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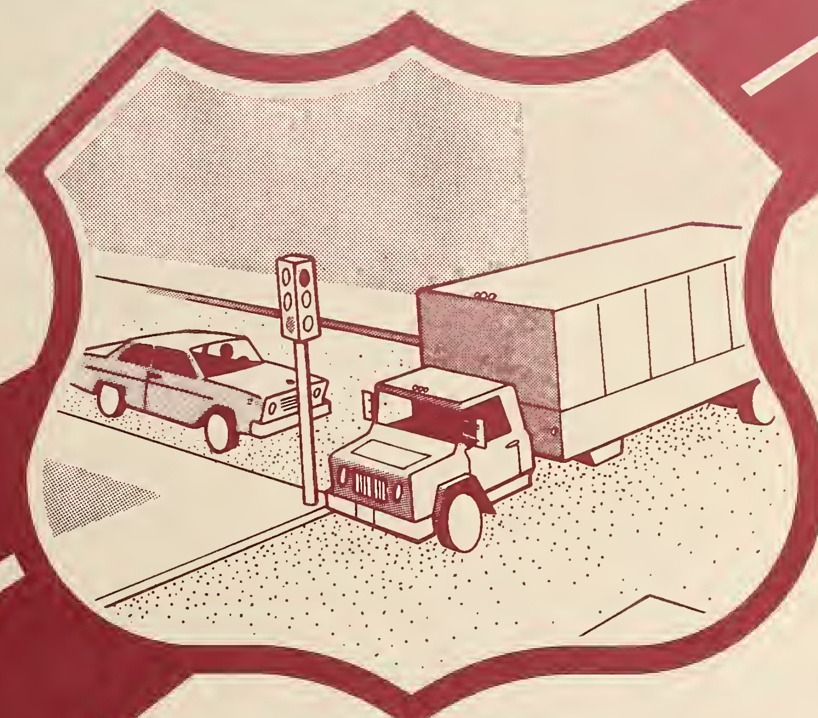
...CTIVENESS OF ALTERNATIVE SKID REDUCTION MEASURES

Vol. IV. Criteria for Improvement of Pavement Surface Macrotexture

November 1978

Final Report



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Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Environmental Division
Washington, D.C. 20590

FOREWORD

This report is part of a final report consisting of an executive summary and four volumes. The executive summary provides a synopsis of the research. Volume I describes the evaluation of accident rate-skid number relationships; Volume II describes the development of the benefit-cost model; Volume III presents the computerized benefit-cost model and instructions for its use; and Volume IV summarizes methods of measuring and achieving macrotexture. It will interest those concerned with pavement surface characteristics and the selection of accident reduction measures.

This research is included in Project 1H, "Skid Accident Reduction" of the Federally Coordinated Program of Research and Development. Mr. George B. Pilkington II is the Project Manager and Mr. Philip Brinkman is the Task Manager.

One copy of this report is being distributed to each FHWA regional office.

Charles F. Scheffey
Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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16. Abstract

This volume presents a guide to the role of pavement macrotexture in pavement skid resistance and accidents. An overview of the fundamentals of tire-pavement skid resistance is presented. The methods of measuring macrotexture in the field and in the laboratory are described based on a review of relevant literature. The measurement methods currently employed by state highway departments are identified; the sand patch method is the most widely used and accepted method in the United States for measuring pavement macrotexture.

Methods of providing macrotexture in new pavements and restoring macrotexture in existing pavements are described. The methods of providing macrotexture in new pavement include open-graded asphalt surface courses and texturing of portland cement concrete surfaces; the methods of restoring macrotexture to existing surfaces include open-graded asphalt overlays, pavement grooving, cold milling, and seal coats. A cost-effectiveness analysis procedure for alternative methods for improving pavement macrotexture is presented. Such analyses can be used as the basis for cost-effective warrants for pavement macrotexture improvements. The development of cost-effective warrants is illustrated by a numerical example.

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PREFACE

This is volume four of a four-volume set prepared by Midwest Research Institute for the Federal Highway Administration under Contract No. DOT-FH-8120. Mr. Henry C. Huckins of the Implementation Division of the FHWA Office of Development served as Contract Manager for the preparation of this volume. The project also benefited from the comments and suggestions of staff members of the FHWA Office of Research including Mr. Charles P. Brinkman, Mr. George Pilkington, and Mr. Burton Stephens.

We also wish to acknowledge the contributions of 18 state highway and transportation agencies who cooperated with the project. The cooperating agencies are the California Department of Transportation, the Connecticut Department of Transportation, the Kansas Department of Transportation, the Louisiana Department of Highways, the Maine Department of Transportation, the Maryland State Highway Administration, the Massachusetts Department of Public Works, the Michigan Department of State Highways, the Mississippi State Highway Department, the North Carolina Department of Transportation, the Ohio Department of Transportation, the Pennsylvania Department of Transportation, the Rhode Island Department of Transportation, the South Carolina State Highway Department, the Texas Department of State Highways and Public Transportation, the Washington State Highway Commission and the West Virginia Department of Highways.

The work reported in this volume was carried out in the Economics and Management Science Division under the administrative direction of Dr. William D. Glauz. Mr. Robert R. Blackburn, Manager, Driver and Environment Programs, was the Principal Investigator for this effort. Mr. Blackburn, together with Mr. Douglas W. Harwood, Associate Traffic Engineer and Mr. Patrick J. Heenan, Junior Engineer, were co-authors of this volume. Mr. Jerry L. Graham, Associate Traffic Engineer, also contributed to the project.

Approved for:

MIDWEST RESEARCH INSTITUTE

A E Vandegrift

A. E. Vandegrift, Director
Economics and Management
Science Division

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METRIC CONVERSION FACTORS

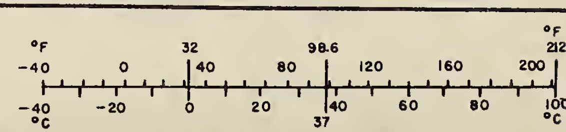
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yerds	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	squere inches	6.5	square centimeters	cm^2
ft^2	squara feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ouncas	28	grems	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	querts	0.95	liters	l
gel	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fehrenheit tempereture	5/9 (efter subtracting 32)	Celsius tempereture	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	cantimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometars	0.4	squere miles	mi^2
he	hectares (10,000 m^2)	2.5	ecres	
		MASS (weight)		
g	grems	0.035	ounces	oz
kg	kilogrems	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yerds	yd^3
		TEMPERATURE (exact)		
°C	Celsius tempereture	9/5 (then add 32)	Fehrenheit temperature	°F



I. INTRODUCTION

This volume, "Criteria for the Improvement of Pavement Macrotexture," is the fourth volume of a four-volume set prepared as the result of a multiyear study of wet-pavement accidents. The project grew out of the increasing national concern over the hazards connected with driving on wet pavements. Under such conditions, the tire-pavement friction level is reduced--often dramatically. If vehicle maneuvers demand higher friction levels, skidding will occur, potentially leading to an accident. The risk of such accidents tends to increase with higher traffic volumes and higher speeds. The traffic safety engineer must therefore develop and implement countermeasures that will either increase the tire-pavement friction level or reduce the demand for friction.

The contract dealt with two aspects of this total problem. The first was to develop the relationships between pavement skid number and wet-pavement accidents for a variety of highway and traffic conditions. This work is documented in Volume I of the set.

The second phase of the study involved the definition of a range of alternative solutions to the problems of maintaining the frictional requirements of drivers during wet-pavement conditions, and the development of a formal means to evaluate the cost-effectiveness of the solutions. This was accomplished through a comprehensive, computerized, benefit-cost model. The model incorporates, in detail, the relationships between accidents and skid number and an indirect relationship developed between accidents and pavement texture, so that the effects on accidents of changes in skid number or pavement texture can be predicted. It also treats explicitly, via self-contained tabular data or user-supplied data, the effects of geometric and traffic control countermeasures on a variety of accident types. The details of the model and its development are given in Volume II.

The third volume of the report, the User's Manual, presents implementation procedures. It shows how the user can apply the benefit-cost model, not only to wet-pavement accident reduction, but to an extremely broad class of benefit-cost evaluations of accident countermeasures. The manual is designed for two types of users: (1) highway engineers or administrators who need to understand the overall aspects of the model to use it effectively in decisionmaking; and (2) engineers and other persons who need to prepare data and actually use the model.

Volume IV is a guide for highway engineers and administrators concerned with the improvement of pavement surface texture as a countermeasure for skidding accidents. It discusses the importance of pavement macrotexture in reducing skidding accidents. The volume then describes the methods of measuring pavement macrotexture and techniques for providing macrotexture in new pavements and restoring macrotexture in existing surfaces. A simplified version of the benefit-cost model described in Volumes II and III is used to perform a cost-effectiveness analysis of alternative techniques for improving pavement macrotexture. The criteria presented for the improvement of pavement surface macrotexture can be directly implemented by state highway departments. Additional criteria, applicable to the local climate and paving materials in specific states or regions of the country, can be developed using the general analysis approach presented in this volume.

II. FUNDAMENTALS OF TIRE-PAVEMENT SKID RESISTANCE

This section provides a summary of the fundamentals of tire-pavement skid resistance. The discussion focuses on the contribution of pavement surface properties to the friction available at the tire-pavement interface. The information presented here is essential background for a complete understanding of the remainder of this report.

A. Overview

The potential for a skidding accident depends mainly on the speed of the vehicle, the cornering path, the magnitude of acceleration or braking, the condition of the vehicle tires, and the characteristics of the pavement surface. On wet pavements, speed is the most significant parameter, not only because the frictional demand increases with the square of the speed, but also because the skid resistance at the tire-pavement interface decreases with increasing speed. Figure 1 is a generalization of these relationships showing, for a given degree of cornering or magnitude of acceleration or braking, how the margin of safety (difference between available skid resistance and friction demand) decreases rapidly with increasing speed until a skid is imminent.

Skid resistance is a general term used to describe the level of friction between a roadway surface and a vehicle tire. The tire-pavement skid resistance can be measured in several testing modes--locked-wheel braking, brake slip, drive slip, and cornering slip. A few states, primarily in the Western United States, operate cornering slip testers, also known as mu-meters. However, the locked-wheel braking mode has gained by far the widest acceptance for skid resistance testing throughout the United States.

The most common measure of skid resistance in the locked-wheel braking mode is the skid number (SN), which is defined as 100 times the coefficient of friction determined with a locked-wheel skid test. The standard procedure for skid number measurements in the United States is established by ASTM Standard E-274-77. Testing is accomplished by a two-wheel trailer towed by a truck at a standard speed of 64.4 km/hr (40 mph). A layer of water with a standard thickness of 0.51 mm (0.02 in.) is placed on the pavement by nozzles located just in front of the trailer wheels. This requires a flow rate of about 0.60 liters/min/mm (4.0 gal./min/in.) of wetted width at 64.4 km/hr (40 mph). The trailer brakes are activated to lock one or both of the trailer wheels on the wetted surface. A standard test tire, described on ASTM Specification E-501 and mounted on a suitable 15 by 6-in. rim, is used. A trace of the wheel torque during braking is made and interpreted either manually or electronically to obtain the frictional force and the resultant skid number.

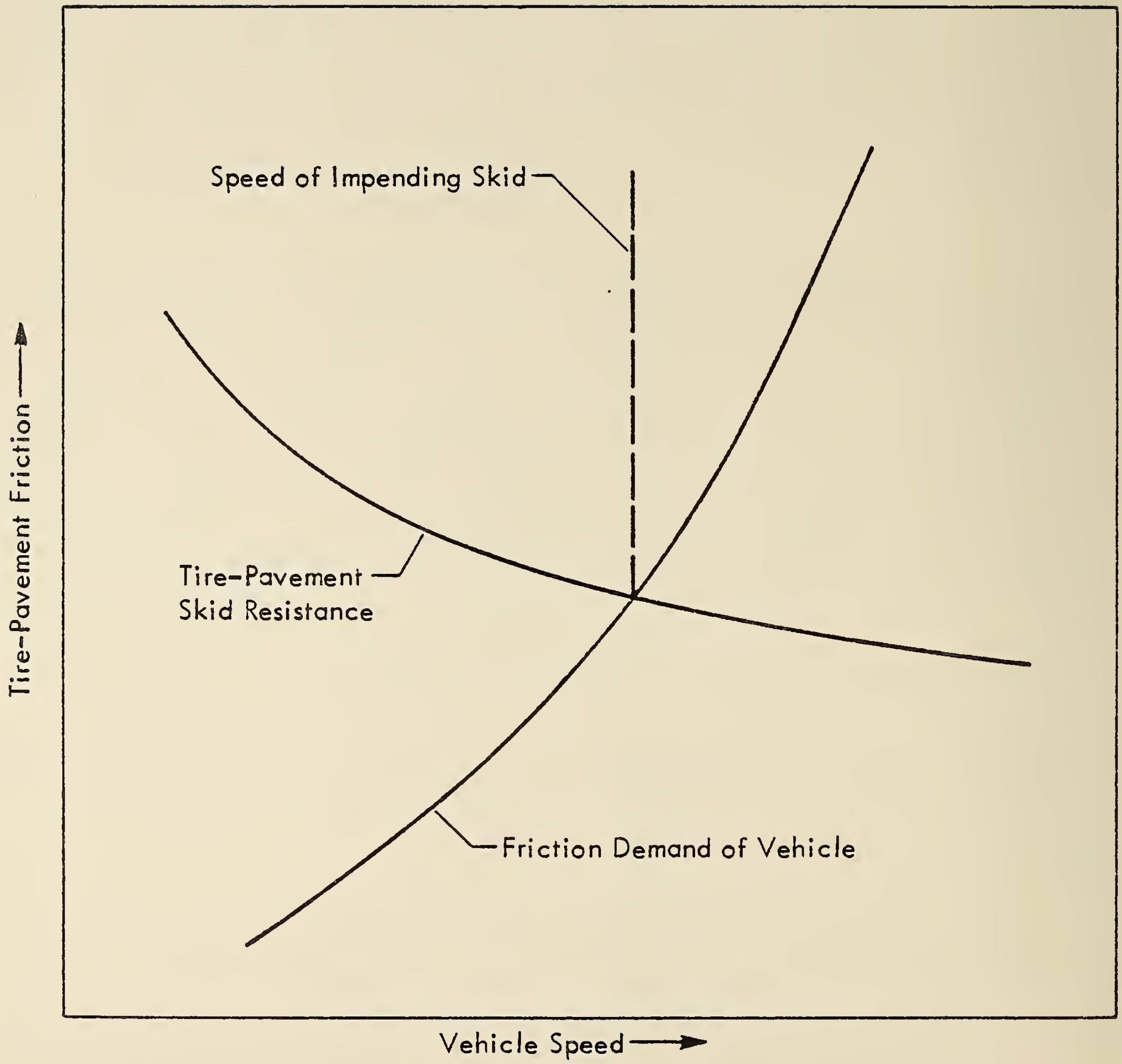


Figure 1 - Relation Between Friction Demand
and Skid Resistance

Skid numbers have been measured for many pavements, and many states conduct a periodic skid number inventory of all state highways. Despite its ready availability, the skid number should not be the only factor on which implementation of skidding accident countermeasures is based. The skid number is measured for one particular tire design and one particular water depth, whereas a variety of tire designs and water depths are encountered in the real highway environment. Also, the skid number by itself is a measure of available friction only and does not reflect the level of friction demand for the maneuvers that are required of drivers at any particular site. Finally the skid number measured at one particular speed, 64.6 km/hr (40 mph), is not adequate to completely define the frictional properties of a pavement surface.

The skid resistance curve in Figure 1 illustrates that the skid number of a pavement depends on the vehicle speed. Two pavements can have similar skid numbers at 64.4 km/hr (40 mph), but very different skid numbers at a higher speed that is more typical of operating conditions on rural highways. Figure 2 is an example of the skid number-speed relationships for a variety of pavement surfaces. Each of the pavements in Figure 2 has a nonlinear skid number-speed relationship, but these relationships are frequently approximated by a straight line. The slope of this line is represented by the skid number-speed gradient. It is conventional to define the skid number-speed gradient as a positive number. This quantity can be determined from the results of skid tests at two speeds as:

$$SNG = - \frac{SN_A - SN_B}{V_A - V_B}$$

where SNG = Skid number-speed gradient between speeds A and B (SN/km/hr or SN/mph)

SN_A = Skid number at speed A,

SN_B = Skid number at speed B,

V_A = Lower testing speed (mph or km/hr), and

V_B = Higher testing speed (mph or km/hr).

In the recommended ASTM test method, speed A is 48.3 km/hr (30 mph) and speed B is 80.5 km/hr (50 mph). These speeds define a 32.2 km/hr (20 mph) interval centered on the standard test speed of 64.4 km/hr (40 mph). However, other combinations of speeds have also been used by the states to determine the skid number-speed gradient.

The magnitude of the skid number-speed gradient indicates the rate at which skid number decreases with increasing speed. A pavement with a relatively flat skid number-speed relationship has a small skid number-speed gradient and will retain a higher skid resistance at increased speeds than a pavement with a relatively steep skid number-speed gradient.

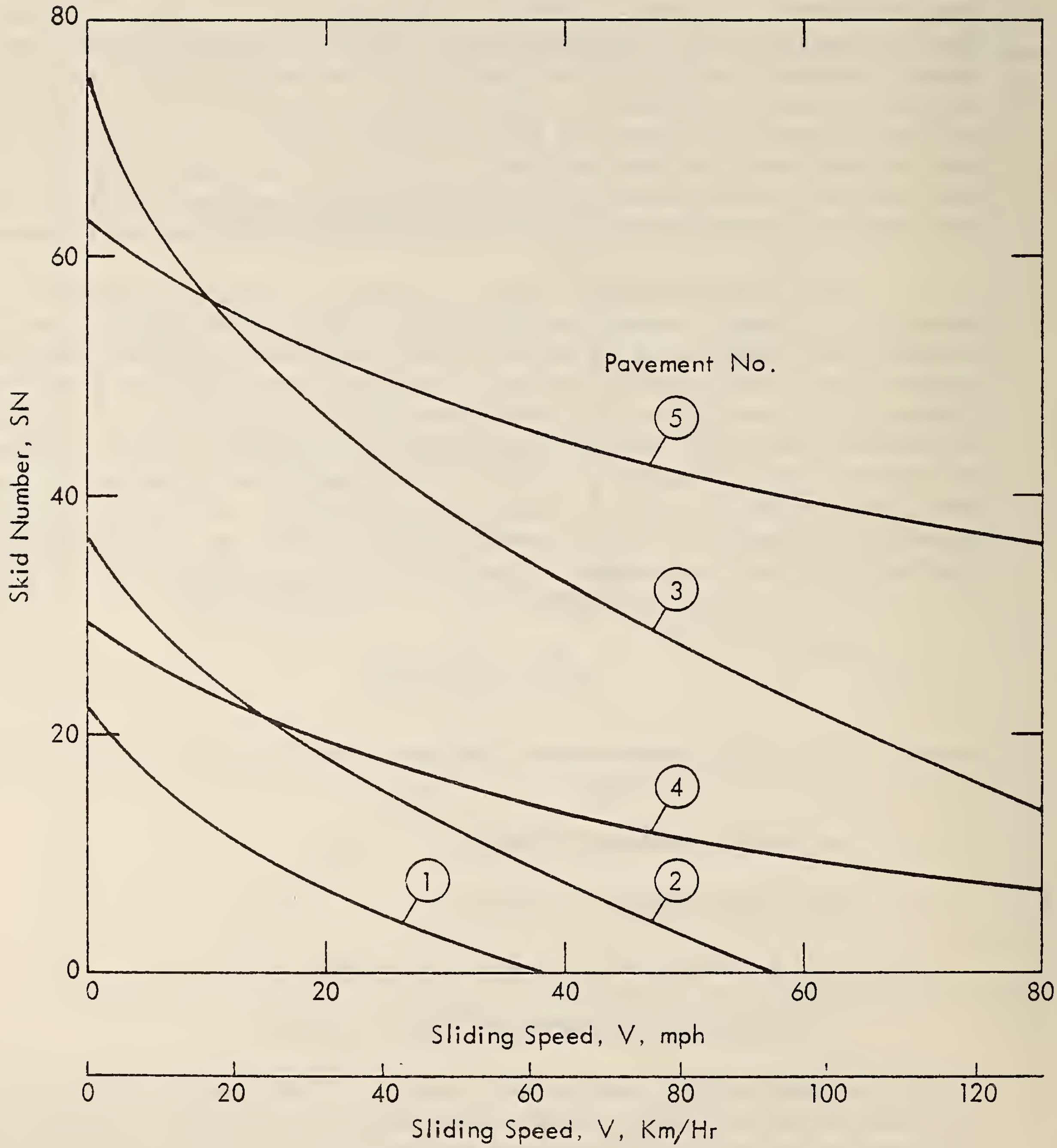


Figure 2 - Skid Number-Speed Relationships for a Variety of Pavement Surfaces[42]

It should be kept in mind that both the skid number and the skid number-speed gradient must be considered in evaluating a pavement. For example, Pavement 3 in Figure 2 has skid resistance qualities preferable to Pavement 4, despite Pavement 3's higher skid number-speed gradient, because it retains a higher skid number at speeds above 64.4 km/hr (40 mph).

The skid number of a pavement is known to change with time due to the action of traffic passages. Different aggregates polish at different rates. Several states have developed wear and polish machines for testing the polish-susceptibility of aggregates in the laboratory. A discussion of wear and polish mechanisms that influence skid resistance can be found in Appendix B of Volume II.[86]

Although the skid number is the most commonly used measure of skid resistance, it has one important limitation that must be recognized by anyone who manages a wet-pavement accident prevention program based on skid number improvements. Pavement skid numbers are known to vary with the season of the year. The highest skid number values are generally observed in the spring months and the lowest values in the fall. Extreme seasonal variations as high as 30 have been observed, with more typical amplitudes in the range of 5 to 15.[64] This phenomenon is presumed to result from variations in ADT aggregate composition, temperature, precipitation, and other factors. The pattern of seasonal skid number variations is inconsistent. Gramling and Hopkins[29] have assumed an "ideal" form of the seasonal variation curve that is roughly sinusoidal, but currently there is no reliable model for predicting these variations. The Federal Highway Administration plans a major research effort, in cooperation with several state highway departments, to model and predict seasonal variations in skid number. Until such information is available, engineers should exercise caution so that whenever possible skid number data used to justify improvement projects not overly biased by seasonal effects.

Finally, it should be recognized that when the depth of water on a pavement is excessive, skidding accidents can occur even when the friction required by a particular maneuver does not exceed the pavement skid number, measured with the ASTM Standard skid test. In this phenomenon, known as hydroplaning, the tire loses contact with the pavement and rides on a thin film of water. Very small accelerating, braking or cornering forces can initiate a skid in this condition. Therefore, at sites where climatic or geometric conditions are likely to produce excessive water depths on the pavement, consideration of factors other than skid number is required.

An understanding of methods to increase high-speed skid-resistance and reduce the potential for hydroplaning requires consideration of the role of pavement surface properties in skid resistance, which is discussed in the next section.

B. Role of Pavement Surface Properties in Skid Resistance

When skid testing procedures and tire design are controlled, pavement surface properties are the only factors that (theoretically) influence skid resistance. An understanding of the role of pavement surface properties in skid resistance is particularly important to the highway engineer because he exercises some direct, continuing control over the condition of the highway pavement, while tire design, vehicle characteristics, and driver behavior are at least one step removed from his control.

The pavement surface properties that influence skid resistance can be divided into two categories: microtexture and macrotexture. Pavement microtexture consists of the microscopic asperities on the surface of individual pieces of aggregate, and to a lesser extent on the surface of the pavement binder (asphalt or portland cement). Microtexture is what makes a piece of aggregate feel smooth or rough to the touch. By contrast, macrotexture consists of the large scale asperities associated with voids in the pavement surface between pieces of aggregate. Thus, in the simplest terms, microtexture is determined by the aggregate surface and macrotexture is determined by the distribution of aggregate sizes and the manner in which the individual pieces of aggregate are assembled to form a pavement surface. Good skid resistance requires both good macrotexture and good microtexture. Separate discussions of macrotexture and microtexture are found below.

1. Microtexture: The low-speed skid number of a pavement surface is a function of the sliding resistance between the tire and the wetted asperities of a pavement surface, where the tire contacts the pavement. This sliding resistance is increased if the pavement surface is rough or has distinct asperities on a microscopic scale. Thus, low-speed skid number is closely related to microtexture.

Several methods have been used to measure microtexture. A study[27] performed in Texas in 1970 reviewed several methods of microtexture measurement including subjective estimation and stylus tracing. The so-called Surfindicator, a stylus tracing method, was subjected to limited field trials, but did not prove completely satisfactory. Dahir and Henry[16] used profile tracing to determine the microtexture of pavement surface directly. Specific measures of microtexture considered by Dahir and Henry were root mean square texture height, arithmetic mean texture height, and root mean square texture slope. However, profile tracing to determine microtexture parameters is time consuming and not well suited to field measurements.

The most frequently used method for measuring microtexture is the British Portable Tester. The British Portable Tester is a dynamic impact device used to measure the energy loss when a pendulum with a rubber slider contacts a test surface. A standard procedure for use of the British Portable Tester is given by ASTM Standard E-303-74. The tester provides a reading called the British Portable Number (BPN). The BPN of typical pavements ranges from 55 to 90, with the larger values representing pavements with higher microtexture. Leu and Henry[47] have established a relationship between microtexture (represented by BPN) and low-speed skid resistance (represented by the zero-speed intercept of skid number):

$$SN_o = -31.0 + 1.38\ BPN$$

where SN_o = Zero-intercept skid number, and

BPN = British Portable Number.

This relationship is based on data for 20 pavements and was established with a correlation coefficient of 0.75. The relationship indicates that pavement surfaces with good microtexture have good low-speed skid resistance. However, the skid resistance at higher speeds is dependent on macrotexture as well as microtexture.

2. Macrotexture: Pavement surface macrotexture has an extremely important role in the prevention of wet-pavement accidents. Good macrotexture provides a channel for water to escape from the tire-pavement interface. The ability of water to escape from the tire-pavement interface has a significant effect on the skid resistance of pavements at speeds above 48.3 km/hr (30 mph). Two pavements with similar microtexture will differ in skid number at 64.4 km/hr (40 mph), if they differ in macrotexture. Thus, the theory of skid resistance suggests that macrotexture should be closely related to the skid number-speed gradient. Such a relationship has been demonstrated empirically in several studies, including the work of Schulze and Beckmann.[76] More recently, several researchers including Henry and Hegmon,[34] Veres, et al.,[83] and Gallaway and Rose[25] have found that the percent skid number gradient (PSNG), also known as the normalized skid number gradient, and defined as:

$$PSNG = -\left(\frac{dSN}{dV}\right)\left(\frac{100}{SN_{40}}\right) = \left(\frac{SNG}{SN_{40}}\right) \quad (100)$$

is more highly correlated with macrotexture than SNG.

A number of methods have been used to measure pavement macrotexture including the sand patch, sand track, grease patch, outflow meter, profile tracing, stereophotographic, light stylus, laser and light depolarization methods. These and other methods are described in detail in section III of this report. The measures of macrotexture that have been used in studies of pavement surface texture include mean void width, average texture depth and root mean square texture height.

Schulze and Beckmann[76] developed the following relationship between skid number-speed gradient and mean void width:

$$X = 0.876 - \sqrt{0.2376T - 0.04725}$$

where X = Skid number speed gradient between 20 km/hr (12.5 mph) and 60 km/hr (37.5 mph), and

T = Mean void width (mm) (1 mm = 0.04 in.).

This relationship is based on data from 48 pavements and was established with a correlation coefficient of 0.87.

Leu and Henry[47] have developed the following relationship between percent skid number speed gradient and macrotexture:

$$PSNG = 23.04\ (MD)^{-0.47}$$

where MD = Average texture depth (mm) as determined by the sand patch method (1 mm = 0.04 in.).

This relationship is based on data from 20 pavements and was established with a correlation coefficient of 0.96.

3. <u>Prediction of skid number from texture parameters</u>: The prediction of skid number from texture parameters is both useful as a tool in the management of pavement surface improvement programs and provides insight into the roles of microtexture and macrotexture. The model developed recently by Dahir and Henry[16] is presented here for illustrative purposes and is used in the cost-effectiveness analysis in Section VI of this report. This model is known as the Penn State model and it has the general form:

$$SN_V = C_o e^{C_1 V}$$

where SN_V = Skid number at any speed V,

V = Speed (km/hr),

C_o = Zero speed intercept (correlated with microtexture), and

C_1 = Function of macrotexture parameters only.

Expressions for C_1 and C_o have been developed empirically by Leu and Henry from data for 20 pavement surfaces.

The term C_o represents the skid resistance at low speeds and is, therefore, a function of microtexture alone. C_o is the zero-speed intercept of skid number, given above by Leu and Henry as:

$$C_o = -31.0 + 1.38 \text{ BPN}$$

The macrotexture influence is expressed in terms of the percent skid number gradient. By differentiating the general prediction model, it can be shown that:

$$\text{PSNG} = -100 \; C_1$$

Since, Leu and Henry have demonstrated that:

$$\text{PSNG} = 23.0 \; (\text{MD})^{-0.47}$$

Then,

$$C_1 = -0.230(\text{MD})^{-0.47}$$

The relationships for C_o and C_1, when substituted in the general model yield:

$$SN_V = (-31.0 + 1.38\text{BPN})e^{-0.230V(\text{MD})^{-0.47}}$$

The coefficients of this model are based on an extremely small data set and are probably dependent on type of pavement surface. However, the model does illustrate separate influences of microtexture and macrotexture and demonstrates the sensitivity of skid number to each.

III. PAVEMENT MACROTEXTURE MEASUREMENT

The qualitative influence of pavement surface properties, namely microtexture and macrotexture, upon the tire-pavement skid resistance has been known for a number of years. In recent years, there has been a need to quantify this influence. This need has spurred the interest of various agencies in the United States and abroad to engage in the development of methods for measuring the pavement surface texture, and in particular, pavement macrotexture. This section of the report discusses the general subject area of pavement macrotexture measurement. Section A presents an overview of the most commonly or recently used macrotexture measurement methods. Where possible, it also presents a discussion of the precision and accuracy of the measurement techniques together with data on their known intercorrelations. The current practice of a selected sample of state highway departments in making macrotexture measurements is described in Section B.

A. Overview of Measurement Methods

A review of the literature revealed numerous methods for measuring pavement surface texture. Some of the techniques are in very limited use and their descriptions are found only in obscure literature. Descriptions of many of these methods have been reported by researchers including Rose, et al.,[68/] Hegmon and Mizoguchi,[33/] Dahir and Lentz,[17/] Henry and Hegmon,[34/] Lees and Katekhda,[45/] Rose and Gallaway,[67/] and Apostolos, et al.[5/] Succinct descriptions of 28 of the most commonly or recently used methods of surface texture measurement are presented in this section. Descriptions of two additional methods that have not been used to measure pavement surface texture are also included because of their potential use as pavement texture measurement techniques. The descriptions are divided into two main categories: contact methods and noncontact methods. The techniques described below produce measures of a wide variety of pavement texture characteristics including: average texture depth, mean void width, root mean square of the texture depth, the root mean square of the texture slope, and outflow time. The results of attempts to evaluate and validate methods of measuring pavement macrotexture are reported below. Many investigators have attempted to validate macrotexture measurement methods by comparison with pavement friction coefficients. Most such attempts have had very low correlation coefficients. These disappointing results are not surprising, because pavement friction is known to be a function of both microtexture and macrotexture. Therefore, a high correlation cannot be expected in an analysis of pavement friction and macrotexture data unless a measure of microtexture is also utilized. The contact methods of measuring pavement texture are described first followed by the noncontact methods.

1. Contact methods: The contact methods discussed in this section include: sand patch, sand track, grease patch, putty impression, outflow meter, profile tracing, and tire noise. Altogether, 19 different techniques are presented under this general heading.

a. Sand patch methods: The sand patch method was originally developed at the British Road Research Laboratory[66] and is one of the first methods used to measure pavement surface texture. Since its development, several versions of the method have been used by various researchers and state highway departments in the United States. Three versions of the method are discussed below. These include: (1) the method most commonly used by the states and referred to simply as the "sand patch;" (2) the modified sand patch; and (3) the vibrating sand patch.

(1) Sand patch: The procedure for this method is described in an American Concrete Paving Association technical publication[3] and elsewhere.[4,17,33,68] The method involves spreading a known volume of fine, dry ASTM C-109 Ottawa sand (passing No. 50 and retained on No. 100 sieve) over a small area of the pavement surface. The sand is spread with a rubber disc into a circular patch until the pavement surface depressions are filled to the level of the aggregate tips. The area of this patch is determined from an average of diameter measurements taken at four equally spaced locations. The average texture depth is calculated as the ratio of the volume of sand spread to the area of the patch. The average texture depth is the only texture parameter evaluated by this method.

