measurement techniques, performance predictor models, and systematic approaches to safety improvement programs.

C. M. Hayden

Federal Highway Administration, Washington, D. C. 20590; symposium chairman and editor.

W. P. Chamberlin[1] *and D. E. Amsler*[1]

Measuring Surface Texture by the Sand-Patch Method

REFERENCE: Chamberlin, W. P. and Amsler, D. E., **"Measuring Surface Texture by the Sand-Patch Method,"** *Pavement Surface Characteristics and Materials, ASTM STP 763.* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 3-15.

ABSTRACT: Components of variance were analyzed for 720 measures of concrete pavement texture depth obtained by the sand-patch method. The measurements were made in connection with a complete factorial field experiment involving four texturing methods used on two sections each of five different paving jobs. Each pavement section was tested at three different sites by three different operators performing two tests each. The analysis permitted estimates of the repeatability and reproducibility of the sand-patch test, as well as errors that can be expected in measuring the mean texture depths of a section of textured pavement.

KEY WORDS: sand-patch test, textures, surface texture, measurement, concrete pavements, precision, reproducibility, repeatability, pavement surface characteristics

The sand-patch test [*1,2*][2] is the most widely used method of judging the adequacy of texture on new portland cement concrete pavement [*3*]. It consists of spreading a predetermined (loose bulk) volume of dry, uniformly graded, fine (Ottawa) sand on the pavement surface, and working it into a circular pattern of maximum diameter through the revolving motion of a small hand-held spreading tool. Mean texture depth is calculated from the bulk volume of the sand and the mean diameter of the circular sand patch. Results of the test can be used in the field to assess the adequacy of texturing compared to specification requirements, and to improve construction practices, where required. The test is simple and quick. Equipment and methods are described in an American Concrete Paving Association (ACPA) guideline [*2*], and shown in Figs. 1 and 2.

[1]Associate civil engineer and senior civil engineer, respectively, Engineering Research and Development Bureau, New York State Department of Transportation, Albany, N.Y. 12232.

[2]The italic numbers in brackets refer to the list of references appended to this paper.

FIG. 1—*Operator spreading fine sand with a circular motion of the spreading tool in the sand-patch test.*

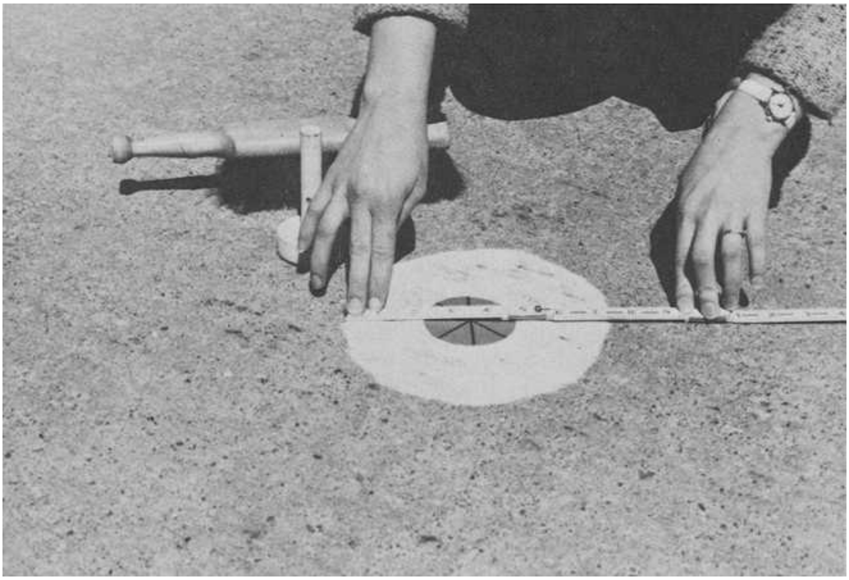


FIG. 2—*Operator making a series of measurements from which he will calculate the mean diameter of the sand patch. (Objects at left are brush used for cleaning pavement surfaces and the wooden spreading tool shown in use in Fig. 1.)*

The sand-patch test was used recently to measure the depths of textures produced in a field investigation of different texturing methods [*4*]. The investigation also was structured to permit an evaluation of pavement variability and testing error, as reflected in results of the test itself.

The experiment designed for the investigation is described in Table 1. Two 36.58-m (120-ft)-long test sections were placed, using each of four different texturing methods, on five different paving jobs. Initial texture depth on each of these 40 test sections then was measured by the sand-patch method at three randomly chosen sites by the same three operators performing two tests each. Results of these measurements then were used to develop precision statements for the sand-patch test and to construct nomographs for determining desired testing frequency. This report describes analysis of the data, development of the precision statements, and construction of the nomographs. Results of the 720 individual sand-patch tests are given in Table 2.

Components of Variance

The total variation in values among the 720 individual measures of texture depth was examined first to identify the major sources of that variation. Such an analysis, it was thought, would be helpful in developing precision statements and recommending sampling plans because it would identify the relative level of variance associated with differences among the test sections, sampling sites within the same test section, operators, and repeat tests by the same operator. Components of variance were estimated in the usual way.

Because the original experiment was designed to measure differences in a variety of texturing methods chosen because they were known to produce different effects, it was expected that a large component of the variance would be associated with texturing method. This in fact was the case. As shown in Table 3, 44.6 percent of the observed variance in texture depth resulted from differences among the methods used to texture the experimental sections.

A second major source of variance, also expected, was that among individual construction jobs. This component, which was higher than an-

TABLE 1—*Complete factorial experimental design.*

Factor	Level	Description
Texturing methods	4	burlap drag, broom, wire brush, fluted float
Jobs	5	Saratoga, Speculator, Buffalo, Rochester, Patroon Island
Test sections	2	placed on different half-days
Test sites	3	randomly chosen on each test section
Operators	3	same at all test sites
Tests	2	one immediately after the other

TABLE 2—*Mean texture depths, in.* $\times 10^3$.

	Bristle Broom						Fluted Float						Wire Brush						Burlap Drag					
	Run 1			Run 2			Run 1			Run 2			Run 1			Run 2			Run 1			Run 2		
Site	A[a]	B[b]	C[c]	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
											Job 1, Section I													
1	31	25	24	31	24	23	31	31	31	32	28	29	61	50	48	61	53	47	14	12	13	14	14	13
2	30	25	30	31	26	26	63	57	55	63	53	53	50	44	41	48	44	39	24	21	23	23	21	21
3	36	31	30	35	29	28	51	51	44	53	48	43	65	55	57	63	51	53	43	40	36	41	39	33
											Job 1, Section II													
1	44	48	44	47	50	47	26	28	27	27	29	27	55	50	57	55	51	53	16	18	17	16	17	18
2	28	30	30	26	29	31	39	36	39	39	37	37	41	38	44	41	37	41	28	26	26	29	25	28
3	28	32	35	28	31	33	37	36	35	36	34	36	23	21	22	22	22	24	31	29	33	33	28	34
											Job 2, Section I													
1	24	24	23	24	26	22	45	44	44	47	41	47	43	41	38	41	40	37	18	20	16	18	22	16
2	26	28	27	26	28	26	53	47	48	50	50	45	24	25	20	23	23	19	17	18	17	18	18	16
3	28	31	26	26	30	25	51	51	48	51	55	45	36	41	33	37	38	34	19	21	19	19	20	19
											Job 2, Section II													
1	20	25	23	20	25	26	48	50	45	48	50	47	51	50	50	51	51	45	16	18	12	14	17	13
2	28	23	23	29	22	22	48	48	50	51	51	47	28	32	31	28	31	31	14	16	14	14	15	13
3	24	17	19	23	19	18	53	50	51	51	51	47	17	20	16	19	19	16	16	16	13	15	15	14
											Job 3, Section I													
1	26	24	23	26	23	24	36	35	32	36	35	32	41	37	35	39	38	35	11	13	11	12	14	12
2	19	21	16	18	21	17	36	34	31	36	35	33	50	48	41	51	47	43	13	13	12	13	13	12
3	14	15	13	13	15	12	36	37	31	36	36	33	39	38	39	40	38	37	13	16	13	13	16	13

											Job 3, Section II													
1	28	31	23	28	29	24	44	43	47	45	44	41	31	33	31	31	32	31	11	14	10	11	14	10
2	20	25	18	21	25	19	25	29	24	26	29	26	24	26	23	23	26	23	15	18	14	16	17	15
3	16	17	13	16	16	13	51	50	47	51	50	47	37	40	40	40	41	40	8	8	8	8	8	8
											Job 4, Section I													
1	48	47	48	51	47	47	48	45	44	45	47	44	20	22	19	20	21	18	14	11	12	13	11	13
2	24	29	26	24	28	27	55	53	55	51	55	53	38	41	38	38	41	37	18	14	14	16	15	14
3	36	40	39	37	41	39	38	38	38	37	44	37	36	39	35	37	38	36	12	11	10	12	11	11
											Job 4, Section II													
1	28	35	28	29	34	27	48	51	43	48	51	44	35	38	34	32	35	32	16	18	15	16	18	16
2	43	44	37	43	45	39	41	44	38	41	43	39	21	26	19	21	24	19	18	19	16	18	19	16
3	23	28	22	25	27	22	47	50	38	47	47	40	34	38	31	36	39	32	14	16	14	14	15	13
											Job 5, Section I													
1	43	44	53	44	44	51	53	53	55	53	53	53	77	83	83	80	77	83	24	28	24	25	28	25
2	47	45	48	48	45	48	50	53	50	50	55	50	90	77	83	90	77	83	24	28	25	25	27	25
3	43	45	51	44	44	51	65	68	74	65	68	71	63	61	59	61	63	61	39	38	33	40	38	34
											Job 5, Section II													
1	53	50	47	51	51	47	44	48	43	43	45	43	61	61	57	61	55	57	19	21	18	18	22	19
2	40	41	38	38	39	38	47	50	45	48	50	45	50	48	48	48	48	48	21	24	18	22	22	18
3	43	50	44	45	45	43	47	50	48	47	50	47	59	61	57	59	59	53	31	28	31	31	28	30

[a,b,c]Operators.

NOTE—1.0 = 25.4 mm.

TABLE 3—*Contribution of components to total variance.*

Source of Variance[a]	Contribution, %
Texturing method	44.6
Job	19.9
Test site	18.9
Texturing method-job interaction	8.5
Operator	3.3
Section	2.6
Section-operator interaction	0.9
Site-operator interaction	0.8
Testing error	0.4
Job-operator interaction	0.2
Texturing method-operator interaction	0.0
Texturing method-job-operator interaction	0.0
Total	100.1

[a]Listed in order of decreasing magnitude.

ticipated, accounted for 19.9 percent of the total observed variance. The significance of this is that job-to-job differences (arising from such factors as materials, climatic conditions, contractors, inspection, and equipment) account for nearly one half as much variation in texture depth as the differences attributable to the range of textures produced on a single job by the four methods studied.

The third major source of variance was that associated with differences among individual sampling sites: that is, the specific locations chosen to perform the test. This component unexpectedly accounted for 18.9 percent of the total variance—roughly equivalent to that associated with jobs. This observation is very significant from the standpoint of sampling plan design (as will be seen later), and means that variations in texture depth within any particular job can be expected to be roughly equivalent to those between jobs. Taken together, these observations indicate that some physical measure of texture depth should be considered in connection with construction inspection.

The other components were found to be relatively small, accounting collectively for 16.7 percent of the total variance. The largest of those remaining—texturing method-job interaction (8.5 percent)—reflects the fact that textures produced by the different methods were not ordered identically on different jobs. Of importance to the following discussion of precision and sampling is the fact that testing error (0.4 percent), which is the effect of differences in repeat tests by the same operator at the same site, contributes very little to the total variance, and substantially less than that associated with different operators (3.3 percent). Also of importance is the relatively small value of the section contribution (2.6 percent) compared to the site contribution (18.9 percent).

Precision

The "precision" of a measurement process refers to the degree of mutual agreement among individual measurements of the same value. With respect to texture depth measurements in this study, precision is governed by the variance associated with testing and operator in Table 3: that is, the ability of a single operator to reproduce the same test value on different occasions at the same site, and the ability of different operators to reproduce the same test value at different times at the same site. Thus, this section describes estimates of single-operator and multiple-operator precision that have been made from the experimental data. The terms "single-operator" and "multiple-operator" precision as used here are analogous to "single-operator" and "multilaboratory" precision defined in the American Society for Testing and Materials (ASTM) Recommended Practice for Preparing Precision Statements for Test Methods for Construction Materials (C 670-77).

Single-Operator Precision (Repeatability)

Estimates of single-operator precision were made from the ranges of the 360 individual pairs of sand-patch test results. The mean value of these 360 ranges would normally be the best single measure of the ability of individual operators to repeat their own measurements on the variety of textures included in the study. However, in this instance, the range of individual data pairs was found to be positively correlated with its own mean (as shown by the solid line in Fig. 3); that is, as texture depth increased, so did the range. As a result, four different estimates of precision corresponding to four different ranges of texture depth (0 to 20, 21 to 40, 41 to 60, and 61 to 80) were calculated and converted to the corresponding standard deviations—a more useful statistic—by dividing by 1.128 [5]. Standard deviations are shown as a second ordinate in Fig. 3, and also are given in Column 2 of Table 4 as estimates of single-operator precision consisting of single measurements. Single-operator precisions for tests consisting of 2, 3, 5, and 10 repeat measurments by the same operator are given in Columns 3 through 6, respectively, in Table 4. Thus, if one is able to approximate the texture depth anticipated, Table 4 can be used to estimate single-operator precision for any number of repeat measurements up to 10.

The nomograph represented by the solid lines in Fig. 4 was constructed from these same data and can be used to estimate, for a given anticipated texture depth, the number of measurements required to assure a desired single-operator precision.

A measure of single-operator precision is most useful when training new operators in the test procedure. It provides a standard for judging when an operator has attained sufficient skill to reproduce his own work within acceptable limits.

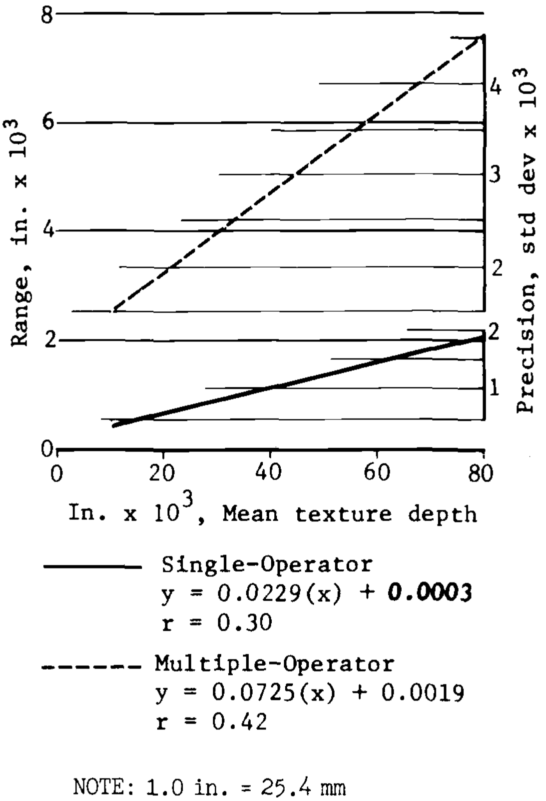


FIG. 3—*Relationship between precision and texture depth.*

Multiple-Operator Precision (Reproducibility)

While single-operator precision is a mesure of one person's ability to reproduce a test result at the same site, multiple-operator precision is a measure of the ability of different operators to reproduce the same test result at the same site. Estimates of multiple-operator precision were calculated in the same general manner as those for single-operator precision, except that they were based on 240 ranges of three tests each by different operators. (Each range consisted of one test by each of three operators at the same site.)

The broken line in Fig. 3 shows the relationship between texture depth and range for these 240 groups of three test results. In this instance, standard deviations were estimated by dividing mean ranges by 1.693 [5]. These values are shown in Table 5 for different numbers of measurements, in a manner similar to Table 4 for single-operator precision.

The broken lines in Fig. 4 can be used to estimate numbers of measurements required for different levels of texture depth and desired precision.

TABLE 4—*Single-operator precision.*[a]

Expected Texture Depth, in. × 10^3	Total Measurements per Test				
	1	2	3	5	10
0 to 20	0.47	0.33	0.27	0.21	0.15
21 to 40	0.87	0.62	0.50	0.39	0.28
41 to 60	1.28	0.91	0.74	0.57	0.40
61 to 80	1.69	1.19	0.97	0.75	0.53

[a]Standard deviation × 10^3 = $\sigma_n = \sigma_1/\sqrt{n}$ (Ref *6*, p. 115), where n = number of measurements per test.
NOTE—1.0 in. = 25.4 mm.

TABLE 5—*Multiple-operator precision.*[a]

Expected Texture Depth, in. × 10^3	Total Measurements per Test				
	1	2	3	5	10
0 to 20	1.53	1.08	0.88	0.68	0.48
21 to 40	2.40	1.70	1.38	1.07	0.48
41 to 60	3.25	2.30	1.88	1.45	1.03
61 to 80	4.11	2.91	2.37	1.84	1.30

[a]Standard deviation × 10^3 = $\sigma_n = \sigma_1/\sqrt{n}$ (Ref *6*, p. 115), where n = number of measurements per test.
NOTE—1.0 in. = 25.4 mm.

The measure of multiple-operator precision is most useful when judging the number of individual tests required to characterize the texture depth at a specific site for a given level of confidence. It is equally applicable whether the same operator or different operators perform the tests.

Sampling Error

To this point, the discussion has dealt only with those sources of variation that might be called testing and operator errors—those inherent in the test itself or associated with the persons performing it. The concept of sampling error deals with the fact that different sites on the same pavement have different depths of texture. It addresses the question of what levels of precision are associated with attempts to characterize the texture depth of a specific area of textured pavement from the mean value of different numbers of tests; or, stated differently, the number of randomly selected sites which need be tested in order to assure a specific degree of confidence in the calculated mean value. Sampling error, as used here, also incorporates elements of both sources of error dealt with previously—the inability of a single operator to

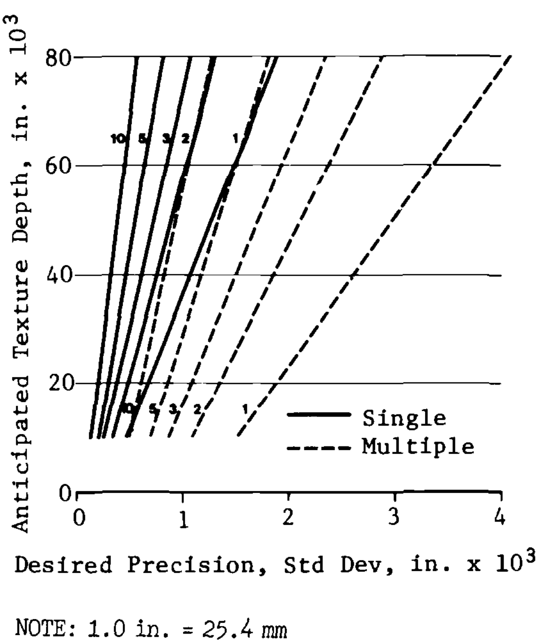
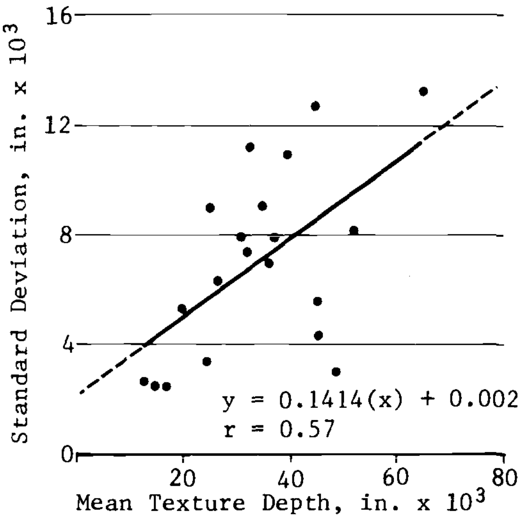


FIG. 4—*Nomograph for estimating numbers of tests for a desired precision.*

repeat his own result, and the inability of different operators to reproduce the results of others.

Referring again to Table 3, the variance associated with site-to-site differences in texture depth (18.9 percent) was far greater than that associated with differences between test sections (2.6 percent), among operators (3.3 percent), or between tests (0.4 percent). Thus, the latter three were considered to be of relative insignificance, and sampling errors were calculated from 20 groups of 36 test results. Each group of 36 included both tests by each of the three operators at each of the three sites on both sections, for each of the 20 combinations of texturing method and job. Figure 5 shows the relationship between texture depth and standard deviation for each group of 36 test results. Typical values of sampling error corresponding to different ranges of texture depth are given in Table 6. The errors associated with different numbers of tests up to 20 are given. As before, a nomograph (Fig. 6) was constructed that permits estimates of the numbers of individual tests required to estimate the mean texture depth of an area of pavement to a predetermined level of confidence.

The measure of sampling error is useful when deciding how many individual tests may be required to estimate the mean texture depth of some unit of pavement—say, a single day's work. It is applicable whether the measurements are made by a single operator or by more than one operator.



NOTE: 1.0 in. = 25.4 mm

FIG. 5—*Relationship between sampling error and texture depth.*

TABLE 6—*Sampling error.*[a]

Expected Texture-Depth, in. × 10^3	Total Measurements per Test						
	1	2	3	5	10	15	20
0 to 20	3.61	2.56	2.09	1.62	1.14	0.93	0.81
21 to 40	6.44	4.56	3.72	2.88	2.04	1.66	1.44
31 to 60	9.27	6.56	5.39	4.15	2.93	2.39	2.07
61 to 80	12.10	8.56	6.98	5.41	3.83	3.12	2.70

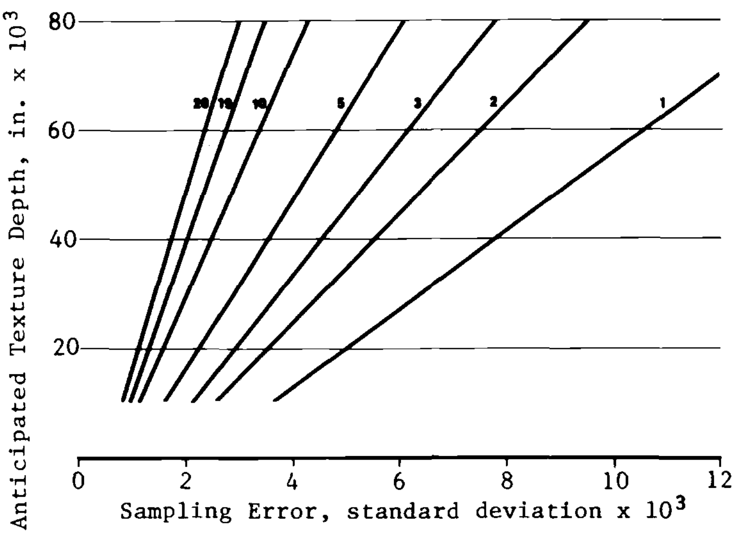
[a]Standard deviation × 10^3 = σ_n = $\sigma_1/\sqrt{n}$ (Ref *6*, p. 115), where n = number of measurements per test.

NOTE—1.0 in. = 25.4 mm.

Level of Precision

The sand-patch test has been criticized as lacking precision [7]. Two thoughts come to mind in this regard.

1. If the variability observed within jobs and among jobs in these experimental pavements is at all typical of what is being obtained generally, the errors assignable to testing appear to be within reason, particularly when multiple tests are employed. The British, for instance, require that the average texture depth obtained from a set of 10 tests be not less than 0.76



NOTE: 1.0 in. = 25.4 mm

FIG. 6—*Nomograph for estimating number of tests for a desired sampling error.*

mm (0.030 in.), and that not more than one of the 10 results be less than 0.64 mm (0.025 in.) [*8*]. This frequency of testing would correspond to a combined sampling and testing error of $\sigma = 0.05$ mm (2.04×10^{-3} in.) for a texture depth of approximately 0.76 mm (0.030 in.) (Table 6), for which the testing error component would account for only 0.01 mm (0.28×10^{-3} in.) (Table 4).

2. If lack of precision is a concern, this may be remedied at least in part by increasing the quantity of sand used in the test. Weller and Maynard [*8*] found that increasing the quantity of sand from 5 to 25 cm^3 (0.30 to 1.52 $in.^3$) reduced the testing error by 40 percent. The British method [*8*] now requires 25 cm^3 (1.52 $in.^3$) of sand, while American practice calls for 1.50 $in.^3$ (24.6 cm^3).

Summary

The components of variance were analyzed for 720 measures of concrete pavement texture depth obtained by the sand-patch method. The measurements were made in connection with a complete factorial field experiment in which four texturing methods were used on two sections each of five different paving jobs. Each pavement section was tested at three different sites by three different operators performing two tests each.

As expected, the variance assigned to differences among the four texturing methods was the largest single component, accounting for 44.6 percent of the total. Unexpected was the large variance assigned to job-to-job differences where texturing was done by the same method (19.9 percent), and to differences within the same job where texturing was done by the same method (the section and site components together, 21.5 percent). Collectively, this "within-texture" variance was roughly equivalent to that between textures, and suggests the need for positive control during construction to assure that desired minimum texture depths and greater uniformity are attained.

The analysis permitted estimates of single-operator precision (repeatability) and multiple-operator precision (reproducibility) of the sand-patch test. These measures are useful in training operators and in judging the number of individual tests required to characterize texture depth at a given site for a given level of confidence. Both measures were found to be correlated positively with the level of texture depth anticipated. It is noted that the test's precision probably could be improved by increasing the quantity of sand used.

The analysis also permitted an estimate of the error likely to be associated with sampling a given area of pavement for its mean texture depth. This value also was found to be correlated positively with texture depth, and is useful in designing sampling plans for construction control or acceptance.

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Macrotexture and Drainage Measurements on a Variety of Concrete and Asphalt Surfaces

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ABSTRACT: As part of a major study to develop methods for predicting tire friction performance on all types of pavements, macrotexture measurements were taken on a variety of concrete and asphalt surfaces using several different volumetric and drainage techniques. Expressions are developed which relate data obtained with each technique. Factors influencing these measurements, including operator technique and type of equipment, are identified. Comparisons also are given with skid resistance values obtained using a British portable pendulum tester. Outflowmeter measurements are presented to show the effect of surface finishes and treatments on drainage characteristics. The need to measure other surface texture parameters, such as microtexture, is suggested from comparative tire friction and surface macrotexture data obtained on two different wet surfaces. The paper concludes with comments relative to the necessity for additional studies to evaluate different surface microtexture measurement techniques in an effort to provide sufficient information to enable researchers to predict tire-pavement friction performance.

KEY WORDS: surface, properties, texture, pavements, correlation, friction, pavement surface characteristics

It long has been recognized that the texture characteristics of a pavement surface directly influence the friction forces which pneumatic tires can develop for accelerating, steering, and braking performance of vehicles. Many different devices and techniques [*1-3*][3] have been developed to provide quantitative measurements of surface texture, and these efforts have identified two texture classifications, namely, micro- and macrotexture. In

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general, microtexture consists of the fine, small-scale, surface features such as those found on individual stone particles, whereas macrotexture encompasses the coarse, large-scale roughness of a pavement surface-aggregate matrix. Results from studies [*4–7*] to evaluate the effects of speed on tire friction have indicated that the slope of the friction-speed gradient curve is primarily a function of the surface macrotexture, and the magnitude of the friction at a given speed is related to the surface microtexture. On that basis, it would appear that an assessment of both surface micro- and macrotexture characteristics is necessary to relate texture measurements with tire frictional performance.

Several different volumetric and drainage measurement techniques for classifying surface texture were evaluated as part of a major study to develop and improve methods for predicting tire friction performance on all types of pavements. The objective of the evaluation was to seek relationships between the results from the different techniques, and to relate those results to surface frictional characteristics obtained using the ASTM Standard Method for Measuring Surface Frictional Properties Using the British Pendulum Tester [E 303-74(1978)]. This paper develops these relationships where possible and discusses the factors which influence them. The paper concludes with comments relative to future efforts to provide researchers with reliable, accurate, and sufficient surface texture information.

Equipment and Test Procedures

Comparative measurements were collected in this study with six different pavement classification techniques which included three volumetric types, two drainage devices, and a skid resistance tester. Recommended calibration procedures and surface preparations were followed during the operation of each technique on fifteen different concrete and asphalt surfaces located at Langley Research Center and Wallops Flight Center of the National Aeronautics and Space Administration (NASA). For each technique, a stiff, wire-bristle brush was applied vigorously to the surface test area prior to measuring, to remove all loose stones, debris, and other contaminants, and a minimum of six measurements were taken at different locations on a given surface. Data were collected with all six techniques at the same test areas of a given surface. For each technique, an average value was calculated from the individual measurements taken on each test surface. These average values are listed in Table 1. A description of the different equipment and test procedures used for the various techniques follows.

Volumetric Methods

The volumetric methods involve spreading a known volume of a given material on the pavement surface to fill all voids, measuring the area

TABLE 1—*Compilation of surface texture measurement data obtained using different techniques.*

Test Surface			Texture Depth Measurement Technique						Outflowmeter				Pendulum Tester, BPN
			Grease		Sand		Putty		ISETH		FHWA		
Number	Type	Location	mm	in.	mm	in.	mm	in.	sec	l/sec	sec	l/sec	
1	concrete	NASA Langley	0.12	0.005	0.30	0.012	0.51	0.020	20.45	0.012	85.53	0.008	63
2	concrete	NASA Langley	0.10	0.004	0.24	0.009	0.51	0.020	34.64	0.007	157.65	0.005	65
3	concrete	NASA Langley	0.35	0.014	0.73	0.029	0.86	0.034	3.41	0.072	14.90	0.049	76
4	concrete	NASA Langley	0.21	0.008	0.52	0.021	0.81	0.032	4.91	0.050	18.75	0.039	65
5	asphalt	NASA Langley	0.55	0.022	1.06	0.042	1.22	0.048	1.34	0.183	7.50	0.097	66
6	asphalt	NASA Langley	0.28	0.011	0.43	0.017	0.38	0.015	12.17	0.020	83.53	0.009	63
7	asphalt	NASA Langley	0.58	0.023	1.93	0.076	2.24	0.088	0.53	0.462	1.77	0.409	53
8	asphalt	NASA Langley	0.76	0.030	1.28	0.050	1.30	0.051	1.18	0.208	4.79	0.151	73
9	asphalt	NASA Langley	0.59	0.023	1.03	0.041	0.97	0.038	1.48	0.166	6.60	0.110	67
10	concrete	NASA Wallops	0.24	0.009	0.65	0.026	0.81	0.032	4.07	0.060	13.20	0.055	55
11	concrete	NASA Wallops	0.13	0.005	0.36	0.014	0.30	0.012	18.10	0.014	84.63	0.009	61
12	concrete	NASA Wallops	0.36	0.014	0.78	0.031	0.89	0.035	...	...	...	...	56
13	asphalt	NASA Wallops	0.47	0.019	0.92	0.036	1.40	0.055	2.07	0.118	7.13	0.102	72
14	asphalt	NASA Wallops	0.27	0.011	0.48	0.019	0.69	0.027	4.02	0.058	20.83	0.035	63
15	asphalt	NASA Wallops	0.39	0.015	0.71	0.028	0.71	0.028	3.03	0.081	15.40	0.047	62

covered, and computing values of average texture depth by dividing the material volume by the spread area. Three materials were employed, and the equipment used for each is shown in Fig. 1. The dissimilar material properties required different measurement procedures, as well as different equipment.

