CTR 3-6-86-490

IMPLICATION OF AGGREGATES IN THE CONSTRUCTION AND PERFORMANCE OF SEAL COAT PAVEMENT OVERLAYS

Mohamed-Asem Usman Abdul-Malak,
Alvin H. Meyer, and David W. Fowler

RESEARCH REPORT 490-2

Project 3-6-86-490

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

NOVEMBER 1991
REVISED: AUGUST 1992

Tec' [...] ige

1. Report No. FHWA/TX-96/490-2	2. Government Accession No.	3. Recip
4. Title and Subtitle IMPLICATION OF AGGREGATES IN THE CONSTRUCTION AND PERFORMANCE OF SEAL COAT PAVEMENT OVERLAYS		5. Report Date November 1991; Revised August 1992
		6. Performing Organization Code
7. Author(s) Mohamed-Asem Usman Abdul-Malak, Alvin H. Meyer, and David W. Fowler		8. Performing Organization Report No. Research Report 490-2
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, Texas 78705-2650		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Research Study 3-6-86-490
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Interim
		14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration
Research study title: "Strategic Research Plan for Achieving Adequate Pavement Friction"

16. Abstract

The ultimate aim of this research was to formulate statistical models that can be used for predicting the frictional performance of seal coat pavement overlays. The methodology involved establishing fifty-nine seal coat test sections in many parts of the State of Texas, including all four environmental regions, and monitoring their performances over time. Many factors, believed to have an influence on performance level and identified in the literature and Texas districts surveys, were considered in this study. These included aggregate properties, construction variables, traffic, and environment and weather variables.

Frictional performance was measured by a skid trailer and expressed as a friction number (FN). Eight sets of FN measurements, spanned over about five years, have been obtained which were used in the analysis. In addition, three rounds of British pendulum and sand patch testing were performed on most of the test sections. The data were used for correlation purposes with the FN and in the interpretation of the trends in frictional performance. Weather data relevant to the period prior to field testing were also collected.

Construction data were gathered which basically dealt with construction application rates and types of aggregates and asphalt. Aggregate samples obtained from construction sites were tested in the laboratory for their basic properties, polish susceptibility, resistance to weathering action, and resistance to abrasion and impact actions. Twenty of the samples were also examined for their mineralogical and petrographical properties. In this examination, the mineralogical constituents were estimated, and the textural characteristics were evaluated.

Correlations among the laboratory tests and among the field tests were studied. The performance data was graphed to detect the sources of variations and were grouped according to the different considered variables. The grouping gave insights into which variables controlled the observed differences in frictional performance. The grouping was followed by extensive statistical modeling which pinpointed the significant variables.

17. Key Words Statistical models, prediction, frictional performance, seal coat pavement overlays, friction number (FN), aggregate properties, construction variables, traffic, environment	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	512	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

IMPLICATION OF AGGREGATES IN THE CONSTRUCTION AND PERFORMANCE OF SEAL COAT PAVEMENT OVERLAYS

by

Mohamed-Asem Usman Abdul-Malak
Alvin H. Meyer
and
David W. Fowler

Research Report Number 490-2

Research Project 3-6-86-490
Implication of Aggregates in the Construction and Performance of Seal Coat Pavement Overlays

conducted for

Texas Department of Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

November 1991
Revised: August 1992

IMPLEMENTATION STATEMENT

The methods for predicting friction performance can be used for design purposes. It will be possible to predict friction numbers over the projected life of the seal coat pavements. These methods may be able to replace current procedures for qualifying aggregates.

Prepared in cooperation with the Texas Department of Transportation

DISCLAIMERS

Alvin H. Meyer, P.E. (Texas No. 31410)
David W. Fowler, P.E. (Texas No. 27859)
Research Supervisors

TABLE OF CONTENTS

IMPLEMENTATION STATEMENT iii
SUMMARY vii

CHAPTER 1. INTRODUCTION 1
1.1 BACKGROUND 1
1.2 OBJECTIVES OF THE STUDY 2
1.3 THE FIRST PHASE OF THE STUDY 3
1.4 SCOPE OF THE FIRST PHASE 4

CHAPTER 2. LITERATURE REVIEW 7
2.1 PURPOSE AND USES OF SEAL COATS 7
2.2 PARAMETERS OF FRICTIONAL RESISTANCE 8
2.3 VARIABILITY IN THE FRICTIONAL RESISTANCE 13
2.4 THE AGGREGATE PROPERTIES AFFECTING FRICTIONAL RESISTANCE 16
2.5 PREDICTION OF FRICTIONAL RESISTANCE FROM MATERIALS PROPERTIES 20
2.6 GUIDELINES FOR ACHIEVING AND MAINTAINING ADEQUATE SEAL COAT FRICTIONAL RESISTANCE 23

CHAPTER 3. SURVEY OF TEXAS DISTRICTS 29
3.1 PURPOSE 29
3.2 FINDINGS 29
3.3 OBSERVATIONS FROM OBTAINED FRICTION DATA 34
3.4 SUMMARY 37

CHAPTER 4. RESEARCH METHODOLOGY AND TEST SECTIONS 39
4.1 RESEARCH METHODOLOGY 39
4.2 TEST SECTIONS 40

CHAPTER 5. LABORATORY TESTS AND PETROGRAPHIC EXAMINATION 51
5.1 ENGINEERING LABORATORY TESTS AND RESULTS 51
5.2 PETROGRAPHIC EXAMINATIONS AND RESULTS 65
5.3 INTERRELATIONSHIPS AMONG TESTING METHODS 91

CHAPTER 6. FIELD-RELATED DATA 123
6.1 CONSTRUCTION DATA 123
6.2 TRAFFIC DATA 129
6.3 FIELD TESTING DATA 140

6.4 VISUAL CONDITION SURVEY DATA....163
6.5 WEATHER DATA....169

CHAPTER 7. AN EXAMINATION OF THE EFFECTS OF THE INVOLVED VARIABLES ON SEAL COAT FRICTIONAL PERFORMANCE....185
7.1 INTRODUCTION....185
7.2 PERFORMANCE VARIATIONS OBSERVED IN THE CONSTRUCTED REPLICATIONS....185
7.3 LEVELS OF PERFORMANCE EXPERIENCED BY THE DIFFERENT AGGREGATE MATERIALS....210
7.4 ISOLATED EFFECTS OF SOME VARIABLES OBSERVED IN TEST SECTIONS CONSTRUCTED END-TO-END....228
7.5 GROUPING OF DATA ACCORDING TO CLIMATIC REGIONS, CONSTRUCTION VARIABLES, AND AGGREGATE PROPERTIES....250
7.6 GROUPING OF FRICTION DATA ACCORDING TO MINERALOGICAL AND PETROGRAPHIC....276

CHAPTER 8. STATISTICAL MODELING OF SEAL COAT PERFORMANCE....286
8.1 DATA BASE AND STATISTICAL METHODS....286
8.2 VARIABLES INVOLVED....286
8.3 STATISTICAL ASSUMPTIONS....288
8.4 DESIGN OF THE EXPERIMENT FOR THE MAIN EFFECTS....289
8.5 MULTIVARIABLE REGRESSION ANALYSIS....293

CHAPTER 9. CONCLUSIONS....312
9.1 SUMMARY....312
9.2 CONCLUSIONS....313
9.3 CONTRIBUTIONS AND SIGNIFICANCE OF FINDINGS....317

APPENDIX A....319
APPENDIX B....373
APPENDIX C....415
APPENDIX D....437

REFERENCES....463

SUMMARY

The ultimate aim of this research was to formulate statistical models that can be used for predicting the frictional performance of seal coat pavement overlays. The methodology involved establishing fifty-nine seal coat test sections in many parts of the State of Texas, including all four environmental regions, and monitoring their performances over time. Many factors, believed to have an influence on performance level and identified in the literature and Texas districts surveys, were considered in this study. These included aggregate properties, construction variables, traffic, and environment and weather variables.

Frictional performance was measured by a skid trailer and expressed as a friction number (FN). Eight sets of FN measurements, spanned over about five years, have been obtained which were used in the analysis. In addition, three rounds of British pendulum and sand patch testing were performed on most of the test sections. The data were used for correlation purposes with the FN and in the interpretation of the trends in frictional performance. Weather data relevant to the period prior to field testing were also collected.

Construction data were gathered which basically dealt with construction application rates and types of aggregates and asphalt. Aggregate samples obtained from construction sites were tested in the laboratory for their basic properties, polish susceptibility, resistance to weathering action, and resistance to abrasion and impact actions. Twenty of the samples were also examined for their mineralogical and petrographical properties. In this examination, the mineralogical constituents were estimated, and the textural characteristics were evaluated.

Correlations among the laboratory tests and among the field tests were studied. The performance data was graphed to detect the sources of variations and were grouped according to the different considered variables. The grouping gave insights into which variables controlled the observed differences in frictional performance. The grouping was followed by extensive statistical modeling which pinpointed the significant variables.

CHAPTER 1
INTRODUCTION

1.1 Background

Wet weather skidding accidents and their consequences have long been a problem of increasing concern to highway engineers and researchers (12, 14). Many variables have been identified as important in causing such accidents. These include pavement frictional resistance, drainage properties of the surface, highway geometrics, vehicle speed and load, tire tread depth and inflation pressure, driver experience and rainfall intensity.

Since pavement frictional resistance has been recognized as being the primary factor (53, 71, 75), highway engineers are even faced with a more serious problem and a challenging task of designing and constructing long-lasting skid-resistant surfaces. The problem is merely what level of frictional resistance should be provided and how it can be maintained. It seems neither possible nor economically feasible, under most circumstances, to build in adequate frictional resistance for the design life of a pavement surface. Therefore, numerous maintenance techniques have been developed which are aimed at correcting for deterioration in surface friction. These range from modification of existing surfaces to the application of new surface friction courses.

The design and construction of surface friction courses to meet a desired level of frictional resistance requires a thorough knowledge and understanding of the complex and interrelated factors that influence the components of surface frictional resistance, surface microtexture and macrotexture. These factors have been identified to include the frictional characteristics of the coarse aggregates, construction design variables and practices, traffic volume, and environment.

Whether a friction course is built as a part of the initial construction or as a corrective measure required by a pavement management system, the use of a suitable nonpolishing aggregate in such a surface becomes a necessity if the surface is to maintain a highly desirable level of frictional resistance, the level being usually dependent on the traffic volume and speed limit expected.

In Texas, the Materials and Tests Division (D-9) of the State Department of Highways and Public Transportation (SDHPT) employs the polish value (PV) test (115) in which an aggregate is subjected to accelerated polishing for evaluating the polish susceptibility of coarse aggregates incorporated in pavement surfaces. The skid resistance test (8) is used by D-9 to measure the frictional resistance of pavement surfaces expressed as skid number, referred to in this report as friction number (FN). Minimum laboratory PVs of coarse aggregates have been established and in use in Texas for years for the purpose of providing acceptable pavement friction. Normally, high traffic volume roads require aggregates with high resistance to polish and wear while low traffic volume roads may operate with lower polish resistant aggregates. The current PV requirements based on average daily traffic (ADT) are as follows:

ADT	PV
Where specified in the plans	35
Greater than 5000	32
5000 to 2000	30
2000 to 750	28
Less than 750	No requirements

1.2 Objectives of the Study

The overall objective of this study, of which this research is the first phase, is to investigate and develop design criteria which will provide and maintain adequate pavement friction. Specifically, these objectives are to

1. develop a comprehensive, long-range strategic research plan which addresses all aspects of pavement friction and
2. investigate the relationship between laboratory frictional properties of coarse aggregates (i. e., PV) and frictional performances of roads built with these aggregates (FN).

While the second objective is included in the scope of achieving adequate pavement friction in the first objective, there is an immediate need to define, if possible, the relationship between PV and FN. Implicitly stated in the second objective is to investigate what predicts the friction number; the PV test by itself or a combination of laboratory tests, performed on the coarse aggregate, along with the construction design variables, traffic, and environment may quantitatively help in the formulation of models for predicting pavement frictional performance.

1.3 The First Phase of the Study

In general, providing skid resistant surfaces for highway pavements involves developing skid resistance design guidelines and incorporating these guidelines into the design of new pavements or into the process of maintaining and rehabilitating existing pavements. Since maintenance and rehabilitation of pavements become increasingly important as road systems grow and mature and fewer new roads are built, the efforts in this phase of the research were directed toward studying the frictional resistance of a pavement overlay construction method used in the rehabilitation of the huge present network of highways in Texas.

Many pavement rehabilitation methods (1, 41, 50, 55, 79, 81), including seal coat and hot mix asphalt concrete (HMAC) overlays, have been used in Texas for the purpose of improving the frictional resistance and other surface characteristics of the highways. In this research, the frictional resistance of seal coat overlays is being investigated. A seal coat overlay is a rehabilitation method for pavements of all classes, from low-volume roads to interstate highways, used mostly on rural highways. The construction of this method involves the application of asphalt to a roadway surface followed by the spreading of cover aggregate to form an overlay, which is generally less than one inch thick.

1.4 Scope of the First Phase

As sources of aggregates of known satisfactory frictional performance are being used up in many areas of Texas, ever-growing consumption will require that more use be made of aggregates that do not have a known performance record. Although existing laboratory tests can provide guidance in the assessment of polishing and wear properties of aggregates, quantitative information is needed on the relationship between the properties of aggregates and the performance of seal coat overlays built with them in a particular environment, in order that efficient use can be made of those aggregate sources without records of acceptable field performance. Also needed is a quantitative evaluation of the influence of construction design variables on the performance of seal coat surfaces. Progress in this area might lead to better construction practices which would make aggregate performance less critical.

The ultimate aim of this research has been to formulate statistical models for predicting the frictional resistance of the seal coat rehabilitation method in terms of factors hypothesized to have influence on either or both of the microtexture and macrotexture components of surface friction. Specifically, the attempt has been to:

1. relate the laboratory properties of aggregates used in seal coat construction to the frictional performance of this rehabilitation method and thus establish criteria for evaluating expected aggregate performance,
2. evaluate the effects of different construction spreading rates and gradation of aggregates on the frictional performance of seal coat overlays constructed with aggregates of common laboratory properties,
3. determine the influence of environment and other climatic variables on seal coat frictional performance, and
4. quantitatively study the interaction between traffic and the performance of various aggregate materials.

The models, if statistically feasible, will provide highway engineers with a tool by which the following may be made possible:

1. An optimization of aggregate use with regard to economics and conservation of resources through an improved procedure for selecting

aggregates for a given class of use, in a given environment of service, and for a given level of performance.

2. Prediction, at the design phase, of the life of seal coat overlays, and, consequently, more effective planning for seal coat rehabilitation projects.
3. A better understanding of the construction practices required for maintaining a desirable level of performance.

The objectives were achieved through the establishment of seal coat test sections in different parts of Texas. The test sections were constructed with various types of aggregate materials, in different climatic regions, using various construction application rates of asphalt and aggregates, and under a wide range of traffic volumes.

Chapter 2 is a review of the literature pertinent to the performance of seal coat surfaces, while Chapter 3 summarizes a survey of selected Texas districts that was intended to gather information on the current policies practiced and problems experienced by those districts concerning frictional performance of pavement surfaces.

Chapter 4 describes the methodology followed in this investigation and the considerations taken into account in the establishment of test sections and replications. Chapter 5 covers the laboratory tests and petrographic examination performed on the aggregate samples obtained from the construction sites of test sections, as well as the studied correlations among the major laboratory tests considered. Chapter 6 covers the collection of all field-related data. It includes sections on the collection of construction data, traffic data, field testing data and interdependencies among field tests, visual condition surveys, and weather data.

Chapter 7 includes graphical illustrations of the magnitude of variability in performance observed in the constructed replications. A large portion of this chapter deals with graphical presentations of the field testing results. Specifically, it covers the groupings of the performance of test sections constructed end-to-end as well as the groupings of the entire skid test data according to aggregate material, aggregate laboratory and petrographic properties, construction design variables, traffic levels, and environment. The various groupings represented graphical presentations of the experimental designs that were used in Chapter 8 for the formulation of prediction models. Chapter 8 covers the data base developed for this research, the types of data manipulation techniques used, the statistical methods involved, and the prediction models developed for the experimental designs studied in Chapter 7.

CHAPTER 2
LITERATURE REVIEW ON THE FRICTIONAL RESISTANCE OF SEAL COAT PAVEMENT SURFACES

2.1 Purpose and Uses of Seal Coats

A seal coat is an economical method for pavement rehabilitation which involves the application of asphalt and aggregate to an existing bituminous surface. The asphalt binder is sprayed uniformly across and along the road surface at a designed rate. The cover aggregate is then spread uniformly on top of the asphalt at a specified rate. The additional pavement thickness supplied by a seal coat is generally less than one inch, providing little increase in the load carrying capacity of a pavement section (133). However, successfully-placed seal coats applied to pavements showing signs of non-traffic-load-associated cracking have proven to somewhat improve the load carrying capacity by satisfactorily bridging the cracks and consequently altering the water content of the materials composing the pavement structure (122).

The main reason for using seal coats is to improve the frictional resistance of highway bituminous pavements, with the improvement being largely dependent on the frictional properties of the aggregate used and the quality of construction. Other purposes for which seal coats may be used (122, 131, 133) are to

1. enrich a raveled surface,
2. increase pavement visibility at night,
3. reduce tire noise,
4. improve demarcation of traffic lanes, and
5. attain a uniform appearing surface.

2.2 Parameters of Frictional Resistance: Microtexture and Macrotexture

The magnitude of frictional resistance developed between the tire and the pavement surface is generally controlled by the characteristics of the pavement surface. It can be explained by the behavior of the rubber as it rolls over the pavement surface. There are two components that make up the developed friction: adhesion and hysteresis (47). The adhesion component is generally considered to be the shear strength developed in the area of the actual contact of the rubber with the surfaces of aggregate particles, while the hysteresis component is caused by damping losses within the rubber when the latter is rolling over and around the aggregate particles. Among the many factors which affect the role of these components, the most important are the microtexture and macrotexture of the pavement surface (61, 101). The microtexture controls the adhesion component, while the macrotexture controls the hysteresis component.

In seal coats, the microtexture is the fine-scaled roughness contributed by individual small asperities on the individual coarse aggregate particles. Ideally, for the aggregate particle to sustain a highly favorable microtexture, it should be composed of hard, coarse, angular minerals well bonded into a softer matrix so that gradual differential wear will occur (23). The macrotexture is the large-scale texture at the surface caused by the size and shape of the coarse aggregate particles. Appropriate angularity and proper maximum size and gradation of aggregate particles are essential for achieving adequate macrotexture.

2.2.1 Relative Merits of Each

There had been conflicting claims on the relative merits of macrotexture and microtexture until Kummer and Meyer (77) proposed the classification of pavement surfaces shown in Fig. 2.1, which delineates the roles of microtexture and macrotexture in the generation of friction. The figure shows the excellent friction that may be attained by a fine textured gritty surface (high microtexture and low

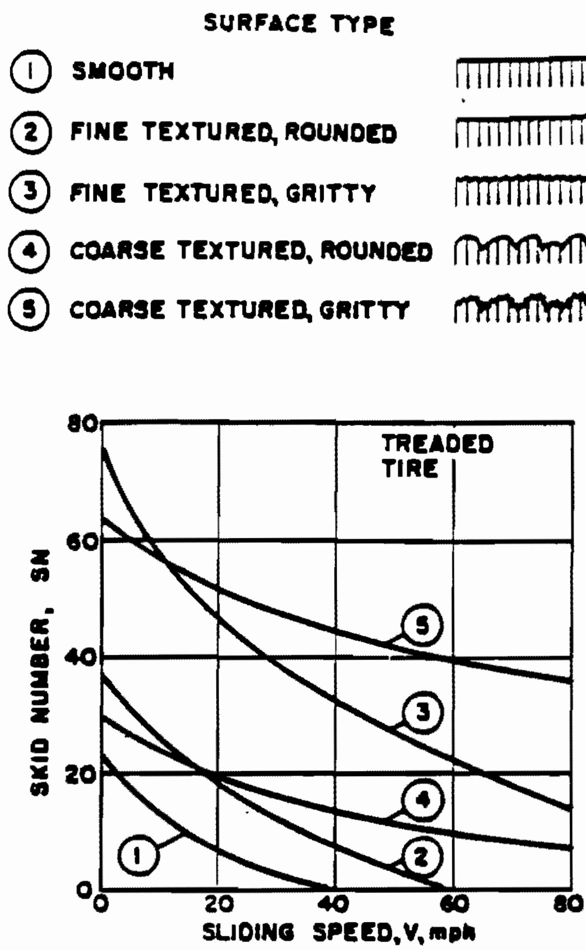


Fig. 2.1 Classification of Pavement Surfaces According to their Friction and Drainage Properties (77)

macrotexture) below 30 mph and the loss in friction above this speed. The figure also indicates that a coarse textured gritty surface (high microtexture and high macrotexture) attains high levels of friction at all speeds. Therefore, there seems to be an indication that both microtexture and macrotexture are equally important if enduring friction is to be maintained. However, it appears that the microtexture effect (adhesion) does interact with that of the macrotexture (hysteresis). This is because the macrotexture plays the role of providing drainage channels for water expulsion between the tire and the pavement surface, which allows the fine features (microtexture) to penetrate the remaining thin water films and thus permit an intimate contact between the tire and the roadway (43). This role becomes of even greater importance when it comes to providing adequate friction at high speed levels, as shown by comparing the friction of surfaces 3 and 5 in Fig. 2.1. Inadequate macrotexture drainage capability may cause the water depth on the pavement surface to increase. This may result in hydroplaning (104), which is a condition in which loss of contact between the tire and the pavement surface occurs.

In a study by Balmer and Hegman (10), skid tests, British pendulum microtexture measurements, and sand patch macrotexture measurements were conducted on 20 bituminous test pavements in west Virginia. The effects of microtexture and macrotexture on frictional resistance were separated and modeled. The friction model indicated that the FN increased rapidly as the sand patch average texture depth (ATD) increased to about 0.02 in. for constant values of British pendulum number (BPN). Although the FN increased above this ATD value, the rate of change was much smaller. Also, it was found that as the level of a constant BPN value increased, the relationship between FN and STD shifted upward.

2.2.2 Quantification Methods

2.2.2.1 Quantification of Microtexture. Microtexture depends largely on the mineral composition and the rugosity of the aggregates. The quantitative measure of microtexture is a very difficult problem. It is best done when the quantification method can evaluate the effect of the factors contributing to the role of microtexture and can give relative measures on the probable change in the microtexture during surface life.

The most commonly used laboratory method is the PV test (115) in which the friction of the coarse aggregate particles is evaluated with the British Portable Tester (BPT) after nine hours of exposure to accelerated polishing in the presence of water and abrasive grit. The PV is believed to represent the ultimate polish and wear that will be reached by an aggregate when it is placed in field service.

The four-cycle magnesium sulfate soundness (MSS) test (110) used to judge the soundness of an aggregate when it is subjected to weathering action (freeze and thaw) gives indications of the strength (or softness) of the cementing matrix that holds the individual grains in the aggregate particles. If the cementing matrix is strong (reflected by low soundness loss) the individual grains will be expected to be tightly held and thus polished by traffic, resulting in an overall low frictional resistance. On the other hand, if the hardness of the cementing matrix is lower (if the matrix is made up of softer minerals) than that of the minerals bonded into it in such a way that a higher but still reasonably acceptable soundness loss will result, the microtexture of an aggregate may be expected to wear differentially under traffic, resulting in a continually renewed non-polished surface.

Other tests of significance in characterizing microtexture and the degree of retention of such microtexture during service life are the various petrographic tests which describe the aggregate and its mineralogical composition (12, 23). This analysis may prove very helpful if the combinations of the results of such tests (quantitative and qualitative results) can be presented in a way that they can be correlated with field performance (31, 91, 116).

2.2.2.2 Quantification Methods of Macrotexture. Several methods have been developed for evaluating or measuring pavement macrotexture (46, 56, 59, 63, 68, 69, 93, 97, 100, 121). Of these, the sand patch and the silicone putty (volumetric measurements), outflow meter (drainage measurement), and stereo photographic interpretation (topography measurement) methods are most commonly used (10, 11, 24). However, since most of these procedures measure only a single attribute, such as depth or drainage characteristics, the correlation of frictional resistance with these measurements is often imperfect (43, 56). In addition, the simplicity, repeatability, and reproducibility are factors affecting the decision to implement any of these methods.

Although the sand patch method is perhaps the most widely used for measuring macrotexture (17, 94), it is difficult to use in wet conditions and has poor reproducibility. Recent work (139) using small glass beads to replace sand has been undertaken by the American Society of Testing and Materials (ASTM), and the findings of the early efforts indicate an improvement in the reproducibility of the test when the work is done in damp conditions.

The Schonfeld method (100) for measuring both microtexture and macrotexture by means of stereophotographs has been used by several researchers, and the photointerpreted values have been found to correlate highly with skid trailer values. While the original method uses visual stereo-interpretation to classify the texture of the pavement surface, a study by Howerter and Rudd (123) demonstrated that the method can be automated through the use of electronic stereophotogrammetic techniques coupled with computer processing. The automation has the effect of removing the human subjectivity associated with visual stereointerpretation and leads to a more efficient implementation of the method.

Most of these texture measurement methods are slow and involve stationary procedures. Other methods, which measure macrotexture from a moving vehicle, have been developed, and some of these have now evolved into workable systems. Examples of these are the laser sensor methods now being used in Europe. The "Numerisateur", a contactless sensor based system, is in use in France (103). In this method, contactless sensor-measured heights taken at 0.5 mm to 1 mm centers are stored so that a three-dimensional model of the surface can be built up. The stored data are then used to calculate texture depths. In the United Kingdom, the Transport and Road Research Laboratory (TRRL) Mini Texture Meter, a laser based monitoring system, is currently the accepted method for measuring the macrotexture of new asphaltic roads and surface dressings (seal coats) (103). The method has many advantages over the sand patch test, including high correlation with the standard sand patch procedure, a high degree of repeatability, speed of operation, and the ability to work in damp conditions and on warm surfaces. This last aspect has made possible the use of the method as a control tool during construction operations.

In the United States, two indirect methods for measuring texture from a moving vehicle have been evaluated (162). The first method is based on the depolarization of light reflected from a surface whereby the degree of depolarization is a function of the texture. The depolarized light number (DPN), which is the output of the depolarized light system, was found to increase with increasing macrotexture measurements and increasing skid resistance. However, the coefficients of determination of the linear regressions between the DPN and the texture data were too low to provide reliable prediction of texture. The other method is based on the use of skid resistance data measured with blank and ribbed test tires. Two-way regressions with ribbed and blank tire data and macrotexture (determined by the sand patch test) and microtexture (determined by the BPT) produced highly significant results. The procedure was thus recommended for implementation using a two-wheeled tester equipped with a blank tire on one side and a ribbed tire on the other.

2.3 Variability in the Frictional Resistance

In the United States, the frictional resistance of highway pavement surfaces (including seal coat surfaces) is generally measured with the locked-wheel skid test standardized in ASTM Designation E274 (8). In this method, the friction force on a locked test wheel equipped with a standard test tire, in accordance with ASTM E249, is measured as the locked wheel is dragged over a wetted pavement surface at constant speed (usually 40 mph) and under constant load. The results are expressed as skid numbers (referred to in this report as FNs) that indicate the relative safety of pavements under wet conditions. Variations in the measured friction may occur due to many factors, including tester calibration techniques and operating procedures, lateral positioning of test trailer on the surface and longitudinal uniformity of the surface (45, 80, 124), and long-term and short-term seasonal changes.

2.3.1 Variability Caused by Friction Test Calibration and Operating Procedures

In a study undertaken by the National Cooperative Highway Research Program (NCHRP) (124), the sources of error in skid testing were identified and their magnitudes determined. Recommendations were made for improving skid-testing equipment and the calibration, operation, and data evaluation procedures. The corrective measures formulated in this study involved stressing the need for high-quality instrumentation, recommending a standardized pavement watering method and calibration procedures, and lengthening skid tests to permit evaluation of longer portions of the skid trace, more precise methods of evaluation, and recognition of the statistical uncertainly associated with the data. Through the incorporation of these corrective measures into ASTM E274, the magnitudes of equipment and measurement errors have been minimized.

2.3.2 Variability Caused by Lateral and Longitudinal Changes in Test Surface

The testing for frictional resistance is normally done in the center of the left wheel path (8). The lateral positioning of the test trailer within the path width and the longitudinal uniformity of the test surface along the path have been reported as major sources of variability in measured FNs. An FN difference as high as five between the center line and either edge was indicated by the NCHRP study (124). Errors of this type can be minimized by making the drivers aware of the problem. Since at least five friction measurements are made in a test section, the arithmetic average is expected to represent the average roadway condition and thus take into account any non-uniformity in the test surface. However, if statistical or other criteria applied to the FN for a long test section indicate that it cannot be considered to be uniform, the section is treated as two or more sections (8).

2.3.3 Variability Caused by Seasonal Changes

It has been recognized for many years that pavement surface characteristics undergo seasonal changes which cause variations in their frictional properties. Two types of seasonally-caused variations have been observed: long-term and short-term. It has been determined that the long-term variations are caused by changes in the microtexture of the exposed aggregates brought about by polishing during long, dry periods and roughening caused by the rejuvenating effects of long wet periods (25). As a result of this mechanism, friction measurements made in the wet periods have been reported to be much higher than those made in the dry periods. In a five-year study by the Pennsylvania Department of Transportation, summarized by Rice (96), it was found that the rejuvenating effects tended to offset the polishing effects in that the curves of frictional performance for the last three years showed no consistent upward or downward trends for the annual minimum levels. Similar observations on this tendency are made in a summary of six-years of research work done in the Federal Republic of Germany recently reported (103) and from many other studies (18, 19, 26, 35, 117, 120). In one of those studies (26), it was reported that the stabilization of the minimum skid number after two years of pavement exposure to traffic was irrespective of the level or volume of traffic.

In the United States, long-term seasonal variations as high as 30 friction numbers were reported in Kansas, with more typical variations in the order of 5 to 15. In Texas, a study of four aggregate types used in seal coats showed average seasonal decreases (wet to dry) of approximately 10 friction numbers (118). A study in Kentucky (15) indicated that the frictional resistance exhibited an annual sinusoidal cycle similar to the annual precipitation and temperature cycles. In fact, correlations between changes in frictional resistance and temperature suggested that the annual changes in friction resulted from a reaction of the surface to temperature over a period of a few weeks (four and eight-week periods prior to the date of test). In addition, when test sections at the same location were compared, the magnitude of the annual variation in frictional resistance was found to be strongly associated with volume of traffic. In a report by NCHRP (101), this magnitude was reported to differ with type of aggregate, with softer materials such as limestone responding more strongly. Superimposed on these long-term (annual) variations are short-term variations attributable to external factors, such as amount and timing of intermittent rainfall, and possibly to contaminations from oily films, drippings, and other deposits on the surface (57, 65).

In the systematic identification of the levels of frictional resistance, occurrence of these variations makes it difficult to precisely evaluate surfacing materials and practices and to take corrective measures. As an interim measure, it was suggested in Pennsylvania and Kentucky that taking of friction measurements be confined to a period between the first of July and the middle of November, during which the friction is minimum and therefore most critical. However, since it would be difficult, if not impossible, to conduct all inventory surveys of frictional resistance in such a short period, it would be desirable to develop procedures that can be applied to make needed adjustments in measured skid numbers. In this respect, Pennsylvania State University has recently done considerable work (60, 66, 98) in which two models, a generalized prediction model and a mechanistic model, were developed to predict seasonal variations in the skid resistance of asphalt pavements due to rainfall conditions, temperature effects, and time of the year. It has been suggested that the models be limited to the geographical area within which the investigation was conducted.

2.4 The Aggregate Properties Affecting Frictional Resistance

Several characteristics should be evaluated in the selection of aggregates for frictional resistant surfaces. For seal coats, the microtexture and the angularity and gradation of the individual coarse aggregate particles composing the surface structure are the main variables controlling the frictional resistance.

2.4.1 Angularity and Gradation

It has long been recognized by paving and construction researchers and engineers that aggregate angularity (shape) and gradation (size) have an important influence on pavement frictional performance (20, 21, 34, 39, 72). The angularity of coarse aggregates contributes to tire-pavement friction in the case of seal coats by establishing points of contact with the tire rubber which protrude above the water level. The contribution lasts as long as the aggregates remain angular. Angularity relates to the rock crushing process, but the retention of angularity depends on such characteristics as mineralogical composition and the amount of polish-wear produced by traffic. In the case of seal coats, the sizes of the aggregate particles in the final surface are a very important consideration in terms of angular projections to different heights so as to provide rubber envelopment and drainage patterns. Variance in maximum aggregate size, which is possible even from the same aggregate source, may result in different frictional responses under equivalent traffic exposure (12).

2.4.2 Microtexture

The microtexture or roughness of the coarse aggregate particles in seal coats is a very important characteristic in terms of frictional resistance, but of greater importance is the change in such texture during the service life of seal coats. Fortunately, at this stage of development, there appears to be an agreement on the basic requirement for a satisfactory aggregate microtexture. That is, an aggregate should be composed of sand-size hard grains weakly cemented in a soft matrix so that it will wear differentially under traffic and expose a continually renewed nonpolished surface. On the other hand, if the cementing matrix is strong, the individual grains will be tightly held and consequently polished by traffic. The rate of polish depends on the hardness of the grains, the frequency of contacts with traffic, and the type of abrasive material on the roadway surface(12). The characteristics that describe, to some extent, the microtexture qualities of an aggregate include polish and wear resistance, strength and toughness, and resistance to weathering. Discussions of these characteristics and their evaluation methods follow.

2.4.2.1 Polish and Wear Resistance. Polishing may be defined as reduction of microtexture whereas wear is the loss of macrotexture. Researchers have found that the two phenomena are not strictly separable (21, 22, 73). That is, wear is promoted by the presence of abrasive material. The finer the particles constituting this material are, the smoother the resulting wear surface and the slower the wear rate will be. Consequently, for similar materials, slower wear means more polishing of the exposed mineral grains.

During the past two decades, several polish susceptibility tests have been proposed and used by various investigators (48, 49, 58, 67, 85, 86, 87, 88, 90, 92). However, progress towards acceptance and universal use of these methods has been slow because many questions remain unanswered. Considerable work has been done in Texas on the polishing properties of coarse aggregates (118). The results of this work led to the adoption of the PV test for ranking aggregates with respect to their frictional characteristics. However, recognizing that some low PV aggregates had exhibited satisfactory performance, experience with the field frictional performance was substituted for the PV requirements.

The term *petrography* is used to refer to mineral composition; constituent mineral hardness; mineral grain size, shape, and distribution; grain interlocking; and mineral susceptibility to chemical attack and alteration. Most of these properties were found to have been repeatedly mentioned in the literature as important in terms of resistance to polish and wear (9, 12, 23, 27, 40, 76, 91). Dahir and Mullen (23) found that there exists an optimum proportion of hard to soft mineral content that produces satisfactory and lasting polish and wear properties as evaluated in the laboratory by using the circular track and jar mill methods. The optimum proportion seemed to fall in the range of 50 to 70 percent of hard minerals to 30 to 50 percent of soft minerals. In their study and in many others, it was found that the coarser and the more angular the hard grains were and the more uniform their distribution was in the softer bonding matrix, the more likely that a differential wear would occur in the aggregate. It has been also repeatedly reported in the literature that the petrographic tests may prove very useful in the selection of aggregates if such test or combination of test results are correlated with field performance (31, 91, 116).

In the case of carbonate aggregates, which generally polish very rapidly, the acid insoluble residue test (6) has been explored and currently adopted by many highway agencies. Several investigators found that the amount of sand-sized insoluble residue, the residue gradation, and the total amount of insoluble residue are significant factors in determining the polish susceptibility of carbonate aggregates, with the sand-sized residue tending to be more significant than the total residue (23, 118).

2.4.2.2 Strength and Toughness. In highway pavement surfaces, particularly in seal coats, the coarse aggregate is in direct contact with the tire. The aggregate is thus subject to forces of shear, abrasion, and impact. These forces may break up the aggregate, altering its gradation, and they may abrade the aggregate particles, reducing their texture. Therefore, the aggregate should provide the mechanical stability and strength to resist these forces over the surface life. The most commonly used test for resistance to abrasion and impact is the Los Angeles abrasion test (109). Typically, not more than 40 percent loss is permitted when this test is used for surface aggregates.

2.4.2.3 Resistance to Weathering. Resistance to degradation by weathering actions is a major factor affecting the wear rate of aggregates (20, 71, 101). The MSS test (110) is often used in determining this resistance. The test gives a useful indication of the expected aggregate resistance to freeze-thaw and salt recrystallization effects. As indicated earlier, the test is also indicative of the strength and hardness of the cementing matrix that holds the crystal grains of aggregate particles together. As with the many other tests used for determining the suitability of aggregate for incorporation in pavement work, the test does not always appear to be reliable in distinguishing poor from good aggregate performers (89). Recognizing this fact, some agencies use performance history along with the information furnished by the test when judging the soundness of aggregates. Another method currently intended for use only in synthetic aggregates is the freeze-thaw test (112).

2.5 Prediction of Frictional Resistance from Materials Properties

After the dependence of frictional resistance on the polishing and wear properties of aggregates had been established, several researchers attempted to predict the frictional resistance from these aggregate properties. A major advancement in this field was achieved in a study by the British TRRL in 1972. In this study, summarized by Salt (99), a regression analysis was carried out to relate the polished stone value (PSV) and traffic simultaneously to the side friction coefficient measured at 50 km/h (SFC_{50}). One hundred and thirty-nine different sections of bituminous surfaces (including seal coat surfaces) with traffic volumes of up to 4000 commercial vehicles per day were examined for the purpose of the investigation. The following highly significant relationship (R^2 =0.83) was obtained:

$$SFC_{50} = 0.024 - 0.663 \times 10^{-4}\, q_{cv} + 1 \times 10^{-2}\, PSV$$

where

q_{cv} = flow of commercial vehicles per lane per day (in one direction)

The relationship applies only to tangent sections.

The publication of this finding has been regarded as a major advancement in the field of frictional resistance as it provides a method for nominating at the design stage the properties of the aggregate required to provide a given ultimate frictional resistance provided that the traffic can be estimated. It is believed that the high significance obtained by the TRRL model was due to the use of a low speed measure of skid resistance, SFC, which depends primarily on surface microtexture (140).

In North America, almost all research concerned with the prediction of skid resistance has been on the correlation of friction measured by the ASTM Standard E274 locked wheel skid trailer with laboratory aggregate polishing values. Mullen (87) established a method which allows prediction from laboratory tests of maximum field polish that may be anticipated for a given pavement mixture (open-graded and dense-surface mixes). First, usable correlations were found between field British portable tester measurements, British pendulum numbers (BPN), and skid trailer measurements (FN) at different test speeds. These correlations were used to draw BPN-FN-velocity nomograph for the types of mixes investigated. Second, field wear versus laboratory wear correlation was attempted by coring pavements after field testing and then polishing the cores to terminal polish in the circular-track machine. Then, the unworn portions of the field cores were remixed and molded into laboratory specimens and later polished in the circular track machine to obtain the full "as new" polish curve. The new and worn polish curves when compared gave the extent of circular-track wear experienced in the field. An upper limit for field wear equivalent to three hours or less of machine wear was therefore established. The established limit, in terms of hours, may then be used to obtain the laboratory BPN associated with it, and the BPN when entered in the BPN-FN-velocity nomographs will give the predicted FN values for the different velocity values.

In a study by Dahir et al (26), correlations between the results of laboratory polishing tests on aggregate panels and FNs were poor with correlation coefficients ranging from 0.55 to 0.65. The laboratory tests were the Penn State rotary drum polishing machine (RDM), the modified Penn State reciprocating pavement polisher (RPP), and a modified small drum machine (SDM) used for aggregate wear at Penn State. In the RDM and RPP methods, the frictional resistance was measured by the BPT, while in the SDM method the average frictional resistance of ten aggregate particles was measured electronically.

A skid resistance model appropriate for asphaltic concrete mixes was developed in Ontario, Canada, by Heaton et al (140). The model contained parameters associated with the ability of the mixes to resist consolidation under traffic as well as those contained in the TRRL model. The multiple regression model with a multiple coefficient of correlation (R^2) of 0.86 is shown below:

$$SN_{100} = (0.17 \times PSV) + (1.7 \times MS) + (3.6 \times FLOW) + (0.9 \times VOID) - (0.24 \times EQT) - 9$$

where

SN_{100}	=	skid number at 100 km/h,
PSV	=	polish stone value,
MS	=	Marshall Stability,

FLOW	=	flow of the mix,
VOID	=	void content in the mix, and
EQT	=	an equivalent traffic factor

The model was based on 56 independent site cases, all of dense-graded asphaltic concrete of 12 mm maximum aggregate size. The cases covered two of the more widely used aggregates in Ontario, limestone and traprock with low and medium PSV, 41 and 45, respectively, and two aggregates of high PSV, blast furnace slag and steel slag, 55 and 59, respectively.

Refinement of this model continued with the analysis of other site cases and extended to include a wider range of mix compositions and aggregate types (35). As a result, improved predictive models have been developed for various traffic volumes and surface types. The work has confirmed the overall importance of mix designs in achieving desired skid resistance with accumulated traffic influences, particularly in preventing coarse aggregate immersion due to traffic compaction. High stability mixes have proven most suitable, and coarse aggregate properties such as PSV and Los Angeles abrasion value are of secondary importance once adequate levels are provided. Continued monitoring of the site cases showed that the SN_{100} values have leveled off rather than continuing to decrease at a reduced rate, which the predictive model could not describe. This pertained to weathering influences which appeared to be regenerating microtexture at about the same rate that traffic polishing is involved.

In Texas, several researchers have attempted to formulate relationships similar to those discussed above (33, 39, 125, 126); however, reliable relationships could not be established. In a study by Elmore and Hankins (33), it was found that a relationship does exist between the ultimate PV of the aggregate reached in the laboratory and the stable value of skid resistance reached after exposure to traffic. However, one problem with that relationship was that the friction numbers used were those predicted to represent the friction level at 1 x 10^6 traffic applications from traffic-friction regression equations which had poor prediction ability (R^2s for the traffic-friction regressions for all aggregate types investigated were too low). The poor prediction ability was due to the variability found in the measured skid numbers. The variability was indicated to have been probably caused by the effects of seasonal and climatological changes and the effect of different construction techniques. Therefore, it was suggested that the results of the study not be implemented.

In a more recent study (13), which involved aggregates used in many types of pavement surfaces, relationships were obtained between skid number, cumulative traffic per lane, and aggregate properties (PV and Los Angeles abrasion tests). The all-pavement-type model was based on about 600 observations and had an R^2 of 0.40. The model for seal coats was based on about 150 observations and had an R^2 of 0.32. The low predictive ability of the models can probably be attributed to the exclusion of variables such as macrotexture and seasonal variations.

2.6 Guidelines for Achieving and Maintaining Adequate Seal Coat Frictional Resistance

Adequate frictional performance of seal coats is achieved mainly by the selection of satisfactory aggregates, by the use of properly designed application rates of asphalt and aggregate, and by assuring a careful quality control of construction operations. Guidelines related to each of these factors are discussed as follows.

2.6.1 Selection of Aggregates

It has been accepted worldwide that, for a natural aggregate to have high, prolonged frictional resistance, it should be comprised of sand-size hard grains weakly bonded in a softer matrix so that differential wear in the aggregate occurs. Generally, it has been proved that sandstone aggregates with high PVs have high, long-lasting frictional resistance, whereas carbonate aggregates with low PVs, such as limestones and dolomites and some siliceous gravels lose their initial frictional resistance rapidly under traffic exposure (28, 29, 36, 84). On the other hand, synthetic aggregates, particularly lightweight aggregates, have proven to be highly superior to most natural aggregates in terms of maintaining comparatively highly favorable frictional resistance (37, 38, 42, 70, 101).

In selecting the aggregate to be used in a seal coat, attention is paid to the level of friction to be maintained on the roadway. The level of friction is, in turn, decided upon in view of the estimated traffic volumes and speed limits (32, 44, 55, 102, 104); the roadway features, such as hills, curves, and intersections, which require unusual vehicle maneuvering (83, 102, 104); and the pavement's drainage capabilities as influenced by the surface cross-slope and the capacity and adequacy of drainage facilities in the vicinity of vertical and horizontal curves and other transition locations (83, 127).

After the required level of frictional resistance has been decided, the aggregate is selected. The aggregate is expected to have the following properties:

1. adequate polish and wear resistance,
2. ability to transmit traffic loads to the underlying surface,
3. abrasion resistance, and
4. resistance to the deteriorating effects of weathering.

Due to the lack of reliable frictional resistance predicting models that would relate the aggregate properties to the desired frictional resistance, the several laboratory tests discussed previously and the aggregate performance history can be used to determine the suitability of the aggregate under consideration. The tests may include the PV test, the Los Angeles abrasion test, the four-cycle soundness test, the insoluble residue test for carbonate aggregates, and many others. In Texas, PV guidelines have been used for many years. The SDHPT issues a catalogue of RSPVs to serve as a guide for prospective bidders concerning the furnishing of aggregates for pavement surfaces. Recognizing the fact that some low PV aggregates (especially siliceous gravels) have been observed to show satisfactory frictional performance (44), the SDHPT has permitted the qualification of an aggregate source based on historical friction data as an alternate procedure. Several studies have been conducted in Texas (13, 39, 117) which have resulted in the development of plots of frictional performance history for numerous aggregate sources. As expected, the major problem encountered has been the high variability in the friction data caused by seasonal variations, which makes the qualification procedure not very dependable.

One of the difficult problems in proper selection of aggregates in Texas and many other states is the enormous amount of limestone available as aggregate. As stated previously, most limestone aggregates polish rapidly and lead to low frictional resistance in a relatively short period. Some studies (12, 86, 128) have shown that it is possible through proper combinations of aggregates of different wear rates to use most sources of limestone aggregates. Studies indicated that blending in proportion to the amount blended and the polish and wear resistance of the blending aggregates is effective. In Texas, problems have been experienced with blends of aggregates used to meet PV or soundness requirements. From a production standpoint, blends were reported to be difficult to control (141). For example, in blending a lightweight aggregate with a limestone aggregate, the tendency is to cut back on the amount of the lightweight aggregate. The specific gravity of the resulting mix is used to check the adequacy of blending rates. From a performance standpoint, it was experienced that surfaces constructed with blends may start with improved initial friction, but eventually this will decrease to take on the frictional characteristics of the poorer aggregate (141). Another observation reported (89) was on a highway section overlayed in 1985 with a HMAC blend of a sandstone aggregate and a siliceous gravel aggregate. The blend met the 40 percent soundness requirement, but the individual sandstone soundness loss was much higher. After two years of exposure to traffic, the road suffered deterioration due to rapid degradation of soft sandstone particles. It is recommended that, when blends of aggregates are to be used, each aggregate be required to meet all quality tests (89, 141).

Shape and size of aggregate particles are other features important to satisfactory frictional performance of an aggregate. Angular aggregate particles have proven to provide higher frictional resistance than subrounded or rounded particles, particularly in the case of siliceous gravels (122). Rounded siliceous gravels have provided satisfactory performance on low traffic volume roads. Although lightweight aggregate particles are often not angular, they tend to have the rough surface features desired for good seal coat surfaces. The presence of flat and elongated particles should be minimized and, if possible, avoided.

An aggregate with a "one size" gradation which will produce superior particle interlocking and will result in an optimum contact area between the tire and seal coat surface is preferred for seal coats. In Texas, it is a common practice to select large maximum size aggregate, grade 3 or 4, for high traffic volume roads (122). A large maximum size aggregate improves pavement surface drainage and thus reduces the potential for hydroplaning.

Although precoated aggregates are more expensive than non-coated aggregates, they have been extensively utilized in Texas for many years and found to have the following advantages (104, 122). They (1) reduce the effect of dusty aggregates, thus promoting the bond with asphalt, and (2) reduce automobile glass damage due to flying aggregate particles.

2.6.2 Construction Design Variables

After the aggregate type has been selected, the asphalt type and the design application rates of asphalt and aggregate are determined. Guidelines for selection of asphalt type and grade are available in the literature (122, 129, 130, 131). The guidelines indicate that a careful selection is one based on the following:

1. the type of aggregate to be spread on the asphalt layer, as related to the percentages of silica and alkali contents;
2. the climatic region in which the seal coat is to be constructed (minimum temperature, rainfall, and humidity); and
3. the limitations on the minimum surface and ambient temperatures for a few days prior to construction, at time of construction, and for several days after construction.

In designing the distribution rate of asphalt and the spreading rate of aggregate, the procedures should be aimed at producing (1) an adequate surface macrotexture in terms of providing uniformly distributed aggregate particles spaced in a way that would assure rapid escape of water from underneath the tire and (2) proper adhesion between the asphalt binder and the aggregate and adequate embedment of the aggregate into the asphalt film in order to minimize loss of aggregate. In practice, large maximum size aggregates require larger amounts of asphalt than small maximum size aggregates (i.e., 0.40 gal/sq yd for a Grade 3 aggregate compared with 0.20 gal/sq yd for a Grade 5 aggregate). It has been evident that Grade 3 aggregates have provided more effective seal coats because of the thickness of the applied asphalt film. In addition, although distributors, when kept in proper condition, are capable of accurately distributing desired amount of asphalt in a uniform transverse and longitudinal direction, experience has shown that field variations in applied asphalt quantities are much less critical for Grade 3 aggregates than for smaller grade aggregates (122, 130, 132). However, under no circumstances should the amount of the applied asphalt be in excess of that required to develop enough bond to the aggregate and produce adequate embedment. Excessive amounts of asphalt may cause the aggregate to be completely embedded into it, inducing bleeding of the surface, which lowers the surface frictional properties. Epps, Gallaway, and Hughes (122) gave excellent guidelines concerning design methods of seal coat surfaces.

2.6.3 Quality of Construction

The performance of seal coats depends to a large degree on the quality of construction. Key factors which may contribute to successfully constructing high quality seal coats include (122, 130, 131):

1. proper preparation of the existing surface upon which the seal coat is to be placed -- for example, if the surface is exhibiting a bleeding distress, special corrective measures should be taken to reduce the potential for bleeding in the new seal coat;
2. satisfactory environmental conditions -- experience has shown that the ideal environment for the construction of seal coats is hot, dry weather with no rain for the next several days;
3. selection of equipment in good operating condition and proper handling of equipment during construction;
4. carefully planned sequence and timing of construction operations;
5. implementation of an adequate field inspection and quality control plan; and
6. adequate traffic control during construction and in the first hours after completion of construction.

CHAPTER 3
SURVEY OF TEXAS DISTRICTS

3.1 Purpose

Nine Texas districts were surveyed to obtain information on the current policies practiced and problems experienced by those districts concerning methods for laboratory evaluation of coarse aggregate and aggregate frictional performance. Although the research efforts at this phase are directed towards investigating the frictional resistance of seal coat surfaces only, the survey of districts was intended to gather information on the use and friction of aggregates in seal coat and HMAC surfaces. It is believed that a full understanding of these policies and problems by the researchers would play a vital role in prioritizing and refining the study objectives. This in turn might lead to a better orientation of the research towards solving the current problems. Information sought included requirements regarding PVs of aggregates used, other methods for laboratory evaluation of coarse aggregate, FNs obtained, correlation between PV and FN where applicable, visual inspection of typical sections for high and low polish values and high and low frictional resistance, and personal observations of district personnel.

3.2 Findings

Enthusiastic response to the opportunity to participate in this research was observed in all surveyed districts. Valuable information was gathered and summarized for each district as follows.

3.2.1 District 2

The District uses the MSS and PV tests for evaluating the polish susceptibility of coarse aggregates. District personnel contacted feel that both tests are equally important in determining aggregate properties. The percentage loss in the MSS test should not exceed 30 percent if an aggregate is to be accepted. An aggregate with an MSS loss less than 20 percent is considered good; one with an MSS loss greater than 40 percent is poor. The District's PV requirements are as follows:

ADT	PV
Greater than 30,000	35
30,000 to 5,000	32
5,000 to 2,000	30
2,000 to 750	28
Less than 750	No requirement

District personnel have found that when aggregates meet the soundness test requirement, the FN tends to equal the PV of the aggregate at about 40 million passes; afterwards the FN remains constant. For example, for an aggregate with an MSS loss of 10 percent and a PV of 28, the FN tends to be in the range of 27 to 28 at 40 million passes. They have also found that aggregates with high MSS losses have low FNs even though their PVs are relatively high.

District personnel do not approve the use of aggregate blends as a way to improve FN. They feel that, although the initial FN is improved, it eventually drops and tends to decrease to the PV of the poor material with increases in traffic passes.

The District has the FN incorporated in its pavement management rehabilitation system. Pavements are ranked according to their respective FNs, and their maintenance priorities are set; pavements with the FN less than 20 have a higher priority.

Finally, the District keeps records of aggregate performance. It has 10-year friction data for many seal coat and HMAC projects, along with laboratory information on the results of the MSS, Los Angeles abrasion, PV, and decantation tests performed on the aggregate materials used in the projects documented. Some of the friction data were obtained on hard copies and manipulated along with data from other districts, as discussed later.

3.2.2 District 3

The District adopts the use of the PV test to determine the aggregate acceptability for use in HMAC and seal coat surfaces. The eastern part of the District produces aggregates with a PV of 38 to 39 while the southern part produces aggregates with a PV of 29. Sometimes, a low PV aggregate is blended with sandstone to improve its frictional resistance. The District personnel indicated that the sandstone improves the PV of the blend but reduces its soundness.

In addition, the District personnel stated that there is no need for an aggregate to meet a high PV requirement because, usually, before the FN drops below the acceptable limit the road is resurfaced for other rehabilitation purposes. However, they reported that in the case of low volume roads where low PV aggregates can be used, the relation between the reduction in FN and accumulative traffic is needed.

Finally, the District has never required the four-cycle soundness test for job control because the limestone aggregates used in the District easily pass the test.

3.2.3 District 4

The District uses the PV test. The aggregates should meet a minimum PV of 32 to be accepted. It also runs the Los Angeles abrasion test on siliceous gravel material. The District does not require the MSS test because its aggregates easily meet the requirement. The primary performance problem the District reported was stripping of aggregates from pavement surfaces.

3.2.4 District 5

The District uses the PV and MSS tests. The maximum allowable MSS loss is 25 percent. The PV requirements are a minimum of 32 for high volume roads and a minimum of 28 for low volume roads. However, past history of aggregates having lower PVs showing good frictional performance could waive these requirements. The District's area has a shortage of aggregates that will meet the PV required for high volume roads. Some of their aggregate must have a softer, unsound material combined with it to come up to a minimum required PV. The lightweight aggregate (high PV) is expensive for their area and tends to crush in the wheel paths of high volume roads.

Additionally, it was related that soft aggregates have high PV but are inadequate in soundness. The personnel contacted believe that soundness of aggregates is more important than PV and, therefore, the MSS test is preferred. It was also stated that soft limestone should be tested for soundness while siliceous gravel should be tested for polishing characteristics.

Factors mentioned to be more important for the FN than the PV were flushing of the asphalt in wheel paths and slipperiness of pavement surfaces right after rainfall. Finally, the District's Pavement Evaluation System does not include the FN. Yet, it is believed that safety regulations will eventually require the inclusion of the FN in such a system.

3.2.5 District 15

The District employs the PV and MSS tests in judging the acceptability of aggregates for usage in pavement surfaces; an aggregate should have a PV greater than 30 and a maximum allowable MSS loss of 30 percent to be accepted. Also, the District allows the use of blended aggregates to meet PV requirements. The District preserves records of aggregate properties and performance upon which such judgments and decisions also depend. Some of these records were acquired for evaluation by the researchers. The District has also set up test sections of seal coat surfaces for investigating aggregate performance.

3.2.6 District 16

The District relies on the PV and MSS tests. The soundness limits used are 30 percent for seal coats and 40 percent for blends in HMAC surfaces. A research project has been undertaken in which twelve HMAC test sections were built with limestone, sandstone, and blends of the two aggregates. The purpose of the study is to relate mixture design data including PV and MSS with FN and pavement performance (cracking, rutting, etc.).

3.2.7 District 18

The MSS test is greatly relied upon in this District with a maximum allowable MSS loss of 30 percent. The PV test is also used, though not relied on as much as the MSS test.

The District has several test sections on which PVs, MSS losses, and five-year friction data are available. Some of the District personnel pointed out that the outside lanes have lower FNs than the inside lanes due to heavier traffic volumes. They also believe that blending aggregates with different PVs (e.g., 30 and 34) gives better performance than using one aggregate with a PV of 32.

3.2.8 District 23

The PV and MSS tests are used in this District with 30 percent being the maximum allowable MSS loss. Two types of aggregates are used in the District, a limestone aggregate with a PV of 28 and a lightweight aggregate with a PV of about 45. A few years ago, both aggregates were used extensively. However, the use of the lightweight aggregate during the last few years was observed to have improved the frictional resistance of the roadways built with this aggregate. As a consequence, the lightweight aggregate has replaced the limestone aggregate almost everywhere.

The FN of roads constructed with the limestone aggregate dropped to around 25 within the first year, whereas roads built with the lightweight aggregate maintained a FN in the range of 50 to 45. The limestone aggregate is mostly used on low volume Farm-to-Market roads and roads that have many bends. The lightweight aggregate was reported to break in bends exposing sharp edges detrimental to rubber tires. The District prefers not to use aggregate blends, but it uses the low PV aggregate and then applies a sealant of lightweight aggregate when the FN drops below the acceptable limits.

A case was reported where two HMAC pavement sections were constructed with aggregates from the same source. In the first section, the aggregate was washed, and the section consequently used less asphalt; in the second, the aggregate was not washed. Performance records have shown that the first section maintained higher friction numbers, which might indicate that asphalt content affects pavement friction.

3.2.9 District 25

Both the MSS and PV tests are used in the District. The MSS test is not performed on aggregates for use in HMAC surfaces, because aggregates can pass it very easily, nor on precoated aggregates. The test is used only for seal coat surfaces with non-coated aggregates, with a maximum allowable MSS loss of 30 percent. The MSS test is relied on since the test gives a relative measure of the extent to which their aggregates will deteriorate due to salt, which they apply when they experience ice and snow.

The requirements for the PV test are a minimum of 32 for high traffic volume and a minimum of 28 for low traffic volume. However, the District allows the use of lower PV aggregates only if the aggregates have good performance history. The PV test is used only for new construction.

A study conducted years ago in the District revealed little correlation between PV and FN while some correlation between sand equivalency of surface texture and FN was found.

3.3 Observations from Obtained Friction Data

Friction data, collected over the past six to eight years, were obtained from several districts in Texas, not necessarily from the surveyed Districts only. Data from three districts, for four selected aggregate sources used in HMAC surfaces with various traffic volumes, were combined in order that limited observations could be made. Besides FN, the data included laboratory information on PV and MSS loss. Graphs of the FN versus accumulated traffic per lane are plotted in Fig. 3.1 (89). The aggregate material (AGMT) and the laboratory information are shown in Table 3.1.

As can be noted, there are obvious performance differences among the aggregates considered. First, the overall performance of the sandstone aggregate, which had a high PV of 47 and a MSS loss in the range of 9 to 20 percent, was markedly better than that of the limestone aggregates. A comparison between the PVs and MSS losses of the sandstone aggregate with those of Limestone 1 suggests that, since the MSS loss for the limestone aggregates is less than that of the sandstone aggregate, the markedly better performance of the sandstone aggregate is most likely

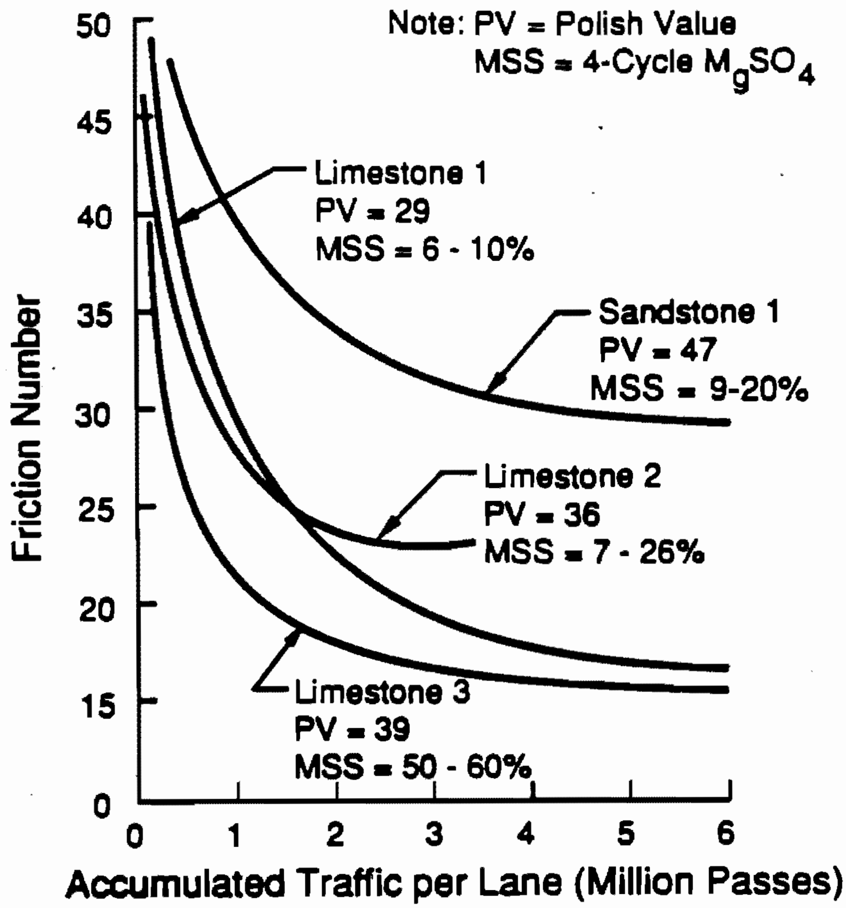


Fig. 3.1 Frictional Resistance of HMAC Surfaces Constructed with Four Aggregates in Three Texas Districts (89)

Table 3.1 Aggregate Considered in the Analysis of Obtained Friction Data

Aggregate Material	Aggregate Properties	
	PV	MSS, %
Sandstone 1	47	9-20
Limestone 1	29	6-10
Limestone 2	36	7-26
Limestone 3	39	50-60

due to the relatively higher PV, 47. Second, up to two million passes, the limestone aggregates exhibited a dramatic decrease in FN compared to the rather flattened decrease exhibited by the sandstone aggregate. Third, Limestone 1, which had the lowest MSS loss, maintained on FN higher than those of the other two limestone aggregates up to about two million accumulated passes, after which the FN of Limestone 2, with a PV of 36 and an MSS loss in the range of 7 to 26 percent, flattened and remained constant. Fourth, Limestone 3, in spite of its good PV of 39, had the worst performance all throughout the life of the road because it was inadequate in soundness (MSS = 52%). Last, the terminal FNs of Limestone 1 and Limestone 3 were about the same at six million passes. Yet, had the roads been resurfaced when the FN dropped below 20, Limestone 1 could have sustained twice as much traffic.

In summary, it seems that good frictional performance could not be achieved with only good polishing properties or with only good soundness characteristics. Two aggregates, limestone 1 and limestone 3, exhibited undesirable frictional performances; the former had a low PV and a low MSS loss, and the latter had a high PV and a high MSS loss. Two other aggregates, sandstone 1 and limestone 2, with approximately the same range of soundness loss had different frictional performances, with the aggregate with the higher PV maintaining higher FNs. The above discussion seems to indicate that both polishing and soundness properties and the interaction between the two contribute to changes in frictional performances.

It should be reemphasized that the above findings apply only to HMAC surfaces and may not necessarily be valid for seal coat surfaces. Also, it should be mentioned that these observations are to be interpreted cautiously since they are tentative and are based on plots that represent best fit curves. Although the trend of the decrease in FN was traceable, the data of each curve suffered from large, unexplained variations to the extent that overlapping between some of the data was observed. These variations could be attributable to factors , which are not easy to determine.

3.4 Summary

Several observations can be drawn from the findings of the surveys and the analysis of obtained friction data.

1. It was found that the PV test is used in all surveyed districts as a laboratory method for evaluating the polish susceptibility of coarse aggregates used in pavements. In seven of the surveyed districts, the MSS test is used along with the PV test (and is even more preferred in Districts 5 and 25). The PV requirements in most of the surveyed districts seemed to be a minimum of 32 for high volume roads and a minimum of 28 for low volume roads. The maximum allowable MSS loss was found to be 30 percent in five of the surveyed districts.
2. The soundness of the aggregates was found to be an important aggregate characteristic affecting the frictional resistance of pavement surfaces. It was reported in the surveys and supported by the findings of the analysis of friction data obtained for HMAC surfaces that aggregates that had high PV but were inadequate in soundness did not have good frictional performance on the roads.
3. Districts 5, 15, and 25 allow the use of aggregates that do not meet the PV requirements only if the aggregates have good frictional performance history. In addition to these districts, District 2 preserves friction data for many seal coat and HMAC projects along with laboratory information on the aggregates used in the documented projects.
4. Districts 15, 16, and 18 have set up seal coat and HMAC test sections for the purpose of investigating frictional resistance. A study conducted

years ago in District 25 revealed little correlation between PV and FN while some correlation between sand equivalency of surface texture and FN was found.

5. Districts 2 and 23 do not like to use aggregate blends while District 18 does. District 2 personnel feel that although the initial FN is improved, the FN eventually drops and tends to decrease to the PV of the poorer material. District 23 personnel prefer to use the low PV aggregate and then apply a sealant of lightweight aggregate (high PV) when the FN drops below the acceptable limits. However, District 18 personnel believe that blending aggregates with different PVs (e. g., 30 and 34) gives better performance than using one aggregate with a PV of 32.

6. District 3 reported that there is no need for an aggregate to meet a high PV requirement because, usually, the road is resurfaced for other rehabilitation purposes before the FN drops below the acceptable limit. However, for low volume roads where low PV aggregates can be used, the relationship between FN drop and accumulated traffic was reported to be of value.

7. Factors mentioned as important for frictional resistance were stripping of aggregates in the wheel paths (District 4), flushing of asphalt in the wheel paths (District 5), and slipperiness of pavement surfaces right after rainfall (District 5). District 18 reported that the outside lanes have lower FNs than the inside lanes due to the heavier traffic volumes passing on the outside lanes.

8. Finally, only District 2 was found to have the FN incorporated in its pavement management rehabilitation system. Yet, District 5 personnel believe that safety regulations will eventually require the inclusion of FN in such a system.

CHAPTER 4
RESEARCH METHODOLOGY AND TEST SECTIONS

4.1 Research Methodology

Many coarse aggregate materials and sources have been used for placing seal coats on Texas highways. The major categories include crushed limestone (LMST), crushed sandstone (SDST), and crushed siliceous gravel (SIGR). Other types include lightweight aggregate (LTWT), limestone rock asphalt (LMRA), traprock (TPRK), granite (GRNT), and rhyolite (RHYO). Differences in field frictional resistance of these aggregates have been observed over the years (13, 28, 29, 36, 39, 84, 117). Numerous factors, including aggregate characteristics, construction variables, traffic volume, and environment, are believed to be major contributors to these performance differences. The objective of this phase of the study was to investigate the effects of these factors on the field frictional resistance of coarse aggregates when used in seal coat surfaces.

The methodology followed in this investigation involved establishing seal coat test sections in different climatic regions of the state of Texas, using as many of the aggregates which are predominant to the area and which are economically obtainable. For each test section a construction survey was made. The survey was comprised of construction variables such as design application rates of asphalt and aggregate, asphalt and aggregate type, weather condition, and type and condition of existing pavement. Aggregate samples were obtained from the job sites of test sections and examined in the laboratory to determine their physical properties, polish and wear characteristics, resistance to weathering, resistance to impact and abrasion, and petrographical and mineralogical qualities. Field tests were then performed on the surfaces of test sections twice a year at random intervals. Six sets of skid resistance readings have been obtained while only three rounds of British Pendulum and sand patch testing were done on most of the sections. Visual condition surveys were also performed that described the surface condition of test sections each time field testing was conducted. Finally, annual and periodic weather information was collected for each section. It basically included short-term and cumulative precipitation data and cumulative number of freeze-thaw cycles.

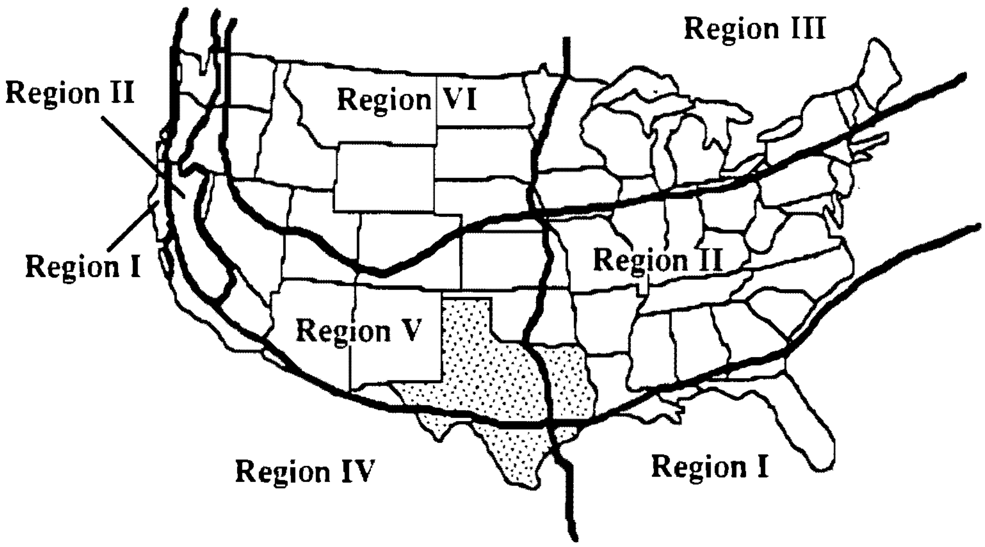
A data base was developed that embodied the bulk of the data collected. The data were manipulated and grouped according to the important variables involved. The graphical presentations of these groups represented the experimental designs that were used in the formulations of prediction models.

4.2 Test Sections

4.2.1 Environmental Considerations

Figure 4.1 provides a map that shows six different climatic regions of the United States and the environmental characteristics associated with each (135). The State of Texas lies within four of these regions (I, II, IV, and V), as shown in Fig. 4.2. The respective environmental characteristics are wet and no freeze, wet and freeze-thaw cycling, dry and no freeze, and dry and freeze-thaw cycling.

Seal coat test sections have been established in all four climate regions. At least one source of each major aggregate category should have been used in all four regions to allow evaluation of the effect of climate on that aggregate category. However, in the course of building test sections it was felt that this was not possible in most cases because, in practice, no aggregate is currently used in all four regions, and hauling an aggregate to locations far from its source is neither feasible nor practical. To make the experimental design more representative of the current practices, D-9 suggested that sections be constructed using different qualities of the main aggregate categories in regions close to the aggregate source. Since laboratory testing was not yet completed, D-9 reviewed the then established sections and grouped some of the major aggregates, according to their soundness (the MSS test) and polishing characteristics (the polish value test), based on historical laboratory data available at D-9. As a result, regions where it might be practical to construct sections were recommended. Table 4.1 provides a summary of the suggested aggregate groups and recommended regions. Unfortunately, after the grouping had been made, it was possible to place only two of recommended sources, one SDST and one LTWT. However, after the laboratory testing had been completed, the aggregates were grouped according to their determined



Region	Characteristics
I	Wet and no freeze
II	Wet and freeze-thaw cycling
III	Wet, hard freeze and spring thaw
IV	Dry and no freeze
V	Dry and freeze-thaw cycling
VI	Dry, hard freeze and spring thaw

Fig. 4.1 The Six Climatic Regions in the United States (135)

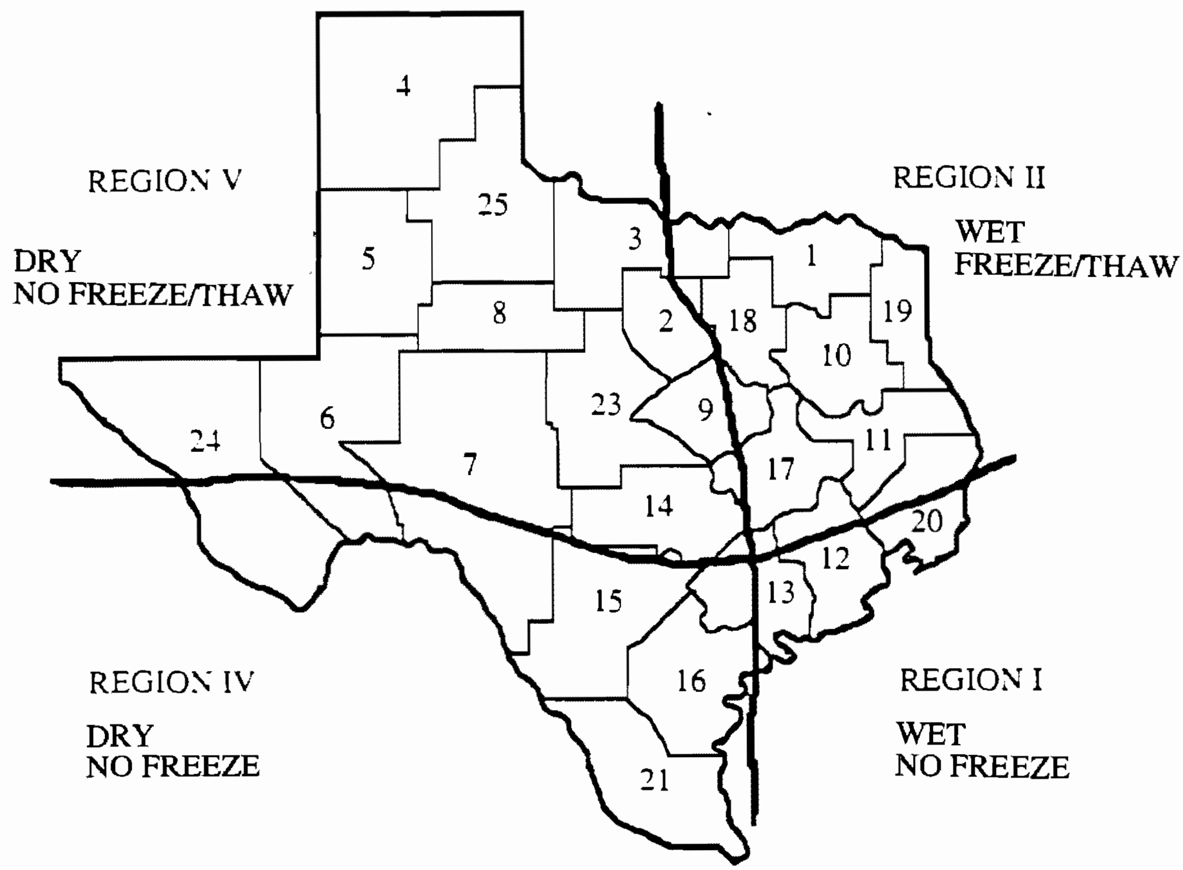


Fig. 4.2 Location of the Four Climatic Regions in Texas (135)

Table 4.1 Summary of the Aggregate Groups and Regions Suggested by D-9 for Evaluating the Environmental Effect

Material	Group	Properties		Aggregate producer / pit	Regions with section	Regions to place section in	Pit location
		MSS	PV				
LMST	1	≤ 5 ≤ 5	< 30 < 30	Dolese Bros. / Coleman Gifford Hill / Ogden	V IV, I	II -	Oklahoma District 15
	2	10-25 30-80	> 33 > 35	TX Crushed Stone / Feld White's Mines / Massey	II V	I, IV -	District 14 District 8
SDST	1	~ 5 ~ 20	36-43 ~ 40	Boorhem Fields / Apple, OK Delta Mat'l / Marble Falls	II -	- I, IV, V	Oklahoma District 14
SIGR	1	< 5 < 5	26-30 25-28	Janes / Blackburn South TX Aggregate / Knippa	V I	II IV	District 8 District 15
LMRA	1	~ 10	35-39	White's Mines / Unalde	I, II, IV	V	District 15
LTWT	1	-	48-50	TX Industries / Streetman	I	II, IV, V	District 18

properties, and the grouping was later used in the evaluation of the environmental effect.