Mixed results have been obtained from use of the sand patch method because of its strong dependence upon possible operator error. Hegmon and Mizoguchi[33] reported poor repeatability with sand patch tests and noted that good repeatability could be obtained only with extreme care by the same operator.

Chamberlin and Amsler[4] analyzed sand patch test data collected from concrete pavement test surfaces during an HPR study. The measurements were made in connection with a field investigation of four pavement surface texturing methods applied to two sections each of five different paving jobs. Sand patch measurements were taken at three different sites within each of the 40 test subsections by three different operators performing two tests each. Thus, 720 individual sand patch measurements were taken during the investigation.

The analysis of the data produced estimates of the repeatability (method precision) and reproducibility (applied precision) of the sand patch test, as well as sampling errors that can be expected in measuring the mean texture depths of a pavement section by the method. All three measures of the sand patch method were described in terms of the standard deviation, σ, of the mean texture depth. Linear regression equations of the

$$\sigma = aX + b \ ,$$

form were derived for each of these quantities for a texture depth, X, range of applicability of 0.25 to 2.03 mm (0.01 to 0.08 in.). The coefficients determined for each of the three quantities along with the associated correlation coefficient, r, are given below:

	a	b	r
Repeatability in mm (in.)	0.0229	0.0076 (0.0003)	0.30
Reproducibility in mm (in.)	0.0725	0.0483 (0.0019)	0.42
Sampling error in mm (in.)	0.1414	0.051 (0.002)	0.57

Using these results, Chamberlin and Amsler[4/] also developed nomographs for estimating the number of tests an operator would need to perform for a desired precision and sampling error.

Some overall findings from the study are worthy of noting. Variations in the texture depth within any particular job were found to be roughly equivalent to hose between jobs. This suggests a need for positive control on surface texturing during construction to assure that desired texture depths and greater uniformity in texture are attained. Differences in repeat tests by the same operator at the same site, account for only 0.4% of the total variance. This is substantially less than the 3.3% found for the operator-to-operator component of the total variance.

In spite of conflicting reports about the sand patch method, most users of the method consider it reasonably precise, rapid to use, low cost and simple. However, the method is not suitable for use on grooved and deeply, interconnected textured surfaces, because the sand tends to run out along the grooves and channels which makes the true area covered extremely difficult to evaluate.

In July of 1974, a task group in ASTM Subcommittee E17.23 was created to develop a simplified method for measuring pavement recommended to the subcommittee. The sand patch procedure is currently being written up for submission to ASTM as a recommended standard method of pavement surface macrotexture determination.

(2) Modified sand patch: Several modified sand patch methods have been used in the past to obtain field estimates of the average macrotexture depth of pavement surfaces.[5,18,33,67,68] These methods differ from the sand patch method just described in that a volume of sand required to cover a specified area is determined, rather than the area that will be covered by a predetermined volume of sand.

In practice, a metal or rubber plate with a cutout of known volume is placed on the pavement surface. Fine, dry sand is used to fill the cavity. This amount of sand, less the amount required to fill the cavity when the plate is on a perfectly flat surface, determines the volume of texture below the cavity. The average macrotexture depth is then computed as the ratio of the volume of texture to the area covered.

Dahir and Lentz[17] report that the FHWA has developed a modified sand patch method that is suitable for use with pavement core samples. This procedure involves installing 6.4 mm (1/4 in.) width rubber band around the periphery of the core sample and level with the surface peaks. The sample is first weighed and then dry ASTM C-109 Ottawa sand is spread on the core surface in a circular motion using a rubber disc. The surface depressions are filled to the level of their peaks and the sample is reweighed. The difference in the sample weights represents the amount of sand required to fill the depressions in the surface of the core sample. The average macrotexture depth is computed from the difference in the sample weights, the specific gravity of the sand, and the circular area of the core sample.

(3) Vibrating sand patch: A vibrating sand patch method was developed by researchers at Pennsylvania State University.[33,68] The process was developed to eliminate the operator effect which is one of the sources of error in the use of the sand patch technique. In practice, a 152 mm (6 in.) diameter pavement core was placed on a pneumatic shaker, and sand was added to fill up the space formed by the interior of a ring sealed to the exposed texture surface of the core.* The weight of sand used was then determined. The process was then repeated using a smooth reference surface instead of the core sample. The weight of this reference volume of sand was subtracted from the first sand weight to determine the weight of sand filling the voids in the surface of the core sample. The average macrotexture depth is computed from the difference in the sand weights, the specific gravity of the sand, and the interior area of the ring sealed to the core surface. Repeatability in the texture measurements was greatly improved over other sand patch methods and was independent of the operator. However, the procedure is suitable only for laboratory use.

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* Hegmon and Mizoguchi[33] report the amount of sand perculating into the voids of the core surface was found to be dependent upon frequency, amplitude and duration of shaking. The reasons for these dependencies were not explained. However, these three vibrational parameters were controlled during core to core testing.

b. Sand track: The sand track device represents another refinement of the sand patch method.[50,68] The unit was also developed at the Pennsylvania State University and is believed to involve less operator error than the sand patch method by controlling the placement of the sand on the pavement surface. A constant volume of either ASTM C-190 or C-109 Ottawa sand is placed in a trapezoidal hopper and then the hopper is slid along a metal frame at a constant rate. Sand flows freely from the bottom of the hopper and the unit is calibrated so that it will deposit a uniform thickness of 3.2 mm (1/8 in.) of sand on a smooth surface. On a textured surface, the unit will deposit sand to fill the surface voids and will place a uniform depth of sand 3.2 mm (1/8 in.) above the peaks of the asperties. The resultant test data are reported as sand track values which are lengths of travel of the hopper required to deposit the known volume of sand on the textured surface. The average macrotexture depth is determined by subtracting the depth of sand deposited on a smooth surface (3.2 mm (1/8 in.)) from the depth of sand deposited on the pavement surface (calculated from the known volume of sand distributed over an area of recorded length and known width).

The device is adapted to routine field tests and is reported to be capable of measuring varying degrees of texture.[50] However, the precision and accuracy of the sand track device cannot be determined from the meager test data reported in the literature.

c. Grease patch: The grease patch or grease smear method was developed by NASA.[25,68] It is very similar to the sand patch method. A known volume of grease contained in a tube with a plunger (for ease in applying the grease) is placed on the pavement between two parallel strips of masking tape. The grease is then worked into the voids using a squeegee with a rubber face similar in hardness to an automobile tire. The average texture depth is determined by dividing the volume of the grease by the area covered.[46] The method is quick and easy to apply. Relatively weak correlations have been found between average texture depths determined by the grease patch method and pavement friction coefficients. No data were found in the literature relating grease patch texture measurements and other texture measurement methods.

d. Putty impression: Basically, three putty impression methods have been used to make macrotexture measurements. Two of the three methods, and the ones most commonly used, resemble the sand patch method in application and yield an estimate of the average macrotexture depth.[68] The third method involves making a solid casting of the surface for detailed laboratory measurements of various texture parameters.[25]

The first putty impression method uses a 25 mm (1 in.) thick, 152 mm (6 in.) diameter metal plate and a fixed weight of putty to measure pavement macrotexture. One side of the metal plate contains a centered recess which is 101.6 mm (4 in.) in diameter and 1.6 mm (1/16 in.) deep. The other side of the plate is flat and without a recess. To determine the texture depth with this method, 15.90 grams of putty (in the shape of a ball) are placed on the pavement surface to be measured. The metal plate with the recessed side facing the pavement is then centered over the putty and pressure is applied to flatten the putty between the plate and the surface. On a perfectly smooth surface, the recess in the plate will be completely filled with the 15.90 grams of putty. As the pavement texture increases, the diameter of the distributed putty will decrease. The average texture depth is determined by dividing the volume of the putty used by the area of the flattened putty. In practice, the area of the flattened putty is calculated from an average of four diameter measurements. The diameter measurements are a source of error which directly affect the accuracy of the average macrotexture depth calculation.

A second putty impression method was developed that attempts to eliminate this error.[82/] In this second method, which is a modified version of the first method, a frame, a top plate assembly, a roller, a 0.03-mm (0.001 in.) to 0.05 mm (0.002 in.) thick plastic film, a thinner household plastic wrap (such as Handiwrap or equivalent) 16 to 20 grams of silicon putty, and a cookie cutter about 67-mm (2-5/8 in.) in diameter are used to determine the average macrotexture depth of the pavement. The frame is a 152-mm (6 in.) by 203-mm (8 in.) by 6.4 mm (1/4 in.) thick plate with a 102-mm (4 in.) by 152-mm (6 in.) center cut of it. The top plate assembly consists of two plates joined together. One plate is 152-mm (6 in.) by 203-mm (8 in.) by 4.8 mm (3/16 in.) thick (the same platform size as the frame) and the other is 102-mm (4 in.) by 152-mm (6 in.) by 4.8-mm (3/16 in.) thick and is fastened to the top plate so that it fits into the cut out in the frame when the top plate completely covers the frame. When the top plate assembly and frame are together a space 1.6 mm (1/16 in.) deep exists between a smooth surface and the top plate assembly. The roller, which is used to check this 1.6 mm (1/16 in.) dimension, rides on the 6.4 mm (1/4 in.) thick frame and also fits into the frame cut out and penetrates the opening by 4.8 mm (3/16 in.) leaving the 1.6 mm (1/16 in.) space.

In practice, the average texture depth is determined by the second putty impression method in the following manner. First, the frame is placed over a smooth piece of shim stock. Next, putty with a known specific gravity is placed on the smooth surface and inside the metal frame. The putty is then flattened by covering it with the plastic film and pressing it with the top plate assembly until contact is made between the plate, frame, and the shim stock (which should sit on top of the surface asperities and conform to large surface variations). The roller is then passed over the flattened putty to check if the putty was pressed down properly. The plastic film is then removed and the 67 mm (2-5/8 in.) diameter cookie cutter is then used to obtain a sample of the flattened putty for weighing to the nearest 0.01 grams. The thickness of the putty is then determined using the specific gravity, the area and weight of the cut out sample of the putty. The same test is repeated again but, instead of using the smooth surfaced shim, the putty is placed on a piece of household plastic wrap which covers the pavement surface to be tested. The difference in indicated thicknesses obtained from using the shim stock and household plastic wrap is the average macrotexture depth.

Measurement errors are reduced using this second method because large surface variations are cancelled when the results of the first test, which include the variations, are subtracted from the results of the second test. Inaccurate estimates of the area of approximate circles of putty are also eliminated by using the cookie cutter, with a fixed area, to obtain the samples.[82]

The third putty impression method involves making a casting of the pavement surface using an RTV silicone which is an elastic solid when dry. A mold of the pavement can be cut into thin sections on a meat slicer. The slices can then be analyzed in a laboratory to determine such quantities as surface void sizes, void distributions, asperity sizes and distributions and to obtain typical surface profiles.[25]

Each of the putty impression methods has some disadvantages. The first putty impression method is reasonably fast but it is not very accurate. The second method effectively eliminates errors due to large scale pavement variations and inaccurate diameter measurements; however, it does take longer in the field to make the necessary measurements and is much more complicated. The third method also takes longer in the field than the first method because the RTV silicone has to cure before it can be removed. However, the third method has an additional advantage over the first two methods in that a more detailed analysis of the impression specimen can be made under laboratory conditions.

Although only limited information is available on the correlation of putty impression results with skid resistance or various texture parameters, Ledbetter and Meyer[44] found that the putty impression texture depth is related to the sand patch texture depth by the following equation:

$$TXD_{\text{sand patch}} = 0.8185\ TXD_{\text{putty impression}},$$

where TXD = texture depth. This regression equation has a correlation coefficient of 0.98 and was determined using 400 observations from two PCC tests sections and 44 observations on laboatory blocks with various finishes. The test section had texture depths ranging from 0.371 mm (0.0146 in.) to 1.892 mm (0.0745 in.).

e. Outflow meter: Two basic types of outflow meters are in general use today. Both types employ the use of a hollow cylinder with a rubber ring affixed to the bottom face. In application, the cylinder is placed vertically on the pavement surface and loaded so the rubber ring contacts the surface asperities in a manner similar to that of an automobile tire. The cylinder is then filled with water and the time for the water to drain between two reference marks on the cylinder is meausred. The two types of outflow meters differ basically only in the manner in which the water is allowed to drain from the cylinder. The first uses what is called the static drainage method wherein water drains from the cylinder solely under the influence of gravitational forces. Water in the cylinder is under atmospheric pressure. The second type uses a pressurized drainage method. In this method, the air above the water in the cylinder is pressurized to a constant amount above atmospheric and is used to force the water out between the rubber ring and the test surface.

The static drainage outflow meter uses a transparent cylinder which is typically 127 mm (5 in.) in diameter and about 305 mm (12 in.) tall.[17,55,68] The bottom of the cylinder has a rubber ring affixed to it. Originally this ring was only 6.35 mm (0.25 in.) wide but was made wider when it was found that the rubber ring must be wider than the mean surface texture asperity spacing to produce useable outflow times. The rubber ring contacts the asperities in a manner similar to a tire when the cylinder is loaded with metal ring weights. Various rubber compositions, rubber ring widths, and ring weights have been used by researchers.[68]

Two pretesting procedures have been used by researchers when testing with the static drainage outflow meter. First, the rubber ring is conditioned to the pavement surface texture by allowing it to set on the surface about 4 min before the tests are conducted.[33] Secondly, the test surface is flooded with water for approximately 1 min before testing.[18] Following these steps, the cylinder is filled with water to a certain level and the time for a definite volume of water to drain out of the cylinder is measured. The timing can be accomplished manually using a stopwatch and to time the water as it passes two marked positions, usually in the middle one third of the cylinder. The outflow times can be measured to the nearest tenth of a second with this method, but there are inherrent errors associated with the manual measurements. Errors in judgment in stopping and starting the stopwatch can be eliminated by installing electrodes in the wall of the cylinder to electrically stop and start precision timers. The electrodes are capable of determining the time to the nearest milli-second. However, outflow times measured to the nearest tenth of a second are considered satisfactory.[33] The outflow times of the static drainage method usually vary from less than 10 sec for an open-graded texture surface to greater than 200 sec for very fine textured surfaces.[8]

A very similar static drainage outflow meter was developed by the Transportation and Road Research Laboratory (TRRL).[45] The only differences between the use of the TRRL device and the one just described are that the pavement surface is not pretreated by flooding before testing and rather than allowing the water to flow out of the base during filling, a plug is used to hold the water and to release it when actually testing the surface. The TRRL also tested an outflow meter which uses an elliptical rubber plate similar to the shape and size of a tire footprint. This unit is used to detect an anisotropy in the drainage characteristics of a surface.[45]

The second type of outflow meter uses a pressurized drainage method to force the water through the base of the meter. The development of this device grew out of a number of modifications made to the static drainage outflow meter. When the width of the rubber ring used with the first type was increased to make it wider than the mean asperity spacing, the contact area on the pavement surface was increased from 1,930 to 7,100 sq mm (3 to 11 sq in.). To maintain the same contact pressure over the larger area the load had to be increased by a factor of 4. The use of simple weight rings was now too cumbersome so the outflow meter was pneumatically loaded. The loading was later increased to around 159 KPa (23 psi) which is typical of the pressures in a passenger car tire. It was felt that this pressure would be more representative of actual tire-pavement interactions. This increased pressure caused the rubber ring to have a tighter seal with the surface asperities which increased the outflow time beyond practical limits. To reduce the outflow time, the top of the cylinder was sealed and pressurized and a water input line was added to allow filling of the cylinder prior to the test. During testing with later units, the air pressure is maintained below 138 KPa (20 psi) to keep the meter from lifting off of the ground. Typical outflow times range from 0.7 seconds to 5.0 seconds.

The outflow meters which have been described can be used to estimate a pavement performance parameter termed the mean hydraulic radius (MHR). This parameter is a measure of the pavement surface drainage capability and is a function of the macrotexture and porosity of the pavement in addition to the properties of the outflow meter and fluid used in the tests. An empirical equation for the determination of the MHR is given by Moore.[55]

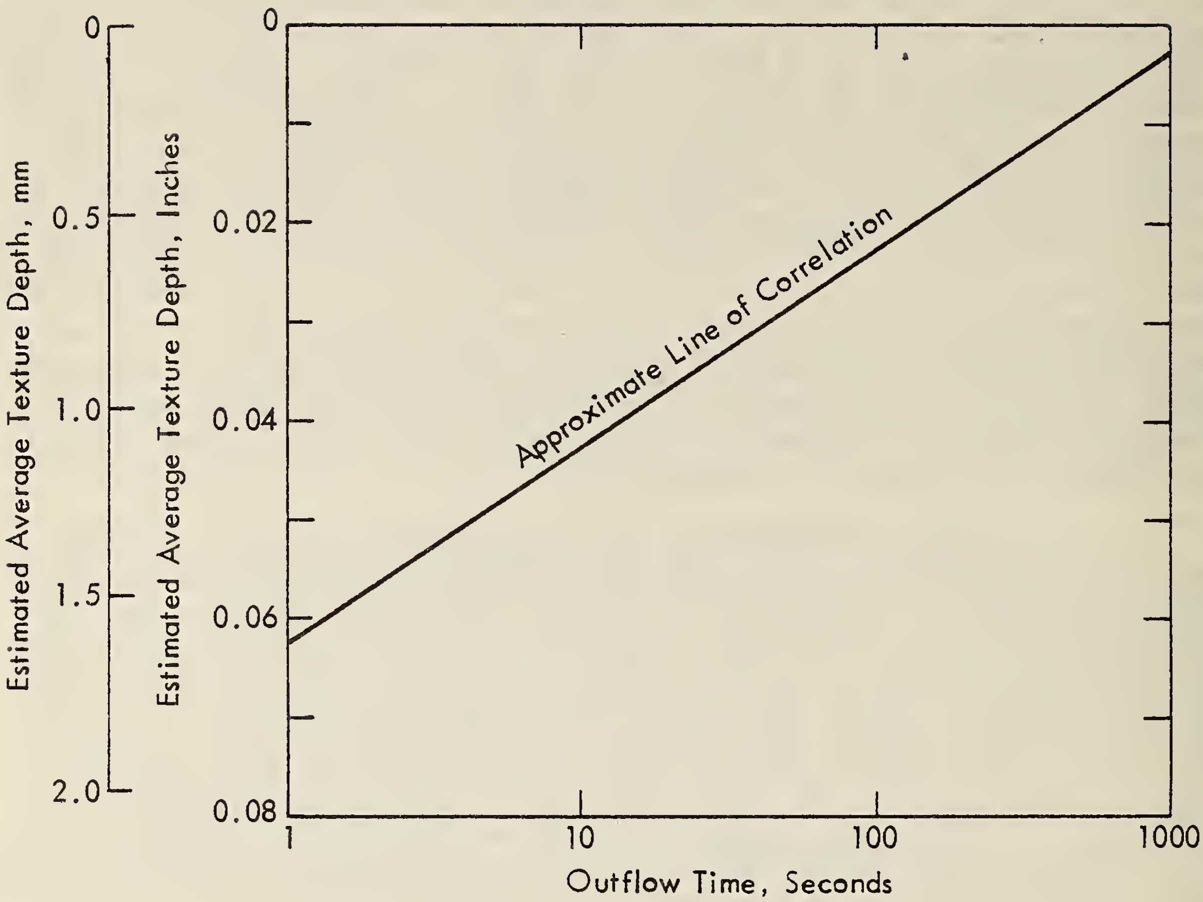
Several sources of error common to both types of outflow meters have been investigated by various researchers.[31,33,34] Gustafson[31] investigated the effects of water temperature on outflow times. He found that the temperature dependence is negligible on coarse pavement textures but is quite strong on smooth pavement surfaces for the temperature range of about 3°C to 38°C (38°F to 100°F).[34]

Other sources of error include the inclination of the outflow meter and various outflow meter-pavement interfacial parameters. Large deviations from the vertical in the inclination of outflow meter can produce significant errors in the volume of water that is timed. However, for cylinder and/or pavement inclination of less than 15 degrees from the vertical, the change in the water volume is less than 4% and is considered to be an insignificant variation.[31] The other parameters which affect the device are the water pressure, contact pressure, and the dimensions and characteristics of the rubber ring. Additional work is needed to explore these variables and their effects more fully. Most of the recommendations to date indicate that the outflow meter is not suitable for use in the field because it is cumbersome, temperamental and generally not very practical[31] nor is it directly useful in establishing pavement design specifications.[34] Also, it is not possible to ascertain the mean hydraulic radius from other texture measurement methods for purposes of comparison.[31]

The static drainage type outflow meter has shown variable repeatability ranging from excellent for rough textured surfaces to unacceptably poor on smooth textured surfaces.[5,18] The outflow meter has better repeatability than the sand patch method and is more sensitive and reliable for pavements that are not too smooth.[33] A graphical relationship between estimated average texture depth and outflow times was reported in a study conducted by the State of Colorado and is reproduced in Figure 3.[8] The outflow times shown are for 620 ml (48.9 cu in.) of water to flow beneath a 95.25 mm (3.75 in.) I.D. rubber ring with an average head of 159 mm (6.25 in.) of water. The average texture depths are not measured at the same locations as the outflow meter measurements but are estimated measurements.

The mean hydraulic radius can be determined from the pressurized drainage outflow meter results with a precision of 1.5%. No correlations were found between the pressurized drainage outflow meter and other texture measuring devices.

Currently six state highway departments are participating in a study sponsored by the Implementation Division of FHWA involving the field evaluation of the static drainage outflow meter. The states involved are Colorado, Louisiana, Mississippi, Nebraska, New Mexico, and West Virginia. The data being collected under this study will be correlated with pavement skid resistance and/or skid resistance-speed gradients under a variety of pavement and weather conditions. Based upon the results of the study, the states are to recommend to the FHWA improvements in the device and items for the preparation of an effective field procedure for the use of the outflow meter.



Source: Reference 8.

Figure 3 - Graph Showing Approximate Correlation Between Average Pavement Texture Depth and Outflow Time

f. Profile tracing: Several types of profile tracers have been developed and used for making both macrotexture and microtexture measurements of pavement surfaces. Five types of macrotexture profile tracers are described in this section. These are: (1) the profileograph (or profileometer); (2) modifed versions of the profileograph; (3) a unit developed by the University of New South Wales; (4) the linear traverse device; and (5) the Texturemeter. The first three macrotexture profile tracers use a stylus which scribes the pavement surface and, through the use of linkages or transducers, reproduces and magnifies the motion of the stylus on recording paper. The linear traverse method uses a microscope whose focus is constantly adjusted as it is moved across the pavement surface. The movement of the focus control is used to plot the profile of the surface on recording paper. The Texturemeter uses a series of rods with a string passed through them. These rods are pressed down onto the pavement surface. As the rod movements conform to the surface, they cause deflections in the string which is indicated on a dial gage calibrated to read average texture depth.

The five types of profile tracers identified above are all employed to measure pavement macrotexture. The profileograph device can also be employed, with appropriate modifications, to measure pavement microtexture. Therefore, for the sake of completeness, two types of microtexture profile tracers are also described in this section. These are: (1) the Gould Surfanalyzer; and (2) the Surfindicator. Both of these units use a stylus to record the microtexture.

Each of the profile tracers is described more fully in the following subsections. These descriptions are followed by a discussion of the distinctions between microtexture and macrotexture profiles, the sources of errors in the profile tracers, and finally, the texture measures determined from profile tracing data.

(1) Macrotexture profile tracing

(a) Profileograph or profileometer: This device was developed by the Texas Highway Department and is strictly a mechanical device for evaluating the macrotexture of pavement surfaces. It is designed to scribe a magnified profile of the pavement surface texture as a motor driven feeler probe is drawn across the surface. A mechanical linkage system magnifies the vertical movement of the probe, and the resulting profile is drawn on a chart. Upward vertical excursions of the probe are also recorded on a counter as the cumulative vertical peak height over the length traversed by the probe.[25,67,68] A reading of 29 on the counter is equivalent to 25.4 mm (1 in.) of cumulative vertical motion of the probe. Therefore, cumulative peak height of the asperities, in inches, is obtained by dividing the counter reading by 29. Average peak height is obtained by dividing the cumulative peak height by the number of peaks. A peak is arbitrarily defined as any magnified asperity profile with a minimum height of 1.6 mm (1/16 in.) and a maximum base length of 6.4 mm (1/4 in.) or any combination of these dimensions with the same total magnitude.

Microscopic damage to surface being traced has been noted with this device. The probe tip has been noted to shear off tips of weak asperities. These conditions plus the hysteresis losses and sluggish motions associated with the mechanical linkage are suspected sources of error in the measurement technique.

(b) Modified versions of the profileograph: Additional profile tracers to measure macrotexture have been developed by Pennsylvania Transportation Institute (PTI) and others.[15,19,34,43] These units are similar to the profileograph described above in that they still use a motorized drive to draw the stylus across the pavement surface. However, these versions use a transducer to transform the stylus motions into electrical signals which drive the profile plotter. Two types of transducers can be used: a displacement transducer that produces signals proportional to the texture height or a linear velocity transducer that produces a signal proportional to the profile slope. Comparisons of the results from both types of transducers show that the integration of the velocity transducer output was "nearly indistinguishable" from the displacement transducer output.[34] The stylus of these units has a diameter of 1.60 mm (0.063 in.) with a cone angle of 20 degrees to 22 degrees and a tip radius of 0.125 mm (0.00492 in.). The stylus is oil quench hardened to a Rockwell hardness of 60 and has a life of around 100 passes of 2,662 mm (11.81 in.) long at a velocity of 3.91 mm/sec (0.154 in./sec).

Profileographs have been found to be rugged, easy to use, rapid in profile tracing and to compare favorably with other profile tracing devices in resolution and output presentation.[15] In addition, they can produce profile tracings with a high degree of reproducibility.[19] The precision of the PTI unit was found to be 0.5% of full scale. Profileometer data collected from 41 test surfaces in Texas were found to be poorly correlated with skid resistance.[25,67] The test pavements consisted of 21 hot-mix asphalt concrete surfaces, 9 portland cement concrete surfaces, 9 surface treatments, and 2 seal coat surfaces. The macrotexture depth of these surfaces ranged from 0 to 1.8 mm (0 to 0.07 in.).

An extremely high correlation of 0.9463 was found between profile data and sand patch data.[69] These results were obtained for a limited number of dense graded asphalt surfaces (20) that had a macrotexture range of from 0.10 to 0.84 mm (0.004 to 0.033 in.) as determined by the sand patch method.

(c) University of New South Wales unit: Another type of profile tracer which uses a transducer was developed by the University of New South Wales.[60] It consists of a counterbalanced lever arm with a diamond tipped stylus with a radius of 0.01 mm (0.0005 in.) mounted in one end. The arm sits on a pivot and as the arm moves up and down a rod, positioned immediately above the arm, is also deflected. This rod is attached to a moving anode triode electro-mechanical transducer. The output of the transducer is monitored for current changes which have been determined to be proportional to the stylus deflection. A chart recorder was used to record the stylus deflections on a chart drum. The drum surface had tape with the sticky side facing out. Fine thread was fed through the chart recorder pen and adhered to the sticky drum as the pen carriage traced the profile of the pavement. When a trace of a fixed or known length was completed the thread was removed and measured. The ratio of profile length (thread length) to center line length (length of trace) provides an estimate of the average peak height of the asperities.

(d) Linear traverse device: The linear traverse device consists of a motorized lathe and a stereo-microscope with the shaft of a potentiometer attached to the microscope focusing shaft.[25,68] The potentiometer is fixed to the microscope and is adjusted with the focus. The output of the potentiometer is fed through an amplifier to a strip chart recorder. In measuring texture, the pavement specimen is moved transversely under the microscope while the operator manually keeps the microscope in constant focus on the surface of the specimen. Focusing on the varying surface elevation results in an amplified tracing on a chart of the surface texture of the specimen. It is then necessary to make measurements of this tracing to produce measures of the pavement macrotexture. Sources of error are associated with the manual focusing and measurements of the tracing. No data were found in the literature relating the linear traverse macrotexture measurements and other macrotexture measurement methods.

(e) Texturemeter: The Texturemeter was developed by the Texas Transportation Institute to measure the average macrotexture depth of a pavement surface in situ.[25,67,68] The device consists of a number of evenly spaced vertical rods mounted in a frame. All but two of the rods can be moved vertically, against spring pressure, independent of one another. One rod at each end of the device is fixed for support. Each movable rod has a hole through which a taut string is passed. One end of the string is fixed to the Texturemeter frame and the other end is attached to the stem of a dial gage extentionmeter mounted on the frame. In application, the rods are held in a vertical position with their ends resting against the pavement surface. If the surface is smooth, the string will form a straight line, and the dial will read zero. Any measurable irregularities in the surface will cause the string to form a zig-zag line and will result in a dial reading from which the average peak height can be calculated or directly indicated. The coarser the pavement macrotexture, the higher the dial reading. The TTI Texturemeter uses a 0.03 mm (0.001 in.) dial gauge and has rods spaced at 15.9 mm (5/8 in.) intervals over a 254 mm (10 in.) length. A similar device, the Rainhart Text-Ur-Meter uses a 0.03 mm (0.001 in.) dial gauge with 29 probes spaced evenly over 254 mm (10 in.).[17] Both instruments are noted to be simple to operate in the field.

(2) Microtexture profile tracing: The profile tracers discussed above have been used to measure the macrotexture of the pavement. A macrotexture profile tracer using a stylus and transducer to produce a trace of the surface can be converted to measure microtexture by changing the stylus size, the stylus traverse velocity, and the transducer sensitivity. Two type of microtexture profile tracing devices that have been used are briefly described below.

(a) Gould Surfanalyzer: The Gould Surfanalyzer (Model 150) has been used to measure microtexture of pavement surfaces under laboratory conditions.[15] It consists of a stylus probe, some associated electronics, and a display for roughness readings. The stylus has a radius of 2.54 x 10^{-3} mm (1 x 10^{-4} in.) with a 90 degree cone angle and a traverse speed of 2.54 mm/sec (0.10 in./sec). The sensitivity of the device is such that a maximum vertical movement of approximately 1.3 mm (0.05 in.) is permitted. No additional data could be found in the literature on this device. The device has a precision of 1% of full scale.

(b) Surfindicator: The Surfindicator is another device that has been used to measure very fine-scaled textures of pavement surfaces.[22,27,68] It was originally developed to measure surface textures of machined surfaces. It consists of a surface datum pickup with a stylus, some associated electronics, and a dial gauge for displaying roughness from 2.54 to 2,540 10^{5} mm (1 to 1,000 10^{-6} in.). The stylus has a conical diamond tip with a radius of about 0.01 mm (0.0005 in.) and is permitted to move vertically a maximum distance of 1.5 mm (0.06 in.). Changes in the speed at which the stylus traverses the pavement surface causes errors in the readings. Consequently, the traverse speed must be controlled.

(3) Distinctions between microtexture and macrotexture profiles: Because most microtexture measuring devices are also capable of measuring small macrotexture features, some technique must be employed to isolate or eliminate this data.[22,27] One technique, utilized on the Surfindicator, is to install a small foot at or around the stylus which rides on top of the large asperity features. The microtexture is measured from a reference point that varies as the macrotexture features vary. This is extremely important with the Surfindicator its direct readings would include the influence macrotexture in the results had it not been eliminated.

Most of the profile tracers discussed use transducers which produce electrical signals that indicate the texture using the magnitude and frequency of the output. A filter can modify the transducer output allowing only microtexture signals to be provided.[15] However, this requires some decision to be made about the cutoff points for microtexture or macrotexture sizes. In a recently completed study involving a computer evaluation of pavement texture, it was determined that texture less than 0.50 mm (0.020 in.) could be considered microtexture and values greater than 0.50 mm

(0.020 in.) could be classified as macrotexture.[69] These same findings were verified by researchers at the Pennsylvania Transportation Institute when pavement profile tracers were subjected to power spectral density analysis.[11,16] Thus, based on these findings, a filter which provides a cutoff frequency of 2,000 cycles per meter (roughly equivalent to one cycle per 0.02 in.) can be used to separate microtexture from macrotexture in the profile tracing. For instance, a filter passing only frequencies greater than 2,000 cycles per meter would eliminate macrotexture influences and yield only a microtexture profile tracing.

(4) Sources of error in profile tracings: Profile tracings made using a stylus can contain errors due to the geometry and dimensions of the stylus. For instance, the contact point of the conical probe tip will vary if the tip is too large to penetrate to the bottom of some of the voids. A profileograph using a stylus with a tip radius of 0.125 mm (0.00492 in.) and a cone angle of 20 degrees will not be able to trace curvatures less than 0.125 mm (0.00492 in.), and slopes greater than 45 degrees will be distorted.[19,34] Thus, care must be taken to see that the geometry and dimensions of the stylus do not fall within the range of texture sizes to be measured. One study of profileometer data[15] indicated that profileometer measurements errors of 10% or less could be expected due to nonlinearity of the electronic components, the stylus size and tilt of the stylus during the tracing.