The silicone putty method [8], which was developed by researchers at Texas Transportation Institute, uses 15.9 g (0.56 oz) of putty and a flat acrylic plastic disc (see Fig. 1) which has a recess, 10.2 cm (4 in.) in diameter by 0.16 cm (0.06 in.) deep, machined on one side. Prior to each measurement, the surface is wetted with a solution of water and liquid dishwashing detergent to minimize adherence of the putty to the surface. The putty specimen is formed into an approximate sphere, and placed on the wetted pavement surface. The recess side of the plastic disc is then centered over the putty and firmly pressed down to contact the surface. A weight of 22.7 kg (50 lb) is next placed on top of the plastic disc for approximately 1 min. The weight is then removed, and the average diameter of the deformed putty is recorded. The texture depth is defined by the diameter of the deformed putty: the larger the diameter, the smaller the texture depth. When tested on a flat,

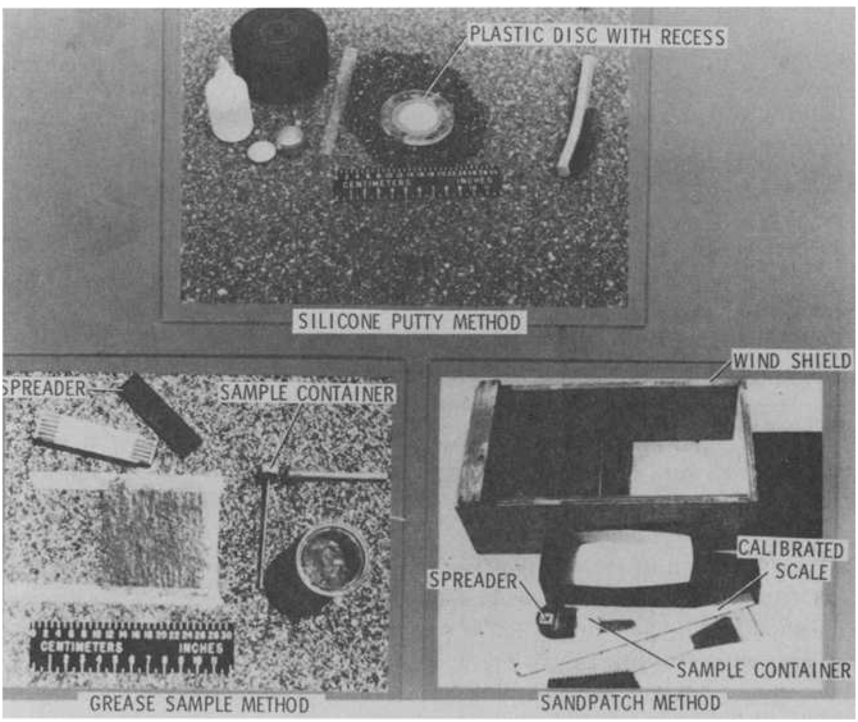

FIG. 1—*Surface texture depth measurement techniques.*

smooth surface with negligible texture, the putty specimen will completely fill the recess. The following equations are used to calculate surface texture depth values from the silicone putty measurement method. For SI Units

$$T_p = \frac{16.3871}{D^2} - 0.1588$$

For U.S. Customary Units

$$T_p = \frac{1}{D^2} - 0.0625$$

where

T_p = texture depth, cm (in.) and
D = average putty circle diameter, cm (in.).

The grease specimen technique [*4*] was developed at the NASA Langley Research Center, and the equipment used (see Fig. 1) includes a grease tube of known volume, a tight-fitting cork plunger used to expel the grease from the tube, and an aluminum squeegee, faced with rubber, used to work the grease into the pavement surface voids. The grease is a general-purpose lubricant, and a minimum amount of 7.5 cm^3 (0.5 $in.^3$) is applied during each measurement. The grease specimen generally is deposited between two parallel lines of masking tape placed on the surface to assist in defining the area covered. In spreading the grease with the rubber squeegee, care must be taken to assure that no appreciable amount of grease is left on the masking tape or on the squeegee. A measurement is made of the pavement surface area covered by the grease, and this is recorded together with the grease specimen volume. The average texture depth of the surface is obtained by dividing the volume of grease used by the area covered.

The sandpatch method [*9*] was developed at the British Road Research Laboratory, and is one of the first methods used to determine surface texture. Some previous surface texture evaluations [*3,10,11*] have indicated poor repeatability in measurements obtained when using the initial sandpatch method, and various modifications have been made since to improve reliability of results. The technique used in this study, however, followed the initial procedure of pouring a known volume of fine, dry, sand on the pavement, spreading it over a circular area, and, using a hard round puck similar to that used in ice hockey, leveling the sand with the tips of the asperities (peaks). A small, open-ended, wood frame (see Fig. 1) provided the test surface area with protection from the wind during each sandpatch application. The average surface texture depth is obtained by dividing the volume of sand used by the area covered. In this investigation, 25 cm^3 (1.5 $in.^3$) of quartz sand with a particle grain size of 0.09 to 0.20 mm (0.003 to 0.01 in.) was used, and a calibrated scale was available which indicated texture depth values directly from the sandpatch diameter.

Drainage Methods

The basic static water drainage meter, commonly referred to as an "outflowmeter" [*1,3,11,12*], consists of a transparent plastic cylinder to contain water and a brass base plate with a rubber ring attached to the bottom face. The cylinder is placed on the pavement and loaded so that the rubber ring will contact the surface aggregate particles in a way similar to that expected of tire tread elements. Water is poured into the cylinder, and a clock measures the time required for a known volume of water (usually indicated by two marks on the side of the cylinder) to escape through the pores or channels (grooves) in the pavement and between the rubber ring and pavement surface. Water in the cylinder is under atmospheric pressure. Short durations of time or high rates of water flow are associated with pavement surfaces having high macrotexture, high permeability characteristics, or both.

Two types of static outflowmeters, shown in Fig. 2, were used in this study to relate drainage times and flow rates between the two different outflowmeters, as well as to establish their relationship to texture depth measurements. The Federal Highway Administration (FHWA) provided one outflowmeter equipped with a rubber plug to prevent water from escaping the cylinder during filling, and an electronic timer actuated by metal probes placed inside the cylinder. The timer proved to be unreliable, which necessitated use of a handheld stopwatch. The second outflowmeter, furnished by the Institute for Highways, Railroads, and Rocks Engineering (ISETH), Zurich, Switzerland, had neither a water discharge plug nor an electronic timer. Hence, a stopwatch was used for both meters to obtain water drainage times, and for comparison purposes, the test surface was wetted prior to obtaining measurements. Other differences noted between the FHWA and ISETH outflowmeters are given in the following table.

Parameter	FHWA	ISETH
Empty weight, kg (lb)	6.7 (14.8)	3.5 (7.7)
Cylinder inside diameter, mm (in.)	95 (3.75)	57 (2.2)
Timed discharged water volume, cm^3 ($in.^3$)	724 (44.16)	245 (14.97)
Timed discharged water amount, litres (gal.)	0.724 (0.191)	0.245 (0.065)
Discharged water opening dia, mm (in.)	51 (2.0)	57 (2.2)
Base plate rubber ring	...	...
Thickness, mm (in.)	6 (0.25)	5 (0.2)
Width, mm (in.)	20 (0.78)	5 (0.2)
Area, mm^2 ($in.^2$)	4791 (7.43)	974 (1.51)
Effective loading, kg (lb)	7.6 (16.71)	3.5 (7.7)
Bearing pressure, kPa (lb/in^2)	15.5 (2.25)	35.2 (5.11)

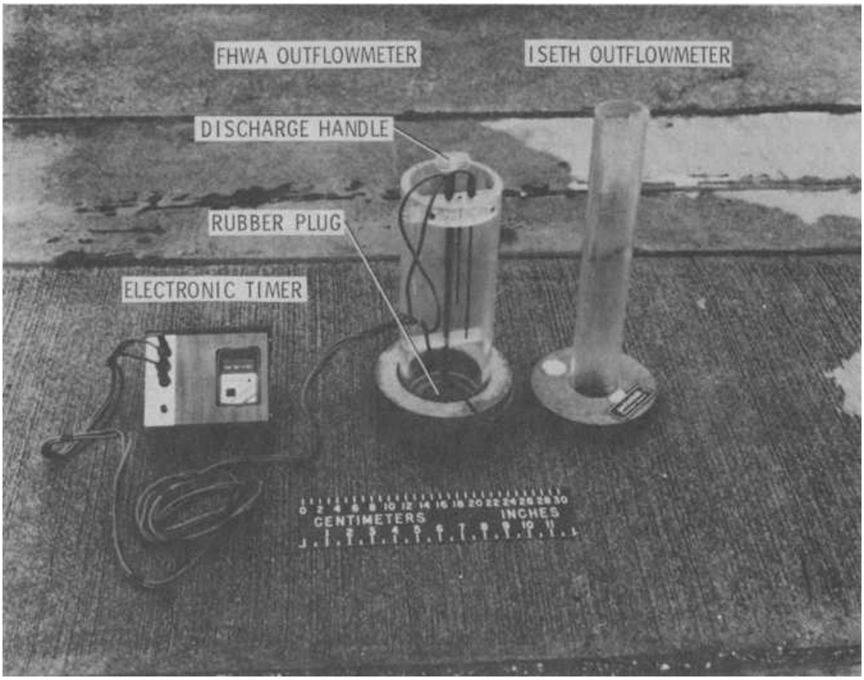

FIG. 2—*Surface drainage measurement equipment.*

Despite the larger volume and greater loading, the effective bearing pressure for the FHWA outflowmeter is less than half that imposed on the pavement surface by the ISETH outflowmeter, because of the large difference in rubber ring areas. The average values of water drainage time and flow rate obtained from measurements with these outflowmeters are given in Table 1 for each test surface.

British Pendulum Tester

The British pendulum skid resistance tester [*13,14*] shown in Fig. 3 is a dynamic pendulum impact-type tester used to measure the energy lost when a rubber slider edge is propelled over a test surface. The British Pendulum (Tester) Number (BPN) values measured for flat surfaces represent the frictional properties obtained with the apparatus, using recommended procedures. Since pavement surface temperature measurements, taken during all tests, varied from 15 to 40°C (59 to 104°F), the average BPN values on each test surface were temperature-corrected by a nonlinear factor established during a British Road Research Laboratory study. This temperature correction factor varied from zero BPN at a surface temperature of 20°C

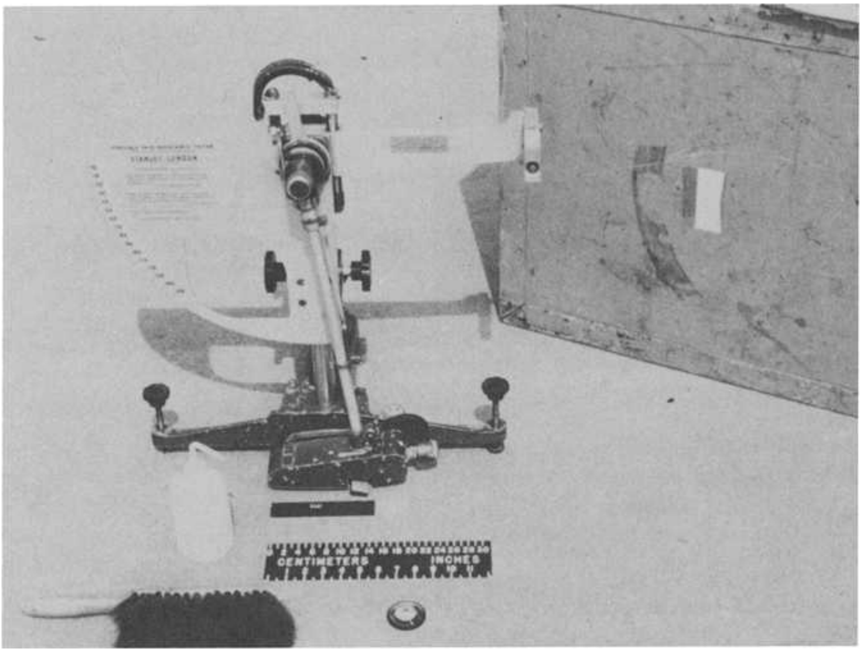

FIG. 3—*British portable pendulum skid resistance tester.*

(68°F), to −1.45 BPN at 15°C (59°F), and +3.0 BPN at 40°C (104°F). A compilation of the average, temperature-corrected, BPN values obtained on each test surface is given in Table 1.

Results and Discussion

With the exception of the British pendulum skid resistance numbers obtained in this study, a linear regression analysis was performed on each data set to determine the equation of best agreement in a least squares fashion. Figure 4 shows the relationships between the grease, putty, and sand techniques on the basis of average texture depth (ATD) values. The figure shows that the relationships between the results obtained using the different techniques varied, depending upon the techniques being compared. A well-established relationship is noted between the grease specimen and sandpatch data, as verified by the highest correlation coefficient r of 0.95. On the other hand, the relationship between the grease specimen and putty specimen data is not well established, with a correlation coefficient of only 0.64. The ATD values obtained using the sandpatch and putty techniques are shown in Fig. 4 and noted in the table to be roughly twice those obtained on the same surface using the grease specimen technique. One possible explanation for this

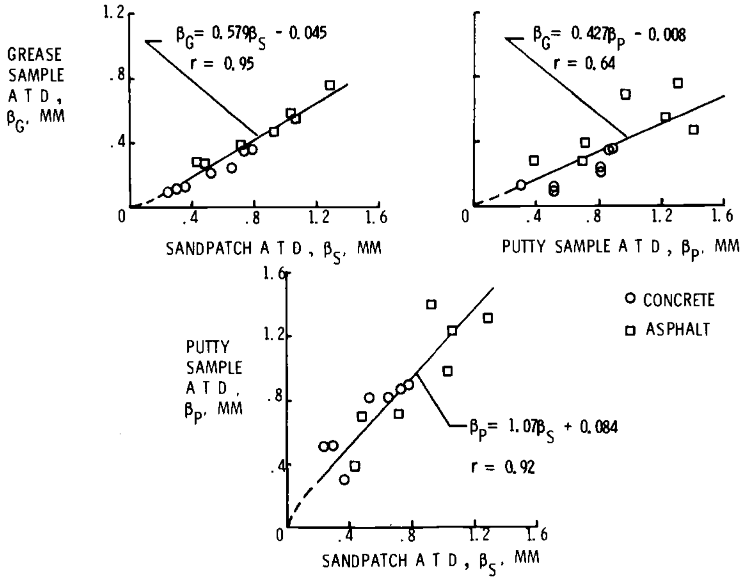

FIG. 4—*Relationships between surface texture depth measurement techniques.*

difference is that in using the rubber squeegee to spread the grease on the surface, the force exerted by the operator is more likely to fill the surface voids with grease and wipe the asperities (peaks) nearly clean. The measurement procedures followed in the sandpatch and putty methods, combined with the type of measurement material used, do not produce the same effects from surface voids and asperities in the area covered and hence, higher ATD values are obtained.

The relationships established between the two outflowmeters on the basis of water drainage time and drainage rate are shown in Fig. 5. The FHWA outflowmeter drainage time is approximately four times that demonstrated by the ISETH outflowmeter on the same surface, as a result of the larger water volume in the former. Outflowmeter drainage rates normalize this difference in drainage time; however, agreement between the two outflowmeters is only fair because of the differences in the base plate rubber ring effective bearing pressure.

Figures 6 and 7 give comparisons between BPN's obtained with the portable skid resistance tester, the three texture depth measurement techniques (Fig. 6), and the two outflowmeter drainage techniques (Fig. 7). The fifteen different surfaces evaluated in this study did not provide a significant range in skid resistance or BPN values (for example, 53 to 76) despite a fairly sizable range in measured texture depths. As a result of this relatively narrow BPN data band, relationship equations were not derived for these data sets.

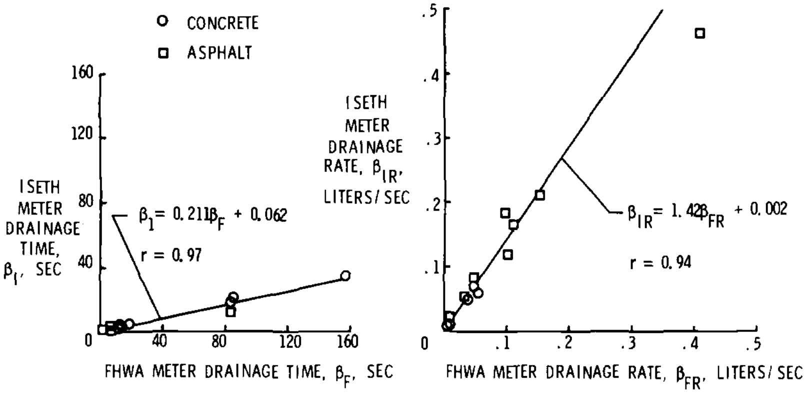

FIG. 5—*Relationships between surface outflowmeter drainage measurements.*

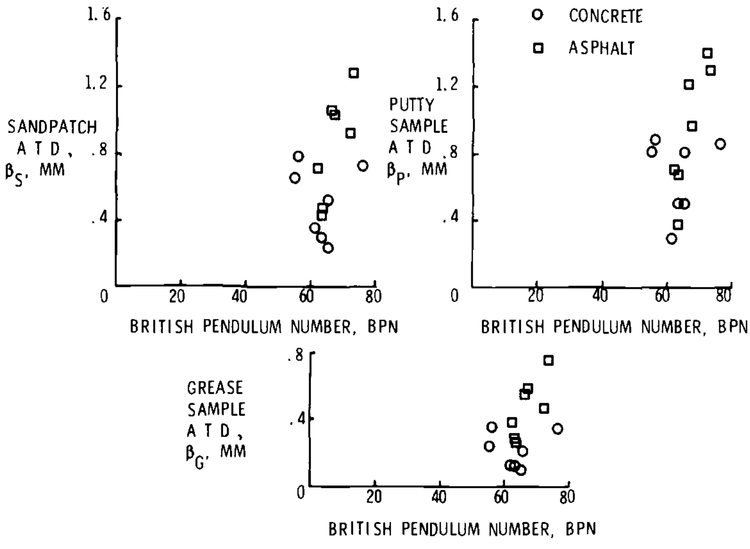

FIG. 6—*Comparison of British pendulum tester numbers and texture depth measurements.*

It is suspected that surface microtexture has a greater influence on the British pendulum skid resistance tester data than does the surface macrotexture, which influences the texture depth and drainage data.

The relationship between the surface texture depth data and the drainage time data from the outflowmeters is presented in Figs. 8 and 9 in log-log form. The figures shows that in general, the outflowmeter drainage times increase with decreasing surface macrotexture depth values. The grease specimen data exhibited the greatest sensitivity (slope), but the sandpatch

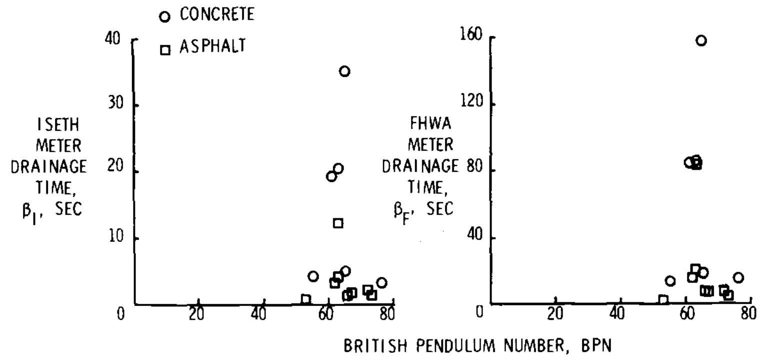

FIG. 7—*Comparison of British pendulum tester numbers and outflowmeter drainage measurements.*

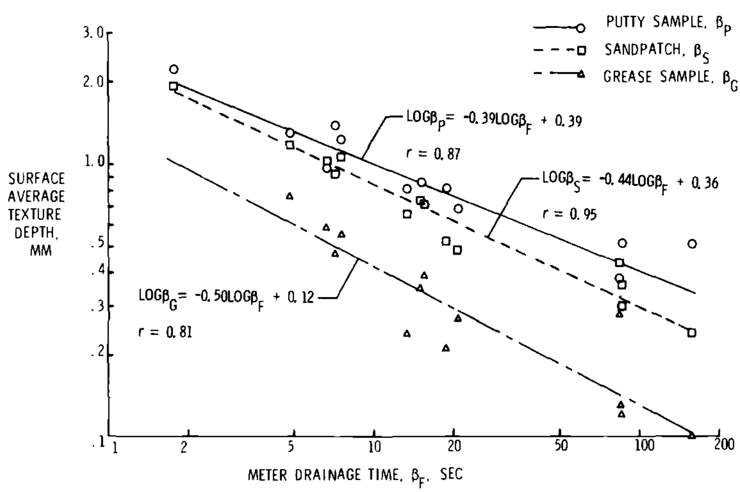

FIG. 8—*Relationships between surface texture depth and outflowmeter drainage measurements, FHWA outflowmeter.*

data exhibited the best relationship with the drainage times. Although the linear regression curves for the data sets have nearly similar negative slopes for both outflowmeters, the y-intercept terms differ because one data set obtained on asphalt Test Surface 6 (see Table 1) was omitted from the ISETH outflowmeter drainage data shown in Figure 9.

The effect of various pavement surface finishes and treatments on surface macrotexture is indicated in Fig. 10 by the wide variation in FHWA outflowmeter water drainage times. The drainage measurements for this figure were taken on a canvas belt-finished concrete runway which was constructed level, both longitudinally and transversely, at NASA Wallops Flight

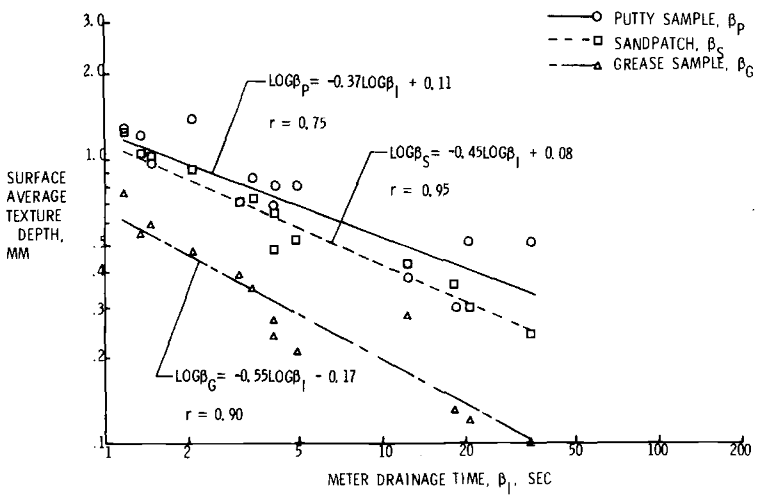

FIG. 9—*Relationships between surface texture depth and outflowmeter drainage measurements, ISETH outflowmeter.*

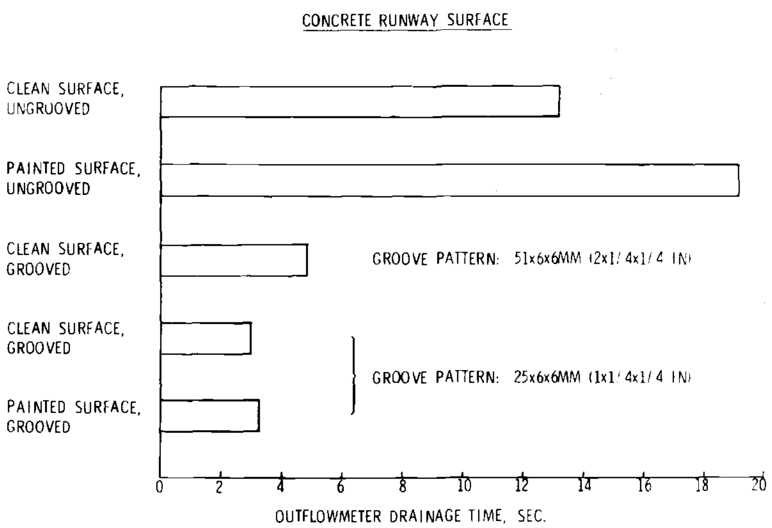

FIG. 10—*Effect of surface treatments on outflowmeter drainage measurements.*

Center. The paint markings on the runway centerline significantly reduced the ungrooved surface macrotexture as indicated in Fig. 10 by the relatively long drainage time (19 versus 13 s). Figure 10 also shows that the saw-cut grooving greatly improved the surface drainage rates. The outflowmeter average drainage time measured on the 51-mm (2-in.) spaced groove pattern was approximately twice as long as that measured on the 22-mm (1-in.) spaced groove pattern. The drainage time differences shown between the two groove patterns, which differed only in groove spacing, may be due partially to the random placement of the outflowmeter with respect to the surface grooves, since the water discharge opening is only 51 mm (2 in.) in diameter.

Some recently obtained tire friction and surface macrotexture values measured on concrete and asphalt surfaces located at a test track in San Angelo, Texas suggest that additional surface texture information, such as microtexture, is required to adequately define tire frictional performance. Figure 11 shows the effect of speed and pavement surface on the locked wheel friction coefficient μ_{skid} of both test tires [in accordance with the ASTM Specification for Standard Tire for Pavement Skid Resistance Tests (E 501-76) and ASTM Specification for Smooth Tread Standard Tire for Special-Purpose Pavement Skid Resistance Tests (E 524-76)] installed on a skid trailer [in accordance with the ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79)] equipped with an on-board wetting system adjusted to provide a surface water depth of 0.51 mm (0.02 in.). The data in the figure are numerical averages of several μ_{skid} values obtained at each of six speed increments up to 97 km/h (60 mph). In general, the locked-wheel friction developed by both tires on the two wet surfaces decreased with increasing speed as expected, but the higher friction levels developed on the asphalt surface are contrary to previously noted [*4*] trends of higher friction being associated with higher surface texture depths. (Average surface macrotexture depth values obtained using the silicone putty specimen technique indicated that the asphalt surface had considerably less macrotexture than the concrete surface.) Apparently, surface microtexture characteristics as well as aggregate shape and surface finish treatment must contribute

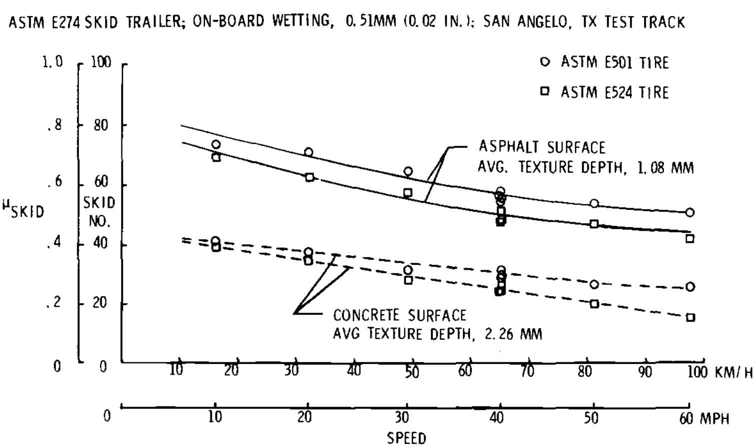

FIG. 11—*Tire friction performance on two different wet surfaces.*

significantly to the magnitude of friction forces developed between tires and wet surfaces. Additional studies of several promising microtexture measuring techniques, including use of a profilometer, are planned.

Summary

Several different volumetric and drainage measurement techniques for classifying surface macrotexture were evaluated on 15 concrete and asphalt surfaces. Equations were derived, using linear regression analysis, to indicate the relationship between data obtained using three macrotexture depth-measuring techniques and two outflowmeter drainage devices. Comparisons also were made with skid resistance values using a British pendulum tester, but due to the relatively small range of these measurements, no relationship could be established with data from other measurement techniques. The outflowmeter was shown to provide an indication of the effect of various pavement surface finishes and treatments on surface drainage characteristics. The need to define other surface texture parameters, such as microtexture, was implicated from comparative tire friction and surface macrotexture data obtained on two different wet surfaces. Further investigations will include an evaluation of different surface microtexture measurement techniques and the development of new, improved devices for providing sufficient surface texture information to enable researchers to predict tire-pavement frictional performance.

Acknowledgments

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Surface Texture Classification: A Guide to Pavement Skid Resistance

REFERENCE: Holt, F. B. and Musgrove, G. R., "**Surface Texture Classification: A Guide to Pavement Skid Resistance,**" *Pavement Surface Characteristics and Materials, ASTM STP 763,* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 31–44.

ABSTRACT: The ability of a driver to stop on a wet pavement surface has been shown as a relationship between a tire and the pavement surface texture. For a number of years, the Ontario Ministry of Transportation and Communications has been classifying pavement surface textures and correlating these textures with the skid number of the surface, as measured by a brake-force trailer. The skid trailer number can be explained now in terms of the pavement surface texture. This paper contains a summary of the test procedure, a discussion of the texture classification system, and the relationship of the six texture parameters is discussed in terms of the equation used to generate a skid number at 100 km/h (62 mph). The advantages of using this system as a complement to present skid testing procedures are outlined, with particular attention to its use in those areas which cannot be tested by the brake-force trailer and in the area of mix design studies. The paper concludes with a discussion of some potential uses for the test method in future research work in Ontario, including the semiautomation of the interpretation procedure.

KEY WORDS: pavement skid resistance, skid number, surface texture, surface profile, texture parameters, photo-interpretation (semiautomated), skid trailer correlation, computer graphics, skid numbers (SN), instrumentation, stereo photography, pavement surface characteristics

A number of texture evaluation methods have been developed over the years. These include the sand patch test, the grease smear test, laser profilometers, etc. Each of these has been used strictly as a texture evaluation device, or as a measure of a particular profile characteristic, such as macrotexture.

During the past few years, the Ontario Ministry of Transportation and Communications has developed a photo-interpretation technique not only to

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analyze and classify the pavement surface texture, but also to generate a skid number that correlates to the skid number (SN) in the ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79). This method was developed by R. Schonfeld [*1,2*][2] and forms the basis for the new ASTM Standard Method for Classifying Pavement Surface Textures (E 770-80).

The reasons for a certain level of skid resistance now can be explained and evaluated using this new technique. The generation of skid characteristics has been semiautomated under an investigation project by the Ministry with Carl Zeiss (Canada) Ltd. Using a Zeiss Stereocord G-2, which offers a stereoscopic viewing/measuring system coupled to a microprocessor for data manipulation, the photo-interpretation system has the flexibility to produce not only texture parameters and skid numbers, but also visual and graphic profiles of the pavement surface.