4.2.2 Statistical Considerations

For the experimental design to be statistically sound, some requirements have been established.

1. Sections should be at least 1000 feet long to allow five friction or texture values to be measured.
2. Ideally, for each major aggregate material, as many as four sections with aggregates obtained from four different sources are to be constructed in each environmental region. However, the total number of test sections should not exceed that which may result in effective handling of sections.
3. Replications of sections should be established wherever possible. These will be used to examine the variation in field response under various experimental conditions. The variation expected can be caused mainly by the following:

    (a) time: which reflects possible changes in the quality of an aggregate pit with time. For example, an aggregate is used to construct a test section, and later, after a few weeks or few months, the same aggregate is used to build another test section, preferably in the same climatic region and under the same traffic.

    (b) traffic count: in the case of a divided highway, the ADT is provided as a total figure for both directions. Replications built in both directions may clarify whether traffic is divided equally. Also, in the case of more than one lane per direction, replications are built in all lanes to account for differences among the traffic volumes passing over the different lanes.

(c) less importantly, construction practices: two sections are placed a few miles apart on the same lane to evaluate any variations due to construction equipment, such as changes in application rates of asphalt or aggregate.

However, it should be mentioned that replications were not constructed for all sections. They were considered only for few sections where the circumstances permitted.

4.2.3 Criteria for Selection

Several criteria have been followed, when possible, in selecting test sections. These are as follows:

1. All sections are tangent sections.
2. Consider only sections with minimal slope, up to two percent.
3. Have no major intersections within or between sections.
4. Ideally, have as many sections end-to-end as the number of different aggregates used.
5. In the case of divided highways, have sections in only one direction, preferably the direction of heavier traffic. Where possible, replications are recommended as discussed earlier.

4.2.4 Selected Test Sections

Fifty-nine seal coat test sections were established in eleven districts of the four environmental regions, about twenty of them were replications. Various aggregate materials and sources were used. Specifically, aggregates from nine sources of crushed LMST, one source of LMRA, four sources of crushed SDST, eight sources of SIGR, three sources of LTWT two sources of crushed RHYO, and one source of crushed TPRK were placed. More than fifty percent of the aggregates were non-coated and the others were precoated. Different aggregate grades were also considered in view of the effect gradation may have on pavement surface texture and friction.

Table 4.2 provides a comprehensive summary of the selected test sections. The summary shows the region, district, county, and highway designation where each aggregate was placed. The aggregate type, material, grade, producer, and pit are also included. The map in Fig. 4.3 shows the districts, where test sections were established, in regard to the four environmental regions.

Table 4.2 Summary of Selected Test Sections and Aggregates Used

District	County	Section No.	Aggregate				Highway	Const. Forces	RG
			Type	Material	Grade	Producer / Pit			
23 Brown- wood	Stephens	1	B[a]	LMST	4	White's Mines / Brownwood	FM 1148	C	V
	Stephens	2[d]	B	LMST	4	White's Mines / Brownwood	FM 1287	C	V
	Stephens	3	B	LTWT	4	Featherlite / Ranger	US 193	C	V
	Eastland	4[d]	B	LTWT	4	Featherlite / Ranger	SH 36	C	V
8 Abilene	Shackelford	5[d]	B	LTWT	3	Featherlite / Ranger	SH 351	C	V
	Shackelford	6	B	SIGR	3	Janes Gravel / Blackburn	SH 351	C	V
	Shackelford	7	PB[b]	LMST	3	White's Mines / Massey	SH 351	C	V
4 Amarillo	Deaf Smith	8	B	SIGR	3	Vega S & G / Tom Green	FM 2943	M	V
	Deaf Smith	9	B	SIGR	4	Panhandle Gravel / Box Canyon	FM 2943	M	V
	Deaf Smith	10	B	LMST	4	Dolese Brothers / Coleman, OK	FM 2943	M	V
	Deaf Smith	11	B	SIGR	4	Western S & G / Tascosa	FM 2943	M	V
	Deaf Smith	12	PD[c]	SIGR	4	TX S & G / Mansfield	FM 2943	M	V
	Deaf Smith	13[g]	B	SIGR	3	Vega S & G / Tom Green	FM 1062	M	V
	Deaf Smith	14[g]	B	SIGR	3	Vega S & G / Tom Green	FM 2943	M	V
16 Corpus- Christi	Nueces	15	PB	LMRA	4	White's Mines / Uvalde	P 22	M	IV
	Nueces	16	PB	SIGR	4	Bay Inc. / Perez	P 22	M	IV
	Nueces	17	PB	LMST	4	Pioneer / Tradesman Dr.	P 22	M	IV
	Nueces	18	PB	LMST	3M	Gifford Hill / Ogden	P 22	M	IV
	Nueces	19	PB	LMRA	4	White's Mines / Uvalde	P 22	M	IV
	Nueces	20[e]	PB	LMRA	4	White's Mines / Uvalde	P 22	M	IV
	Nueces	21[d]	PB	LMRA	4	White's Mines / Uvalde	SH 358	M	IV

(continued)

Table 4.2 Summary of Selected Test Sections and Aggregates Used (Continued)

District	County	Section No.	Aggregate				Highway	Const. Forces	RG
			Type	Material	Grade	Producer / Pit			
19 Atlanta	Titus	22	B	SIGR	3	Gifford Hill / Little River	FM 2882	M	II
	Panola	23	B	LTWT	3M	TX Industries / Streetman	US 59	C	II
	Panola	24[e]	B	LTWL	3M	TX Industries / Streetman	US 59	C	II
	Panola	25	B	SDST	3	Boorhem Fields / Apple,OK	SH 315	C	II
	Bowie	31	B	SDST	3	HMB / Dequeen, AK	FM 989	M	II
	Upshur	35[h]	B	LTWT	3M	TX Industries / Streetman	US 271	M	II
1 Paris	Delta	26[d]	PB	SDST	3	Boorhem Fields / Apple,OK	SH 154	C	II
	Hopkins	27[d]	PB	SDST	3	Boorhem Fields / Apple,OK	SH 19	C	II
	Hopkins	28[d]	PB	SDST	3	Boorhem Fields / Apple,OK	SH 19	C	II
	Grayson	29	PB	LMST	3	Amis Mat'l / Stringtown, OK	US 69	C	II
	Grayson	30[e]	PB	LMST	3	Amis Mat'l / Stringtown, OK	US 69	C	II
11 Lufkin	Houston	32	B	LMST	3M	Tx Crushed Stone / feld	US 287	C	II
	Houston	33[h]	PB	LTWT	3M	TX Industries /Streetman	US 287	C	II
	Houston	34[h]	PB	LMRA	4	White's Mines / Uvalde	US 287	C	II
13 Yoakum	Victoria	36	PB	LMST	4	Colorado Mat'l / Hunter	US 77	C	I
	Victoria	37[h]	PB	LMST	4	Gifford Hill / Ogden	US 77	C	I
	Victoria	38	PB	LMST	4	Redland Worth / Beckman	US 77	C	I
	Victoria	39	PB	SIGR	4	South TX Aggreg. / Knippa	US 77	C	I
	Victoria	40	PB	LTWT	4	TX Industries / Clodine	US 77	C	I
	Victoria	41[h]	PB	LTWT	4	TX Industries / Streetman	US 77	C	I
	Victoria	42[h]	PB	LMRA	4	White's Mines / Uvalde	US 77	C	I
	Victoria	43[h]	PB	LMRA	4	White's Mines / Uvalde	US 77	C	I

(continued)

Table 4.2 Summary of Selected Test Sections and Aggregates Used (Continued)

District	County	Section No.	Aggregate				Highway	Const. Forces	RG
			Type	Material	Grade	Producer / Pit			
23 Beaumont	Liberty	44[h]	B	LTWT	3M	TX Industries / Streetman	US 90	C	I
	Liberty	45[h]	PB	LMRA	3	White's Mines / Uvalde	US 90	C	I
	Liberty	46[e]	B	LTWT	3M	TX Industries / Streetman	US 90	C	I
	Liberty	47[e]	PB	LMRA	3	White's Mines / Uvalde	US 90	C	I
	Liberty	48[h]	B	LTWT	3M	TX Industries / Streetman	US 90	C	I
	Liberty	49[e]	B	LTWT	3M	TX Industries / Streetman	US 90	C	I
7 San Angelo	Sutton	50[e]	B	LMST	4M	White's Mines / Massey	US 277	M	V
	Sutton	51	B	SDST	4	Delta Mat'l / Marble Falls	US 277	M	V
	Sutton	52	B	SDST	4	East TX Stone / Blue Mountain	US 277	M	V
	Sutton	53	B	LMST	4	Reese Albert / Willeke	US 277	M	V
	Sutton	54[d]	PB	LMST	4	Reese Albert / Willeke	US 277	M	V
	Sutton	55	B	TPRK	4	White's Mines / Knippa	US 277	M	V
24 El Paso	Hudspeth	56	PB	RHYO	3M	Allamore Pit / W. of Boracho	IH 10	M	V
	Hudspeth	57[e]	PB	RHYO	3M	Allamore Pit / W. of Boracho	IH 10	M	V
	Hudspeth	58	PB	RHYO	3M	Hoban Pit / W. of Boracho	IH 10	M	V
	Hudspeth	59[e]	PB	RHYO	3M	Hoban Pit / W. of Boracho	IH 10	M	V

[a] An aggregate which consists of crushed gravel, crushed slag, crushed stone, or natural limestone rock asphalt.
[b] The same as above but precoated.
[c] A precoated aggregate which consists of crushed gravel, crushed slag, crushed stone.
[d] A traffic replication.
[e] A construction replication.
[f] A time or pit replication.
[g] A replication that complies with both "d" and "e".
[h] A replication that complies with both "d" and "f".

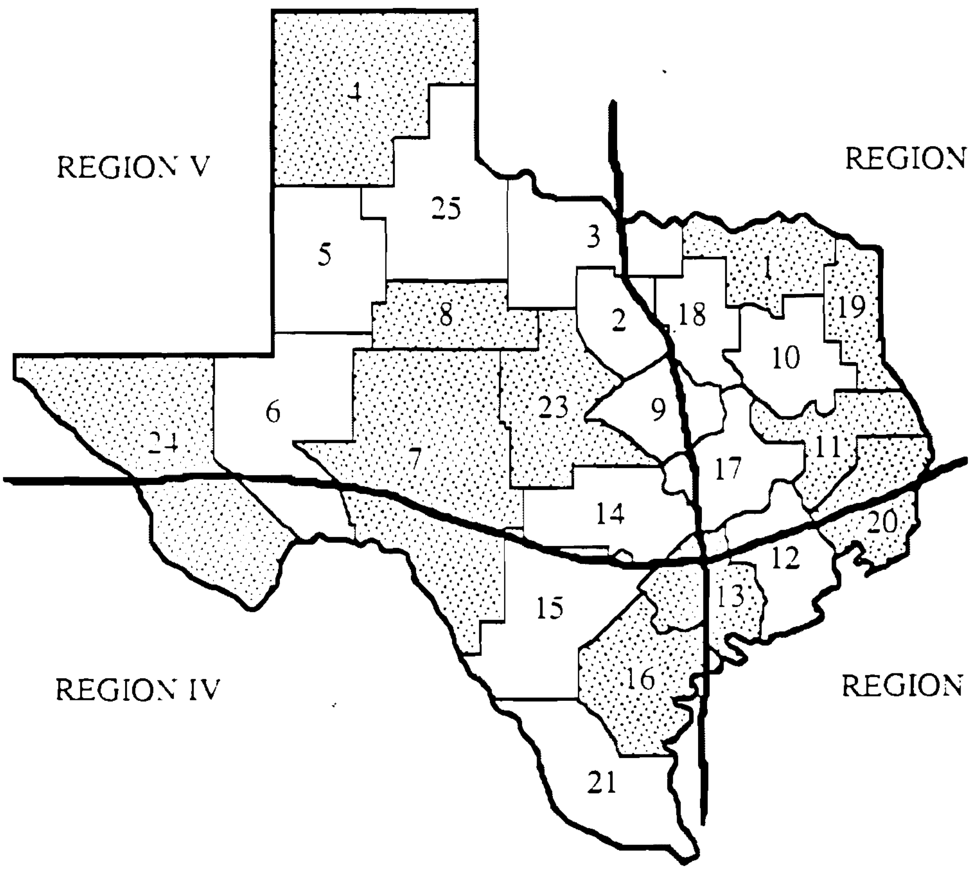


Fig. 4.3 Texas Districts where Test Sections were Placed

CHAPTER 5
LABORATORY TESTS AND PETROGRAPHIC EXAMINATION

Two types of laboratory data were collected. The first type, which constituted the larger portion, was concerned with the physical properties of the aggregates and was obtained from the results of numerous tests performed at the engineering laboratories. The other type dealt with aggregate mineralogy and petrographic characteristics and was gathered from examinations done by the petrographers of a construction materials consulting office.

5.1 Engineering Laboratory Tests and Results

Four groups of tests were found applicable for measuring the degree of deterioration an aggregate may exhibit when placed in field service. These were:

Group One: testing for basic properties,
Group Two: testing for polish and wear characteristics,
Group Three: testing for resistance to disintegration due to weathering action, and
Group Four: testing for resistance to degradation due to abrasion, impact, and grinding.

It was first thought that laboratory testing would be performed on both precoated and uncoated aggregate samples. Therefore, the asphalt coating on the particles of precoated aggregate samples needed to be extracted in order that testing could be done. As the extraction process had progressed and after it had been completed on the samples of two aggregate sources, it was felt that the process was very costly and time consuming because of the huge quantity of aggregate needed to be cleaned in order that most of the tests, that constitute the above four groups, could be performed. It was then decided that only those tests that did not necessitate prior extraction of asphalt would be performed. These were the sieve analysis, accelerated polish, insoluble residue, and crushed particles in gravel tests. In addition, it was decided to gather data on two major tests from the laboratory history of aggregate sources available at D-9, the tests being the MSS test from group three and the Los Angeles abrasion (LA) test from group four.

In the engineering laboratories, the aggregate samples were first reduced to testing sizes, required by each of the tests considered, according to the procedures described in ASTM C702. Then, most of the selected tests were performed on all prepared samples in conformity with the Test Methods described in the Manual of Testing Procedures of the SDHPT, D-9. Most of these methods are modifications of Standard Test Methods of the ASTM; some have procedures identical to those prescribed by ASTM. Two of the selected tests, the insoluble residue test and the aggregate durability test, were preformed in accordance with ASTM Standards.

A comprehensive summary of the test results for the uncoated aggregates is presented in Table 5.1. Table 5.2 summarizes the laboratory data and test results collected for the precoated aggregates. The following is a review of the selected tests and their grouping and significances.

5.1.1 Group One: Testing for Basic Properties

5.1.1.1 Sieve Analysis. This procedure was performed in accordance with Test Method Tex-401-A (106). It was used for the determination of the particle distribution, gradation, of the obtained aggregate samples. The gradation of an aggregate is a primary factor in the forming of the texture of a seal coat surface and thus affects surface friction.

Aggregates from four different grades were used. These were grade 3, 3 modified (3M), 4 , and 4 modified (4M). The larger the grade number is, the smaller the size of the aggregate particles are. The results in Tables 5.1 and 5.2 represent the cumulative percent retained on each of the sieves used in this test. Since single-sized aggregates are usually used in seal coat overlays, the cumulative percents retained on No. 4 and No. 8 sieves seem not to have varied significantly for the different aggregates tested. It is the cumulative percents retained on No. 1/2 in. and No. 3/8 in. that seems to explain how the size of aggregate particles varies from one grade to

Table 5.1 Summary of Results of the Engineering Laboratory Tests Performed on the Uncoated Aggregates

AGMT	Agg. No.	SC	GR	No. 1/2	No. 3/8	No. 4	No. 8	SPGR	ABSP	PV	INRD	MSS	FRTH	LA	ADI	TDT	DCL
LMST	1	U	4	0.7	26.5	93.6	98.5	2.63	1.3	28	0.45	2.7	0.85	24	67	0.9	0.30
	2	U	4	2.3	37.8	98.9	99.5	2.66	0.8	28	0.53	6.1	2.60	24	73	3.6	0.34
	10	U	4	1.1	33.7	96.9	99.6	2.70	0.8	27	9.36	1.7	1.11	22	76	3.5	0.51
	30	U	3	41.8	87.2	99.5	99.6	2.52	1.4	34	72.27	2.1	4.25	16	74	2.6	-
	32	U	3M	23.5	76.5	94.9	98.2	2.32	4.3	36	4.24	17.1	1.90	36	63	8.3	1.06
	50	U	4M	0.0	18.0	88.3	97.7	2.47	3.5	34	0.30	36.6	10.00	26	49	3.9	0.72
	53	U	4	0.3	22.3	94.6	98.5	2.50	3.3	33	0.80	27.6	4.78	29	57	1.8	0.53
	54	E	4	1.0	25.1	96.2	99.3	2.48	3.4	33	0.41	32.0	6.08	29	66	6.4	-
LTWT	3/4	U	4	3.3	45.7	98.5	99.1	1.53	4.6	42	-	-	3.41	25	88	8.1	0.25
	5	U	3	25.6	58.9	96.7	98.0	1.38	4.8	45	-	-	13.14	32	93	5.4	0.07
	23/24/35	U	3M	14.3	60.1	98.2	98.9	1.57	7.3	49	-	-	9.53	26	79	8.1	0.35
	44/46/48/49	U	3M	8.9	52.0	98.8	99.1	1.49	8.2	48	-	-	3.00	18	85	2.0	-
SDST	25	U	3	36.7	89.9	99.5	99.6	2.49	2.3	36	100.00	2.6	1.41	20	89	4.6	0.15
	26/27/28	E	3	26.4	84.3	99.3	99.7	2.50	1.4	36	100.00	0.0	1.58	20	93	2.6	-
	31	U	3	44.4	90.2	99.2	99.3	2.58	1.2	41	99.80	2.5	1.11	19	80	3.6	0.35
	51	U	4	0.0	24.8	96.3	99.4	2.53	2.4	38	55.00	12.2	2.00	22	73	1.2	0.30
	52	U	4	2.8	33.5	96.6	98.7	2.65	0.6	40	63.00	4.0	1.45	28	88	3.0	0.35
SIGR	6	U	3	34.9	93.7	99.2	99.5	2.67	0.5	27	10.75	2.9	1.41	22	73	3.7	0.19
	8/13/14	U	3	44.0	94.9	99.6	99.7	2.65	0.7	33	84.10	3.7	1.95	25	74	3.8	0.13
	9	U	4	0.3	34.3	98.2	99.6	2.61	1.0	30	85.40	8.6	5.30	24	76	6.5	0.27
	11	U	4	0.0	32.7	98.2	99.2	2.60	1.2	34	85.33	4.7	2.08	24	81	5.4	0.23
	22	U	3	37.7	89.7	99.7	99.7	2.57	0.8	34	100.00	1.3	0.66	18	96	2.2	0.10
TPRK	55	U	4	2.5	37.0	97.7	99.5	2.96	1.5	34	97.40	7.7	4.60	10	52	0.8	0.34

Table 5.2 Summary of the Engineering Laboratory Data and Tests Results Collected for the Precoated Aggregates

AGMT	Agg. No.	GR	No. 1/2	No. 3/8	No. 4	No. 8	PV	MSS	LA	INRD	FRTH
LMRA	15	4	1.3	30.6	97.2	99.6	34	11.0	32	10.41	-
	19/20/21	4	0.0	34.4	98.3	99.4	37	11.0	32	13.26	-
	34	3	38.3	90.5	99.5	99.5	40	11.0	32	1.28	-
	42/43	4	0.5	25.6	97.0	99.4	35	11.0	32	3.80	-
	45/47	3	27.2	87.2	99.6	99.6	37	11.0	32	14.71	-
LMST	7	3	33.4	91.4	100.0	100.0	37	41.0	28	2.19	-
	17	4	0.0	17.4	99.3	99.8	28	4.0	25	4.71	-
	18	3M	15.7	54.4	96.3	99.9	27	2.0	26	5.16	-
	29	3	30.9	87.2	99.8	99.8	34	2.1	16	72.27	4.25
	36	4	0.3	24.7	99.5	99.7	28	18.0	27	0.80	-
	37	4	0.4	22.4	96.8	99.8	29	2.0	26	2.14	-
	38	4	0.2	33.3	97.5	99.9	27	5.0	25	2.61	-
LTWT	33	3M	25.9	70.5	99.1	99.7	48	-	19	-	3.00
	40	4	0.3	28.2	98.3	99.9	51	-	25	-	7.50
	41	4	1.0	23.8	95.2	96.9	50	-	19	-	3.00
RHYO	56/57	3M	22.5	79.1	99.4	99.9	37	15.0	20	100.00	-
	58/59	3M	31.1	65.5	99.9	99.9	36	4.0	15	100.00	-
SIGR	12	4	2.5	39.6	95.1	99.7	30	4.5	24	83.75	-
	16	4	0.3	18.3	88.3	98.9	27	8.0	17	92.90	-
	39	4	0.2	23.8	96.7	99.3	25	1.0	23	20.23	-

another. It can also be seen that aggregates assigned to have the same grade may have significantly different percents retained on No. 1/2 in. sieve. As an example, aggregate 5 has 25.6 percent retained on No. 1/2 in. sieve compared with 44.4 percent for aggregate 31, both of grade 3.

5.1.1.2 Specific Gravity and Absorption Test. The test was performed according to Test Method Tex-403-A (107) to determine the saturated surface-dry specific gravity and water absorption of natural aggregates and according to Test Method Tex-433-A (113) to determine the dry bulk specific gravity and absorption of lightweight aggregates. The specific gravity (SPGR) and absorption (ABSP) give an indication of the porosity of an aggregate. Both characteristics are viewed as important factors influencing aggregate frictional properties. The results of this test showed lower SPGR and higher absorption values for the LTWT aggregates. Among the natural aggregates, some of the LMST ones had high ABSP values.

5.1.1.3 Decantation Test. This test was performed in conformity with Test Method Tex-406-A (108). During the test, the amount of material finer than the No. 200 sieve was removed from an aggregate by washing and the percentage by weight is calculated. The removed materials normally include water-soluble materials as well as aggregate particles and silt and clay particles that can be dispersed by water. The decantation loss (DCL) ranged from 0.07 percent for a LTWT aggregate to 1.06 percent for a LMST one. The only four values above 0.50 percent belonged to the LMST group.

The amount of material removed may be related to the relative stability of aggregate particles in seal coat surfaces (adhesion between aggregate particles and asphalt) and to the amount of asphalt needed to assure a desired stability. This measure along with some of the construction variables may explain some of the problems pertaining to surface texture, particularly to dislodgement or loss of aggregates from seal coat surfaces.

5.1.2 Group Two: Testing for Polish and Wear Characteristics

5.1.2.1 Accelerated Polish Test. Test Method Tex-438-A (115) for the accelerated polish test was employed; it provides an estimate of the extent to which coarse aggregates in the wearing surface of the roadway are likely to polish when subjected to traffic. In this test, the aggregate samples are prepared to required size and placed in metal molds, shown in Fig. 5.1. For uncoated aggregates, a polyester resin and a catalyst are normally used to bond the aggregate particles together in the mold. Seven specimens are made for each aggregate material. The specimens are then mounted around the periphery of a specimen wheel, shown in Fig. 5.2, and subjected to accelerated polish by the rolling action of a rubber tire in the presence of water and abrasive grit used to accelerate the rate of wear. The state of polish reached by each sample after nine hours of accelerated polish is expressed as the PV, and it is measured by the BPT, shown in Fig. 5.3. The PV is calculated as the average of the seven specimens.

In the case of precoated aggregates, the normally-used resin, when tried on some of these aggregates, did not perform well, and problems with aggregate particles stripping from specimen surfaces were encountered. A high molecular weight methacrylate (HMWM) monomer was tried which proved to have the ability to penetrate the asphalt coating film and hold the aggregate particles together very well. However, because of the brittleness of this monomer, some of the first specimens made with it were breaking under the impact actions resulting from vibrations in the accelerated polishing tire used in this test. It is thought that the impact actions created tensile stresses in those specimens due to the presence of imperfections at the interface between the specimens and specimen wheel. This problem was remedied by using corrugated Xorex II Ribtec® steel fibers to provide some flexibility to the system. Fifteen one-inch long fibers were placed longitudinally in five rows on the tips of the aggregate particles, and five others were placed in the other direction before each specimen was cast.

Fig. 5.1 Preparation of an aggregate sample in test molds

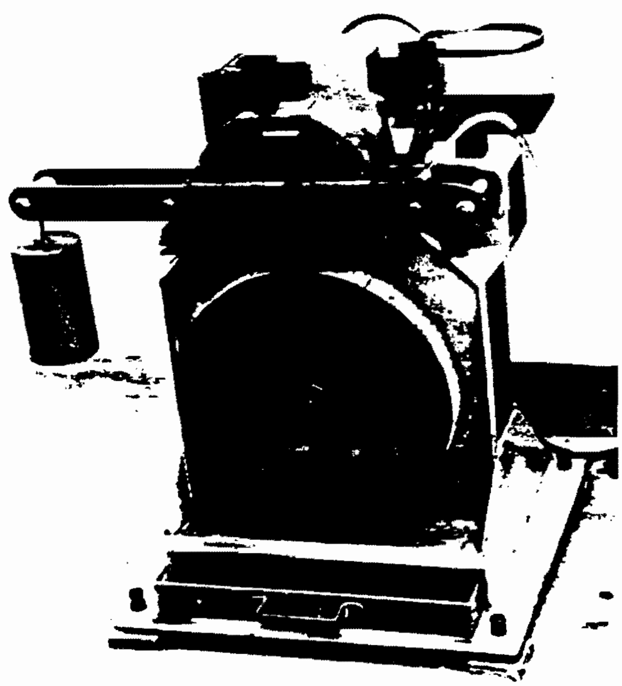


Fig. 5.2 Specimens mounted on specimen sheel

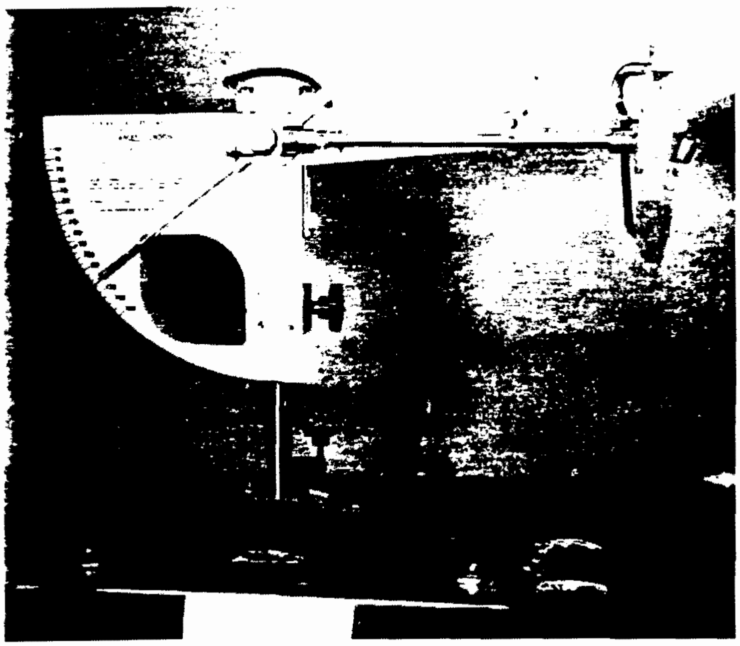


Fig. 5.3 The British pendulum tester

Natural sand along with fly ash were used as fillers in the polymer mix in the following proportions:

	Sieve	% Retained
Sand:	No. 8	5
	No. 16	20
	No. 30	20
	No. 50	17
	No. 100	13
Fly Ash:	----------	25
		100

The mix was proportioned as follows:

HMWM (1540)	100	parts
Filler (untapped)	500	parts
Cumene Hydroperoxide (as an initiator)	4	parts
6% Cobalt solution (as a promoter)	2	parts

The PV ranged from a lowest value of 25 for a SIGR aggregate to a highest value of 51 for a LTWT aggregate. The LTWT aggregates had PVs greater than 41, while all the natural aggregates had PVs less than 42. The PV range was 36 to 41 for the SDST and RHYO aggregates and 34 to 40 for the LMRA ones. For the LMST and SIGR groups, the PV ranged between 27 and 37 for the former and between 25 and 34 for the latter. The TPRK source had a PV of 34.

5.1.2.2 Insoluble Residue Test for Carbonate Aggregates. This test was conducted in accordance with ASTM D-3042 (6). It provides an estimate of the amount of non-carbonate (insoluble) material in carbonate aggregate and involves a grain size distribution of these insoluble particles. This is done by chemically separating the non-carbonate minerals from the carbonate particles. The procedure is based on the chemical reaction that occurs when dilute hydrochloric acid is allowed to react with the carbonate portion of the aggregate. The leaching process dissolves the carbonate fraction, leaving the non-carbonate fraction in the form of a residue, which must not be destroyed or altered in any manner. Then, the grain size distribution of the residue is analyzed and the plus No. 200 sieve fraction (sand-sized fraction) determined. This sand-sized fraction is believed to be a major factor affecting the polish susceptibility of a carbonate aggregate (118). In case of precoated aggregates the asphalt portion of the residue was washed away thoroughly over the No. 200 sieve using a methyl chloride solution.

The aim of the test was to establish a relationship between the friction properties exhibited by a carbonate aggregate and the physical characteristics measured by this test. The theory is based on the concept that the frictional resistance of carbonate aggregates is related to the differential hardness of the minerals that comprise the structure of the aggregate (118). According to this concept, when a carbonate aggregate is subjected to polish, the softer minerals will wear away at a faster rate than the harder ones, and there will be some attrition in the aggregate caused by loss of the softer minerals. The result will leave the wearing surface of the aggregate with a rough, uneven texture, which increases or maintains the friction properties of the carbonate aggregate.

The insoluble residue (INRD) values obtained for the LMST aggregates were generally low, with only one aggregate source having an INRD of 72.27 percent. This source is known to have a high amount of chert, which is a hard and insoluble mineral. The residues for the LMRA group were relatively higher with three values were above ten percent. Two of the SDST sources were almost insoluble, and so were the RHYO and TPRK sources. However, the other two SDST sources were composed of large amounts of soluble carbonate minerals. Six of the SIGR aggregates had INRD values above 80 percent, while the other two, aggregates 6 and 39, had INRD values of 10.75 and 20.23, respectively. This test was attempted on the LTWT aggregates, but no reaction was found. It was then considered to be unsuitable for this aggregate group.

5.1.2.3 _Crushed Particles in Gravel Aggregates._ This test was performed in accordance with Test Method Tex-413-A (111) and was used to determine the percentage by particle count of crushed particles in aggregates. This characteristic is of interest because the asperities of the texture of crushed particles as opposed to the smooth texture of non-crushed particles are very important features of an aggregate's polish susceptibility. According to the SDHPT standards, a gravel aggregate shall have a minimum of 85 percent of the particles retained on the No. 4 sieve with at least two crushed faces for it to be considered crushed. The test results in Table 5.3 indicated that only aggregates 11, 12, and 39 may be considered crushed.

5.1.3 Group Three: Testing for Resistance to Disintegration Due to Weathering Action

5.1.3.1 Four-Cycle Magnesium Sulfate Soundness Test. In this test, only the natural, uncoated aggregates were examined according to Test Method Tex-411-A (110) to estimate their soundness when subjected to weathering action. Data on the MSS losses were collected from D-9 for the precoated aggregates. In each of the four cycles of this test, the aggregate samples are immersed in saturated solutions of magnesium sulfate followed by oven drying to partially or completely dehydrate the salt precipitated in permeable pore spaces. The dehydration of salt upon re-immersion causes internal expansive forces which simulate the expansion of water on freezing. When water repeatedly freezes in the pore spaces of an aggregate in field service, the expansive forces created cause the aggregate particles to disintegrate, resulting in increased deterioration in surface texture and decreased frictional resistance.

Soundness losses ranging from 1.7 to 41.0 were experienced in the LMST group. Low losses in the range of 1.7 to 6.1 percent were found to be associated with low PVs in the range of 27 to 29 with only one exception. This was the cherty LMST, aggregate 29/30, which had a PV of 34 and an MSS loss of 2.1 percent. On the other hand, higher losses ranging from 17.1 to 41.0 percent were noticed to be associated with higher PVs ranging from 33 to 37 with also one exception, that being precoated aggregate 36 with a PV of 28 and an assigned MSS loss of 18.0 percent. The laboratory history of this source revealed a wide range of experienced MSS losses, with the losses increasing as the PVs increased. The assigned MSS loss was more or less the average of those losses that had been associated with low PVs.

The MSS losses for the SIGR, SDST, RHYO and TPRK groups were generally low, with only SDST aggregate 51 and precoated RHYO aggregate 56/57 showing losses of 12.2 and 15.0, respectively. The LMRA source was assigned an

Table 5.3 Results of the Crushed Particles in Gravel Test

Aggregate Number	Gravel Particles with Least Number of Crushed Faces					
	6CF	5CF	4CF	3CF	2CF [a]	1CF
6	0.0	1.4	9.7	22.8	38.0	55.8
8/13/14	1.2	3.8	10.5	21.2	37.4	55.3
9	0.0	0.0	4.0	29.0	65.9	81.9
11 [b]	0.0	1.2	8.3	48.1	89.3	96.9
12 [b]	4.0	17.0	40.0	71.0	92.0	96.0
16	0.5	3.0	16.5	37.9	66.8	83.5
22	0.0	0.2	4.2	25.9	60.4	93.5
39 [b]	2.0	15.4	38.6	59.3	84.5	89.7

[a] Percentage of particles with at least two crushed faces (CF).
[b] Gravel aggregates that are considered "crushed" according to the SDHPT standards.

MSS loss of 11.0 percent, which was considered to be a good estimate of the average for a range in MSS losses of 7.0 to 14.0 percent experienced with this source.

5.1.3.2 *Coarse Aggregate Freeze-Thaw Test.* This test was performed in accordance with Test Method Tex-432-A (112) which was developed for testing synthetic coarse aggregates. As in the MSS test, the aggregates are tested to judge their soundness when subjected to weathering action. This is accomplished by subjecting the aggregates to 50 cycles of freezing and thawing in the presence of water. The internal expansive forces created by repeated freezing of water in the pore spaces cause the aggregate to disintegrate. This action is supposed to simulate what happens to the aggregates placed in the environmental regions characterized by freeze-thaw cycling.

The results of this test varied widely for the LMST and LTWT groups, with three freeze-thaw (FRTH) losses being associated with high PVs in the LMST group. The FRTH losses were relatively lower, and the loss range was narrower for the SDST and SIGR groups.

The soundness and freeze-thaw losses are believed to be indicative of the strength and hardness of the cementing material that holds the crystal grains of aggregate particles together.

5.1.4 Group Four: Testing for Resistance to Degradation Due to Abrasion, Impact, and Grinding

5.1.4.1 *Los Angeles Abrasion Test.* The Los Angeles abrasion test provides a measure of degradation resulting from a combination of actions, including abrasion or attrition, impact, and grinding. This is done, in accordance with Test Method Tex-410-A, by placing the aggregate in a rotating steel drum containing a specified number of steel spheres, the number depending upon the gradation of the test sample. As the drum rotates, the sample and the steel spheres are picked up by a shelf plate and carried around until they are dropped to the other side of the drum, creating an impact-crushing effect. The contents then roll within the drum with abrading and grinding actions until the shelf plate impacts and the cycle is repeated. After the prescribed number of revolutions, the aggregate is removed and then sieved to measure the degradation as percent loss.

The variation in the results of this test was similar to that of the soundness results. In the LMST group, LA losses of 28 and above were experienced by almost all of the aggregates possessing high PVs. Another observation is the wide variability in LA losses encountered with the LTWT aggregates, with the results correlating well with those of the FRTH test.

5.1.4.2 Aggregate Durability Test. This test was performed in compliance with ASTM D-3744 (7). The test establishes an empirical value, the durability index, indicative of the relative resistance of an aggregate to generating detrimental clay-like fines when subjected to mechanical degradation in the presence of water. The coarse aggregate test sample is agitated in a mechanical washing vessel for a period of 10 minutes. The resulting wash water and minus No. 200 size fines are collected and mixed with a stock calcium chloride solution and placed in a plastic cylinder. After a 20-minute sedimentation time, the height of the sediment is read and then used to calculate the aggregate durability index (ADI). The smaller the ADI is, the less durable the aggregate is expected to be.

An ADI of about 75 seemed to have divided the results into two groups. The group with indices below 75 constituted almost all of the LMST, two SIGR, and one SDST aggregate. All of these aggregates were largely comprised of carbonate minerals, except LMST aggregate 30 and SIGR aggregate 8/13/14. The TPRK aggregate was evaluated to have unexpectedly low ADI of 52. Also in this group, the lowest indices were obtained for four of the LMST aggregates with high PVs and MSS losses. The other group with indices above 75 constituted the LTWT and the rest of the SDST and SIGR aggregates.

5.1.4.3 Texas Degradation Test. This test was developed as a part of another research project at The University of Texas at Austin (89). It is intended for determining the resistance of aggregates to degradation in HMAC and seal coat surfaces. The test includes information from Test Methods Tex-116-E, ASTM C-132, ASTM D-3744, and California Test 229 and research reports "Modification of the

Standard Los Angeles Abrasion Test" (HRB), and "Concrete Aggregate Durability Tests" (California Department of Transportation).

The procedure is to subject an aggregate to mechanical degradation by agitation in the wet ball mill apparatus in the presence of water, and the percent loss of minus No. 16 material is measured. The loss by weight in the Texas degradation test (TDT) is created by the resulting interparticle impact, abrading, and grinding actions. The results of this test differed largely within each aggregate group. Its correlation with other tests is discussed in a later section.

The mechanical degradation an aggregate is subjected to in this group of tests is expected to simulate (1) the impact action of axle loads on aggregate particles in the wearing surface of a roadway and (2) the abrasive and grinding actions created between the rubber tires and aggregate particles in the presence of fines and grits accumulated on the roadway surface.

5.2 Petrographic Examinations and Results

5.2.1 Purposes

The petrographic examinations were intended to provide information that may prove to be helpful in judging the mineralogical and the petrographic characteristics of an aggregate as related to the aggregate's frictional resistance. Specifically, the petrographic examinations were made for the following purposes:

1. to determine the physical properties of an aggregate that may be observed by petrographic methods and that may have a bearing on the performance of the aggregate in seal coat surfaces,
2. to identify, describe, and classify the constituents of the aggregate sample,
3. to determine the relative amounts of the constituents of the samples when the constituents differ significantly in a property, such as

hardness, that may be expected to influence the frictional behavior of the aggregate when used in these surface, and

4. to help develop a better understanding of the results of the engineering laboratory tests.

5.2.2 Background

Rocks can be classified into three major groups: igneous, sedimentary or metamorphic. Igneous rocks are formed by cooling and solidification of magma, which is made up wholly or in appreciable portion of molten rocks having the composition of a silicate melt. Sedimentary rocks are formed in two different ways. The first is through mechanical accumulations of minerals and rock fragments produced by disintegration of older rocks by subaerial weathering and erosion. The second is by minerals originating by precipitation from solution. Precipitation may be a purely inorganic chemical process (by evaporation), or it may be caused by organisms (bioganic). Metamorphic rocks are formed by transformations of preexisting rocks while they remain in the solid state. The transformations may be textural, mineralogical, or more commonly both and may or may not involve changes in the overall chemical composition of the rock. High temperatures, high pressures, and/or chemically active pore fluids promote the metamorphic changes.

Since most of the aggregates used in this study are believed to be from the sedimentary group, a review on the formation, composition, and texture of this type of rocks was made. The review was used to develop a procedure for identifying the mineralogical and petrographic properties that may be relevant to the study.

5.2.2.1 Formulation of Sedimentary Rock. As mentioned in the previous section, sedimentary rocks are formed either by the accumulation of decayed mineral and rock fragments or by precipitation. However, postdepositional changes are common in these rocks, which affect both their texture and composition (133). When these changes occur at relatively low temperatures, they are termed diagenetic, rather than metamorphic. Processes that commonly take place during diagenetic changes include compaction, solution, and authigenesis and replacement (3, 4). In the compaction process, the solid particles of a sediment are pressed together by the weight of overlaying material, and the bulk volume is reduced. In the solution process cavities may be formed or stylolites may develop; stylolites are highly irregular surfaces and typically marked by thin seams of insoluble residue. Authigenic changes are characterized by mineral components crystallizing within the deposit itself, either precipitating in open pores or replacing original constituents. Authigenic cements composed of calcite are widespread, but many other cement minerals also occur. The most common authigenic minerals replacing calcite in limestones are quartz, chert, and dolomite.

5.2.2.2 *Composition of Sedimentary Rocks.* Limestone, dolomites, sandstones, and cherts are of the most common sedimentary rocks. Any of these rocks may be composed of a variety of different minerals with various chemical composition and hardness. Common minerals occurring in these rocks include carbonate minerals such as calcite and dolomite and non-carbonate ones such as quartz, chert, pyrite, and feldspar. The calcite and dolomite minerals are soluble by diluted hydrochrolic acid and have hardness values of less than 5 according to Mohs' scale of hardness; calcite has a hardness of 3, while dolomite is slightly harder with a hardness of 3.5 to 4.0. The non-carbonate minerals are harder and insoluble with hardness values ranging from 6 to 7.

5.2.2.3 *Texture of Sedimentary Rocks.* Texture refers to the physical make-up of a rock in particular to its crystallinity and grain size and to the mutual relationship of the individual components (2, 3). Some rocks are composed of grains of all single size, whereas others are made up of large crystals scattered through a matrix of smaller ones.

In aggregates having a wide range of particle sizes, the material is subdivided, for purposes of description, into grains and matrix. No particular size is implied; the terms have reference to the relative size of particles and their deposition in the aggregate. If grains compose more than two-thirds of an aggregate, they will be in contact, and the texture is said to be grain-supported. In this texture, the matrix simply fills in the potential pores between the grains. On the other hand, if the grains are much fewer, loosely packed, and scattered in the matrix, they will appear to be suspended in the matrix, and the texture is said to be matrix-supported.

In the case of limestones and some sandstones, the matrix may be composed of microcrystalline calcite and/or sparry calcite. Microcrystalline calcite, also referred to as calcite mud, consists of interlocking fine calcite crystals which are considered to represent original carbonate mud. Sparry calcite (spar) is relatively clear calcite that has crystallized within the void spaces between grains. Calcite grains occurring in carbonate rocks may take the form of fossils, oids, pellets, and/or intraclasts, all of which consisting of calcite spar or mud with different internal structures.

Crystal grains may be packed together in any fashion, from loosely to very tightly. Packing refers specifically to the arrangement of grains, entirely apart from any authigenic cement that may have crystallized between them. Grains are expected to be loosely packed when authigenic cementation occurs soon after deposition, before progressive postdepositional compaction can occur. Grains, found to be tightly packed, are assumed to have been forced into intimate contact by compaction or to have enlarged by authigenic growth until they interfaced, and the pores between them have been closed. Therefore, the level of intergranular porosity in sedimentary aggregates can be assumed to be greatly dependent on the postdepositional changes that commonly occur in them.

5.2.3 Step-by-Step Examination Procedure

With the previously discussed background in mind, a simple petrographic procedure was developed that included those mineralogical and textural properties identified. Table 5.4 was designed to facilitate the organization of the information to be obtained from the examination. The second part of the table was adapted from Dunhan's limestone classification and modified in a way that it can also be used for the examination of rocks other than sedimentary ones. A total of twenty handful-sized aggregate samples, representing the LMRA, LMST, SIGR, and SDST groups, were examined using the developed procedure. The asphalt coating in the precoated samples had been extracted in accordance with Test Method Tex-210-F-1986 (105) before these samples were examined.

Table 5.4 Summary of Mineralogical and Petrographic Properties

Aggregate Number:	**Color:**
Group: of	**Sphericity:**
Percentage: %	**Roundness:**
Name:	

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
%	Total= %	Total= %	Total= %	Total= %

Grain size	**Roundness**	**Sphericity**

The following steps are suggested for the examination of an aggregate sample:

1. The first step is to acid-etch the handful-sized sample in order that the depositional texture of the carbonate particles was exposed and dust coating removed. Etching is done by placing the sample in a container and adding enough dilute hydrochloric acid to cover the particles being examined. The container is gently swirled for approximately twenty seconds, then rinsed thoroughly with water.

2. The aggregate sample is then subdivided into groups, each having particles with certain textural and mineralogical characteristics that may be related similarly to the overall expected performance. To accomplish this, each aggregate particle is examined on all of its sides using a stereomicroscope at a magnification of 10 to 45 and then placed in a group that best represents its overall textural and mineralogical characteristics. A balance should be sought between a reasonable number of groups and properly representing the expected performance. It is suggested that the maximum number of groups be limited to five, preferably three for crushed limestones.

3. The data for each subdivided group are summarized in separate tables. In the first part of each table, basic information on each group is obtained. This includes the approximate percentage of total sample, name, color, roundness and sphericity of group particles. Roundness is concerned with the curvature of the corners of a particle, and six classes can be distinguished. They are very angular, angular, subangular, subrounded, rounded, and well rounded. Sphericity is a measure of how closely the particle shape approaches that of a sphere. It can be characterized as low, medium, or high.

4. In the second part, each group is classified according to its overall textural appearance into either a matrix-supported or grain-supported texture. Matrix-supported texture particles generally contain mostly groundmass with less amount of grains dispersed in it. Grain-supported texture particles contain more clasts or grains compared to groundmass.

5. The textural evaluation is taken one step further by classifying matrix-supported textures into either less than 10 percent grains or more than 10 percent grains (examples are mudstone limestone for the former and wakestone limestone for the latter). Matrix-supported textures are subdivided into either matrix present between

grains or no matrix present (by the end of this step, only one out of the four texture classes will have been selected for a particular group).

6. Once a texture class is selected, the aggregate group is described in terms of its overall average percentage of voids and the percentage and mineralogy of its matrix and grains. The mineral constituents of the matrix and grains categories are then divided into carbonates and non-carbonates. The percent subtotal in this third part of the table must add up to 100 (i.e., % void + % carbonate matrix + % non-carbonate matrix + % carbonate grains + % non-carbonate grains = 100). In each class under the matrix and grains categories, the different minerals are expressed in terms of their percentages of the percent subtotal for that class.

7. In this step the grains are characterized with respect to their size, roundness, and sphericity. Grain size is evaluated from very-fine to very-coarse, while grain roundness and sphericity were evaluated in the same manner as those of the aggregate particles in the first part of the table.

8. The last step is to take photomicrographs of the texture representing each of the groups encountered in a particular aggregate.

5.2.4 Examination Results

The results for each of the examined aggregate samples are included in Tables A.1 through A.20 of Appendix A. Table 5.5 summarizes information on the groups encountered in each examined aggregate sample. The results collected for each group in a particular aggregate sample was used to calculate weighted results that represent the sample as a whole. For each of the variables included in Table 5.4, this was done simply by multiplying the results obtained in each of the groups by the respective percentages of the groups to the whole sample. The weighted results, shown in Table 5.6, will be used in a later section in a comparison with the engineering laboratory test results and will also be used in the statistical analysis in Chapter 8. The variables in this table stand for the following:

Table 5.5 Summary of Aggregate Sample Groups

Aggregate Number	Description	Group Name	Percentage
1	Crushed limestone	Mudstone Wackestone Packstone	55 40 5
6	Partially crushed mixed gravel	Mudstone Packstone Quartz & Chert Quartzite	50 41 6 3
7	Crushed limestone	Packstone Mudstone Wackestone	65 20 15
9	Partially crushed mixed gravel	Granite gneiss Wackestone Calcareous sandstone	80 10 10
12	Partially crushed mixed gravel	Granite gneiss Sandstone Calcareous sandstone Quartz	40 35 15 10
15	Crushed limestone	Packstone - 1 Packstone - 2	60 40
16	Partially crushed mixed gravel	Chert Dolomitic wackestone Quartz	90 5 5
18	Crushed limestone	Mudstone Wackestone Packstone	55 40 5
22	Partially crushed mixed gravel	Chert Sandstone	65 35

(continued)

Table 5.5 Summary of Aggregate Sample Groups (Continued)

Aggregate Number	Description	Group Name	Percentage
25	Crushed Sandstone	Sandstone	100
31	Crushed Sandstone	Sandstone	100
32	Crushed limestone & dolomitic limestone	Mudstone Packstone Dolomitic wackestone	55 35 10
36	Crushed limestone	Mudstone Packstone Wackestone	55 25 20
37	Crushed limestone	Mudstone Wackestone Packstone	80 15 5
38	Crushed limestone	Mudstone Wackestone Packstone	75 15 10
39	Partially crushed mixed gravel	Mudstone Chert Sandstone	73 24 3
50	Crushed limestone & dolomitic limestone	Wackestone Packstone Dolomitic wackestone Mudstone	45 35 10 10
51	Crushed calcareous sandstone & sandy limestone	Calcareous sandstone Sandy limestone	70 30
52	Crushed calcareous sandstone	Calcareous sandstone	100
53	Crushed dolomite & dolomitic limestone	Dolomite Dolomitic mudstone	75 25

Table 5.6 Weighted Mineralogical and Petrographic Results for the Aggregate Samples Examined

		Laboratory Test		Texture		Matrix			Grains				Mineralogy		Voids			Particle Shape	
AGMT	Agg. No.	MSS	PV	MST	GST	TM	CM	NCM	TG	CG	DG	NCG	CC	NCC	VC	VCG	VCM	SPH	RND
LMRA	15	11.0	34	0	100	14.00	14.00	0.00	69.00	62.00	13.20	7.00	76.00	7.00	17.00	17.00	0.00	LTM	ATSA
LMST	1	2.7	28	95	5	85.00	85.00	0.00	14.75	13.00	0.55	1.75	98.00	1.75	0.25	5.00	0.00	LTM	ANGU
	7	41.0	37	35	65	51.30	51.30	0.00	44.70	43.70	4.00	1.00	95.00	1.00	4.00	5.00	2.14	LOW	ANGU
	18	2.0	27	95	5	80.50	80.50	0.00	14.50	13.50	0.00	1.00	94.00	1.00	5.00	5.00	5.00	LTM	ATSA
	32	17.1	36	65	35	63.25	63.25	0.00	16.00	15.00	0.80	1.00	78.25	1.00	20.75	25.00	18.46	LTM	ATSA
	36	18.0	28	75	25	73.25	73.25	0.00	25.50	25.30	0.00	0.20	98.55	0.20	1.25	5.00	0.00	LOW	ATSA
	37	2.0	29	95	5	84.00	84.00	0.00	10.00	9.20	0.00	0.80	93.20	0.80	6.00	10.00	5.80	MED	ANGU
	38	5.0	27	90	10	84.50	84.50	0.00	12.25	12.25	0.00	0.00	96.25	0.00	3.25	10.00	2.50	LOW	ATSR
	50	36.6	34	65	35	57.80	57.80	0.00	30.45	29.10	1.50	1.35	86.90	1.35	11.75	10.00	12.70	LTM	ANGU
	53	27.6	33	25	75	36.25	36.25	0.00	58.75	57.70	56.75	1.00	94.00	1.00	5.00	5.00	5.00	LOW	ANGU
SIGR	6	2.9	27	56	44	63.75	57.75	6.00	36.25	32.34	0.00	3.91	90.09	9.91	0.00	0.00	0.00	MED	WRTA
	9	8.6	30	10	90	9.00	9.00	0.00	90.00	2.90	1.00	87.10	11.90	87.10	1.00	0.55	5.00	LOW	ATSR
	12	4.5	30	10	90	20.00	3.00	17.00	80.00	0.00	0.00	80.00	3.00	97.00	0.00	0.00	0.00	LTM	ANGU
	16	8.0	27	100	0	94.25	3.75	90.50	5.50	0.74	0.75	4.75	4.50	95.25	0.25	0.00	0.25	LOW	ATSR
	22	1.3	34	65	35	65.50	0.00	65.50	29.50	0.00	0.00	29.50	0.00	95.00	5.00	5.00	5.00	MED	ATSR
	39	1.0	25	94	6	92.00	67.00	25.00	8.00	4.50	0.25	3.50	71.50	28.50	0.00	0.00	0.00	LTM	ATSA
SDST	25	2.6	36	0	100	25.00	0.00	25.00	75.00	0.00	0.00	75.00	0.00	100.00	0.00	0.00	0.00	LOW	ANGU
	31	2.5	41	0	100	25.00	0.00	25.00	75.00	0.00	0.00	75.00	0.00	100.00	0.00	0.00	0.00	LOW	ANGU
	51	12.2	38	30	70	38.00	38.00	0.00	62.00	8.50	8.50	53.50	46.50	53.50	0.00	0.00	0.00	LOW	ANGU
	52	4.0	40	0	100	30.00	30.00	0.00	70.00	2.00	0.00	68.00	32.00	68.00	0.00	0.00	0.00	LOW	ANGU

MST:	percent matrix-supported texture particles,
GST:	percent grain-supported texture particles,
TM:	percent total matrix,
CM:	percent carbonate matrix,
NCM:	percent non-carbonate matrix,
TG:	percent total grains,
CG:	percent carbonate grains,
DG:	percent dolomite grains,
NGG:	percent non-carbonate grains,
CC:	percent carbonate content,
NCC:	percent non-carbonate content,
VC:	percent void content,
VCG:	percent void content in grain-supported texture particles,
VCM:	percent void content in matrix-supported texture particles,
SPH:	particle sphericity, and
RND:	particle roundness.

The abbreviations under the sphericity and roundness variables stand for the following:

LTM:	low to medium,
MED:	medium,
ANGU:	angular,
ATSA:	angular to subangular,
ATSR:	angular to subrounded, and
WRTA:	well rounded to angular.

5.2.5 Texture and Minerals Encountered

The rocks encountered in the examinations were from all of the basic rock types and exhibited a wide variety of textures. The sedimentary rocks were carbonates, sandstones, and cherts. The igneous and methamorphic rocks included quartzite, quartz, granite gneiss, and a minor amount of volcanic rocks.

The carbonate rocks were mudstones, wackestones, packstones, dolomitic limestones, sandy limestones, and dolomites. The matrix in these rocks was comprised predominantly of microcrystalline calcite mud, with partial recrystallization of the mud to sparry calcite occurring in some of them. The carbonate grains observed were comprised of calcite spar, calcite mud, or dolomite. However, siliceous, pyrite, pyrolusite, clay, limonite, and stylolite grains were also found. In addition these rocks were found to have occurred with a wide range of visible porosity. The voids were created by dissolution and/or unconsolidation. Their presence in some of these rocks is considered to increase frictional properties.

Mudstones are matrix-supported limestones with less than ten percent grains. Some mudstones were dense and firm comprising of well compacted microcrystalline calcite, and some others occurred with considerable amount of voids constituting their textures. An example of a dense mudstone, observed in aggregate 36, is shown in Fig. 5.4. A mudstone with 5 percent voids, found in aggregate 37, is shown in Fig. 5.5.

Wackestones are also matrix-supported limestones but with more than ten percent grains. These were found with a wide range of percent voids in their structure. Figures 5.6, 5.7, and 5.8 illustrate wackestones with three level of voids, 0, 5, and 10 percents, seen in aggregates 1, 16, and 32, respectively.

Packstones, which are grain-supported limestones, were found to have occurred with the widest percent of voids seen in this study. Some of them were firmly dense while others were so porous that were described as very soft and friable. The latter were frequently seen to have comprised of calcite and dolomite grains that were not well compacted. Figures 5.9 through 5.12 show packstones with four level of voids, 0, 5, 10, and 25 percents, from aggregates 6, 1, 50, and 32, respectively.

The presence of dolomite grains, which have a hardness slightly greater than that of calcite grains, is expected to enhance the frictional properties of these carbonate rocks. When more than a trace of dolomite grains was observed in a limestone, the rock was referred to as a dolomitic limestone. Similarly, when quartz grains were found in sizable amounts, the limestone was referred to as a sandy limestone. An example of a sandy limestone from aggregate 51 is shown in Fig. 5.13.

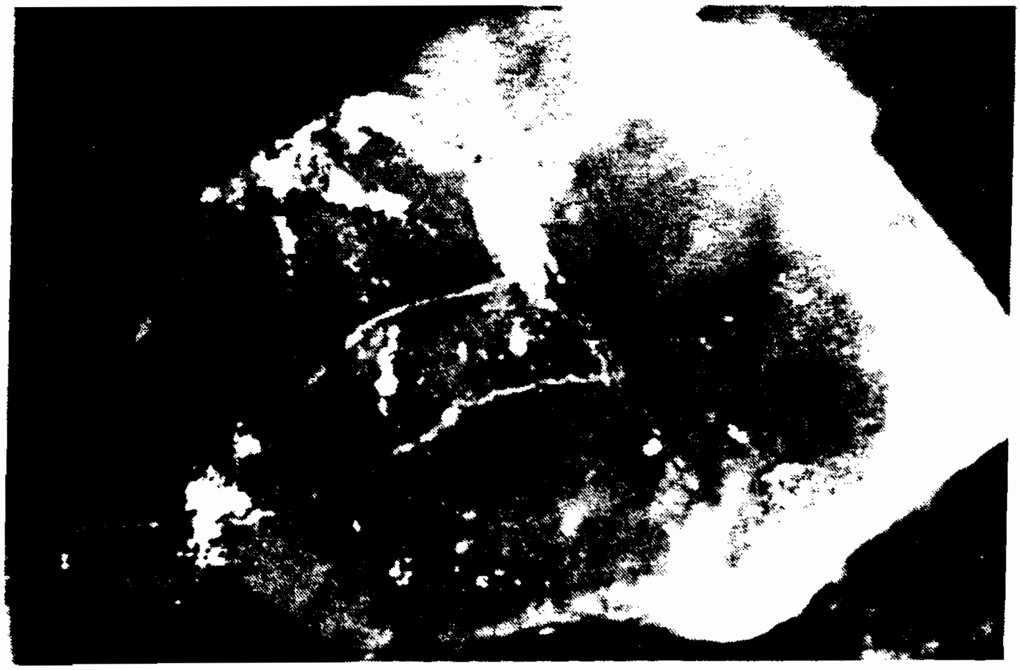


Fig. 5.4 Dense mudstone from Aggregate 36

Fig. 5.5 Mudstone with 5 percent voids from Aggregate 37

Fig. 5.6 Wackestone with 0 percent voids from Aggregate 1

Fig. 5.7 Wackestone with 5 percent voids from Aggregate 16

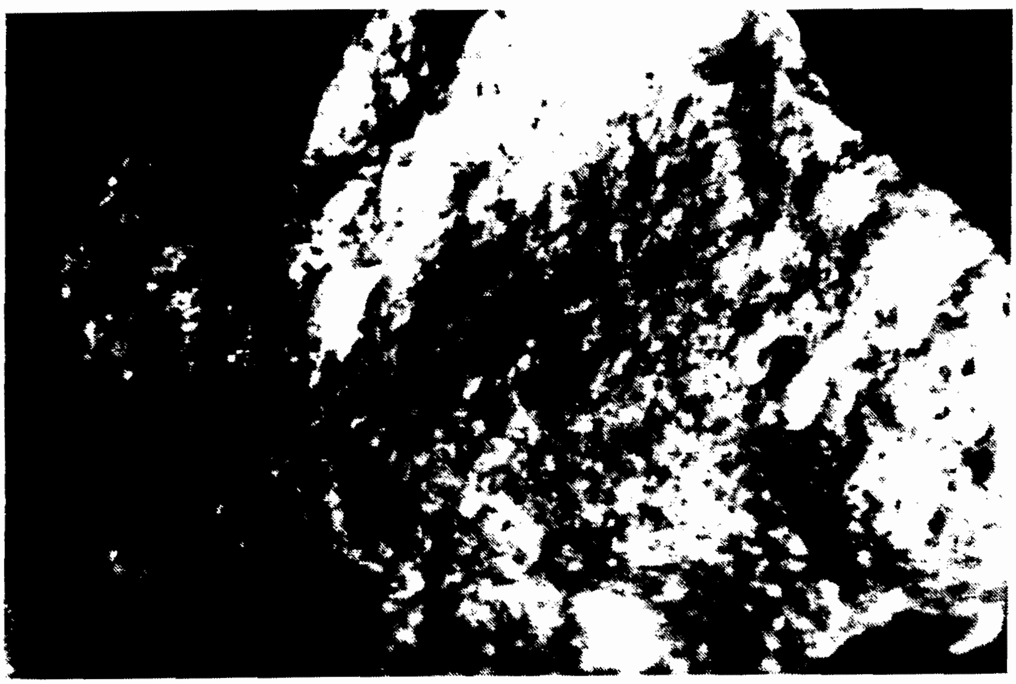


Fig. 5.8 Wackestone with 10 percent voids from Aggregate 32

Fig. 5.9 Packstone with 0 percent voids from Aggregate 6

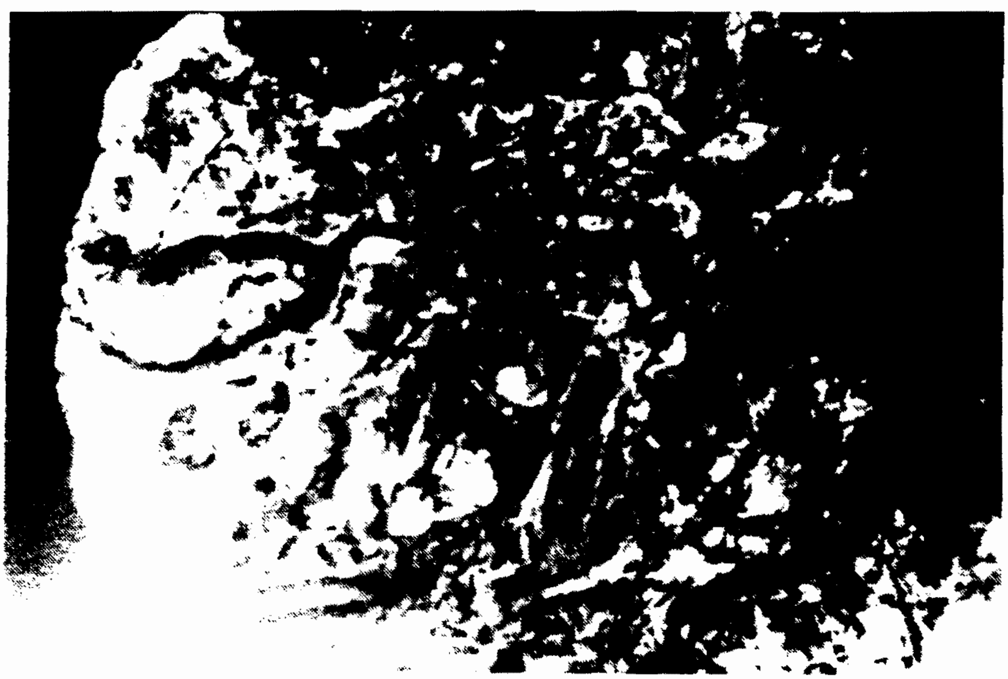


Fig. 5.10 Packstone with 5 percent voids from Aggregate 1

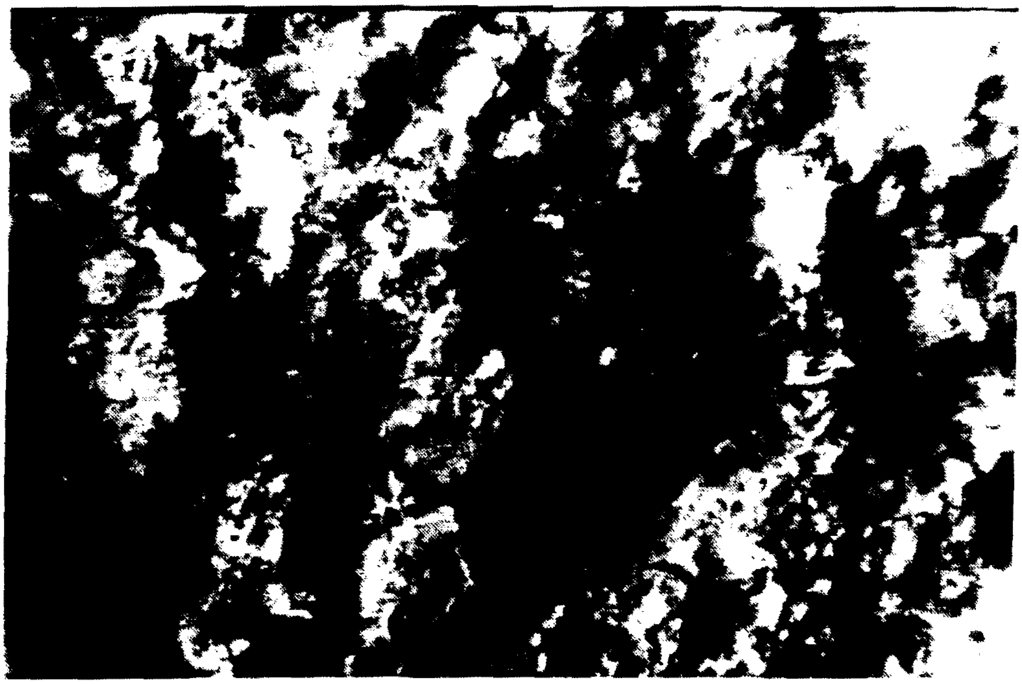


Fig. 5.11 Packstone with 10 percent voids from Aggregate 50

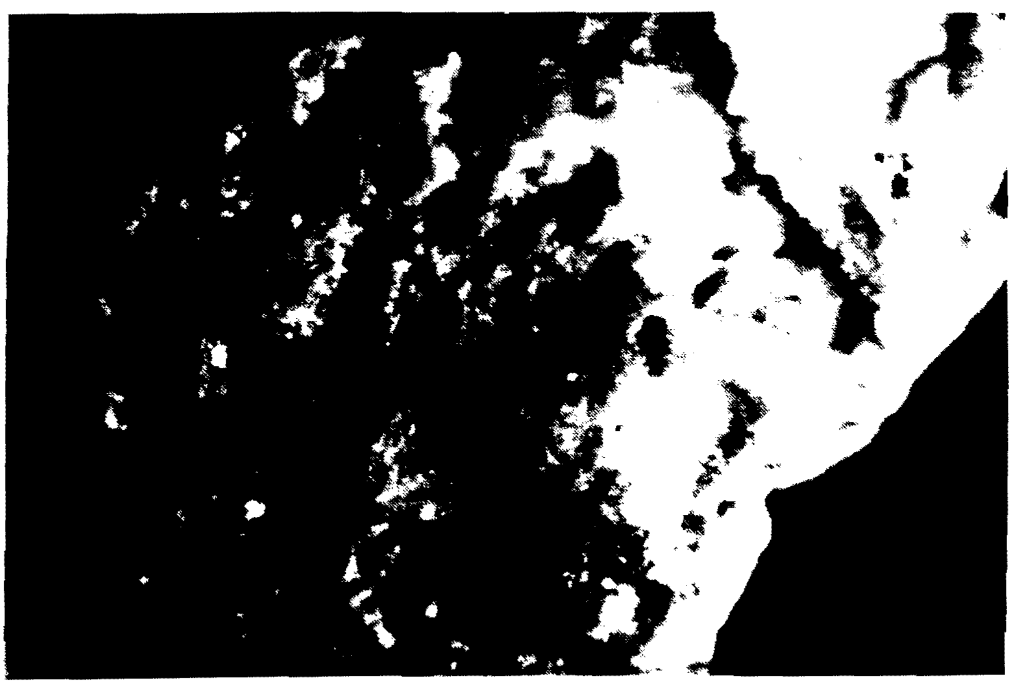


Fig. 5.12 Packstone with 25 percent voids from Aggregate 32

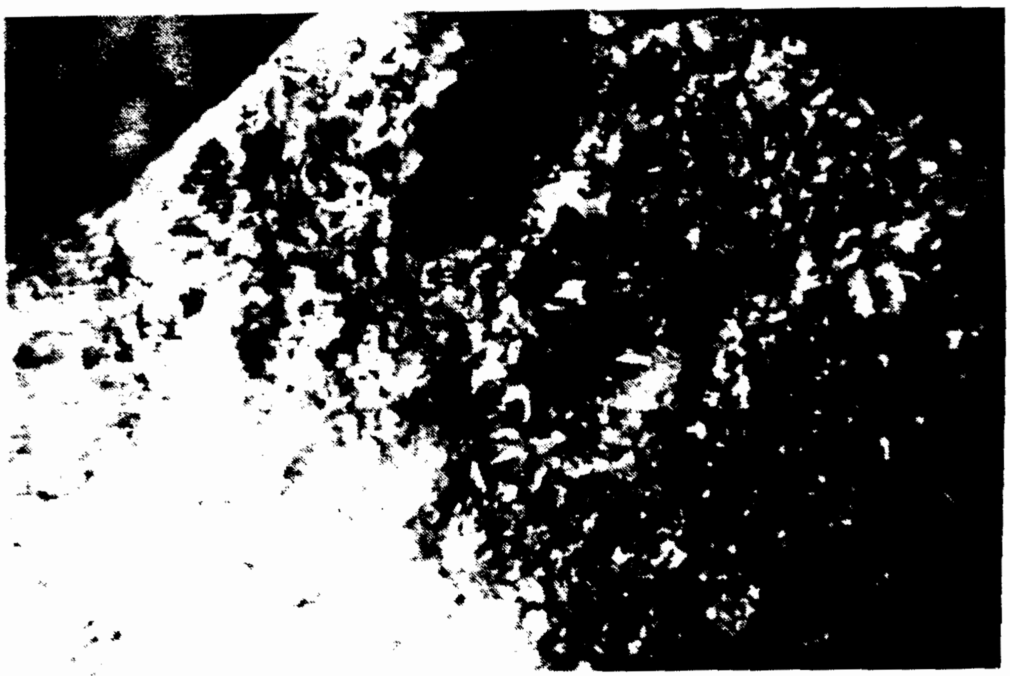


Fig. 5.13 Sandy limestone from Aggregate 51

Dolomites were also observed that were predominantly comprised of dolomite grains with calcite mud filling the pores between them. Thus, they were classified as grain-supported. A dolomite from aggregate 53, with 5 percent voids, is manifested in Fig. 5.14.

The other types of sedimentary rocks encountered in this study were sandstones and cherts. Sandstones are commonly composed of abundant quartz, feldspar, and/or carbonate grains cemented together with calcite or silica and perhaps some minor hematite and other impurities. Therefore, they are regarded to have an excellent grain-supported texture. When the grains are cemented with calcite, the rock is referred to as a calcareous sandstone. This rock is believed to have excellent differential wear, which is a quality of a great return on frictional properties. Figures 5.15 and 5.16 illustrate two cases of calcareous sandstones seen in aggregates 9 and 51, respectively. A sandstone made up of quartz grains cemented with silica is shown in Fig. 5.17.

Chert is a microcrystalline quartz formed by either the accumulation of organism sediment or precipitation from solution. Although cherts are hard and, thus, are expected to aid in frictional resistance, they were classified as matrix-supported because of their lack of surface texture. Cherts with 0 and 5 percents voids, observed in aggregate 39 and 22, respectively, are shown in Figs. 5.18 and 5.19.

From the metamorphic and igneous groups, quartzite and granite gneiss were identified. Quartzite is a metamorphosed quartz sandstone composed of quartz grains with silicate cement that has grown in optical continuity around each fragment. Granite gneiss, also referred to as metagranite, is a metamorphic rock which contains mostly quartz with some feldspar and mica in a conspicuously foliated, banded, or aligned pattern. Both rocks were considered to have a good texture. They were classified to be lacking in matrix. In aggregate 12, the quartzite observed was thought to have been from an igneous origin although its texture was granular. The quartzite-appearing particles, an example of which is shown in Fig. 5.20, were grouped with the granite gneiss of that aggregate, seen in Fig. 5.21. Another granite gneiss found in aggregate 9 is shown in Fig. 5.22.