For microtexture measurements, the stylus probe in its uncertain movements will indent the surface creating an error of around 1×10^{-3} mm (0.3937×10^{-4} in.) in the roughness readings. Indentation does not present a significant source of error in macrotexture measurements because the smallest details measured are generally much larger than the indentation depth.[15]

(5) Texture measures determined from profile tracings: Some of the profile tracing devices provide a direct measure of pavement surface texture through use of a dial gauge or counter. Other devices, such as the profileographs and the linear traverse produce a recording of the profile trace which, in turn, must be analyzed to obtain useful estimates of the pavement texture. The profile analyses can be accomplished by making manual measurements of the profile and performing manual calculations or by digitizing the profile for computer analysis. Both macrotexture and microtexture profile tracings can be processed to obtain such variables as the root mean square (RMS) of the texture height (depth), the arithmetic mean texture depth, and the RMS slope of the texture. Of course, very detailed and complex analysis of the profile traces can be performed quickly through computer analysis. One study[34] used digitizing and computer analysis to obtain: (a) values for the root mean square (RMS) of the texture height and the first and second derivatives of the profile trace; (b) the autocorrelation functions and their first and second derivatives; and (c) the results of various types of power spectral density analyses.

Profile tracing, with careful statistical analyses and data reduction by profile spectral analyses, can be successfully used to predict values for the zero intercept skid number (SN_o) and the percent skid number-speed gradient.[15,43] Strong correlations have been found between SN_o and the microtexture RMS height and between the percent skid number-speed gradient and the macrotexture RMS height.

g. Tire noise: Studies by Eaton[19] and Mitrey, et al.,[54] have atempted to relate tire noise to pavement surface macrotexture. The approach followed in these two studies was to correlate the average texture depth of test pavement surfaces, as determined by another contact method, with the tire noise in a specific frequency band. The average texture depth of a given pavement surface could then be obtained by recording the tire noise for these pavements and applying the derived texture correlations to the recorded data.

In order to evaluate road surface macrotexture from tire noise, six operating parameters need to be held constant to assure repeatability in the measurements. They are:

Vehicle speed,
Wheel load,
Inflation pressure,
Tread pattern,
Degree of tread wear, and
Tire size and construction.[19,54]

Two methods have been used to measure tire noise. Both involve the use of a microphone with a windscreen which is attached to a recording or measuring device. In the first method, the microphone is placed behind the left rear tire, centered with respect to the tire tread, pointing directly at the rear of the contact patch area, and set about 203 mm (8 in.) from the road surface.[19] The microphone output is amplified in a variable gain DC amplifier and this signal is recorded on two channels of magnetic tape. One channel records the entire microphone output and is called the broadband channel. The other channel is used to record a signal which has been filtered to allow only the weaker high frequency signals to pass. These high frequency signals are separately amplified to provide a signal at a high enough energy level to make analysis of these signals easier. The filter is used because the low frequency signals are already at high energy levels and would saturate the tape if they were amplified. A white noise signals of a known sound pressure level (SPL) is recorded periodically as a calibration signal. Various types of spectral analyses can be utilized to reduce the noise data.

The second method[54] uses a totally different procedure to record the tire noise. Rather than continuously monitoring the tire noise from the moving vehicle, the maximum sound pressure level is recorded as the car passes a microphone located 4.6 or 7.6 m (15 or 25 ft) from the pavement edge and 1.4 m (4.5 ft) above ground level. Readings are taken from an SPL meter using an A weighted scale. This method has certain restrictions on the test site. The site needs to be free of reflecting surfaces, wind speeds must be less than 19.3 km/hr (12 mph), the SPL of the tire noise must be at least 10 db above ambient sound levels and the vehicle being measured must be the only vehicle in the test area.

Tests utilizing the first method were run on a test track with five asphaltic surfaces and one portland cement surface. Using a profileograph to determine the texture depth, it was found that the sound pressure level increased as the pavement texture decreased. Significant correlations were found between macrotexture, as measured by the profileograph, in the frequency band of 25 to 8,000 cpm and tire noise in the frequency band of 2,000 to 8,000 Hz. The correlation coefficient, r, for these frequency bands ranges from 0.70 to almost 1.0. The highest correlations were found in the bands of 500 to 1,600 cpm at 2,000 to 4,000 Hz with a range of r from 0.90 to almost 1.0. Resolution, the ability to distinquish between sites, is excellent at 2,000 to 4,000 Hz. Relative repeatability (precision) has been demonstrated and the tire noise data can consistently rank the pavement surfaces in the same order as ranking them using SN_{40} and percent skid number-speed gradient. The technique, however, does not have absolute repeatability because variations of SPL readings on the same or similar surfaces but at different times and days has been noticed.

The second method of texture evaluation was conducted on 11 portland cement surfaces which had five different types of transverse texturing. Using the sand patch method to determine the texture depths it was found that the tire noise increased with increasing texture depths (from 0.2 mm (0.008 in.) to 1.91 mm (0.075 in.)). (It should be noted that this is exactly opposite to the findings of the first method on primarily asphaltic surfaces.) The precision of this second method was based upon three measurements (one each at 64.4, 80.5, and 96.6 km/hr (40, 50, and 60 mph)) at four test sites and 50 independent measurements at each of the three speeds at a fifth location. It was found that, at the 95% confidence level, there were significant differences at all three speeds. It is possible that interactions of vehicle noises could cause difficulty in using this second method. For instance, it was found that engine and exhaust noise dominate the vehicle generated noises below 56.3 km/hr (35 mph) and that tire pavement noises dominate at speeds above 80.5 km/hr (50 mph).[54] In light of this, at least two of the test speeds may be suspected of providing inaccurate data.

2. Noncontact methods: The noncontact methods discussed in this section include: stereophoto-interpretation, modifications of the stereophoto-interpretation, light stylus, laser, light depolarization, photoestimation, and others. The shadow interpretation and white light speckle techniques are included in the other category. These two methods have not been used to measure pavement surface texture characteristics; however, both have potential for application to texture determination. Altogether, 11 different techniques are presented under this general heading.

a. Stereophoto-interpretation method: This method was developed by Schonfeld of the Ontario Highway Department to estimate the skid-resistance of pavement surfaces by analyzing stereophotographs of the surface texture.[68,73] The method has undergone a number of modifications since its conception. What follows is a summary of the latest published description of Schonfeld's method.[74]

In practice, stereophotographs of approximately 101 mm (3.94 in.) square sections of pavement surfaces are obtained with a specially designed camera/box arrangement. The field equipment used is similar to that used in the stereophotography method developed in Europe for measuring pavement texture.[71] In the Schonfeld method, a 35 mm single reflex camera with a focal length of 55-mm is used for taking pairs of stereophotographs. The camera is mounted on a light tight box 457 mm (18 in.) above the pavement. The box is equipped with an electronic flash unit that illuminates the photographed area at an angle of about 45 degrees. The camera is attached to a sliding seat for taking pavement photographs from two positions, about 95 mm (3.75 in.) apart. A horizontal and vertical reference scale, included in the field of view, is also photographed.

The pavement stereophotographs are viewed under a mirro-stereoscope or under a microstereoscope. If a mirro-stereoscope is used, the 35-mm pavement photographs are first enlarged into natural scale black and white prints. These are then viewed at a magnification of 6. A microstereoscope with a magnification of 25, can be used for viewing the 35-mm color transparencies.

Texture elements of the pavement surface are classified visually and are rated subjectively according to an established severity rating for each of six pavement surface parameters. Four of the parameters are used to describe the macrotexture characteristics of the surface: the average width, height, angularity, and density of the larger aggregates or macroparticles. The other two parameters are used to describe the surface microtexture characteristics. One of the two is used to characterize the background microtexture of the pavement surface; the other one characterizes the microtexture on top of the macroparticles.

The surface texture of a pavement is classified by a texture code number which is composed of the set of the six texture parameter numbers. In the stereo-interpretation of the photographs, an operator places a transparent grid with 10-mm squares either over one of the prints or under the transparency depending upon which is used in the analysis. Ten randomly selected squares are marked and numbered on the grid. Each of the numbered squares of pavement surface is then examined under the stereoscope and the number for each of the six texture parameters is recorded. The results of classifying the 10 random 10-mm square areas are then combined to yield the texture code number for the complete photographed surface area under investigation.

Friction weights are determined from correlation plots for each of the six texture parameter numbers and associated severity levels. The friction weights are skid number increments derived from tests with the Ontario Highway Department's locked wheel skid trailer. The photo-interpreted skid number of a pavement surface is determined as the sum of the friction weights for the six parameters. The method yields photo-interpreted skid numbers for test speeds of 49.9 km/hr (31 mph) and 99.8 km/hr (62 mph).

Holt and Musgrove[36] of the Ontario Ministry of Transportation and Communications have developed a skid-resistance photo-interpretation manual based upon Schonfeld's latest work. The manual describes the method of determining and analyzing the six Schonfeld parameters, as well as explaining field sampling procedures, equipment requirements and sample calculations for friction weights and skid resistance numbers.

Schonfeld, in his earlier work,[73] reported a correlation coefficient of 0.9 between SN's obtained from actual skid trailer tests and those obtained from photo-interpretations. However, no estimation of the correlation coefficient was available for the same comparisons using the revised correlation curves.

b. Automation of the Schonfeld method: Automation of the Schonfeld method was performed by Howerter and Rudd[37] to remove the human subjectivity associated with the visual stereo-interpretation and also to provide a more efficient way of implementing the method. This work was pursued by ENSCO, Inc., under an HP&R contract with the Maryland State Highway Administration.

Electronic stereophotogrammetric techniques, previously used in the field of aerial mapping, were adapted to obtain digital data describing the pavement surface from stereophotographs. Comprehensive computer algorithms were developed to process these data and to classify the pavement surface texture automatically. In a demonstration of the automated technique using test surfaces, it was shown that the computer algorithms yielded Schonfeld surface parameter values that were in reasonable agreement with those obtained by the manual method.

Preliminary designs of a camera system, which could be mounted on a highway vehicle and collect the required data, were developed and shown to be feasible for a moderate expenditure. However, the automated system is not cost effective as an operational tool based on current costs for digitization of the stereophotographs and for the associated computer processing of the data.

Subsequent to this work, Rudd[69] reported that ENSCO developed a modified automated approach that requires a much smaller set of data than the original automated method to evaluate the surface texture. This modified method uses a simplified computer algorithm that can yield good estimates of the Schonfeld parameters from single-line profile data. This effort was undertaken to avoid the use of cumbersome stereophotographs and the need to perform off-line stereophotographic digitization.

A recently completed phase of the ENSCO HP&R contract involved the development of specifications for a practical single-line profile scanning instrument for the real-time measurement of surface texture from a moving vehicle.[11] The output of the instrument would be used in a modified version of the automated Schonfeld method using the simplified algorithms.

In the latter ENSCO study, existing instrument concepts (discussed later in this section under laser noncontact methods) were investigated and evaluated, but proved to be inadequate for use on a vehicle moving at 64.4 km/hr (40 mph). A new instrument concept was devised that showed the potential for use in the moving vehicle application. The concept involves projection of a very short duration, slit of light onto the road surface and detection of the resultant illuminated profile of surface texture with a sensitive vidicon television camera. A more detailed description of the concept is presented in the report by Cantor.[11]

Analytical and experimental evaluation of this new concept have verified its feasibility. A prototype of this instrument is currently being built and will be field tested in the near future.

c. Light stylus: The light stylus concept was developed as a noncontact method of measuring pavement surface texture from a moving vehicle traveling at acceptable traffic speeds.[30] Basically, three types of light stylus devices have been designed and laboratory tested under simulated field conditions.

The first type of light stylus uses a narrow beam of intense light which is projected perpendicular to the pavement surface. As the light source moves, the projected light spot on the surface will move up and down with the texture of the surface. By using a focusing lens and a ground glass viewing screen located at an angle, Θ, to the horizontal, an image of the moving light spot will be visible on the ground glass screen. In theory, then, the actual texture depths of the pavement surface can be obtained from direct measurements of the vertical displacement of the light spot (as seen on the viewing screen) that have been adjusted to account for the image magnification and the angle Θ. The nature of the light image is such that it cannot be converted at this time to an electrical signal for rapid processing. Instead, manual measurements must be made of the light image to obtain an estimation of the texture depths. This device has been abandoned in favor of the other two types of light stylus devices.

The second type of light stylus device is a modification of the first type in that it uses a projected narrow slit of light instead of a light beam. The projected light trace is photographed for later analysis. The photographic analysis consists of enlarging the image, measuring the areas between major asperities with a planimeter, summing the areas and then dividing by the length of the profile to obtain a measure of the texture depth. The projected trace can also be analyzed by direct computer analysis if the profile image is illuminated on a phototube. The masking of the project light slit on the pavement by adjacent asperities is a complicating factor in this second type. These confused areas are excluded from the analysis.

The third type of light stylus uses what is called the zero-slope detector. This device also uses a narrow beam of intense light projected perpendicular to the pavement surface. Two photocells are located symmetrically on each side of the light source and in the same plane that the profile is to be traced. As the light source moves across the pavement the reflected light will vary between the two photocells depending on the slope direction (positive or negative) of the pavement asperities. At the peaks and valleys of the asperities, the light will be evenly distributed to each photocell. These points have zero-slopes. If the photocells are used as two legs of a wheatstone bridge, the variations of the output voltage will indicate the direction of slope and the points of zero slope. Because the valleys can be obscured by adjoining aggregate peaks it is advantageous to accurately note only the peaks. Using a diode to block the negative voltages, the peaks will be the only zero slope points counted. The mean void width can be determined by dividing the length of traverse on the surface of the light spot by the number of peaks.

The light stylus devices were developed with the intent of recording pavement macrotexture profile data from a vehicle moving at high speeds. The first device is presently not suitable for practical use. The second and third devices are discussed below.

The second device can be used at high speeds by using a high intensity, short duration strobe as a light source. The strobe will provide an essentially still profile on the pavement which can be photographed for later data reduction in the laboratory. The effects of vehicle bouncing, pitching and rolling are a potential problem for the light stylus techniques. The focus of the light changes with the height of the source above the pavement. A solution to this problem for the second type of device has been to mount the unit close to a fifth wheel trailing the instrumented vehicle at a predetermined distance. This approach appears to filter out the large vehicle motions and provides an acceptable control of the distance between the light source and the pavement. The second light stylus device has produced, under laboratory conditions, macrotexture profiles that compared favorably with other profile tracers and at a faster, but still relatively slow rate of collection. Currently, the second device is not capable of providing immediate texture depth readings in the field. The planimetering technique is the only form of data reduction at present.

This approach for filtering out large vehicle motions is apparently not suitable for the third device (zero-slope detector) because this unit is more sensitive to the divergence of the light beam. It has been suggested that a nondiverging light source, such as a laser beam, be used with the third device in place of the standard light source. However, no data could be found for a unit using a laser beam. The zero-slope detector can provide counts of asperity peaks to within ± 5% as they are encountered at high speed. These data can be used automatically in the field to provide values for mean void width of the pavement texture.

While the light stylus devices were developed to provide data on pavement texture from a moving vehicle, no reports of field test results could be found. Likewise, no correlations between the data from the light stylus devices and other texture measurement methods are available in the literature.

d. Laser: There are two basic types of laser pavement texture measuring devices. The first of these devices is the Autech Laser Dimension Gauge.[11,69/] This unit uses a low power laser to project a small spot of light onto the pavement in the normal direction. Two mirrors are located symetrically about the projection axis of the laser beam and receive the light reflected from the pavement. The two images from the mirrors are then focused onto an image converter. The distance between the two images of the light spot on the image converter will vary with the pavement surface texture. The image converter measures the time it takes to sweep across the two images and converts this information into texture depth measurements made from an arbitrary reference location.

The Autech device samples at a rate of 1,000 hertz and averages the data over 0.01, 0.1 or 1 sec time periods.[11,69/] At this sampling rate and a typical vehicle speed of 64.4 km/hr (40 mph), the device would only be able to measure the pavement surface every 18 mm (0.7 in.).[11/] The low sampling rate, therefore, limits the use of the device to stationary or very slow vehicle speed applications. From calibration tests and manufacturer' data, the device is capable of measuring pavement texture dimensions of from 0.03 to 1.59 mm (0.001 to 0.0625 in.).

The second device was developed independently by both ENSCO and the British Transportation and Road Research Laboratory.[11] The ENSCO/TRRL device consists of a laser light source and an array of photodetectors which is used as the detection device. The light source and detector are located symmetrically about the normal to the pavement surface and at an angle of 45 degrees from the normal. As the light spot is moved across a textured pavement surface, the reflected image will also move across the array of photodetectors. The difference in position of the reflected image from a reference location can be determined and these movements are directly related to pavement texture variations.

The geometric configurations of this device will cause data "drop outs" in the presence of "steep texture geometries." On wet surfaces, false texture depths will be indicated due to the water level in the voids. This device does not sample surface textures fast enough to allow it to be used from a moving vehicle at high speeds but it does provide useful texture depth data when used in a stationary mode. A TRRL report by D. R. C. Cooper[14] contains correlation data of this device with the sand patch method.

e. Light depolarization: The light depolarization device was developed by the Naval Ordinance Laboratory for the Federal Highway Administration and was intended to be used for making texture measurements from a moving vehicle.[28,69] This system involves the use of a linearly polarized laser light source which is directed towards the pavement surface at some angle from the normal. If the pavement were a perfect reflector, no light would be transmitted into the surface and the beam would be reflected completely at an equal but opposite angle to the normal with no change in polarity. However, if the surface were less than an ideal reflector and textured, the light will then be reflected back at other angles and become diffused or scattered. The composite polarization of the reflected wave will also cease to be linear and will exhibit an elliptical polarization. The more textured the surface is, the larger the degree of ellipticity will be. It is this ellipticity, or departure from linearity, that is used to measure the surface texture.

For this device, the laser light source is a helium-neon gas laser radiating at a wavelength of 0.6328 microns (2.491×10^{-5} in.), but any visible or near infrared laser would also work.[28] Th incident wave polarization must be constant and the laser must be of sufficient power (in this case 10 milliwatts) to provide a useful signal to noise ratio.

The receiver consists of a rotating polarizing sheet in front of a photodetector. The rotating polarizer modulates the reflected light according to the degree of ellipticity. The reflecting light passing through the polarizer is sinusoidal and produces a sinusoidal output from the photodetector. The measure of ellipticity is the ratio of the minimum to maximum values of the sinusoidal voltage from the photodetector output.[28,69]

The light depolarization method in its present form cannot be used to directly measure pavement texture; but, it can be used to rank order pavement surfaces according to their degree of texture. The device was used in the laboratory to rank order a number of test specimens of pavements. An area approximately 51 mm (2 in.) in diameter, or a span of about 10 void widths, was illuminated by the laser light source. The test results were then compared with the rank ordering of the specimens using conventional texturing measuring techniques, including sand patch, texturemeter and outflow meter. The rank difference correlations were at least 0.5 in all comparisons. The laboratory measurements with the light depolarization device were found to be repeatable to within ± 3% of the initial test values.

The device has also been field tested from a moving vehicle, but these data were not available to the authors of this report. However, it is known that the test data were not compared with data from other texture measurement devices.

Besides the pavement surface texture, it was found that pavement color and "material properties" of the surface also affect the amount of light depolarization.[28] This effect was not quantifiable and further tests were recommended to explore the extent and significance of these variations. The system did not appear to be significantly affected by simulated vehicle bouncing which would be encountered in real-life high speed applications.

f. Photoestimation (MRI Method): The photoestimation technique was developed by MRI to determine the skid number-speed gradients of pavement surfaces from a moving vehicle. A complete description of this technique is presented in Appendix C of Volume I of this report. What follows is a brief summary of the photoestimation method.

In application of this method, a high-intensity short-duration flash was used to project a beam of light containing shadow lines onto the pavement surface. These lines were then photographed with a 35 mm data camera. The shadow lines were produced by a photographic glass slide with sharp edged opaque bars placed in the light path and focused onto the pavement surface. The slide was positioned at an angle so that a portion of the shadow lines would remain in focus even though the optical path length changed as the vehicle bounced up and down. This beam of light with the shadow bars was projected at a low incidence angle to the horizontal to project shadows across the peaks and valleys of pavement macrotexture.

The illuminated area of the pavement surface was shielded from ambient light sources so that pavement photographs could be taken during the day and at vehicle speed up to 64.4 km/hr (40 mph). The field photographs were rated on a scale of 1 to 5 by comparing them to standard photographs selected through a series of studies using photographs of pavements with known skid number-speed gradients. The ratings of the field photographs were converted to estimated skid number-speed gradient using regression equations.

The photoestimation procedure was found to have a correlation of 0.81 when the estimated gradients were compared to the actual gradient. The method also has a reasonable reliability which is shown by a correlation of 0.69 or greater between the first reading and later rereading of some of the photographic data.

g. Other noncontact methods: Two additional noncontact methods are described below. One is the shadow interpretation method and the other is the white light speckle method. Neither of these techniques have been used to measure pavement surface texture characteristics. However, both have potential for application to texture determination.

(1) Shadow interpretation: The shadow interpretation is a photographic method developed by the Ontario Highway Department to provide visual records of the road surface wear due to studded tires.[23] The method is indicated to also be applicable to macrotexture measurements.

In application, elevation reference studs are countersunk into the pavement and are used to provide a fixed height support for a piano wire strung on a bow shaped angle iron frame. A light-tight box is then placed over the piano wire. This box contains a 35-mm camera which looks vertically downward at the pavement and a photoflash unit which is offset to one side of the wire. The flash unit projects a shadow of the piano wire at an angle to show an oblique cross section of the pavement surface. The shadow is then photographed for later analysis.

The variations in the shadow of the wire in the photographs reflect changes in the pavement texture depth. Estimates of the average texture depth can be calculated using profile analysis techniques and shadow measurement adjusted for the projection angle.

No correlation between data from this method and other texture measurement methods are available in the literature.

(2) White light speckle: The white light speckle method was developed to measure the fine-scaled surface texture on machined metal surfaces without damaging the surface by traversing it with a probe.[80] A white light source is made spatially coherent using a lens and the coherent light source is used to illuminate the surface being measured. When the light

is reflected, the texture of the surface will cause the light to have different pathlengths and to be reflected at some angle depending on the texture depth and the texture slope. When the light is collected and refocused, the light received at each point in the image will be coming from several different points on the object. If these variations are significant, the resulting interference patterns will be seen as speckles.

To provide a useable output for actual determination of the texture parameters, a mask with a pin hole in the center is placed between the refocused light and a photodetector. The pinhole must be smaller than the smallest speckle. It has been determined that the speckle contrast increases as texture depth increases which means that variations in the contrast read by the photodetector will correspond to variations in the texture depth. A narrow trace is made across a two-dimensional speckle pattern and the resulting output of the photodetector can be used to create a profile of the surface. By adjusting the pinhole ahead of the image plane it is possible to create variations in the width cutoff point to effectively filter out unwanted data. This technique shows much promise as a texture measurement device because "it can separate out the average surface height deviations from other surface characteristics and is flexible enough to allow for some choice of roughness width cutoff."[80/]

B. Current State Practice for Macrotexture Measurement

Sixteen state highway departments were contacted by telephone and letters were sent to all the FHWA Regional Offices to determine the methods that have been and are being used by the states to measure macrotexture. This included techniques used both in the laboratory and in field work. We also wanted to know which of the many techniques does the state prefer and on what basis. The states contacted by phone were: California, Connecticut, Kansas, Louisiana, Maine, Maryland, Massachusetts, Michigan, Mississippi, North Carolina, Ohio, Pennsylvania, Rhose Island, South Carolina, Texas, and West Virginia. Personal visits were made to six of these states (California, Kansas, Maryland, Ohio, Pennsylvania, and Texas) to obtain additional information. Also, follow-up telephone contacts were made with FHWA's Region 3, 4, and 7 offices and a personal visit was made with the highway engineers at the Region 7 office.

A majority of the states surveyed had some experience with texture measurement techniques. Those that did have some experience, generally used the methods as an experimental or research tool and not as a source of data for pavement construction acceptance, surface specifications, or surface condition evaluation. The two exceptions to this were the states of Texas and Florida. Texas uses the sand patch method in their specifications for surface texture requirements of portland cement concrete. Florida uses the Text-Ur-Meter results as a variable in their Present Serviceability Index to predict the serviceability rating for pavements and in most of their pavement condition surveys.

A brief summary is given below of some of the state's experience with texture measurement techniques. General comments by some of the state highway engineers on certain measurement techniques are also presented.

1. California: California has tried a number of texture measurement techniques. These include: sand patch, modified sand patch, stereophotographic, profileometer, outflow meter, putty impression, clay casting, and resin casting. A study of the repeatability of the sand patch method has shown the technique to be operator dependent. They have used a stereophotographic method, different than the Schonfeld method, which involves interpreting a stereophoto of the pavement surface using photogrammetric techniques common in the field of aerial mapping. The stereophotograph covers only 3,900 sq mm (6 sq in.) of the pavement surface and it cost about $200 per frame to interpret.

2. Connecticut: Connecticut has used both the putty impression and sand patch methods. Of the two, they feel the putty impression method is more accurate where there are extremes in texture and is simpler to use. They also like the repeatability of the putty impression results. Connecticut has noted three problems with the sand patch method:

- It does not work well on very smooth pavement surfaces;
- It does not work well on open-graded pavement surfaces; and
- The wind distrubs the sand during the testing.

Connecticut has found that the sand patch method tends to indicate a deeper texture depth than the putty impression method on extremely coarse textures. On open-graded friction courses, the sand tends to percolate down below the reach of the putty. In this situation, they feel the putty impression method provides a more realistic measure of the effective texture of the pavement surface in contact with the vehicle tires.

3. Louisiana: Louisiana has used the sand patch, Schonfeld stereophotographic, and outflow meter techniques in their research studies. Of the three, they consider the sand patch to be the best method for their needs. They are one of the six states currently participating in the FHWA-Implementation Division sponsored study involving the field evaluation of the static drainage outflow meter. Louisiana made the first set of outflow meter measurements under this study in the fall of 1977 with the FHWA supplied unit. The measurements will be repeated in April 1978 after which an evaluation report will be written.

4. Maryland: Maryland has been actively supporting, for the past several years, attempts to automate the Schonfeld method for highway surface texture classification. The work has been done by ENSCO under an HP&R contract with the state. Recently, the state's interest has been directed towards the development of speicifications for, and a prototype of a single-line profile scanning instrument for measuring pavement surface macrotexture from a moving vehicle. This work is also being conducted by ENSCO under contract with the state and was described earlier in this section of the report under automation of the Schonfeld method.

Maryland does not like contact methods for measuring pavement surface macrotexture. They feel the methods are too dangerous from a traffic control standpoint. For this reason, their interest have been directed towards methods of making pavement macrotexture measurements from a vehicle moving at highway speeds of 64.4 km/hr (40 mph).

5. Mississippi: Mississippi has used the sand patch, putty impression, and outflow meter methods as research tools and have no preference. In the past, they have not conducted any correlation or repeatability studies of the techniques. However, Mississippi is currently participating in the FHWA sponsored outflow meter evaluation study.

6. North Carolina: North Carolina has used the sand patch method only on an experimental basis and has not performed any study of the repeatability of the technique.

7. Ohio: Ohio has also used the sand patch method on an experimental or trial basis and has not evaluated the repeatability of the technique.

8. Pennsylvania: Pennsylvania has used both the sand patch and sand track methods of measuring pavement surface macrotexture. These methods have been used as a research tool and are not used as part of pavement surface texture specifications.

9. Rhode Island: Rhode Island has frequently used the sand patch method recommended by the American Concrete Paving Association in their Technical Bulletin 19, "Guidelines for Texturing PCC Highway Pavements."[3/] The state has developed a statistical sampling procedure for selecting the roadway locations for making the sand patch measurements.

10. South Carolina: South Carolina has extensively used the sand patch method and has used the results to compare their typical mix designs. However, they do not consider sand patch results appropriate for use in pavement surface specifications. South Carolina believes the technique to be reliable, but has not formally evaluated the method. They use the ACPA's recommended sand patch method with one exception--they substitute local sand

for the specified Ottawa sand because of cost considerations. Sand patch measurements in the field are conducted in the wheel path of the traffic lanes. They have not experienced any particular traffic control problems when making the field measurements and they estimate it takes at most 10 min to make a single sand patch measurement at a given location.

South Carolina has studied other methods of making texture measurements, including the sand track method, but they have not used any of them because of satisfaction with the sand patch technique. Although they favor the sand patch method, they feel it has two disadvantages:

- The pavement must be completely dry before the texture measurements can be made. The method will not work if the pavement is wet.

- Tests cannot be performed in the wind without the use of a wind screen because the sand is susceptible to being blown away by the wind.

11. Texas: Texas has used both the sand patch and modified sand patch methods. The sand patch test used is described in the Texas standard procedure document as procedure Tex 436-A and involves deploying a known volume of sand. (The modified sand patch method involves spreading sand over a known surface area.) Texas has also tried profile tracing, putty impression, the Text-Ur-Meter, and the Schonfeld technique. A report by Ledbetter, et al.,[44] presents a linear correlation between results obtained with the sand patch method (Tex 436-A) and the putty impression method ($r = 0.98$). Texas uses the sand patch method in their specifications for surface texture requirements of portland cement concrete.

12. West Virginia: West Virginia has used both the sand patch and the stereophoto-interpretation methods, but only on an experimental basis. The state has not conducted any study of the intercorrelation of different texture measurement techniques. West Virginia is also participating in the FHWA sponsored outflow meter evaluation study.

13. Kansas: Kansas has used the linear traverse, Schonfeld stereophoto-interpretation, sand patch and outflow meter techniques. Both the linear traverse and stereophoto-interpretation methods have been used in a laboratory study of wear and polish susceptibility of pavement specimens. The sand patch was used only once on an experimental basis. Kansas has used the outflow meter in the field. They do not like to use the outflow meter on dense-graded asphalt pavement surfaces because the outflow times tend to be excessive--in some cases up to 15 min.

14. FHWA Region 3 response: Pennsylvania, Maryland and West Virginia are the states in this region that have the most macrotexture measurement experience. The activities of each of these three states have been discussed.

15. FHWA Region 4 response: The methods most frequently used in this region to measure pavement texture are the sand patch, putty impression, grease patch, and the Text-Ur-Meter. The sand patch is used more commonly than the other methods and, generally, only as a research tool. For example, South Carolina uses the sand patch method results to compare the texture of different types of asphalt mixes, but not for pavement construction acceptance, surface specifications, or condition evaluation. An exception to this is Florida which uses the Text-Ur-Meter. Florida uses the Text-Ur-Meter data as a variable in the Present Serviceability Index to predict the serviceability rating for pavements. These data are also used in most all of Florida's pavement condition surveys.

16. FHWA Region 7 response: The two states most active in this region in making pavement surface macrotexture measurements are Kansas and Nebraska. The work of Kansas has already been discussed. Nebraska is another one of the participating states in the FHWA evaluation study of the outflow meter. Nebraska's outflow meter study began in July of 1977. In addition to evaluating the outflow meter, they also intend to develop a correlation between the sand patch and outflow meter results.

IV. METHODS OF ACHIEVING MACROTEXTURE IN NEW PAVEMENTS

There are two types of pavement surfaces that can be used to achieve a high level of macrotexture in new pavements: open-graded asphalt surface courses for bituminous pavements and textured surfaces for portland cement concrete pavements. Both of these techniques are described in this section. Open-graded asphalt friction courses can also be used to overlay existing pavement and, therefore, are also discussed briefly in Section V. Texturing of new portland cement concrete surfaces is accomplished when the concrete is in the plastic state and, therefore, is discussed only in this section. Retexturing of existing portland cement concrete surface can be accomplished through pavement grooving and cold milling and a discussion of these techniques is given in Section V.

A. Open-Graded Surfaces for New Bituminous Pavements

The highest levels of macrotexture depth for new bituminous pavements have been achieved with open-graded asphalt surface courses. The macrotexture depth of an open-graded surface, as determined by the sand patch method, is usually in the range of 1 to 3 mm (0.04 to 0.12 in.) as compared to less than 0.3 to 3 mm (0.01 to 0.04 in.) for conventional dense-graded bituminous surfaces.

An open-graded asphalt mixture is one containing relatively little fine aggregate and mineral filler. The standard FHWA specification for open-graded asphalt requires between 5 and 15% of aggregate by weight passing the No. 8 sieve and between 2 and 5% passing the No. 200 sieve. This gradation requirement eliminates many of the fines that would fill in the voids in a dense-graded mixture. The increased size of voids provides an open texture that allows water to escape more readily from the tire-pavement interface providing a smaller skid number-speed gradient. Thus, an open-graded surface provides increased skid number at high speeds due to the flatter slope of the skid number-speed relationship.