This paper will look at the test method and its texture classification system, the uses of the method, and the potential uses in the future as a production line tool using the semiautomated system.

Method

The test method involves the analysis of a series of stereo photographs taken on a section of highway. The photos are analyzed and the surface texture is classified in terms of six texture parameters: *A* is the height of the macroprojections; *B* is the width of the macroprojections; *C* is the angularity of the macroprojections; *D* is the density of the distribution of the macroprojections; *E* is the harshness of the macroprojections' surfaces; and *F* is the harshness of the matrix surface [*1-3*].

Figure 1 shows a pavement surface profile, with the various parameters noted. The parameters developed from the photo-interpretation analysis are fed into a computer program that generates skid numbers at 50 and 100 km/h (31 and 62 mph).

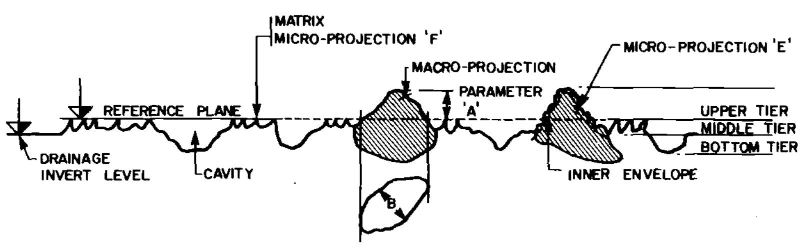

FIG. 1—*Pavement surface profile.*

[2]The italic numbers in brackets refer to the list of references appended to this paper.

Sampling and Camera Equipment

The pavement is divided into a number of test sectors that are representative of the performance of that section of highway in terms of age, alignment, structural condition, and wear consistency [3]. The number of test sections also will vary according to the nature of the testing program. Using 300 m (980 ft) as a minimum length, the test section is photographed in five equally spaced locations. Statistically, this number of photographic specimens gives an acceptable specimen of the pavement texture. The photographs are taken in the outer wheelpath of the lane, as this wheelpath generally exhibits the worst conditions of wear and polishing. It should be noted that for various reasons, including the nature of the test program, safety, and economics, the number and locations of the photographs may change. However, the above procedure can be used as a guideline for textural evaluations under most circumstances.

A 35-mm single lens reflex camera, mounted on a specially designed camera box (Fig. 2) is used to take the stereo photographs. The camera is equipped with a macrolens to maximize the definition of the pavements's surface texture.

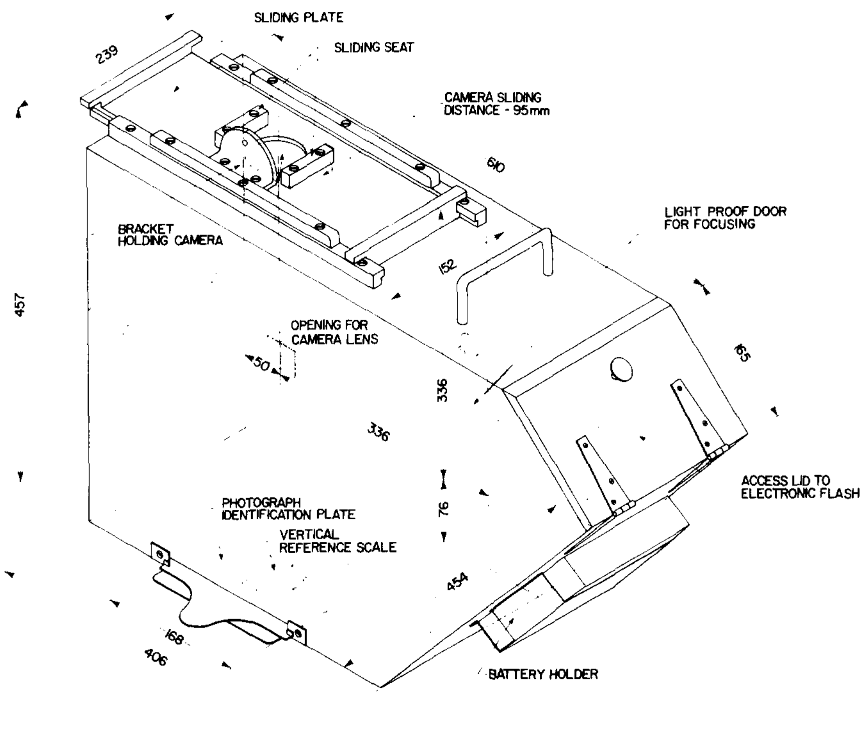

FIG. 2—*Camera box specifications.*

The camera box is designed to ensure the photos have a constant scale and a constant vertical height. A consistent 60 percent stereo overlap is acheived by mounting the camera on a sliding plate on top of the camera box (Fig. 3).

A flash unit mounted inside the box illuminates the pavement at a 45-deg angle. This angle minimizes the shadows in the photos and ensures an even illumination of the specimen area.

Low-speed, fine-grained, ASA 25 color slide film is used to maximize photo definition for the interpreter and minimize grain in the images, which are interpreted under varying magnifications, from ×1 to ×25.

Analysis Procedure

Mounted in the camera box is a reference plate (Fig. 4). This plate is visible in both of the stereo photos. The plate contains reference scales for both horizontal and vertical measurement. The color slides, when viewed stereoscopically, have a three-dimensional surface which allows the interpreter to measure and classify the six texture parameters.

The interpreter uses an instrument such as the Bausch and Lomb Zoom 240 Stereoscope (Fig. 5) to interpret the slides. The left slide of the stereo pair is overlayed with a transparent grid, marked off in 1-cm (25/64 in.) squares. Ten of the squares are selected at random, and the surface texture is analyzed and coded. Table 1 shows the coding breakdown used by the inter-

FIG. 3—*Camera being mounted on sliding plate.*

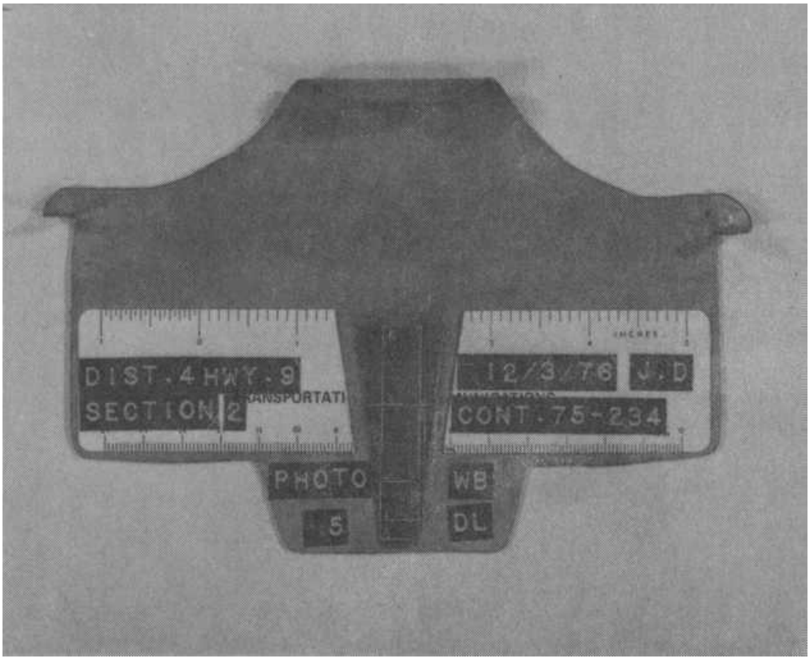

FIG. 4—*Reference plate with horizontal and vertical reference scales.*

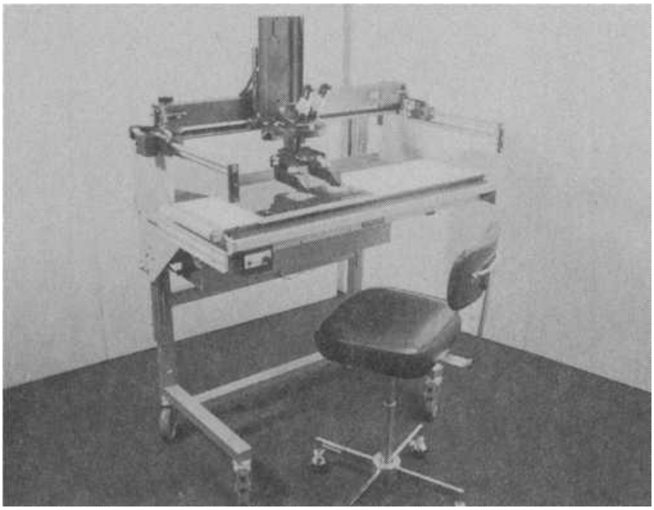

FIG. 5—*Bausch and Lomb Zoom 240 stereoscope.*

TABLE 1—*Surface texture classification.*

	Macrotexture of Projections				Microtexture	
Parameter Code	Approximate Height A, mm	Approximate Width B, mm	Angularity C	Density of Distribution of Projections as a Percentage of Total Area D	Projections Harshness E	Matrix Harshness F
0	0	16	...	...[b]		cavity in Surface of Matrix
1	0.25	8	round	...	polished (no texture visible)	
2	0.50	4	subangular	...	smooth (texture visible but microprojections too small for visual estimate of height)	
3	1	2	angular	...	fine-grained (height of microprojections less than 0.25 mm and less than one half of their width	
4	2	...[a]	...	...	fine-grained (microprojections approximately 0.25 mm and less than 1.0 mm high)	
5	4	...	...	...	coarse-grained subangular (microprojections approximately 0.5 mm high or more)	
6	8	...	...	...	coarse-grained angular (microprojections approximately 0.5 mm high or more)	

[a]Particles less than 2 mm (5/64 in.) wide are regarded as microtexture.
[b]Enter actual percentage of total area.

preter. Each parameter is analyzed and coded such that the interpreter has ten sets of six parameters for the slide.

The interpreter must refer to the reference plate and its reference scales to establish the various parameters. The angularity and harshness parameters are interpreted on the basis of the descriptions in Table 1 and the photo-interpreter's manual [*3*].

Once the ten squares have been interpreted, the data are fed into a computer program which generates a skid number at 50 and 100 km/h (31 and 62 mph) for each square and for the stereo pair as an entity. The program was developed by testing 1500 stereo pairs of various pavement textures. By trial-and-error techniques, the six parameters were weighted to a best-fit correlation with the ASTM Test E 274-79 skid number.

Recently, this program was revised to widen the statistical base, and an equation was developed to represent better the impact of each parameter on the skid numbers as generated by the photo-interpretation technique. The equation was developed by multiple regression techniques using the same six texture parameters, and certain combinations of parameters. The skid numbers at 100 km/h (62 mph) can be equated as follows

$$\begin{aligned} \text{SN } 100\,\text{km/h} = {} & -0.67 + 10.33(A) - 0.33(B) - 1.13(D) \\ & + 5.10(C \text{ and } E) + 2.55\,(F) - 3.30(A/BD) \end{aligned}$$

With a limited data base of 500 stereo pairs, the coefficient of correlation (r) is 0.918 with a standard error of estimate of 2.2. With the purchase of a new ASTM Test E 274-79 trailer, work will be carried out to expand the data base and improve the equation. It is expected that with a wider data base, the error of estimate will decrease, and the correlation coefficient will improve.

This equation accounts for a number of factors that the six single parameters do not. The C and E factor accounts for the polishing and wear characteristics of the macroprojections by combining the effect of the projections' angularity with the harshness of the projections' surface.

The A/BD factor accounts for the bulk water drainage capability of the pavement surface. The six texture parameters, in themselves, quantify the abrasive characteristics of the pavement.

Precision

Using this method to generate skid numbers, an experienced interpreter can expect to have a correlation between the ASTM Test E 274-79 skid number and the photo-interpreted skid number in the area of 0.96, with a standard error of estimate of 1.6. These statistics are based on Ontario's experience for over 1500 stereo pairs of varying asphalt and concrete pavements having random textures.

The interpreter can achieve this level of accuracy and repeatability with proper training and continued use. To this end, a user's manual [*3*] and an appendix to assist as a reference have been produced. The appendix offers both black-and-white enlargements and color slides by which the interpreter can gauge his performance. These photos also can act as an interpretation data base to upgrade and maintain the interpreter's work.

Benefits of the Method

The complexity of the test method is offset by a number of important assets. While generating a skid number that correlates well with the ASTM Test E 274-79 skid number, the texture parameters explain the reasons for the magnitude of the number. This explanation is important when deciding the type or need for a rehabilitation strategy. The parameters can indicate the need for a short-term strategy such as scabbling or burning, or indicate that the section of highway must be resurfaced in the near future.

The method can be used to establish the skid resistance life of various mixes while recording the textural changes through photographs. This asset enables the pavement evaluation engineer to assess the mix design from a point of view hitherto unseen.

Another benefit of the method is its applicability to all types of geometric alignment, both horizontal and vertical.

Skid trailers similar to the Ontario trailer are limited to testing certain types of locations. Some intersections such as T-intersections, plus curves and grades greater than 4 percent, now can be tested using the photo-interpretation technique.

Where the skid trailer is restricted to testing in weather conditions which are dry with above-freezing temperatures, the photo-interpretation technique can be used in any temperature as long as the pavement is dry and clear of ice, snow, and extraneous materials.

Semiautomation of the Photo-Interpretation Classification System

Background

Recent developments in the photogrammetric and computer industries have made available accurate and simple mensuration devices which have on-line computing capabilities. These instruments lend themselves to accurate determination and evaluation of pavement surface textures using the photo-interpretation technique. The instrumentation would allow the objective extraction of both qualitative and quantitative data, and be less labor-intensive and more cost-effective than the present photo-interpretation technique.

During the past few months, a project with Carl Zeiss (Canada) Ltd. investigated the possibility of semiautomation of the photo-interpretation procedure.

The present photo-interpretation technique is somewhat labor-intensive, and relies on the interpreter making subjective data decisions. A cost-effective approach based on rapid, nonsubjective digitization procedures was needed if the technique was to be used as a production tool.

The project has a number of objectives:

1. To determine if the Stereocord G-2 could be used as a tool for semiautomation.

2. To decrease subjective operator input by generating software routines to define as many of the six texture parameters as possible.

3. To compare machine data with manually-developed data to ensure repeatability and consistency.

4. To obtain digital data that are storable for future use, and to obtain graphical data such as real-time line profiles and enlarged macroprofiles.

5. To identify those areas requiring further refinement in a full-scale study.

Hardware Components

The Carl Zeiss (Oberkochen) Stereocord G-2 system, with its peripherals, is an image measuring system suited to the gathering, processing, and storage of digital data from all types of photographs, slides, or plans (Fig. 6).

The system is designed to produce accurate digital data, manipulate the data, record, and store data for future manipulation. The system consists of a basic optical-mechanical unit (the Stereocord G-2) for viewing and making measurements; the Direc-1 electronic system for counting, display, and transmission of the measurements; and the system is coupled to a Hewlett-Packard 9825A for on-line data manipulation and storage.

Software Components

In order to meet the objectives of the semiautomation project, a number of software components were put together. These included: (*a*) algorithms for measuring parameters *B, D, A,* and *C;* (*b*) programs for the microprocessor to evaluate parameters *B, D, A,* and *C;* (*c*) programs to scan the photographs in two directions and digitize the profiles of each scan line; (*d*) plotting programs to produce both visual and hard copy profiles; (*e*) leveling routines to establish reference plane; (*f*) parameter coding generation programs; and (*g*) programs to evaluate parameters *E* and *F* from the line scans.

A detailed outline of the software routines can be found in the project report [5]. Figure 7 shows the program interaction in flow chart form. All of the programs were developed to create a dialogue between the operator and the computer.

The final package digitizes, records, plots, and establishes the reference plane of the textures with a minimum of operator input. The hard copy out-

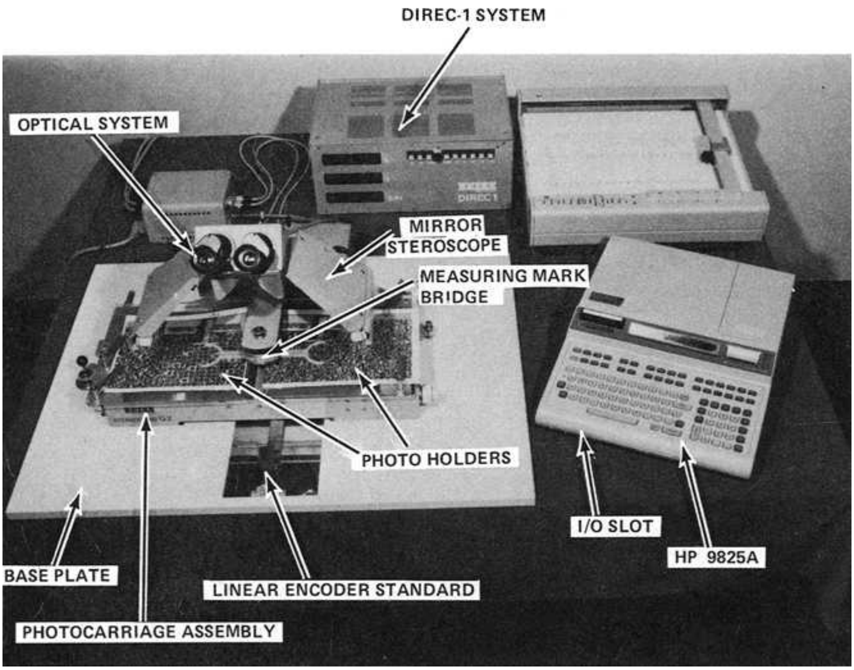

FIG. 6—*G-2 stereocord with Direc-1 and HP 9825 desk calculator.*

put is a plot of the line scan, profiles, and a summary of the required numerical data (Fig. 8).

By minimizing the operator input, and creating a dialogue between the operator and the software, the amount of subjective interpretation is reduced to a minimum. The result is a more consistent, repeatable, and accurate texture profile and skid number.

Study Results

The initial semiautomation study drew a number of conclusions.

1. The semiautomation of the photo-interpretation technique is feasible using the Zeiss Stereocord G-2.

2. Semiautomation will reduce the amount of subjective operator input found in the present manual procedures.

3. Semiautomation will reduce the amount of interpretation time by as much as 50 percent, thus increasing productivity.

4. The amount of operator training can be significantly reduced, due to the microprocessing unit carrying out all computations and classification procedures.

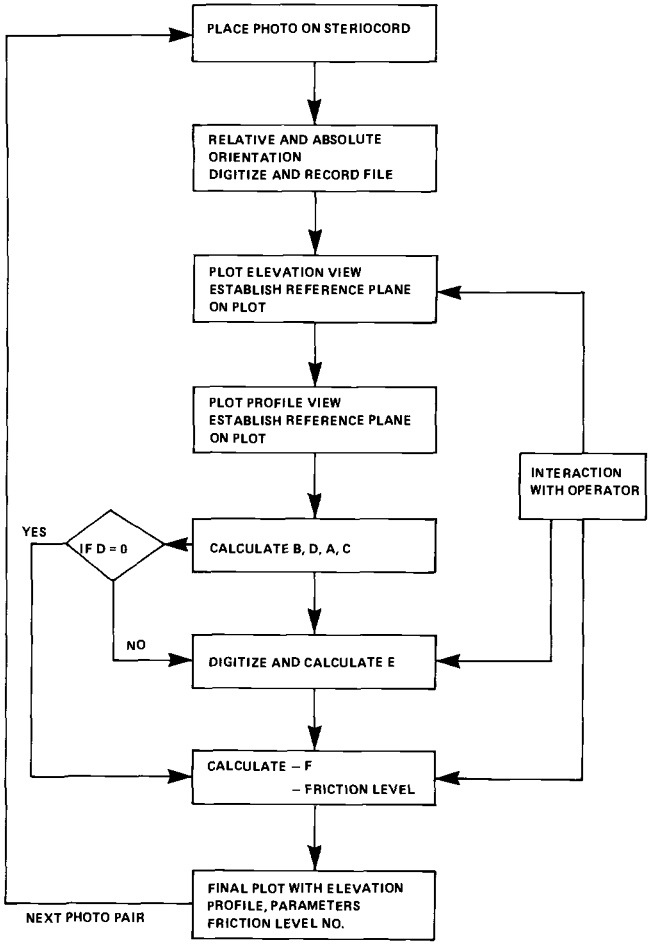

FIG. 7—*Schematic flowchart of program routine.*

5. Accuracy and repeatability of measurements and classifications are improved through a decrease in subjective operator interpretation.

6. Further in-depth study of the system is required to produce a production line system.

Uses of the Method

The photo-interpretation technique is being used in a number of projects within the Ministry. The study of the pavement surface texture enables the establishment of guidelines for desirable texture features of high-speed skid resistant pavements, where a prime requirement is adequate surface

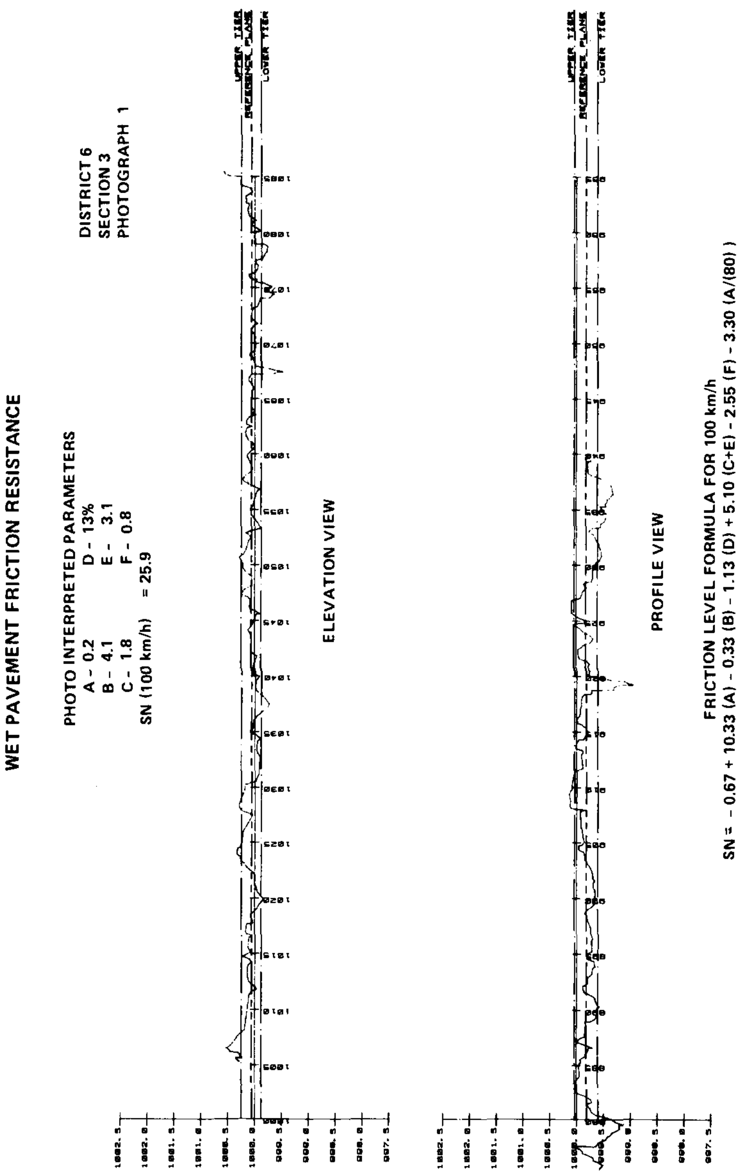

FIG. 8—*Hard copy output.*

drainage [4]. These guidelines include: height of macroprojections (*A*), 0.55 mm (1/32 in.) minimum; and width of macroprojections (*B*), 2.0 mm (5/64 in.) minimum.

These guidelines have been set up as a result of a 5-year study of the changing surface texture on 18 test sections in a high-volume freeway in the Toronto, Ontario area [4].

New mix designs have been studied at the laboratory stage by photographing specimens of the mix and studying the initial pavement textures. Although a pavement texture changes rapidly during its initial life, the initial skid resistance level should not be so low that it will present a hazard to the driving public. The photo-interpretation technique allows the mix design personnel to assess the differences between various mixes and choose that design which offers an optimum skid resistance level while achieving an optimum structural design.

The study of in-place mix designs has allowed us to study the variations in skid resistance with time, and measure the effect of various parameters on these variations. Some of the reasons for seasonal variations in the skid number can be accounted for using this technique. The particulate material on the pavement surface and changes in the asphalt cement matrix due to temperature and kneading by traffic can be observed in the photographs.

The technique currently is being used to study skidding accident sites in Ontario. The areas are being studied prior to resurfacing; therefore, the area is rephotographed to assess not only the change in skid resistance, but also the change in the texture parameters.

Potential Uses of the Semiautomated Method

By means of semiautomation, the photo-interpretation technique can become a fully operational system with increased productivity, efficiency, and accuracy.

The significance of macro- and microtextures could be defined more clearly, and optimum textures could be defined. Specific changes in mix designs could be shown graphically in terms of the surface texture changes. Knowing these changes, more economical and optimum mix designs could be generated.

Using the digital data produced by the semiautomation, particle orientation for coarse aggregates could be evaluated.

The effectiveness of aggregate crushing devices and paving procedures could be assessed in terms of the initial texture profiles in new pavements. These profiles can be compared to profiles taken later in the life of the pavement. Texture deterioration could be studied in terms of stability of the mix design, retention of macroprojection angularity, and projection orientation. Comparisons also could be made with laboratory-produced mix designs.

Using appropriate programs, the relationships of texture profiles and light reflectance could be studied. The resultant information could be used with the light absorption rates to design pavements which have a maximum skid resistance and appropriate reflectance values for lighting.

These are but a few of the potential uses of the semiautomated texture classification systems.

Conclusions

1. The ability of the photo-interpretation method to provide an explanation of skid resistance generated by mechanical testing devices makes it a valuable tool and a viable alternative.
2. The method offers a texture classification system that can be used to study mix designs and predict the reaction of these profiles to traffic wear.
3. The potential of the semiautomated system for improved accuracy, consistency, and repeatability of texture classification has been demonstrated in the initial study.
4. Further work is necessary to refine the semiautomated system so that the benefits and potential uses of the texture classification system can be realized.

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B. J. Hill[1] *and J. J. Henry*[1]

Surface Materials and Properties Related to Seasonal Variations in Skid Resistance

REFERENCE: Hill, B. J. and Henry, J. J., "**Surface Materials and Properties Related to Seasonal Variations in Skid Resistance,**" *Pavement Surface Characteristics and Materials, ASTM STP 763.* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 45–60.

ABSTRACT: A three-year research program was initiated in 1978 at the Pennsylvania Transportation Institute by the U.S. Department of Transportation to investigate possible causes for seasonal and short-term skid resistance variations. The primary objective is to determine the parameters which can be used to predict the influence of seasonal and short-term effects. This paper is concerned with the material parameters influencing the long-term seasonal variations.

Data are analyzed from 21 test surfaces in State College, Pennsylvania and 10 test surfaces in Tennessee and North Carolina. The data include skid resistance measurements according to the ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79), British Pendulum Number measurements, calculated percent normalized skid number gradient, and average daily traffic volumes. An exponential curve is fitted to the skid number data for the asphalt pavements, while a linear relationship best fits the data for portland cement concrete surfaces. The coefficients of the resulting seasonal variation curves are regressed against pavement and traffic parameters to provide predictors for the long-term effects. Significant predictors are found to be British Pendulum Number and average daily traffic. Further predictors are suggested by the results of a pavement polishing experiment carried out on the 21 Pennsylvania test surfaces. Good agreement is observed between the two sets of test data.

KEY WORDS: skid resistance, seasonal variations, surface properties, texture, correlation, predictions, polishing, pavements, pavement surface characteristics

Skid resistance measurements according to the American Society for Testing and Materials (ASTM) Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79) exhibit both long-term seasonal and short-term variations on public highways in Pennsylvania and other states [*1,2*].[2]

[1]Assistant professor and associate professor, respectively, of mechanical engineering, The Pennsylvania State University, University Park, Pa. 16802. Author Hill is a visiting professor from the University of Newcastle, New South Wales, Australia.

[2]The italic numbers in brackets refer to the list of references appended to this paper.

These variations make it difficult to establish a rational maintenance program in which skid resistance is one of the important factors. An annual cycle has been observed, in the northern states at least, where the skid resistance tends to be higher in winter through spring than summer through fall. Superimposed on this annual cycle are short-term variations, attributed to rainfall and local weather conditions. Figure 1 shows a five-year skid resistance history for a typical Pennsylvania pavement.

During the past two decades, several state highway departments and other agencies in the United States have conducted extensive skid resistance surveys, but until the last few years little attention was paid to seasonal variations. Until recently, the most comprehensive, documented studies involving both seasonal and short-term skid resistance variations were the ones undertaken by the Pennsylvania Department of Transportation [*1,3*]. In the course of evaluating skid resistance measurements, some cyclic patterns were observed. Measurements showed that, once a pavement surface had stabilized after being exposed to weather and traffic for one to two years, the surface exhibited cyclic skid resistance variations.

Several other states have reported their observations of seasonal skid resistance variations to the Federal Highway Administration (FHWA). These observations were summarized in 1977 by Rice [*2*]. Although these and other observations from various agencies are helpful in providing qualitative information about trends and magnitudes of seasonal variations in skid resistance, the measurements are spaced too far apart in time to offer sufficient information for developing a model for predicting the low skid number expected to occur during the year on a given pavement.

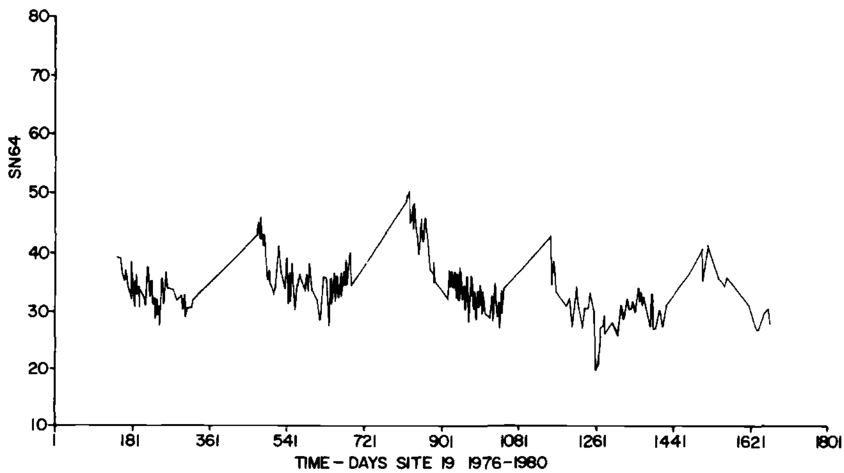

FIG. 1—*Five-year history for Pennsylvania Site 19.*

To establish further means of interpreting skid resistance data subject to seasonal and short-term variations, the U.S. Department of Transportation (USDOT) initiated a three-year research program at the Pennsylvania Transportation Institute (PTI) in 1978. This paper describes the findings regarding the long-term seasonal skid resistance variations on 21 pavements in Pennsylvania. Additional evidence supporting these findings is presented with a comparison of the seasonal variations experienced on 10 pavements in Tennessee and North Carolina (FHWA Region 15). The results of studies of the effects of weather on the short-term skid resistance variations are reported by Hill and Henry [*4*].