Lastly, quartz, shown in Fig. 5.23, was encountered in this study. This is a

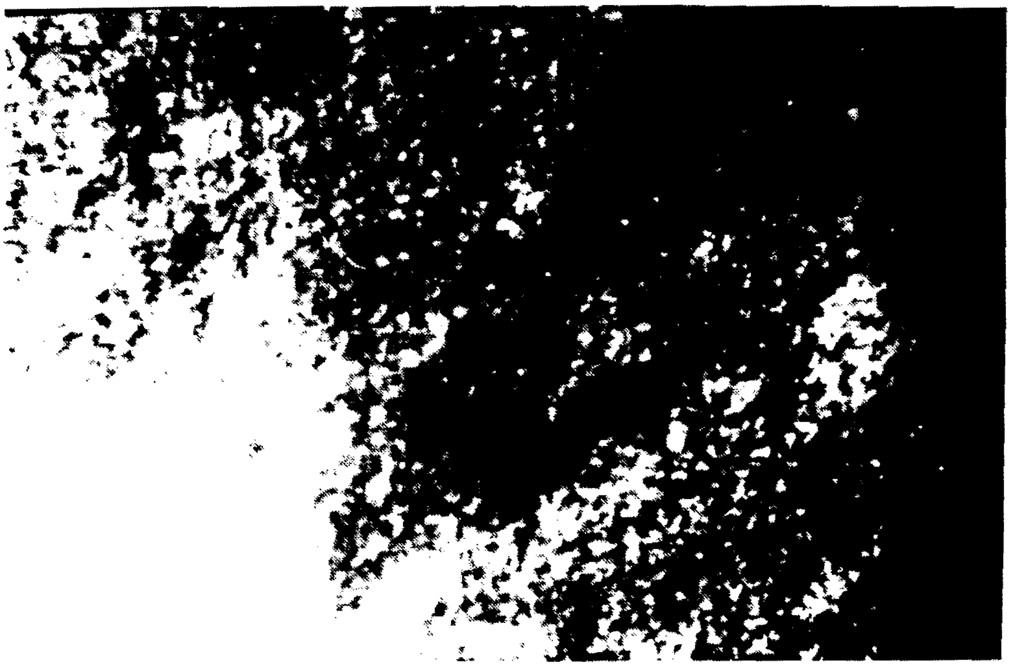


Fig. 5.14 Dolomite with 5 percent voids from Aggregate 53

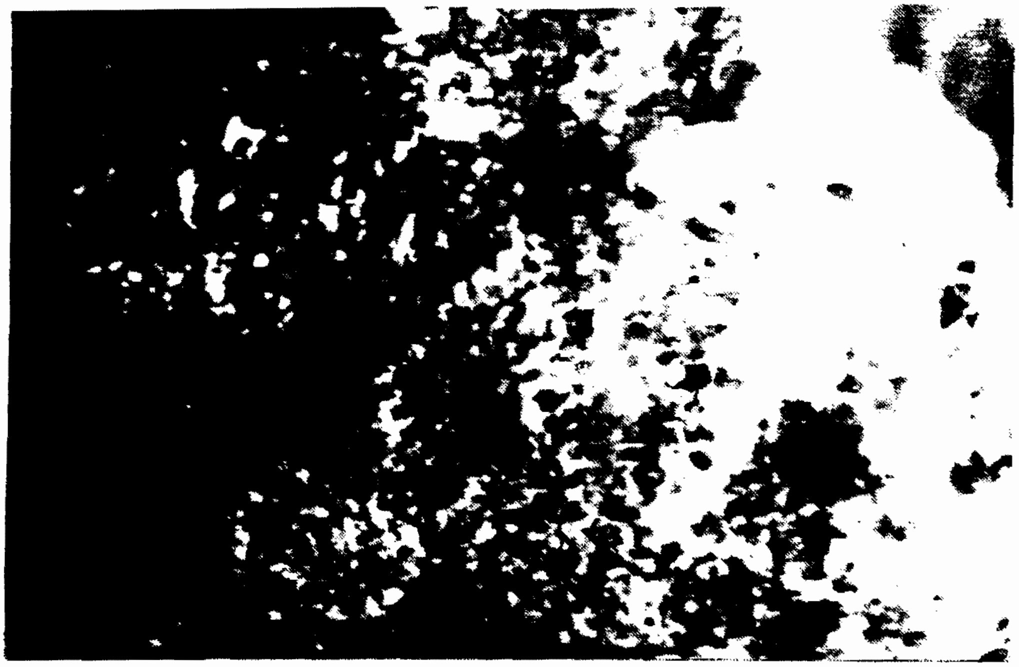


Fig. 5.15 Calcareous sandstone from Aggregate 9

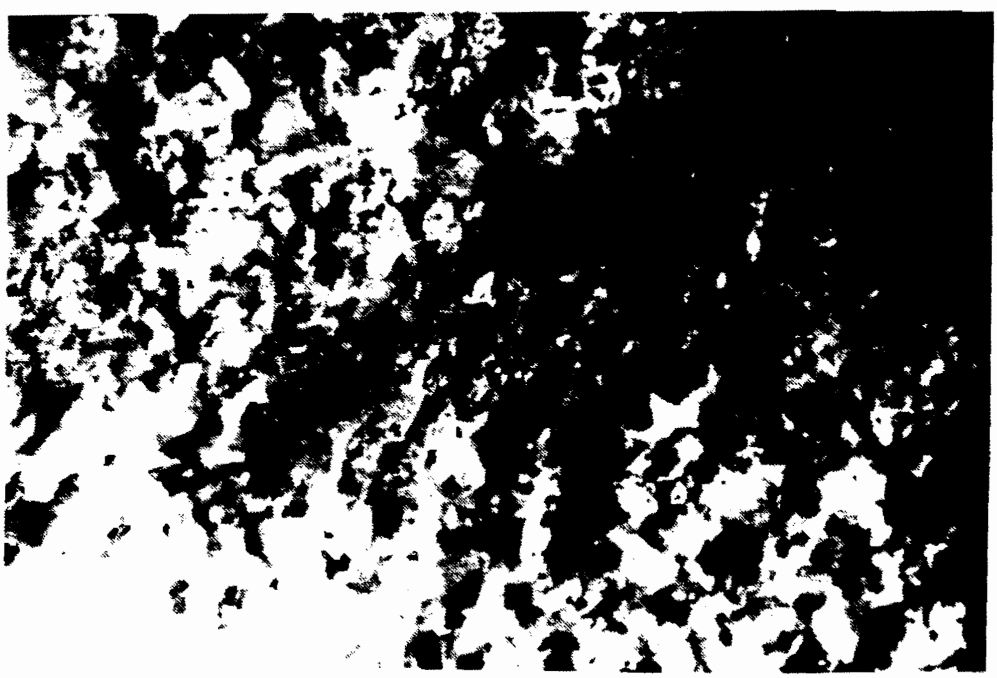


Fig. 5.16 Calcareous sandstone from Aggregate 51

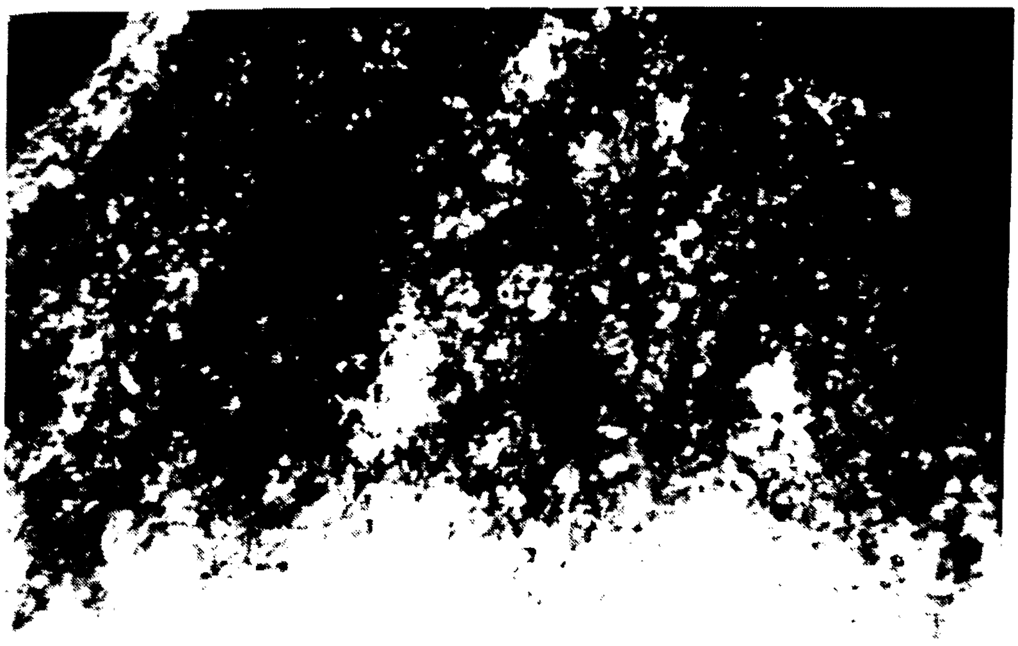


Fig. 5.17 Sandstone from Aggregate 25

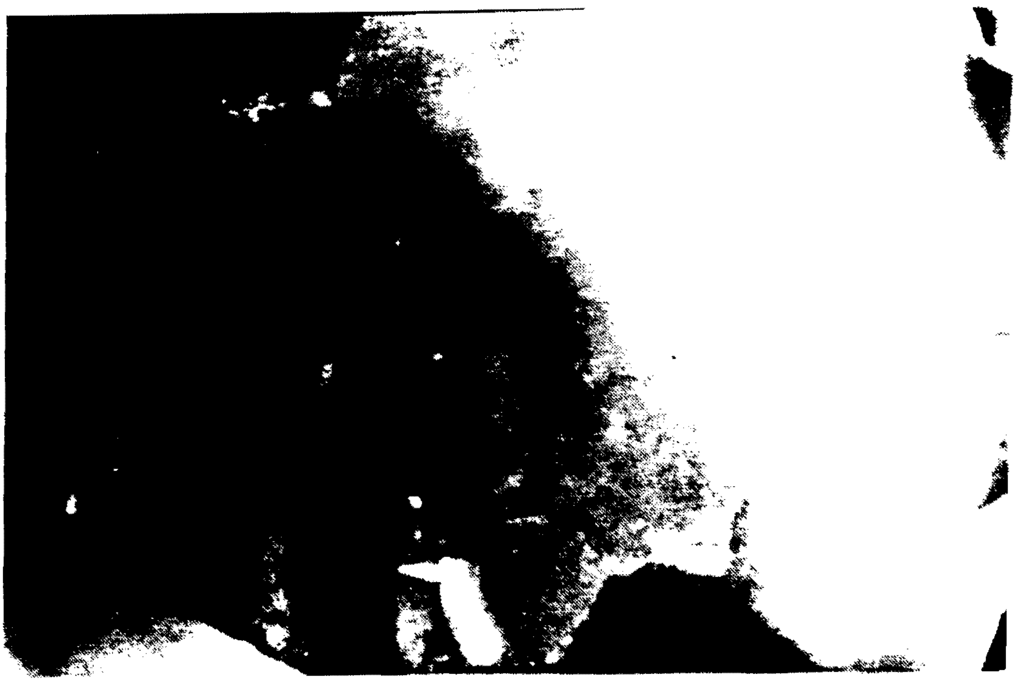


Fig. 5.18 Chert with 0 percent voids from Aggregate 39

Fig. 5.19 Chert with 5 percent voids from Aggregate 22

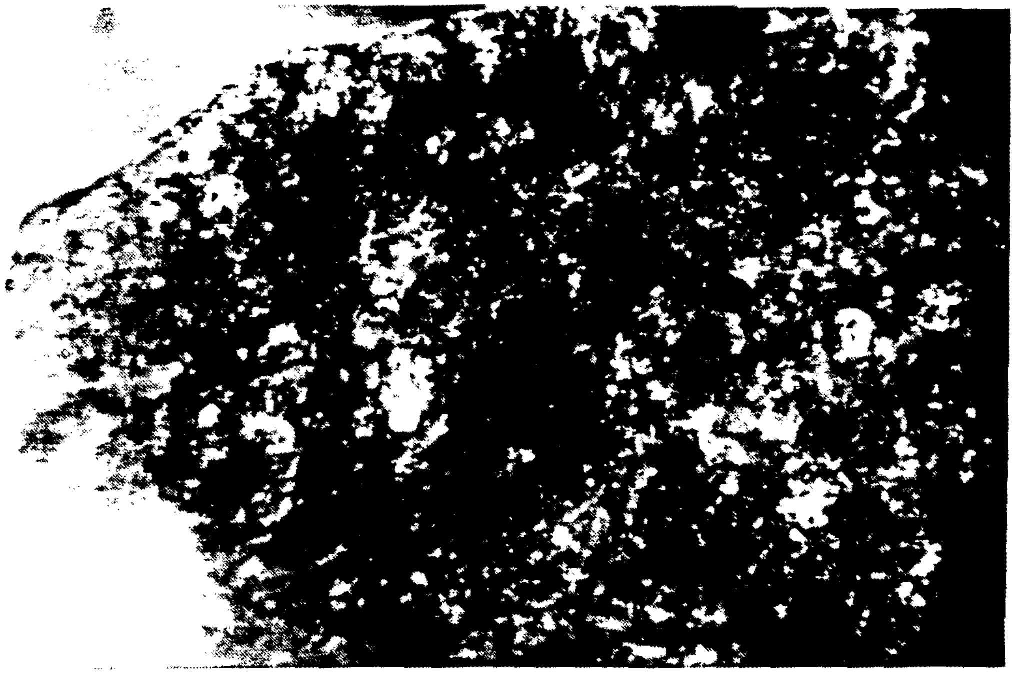


Fig. 5.20 Quartzite from Aggregate 12

Fig. 5.21 Granite Gneiss from Aggregate 12

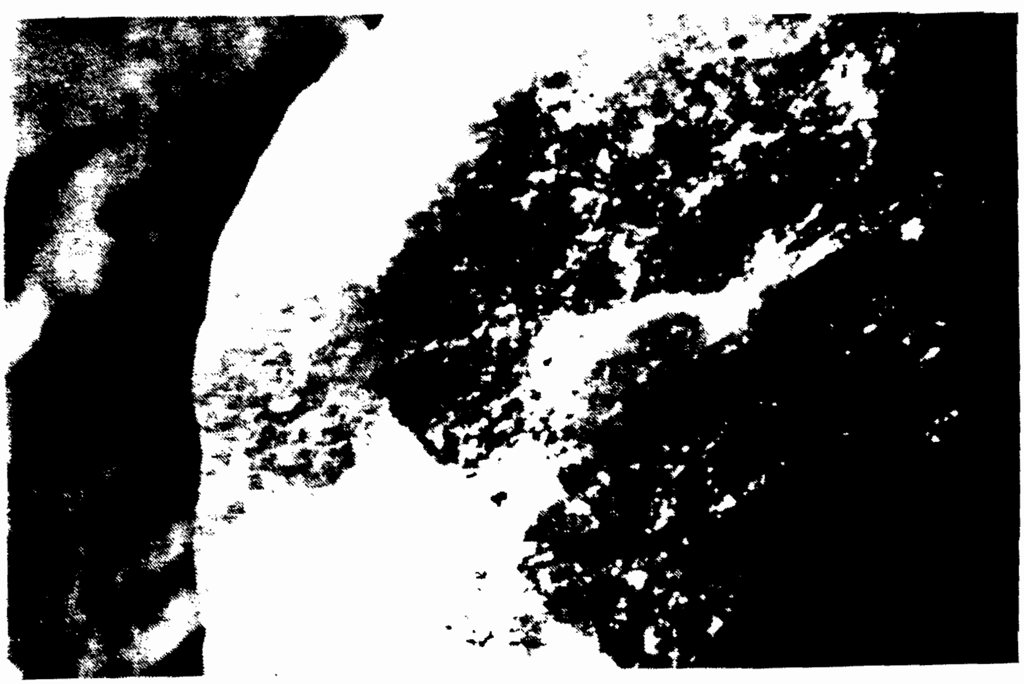


Fig. 5.22 Granite Gneiss from Aggregate 9

Fig. 5.23 Quartz from Aggregate 6

hard rock with no surface texture, that was therefore considered to be matrix-supported with less than ten percent grains.

5.3 Interrelationships Among Testing Methods

5.3.1 Correlations Among Engineering Laboratory Tests

Aggregate properties are expected to have a major role in explaining the variability in the frictional performance of seal coat overlays. The consideration of including any two or more laboratory tests in the formulation of prediction models necessitates that their intercorrelations be first understood. This is because when two tests are found to be highly correlated, their inclusion in any prediction model may cause a multicollinearity problem. Multicollenearity may arise because the two tests contribute overlapping information to the reduction in the error sum of squares of a particular model.

Since the PV test is regarded as the first test to be considered for inclusion in the prediction models, its correlation with all the tests describing some form of aggregate physical deterioration was studied. Also, the correlations among most of the other tests were investigated. The discussion is divided into two sections. The first section deals with the correlations among those tests for which data is available on all aggregates. The second section covers the correlations studied using the data of the uncoated aggregates.

5.3.1.1 All Aggregates. The correlations between the PV test and each of the MSS, LA, and INRD tests were first studied. These are included in Figs. 5.24 through 5.32. When all natural aggregates were considered, no correlation was found between the PV and MSS tests. However, when the data were grouped according to AGMT, a high correlation coefficient of 0.85 was found for the LMST group, not including

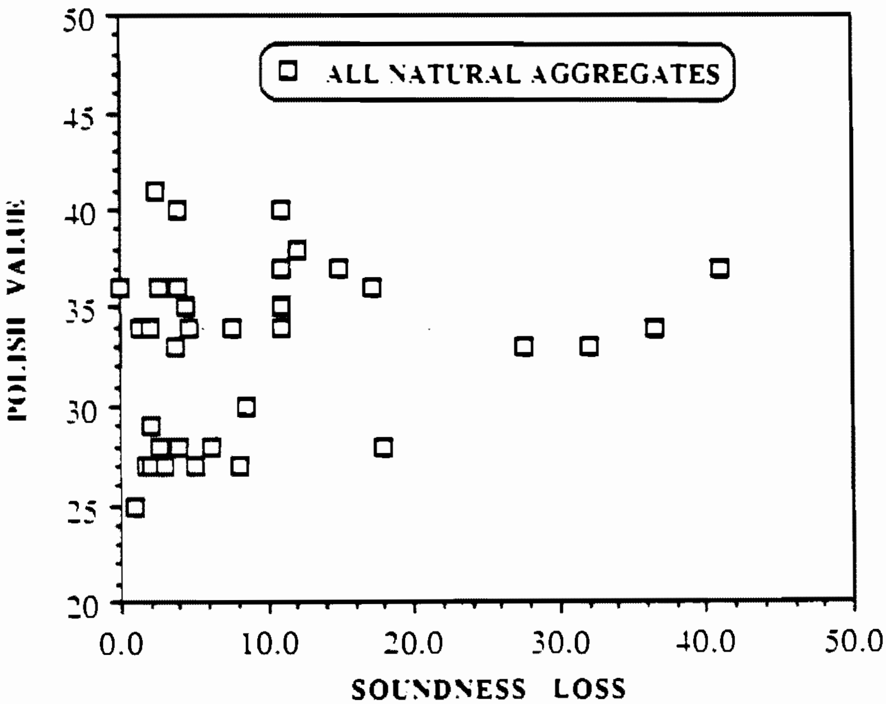


Fig. 5.24 Correlation between the Results of the Polish Value and Soundness Tests for All Natural Aggregates

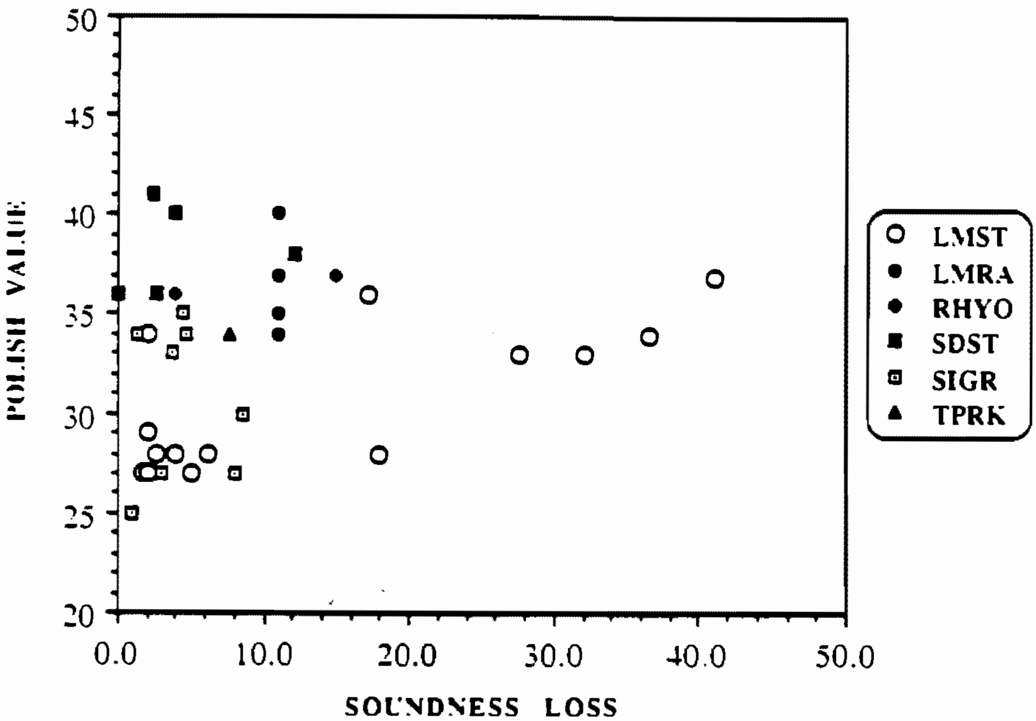


Fig. 5.25 Correlation between the Results of the Polish Value and Soundness Tests for the Different Natural Aggregate Materials Used

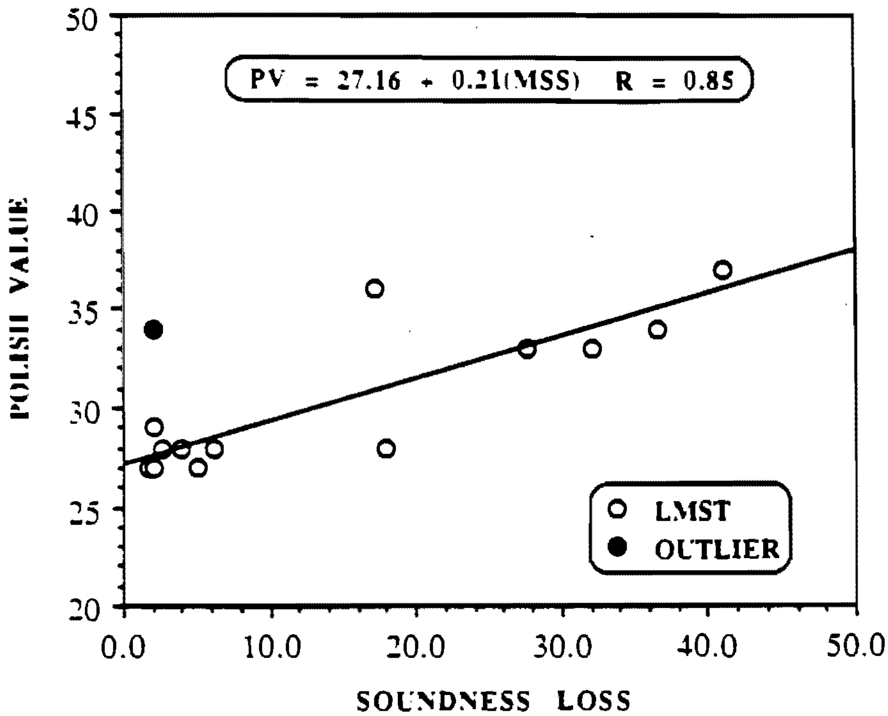


Fig. 5.26 Correlation between the Results of the Polish Value and Soundness Tests for the LMST Aggregates

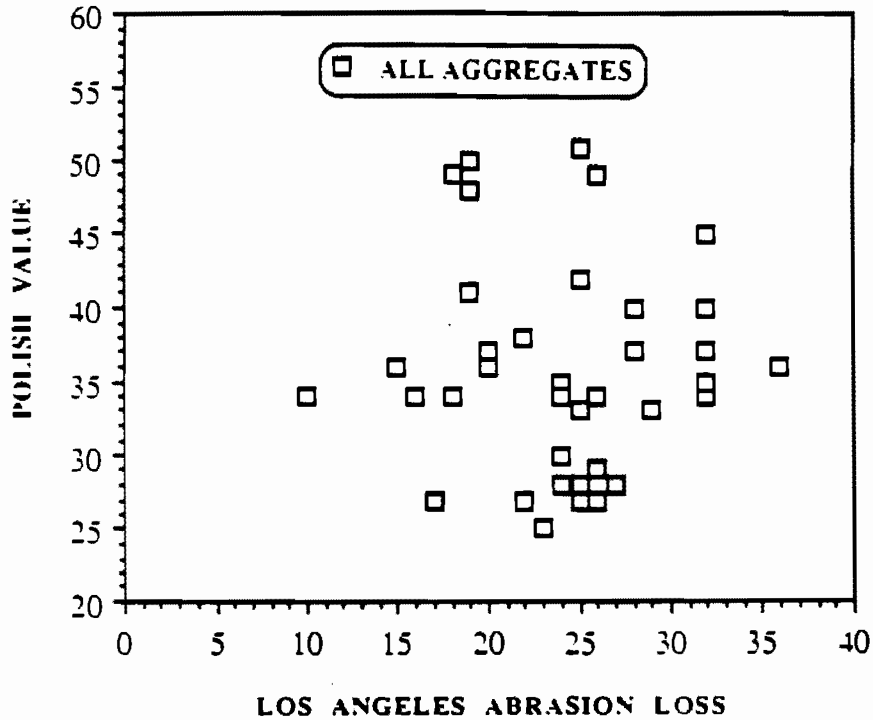


Fig. 5.27 Correlation between the Results of the Polish Value and Los Angeles Abrasion Tests for All Aggregates

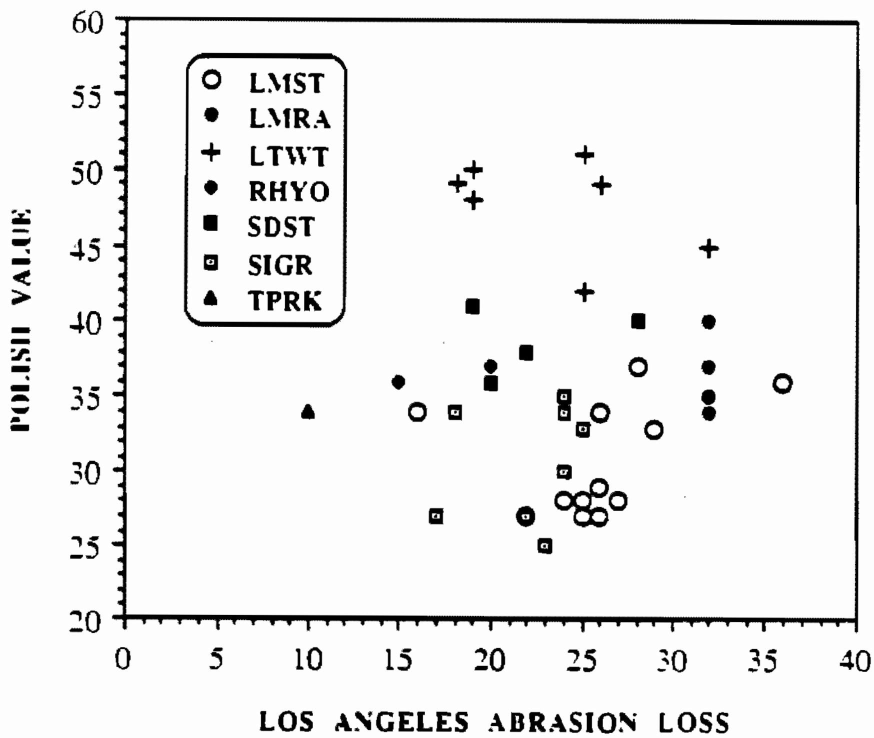


Fig. 5.28 Correlation between the Results of the Polish Value and Los Angeles Abrasion Tests for the Different Aggregates Used

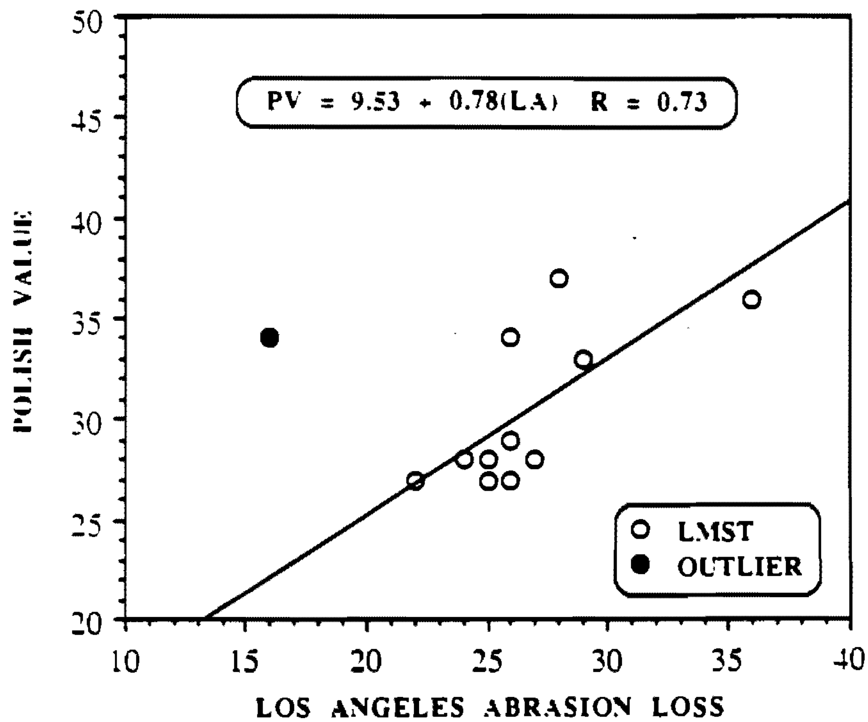


Fig. 5.29 Correlation between the Results of the Polish Value and Los Angeles Abrasion Tests for the LMST Aggregates

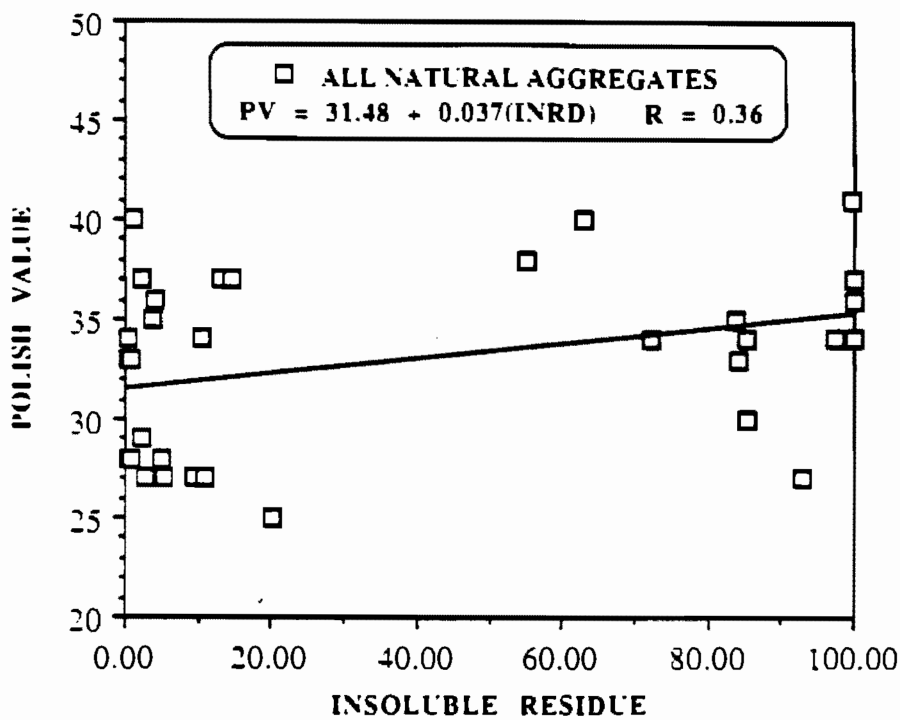


Fig. 5.30 Correlation between the Results of the Polish Value and Insoluble Residue Tests for All Natural Aggregates

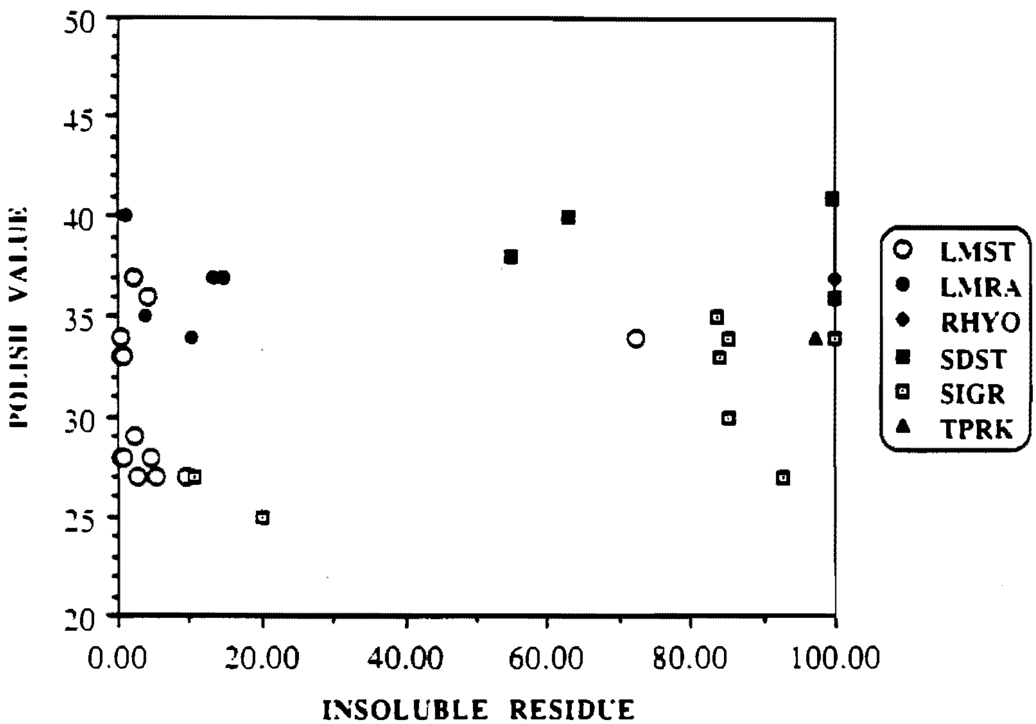


Fig. 5.31 Correlation between the Results of the Polish Value and Insoluble Residue Tests for the Different Natural Aggregates Used

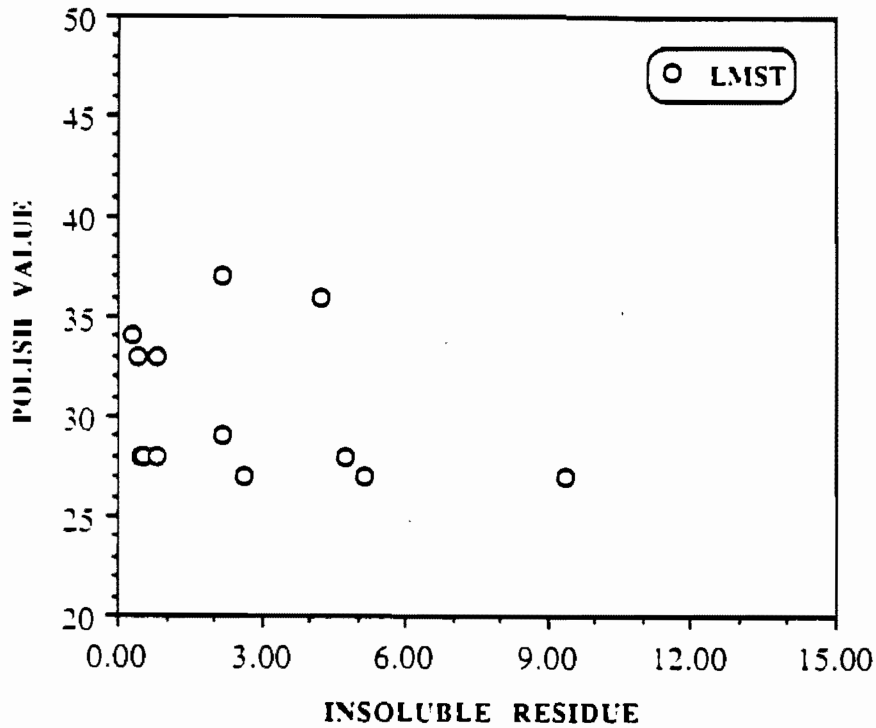


Fig. 5.32 Correlation between the Results of the Polish Value and Insoluble Residue Tests for the LMST Aggregates

aggregate 30 which was viewed as an outlier. Similarly, the PV and LA tests did not exhibit any general correlation, but a high correlation coefficient of 0.73 was found for the LMST group. A poor overall correlation coefficient of 0.36 was detected between the PV and INRD, but no correlation was seen between them in the LMST group nor in any of the other AGMT groups.

The correlation between the MSS and LA tests was then examined and found to have followed a nearly exponential path, shown in Fig. 5.33. When the data were grouped in Fig. 5.34 it was noticed that the LMST group had significantly contributed to the shape of that path. That was because the MSS losses for this group varied widely, from 17.1 to 41.0 percent, for a narrow range of LA losses, 26 to 30 percent.

5.3.1.2 Uncoated Aggregates. In this section, the correlation study included the PV test with each of the SPGR, ABSP, FRTH, ADI, and Texas degradation tests. Figures 5.36 through 5.43 graphically illustrate the observed correlations. Poor overall correlations were noticed between the PV and all of these tests, whereas good correlations were seen between the PV and some of these tests in the LMST group. In this group, high correlation coefficients of 0.95 and 0.84 were found between the PV and SPGR and ABSP, respectively. In addition, a moderate coefficient of correlation of 0.53 was obtained for the correlation between the PV and Texas degration tests.

Also investigated were the correlations between each of the LA and MSS tests and the other tests in their respective test groups. These are presented in Figs. 5.44 through 5.49. The ADI test was found to have no correlation with the LA test, while a moderate overall coefficient of 0.58 was obtained for the correlation between the LA and Texas degradation tests. The overall correlation of the FRTH test with the MSS test revealed a high coefficient of 0.81. Another high correlation coefficient of 0.82 was also seen in the LMST group, with an almost perfect coefficient of 0.98 observed in the SIGR group.

Lastly, in an attempt to find out what causes some aggregates, especially the LMST ones, to have high MSS losses, the correlation between the ABSP and MSS tests was investigated. As illustrated in Fig. 5.49, the two test were found to be highly correlated with an overall correlation coefficient of 0.81. Much of the strength of this correlation can be attributed to the slightly stronger correlation observed between the

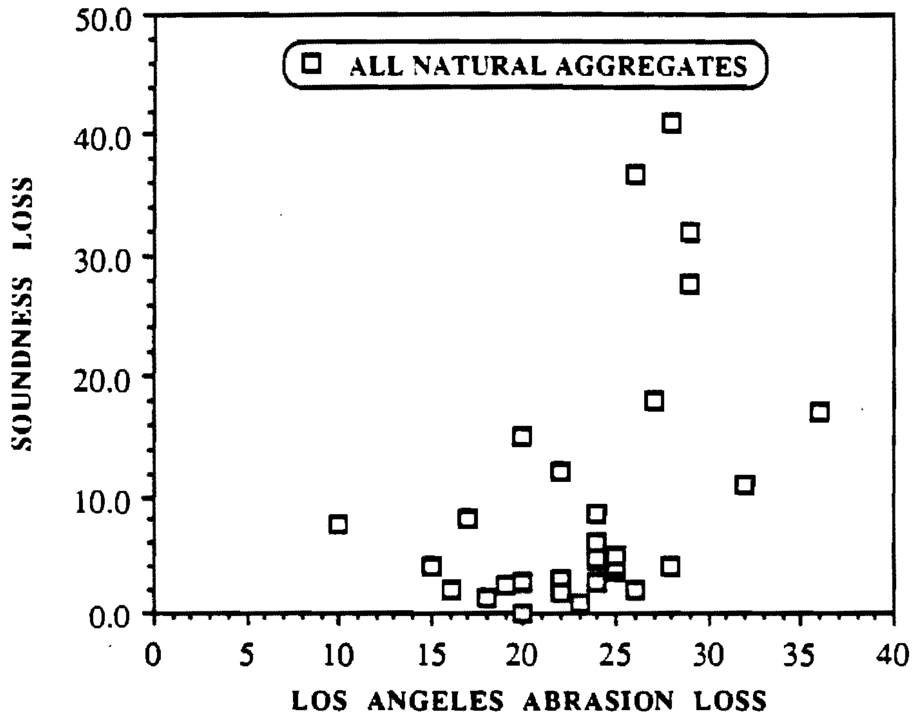


Fig. 5.33 Correlation between the Results of the Soundness and Los Angeles Abrasion Tests for All Natural Aggregates

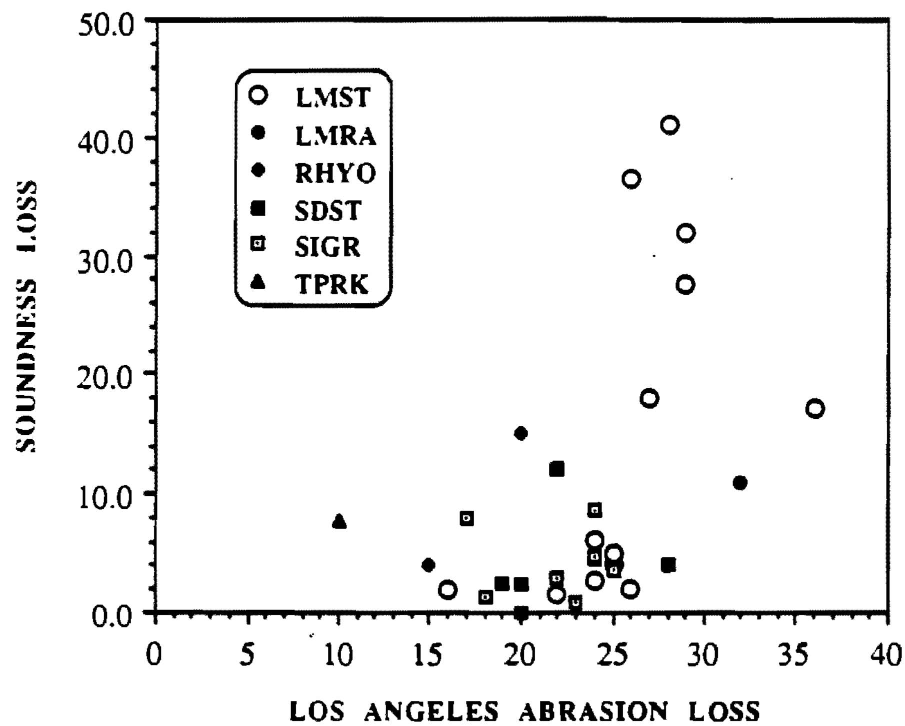


Fig. 5.34 Correlation Between the Results of the Soundness and Los Angeles Abrasion Tests for the Different Natural Aggregates Used

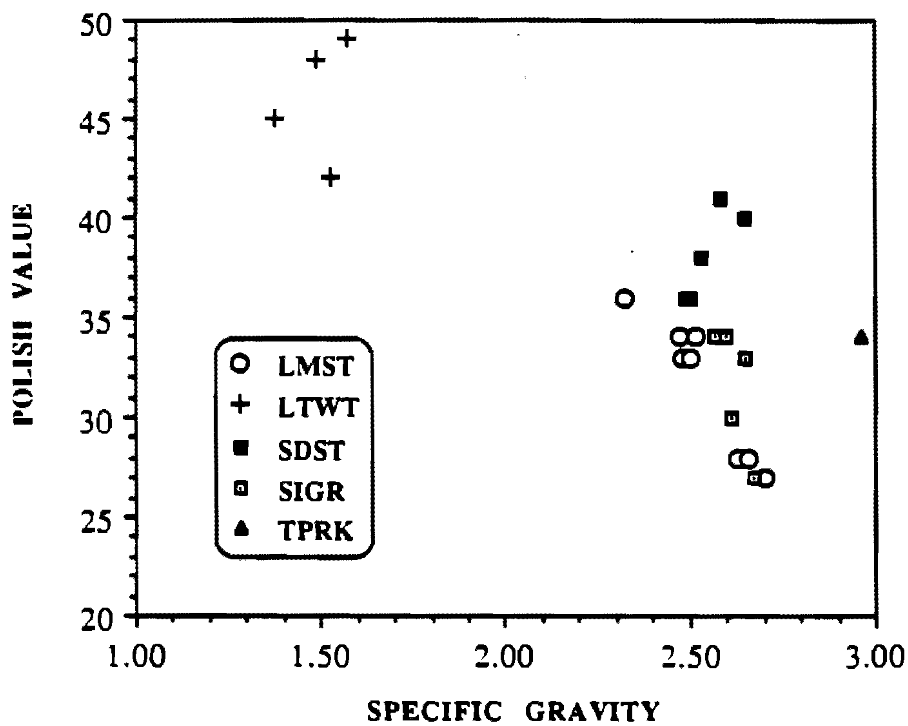


Fig. 5.35 Correlation Between the Results of the Polish Value and Specific Gravity Tests for the Different Uncoated Aggrgates Used

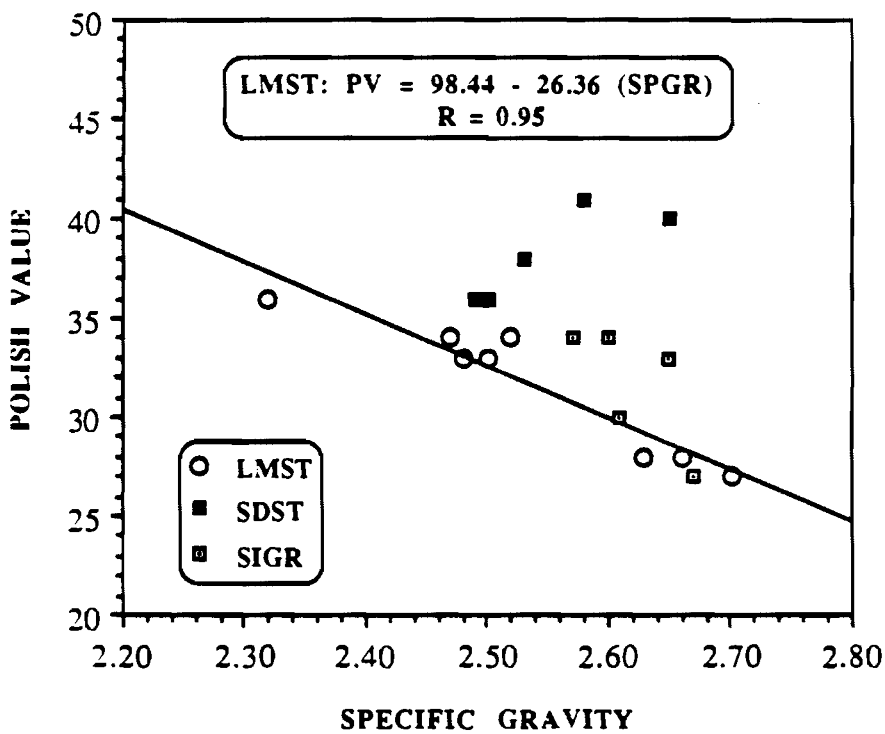


Fig. 5.36 Correlation Between the Results of the Polish Value and Specific Gravity Tests for the Uncoated LMST, SDST, and SIGR Aggregates Used

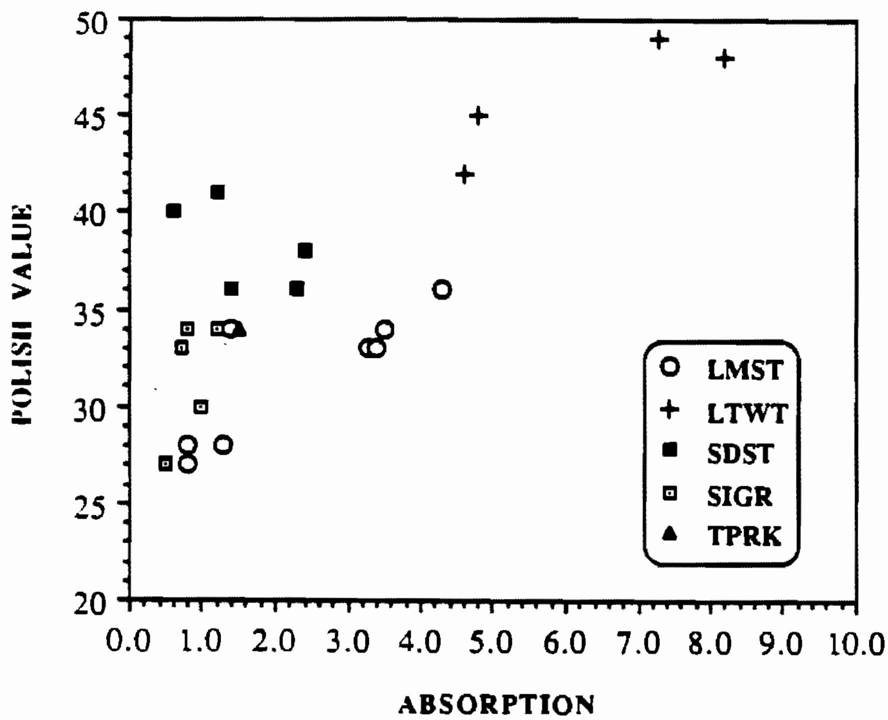


Fig. 5.37 Correlation Between the Results of the Polish Value and Absorption Tests for the Different Uncoated Aggregates Used

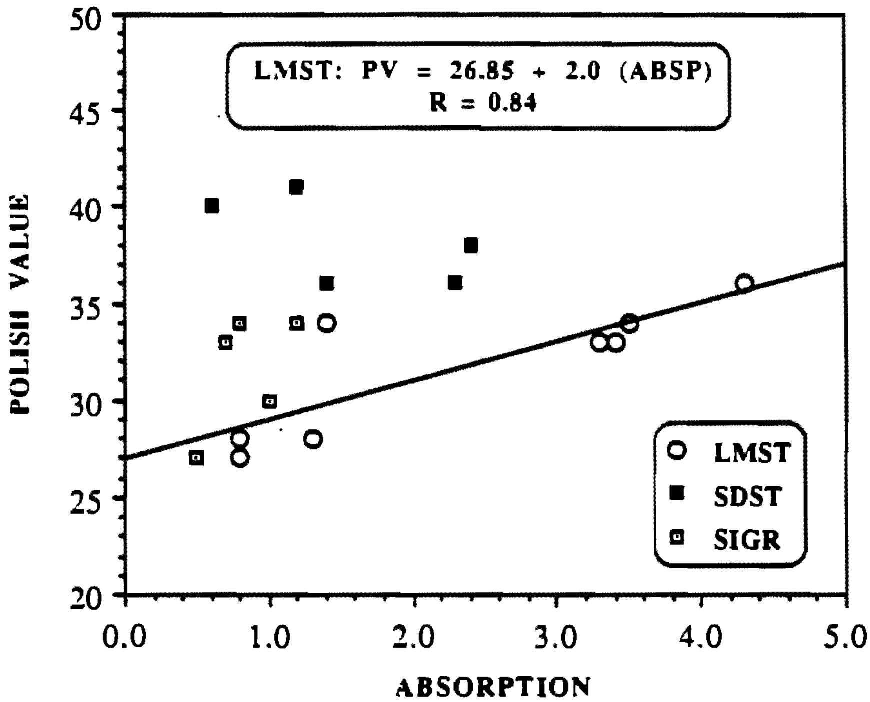


Fig: 5.38 Correlation Between the Results of the Polish Value and Absorption Tests for the Uncoated LMST, SDST, and SIGR Aggregates Used

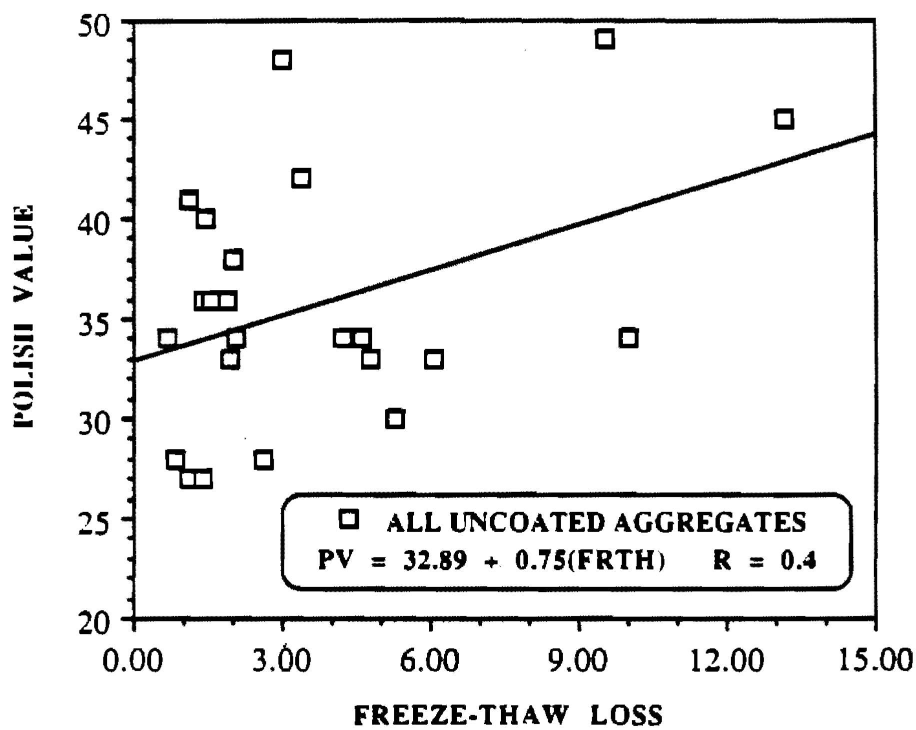


Fig. 5.39 Correlation Between the Results of the Polish Value and Freeze-Thaw Tests for All Uncoated Aggregates

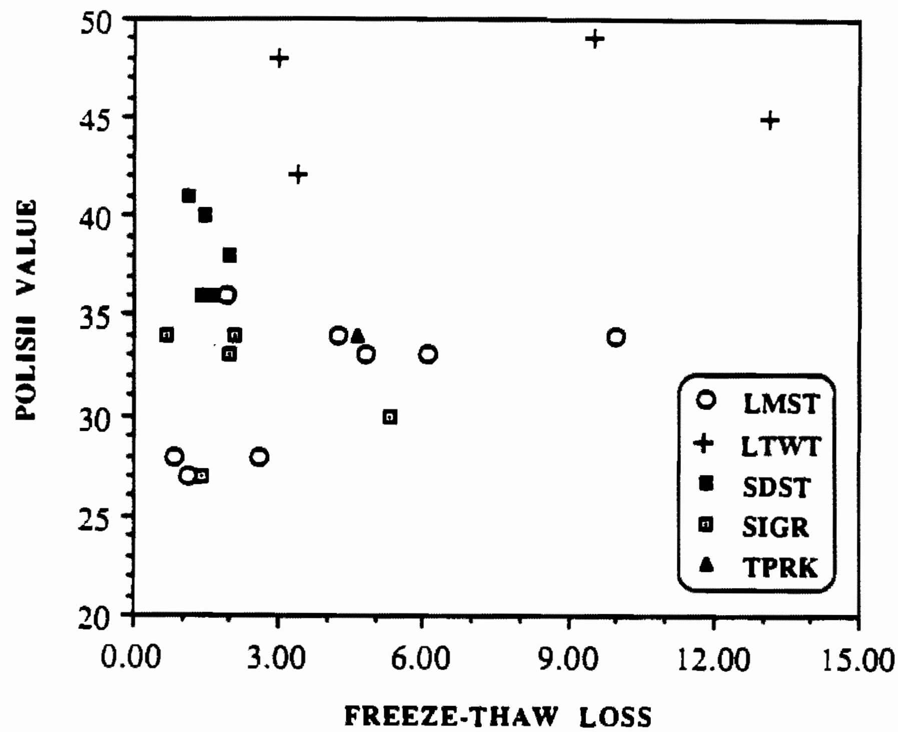


Fig. 5.40 Correlation Between the Results of the Polish Value and Freeze-Thaw Tests for the Different Uncoated Aggregates Used

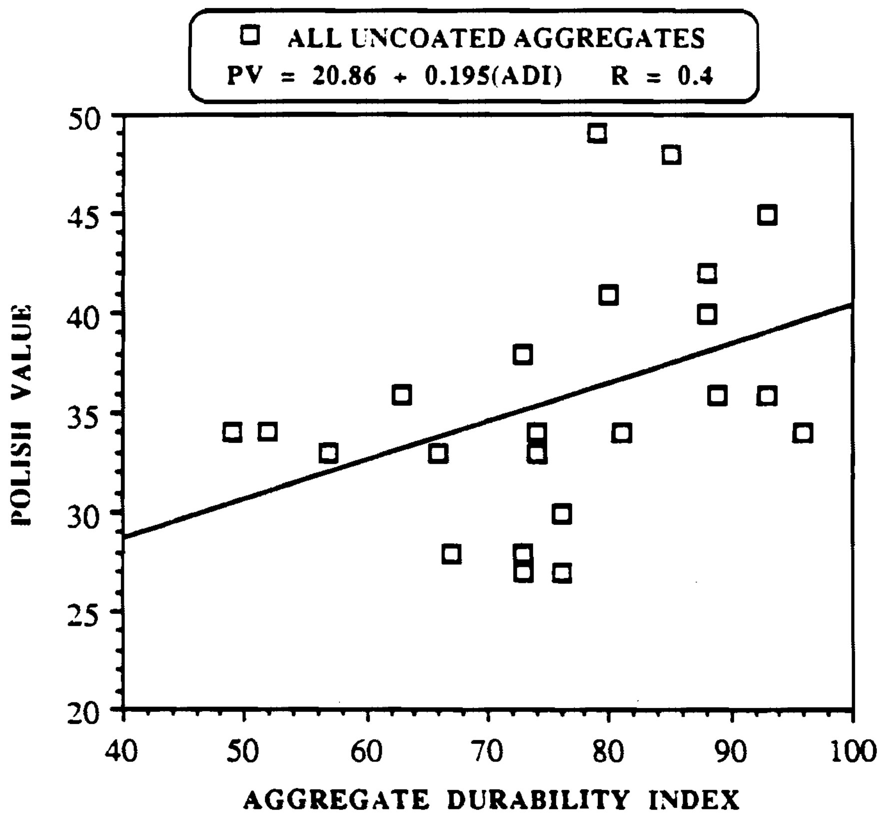


Fig. 5.41 Correlation Between the Results of the Polish Value and Aggregate Durability Tests for All Uncoated Aggregates

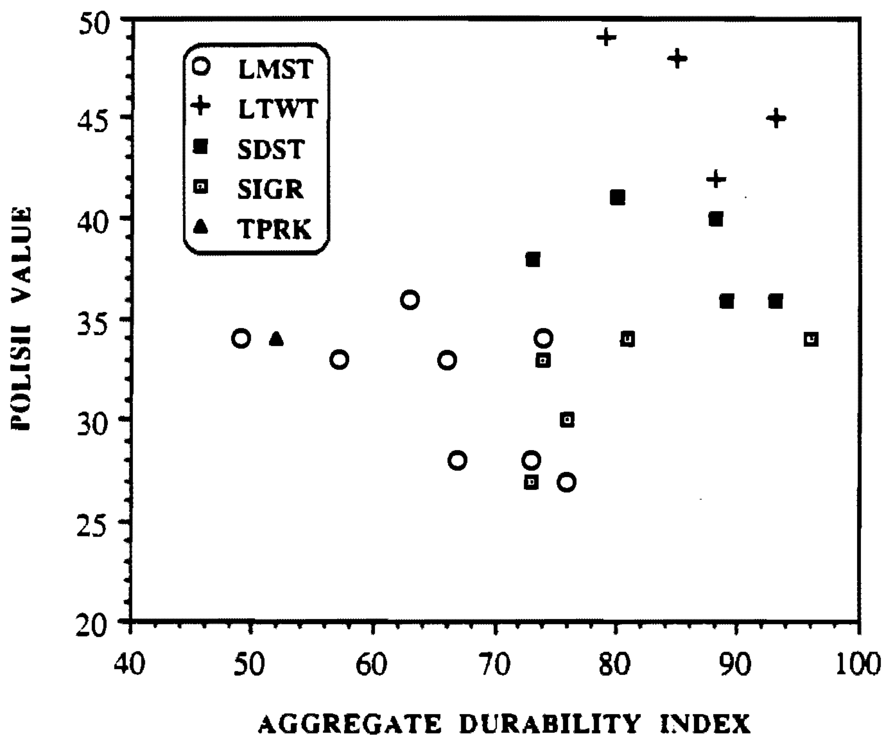


Fig. 5.42 Correlation Between the Results of the Polish Value and Aggregate Durability Tests for the Different Uncoated Aggregates Used

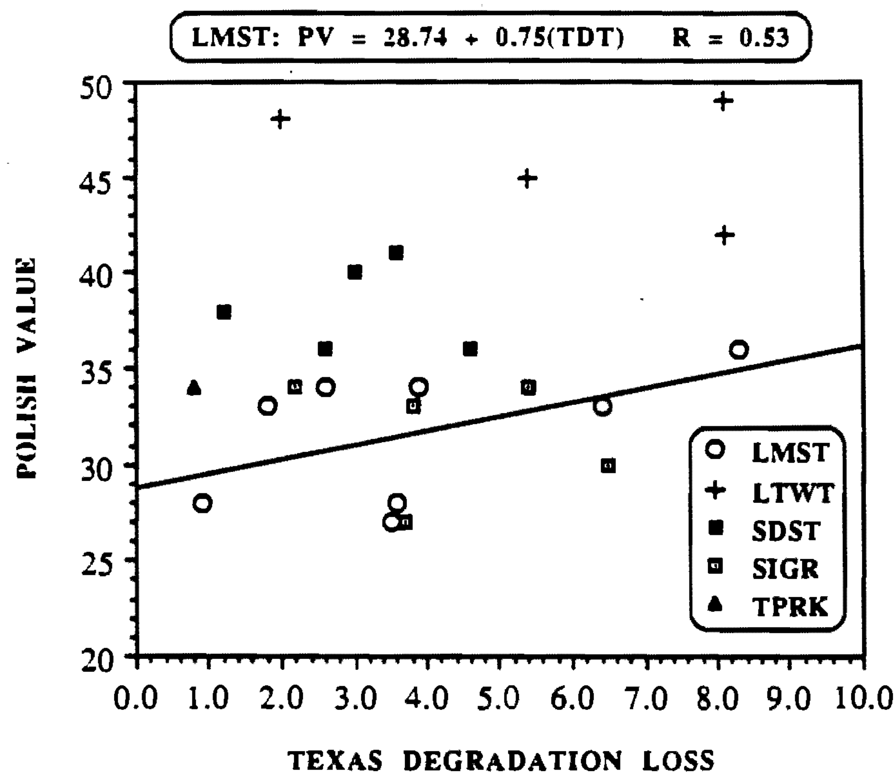


Fig. 5.43 Correlation Between the Results of the Polish Value and Texas Degradation Tests for the Different Uncoated Aggregates Used

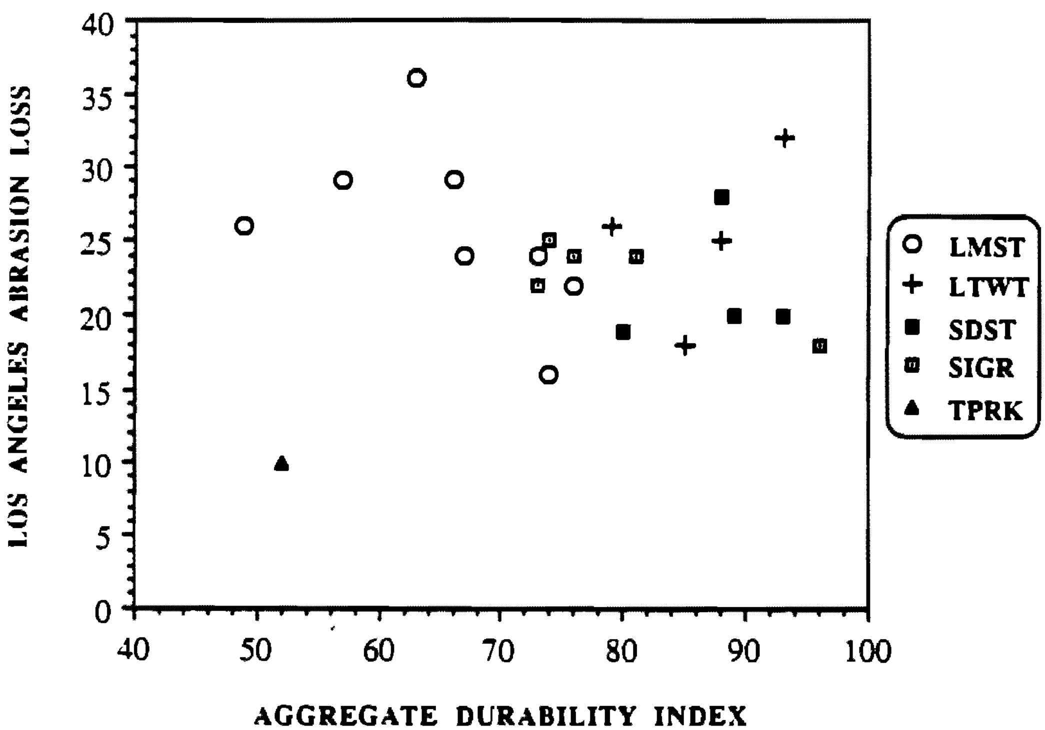


Fig. 5.44 Correlation Between the Results of the Los Angeles Abrasion and Aggregate Durability Tests for the Different Uncoated Aggregates Used

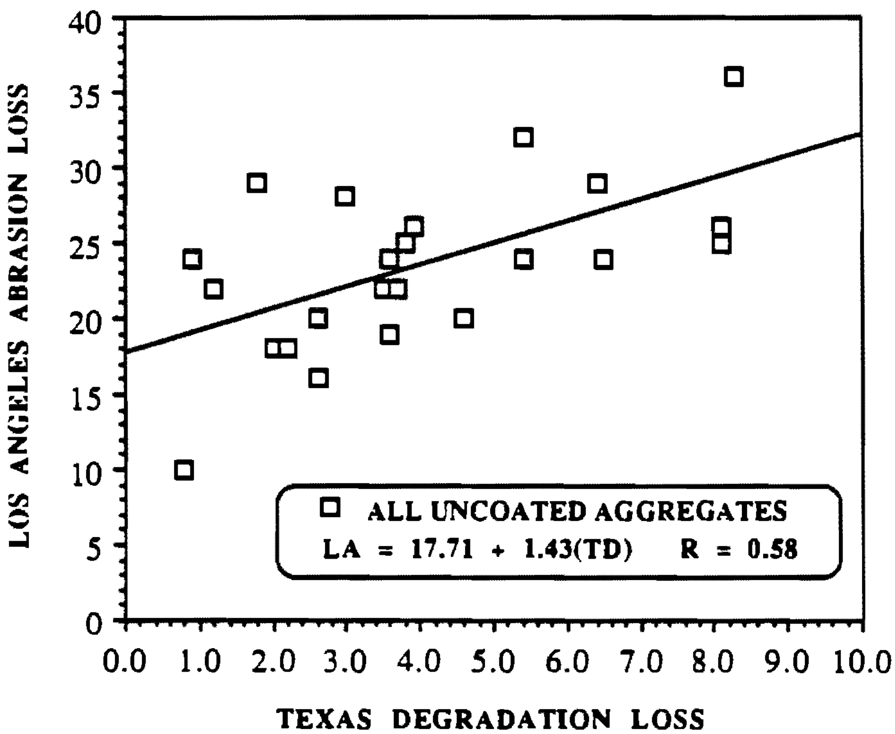


Fig. 5.45 Correlation Between the Results of the Los Angeles Abrasion and Texas Degradation Tests for All Uncoated Aggregates

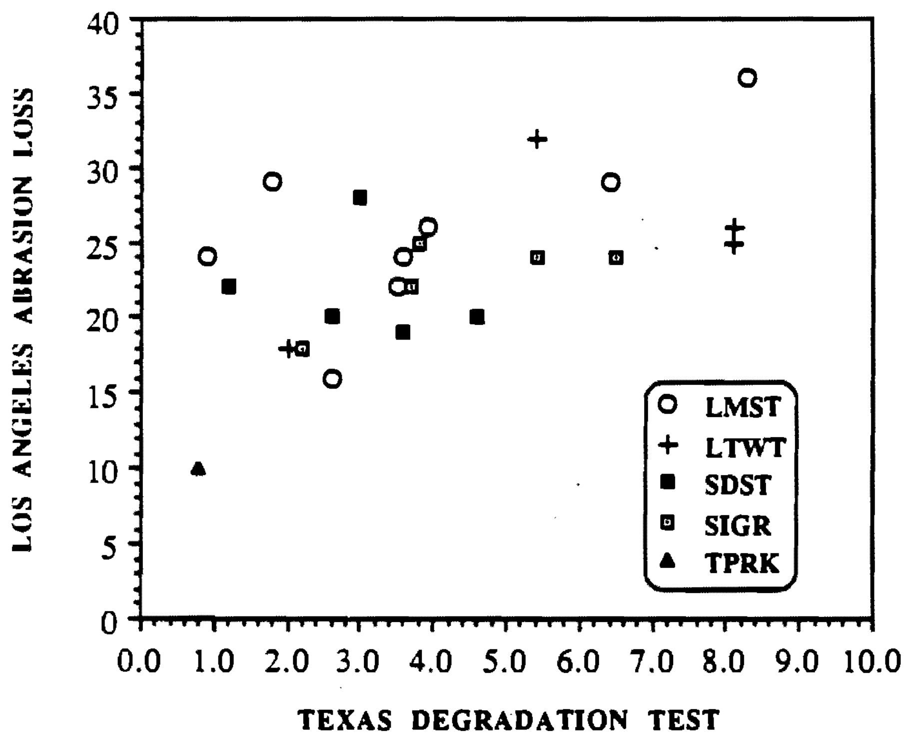


Fig. 5.46 Correlation Between the Results of the Los Angeles Abrasion and Texas Degradation Tests for the Different Uncoated Aggregates Used

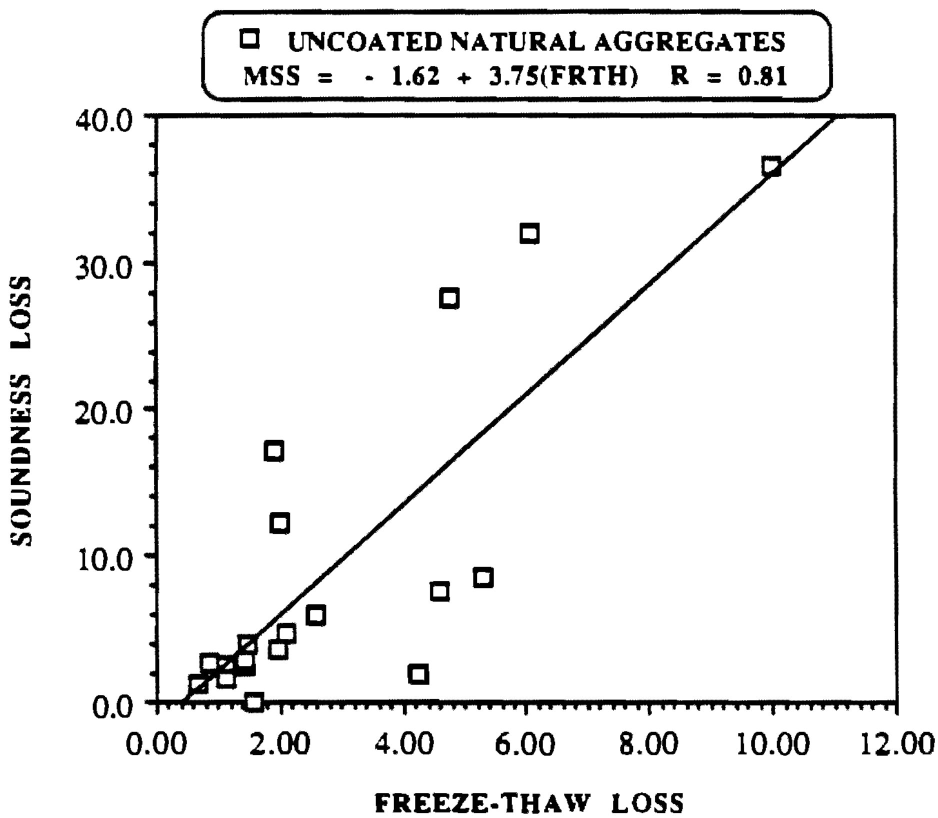


Fig. 5.47 Correlation Between the Results of the Soundness and Freeze-Thaw Tests for All Uncoated Natural Aggregates

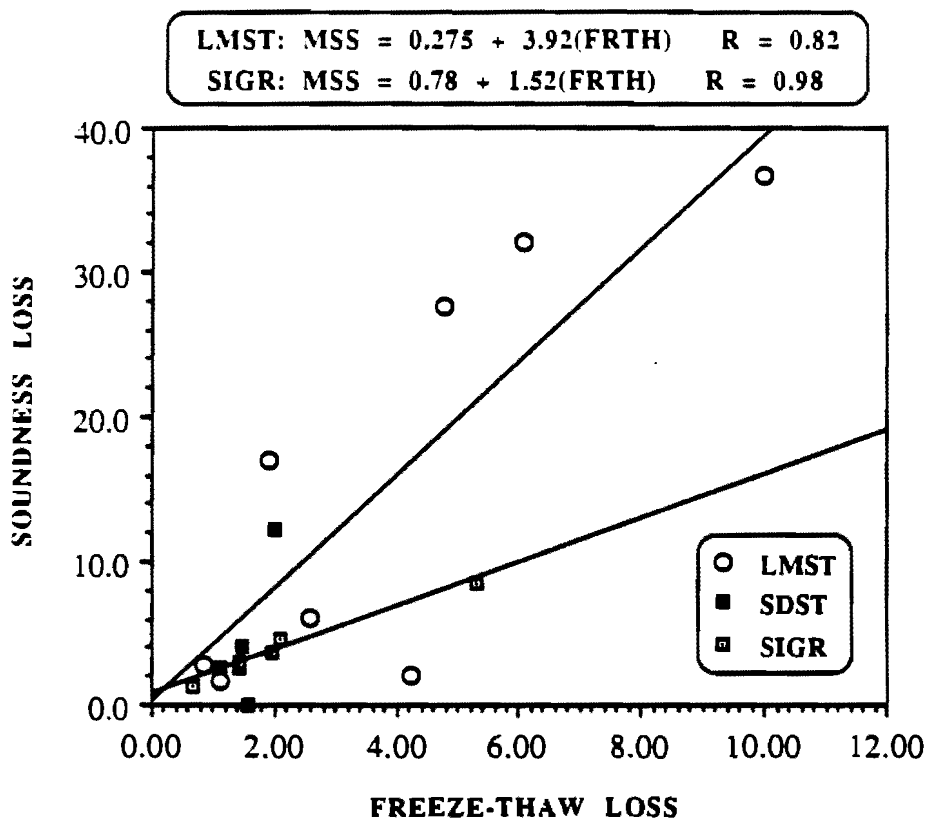


Fig. 5.48 Correlation Between the Results of the Soundness and Freeze-Thaw Tests for the Uncoated LMST, SDST, and SIGR Aggregates Used

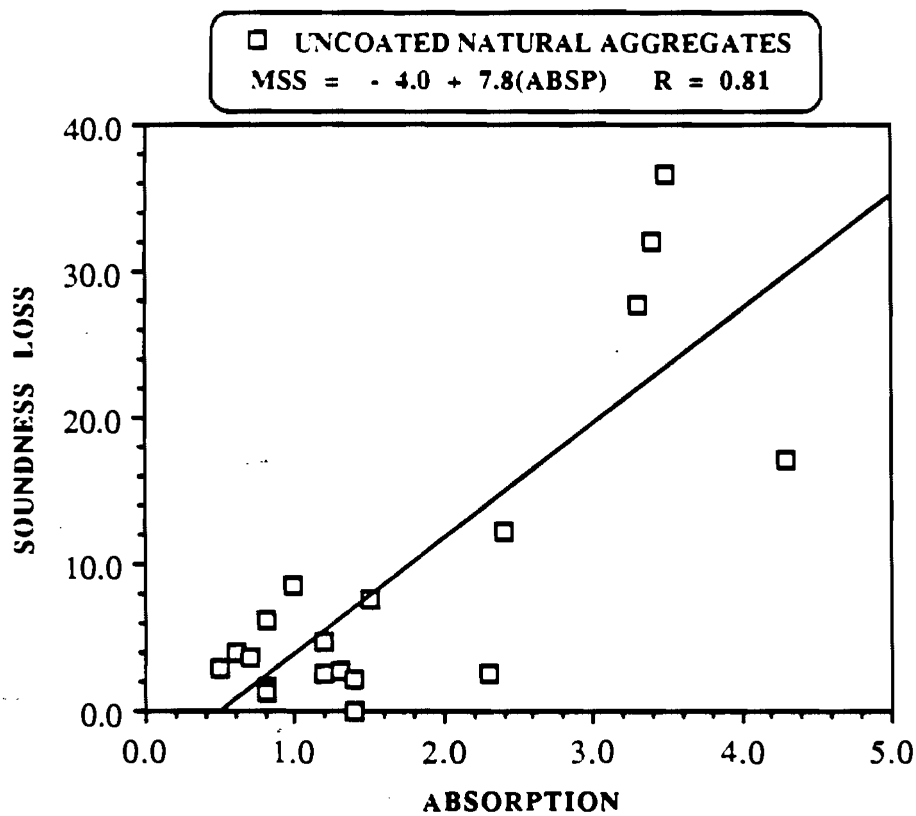


Fig. 5.49 Correlation Between the Results of the Soundness and Absorption Tests for All Uncoated Natural Aggregates

two tests in the LMST group. Figure 5.50 shows the LMST correlation with a coefficient of 0.82.