Open-graded surfaces have been used for bituminous pavements since the late 1940's in the Western United States. A landmark study, published in 1968, found that open-graded surfaces had a higher level of skid resistance than any other type of bituminous pavement studied.[53] As a result of this finding and a subsequent demonstration program sponsored by the Federal Highway Administration, open-graded surfaces have been constructed in 49 of the 50 states plus the District of Columbia and Puerto Rico.[57]

Open-graded surfaces have been known variously as "plant mix seal coats," "open-grades asphalt friction courses," "porous friction courses," and "popcorn mixes" in the states where they have been used. The multiplicity of terms has developed because of the variety of objectives for which open-graded surfaces have been used. Originally, open-graded surfaces were used as an overlay to seal an existing pavement surface, hence, the name "plant mix seal coat." When the superior friction properties of the material became apparent, the Federal Highway Administration adopted the name "open-graded asphalt friction course." This term is used solely to identify materials that meet the FHWA specification discussed below. The Federal Aviation Administration prefers the term "porous friction course" for open-graded asphalt applied to airport runways. The term "popcorn mix," describing the general appearance of an open-graded surface, has been used by some highway engineers in a colloquial sense and is not considered a standard phrase.

1. Specification: Each of the states that have used open-graded asphalt surfaces have developed specifications for designing, mixing, and placing them. In some states, the specifications have been adopted as a published standard. In other states, the specification is still experimental and may be varied from job-to-job.

The macrotexture of an open-graded asphalt mixture is determined by the aggregate gradation. All states contacted by the authors have chosen to control macrotexture by specifying the gradation of the aggregate used in the open-graded asphalt mixture rather than by requiring a minimum level of macrotexture depth in the surface course. The reluctance of states to specify macrotexture depth as an acceptance criterion is based on lack of agreement on the best method of measuring macrotexture and experience in obtaining predictable skid resistance and texture from their specified aggregate gradation.

The following master ranges of aggregate gradation for open-graded asphalt mixtures are recommended by the Federal Highway Administration.

Sieve Size[a/]	Percent Passing[b/]
12.7 mm (1/2 in.)	100
9.5 mm (3/8 in.)	95-100
No. 4	30-50
No. 8	5-15
No. 200	2-5

a/ U.S. Standard Sieve Series.

b/ By volume (this is the same as by weight unless the specific gravities of aggregate being combined are different).

The recommended aggregate gradation has a nominal maximum size of 9.5 mm (3/8 in.), but experience has shown that up to 5% of material between

95 mm (3/8 in.) and 12.7 mm (1/2 in.) does not change the skid resistant properties of the surface. A specific aggregate gradation within the recommended master ranges for each sieve size is used for each job.

A recent survey of state highway departments conducted by the National Cooperative Highway Research Program (NCHRP) found a wide variation in master ranges specified by individual states.[57] The authors of the NCHRP report suggest that these variations result from the need for each state to adapt its specification to a locally available aggregate. However, the NCHRP survey found the typical gradations used by the states on specific jobs were more similar than the master ranges might suggest. The average gradation for all of the states surveyed fell within the FHWA specification band.

Open-graded mixtures require good, durable aggregate to avoid rapid loss of their skid resistant properties due to traffic polishing. There are two possible approaches to the problem of eliminating soft, polish-susceptible aggregates from use in open-graded surfaces. The first approach uses physical testing to establish durability and polish-susceptibility. FHWA recommends that aggregate in open-graded mixtures not exceed 40% abrasion loss using the standard abrasion test procedure (AASHTO T 96). Another physical method used by some states is the acceptance of aggregates after simulated traffic exposure on a polishing machine and satisfactory test results with the British Portable Tester. The second approach is to exclude aggregates on the basis of petrographic analyses of their components. Many, but by no means all, carbonate aggregates (such as limestone) are too polish-susceptible for use in open-graded surfaces. However, most states have not had good success in trying to predict pavement performance from the results of petrographic analyses. One procedure that has been used involves the determination of the percentage of acid insoluble residue which is an indicator of carbonate content and, by inference, polish-susceptibility. An interesting example of the use of the insoluble residue test in specifying aggregate for skid-resistant bituminous surfaces in New York is provided by Kearney, McAlpin and Burnett.[41] Carbonate aggregates that contain less than 10% sand-size impurities or mixtures of carbonate and noncarbonate aggregates that contain less than 20% noncarbonates are upgraded by blending with 20% noncarbonate stone.

A formal mix design procedure is used to select the asphalt content of an open-graded mixture. The design procedure recommended by FHWA is described in Appendix A. In this procedure, the asphalt content is determined by use of the modified California oil equivalent test to measure the surface capacity for the course aggregate fraction (passing the No. 4 sieve). The surface capacity, K_c, is determined from the percentage of SAE No. 10 oil retained when an aggregate sample is immersed in oil and drained. The oil equivalent test should be performed with a sample of the same aggregate that is planned for use on the job. The appropriate percent asphalt to be used in the mixture is determined from the relationship:

$$\text{Percent Asphalt} = 2.0\ K_c + 4.0$$

In the FHWA procedure, this asphalt content is used in the job mix without further testing.

A somewhat more laborious procedure has been developed by the Colorado Department of Highways.[8/] An estimated optimum asphalt content is estimated using the relationship:

$$\text{Estimated Optimum Asphalt Content} = 1.5\ K_c + 3.5$$

Then, the actual asphalt content to be used on the job is selected from laboratory testing of trial mixes below, at, and above the estimated optimum asphalt content.

2. Costs: Costs of open-graded surface courses vary substantially throughout the United States. Typical costs for open-graded asphalt mixtures were recently provided by the states of Maine, Maryland, Michigan, Pennsylvania, and South Carolina. These states provide costs on a common basis--cost per ton in place. The average cost per ton for open-graded asphalt mixtures for these five states is $19.90. For example, the yield for a 25.4 mm (1 in.) open-graded asphalt surface course might be 51.5 km/sq m (95 lb/sq yard) (based on 6.5% asphalt by weight, 15% air voids and specific gravities of 2.65 and 1.00 for the aggregate and asphalt cement, respectively). Based on this yield and the average cost per ton, a typical cost per square yard for a 25.4 mm (1 in.) open-graded asphalt surface course in these five states is $0.95. The State of California reports placing 25.4 mm (1 in.) open-graded asphalt surface courses for approximately $1.44 per sq m ($1.20 per sq yard). Combining this cost estimate with those of the other five states yields an average cost per square meter for open-graded surface courses of $1.19 ($1.00 per sq yard).

The limitation on the polish-susceptibility of aggregates is often cited as an economic restriction on the widespread use of open-graded asphalt friction courses in some areas. A number of states do not have suitable aggregate available locally and must import aggregates substantial distances to construct open-graded surfaces. This added expense, naturally, makes these surfaces less attractive on a budgetary and cost-effectiveness basis. However, some states report that substantial aggregate haul distances result in only modest increases to the cost of open-graded mixes. For example, one state reports that open-graded surfaces have been placed for $1.44 per sq m ($1.20 per sq yard), despite an aggregate haul distance of over 804.7 km (500 miles).

3. Service life: There are no reliable data to compare the service lives of open-graded and conventional dense-graded surfaces. Some open-graded surfaces have been in service as long as 12 years with no apparent problems. However, other open-graded surfaces have required early replacement. Many states have only begun placing open-graded surfaces within the last 3 years, so these states do not yet have any basis for service life comparisons.

4. Performance: Most states have found open-graded surfaces to perform adequately. In some cases, performance problems such as flushing or raveling occurred on the first open-graded surfaces placed in an area. Flushing results when the asphalt content of the mix is too high and raveling results when the asphalt content is too low or when the aggregate is not completely coated with asphalt. However, such problems have largely disappeared as pavement designers and contractors have become familiar with the material, and the appropriate construction practices. In some states, field engineers and inspectors were reluctant to use open-graded mixtures since they often require unusually high asphalt contents and relatively low mixing temperatures. These concerns have largely disappeared when the pavement surfaces have performed adequately. The California Department of Transportation has found that open-graded asphalt surface perform poorly on steep grades in severe winter climates where use of tire chains is common. Therefore, California has restricted use of open-graded asphalt surfaces to elevations less than 0.9 km (3,000 ft) above sea level. An excellent discussion of construction and performance problems with open-graded surface courses can be found in a recent report from Texas by Gallaway and Epps.[23]

An important consideration in effectively constructing an open-graded surface is that the open-graded asphalt mixture must be placed over both the roadway and shoulders to obtain the full drainage advantages of the mixture. If the open-graded surface covers the roadway, but not the shoulders, then the flow of water off the pavement is restricted and much of the benefit of the open-graded surface may be lost. There is also a consensus that open-graded asphalt surfaces must not be placed under ambient temperatures less than 15.6°C (60°F).

Texas has found that open-graded asphalt surfaces require routine preventive maintenance. The asphalt cement binder tends to become brittle and needs to be renewed every 9 to 12 months by spraying an asphalt emulsion, known as a "fog seal," onto the pavement. This seal creates a very slick surface for 1 or 2 days, but prolongs the life of the open-graded surface.

Many engineers were initially concerned that open-graded surfaces would produce higher traffic noise levels than dense-graded surfaces. However, the experience of many agencies appears to be just the opposite. Kay and Stevens[40] evaluated the noise generated by three types of tires on pavements in Arizona, California and Nevada and found that open-graded asphalt surfaces produce slightly lower noise levels than dense-graded surfaces, portland cement concrete surfaces or chip-seal coats.

Open-graded asphalt surfaces were found by Mills[53] to have higher skid numbers at 64.4 km/hr (40 mph) than conventional dense-graded surfaces in four western states. Subsequent studies such as the work by Adam and Shah,[1] have also shown that open-graded asphalt surfaces have flatter skid number-speed gradients than dense-graded surfaces. These comparative findings concerning the skid numbers and skid number-speed gradients of dense- and open-graded mixes are also illustrated by the test results in Table 1, which were obtained by the Texas Department of State Highways and Public Transportation.[59] Therefore, many proponents of open-graded surfaces see them as the best available method to achieve good skid resistance at speeds higher than 64.4 km/hr (40 mph). However, some critics contend that the differences in skid resistance between open-graded and dense-graded surfaces arise primarily because higher-quality (higher microtexture) aggregate is often used in open-graded mixes. It is argued, quite reasonably, that good skid resistance can also be achieved by the use of high-quality aggregates in dense-graded surfaces. The Penn State model for skid resistance, presented in Section II, shows that both microtexture and macrotexture have a role in providing high skid resistance. Therefore, open-graded surfaces should generally have higher skid numbers than dense-graded surfaces made from the same aggregate. A rational choice must involve consideration of the cost-effectiveness of these alternatives. A formal analysis of the costs and benefits of open- and dense-graded surfaces is illustrated in Section VII of this report. However, no single analysis can adequately represent conditions in all states and regions, so determination of the actual cost and benefits experienced by individual states should be encouraged.

5. Advantages and disadvantages: The major advantages and disadvantages of using open-graded asphalt friction courses for new pavements are presented in Table 2. Additional advantages and disadvantages that apply only to the use of open-graded asphalt friction courses for pavement overlays are discussed in Section V.

The advantages presented for open-graded asphalt friction courses show that they perform well in many ways. The disadvantages do not limit the performance of open-graded surfaces but do contribute to increased costs. For a more complete discussion of the advantages and disadvantages of open-graded asphalt friction courses the reader is referred to the recent NCHRP Synthesis of Highway Practice, entitled "Open-Graded Friction Courses."[57]

TABLE 1

TEST RESULTS FOR DENSE-GRADED AND OPEN-GRADED ASPHALT PAVEMENT SURFACES[59]

	Average Skid Number			Average Skid Number-Speed Gradient (SN/kph (SN/mph))[a]	Average Percent Skid Number-Speed Gradient[a]	Average Texture Depth[b] (mm (milli-in.))	Average Peak Noise Level (dBA)		
	64.4 kph (40 mph)	80.5 kph (50 mph)	96.6 kph (60 mph)				64.4 kph (40 mph)	80.5 kph (50 mph)	96.6 kph (60 mph)
Hot-mix asphalt (conventional dense-graded)	33	31	28	0.17 (0.27)	0.82	0.65 (25.6)	68	71	74
Seal coat (open-graded)	48	45	43	0.14 (0.22)	0.47	1.87 (73.5)	69	72	76

a/ The definitions of skid number-speed gradient and percent skid number-speed gradient are given in Section II. The values in this table are averages of values from several pavement sections, so the average gradient cannot be calculated directly from the average skid numbers listed.

b/ Determined by sand patch method.

TABLE 2

OPEN-GRADED ASPHALT FRICTION COURSES FOR NEW PAVEMENTS

Advantages	Disadvantages
1. Skid resistance at high speed is higher than for dense-graded surfaces.	1. Durable, polish-resistant aggregate is not readily available in many states.
2. Hydroplaning potential is lower than for dense-graded surfaces.	2. Early preventive maintenance is necessary.
3. Splash and spray from vehicles on wet-pavements is less than for dense-graded surfaces.	3. Petroleum product spills cause rapid deterioration of the surface course.
4. Noise levels are lower than for dense-graded surfaces.	4. Service life may be shorter than for dense-graded surfaces.
5. Smooth ride.	5. If handwork is required in placement, a different texture results.
6. Minimum thickness requires less high-quality skid-resistant aggregate.	6. Patching is difficult.
7. Traffic can use surface almost immediately after placement.	7. Must be placed at ambient temperatures above 15.6°C (60°F).
8. Wheel path rutting is minimized.	8. Susceptible to damage by tire chains on steep grades.
9. Less glare at night during wet weather, than for dense-graded surfaces.	9. Relatively more deicing salts may be necessary to keep the surface free of snow and ice than for dense-graded surfaces.
10. Better wet weather night visibility of pavement markings than for dense-graded surfaces.	10. Pavement appears to remain wet even after the surface asperities are dry.
11. Frost formation on the surface may be retarded.	11. Some overlays adhere poorly to an old open-graded surface.

B. Texturing of Portland Cement Concrete Surfaces

The macrotexture of new portland cement concrete surfaces must be established by texturing or finishing the surface of the roadway while the concrete is in the plastic state, i.e., before the concrete has hardened. Historically, most portland cement concrete pavement surfaces have been finished by the burlap drag technique, which is described below. However, many other methods for finishing pavements are available. Thirteen alternative methods, together with their resulting initial macrotexture depths, as determined by the sand patch method are listed in Table 3 to illustrate the variety of texturing methods. However, the initial texture depths for some finishing methods have been found to decrease rapidly due to traffic wear. As state highway departments began perceiving the need for deeper pavement texture than could be obtained with the burlap drag, a number of these texturing methods were tried. The most commonly used methods have included belting, brooming, and wire tining. Most research has identified wire tining methods (also called wire combing or plastic grooving) as being the most effective in creating deeper texture depths and higher skid numbers. In 1976, the Federal Highway Administration, recommended transverse wire tining as the most practical and dependable method of providing macrotexture for new portland cement concrete pavements.

TABLE 3

INITIAL TEXTURE DEPTHS FOR VARIOUS FINISHES[13/]

Method of Finish	Texture Depth (mm (in.))
Wood float	0.36 (0.014)
Light belt	0.38 (0.015)
Light burlap drag	0.43 (0.017)
Heavy belt	0.51 (0.020)
Steel wool	0.56 (0.022)
Heavy burlap drag	0.64 (0.025)
Wallpaper brush	0.66 (0.026)
Medium paving broom	0.74 (0.029)
Door mat (cocoa matting)	0.81 (0.032)
Wire drag	0.91 (0.036)
Heavy paving broom	0.94 (0.037)
Flexible wire brush	1.30 (0.051)
Stiff wire brush	1.91 (0.075)

1. Specification: This section describes the methods for texturing portland cement concrete in the plastic state including burlap drags, brooming, belting and wire tining. The other methods listed in Table 3 are not described because they are not in general use.

a. Burlap drag finishes: Burlap drag finishes are the oldest method of texturing PCC pavements. In 1963 about 60% of highway departments used the burlap drag method exclusively.[13/] In this method, longitudinal striations are formed in the concrete surface by dragging burlap or other material behind the paving machine. Refinements of this method have used artificial grass or carpet instead of burlap to create a deeper texture. In another variation, deeper texture is created by allowing mortar to accumulate on the trailing threads of the burlap.

A typical specification for burlap drag used by the Mississippi State Highway Department reads: "A drag shall be used which shall consist of a seamless strip of damp burlap or cotton fabric, which shall produce a uniform surface of gritty texture after dragging it longitudinally along the full width of pavement. For pavement 4.9 m (16 ft) or more in width, the drag shall be mounted on a bridge which travels on the forms. The dimensions of the drag shall be such that a strip of burlap or fabric at least 0.9 m (3 ft) wide is in contact with the full width of pavement surface while the drag is used. The drag shall consist of not less than two layers of burlap with the bottom layer approximately 152.4 mm (6 in.) wider than the upper layer. The drag shall be maintained in such condition that the resultant surface is of uniform appearance and reasonably free from grooves over 1.6 mm (1/16 in.) in depth. Drags shall be maintained clean and free from encrusted mortar. Drags that cannot be cleaned shall be discarded and new drags substituted."

b. Broom finishes: In this method, a broom is used to form striations in the pavement surface. Although the burlap drag forms longitudinal ridges in the plastic concrete, brooming is usually done in a transverse direction across the pavement. A typical specification for transverse brooming used by the Connecticut Department of Transportation reads: "Brooming operations shall be performed when the water sheen has practically disappeared, prior to initial set and before the concrete is in such a condition that the surface would be torn or unduly roughened. The brooming shall extend transversely from edge to edge of the pavement. The broom shall be 3 m (10 ft) long and the machine shall be operated in such a manner that successive passes of the broom will overlap the previous pass by about 0.3 m (1 ft)." The types of brooms used include nylon bristle brooms, steel brooms, and poly-plastic brooms.

c. Belt finishes: Like broom finishes, belt finishes are used to form transverse striations in the pavement surface. Belts are usually made of two-ply canvas and can be used manually to texture pavement surfaces. The Mississippi State Highway Department specification for a belt finish reads: "...when straight-edging is complete and water sheen has practically disappeared and just before the concrete becomes nonplastic, the surface shall be belted with a two-ply canvas belt not less than 203 mm (8 in.) wide and at least 0.9 m (3 ft) longer than the pavement width. Hand belts shall have suitable handles to permit controlled uniform manipulation. The belt shall be operated with short strokes transverse to the road center line and with a rapid advance parallel to the center line."

d. Wire tine finishes: Wire tine finishes are placed with a steel comb with teeth or tines that are dragged across the pavement surface. The texture pattern produced in the plastic surface of the portland cement concrete is determined by the size and spacing of the tines. Both longitudinal and transverse tining have been used by state highway departments. A typical specification for transverse tining used in Mississippi reads: "The final surface texture shall be produced with a metal tine finishing device. The tines shall be approximately 0.81 x 2.11 mm (0.032 x 0.083 in.) steel flat wire, 102 to 127 mm (4 to 5 in.) in length spaced on 12.7 mm (1/2 in.) centers. The texturing device shall be so constructed and operated as to produce uniform transverse parallel grooves normal to the centerline of the pavement and 12.7 mm (1/2 in.) on centers and having a depth of 4.8 mm (3/16 in.) determined as set out herein below...The final depth of the finished grooves will be determined by the accurate use of a standard commercial tire tread depth measuring gauge with 0.8 mm (1/32 in.) graduations so arranged as to be easily and accurately read."

2. Cost: The cost of texturing of portland cement concrete in the plastic state is difficult to estimate because most state highway departments do not pay for texturing as a separate item. Ledbetter, et al.,[44] found no cost difference in several experimental finishes that they compared and most engineers agree that the difference in cost between texturing methods is negligible. Most finishes including longitudinal tining are produced with a float machine or curing rig, so construction contractors do not find it necessary to purchase a new piece of equipment. Transverse brooming and tining may be an exception to this general rule and may, therefore, involve a small additional cost. Since cost is not generally a factor, the texturing method that provides the deepest macrotexture, highest skid number and lowest wet-pavement accident experience should be adopted.

3. Service life: The service life of portland cement concrete finishes is difficult to specify exactly because it depends on the depth of the initial macrotexture for the type of finish used, the traffic volume and the use of tire chains or studded tires. There are, however, several sources that provide an indication of the rate at which the texture degrades. For example, the results of a Texas study by Ledbetter, et al.,[44] summarized in Table 4, indicate the amount of loss in texture depth of

seven surface finishes over a 30-month period. The study also evaluated 11 additional surfaces over a 7-month period. The study concluded that the texture depth decreased an average of 25 to 35% with traffic wear and then leveled off. A Georgia Department of Transportation study reported by Thornton[82] established a relationship between skid number and cumulative traffic passages. These results are shown in Figure 4, for six different surface finishes. Both the Texas and Georgia studies involved pavement surfaces that are not exposed to tire chains or studs.

TABLE 4

TEXTURE DEPTH DEGRADATION AFTER 30 MONTHS[44]

Type of Finish	Texture Depth (mm (in.)) Dec. 71[a]	Texture Depth (mm (in.)) June 74[b]	Percent Loss Over 30-Month Period
Transverse broom	1.45 (0.057)	0.76 (0.030)	47
3.2 mm (1/8 in.) transverse tines	1.63 (0.064)	1.27 (0.050)	22
Longitudinal broom	0.91 (0.036)	0.46 (0.018)	50
3.2 mm (1/8 in.) longitudinal tines	2.36 (0.093)	1.30 (0.051)	45
Burlap + 3.2 mm (1/8 in.) longitudinal tines	2.11 (0.083)	1.57 (0.062)	25
Burlap drag (control)	0.71 (0.028)	0.58 (0.023)	18
Transverse brush	0.79 (0.031)	0.77 (0.026)	16

a/ Determined by putty impression method and corrected to correspond to sand patch method.

b/ Determined by sand patch method.

4. Performance: The Texas and Georgia studies mentioned above also provide an indication of the performance of portland cement concrete finishes. Ledbetter, et al.,[44] conclude that, under simulated rain conditions, deep transverse texturing resulted in the greatest improvement in skid number. The results of the Thornton study,[82] presented in Figure 4, show the clear superiority of wire tine finishes over broom and burlap drag finishes. Tines provided the highest skid resistance of any finish tested and the traffic wear rate for the 12.7 mm (1/2 in.) tine spacing is less than any other finish except the burlap with grout build-up, which had much lower skid numbers. A report by Balmer[6] stated that finishes produced by nylon brooms, flexible fine wire brushes, wire drags, and modified burlap drags

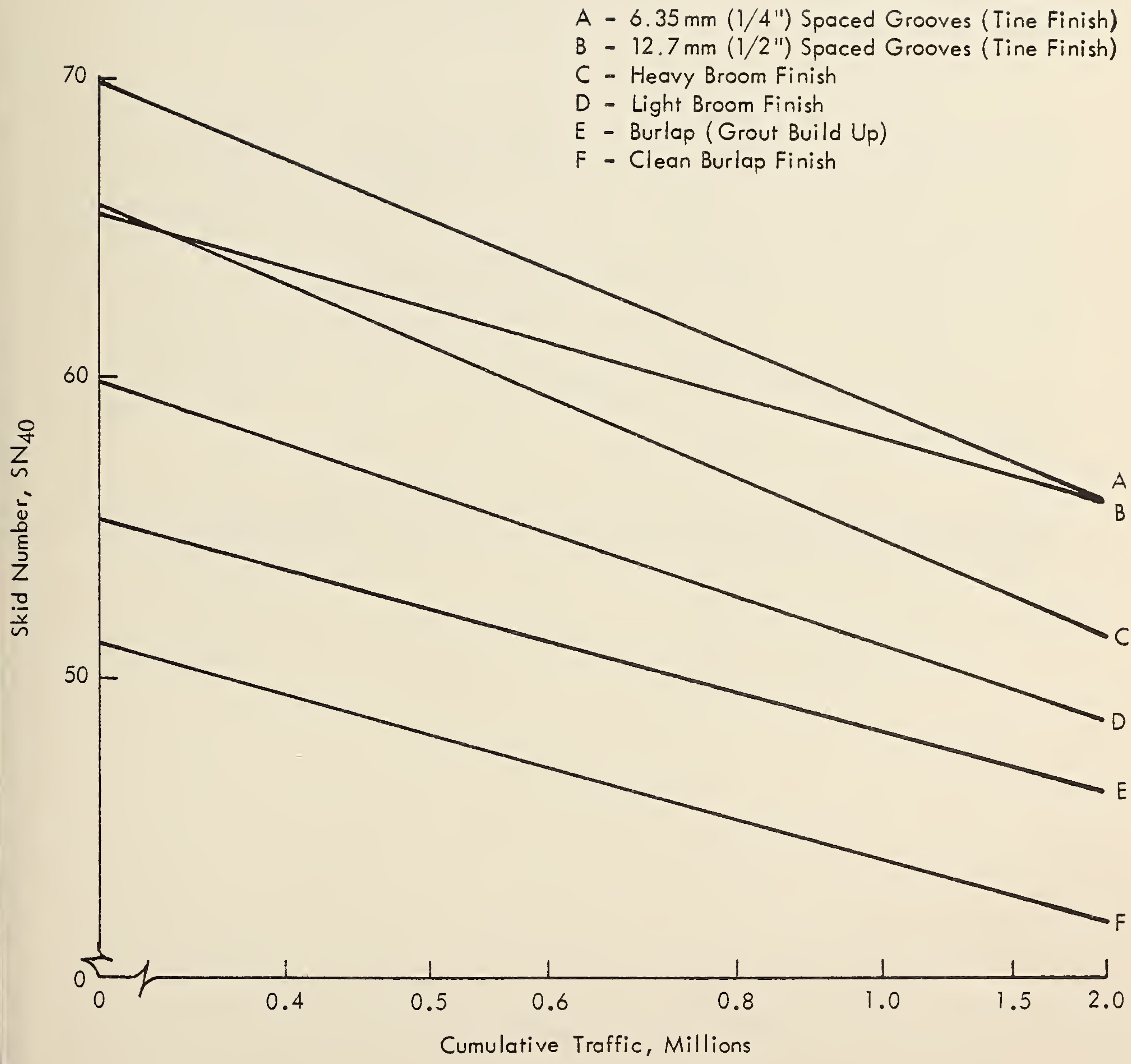


Figure 4 - Skid Resistance Change From Traffic on PCC Pavements

met with only limited success. The report also stated that greater success was achieved by wire tining, and that it is desirable to finish pavements with a burlap drag before tining to produce a gritty texture between the grooves.

Some concern has been expressed that deeper textures for portland cement concrete surfaces would produce higher noise levels. Scarr[72] evaluated nine surface finishes including burlap drag, brooming, and metal tining and found no substantial differences in noise levels. Thus, any apparent differences in the sound produced by different surface finishes may result from differences in pitch and not from increased noise levels.

Each study of texturing has concluded that wire tining is the most desirable finish for portland cement concrete surfaces. On the basis of this strong evidence, the Federal Highway Administration has recommended the use of wire tine finishes. Transverse rather than longitudinal tining was recommended by FHWA because transverse texturing provides higher skid numbers due to superior drainage. Recent conversations by the authors with 15 state highway departments determined that all but one of these states have adopted longitudinal or transverse tining as the standard finish for portland cement concrete. The impact of these new texturing specifications in many states has not been great, however, because very few portland cement concrete pavements are being constructed. One important area where wire tine finishing is having an impact is in texturing the surfaces of portland cement concrete bridge decks that are replaced.

5. Advantages and disadvantages: The advantages and disadvantages of the four most commonly used methods of texturing portland cement concrete bridge decks are presented in Table 5.

TABLE 5

TEXTURING OF NEW PORTLAND CEMENT CONCRETE SURFACES

Advantages	Disadvantages
Texturing Method: Burlap Drag Finish	
1. Requires no additional equipment	1. Provides small macrotexture depth 2. Provides inconsistent results 3. Timing is critical
Texturing Method: Belt Finish	
1. May be done manually	1. Timing is critical
Texturing Method: Broom Finish	
1. Texture depth superior to burlap drag finish	1. May require an additional piece of equipment 2. Timing is critical
Texturing Method: Wire Tine Finish	
1. Higher skid resistance then other finish methods 2. Lower wear rate than other finishing methods 3. Excellent drainage with transverse tining	1. Transverse tining may require an additional piece of equipment 2. Tines may dislodge aggregate near the pavement surface 3. Overlaps of tine passes may cause weak pavement strips 4. Timing is critical

V. METHODS OF RESTORING MACROTEXTURE FOR EXISTING PAVEMENTS

Existing pavements provide the greatest potential for reducing wet-pavement accidents by improving pavement surface macrotexture. Only a limited mileage of new pavements are constructed in any given year, but the mileage of existing pavements that are candidates for macrotexture improvement is nearly unlimited--far greater than the funds available for such improvements could treat. This section presents an overview of the four most common methods of increasing the macrotexture of pavement surfaces. These are: open-graded asphalt overlays, pavement grooving, cold milling, and seal coats. This section is intended as a guide to provide a basic description of these methods for engineers who are unfamiliar with them. A cost-effectiveness analysis of two of these techniques--open-graded asphalt overlays and pavement grooving is found in Section VII of this report. A fifth method for restoring the macrotexture of existing pavements is the sprinkle treatment. However, recent experience with sprinkle treatments has been disappointing due to aggregate pickup. For this reason, sprinkle treatments are not included in this report.

A. Open-Graded Asphalt Overlays

The use of open-graded asphalt pavement surfaces for new pavements was discussed in Section IV.A. of this report. Most of the information on new open-graded asphalt surfaces discussed in that section is also applicable to open-graded asphalt overlays, and will not be repeated here. This section focuses on the aspects of open-graded asphalt surfaces that are unique to their use on existing pavements.

1. Specification: The typical specification for open-graded asphalt surfaces given in Appendix A and the mix design procedure given in Appendix B are applicable to overlays as well as new surfaces.

Proper preparation of the existing surface is vital to assure the success of an open-graded overlay. If the existing surface is rough, uneven or extremely cracked, a leveling course should be used under the open-graded surface course. Such leveling courses will provide a smooth ride and help extend the life of the surface course by preventing early failures.

2. Cost: The cost estimates discussed in Section IV for open-graded asphalt surface courses also apply to open-graded asphalt overlays. The average in-place cost of a 25.4 mm (1 in.) overlay is estimated at $1.20 per sq m ($1.00 per sq yard). In current state practices, the thickness of an open-graded asphalt overlay can range from 12.7 to 38.1 mm (1/2 to 1-1/2 in.). The Federal Highway Administration recommends a maximum thickness of 25.4 mm (1 in.) for an open-graded asphalt overlay. Naturally,

the overaly thickness has important cost implications. If overlays as thin as 12.7 mm (1/2 in.) perform adequately, this type of improvement will be more attractive economically. More evaluations of such overlays are needed. In the meantime, a suggested rule is that an open-graded asphalt overlay should be at least twice the thickness of the maximum aggregate size, e.g., at least 25.4 mm (1 in.) thick for a mix with a maximum aggregate size of 12.7 mm (1/2 in.).

3. Service life: The discussion of service life for open-graded surfaces in Section IV is also applicable to open-graded overlays. There are no reliable data to determine the relative service life of open-graded and dense-graded surface courses. However, there is currently no strong evidence to indicate that the service life of a properly constructed open-graded surface is shorter than that of a dense-graded surface, approximately 10 to 15 years.

4. Performance: Performance problems such as raveling and flushing mentioned for new open-graded asphalt surfaces can also occur with open-graded overlays. In addition, reflective cracking of the overlay is a potential problem that often occurs more quickly on open-graded overlays than on dense-graded overlays, particularly for surfaces placed over portland cement concrete. This problem can be alleviated with use of a binder or leveling course.

There has been no comprehensive evaluation of the accident reduction effectiveness of open-graded asphalt overlays. Studies from California and Virginia provide some indication of the effectiveness. A before-after study of 10 sections in California overlayed with open-graded asphalt concrete found that the wet-pavement accident rate was reduced by 70% in the after period.[39/] An evaluation of one section in Virginia resurfaced with an open-graded asphalt overlay found that 39% of all accidents before the overlay occurred on wet pavement, whereas only 17% of all accidents after the overlay occurred on wet pavement.[49/] However, neither study provides any quantitative measure of the improvement in macrotexture or skid number or any comparison with the accident reduction effectiveness of dense-graded overlays.

5. Advantages and disadvantages: The advantages and disadvantages enumerated in Table 2 apply also to open-graded asphalt overlays. The only additional disadvantage to be added for overlays of open-graded asphalt is the problem of reflective cracking described above.