Data Collection

In 1979, skid resistance testing was performed on 21 test pavements in Pennsylvania between March and November. The 21 sites contain a variety of aggregates and mix designs, and include both asphalt and portland cement concrete (PCC) pavements which are subjected to a wide range of average daily traffic (ADT). The FHWA Region 15 data were collected for 10 sites during the 12-month period from July 1979 to July 1980. A full description of the test site construction materials and locations is given by Henry and Dahir [*5*].

The 21 Pennsylvania sites include: 10 dense graded asphalt (DG), 5 open graded asphalt (OG), and 6 PCC. The 10 sites in FHWA Region 15 include: 4 dense graded asphalt, 2 open graded asphalt, 2 bituminous surface treatments (BST), and 2 PCC.

For the Pennsylvania sites, the skid tests were made in the transient slip mode [*6*] which, while providing skid number data at 64 km/h (40 mph) (SN_{64}) data according to ASTM Method E 274-79, also provide brake slip numbers at 16, 32, and 48 km/h (10, 20, and 30 mph) which can be used to approximate SN_{16}, SN_{32}, and SN_{48} [*6*]. For the FHWA Region 15 sites, separate tests were made at speeds of 48, 64, and 80 km/h (30, 40, and 50 mph). Both approaches permit the skid resistance-speed relationship to be developed in terms of the percent normalized gradient [*6*] and the zero speed skid number intercept according to the exponential relationship [*6*] below

$$SN_v = SN_0 e^{-PNG/100\,V} \tag{1}$$

where

SN_v = the skid number at velocity V km/h,
SN_0 = the zero speed skid number intercept (a function of pavement microtexture), and
PNG = the percent normalized gradient (a function of pavement macrotexture) $= \dfrac{-100}{SN}\dfrac{d(SN)}{dV}\left(\dfrac{h}{km}\right)$.

Monthly texture measurements made at each site included British Pendulum Number (BPN) according to the ASTM Method for Measuring Surface Frictional Properties Using the British Pendulum Test [E 303-74(1978)]. The temperature of the test tire and test pavement and the ambient temperature were recorded in each case.

Mechanistic Model

The SN_0 deduced from data collected throughout the year typically exhibits long-term variations as shown in Figs. 2, 3, and 4. Figures 2 and 3 show the results for a dense-graded and open-graded asphalt surface. The long-term trend for these cases can be considered to be exponential in nature, while the trend in the data for PCC surfaces (Fig. 4) is linear.

The value of SN_0 at any time t can be expressed [4]

$$SN_0 = SN_{OR} + SN_{OL} + SN_{OF} \tag{2}$$

where

SN_{OR} = the short-term weather related residual,
SN_{OL} = the long-term seasonal variation, and
SN_{OF} = a measure of SN_0 which is independent of both short- and long-term variations.

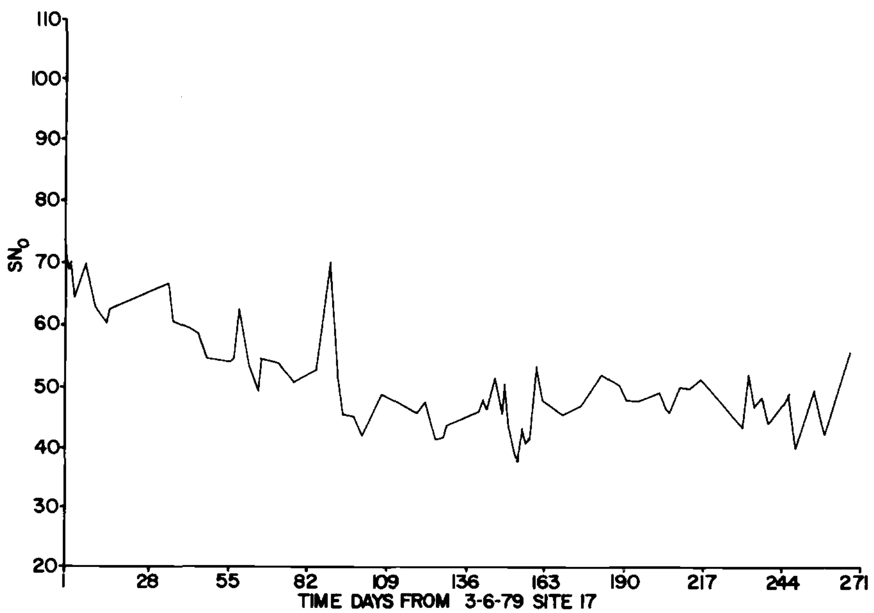

FIG. 2—*SN_0 versus time for dense graded Pennsylvania Site 17, 1979.*

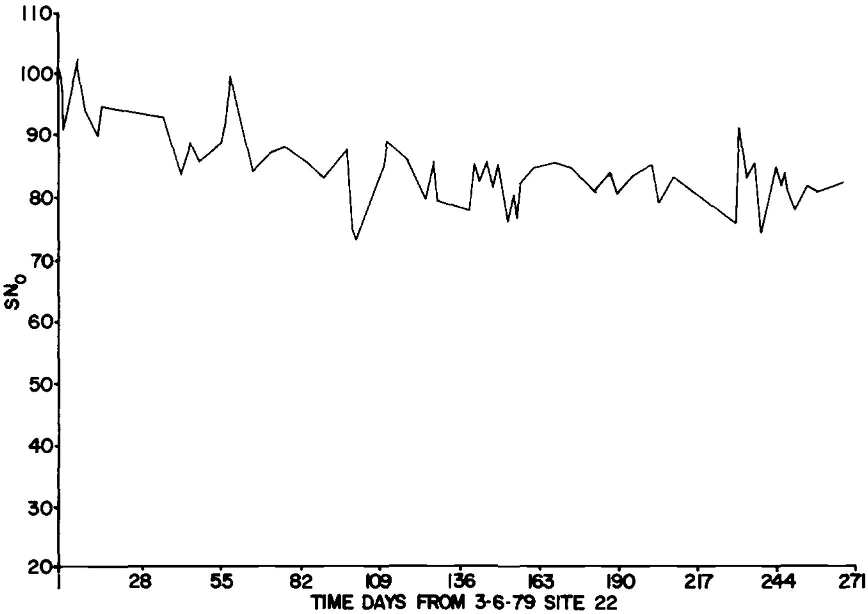

FIG. 3—*SN_0 versus time for open graded Pennsylvania Site 22, 1979.*

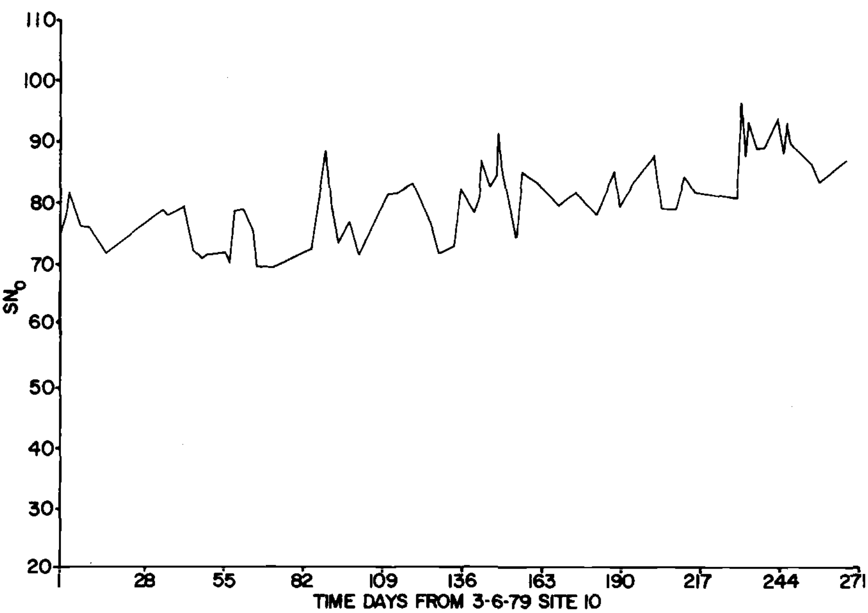

FIG. 4—*SN_0 versus time for portland cement concrete Pennsylvania Site 10, 1979.*

For asphalt surfaces an exponential relationship is written

$$SN_{OL} = \Delta SN_0 e^{-t/\tau} \tag{3}$$

while for PCC surfaces, a linear relationship better fits the observations

$$SN_{OL} = \frac{\Delta SN_0}{\Delta t}\, t \tag{4}$$

where

ΔSN_0 = the change in SN_0 over the testing season,
τ = the rate at which long-term effects occur, and
Δt = the length of the testing season in days.

Reduction of the measured SN_{64} data using Eq 1 results in a value for SN_0 and PNG for each data point. For each site, however, PNG is assumed constant [6] with the variations observed being a result of random errors in the data. These variations in PNG for any one site are small, and are distributed approximately normally, as illustrated by Figs. 5 and 6. Modeling is simplified, with no sacrifice in accuracy, if the average value for PNG is used for each site. Figure 7 shows the originally calculated SN_0 for Pennsylvania Site 22 compared with corresponding values of SN_{OP}, which are calculated using the average PNG. Tables 1 and 2 list the average PNG for each site along with average values for BPN and ADT.

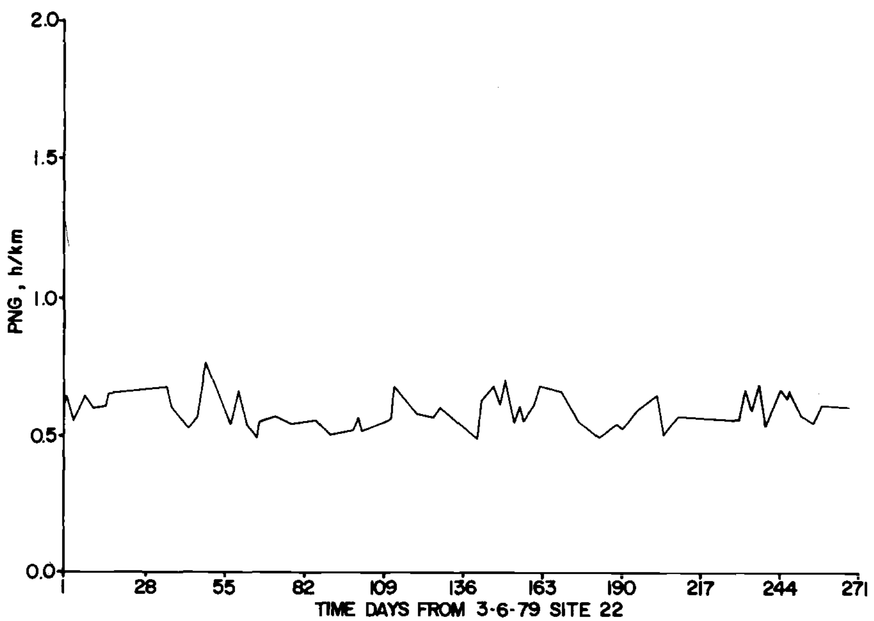

FIG. 5—*Variation of PNG with time for Pennsylvania Site 22, 1979.*

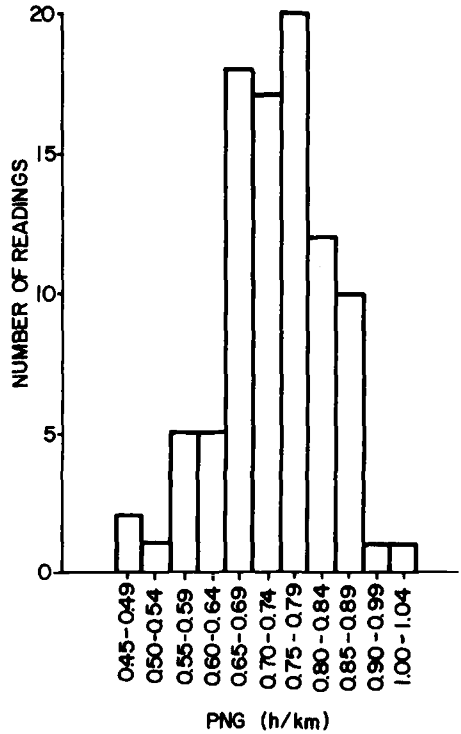

FIG. 6—*Distribution of PNG for Pennsylvania Site 1, 1979.*

The skid number at 64 km/h (40 mph) will contain similar long- and short-term effects according to Eq 1, and the value of SN_{64} after removal of these effects can be expressed

$$SN_{64F} = SN_{OF}e^{-0.64\,PNG} \tag{5}$$

Combining Eqs 1, 2, and 5 yields

$$SN_{64F} = SN_{64} - (SN_{OR} + SN_{OL})e^{-0.64\,PNG} \tag{6}$$

Hill and Henry [*4*] detail the effect of weather on the short-term residuals SN_{OR}; this study concerns the effect of pavement properties on seasonal skid resistance variations (SN_{OL}).

Fitting of Long-Term Relationship

Asphalt Surfaces

The reduction in skid resistance on asphalt pavements that occurs over the warmer months appears to be exponential. The fitting of an exponential rela-

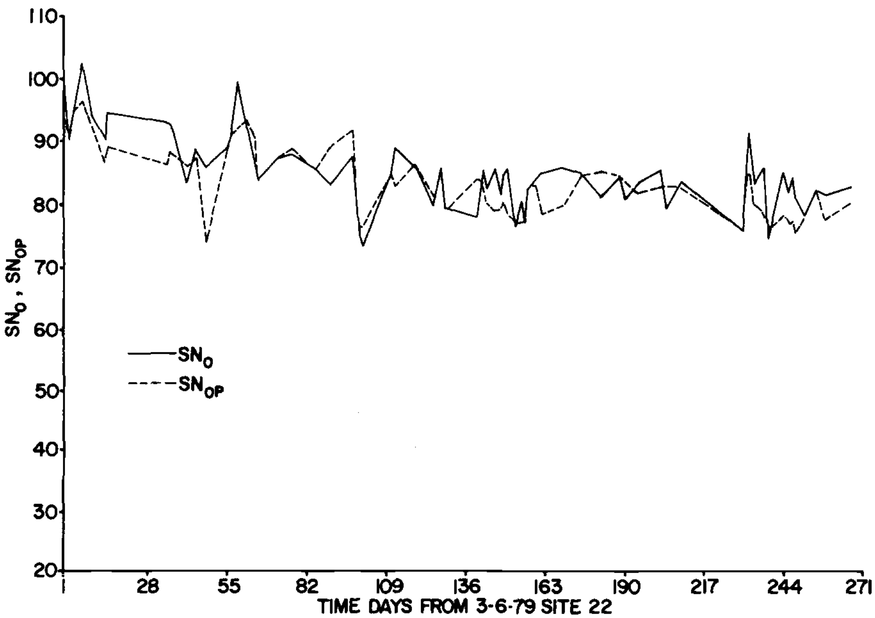

FIG. 7—*SN_0 and SN_{OP} versus time for Pennsylvania Site 22, 1979.*

TABLE 1—*Pavement and traffic parameters for Pennsylvania sites.*

Site Number	Type	Average PNG, h/km	Average BPN	ADT
1	DG	0.81	44.2	6 630
2	PCC	0.88	53.4	7 700
3	PCC	0.75	67.2	3 640
4	DG	0.76	50.3	3 640
7	PCC	0.78	64.7	1 820
8	DG	0.62	44.6	1 820
9	DG	0.70	53.1	1 710
10	PCC	0.81	65.5	1 710
11	DG	0.83	45.4	4 490
12	DG	0.67	56.2	4 490
13	OG	0.56	85.0	7 920
14	PCC	0.83	58.9	8 770
15	OG	0.56	83.4	7 920
16	DG	0.90	43.3	6 500
17	DG	0.66	48.7	800
18	PCC	0.69	65.6	1 200
19	DG	0.82	46.9	7 000
21	OG	0.64	48.5	2 500
22	OG	0.60	80.3	2 500
24	DG	0.84	43.7	4 490
25	OG	0.74	73.2	7 920

TABLE 2—*Pavement and traffic parameters for FHWA Region 15.*

Site Number	Type	Average PNG, h/km	Average BPN	ADT
1	asphalt	0.54	74.6	6 790
2	asphalt	0.43	84.8	6 020
3	PCC	0.92	59.3	32 420
4	asphalt	0.55	73.9	9 220
5	asphalt	0.41	54.6	11 820
6	BST	0.41	57.0	1 280
7	asphalt	0.60	70.3	11 350
8	PCC	0.68	62.7	NA[a]
9	BST	0.48	77.8	NA
11	asphalt	0.74	65.3	NA

[a]NA—not available.

tionship to the data is not strictly a least squares regression, but requires a qualitative judgment on the part of the investigator. First, a simple log-linear regression is not possible as there are three variables for which values are to be found: SN_{OF} (the magnitude of SN_0 at $t = \infty$); ΔSN_0 (the intercept at $t = 0$); and τ (the time constant for the curve). Secondly, the existence of short-term variations due to weather effects must be accounted for in fitting the long-term variation. Weather conditions, particularly during summer when there may be long periods of high temperature with little or no rain, may be responsible for large sections of the data being lower than normal. These usually low (or high during cold, wet periods) data points should not be allowed to unduly influence the fit of the long-term relationship. In this case, a curve which provides the best fit to the data in the opinion of an experienced investigator is better than the one which has the highest correlation resulting from a least squares fit.

The procedure used is detailed in the following paragraphs. For each of the sites, a value for SN_{OF} was chosen which appeared to be the level of SN_0 that would be reached over a long period of time if short-term fluctuations were not present; that is, it is a characteristic value for the pavement. Then a least squares regression was carried out to determine the corresponding values for ΔSN_0 and τ. For some of the sites, the resulting exponential curve did not appear to be a suitable fit to the data; in every case the magnitude of the time constant τ appeared at fault. A qualitative adjustment was made to τ in the appropriate direction, and a further least squares regression was performed to determine new values for ΔSN_0 and SN_{OF}. This last step was repeated until the resulting curve appeared suitable. Figure 8 shows a typical result.

In view of the magnitude of the short-term fluctuations, particularly for those sites with a small value for ΔSN_0, a high correlation coefficient cannot be expected. Figure 9 shows that for sites with a high value of ΔSN_0, the cor-

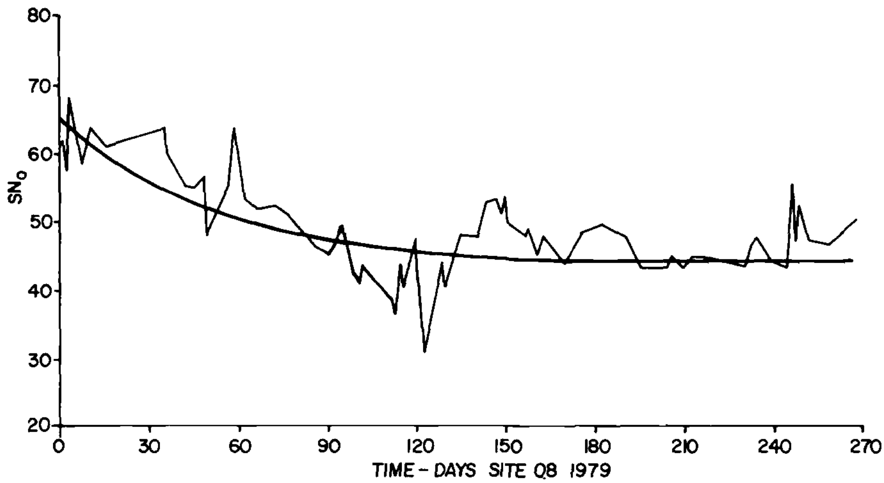

FIG. 8—*Seasonal exponential curve for Pennsylvania Site 8, 1979.*

relation coefficient obtained using the previously mentioned procedure is high, with the value falling off to a lower (but still acceptable) value with lower ΔSN_0. Table 2 lists the values obtained for SN_{OF}, ΔSN_0, and τ.

Portland Cement Concrete Surfaces

All the PCC surfaces exhibited a linear increase in SN_0 with time over the testing season. This increase is unexpected, and may not be typical of rigid pavements. More data than those from one single year would be necessary to determine the true nature of the long-term behavior of PCC surfaces. For the data available, a simple linear regression was performed to yield the average value SN_{OF} and the rate of increase $\Delta SN_0/\Delta t$. These results are listed in Tables 3 and 4.

Prediction of Long-Term Parameters

SN_{OF} is essentially a measure of the microtexture of the pavement after removal of the long-term seasonal and short-term weather-related effects. As such, it would seem likely that a microtexture parameter could be used to predict SN_{OF}. Monthly measurements of BPN were available for each of the test pavements, and a regression of SN_{OF} against the average BPN for the season yields

Pennsylvania (21 observations)

$$SN_{OF} = 1.19\,\text{BPN} - 7.90\,[r = 0.97] \tag{7}$$

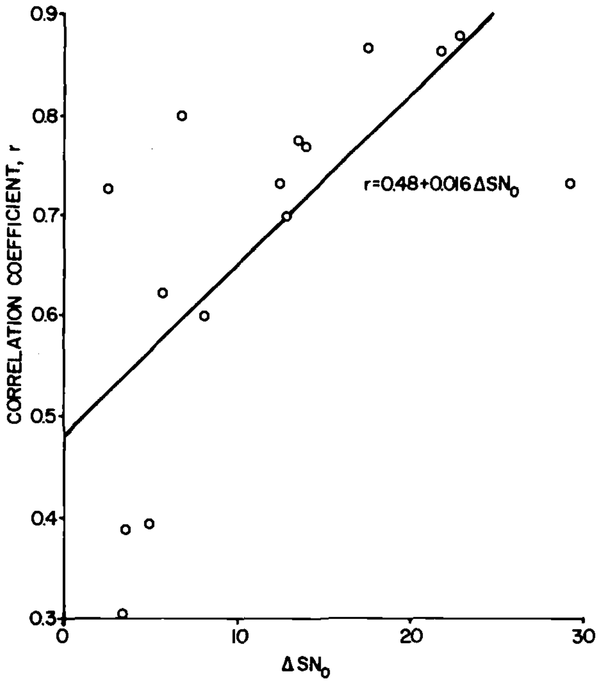

FIG. 9—*Correlation coefficient versus ΔSN_0 for exponential curve fits.*

TABLE 3—*Parameters for seasonal variations of Pennsylvania sites.*

Site Number	SN_{OF}	ΔSN_0	$\Delta SN_0/\Delta t$	τ days
1	50.5	8.28	...	55
2	49.4	...	0.036	...
3	72.4	...	0.040	...
4	55.4	9.60	...	45
7	69.8	...	0.058	...
8	43.0	22.57	...	75
9	57.7	21.90	...	75
10	77.6	...	0.053	...
11	44.6	12.87	...	50
12	59.8	6.89	...	27
13	89.9	2.61	...	58
14	63.6	...	0.034	...
15	92.1	4.98	...	25
16	39.1	13.56	...	45
17	44.3	29.28	...	53
18	73.7	...	0.008	...
19	49.2	12.56	...	30
21	45.2	17.35	...	65
22	80.5	14.04	...	65
24	44.4	3.57	...	27
25	80.5	3.36	...	63

TABLE 4—*Seasonal variation parameters for FHWA Region 15.*

Site Number	SN_{OF}	ΔSN_0	$\Delta SN_0/\Delta t$	τ days
1	76	6	...	250
2	67	10	...	140
3	60	...	0	...
4	78	4	...	100
5	38	14	...	190
6	37	15	...	110
7	73	0	...	...
8	55	...	0	...
9	68	12	...	110
11	57	15	...	120

FHWA Region 15 (10 observations)

$$SN_{OF} = 1.16\,\text{BPN} - 17.5\,[r = 0.79] \tag{8}$$

ΔSN_0 is a measure of the rejuvenation of skid resistance that occurs during the winter months as a result of the depolishing effects of winter conditions [*3*]. As such, BPN again would seem a likely parameter to be used as a predictor; however, a linear regression of the data yields a poorer-than-expected correlation. The resulting relationships are

Pennsylvania

$$\Delta SN_0 = 26.11 - 0.24\,\text{BPN}\,[r = -0.46] \tag{9}$$

FHWA Region 15

$$\Delta SN_0 = 26.39 - 0.24\,\text{BPN}\,[r = -0.44] \tag{10}$$

The introduction of the traffic volume parameter ADT is found to improve significantly the prediction of ΔSN_0, yielding

Pennsylvania

$$\Delta SN_0 = 28.5 - 0.0023\,\text{ADT} - 0.09\,\text{BPN}\,[r = -0.82] \tag{11}$$

FHWA Region 15

$$\Delta SN_0 = 66.1 - 0.0049\,\text{ADT} - 0.29\,\text{BPN}\,[r = -0.66] \tag{12}$$

This result indicates that the depolishing of the pavement as a result of winter deicing chemicals is offset by the mechanical polishing occurring with large traffic volumes. The mechanical aspects of pavement rejuvenation become important when the winter use of studded tires is considered. Data are available to allow the calculation of ΔSN_0 for five of the asphalt pavements in Pennsylvania over a period of three consecutive winters, the middle winter being one for which the use of studded tires was prohibited. Table 5 shows these results; ΔSN_0 is consistently greater for the two winters during which studded tires were used, supporting the theory that a significant factor in the winter rejuvenation of surface texture is the mechanical interaction of the tires and the pavement.

The time constant τ associated with the rate of decrease in skid resistance over an annual cycle was correlated against ADT for each of the asphalt pavements with the following results

Pennsylvania

$$\tau = 67.67 - 0.0037\,\text{ADT}\,[r = -0.53] \tag{13}$$

FHWA Region 15

$$\tau = 124.1 - 0.0048\,\text{ADT}\,[r = -0.30] \tag{14}$$

The relatively poor correlations may be a result of the uncertainty in values for τ obtained from the curve fitting technique. Because of the relative magnitudes of the short-term fluctuations in the data, as well as the magnitude of τ compared with the length of the testing season, relatively large absolute changes in τ result in only minor variations in the predicted seasonal variation curve. Table 6 lists the maximum change in SN_{OL} predicted by Eq 3 if the values predicted by Eq 14 are used for τ instead of those listed in Table 2, indicating that the predictor model for SN_{OL} is not highly sensitive to changes in τ. Equations 13 and 14 suggest that ADT is an adequate estimator, with higher traffic volumes causing the pavement to reach a terminal polished state more quickly.

Pavement Polishing Experiments

During the month of July 1980 a series of tests was carried out on the Pennsylvania test sites using the Penn State Reciprocating Pavement Polisher [7] to develop further predictors for seasonal parameters. Each pavement was

TABLE 5—*ΔSN_0 for five Pennsylvania sites over three consecutive winters.*

	ΔSN_0		
Site Number	1977-78	1978-79	1979-80
16	20	14	23
17	23	30	30
19	26	13	24
21	23	17	23
22	19	14	21

TABLE 6—*Sensitivity of predicted seasonal variations to changes in* τ.

FHWA Region 15 Site Number	τ from Curve Fit	τ from Eq 14	Change in SN_{OL} from Eq 3
1	250	160	0.98
2	140	150	−0.25
4	100	170	−0.77
5	190	180	0.28
6	110	130	−0.92
7	...	...	...
9	110	NA[a]	...
11	120	NA	...

[a]NA—not available.

subjected to 2000 polishing cycles using an abrasive size of 44 μm with BPN measurements taken initially, at 500 cycles, and at 2000 cycles. The polishing was performed near the right-hand edge of the pavement, out of the wheel track. The results are listed in Table 7. In many cases, the BPN value was higher after 500 cycles of polishing than initially; this is thought to be a result of removal of the surface glaze. The results for three of the sites (4, 21, and 24) are to be used with caution since the upward trend in BPN continues to 2000 polishing cycles, a result for which no reasonable explanation can be made. Leaving out the data for these sites, the best predictor for ΔSN_0 for the asphalt surfaces is

$$\Delta SN_0 = 41.4 - 0.068\left(\frac{BPN_{500} - BPN_{2000}}{BPN_{500}}\right) - 0.41\ BPN_{2000}\ [r = -0.74] \qquad (15)$$

For the PCC sites, the best predictor for the slope of the seasonal line, $\Delta SN_0/\Delta t$ is

$$\frac{\Delta SN_0}{\Delta t} = 0.045 - 0.0002\left(\frac{BPN_0 - BPN_{2000}}{BPN_{2000}}\right)[r = -0.85] \qquad (16)$$

Conclusions

The following conclusions can be made based on the results of this study.

1. The level of skid resistance at the beginning of spring is a function of surface microtexture as measured by BPN, the ADT volume, and mechanical effects such as the roughening of the surface by the use of studded tires during winter. Surfaces with high microtexture exhibit the smallest variations in zero speed skid number intercept over an annual cycle.

TABLE 7—*Results of polishing tests on Pennsylvania sites.*

Site Number	Initial BPN (BPN_0)	BPN after 500 cycles (BPN_{500})	BPN after 2000 cycles (BPN_{2000})
1	59	60	59
2	68	75	64
3	74	79	70
4	58	68	64
7	68	70	71
8	56	51	50
9	71	66	69
10	70	72	75
11	67	68	66
12	87	82	73
13	89	85	87
14	73	68	66
15	87	85	81
16	70	62	56
17	...[a]	...	...
18	74	73	67
19	65	62	63
21	67	74	68
22	81	76	78
24	50	59	56
25	79	77	71

[a] Site has been resurfaced.

2. The level of SN_0 after removal of long- and short-term effects can be predicted by the average BPN value obtained over a number of tests.

3. The rate of decrease τ in skid resistance due to polishing of the aggregate can be adequately predicted by ADT, allowing for the relative insensitivity of the seasonal skid resistance variations model to changes in τ.

4. The seasonal variations observed at a speed of 64 km/h (40 mph) are a function of surface macrotexture as well as the microtexture of the aggregate. The microtexture, however, is the controlling factor at all speeds.

5. A pavement polishability test has been devised, which may be used in place of BPN and ADT to predict the long-term change in SN_0 of a pavement.

6. Good agreement between the results for the Pennsylvania sites and the FHWA Region 15 sites has been found. Further studies need to be directed toward similar investigations of skid resistance variations in warmer climates.

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Skid Resistance Predictive Models for Asphaltic Concrete Surface Courses

REFERENCE: Emery, J. J., Lee, M. A., and Kamel, N., "**Skid Resistance Predictive Models for Asphaltic Concrete Surface Courses,**" *Pavement Surface Characteristics and Materials, ASTM STP 763,* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 61-72.

ABSTRACT: Skid resistance performance models for dense and open graded asphaltic concrete surface courses are presented. Previous studies for dense graded mixes and high traffic volumes resulted in a predictive linear model for the skid number (SN) at 100 km/h (60 mph) (SN_{100}) in terms of known aggregate and mix parameters and available traffic data. However, the SN_{100} does approach a constant level requiring a rational function to describe traffic influences. Further work has confirmed the overall importance of mix designs in achieving desired skid resistance with accumulated traffic influences, particularly in preventing coarse aggregate immersion due to traffic compaction. High stability mixes (all steel slag, blast furnace slag, or traprock, for instance) have proven most suitable, and coarse aggregate factors such as polished stone value and aggregate abrasion value are of secondary importance once adequate levels are provided. Using a wider range of test sections, improved predictive models have been developed for various traffic volumes and surface types. Full details on model development are given.