5.3.2 Petrographic vs Laboratory Test Results

Before any conclusions can be drawn as to whether the petrographic results may or may not be helpful in explaining the laboratory results, the precision achieved in subjectively judging the percentages for the different variables involved in the petrographic study should be tested. This can be best done by studying the correlation of the percentages of non-carbonate contents (NCC) as determined petrographically with the results of the INRD test. When studied, a perfect correlation with a coefficient of 0.99 was revealed, indicating that extreme caution was taken by the petrographers in making those subjective judgments. The correlation is shown in Fig. 5.51.

Two major observations can be made. First, low PVs, ranging from 25 to 29, were found to be associated with aggregates having high percentages of MST particles. However, the PV test did not clearly distinguish between aggregates exhibiting similar textural properties (MST and GST) but different percentages of void contents. Aggregates 1 and 18, with respective PVs of 28 and 27, may be used as an example. Both had the same amount of MST particles (95 percent) with aggregate 18 showing to have 5 percent voids in these particles compared to zero percent for aggregate 1. Furthermore, aggregate 16 was comprised almost entirely of dense, hard chert (NCC = 95.25) that was classified to be matrix-supported, and still its PV of 27 did not seem to reflect the effect of hardness. The matrix-supported porous chert along with the grain-supported sandstone could have both contributed to the PV of 34 obtained for aggregate 22.

Second, aggregates 7, 32, 50, and 53, with high PVs and MSS losses, were petrographically evaluated to have either a considerable amount of GST particles with a moderate level of porosity throughout the entire sample (aggregates 7 and 53) or a lesser amount of GST particles with a high level of porosity throughout the entire sample (aggregate 32 and 50). Since the presence of a moderate amount of voids (about 5 percent) in MST particles was not found to be associated with a high MSS loss, as in aggregate 18 and 37, it is now understood that the presence of loosely

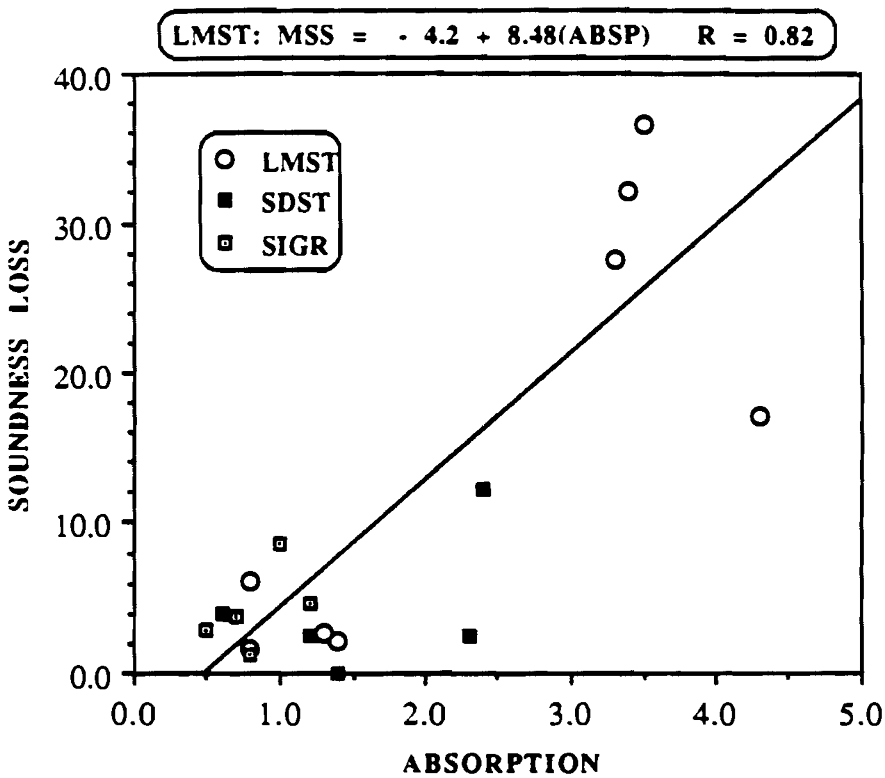


Fig. 5.50 Correlation Between the Results of the Soundness and Absortion Tests for the Uncoated LMST, SDST, and SIGR Aggregates Used

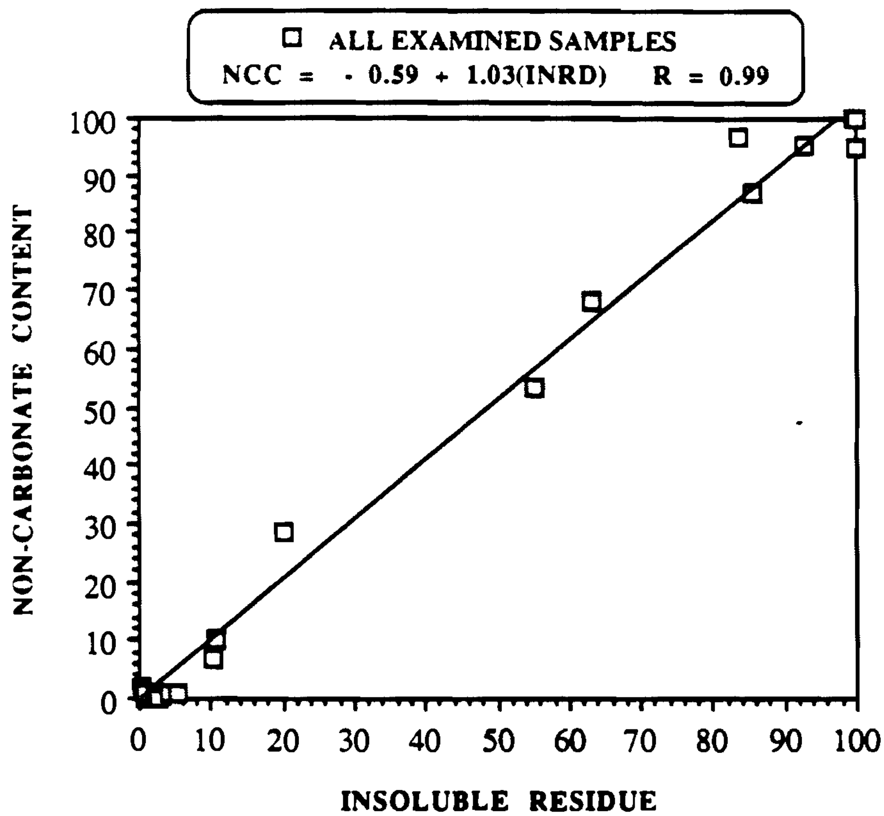


Fig. 5.51 Correlation Between the Petrographically Determined Non-Carbonate Contents and the Results of the Insoluble Residue test

compacted grains in the GST particles of aggregates 7 and 53 has contributed to the high MSS losses experienced by these aggregates. Both the presence of higher amounts of voids in the GST particles of aggregate 32 and 50, indicating an even looser compaction of grains, and the presence of higher levels of porosity in the MST particles of these aggregates (about 18 and 13 percents, respectively) might have contributed to the high MSS losses in these aggregates. Lastly, it was noticed that the presence of a large amount of tightly compacted carbonate grains in the GST particles of aggregate 6 did not produce a high PV; the PV was only 27 for this aggregate.

CHAPTER 6
FIELD-RELATED DATA

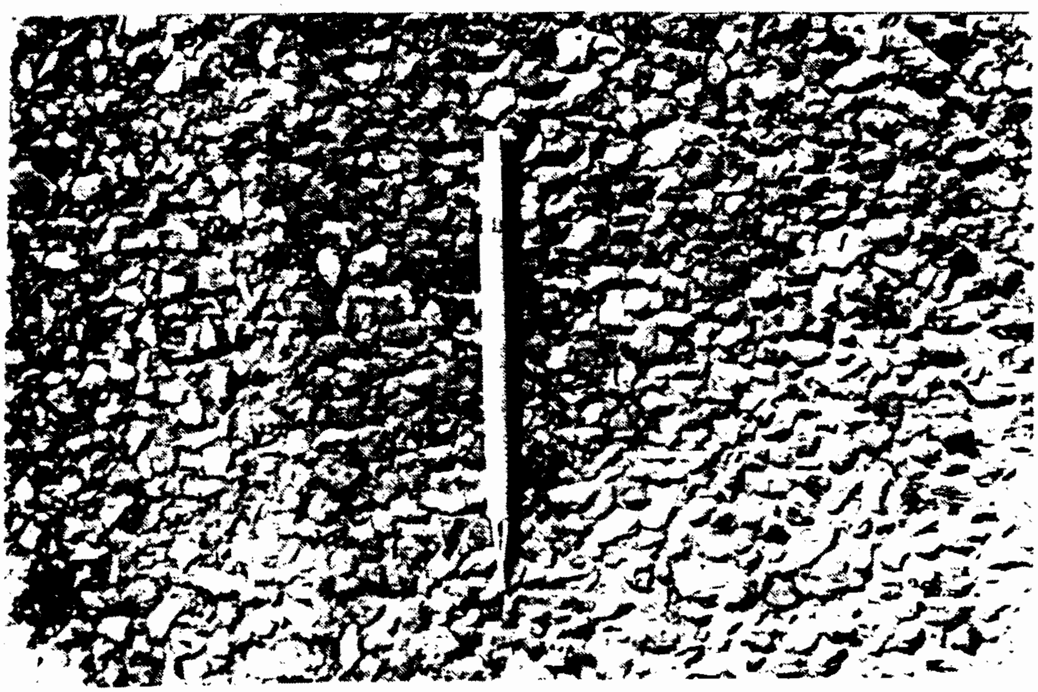
Field data were gathered on all established test sections. They constituted information obtained by monitoring the sections since the date of construction. Specifically, the data for each test section consisted of a survey of construction variables performed at the construction site, traffic data collected annually, results of field testing, evaluations of a visual condition survey done concurrently with field testing, and long-term and periodical weather data.

6.1 Construction Data

Typical photographs on the sequence of operations used in the construction of seal coat overlays are included in Figs. 6.1 through 6.5. Figure 6.1 shows the condition of a patched, existing surface before the application of the seal coat overlay. The patching, evident along an old wheel path, had been made with a LTWT material for the purpose of improving frictional resistance. Brooming of the old surface, shown in Fig. 6.2, is first done to insure a clear interface between the old surface and the seal coat overlay. This operation is followed by the uniform distribution of asphalt on the old surface. Figure 6.3 shows (a) an asphalt distributor at the start of asphalt application and (b) the distribution operation in progress. The aggregate is then uniformly applied on top of the asphalt film as illustrated in Fig. 6.4. Finally, pneumatic tired rollers, shown in part (a) of Fig. 6.5, are used to firmly press the aggregate particles into the asphalt layer so as to insure proper embedment and to promote adhesion. A newly finished (rolled) seal coat surface is shown in part (b) of Fig. 6.5.

For each constructed test section, a survey was made which consisted of information on the location of each section, condition and type of existing pavement, personnel contacted at the site, type of construction forces, coarse aggregate material and asphalt type (ASTY) used, design spreading rate of aggregate (AGSR) and

(a) Longitudinal patching in a wheel patch



(b) LTWT patching material vs. old LMST material

Fig. 6.1 Condition of an existing surface before the application of a seal coat overlay

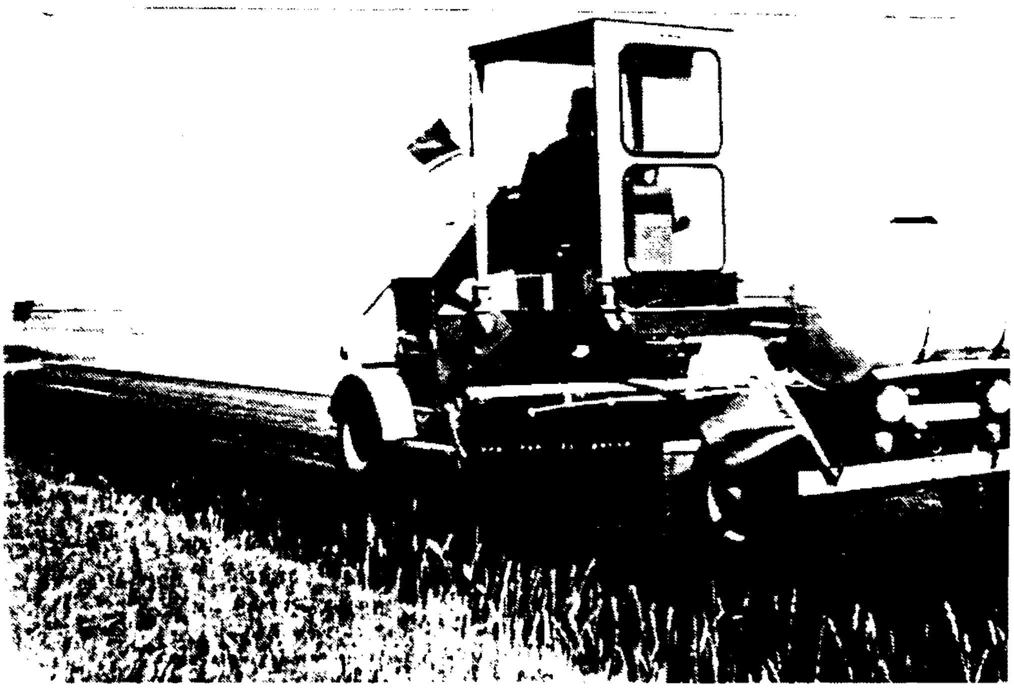
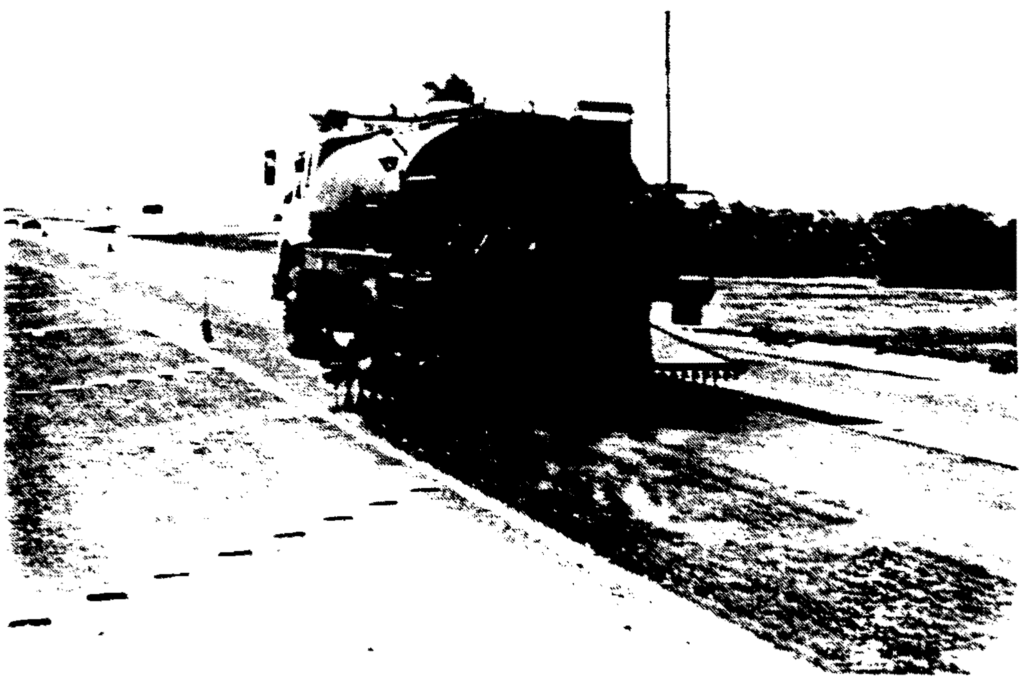


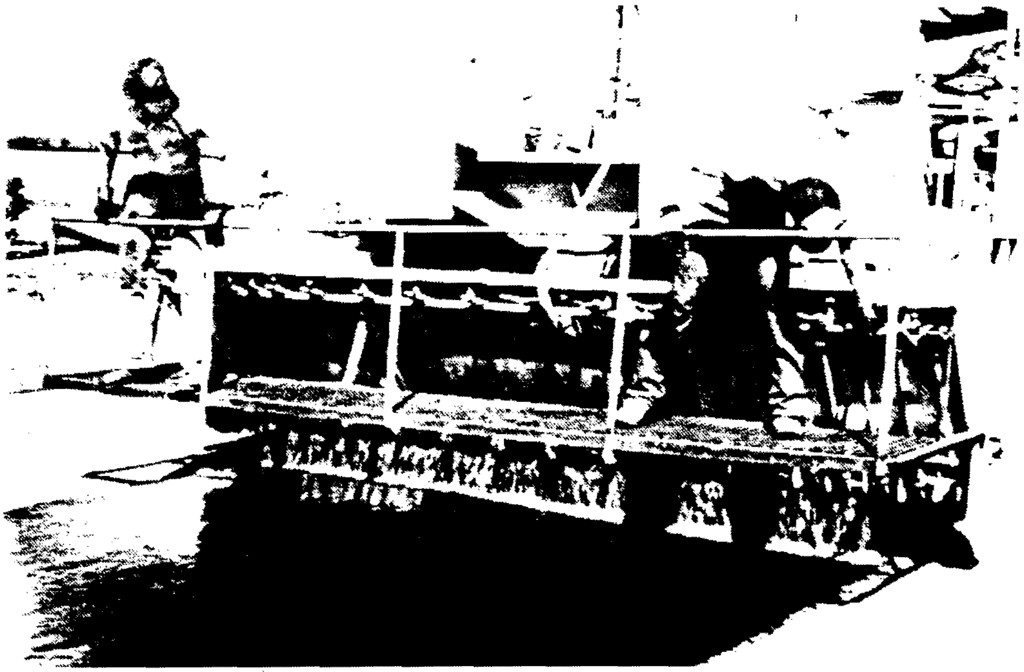
Fig. 6.2 Brooming operations of old surface

(a) Start of asphalt distribution



(b) Asphalt distribution in Progress

Fig. 6.3 Asphalt distribution operation

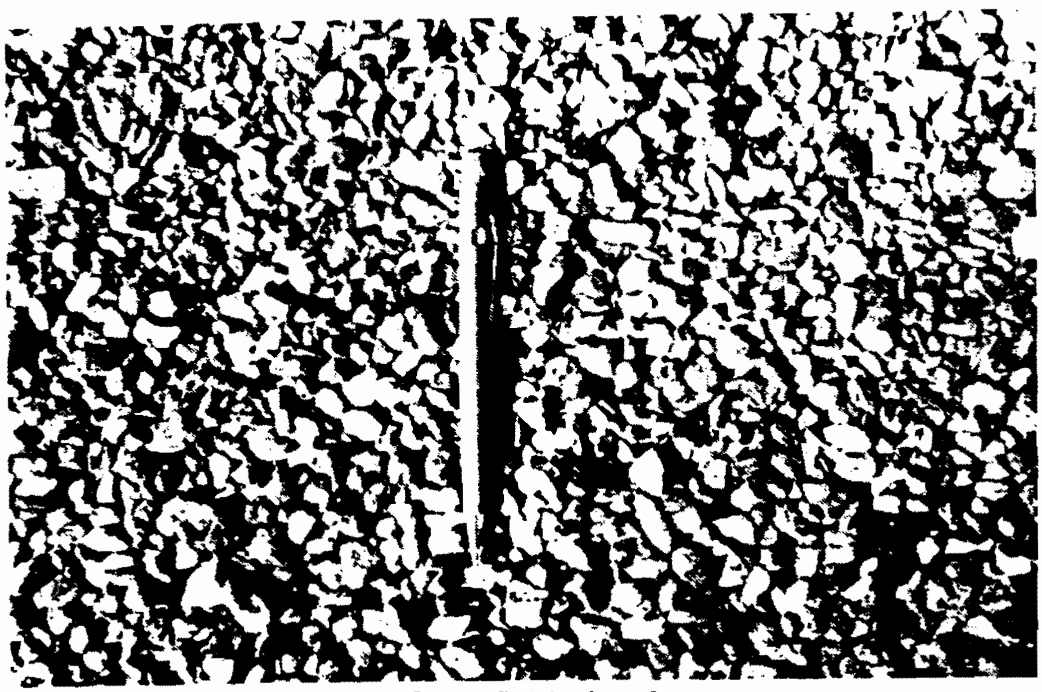


(a) Start of aggregate spreading

(b) Aggregate spreading in progress

Fig. 6.4 Aggregate spreading operation

(a) Rolling operations in progress



(b) Newly finished surface

Fig. 6.5 Rolling Operations of new surface distribution rate of asphalt (ASDR), and weather conditions. Also, photographs of the construction operations and notes on any problems encountered have been documented. The form shown in Fig. 6.6 was utilized for collecting the construction data.

Various aggregate materials and asphalt types were used in the construction of the established sections, along with wide ranges of AGSR and ASDR. A complete summary is included in Table 6.1.

6.2 Traffic Data

6.2.1 Annual Average Daily Traffic

Information on annual ADT was sought for each of the established sections. It was obtained from the district traffic maps produced annually by the Transportation Planning Division (D-10) of the SDHPT. In each of the cases, the annual ADT count taken for the portion of the highway located closest to the section being studied was assumed to represent the traffic passing on that section. Traffic data have been collected for 1986, 1987, and 1988. Traffic maps for a particular year normally becomes available in the last quarter of the following year. Table 6.2 summarizes the traffic data collected on all of the sections.

6.2.2 Directional and Lane Distribution of Traffic

The ADT counts of Table 6.2 were given as total counts for all lanes. Test sections, depending on whether or not they were replicated, were established either in only one direction of a two-lane highway or in both directions. Similarly, they were established either in only the outer lane of a particular direction of a four-lane highway or in both inner and outer lanes. In order that the traffic passing on a particular lane in a certain direction could be determined, information on percentages for directional and lane distribution of traffic was searched for.

Total annual directional counts of traffic were obtained from D-10 for twenty-one of the test sections. The counts were available from a summary of information obtained from permanent automatic traffic recorders, which are installed at representative highway locations throughout the state. Table 6.3 shows the counts

CONSTRUCTION SURVEY

Test Section No..:____________________________Date:____________________

DISTRICT:____________________________County:_______________________

Contact:________________________Title:______________Phone:_____________

Contact:________________________Title:______________Phone:_____________

Contact:________________________Title:______________Phone:_____________

Location: Highway No.:___CSN: ____________Station No._________

MAP: Indicate distances from permanent nearby features (like bridges etc.) and test section. Also indicate if steel markers and/or paint markings were used.

Construction Date:________________Temperature:________Weather:____________

Construction by Contract:_____________or Maintenance Forces________________

Contractor Name:____

Asphalt Design Data:

Type: ______________________________________

Admixture: ________________________________

Percent Admixture: ____________________________

Distribution Rate: ______________________________

Coarse Aggregate Data

Type: ____

Procedure: ___________________________________

Pit Location: __________________________________

Spreading Rate: _______________________________

Climatic Region:___________No. of Freeze-Thaw Cycles Per Year:_____________

Winter Temperature:______________Rainfall:______________________________

Summer Temperature:_____________Rainfall:______________________________

A.D.T. :___________________% Trucks:__________Speed Limit:_______________

Condition and Type of Existing Pavement:____________________________________

____

Fig. 6.6 Seal coat construction survey form

Table 6.1 Summary of the Data for the Construction Design Variables

Section	Lane	Construction Date	AGMT	Surface Coating	Grade	AGSR	ASTY	Admixture	Percent Admix.	ASDR
1	.	14JUL86	LMST	U[a]	4	125	HFRS-2	-	-	0.400
2	.	14JUL86	LMST	U	4	125	HFRS-2	-	-	0.400
3	.	07JUL86	LTWT	U	4	125	HFRS-2	-	-	0.400
4	.	20JUN86	LTWT	U	4	125	HFRS-2	POLYMER	-	0.350
5	.	24JUL86	LTWT	U	3	95	AC-5	LATEX	3.0	0.340
6	.	24JUL86	SIGR	U	3	95	AC-5	LATEX	3.0	0.340
7	.	24JUL86	LMST	P[b]	3	95	AC-5	LATEX	3.0	0.340
8	.	24JUL86	SIGR	U	3	100	RS-2	-	-	0.355
9	.	24JUL86	SIGR	U	4	100	RS-2	-	-	0.355
10	.	24JUL86	LMST	U	4	100	RS-2	-	-	0.355
11	.	24JUL86	SIGR	U	4	100	RS-2	-	-	0.355
12	.	24JUL86	SIGR	P	4	100	RS-2	-	-	0.355
13	.	24JUL86	SIGR	U	3	100	RS-2	-	-	0.355
14	.	24JUL86	SIGR	U	3	100	RS-2	-	-	0.355
15	I	30JUL86	LMST	P	4	110	HFRS-2	POLYMER	3.0	0.300
16	I	30JUL86	SIGR	P	4	110	HFRS-2	POLYMER	3.0	0.300
17	I	30JUL86	LMST	P	4	110	HFRS-2	POLYMER	3.0	0.300
18	I	30JUL86	LMST	P	3M	110	HFRS-2	POLYMER	3.0	0.300
19	O	30JUL86	LMRA	P	4	110	HFRS-2	POLYMER	3.0	0.300
20	O	30JUL86	LMRA	P	4	110	HFRS-2	POLYMER	3.0	0.300
21	O	26JUL86	LMRA	P	4	-	-	-	-	-
22	.	31JUL86	SIGR	U	3	70	CRS-2	-	-	0.635
23	I	31JUL86	LTWT	U	3M	103	AC-10	-	-	0.360
23	O	31JUL86	LTWT	U	3M	103	AC-10	-	-	0.330
24	I	31JUL86	LTWT	U	3M	103	AC-10	-	-	0.360
24	O	31JUL86	LTWT	U	3M	103	AC-10	-	-	0.330

(continued)

Table 6.1 Summary of the Data for the Construction Design Variables (Continued)

Section	Lane	Construction Date	AGMT	Surface Coating	Grade	AGSR	ASTY	Admixture	Percent Admix.	ASDR
25	.	28JUL86	SDST	U	3	95	AC-10	-	-	0.370
26	.	05AUG86	SDST	E[c]	3	100	AC-5	LATEX	3.0	0.410
27	.	05AUG86	SDST	E	3	100	AC-5	LATEX	3.0	0.420
28	.	05AUG86	SDST	E	3	100	AC-5	LATEX	3.0	0.420
29	.	11JUL86	LMST	P	3	100	AC-5	-	-	0.400
30	.	11JUL86	LMST	U	3	100	AC-5	-	-	0.400
31	.	01JUL86	SDST	U	3	70	AC-10	-	-	0.520
32	I	08AUG86	LMST	U	3M	100	AC-10	-	-	0.330
32	O	08AUG86	LMST	U	3M	100	AC-10	LATEX	-	0.300
33	I	08AUG86	LTWT	P	3M	100	AC-10	-	-	0.330
33	O	08AUG86	LTWT	P	3M	100	AC-10	LATEX	-	0.300
34	I	08AUG86	LMRA	P	3	100	AC-10	-	-	0.330
34	O	08AUG86	LMRA	P	3	100	AC-10	LATEX	-	0.300
35	O	24JUL86	LTWT	U	3M	70	AC-10	-	-	0.410
36	.	26AUG86	LMST	P	4	130	AC-10	LATEX	2.0	0.295
37	.	26AUG86	LMST	P	4	130	AC-10	LATEX	2.0	0.295
38	.	26AUG86	LMST	P	4	130	AC-10	LATEX	2.0	0.295
39	.	26AUG86	SIGR	P	4	130	AC-10	LATEX	2.0	0.300
40	.	26AUG86	LTWT	P	4	130	AC-10	LATEX	2.0	0.300
41	.	26AUG86	LTWT	P	4	130	AC-10	LATEX	2.0	0.300
42	.	26AUG86	LMRA	P	4	130	AC-10	LATEX	2.0	0.300
43	.	26AUG86	LMRA	P	4	130	AC-10	LATEX	2.0	0.300

(continued)

Table 6.1 Summary of the Data for the Construction Design Variables (Continued)

Section	Lane	Construction Date	AGMT	Surface Coating	Grade	AGSR	ASTY	Admixture	Percent Admix.	ASDR
44	.	15SEP86	LTWT	U	3M	100	CRS-2	POLYMER	3.0	0.333
45	.	15SEP86	LMRA	P	3	100	CRS-2	POLYMER	3.0	0.333
46	.	15SEP86	LTWT	U	3M	100	CRS-2	POLYMER	3.0	0.333
47	.	15SEP86	LMRA	P	3	100	CRS-2	POLYMER	3.0	0.333
48	.	15SEP86	LTWT	U	3M	100	AC-5	LATEX	3.0	0.335
49	.	15SEP86	LTWT	U	3M	100	AC-5	LATEX	3.0	0.335
50	.	07AUG87	LMST	U	4M	120	AC-5	LATEX	3.0	0.350
51	.	07AUG87	SDST	U	4	120	AC-5	LATEX	3.0	0.350
52	.	07AUG87	SDST	U	4	120	AC-5	LATEX	3.0	0.350
53	.	07AUG87	LMST	U	4	120	AC-5	LATEX	3.0	0.350
54	.	07AUG87	LMST	E	4	120	AC-5	LATEX	3.0	0.350
55	O	04AUG87	TPRK	U	4	115	AC-5	LATEX	3.0	0.350
56	I	11AUG87	RHYO	P	3M	96	AC-10	-	-	0.500
56	O	11AUG87	RHYO	P	3M	96	AC-10	-	-	0.410
57	I	11AUG87	RHYO	P	3M	96	AC-10	-	-	0.500
57	O	11AUG87	RHYO	P	3M	96	AC-10	-	-	0.410
58	I	11AUG87	RHYO	P	3M	100	AC-10	-	-	0.500
58	O	11AUG87	RHYO	P	3M	100	AC-10	-	-	0.410
59	I	11AUG87	RHYO	P	3M	100	AC-10	-	-	0.500
59	O	11AUG87	RHYO	P	3M	100	AC-10	-	-	0.410

[a] Uncoated.
[b] Precoated.
[c] Asphalt was extracted.

Table 6.2 Annual Average Daily Traffic Data

District	Sections	County	Highway	No. of Lanes	Shoulder Yes/No	ADT*		
						1986	1987	1988
23- Brown-wood	1	Stephens	FM 1148	2	No	640	660	600
	2	Stephens	FM 1287	2	No	950	950	890
	3	Stephens	US 183	2	Yes	850	660	680
	4	Eastland	SH 36	2	Yes	2500	2500	2400
8- Abilene	5-7	Shackelford	SH 351	2	Yes	2400	2500	2500
4- Amarillo	8-12	Deaf Smith	FM 2943	2	No	230	210	200
	13	Deaf Smith	FM 1062	2	No	270	210	220
	14	Deaf Smith	FM 2943	2	No	260	210	200
16- Corpus Christi	15-20	Nueces	P 22	4	Yes	14100	13900	14000
	21	Nueces	SH 358	4	No	34000	36000	35000
19- Atlanta	22	Titus	FM 2882	2	No	950	910	890
	23-24	Panola	US 59	4	No	6900	7200	7400
	25	Panola	SH 315	2	Yes	3900	4600	4800
	31	Bowie	FM 989	2	No	2200	2500	2800
	35	Upshur	US 271	4	Yes	7000	7500	7400
1- Paris	26	Delta	SH 154	2	No	650	890	810
	27-28	Hopkins	SH 19	2	Yes	4600	5000	5800
	29-30	Grayson	US 69	2	Yes	2900	3200	3300
11- Lufkin	32-34	Houston	US 287	4	Yes	3700	3700	3900
13- Yoakun	36-43	Victoria	US 77	2	Yes	5600	5700	5900
23- Beaumont	44-49	Liberty	US 90	2	Yes	5000	5700	5000
7- San Angelo	50-54	Sutton	US 277	2	Yes	1700	1850	2000
	55	Sutton	US277	4	No	4800	4500	3200
24- El Paso	56-59	Hudspeth	IH 10	4	Yes	7000	7600	7900

* ADT is given as a total count for all lanes.

Table 6.3 Directional Distribution of Annual Traffic Volumes for Twenty-One of the Established Test Sections

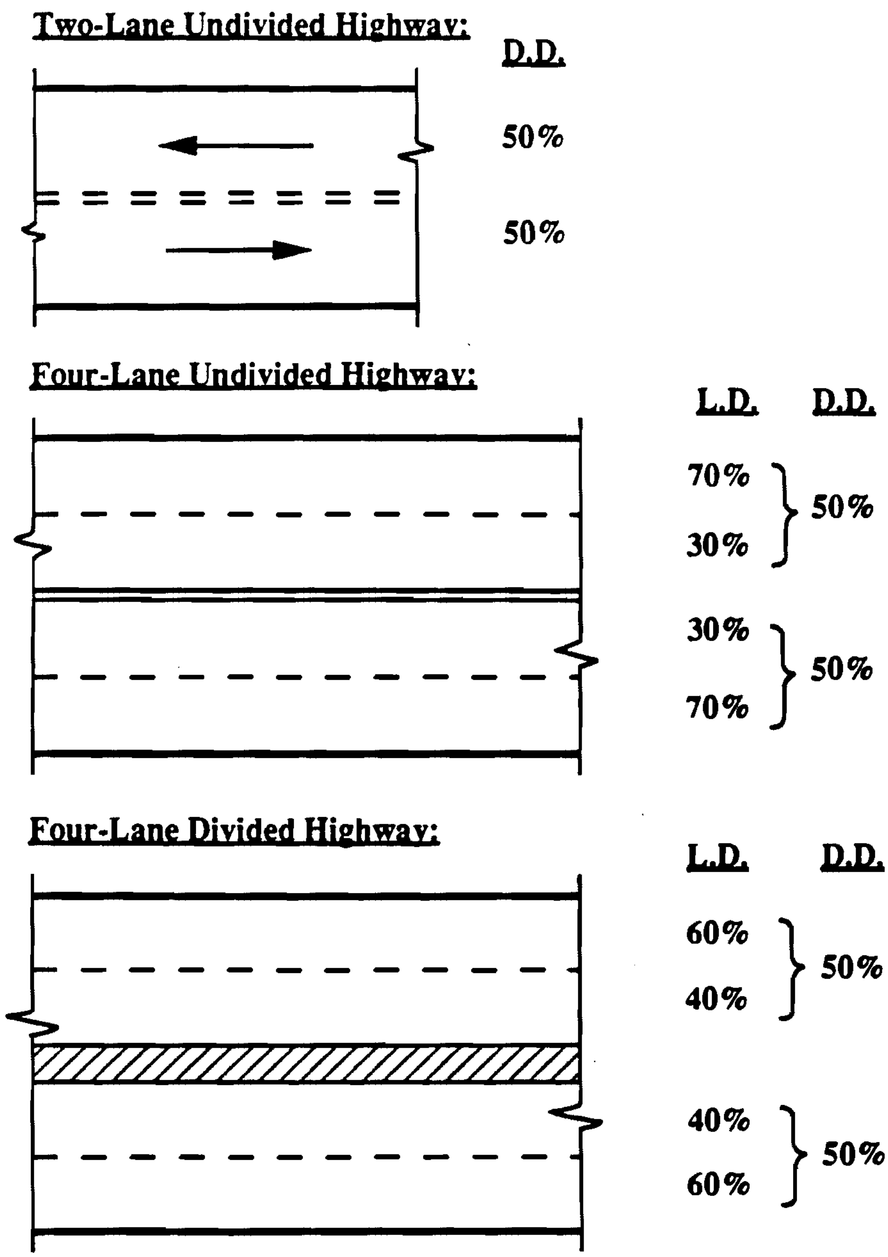
District	Sections	Highway & Bound	Permanent Sta.&Hwy	Count Bound	1986 Total Count	1986 D.D.*	1987 Total Count	1987 D.D.	1988 Total Count	1988 D.D.
8-Abilene	5 - 7	SH 351 W	S-18/US 84	N	979,600	0.50	973,280	0.50	1,010,690	0.50
				S	988,290	0.50	989,770	0.50	1,026,370	0.50
			S-153/IH 20	E	2,557,560	0.50	2,675,580	0.50	2,815,600	0.50
				W	2,571,190	0.50	2,701,780	0.50	2,831,960	0.50
11-Lufkin	32A-34A	US 287 N	S-36/US 287	N	712,900	0.49	998,380	0.49	1,061,310	0.50
				S	733,880	0.51	1,054,730	0.51	1,072,570	0.50
16-Corpus Christi	15 - 21	P 22 E	S-161/P 22	E	2,555,120	0.50	2,564,480	0.51	1,010,540	0.49
				W	2,542,650	0.50	2,509,130	0.49	1,056,650	0.51
20-Beau-mont	44 - 49	US 90 E&W	S-117/US 90	E	8,933,760	0.50	8,915,210	0.50	9,238,630	0.50
				W	8,955,130	0.50	8,903,810	0.50	9,256,880	0.50
			S-125/IH 10	E	6,858,040	0.49	7,090,790	0.50	7,460,370	0.49
				W	7,050,500	0.51	7,198,540	0.50	7,620,330	0.51
			S-147/US 59	N	11,147,730	0.45	11,535,760	0.45	12,102,190	0.45
				S	13,434,010	0.55	13,989,590	0.55	14,489,400	0.55
24-El Paso	56 - 59	IH 10 E	S-152/IH 10	E	1,276,490	0.49	1,382,560	0.49	1,464,940	0.49
				W	1,314,600	0.51	1,432,520	0.51	1,501,020	0.51

* Directional Distribution.

collected for 1986, 1987, and 1988. In this table, the permanent stations used were located either on the same highways where test sections were placed or on highways in the same areas believed to have similar traffic patterns. The directional distribution results suggested, as expected, a factor of 0.5 to be used for directionally dividing the total traffic.

No hard data were found on lane counts because, practically, highway engineers conservatively use a lane distribution factor of 0.8 to 1.0 for pavement design purposes. The use of such a high lane distribution factor in studying performance (as opposed to design) may lead to making erroneous conclusions in the interpretation of performance results versus accumulated traffic passes per lane. In light of this, it was decided to seek suggestions from the personnel of the Traffic Section at D-10 and from the Transportation Group of the Department of Civil Engineering at The University of Texas at Austin. The results of meetings held with both groups suggested that, for four-lane undivided and divided rural highways, distribution factors of 0.7 and 0.6 be used, respectively, in the calculation of traffic passing on the outer lane. A higher factor was suggested for the undivided highways because it was thought that, for safety reasons, more people tend to drive on the outer lane of these highways. In any case none of the four-lane highways involved in this study was undivided, but one case of a four-lane divided urban highway, Park Road 22 in District 16, was encountered for which the traffic was suggested to be divided equally between the inner and outer lanes. Figure 6.7 summarizes all assumed directional and lane distribution factors for rural highways.

It is worthy to mention that the 60-to-40 assumption was tested by performing a limited traffic count on a rural section of north bound U.S. Highway 183 located about 20 miles north of the Austin metropolitan area. The section was believed to be representative of the traffic pattern of the four-lane divided rural highways involved in this study. The results revealed a lane distribution of traffic of about 63 percent for the outer lane to about 37 percent for the inner lane, which was considered to be satisfactorily close to the assumed distribution.



D.D. : Directional Distribution
L.D. : Lane Distribution

Fig. 6.7 Assumed Directional Distribution and Lane Distribution of Traffic for the Different Types of Rural Highways

6.2.3 Percent Trucks

For rural highways carrying low volumes of traffic, the percentage of trucks may be expected to be somewhat low and restricted in loads. Therefore, its effect is usually disregarded in the design of seal coats to be used on such highways. However, for rural highways with high traffic volumes, it is assumed that about 15 percent of the traffic is heavy trucks. In view of the effects that higher percents of trucks may have on the performance of seal coats, efforts were made to collect information on these percents, especially for those sections placed on highways with traffic volumes higher than average .

Manual traffic stations are located throughout the state on various classes of highways. Twenty-four manual traffic counts are done once a year in these stations, in which traffic is classified from passenger cars to much heavier trucks based on the number of axles in each passing vehicle. Table 6.4 summarizes the 1988 data collected for most of the test sections. Data for 1986 and 1987 were not available for most of the identified stations. The 1988 data revealed a normal percent trucks and buses excluding light trucks, ranging from 12 to 22 with an average of about 17. These did not include the 37 and 57 percents obtained for section 25 and sections 56 through 59, respectively. The fact that sections 56 through 59 are located on Interstate Highway 10, connecting the City of El Paso with the rest of the state, explains the high percentage of trucks obtained for these sections.

For some of the sections, where representative manual count stations could not be identified, the percent trucks data were collected from the highway offices of the districts or counties where the sections are located. A low percentage of trucks equal to 4 was obtained for sections 15 through 20, that were established on an urban highway in District 16. A percentage of about 13 was suggested to be reasonable for sections 44 through 49, located in District 20.

These percentages may turn out useful in explaining the variations in field performance among aggregates with similar laboratory properties and placed under the same level of ADT. They can also be used in the interpretation of the extent of asphalt bleeding in sections with similar construction design characteristics and traffic volumes.

Table 6.4 Percent Trucks Based on 24-hour Manual Count

Sections	Highway & Bound	Station Number	1988					
			Count Bound	T & B[a] Count	T&B-LT[b] Count	Total Count	Percent T & B	Percent T&B-LT
1-3	US 183 N	S-46	NDD[c]	909	255	1516	60	17
4	US 36 E	944	SE	1798	422	3566	51	12
5-7	SH 351 W	S-47A	NDD	792	216	1572	50	14
23-24	US 59 S	1234	NDD	3803	1542	7099	54	21
25	SH 315 SW	1228	NE	1221	725	1951	63	37
26	SH 154 W&S	S-179	NDD	1254	387	2333	54	17
27-28	US 19 N&S	500	NE	2524	1029	4710	54	22
			SW	1562	596	3100	50	19
29-30	US 69 N&S	1087	SE	2403	618	4995	48	12
			NW	2253	609	4714	48	13
32-34	US 287 N	S-36	NDD	1744	397	3324	52	12
35	US 271 N&S	1296	N	2586	1018	4562	57	22
			S	2654	990	4704	56	21
		1088	S	128	32	185	69	17
			N	155	32	247	63	13
36-43	US 77 S	1042-A	SW	3743	1495	6935	54	22
		913	S	1630	479	3051	53	16
50-55	US 277 N	S - 51	NDD	2118	459	3789	56	12
		1003	N	566	178	983	58	18
56-59	IH 10 E	S-152	NDD	5224	4174	7280	71	57

[a] T&B : Trucks and Buses Count.
[b] T&B - LT : Trucks and Buses Count minus Light Trucks Count.
[c] NDD : No Directional Distribution.

6.3 Field Testing Data

6.3.1 Field Tests

Three tests were performed on the surfaces of test sections for evaluating their frictional properties and texture depths. These were the skid resistance test and the British pendulum test for measuring friction and the sand patch test for measuring texture. The skid resistance tests were conducted by D-10 of the SDHPT while the other two tests were performed by the researchers.

The testing was normally done in the center of the left wheel paths of test sections. Friction and texture measurements were taken only on parts of the surfaces that were free of obvious contamination. For each of the tests considered, five measurements were made at intervals of about 200 feet in each test section with the instrument of each test at the same lateral position in any one test section. The arithmetic average of the five measurements was considered to be the representative friction or texture measure of a test section.

Three rounds of British pendulum and sand patch testing were done, whereas, the skid resistance test has been conducted twice a year. According to the findings in Chapter 2, it was first thought that it would be best to conduct the tests after long periods of dryness when the pavement surface is expected to exhibit a minimum frictional resistance and texture depth. Later, it was felt that this would eliminate the chance of detecting and understanding the effects of long-term seasonal variations, caused by long periods of wetness or dryness. Then, the decision was made to perform the tests on a random basis, and the season in which the tests were undertaken was regarded as a variable affecting the obtained measurements and expected to explain the anticipated variations. This weather-caused variable is further discussed subsequently in this chapter.

6.3.1.1 *Skid Resistance Test.* The test provides a method for characterizing the capability of pavements to contribute to tire-pavement friction under wet conditions. The test has been conducted in accordance with the ASTM E274 (8) using the SDHPT research skid trailer, borrowed from District 5. The test apparatus consists of an automotive vehicle with a test wheel forming part of a suitable trailer towed by a vehicle. The test wheel is equipped with a standard pavement test tire according to ASTM E501. The apparatus contains a transducer and instrumentation which feeds into a computer, a water supply and proper dispensing system, and actuation controls for the brake of the test wheel.

The test apparatus is brought to the desired speed, 40 mph for the purpose of this research. Water is delivered to the surface ahead of the test tire and the test wheel brake is applied so as to lock the wheel completely. The magnitude of friction is then measured, and the result is expressed as a FN.

The FN is determined from the resulting friction force acting between the test tire and the pavement surface. It is the force required to slide the locked test tire at the stated speed, divided by the effective wheel load, and multiplied by 100.

Eight sets of skid resistance measurements have been obtained, which were spanned over a period of about five years. The results of the first four sets were adjusted to meet a standard Federal Highway calibration using the following equation:

$$\text{Adjusted FN} = 2.2 + 0.92\ (\text{Measured FN})$$

The last four sets were obtained after a new computer system had been installed in the trailer, which automatically adjusted the FN readings as they were being obtained. The skid resistance test results along with the accumulated traffic per lane associated with them are presented in Table B.1 of Appendix B.

6.3.1.2 British Pendulum Test. This test was performed with a modification to ASTM Designation: E 303, Measuring Surface Frictional Properties Using the British Pendulum Tester. The tester, suited for laboratory as well as field friction measurements, is described by ASTM as a dynamic pendulum impact type tester used to measure the energy loss when a rubber slider edge is propelled over a test surface. To elaborate, the test surface is freed of loose particles, and sufficient clear water is applied to flush the surface thoroughly. The pendulum slider is positioned to allow a specified length of slider path to make contact with the test surface. The pendulum is raised to a near horizontal position as it is locked, then released to swing freely, allowing the slider to make contact with the test surface. A drag pointer is carried by the pendulum arm to maximum upswing, where it remains in place to indicate the BPN. Five swings are made with the test surface rewet each time. The modification to ASTM Standard Method was that only the BPN of the last swing was recorded.

The significance of the test lies in the fact that it describes the microtexture of the surface, i.e., the fine-scaled roughness contributed by individual small asperities on the individual aggregate particles. The results of this test, shown in Table B.2 of Appendix B, were used for correlation purposes with the skid test results and in the interpretation of frictional performance variations.

6.3.1.3 Sand Patch Test. The sand patch test is a volumetric method used for determining the ATD of a selected portion of a pavement surface. The test was performed according to Test Method Tex-436-A, described in the Manual of Testing Procedures of the SDHPT, D-9 (114).

A known volume of dry natural silica sand is spread over a circular dry area until it is flush with the aggregate peaks of the pavement surface. The area of the formed patch is determined from an average of four or more diameters measured at equally spaced locations. Then, the ATD, calculated as the ratio of the volume to the area, is considered to be a measure of surface texture.

The measured ATD does not adequately assess the degree of roundness or grittiness the individual aggregate particles possess. However, it does represent the large scale texture of the pavement surface caused by the size and shape of the surface aggregate particles. The results of this test, presented in Table B.3 of Appendix B, were used in the same way as the British pendulum results.

6.3.2 Interdependencies Among Field Tests

The primary objective of performing the British pendulum and sand patch tests on the established test sections was to determine whether the skid test results did indeed measure both the microtexture (BPN) and macrotexture (ATD) properties of seal coat surfaces. In this section, the correlation between FN and BPN is established, and the dependency of the relationship between FN and ATD on BPN is studied. Since, in most cases, British pendulum and sand patch testing was not conducted concurrently with skid testing, the skid data used in the study of these relationships were those of the skid testing rounds performed closest in time to the rounds of the other two tests.

6.3.2.1 <u>*FN versus BPN.*</u> The overall correlation between FN and BPN is shown in Fig. 6.8. The correlation, based on 139 observations, had a good correlation coefficient of 0.74, indicating that the FN and BPN are directly related. As expected, the BPN, by itself, did not fully explain the variability in the FN. In fact, the BPN was shown to account for about 55 percent of the variation ($R^2 = 0.548$). The unexplained variation can be largely attributed to the effect of macrotexture and, to some extent, to the fact that some of skid testing results could have been affected by the asphalt bleeding and aggregate loss distresses encountered in some of the sections.

The data was then grouped according to the different aggregate materials used. The grouping, illustrated in Fig. 6.9, revealed higher levels of FN and BPN readings for the LTWT and SDST groups, wide ranges of FN and BPN readings for the LMRA and SIGR groups, and relatively low levels of FN and BPN readings for the LMST group. The correlation between FN and BPN was further studied in each aggregate group, as shown in Figs. 6.10 through 6.14. Moderate correlation coefficients of 0.47 and 0.44 were obtained in the LMST and LTWT groups, respectively. Good correlation coefficients of 0.58 and 0.6 were observed in the LMRA and SDST groups, respectively. Finally, a high correlation coefficient of 0.79 was revealed in the SIGR group. This group had the widest range of FN and BPN readings, which is believed to be associated with the different levels of laboratory polish susceptibility experienced by the aggregates of this group.

In conclusions, the overall correlation was found to be stronger than most of the correlations for the individual aggregate groups. Much of the strength of the overall correlation is believed to have been contributed by the SIGR group and by the fact that the other aggregate groups, when all combined, represented all levels of microtexture.

6.3.2.2 <u>*FN versus ATD.*</u> When FN was plotted against ATD, as shown in Fig. 6.15, no correlation was found indicating that the two are not directly related. This was expected because, according to the literature review, the relationship between these two tests has been shown to be dependent on the level of BPN. In view of this, the data was sorted according to BPN and divided into groups, each with a narrow range of BPN readings. The correlation between FN and ATD was then studied in each of the BPN groups; the correlations are presented in Figs. 6.16 through 6.25.

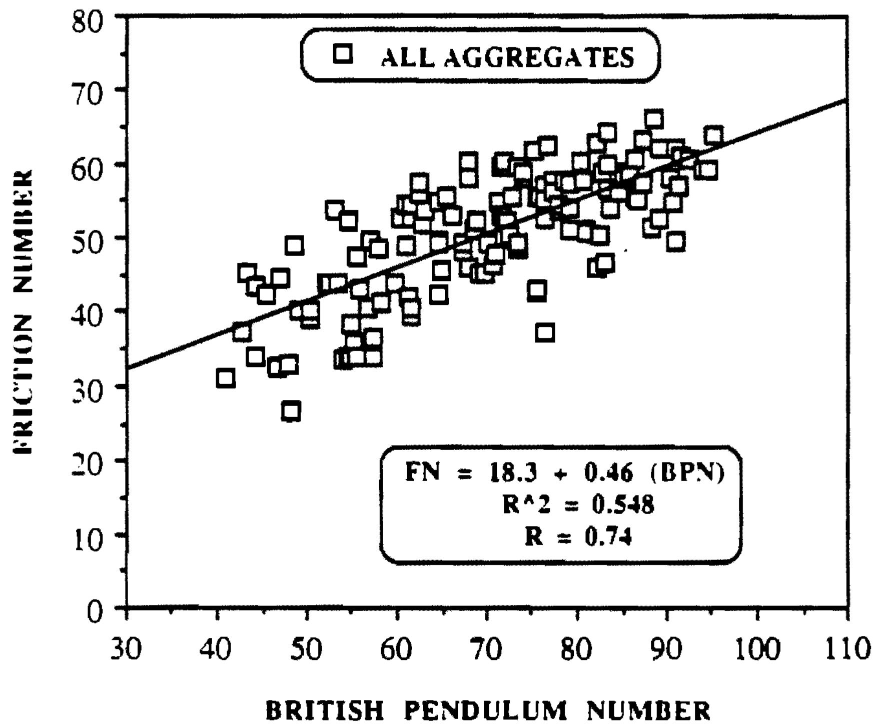


Fig. 6.8 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for All Aggregates

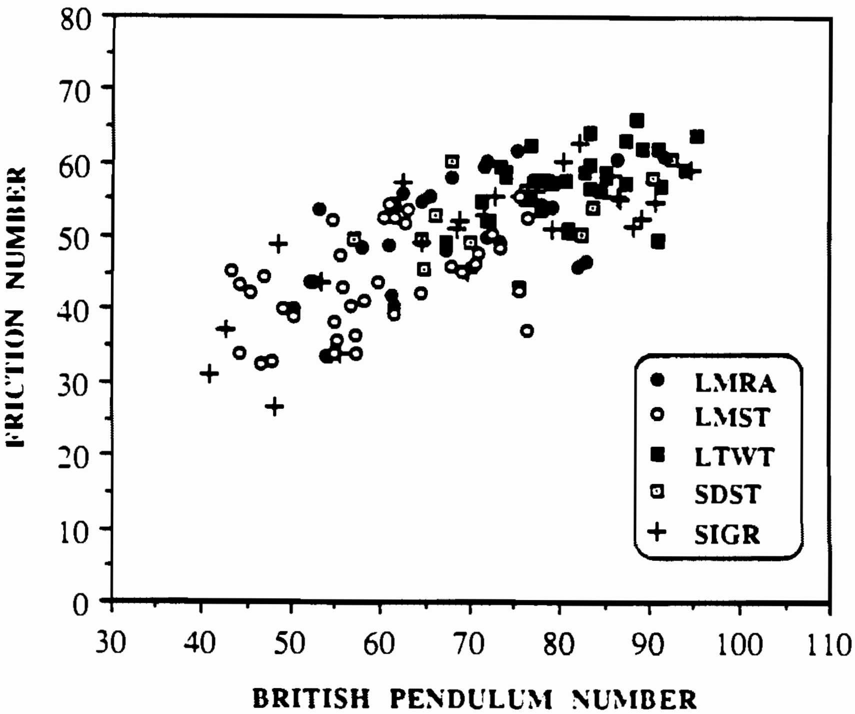


Fig. 6.9 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the Different Aggregate Materials Used

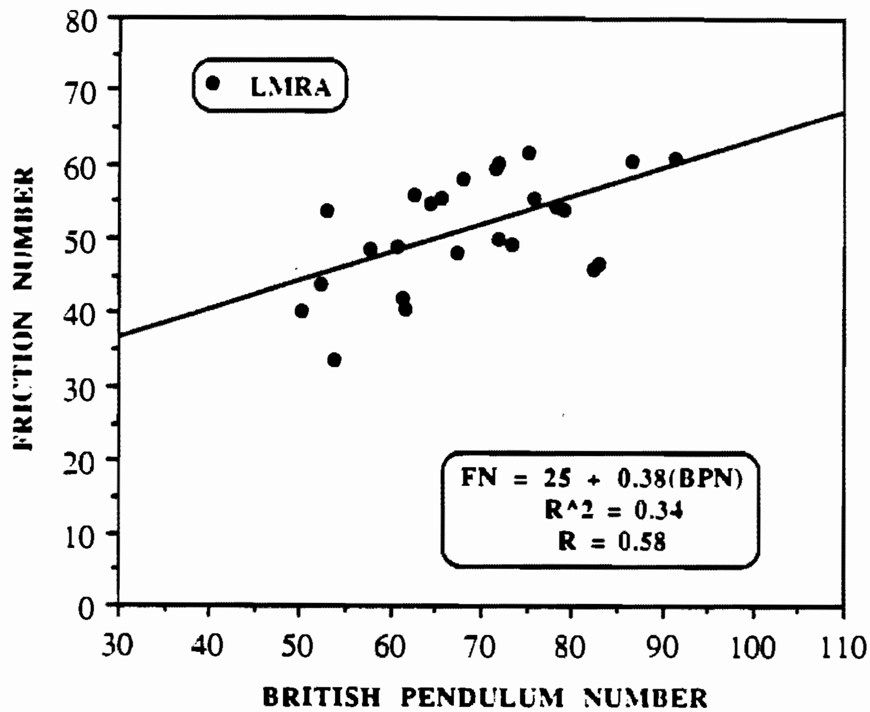


Fig. 6.10 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the LMRA Aggregates

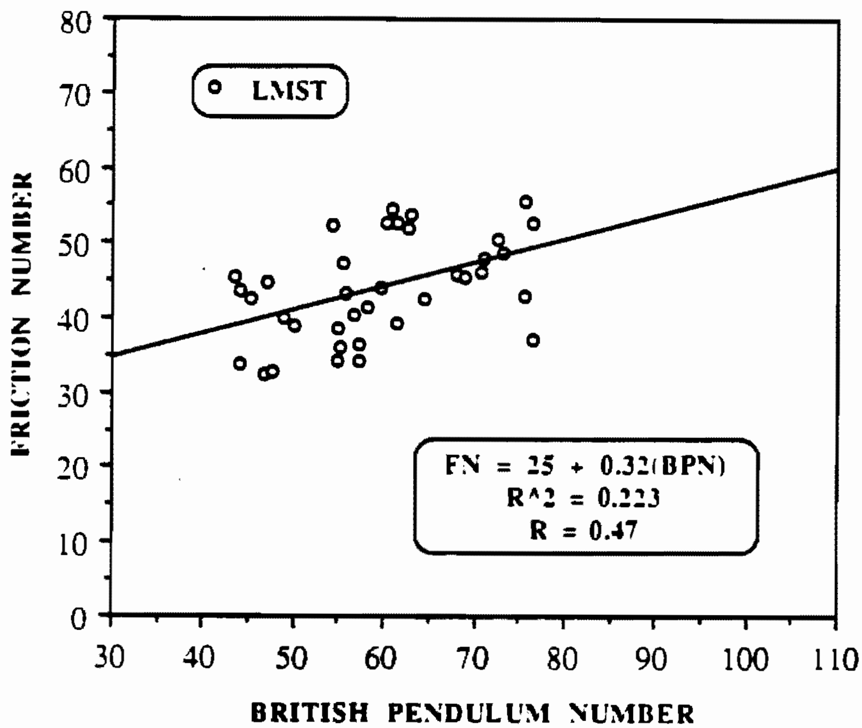


Fig. 6.11 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the LMST Aggregates

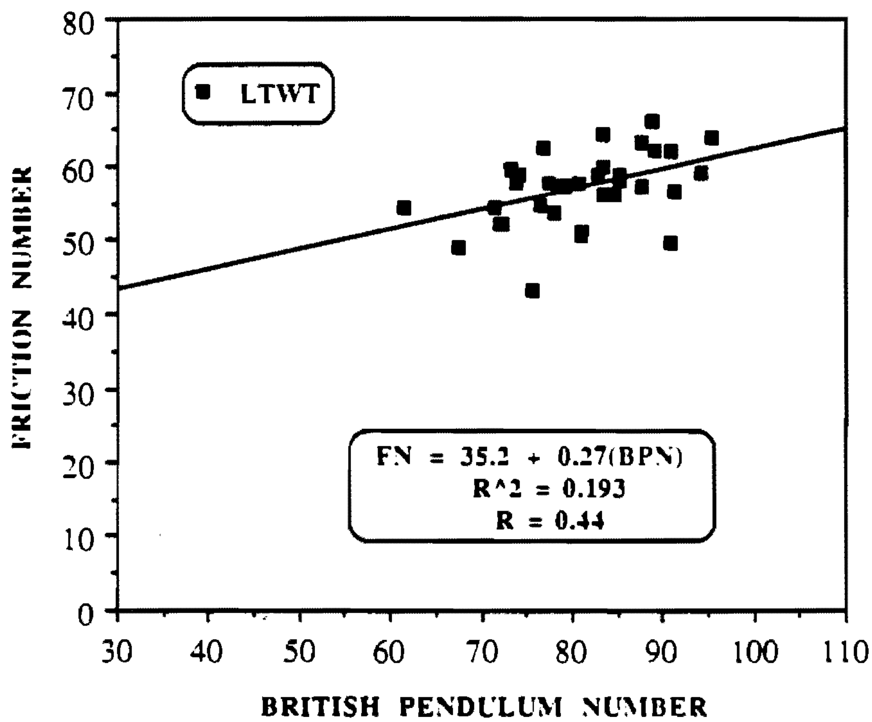


Fig. 6.12 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the LTWT Aggregates

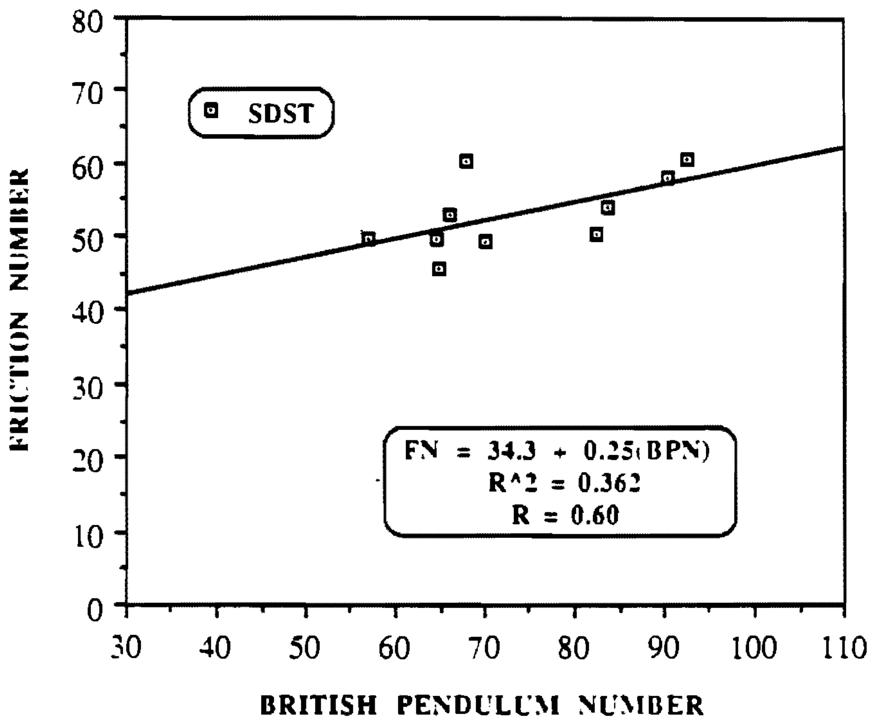


Fig. 6.13 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the SDST Aggregates

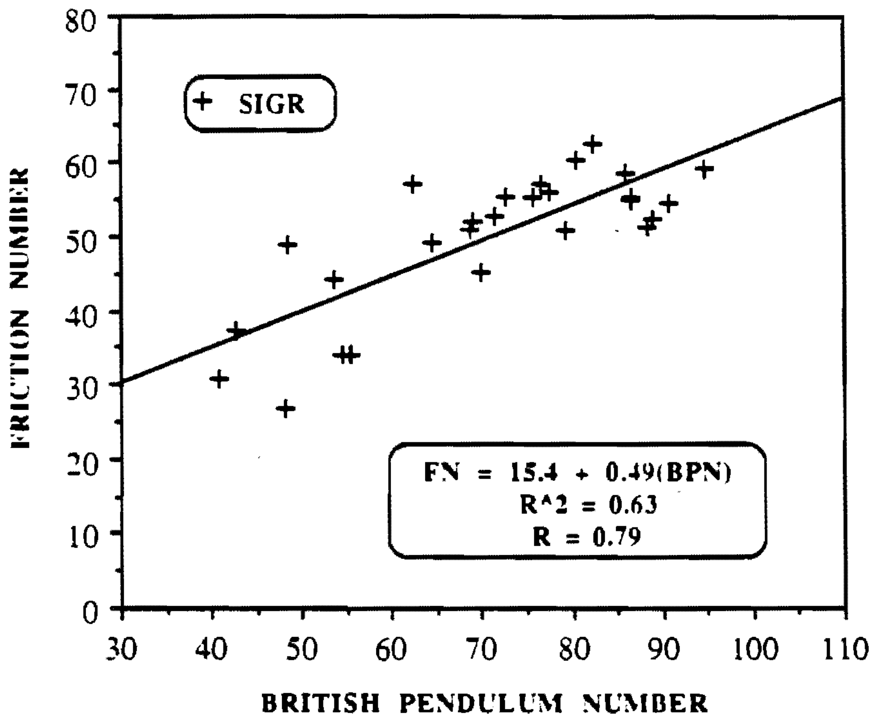


Fig. 6.14 Correlation Between the Results of the Skid Resistance and British Pendulum Tests for the SIGR Aggregates

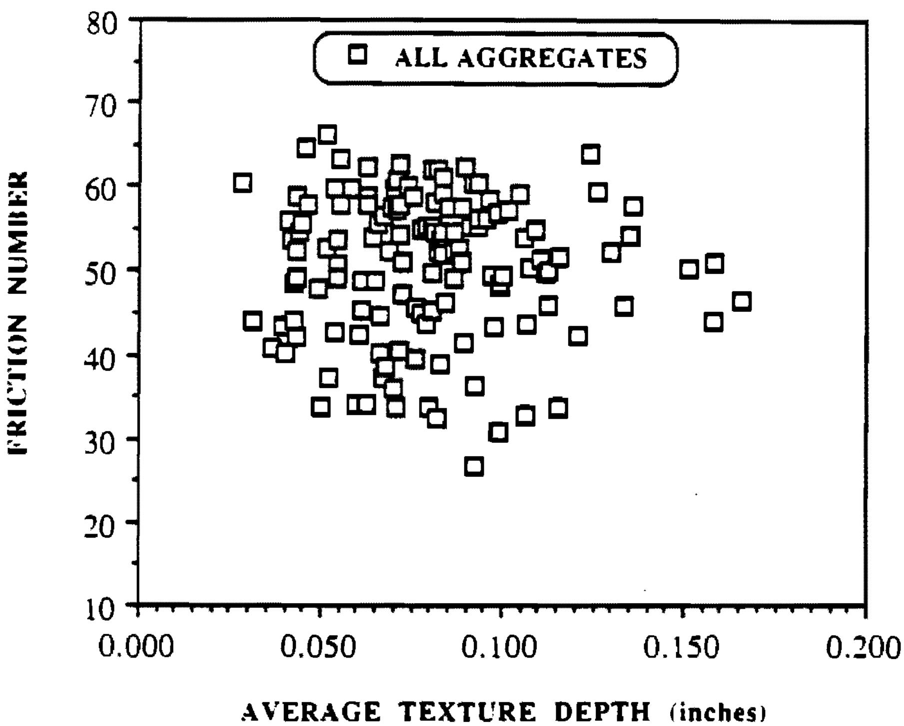


Fig. 6.15 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for All Aggregates

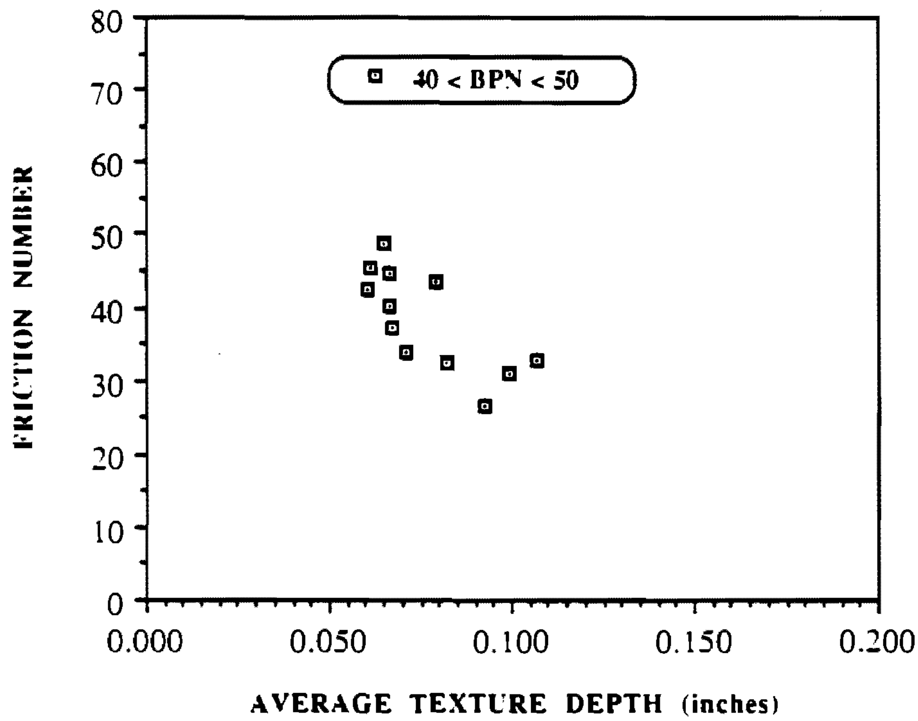


Fig. 6.16 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 40 to 50

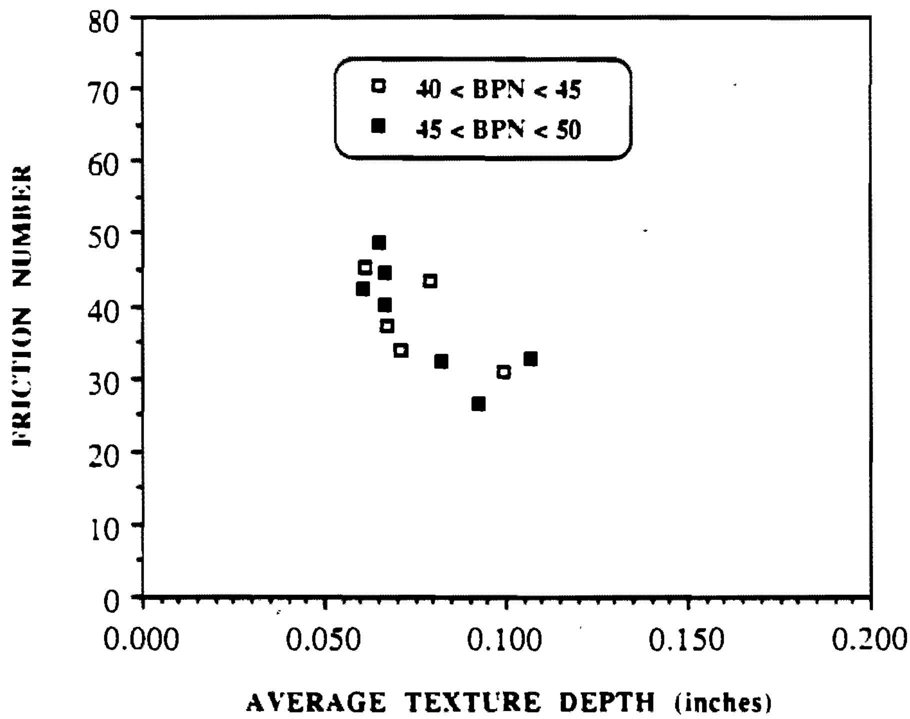


Fig. 6.17 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for Subgrouped BPNs in the Range of 40 to 50