B. Pavement Grooving

Pavement grooving is the process of making a pattern of parallel, shallow cuts of uniform depth, width, and shape in the surface of an existing pavement. The grooves are usually cut with a diamond saw. Portland cement concrete pavements are most frequently grooved, but grooving has been accomplished successfully on older bituminous pavements where the asphalt is well-cured. Grooving should not be confused with texturing of new portland cement concrete pavements, which is a finishing process that is accomplished while the concrete is in the palstic state. The International Grooving and Grinding Association makes the distinction that patterns spaced less than 12.7 mm (1/2 in.) center-to-center are considered to be "texturing" and patterns spaced more than 12.7 mm (1/2 in.) center-to-center are considered to be "grooving."[63/]

Most grooving in the United States uses center-to-center spacings between 12.7 mm (1/2 in.) and 38.1 mm (1-1/2 in.). Both longitudinal and transverse grooves have been used, but longitudinal grooves are more common. The objective of this treatment is to place grooves in the tire-pavement interface which provides a path for water to escape from under the tire. Thus, grooving acts like other forms of pavement macrotexture in reducing the potential for hydroplaing. However, despite its proven wet-pavement accident reduction effectiveness, longitudinal grooving does not normally increase the skid number of a pavement surface. Thus, this form of texturing does not influence skid number in the same manner as the random macrotexture of open-graded asphalt surfaces or the pattern macrotexture of finished portland cement concrete surfaces. There is some indication that transverse grooving may increase skid number.

Some foreign countries have employed pavement grooving effectively for a different purpose than used in the United States. Zipkes reports on a highway section in Switzerland that has been grooved transversely at center-to-center spacings of 254 to 1,016 mm (10 to 40 in.).[84/] This wide spacing does not necessarily keep a groove within the tire footprint at all times, as with more closely spaced grooves. The only objective of widely spaced grooving is to facilitate the flow of water off the pavement to the shoulder. The reduction in the depth of water on the pavement reduces the potential for hydroplaning by increasing the critical speed at which hydroplaning will occur.

1. Specification: There are some variations in the depth, width and spacing of grooves that have been used in the United States. Typical ranges are 2.4 to 6.4 mm (0.095 to 0.25 in.) for groove width, 3.2 to 0.64 mm (0.125 to 0.025 in.) for groove width, 3.2 to 0.64 mm (0.125 to 0.025 in.) for groove depth and 12.7 to 38.1 mm (0.5 to 1.5 in.) for center-to-center spacing. The most common specification for grooves is 2.4 mm (0.095 in.) width, 6.4 mm (0.25 in.) deep, and 19.1 mm (0.75 in.) center-to-center spacing. The grooves cut by a diamond saw have a rectangular cross-section.

The usual practice in most states is to groove the center 3 m (10 ft) portion of a 3.7 m (12 ft) lane and leave a 0.3 m (1 ft) strip ungrooved at the edge of each lane. Appendix B contains a sample specification for pavement grooving used by the Louisiana Department of Highways.

The depth of pavement grooves is extremely important to their proper functioning as a wet-pavement accident countermeasure. Most state highway departments have adopted a modified tire tread depth gauge to inspect the depth of grooves on construction projects. This relatively simple, but effective, procedure is illustrated by Pennsylvania Department of Transportation Test Method No. 629 also discussed in Appendix B.

2. <u>Cost</u>: The cost for pavement grooving is quite variable and depends on the construction contractor's familiarity with grooving equipment and the hardness of the aggregate in the pavement surface course. Several contractors report typical productivity rates for longitudinal grooving of 0.6 lane-km per hour (0.4 lane-miles per hour). However, extremely hard aggregates produce a noticeable decrease in productivity rates for grooving.

The best available cost estimate for longitudinal pavement grooving is $1.20 per sq m ($1.00 per sq yard). However, lower unit costs for grooving are reported in areas where grooving is used extensively. For example, a typical cost for pavement grooving in the Los Angeles area is $0.72 per sq m ($0.60 per sq yard), while a cost of $1.20 per sq m ($1.00 per sq yard) is more common elsewhere in the State of California. Transverse grooving of in-service pavements is more time-consuming and more expensive than longitudinal grooving. While longitudinal grooving can be accomplished by closing one lane of traffic at a time, transverse grooving requires at least two lanes to be closed for equipment moving. One equipment manufacturer reports that longitudinal grooving can be accomplished 50 times as fast as transverse grooving.

3. <u>Service life</u>: The service life of pavement grooves depends on the type of traffic to which they are exposed. High traffic volumes shorten the service life of pavement grooves, but this effect has not been adequately quantified. The presence of tire chains or studs on vehicles in the traffic stream has an important effect on grooving service life. California reports that grooves 3.2 mm (1/8 in.) deep on highways where tire chains and studs are not used have a service life of 8 to 10 years,[20] but Pennsylvania reports service life of 3 years or less where tire chains and studs are used.[9]

4. <u>Performance</u>: Dramatic reductions in wet-pavement accidents have resulted from pavement grooving. Two California studies completed in 1972 and 1975 have found reductions in wet-pavement accident rate of 73 and 70%, respectively.[39,77] The largest decreases reported were in

sideswipe, fixed object and rear-end accidents. However, the accident reduction effectiveness of grooving does not appear to be consistent. In the 1972 California study, 27 projects decreased in total accident rate, while 11 projects increased. The change in total accident rate with grooving for these 38 projects ranged from a 100% reduction to a 45% increase. A review of 77 grooving projects in 13 states reported by Rasmussen[63] showed an overall decrease of 75% in the number of wet-pavement accidents. The before and after periods in this evaluation range from 2 months to 5 years in length and the decreases in the number of wet-pavement accidents for individual projects ranges from 16 to 100%.

Some users have also observed that longitudinal grooving is effective in increasing the directional control of automobiles. Apparently, the automobile tires penetrate slightly into the grooves and form a mechanical interlock that helps to hold the vehicle in alignment with the roadway.[81] However, a persistent concern exists about handling difficulties of motorcycles and small cars on grooved pavements. Pavement grooves do produce a sensation of instability while riding a motorcycle, but a recent study sponsored by the California Department of Transportation in which seven motorcycles of different sizes were driven by two riders on grooved pavements, found no significant control problem.[77] Furthermore, the 1975 accident study in California found decreases in the number of motorcycle accidents after grooving on both wet and dry pavements, even though total motorcycle registrations and, presumably, motorcycle traffic on the study sections increased by 14.5% between the before and after study periods.[77] However, because the sensation of instability is unsettling to motorcyclists, even though it does not lead to loss of control, the use of warning signs at the beginning of grooved pavement sections is recommended.

5. Advantages and disadvantages: The advantages and disadvantages of grooved pavement as a wet-pavement accident countermeasure are summarized in Table 6.

C. Cold Milling

Cold milling is a technique for retexturing pavements that has come into widespread use recently. Major technological advances within the past 2 years have produced a new type of rotary milling machine that can grind or scarify pavement surfaces more efficiently and accurately than ever before. The milling machine used has a rotating drum with carbide steel teeth that can texture a pavement surface with a pattern of grooves that provide macrotexture for water drainage at the tire-pavement interface. Unlike grooving accomplished with a diamond saw, the grooves made by a milling machine are short and discontinuous. The depth of the grooves can be varied by controlling the forward speed of the milling machine and the rotating speed of the drum. The microtexture of the surface between the grooves is a function of the type of aggregate used in the pavement. Cold milling can be used to retexture both asphalt and portland cement concrete surfaces.

TABLE 6

PAVEMENT GROOVING

Advantages	Disadvantages
1. Wet-pavement accident experience may be substantially reduced.	1. Grooving cannot be used for bituminous concrete pavements unless the asphalt is well cured.
2. Longitudinal grooving can be accomplished quickly and only one lane of traffic at a time has to be closed.	2. The use of studded tires or tire chains reduces the service life of grooved pavement.
3. Traffic can use the pavement surface soon after grooving.	3. Motorcyclists and drivers of small cars may have a sensation of instability.

Several state highway departments have recently experimented with cold milling to improve skid resistance of pavement surfaces. The Texas Department of State Highways and Public Transportation has recently milled several highway sections using machines from two manufacturers. Milling was used successfully on one recent job on 22.5 km (14 mile) portion of the North Central Expressway in Dallas. One pass of the milling machine removed a 38.1 mm (1-1/2 in.) asphalt overlay and retextured the underlying portland cement concrete. The Iowa Department of Transortation has retextured two portland cement concrete pavement projects totaling 0.8 km (1/2 mile) in length. Cold milling has also been used on highways in Michigan, Ohio, Pennsylvania, and Wyoming.

1. Specifications: The advances in cold milling technology have been so recent that, to our knowledge, no states have yet adopted formal, published specifications for the use of milling to increase skid resistance. The Texas Department of State Highways and Public Transportation reports that milled portland cement concrete surfaces have average texture depths of 2 to 2.4 mm (80 to 95 milli-in.), as determined with the sand patch method. This is substantially above Texas' specified texture depth for new portland cement concrete surfaces, but it should be recognized that comparisons of sand patch test results between milled surfaces and finished new surfaces may not be appropriate.

2. Cost: The only available information on the cost of cold milling comes from the recent jobs in Texas. The cost of retexturing is \$0.48 to \$0.54 per sq m (\$0.40 to \$0.45 per sq yard) and the cost for both removing an asphalt overlay and retexturing is \$0.84 to \$1.32 per sq m (\$0.70 to \$1.10 per sq yard). Both of these cost estimates include hauling the removed paving material from the site. Furthermore, the paving material removed from the surface may be recycled into new pavements resulting in additional cost savings. Productivity rates have approached 16,700 sq m (20,000 sq yards) per day.

3. Service life: No pavement surfaces retextured with the new milling machines have been in service for longer than about 1 year. Therefore, there are no data available to evaluate the service life of milled pavement surfaces.

4. Performance: There are no reports of performance problems with milled pavement surface during their initial period of service. The Iowa Department of Transportation has reported an increase in skid number at 64.4 km/hr (40 mph) from 25 to 57 on one portland cement concrete pavement and from 39 to 60 on the other.[75] On the second project, skid numbers were measured at several speeds. The skid number-speed gradient of the surface was not appreciably changed by milling. (It remained at a relatively low level of 0.75.) There are no other skid resistance data available for evaluation and the rate of skid resistance degradation after milling is unknown.

Milling has other benefits besides increased skid resistance. Profiling of the pavement surface by removal of material reduces pavement roughness and produces a smoother ride. Bridge clearances on controlled-access highways are increased rather than reduced as they would be by an overlay. However, there are two potential drawbacks to milled surfaces. The indentations in the pavement are not continuous, as with sawed grooves, so ponding of water might be a problem. Also, the high texture may increase tire noise levels. Currently, there are no reliable data on these potential problems, but they merit further investigation.

5. Advantages and disadvantages: Table 7 summarizes the advantages and disadvantages of cold milling to restore texture of existing pavement surfaces.

D. Seal Coats

Two kinds of seal coats that are frequently used to improve skid resistance are described in this section: chip-seal coats and slurry seals. Both of these improvements fall in the general category of asphalt surface treatments, a term that describes a broad range of asphalt and asphalt-aggregate applications usually less than 25.4 mm (1 in.) thick. These types of seal coats are generally used only under very low traffic volume conditions, but are described here for the sake of completeness.

1. Specification: A chip-seal coat consists of a coat of asphalt sprayed on an existing pavement followed by a layer of cover aggregate. Beaton[7/] states that an advantage of this type of seal coat is that it provides the pavement designer a great deal of flexibility, since his choice of a maximum aggregate size allows development of gradations that will produce a variety of macrotexture depths. It is critical, however, that good polish-resistant aggregate be used to maximize the life of the chip-seal coat. Most state highway department specifications require the use of a crushed aggregate. The usual thickness of a chip-seal coat is approximately the same as the maximum size of aggregate used. The design method used by most agencies to determine the required asphalt content for chip-seal coats is presented by McLeod.[48/] Gallaway and Epps[24/] provide an excellent discussion of material selection considerations for chip-seal coats including the selection of aggregates and gradation requirements.

A slurry seal, as specified by most agencies, is a form of sand asphalt mixture. It consists of a mixture of asphalt emulsion, fine aggregate, mineral filler, and water. The first slurry seals used an anionic emulsion that required 12 to 36 hr to cure before it could be opened to traffic, but the newer slurry seals constructed with cationic asphalt emulsions require only 3 to 5 hr, with proper weather conditions, before opening to traffic.[50/] Typically, slurry seals are placed 6.4 to 9.5 mm (1/4 to

TABLE 7

RETEXTURING PAVEMENTS BY COLD MILLING

Advantages	Disadvantages
1. Skid resistance is increased initially.	1. Service life is unknown.
2. Surface roughness is reduced.	2. Ponding of water is possible if voids are not interconnected.
3. Traffic can use surface almost immediately after milling.	3. Noise levels may increase.
4. Bridge clearances are increased.	

3/8 in.) thick. Slurry seals are frequently used to increase skid resistance on low volume facilities. However, because this type of seal consists primarily of fine aggregate, the skid resistance improvement comes from increased microtexture.

2. Cost: A typical current construction cost for chip-seal coats, based on recent experience in Texas and California is $0.42 to $0.60 per sq m ($0.35 to $0.50 per sq yard). A typical cost for a 6.4 mm (1/4 in.) thick slurry seal is $0.36 to $0.54 per sq m ($0.30 to $0.45 per sq yard), based on 1975 prices in Pennsylvania and New York. This estimate agrees well with an estimate of $3,700 per km ($6,000 per mile) for two lanes ($0.51 per sq m ($0.43 per sq yard)) for slurry seals reported by Runkle and Mahone.[70/] Costs for a double application of slurry seal (9.5 mm (3/8 in. thick)) in Pennsylvania ranged as high as $1.61 per sq m ($1.35 per sq yard) in 1975. The prices quoted in 1975 would undoubtedly be higher today.

3. Service life: Pennsylvania has attained up to 4 years service from a 6.4 mm (1/4 in.) thick slurry seal when properly constructed at sites where the traffic volume is less than 2,000 vehicles per day. Double applications of slurry seal (9.5 mm (3/8 in. thick)) are recommended to preserve the service life when ADT is greater than 2,500.[50/] The service life of chip-seal coats is assumed to be similar to the service life of slurry seals.

4. Performance: Chip-seal coats have been found to perform well when properly designed and constructed. Good performance requires proper preparation of the surface before placing the chip-seal coat. Binder or leveling courses should be placed on the existing surface, if needed to improve structural strength or rideability. It is important that the existing surface or any added binder course be adequately compacted before a chip-seal surface is placed so that the aggregate will not penetrate the underlying surface and lead to flushing. The asphalt content should be determined carefully, because flushing will occur if the mixture is too rich while raveling will occur if it is too lean. The application rate of cover aggregate should be minimized, consistent with the desired thickness of the chip-seal coat, to avoid problems such as flying chips, and crushing of aggregate. It is extremely important that clean aggregate be used in seal coats, because asphalt will not adhere to dirty aggregate.

The expected skid-resistance characteristics of chip-seal surfaces are not well defined and vary with the microtexture and macrotexture of the surface in a manner similar to other pavements. The initial skid number of a chip-seal surface could be predicted using British Portable and sand patch test results from sample surfaces constructed for a particular choice of aggregate and gradation. Zube and Skog[85/] suggest an alternative procedure for estimating the initial skid number using the same California oil equivalent test that was described in connection with the design of open-graded asphalt surfaces.

The performance of slurry seals is similar to chip-seal coats. Raveling and flying chips do not present a problem, because no coarse aggregate is used. However, preparation of the underlying surface is extremely important. Excessively cracked or deteriorated bases and bases that are not structurally sound will cause the slurry seal to perform poorly.[50] Also, excessive asphalt contents should be avoided to prevent bleeding.[7]

Both chip-seal coats and slurry seals are intended for use under low traffic volume conditions, but there is no agreement on an appropriate upper limit of traffic volume for their use. For example, the Iowa Department of Transportation recommends the use of chip-seal coats and slurry seals only for highways with traffic volumes less than 1,500 vehicles per day.[61] On the other hand, Texas reports that chip-seals have performed well under traffic volumes up to 20,000 vehicles per day, if high-quality aggregate is used.

5. Summary: No formal cost-effectiveness analysis of chip-seal coats or slurry seals is made in this report because there are no reliable data on their accident reduction effectiveness and service lifes. However, it is clear that chip-seal coats provide a level of macrotexture that contributes both to increasing skid number and reducing hydroplaning potential. Slurry seals increase skid number due to an increase in microtexture alone. Both treatments are suitable only for lower traffic volume sites where short-term improvements are desired. With these precautions, local experience is the best guide for selecting situations where seal coats are appropriate.

VI. COST-EFFECTIVENESS OF PAVEMENT MACROTEXTURE IMPROVEMENTS

An example of a cost-effectiveness analysis for alternative pavement macrotexture improvement techniques is presented in this section. The purpose of this analysis is to compare the costs and benefits of the available techniques in typical situations where they might be applied. The basic analysis approach is provided by the benefit-cost model described in Volumes II and III of this report. However, because the computerized model is intended for application at specific sites, the model logic has been adapted and generalized for application to representative sites. The cost-effectiveness analysis considers the application of each pavement macrotexture improvement at sites of two area types (urban/rural) and three highway types (two-lane/multilane, uncontrolled access/multilane, controlled access). Two typical ADT levels are analyzed for each combination of area type and highway type.

The cost-effectiveness analysis makes use of the best available estimates of the costs and benefits for the following wet-pavement accident countermeasures: open-graded asphalt surfaces for new pavements, open-graded asphalt overlays, and grooving of existing pavements. For illustrative purposes, dense-graded overlays with both high and average microtexture aggregate are also included in the analysis. The analysis incorporates estimates of countermeasure costs, initial skid resistance improvement and skid number degradation with time that are assumed to represent national average conditions. Such estimates are not well established and are highly dependent on local materials and construction practices. Therefore, the analysis results should not be interpreted as being representative of all states and regions of the United States. The analysis results indicate generally which pavement macrotexture improvement techniques have the greatest potential for cost-effective reduction of wet-pavement accidents, but should not be applied indiscriminately. The analysis does, however, provide a framework within which individual values can be modified by highway agencies to obtain results that are consistent with local experience for a particular state or region.

Several pavement macrotexture improvement techniques discussed earlier in this report are not included in the cost-effectiveness analysis. For example, the accident reduction effectiveness and service life of seal coats and milled surfaces are not well enough defined to make intelligent estimates for a formal analysis. Also, the texturing of new portland cement concrete pavements by wire tining produces an improvement in pavement macrotexture at little or no increased cost. Any such "no-cost" improvement can be adopted as a standard without reference to formal cost-effectiveness considerations.

The remainder of this section presents the general approach to the cost-effectiveness analysis, the estimates used for construction cost, accident reduction effectiveness, and benefit-cost comparisons of pavement macrotexture improvements. The final portion of this section provides an interpretation of the analysis results.

A. General Approach

The general approach to the cost-effectiveness analysis is to consider the application of each pavement macrotexture improvement at sites representative of each of the 12 cells in the analysis reported in Volume I. These cells represent two area types (urban/rural), three highway types (two-lane/multilane uncontrolled access/multilane, controlled access) and two ADT categories (under 10,000/over 10,000). The analysis employs procedures to estimate both the construction cost and the savings of accident costs on an equivalent uniform annual cost basis. The benefit-cost ratio is then calculated as the ratio of accident costs savings to construction costs. In order to accomplish this, the following assumptions are made:

- Each ADT category is represented by a typical ADT, which does not change with time.
- The pavement at the site is wet 20% of the time.
- The countermeasures analyzed reduce the wet-pavement accident rate, but have no effect on the dry-pavement accident rate.
- The skid number for open-graded and dense-graded asphalt surfaces are predicted from assumed values of microtexture and macrotexture by the Penn State relationship.
- The change of skid number with cumulative traffic passages follows the logarithmic relationship used in the computerized benefit-cost model. The coefficients of this relationship are estimated from the literature.
- The effect of changes in skid number on wet-pavement accident experience can be predicted using the relationships developed in Volume I of this report.
- The effect of pavement grooving on accident experience is a 70% reduction in wet-pavement accident rate.
- The cost of fatal, injury, and property-damage-only accidents used are those developed by NHTSA. The average cost per accident reflects the accident severity distribution for each area type and highway type.
- The service life of new pavement surfaces and overlays is 10 years. The service life of a grooved pavement is 3 years when tire chains and studded tires are present and 8 years when they are not.
- The interest rate (minimum attractive rate of return) is 7%.

These assumptions simplify the computerized benefit-cost model so that useful results can be obtained with simple manual calculations. This simplified approach makes it easy for the interested reader to revise the analysis results by substituting data applicable to his local area. The results of the benefit-cost analysis are quite sensitive to three factors: percentage of wet-pavement time, dry-pavement accident rate and ADT. Typical values of these three factors have been assumed in the following analyses. Section VI.D. explains how the analysis results can be adjusted for sites that deviate from the assumed values.

B. Construction Cost Estimates

Construction costs for pavement macrotexture improvements are highly variable and are dependent on local materials and construction practices. The construction cost estimates used for the cost-effectiveness analysis are based on averages of recent bid prices reported by several state highway departments or obtained from expert opinion of state highway department materials engineers and estimates by the authors. The following construction cost estimates were used:

Improvement	Cost Per Sq M (Sq Yd) ($)
Open-graded asphalt surface course	1.20 (1.00)
Dense-graded asphalt overlay with high microtexture aggregate	1.20 (1.00)
Dense-graded asphalt overlay with average microtexture aggregate	0.90 (0.75)
Pavement grooving	1.20 (1.00)

The above costs are used directly when no prior decision to resurface the roadway is made. However, when it has already been decided that a roadway required resurfacing to improve strength or rideability, an additional improvement in pavement macrotexture may also be considered. In this case, only the incremental cost of the pavement macrotexture improvement, over and above the resurfacing cost, is considered. A highway section where a dense-graded asphalt overlay is planned to increase the structural strength or rideability of the pavement provides an example of this situation. The estimated cost of a dense-graded overlay is $0.90/sq m ($0.75/sq yd). Suppose that an alternative design using an open-graded asphalt overlay is also considered, the total cost of the open-graded asphalt overlay is $1.20/sq m ($1.00/sq yd), but the incremental cost is only $0.30/sq m ($0.25/sq yd) greater than the dense-graded overlay. The use of an open-graded asphalt surface for a new pavement presents a unique case. Open-graded surface courses have no inherent structural strength, so they cannot be included in the design pavement thickness. The approach commonly employed is to use an increased thickness (typically 38 mm (1-1/2 in.)) of base course material is less expensive than surface course asphalt. The incremental cost of an open-graded surface course for a new pavement is estimated as \$0.57/sq m (\$0.475/sq yd). A summary of all incremental costs used in the cost-effectiveness analysis follows:

Improvement	Base Condition	Incremental Cost Per Sq M (Cost Per Sq Yd)
Open-graded asphalt surface for new pavement	Dense-graded asphalt surface (with average microtexture aggregate)	0.568 (0.475)
Open-graded asphalt overlay	Dense-graded asphalt overlay (with average microtexture aggregate)	0.30 (0.25)
Dense-graded asphalt overlay (with high microtexture aggregate)	Dense-graded asphalt overlay (with average microtexture aggregate)	0.30 (0.25)

The unit cost per square yard is used to determine the improvement cost per mile of roadway for pavement surface improvements. For example, pavement grooving of a highway with two, 3.7 m (12 ft) lanes costs \$1.20/sq m (\$1.00/sq yd) or:

$$\frac{\$1.20}{\text{sq m}} \times \frac{1{,}000 \text{ m}}{\text{km}} \times 7.3 \text{ m} = \$8{,}760/\text{km} ,$$

$$\text{or} \quad \frac{\$1.00}{\text{sq yd}} \times \frac{1 \text{ sq yd}}{9 \text{ sq ft}} \times \frac{5{,}280 \text{ ft}}{1 \text{ mile}} \times 24 \text{ ft} = \$14{,}080/\text{mile}$$

For cost-estimation purposes, a 7.3 m (24 ft) width is used for all two-lane highways and a 14.6 m (48 ft) width for all multilane highways. However, because open-graded asphalt surfaces should be carried onto the shoulder to allow a continuous drainage path and avoid ponding water on the pavement, the cost estimates for such surfaces include an allowance for 1.8 m (6 ft) shoulders on both sides of the roadway.

The equivalent uniform annual cost of each macrotexture improvement is computed by multiplying the total construction cost by capital recovery factor.

$$CRF = \frac{i(1 + i)^n}{(1 + i)^n - 1}$$

where CRF = Capital Recovery Factor,

i = Decimal interest rate, and

n = Service life (years).

A service life of 10 years is used for resurfacing improvements, 8 years for pavement grooving on sections not exposed to tire chains or studs and 3 years for pavement grooving where tire chains or studs are used. The interest rate for all analyses is 0.07 or 7% per annum.

C. Accident Reduction Effectiveness

The accident reduction effectiveness of each pavement macrotexture improvement by resurfacing (i.e., not including pavement grooving) is determined in six steps, as follows: (1) estimating the microtexture and macrotexture of the new pavement surface shortly after installation; (2) determining the skid number from the microtexture and macrotexture estimates using the Penn State relationship (discussed in Section II); (3) estimating the rate of degradation of skid number with traffic passages; (4) determining the skid number during each year of the improvement service life; (5) calculating the accident rate reduction associated with that level of skid number; and finally (6) converting these accident rate reductions to a uniform annual basis. This six step procedure will be illustrated below by reference to an example.

Table 8 shows the estimates of initial microtexture and macrotexture for various pavement surface types. The estimates were developed from literature, including the work of Nixon et al.[59/] and the authors judgment. The initial skid number at 64.4 kph (40 mph) for each pavement surface is calculated from the microtexture and macrotexture values using the Penn State relationship, developed by Leu and Henry:[47/]

$$SN_{64.4} = (-31.0+1.38BPN)e^{-0.230(64.4)(MD)^{-0.47}}$$

where $SN_{64.4}$ = Skid number at 64.4 kph (40 mph),

BPN = British Portable Number, and

MD = Mean texture depth (mm), as determined by sand patch method (1 in. = 25.4 mm).

TABLE 8

ESTIMATES OF INITIAL MICROTEXTURE, MACROTEXTURE AND SKID NUMBER USED IN THE COST-EFFECTIVENESS ANALYSIS

Pavement Surface Type	Microtexture (British Portable Number)	Macrotexture (Mean Texture Depth)a/ (mm (milli-in.))	Estimated Skid Number at 64.4 kph (40 mph)b/
Open-graded asphalt surface	80	1.8 (70)	63.6
Dense-graded asphalt surface with high microtexture aggregate	80	0.6 (25)	55.3
Dense-graded asphalt surface with average microtexture aggregate	65	0.6 (25)	40.9
Worn AC or PCC surface (used as estimate for existing pavement)	55	0.6 (25)	31.3

a/ Determined by sand patch method.
b/ Determined from Penn State relationship.

These estimated skid numbers are also shown in Table 8. The analysis assumes that the existing pavement surface is worn AC or PCC, estimated in Table 8 to have a skid number of 31.3. This existing surface is to be replaced with one of the alternative improvements shown in Table 8.

The benefit-cost model described in Volumes II and III uses the following form for the change of skid number as a function of cumulative traffic passages:

$$SN = SN_i + C_s \ln(T)$$

where SN = Pavement skid number at any time,

SN_i = Initial skid number,

C_s = Coefficient representing rate of change of skid number with traffic wear, and

T = Cumulative traffic passages per lane/10^5,

and subject to the constraints:

$T \geq 1$,

$SN \leq SN_f$ when $C_s > 0$, and

$SN \geq SN_f$ when $C_s < 0$,

where SN_f = Final or limiting value of skid number.

The initial skid numbers for alternative surface improvements in Table 9 are used as values for SN_i. The typical skid number for a worn AC or PCC surface, 31.3, is used as the limiting value for skid number, SN_f. The cost-effectiveness analysis is sensitive to C_s, the rate of decrease of skid number with traffic wear. Rizenburgs et al.,[65] determined a value of -8.3 for C_s for typical asphalt surfaces in Kentucky. However, these asphalt surfaces contained predominately limestone aggregate, which is highly polish-susceptible and not suitable for open-graded surfaces or other surfaces that require high-microtexture aggregates. Rizenburg et al., found a smaller value of C_s, -4.7, for Kentucky rock asphalt which is a porous material with surface voids similar to an open-graded surface. This latter value for the polishing rate has been used in this analysis.

The skid number of the improved pavement surface at the midpoint of each year is determined by substituting the initial skid number, SN_i, rate of change of skid number, C_s, and the cumulative traffic passages per lane into the expression given above. It is assumed that the pavement surface that existed before the improvement has reached its limiting value, 31.3, and does not change with time. The difference between the skid number of the improved pavement at the midpoint of a given year and the skid number of the existing pavement is used to determine the accident rate reduction for that year.

The reduction in wet-pavement accident rate for a given change in skid number can be predicted from the Phase I analysis results presented in Volume I of this report. The following wet-pavement accident rate--skid number relationship was reported in Volume I:

$$AR = AR_o + a_1\ SN_{40}$$

where AR = Wet-pavement accident rate (accidents/MVK),

AR_o = Zero-intercept of wet-pavement accident rate (a function of highway type, area type, and ADT),

a_1 = Slope coefficient (accidents/MVK/SN), and

$SN_{64.4}$ = Skid number at 64.4 km/hr (40 mph).

From the last equation it follows that the effect on wet-pavement accident rate of an improvement in skid number is:

$$\Delta AR = AR_a - AR_b = a_1\ (SN_a - SN_b)$$

where ΔAR = Decrease in wet-pavement accident rate (accidents/MVK),

AR_a = Wet-pavement accident rate after improvement (accidents/MVK),

AR_b = Wet-pavement accident rate before improvement (accidents/MVK),

SN_a = Skid number at 64.4 km/hr (40 mph) after improvement, and

SN_b = Skid number at 64.4 km/hr (40 mph) before improvement.

This relationship is used to determine the accident rate reduction from the change in skid number. The slope coefficient, a_1, was found to be -0.0286 accidents/MVK/SN (-0.046 accidents/MVM/SN) for all highway types, area types and ADT levels in the Phase I analysis. This value of the slope coefficient is used in the cost-effectiveness analysis. However, the Phase I analysis also found that the slope coefficient is sensitive to the level of dry-pavement accident rate. This sensitivity has important implications for the cost-effectiveness of pavement macrotexture improvements, but will not be introduced until a later point in the analysis.

The expected wet-pavement accident rate reduction during each year of the improvement service life is placed on an equivalent uniform annual cost basis for comparison with the improvement construction costs. The accident rate reduction for each year could be converted to number of accidents reduced or accident cost reduction but, at this point, it is most useful to retain them as accident rates. The procedure used is to obtain a present worth of wet-pavement accident rate reduction by multiplying accident rate reduction during each year by the appropriate present worth factor. The total present worth of wet-pavement accident rate reduction is the sum of the present worth of the wet-pavement accident rate reductions for each individual year. This sum multiplied by the appropriate capital recovery factor to yield an equivalent uniform wet-pavement accident rate reduction.*

Table 9 illustrates the calculation of the equivalent uniform annual accident rate reduction. The site chosen for this example is a two-lane highway section with average daily traffic of 2,500 vehicles that is resurfaced with an open-graded asphalt friction course. Columns 1 through 3 illustrate the prediction of skid number values for each year of the analysis period. Column 2 shows the cumulative number of vehicle passages per lane at the midpoint of each year. The corresponding values of skid number, decreasing from an initial value of 63.6 to a value of 45.9 in the 10th year, are shown in Column 3. Column 4 contains the difference between each yearly value of skid number and the expected value if the pavement were not resurfaced. The wet-pavement accident rate reduction in Column 5 is determined from the change in skid number in Column 4. The present worth factors for each year are given in Column 6. The present worth of accident rate reduction for each year, shown in Column 7, is the product of the entries in Columns 5 and 6. The total present worth of wet-pavement accident rate reduction is 3.94 accidents/MVK (6.34 accidents/MVM), the sum of all entries in Column 7. This value is multiplied by the capital recovery factor ($i = 7\%$, $n = 10$ years, CRF = 0.14238) to obtain the equivalent uniform annual wet-pavement accident rate reduction for an open-graded asphalt surface--0.561 accidents/MVK (0.903 accidents/MVM).

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* The equivalent uniform annual accident cost reduction could have been computed here instead of the equivalent uniform wet-pavement accident rate reduction. However, this would not have been advantageous at this point because of the dependence of accident costs on area type and highway type. Instead, the assignment of costs to accidents is made just prior to the calculation of the benefit-cost ratios for different pavement macrotexture improvements. The procedure also makes it convenient for a highway engineer to use accident cost figures other than those assumed with a minimum amount of recomputation.