KEY WORDS: skid resistance, models, predictive, asphaltic concrete, dense graded, open graded, friction courses, mix design, aggregates, weathering, stability, pavement surface characteristics

In previous studies, multiple linear regression analyses were used to evaluate the significance of various aggregate and mix parameters on the skid resistance performance of dense graded asphalt mixes under extremely high traffic volumes, and to develop appropriate predictive models [*1-3*].[4] The parameters considered were capable of being determined in the laboratory prior to mix placement—aggregate gradation, polished stone value (PSV), Los Angeles abrasion value (LAAV), and aggregate abrasion value (AAV),

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[4] The italic numbers in brackets refer to the list of references appended to this paper.

and mix Marshall stability (MS), Marshall flow (FLOW), and air voids (VOID)—or readily projected in the case of traffic (total and percent commercial). While aggregate microtexture and macrotexture features, and some measure of traffic volume (mainly commercial), are identified in the literature as strongly influencing skid resistance performance, test section results suggested that parameters describing the ability of the mix to withstand immersion of the coarse aggregate into the matrix might also be significant in a predictive model [4-6]. Using monitored skid resistance data [determined in accordance with the ASTM Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79)] for high traffic volume test sections, aggregate and mix test results, and the annual average daily traffic (AADT) and percent commercial vehicles (COMM) for each section, the significant parameters and form of skid resistance predictive relationship for dense graded asphalt mixes were determined using the Statistical Package for the Social Sciences (SPSS) computer program [1-3]

$$\begin{aligned} SN_{100} = {} & A(\mathrm{PSV}) + B(\mathrm{MS}) + C(\mathrm{FLOW}) + D(\mathrm{VOID}) \\ & + E[\mathrm{EQT}(F)] + G \end{aligned} \tag{1}$$

where SN_{100} is the predicted skid number (SN) at 100 km/h (60 mph). The equivalent traffic is given by

$$\mathrm{EQT}(F) = (3 \times 10^{-5})\left[1 + \left(\frac{F-1}{100}\right)(\mathrm{COMM})\right](\mathrm{AGE})(\mathrm{AADT}) \tag{2}$$

where: F is the commercial vehicle equivalence factor, and the service life (AGE) is in months.

The skid resistance predictive model (Eqs 1 and 2) was applied to monitored data for the test sections (Spring and Fall measurements at 8, 14, 21, 27, 35, and 39 months), and a correlation coefficient of greater than 0.91 was found for all cases using the most significant EQT(F) [2]. As further monitoring data and test sections became available, it was intended to refine the predictive model. However, it became apparent that the monitored SN_{100} values had attained constant levels for the high traffic volume test sections after about four years, which the predictive model could not describe. In addition, there were low traffic volume test sections and dense graded and open graded friction courses to consider [6, 7]. The extension of the skid resistance modeling to develop improved predictive models is described herein.

Test Sections

While the test sections have been fully documented [1, 2, 6, 7], some general background information will be given to indicate the range of input data available.

Highway 401 Skid Resistance Test Sections

During 1974, the Ontario Ministry of Transportation and Communications (MTC) constructed 17 asphaltic concrete test sections on Highway 401 (Toronto bypass section) to evaluate methods for improving rigid pavement skid resistance with thin overlays. Extremely high traffic volumes are involved with a total daily traffic of 199 000 (1976) using a 12-lane collector and express lane system. Dense graded surface course mixes, dense graded friction course mixes, and open graded friction course mixes were placed using limestone, traprock, steel slag, and air-cooled blast furnace slag aggregates, or combinations. These test sections have yielded valuable general information on the skid resistance for high traffic volume conditions: initial target skid numbers must be substantially greater than recommended minimum levels; early decline in skid resistance is due to coarse aggregate immersion into the matrix under wheel loads (Fig. 1); dense and open graded mixes of 100 percent crushed aggregates with fairly high PSV (traprock, steel slag, and blast furnace slag, with PSV greater than about 45 to 50) and high stability provide skid numbers close to, or above, recommended minimum levels; crushed fines result in much better macrotexture than mixes containing sand; and adequate macrotexture is achieved when the stone projection above the matrix is 0.5 mm (1/64 in.), or greater. It is clear that factors contributing to high mix stabilities are important to maintaining skid resistance and must be considered in predictive models. Continued monitoring also shows that the SN's have levelled off (dotted line in schematic of Fig. 1), rather than continuing to decrease at a reduced rate (solid line in schematic of Fig. 1). Weathering influences, which are generally seasonal, appear to be regenerating microtexture at about the same rate that traffic polishing is involved. These important weathering influences, including factors such as erosion of the matrix that improve macrotexture, are the subject of current graduate research at McMaster University, Hamilton, Ontario, Canada.

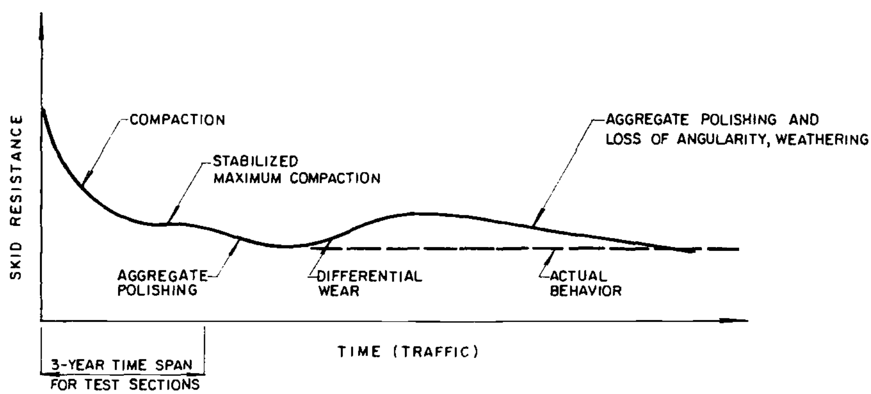

FIG. 1—*Schematic of skid resistance changes with time (accumulated traffic).*

Highway 7 Test Sections

During 1978, the MTC constructed 17 asphaltic concrete test sections on Highway 7 near Lindsay to develop mixes which provide, and maintain, adequate surface texture and skid resistance, so that skidding accidents are minimized for relatively low volumes (moderate) traffic. Mix types were similar to the Highway 401 sections except that limestone and igneous aggregates, more typical of local sources, were incorporated. While the evaluation of these test sections continues, some skid resistance monitoring data were available for predictive model development [7].

Improved Predictive Models

Starting with the overall Highway 401 skid resistance monitoring for 1974 to 1979, and aggregate and mix data, the significant parameters and a series of radically modified predictive models were investigated with the SPSS program. It was found that PSV was not a significant parameter over the longer time period, and the rather narrow PSV range involved (45 to 64) provided an adequate level [greater than about 45 to 50, testing in accordance with British Standards Institution Methods for Sampling and Testing of Mineral Aggregates, Sands, and Filler—Part 3. Mechanical Properties (BS812: Part 3: 1975, British Standards Institution, London)]. AAV and LAAV (not considered appropriate for measuring wear) were not found to be significant parameters since wear was either "masked" by coarse aggregate immersion or was not significant (the highest AAV, for blast furnace slag, was less than 14, testing in accordance with BS812). Factors related to mix stability and accumulated traffic, emphasizing commercial vehicles, were found to be the most significant parameters.

In order to predict the constant SN_{100} levels attained for the test sections, a rational SN_{100}-to-traffic relationship was generally required in the predictive models

$$SN_{100} \propto \frac{a}{[\text{EQT}(F)]^b} \tag{3}$$

where a and b are constants established by regression analyses. Starting from this basic model, the Highway 401 and Highway 7 data were then used to develop a series of predictive models. Models for SN at lower speeds were also evaluated.

High Traffic Volume Predictive Models Using Highway 401 Data

Dense Graded Surface Course and Friction Course Mixes—The final model developed for the Fall, and combined Spring and Fall data, involves mix parameters and the equivalent traffic (MS in kN, FLOW in 0.25 mm and VOID in percent throughout)

$$SN_{100} = A(\mathrm{MS}) + B(\mathrm{FLOW}) + C(\mathrm{VOID}) + D/E + F \tag{4}$$

where for Fall data

$A = 0.714$,
$B = 0.356$,
$C = 1.048$,
$D = 40.904$,
$E = [\mathrm{EQT}(36)]^{0.081}$, and
$F = -17.323$.
(multiple R coefficient (R) = 0.926)

$$SN_{50} = 0.98\, SN_{100} + 8.46 \quad (R = 0.830) \tag{5}$$

where for combined Spring and Fall data

$A = 0.738$,
$B = 0.377$,
$C = 1.116$,
$D = 42.772$,
$E = [\mathrm{EQT}(28)]^{0.092}$, and
$F = -18.150$.
(R = 0.924)

$$SN_{50} = 0.87\, SN_{100} + 12.05 \quad (R = 0.902) \tag{6}$$

Very high correlation was observed between the measured SN_{100} and the Marshall parameters, particularly stability. The importance of high mix stability to the provision of adequate skid resistance for high traffic volumes cannot be overemphasized (use of 100 percent crushed angular aggregates, coarser blends, staffer binders, and fillers, for instance). Somewhat better multiple R coefficients were obtained by using the rational function approach, as compared to the previous linear function approach, for incorporating the influence of cumulative traffic. A typical scattergram plot of measured SN_{100} values against SN_{100} values predicted by the model is given in Fig. 2, indicating the good accuracy of the predictive model. While the expression for SN_{50} in terms of SN_{100} is adequate, further model development is required in the general area of relating skid resistances at various speeds.

Open Graded Friction Course Mixes—The final model developed for the Fall and combined Spring and Fall data involves the equivalent traffic, but not mix properties

$$SN_{100} = A/B + C \tag{7}$$

where for Fall data

$A = 57.753$,
$B = [EQT(3)]^{0.198}$, and
$C = -0.474$.
$(R = 0.962)$

$$SN_{50} = 1.227\ SN_{100} - 1.753 \quad (R = 0.878) \tag{8}$$

where for combined Spring and Fall data

$A = 50.916$,
$B = [EQT(4)]^{0.156}$, and
$C = 0.551$.
$(R = 0.899)$

$$SN_{50} = 1.180\ SN_{100} - 0.295 \quad (R = 0.864) \tag{9}$$

Using the rational function approach, a very high correlation was obtained between SN_{100} and the accumulated traffic without considering mix parameters. It should be noted that the available data were limited for model development, and a full evaluation of the parameters involved was not possible.

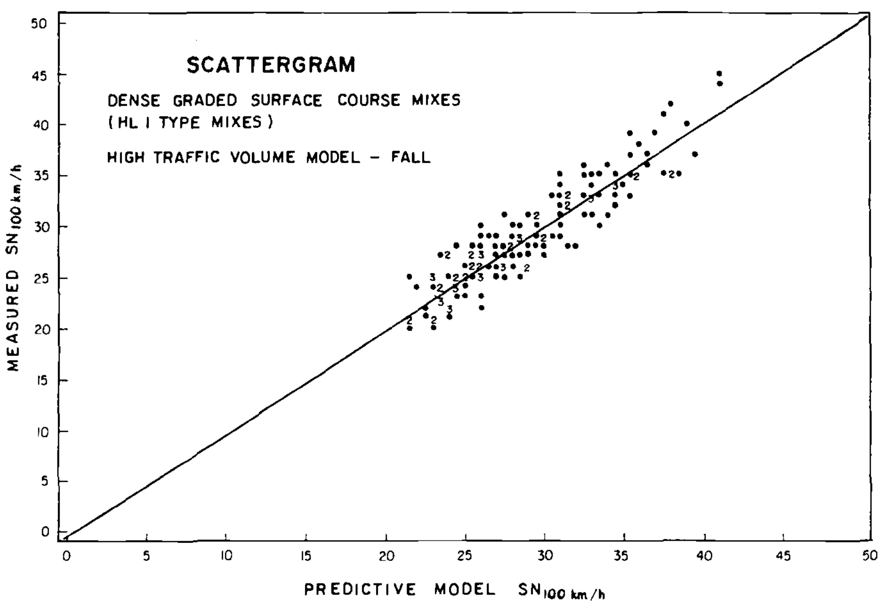

FIG. 2—*Scattergram (SPSS) of measured SN_{100} and predicted SN_{100}.*

Low Traffic Volume Predictive Models Using Highway 7 Data

Dense Graded Friction Course Mixes—The final model developed for the combined Spring and Fall data involves only the mix parameters

$$SN_{80} = A(\text{MS}) + B(\text{FLOW}) + C(\text{VOID}) + D \tag{10}$$

where

$A = 2.155$,
$B = 0.192$,
$C = 4.418$, and
$D = -8.57$.
$(R = 0.876)$

$$SN_{50} = 1.024\,SN_{80} + 6.239 \quad (R = 0.822) \tag{11}$$

All data for the dense graded friction course sections were used and treated equally, although some sections were located on curves and intersections. SN readings were separated into eastbound and westbound values, rather than taking the average value for each section. The attempt to investigate the significance of PSV and AAV in the model was abandoned, since some sections contained a blend of two coarse aggregate types. Relatively good correlation was observed between SN_{80} and the mix parameters. As in previous predictive model development, various combinations of MS, FLOW, and VOID were tried. However, the best correlation was found for these three parameters in separate form. Very poor correlation was obtained when traffic parameters were incorporated in the model, as relatively small overall changes in measured SN_{80} have been observed for the dense graded sections.

Open Graded Friction Course Mixes—The final model developed for the combined Spring and Fall data involved mix parameters and the equivalent traffic

$$SN_{80} = A(\text{MS}) + B(\text{VOID}) + CD + E \tag{12}$$

where

$A = 0.196$,
$B = 5.472$,
$C = 37.320$,
$D = [\text{EQT}(4)]^{0.016}$, and
$E = -40.32$.
$(R = 0.865)$

$$SN_{50} = 0.953\,SN_{80} + 7.338 \quad (R = 0.772) \tag{13}$$

The data were treated in the same way as for the previous case. Relatively good correlations were obtained for MS and VOID; however, poor correlation was observed for FLOW. A fair correlation was obtained when equivalent traffic was incorporated; however, in contrast with the high traffic volume model (Highway 401) the SN is increasing actively with accumulated traffic at early stages.

It must be emphasized that the models for low traffic volumes (and open graded friction courses for high traffic volumes) are preliminary, as the data are limited, and the test sections are still at early stages of monitoring. With the use of the SPSS program, it is relatively straightforward to both modify and update the predictive models.

Typical Predicted SN Ranges

The major aim of the predictive model development study was to provide anticipated SN's for use at the design stage, in terms of known aggregate and mix properties and projected traffic. This has been done in the graphical form of Figs. 3 to 8 in terms of high and low traffic volumes, aggregate, and mix types. For each case, a range of typical Marshall laboratory design data, supplied by the MTC, was used to calculate the maximum and minimum SN_{100} or SN_{80} at a given accumulated traffic value. The anticipated SN range is then easily estimated from the appropriate figures for a given traffic situation, mix type, and surfacing age (accumulated traffic). Once again, while there is fair confidence in the predictions for dense graded mix types and high traffic volumes, further developmental work is recommended for other

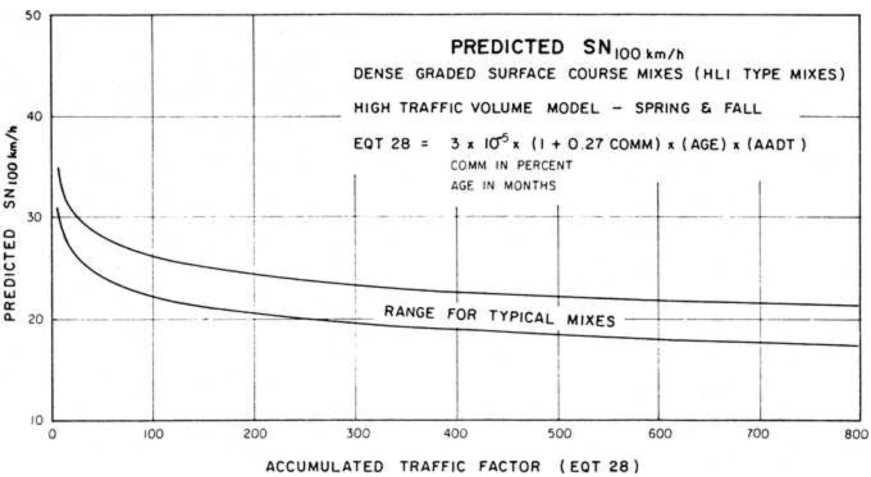

FIG. 3—*Predicted SN_{100} for dense graded surface course mixes (HL1-traprock/limestone screenings) and high traffic volume.*

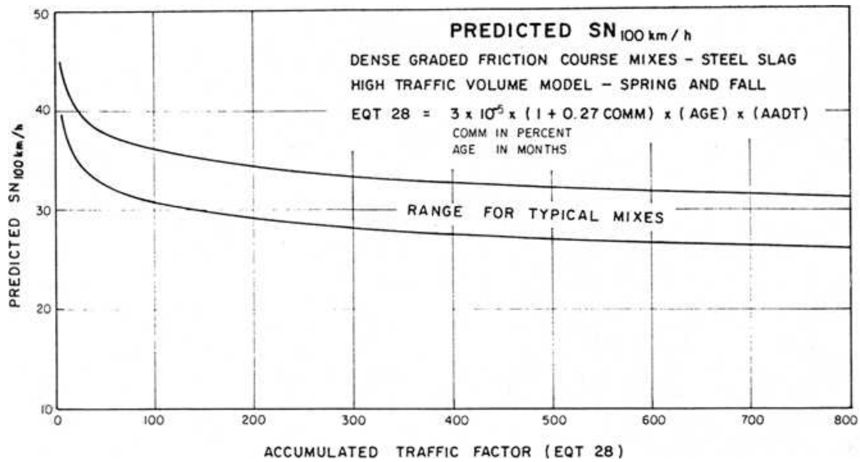

FIG. 4—*Predicted SN_{100} for dense graded friction course mixes (steel slag) and high traffic volume.*

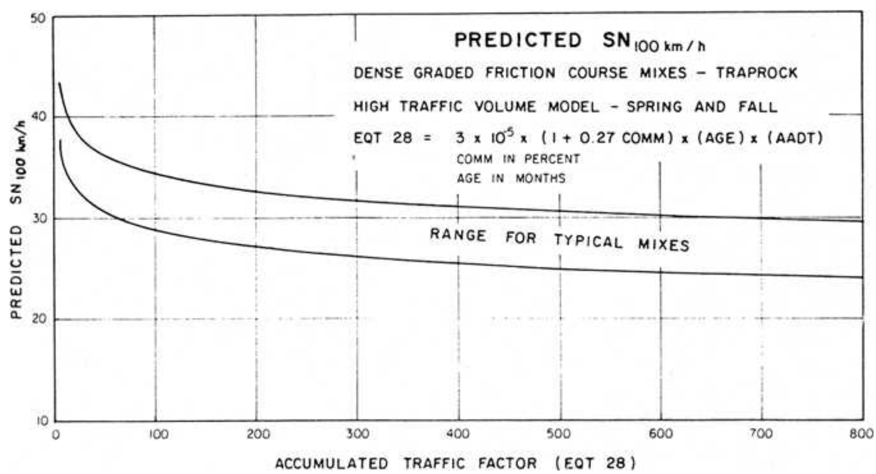

FIG. 5—*Predicted SN_{100} for dense graded friction course mixes (traprock) and high traffic volume.*

cases. The importance of high stability mixes shows up again in the high traffic volume cases, Figs. 3 to 7.

Conclusions

While further study is required to refine the predictive models for low volume traffic, the overall methodology is valuable in that it both isolates the

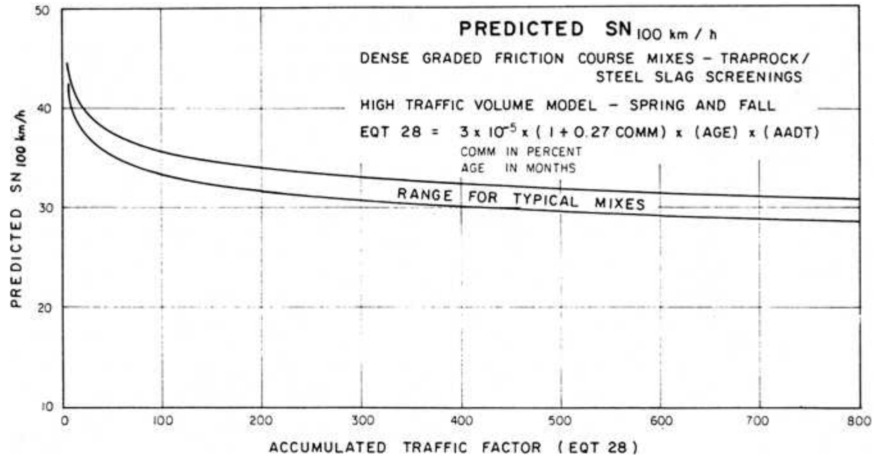

FIG. 6—*Predicted SN_{100} for dense graded friction course mixes (traprock/steel slag screenings) and high traffic volume.*

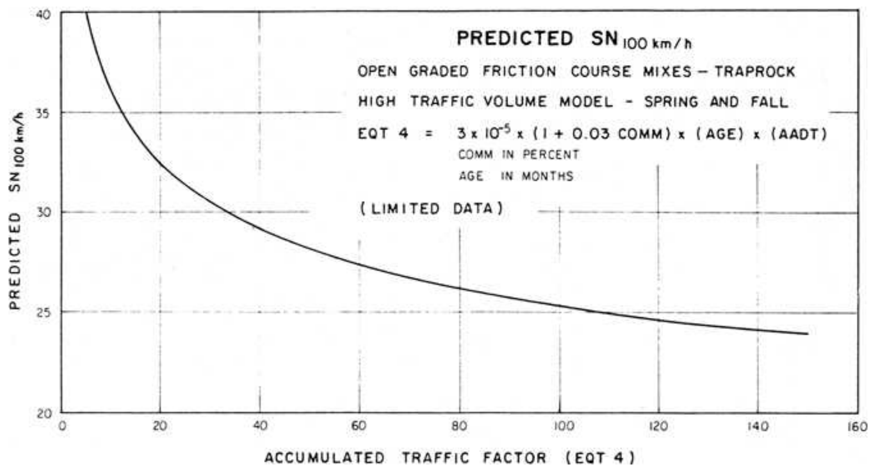

FIG. 7—*Predicted SN_{100} for open graded friction course mixes (traprock) and high traffic volume.*

key surfacing parameters influencing skid resistance, and allows the designer to evaluate potential skid resistance performance at the design stage. Further study will focus on updating the models as monitoring continues, and the evaluation of coupled weathering-polishing influences that appear to result in a long-term constant skid resistance level. It is critical that this constant level (terminal level) be adequate to minimize wet weather skidding accidents.

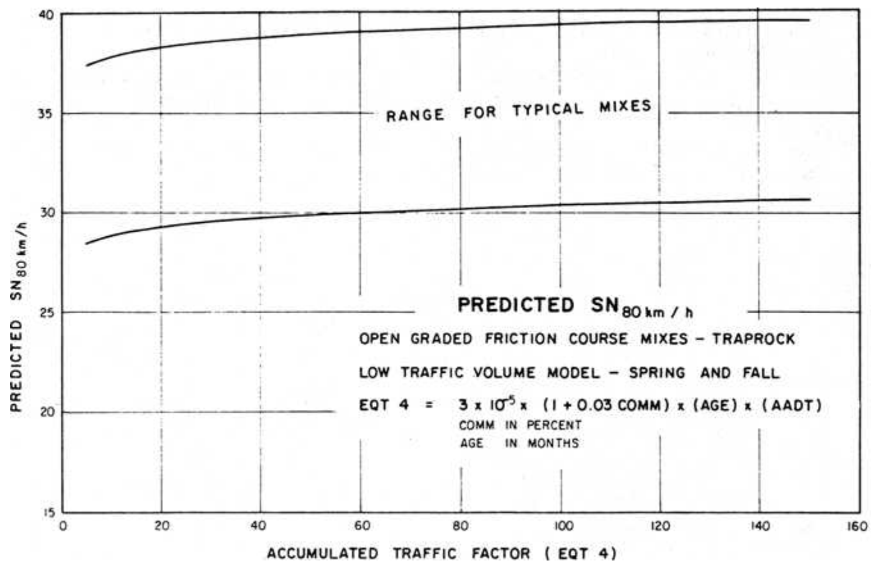

FIG. 8—*Predicted SN80 for open graded friction course mixes (traprock) and low traffic volume.*

Acknowledgments

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This work was completed while authors Emery and Lee were associated with the Construction Materials Laboratory at McMaster University, Hamilton, Ontario, Canada, and author Kamel was with the Research and Development Division, Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada.

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Frictional Performance of Pavements and Estimates of Accident Probability

REFERENCE: Burchett, J. L. and Rizenbergs, R. L., "**Frictional Performance of Pavements and Estimates of Accident Probability**," *Pavement Surface Characteristics and Materials, ASTM STP 763,* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 73-97.

ABSTRACT: Objectives of this study were to evaluate standard and experimental surfaces throughout Kentucky in terms of skid resistance and effects of traffic, and to provide criteria for judging suitability of these surfaces to satisfy requirements for skid resistance and economics. The effects of traffic were quantified by regression analysis and scatter of data. Criteria included an estimate of accident risks, effects of speed on skid resistance, and seasonal variations in skid resistance.

Pavements on low volume roads (less than 1000 vehicles per day) maintained adequate skid resistances. Open-graded friction courses, with the possible exception of sections using phosphate slag aggregate, maintained adequate skid resistance to meet design requirements. The adequacy of other pavements may be judged from the criteria provided herein.

Estimates of accident reduction were made by combining the relationship between skid numbers and accidents with the distribution of skid numbers for each pavement type. Those reductions were used to calculate benefits that, along with costs of overlay, were used to determine benefit-cost ratios. Benefits exceeded costs for roads having annual average daily traffic (AADT) greater than 750, 2500, and 5000 and skid numbers (SN) less than 24, 30, and 35, respectively.

KEY WORDS: skid resistance, friction, pavements, wet-pavement accidents, aggregates, surfaces, polishing, wear, traffic, pavement surface characteristics

Reduction in skid resistance of pavements is related to loss of macrotexture and to polishing of aggregates. Loss of macrotexture is caused by wear and, in case of asphaltic concrete, consolidation induced by traffic. Skid resistance varies with cumulative traffic [*1*].[2] Variances are influenced by and attributed to differences in volume and composition of traffic, aggregate

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[2] The italic numbers in brackets refer to the list of references appended to this paper.

types (polish susceptibility), and weathering (including frequency and duration of rainfall). Variations occur seasonally [2] and are affected by traffic volume. Survey testing of roads in Kentucky since 1974 has generated data to evaluate the performance of several types of pavements. These include Class I bituminous, portland cement concrete, and Kentucky rock asphalt. Other types of pavement with up to about 12 million vehicular passes include sand asphalts and open-graded friction courses.

Degrees of hazard are related to needs for traction and, therefore, to speed and density of traffic, turning and stopping movements, and roadway geometrics. Indeed, there are degrees of risk associated with highway hazards. Nevertheless, expedient judgments are being made in regard to the significance or meaning of skid numbers (SN). Critical SN's have been derived for interstate and toll roads [3] and for principal two-lane roads (U.S. routes) [4]. Speed limits were reduced from 97 km/h (60 mph) (daytime) to 88 km/h (55 mph) in March 1974. The relationship between accidents and pavement friction, therefore, may have been altered. A study of those aftereffects is going forward. Preliminary results from two-lane roads [about 8000 km (5000 miles)] are presented herein.

Perhaps the surface providing the highest SN's may seem desirable, to minimize risks. Otherwise, minimizing risks must be balanced with benefits to obtain the greatest safety with monies available. Thus, final criteria for adequacy of surface courses must include a best-good-for-all approach and priority-type programming; and the criteria may be different for various classes of roads.

Data Sources

Skid Test

Beginning in 1974, testing was done from June through November. Skid tests were made with two two-wheel trailers. The equipment, methods, and procedures have been described previously [5, 6]. Some pavements were tested at 40 and 88 km/h (25 and 55 mph), and all pavements were tested at the standard test speed of 64 km/h (40 mph).

Accident Information

Accidents reported during calendar years 1976 and 1977 were used in conjunction with SN's obtained in 1975 and 1976, on two-lane roads, to determine a relationship between wet-pavement accidents (as a percentage of total accidents) and SN's. Accidents for the two-year period totaled 29 783—of which 5930 occurred during wet-pavement conditions—on 1209 sections. Accidents reported during 1979 were used in conjunction with SN's obtained during 1977 and 1978, on two-lane roads, to determine wet-pavement accidents per km (mile) per year to ascertain potential benefits from deslicking. Accidents totaled 16 533—of which 3785 occurred during wet-pavement conditions—on 1132 sections.

Precipitation

Precipitation data were obtained from monthly tabulations of local climatological data [7] for seven weather stations in and around Kentucky. Yearly averages of precipitation in Kentucky since 1969 are presented in Table 1.

Traffic Volume

Annual average daily traffic (AADT) was determined for each pavement section using traffic flow maps published biennially. The 1975 AADT's were used with accidents occurring during 1976 and 1977 on sections tested in 1975 and 1976. The 1977 AADT's were used with accidents during 1979 on sections tested in 1977 and 1978. The AADT's also were used to calculate cumulative traffic.

Procedures

Cumulative Traffic Calculations

For two-lane roads, the cumulative traffic was calculated from the AADT value, divided by two, times the number of days in the time frame. For four- and six-lane roads, the values were adjusted according to lane distribution factors reported by Pigman and Mayes [8]. All values were as of the date of test. No weighting factors for trucks were applied.

TABLE 1—*Percent of time of precipitation in Kentucky (trace or more).*

Year	Rainfall	Snow and Ice	No Precipitation
1969	11.5	2.6	85.9
1970	11.5	3.1	85.4
1971	10.5	2.4	87.1
1972	14.3	2.3	83.4
1973	13.1	2.3	84.6
1974	13.8	2.4	83.8
1975	13.5	2.4	84.1
1976	9.9	2.1	88.0
1977	10.1	3.9	86.0
1978	11.5	4.2	84.3
1979	13.8	3.9	82.2
All	12.1	2.9	85.0

An effective AADT was determined for each pavement section by dividing the cumulative traffic by the number of days the pavement was open to traffic. The effective AADT then is the average number of vehicles per day that traversed the pavement.