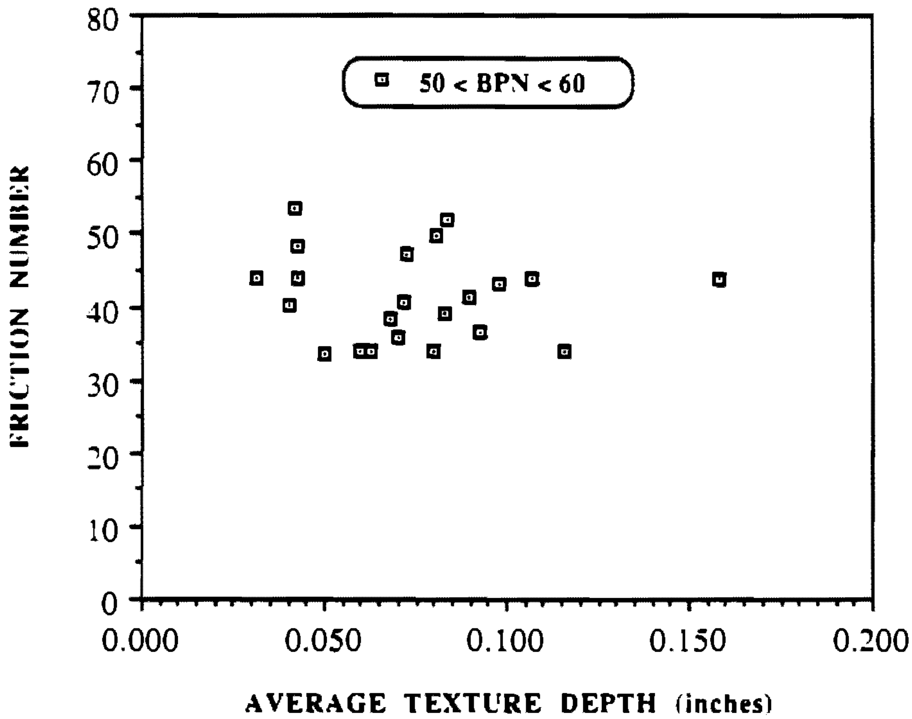


Fig. 6.18 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 50 to 60

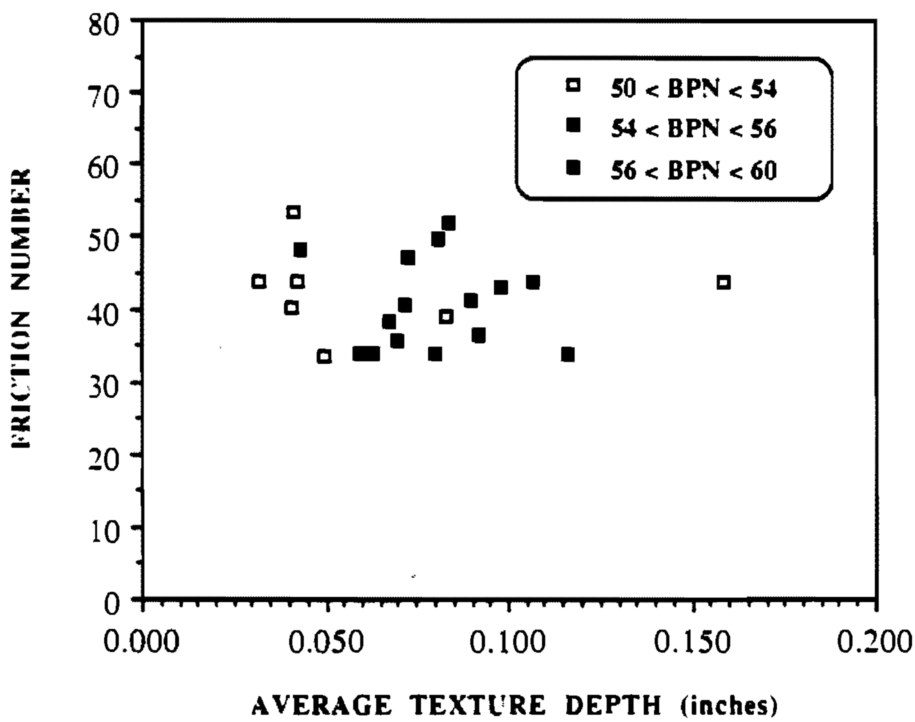


Fig. 6.19 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for Subgrouped BPNs in the Range of 50 to 60

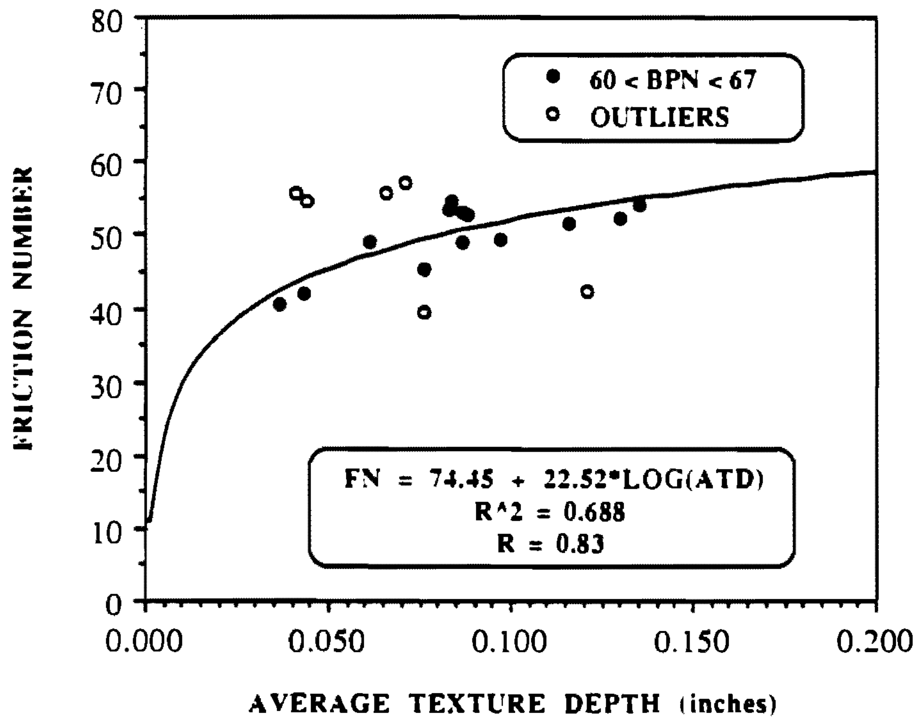


Fig. 6.20 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 60 to 67

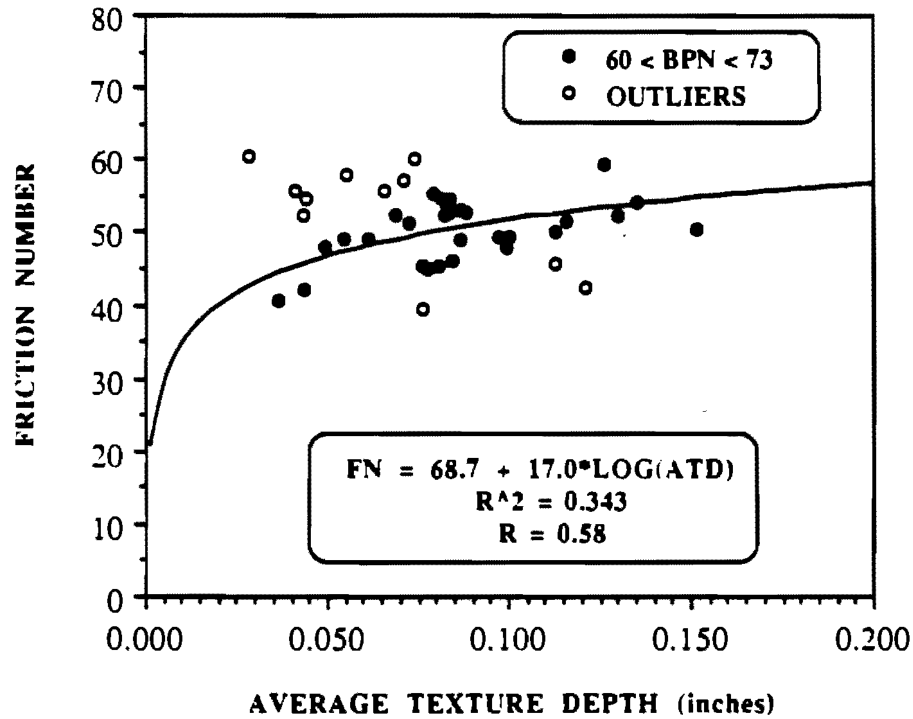


Fig. 6.21 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 60 to 73

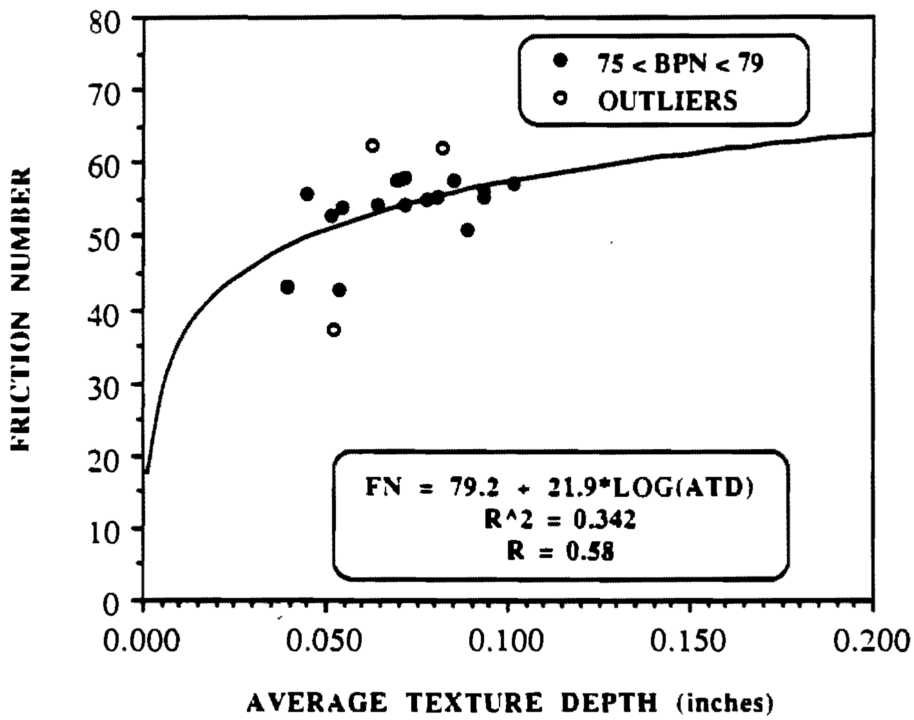


Fig. 6.22 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 75 to 79

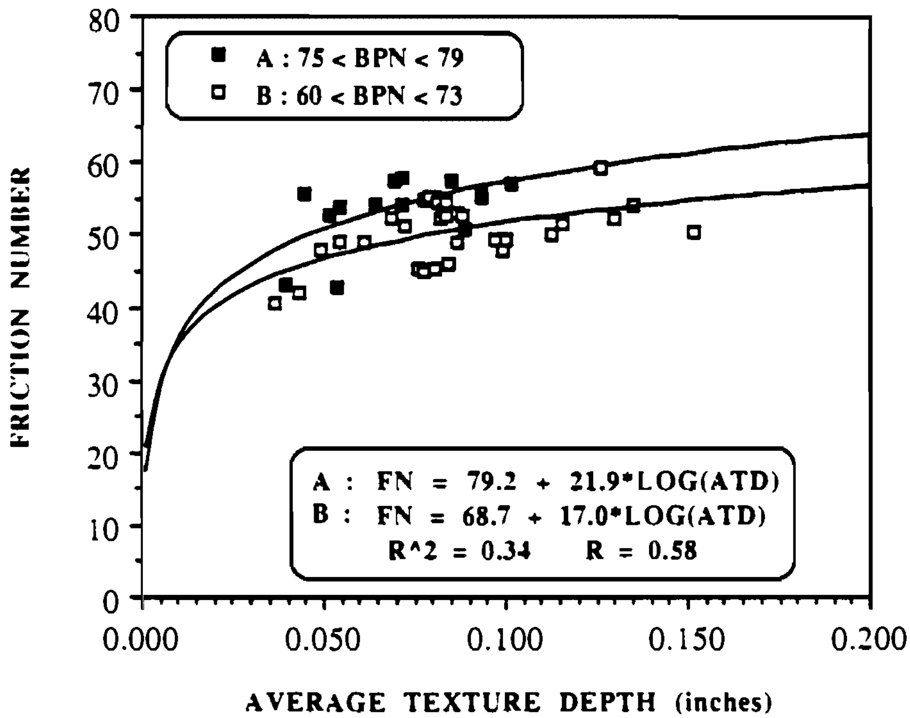


Fig. 6.23 Friction Number vs Average Texture Depth for BPN Ranges of 60 to 73 and 75 to 79

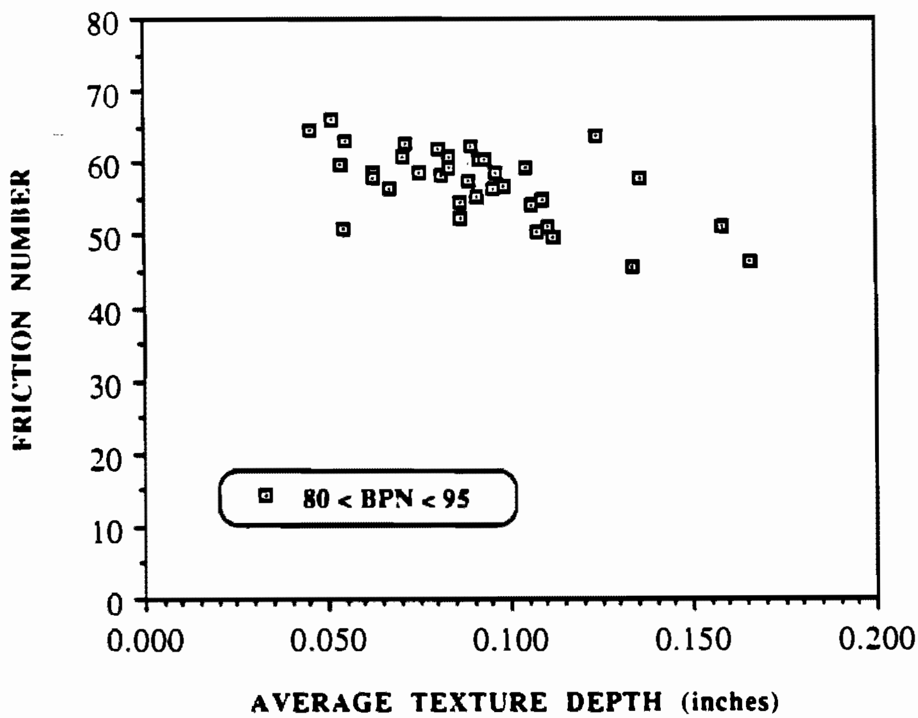


Fig. 6.24 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for BPNs in the Range of 80 to 95

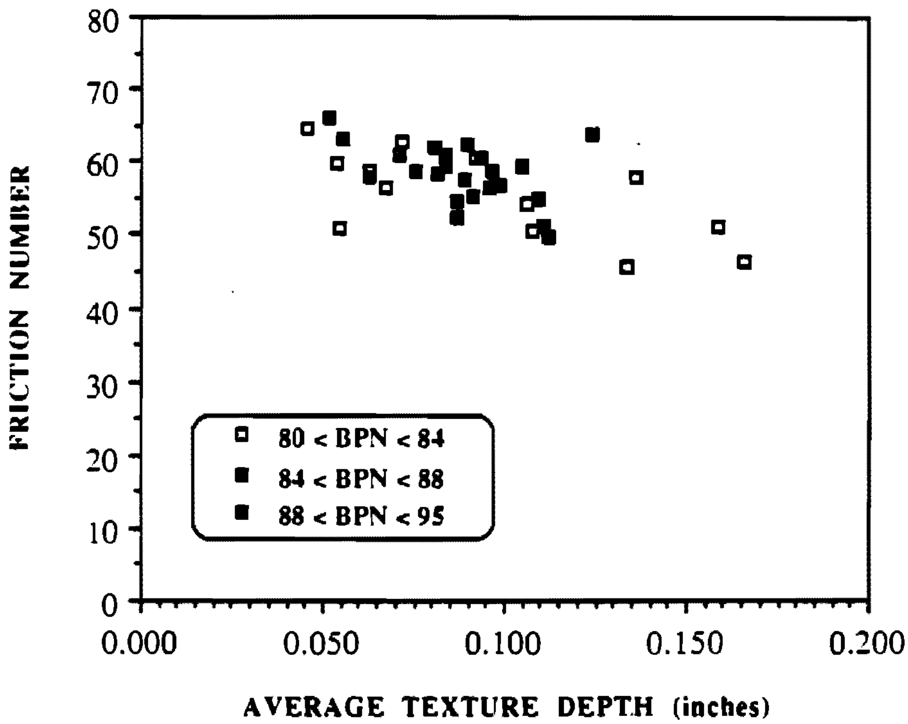


Fig. 6.25 Correlation Between the Results of the Skid Resistance and Sand Patch Tests for Subgrouped BPNs in the Range of 80 to 95

The correlation for a BPN range of 40 to 50 was contrary to what was expected, with the FN decreasing as the ATD increased. The data of this group were then subdivided into two subgroups with narrower BPN ranges to see if any sorting pattern exists in it. However, both subgroups were found to exhibit trends similar to that of the original group. When the second level of BPN, BPN ranging between 50 and 60, was studied, the relationship between FN and ATD, shown in Fig. 6.18, was found to be somewhat linear. The subgrouping of this group's data in Fig. 6.19 did not change the trend of the relationship.

The third level of BPN included BPN readings in the range of 60 to 67. A plot of this data in Fig. 6.20 revealed that most of the observations were concentrated along a logarithmic fit, in which the FN was found to be increasing at a low rate as the ATD increased. A few observations, considered to be outliers, fell on both sides of the logarithmic curve. With the exclusion of these outliers, the correlation had a high correlation coefficient of 0.83 with ATD being accounted for explaining about 70 percent of the variation in the FNs of this group. When the BPN range was expanded to include BPN readings as high as 73, it was found that more observations coincided with the logarithmic fit, with a few others added to the group of outliers. This is illustrated in Fig. 6.21. Because of the increased variability across the fitted correlation caused by the inclusion of more observations, the correlation coefficient decreased to 0.58, and ATD was found to explain no more than 35 percent of the variation in FN. The presence of those outliers falling below the fitted curve is thought to be attributed to some FNs having been affected by a type or another of surface distress. The outliers falling above the fitted correlation are believed to be associated with aggregates having good polish properties, which did not experience any noticeable decrease in FN under the exposure of traffic as a result of having maintained continuously rejuvenated microtextures. In these cases, the FN seems not to have been affected by the decrease in ATD that might have been caused by the exposure to traffic.

The next level of BPN involved readings ranging from 75 to 79. The relationship between FN and ATD for this range, shown in Fig. 6.22, was similar to that of the preceding one, with the exception that the logarithmic fit was shifted upward due to the higher level of microtexture possessed by this group. Figure 6.23 depicts the upward shifting of the FN-ATD correlation for a higher level of BPN, thus demonstrating the dependency of the relationship between the two on the BPN.

The last BPN level studied was largely comprised of those aggregates with excellent polish value properties, namely the LTWT aggregates. The BPN ranged from 80 to 95 in this group. The data, when plotted in Fig. 6.24, showed an increase in FN as opposed to a decrease in ATD. The fact of the matter is that FN and ATD are totally unrelated in this group. As will be discussed in Chapter 7, the individual LTWT aggregates were shown either to have maintained a constant level of FN in the climatic regions with no temperature freeze-thaw cycling or to have exhibited an increase in FN, with exposure to traffic, in the climatic regions with temperature freeze-thaw cycling. Therefore, the frictional performance of the individual LTWT aggregates was independent of the traffic-related decrease in the texture depths of the sections built with these aggregate. The subgroupings of the data in Fig. 6.25 revealed trends similar to that of the original group.

In conclusion, the FN was found to be somewhat related to the ATD for two medium levels of BPN. In addition, the FN was found to be independent of ATD for a high level of BPN. Lastly, a rational correlation between FN and ATD could not detected for low levels of BPN.

6.4 Visual Condition Survey Data

The visual condition survey was made for the purpose of determining the condition of a test section at the time of field testing. The survey is qualitative in nature in that subjective evaluations are made of those surface distresses whose occurrence would undoubtedly influence the friction and texture measurements. The evaluations are made mainly in the wheel paths of a test section, but more attention is given to the left wheel path, where field testing normally takes place.

Three types of distress have been found to affect the friction and texture measurements of seal coat surfaces. These are poor aggregate retention, inadequate aggregate embedment, and bleeding or flushing of asphalt (122, 138). The form shown in Fig. 6.26 was used for gathering data on the extent of these distresses in each test section (122). The results are included in Tables B.1, B.2, and B.3 of Appendix B. In addition, slides of each test section were taken to show the surface condition, particularly the distressed areas.

VISUAL CONDITION SURVEY

Test Section Number : ________ Date : ________ Rater(s) : ________

District : ________ County : ________ Highway : ________

Overall Condition : Poor | Fair | Good
0 2 4 6 8 10

Aggregate Retention :

Outer Wheel Path 100 50 25 15 10 5 2 0
0 2 4 6 8 10

Inner Wheel Path
0 2 4 6 8 10

Between Wheel Path
0 2 4 6 8 10

Bleeding :

Outer Wheel Path Severe | Moderate | Slight
0 2 4 6 8 10

Inner Wheel Path
0 2 4 6 8 10

Between Wheel Path
0 2 4 6 8 10

Aggregate Embedment :

Outer Wheel Path ________%

Inner Wheel Path ________%

Between Wheel Path ________%

Comments : ________

Fig. 6.26 Seal Coat Visual Condition Evaluation Form (122)

Since the results of field testing done on severely distressed sections were thought not to properly represent the frictional properties of the surface aggregate, field testing of such sections continued only for those displaying a differential or discontinuous type of distress along a wheel path. It was then carried on parts of the wheel path where the distress seemed to be minimal. As discussed in the previous section, some discrepancies were observed in the study of the interrelationships among field tests, some of which were due to the fact that skid testing could not be controlled so as to test the parts of a wheel path with the minimal distress.

6.4.1 Aggregate Retention

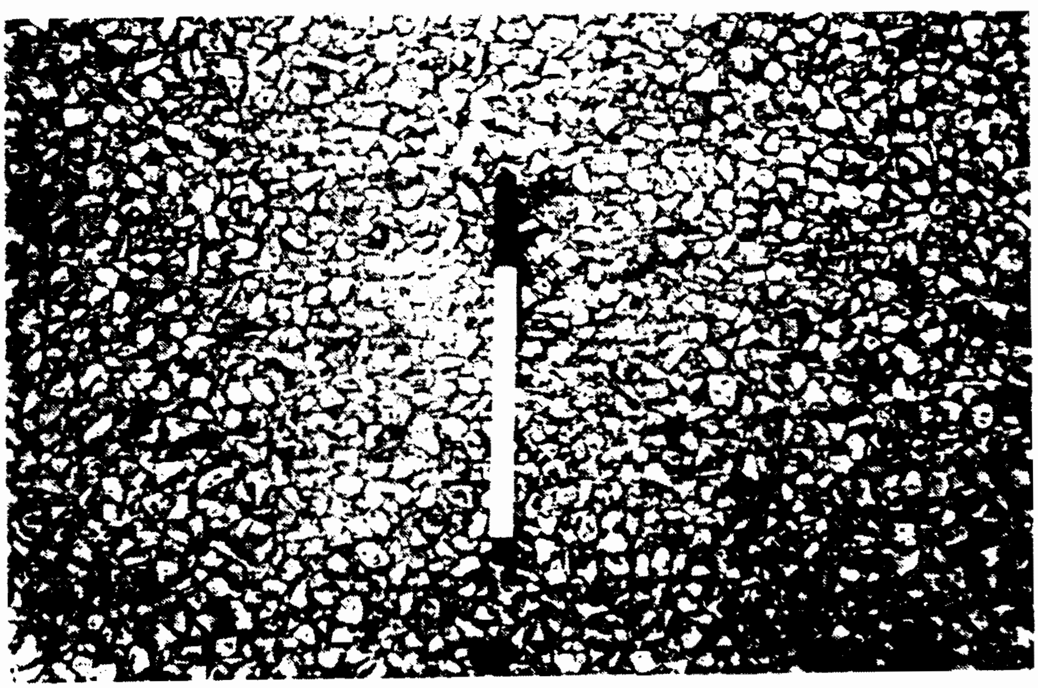
Aggregate retention (AGRT) is usually measured by the percent loss of aggregate particles from seal coat surfaces. As shown in Fig. 6.26, a score of six for aggregate retention refers to aggregate loss in 15 percent of a wheel path area. Factors which may cause poor aggregate retention include unsuitable application rates of asphalt and aggregate, poor construction methods, improper adhesion between the asphalt binder and the aggregate, and inadequate embedment of the aggregate into the asphalt film. Excessive loss of aggregate may result in loss of surface friction. Figures 6.27, 6.28, 6.29 show the three levels of aggregate retention distress exhibited by the test sections. These were good, moderate, and poor referring to score ranges of 8 to 10, 5 to 7, and 0 to 4, respectively.

6.4.2 Bleeding

Bleeding (BLG) or flushing is the presence of excess asphalt or a film of asphalt on the pavement surface. The extent of distress for bleeding is defined by three levels of severity, as shown in Fig. 6.26: slight, moderate, and severe.

Bleeding of asphalt on seal coat surfaces can be caused by a variety of factors, including bleeding in the old surface of the roadway, improper design value of asphalt spreading rate, asphalt too soft for the climate, and poor construction method. Moreover, the bleeding defect is traffic related and occurs in the wheel paths. That is, heavy traffic and/or high traffic volumes can force the asphalt to the surface, especially in hot weather. In any case, bleeding is very detrimental to the surface friction and

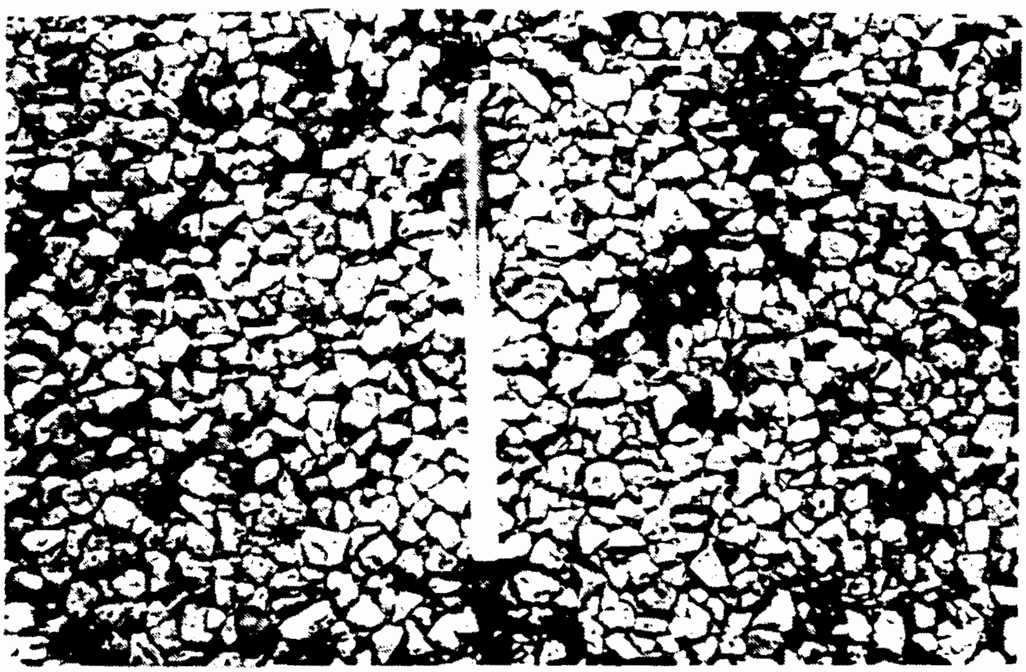
(a) Overall section



(b) Section surface

Fig. 6.27 A seal coat surface with good aggregate retention

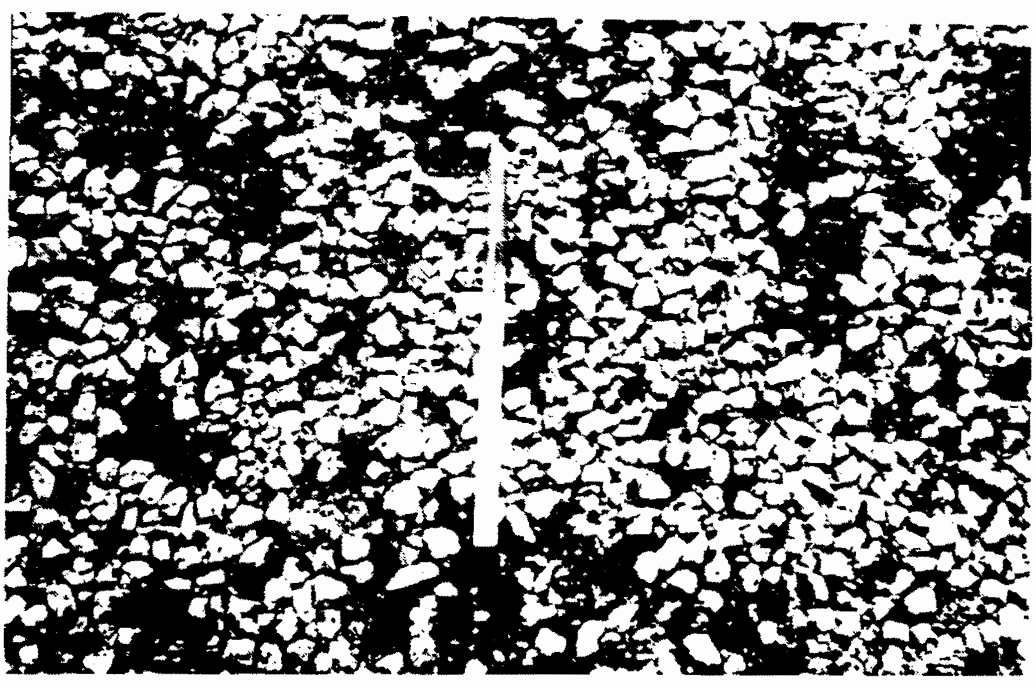
(a) Overall section



(b) Section surface

Fig. 6.28 A seal coat surface with moderate aggregate retention

(a) Overall section



(b) Section surface

Fig. 6.29 A seal coat surface with poor aggregate retention causes problems when measuring the frictional properties of the surface aggregate is of concern.

All three levels of bleeding distress were seen in the test sections. They are illustrated in Figs. 6.30, 6.31, and 6.32.

6.4.3 Aggregate Embedment

Aggregate embedment is determined as a percent of the depth of the aggregate embedded into the asphalt film. The seal coat design methods, the construction operations, and considerations for climatic conditions and traffic should be aimed at providing proper embedment of aggregate particles throughout the life of a seal coat surface. However, if during the life of seal coats the aggregate particles sink excessively into the asphalt, the surface loses texture and friction and may suffer from bleeding of the asphalt.

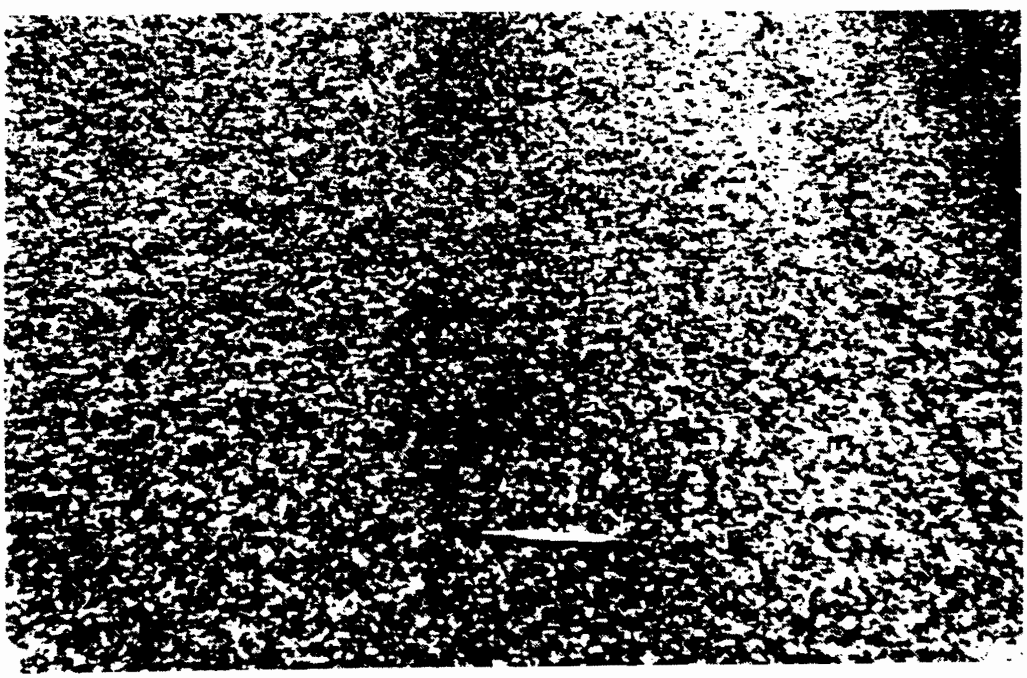
6.5 Weather Data

6.5.1 Short-Term Weather Data

There are many recording weather stations in the State of Texas for which climatological data are published. The primary components of the climatic description furnished by the majority of these stations include precipitation, snowfall, snow on ground, temperature, evaporation, and wind. Eighteen major stations, which are basically located in the large cities, provide more detailed meteorological data.

Climatological data are published by the National Oceanic and Atmospheric Administration have been collected on cumulative precipitation (CUPR) and cumulative number of freeze-thaw cycles (CUFT) for each test section from the nearest recording weather station. Detailed climatological data for the periods prior to and during field testing have also been sought. Specifically, the data have been concerned with the length of the last rainfall period (LRP), the number of days between the last rainfall that occurred in that period and the day of field testing (DLR), and the total precipitation (TPP) that fell in that period. This information, presented in Tables C.1 and C.2 of Appendix C, was to serve as the basis for understanding the effects of short-term

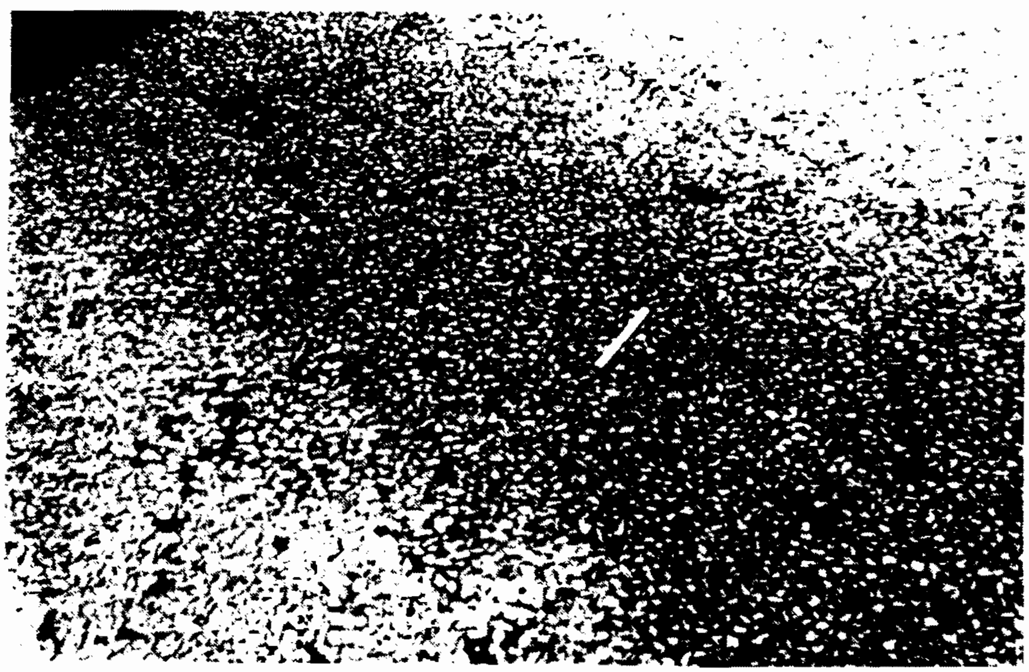
(a) Overall section



(b) Section surface

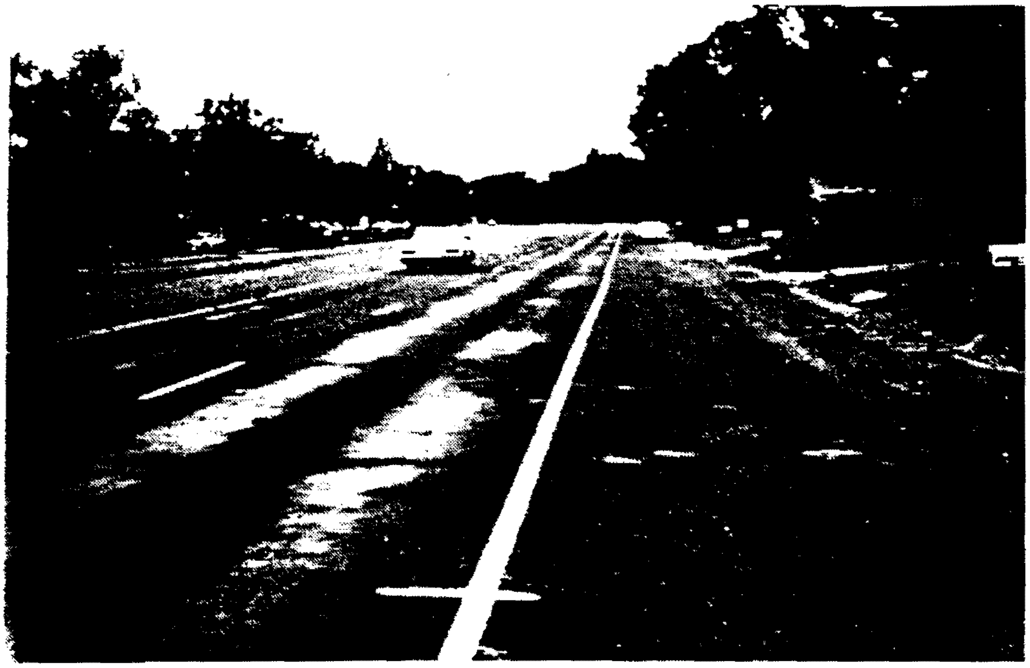
Fig. 6.30 A seal coat surface with slight bleeding

(a) Overall section



(b) Section surface

Fig. 6.31 A seal coat surface with moderate bleeding



(a) Overall section

(b) Section surface

Fig. 6.32 A seal coat surface with severe bleeding weather variations, caused mainly by localized showers, on the frictional properties of roadway surfaces.

6.5.2 Long-Term Weather Data

It was stated earlier in this chapter that the season, dry or wet, in which field testing was undertaken is regarded as a variable affecting the obtained field measurements, and that may account for long-term seasonal variations in pavement surface frictional properties caused by long periods of dryness or wetness. For the purpose of properly defining this weather-caused variable, detailed information on climatic, precipitation related patterns in the State of Texas was searched for (78). The state was found to have ten climatic subdivisions formed by blocks of counties having similar rainfall amounts (16). The climatic subdivisions are East Texas (ET), Upper Coast (UC), South Central (SC), North Central (NC), Southern Division (SD), Lower Valley (LV), Edwards Plateau (EP), Low Rolling Plains (LR), High Plains (HP), and Trans-Peco (TP). These can further be grouped to form three climatic regions having common seasonal rainfall characteristic: summer maximum, summer drought, and May and September maximum. The climatic subdivisions, the climatic regions, and the rainfall characteristics associated with each region are shown in Fig. 6.33.

Monthly precipitation data based on the averages for the 30-year period 1951 to 1980 were obtained from the Texas Water Development Board; use of the 30-year data, as currently computed by the U.S. Weather Bureau, was suggested to be a more realistic approach than use of long-time averages when describing the climate of an area. The data were manipulated and Figs. 6.34 through 6.41 generated. Each of the figures represents a climatic subdivision of Texas. It seems that most of the curves agree with the rainfall patterns of their respective climatic regions, shown in Fig. 6.33. Exceptions are made for the ET curve because of the unusually, relatively high precipitation that occurred in September; for the HP curve, owing to the unexpectedly decreased precipitation in September; and, to some extent, for the UC curve due to the relative decrease in the precipitation trend which occurred in the months of July and August as compared with that of the TP curve. Table 6.5 shows the 30-year average annual precipitation amounts for each subdivision.

Fig. 6.33 The Ten Climatic Subdivisions (Blocks of Counties Having Similar Rainfall Amounts) and the Three Climatic Regions in Texas (Blocks of Climatic Subdivisions Having Common Seasonal Rainfall Characteristics) (16)

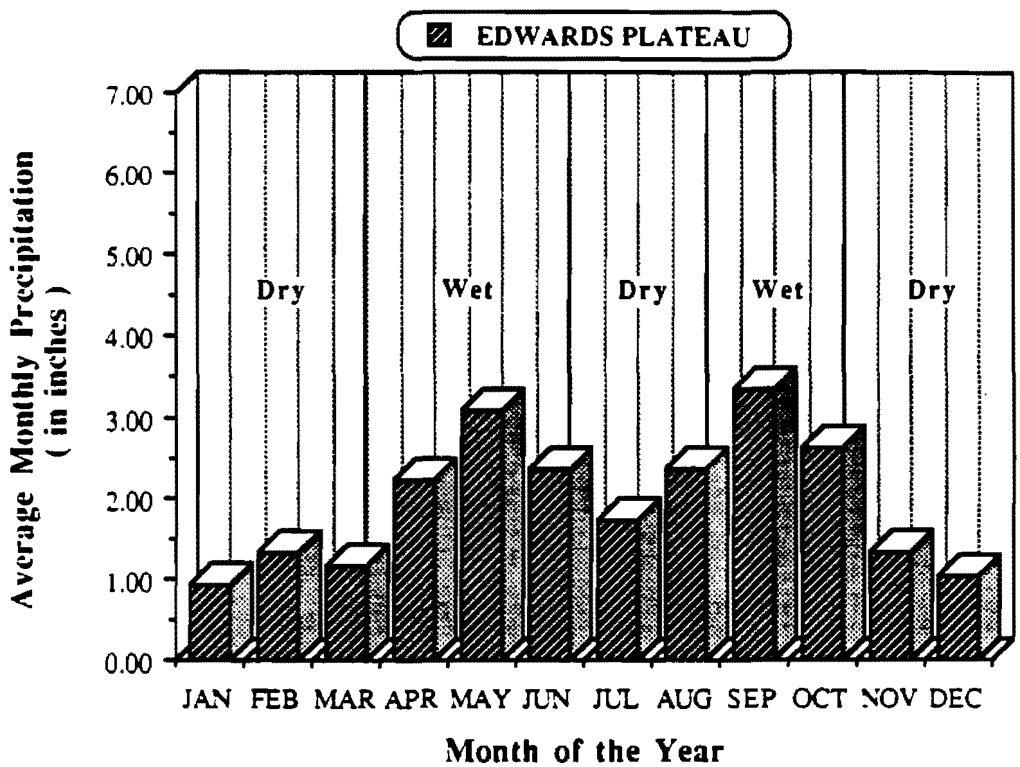


Fig. 6.34 Thirty-Year Average Monthly Precipitation for the Edwards Plateau Climatic Subdivision

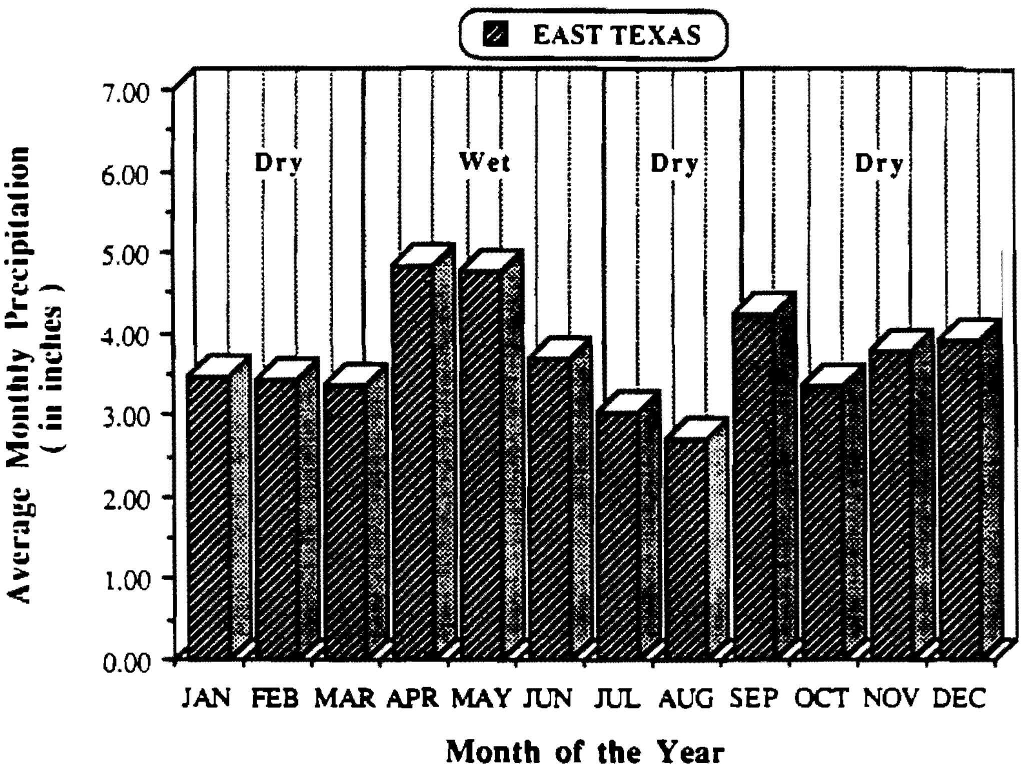


Fig. 6.35 Thirty-Year Average Monthly Precipitation for the East Texas Climatic Subdivision

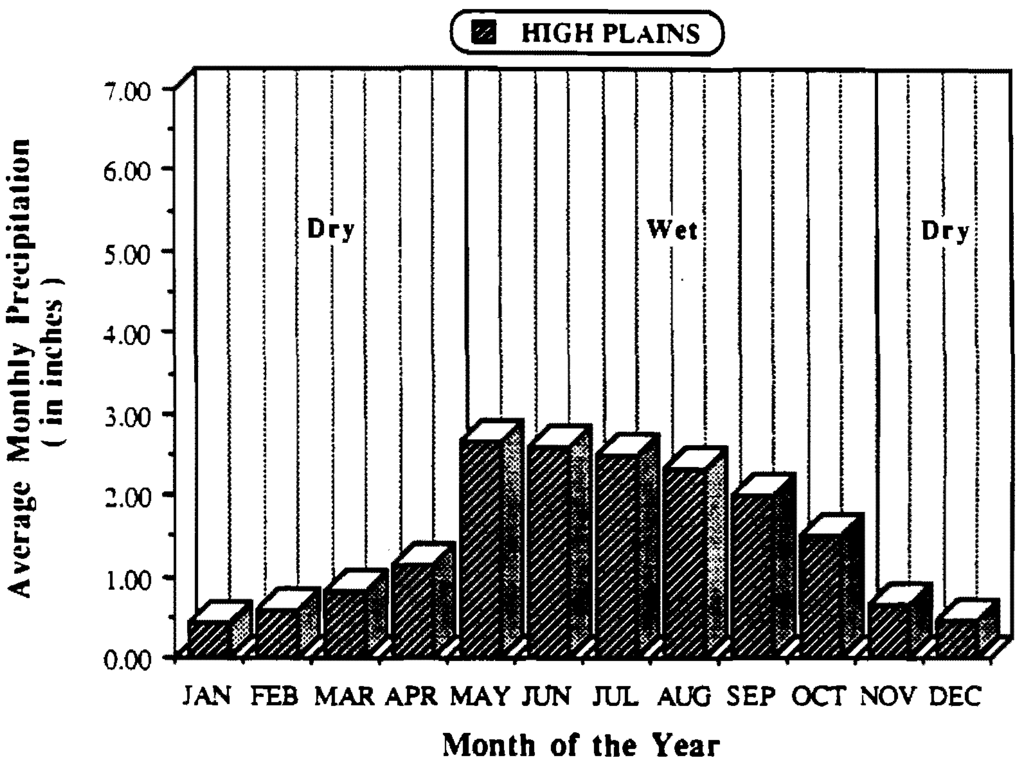


Fig. 6.36 Thirty-Year Average Monthly Precipitation for the High Plains Climatic Subdivision

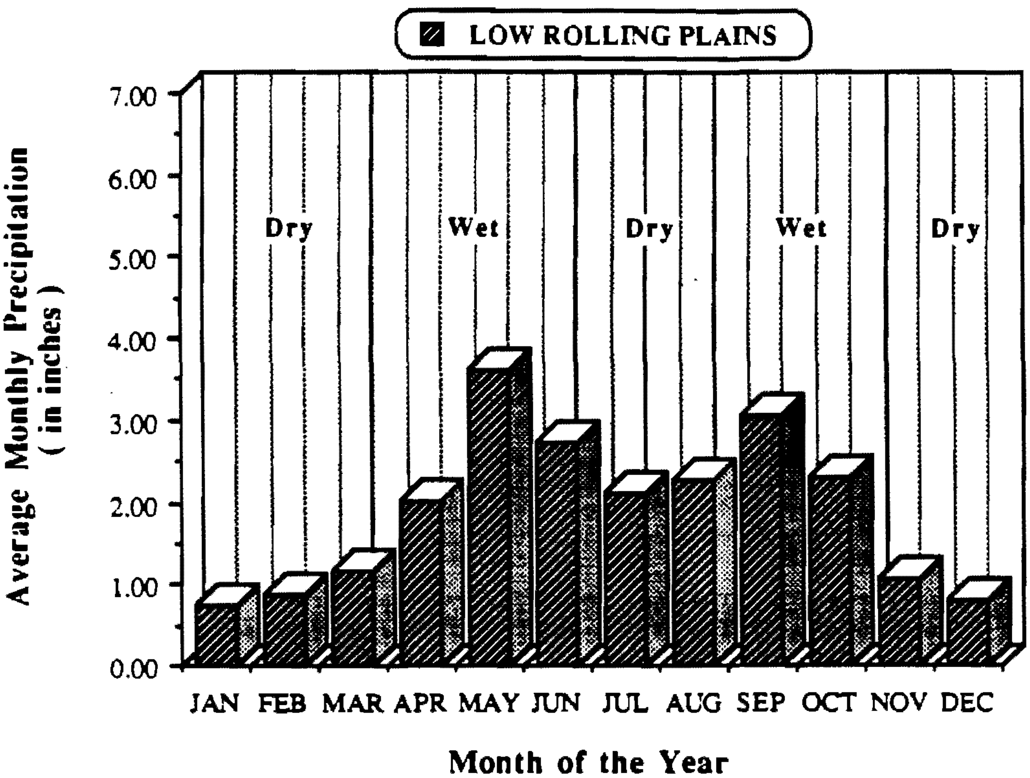


Fig. 6.37 Thirty-Year Average Monthly Precipitation for the Low Rolling Plains Climatic Division

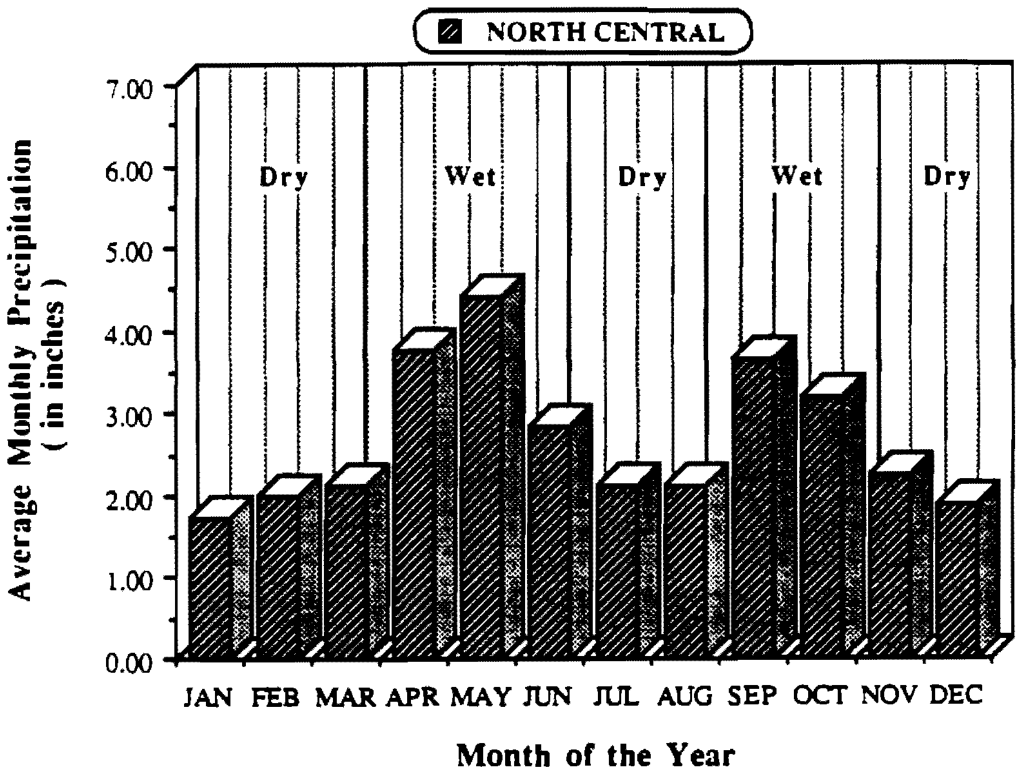


Fig. 6.38 Thirty-Year Average Monthly Precipitation for the North Central Climatic Subdivision

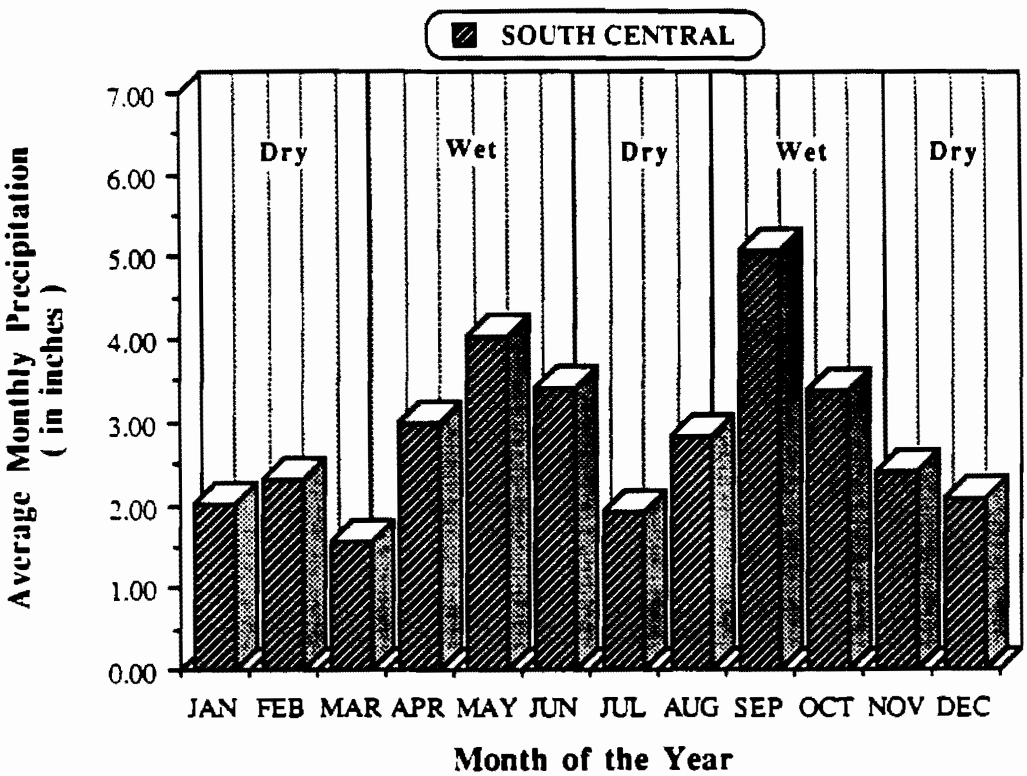


Fig. 6.39 Thirty-Year Average Monthly Precipitation for the South Central Climatic Subdivision

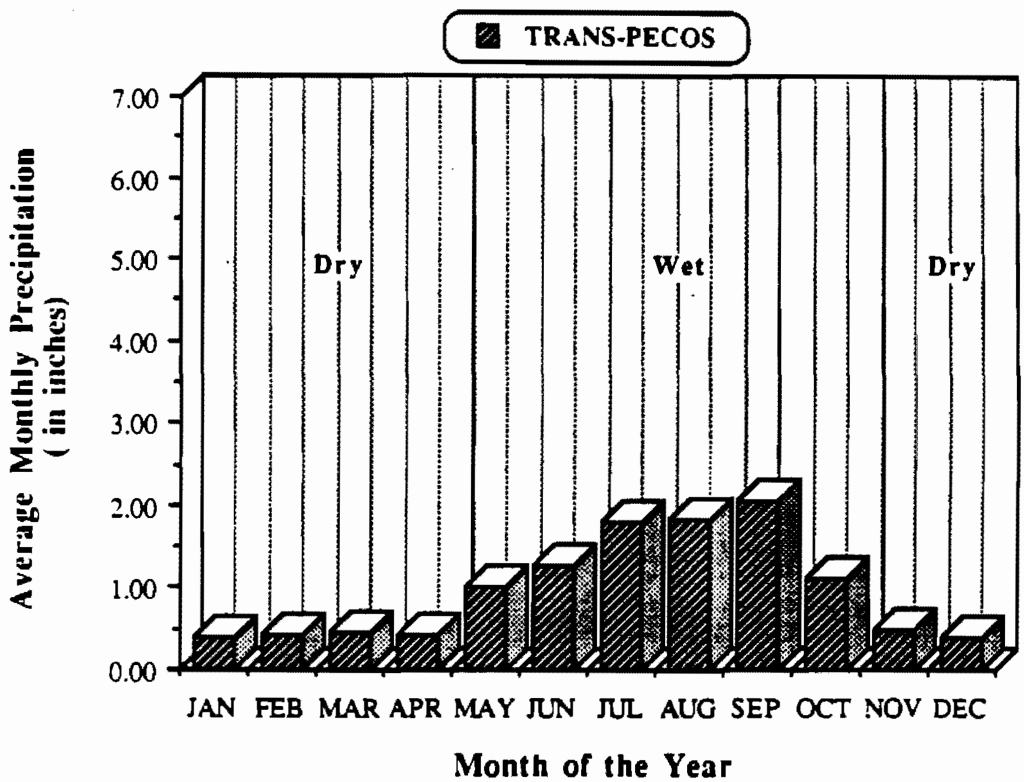


Fig. 6.40 Thirty-Year Average Monthly Precipitation for the Trans-Pecos Climatic Subdivision

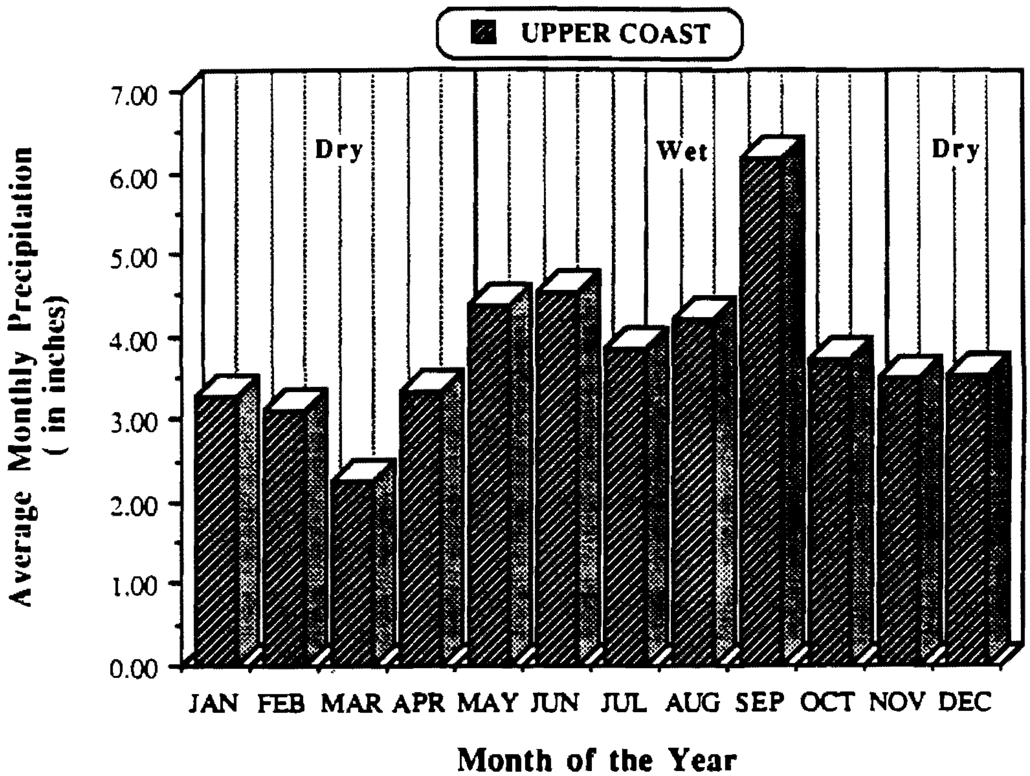


Fig. 6.41 Thirty-Year Average Monthly Precipitation for the Upper Coast Climatic Subdivision

Table 6.5 Thirty-Year Average Annual Precipitation Amounts for Each Climatic Subdivision

Climatic Division	Average Annual Precipitation
HP	17.73
LR	22.80
NC	32.14
ET	44.70
TP	11.65
EP	23.52
SC	34.03
UC	45.93
SD	22.91
LV	24.73

Based on these observations, the following was warranted. First, the HP subdivision was considered to have manifested a rainfall pattern somewhat similar to that of the TC subdivision. It was thus grouped with the TC and UC subdivisions in the region of summer maximum as far as the determination of wet and dry periods was concerned. Testing seasons, included in Table C.1 and C.2 of Appendix C, were then determined based on a tentative segmentation of a year into wet and dry seasons as follows:

1. For the UC, TP, and HP subdivisions, from the beginning of May until the end of October is wet, while from the beginning of November until the end of April is dry.
2. For the ET region, from the middle of March until the end of June and from the beginning of September until the middle of December are wet,

whereas from the beginning of July until the end of August and from the middle of December until the middle of March are dry.

3. For the Interior-Region excluding the HP Subdivision, from the middle of March to the middle of June and from the middle of August until the end of October are wet, while from the middle of June until the middle of August and from the beginning of November until the middle of March are dry.

Finally, it should be noted that the climatic regions described in this section are not to be confused with the four environmental regions discussed in Chapter 4 which are based on precipitation and temperature freeze-thaw cycling. In fact, it is worthwhile to mention that the 30-year average annual precipitation amounts, shown in Table 6.5, do justify how the state was divided vertically into dry and wet zones as shown in Fig. 4.2. That is, the ET and UC subdivisions have average annual precipitation amounts of 44.7 inches and 45.93 inches respectively, which by far exceed those of all adjacent subdivisions. Moreover, the western borders of these subdivision in Fig. 6.33 almost coincide with the vertical dividing line in Fig. 4.2.

CHAPTER 7
AN EXAMINATION OF THE EFFECTS OF THE INVOLVED VARIABLES ON SEAL COAT FRICTIONAL PERFORMANCE

7.1 Introduction

In this chapter, a detailed examination of the frictional performance of the established test sections is presented. The objectives were to develop an understanding of the types and sources of the encountered variations and to evaluate and/or isolate the effects of the important variables involved. To achieve these objectives, the magnitude of the random performance variations in the constructed seal coat replicates was studied. Then, the effects of some variables were evaluated and isolated by examining the performance of test sections constructed end-to-end. Finally, the frictional performance data was grouped according to the major variables involved to graphically illustrate the experimental designs needed for the formulation of prediction models. The variables identified in these designs represented the main components of the statistical models formulated in Chapter 8. Only six of the obtained eight sets of friction measurement were used for the examinations presented in this chapter.

7.2 Performance Variations Observed in the Constructed Replications

As discussed in Chapter 4, three major types of replications were considered. These were traffic, construction, and time or pit replications, all of which are expected to introduce some sort of random variations to the performance data. The frictional performances of the replicates of each type are examined as follows.

7.2.1 Traffic Replications

7.2.1.1 Bound Replications. According to the findings in Chapter 6, traffic was considered to be divided equally between the two directions of a highway section. Therefore, the frictional performance of aggregates is expected to be similar for both bounds, and any variation is considered to be random unless a logical reason exists that may explain the variation.

Figures 7.1 through 7.7 show the performance of six groups of bound replicates, constructed with LMST, LTWT, and SDST aggregates. The variations in these groups were observed to be minimal except for sections 29 and 30. These sections, constructed with the same LMST material, were established on the same highway about less than one mile apart. In both sections, it was the south bound that exhibited lower FNs throughout. The condition surveys revealed that both bounds suffered from an asphalt bleeding distress, but the distress was evaluated to be severe in the south bound as opposed to moderate in the north bound. Thus, the variation between these bounds can no longer be considered random.

7.2.1.2 Lane Replications. These replications were built on the inner and outer lanes of four-lane highways with the objective of determining whether any performance variation between the two lanes might result which could be attributed to the level of traffic passing on each lane. Ten groups of lane replicates are presented in Figs. 7.8 through 7.17. In most of them, the inner lanes were observed to have maintained levels of frictional performance higher than those exhibited by the outer lanes. This indicates that the level of ADT was truly lower on these lanes, and the variations in most of these groups can no longer be considered random. The effect of the level of ADT will be further discussed later in this chapter.

The LMRA replicates, shown in Fig. 7.8, were constructed on an urban highway where traffic was assumed to be divided equally between the inner and outer lanes. The performance of section 15, placed on the inner lane, seemed not to have differed much from that of sections 19 and 20, placed on the outer lane, thus indicating the traffic was indeed distributed equally on these lane.

7.2.2 Construction Replications

These replications were constructed to evaluate any performance variations that might be due to changes in construction equipment application rates or condition of the existing surfaces. The magnitude of the variations encountered in these replications is shown in Figs. 7.18 through 7.34. In most of these groups of replicates, the variations were seen to be minimal, with only few exceptions found.