TABLE 9

EXAMPLE OF DETERMINING EQUIVALENT UNIFORM ANNUAL REDUCTION IN ACCIDENT RATE
(Open-Graded Asphalt Surface, Two-Lane Highway, ADT = 2,500)

(1) Time (n) (years)	(2) Cumulative Traffic Passages Per Lane/ 10^5	(3) Skid Number at 64.4 kph (40 mph)	(4) Increase in Skid Number From Present Values	(5) Wet-Pavement Accident Rate Reduction (ΔAR_n) Per MVK (per MVM)	(6) Present Worth Factor ($PW_{i,n}$)	(7) Present Worth of Accident Rate Reduction Per MVK (per MVM)
Present	-	31.3	-	-	-	-
Initial	0	63.6	-	-	-	-
1	2.28	59.7	28.4	0.81 (1.31)	0.9346	0.76 (1.22)
2	6.84	54.6	23.3	0.66 (1.07)	0.8734	0.58 (0.93)
3	11.41	52.2	20.9	0.60 (0.96)	0.8163	0.48 (0.78)
4	15.97	50.6	19.3	0.55 (0.89)	0.7629	0.42 (0.68)
5	20.53	49.4	18.1	0.52 (0.83)	0.7130	0.37 (0.59)
6	25.09	48.5	17.3	0.49 (0.79)	0.6663	0.33 (0.53)
7	29.65	47.7	16.4	0.47 (0.75)	0.6227	0.29 (0.47)
8	34.22	47.0	15.7	0.45 (0.72)	0.5820	0.26 (0.42)
9	38.78	46.4	15.1	0.43 (0.69)	0.5439	0.24 (0.38)
10	43.34	45.9	14.6	0.42 (0.67)	0.5083	0.21 (0.34)
					Total	3.94 (6.34)

Present worth factor $= PW_{sn} = \frac{1}{(1+i)^n}$

Total present worth of accident rate reduction (PWARR) $= \sum_{n=1}^{10} (\Delta AR_n) \left(\frac{1}{(1+i)^n}\right)$

Equivalent uniform annual accident rate

reduction $= (\text{PWARR}) \left(\frac{i(1+i)^n}{(1+i)^{n-1}}\right)$

$= 3.94\ (0.14238) = 0.561$ accidents/MVK

$= (6.34)\ (0.14238) = 0.903$ accidents/MVM

The equivalent uniform annual accident reduction for other countermeasures, except pavement grooving, is determined in an analogous manner.

The effect of pavement grooving on wet-pavement accident rate is handled by a different approach, because pavement grooving is known to reduce wet-pavement accident experience with no improvement in skid number. Recent evaluations in California have found that pavement grooving results in a 70% reduction in wet-pavement accident rate.[77] Therefore, the average wet-pavement accident rate for each combination of area type, highway type, and ADT level were taken from the Phase I analysis results and a 70% reduction in these rates was assumed during each year of the service life of the grooving. The presence of tire chains or studs in the traffic stream was accounted for by a reduction in the service life of the improvement.

D. Benefit-Cost Comparisons

The benefit-cost ratio for investigating pavement macrotexture improvements is formed as:

$$B/C = \frac{ACR}{ACC}$$

where ACR = Equivalent uniform annual accident cost reduction (dollars), and

ACC = Equivalent uniform annual construction cost (dollars).

The method of determining construction costs has already been discussed. The annual accident cost reduction is calculated as the product of the annual number of accidents reduced and the appropriate cost per accident.

The annual number of wet-pavement accidents reduced is determined from the accident rate reduction in the following manner:

$$AAR = \frac{(ARR)(ADT)(FWET)(365)(L)}{1{,}000{,}000}$$

where AAR = Equivalent uniform annual number of wet-pavement accidents reduced

ARR = Equivalent uniform annual accident rate reduction (accidents/MVK),

ADT = Average daily traffic (vehicles/day),

FEWT = Fraction of time with wet-pavement = 0.20,

365 = Number of days per year, and

L = Length of section = 1.6 km = 1 mile.

The costs per accident used in the analyses are taken from the computerized benefit-cost model logic and are shown in Column 9 of the benefit-cost tabulations. These costs are based on the NHTSA accident costs ($200,700 per fatality, $7,300 injury, and $300 per property-damage-only accident involvement) and a typical distribution of accident severities for each combination of highway type and area types. The benefit-cost ratios computed by this approach are referred to as unadjusted benefit-cost ratios, because three adjustment factors discussed below have not yet been applied.

The benefit-cost ratios developed for macrotexture improvements where no prior decision to resurface has been made are found in Tables 10 through 14. These tables illustrate:

- Open-graded asphalt overlay (Table 10).

- Dense-graded asphalt overlay with high microtexture aggregate (Table 11).

- Dense-graded asphalt overlay with average microtexture aggregate (Table 12).

- Pavement grooving (Tables 13 and 14).

The incremental benefit-cost ratios for pavement macrotexture improvements when a previous decision has been made to build a new pavement or resurface an existing pavement are found in Tables 15 through 17. The tables illustrate:

- Open-graded asphalt surface for new pavement (Table 15).

- Open-graded asphalt overlay (Table 16).

- Dense-graded asphalt overlay with high microtexture aggregate (Table 17).

TABLE 10

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Open-Graded Asphalt Overlay

Prior decision to resurface? No Service life: 10 Years
Base condition Existing Pavement

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	21,120	3,007	0.564	0.903	0.165	9,348	1,540	0.51
		Over 10,000	15,000	21,120	3,007	0.323	0.517	0.566	9,348	5,292	1.76
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	42,240	6,014	0.51	0.816	0.447	7,862	3,512	0.58
		Over 10,000	20,000	42,240	6,014	0.378	0.605	0.883	7,862	6,945	1.15
Rural	Multilane Controlled Access	Under 10,000	7,500	42,240	6,014	0.51	0.816	0.447	9,320	4,164	0.69
		Over 10,000	20,000	42,240	6,014	0.378	0.605	0.883	9,320	8,232	1.37
Urban	Two-Lane	Under 10,000	7,500	21,120	3,007	0.416	0.666	0.365	5,152	1,879	0.62
		Over 10,000	15,000	21,120	3,007	0.323	0.517	0.566	5,152	2,917	0.97
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	42,240	6,014	0.51	0.816	0.447	4,897	2,188	0.36
		Over 10,000	20,000	42,240	6,014	0.378	0.605	0.883	4,897	4,326	0.72
Urban	Multilane Controlled Access	Under 10,000	7,500	42,240	6,014	0.51	0.816	0.447	5,672	2,534	0.42
		Over 10,000	20,000	42,240	6,014	0.378	0.605	0.883	5,672	5,010	0.83

Adjustment Factors:
Wet-pavement exposure time
Traction demand
Traffic volume

TABLE 11

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Dense-Graded Asphalt Overlay (High Microtexture Aggregate)
Prior decision to resurface? No Service life: 10 Years
Base condition Existing Condition

(1) Area Type	(2) Highwsy Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	(7) acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	14,080	2,005	0.326	0.523	0.095	9,348	892	0.44
		Over 10,000	15,000	14,080	2,005	0.095	0.152	0.166	9,348	1,556	0.77
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	4,010	0.270	0.433	0.237	7,862	1,864	0.46
		Over 10,000	20,000	28,160	4,010	0.140	0.224	0.327	7,862	2,571	0.64
Rural	Multilane Controlled Access	Under 10,000	7,500	28,160	4,010	0.270	0.433	0.237	9,320	2,209	0.55
		Over 10,000	20,000	28,160	4,010	0.140	0.224	0.327	9,320	3,048	0.76
Urban	Two-Lane	Under 10,000	7,500	14,080	2,005	0.176	0.282	0.154	5,152	795	0.40
		Over 10,000	15,000	14,080	2,005	0.095	0.152	0.166	5,152	857	0.43
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	4,010	0.270	0.433	0.237	4,897	1,161	0.29
		Over 10,000	20,000	28,160	4,010	0.140	0.224	0.327	4,897	1,602	0.40
Urban	Multilane Controlled Access	Under 10,000	7,500	28,160	4,010	0.270	0.433	0.237	5,672	1,345	0.34
		Over 10,000	20,000	28,160	4,010	0.140	0.224	0.327	5,672	1,855	0.46

Adjustment Factors:
Wet-pavement exposure time
Traction demand
Traffic volume

TABLE 12

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermessure: Dense-Graded Asphalt Overlay (Average Microtexture Aggregate)
Prior decision to resurface? No Service life: 10 Years
Bsse condition Existing Condition

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	(7) acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	10,560	1,504	0.023	0.038	0.007	9,348	65	0.04
		Over 10,000	15,000	10,560	1,504	0	0	0	9,348	0	0
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	21,120	3,007	0.014	0.023	0.013	7,862	102	0.03
		Over 10,000	20,000	21,120	3,007	0	0	0	7,862	0	0
Rural	Multilane Controlled Access	Under 10,000	7,500	21,120	3,007	0.014	0.023	0.013	9,320	121	0.04
		Over 10,000	20,000	21,120	3,007	0	0	0	9,320	0	0
Urban	Two-Lsne	Under 10,000	7,500	10,560	1,504	0.002	0.003	0.002	5,152	10	0.01
		Over 10,000	15,000	10,560	1,504	0	0	0	5,152	0	0
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	21,120	3,007	0.014	0.023	0.013	4,897	64	0.02
		Over 10,000	20,000	21,120	3,007	0	0	0	4,897	0	0
Urban	Multilsne Controlled Access	Under 10,000	7,500	21,120	3,007	0.014	0.023	0.013	5,672	74	0.02
		Over 10,000	20,000	21,120	3,007	0	0	0	5,672	0	0

Adjustment Factors:
Wet-psvement exposure time
Traction demand
Trsffic volume

TABLE 13

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Pavement Grooving (No Tire Chains or Studded Tires)

Prior decision to resurface? No Service life: 8 Years
Base condition Existing Pavement

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	14,080	2,358	1.28	2.05	0.374	9,348	3,497	1.48
		Over 10,000	15,000	14,080	2,358	1.79	2.86	3.13	9,348	29,275	12.42
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	4,716	0.80	1.28	0.701	7,862	5,580	1.18
		Over 10,000	20,000	28,160	4,716	1.75	2.80	4.09	7,862	32,140	6.82
Rural	Multilane Controlled Access	Under 10,000	7,500	28,160	4,716	0.31	0.49	0.268	9,320	2,500	0.53
		Over 10,000	20,000	28,160	4,716		0.45	0.657	9,320	6,123	1.30
Urban	Two-Lane	Under 10,000	7,500	14,080	2,358	0.28	2.42	1.33	5,152	6,826	2.89
		Over 10,000	15,000	14,080	2,358	2.06	3.30	3.61	5,152	18,617	7.90
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	4,716	3.07	4.91	2.69	4,897	13,164	2.79
		Over 10,000	20,000	28,160	4,716	4.19	6.70	9.78	4,897	47,902	10.16
Urban	Multilane Controlled Access	Under 10,000	7,500	28,160	4,716	0.71	1.14	0.624	5,672	3,540	0.75
		Over 10,000	20,000	28,160	4,716	0.95	1.56	2.28	5,672	12,919	2.74

Adjustment Factors:
Wet-pavement exposure time
Traffic volume

TABLE 14

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Pavement Grooving (Tire Chains and/or Studded Tires

Prior decision to resurface? No   Service life: 3 Years
Base condition Existing Pavement

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	14,080	5,365	1.28	2.05	0.374	9,348	3,497	0.65
		Over 10,000	15,000	14,080	5,365	1.79	2.86	3.13	9,348	29,275	5.46
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	10,730	0.80	1.28	0.701	7,862	5,580	0.52
		Over 10,000	20,000	28,160	10,730	1.75	2.80	4.09	7,862	32,140	3.00
Rural	Multilane Controlled Access	Under 10,000	7,500	28,160	10,730	0.31	0.49	0.268	9,320	2,500	0.23
		Over 10,000	20,000	28,160	10,730	0.28	0.45	0.657	9,320	6,123	0.57
Urban	Two-Lane	Under 10,000	7,500	14,080	5,365	1.51	2.42	1.33	5,152	6,826	1.27
		Over 10,000	15,000	14,080	5,365	2.06	3.30	3.61	5,152	18,617	3.47
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	28,160	10,730	3.07	4.91	2.69	4,897	13,164	1.27
		Over 10,000	20,000	28,160	10,730	4.19	6.70	9.78	4,897	47,902	4.46
Urban	Multilane Controlled Access	Under 10,000	7,500	28,160	10,730	0.71	1.14	0.624	5,672	3,540	0.33
		Over 10,000	20,000	28,160	10,730	0.97	1.56	2.28	5,672	12,919	1.20

Adjustment Factors:
Wet-pavement exposure time
Traffic volume

TABLE 15

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Open-Graded Asphalt Surface for New Pavement

Prior decision to resurface? N/A Service life: 10 Years
Base condition Dense-Graded Asphalt Surface for New Pavement

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	(7) acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	6,688	952	0.540	0.865	0.158	9,348	1,476	1.55
		Over 10,000	15,000	6,688	952	0.323	0.517	0.566	9,348	5,292	5.55
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	13,376	1,904	0.495	0.793	0.434	7,862	3,413	1.79
		Over 10,000	20,000	13,376	1,904	0.378	0.605	0.883	7,862	6,945	3.65
Rural	Multilane Controlled Accesa	Under 10,000	7,500	13,376	1,904	0.495	0.793	0.434	9,320	4,046	2.13
		Over 10,000	20,000	13,376	1,904	0.378	0.605	0.883	9,320	8,232	4.32
Urban	Two-Lane	Under 10,000	7,500	6,688	952	0.414	0.663	0.363	5,152	1,870	1.96
		Over 10,000	15,000	6,688	952	0.323	0.517	0.566	5,152	2,917	3.06
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	13,376	1,904	0.495	0.793	0.434	4,897	2,126	1.12
		Over 10,000	20,000	13,376	1,904	0.378	0.605	0.883	4,897	4,326	2.27
Urban	Multilane Controlled Access	Under 10,000	7,500	13,376	1,904	0.495	0.793	0.434	5,672	2,463	1.29
		Over 10,000	20,000	13,376	1,904	0.378	0.605	0.883	5,672	5,010	2.63

Adjustment Factors:
Wet-pavement exposure time
Traction demand
Traffic volume

TABLE 16

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Open-Graded Asphalt Overlay

Prior decision to resurface? Yes Service life: 10 Years
Base condition Dense-Graded Asphalt Overlay (Average Microtexture Aggregate)

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	10,560	1,504	0.540	0.865	0.158	9,348	1,476	0.98
		Over 10,000	15,000	10,560	1,504	0.323	0.517	0.566	9,348	5,292	3.52
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	21,120	3,007	0.495	0.793	0.434	7,862	3,413	1.14
		Over 10,000	20,000	21,120	3,007	0.378	0.605	0.883	7,862	6,945	2.31
Rural	Multilane Controlled Access	Under 10,000	7,500	21,120	3,007	0.495	0.793	0.434	9,320	4,046	1.35
		Over 10,000	20,000	21,120	3,007	0.378	0.605	0.883	9,320	8,232	2.74
Urban	Two-Lane	Under 10,000	7,500	10,560	1,504	0.395	0.663	0.363	5,152	1,870	1.24
		Over 10,000	15,000	10,560	1,504	0.323	0.517	0.566	5,152	2,917	1.94
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	21,120	3,007	0.495	0.793	0.434	4,897	2,126	0.71
		Over 10,000	20,000	21,120	3,007	0.378	0.605	0.883	4,897	4,326	1.44
Urban	Multilane Controlled Access	Under 10,000	7,500	21,120	3,007	0.495	0.793	0.434	5,672	2,463	0.82
		Over 10,000	20,000	21,120	3,007	0.378	0.605	0.883	5,672	5,010	1.67

Adjustment Factors:
Wet-pavement exposure time
Traction demand
Traffic volume

TABLE 17

BENEFIT-COST CALCULATION FOR A 1.6-km (1-mile) SECTION

Countermeasure: Dense-Graded Asphalt Overlay (High Microtexture Aggregate)

Prior decision to resurface? Yes Service life: 10 Years

Base condition Dense-Graded Asphalt Overlay (Average Microtexture Aggregate)

(1) Area Type	(2) Highway Type	(3) ADT Category	(4) ADT	(5) Construction Cost ($)	(6) Equivalent Uniform Annual Cost ($)	(7) Equivalent Uniform Annual Accident Rate Reduction acc/MVK	(7) acc/MVM	(8) Annual Accidents Reduced	(9) Cost Per Accident ($)	(10) Unadjusted Annual Accident Cost Savings ($)	(11) Unadjusted B/C Ratio
Rural	Two-Lane	Under 10,000	2,500	3,520	501	0.303	0.485	0.088	9,348	827	1.65
		Over 10,000	15,000	3,520	501	0.095	0.152	0.166	9,348	1,556	3.11
Rural	Multilane Uncontrolled Access	Under 10,000	7,500	7,040	1,002	0.256	0.410	0.224	7,862	1,765	1.76
		Over 10,000	20,000	7,040	1,002	0.140	0.224	0.327	7,862	2,571	2.57
Rural	Multilane Controlled Access	Under 10,000	7,500	7,040	1,002	0.256	0.410	0.224	9,320	2,092	2.09
		Over 10,000	20,000	7,040	1,002	0.140	0.224	0.327	9,320	3,048	3.04
Urban	Two-Lane	Under 10,000	7,500	3,520	501	0.174	0.279	0.153	5,152	787	1.57
		Over 10,000	15,000	3,520	501	0.095	0.152	0.166	5,152	857	1.71
Urban	Multilane Uncontrolled Access	Under 10,000	7,500	7,040	1,002	0.256	0.410	0.224	4,897	1,099	1.10
		Over 10,000	20,000	7,040	1,002	0.140	0.224	0.327	4,897	1,602	1.60
Urban	Multilane Controlled Access	Under 10,000	7,500	7,040	1,002	0.256	0.410	0.224	5,672	1,273	1.27
		Over 10,000	20,000	7,040	1,002	0.140	0.224	0.327	5,672	1,855	1.85

Adjustment Factors:

Wet-pavement exposure time

Traction demand

Traffic volume

The unadjusted benefit-cost ratios for each pavement macrotexture improvement for each combination of area type, highway type and ADT level are found in Column 11 of Tables 10 through 17. For the sake of convenience, all of the benefit-cost ratios are summarized in Table 18.

These benefit-cost ratios can be used in two ways. First, in the form presented in Table 18, the benefit-cost ratios can be used to evaluate the cost-effectiveness of pavement macrotexture improvements under typical conditions. In the second use, the typical benefit-cost ratios can also be adjusted to be more appropriate to a particular site whose actual conditions are known. Three adjustment factors are used: a wet-pavement exposure factor; a traction-demand factor; and a traffic volume factor. Tables 10 through 17 indicate which of these three factors are used in the evaluation of each pavement macrotexture improvement. The equivalent uniform annual accident cost savings can be converted to apply to specific site conditions by multiplying by the adjustment factors. Because the equivalent uniform annual cost savings appear in the numerator of the benefit-cost ratio expression, the benefit-cost ratios themselves can be adjusted by direct multiplication of the adjustment factors. In all three cases, increases in wet-pavement exposure, traction demands and traffic volumes above those assumed in the primary benefit-cost analysis result in higher benefit-cost ratios, and vice versa.

The first adjustment factor is based on wet-pavement exposure time. A typical value of 20% wet-pavement exposure time, was assumed in the benefit-cost analysis. For sites which deviate from this typical value, the appropriate adjustment factor is the ratio between the actual wet-pavement exposure time and the 20% assumed value. A convenient tabulation of this factor is found in Table 19.

The second adjustment factor accounts for traction demand. This factor is determined by the dry-pavement accident rate for the site. The Phase I analysis results presented in Volume I of this report show that the rate of change of wet-pavement accident rate with skid number is sensitive to the dry-pavement accident rate. This finding is interpreted to mean that highways with high traction demands have both relatively high dry-pavement accident rates and an increased sensitivity of wet-pavement accident rate to skid number. An adjustment factor based on the dry-pavement accident rate is tabulated in Table 20. If the value for dry-pavement accident rate is not known, an adjustment factor of 1.00 should be assumed.

The final adjustment factor is based on the actual at the site traffic volume. This factor is calculated as:

$$\text{Traffic Volume Factor} = \frac{\text{Actual ADT}}{\text{Assumed ADT}}$$

TABLE 18

SUMMARY OF UNADJUSTED BENEFIT-COST RATIOS FOR PAVEMENT MACROTEXTURE IMPROVEMENTS

Area Type	Highway Type	ADT	No Prior Decision to Resurface: Open-Graded Asphalt Overlay	No Prior Decision to Resurface: Dense-Graded Asphalt Overlay With High Microtexture Aggregate	No Prior Decision to Resurface: Dense-Graded Asphalt Overlay With Average Microtexture Aggregate	Prior Decision to Resurface: Open-Graded Asphalt Overlay	Prior Decision to Resurface: Dense-Graded Asphalt Overlay With High Microtexture Aggregate	Pavement Grooving: No Tire Chains or Studded Tires	Pavement Grooving: Tire Chains and/or Studded Tires	Open-Graded Asphalt Surface for New Pavement
Rural	Two-Lane	2,500	0.51	0.44	0.04	0.98	1.65	1.48	0.65	1.55
		15,000	1.76	0.77	0.00	3.52	3.11	12.42	5.46	5.55
Rural	Multilane Uncontrolled Access	7,500	0.58	0.46	0.03	1.14	1.76	1.18	0.52	1.79
		20,000	1.15	0.64	0.00	2.31	2.57	6.82	3.00	3.65
Rural	Multilane Controlled Access	7,500	0.69	0.55	0.04	1.35	2.09	0.53	0.23	2.13
		20,000	1.37	0.76	0.00	2.74	3.04	1.30	0.57	4.32
Urban	Two-Lane	7,500	0.62	0.40	0.01	1.24	1.57	2.89	1.27	1.96
		15,000	0.97	0.43	0.00	1.94	1.71	7.90	3.47	3.06
Urban	Multilane Uncontrolled Access	7,500	0.36	0.29	0.02	0.71	1.10	2.79	1.27	1.12
		20,000	0.72	0.40	0.00	1.44	1.60	10.16	4.46	2.27
Urban	Multilane Controlled Access	7,500	0.42	0.34	0.02	0.82	1.27	0.75	0.33	1.29
		20,000	0.83	0.46	0.00	1.67	1.85	2.74	1.20	2.63

TABLE 19

WET-PAVEMENT EXPOSURE TIME ADJUSTMENT FACTOR

Wet-Pavement Exposure Time (%)	Adjustment Factor	Wet-Pavement Exposure Time (%)	Adjustment Factor
0	0.00		
1	0.05	21	1.05
2	0.10	22	1.10
3	0.15	23	1.15
4	0.20	24	1.20
5	0.25	25	1.25
6	0.30	26	1.30
7	0.35	27	1.35
8	0.40	28	1.40
9	0.45	29	1.45
10	0.50	30	1.50
11	0.55	31	1.55
12	0.60	32	1.60
13	0.65	33	1.65
14	0.70	34	1.70
15	0.75	35	1.75
16	0.80	36	1.80
17	0.85	37	1.85
18	0.90	38	1.90
19	0.95	39	1.95
20	1.00	40	2.00

TABLE 20

TRACTION DEMAND ADJUSTMENT FACTOR

Dry-Pavement Accident Rate		
(accidents/MVK)	(accidents/MVM)	Adjustment Factor
under 0.67	under 1.082	0.00
0.68	1.10	0.02
0.75	1.20	0.11
0.81	1.30	0.20
0.87	1.40	0.29
0.93	1.50	0.39
0.99	1.60	0.48
1.06	1.70	0.57
1.12	1.80	0.67
1.18	1.90	0.76
1.24	2.00	0.85
1.30	2.10	0.94
1.37	2.20	1.04
1.43	2.30	1.13
1.49	2.40	1.22
1.55	2.50	1.31
1.62	2.60	1.41
1.68	2.70	1.50
1.74	2.80	1.59
1.80	2.90	1.68
1.86	3.00	1.78
over 1.88	over 3.02	1.79

The value of assumed ADT appropriate for the site under consideration should be selected from Column 4 of Tables 10 through 17. This adjustment factor is only approximate, since it accounts for the influence of traffic volume on vehicles-kilometers of exposure to wet-pavement, but not the influence of increased or decreased traffic passages on skid number. If the actual ADT differs substantially from the assumed ADT, then this adjustment factor should not be used and the calculation of equivalent uniform annual accident cost savings should be repeated using the actual ADT.

A simple example illustrates the use of the three adjustment factors. Consider a rural, two-lane highway section with average daily traffic of 5,000 vehicles, wet-pavement exposure time of 25% and dry-pavement accident rate of 1.55 accidents per million dry-pavement vehicle-kilometers (2.50 accidents per million dry-pavement vehicle-miles). Let us assume that an open-graded asphalt overlay is considered for this section as a wet-pavement accident countermeasure. No prior decision to resurface the pavement has been made. Table 18 indicates that the benefit-cost ratio for an open-graded asphalt overlay on a rural, two-lane highway with traffic volume less than 10,000 vehicles per day is 0.51. Since this benefit-cost ratio is less than 1.0, it does not initially appear that the overlay is justified. However, the tabulated benefit-cost ratio is only appropriate for sites with traffic volumes of 2,500 vehicles per day, wet-pavement exposure time of 20% and average dry-pavement accident rate. The appropriate wet-pavement time adjustment factor from Table 19 is 1.25; the traction demand adjustment factor is 1.31. The traffic volume adjustment factor is:

$$\frac{\text{Actual ADT}}{\text{Assumed ADT}} = \frac{5{,}000}{2{,}500} = 2.0$$

The adjusted benefit-cost ratio is:

$$0.51(1.25)(1.31)(2.00) = 1.67 > 1.0$$

Thus, an open-graded asphalt overlay for this site is economically justified, even though it would not be justified for a typical rural, two-lane site.

E. Interpretation of Benefit-Cost Ratios

The benefit-cost ratios presented in the previous section after appropriate adjustment factors are applied, provide an indication of whether a pavement macrotexture improvement is economically justified under generalized site conditions. Improvements whose benefits exceed costs have benefit-cost ratios greater than 1.0 and are economically justified. Conversely, improvements with benefit-cost ratios less than 1.0 are not justified economically. When several alternative pavement macrotexture improvements are considered at a particular site the final selection should be based on an incremental benefit-cost analysis. For an incremental analysis, the alternatives must be arranged in order of increasing costs. Each additional expenditure (incremental cost) must be exceeded by a corresponding incremental benefit in order to justify a higher-cost improvement. This section discusses the significance of the benefit-cost ratios for each pavement macrotexture improvement. The use of the adjustment factors to establish cost-effectiveness warrants for pavement macrotexture improvements is presented in Section VII.

Based on the assumption about costs and benefits in this example, open-graded asphalt overlays cannot generally be justified economically under the assumed conditions except for rural highways in the high ADT category. However, benefit-cost ratios over 1.0 could be attained for any type of site with high wet-pavement exposure or traction demand. Also, the comparison of open-graded and dense-graded overlays shows that the use of open-graded asphalt is easily justified when a prior decision to resurface a section has been made for reasons other than the reduction of wet-pavement accidents. Similarly, open-graded asphalt surfaces for new pavements are also cost-effective under typical conditions.

In this example, dense-graded asphalt surfaces with high microtexture aggregate do not appear cost-effective as a wet-pavement accident countermeasure unless extremely high adjustment factors can be applied or unless a previous decision to resurface has been made for other reasons. However, even under conditions where dense-graded asphalt surfaces with high-microtexture aggregate are justified, the open-graded asphalt surfaces considered in this example are more cost-effective. A substantial change in the construction costs or rate of change of skid number used in this analysis would be required to reverse this general conclusion. Dense-graded asphalt surfaces with average microtexture aggregate do not appear cost-effective as a wet-pavement accident countermeasure, under any of the conditions considered in this example.

The benefit-cost analysis demonstrates that pavement grooving is a very cost-effective countermeasure when there are no tire chains or studs in the traffic stream, and is cost-effective in many situations even when tire chains and studs are present. Furthermore, this analysis probably overstates the cost of grooving because nearly the same accident reduction might result from grooving only those areas with high traction demands, such as horizontal curves.

The variety of factors that must be considered in the cost-effectiveness analysis indicate that there can be no single level of macrotexture and no one high-macrotexture surfacing material that is optimal for all site conditions and all geographic areas. It is recommended that the selection of pavement macrotexture improvements for a particular site or a particular geographic area be based on cost-effectiveness considerations such as those presented in this section. The most reliable analysis results will be obtained if users incorporate actual construction costs, traffic wear rates, wet-pavement exposure estimates, etc., that are valid for particular locations and local materials. Benefit-cost analysis provides a strong justification for pavement macrotexture improvements at sites where they are appropriate, and can be used to establish priorities between improvement sites. However, it is important in establishing a pavement surface improvement program that agencies also consider the relative cost-effectiveness of other traffic safety improvements not related to pavement macrotexture, such as signing, geometric improvements, and roadside safety.

VII. WARRANTS FOR HIGH-MACROTEXTURE SURFACE COURSES

This section discusses the subject of warrants for high-macrotexture improvements. The initial portion of this section discusses the current practices employed by state highway departments to implement pavement macrotexture improvements, while the final portion provides an example of the development of warrants from the results of the cost-effectiveness analysis presented in Section VI. Because the cost-effectiveness of pavement macrotexture improvements depends on wet-pavement exposure time, traction demand, and traffic volume, no single set of warrants can be appropriate for all conditions. The final portion of this section provides guidance for a user to develop cost-effective warrants for high-macrotexture surface courses that are appropriate for a particular locality.

A. Current State Practices

Most states have not established formal warrants for pavement macrotexture improvements or other wet-pavement accident countermeasures. However, most states have established procedures for monitoring wet-pavement accident experience, identifying locations with higher than expected wet-pavement accident experience and implementing countermeasures at such sites. The implementation decisions in some states are based on some form of cost-effectiveness analysis, but in most states countermeasures are selected on the basis of engineering judgment, tempered by budget constraints. To a great extent, judgment has been the only basis on which such decisions could be based because of the dearth of proven evaluations of accident reduction effectiveness. In nearly every case where cost-effectiveness analyses have been employed, their purpose has been to justify the need for some improvement rather than to compare alternative improvements. Other criteria employed in the decisionmaking process include skid number and traffic volume levels.

Three states that have employed cost-effectiveness considerations in the selection of pavement surface improvements are Virginia, California, and Texas. Virginia's approach has been described by Runkle and Mahone.[70] In this approach, the annual accident reduction benefit is determined by assuming that the effect of any surface improvement is to reduce the percentage of all accidents that occur on wet-pavement from its level before improvement of a particular highway section to the statewide average--20%. Property-damage accident costs are estimated as the average of the property damage costs reported for the analysis site; injury costs are based on the National Safety Council estimate of $4,000 per injury; and, fatal accident costs are considered equal to injury costs. The ratio of the total construction cost to the annual accident cost savings, known as the "breakeven value" or payback period, is used as one basis for establishing improvement priorities. The Virginia approach can only be used to establish the need for an improvement and not to compare alternatives, because alternative improvements have the same estimated annual benefit and differ only in the cost. This approach does not incorporate the concept of an interest rate or minimum attractive rate of return; does not consider the service life of an improvement; and assumes implicitly that the improvement is equally effective throughout its service life. Additional factors considered by Virginia in establishing priorities for field review (including skid testing) and subsequent improvement are number of wet-pavement accidents, percentage of wet-pavement accidents and skid number level.

The California Department of Transportation has developed a cost-effectiveness criterion for pavement surface improvements (and other types of safety improvements) called the Safety Index.[10] The Safety Index is equal to the conventional benefit-cost ratio multiplied by 100, except that the computation does not involve an interest rate. However, as in Virginia, this technique is not employed to compare improvement alternatives. There is a surprising reluctance, even by states that choose a formal benefit-cost approach, to require a minimum rate of return from improvements through use of an interest rate in the analysis.

Texas has established a priority system for ranking pavement surface improvements based on their Skid-Prone Index,[32] which they define as:

$$SPI = B/C \times \left(\frac{ADT}{1,000}\right) \times SNF$$

where SPI = Skid-Prone Index,

B/C = Benefit-cost ratio,

ADT = Average daily traffic (vehicles/day), and

SNF = Skid number factor =

$$= \frac{1}{30} \text{ if } SN_{40} < 30$$

$$= \frac{1}{35} \text{ if } 30 \le SN_{40} \le 40$$

$$= \frac{1}{40} \text{ if } SN_{40} > 40.$$

The benefit-cost ratio in the Skid-Prone Index is determined in the conventional manner from estimated accident cost savings and construction costs using an interest rate of 8%. This technique is used to establish improvement priority rankings, but not to compare improvement alternatives. It is not at all apparent why the ADT or skid number factor should receive independent consideration in the establishment of priority rankings, if the influence of ADT and skid number on wet-pavement accident reduction is incorporated in the benefit-cost ratio on a rational and consistent basis.