Regression Analysis

The relationships between skid resistance and cumulative traffic were determined by regression analysis and the method of least squares. Previous research [*1*] had shown that skid resistance could be related to the logarithms of cumulative traffic; therefore, a logarithmic equation was used here as the model. Cumulative traffic was expressed in terms of millions of vehicle passes. New surfaces subjected to little or no traffic yielded spurious skid numbers. For this reason, data associated with cumulative traffic of less than 0.1 million vehicle passes were omitted from the regression analysis.

Preliminary analysis of Class I bituminous and portland cement concrete pavements indicated the best-fit equations were influenced unduly by sections having low volumes of traffic. Cumulative traffic for these low-volume sections was also low, and SN's were high. This resulted in best-fit equations that predicted unduly low skid numbers at high values of cumulative traffic. For this reason, data were grouped by effective AADT, and the performance equations were determined for each group. Also, scatter of data at low values of cumulative traffic was greater than at high values. Thus, the standard errors of estimate, from regression analyses, were not an appropriate indicator of scatter throughout the range of cumulative traffic. Instead, standard deviations were determined using data stratified by cumulative traffic.

The first part of the procedure to determine standard deviation was to establish a data set representing the differences between measured SN and predicted SN. These differences were then grouped in a five-point moving average beginning with the five highest cumulative traffic values. The standard deviation of the differences in SN and the average cumulative traffic were determined for this group of five points. The data for the highest cumulative traffic were then dropped, and the sixth highest value was added. Again, the standard deviation of the differences in SN and the average cumulative traffic were determined. The procedure continued until the last group consisted of data associated with the five lowest values of cumulative traffic.

A multiple of the calculated standard deviation was subtracted from the SN predicted for the average cumulative traffic. This was done for each five-point group and resulted in a set representing a lower limit of SN's above which a known percentage of measured SN's occurs. The percentage depends on the multiple of the standard deviation. Here, a multiple of 2.5 was used to establish SN levels that should be exceeded by 99.4 percent of the measured SN's. Additional analysis was done to determine a multiple, and consequently a percentage, for predetermined levels of SN's. Relationships between the lower limit of SN's and cumulative traffic were determined.

Criteria for Prequalifying Pavements

Friction Requirements

The relationship between percentage of accidents on wet pavements and SN's on two-lane roads [about 8000 km (5000 miles)] is presented in Fig. 1. Here, the points represent averages of groupings by two SN's. The data were fitted by regression analysis such that the line would indicate nearly 100 percent at SN of 0. Also, at high values of SN, the percentage of wet-pavement accidents would be at least as high as the percentage of time the pavements were wet. In Kentucky, for the two-year period of accident statistics (1976 and 1977) included in the analysis, this percentage was 10, but it was adjusted to 12. Regression equations were determined for various percentages at which the curve becomes asymtotic. The best-fit line indicated that, even if skid resistance remained equivalent to dry-pavement values, wet-pavement accidents comprised 16 percent of the total. This four-percentage point increase (from 12 percent wet time) resulted because reduced visibility, road-

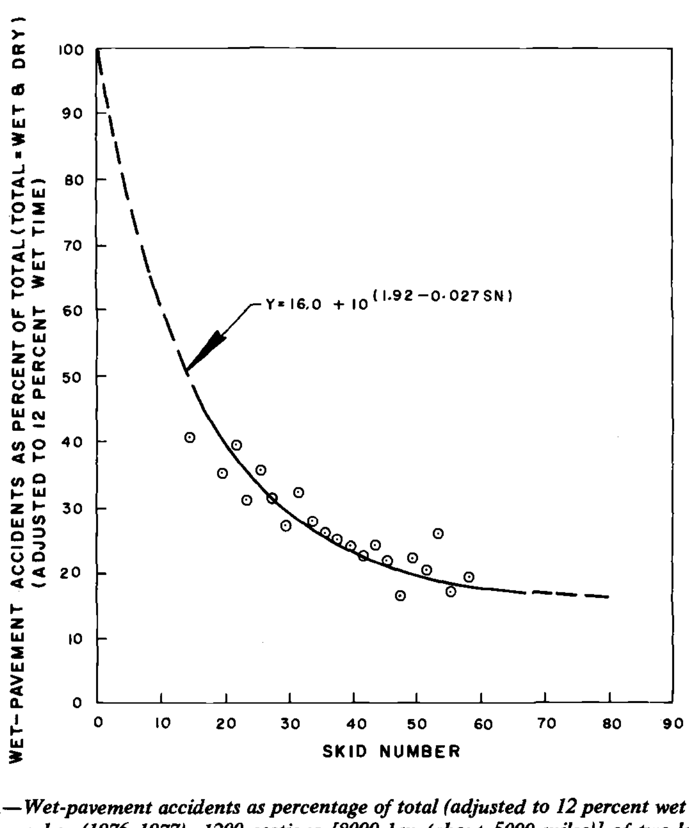

FIG. 1—*Wet-pavement accidents as percentage of total (adjusted to 12 percent wet time) versus skid number (1976–1977); 1200 sections [8000 km (about 5000 miles)] of two-lane roads; grouped by two skid numbers: AADT's above 750.*

way spray, and hydroplaning contributed to accidents. The data were greatly scattered. Thus, use of this trend line for evaluating specific locations must be in conjunction with other supporting statistics, such as occurrences of accidents or, perhaps, number of conflicts.

The best-fit line of Fig. 1 may be used to determine the increased risk of an accident being a wet-pavement accident at SN's less than the equivalent dry-pavement values. For example, from Fig. 1 at SN 60, 18 percent of the accidents occurred on wet pavements; this was a 12.5 percent increase from the 16 percent that occurred for equivalent, dry-pavement values. Likewise, at SN 27, 32 percent of the accidents occurred on wet pavements; this was a 100 percent increase from the 16 percent that occurred on dry pavements.

Of course, pavements cannot be maintained at SN's equivalent to dry-pavement values; and obtainable levels of skid resistance for new pavements must be selected on the basis of other criteria. Moreover, the relationship shown in Fig. 1 indicates the desirability of establishing a maximum risk for existing pavements and provides a means of assessing the relative consequences. The selection of maximum risk must be tempered with realism. For example, a maximum increased risk of only 50 percent (SN 38) would mean that over one-half of the road mileage (AADT more than 1000) would not qualify [almost 8000 km (5000 miles)]. However, a maximum risk of 91 percent (SN 28) would mean that a more manageable six percent of the road mileage would not qualify [almost 1000 km (600 miles)].

Present criteria for identifying pavements in need of deslicking [*9*] specify that any highway section with an AADT greater than 1000 should be deslicked if the SN of the pavement is 28 or less. A total of 1011 km (632 miles) of state roads met this criterion. In addition, highway sections with SN's between 29 and 32 were selected if accident experience indicated a wet- to dry-pavement accident ratio of at least 0.30. These sections totaled an additional 48 km (30 miles). As efforts to deslick candidate roads are successful, increasing the minimum SN allowed on existing pavements may be feasible.

Based on these present criteria for identifying existing pavements in need of deslicking, the criterion for new pavements, for this category of road, was set to prevent future occurrences. The criterion specifies that the mature SN of a surface, at −2.5 standard deviations (99.4 percent assurance), must exceed 32.

Pavement Life

Judgments of the suitability of surfaces must include a consideration of service life and traffic volumes to determine when an SN is a mature value. A surface, during its life, may provide suitable SN's for a road with low traffic volume, but may not be adequate for a road with high traffic volume. For example, if a pavement provides adequate SN through 10 million vehicle passes and its service life is estimated as 12 years, the pavement is suitable for use on roads with traffic volumes as high as 4600 vehicles per day (average volume for the 12 years). At lower traffic volumes, the pavement would age 12 years prior to accumulating 10 million vehicle passes. At higher traffic volumes, the surface may exhibit SN's of 32 or less before reaching the 12 years of life and, thus, may require a premature (or planned) surface renewal.

The useful life of an overlay depends on such variables as type and thickness of the overlay, traffic volume, numbers and types of trucks, and weather conditions [*10*]. The useful life of an overlay ends when it becomes unusually slick, rough, cracked, or rutted. Predicting the number of years when any of these failing conditions will occur is quite difficult. The actual term of service ends when the pavement is resurfaced again or when the road is abandoned.

Effects of Speed

Another characteristic to be considered in judging the suitability of surfaces is the relationship between skid resistance and speed. Skid resistance decreases with increasing speed. Many of the pavements were tested at 40 km/h (25 mph) and 88 km/h (55 mph). A representative curve for each pavement type is shown in Fig. 2. The decrease in skid resistance, from 64 km/h (40 mph) to 88 km/h (55 mph), ranged from a high of 10 SN's on portland cement concrete (burlap drag texturing) to 3 SN's on open-graded friction courses.

A vast majority of pavements involved in the study to relate accidents with pavement friction on two-lane roads (see Fig. 1) were Class I bituminous. Therefore, surfaces for high-speed roads, and with decrease in skid resistance lower than for Class I, may be viewed more positively; and surfaces

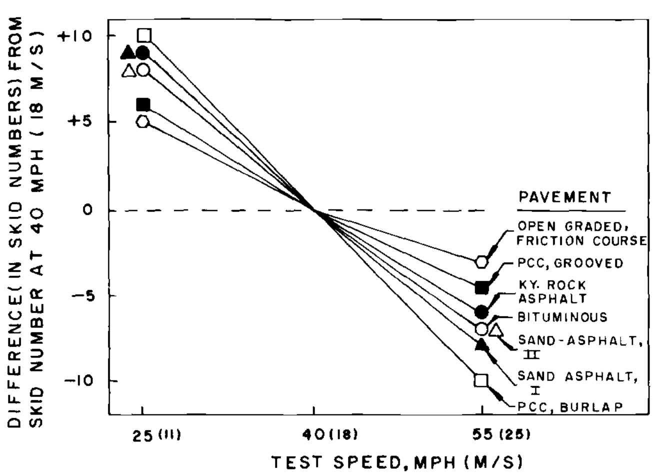

FIG. 2—*Effect of speed on skid resistance of several pavement types.*

with higher decreases in skid resistance may be viewed more negatively. On the other hand, surfaces for low-speed roads may be viewed more positively if increases in skid resistance, from 64 km/h (40 mph) to 40 km/h (25 mph), are higher.

Seasonal Variations

Evaluations herein were based on tests conducted during the summer and fall. Research has shown that skid resistance varies seasonally and is lowest during the late summer and early fall [*2*]. Class I bituminous surfaces, on higher volume roads, were as much as 14 SN's higher during the winter than during the late summer. Sand asphalt exhibited as much as 11 SN's higher, and portland cement concrete pavements were 5 SN's higher. Data on one section of an open-graded friction course indicated the skid resistance of that surface varied little. A higher SN during the winter and spring is a positive attribute of a pavement and should be considered in the selection of surfaces.

Pavement Performance

Class I Bituminous

Class I bituminous is a densely graded, high-type asphaltic concrete. Limestone was the predominant coarse aggregate in these surfaces. Unfortunately, most, if not all, limestones are susceptible to polishing. The surfaces also contained natural or conglomerate sand in the proportion of not less than 40 percent of the combined aggregate. Mineral composition, gradation, and particle-shape requirements for sand, however, were not specified.

Best-fit equations for the two effective AADT groups of interstate and toll roads are plotted in Fig. 3. Also plotted, for pavement sections with more than 2500 vehicles per day, is the best-fit line representing a lower limit of −2.5 standard deviations. Equations for this and other pavements are presented in Table 2. The estimated SN's, representing medians for each pavement, at 0.1, 1, 5, 10, and 60 million vehicle passes, and the lower limit at 10 million vehicle passes are presented in Table 3.

As mentioned previously, tests on new bituminous surfaces that had experienced little or no traffic yielded spurious skid numbers. Valid tests were conducted only after asphalt coating on the surface of the aggregates, and other contaminates, had worn away. Surfaces having low cumulative traffic had initial SN's in the order of 50 for interstate and toll roads and 45 for U.S. and Kentucky routes. Subsequent loss of skid resistance occurred as the sections accumulated more traffic. Both the cumulative traffic and traffic volume (effective AADT) were significant variables.

For interstate and toll roads with 1000 to 2500 vehicles per day, the SN decreased to 46 after only 1 million vehicle passes and decreased less rapidly to

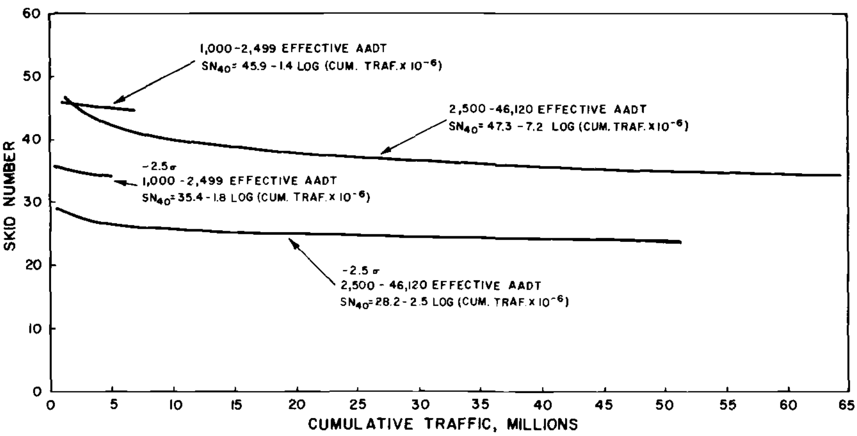

FIG. 3—Effect of traffic on skid resistance of bituminous concrete surfaces (interstate and toll roads).

an SN of 45 at 7 million vehicle passes. The lower limit indicated that 99.4 percent of these sections maintained SN's greater than 34 at 5 million vehicle passes. For surfaces with more than 2500 vehicles per day, the SN was 47 at 1 million vehicle passes, decreased to 40 at 10 million vehicle passes, and decreased to 35 after 60 million vehicle passes. The lower limit indicated that 99.4 percent of these sections maintained SN's greater than 24 at 50 million vehicle passes.

For U.S. and Kentucky routes with 1000 to 2500 vehicles per day, the SN decreased to 42 after 1 million vehicle passes and continued decreasing to 39 at 5 million vehicle passes. The lower limit indicated that 99.4 percent of these sections maintained SN's greater than 28 at 5 million vehicle passes. For surfaces with more than 2500 vehicles per day, the SN was 40 at 1 million vehicle passes and continued decreasing gradually to an SN of 37 at 27 million vehicle passes. The lower limit indicated that 99.4 percent of these sections maintained SN's greater than 21 at 20 million passes.

Portland Cement Concrete

Limestone has been used as the coarse aggregate in most portland cement concrete pavements. Projects on I-75 in northern Kentucky and projects on I-71, however, contained crushed calcareous glacial gravel. Fine aggregates were natural sand, comprising 34 to 40 percent of the combined solid volume of the fine and coarse aggregate. Sections of road containing crushed calcareous glacial gravel aggregate exhibited the same performance histories as sections with limestone aggregate. There was, however, a slight difference in the −2.5 standard deviation. The lower limit for sections of road with

TABLE 2—*Best-fit equations relating skid number and cumulative traffic for various types of pavements.*

Pavement	Effective AADT		Number of Sections	Number of Data Points	Best-Fit Equations, SN = $A + B \times \mathrm{LOG}(CT^a)$			
					Median		Lower Limit[b]	
	Range	Average			A	B	A	B
Class I, bituminous:								
Interstate and toll roads	1 000 to 2 499	1 560	43	83	45.9	−1.4	35.4	−1.8
	2 499 to 46 120	8 380	41	95	47.3	−7.2	28.2	−2.5
U.S. and Kentucky roads	1 000 to 2 499	1 770	100	99	42.3	−4.3	28.2	+1.0
	2 499 to 34 000	5 080	130	132	40.2	−2.0	25.8	−3.1
Portland cement concrete	1 000 to 2 499	2 070	46	68	48.9	−0.6	36.0	−1.5
	2 499 to 38 200	9 490	167	499	49.3	−7.9	22.7	+3.9
Kentucky rock asphalt	1 180 to 7 590	2 950	20	20	57.2	−7.9	51.2	−15.3
Sand-asphalt, type 1	690 to 20 130	8 680	17	58	39.9	−1.2	28.1	−4.2
Sand-asphalt, special provision 59B	4 000 to 14 550	8 900	3	58	47.3	−1.7	35.1	0.0
Sand-asphalt, type 2:								
Rural	300 to 10 560	4 070	16	49	49.3	−3.8	35.8	−2.8
Urban	1 040 to 18 650	8 040	9	39	36.6	−7.0	8.5	+4.6
Open-graded friction course, type 1:								
Green River gravel	2 220 to 19 400	6 610	10	63	48.2	+4.5	38.9	+0.7
Slag	400 to 43 610	12 030	12	48	49.0	−3.7	39.3	−6.2
Gravel	1 100 to 10 400	6 680	6	21	52.8	−4.4	32.2	−4.7
Granite	5 300 to 11 500	6 520	7	6	48.6	+5.4	38.3	+4.3
Type 2: all aggregate	2 400 to 6 900	3 360	6	16	47.1	+1.2	...	...

[a] Cumulative traffic in millions of vehicle passes.
[b] At −2.5 standard deviations.

TABLE 3—*Skid number at several values of cumulative traffic for various types of pavements.*

Pavement	Skid Numbers					
	Cumulative Traffic, Millions					Minus 2.5 σ^b
	0.1	1	5	10	60	
Class I, bituminous, interstate and toll roads:						
AADT 1 000 to 2 499	50	46	45	44[a]	...	34[a]
AADT 2 500 to 46 120	50	47	42	40	35	26
Class I, bituminous, U.S. and Kentucky roads:						
AADT 1 000 to 2 499	45	42	39	38[a]	...	29[a]
AADT 2 500 to 34 000	45	40	39	38	37[a]	23
Portland cement concrete	55	49	48	48[a]	...	34[a]
	55	49	44	41	35	26
Kentucky rock asphalt	...	57	52	49	...	36
Sand-asphalt, type 1	42	40	39	39	...	24
Sand-asphalt, special provision 59B	...	47	46	46	...	35
Sand-asphalt, type 2:						
Rural	54	49	47	45[a]	...	33[a]
Urban	44	37	32	30	...	13
Open-graded friction course, type 1:						
Green River gravel	42	48	51	53	...	40
Slag	53	49	46	45	...	33
Gravel	57	53	50[a]	48[a]	...	28[a]
Granite	45	49	52[a]	...	...	43[a]
Type 2: all aggregate	45	47	48[a]	...	...	...

[a] Extrapolated using best-fit equation.
[b] At 10 million vehicle passes.

glacial gravel aggregate was about one SN less than that obtained for all sections; the values for sections of road with limestone coarse aggregate were the same as for all sections.

Tests of portland cement concrete surfaces (burlap drag texturing) with low values of cumulative traffic indicated initial SN's on the order of 55. Subsequent loss of skid resistance occurred as the roadway sections accumulated traffic. SN's for surfaces with less than 2500 vehicles per day dropped to about 49 after only 1 million vehicle passes, and continued decreasing less rapidly to an SN of 48 after 7 million vehicle passes. For these sections, the lower limit indicated that 99.4 percent maintained SN's greater than 34 at 7 million vehicle passes. For surfaces with more than 2500 vehicles per day, the SN was 49 at 1 million vehicle passes, dropped to 41 at 10 million vehicle passes, and continued decreasing to SN 35 at 60 million vehicle passes. Also, for sections with more than 2500 vehicles per day, the lower limit indicated that 99.4 percent maintained SN's greater than 29 at 60 million vehicle passes.

Kentucky Rock Asphalt

Although Kentucky rock asphalt is not currently being applied in Kentucky, the skid resistance performance is useful for comparison. Twenty projects were tested during 1975. Data were insufficient to determine initial skid resistance. Skid numbers decreased to about 56 after 1 million vehicle passes, and continued decreasing to about 49 at 10 million vehicle passes. The lower limit indicated that 99.4 percent of these sections maintained SN's greater than 36 at 9 million vehicle passes.

Sand Asphalt

Limestone sand obviously reduced the frictional levels of sand-asphalt surfaces constructed prior to 1970 [*1,11*]. Continued study demonstrated that better sands could be selected on the basis of mineral composition, gradation, and particle shape [*12*]. Sand-Asphalt (Skid Resistant), Special Provision 59B, resulted. With continued refinement of mineral composition and gradation, the mixture evolved into Sand-Asphalt Surface, Type 1, and Sand-Asphalt Surface (Skid Resistant), Type 2.

Sand-Asphalt, Type 1—Sand-Asphalt, Type 1, is intended to provide a thin, fine-textured wearing surface from aggregates generally available from commercial sources. This mixture has been used since 1974. Aggregates included natural sand, natural sand with slag sand, natural sand with limestone sand, pit sand with limestone sand, and slag sand with natural sand. Initial SN's were about 42. The best-fit line indicated a decrease in SN to 39 at 10 million vehicle passes. Scatter caused the −2.5 standard deviations to drop to an SN of 24 at 10 million vehicle passes. However, much of the scatter resulted from combining data for different aggregate types.

Sand-Asphalt (Skid Resistant), Special Provision 59B—Three adjacent sections of road were surfaced in 1972 and 1973. The aggregate was crushed quartz gravel. Performance was expected to depend on the degree of crushing and sharpness achieved. SN's were about 47 after 1 million vehicle passes, and decreased slowly to about 45 at 10 million vehicle passes. The lower limit, determined by regression analysis, indicated that 99.4 percent of the SN's were greater than 35 up to 10 million vehicle passes.

Sand-Asphalt (Skid-Resistant), Type 2—Sand-Asphalt (Skid Resistant), Type 2, is fine-textured and has been used since 1974. Aggregates included slag sand, slag sand with natural sand, quartz sand, quartz sand with mortar sand, and crushed gravel sand with crushed limestone sand. Several projects yielded low SN's. The data were divided into two groups, urban and rural. Almost all of the low SN's were in urban areas. The reasons remain a point of conjecture and require further study. Sections in rural areas had initial SN's near 54, and maintained SN's near 46. The lower limit indicated that 99.4 percent of the sections had SN's greater than 34 at 4 million vehicle passes. Sections in urban areas had initial SN's near 44, and maintained SN's near 30. The lower limit for these sections was less than 14 at 10 million vehicle passes.

Open-Graded Friction Courses

Open-graded friction courses were first used in Kentucky in 1973. Since then, over 50 sections have been paved. Most of these sections were Type 1—allowing aggregate sizes up to 13 mm (½ in.). Six of the sections were Type 2—allowing aggregate sizes up to 10 mm (⅜ in.). Aggregate included crushed quartz gravel, crushed quartz gravel with limestone aggregate, crushed slag, crushed granite, crushed conglomerate gravel, crushed conglomerate gravel with limestone aggregate, limestone with crushed gravel, and limestone aggregate with crushed granite.

For Type 1 OGFC using crushed quartz gravel (Green River) aggregate, with and without limestone aggregate (Fig. 4), initial SN's were less than 43. SN's increased to 48 at 1 million vehicle passes and to 53 at 10 million vehicle passes. The lower limit was initially lower but improved to 40 after 10 million passes.

For Type 1 OGFC using crushed slag with and without limestone aggregate, initial SN's were about 53. After 1 million passes, the SN had decreased to 49. Mature values were about 44. There was sufficient scatter of data to result in a lower limit SN of 32 at 15 million passes.

For Type 1 OGFC using crushed gravel aggregate with and without limestone, initial SN's varied considerably—from 37 to 68—with an average of about 54. They maintained an average SN near 50. The lower limit was an SN of 29 at 5 million passes.

Data for Type 1 OGFC using crushed granite were limited, but indicated mature SN's greater than 50 and a lower limit of about 40.

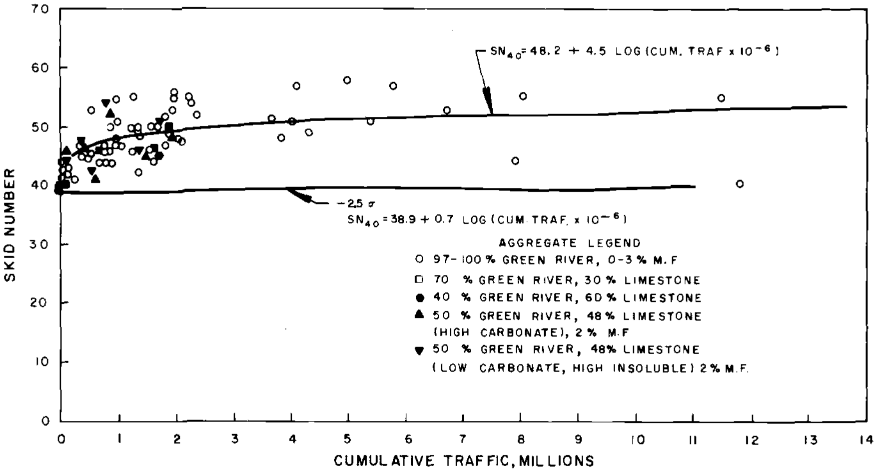

FIG. 4—*Effect of traffic on skid resistance of open-graded friction courses, Type 1: crushed quartz gravel (Green River).*

The limited data for Type 2 OGFC, for all aggregate types, indicated mature SN's near 47. Data were insufficient to determine a lower limit.

At three locations, sections of the Type 1 OGFC were placed without limestone aggregate, and adjacent sections included limestone aggregate. The SN's at 2 million vehicle passes for each of the nine sections are plotted in Fig. 5. Limestone aggregate reduced the skid resistance of the Type 1 OGFC using crushed quartz gravel (Green River) by 1.64 SN for each 10 percent of limestone aggregate in the mixture. The low-carbonate, high-insoluble limestone performed slightly better than average, reducing the skid resistance by 1.25 SN for each 10 percent of limestone aggregate in the blend. The high-carbonate limestone aggregate performed slightly worse than average, reducing the skid resistance by 2.0 SN for each 10 percent of limestone aggregate in the blend. Limestone aggregate reduced the skid resistance of the Type 1 OGFC using crushed bank gravel by 1.33 SN for each 10 percent of limestone used.

Benefits and Costs

Benefits

Benefits herein were derived from calculations of the reduction in the number of wet-pavement accidents. The reduction depends on the previous SN of the road, the SN after deslicking, and the traffic volume. To quantify these relationships, wet-pavement accidents per kilometre (mile) during 1979, on roads skid tested in 1977 and 1978, were analyzed. The data were stratified

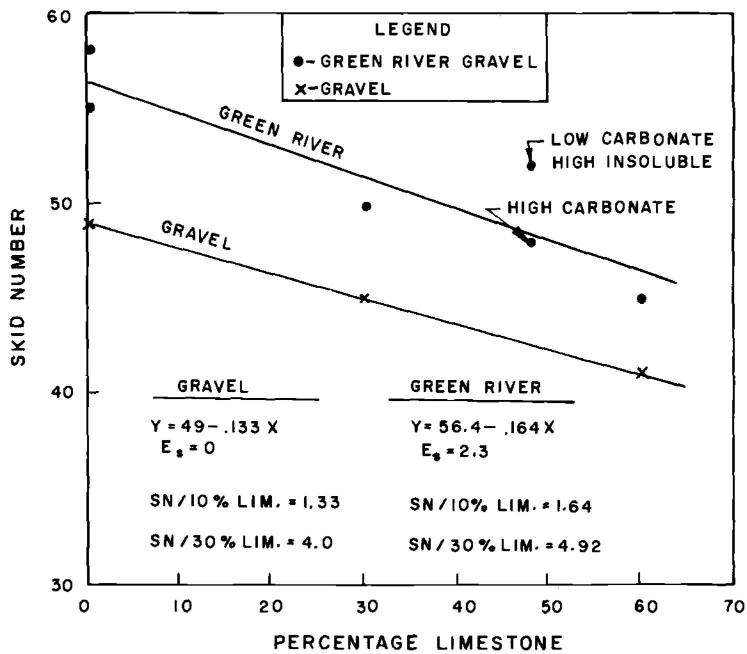

FIG. 5—*Effect of limestone sand on skid resistance of open-graded friction courses, Type 1, at two million vehicle passes.*

by AADT and each AADT group was subdivided, by equal number of wet-pavement accidents, into six groups. The resulting values and best-fit lines are shown in Fig. 6. Here, SN was the independent variable. The three curves were also converted so that traffic volume was the independent variable, and the resulting family of curves is shown in Fig. 7. These relationships show, for example, that, if a road with an SN of 20 and AADT of 8000 were de-slicked and improved to an SN of 40, wet-pavement accidents would be reduced from 2.8 to 1.0 per kilometre (4.4 to 1.5 per mile) per year, and result in a benefit of $11 700 per km ($18 850 per mile) per year. The average cost of a wet-pavement accident was calculated based on accidents on rural, two-lane roads in Kentucky and cost of fatal, injury, and property-damage-only accidents cited by the National Safety Council [*13*].

Performance evaluation of pavements has shown that SN's obtained vary considerably for each type of surface. Thus, to ascertain benefits expected, the deviations of SN's expected must be included. The analysis used the mean SN and three standard deviations for each surface at the cumulative traffic corresponding to the half-life of the pavement (see Table 4) and at the three levels of AADT cited in Fig. 6. These distributions were combined with the curves of Fig. 6 to yield the number of wet-pavement accidents per kilometre (mile) per year expected after surface renewal. This value was sub-

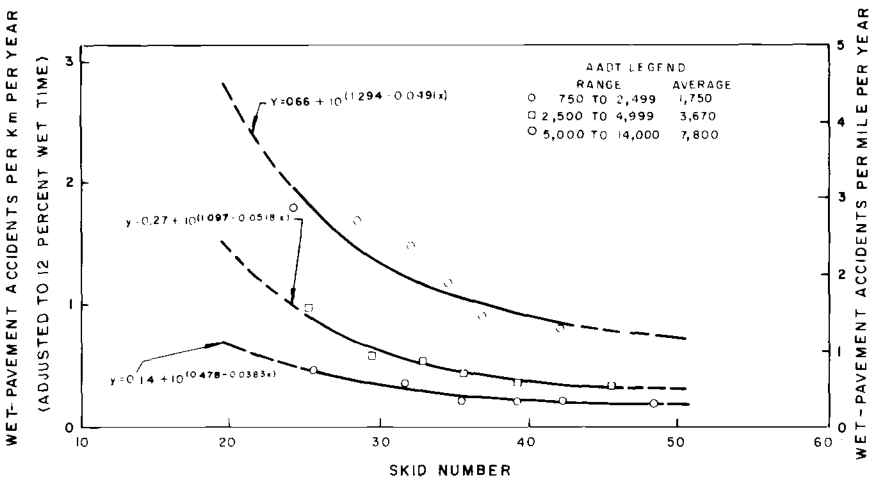

FIG. 6—*Wet-pavement accidents per kilometre (mile) per year (adjusted to 12 percent wet time) versus skid number (1979); 1132 sections [7100 km (about 4400 miles)] of rural, two-lane roads; stratified by AADT; grouped by equal number of wet-pavement accidents.*

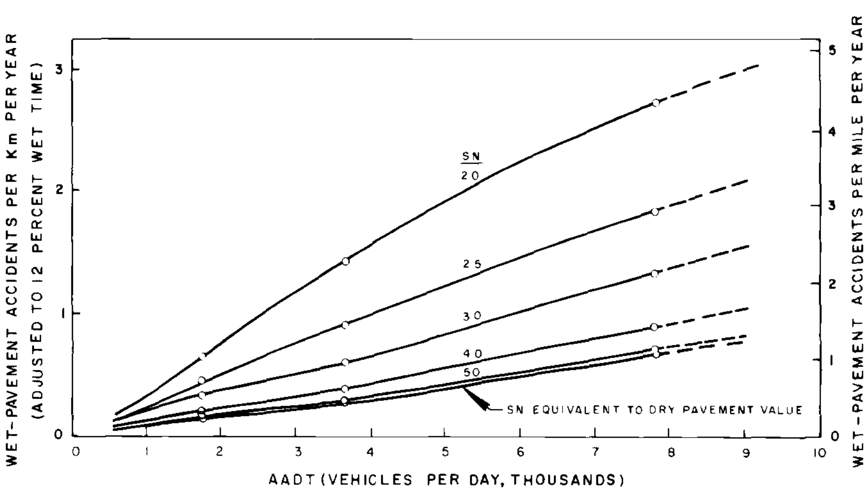

FIG. 7—*Wet-pavement accidents per kilometre (mile) per year (adjusted to 12 percent wet time) (1979), from best-fit curves of Fig. 6, versus traffic volume.*

tracted from the number of wet-pavement accidents before surface renewal, indicated in Fig. 6, for several SN's. The differences times the average cost of a wet-pavement accident in Kentucky in 1979 ($6500) yielded the benefits per kilometre (mile) per year.