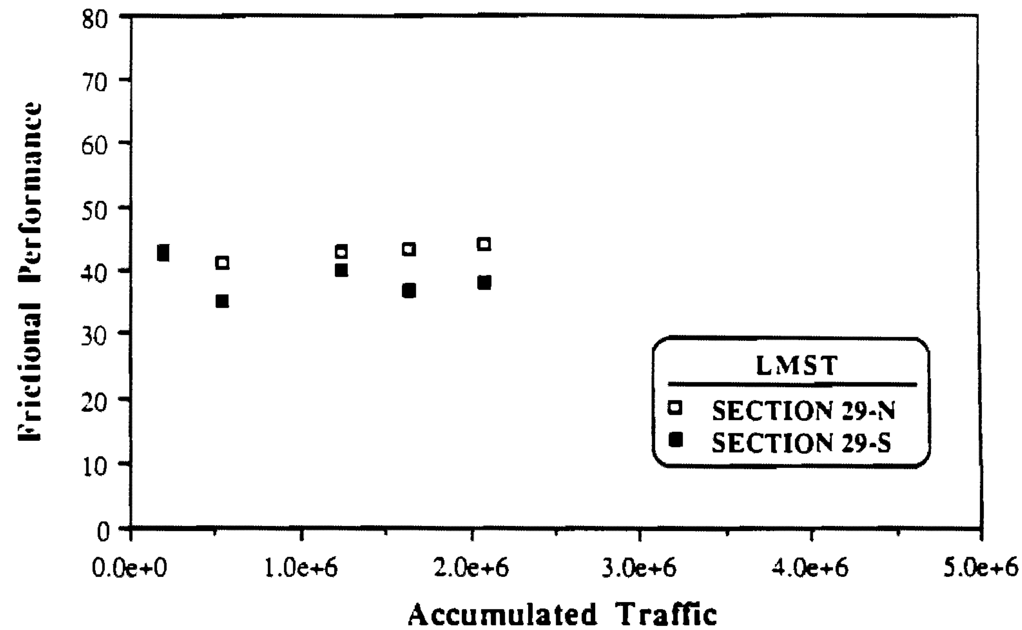


Fig. 7.1 Frictional Performance Variation Observed in the North and South Bounds of Section 29

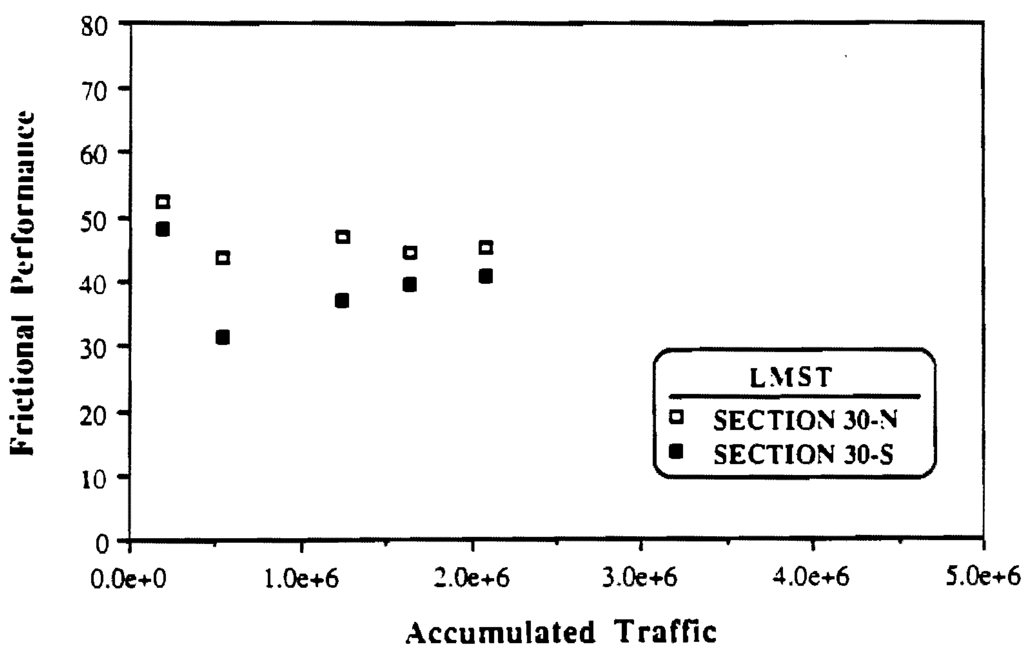


Fig. 7.2 Frictional Performance Variation Observed in the North and South Bounds of Section 30

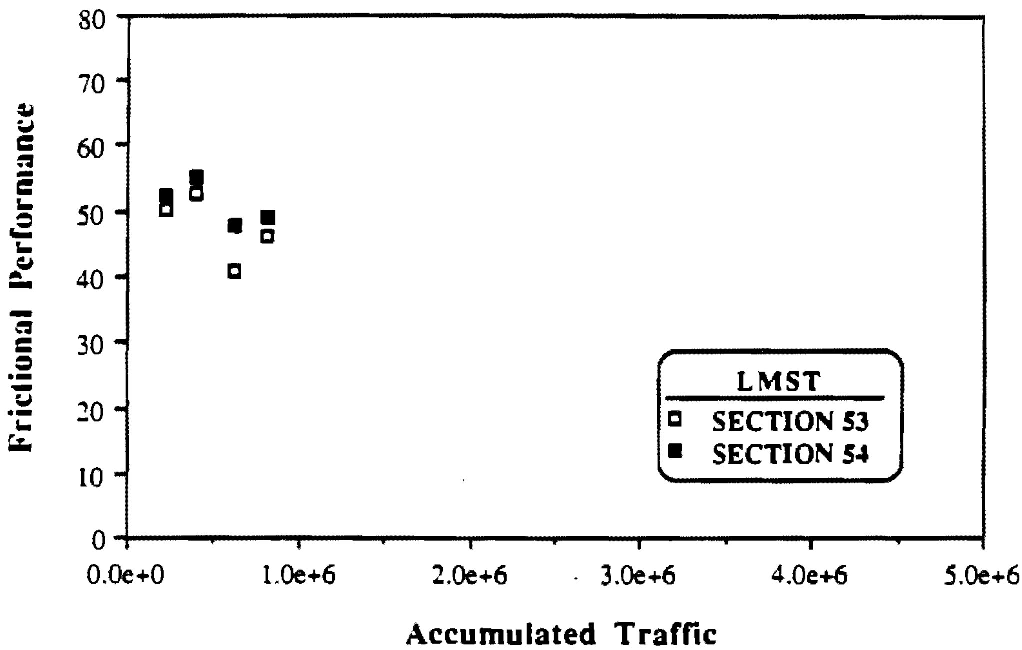


Fig. 7.3 Frictional Performance Variation Observed in Sections 53 and 54

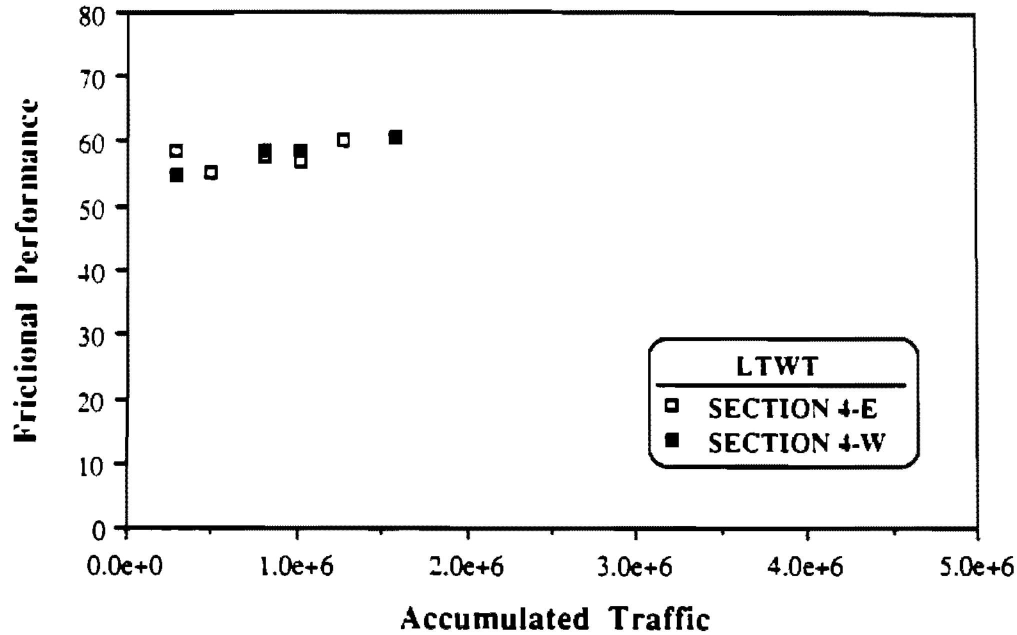


Fig. 7.4 Frictional Performance Variation Observed in the East and West Bounds of Section 4

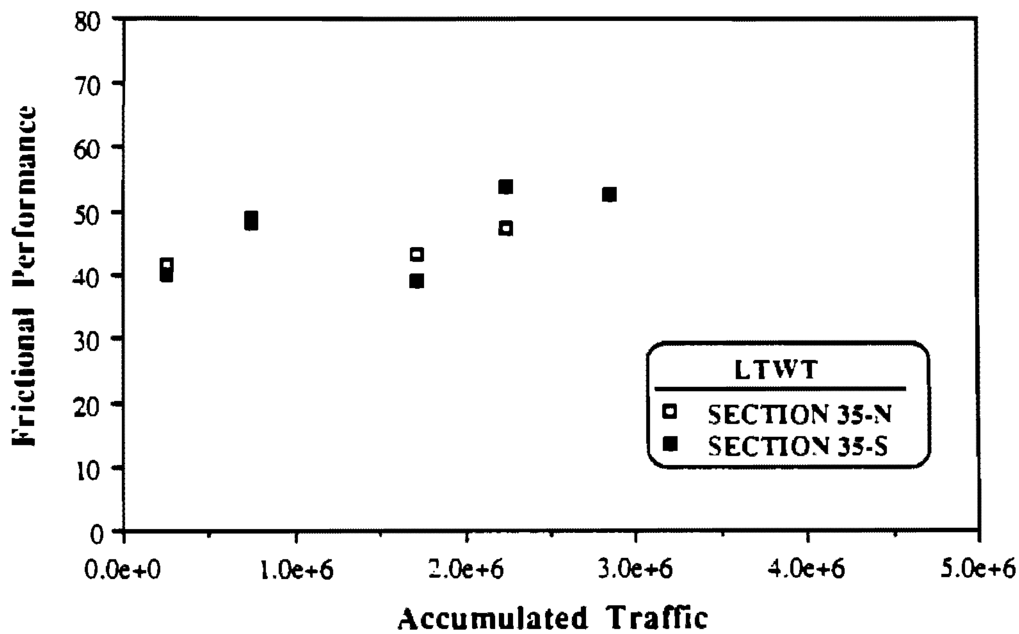


Fig. 7.5 Frictional Performance Variation Observed in the North and South Bounds of Section 35

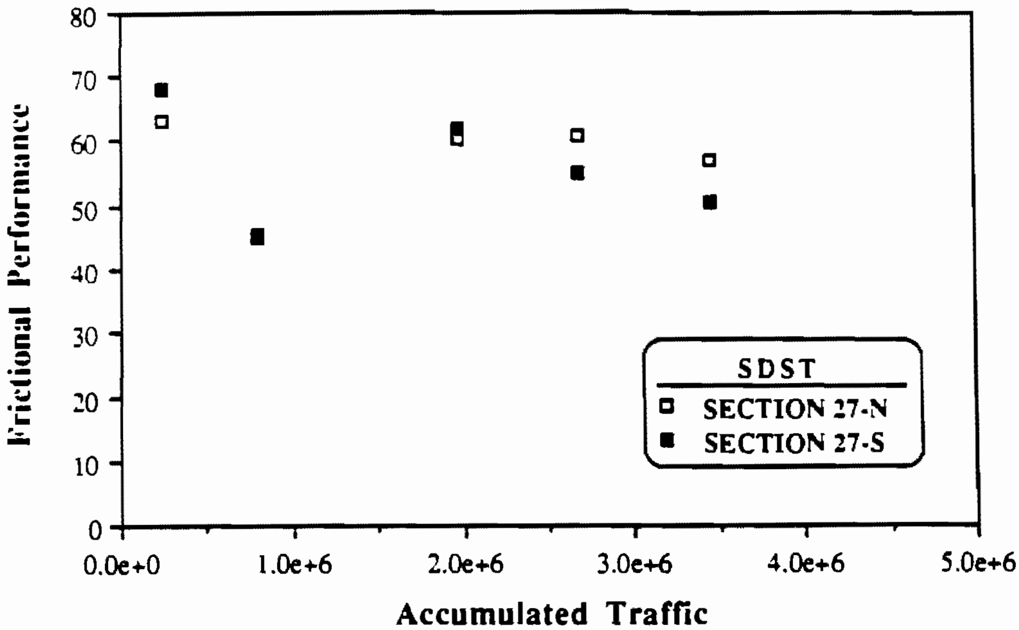


Fig. 7.6 Frictional Performance Variation Observed in the North and South Bounds of Section 27

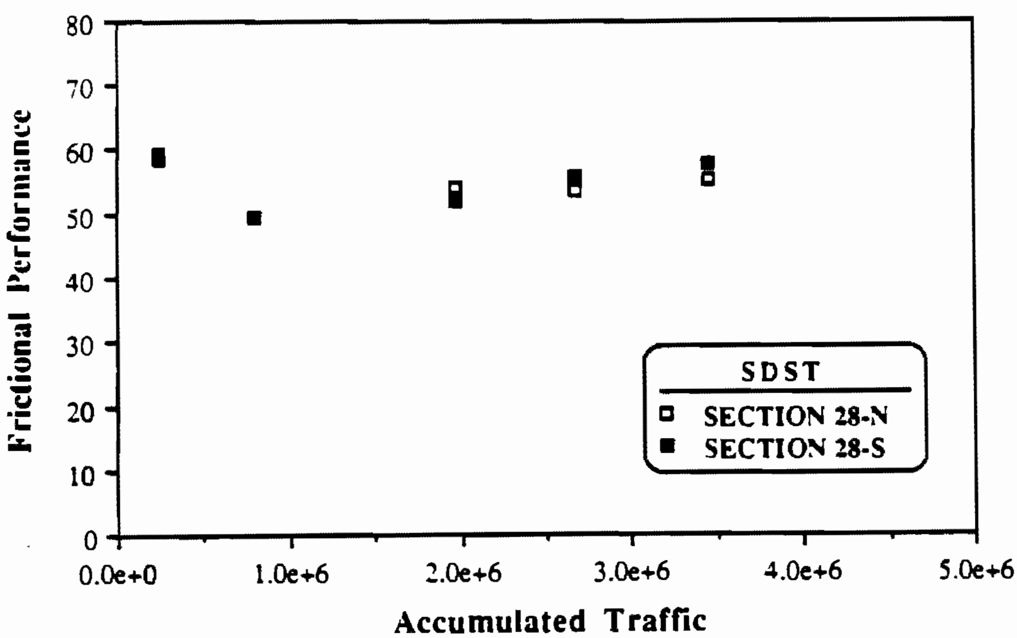


Fig. 7.7 Frictional Performance Variation Observed in the North and South Bounds of Section 28

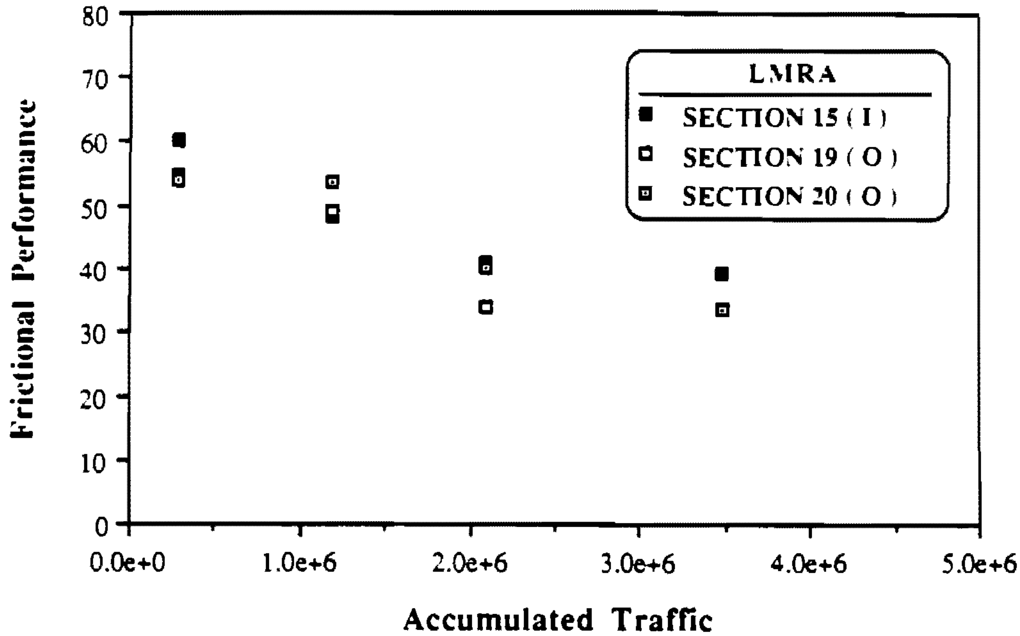


Fig. 7.8 Frictional Performance Variation Observed in Sections 15, 19, and 20

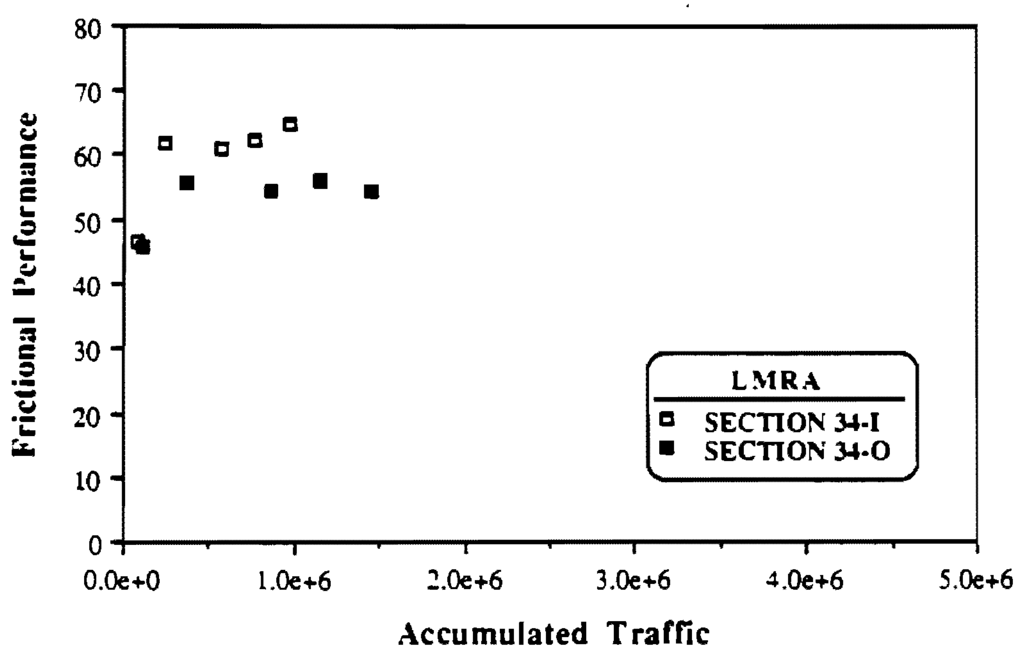


Fig. 7.9 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 34

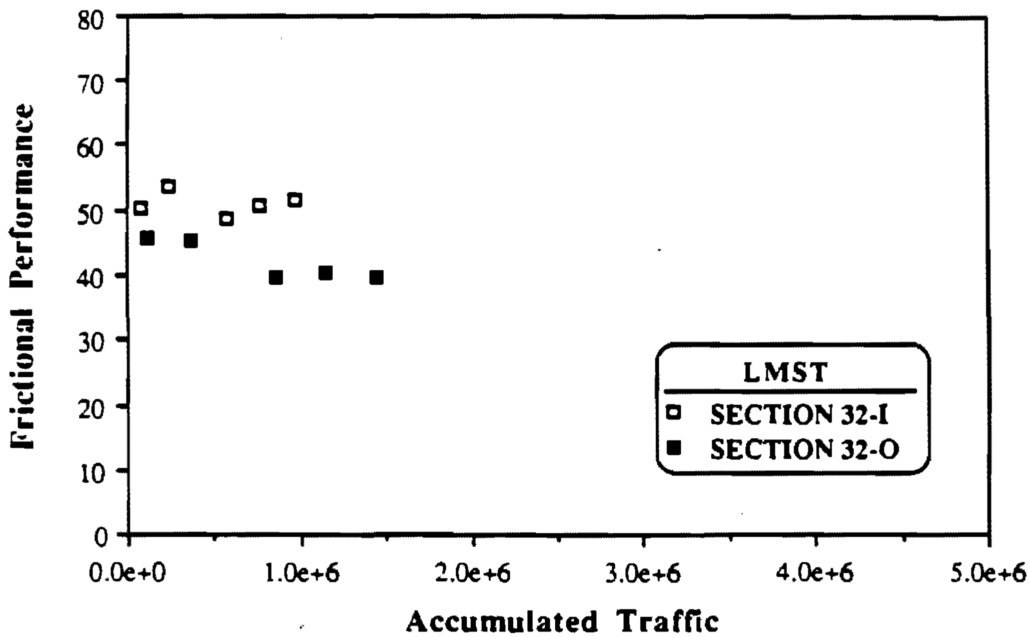


Fig. 7.10 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 32

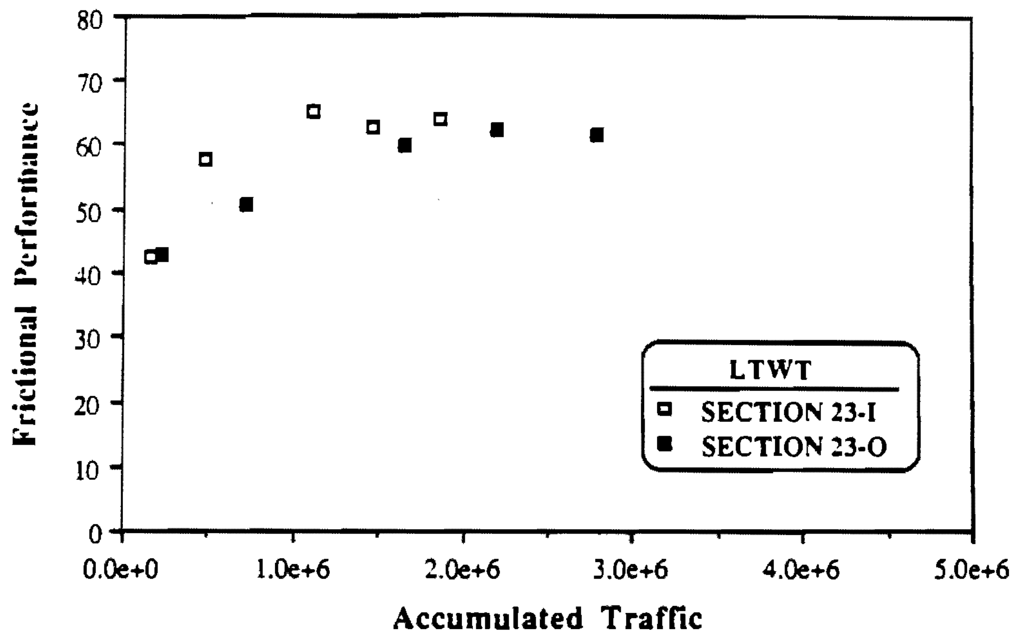


Fig. 7.11 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 23

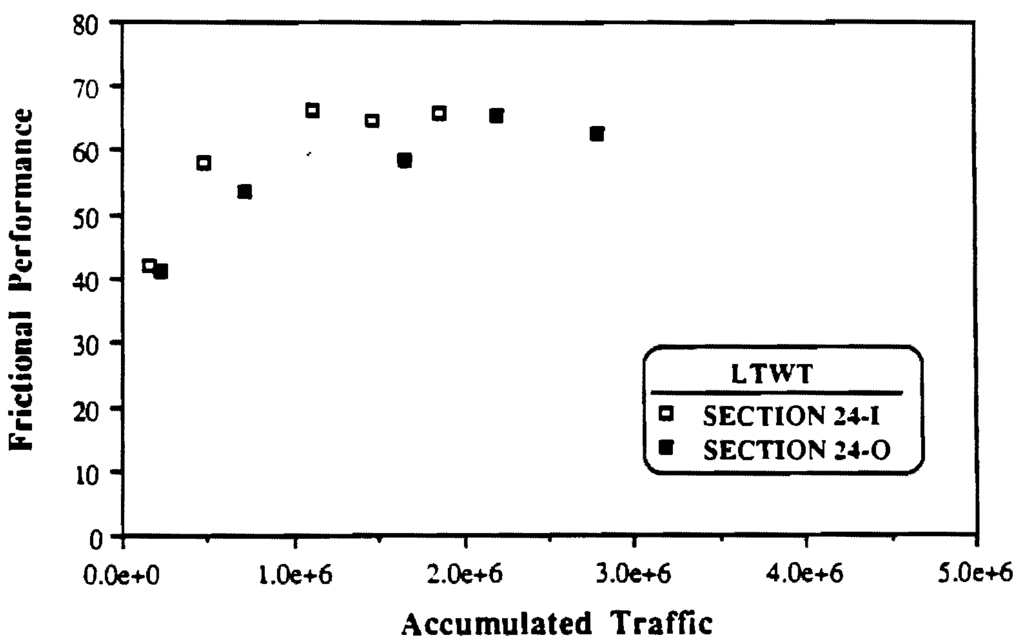


Fig. 7.12 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 24

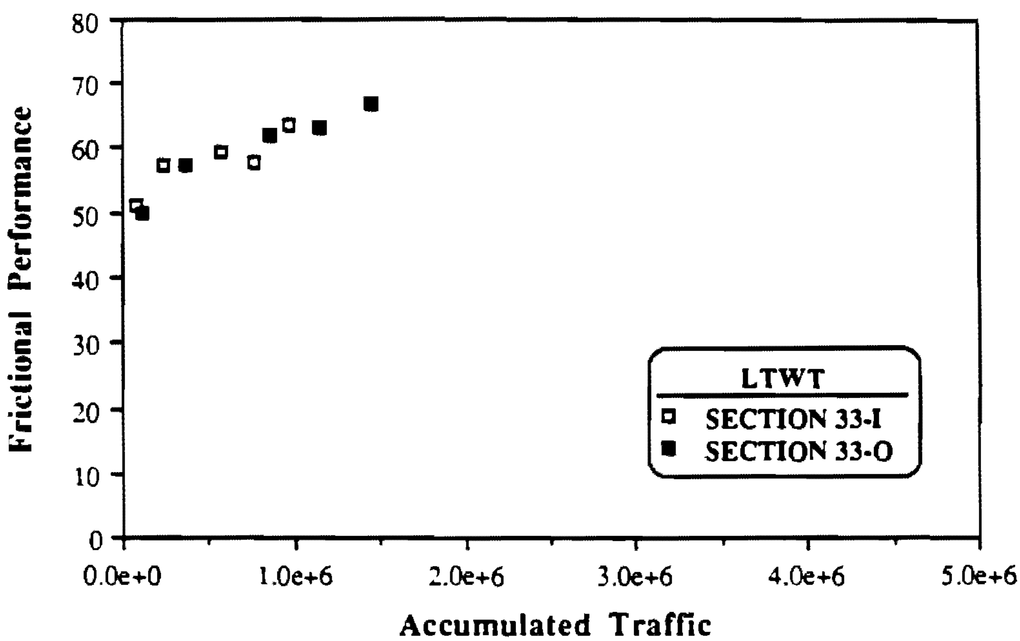


Fig. 7.13 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 33

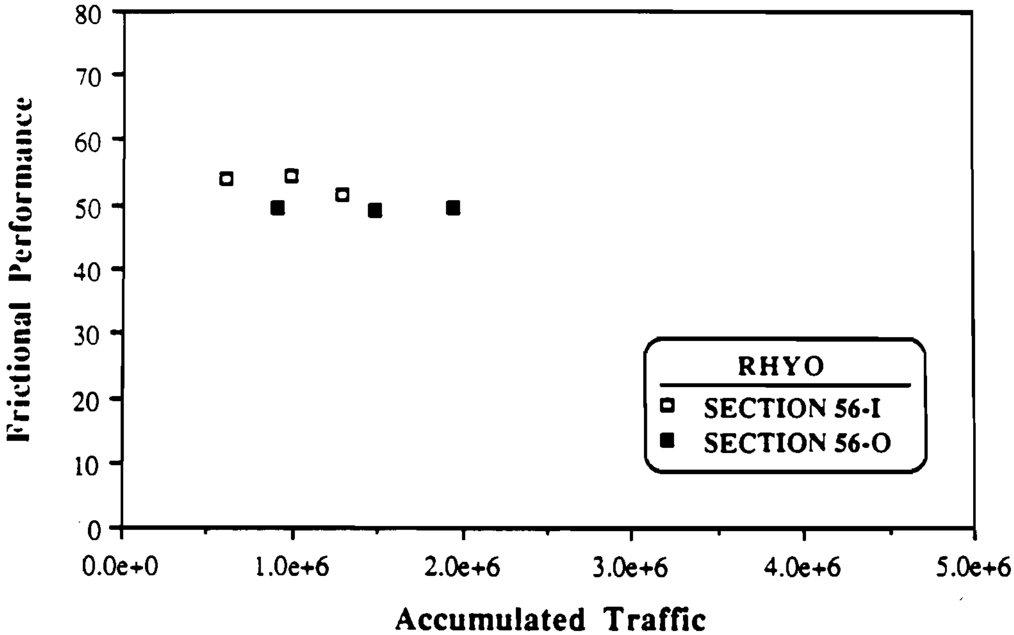


Fig. 7.14 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 56

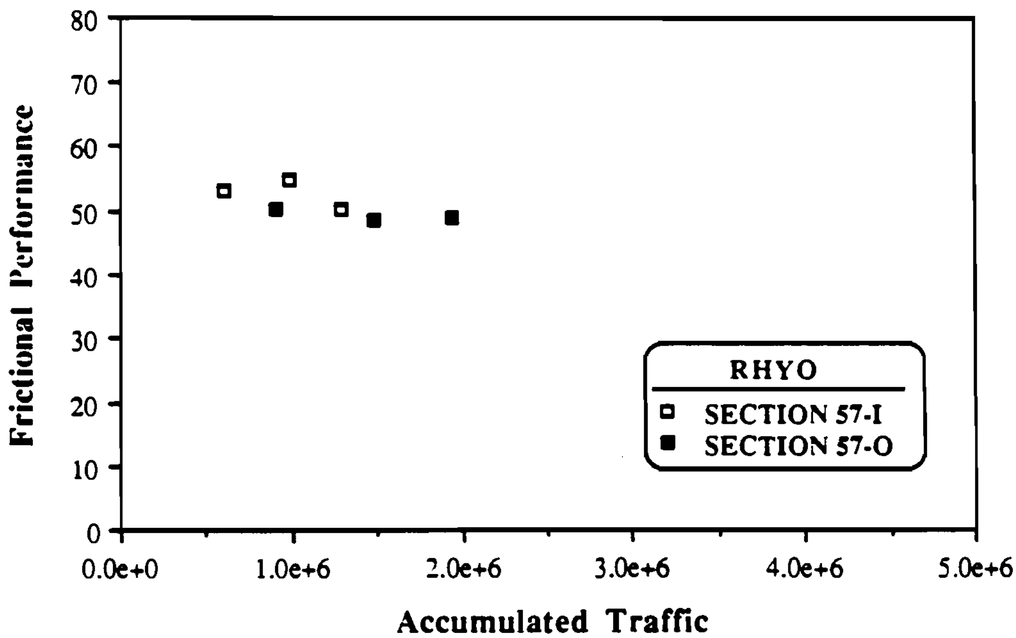


Fig. 7.15 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 57

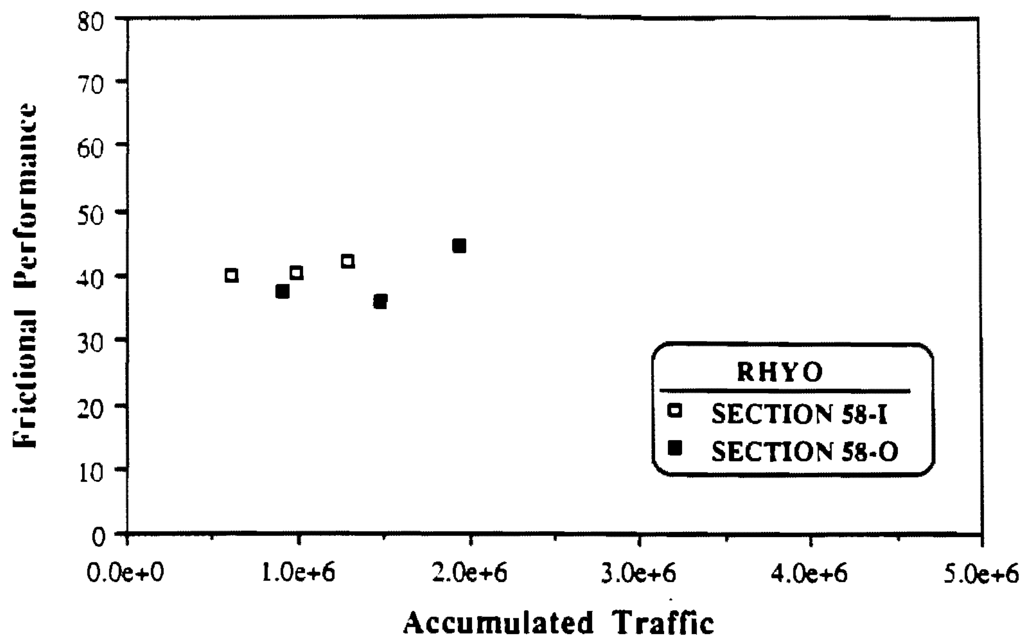


Fig. 7.16 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 58

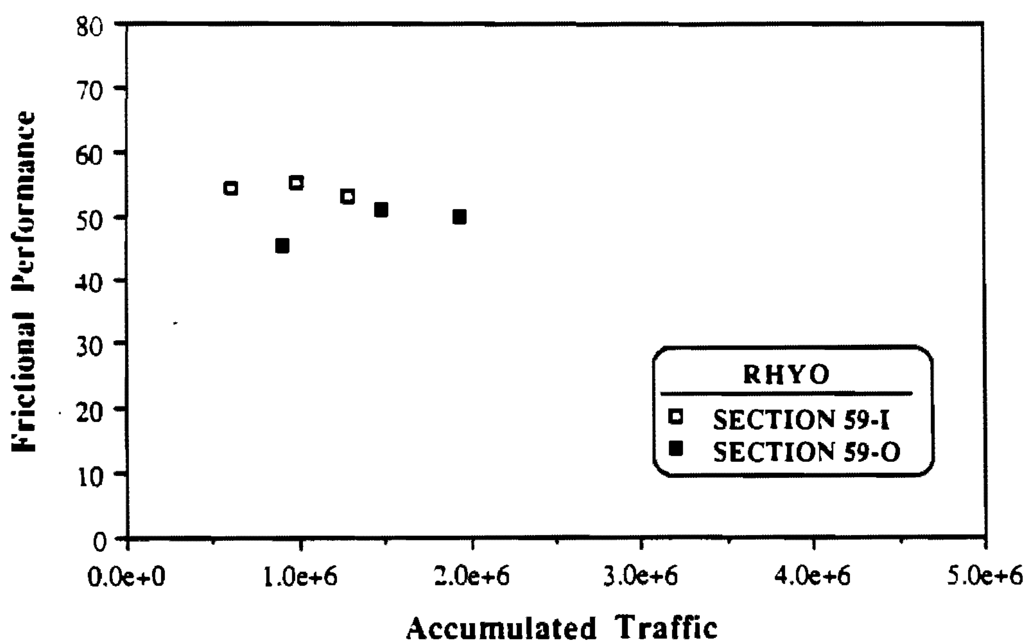


Fig. 7.17 Frictional Performance Variation Observed in the Inner and Outer Lanes of Section 59

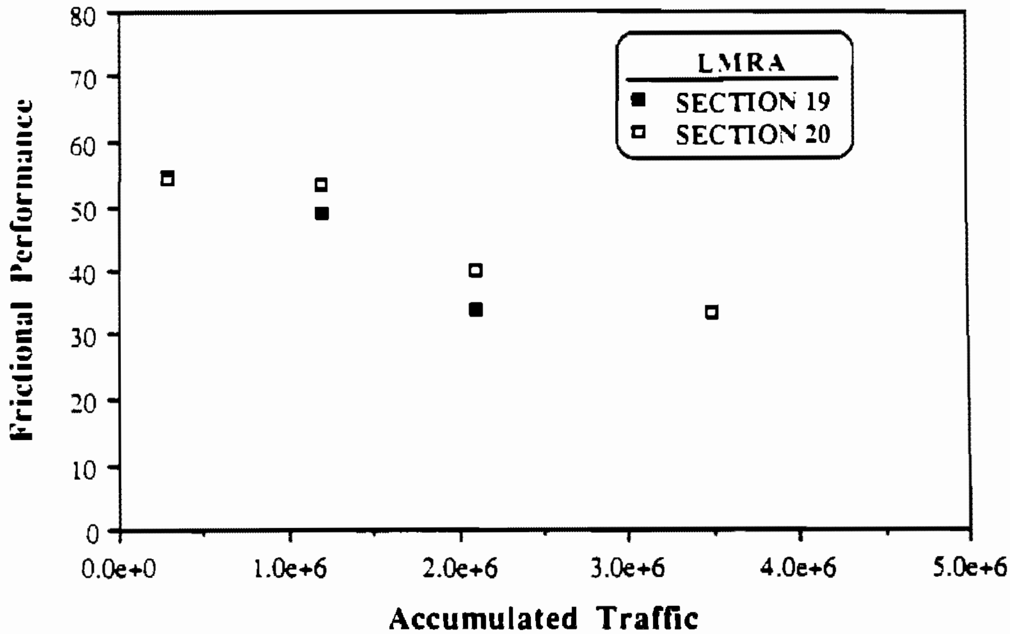


Fig. 7.18 Frictional Performance Variation Observed in Construction Replicates 19 and 20

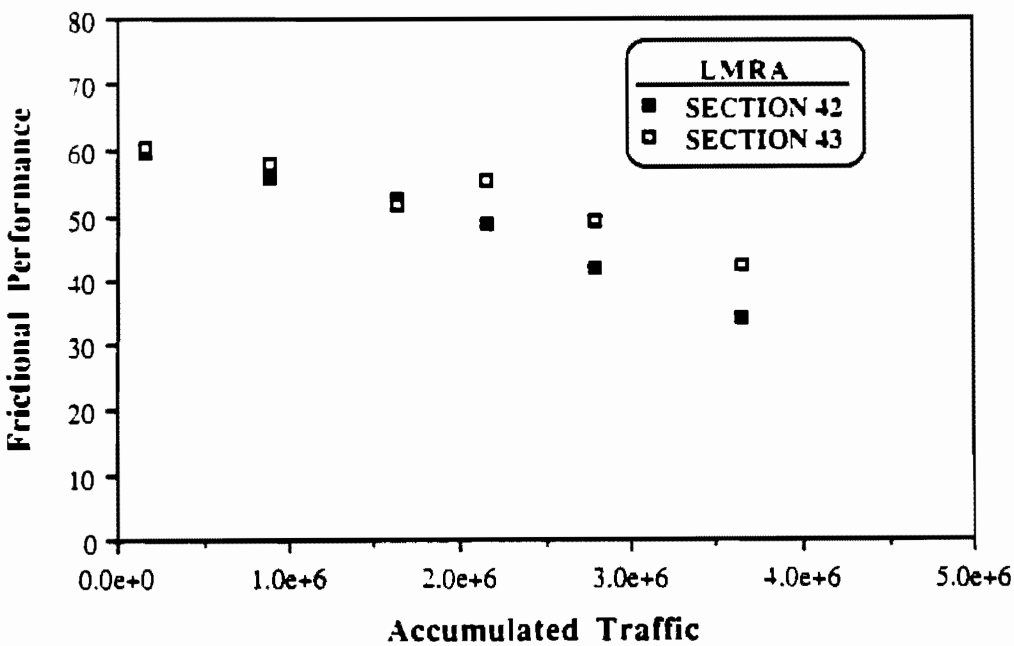


Fig. 7.19 Frictional Performance Variation Observed in Construction Replicates 42 and 43

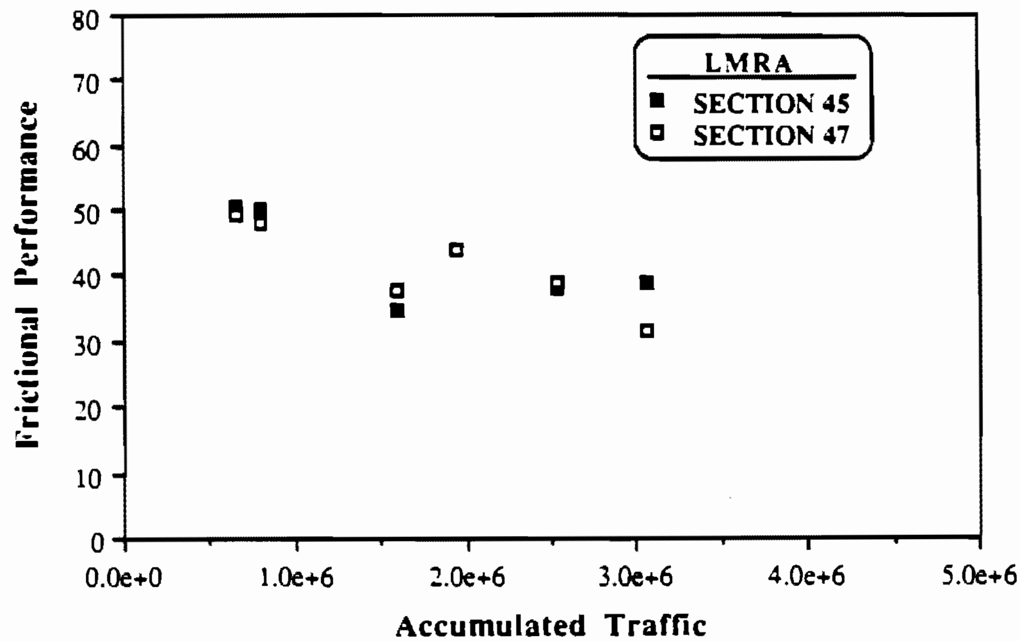


Fig. 7.20 Frictional Performance Variation Observed in Construction Replicates 45 and 47

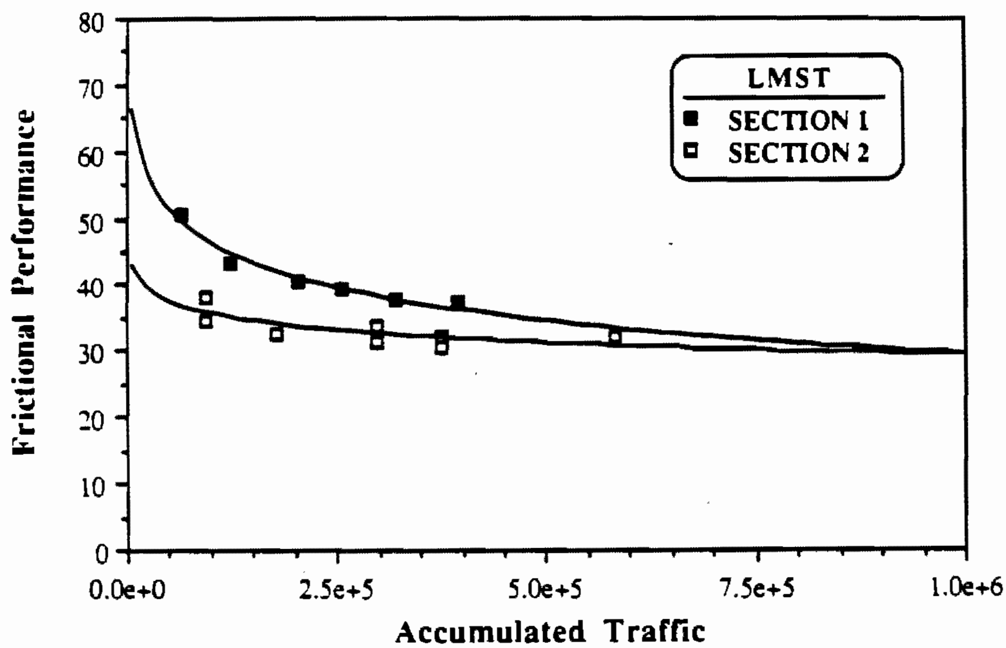


Fig. 7.21 Frictional Performance Variation Observed in Construction Replicates 1 and 2

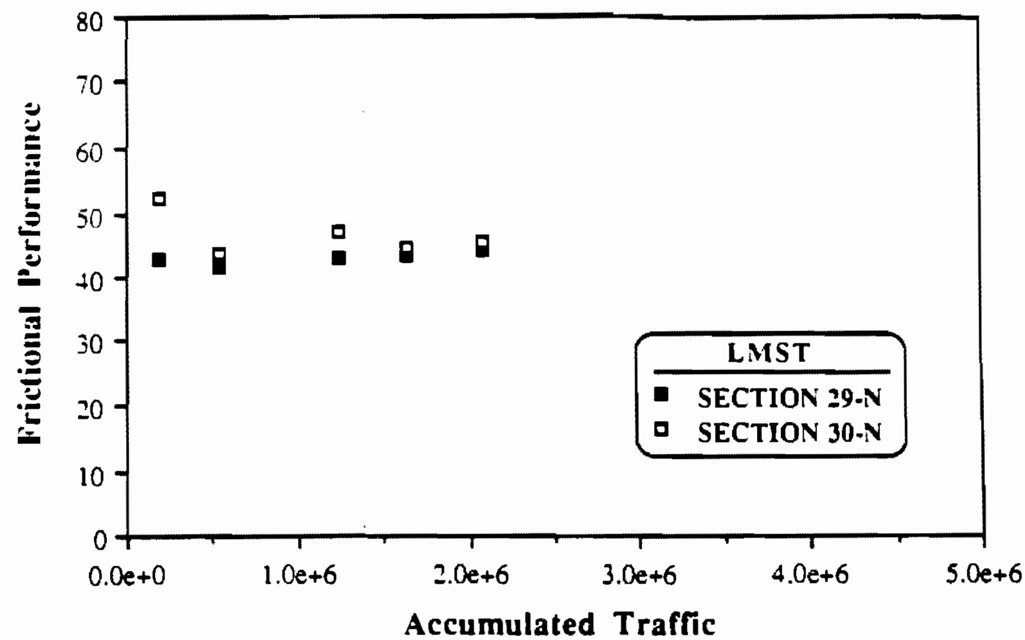


Fig. 7.22 Frictional Performance Variation Observed in Construction Replicates 29-N and 30-N

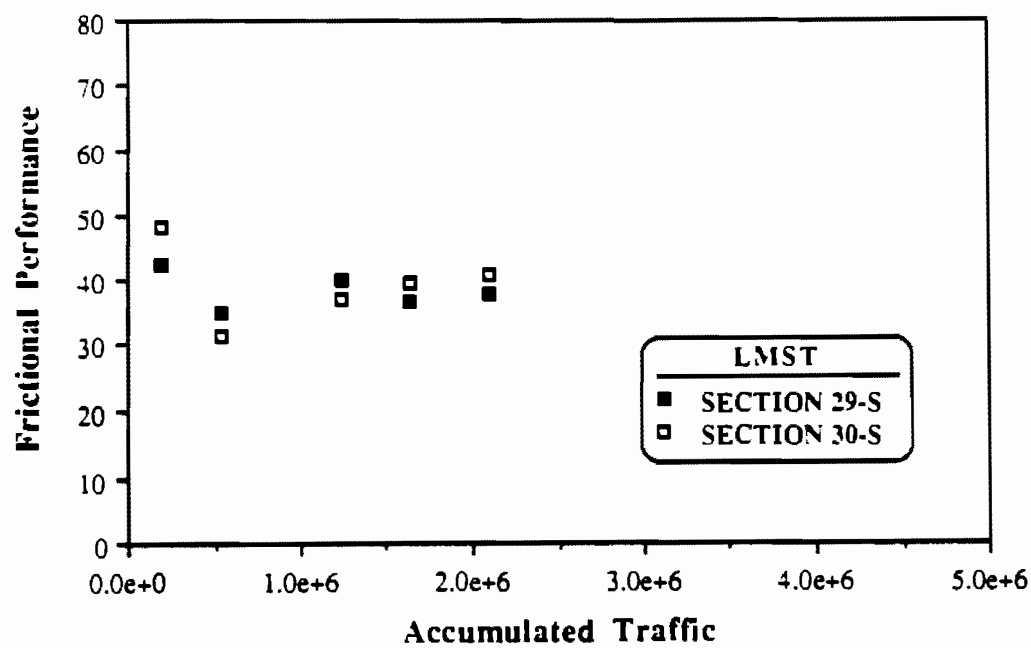


Fig. 7.23 Frictional Performance Variation Observed in Construction Replicates 29-S and 30-S

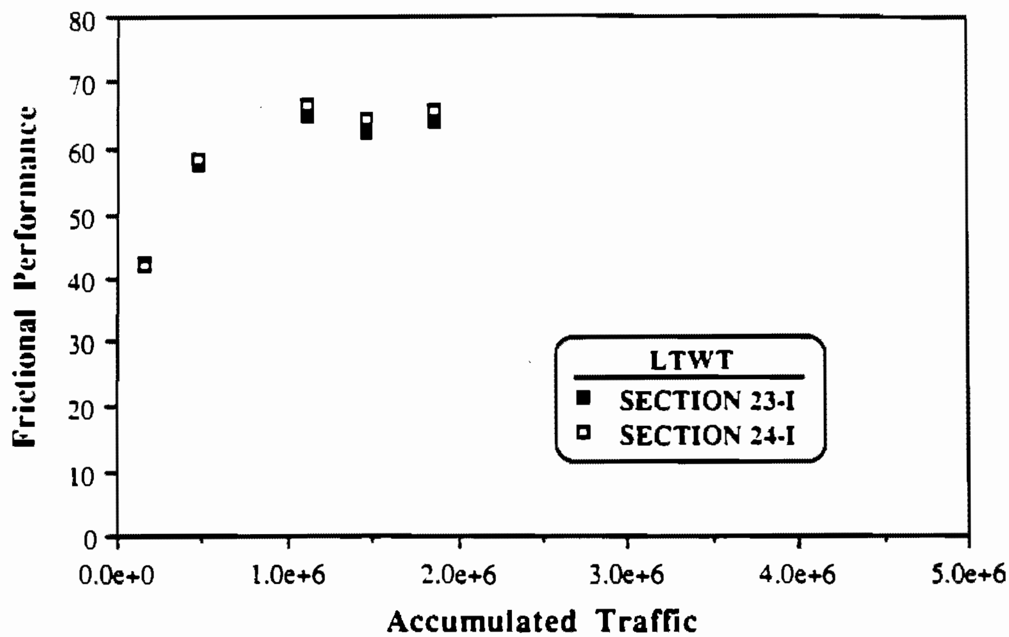


Fig. 7.24 Frictional Performance Variation Observed in Construction Replicates 23-I and 24-I

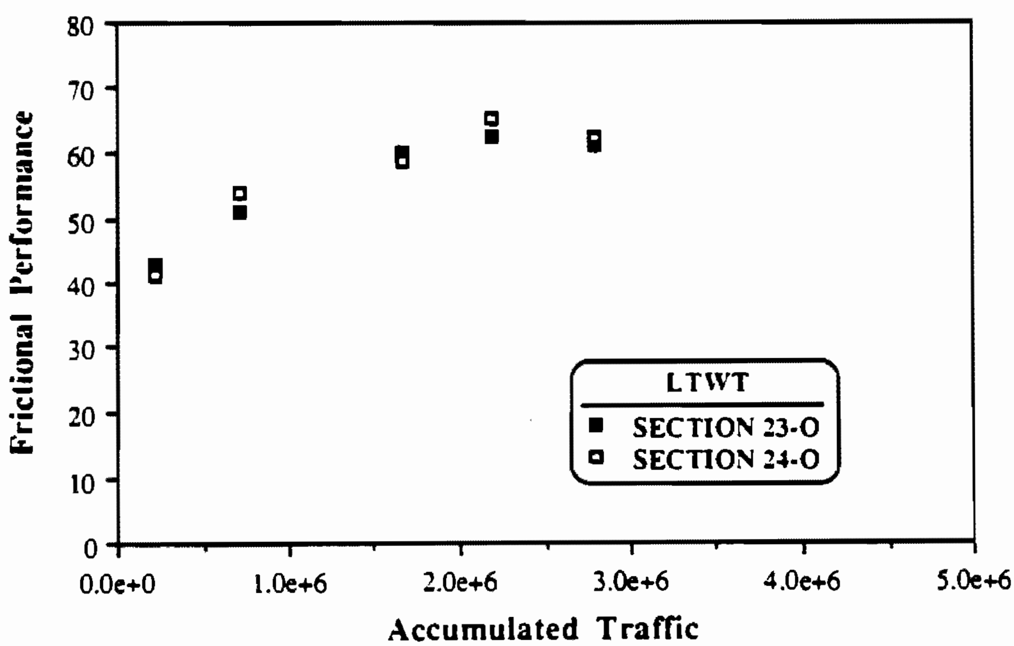


Fig. 7.25 Frictional Performance Variation Observed in Construction Replicates 23-O and 24-O

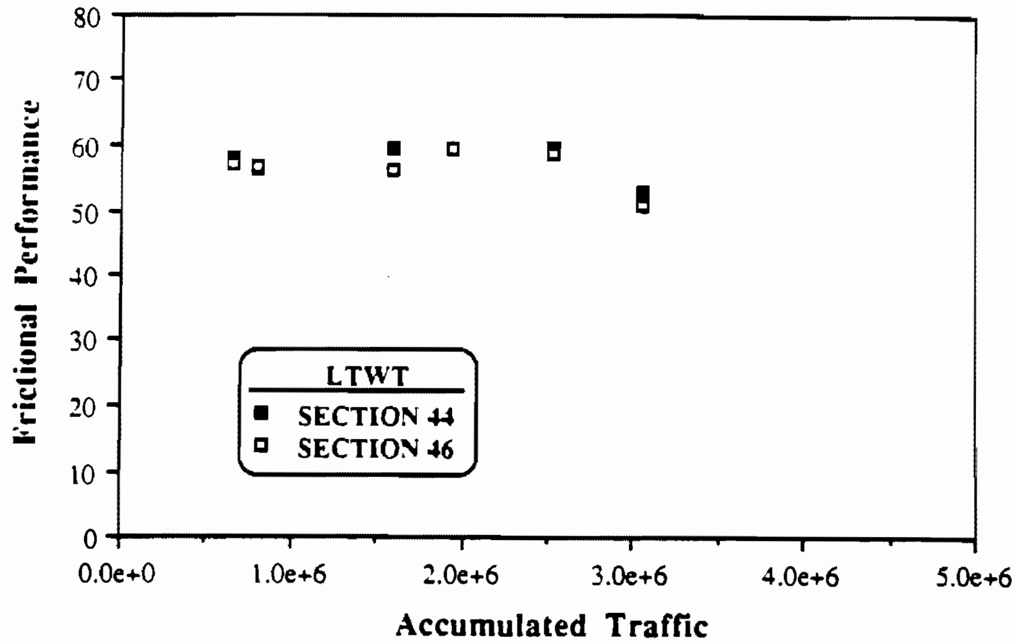


Fig. 7.26 Frictional Performance Variation Observed in Construction Replicates 44 and 46

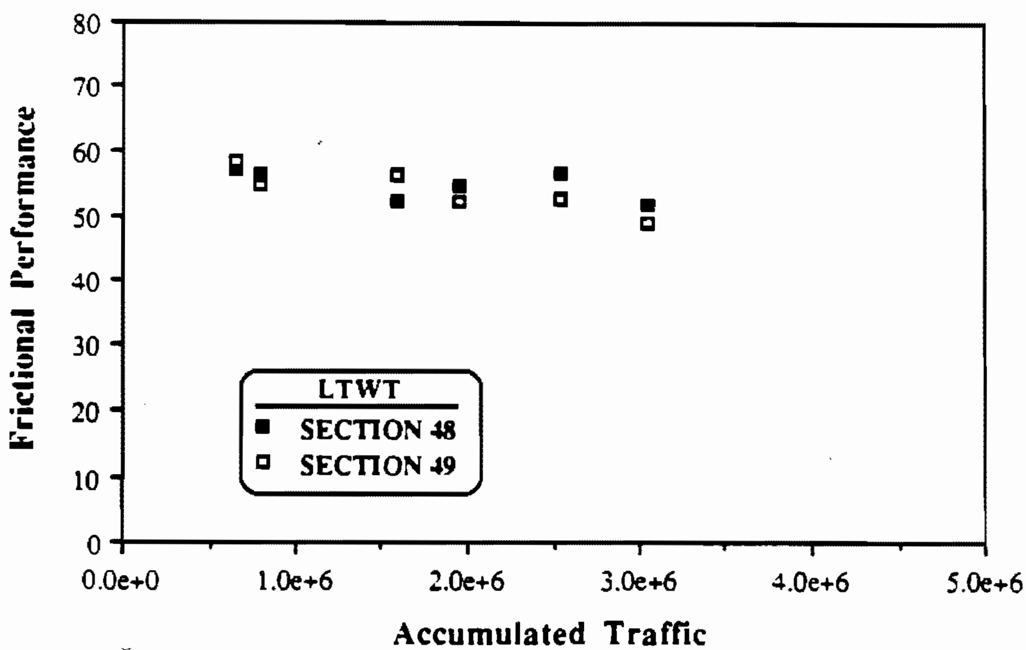


Fig. 7.27 Frictional Performance Variation Observed in Construction Replicates 48 and 49

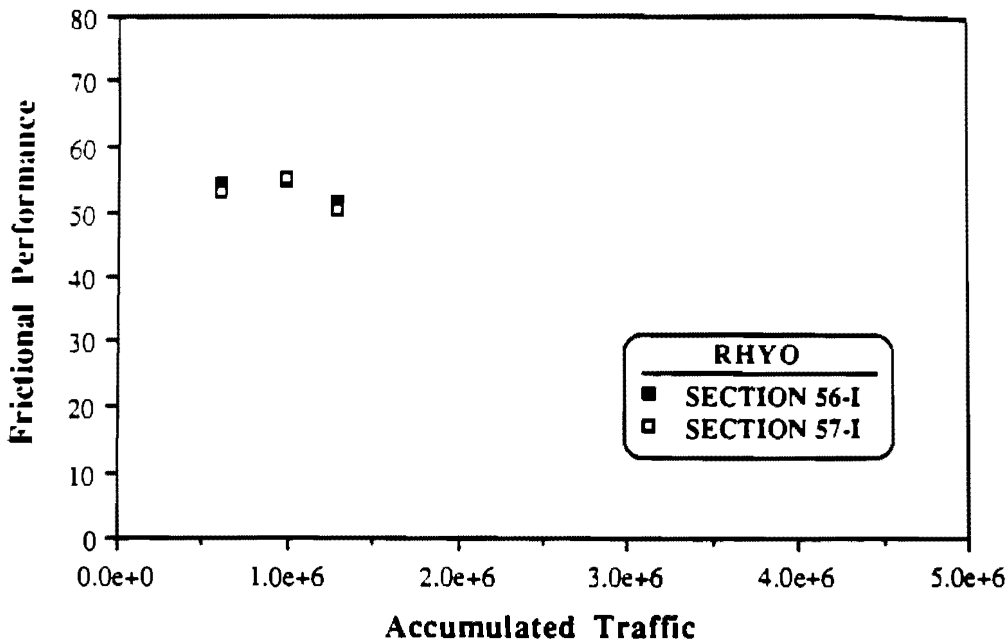


Fig. 7.28 Frictional Performance Variation Observed in Construction Replicates 56-I and 57-I

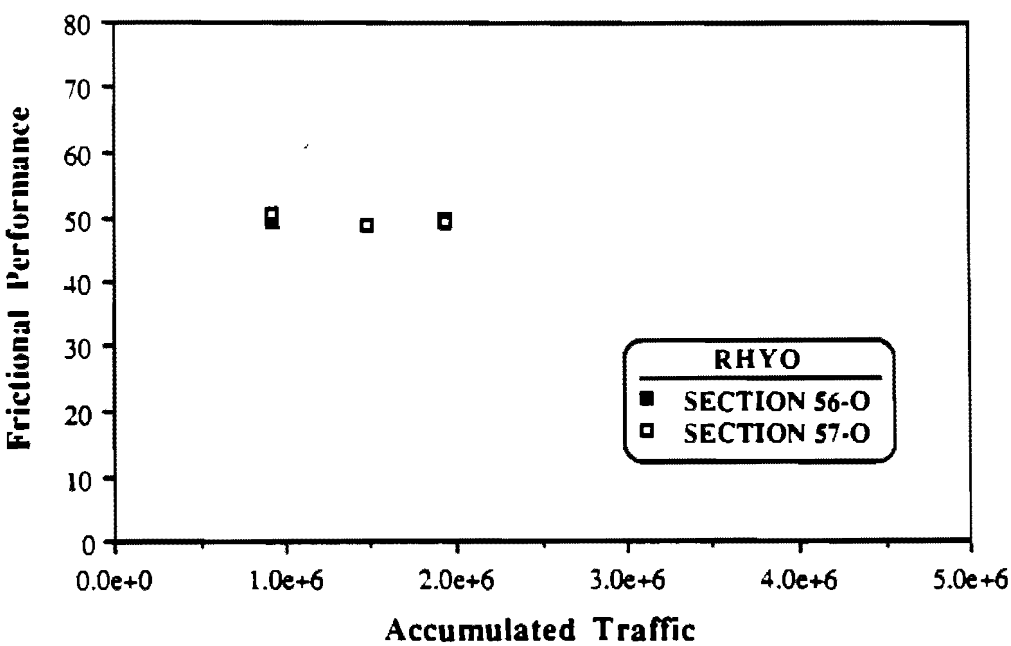


Fig. 7.29 Frictional Performance Variation Observed in Construction Replicates 56-O and 57-O

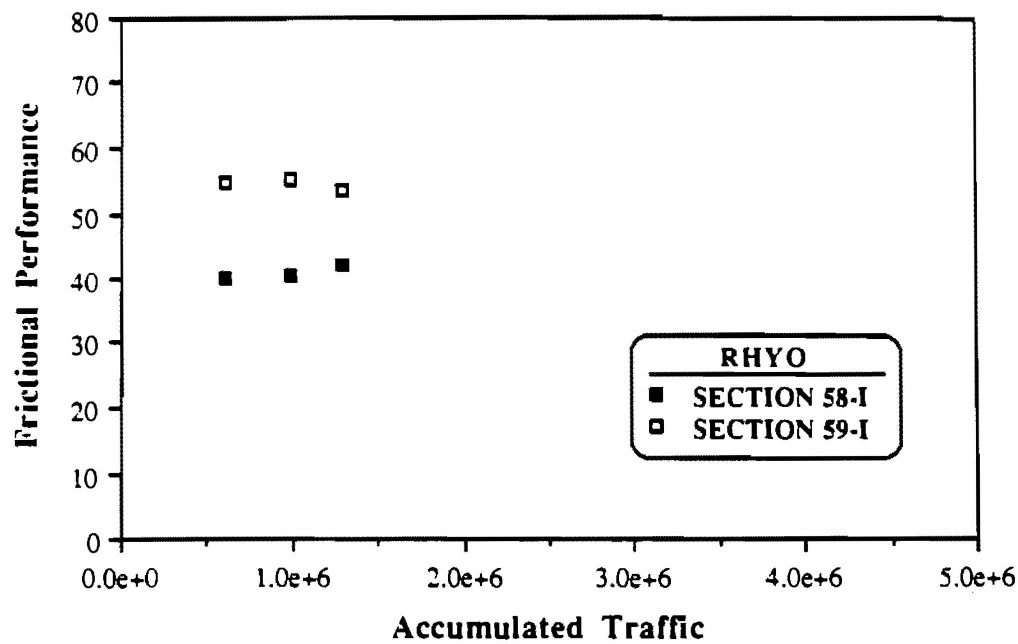


Fig. 7.30 Frictional Performance Variation Observed in Construction Replicates 58-I and 59-I

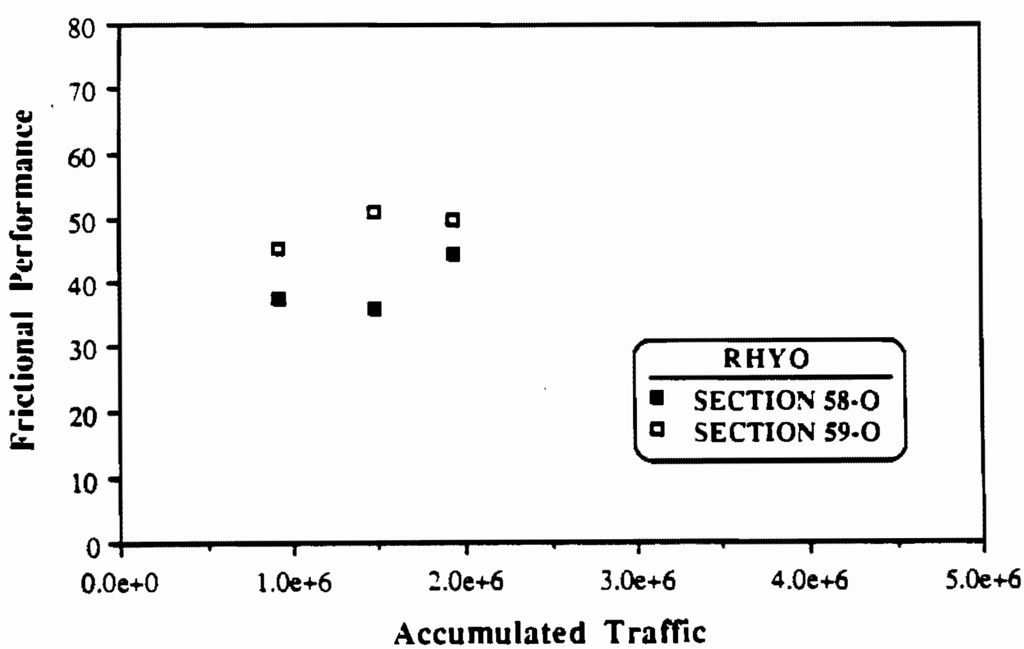


Fig. 7.31 Frictional Performance Variation Observed in Construction Replicates 58-O and 59-O

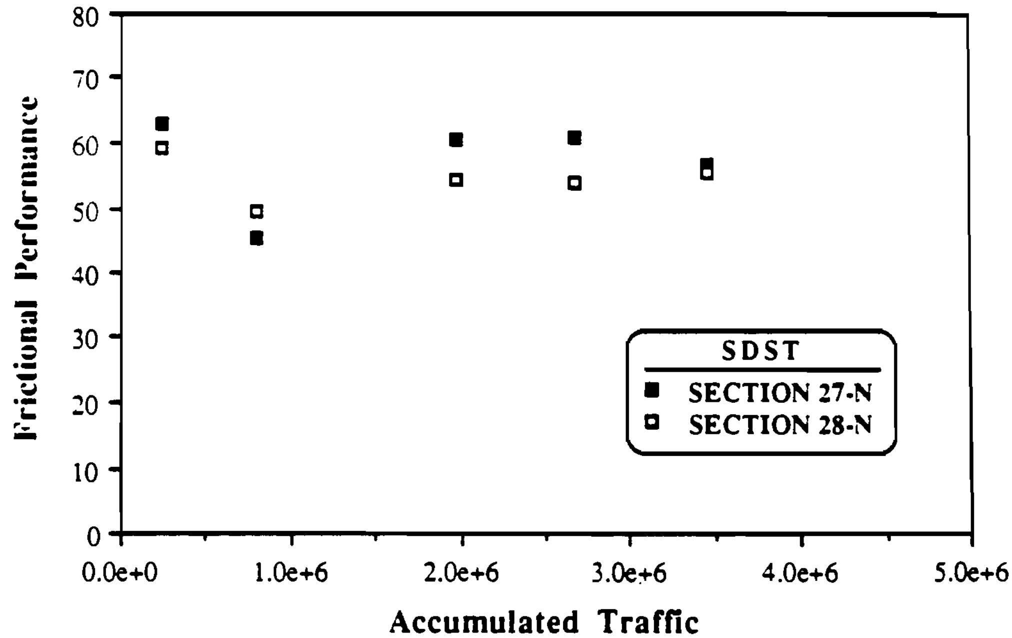


Fig. 7.32 Frictional Performance Variation Observed in Construction Replicates 27-N and 28-N

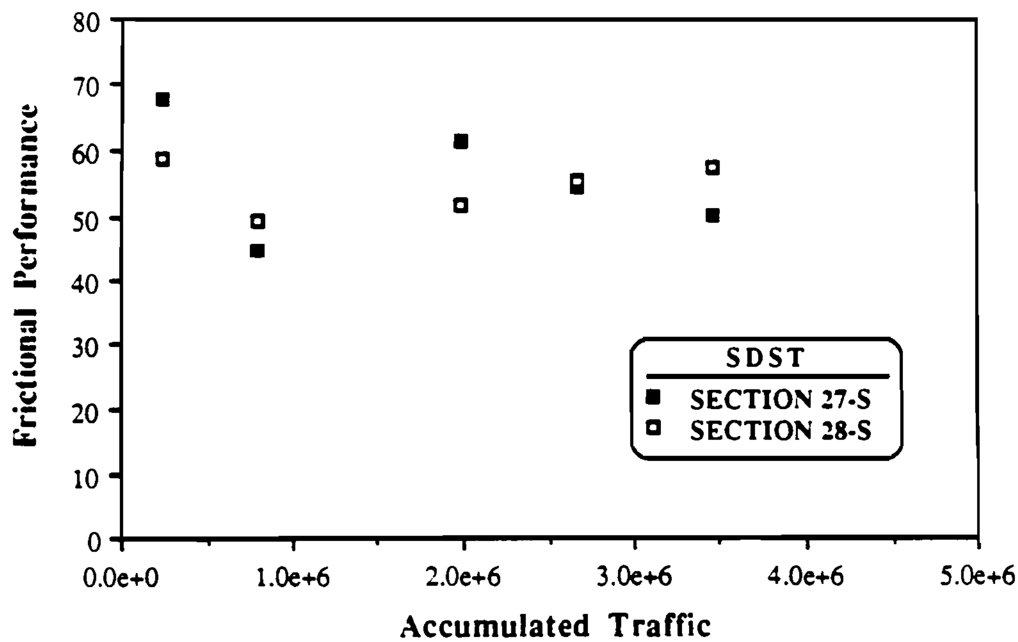


Fig. 7.33 Frictional Performance Variation Observed in Construction Replicates 27-S and 28-S

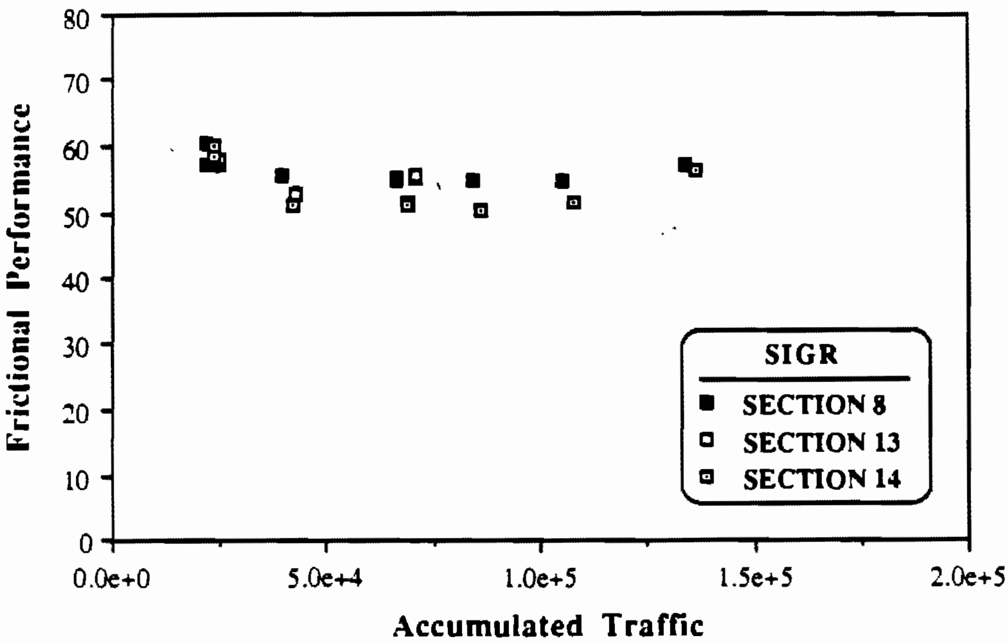


Fig. 7.34 Frictional Performance Variation Observed in Construction Replicates 8, 13, and 14

First, in Fig. 7.19, the last three FN readings obtained for section 42 were lower than those of section 43. The condition survey results revealed that the extent of bleeding experienced by section 42 was higher than that of section 43. Since both sections have been under the exposure of the same traffic, it is believed that possible changes in the construction application rate of asphalt might have been attributed to this variation. It could also be that uncompacted patched areas in the old surface, if there were any, might have made it possible for the aggregate particles in section 42 to sink deeper in the asphalt layer that a greater extent of asphalt bleeding resulted. In any case the source of variation is considered random.

Another sizable variation was observed in construction replicates 1 and 2, shown in Fig. 7.21, both constructed with the same low polish resistant LMST aggregate but on two different low traffic highways located in the same area. The ADT passing on section 2 was found to be about one and a half times that of section 1, which indicates that the observed diminishing variation could have been the results of lower macrotexture and microtexture experienced in section 2, particularly in the early stage of performance. The initial sand patch reading was 0.098 in. for section 1 compared to 0.082 in. for section 2. In addition, the obtained BPNs were, on the average, about 10 numbers lower in the case of section 2. Therefore, the variation in these two sections cannot be considered construction-related.

Lastly, a large variation was seen in the performance of the RHYO source used in sections 58 and 59. As shown in Figs. 7.30 and 7.31, that was due to section 58 exhibiting lower level of performance in both inner and outer lanes. This poor performance was viewed as untypical of RHYO aggregates, particularly that the section was evaluated to have experienced only a slight bleeding problem. Although the source of variation could not identified, the variation was too large to be considered a random construction variation, and its inclusion in the analysis was thought to be unreasonable.

7.2.3 Pit Replications

Pit replicates were constructed to study the variation that may be caused by changes in the quality of an aggregate source. Figures 7.35 through 7.38 illustrate four examples of this type of replication. In the first example, shown in Fig. 7.35, section 50, starting with a higher level of performance, was found to have experienced a drastic

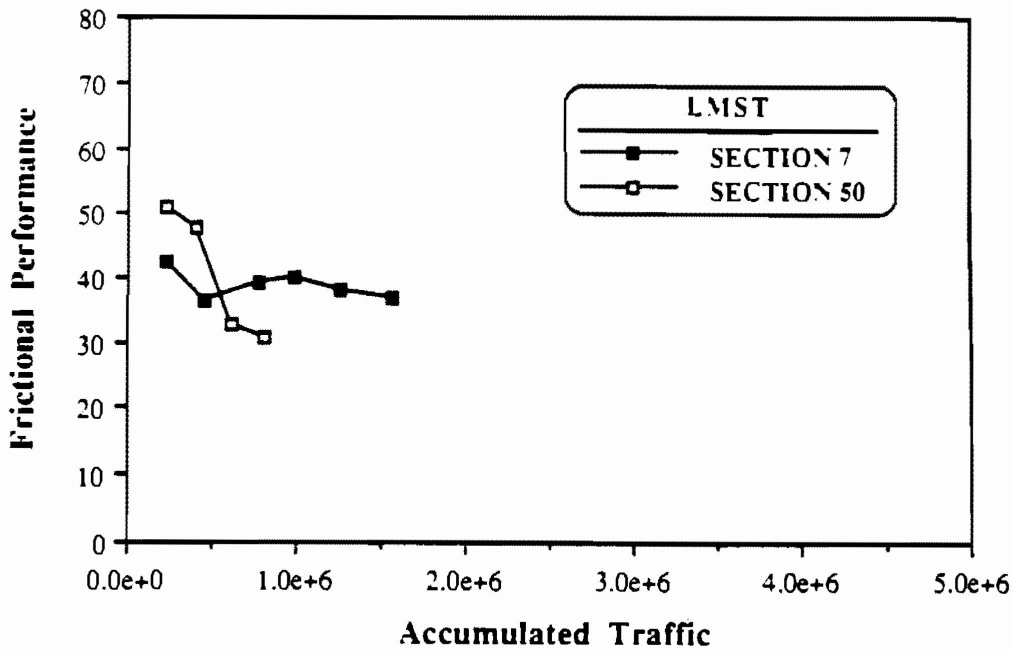


Fig. 7.35 Frictional Performance Variation Observed in Pit Replicates 7 and 50

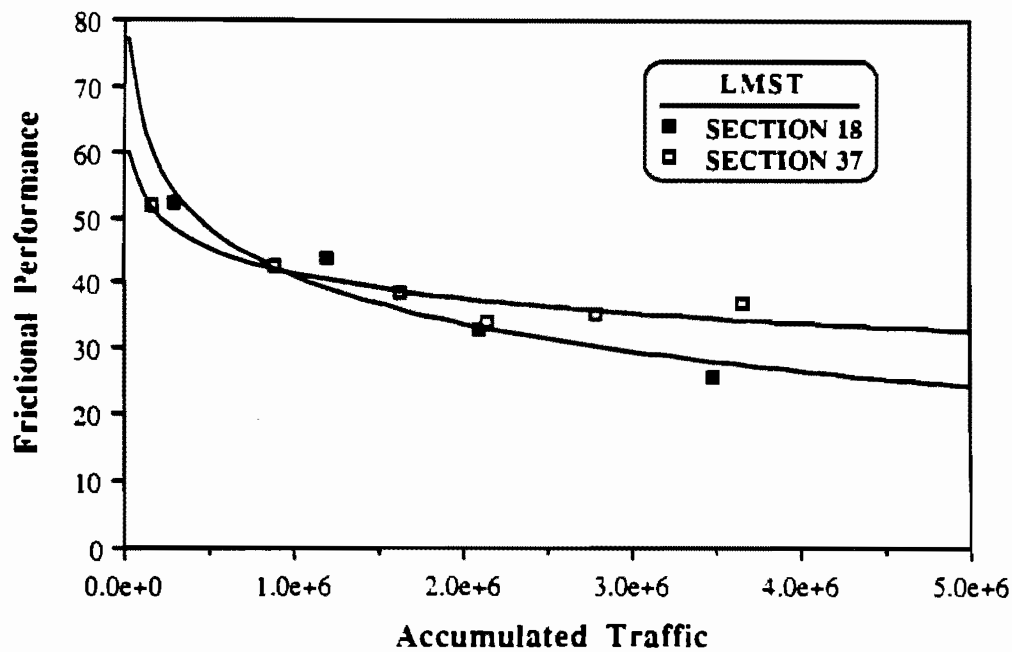


Fig. 7.36 Frictional Performance Variation Observed in Pit Replicates 18 and 37

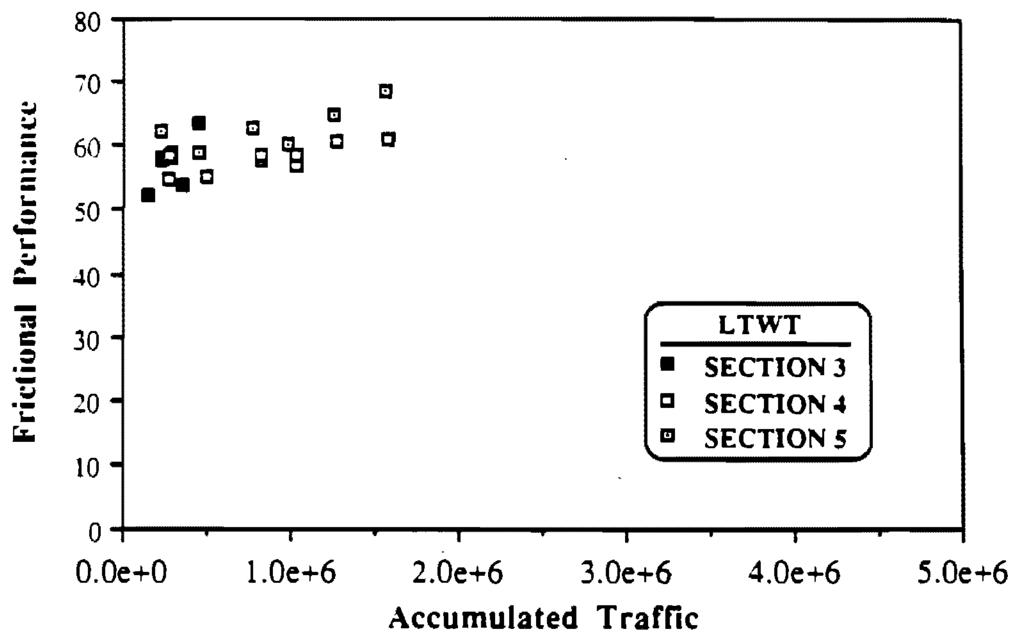


Fig. 7.37 Frictional Performance Variation Observed in Pit Replicates 3, 4, and 5

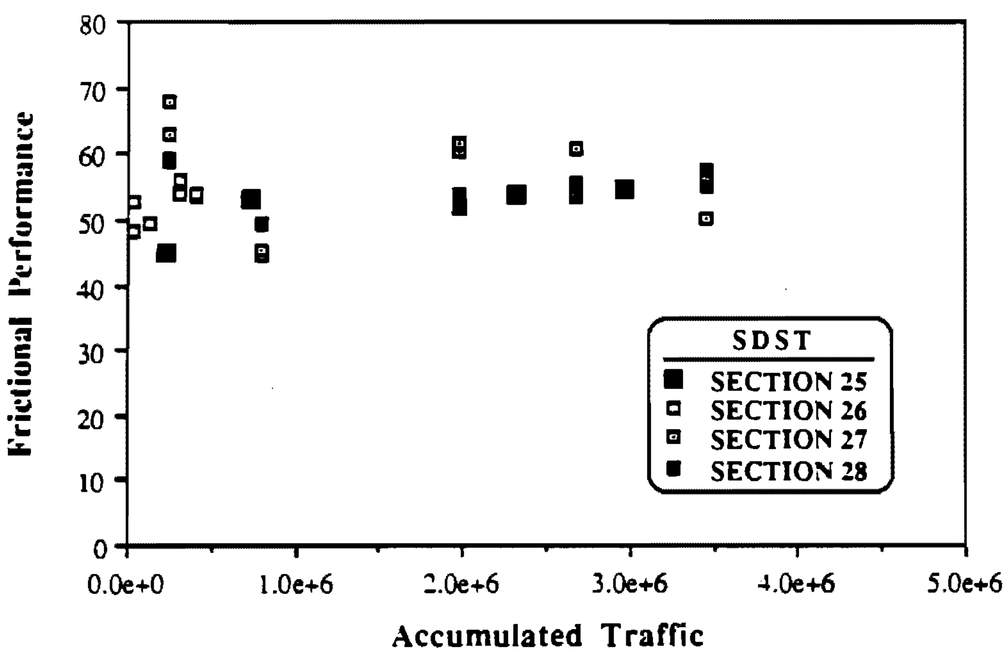


Fig. 7.38 Frictional Performance Variation Observed in Pit Replicates 25, 26, 27, and 28

decrease in the FN under less than one million of accumulated traffic passes. The results of the condition survey indicated that by the time the third friction measurement was taken, the section had suffered from a severe bleeding distress. Both sections 7 and 50 had high PVs and high MSS losses.