Other states have established formal or informal priorities from their analysis of locations with high wet-pavement accident experience on some basis other than a cost-effectiveness analysis. The most common criteria are based on wet-pavement accident frequency or ratio of wet-pavement accidents to total accidents.

Other factors that have been used to establish improvement criteria are skid number and traffic volume level. Several states have established surface improvement guidelines based on skid number measurements, without establishing a formal, minimum skid number requirement. For example, one state highway department requires continued surveillance of pavements that are tested and found to have skid numbers between 35 and 49; corrective action is scheduled in the next fiscal year for pavements with skid numbers between 31 and 34; and, immediate resurfacing is required for pavements with measured skid numbers less than 30. This kind of guideline establishes the need for a surface improvement, but does not indicate which of the available alternatives should be selected. The Iowa Department of Transportation,[61/] has established usage limits for various surfacing alternatives that serve as traffic volume warrants. For example, chip-seal coats and slurry seals are recommended only for traffic volumes less than 1,500 vehicles per day, conventional dense-graded overlays for traffic volumes less than 3,000 vehicles per day and dense-graded overlays with polish-resistant aggregate for traffic volumes from 1,500 to 5,000 vehicles per day. However, no traffic volume limits have been established for open-graded surfaces, sprinkle treatments, and pavement grooving. Such usage limits prevent the application of a surface improvement at an inappropriate traffic volume level. However, traffic volume alone is not sufficient to warrant a pavement surface improvement. Agencies that have established traffic volume usage limits also consider other factors in selecting pavement surface improvements.

B. Development of Cost-Effective Warrants for Use of High-Macrotexture Surface Courses

The cost-effectiveness analysis presented in Section VI can be utilized to develop warrants for pavement macrotexture improvements. Warrants alone cannot guarantee that the optimum pavement macrotexture improvement is selected for a particular site--that can only be accomplished through an incremental benefit-cost analysis. However, the warrants can assure the user that each pavement macrotexture improvement that is implemented is economically justified.

The central principle in the development of cost-effective warrants is that pavement macrotexture improvements with benefit-cost ratios greater than 1.0 are warranted and improvements with benefit-cost ratios less than 1.0 are not warranted. Three adjustment factors presented in Section VI are applicable to sites whose wet-pavement exposure time, dry-pavement accident rate and ADT deviate from assumed typical conditions. The boundary between site conditions that warrant an improvement and site conditions that do not is established by defining combinations of the adjustment factors that produce a benefit-cost ratio of exactly 1.0. The warrants developed in this manner are presented in a convenient graphical form that can be used as a decisionmaking tool.

The three adjustment factors are applied to the benefit-cost ratio in the following manner:

$$\text{Adjusted B/C} = (\text{B/C})(\text{WPF})(\text{TDF}) \left(\frac{\text{Actual ADT}}{\text{Assumed ADT}}\right)$$

where B/C = Unadjusted benefit-cost ratio,

WPF = Wet-pavement exposure time factor (see Table 19),

TDF = Traction demand factor (determined from dry-pavement accident rate; see Table 20),

Actual ADT = Average daily traffic at site (vehicles/day), and

Assumed ADT = Assumed average daily traffic for site conditions (vehicles/day) (see Tables 10 to 17).

The influence of the wet-pavement exposure time and traction demand factors is completely accounted for by multiplying the unadjusted benefit-cost ratio by the factors shown above. However, the influence of ADT is only accounted for approximately by multiplying by the ratio of actual ADT to assumed ADT as illustrated above. This occurs because the traffic volume also affects the rate of change of skid number during the analysis period. To provide accurate warrants, the unadjusted benefit-cost ratios have been recomputed for each ADT level considered using the appropriate rate of change of skid number.

Warrants have been developed for illustrative purposes for open-graded asphalt overlays under two types of site conditions. The two site conditions considered are rural, two-lane highways with traffic volumes less than 10,000 vehicles per day and rural, multilane, controlled-access highways with traffic volumes greater than 10,000 vehicles per day. Similar warrants could be developed for each of the 96 combinations of area type, highway type, ADT level, and pavement macrotexture improvement type shown in Table 18. Also, the warrants presented here can be easily modified to reflect local experience in areas such as construction costs. Only warrants that are consistent with local conditions and experience are likely to be used.

Figure 5 presents the warrants for open-graded asphalt overlays on rural, two-lane highways and Figure 6 presents the warrants for rural, multilane, controlled access highways. The abcissa of each figure represents the

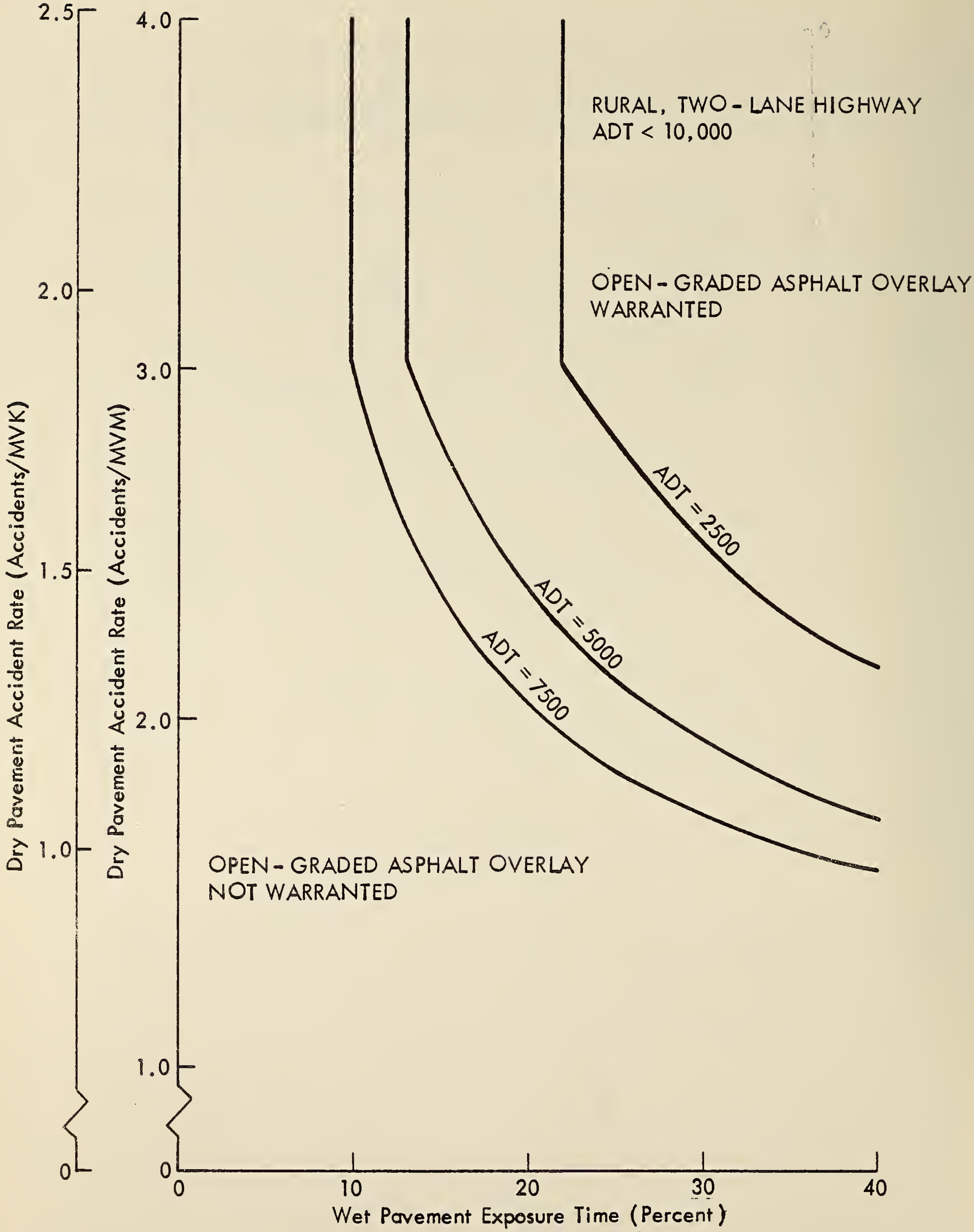

Figure 5 - Warrants for Open-Graded Asphalt Overlays

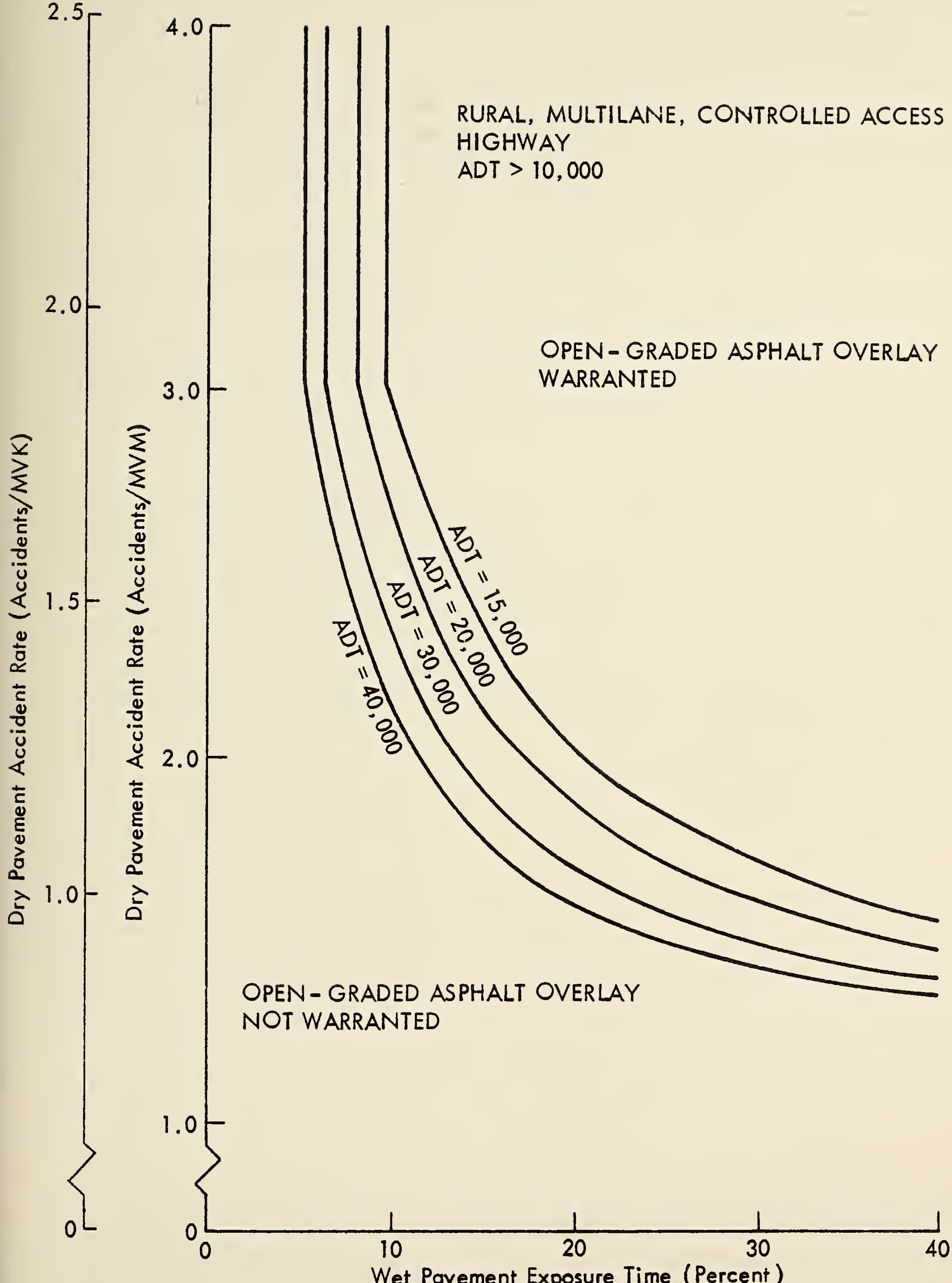

Figure 6 - Warrants for Open-Graded Asphalt Overlays

percentage of wet-pavement exposure time. The ordinate represents the dry-pavement accident rate. The plotted curves for different ADT levels represent conditions where the adjusted benefit-cost ratio is equal to 1.0. Because of the manner in which the traction demand adjustment factor is defined (see Table 20), these curves become vertical at dry-pavement accident rates above 1.88 accidents/MVK (3.02 accidents/MVM) and approach an asymptote at the dry-pavement accident rate of 0.67 accidents/MVK (1.08 accidents/MVM).

The figures are interpreted in the following manner. The point defined by the wet-pavement exposure time and dry-pavement accident rate for a particular site is located in the figure. If this point lies on or to the right of the curve defined for the ADT at the site, the pavement macrotexture improvement is warranted. However, if this point is to the left of the appropriate ADT curve, the pavement macrotexture improvement is not warranted. If no curve is given for a specific ADT level an approximate interpolation should be made.

The procedures presented in this report reflect the current state of the art of pavement macrotexture improvement evaluation and analysis. It is emphasized that no single set of warrants for high-macrotexture surface courses can be appropriate for all circumstances. However, the cost-effectiveness procedures can be applied to develop warrants applicable to particular geographic areas and particular materials. As further evaluations of high-macrotexture surface courses are performed, permitting refinements of the benefit-cost analyses, the warrants should be updated to reflect the current state of knowledge.

VIII. CONCLUSIONS

The following findings and conclusions were drawn from the investigations reported in this volume:

1. Pavement microtexture is determined primarily by aggregate properties; pavement macrotexture is determined primarily by mix design and aggregate gradation.

2. Microtexture determines the skid resistance at low speeds. Both microtexture and macrotexture contribute to the skid resistance at high speeds. Macrotexture is particularly important because it provides a channel for water to escape from the tire-pavement interface.

3. Skid number can be predicted from pavement macrotexture and microtexture using a relationship developed recently by Penn State University.

4. Numerous methods have been applied by agencies in the United States and abroad to measure pavement macrotexture. These methods are described in the text of the report.

5. The sand patch method is the most widely used and accepted method in the United States for measuring pavement macrotexture.

6. There has been no organized effort to appraise the precision, accuracy and intercorrelation of the various macrotexture measurement methods. The literature provided only fragmented information available from obscure sources.

7. Many researchers have attempted to validate pavement macrotexture measurement techniques by comparison with pavement friction coefficients (skid numbers). The results of such efforts are bound to be disappointing because skid number is a function of both macrotexture and microtexture.

8. There are two primary methods for providing a high level of macrotexture in new pavements: open-graded asphalt surfaces and texturing of portland cement concrete surfaces.

9. Open-graded asphalt surfaces have had wide use in some states and very limited use in others. The usual reasons for limited use of open-graded asphalt surfaces are high cost and lack of suitable aggregates.

10. Wire tining has achieved nearly universal acceptance as the best method for finishing portland cement concrete surfaces in the plastic state.

11. The methods for restoring a high level of macrotexture in existing pavements include: open-graded asphalt overlays, grooving, cold milling and seal coats. Open-graded asphalt overlays and pavement grooving are widely used in some areas. Cold milling has had only limited use as a technique for improving skid resistance. Seal coats are appropriate only for highways with low traffic volume levels.

12. Tabulations of benefit-cost ratios for various pavement macrotexture improvements have been developed and are presented as an example.

13. Factors such as traffic volume, wet-pavement exposure time and traction demand are often critical in determining whether or not a pavement macrotexture improvement is cost-effective. For this reason, no single set of warrants for pavement macrotexture improvements can be appropriate for all site conditions and all geographic areas.

14. The development of cost-effective warrants for selected improvements from the results of the benefit-cost analysis has been illustrated. The warrants are determined from area type (urban and rural), highway type, traffic volume, wet-pavement exposure time and traction demand (represented by dry pavement accident rate). Warrants developed in this manner are applicable to particular site conditions and particular geographic areas.

IX. RECOMMENDATIONS

The following recommendations were developed in the investigation of criteria for improvement of pavement macrotexture:

1. A standard method for macrotexture measurement should be established and applied consistently throughout the highway community. The establishment of a standard will require close cooperation between FHWA, state highway departments, ASTM and other interested organizations.

2. More complete information is needed on the relationship between pavement macrotexture and wet-pavement accident rate. The indirect relationships used in this study should be refined as new information becomes available.

3. Evaluations of the performance of pavement macrotexture improvement methods should continue. Additional data on the service lives of pavement macrotexture improvements under various traffic conditions would be valuable.

4. Highway agencies are encouraged to select pavement macrotexture improvements on a cost-effectiveness basis using techniques such as those presented in this report. The benefit-cost ratios and warrants presented here will be most accurate and most accepted if they are refined by individual users to reflect local experience with construction costs and paving materials.

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APPENDIX A

FHWA DESIGN PROCEDURE FOR OPEN-GRADED ASPHALT MIXTURES

This appendix presents the design procedure recommended by the Federal Highway Administration for open-graded asphalt mixtures. The design procedure described is taken from the final draft of the National Cooperative Highway Research Program Synthesis of Highway Practice, entitled "Open-Graded Friction Courses." This procedure is essentially the same as that presented in a 1974 Federal Highway Administration report by Smith, Rice and Spelman,[78] but is updated to reflect the changes recommended in the supplement to that report prepared in 1975 by Smith.[79]

1.0 Material Requirements

1.1 It is recommended that relatively pure carbonate aggregates or any aggregates known to polish be excluded from the coarse aggregate fraction (material retained on the No. 8 sieve). In addition, the coarse aggregate fraction should have at least 75 percent (by weight) of particles with at least two fractured faces and 90 percent with one or more fractured faces. The abrasion loss (AASHTO T 96) should not exceed 40 percent.

1.2 Recommended Gradation for Open-Graded Asphalt Friction Course.

Sieve Size[a]	Percent Passing[b]
1/2"	100
3/8"	95-100
#4	30-50
#8	5-15
#200	2-5

a/ U.S. Sieve Series.

b/ By volume. (This is the same as by weight unless specific gravities of aggregates being combined are different.)

1.3 The recommended grade of asphalt cement is AC-10, AC-20, or AR-40, AASHTO M 226. For AC-10 and AC-20, the M 226 Table 2 requirements should apply where such asphalt is available. AR-40 requirements are given in Table 3 of M 226.

2.0 Preliminary Data

2.1 Test coarse and fine aggregates as received for the project for gradation unless otherwise provided. If mineral filler is submitted as a separate item, it should also be tested for specification compliance. Analyze gradation results to determine if proportions of aggregates and batching operations proposed by the contractor will meet the job-mix formula and the specification limits of step 1.2.

2.2 Determine bulk and apparent specific gravity for the coarse and fine aggregate fractions (retained and passing the No. 8 sieve) for each type of material submitted. Additional specific gravity tests are not warranted when the only distinction between aggregates is size of grading. Using the information verified in step 2.1, mathematically compute the bulk and apparent specific gravity for the coarse and fine aggregate fractions (retained and passing the No. 8 sieve) for the proposed job-mix gradation.

2.3 Test the asphalt cement to be used for specification compliance (AASHTO M 226), viscosity-temperature data, and specific gravity at 77.0°F.

3.0 Asphalt Content

3.1 Determine the surface capacity of the aggregate fraction that is retained on a No. 4 sieve in accordance with the following procedure.

Note: For highly absorptive aggregates, use the procedure described in step 3.3.

K_c is determined from the percent of SAE No. 10 oil retained, which represents the total effect of superficial area, the aggregate's absorptive properties, and surface roughness.

3.1.1 Quarter out 105 g representative of the material passing the 3/8 in. sieve and retained on the No. 4 sieve.

3.1.2 Dry sample on hot plate or in 230 $\pm$ 9°F oven to constant weight and allow to cool.

3.1.3 Weigh out 100.0 g and place in a metal funnel (top diam) 3-1/2 in., height 4-1/2 in., orifice 1/2 in., with a piece of No. 10 sieve soldered to the bottom of the opening).

3.1.4 Completely immerse specimen in SAE No. 10 lubricating oil for 5 min.

3.1.5 Drain for 2 minutes.

3.1.6 Place funnel containing sample in 140°F oven for 15 min of additional draining.

3.1.7 Pour sample from funnel into tared pan; cool, and reweigh sample to nearest 0.1 g. Subtract original weight and record difference as percent oil retained (based on 100 g of dry aggregate).

(a) If specific gravity for the fraction is greater than 2.70 or less than 2.60, apply correction to oil retained, using formula at bottom of chart in Figure 7.

(b) Start at the bottom of chart in Figure 7 with the corrected percent of oil retained; follow straightedge vertically upward to intersection with the diagonal line; hold point, and follow the straightedge horizontally to the left. The value obtained is the surface constant for the retained fraction and is known as K_c.

3.2 Determine the required asphalt content, which is based on weight of aggregate, from the following relationship.

$$\text{Percent asphalt} = (2.0\ K_c + 4.0) \times \frac{2.65}{(SG)ca}$$

Where K_c = surface constant

(SG)ca = apparent specific gravity of coarse aggregate (3/8 in. to No. 4)

3.3 For highly absorptive aggregates, use the following procedure for determining K_c and asphalt content.

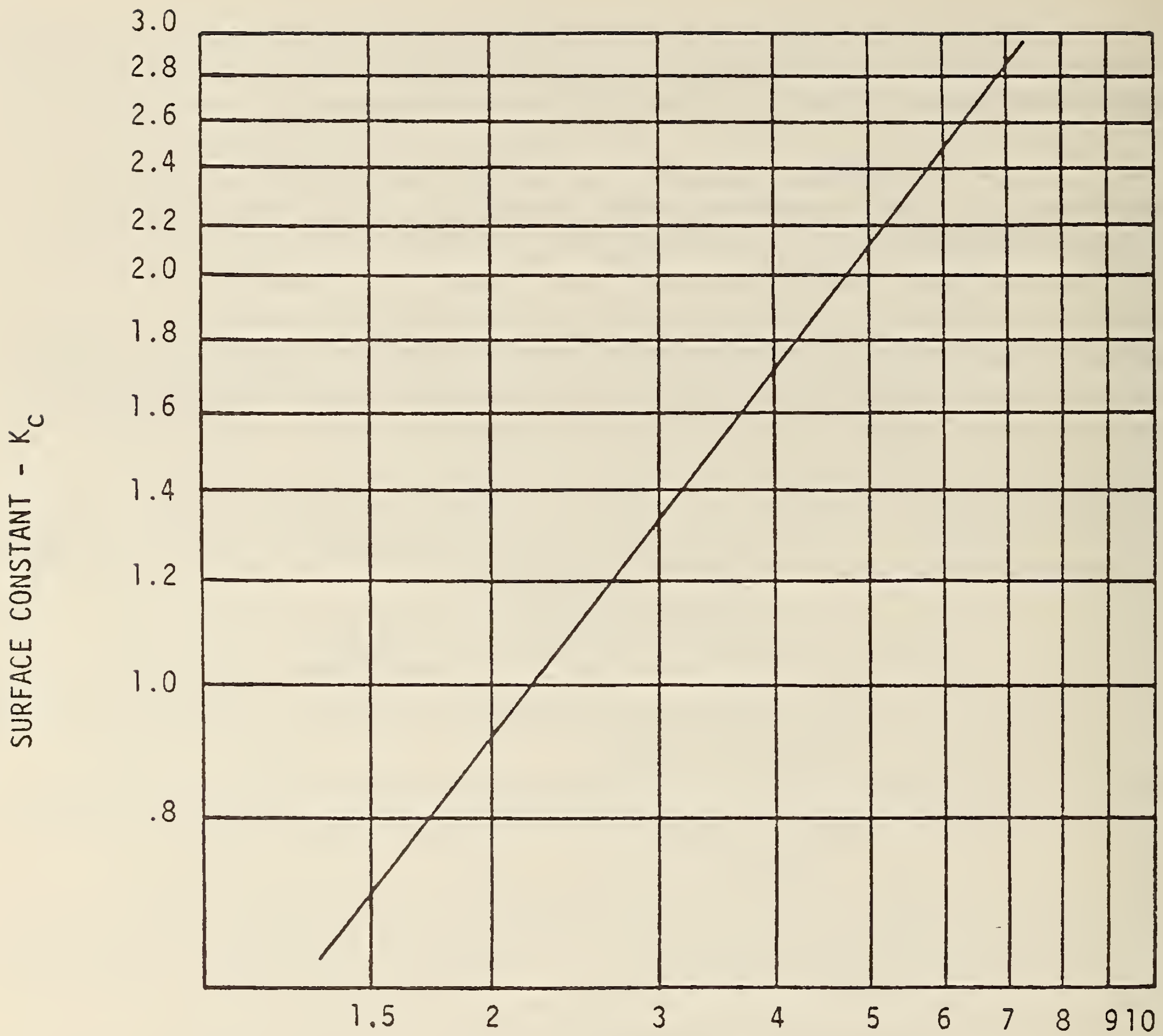
3.3.1 Follow the recommended design procedure from step 3.1 through step 3.1.3.

3.3.2 Follow the instructions in step 3.1.4, except immerse the specimen for 30 min.

3.3.3 Follow the recommended procedure from step 3.1.5 through step 3.1.7.

3.3.4 Pour the sample onto a clean, dry, absorptive cloth; obtain a saturated surface dry condition; pour sample from cloth into a tared pan; reweigh sample to nearest 0.1. Subtract original weight of aggregate and record difference as percent oil absorbed (based on 100 g of aggregate).

3.3.5 Subtract the percent oil absorbed value (see 3.3.4 above) from the percent oil retained value (see 3.3.3 above), and obtain the percent (free) oil retained value. This value represents the percent oil retained value that would have been obtained had the aggregate been a nonabsorptive type. The above technique allows one to evaluate the aggregate's



PERCENT OIL RETAINED--CORRECTED FOR SPECIFIC GRAVITY OF AGGREGATE

Material used: Aggregate--passing 3/8" sieve, retained on No. 4 sieve
Oil--SAE No. 10

$$\text{Oil Retained Corrected (\%)} = \text{Oil Retained (\%)} \times \frac{\text{apparent specific gravity of course aggregate}}{2.65}$$

Figure 7 - Chart for Determining Surface Constant (K_c) of Coarse Aggregate

surface and shape characteristics without the overwhelming influence of a large quantity of absorbed oil.

3.3.6 Follow the procedure recommended in steps 3.1.8 and 3.2. The only exception is that the percent (free) oil retained value is used (from step 3.3.5) to obtain K_c. Thus, the asphalt quantity determined is the "effective" asphalt content.

3.3.7 Follow the recommended procedure indicated through sections 4 and 5. Because asphalt absorption is not presently included in the formula for the determination of fine aggregate content, it is particularly desirable that the effects of oil absorption in the K_c test be excluded in the case of the highly absorptive aggregate.

3.3.8 Prepare a trial mixture using an asphalt content equal to or somewhat greater than (try to estimate amount that will be absorbed) the effective asphalt content determined in step 3.3.6 and also using the aggregate gradation as determined in step 3.3.7. Using a suitable technique, such as the test for maximum specific gravity of asphalt mixtures (AASHTO T 209), determine the actual quantity of asphalt absorbed (in percent, based on total weight of aggregate).

3.3.9 Determine the total asphalt content of the subject mixture by adding the effective asphalt content (from step 3.3.6) to the absorbed asphalt content (from step 3.3.8).

3.3.10 Follow the recommended procedure indicated in sections 6 and 7, using the total asphalt content for all subsequent computations and trials (from step 3.3.9).

4.0 <u>Void Capacity of Coarse Aggregate</u>

4.1 Use the following procedure to determine the vibrated unit weight and void capacity of the coarse aggregate fraction (material retained on a No. 8 sieve) of the proposed job-mix gradation.

4.1.1 Apparatus

Rammer--A portable electromagnetic vibrating rammer as shown in Figure 8, having a frequency of 3,600 cycles per min, suitable for use with 115-Vac. The rammer shall have a tamper foot and extension as shown in Figure 9.

Mold--A solid-wall metal cylinder with a detachable metal base plate and a detachable metal guide-reference bar as shown in Figure 10.