TABLE 4—*Mean skid number of pavements and standard deviation at indicated cumulative traffic.*

AADT range	750 to 2 499		2 500 to 4 999		5 000 to 14 000	
AADT average	1 750		3 670		7 800	
Half-life, years	7.1		6.1		4.1	
Cumulative traffic, millions	2.3		4.1		5.8	
Pavement	Mean SN	Standard Deviation	Mean SN	Standard Deviation	Mean SN	Standard Deviation
Class I, bituminous, interstate and toll roads:						
AADT 1 000 to 2 499	45.4	4.3	...	...	...	...
AADT 2 500 to 46 120	...	...	42.9	6.5	41.8	6.2
Class I, bituminous, U.S. and Kentucky roads:						
AADT 1 000 to 2 499	40.7	4.9	...	...	...	...
AADT 2 500 to 34 000	...	...	39.0	6.1	38.7	6.1
Sand-asphalt, type 1	39.5	5.2	39.2	5.5	39.0	5.6
Sand-asphalt, type 2:						
Rural	47.9	5.2	47.0	5.2	46.4	5.1
Urban	34.1	9.6	32.3	8.4	31.3	7.7
Open-graded friction course, type 1:						
Green River gravel	49.8	4.2	51.0	4.6	51.6	4.8
Slag	47.7	4.3	46.7	4.5	46.2	4.7
Gravel	51.2	8.3	50.1	8.3	49.4	8.3
Granite	50.6	4.3	51.9	4.4	52.7	4.4

Costs

Initial costs (1979 dollars) of the various surface courses for the minimum thicknesses required for surface renewal are cited in Table 5. Those costs are for a 7.3-m (24-ft)-wide two-lane road, and do not include leveling or other incidental work.

The other input for determining cost is the estimated service life. The service life, for the ranges of AADT cited in Fig. 6, was estimated as 14, 12, and 8 years, for low, medium, and high values of AADT, respectively. Dividing the cost by the estimated life and allowing a 10 percent cost per year of money gave the estimated costs per kilometre (mile) per year, as cited in Table 5.

Benefits-Cost Analysis

The benefit-cost ratios (Table 6) indicated that use of any surface is cost effective for surface renewal if the AADT is more than 2500 and the existing SN's are low. In fact, if an existing pavement with an AADT of 5000 or more has a SN as high as 34, application of an overlay using Open-Graded Friction Course, Type 1, yields a ratio of 1.0. Thus, from a cost-effectiveness perspective, efforts should continue, as in the past, to use available monies to deslick pavements with the lowest SN's to reduce wet-pavement accidents the most and achieve the greatest benefits.

In most cases, surfaces yielding the highest benefit-cost ratios should be selected for the overlay. Open-graded friction course provides the best ratio. However, if future costs change or vary for certain locations, another surface may be selected. Additionally, other considerations may warrant selection of other surfaces. Ultimately, benefit and cost information should be an input to priority programming of a pavement management system.

Summary

The estimated SN's at 0.1, 1, 5, 10, and 60 million vehicle passes (Table 3) represent median values for each type of pavement; half of the sections had higher SN's and half lower SN's. The highest median SN's were for Open-Graded Friction Course, Type 1, with crushed Green River gravel. The other pavements had SN's of 38 or higher, except for Sand-Asphalt, Type 2, constructed in urban areas. The SN's at −2.5 standard deviations represent values that are exceeded by 99.4 percent of the paving projects. These values are presented in Fig. 8, and provide an indication of worst-case performance.

The criterion for new pavements specifies that the mature SN of a surface, at −2.5 standard deviations (99.4 percent assurance), must exceed 32. Class I bituminous and portland cement concrete pavements (burlap drag texturing) with AADT's more than 2500 and Sand-Asphalt (Type 1) pavements did not provide the necessary assurance of SN's greater than 32. Class I bitumi-

TABLE 5—*Cost of various surface courses.*

Surface Mix	Coarse Aggregate	Thickness, mm (in.)	Tons per km (mile)	Dollar Cost (1979)		Costs per km (mile) per Year, Thousand Dollars		
				per Ton (ton)	per km (mile)[a]	Low AADT	Medium AADT	High AADT
Class I, bituminous	limestone	25 (1)	436 (774)	22.12 (24.38)	11 725 (18 866)	3.20 (5.14)	3.08 (4.95)	3.12 (5.02)
Class I, bituminous	crushed gravel	25 (1)	432 (767)	21.34 (23.52)	11 214 (18 043)	3.06 (4.92)	2.94 (4.74)	2.98 (4.80)
Class I, bituminous	slag	25 (1)	405 (718)	20.75 (22.87)	10 206 (16 422)	2.78 (4.48)	2.68 (4.31)	2.71 (4.37)
Class AA, bituminous	crushed gravel	25 (1)	436 (774)	26.67 (29.40)	14 143 (22 756)	3.85 (6.20)	3.71 (5.98)	3.76 (6.05)
Open-graded friction course	various	19 (3/4)	258 (458)	26.76 (29.50)	8 397 (13 511)	2.29 (3.68)	2.21 (3.55)	2.23 (3.59)
Sand-asphalt, type 1	various	16 (5/8)	258 (458)	29.20 (32.19)	9 162 (14 741)	2.50 (4.02)	2.41 (3.87)	2.44 (3.92)
Sand-asphalt, type 2	various	16 (5/8)	258 (458)	27.08 (29.85)	8 497 (13 671)	2.32 (3.73)	2.23 (3.59)	2.26 (3.63)

[a] Cost for a 7.3-m (24-ft)-wide, two-lane road; does not include leveling or other incidental work.

TABLE 6—*Benefit-cost ratios from overlaying.*

Pavement	AADT[a]	Wet per Kilometre (mile) Expected[b]	Skid Number Before Overlaying										
			20	22	24	26	28	30	32	34	36	38	40
Class I, bituminous, interstate and toll roads	low	0.20 (0.32)	0.9	0.8	0.6	0.5	0.4	0.3	0.3	0.2	0.1	0.1	0.1
	medium	0.40 (0.64)	2.2	1.7	1.3	1.0	0.7	0.5	0.4	0.3	0.2	0.1	0.0
	high	0.92 (1.48)	3.8	3.0	2.3	1.7	1.3	0.9	0.6	0.4	0.2	0.1	0.0
Class I, bituminous, U.S. and Kentucky roads	low	0.25 (0.40)	0.9	0.7	0.6	0.4	0.3	0.3	0.2	0.1	0.1	0.0	...
	medium	0.44 (0.70)	2.1	1.6	1.2	0.9	0.6	0.4	0.3	0.1	0.0	0.0	...
	high	1.00 (1.60)	3.6	2.8	2.1	1.5	1.1	0.7	0.4	0.2	0.0	...	...
Sand-asphalt, type 1	low	0.25 (0.40)	1.1	0.9	0.7	0.5	0.4	0.3	0.2	0.1	0.1	0.0	0.0
	medium	0.42 (0.68)	2.7	2.1	1.6	1.1	0.8	0.6	0.4	0.2	0.1	0.0	...
	high	0.98 (1.56)	4.0	3.6	2.7	2.0	1.4	1.0	0.6	0.4	0.1	0.0	...
Sand-asphalt, type 2													
Rural	low	0.20 (0.32)	1.3	1.1	0.9	0.7	0.6	0.5	0.4	0.3	0.2	0.2	0.1
	medium	0.33 (0.53)	3.2	2.5	1.9	1.5	1.1	0.9	0.6	0.5	0.3	0.2	0.1
	high	0.80 (1.28)	5.6	4.4	3.4	2.6	2.0	1.6	1.2	0.9	0.6	0.4	0.3

Urban	low	0.39 (0.63)	0.9	0.6	0.4	0.3	0.1	0.0	...	...	...	...	...
	medium	0.77 (1.21)	2.1	1.4	0.8	0.4	0.0	...	...	...	...	...	...
	high	1.56 (2.50)	3.5	2.3	1.4	0.6	0.0	...	...	...	...	...	...
Open-graded friction course, type 1													
Green River gravel	low	0.17 (0.27)	1.4	1.1	0.9	0.8	0.6	0.5	0.4	0.3	0.3	0.2	0.1
	medium	0.30 (0.48)	3.3	2.6	2.0	1.6	1.2	0.9	0.7	0.5	0.4	0.3	0.2
	high	0.73 (1.17)	5.8	4.6	3.6	2.8	2.2	1.7	1.4	1.0	0.8	0.6	0.4
Slag	low	0.20 (0.32)	1.3	1.1	0.9	0.7	0.6	0.5	0.4	0.3	0.2	0.2	0.1
	medium	0.34 (0.54)	3.2	2.5	2.0	1.5	1.2	0.9	0.6	0.5	0.3	0.2	0.1
	high	0.81 (1.29)	5.6	4.4	3.4	2.7	2.1	1.6	1.2	0.9	0.6	0.4	0.3
Gravel	low	0.18 (0.29)	1.4	1.1	0.9	0.7	0.6	0.5	0.4	0.3	0.2	0.2	0.1
	medium	0.32 (0.51)	3.3	2.5	2.0	1.5	1.2	0.9	0.7	0.5	0.4	0.3	0.2
	high	0.78 (1.25)	5.6	4.4	3.5	2.7	2.1	1.6	1.2	0.9	0.7	0.5	0.3
Granite	low	0.17 (0.27)	1.4	1.1	0.9	0.7	0.6	0.5	0.4	0.3	0.3	0.2	0.2
	medium	0.29 (0.47)	3.3	2.6	2.0	1.6	1.2	0.9	0.7	0.6	0.4	0.3	0.2
	high	0.72 (1.15)	5.8	4.6	3.6	2.9	2.2	1.8	1.4	1.1	0.8	0.6	0.4

[a] Refer to legend of Fig. 6.

[b] Wet-pavement accidents per kilometre (mile) per year after overlaying.

NOTE—Benefits from reduction of wet-pavement accidents only.

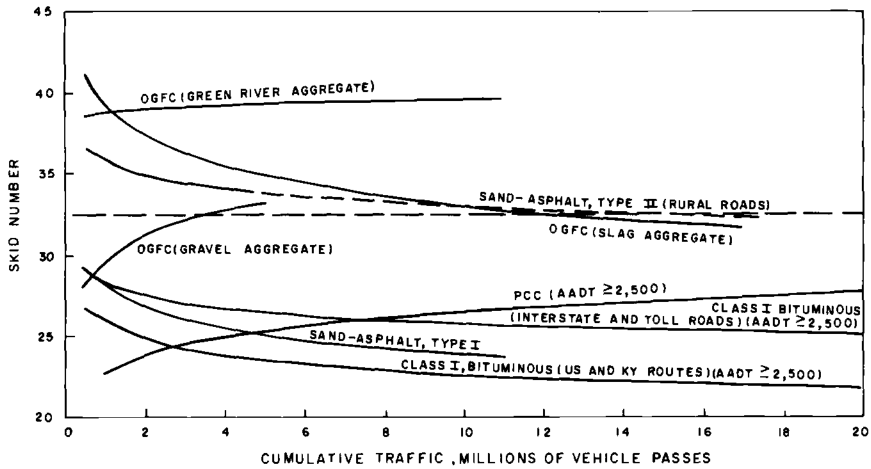

FIG. 8—*Minus 2.5 standard deviations (99.4 percent assurance) for several pavement types.*

nous (interstate and toll-road quality) and portland cement concrete, with AADT's less than 2500 (not shown in Fig. 8), provided suitable SN's throughout their lives. Open-graded friction courses with Green River aggregate provided suitable SN's through the number of vehicle passes accumulated to date and, by interpolation, through the life of the pavement. Open-graded friction courses with slag aggregate provided adequate SN's through 12 million vehicle passes. For eight-year service life, this surface is suitable for roads with AADT less than 8200. Conversely, if applied to a road with AADT of 11 000 vehicles per day, the surface may exhibit SN's of 32 or less after only six years, and may require surface renewal at that time. Open-graded friction courses, with other gravel aggregate, provided necessary assurance against low SN's to 1 million vehicle passes; however, data were too limited to allow final assessment. Sand-Asphalt, Type 2, on rural roads, provided adequate SN's through 15 million vehicle passes. For an eight-year service life, this corresponds to an AADT of 10 300.

Service life has been estimated based on AADT (see Table 4). Using current costs of overlay (see Table 5), benefit-cost analyses indicated that overlaying an existing pavement having an SN less than 35 and AADT greater than 5000 yields benefits from reduction of wet-pavement accidents that equal or exceed the cost of overlay. Benefits also exceeded costs for roads with SN's less than 30 and AADT greater than 2500, and for roads with SN's less than 24 and AADT greater than 750. Additional benefits [*10*], which may be included in an expanded analysis, include increased comfort, time savings, fuel savings, maintenance savings, and reduction of other types of accidents.

A minimum SN of 28, for roads with more than 1000 vehicles per day, has

TABLE 7—*Percent of pavement sections with skid number equal to or exceeding selected minimum value at 10 million vehicle passes.*

Pavement	Minimum Skid Number at 10 Million Vehicle Passes																	
	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
Class I, bituminous, interstate and toll roads																		
AADT 1 000 to 2 499	100	100	100	100	100	99	99	98	97	96	93	90	85	79	72	64	55	<50
AADT 2 500 to 46 120	98	98	96	95	93	90	86	82	77	71	65	58	50	<50	...	...	...	...
Class I, bituminous, U.S. and Kentucky roads																		
AADT 1 000 2 499	99	99	98	97	95	92	87	80	72	62	52	<50	...	...	...	...	...	...
AADT 2 500 to 34 000	95	93	90	87	84	80	75	70	64	58	51	<50	...	...	...	...	...	...
Portland cement concrete																		
AADT 1 000 to 2 499	100	100	100	100	100	99	99	98	98	97	95	93	91	88	85	81	76	70
AADT 2 500 to 38 200	99	98	97	96	95	93	90	86	83	78	72	66	60	53	<50	...	...	...
Kentucky rock asphalt	100	100	100	100	100	100	100	100	99	99	98	97	96	94	91	88	84	79
Sand-asphalt, type 1	97	95	93	90	87	85	79	75	68	61	54	<50	...	...	...	...	...	...
Sand-asphalt, special provision 59B	100	100	100	100	100	100	99	99	99	98	96	94	91	86	80	73	65	56
Sand-asphalt, type 2																		
Rural	100	100	100	100	100	99	99	98	97	95	93	90	86	82	76	69	62	54
Urban	61	54	<50	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Open-graded friction course, type 1																		
Green River Gravel	100	100	100	100	100	100	100	100	100	100	100	99	99	99	98	97	95	93
Slag	100	100	100	100	100	100	99	98	97	95	93	90	86	81	75	68	61	52
Gravel	99	99	98	98	97	97	96	95	93	91	89	87	84	81	78	74	70	66
Granite	100	100	100	100	100	100	100	100	100	100	100	100	100	99	99	99	98	97

been recommended to safeguard the public from undue hazards associated with slippery pavements regardless of the accident history of the road. Also, as indicated from the relationship between skid resistance and cumulative traffic, the best surface does not assure mature SN's above 45. Thus, criteria for the design of surface courses concern primarily the range of SN's between 28 and 45. The percentages of pavement sections estimated to equal or exceed, at 10 million vehicle passes, these values were determined (Table 7). At least 95 percent of all pavement sections, except Sand-Asphalt, Type 2 (urban), provided SN's greater than or equal to a SN of 28. However, if the level of skid resistance required is SN of 32 and the desired percentage level is again 95 percent, then Class 1 bituminous (high AADT roads) and Sand-Asphalt, Type 1, in addition to Sand-Asphalt, Type 2 (urban), are not suitable. The percentages are useful for selecting pavement types to meet different requirements and to assure due margin of safety. Other criteria for selecting surface courses include speed effects (see Fig. 2) and seasonal variations in skid resistance.

Acknowledgments

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N. Kamel[1] *and T. Gartshore*[2]

Ontario's Wet Pavement Accident Reduction Program

REFERENCE: Kamel, N. and Gartshore, T., **"Ontario's Wet Pavement Accident Reduction Program,"** *Pavement Surface Characteristics and Materials, ASTM STP 763,* C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 98–117.

ABSTRACT: Ontario's approach to the identification and treatment of black spot highway locations is presented. Highway locations with an excessive rate of wet pavement accidents are identified and ranked utilizing Ontario Ministry of Transportation and Communications computerized accident data files. Criteria for site selection, procedures for subsequent site investigation, and selection of appropriate remedial measures are outlined and discussed.

Rehabilitation of pavements with low friction levels, and experiencing a high rate of wet pavement collisions, has resulted in substantial reductions in accidents. Collision data before and after treatment at various sites are presented.

This paper provides design and performance information on modified bituminous surface course mixes currently used by the Ministry. Such mixes maintain better surface textures and provide longer lasting skid resistance characteristics. These mixes are used for black spot treatments, and in new surface construction on main highways.

KEY WORDS: black spots, skid resistance, wet pavement accidents, pavement friction, macrotexture, microtexture, friction course mixes, polish and wear resistance, pavement surface characteristics

Over the past 20 years, Ontario has been active in the pavement skid resistance research area. Early pavement friction measurements were carried out in 1962 [*1*][3] utilizing the British portable skid tester. Organized high speed skid testing started in 1967 with an Ontario Ministry of Transportation and Communications (MTC)-built brake force trailer meeting the requirements of the American Society for Testing and Materials (ASTM) Test for Skid Resistance of Paved Surfaces Using a Full-Scale Tire (E 274-79) [*2*]. In 1970 Shonfeld's photo interpretation method for pavement texture

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[3]The italic numbers in brackets refer to the list of references appended to this paper.

classification [3-5] was introduced. In 1975, the MTC implemented a program for the erection of "Slippery When Wet" signs and the posting of wet pavement advisory speed limits at highway locations where more than one third of the accidents were occurring under wet conditions. The purpose of the new signs was to provide a temporary measure until the surface deficiencies were corrected.

In the mid-1970s, considerable attention was given to the construction and maintenance of skid-resistant pavements so that wet weather skidding accidents would be reduced and kept to minimum practicable levels. The MTC, in 1974, commenced an extensive program of transverse grooving on slippery concrete pavements at highway locations where an excessive number of wet pavement accidents occurred. As described in earlier reports [6, 7], grooving has resulted, in many instances, in significant reductions in wet pavement accidents. Other surface retexturing methods, using the CMI rotomill and Klarcrete machines, were also evaluated [7].

Two major experimental projects were carried out to develop bituminous surface course mixes with improved skid resistance characteristics for new construction. Eighteen test mixes were constructed on a section of the Highway 401 Toronto Bypass to evaluate modified surface mixes for freeways and other heavy-trafficked main highways [8, 9]. Seventeen other test sections were constructed in 1978 on Highway 7 near Lindsay, Ontario, to evaluate mixes for highways with lower traffic volumes [10]. These efforts led to the identification and adoption of improved bituminous surface course mixes that provide and maintain better textures and longer lasting skid resistance qualities.

In 1978 the MTC implemented systematic procedures for the identification and treatment of highway locations with high rates of wet pavement accidents (black spots). Rehabilitation of pavements deficient in skid resistance, and experiencing an excessive rate of wet pavement collisions, in most cases has resulted in substantial reductions in accidents. It is the objective of this paper to:

1. Outline Ontario's system for skid resistance improvements at black spot highway locations.
2. Present results of accident data before and after treatments at black spots.
3. Provide design and performance information on skid resistant mixes used in Ontario.

Systematic Procedures for Identification and Treatment of Black Spots

The prime objective in establishing such a systematic procedure is to allow identification and review of all black spot locations on the system so that site investigation and treatment may be established on a priority basis. Thus, available funds may be spent at these locations where maximum benefits are likely to be obtained.

In Ontario, a considerable amount of rehabilitation work to improve pavement skid resistance was carried out prior to developing such procedures. This work, however, was confined to grooving existing polished concrete on one major highway with known low skid resistance levels and high wet pavement accident experience [7].

Ontario's procedure for the identification and treatment of black spots was developed and introduced in 1978. The MTC approved implementation of a pilot project to carry out skid resistance improvements at black spot highway locations on the MTC system. The intention of the project was to determine the overall effectiveness of such improvements in terms of cost and accident reductions achieved. Based on the results of this project, it was felt that a future decision would be made to determine whether or not such a program would become a normal routine maintenance activity. The sum of $0.5 million was assigned to carry out the skid resistance rehabilitation work. The procedures are outlined in Fig. 1 and are discussed in the following sections.

Identification of Black Spots

The MTC Traffic Engineering Office produces, on an annual basis, a priority black spot listing for each of the five MTC Operational Regions. A black spot highway location is defined as any 0.10 km of a highway having three or more wet pavement accidents and a ratio of wet to wet-plus-dry accidents equal to or in excess of 30 percent. The MTC computerized accident data files are used with a special computer program to produce the priority black spot listings. Identification of black spots is done by scanning the accident data files.

Ranking of Black Spots

The program ranks the black spots based on evaluation of the cost and benefits of treatment at the location. Assumed rehabilitation costs vary with facility type and are based on a total resurfacing thickness of 95 mm (3¾ in.) of hot mix for freeways, 76 mm (3 in.) for four-lane, and 57 mm (2¼ in.) for two-lane facilities. Resurfacing thickness for ramps is an assumed 38 mm (1½ in.). In all cases the surface course layer is assumed to contain prime quality coarse and fine aggregates.

Resurfacing a road using these thicknesses would have a number of advantages in addition to upgrading pavement skid resistance. These include increasing structural strength, extending the service life of pavement, reducing routine maintenance costs, reducing user operating costs, etc. Benefits from these additional advantages, however, are not taken into account in this

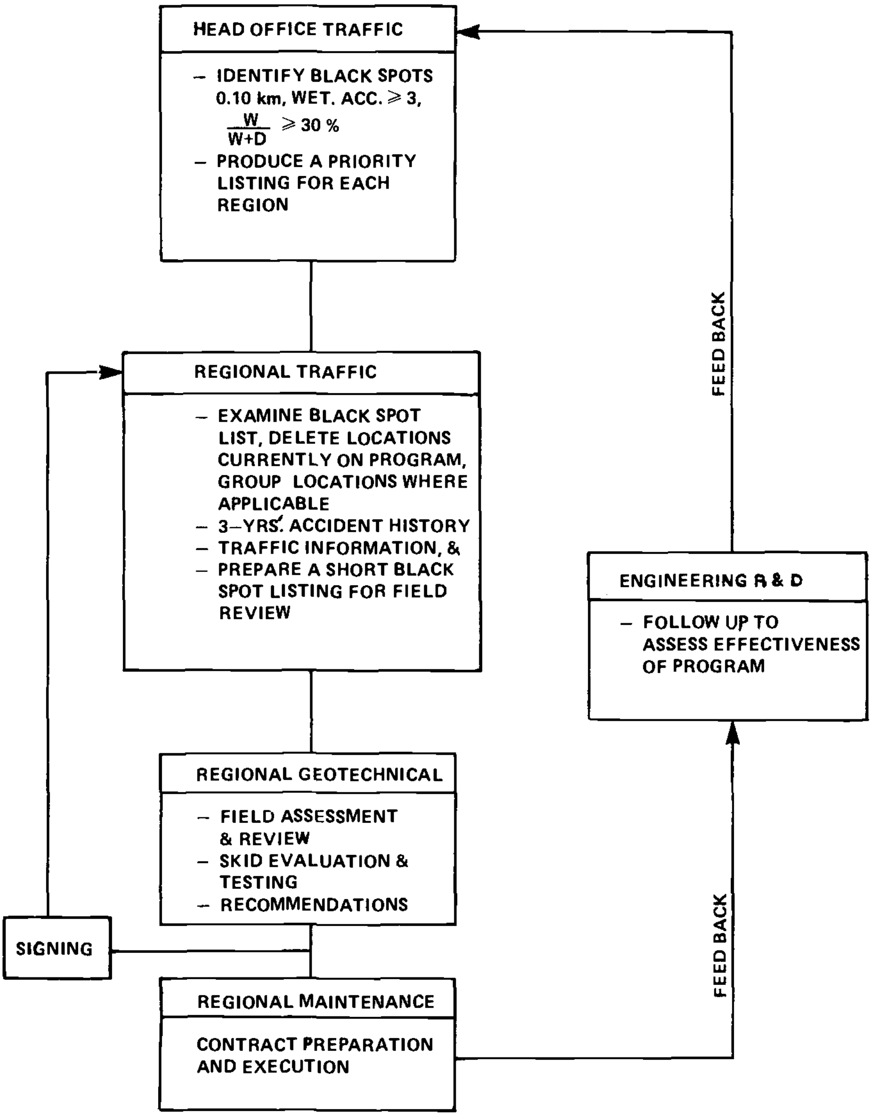

FIG. 1—*Ontario's procedures for black spot treatment program.*

analysis. Only the benefits associated with skid resistance improvements (that is, savings in accident costs due to accident reductions) are evaluated.

For the purpose of estimating rehabilitation benefits, the program assumes a 40 percent average reduction in wet pavement accidents and a 10-year effective service life span of the new pavement. The present value of total savings in wet accident costs over the 10-year period is calculated using a discount rate of 8 percent and the following accident cost figures: \$213 500 per fatal accident; \$5400 per injury accident; and \$1800 per property damage accident. The assumed 40 percent average reduction in wet collisions is selected based upon previous MTC experience.

The benefit/cost ratio of treatment at each black spot is then derived by dividing the present value of total savings in wet accident costs by the estimated rehabilitation cost.

The program produces a listing in descending order of benefit/cost ratios for identified black spots. A typical computer output is shown in Fig. 2.

Analysis of Black Spot Listings

The black spot listings are sent out to the Regional Traffic Sections. The Regional Traffic Section examines the list, deletes locations that are currently on the construction program, groups nearby locations where applicable, and produces a short listing for further consideration. A three-year accident history at each black spot is analyzed along with other data on traffic, road geometry, and alignment, and a site investigation is initiated by completing the top portion (that is, Sections 1, 2, and 3) of the accident site review form shown in Fig. 3. Completed accident site review forms are forwarded to the Regional Geotechnical Office for field assessment and skid resistance evaluation, as indicated in Fig. 1.

Field Assessment and Review

The prime objective of the field review is to determine whether or not pavement skid resistance is a major contributing factor in the excessive rate of wet pavement accidents at the location. The effects of skid resistance and other important factors such as traffic volume, speed limit, congestion, alignment, grade, curvature, etc. are carefully reviewed during the field assessment, and a judgment call is made on whether or not improving skid resistance of the pavement would benefit the reduction of wet weather accidents at the location.

Pavement skid resistance is evaluated using the MTC brake force trailer. In such cases where roadway geometry does not allow the use of the brake force trailer, skid resistance is evaluated by the photo interpretation method [5]. Skid resistance is evaluated by considering the ΔSN, the difference between desirable and existing friction levels. The tentative guidelines shown in Table 1 are used for this purpose.