In the second example, the LMST source in Fig. 7.36 did not change in quality, but the two sections had different construction aspects and were placed in different environmental regions. The effects of these two factors will be discussed in detail in a later section of this chapter.

Fig. 7.37 shows a third example of pit replicates where the same LTWT source was used in three different sections. The variation seen between the performances of sections 4 and 5 is believed to have been caused by a change in the quality of this source. The PV was 45 for section 5 as compared to 42 for section 4.

The last example is on a SDST source, placed in the same environmental region, whose quality did not change with time. Much of the variation seen in Fig. 7.38 is believed to have been caused by the construction and lane replications constructed with this source. The increase in frictional resistance under low traffic exposure noticed in section 26 is an interesting feature of this SDST.

Two more examples, one on a LMRA source and the other on another LTWT source, could have been discussed in this section. However, because of the numerous factors involved in explaining the observed variations, it was decided to include them in a later section of chapter.

7.3 Levels of Performance Experienced by the Different Aggregate Materials

In this section, a grouping of the entire friction data was done according to the different aggregate materials used in the study. The presentation of the frictional performance exhibited by each of the groups was not intended for purposes of making comparisons, based on the laboratory results, among the performances of the sources that constituted each group. The objective was rather to visualize the levels of performance achieved by the various groups as well as the scatter of the friction data in each of them. This was thought to provide insights as to how the data could be later grouped so that the experimental designs could be studied.

The scatter of the entire friction data is shown in Fig. 7.39. As can be seen, the accumulated traffic has not gone beyond four million passes in all sections, with only one exception. That was section 21 which was placed on an urban highway with a very high volume of traffic. The section started to show severe bleeding in the surface soon after it had been constructed. Therefore, the poor frictional performance exhibited in this section, shown in Fig. 7.40, was considered not to represent the frictional properties of the LMRA aggregate used in its surface and was thus excluded from the analysis.

A better visualization of the scatter in the data can be seen in Fig. 7.41, with section 21 being excluded. The FNs were found to have ranged from a lowest of about 25 to a highest of about 70. The plot in Fig. 7.42 explains this variability in the FN in terms of the different AGMT groups used. Then, plots showing the overall performance of each AGMT group and the performance of the individual aggregates comprising each group were made; they are presented in Figs. 7.43 through 7.54. The detailed graphs were included so that reference to the performance of individual test sections in each group can later be made as needed.

The performance of each AGMT group was judged upon based on the rate of decrease in FN exhibited and the number of accumulated traffic passes withstood by the material before the FNs intercepted with the zone of minimum friction. This zone was assumed to be confining FNs in the range of 30 to 40. The lower boundary, a FN of 30, was that thought by highway engineers to represent the level of friction below which a corrective measure should be considered. The upper boundary was chosen based on a study involving wet-pavement accidents (11), in which a significant decrease in the accident rate was revealed when pavement FNs were greater than 44.

A wide scatter was observed in the data of the LMRA group, as shown in Fig. 7.43. The rate of decrease in the FN from about 60 to about 30 was seen to be moderate. The FNs of some sections intercepted with the zone of minimum friction at about one and a half million accumulated traffic passes while some sections withstood about three and a half million accumulated traffic passes before reaching that zone.

In the LMST group, presented in Fig. 7.45, the rate of decrease in the FN was seen to be higher than that of the LMRA group with almost all of the sections intercepting with the zone of minimum friction at accumulated traffic passes of less than

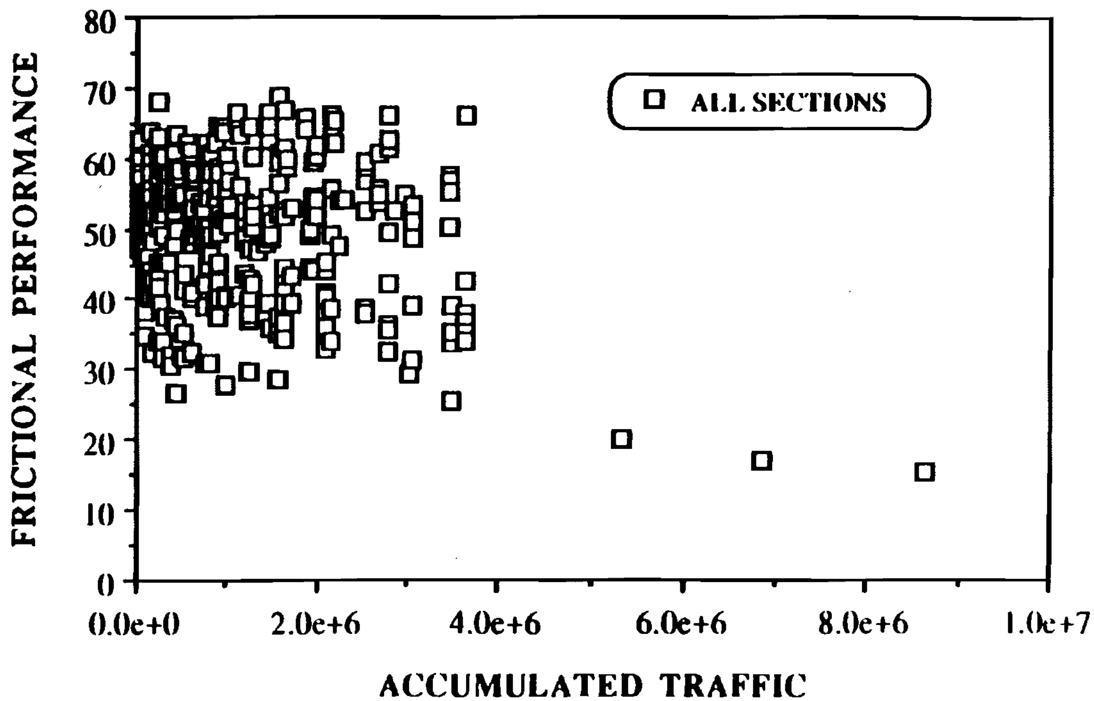


Fig. 7.39 Frictional Performance of All Established Test Sections

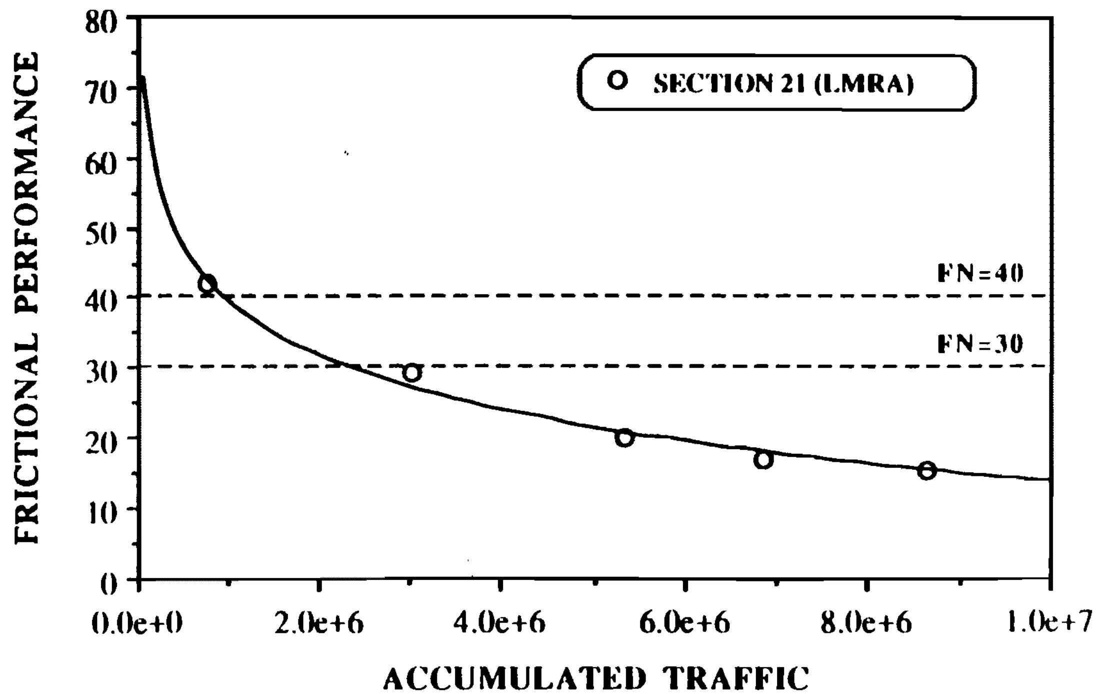


Fig. 7.40 Frictional Performance of Section 21 Constructed with a LMRA Aggregate

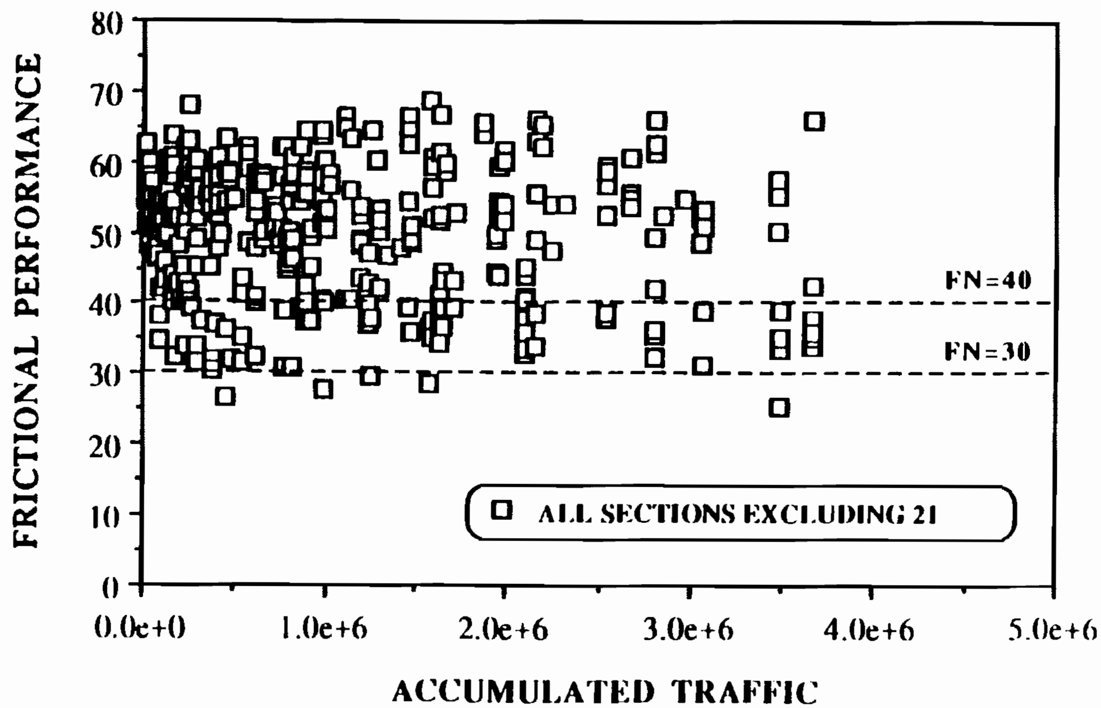


Fig. 7.41 Frictional Performance of All Test Sections Excluding Section 21

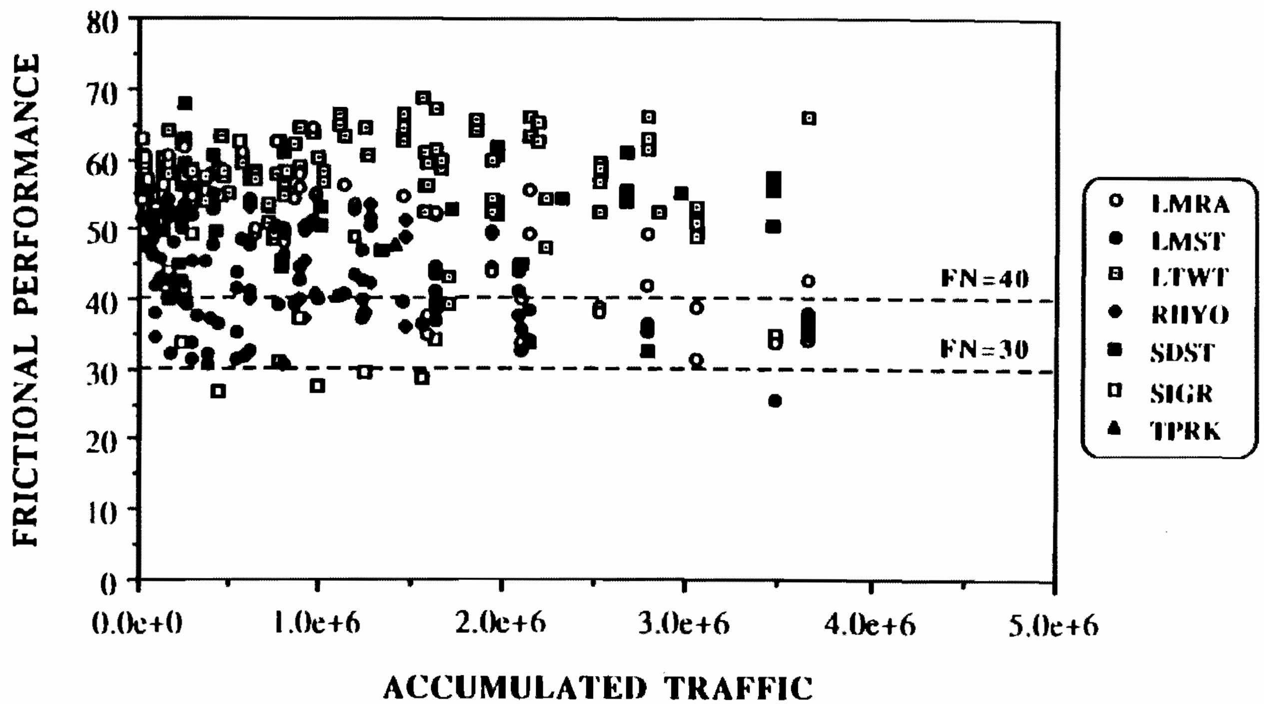


Fig. 7.42 Frictional Performance of All Test Sections--Grouped by Aggregate Material

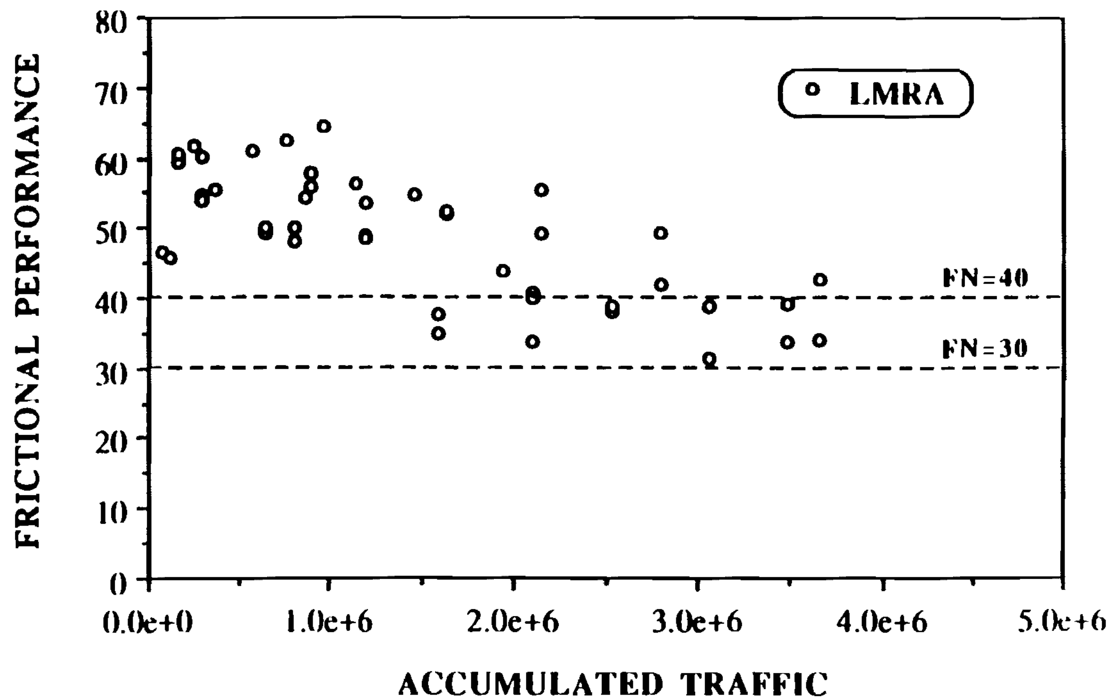


Fig. 7.43 Frictional Performance of Test Sections Constructed with LMRA Aggregates (Excluding Section 21)

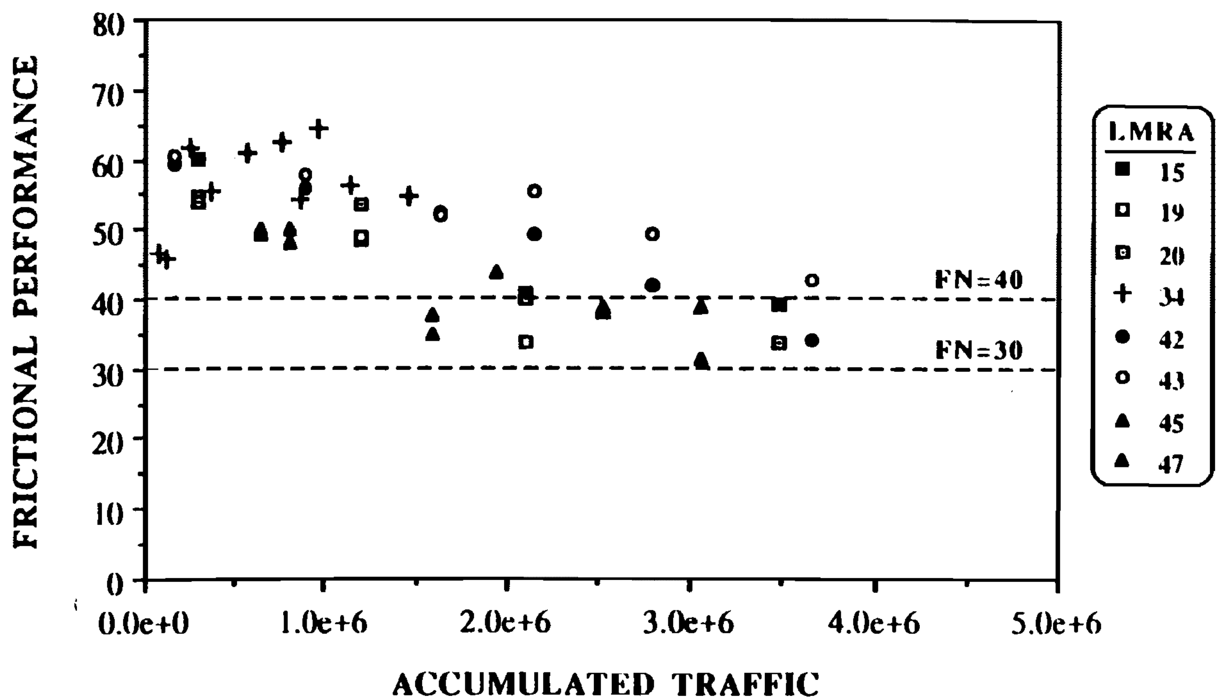


Fig. 7.44 Frictional Performances of the Different Test Sections Constructed with LMRA Aggregates

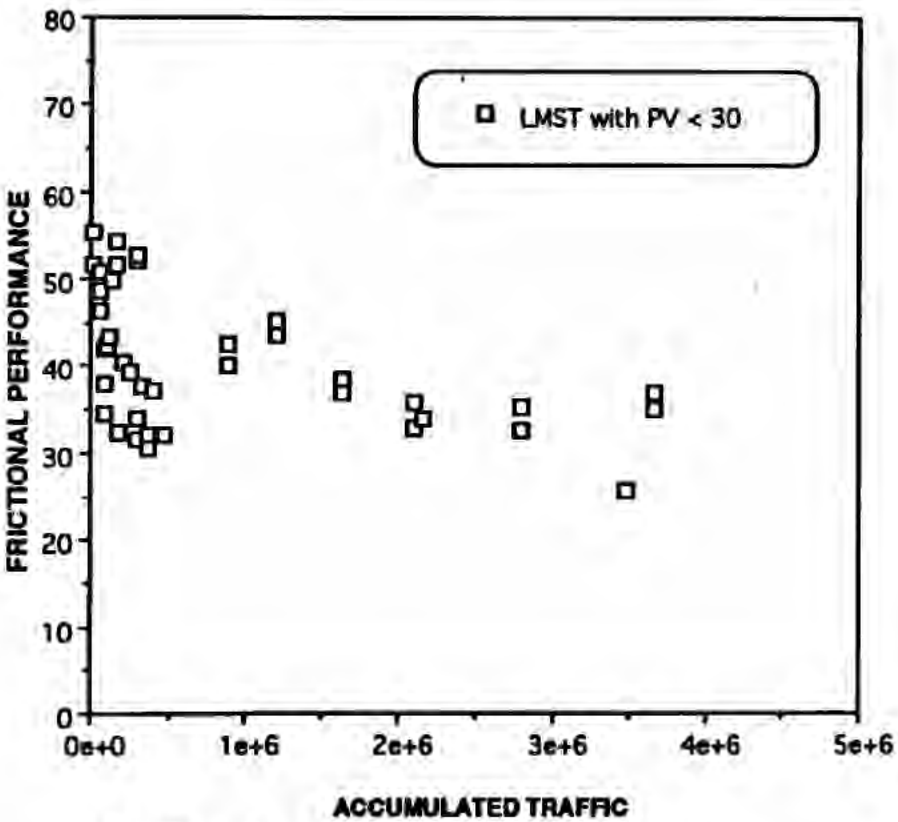


Fig 7.45A **Frictional Performance of Test Sections Constructed with LMST Aggregates for Polish Value less that 30**

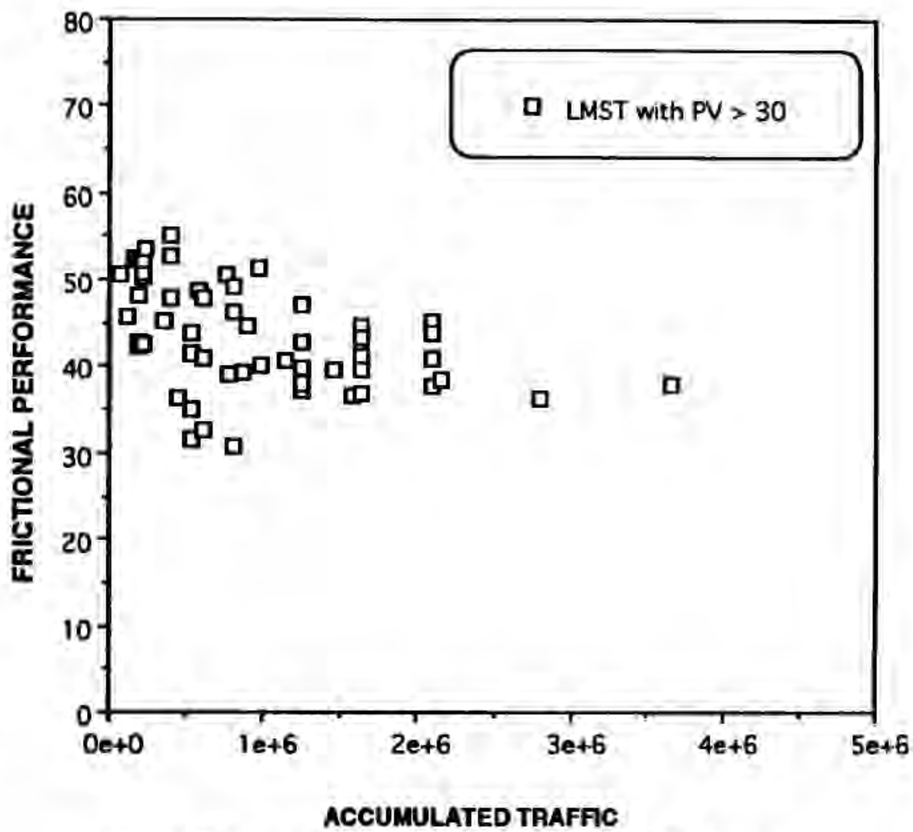


Fig 7.45B **Frictional Performance of Test Sections Constructed with LMST Aggregates for Polish Value more that 30**

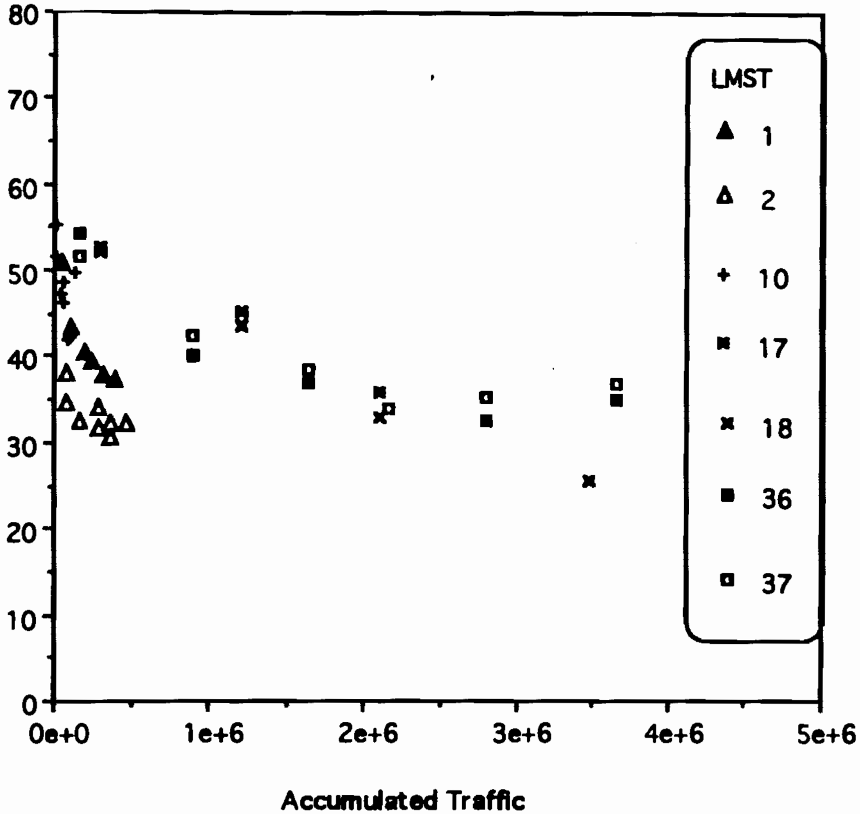


Fig 7.46A Frictional Performance of the Different Test Sections Constructed with LMST Aggregates for Polish Value less than 30

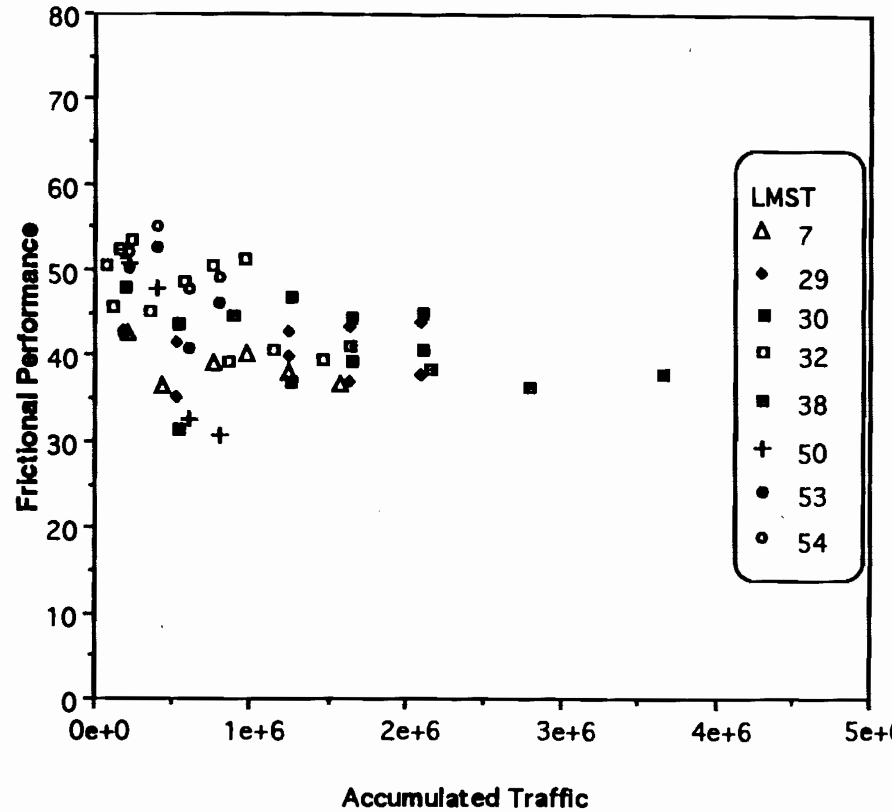


Fig 7.46B Frictional Performance of the Different Test Sections Constructed with LMST Aggregates for Polish Value more than 30

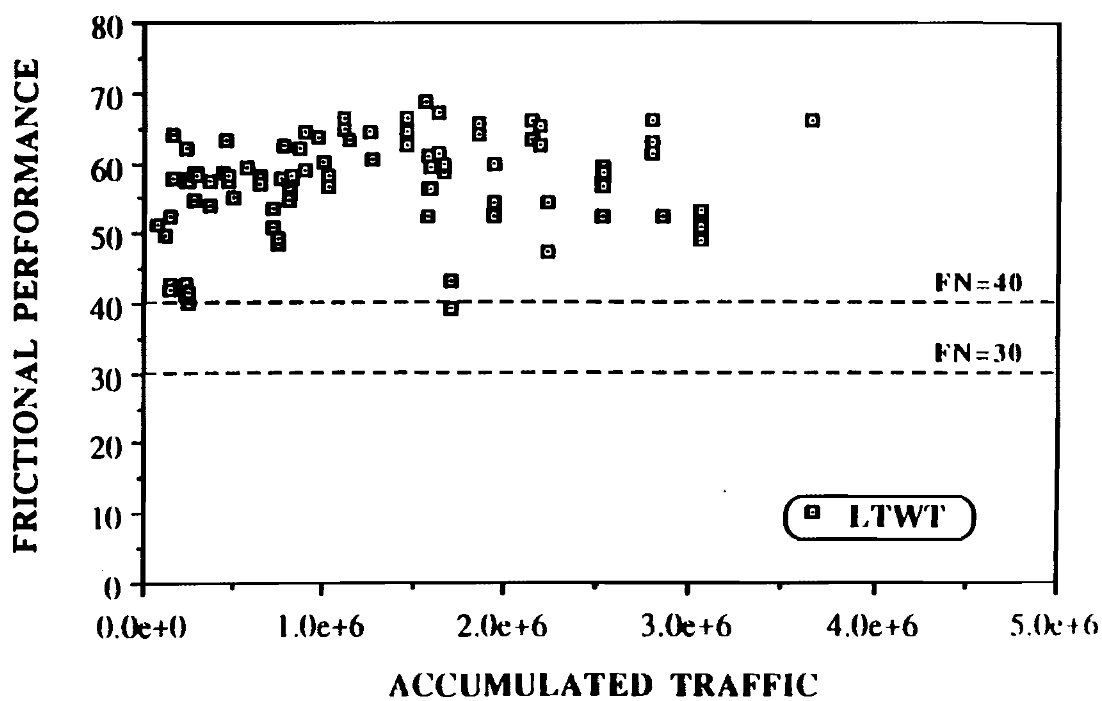


Fig. 7.47 Frictional Performance of Test Sections Constructed with LTWT Aggregates

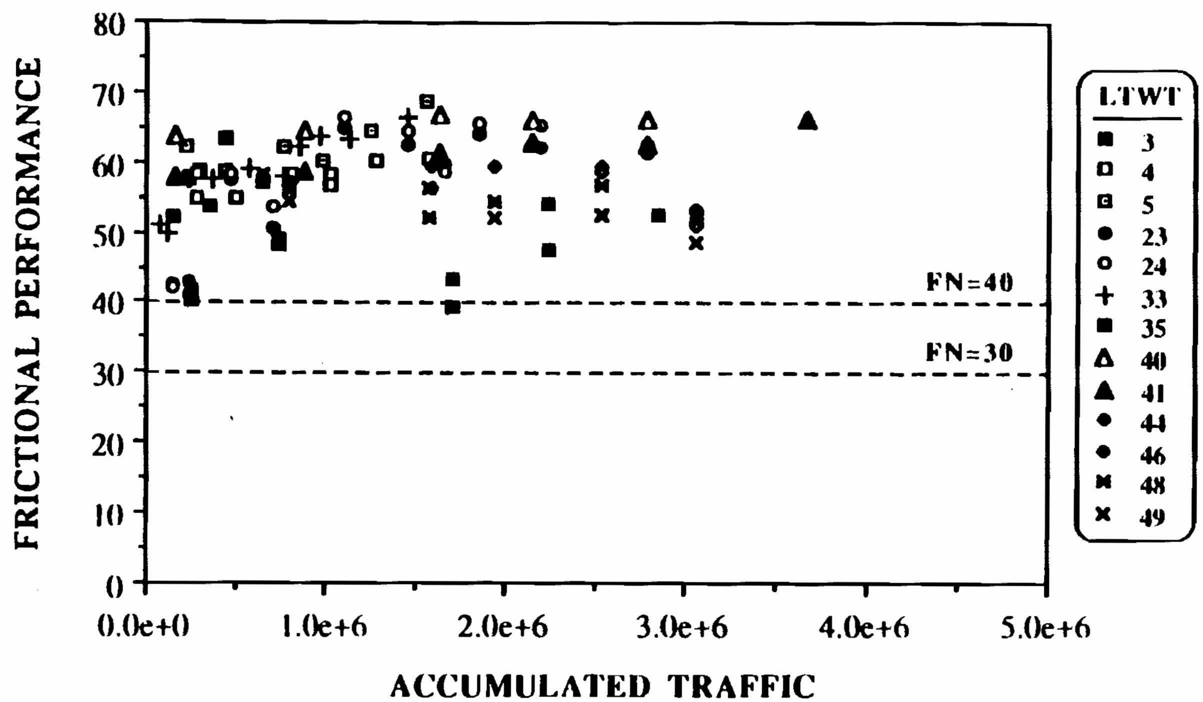


Fig. 7.48 Frictional Performances of the Different Test Sections Constructed with LTWT Aggregates

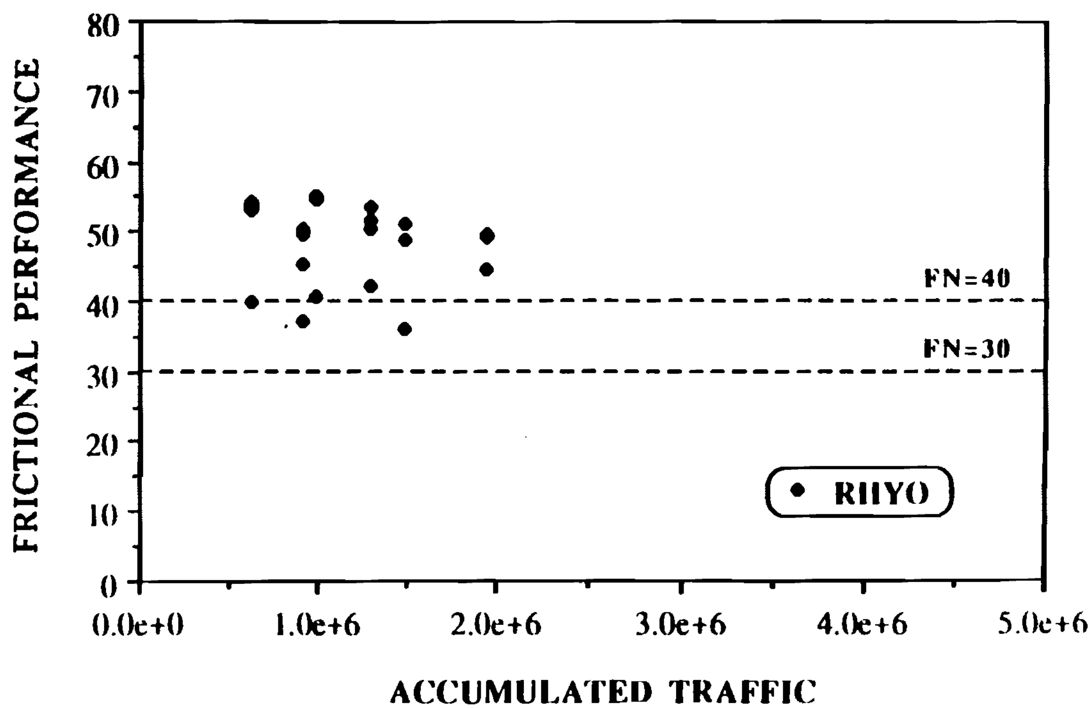


Fig. 7.49 Frictional Performance of Test Sections Constructed with RHYO Aggregates

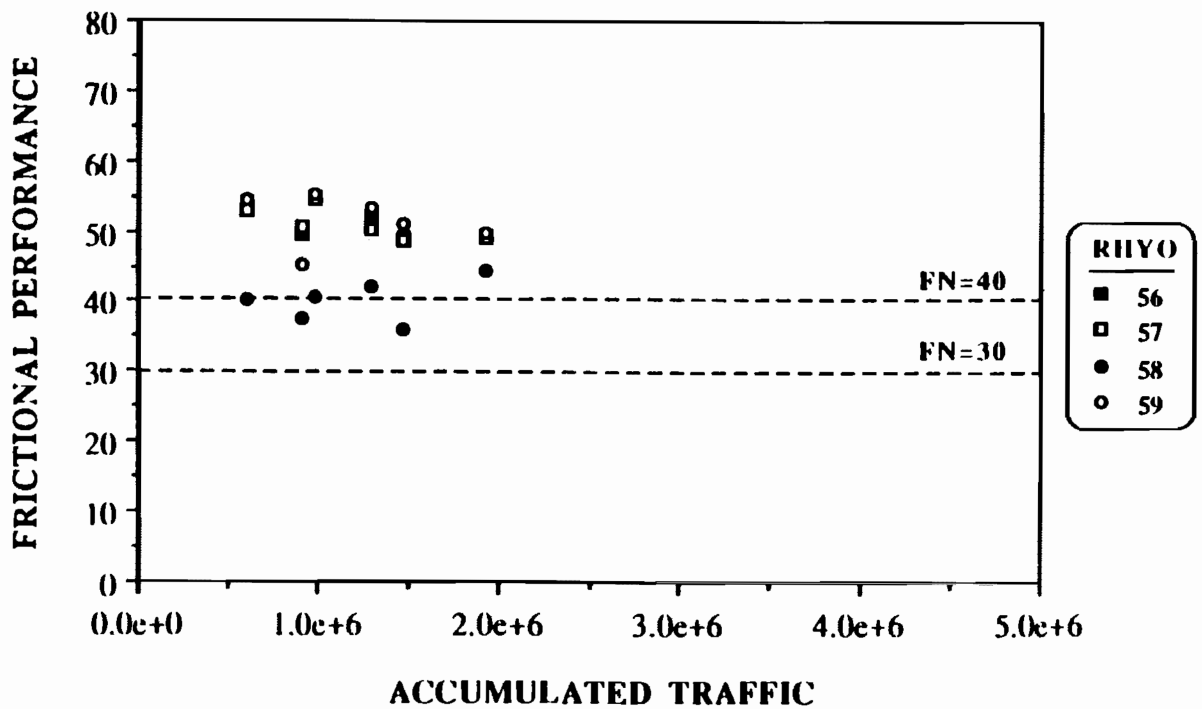


Fig. 7.50 Frictional Performances of the Different Test Sections Constructed with RHYO Aggregates

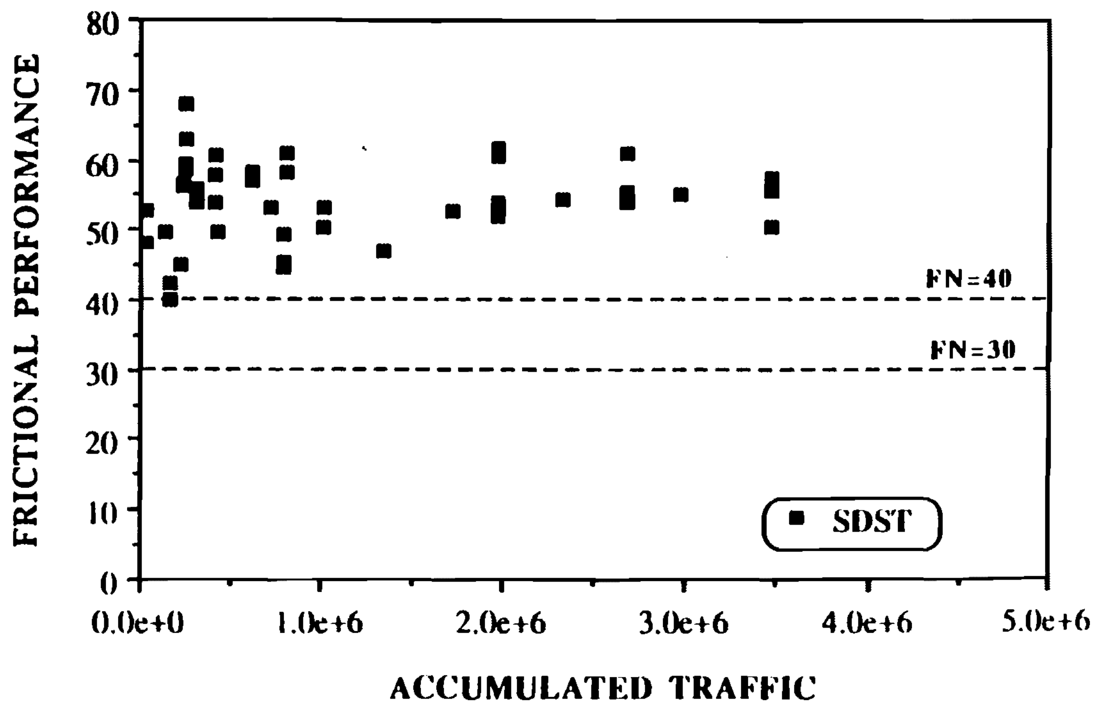


Fig. 7.51 Frictional Performance of Test Sections Constructed with SDST Aggregates

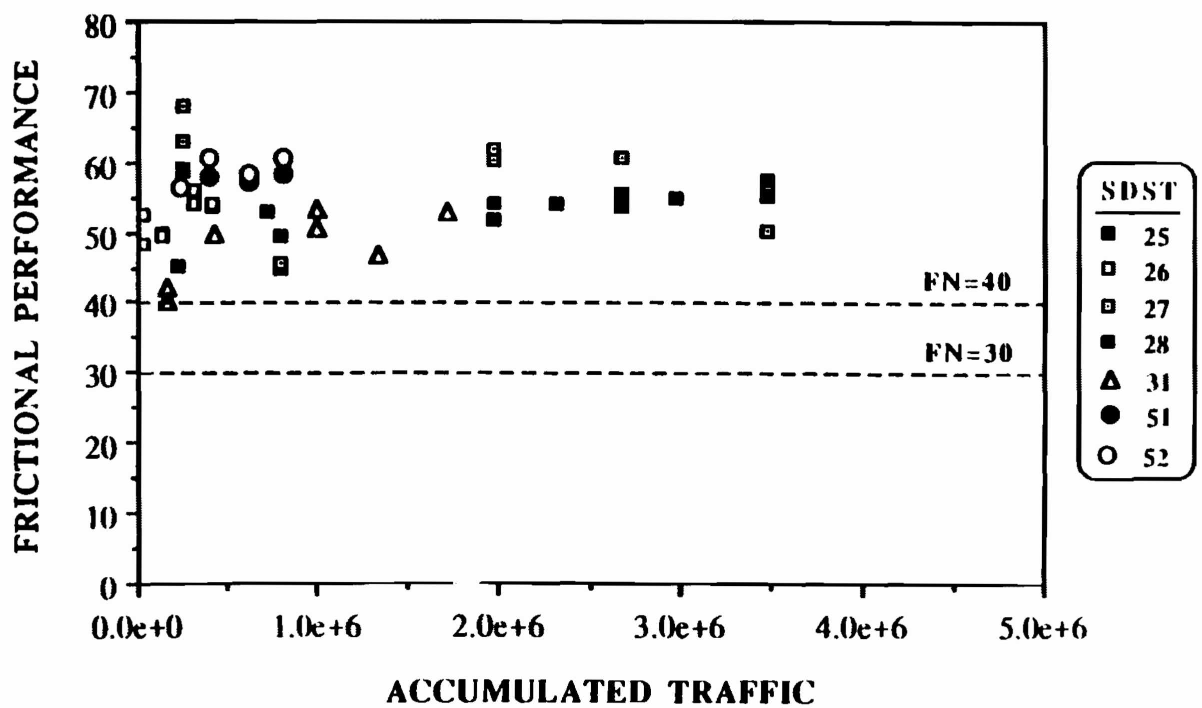


Fig. 7.52 Frictional Performances of the Different Test Sections Constructed with SDST Aggregates

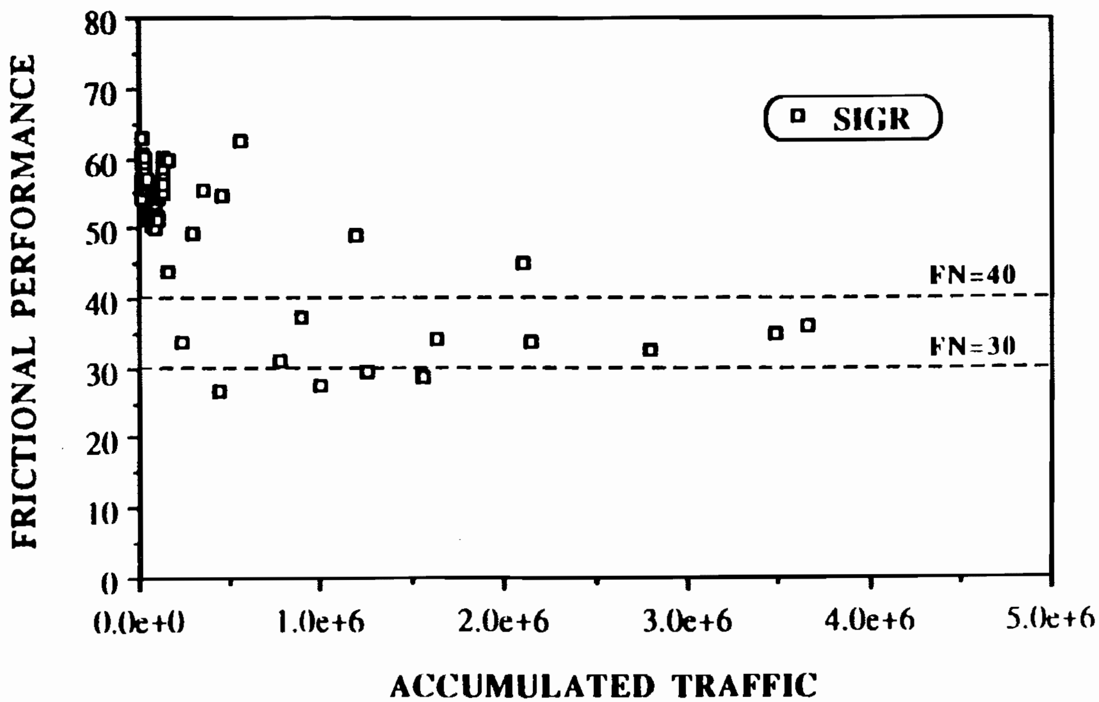


Fig. 7.53 Frictional Performance of Test Sections Constructed with SIGR Aggregates

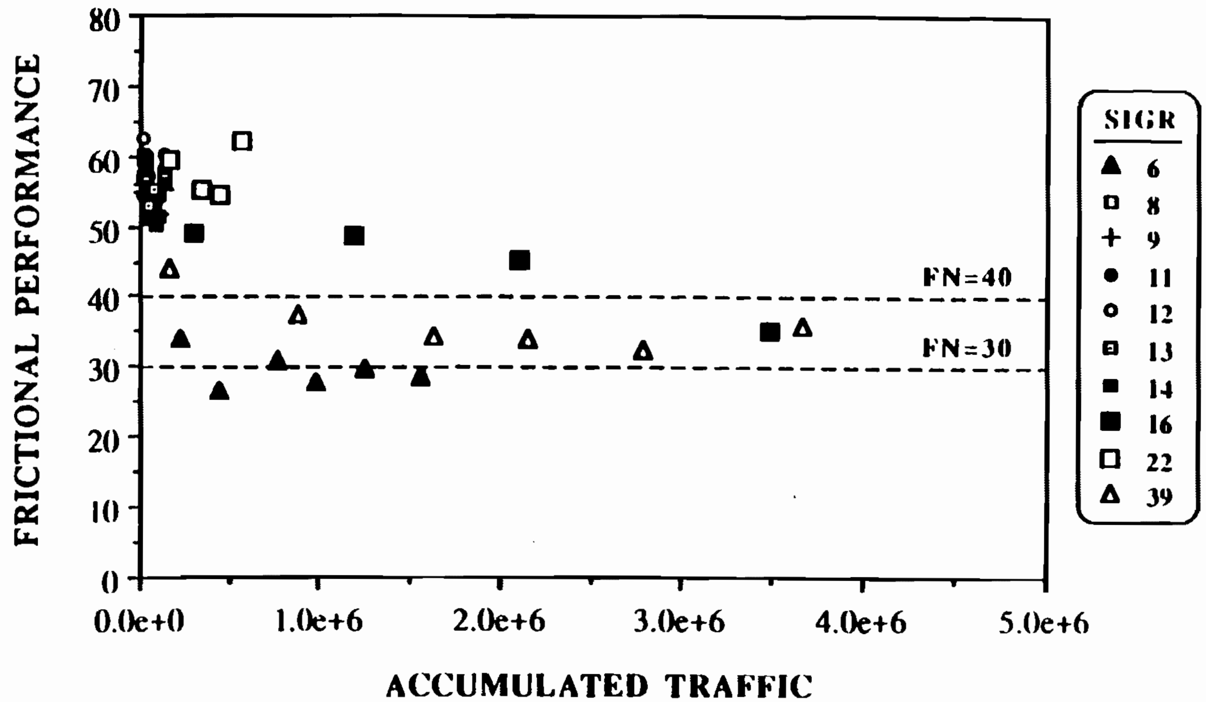


Fig. 7.54 Frictional Performances of the Different Test Sections Constructed with SIGR Aggregates

two million. The performances of some sections have almost gone through the zone at less than half a million passes.

As expected, the performance of the LTWT group, shown in Fig. 7.47, was found to be superior. However, the FNs were widely scattered, with the majority of the data falling in a FN range of 50 to 70.

The RHYO group, presented in Fig. 49, by excluding section 58 which was discussed in an earlier section, showed a slight decrease in frictional performance. The obtained FNs ranged from about 45 to about 55.

The SDST group was seen to have maintained excellent frictional performance. However, the FNs were widely scattered at the early stages of performance, as depicted in Fig. 7. 51, ranging from about 40 to about 70.

The friction data of the SIGR group, shown in Fig. 7.53, was noticed to have experienced the widest scatter, with FNs ranging from about 25 to about 65. Some low traffic sections maintained a high level of performance, while other sections had frictional performance levels that were either within or below the zone of minimum friction. Finally, one of the sections maintained FNs greater than 40 before intercepting with the zone of minimum friction at about three million accumulated traffic passes.

In conclusion, the grouping of the entire friction data according to the various aggregate materials was found to explain, in broad terms, much of the scatter observed in the data. Superior performance was observed in the LTWT and SDST groups, and a slight decrease in performance was seen in the RHYO group. The LMRA and LMST groups exhibited different rates of noticeable decrease in frictional performance. Lastly, different levels of performance were encountered in the SIGR group.

7.4 Isolated Effects of Some Variables Observed in Test Sections Constructed End-to-End

One of the criteria followed in selecting test sections was to have, when possible, as many sections constructed end-to-end as the number of the different aggregates used. It was thought that when sections could be constructed as such, the environmental, construction, and traffic variables would be held constant, and, therefore, the effects of aggregate properties could be isolated. Segments of highways with as many as seven aggregate sources were possible to construct in seven of the eleven districts involved in the study. In the following discussions, the performances of the aggregates used in each segment are examined, and the effects of aggregate properties are evaluated.

7.4.1 Highway Segment in District 8

In this district, sections 5, 6, and 7 were constructed end-to-end on a medium traffic highway in District 8 using three different aggregates, a LTWT, a SIGR, and a LMST, respectively. The frictional performances encountered in this segment are shown in Fig. 7.55. The performance of the LTWT aggregate, having a PV of 45, was superior to those of the SIGR and LMST aggregates, with PVs of 27 and 37, respectively. The performance level attained by the SIGR aggregate was very inferior, with most of the obtained FNs falling below the zone of minimum friction. This aggregate was comprised of a good amount of grain-supported particles in which calcite grains were tightly compacted. In addition, the aggregate was classified as uncrushed according to the SDHPT standards.

Although the LMST aggregate showed a performance level higher than that of the SIGR, the level achieved fell within the zone of minimum friction. It is believed that this poor performance is attributed to the high MSS losses experienced with this aggregate. This LMST source was described in the petrographic examination as very soft and friable. It largely consisted of grain-supported particles in which polish susceptible calcite grains were loosely cemented with calcite mud. More discussion will follow later in this chapter on why such aggregates may not have acceptable field performances in spite of their high PVs.

The PBNs and ATDs, obtained from this highway segment and plotted in Figs. 7.56 and 7.57, revealed that the superior performance of the LTWT aggregate was due to the excellent microtexture possessed by its particles, and that performance was not affected by the lower macrotexture of its section's surface. The lower macrotexture was probably caused the LTWT aggregate having less amount of particles retained on No. 1/2 in. and No. 3/8 in. than the LMST and SIGR aggregates. In addition to the LMST being unsound, the high LA losses in both of the LMST and LTWT aggregates could have controlled the rate of decrease in macrotexture in sections 5 and 7, built with these aggregates.

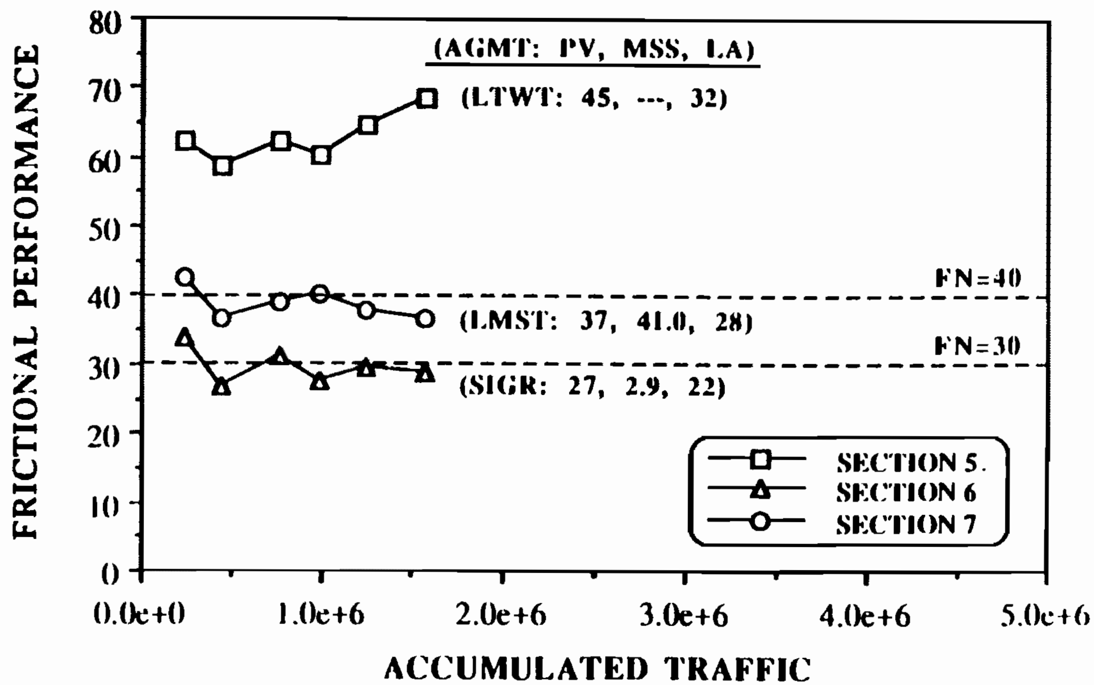


Fig. 7.55 Frictional Performances of the Test Sections Constructed End-to-End in District 8

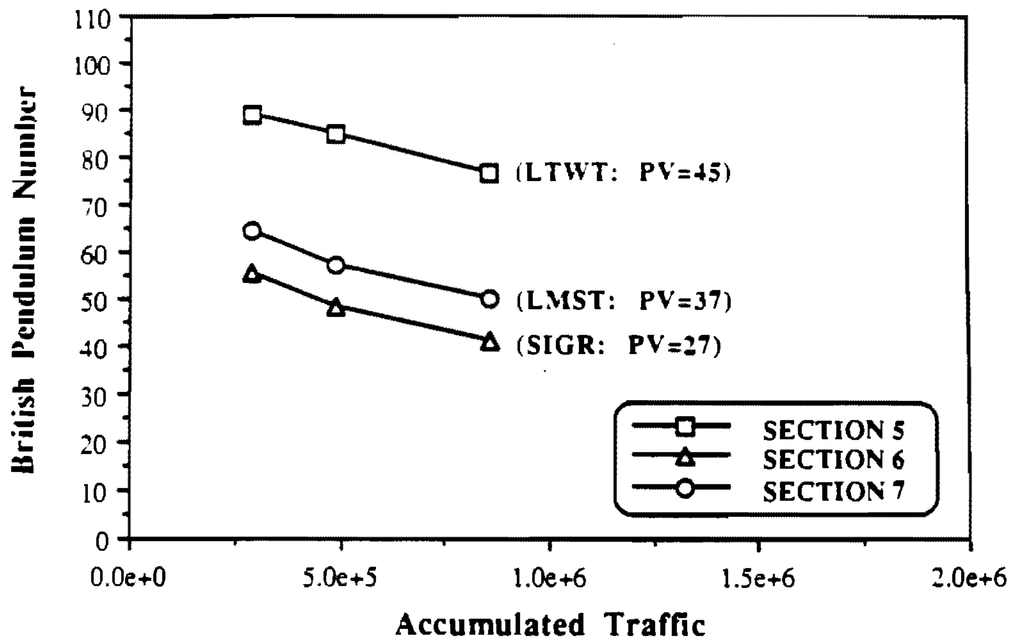


Fig. 7.56 British Pendulum Readings Taken from the End-to-End Sections Placed in District 8

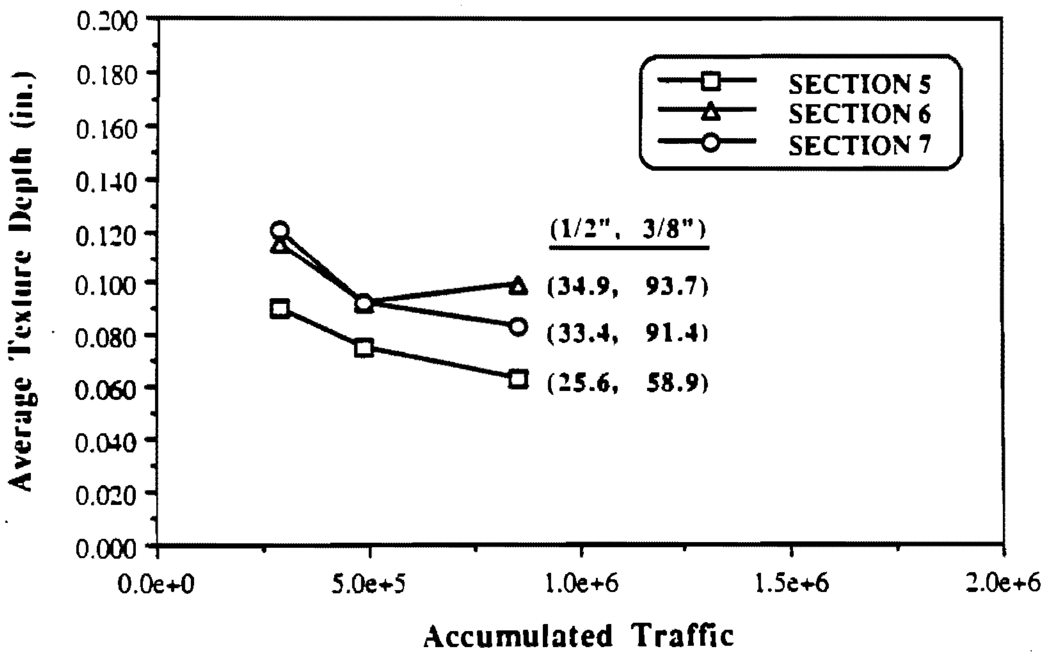


Fig. 7.57 Sand Patch Readings Taken from the End-to-End Sections Placed in District 8

7.4.2 Highway Segment in District 4

Four SIGR and one LMST aggregates were placed end-to-end on a very low traffic highway in District 4. Their frictional performances are illustrated in Fig. 7.58. All of the aggregates were found to have low MSS losses and moderate LA losses. The performances of sections 8 and 11, built with SIGR aggregates having PVs of 33 and 34, respectively, were found to be similar, with FNs ranging between about 65 and 55. Section 9, constructed with a SIGR aggregate having a PV of 30, showed a slightly lower level of performance. However, section 12, with the same quality aggregate as section 9, attained a higher level of performance which almost coincided with those of sections 8 and 11. This was thought to have been the result of aggregate 12 being much better crushed than aggregate 9; aggregate 9 was classified as uncrushed according to the SDHPT standards. Actually, the extent of crushing found in aggregate 12 was even greater than those observed in aggregates 8 and 11.

The LMST source, with a PV of 27, experienced a decrease in frictional performance from an initial FN of about 55 to a fifth FN of about 40 before it bounced back to a sixth FN of about 50. The sixth round of skid testing on this segment had to be redone because, in the first time, the testing gave unreasonably inflated FNs that were as high as 70. The problem, when investigated, was found to be operational and was later corrected. Nevertheless, the FNs obtained after skidding the sections for the second time were still higher than expected. Although no logical explanation could be found for the variation, the decision was not to disregard the sixth round for this segment because it was found later that aggregates similar mineralogically to some of the SIGR aggregates of this segment did experience a somewhat similar type of variation.

The British pendulum and sand patch readings plotted in Figs. 7.59 and 7.60, respectively, were more or less inclusive for this low traffic highway segment. It was only seen that the LMST aggregates had generally lower BPNs than those of the SIGR ones. The second British pendulum reading revealed lower BPNs for aggregates 9 and 12 as compared to those of aggregates 8 and 11.

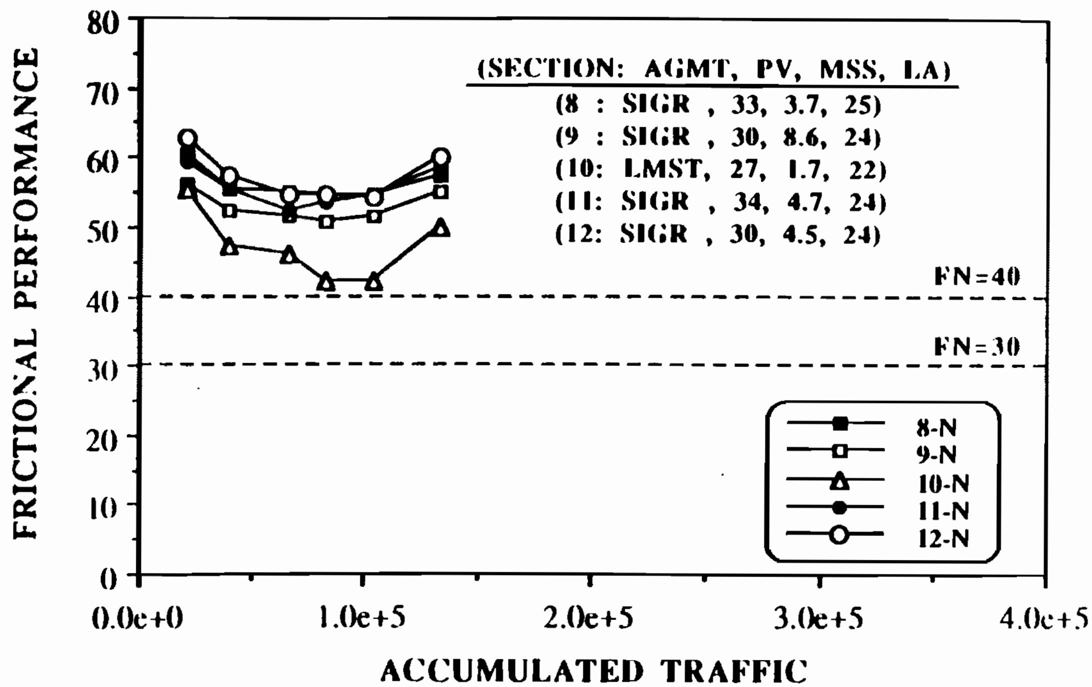


Fig. 7.58 Frictional Performances of the Test Sections Constructed End-to-End in District 4

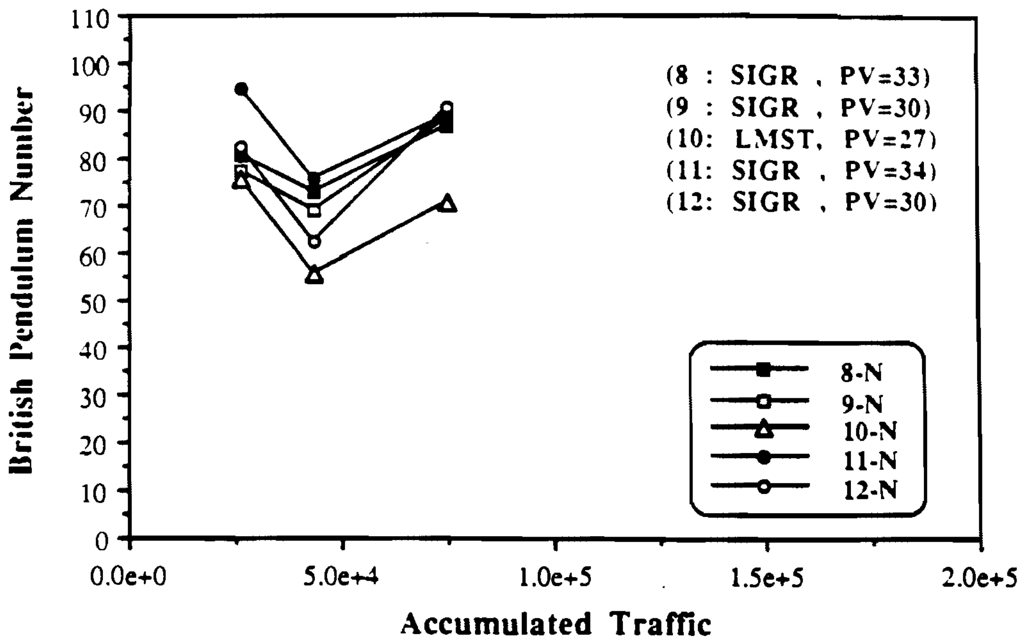


Fig. 7.59 British Pendulum Readings Taken from the End-to-End Sections Placed in District 4

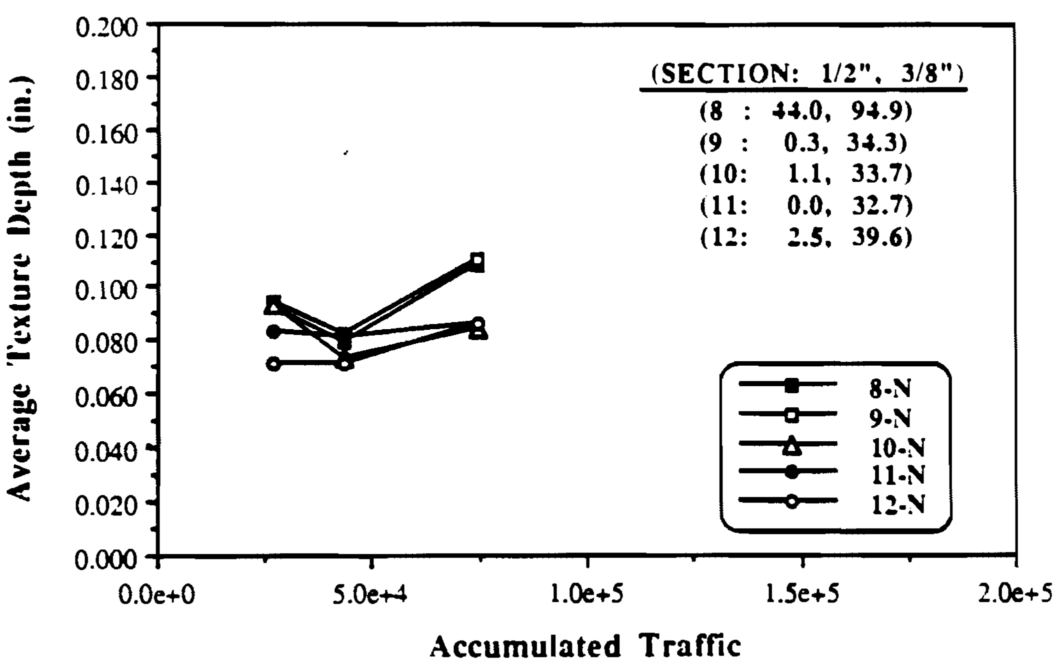


Fig. 7.60 Sand Patch Readings Taken from the End-to-End Sections Placed in District 4

7.4.3 Highway Segment in District 16

Four aggregate sources, one LMRA, one SIGR and two LMST, were placed on the inner lane of a heavily trafficked urban highway in District 16. As depicted in Fig. 7.61, the decrease in frictional performance was logarithmic in relation to accumulated traffic for the LMST and LMRA sources. The two LMST sources, used in sections 17 and 18, had low PVs of 28 and 27, respectively, low MSS losses, and moderate LA losses. The strong logarithmic fit ($R^2 = 0.89$) representing the performances of both of them intercepted with the zone of minimum friction at about one million traffic passes. In addition, their fit reached the lower boundary of the zone at about three and a half million traffic passes.

The SIGR aggregate, used in section 16, had polish and soundness properties similar to those of the LMST aggregates. However, the LA loss was 17 in the SIGR aggregate as opposed to about 25 in the LMST aggregates. This aggregate performance was superior to those of the LMST aggregates. It showed to have maintained a good level of frictional resistance up to about two millions and three quarters of a million of traffic passes, where it intercepted with the zone of minimum friction. The aggregate was evaluated to have more than 90 percent of hard mineral content both by the INRD test and the petrographic examination. According to the latter, the hard mineral was fine chert which had a smooth texture.

The performance of the LMRA aggregate, with a PV of 34, was also seen to be logarithmically related to traffic. Although the level of performance shown by this aggregate was on the average about 7 to 8 FNs higher than the level achieved by the LMST aggregates, its logarithmic fit intercepted with the zone of minimum friction at the same number of accumulated traffic passes as that for the SIGR aggregate.

The British pendulum and sand patch readings, shown in Figs. 7.62 and 7.63, respectively, indicated that since the ATDs of section 15, built with the LMRA aggregate, were lower than those of sections 17 and 18, built with LMST aggregates, the higher BPNs obtained for the LMRA contributed to the better frictional performance. It is believed that the deterioration in the macrotexture of section 15 was the result of the softness of the material as expressed by the high LA loss experienced with it.