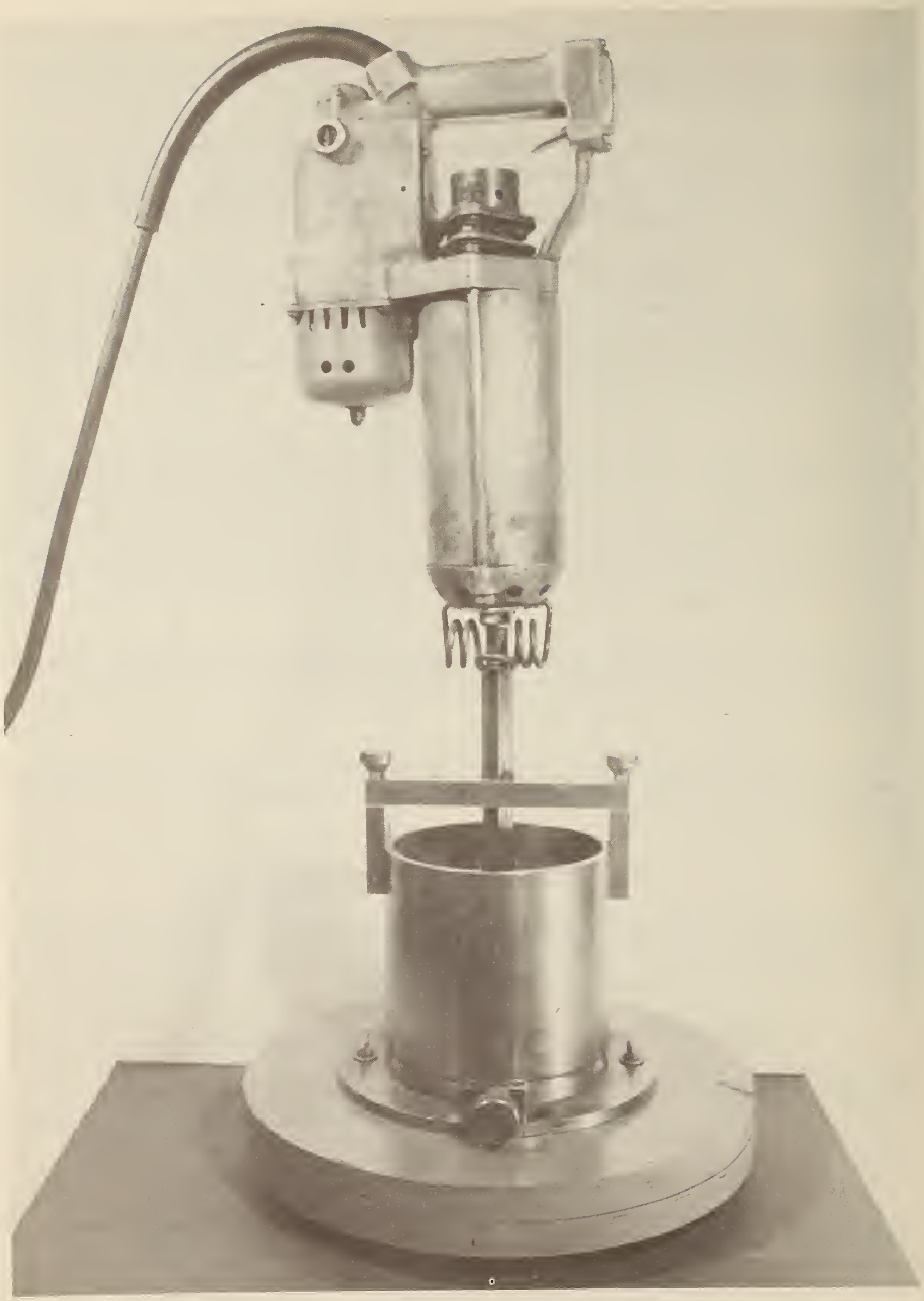


Figure 8 - FHWA Vibratory Compaction Apparatus

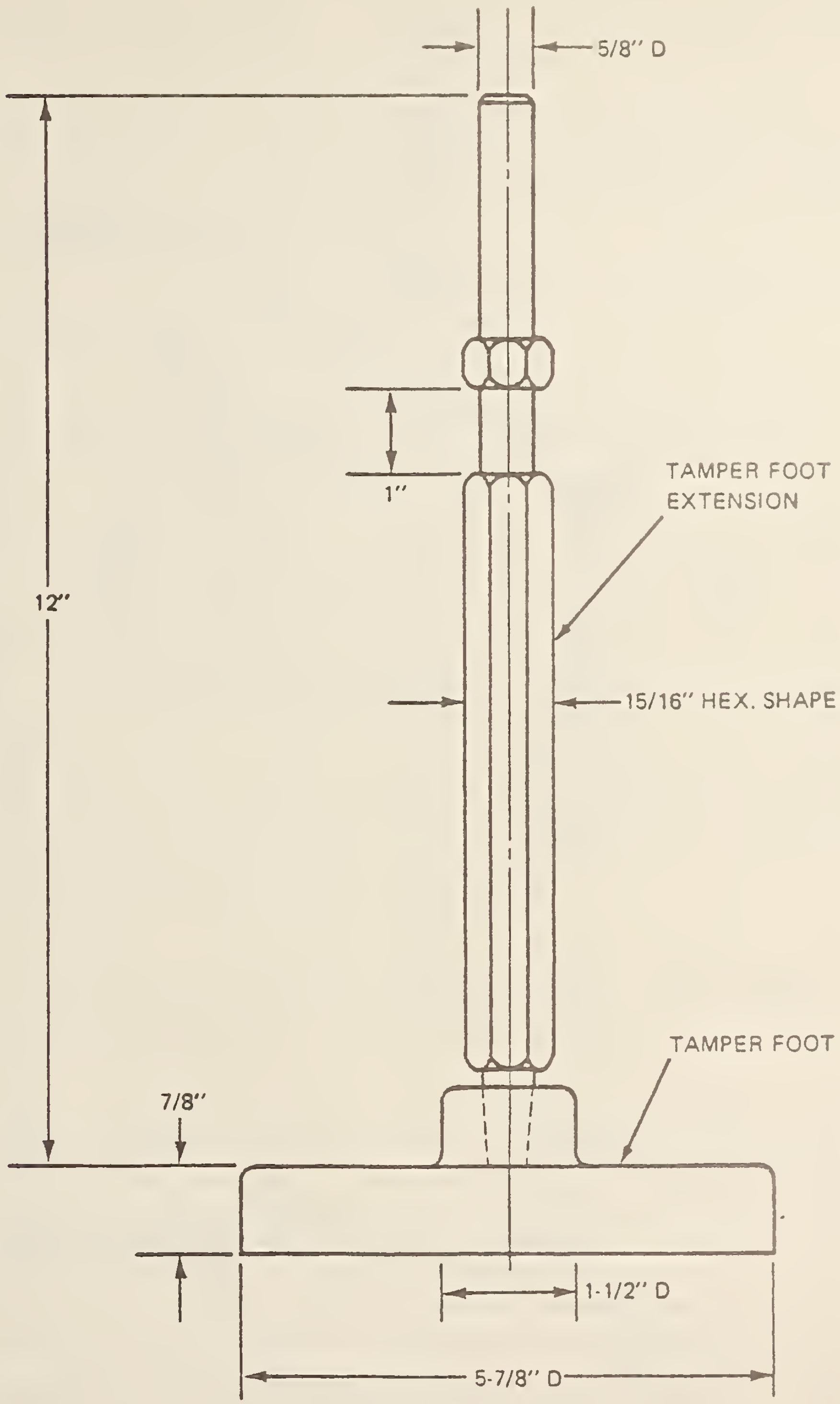


Figure 9 - Tamper Foot and Extension

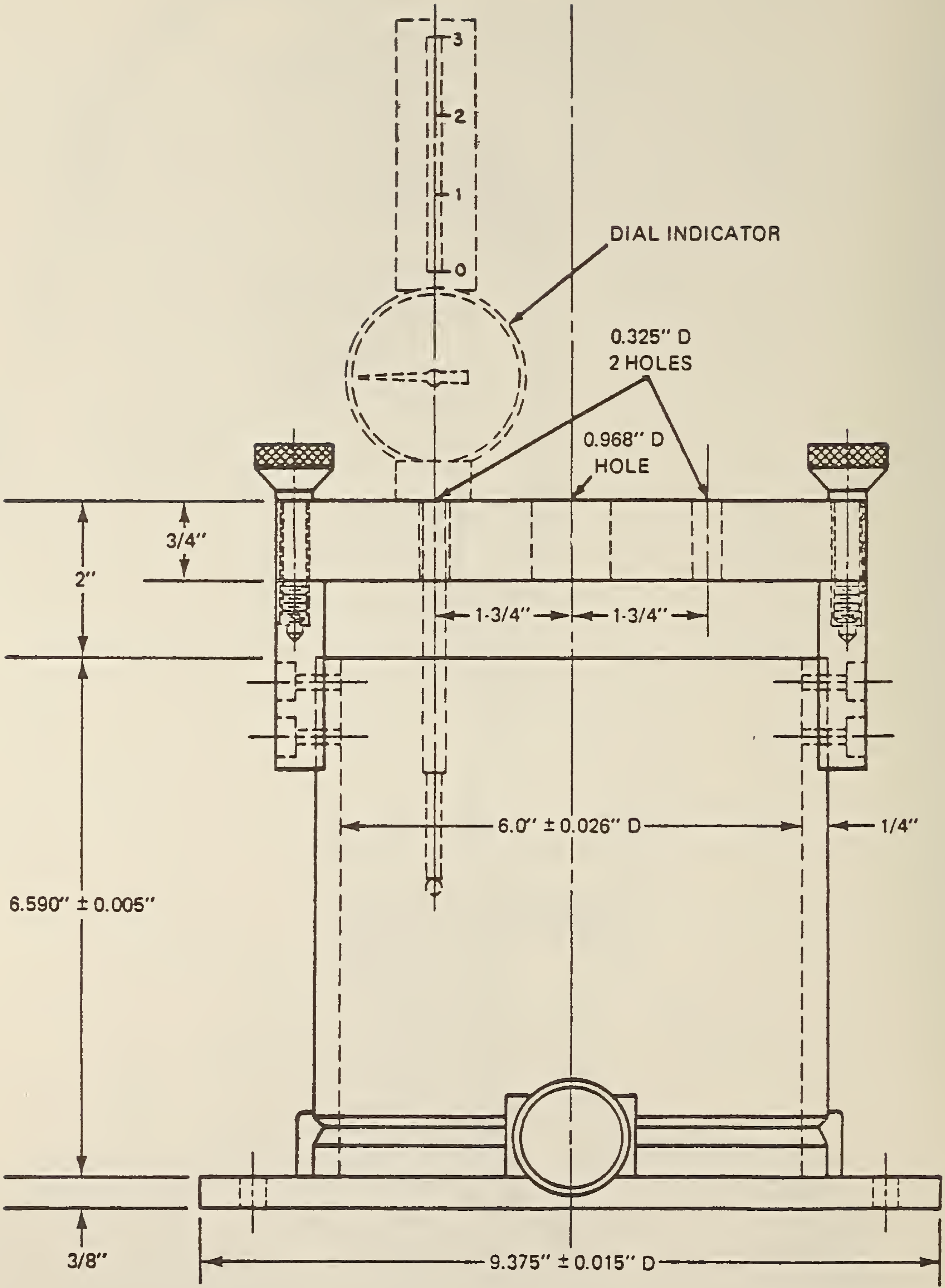


Figure 10 - Cylindrical Mold for Testing Granular Materials

Wooden Base--A plywood disc 15 in. in diam, 2 in. thick, with a cushion of rubber hose attached to the bottom. The disc shall be constructed so it can be firmly attached to the base plate of the compaction mold.

Timer--A stopwatch or other timing device graduated in divisions of 1.0 sec and accurate to 1.0 sec, and capable of timing the unit for up to 2 min. An electric timing device or electrical circuits to start and stop the vibratory rammer may be used.

Dial Indicator--A dial indicator graduated in 0.001-in. increments and having a travel range of 3.0 in.

4.1.2 Sample: Select a 5-lb sample of the coarse aggregate fraction from the proposed job-mix formula as verified in step 2.1.

4.1.3 Procedure

(a) Pour the selected sample into the compaction mold and place the tamper foot on the sample.

(b) Place the guide-reference bar over the shaft of the tamper foot and secure the bar to the mold with the thumb screws.

(c) Place the vibratory rammer on the shaft of the tamper foot and vibrate for 15 sec. During the vibration period, the operator must exert just enough pressure on the hammer to maintain contact between the sample and the tamper foot.

(d) Remove the vibratory rammer from the shaft of the tamper foot and brush any fines from the top of the tamper foot. Measure the thickness (t) of the compacted material to the nearest 0.001 in.

> Note: The thickness (t) of the compacted sample is determined by adding the dial reading, minus the thickness of the tamper foot, to the measured distance from the inside bottom of the mold and the end of the dial gauge when it is seated on the guide-reference bar with stem fully extended.

4.1.4 Calculations

Calculate the vibrated unit weight (X) as follows:

$$X = 6912(w)/\pi(d)^2 t(\mathrm{lb/ft}^3)$$

Where w = wt of coarse aggregate fraction (lb)

d = diam of compaction mold (in.)

If w = 5 lb and d = 6 in.:

$$X = 305.58/t (lb/ft^3)$$

where t is in inches

Determine the void capacity (VMA) as follows:

$$VMA = 100(1 - X/U_c) \text{ (in percent)}$$

where U_c = bulk solid unit weight (lb/ft^3) of the coarse aggregate fraction. U_c is calculated from bulk specific gravity, as determined in step 2.2, multiplied by 62.4 lb/ft^3.

5.0 Optimum Content of Fine Aggregate

5.1 Determine the optimum content of fine aggregate fraction using the following relationship:

$$Y = \left[[\% \text{ VMA} - V] - [(\% \text{ AC}) \ (X)/U_a] \right] \div \left[[(\% \text{ VMA} - V)/100] + [(X)/U_f] \right]$$

Where:	Y	=	Percent passing the No. 8 sieve (by weight)
	X	=	Actual vibrated unit weight of coarse aggregate (retained on the No. 8 sieve)
	U_f	=	Theoretical bulk dry solid unit weight of fine aggregate (passing the No. 8 sieve)
	U_a	=	Unit weight of asphalt cement
	%AC	=	Percent asphalt by total weight of aggregate ($2.0 K_c + 4.0$)
	V	=	Design percent air voids (15.0 percent)
	%VMA	=	Percent voids mineral aggregate of the coarse aggregate (retained on the No. 8 sieve), which is $100 - (100)(X)/U_c$
	U_c	=	Theoretical bulk dry solid unit weight of coarse aggregate (retained on the No. 8 sieve)

Note: X, U_a, U_c, and U_f are in pounds per cubic foot.

In the above relationship, asphalt absorption by aggregate has been assumed to negligible. Because asphalt absorption requirements are considered in the test for K_c (see step 3.1), the estimated air voids of 15 percent in the mixture will actually be greater by an amount equivalent to the volume of asphalt absorbed, in percent. This condition, provides, if anything, an additional safety factor.

As an alternative to the use of the mathematical relationship, one may use the design chart shown in Figure 11, provided that the assumptions used in designing the chart are satisfied; that is, the specific gravity values (bulk dry) for the coarse and fine aggregate fractions do not deviate beyond the limits of 2.600 to 2.700.

If the value thus obtained for fine aggregate content is greater than 15 percent, a value of 15.0 percent shall be used.

5.2 Compare the optimum fine aggregate content (Y) determined in step 5.1 to the amount passing the No. 8 sieve of the contractor's proposed job-mix formula. If these values differ by more than plus or minus 1 percentage point, reconstruct a revised ~~or adjusted~~ job-mix formula using the value determined for optimum fine aggregate content. Recompute the proportions of coarse and fine aggregates (as received) to meet the revised job-mix formula for submission to the contractor.

Note: If the proposed and revised job-mix gradations are significantly different, it may be necessary to rerun portions of this procedure.

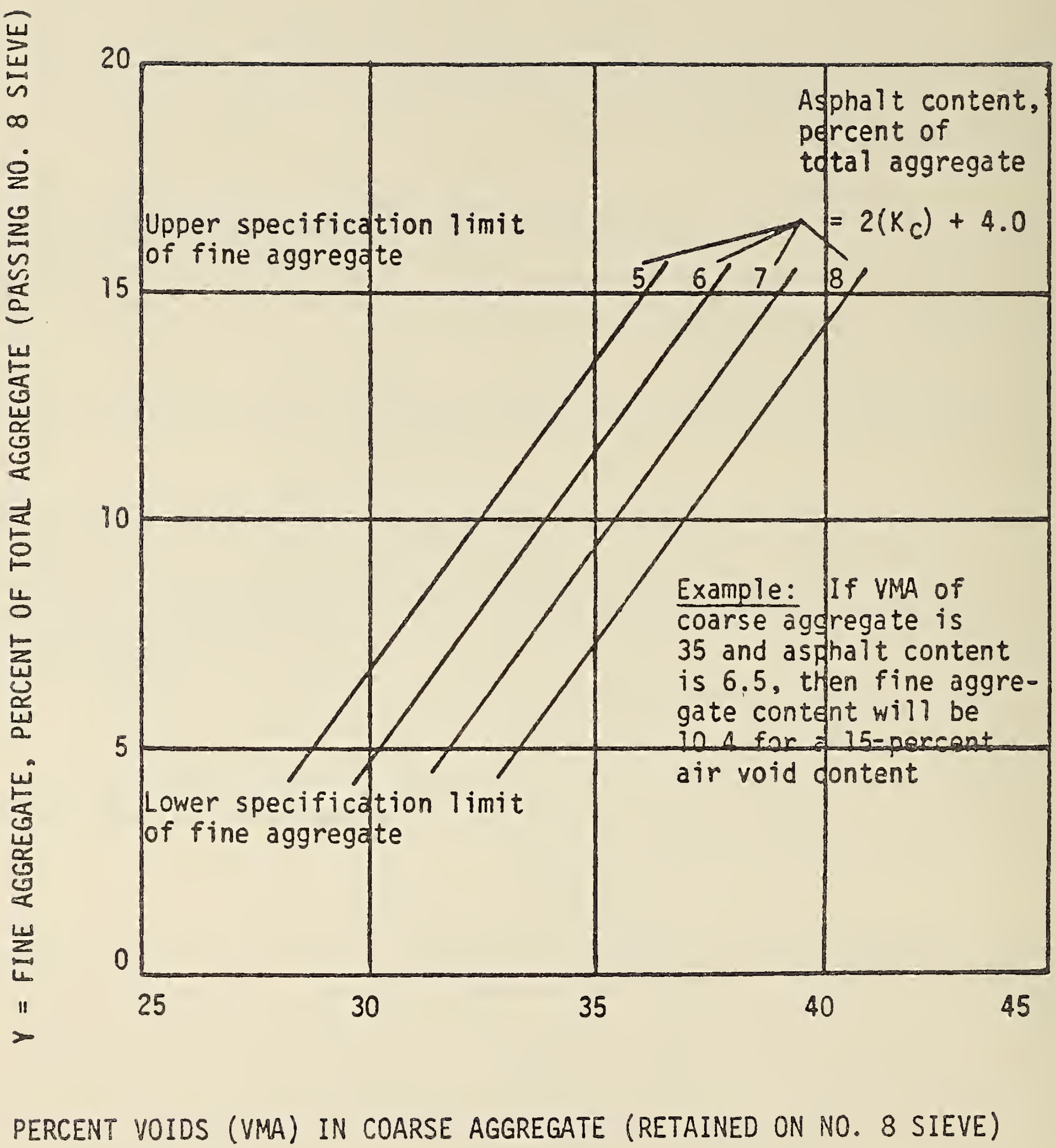
6.0 Optimum Mixing Temperature

6.1 Prepare a 1,000-g sample of aggregate in the proportions determined in section 5. Mix this sample at the asphalt content determined in step 3.2 at a temperature corresponding to an asphalt viscosity of 800 centistokes determined in step 2.3. When the mixture is completely coated, transfer it to a pyrex glass plate (8 to 9 in. diam) and spread the mixture with a minimum of manipulation. Return it to the oven at the mixing temperature. Observe the bottom of the plate after 15 and 60 min. A slight puddle at points of contact between aggregate and glass plate is suitable and desirable. Otherwise, repeat the test at a lower mixing temperature, or higher if necessary.

Note: If asphalt drainage occurs at a mixing temperature that is too low to provide for adequate drying of the aggregate, an asphalt of a higher grade should be used.

7.0 Resistance to Effects of Water

7.1 Conduct the Immersion-Compression Test (AASHTO T 165 and T 167) on the designed mixture. Prepare samples at the optimum mixing temperature determined in step 6.1. Use a molding pressure of 1,000 psi rather than the specified value of 3,000 psi.



<u>Assumptions Used in Deriving Chart:</u>

U_c = 165.4 lb/ft^3 (SG = 2.650)
U_f = 165.4 lb/ft^3 (SG = 2.650)
U_a = 62.4 lb/ft^3 (SG = 1.000)
V = 15.0 percent

Figure 11 - Determination of Optimum Fine Aggregate Content

After a four-day immersion at 120 F, the index of retained strength shall not be less than 50 percent unless otherwise permitted.

Note: Additives to promote adhesion that will provide adequate retained strength may be used when necessary.

8.0 Design Calculations

8.1 The following pages contain a convenient form for recording laboratory test data and performing design calculations.

REPORT ON OPEN-GRADED ASPHALT FRICTION COURSE DESIGN

1. AGGREGATES

A. Proposed Proportions (by weight)

B. Proposed Job-Mix Gradation

Sieve size	Specification limits				Job-mix blend
1/2 in.					
3/8 in.	95-100				
No. 4	30-50				
No. 8	5-15				
No. 16					
No. 200	2-5				

C. Specific Gravity--Unit Weight

	Apparent SG	Bulk SG (dry basis)	Bulk solid unit weight (lb/ft^3)
Coarse aggregate (retained on No. 8 sieve)			________ (U_c)
Fine aggregate (passing No. 8 sieve)			________ (U_f)
3/8 in. - No. 4 Sieve fraction			

D. Void Capacity of Coarse Aggregate

Unit weight (vibrated, lb/ft^3) = ____________________(X)

Voids mineral aggregate (%) = ____________________(VMA)

E. K_c Determination

Oil retention (g oil per 100 g aggregate) = __________

Oil retention (corrected, 2.65 SG) = __________

K_c (from chart) = __________

2. ASPHALT

A. Specific gravity--unit weight

Specific gravity at 77 F (25 C) = __________

Unit weight (lb/ft^3) = __________(U_a)

B. Viscosity--Temperature

Asphalt grade = __________

Temperature (°F)	Viscosity (centistokes)
290	________
275	________
260	________
245	________
230	________
215	________
Target: (-)	(700 - 900)

C. Asphalt Content (AC, %)

Percent asphalt (aggregate basis) =

$$(2K_c + 4) \times \frac{2.65}{\text{apparent specific gravity of coarse aggregate (3/8 in. to No. 4 sieve)}}$$

3. MIXTURE DESIGN

A. Optimum Fine Aggregate Content (Y)

Using: Formula __________ Chart __________

Where: X = __________ lb/ft^3 VMA = __________ %

U_f = __________ lb/ft^3 AC = __________ %

U_c = __________ lb/ft^3 V = __________ %

U_a = __________ lb/ft^3

Find: Y = __________ % (specs limit: $5 < Y < 15$)

Remarks:

B. Optimum Mixing Temperature

Temperature (°F)	Viscosity (centistokes)	Drainage	Use
________	________	________	___
________	________	________	___
________	________	________	___

C. Maximum Specific Gravity of Mixture (AASHTO T 209)

Specific gravity (vacuum saturation) = __________

Unit weight (vacuum saturation) = __________ lb/ft^3

D. Resistance to Effects of Water (AASHTO T 165 and T 167, 2000 psi)

Air dry strength (psi) = __________

Wet strength (psi) = __________ 4 days at 120 F

Retained strength (%) = __________ 50% minimum

Air voids (%) = __________ Bulk volume by dimensional measurement

Remarks:

4. DESIGN SUMMARY

A. Aggregate Proportions (by weight)

B. Job-Mix Gradation

Percent passing

Sieve size	Job-mix blend
1/2 in.	________
3/8 in.	________
No. 4	________
No. 8	________
No. 16	________
No. 200	________

C. <u>Asphalt Content</u>

Aggregate basis (%) = __________

Mixture basis (%) = __________

D. <u>Mixing Temperature</u>

Target value (°F) = __________

Range = __________

E. <u>Additives</u>

F. <u>Recommendations</u> Accepted _____ Rejected _____

APPENDIX B

TYPICAL SPECIFICATIONS FOR CONSTRUCTING AND MEASURING PAVEMENT GROOVES

This appendix presents a typical specification for constructing and measuring pavement grooves. The construction specification is used by the Louisiana Department of Highways and was presented in the April 1976 issue of the Federal Highway Administration publication, Highway Focus. The specification for groove measurement constitutes Pennsylvania Department of Transportation Test Method No. 629 entitled, "Method of Test for Measuring Grooves in Concrete Pavements with a Modified Tire Tread Depth Gauge." The description of the measurement method includes two illustrative examples of its use.

A. Construction Specification for Pavement Grooving

ITEM S-1 GROOVING: The surface of the existing portland cement concrete pavement shall be grooved at the locations shown on the plans and grooving shall conform to the requirements of the plans and these specifications.

Grooved areas shall begin and end at lines normal to the pavement center lane. The grooved area of each lane shall have a minimum width of 3 m (10 ft) and shall be centered within the lane width.

Grooving blades shall be 2.4 mm (0.095 in.) wide $\pm$ 0.08 mm ($\pm$ 0.003 in.) and shall be spaced 19.1 mm (3/4 in.) on centers. The grooves shall be cut not less than 4.8 mm (3/16 in.) nor more than 7.9 mm (5/16 in.) deep.

The actual grooved area of any selected 0.6 m (2 ft) by 30.5 m (100 ft) longitudinal area of pavement specified to be grooved shall be not less than 95 percent of the selected area. Any area within the selected area not grooved shall be due only to irregularities in the pavement surface and for no other reason.

Residue from grooving operations shall not be permitted to flow across shoulders or lanes occupied by public traffic or to flow across shoulders or lanes occupied by public traffic or to flow into gutters or other drainage facilities. Solid residue resulting from grooving operations shall be removed from pavement surfaces before such residue is blown by the action of traffic or wind.

The contractor shall make every effort to insure that noise levels generated by the combined grooving operation shall not be in excess of those levels normally generated by existing truck traffic.

Pavement grooving will be measured by the square meter. The quantity of pavement grooving to be paid for will be determined by multiplying the width of the grooved area by the total horizontal length of lane grooved.

The contract price per square yard for grooving existing concrete pavement shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals and for doing all work involved in grooving the existing concrete pavement, including removing residue, as shown on the plans, as specified in these special provisions, and as directed by the project engineer. Payment will be made under:

Item S-1, Grooving, per square meter.

B. Test Method for Measuring the Depth of Pavement Grooves

1. Scope

1.1 This method of test describes the procedure for sampling, preparing and measuring the depth of grooves in bridge decks, concrete pavements and ramps using a Modified Tire Tread Depth Gage.

1.2 This method of test can measure the depth of grooves in concrete pavements produced by the following methods: Tine Finish, Broom Finish and Pavement Grooving.

2. Apparatus

2.1 Tire Tread Depth Gage - A gage, calibrated in increments of 0.5 mm (1/32 in.) and capable of measuring to a depth of 13 mm (1/2 in.) shall be used. The gage end shall be modified to a shape suitable for the measurement.

2.2 Miscellaneous Equipment - Hand broom or brush, 0.3 m (12 in.) ruler, 31 m (100 ft.) tape measure and notebook.

3. Sampling procedure for securing test area.

3.1 The lot size for bridge decks shall be the length of the span by the width of the lane or lanes in one direction.

3.2 The lot size for concrete pavements shall be a minimum of 4,181 sq m (5,000 sq yards) to a maximum of 8,362 sq m (10,000 sq yards) in one direction. If the contractor's production is below the minimum, the lot size shall be the square yards of pavement placed.

3.3 The lot size for ramps or separate lanes connecting with cross streets shall be the square yards of pavement placed in one direction.

3.4 A lot shall consist of 5 approximately equal sublots.

3.5 Within each sublot, one test area shall be randomly secured in accordance with PTM No. 1.

4. Preparing the test area.

4.1 Brush all loose material from the area to be measured.

5. Measuring the depth of the grooves.

5.1 Measure ten grooves in a straight line perpendicular to the grooves, starting with the point that was randomly secured in Section 3.5.

5.2 Place the Tire Tread Depth Gage on the groove to be measured and firmly seat it to the surface. Make sure that the needle point will fall in the middle of the groove.

5.3 Depress the needle point and determine the depth by reading the scale attached to the gage.

5.4 Repeat the procedures described in Sections 5.2 and 5.3 for the nine remaining grooves.

6. Calculations

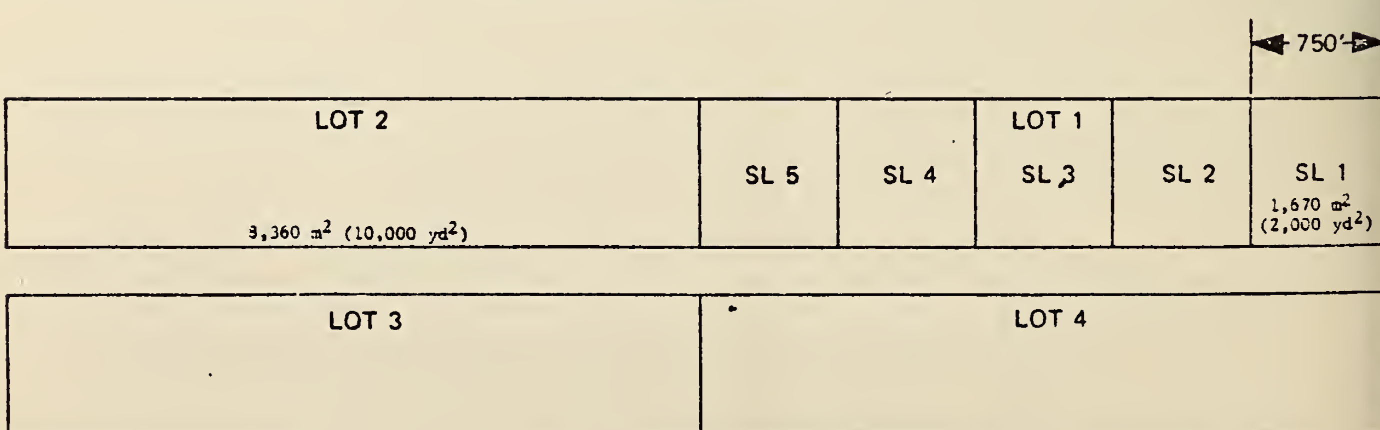
6.1 Calculate the average groove depth for each of the 5 sub-lots.

6.2 Calculate the average groove depth for the lot.

7. Report

7.1 The average groove depth for the lot shall be reported in increments of 0.5 mm (1/32 in.).

ILLUSTRATIVE EXAMPLE NO. 1



Assume a contractor places 33,400 sq m (40,000 sq yards) of separated highway consisting of reinforced cement concrete pavement 7.3 m (24 ft) wide on each side of a traffic separator.

In this case, the pavement can be divided into four lots. Each lot will have an area of 8,360 sq m (10,000 sq yards).

Each lot must then be divided into five approximately equal sublots. Each sublot will have an area of 1,670 sq m (2,000 sq yards) (229 m (750 ft) by 7.3 m (24 ft)).

Assume beginning station is 100 + 00.

Use Table 2 from PTM No. 1 to obtain random decimal fractions in the X and Y columns. These values shall be multiplied by the length and width of the lanes of each sublot to obtain the coordinates of the sample location measured from the starting point of each sublot.

Sublot #1

Coordinate X = 0.47 x 229 m = 107 m or 0.47 x 750 ft = 352.5 ft

Coordinate Y = R 0.20 x 7.3 m = R 1.5 m or R 0.20 x 24 ft = R 4.8 ft

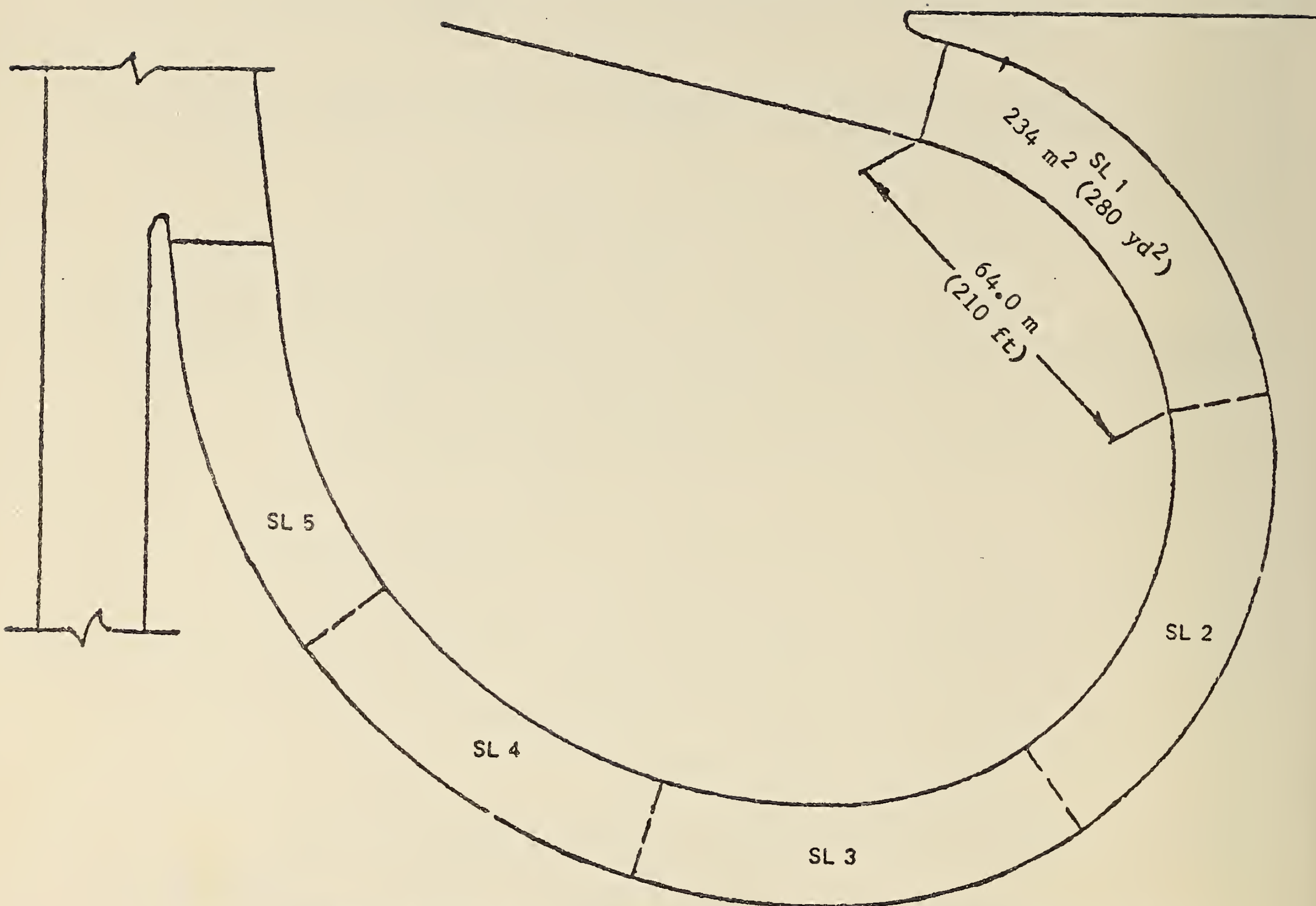
Sample Location = Sta. 100 + 00 plus 352.4 ft = Sta. 103 + 52.5

Measure 4.8 ft from Rt. edge of lane

Calculate the coordinates for the remaining sublots.

Be sure to go through Table 2 before using the same numbers over.

ILLUSTRATIVE EXAMPLE NO. 2



Assume a contractor places a ramp having an area of 1,400 square yards.

In this case, the area of the ramp is the lot size. The lot shall begin where the uniform width starts and end at a point with a uniform width.

The lot must then be divided into five approximately equal sublots. Each sublot will have an area of 234 m^2 (280 sq yards) (64.0 m (210 ft) by 3.7 m (12 ft)).

Measurements for the sublots and X coordinates shall be made along the inner edge.

The Y coordinate shall be measured on a line perpendicular to the sides of the ramp at the X coordinate point.

See ILLUSTRATIVE EXAMPLE NO. 1 for example of how to obtain the coordinates of the sample location.

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

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3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

ADDENDUM

The charts on pages 100 and 101 are not intended for direct application. These charts were developed based upon a specific set of assumptions listed on page 70. The reader should carefully evaluate all assumptions as they pertain to his individual situations. It should be also recognized that pavement friction demands are site specific and that generalized charts of this type are not.