If the reviewers' judgment shows that rehabilitation to improve pavement skid resistance would be beneficial, recommendations are made and forwarded to the Regional Maintenance Office for execution. If, on the other hand, the outcome of the field review is that higher friction values would be

DATE RUN : 80219

1979-BENEFIT/COST PRIORITY LISTING

FOR WET ACCIDENT REDUCTION PROGRAMME

(COMPLEX INTERCHANGE RAMPS EXCLUDED)

PAGE 1

-- CENTRAL REGION --

HWY NO.	DIST	LOCATION DESCRIPTION (LHRS NO.)	OFFSET	ROADWAY TYPE	WET (W)	DRY (D)	W/(W+D) (%)	# KLD	# INJ	# PDO	ACC-COST	BENEFIT/COST RATIO (10 YEARS)
4030	4	48320	0.0	RAMP 66	9	7	56.3	0	8	4	34200	49.6
4040	6	48490	0.0	RAMP 24	8	8	50.0	0	9	4	28800	41.7
4010	6	47663	0.0	E-EXP	24	9	72.7	0	16	13	82800	41.0
480	6	31020	4.2	2-LAN	8	4	66.7	0	16	4	28800	35.8
10	6	10144	0.0	RAMP 36	4	3	57.1	0	4	1	18000	26.1
4010	6	47661	1.2	E-EXP	16	8	66.7	0	10	10	50400	25.0
10	4	10120	0.0	RAMP 55	3	3	50.0	0	6	0	16200	23.5
60	4	13560	0.0	4-D-S	6	3	66.7	0	10	2	25200	23.4
120	6	19430	0.0	2-LAN	4	3	57.1	0	3	1	18000	22.4
70	6	14390	4.5	2-LAN	4	3	57.1	0	9	1	18000	22.4
4010	6	47658	0.0	W-EXP	12	4	75.0	0	17	6	43200	21.4
10	6	10144	0.0	RAMP 61	4	3	57.1	0	2	2	14400	20.9
70	6	14420	0.0	4-UND	13	17	43.3	0	13	7	45000	20.9
10	4	10105	0.0	RAMP 26	5	5	50.0	0	1	4	12600	18.3
4010	6	47643	0.0	RAMP 51	3	0	100.0	0	4	1	12600	18.3
4010	6	47633	0.0	RAMP 51	3	0	100.0	0	3	1	12600	18.3
4270	6	50219	0.0	RAMP 34	5	5	50.0	0	1	4	12600	18.3
10	6	10150	1.9	W-EXP	8	10	44.4	0	6	3	32400	16.1
4010	6	47673	0.0	RAMP 61	4	6	40.0	0	1	3	10800	15.7
10	4	10123	0.0	RAMP 51	4	0	100.0	0	1	3	10800	15.7
90	6	16100	8.7	2-LAN	3	1	75.0	0	3	1	12600	15.6
270	6	25830	0.0	2-LAN	3	6	33.3	0	4	1	12600	15.6
500	6	31520	1.4	2-LAN	3	2	60.0	0	2	1	12600	15.6
4010	6	47665	0.0	E-EXP	10	9	52.6	0	8	7	28800	14.3
10	4	10135	0.0	W-EXP	10	10	50.0	0	6	7	28800	14.3
270	6	25810	0.0	4-UND	9	6	60.0	0	6	5	30600	14.2
4000	6	46815	0.0	S-EXP	7	2	77.8	0	9	3	27000	13.4
4010	6	47631	0.0	RAMP 61	3	1	75.0	0	1	2	9000	13.0
4010	6	47643	0.0	RAMP 61	3	3	50.0	0	1	2	9000	13.0
4010	6	47660	0.0	RAMP 61	3	2	60.0	0	2	2	9000	13.0

FIG. 2—*Typical computer output for black spot listing.*

Ontario

Ministry of Transportation and Communications

Research and Development Division

WET PAVEMENT ACCIDENTS REDUCTION PROGRAM

ACCIDENT SITE REVIEW

1. LOCATION IDENTIFICATION

Hwy. ________ Dist. ________ Location ______________________________

______________________________ Section Length ________ km.

2. TRAFFIC AND ACCIDENT INFORMATION

(list accident data for last 3 years)

Year	# Dry Acc.	# Wet Acc.	$\frac{W}{W\&D}$ %	Wet Fatal Acc.	Wet Injury Acc.	Wet P.D. Acc.	AADT	Comm. Traffic %

Comments: ______________________________

____

____

3. SITE INFORMATION

No. of Lanes ☐

Facility Type: Divided ☐ Undivided ☐

Speed limit ________ km/h

Stop sign or traffic light within 250 m (800 ft.) ☐ Yes ☐ No

Merge: ☐ Yes ☐ No

Alignment: *(Please see classification next page.)* ☐ Straight ☐ Slight curve ☐ Moderate curve ☐ Sharp curve

Grade: ☐ Slight, <2% ☐ Moderate, 2-4% ☐ Steep, >4%

Surface Type ________

Comments: ______________________________

____

____

____

4. SKID RESISTANCE INFORMATION

Ave. SN at posted speed ________ Required SN ________ ΔSN ________

Comments: ______________________________

____

____

5. Recommendations: ______________________________

____

____

____

____

____

Date ____________

FIG. 3—*Accident site review form.*

TABLE 1—*Tentative guidelines for a friction classification system.*

Facility Type	Speed Limit, km/h	Friction Level (*SN*) at Speed Limit		
		Good	Borderline	Low
Freeways and main highways	100	≥31	25 to 30	<25
2-lane and 4-lane	80	≥32	27 to 31	<27
Intersections	80	≥40	31 to 39	<31
	60	≥45	36 to 44	<36

of little benefit in reducing wet pavement accidents at the location, other measures may be recommended, such as posting of warning or advisory traffic signs, reducing speed limits, etc. Sites with low friction levels, and showing a consistent record of high wet pavement accidents, are given top priority for treatment.

Guidelines for Remedial Measures

The following guidelines are used for pavement rehabilitation at black spot highway locations:

1. Open friction course (OFC) Mix: Used as the surface course layer on urban freeway areas where lateral drainage of the pavement can be provided and where no drainage problem is anticipated throughout the site.

2. Dense friction course (DFC) Mix: Used as the surface course layer at all sites other than urban freeways providing that the annual average daily traffic (AADT) is greater than 2000 per lane or the commercial traffic is in excess of 200 vehicles per day, per lane. DFC is also used at all black spot locations on: (*a*) sharp curves [that is, 4 deg or greater for 100 km/h (60 mph) speed limit, 6 deg or greater for 80 km/h (50 mph) speed limit, and 9 deg or greater for 60 km/h (37 mph) speed limit]; (*b*) steep grades (that is, greater than 4 percent); (*c*) merges; and (*d*) approaches to stop signs and traffic lights.

3. HL1 Mix: Used for sites with AADT of less than 2000 per lane and commercial vehicles less than 200 per lane per day.

Details on MTC gradation requirements and the skid resistance performance of the OFC, the DFC, and the HL1 mixes are given later, in the section on skid resistance mixes.

Implementation of Treatment Work

The Regional Maintenance Office is responsible for the execution of black spot rehabilitation work. Treatment may be carried out as part of an ongoing contract in the area, and may be included in upcoming resurfacing or maintenance work, or as a separate maintenance patching job. At his discretion, the maintenance engineer selects the most advantageous method and proceeds with the implementation.

Followup

Followup activities to monitor skid resistance and accidents before and after rehabilitation and the assessment of the overall effectiveness of the program are the responsibility of the Research and Development Branch.

Summary of Procedures

1. The Head Office of Traffic Engineering produces, on an annual basis, a ranked listing of identified highway locations where an excessive number of wet pavement accidents occur.
2. The Regional Traffic Sections examine the accident location list provided, delete sites where resurfacing or reconstruction work is scheduled, and prepare a short list for further consideration.
3. The Regional Traffic Sections analyze three-year accident records of identified sites and select locations where lack of pavement skid resistance appears to be a major contributing factor to accidents.
4. The Regional Geotechnical Office carries out field inspection and pavement friction measurements. Details of related traffic and geometric data are assembled.
5. The Regional Maintenance Office selects rehabilitation methods and proceed with the implementation of the work.
6. The Research and Development Branch assesses overall effectiveness of the program.

Discussion

The black spot treatment program identified 461 highway locations or 46.1 km (29 miles) of pavements with an excessive rate of wet pavement collisions. Table 2 shows a comparison between wet pavement accidents occurring in 1976 at identified black spots versus those for the remainder of the highway system. As can be seen, the 46.1 km (29 miles) of black spots present a frac-

TABLE 2—*Accident comparison: black spot versus remainder of highway system in 1976.*

	Black Spots	Remainder of Highway System	Black Spots in Total Highway System, %
Kilometres	46.1	15 800	0.29
Wet pavement collisions	2 303	6 697	26.0

tion of a percentage point of the total system, but they exhibited 26 percent of the total wet pavement accidents occurring on the system.

A breakdown of black spot accidents by facility type is shown in Table 3. It is noted that wet pavement collisions on freeway black spots account for approximately 46 percent of the total accidents. The combined black spot accidents on freeways and four-lane facilities comprise over 75 percent of the total accidents. These facilities normally are characterized by heavy traffic volumes and a high number of commercial vehicles.

Review of the black spot highway locations on the Highway 401 Toronto Bypass (an Urban Freeway section) revealed that treatment of such short portions [0.10 km (0.06 mile)], individually, would result in a patchwork of high and low skid resistance sections. This was considered an undesirable quality, particularly on such a major freeway with a relatively high speed limit [100 km/h (60 mph)].

Highway 401 across Metro Toronto is 12 lanes wide, carrying in its middle section a traffic volume of approximately 210 000 vehicles per day. Analysis of this highway identified a high number of black spot areas and showed overall low friction levels on the existing old concrete pavement.

Recommendations were made to proceed with normal length resurfacing jobs (that is, rehabilitation is not limited to short sections of black spots), and that black spot accident data be used as one of the criteria for scheduling the resurfacing work. As a result, portions of the Highway 401 Toronto Bypass were placed or advanced on the MTC capital construction program for normal resurfacing.

Analysis of black spot locations on four-lane and two-lane facilities indicated that the majority of these are associated with controlled, grade intersections. In many cases, low skid resistance due to high polishing of coarse aggregate on the pavement surface (caused by vehicles braking and cornering), is not the only factor contributing to the excessive rate of wet pavement accidents at these locations. Low pavement friction, wheel track rutting, unavailability of a left turn storage lane, and difficult alignment, are all common factors at most black spot intersections. Field review staff in such cases must make a judgment call on whether or not rehabilitation to improve pavement skid resistance would benefit the reduction of wet pavement accidents at the location.

Rehabilitation to upgrade pavement friction levels and remove surface rutting can produce appreciable reductions in accidents at locations with low

TABLE 3—*Wet pavement accidents at black spot locations, 1976, by type of highway.*

	Freeways	4-Lane	2-Lane	Ramps	Total
Number of accidents	1010	733	333	227	2303
% of total	46	30	14	10	100

friction levels and consistent records of excessive rates of wet pavement accidents. The following section will present accident data before and after resurfacing at black spot highway locations.

Number of Accidents Before and After Resurfacing

Freeway Locations

Table 4 shows the number of wet and total accidents before and after resurfacing at eight freeway locations treated during the period 1976 to 1978. Six locations are on Highway 401 Toronto Bypass, one on Highway 401 near London, and one on Highway 417 (Ottawa Queensway). Two years' accidents were considered before resurfacing, and one, two, and three years' accidents were considered after treatment for 1978, 1977, and 1976 work.

Reduction in wet pavement accidents after resurfacing ranged between 17 and 73 percent. For all eight locations combined, an average of 54 percent reduction in wet pavement accidents was obtained. Overall reduction in total accidents (that is, wet, dry, snow, and ice) was 29 percent.

An OFC Mix was used on all the Highway 401 locations and a DFC Mix was used on Highway 417. In all projects, the surface coarse mix included traprock coarse aggregate and traprock screening fines, except on Highway 401 at London where steel slag coarse and fine aggregates were used. At Highway 417, rehabilitation included widening of the pavement from two to three lanes in each travel direction.

In all cases the old pavement was concrete. Resurfacing with the OFC and DFC mixes has approximately doubled the skid resistance levels.

It is interesting to compare the number of accidents on the treated portions of the Highway 401 Toronto Bypass with the accidents on the total length of the Bypass (30 km or 19 miles). During the period of observation, traffic volumes increased by approximately 25 percent, a number of geometric improvements were made, and in 1976 the speed limit was reduced from 112 km/h (70 mph) to 100 km/h (60 mph). In the five-year period between 1975 and 1979, the total number of accidents on the 30-km (19-mile)-section remained at relatively constant annual levels. The wet pavement accidents as a percentage of total accidents varied between 31 and 33 percent for the period between 1975 and 1977, and 24 to 26 percent for 1978 and 1979. It would appear that as more rehabilitation work is carried out at black spot locations on the bypass, further reductions in the number and the percentage of wet to total accidents may be achieved.

Signalized Intersections

Table 5 gives the number of wet and total accidents before and after rehabilitation at five black spot signalized intersections treated in 1977 and

TABLE 4—*Change in accidents before and after resurfacing at black spot freeway locations.*

Location	Year of Treatment	Wet Accidents Number	Wet Accidents Change, %	Total Accidents Number	Total Accidents Change, %	Remarks
Highway 401, Toronto, Interchange 51A to 53 Express lanes	1976	B 64 A 30	−53	B 220 A 173	−21	east and westbound, 3 lanes each direction
Collector lanes	1977	B 40 A 23	−43	B 155 A 118	−24	east and westbound, 3 to 4 lanes each direction
Interchange 58 to 57 collector lanes	1978	B 7 A 2	−71	B 16 A 5	−69	westbound, 3 lanes
Interchange 56 to 55 collector lanes	1978	B 22 A 10	−55	B 46 A 35	−24	westbound, 4 lanes
Transfer lanes at Interchange 56	1978	B 15 A 6	−60	B 26 A 15	−42	westbound, 2 lanes, AADT 17 000
Interchange 51A to 50 collector lanes	1978	B 30 A 25	−17	B 98 A 92	−6	westbound, 3 to 4 lanes
Highway 401, London, Interchange 19 to 1.1 km westerly	1978	B 6 A 2	−67	B 13 A 7	−46	only 2 westbound lanes treated, AADT 9000
Highway 417, Ottawa, St. Lanrent Blvd. to Hurdmans Bridge	1977	B 73 A 20	−73	B 168 A 79	−53	east and westbound, 2 lanes before and 3 lanes after treatment, AADT 61 000 in both directions
Total		B257 A118	−54	B 742 A 524	−29	

NOTE—B and A designate number of accidents "Before" and "After" resurfacing.

TABLE 5—*Accidents before and after treatment at black spot intersections.*

Location	Year of Treatment	Wet Accidents		Total Accidents		Remarks	Rehabilitation Type
		Number	Change, %	Number	Change, %		
Highway 7 at Islington Ave., Woodbridge	1978	B 17 A 4	−76	B 35 A 11	−69	4-lane undivided 6% grade, AADT 27 000; only west-bound lanes (down-grade) treated	DFC mix with 100% traprock aggregate plus a warning traffic sign
Highway 2 at Counter Street, Kingston	1978	B 6 A 4	−33	B 13 A 13	no change	4-lane divided, 5% grade, sharp curve, AADT 17 000	DFC mix with 100% traprock aggregate
Highway 40 at Plank Rd., Sarnia	1978	B 5 A 1	−80	B 7 A 3	−57	4-lane undivided, AADT 7500, slight grade	DFC with 100% steel slag aggregate
Highways 2 and 4 at Wonderland Rd., London	1978	B 4 A 1	−75	B 12 A 9	−25	4-lane undivided, AADT 8750, slight grade	as above
Highway 3 at Ridge Rd., Fort Erie	1977	B 3 A −	−100	B 4 A 2	−50	4-lane undivided, 4% grade, AADT 6000, only 2 west-bound lanes "downgrade" treated	CMI retexturing
Total		B 35 A 10	−71	B 71 A 38	−46		

NOTE—B and A designate number of accidents "Before" and "After" treatment.

1978. An average of two years' accidents before and one year after treatment was considered.

The reductions in wet pavement collisions after treatment ranged between 33 and 100 percent. Overall, for the five intersections combined, an average of 71 percent reduction in wet pavement accidents and 46 percent in total accidents was obtained.

With one notable exception, treatment at all sites included resurfacing of existing pavement with a DFC Mix. Traprock coarse and fine aggregates were used on Highways 7 and 2, and steel slag coarse and fine aggregates were used on Highways 40, 2, and 4.

At Highway 3, treatment included retexturing of the pavement surface by cold planing using a CMI rotomill [7].

The old pavement in all cases was bituminous concrete. Major deficiencies included excessive polishing of coarse aggregates on the surface, wheel track rutting, and surface contaminations by oil deposits at the intersections.

Pavement skid resistance levels observed before and after treatment at various sites are shown in Table 6. Skid resistance levels were improved by 13 to 20 skid numbers as measured by the ASTM skid trailer.

Treatment of black spot intersections appears most effective at the intersection of Highway 7 and Islington Ave. in Woodbridge. Recommendations by the field review staff included resurfacing with a DFC Mix plus installation of an electronic, overhead, warning traffic sign with flashing lights at the top of the grade approaching the intersection. The sign operates in conjunction with the traffic lights at the intersection and reads "PREPARE TO STOP" when the amber or red lights are on. During the time prior to rehabilitation, this location had consistently shown high incidence of wet pavement accidents. During the first year after treatment, rehabilitation resulted in a 76 percent reduction in wet accidents and a 69 percent reduction in the total accidents at the intersection.

TABLE 6—*Skid resistance before and after treatment at black spot intersections.*

Location	Pavement Skid Resistance		Test Speed, km/h
	Before	After	
Highway 7, westbound lanes at Islington Ave., Woodbridge	33	50	60
Highway 2, Counter St., Kingston	34	54	60
Highway 40, Blank Rd., Sarnia	28	43	80
Highways 2 and 4, Wonderland Rd., London	23	36	80
Highway 3, Ridge Rd., Fort Erie	26	45	80

Skid Resistance Mixes

The surface properties of a pavement necessary to provide good skid resistance are well defined in available literature. A skid-resistant surface must have sufficient microtexture or harshness, and sufficient macrotexture or stone projections. The macrotexture breaks up the water film and provides drainage channels so that most of the bulk water can be drained from the contact area between the rolling tire and the pavement surface. The microtexture allows penetration of the remaining thin film of water on the roadway surface. Good friction levels can be obtained only with adequate, harsh microtexture on the pavement surface. This is required at all speeds. Adequate macrotexture will limit the drop in friction levels as vehicle speed or water thickness, or both, on the pavement surface increase.

Microtexture may be provided for by use of aggregates with high polish resistance, which show differential polishing or microtexture regeneration characteristics [*11*].

Aggregate size, shape, gradation, type, hardness, and resistance to wear all influence the attainment and maintenance of macrotexture stone projections on the pavement surface [*9-12*].

Ryell, Corkill, and Musgrove [*9*] reported results of a comprehensive testing program for monitoring skid resistance characteristics of 18 test mixes constructed on the Highway 401 Toronto Bypass in 1974. The test sections included both dense and open graded type mixes, and evaluated a variety of aggregate types including traprock, steel slag, and blast furnace slag.

To compare the skid resistance performance of various test mixes, an *SN* value of 31 at 100 km/h (60 mph) is selected. This target *SN* value is based on National Cooperative Highway Research Program (NCHRP) Report 37, "Tentative Skid-Resistance Requirements for Main Rural Highways" [*13*].

During the five years in service (1974 to 1979) under extremely heavy traffic on the Toronto Bypass, the best mixes that provided and maintained friction values close to or above target or desirable levels were: DFC mixes with both coarse and fine aggregates consisting of traprock, steel slag, or blast furnace slag; and OFC mixes using traprock coarse and fine aggregates.

As described in Ref *9*, these mixes also retain a high level of skid resistance during particularly adverse conditions such as heavy rain and light snow/slush on the surface.

Sand mixes containing traprock screenings that produce good microtexture, but little macrotexture, provide a reasonably good level of skid resistance when tested under standard conditions with the brake force trailer, but exhibit a significant decline in skid resistance under conditions of moderate or heavy rain. Such mixes would provide a satisfactory surface texture only for low-speed traffic.

All successful mixes contain traprock or slag screening fine aggregates.

The use of these harsh angular fines not only provides good harshness on the pavement surface, but also results in higher macrotexture depth attained by the protrusion of coarse aggregate particles from the matrix (see Fig. 4). Mixes containing fine aggregates consisting of a blend of natural sand and limestone screenings provide the lowest skid resistance levels. This is due mainly to loss of macrotexture depth by intrusion of coarse aggregate particles into the matrix, under heavy truck traffic at the test site.

Figure 5 shows the high speed skid resistance (SN_{100}) versus accumulated traffic volumes for some selected mixes. Two DFC mixes containing 100 percent traprock and 100 percent steel slag aggregates, one OFC mix with traprock coarse aggregate and washed traprock screening fines, and a standard HL1 (control) mix which contains traprock coarse aggregates and a blend of sand and limestone screening fines. As can be seen, the initial SN_{100} values obtained on the DFC and the OFC mixes are 9 to 15 skid numbers higher than that observed on the standard HL1 mix. Over the five years in service, the OFC and the DFC mixes maintained significantly higher friction levels in comparison with the HL1 mix. With a total of approximately 24 million vehicle passes, 7 million of which are heavy commercial trucks, the skid resistance of the OFC and the DFC are close to or above the identified desirable friction level, whereas the skid resistance of the standard mix declined sharply below target levels during the first year in service.

As a result of this work, the MTC in 1978 introduced a new policy for

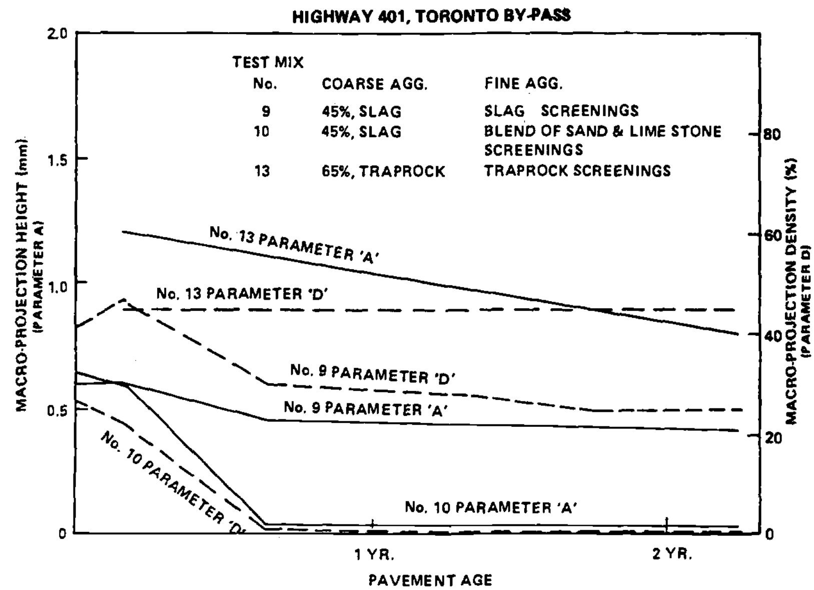

FIG. 4—*Change in height and density of macroprojections for test sections 9, 10, and 13.*

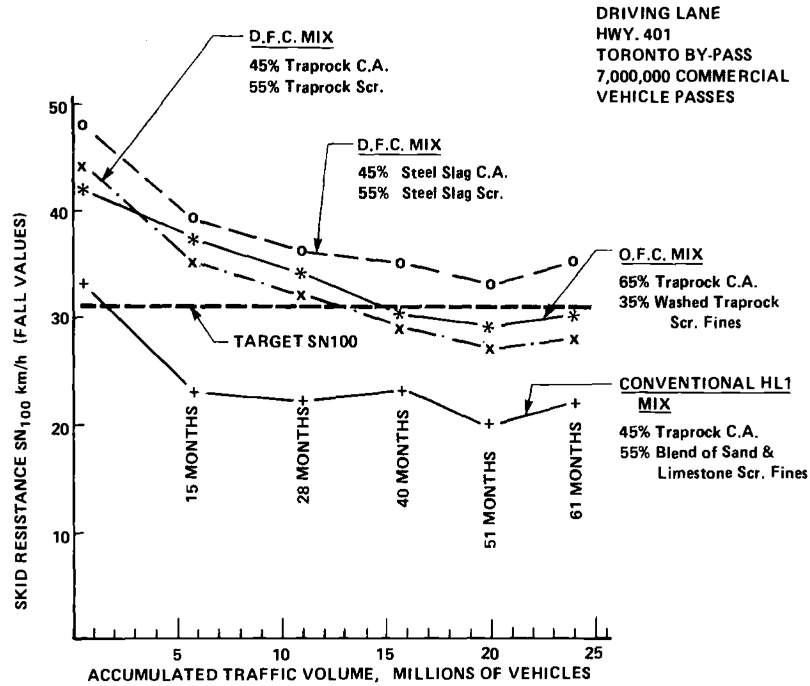

FIG. 5—*Five-years skid performance history of OFC, DFC, and HL1 mixes.*

bituminous surface course construction on freeways and other main highways in Ontario. The new policy specifies the use of OFC mixes on all urban freeway surfacing and the use of DFC mixes on other freeways and main highways carrying a traffic volume in excess of 5000 vehicles per lane per day.

MTC aggregate gradation requirements for OFC and DFC mixes are shown in Tables 7 and 8. The nominal size is 9.5 mm (3/8 in.) for OFC and 13.2 mm (1/2 in.) for DFC mixes. At present, only traprock coarse and traprock washed screening fines are accepted in the construction of OFC mixes. Both traprock and steel slag coarse and fine aggregates are accepted for DFC mixes.

A typical OFC mixture would contain 65 percent coarse aggregates (that is, +4 materials), 35 percent washed screening fines, and approximately 5.2 percent asphalt cement (AC). A typical DFC mix would contain 45 percent coarse aggregates, 55 percent fine aggregates, and approximately 4.5 percent AC. OFC mixes are placed 25 mm (1 in.) in thickness, whereas DFC mixes are placed 38 mm (1 1/2 in.) thick.

Since the introduction of the new surface policy in 1978, a total of approximately 300 lane-km (185 lane-miles) of pavements has been constructed with OFC and DFC on the MTC system.

TABLE 7—*MTC aggregate gradation requirements for OFC mixes.*

Ministry Sieve Designation	Percentage Passing by Weight	
	Coarse Aggregate	Fine Aggregate (Washed Screenings)
13.2 mm (½ in.)	to 100	...
9.5 mm (⅜ in.)	95 to 100	...
6.7 mm (No. 3)	20 to 45	to 100
4.75 mm (No. 4)	0 to 10	85 to 100
2.36 mm (No. 8)	...	50 to 70
1.18 mm (No. 16)	...	25 to 45
600 μm (No. 30)	...	10 to 30
300 μm (No. 50)	...	0 to 20
150 μm (No. 100)	...	0 to 10
75 μm (No. 200)	0 to 2	0 to 3

TABLE 8—*MTC aggregate gradation requirements for DFC mixes.*

Ministry Sieve Designation	Percentage Passing by Weight	
	Course Aggregate	Fine Aggregate
13.2 mm (½ in.)	to 100	...
9.5 mm (⅜ in.)	50 to 73	to 100
4.75 mm (No. 4)	0 to 10	85 to 100
2.36 mm (No. 8)	...	50 to 70
1.18 mm (No. 16)	...	25 to 50
600 μm (No. 30)	...	15 to 40
300 μm (No. 50)	...	10 to 30
150 μm (No. 100)	...	5 to 25
75 μm (No. 200)	0 to 2	0 to 17

Conclusions

1. The amount of available information in the skid resistance field appears sufficient for the development and implementaion of working systems aimed at reducing accidents at black spot highway locations.

2. A large portion of black spot wet pavement accidents was found to occur on freeway and four-lane highways. Such locations are generally characterized by heavy traffic volumes and a large number of commercial vehicles.

3. The majority of black spot locations on four-lane and two-lane facilities was found to be associated with controlled grade intersection locations. At such highway sites low pavement skid resistance is usually not the only factor contributing to high accident rates. Field review staff must make a judgment call on whether or not rehabilitation to improve pavement skid resistance would benefit the reduction of wet pavement accidents at the location.

4. Rehabilitation to improve friction levels at highway locations with low *SN* values and experiencing a high rate of wet pavement collisions appears to produce significant reductions in accidents. An average of 54 percent reduction in wet accidents and a 29 percent reduction in the total collisions were obtained after treatment at eight black spot freeway sections. Treatment at five black spot signalized intersections produced an average of 71 percent reduction in wet accidents and 46 percent reduction in total accidents.

5. Under extremely heavy traffic conditions on the Highway 401, Toronto Bypass, the bituminous mixes which contain harsh, angular screening fine aggregate, such as traprock and slag fines, provide and maintain significantly higher friction levels than standard mixes with sand fines.

6. In the driving lane, after approximately 24 million vehicle passes, of which seven million are heavy commercial vehicles, the bituminous mixes which provide friction levels close to or above the target values are: (*a*) high stone content, dense, and open graded mixes which contain traprock coarse and fine aggregates; and (*b*) dense graded mixes using steel slag and blast furnace slag coarse and fine aggregates.

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Summary

The papers in this Special Technical Publication fall into two general categories: measuring and predicting pavement skid resistance characteristics; and strategies for reducing wet weather accidents on highways. The two topics are related since a wet weather safety program is dependent, among other things, on a good skid resistant pavement surface, which in turn requires methods to test, describe, and predict its characteristics. The papers include a comparison of macrotexture measurements, a method to describe texture characteristics beyond a one number descriptor, methods to predict friction performance and accident risk, and methods used in implementing a safety improvement program.

The Chamberlin and Amsler paper reports the findings of a complete factorial study of the variabilities involved in measuring pavement texture by the sand-patch method. Sand-patch measures of portland cement concrete pavements showed almost 20 percent variance between paving jobs using the same texturing method and a 21.5 percent variance in texture within a job. The authors conclude that better job controls on texturing processes are needed to assure minimum texture depths and uniformity. Considering the large within-texture variability and the reported lack of precision of the sand-patch method, the authors suggest the volume of sand used in the sand-patch method should be increased from the current practice of 25 cm^3 (1.5 in^3).

Yager and Bühlmann also studied macrotexture measurements by comparing the results of three volumetric test methods (sand, putty, and grease) and two drainage test methods (outflow meters). Holt and Musgrove describe continued improvements and utilization of the Schonfeld photo interpretation (PI) method of recording and analyzing surface macro- and microtextures. Their efforts to semiautomate the PI method promise improvements in accuracy and consistency of ratings while reducing operator time. In addition to providing a three-dimensional record of the pavement texture, the PI method also is reported to estimate the skid number. This is significant in respect to the evaluation of locations that are not suitable for normal skid testing (for example, T-intersections).

It has been reported for many years that skid resistance varies in roughly an annual pattern, with high values observed in late winter and low values observed in late summer. This pattern is complicated by rainfall, temperature and other factors. The complex variations make it difficult to predict pavement performance, establish acceptable performance criteria, or evaluate accident experience. Based on a major study of seasonal variations in skid resistance, Hill and Henry report the level of skid resistance at the beginning of spring is a function of microtexture (as measured by the British Pendulum Tester), average daily traffic volume (ADT), and mechanical effects such as studded tire wear. Long-term friction changes related to polishing can be predicted from ADT. A polish resistance test also is suggested to assist the prediction of pavement performance.

Emery, Lee, and Kamel correlated laboratory tests of asphalt pavement mixes to friction performance on a series of test pavements. The authors concluded that factors related to mix stability (air voids, and marshall stability and flow) and traffic volume (especially commercial vehicles) were most significant. The test pavements used aggregates having a narrow range of polish stone values (45 to 64) and three mix design types. Predictive equations relating skid number at either 80 or 100 km/h (50 and 66 mph) based on mix parameters and accumulated traffic are offered for the mixes studied.

Burchett and Rizenbergs' paper shows the relation between a detailed study of pavements and systematic efforts to reduce wet weather accident rates. Their study investigated the relation of skid number, precipitation, traffic and accident experience on 8000 km (5000 miles) of two-lane roads in Kentucky. Relationships are developed to predict wet pavement accidents (as a percentage of the total wet and dry pavement accidents) from skid numbers. Using this data, the benefits from resurfacing a pavement were examined. Wet pavement accident rates per kilometre were stratified by traffic volumes, and benefit/cost ratios were estimated for the service life of an improvement. Following this system, an agency could determine rational criteria for resurfacing roads as normal maintenance or as hazard elimination.

Finally, Kamel and Gartshore describe a general wet pavement accident reduction program to identify and correct high hazard locations, or "black spots." The program involves detection of black spots through computer searches of accident records, estimating benefit/cost ratios of improvements, prioritizing projects, and conducting field reviews of high priority projects. A study of hazardous locations in Ontario showed that less than 1 percent of the total road system accounted for 26 percent of the wet weather accidents. The treatment of black spots on eight freeways resulted in a 54 percent average reduction in wet pavement accidents and a 29 percent reduction in total accidents. The treatment of five signalized intersections produced even more dramatic accident reductions.

These papers demonstrate that significant progress is being made in testing and evaluating pavement friction characteristics, and in using those results to identify and correct hazardous locations and to assure good performance in new pavements. There is still much work to be done to provide universal applicability and acceptance. As the three papers concerning texture measurements demonstrate, methods to measure texture, especially microtexture, need further development and refinement. Chamberlin and Amsler's paper also shows that better construction controls may be needed to assure uniform pavement textures. Skid resistance prediction equations will require significant work to establish coefficients for local aggregates and mix designs. The last two papers (Burchett and Rizenbergs; Kamel and Gartshore) show possible techniques and program structures for implementing a wet weather safety program.

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