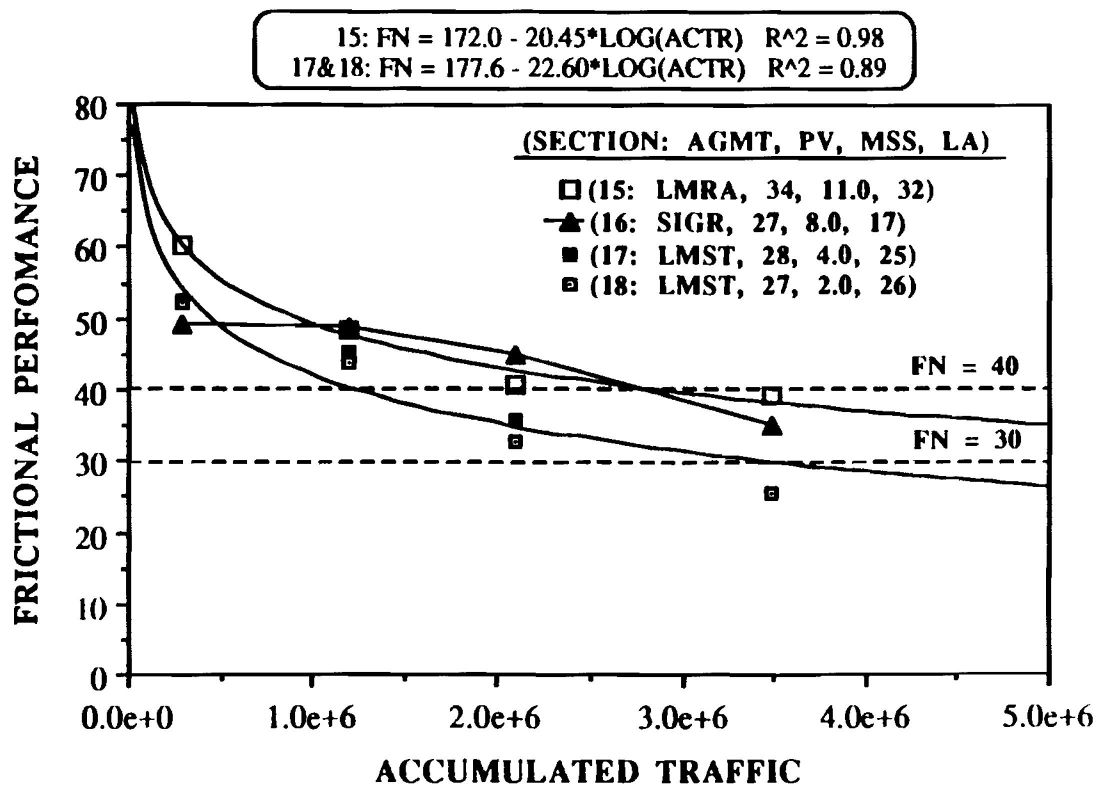


Fig. 7.61 Frictional Performances of the Test Sections Constructed End-to-End in District 16

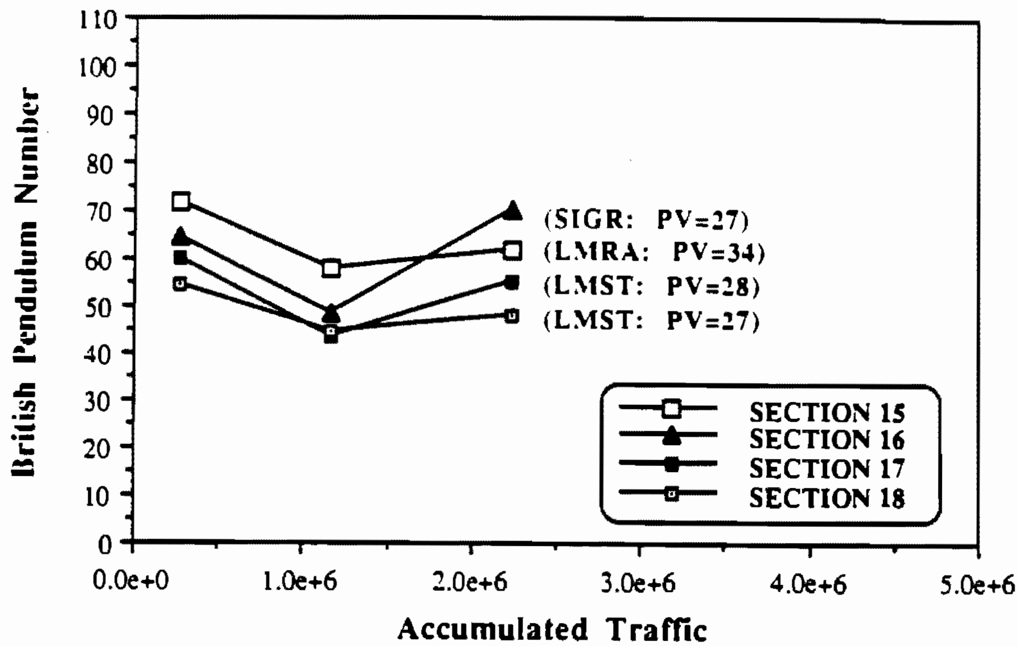


Fig. 7.62 British Pendulum Readings Taken from the End-to-End Sections Placed in District 16

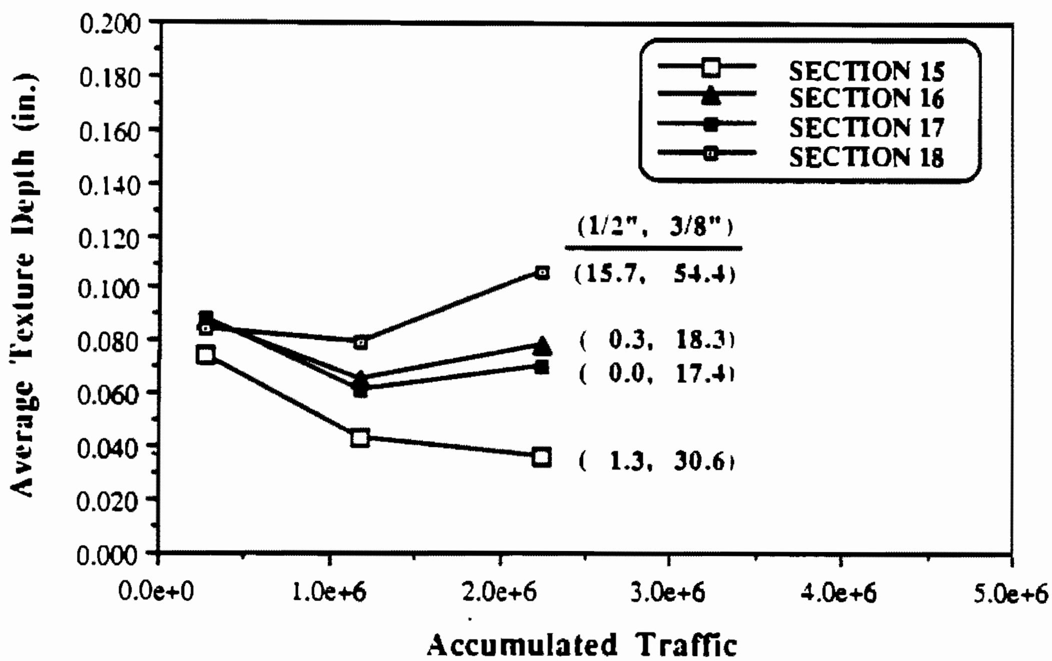


Fig. 7.63 Sand Patch Readings Taken from the End-to-End Sections Placed in District 16

Aggregate 18 had higher percentages of particles retained on sieves No. 1/2 in. and No. 3/8 in. that contributed to the higher ATDs obtained in that section. However, with its low PV, the good macrotexture did not make the aggregate perform any better than aggregate 17.

7.4.4 Highway Segment in District 11

Three different aggregates were placed end-to-end on the inner and outer lanes of this segment. The field results of the inner lane are presented in Figs. 7.64, 7.65, and 7.66; those of the outer lane are illustrated in Figs. 7.67, 7.68, and 7.69. All of the aggregates were on the top of the PV ranges of their respective groups. The LMST and LMRA aggregates of sections 32 and 34, respectively, were evaluated to have high LA losses. In addition to the LMRA and LTWT aggregates being known to have high porosity, the LMST aggregate, with an above than average MSS loss, was found in the petrographic examination to be extremely porous although it was not comprised of a large amount of grain-supported particles.

All of the aggregates were seen to have maintained high FNs on the inner lanes, although the level attained by the LMST aggregate was about 10 FNs lower. In the outer lanes, the levels of performance achieved by the LMRA and LMST aggregates were found to be lower in comparison with theirs of the inner lane. In fact, the LMST aggregate on the outer lane showed a decrease in the FN that was large enough to make it fall in the top portion of the zone of minimum friction.

The variations among the levels of frictional performance observed in each of the lanes were found to be almost fully explained by the variations among the levels of BPNs encountered in the respective lanes. To explain why each of the LMRA and LMST aggregates experienced lower performance levels in the outer lane, the BPNs and ATDs of both inner and outer lane, were plotted versus the age of the sections and the level of ADT was used for making some comparisons. The plots are manifested in Figs. 7.70 and 7.71. For each of the LMRA and LMST aggregates, the BPNs, at the age of about two years, were found to be about 13 numbers lower on the outer lane than on the inner lane, indicating that more polishing have occurred to the minerals constituting these rocks. One the other hand, the BPNs of the LTWT aggregate did

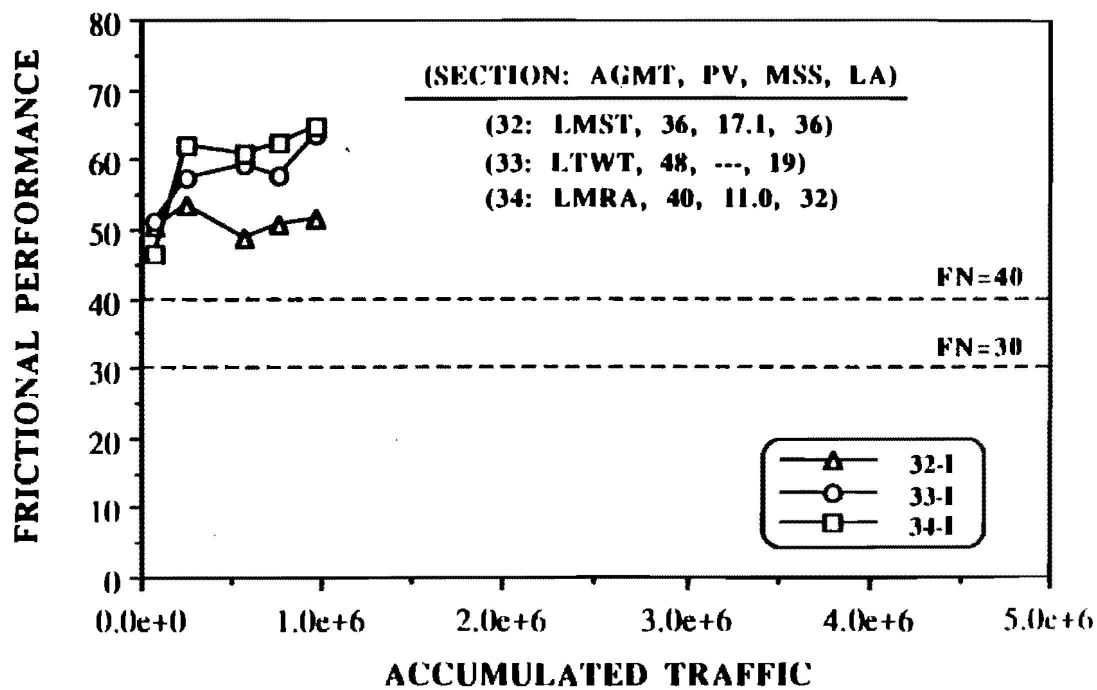


Fig. 7.64 Frictional Performances of the Test Sections Constructed End-to-End in the Inner Lane of Highway US 287 in District 11

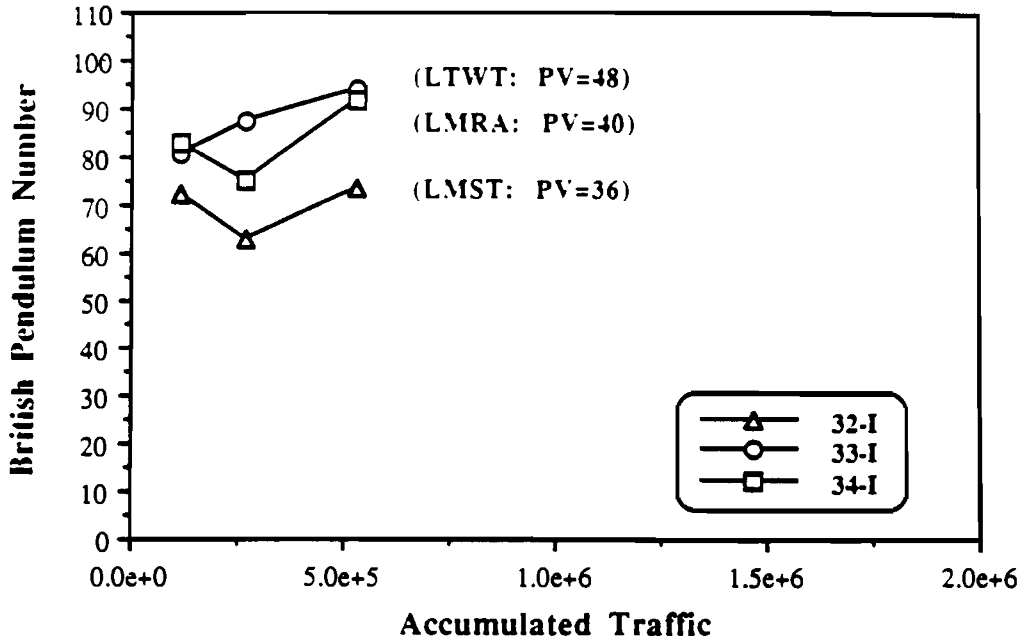


Fig. 7.65 British Pendulum Readings Taken from the End-to-End Sections Placed on the Inner Lane of Highway US 287 in District 11

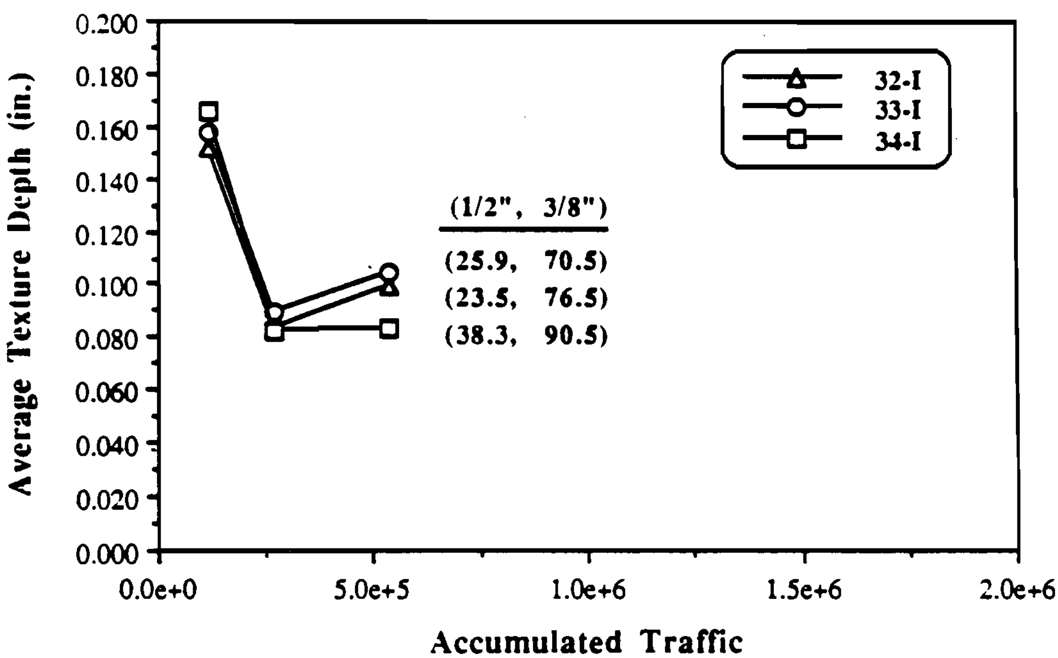


Fig. 7.66 Sand Patch Readings Taken from the End-to-End Sections Placed on the Inner Lane of Highway US 287 in District 11

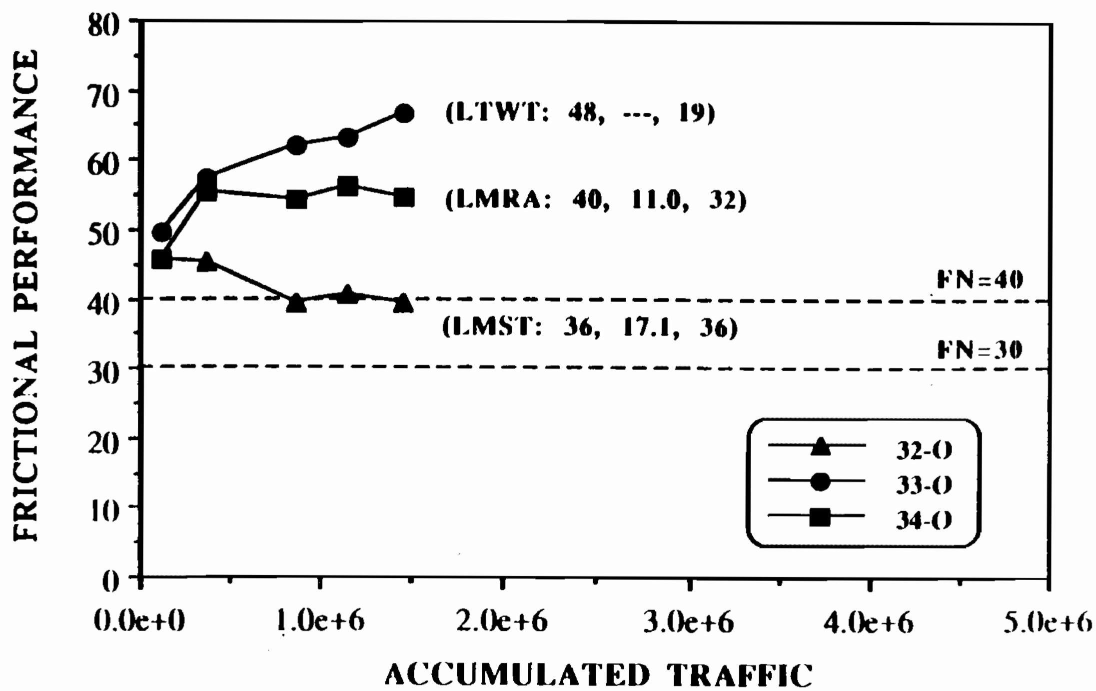


Fig. 7.67 Frictional Performances of the Test Sections Constructed End-to-End in the Outer Lane of Highway US 287 in District 11

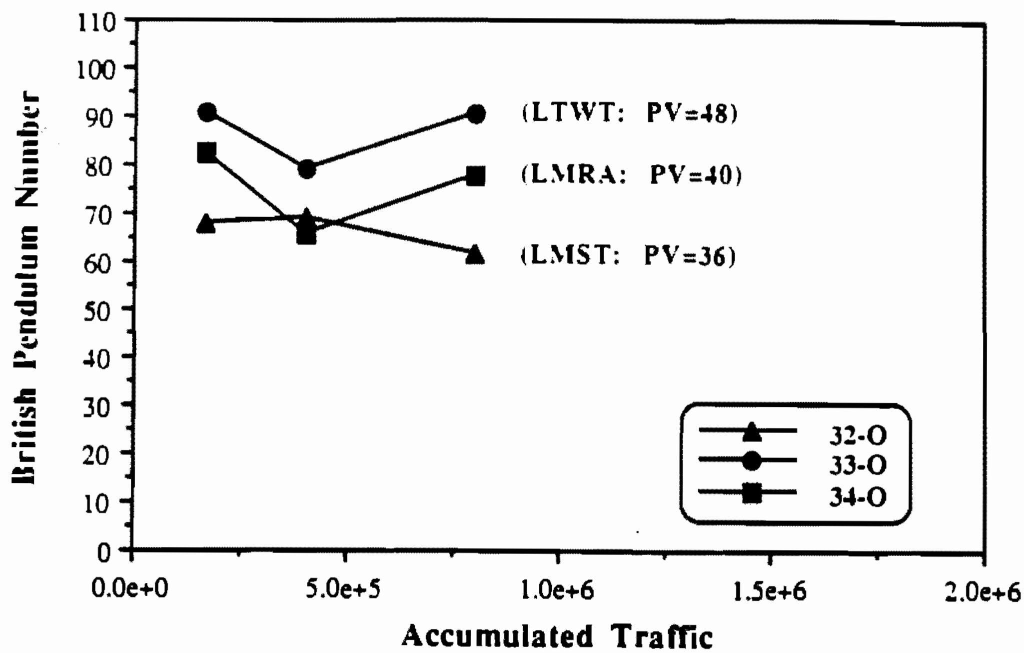


Fig. 7.68 British Pendulum Readings Taken from the End-to-End Sections Placed on the Outer Lane of Highway US 287 in District 11

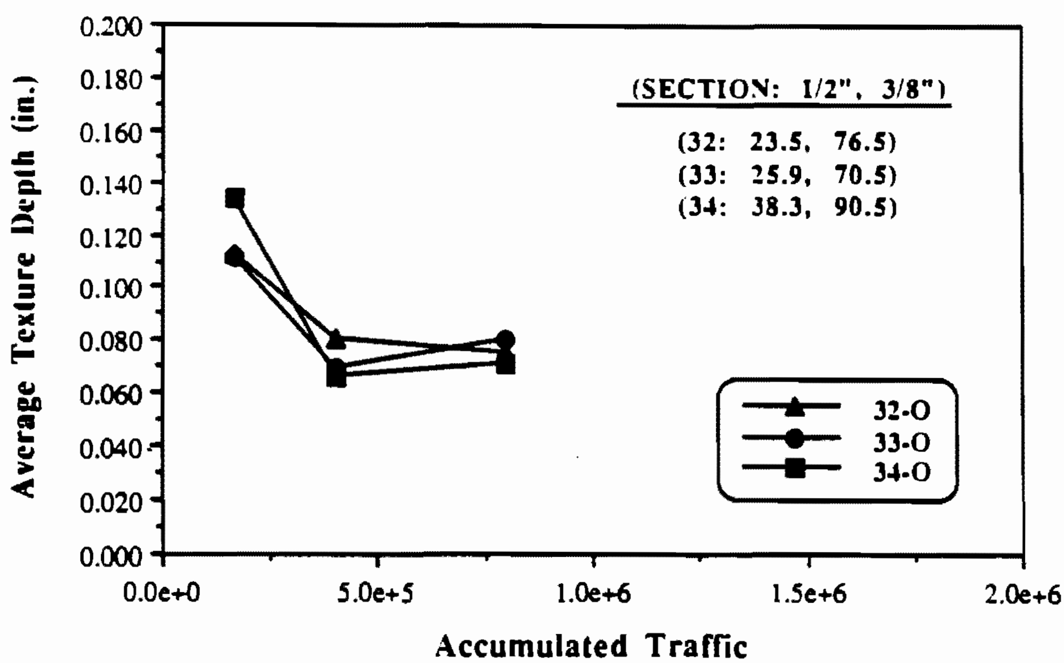


Fig. 7.69 Sand Patch Readings Taken from the End-to-End Sections Placed on the Outer Lane of Highway US 287 in District 11

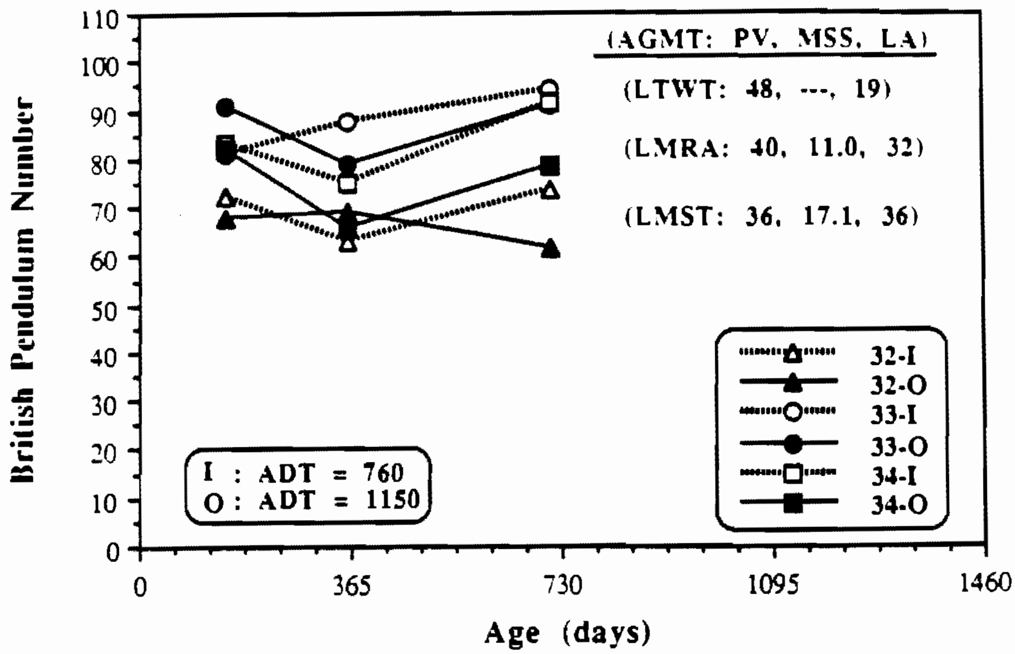


Fig. 7.70 British Pendulum Readings versus Age for the Inner and Outer Lanes of the End-to-End Sections Placed on Highway US 287 in District 11

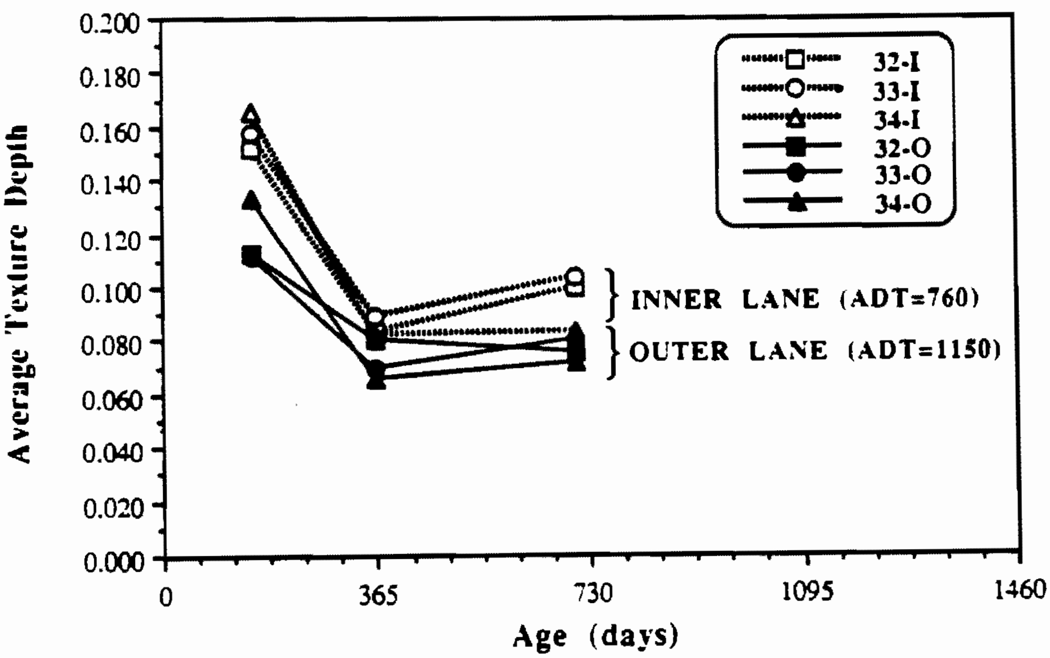


Fig. 7.71 Sand Patch Readings versus Age for the Inner and Outer Lanes of the End-to-End Sections Placed on Highway US 287 in District 11

not differ as much, simply because the viscularity-related frictional phenomenon of these aggregates is not easily comparable with the mineralogy-related polish phenomenon of natural aggregates.

Since the region, where this segment is located, is characterized by temperature freeze-thaw cycling and high amounts of precipitation falling in all seasons, it is now believed that the inner lanes have been experiencing a low level of polishing by the lesser ADT that has been almost completely rejuvenated by the effects of precipitation and freeze-thaw cycling. On the contrary the outer lanes are believed to have been experiencing a rate of polishing higher than that of rejuvenation. Finally, the macrotexture readings were observed to be considerably lower in the outer lane, which could have also contributed to the lower levels of performance experienced by the LMRA and LMST aggregates of this lane.

7.4.5 Highway Segment in District 13

In this segment, three levels of performance were encountered, as shown in Fig. 7.72. First, two LTWT aggregates, used in sections 40 and 41 with respective PVs of 50 and 51, maintained a very high friction level that even seemed to have slightly increased with traffic exposure. Second, a LMRA aggregate, replicated in sections 42 and 43, started with a high level of performance that later decreased gradually until it reached the zone of minimum friction at about three and a half million traffic passes; the LMRA aggregate had a PV of 35 with a moderate MSS loss and a high LA loss. The third level of performance was that seen in the three LMST aggregates and the SIGR aggregate. The LMST aggregates, having PVs in the range of 27 to 29, experienced rapid decreases in frictional resistance making them fall in the zone of minimum friction at about one to one and a half million traffic passes. The relationships between their performances and traffic were found to be perfectly logarithmic. The variation in performance seen among these aggregates is believed to be the result of various levels of surface aggregate loss experienced in sections 36, 37, and 38, built with these aggregates. The performance of the SIGR aggregate was somewhat similar to those of the LMST aggregates. However, this aggregate started with a lower FN and its fit fell in the zone of minimum friction at only half a million traffic passes.

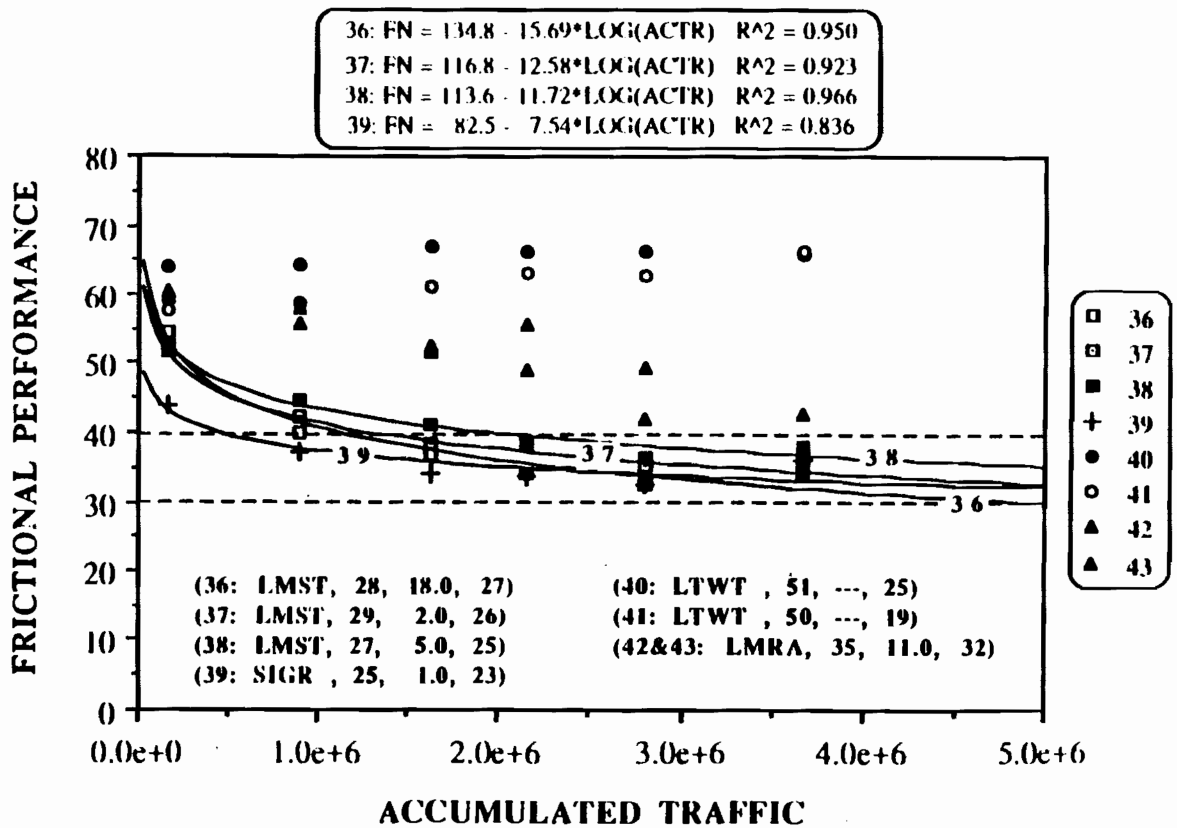


Fig. 7.72 Frictional Performances of the Test Sections Constructed End-to-End in District 13

The British pendulum and sand patch readings, presented in Figs. 7.73 and 7.74, respectively, indicated that the various levels of frictional performance were directly related to the levels of BPN. This was evident in spite of the fact that a sizable variation was seen in the measured ATDs. However, the low ATDs were associated with the LMRA and LTWT aggregates, both are known to loose macrotexture either due to the softness of the material in the case of the former or in the process of maintaining excellent microtexture in the case of the latter.

7.4.6 Highway Segment in District 20

Figure 7.75 shows the frictional performance of two aggregate materials, each was replicated on the same highway segment bound. The LTWT aggregate, replicated in sections 44 and 46 and having a PV of 48, was again shown to have maintained a level of friction superior to that attained by the LMRA aggregate, replicated in sections 45 and 47. The LMRA aggregate, having a PV of 37, showed a decreased in the FN to below the upper boundary of the zone of minimum friction at about one and a half million traffic passes.

7.4.7 Highway Segment in District 7

Figure 7.76 illustrates the differences in frictional performance encountered on this segment. Section 50 suffered from a severe bleeding distress causing the FNs to drop drastically. Sections 51 and 52 were built with good quality SDST aggregates that showed excellent performance. The LMST aggregate, replicated in sections 53 and 54, experienced only some decrease in the FN in spite of its high MSS and LA losses. As will be discussed later, it is believed that the presence of dolomite grains in this aggregate as well as the construction application rate of aggregate contributed to this acceptable performance.

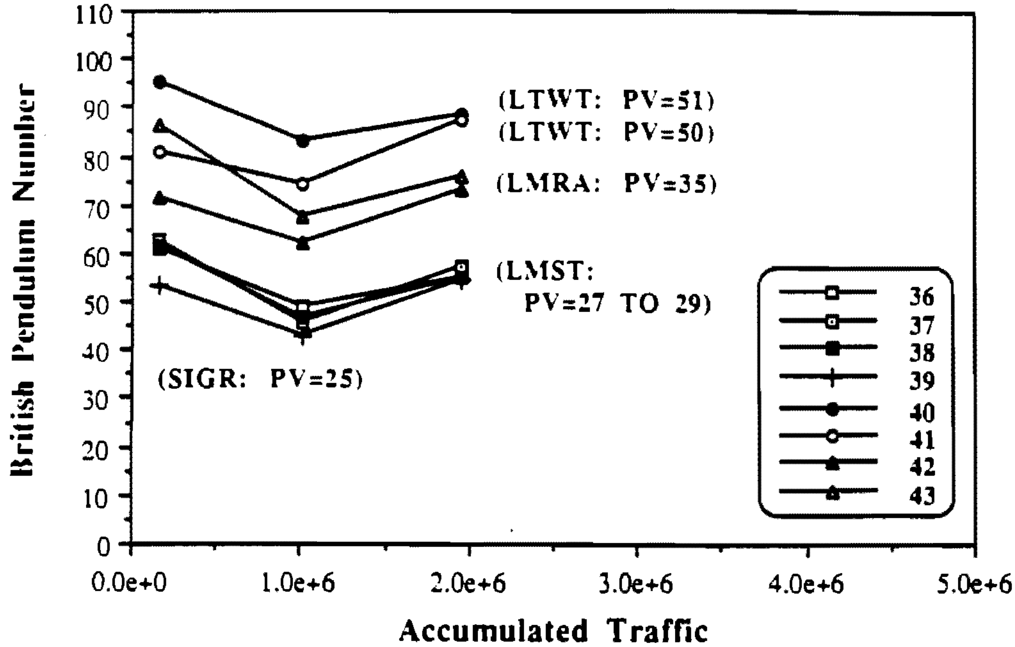


Fig. 7.73 British Pendulum Readings Taken from the End-to-End Sections Placed in District 13

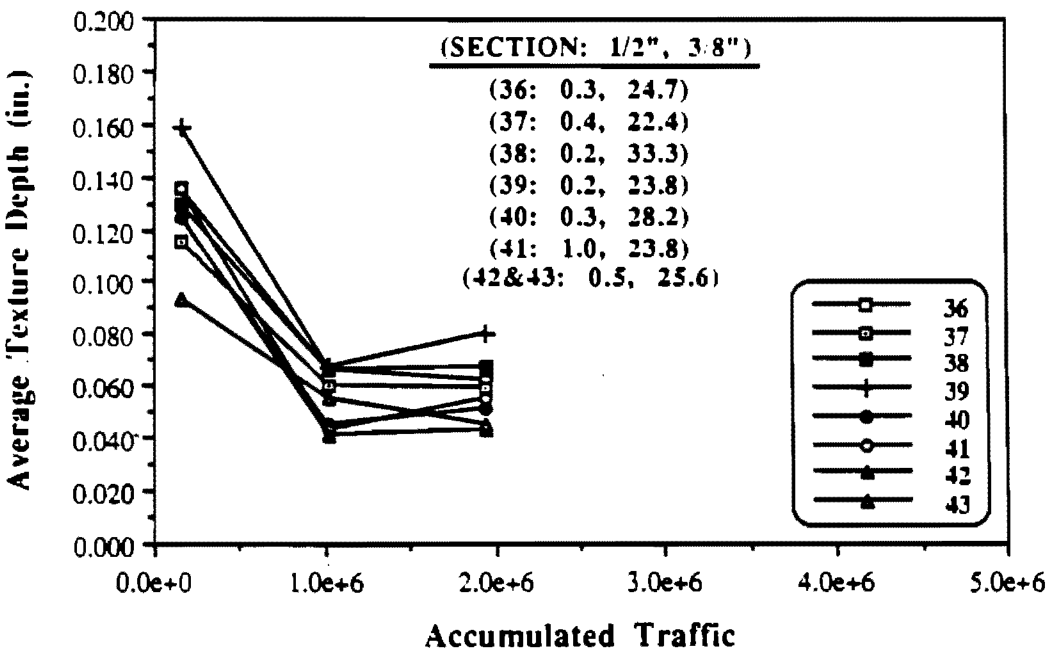


Fig. 7.74 Sand Patch Readings Taken from the End-to-End Sections Placed in District 13

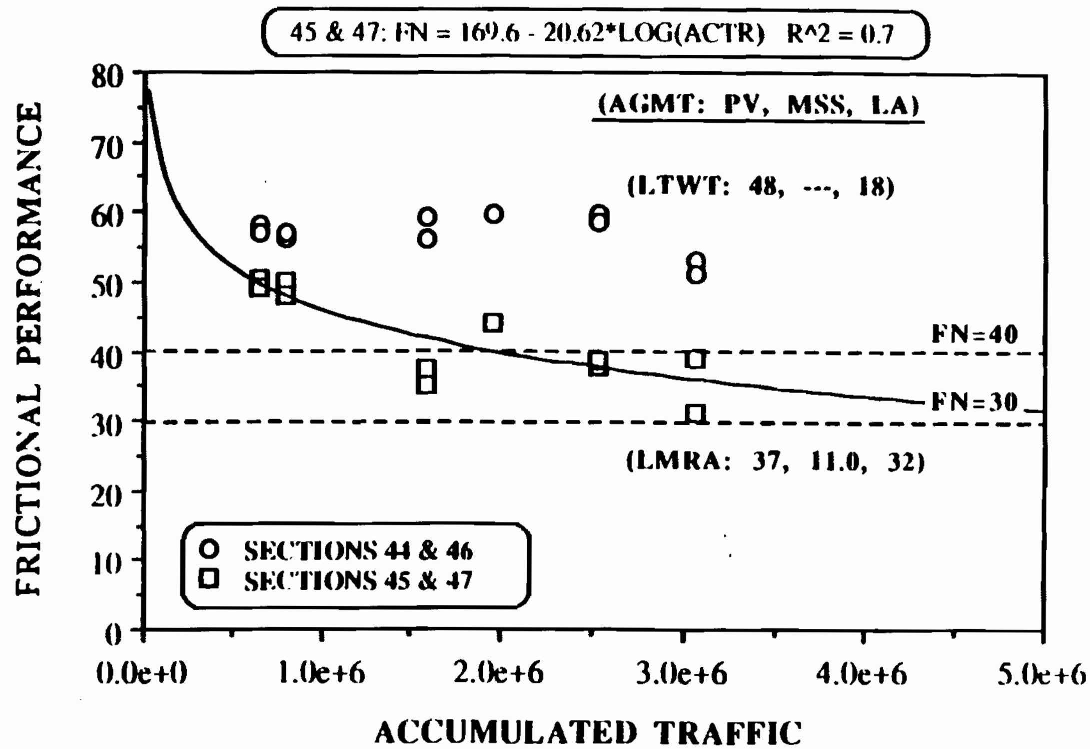


Fig. 7.75 Frictional Performances of the Test Sections Constructed End-to-End in District 20

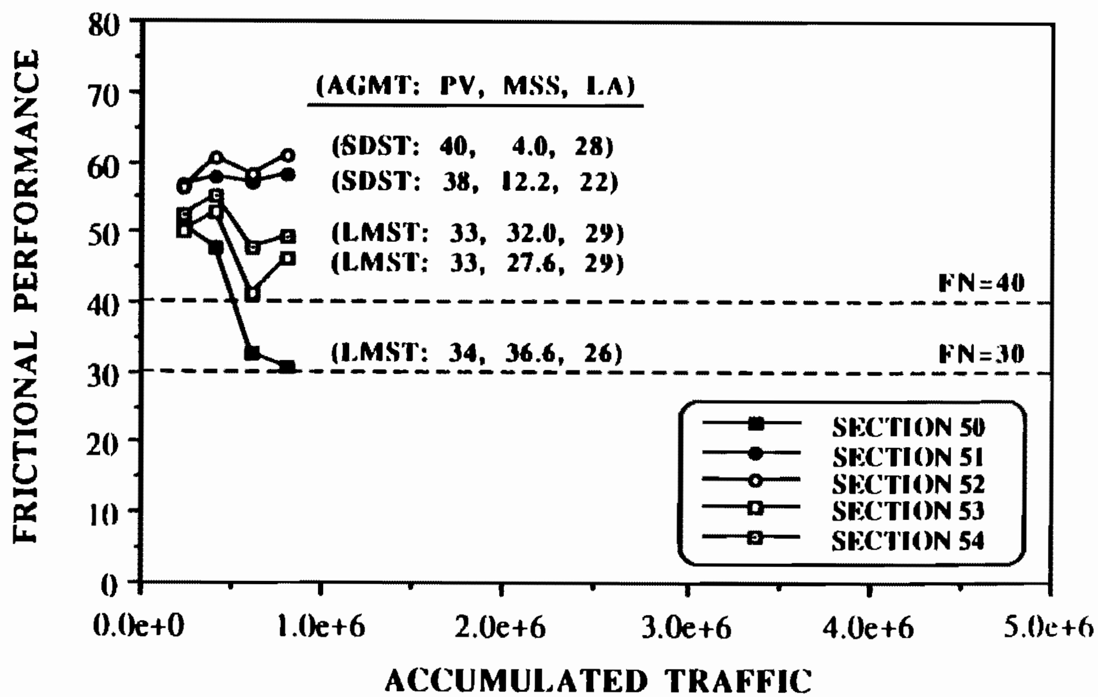


Fig. 7.76 Frictional Performances of the Test Sections Constructed End-to-End in District 7

7.5 Grouping of Data According to Climatic Regions, Construction Variables, and Aggregate Properties

In this section, the objective was to divide the data into groups of sections with similar aspects which experienced similar trends in frictional performance. The groups, if any exists, would represent the experimental design on which the statistical analysis would be based.

7.5.1 Lightweight Aggregates

Since the LTWT aggregates were found to possess laboratory polish properties distinctive from those possessed by the natural aggregates and were seen to have attained the highest frictional performance level, the decision was to treat them separately in the study of the variation encountered in their group. The overall performance of this aggregate group was shown in Fig. 7.47. The data included in this figure were first grouped according to the temperature freeze-thaw division of the climatic regions. The grouping, shown in Fig. 7.77, indicated that the majority of the sections constructed in regions with temperature freeze-thaw cycling, Group B, have experienced any increase in frictional resistance with exposure to traffic. The PV ranged in this group from 42 to 49. The sections constructed in Region I with no freeze-thaw cycling, Group A, maintained a somewhat steady level of frictional performance. The PV ranged from 48 to 51. A further subdivision of Group B into dry and wet regions is shown in Fig. 7.78. Group B.1 consisting of sections located in the wet region did not exhibit a friction level higher than that of Group B.2 of the dry region.

The next step was to further subgroup the data into groups of sections with similar construction aspects. Figure 7.79 shows the subgrouping of the data of Groups B.1 and B.2. Two observations were made. First, in Group B.1, Group B.1.1 consisting of a section constructed with an AGSR of 70 sq.yd/cu.yd showed a lower performance level compared to that of Group B.1.2 made up of sections constructed with an AGSR of about 100 sq.yd/cu.yd. The aggregates in these groups had similar polish properties. Second, in Group B.2, the higher PV of 45 possessed by the aggregates in Group B.2.1 obscured the effect of AGSR. However, Groups B.2.1

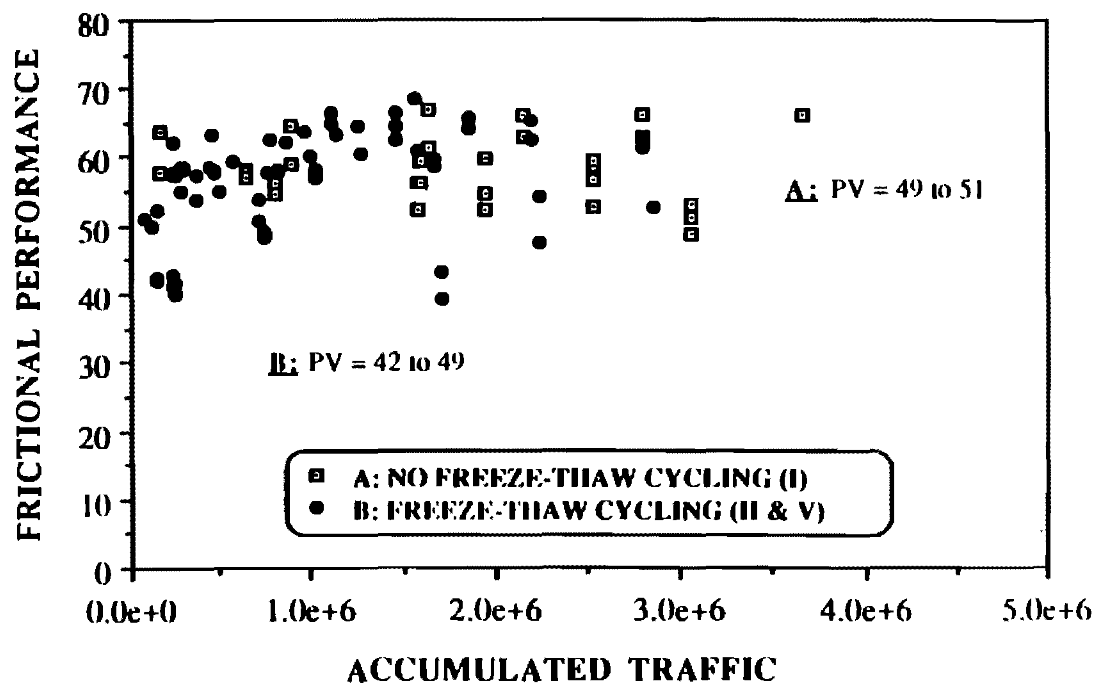


Fig. 7.77 Frictional Performance of Test Sections Constructed with LTWT Aggregates Having a PV in the Range of 42 to 51 -- Grouped According to Freeze-Thaw Climatic Regions

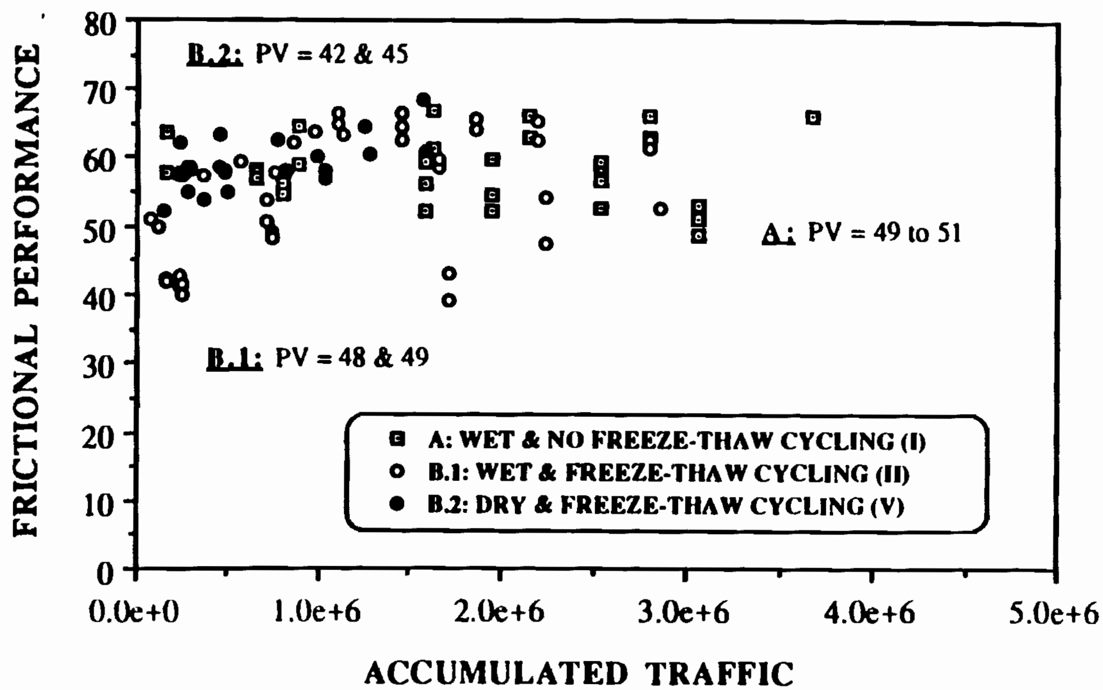


Fig. 7.78 Frictional Performance of Test Sections Constructed with LTWT Aggregates Having a PV in the Range of 42 to 51 -- Grouped According to Climatic Regions

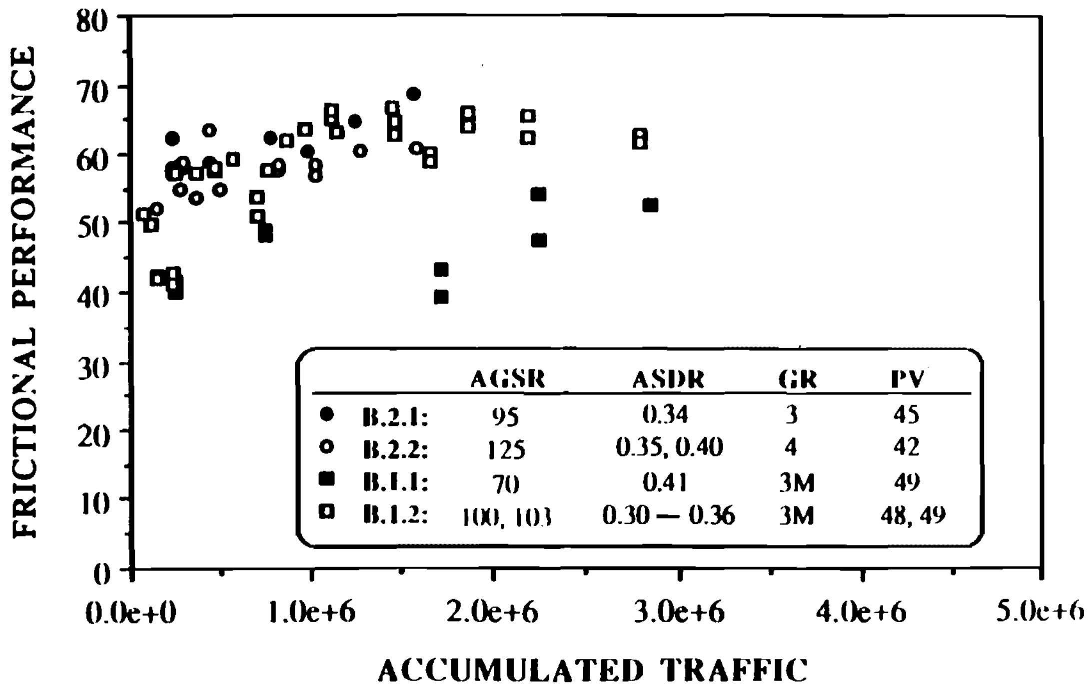


Fig. 7.79 Frictional Performance of Test Sections Constructed with LTWT Aggregates in Region II -- Grouped According to Construction Design Variables

and B.2.2 both performed better than Group B.1.1. Since the PVs of the aggregates in these two groups are lower than that of the aggregate in Group B.1.1, it is believed that the difference in performance can be attributed to the AGSR effect.

When the data of Group B.1.1 was plotted along with that of Group A in Fig. 7.80, it was evident that the level of performance increased as the AGSR increased. The PV range was very narrow for the aggregates used in this plot. In addition, the performance of Group A.1 was still superior to that of the other groups despite the fact that the aggregates in that group were of grade 4 compared to 3M for those in the other groups.

The phenomenon can be explained as follows. Under the exposure to traffic, the aggregate particles in a seal coat surface are pressed more and more into the asphalt layer thus displacing some of the asphalt. When the same cubic yard of aggregates is spread over different surface areas, more asphalt will be required to displace in the case of smaller areas. The displacement of asphalt results in the asphalt filling much of the volume of the interstices between the aggregate particles. This, in turn, results in a reduction in the macrotexture of the surface, and it may or may not cause a bleeding distress in the surface. Moreover, when the aggregate particles are spaced out too closely that excessive interlocking and overlaying between the particle edges result, crushing in the particles may result. Crushing alters the gradation of the aggregate making the surface unstable and causing bleeding problems.

In the example shown in Fig. 7.81, section 35, comprising Group B.1.1, suffered a severe bleeding distress, while sections 44, 46, 48, and 49, constituting Group A.2, suffered from a moderate bleeding distress. In Fig. 7.81, the ATDs obtained from the distressed sections are presented along with the ATDs of undistressed sections 40 and 41 of Group A.1. Unfortunately, since the sand patch readings were obtained from the portions of the wheel paths with minimal distress, the results did clearly show the reduction in the macrotexture caused by bleeding. However, Fig. 7.82 includes photographs of section 35 showing the alteration in the gradation due to crushing after about one year of traffic exposure and the resulting severe bleeding of the surface that took place one year later. A comparison can be made with Fig. 7.83 showing the nicely spaced, uniformly sized aggregate particles and the excellent condition of section 41.

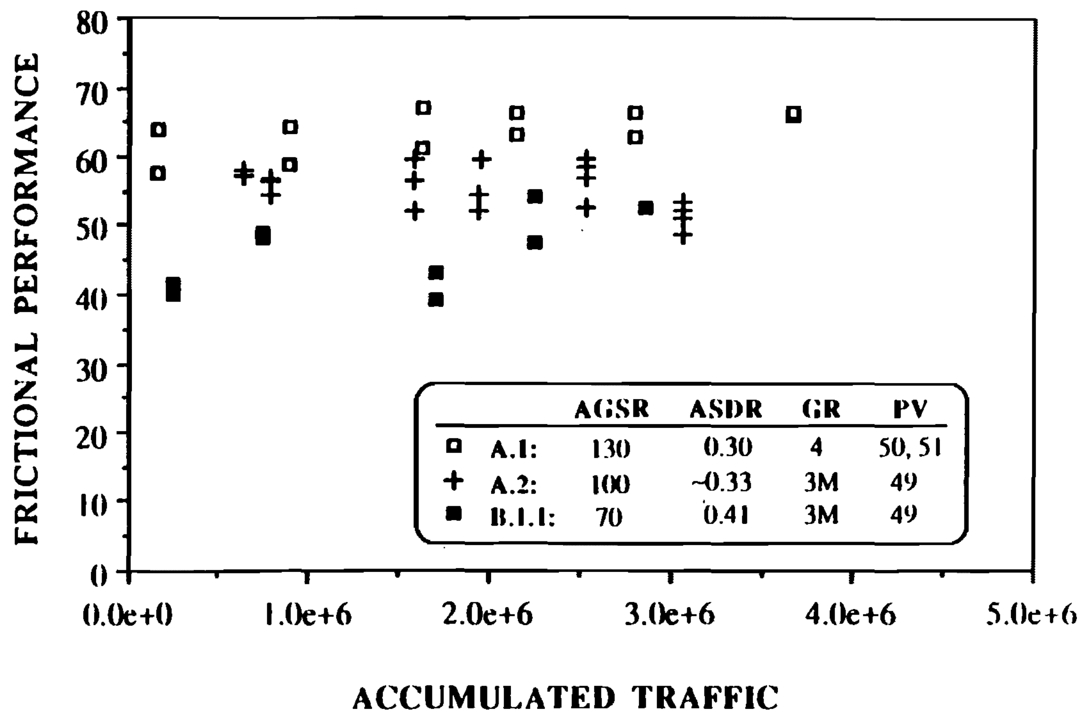


Fig 7.80 Frictional Performance of Test Sections Constructed with LTWT Aggregates Having a PV in the Range of 49 to 51 -- Grouped According to Construction Design Variables

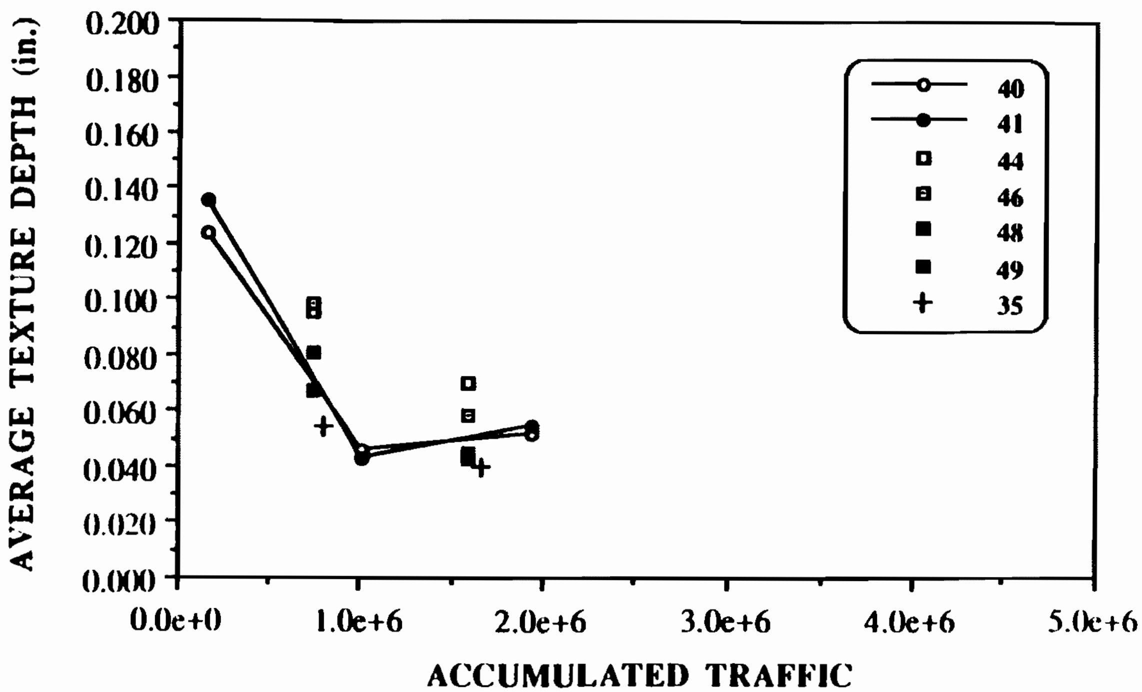
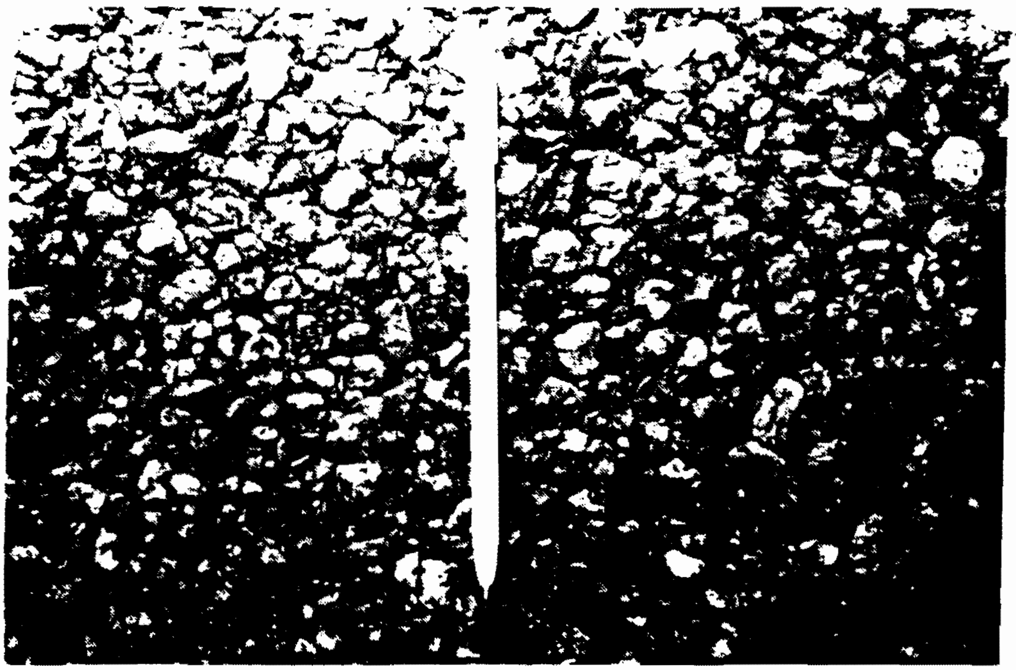
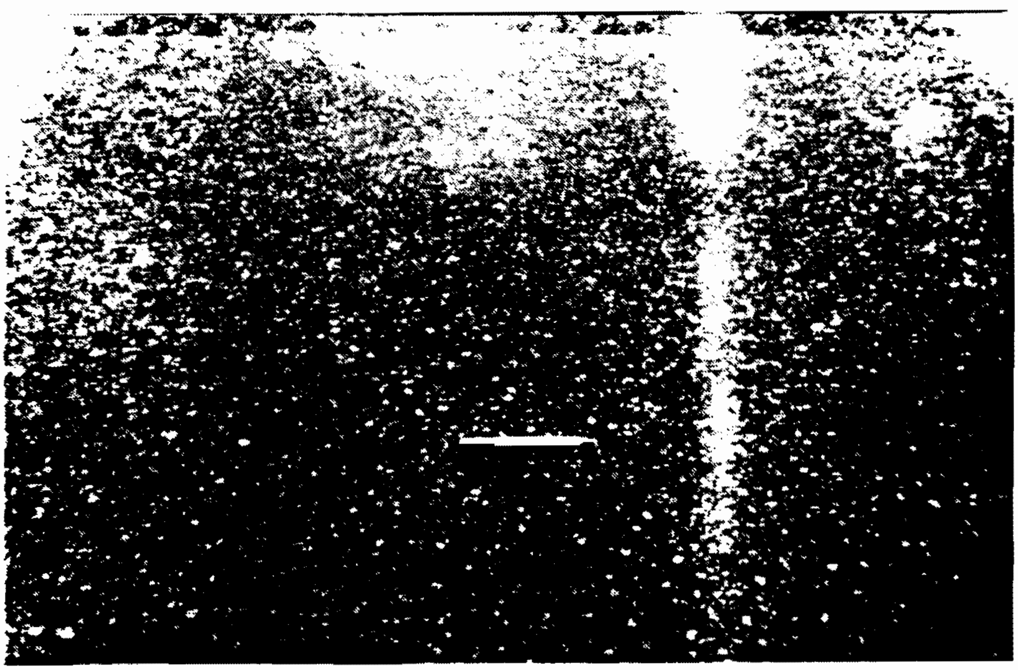


Fig. 7.81 Sand Patch Readings Taken from Sections 35, 40, 41, 44, 46, 48, and 49

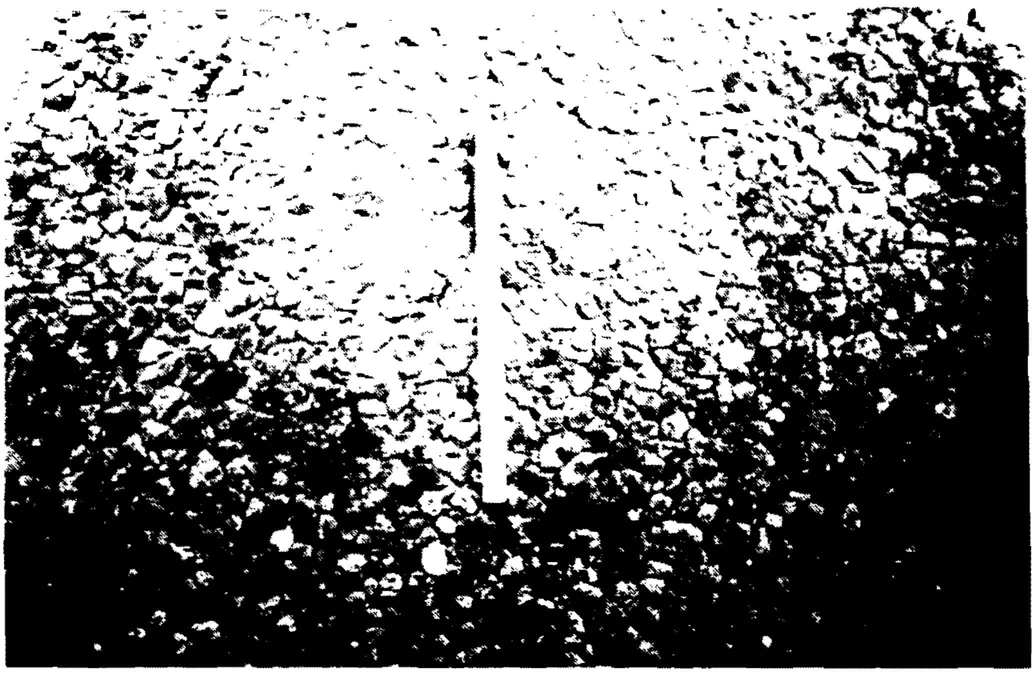


(a) alteration in the gradation after one year

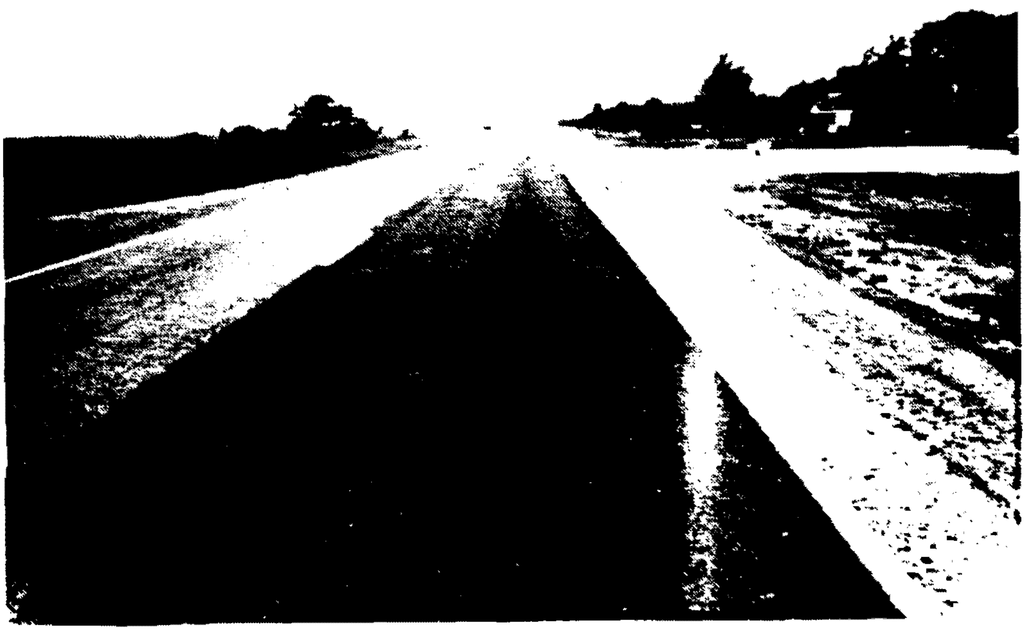


(b) asphalt bleeding after two years

Fig. 7.82 Condition of section 35



(a) no reduction in particless size



(b) no bleeding distress

Fig. 7.83 Condition of section 41

7.5.2 Natural Aggregate

The friction data of the natural aggregate were first divided into four major groups based on the PV, MSS loss, and LA loss. Group 1 consisted of the SDST and RHYO aggregates with PV ranging from 36 to 41, low MSS and LA losses. Only one SDST experienced an LA loss of 28 percent and one RHYO experienced an MSS loss of 15.0 percent. Group 2 was comprised of the LMRA aggregates with high PVs in the range of 34 to 40, moderate MSS losses in the range of 7.0 to 14.0 percent and high LA losses in the range of 28 to 34. Group 3 included the LMST and SIGR aggregates with PVs ranging from 30 to 37, a wide range of MSS losses from 1.3 to 41.0 percent, and a wide range of LA losses from 10 to 36 percent. Finally, Group 4 was made up of the LMST and SIGR aggregates possessing low PVs in the range of 25 to 29, low MSS losses (with one exception in aggregate 36), and LA losses ranging from 17 to 27. The frictional performances of these groups are illustrated in Fig. 7.84. The variation observed in each group was studied as follows.

Group 1 exhibited excellent performance as shown in Fig. 7.85. When its data were grouped in Fig. 7.86 according to the construction variables, some evidence was prevalent as to the effect of AGSR on the difference in performance. Group 1.1 with a AGSR of 120 sq.yd/cu.yd was shown to have maintained a higher level of performance as compared with that of Group 1.3 with a AGSR of 70 sq.yd/cu.yd. Figure 7.87 shows the overlapping between aggregate particles in the finished surface of section 31 comprising Group 1.3. It also shows the resulting bleeding in the wheel paths and aggregate loss in the middle of the lane after about two years of service. The high ASDR used in this section could have also resulted in needing to displace an even larger quantity of asphalt. Figure 7.88 shows photographs of section 52 from Group 1.1 after one year of traffic exposure depicting the excellent condition maintained with a higher AGSR and lower ASDR. The level of ADT was about 1000 on section 52 compared to 1250 on section 31.

The first subgrouping of Group 2, shown in Fig. 7.89, was done according to the temperature freeze-thaw division of climatic regions. Group 2.1 consisting of an aggregate with a PV of 40 showed a steady performance compared to the decrease in performance observed in Group 2.2. It is believed that the difference is due to the aggregate in Group 2.1 having constantly rejuvenated microtexture caused by the high

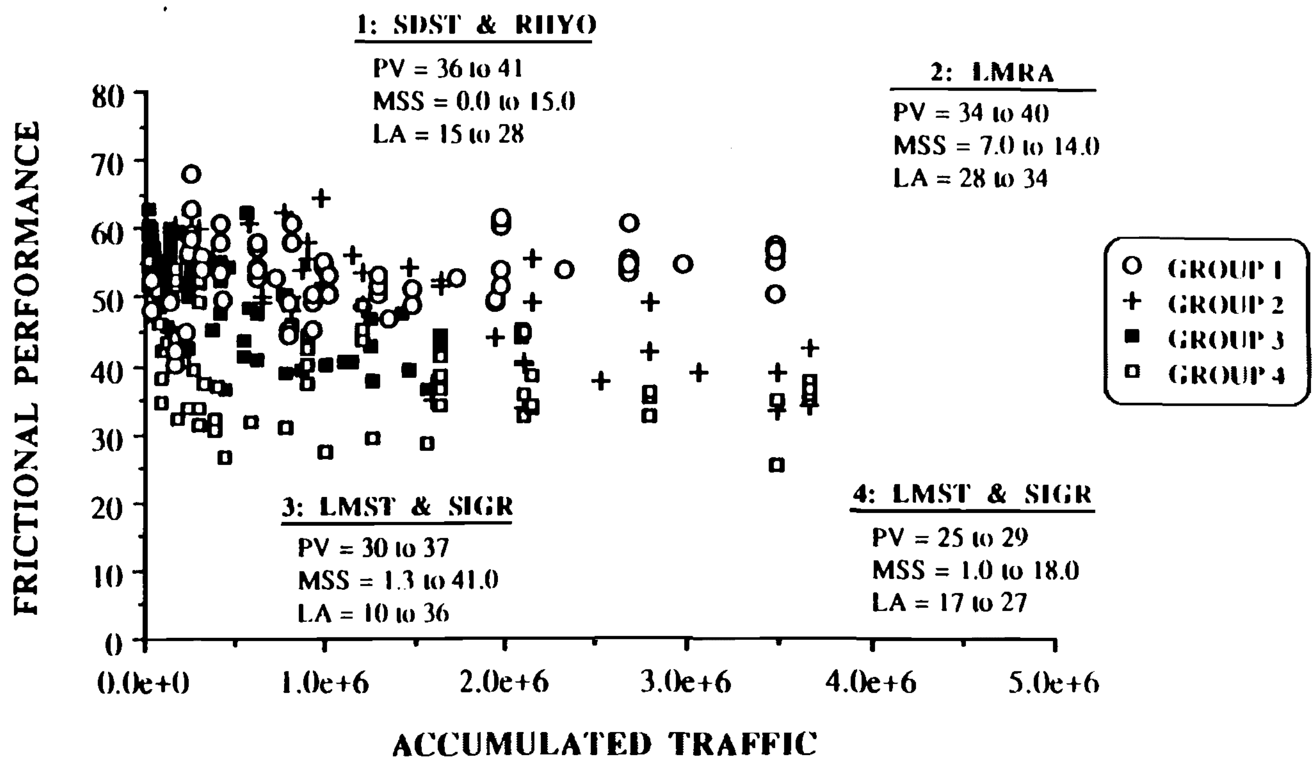


Fig. 7.84 Frictional Performance of Test Sections Constructed with Natural Aggregates -- Grouped According to Polish, Soundness, and Abrasion Characteristics

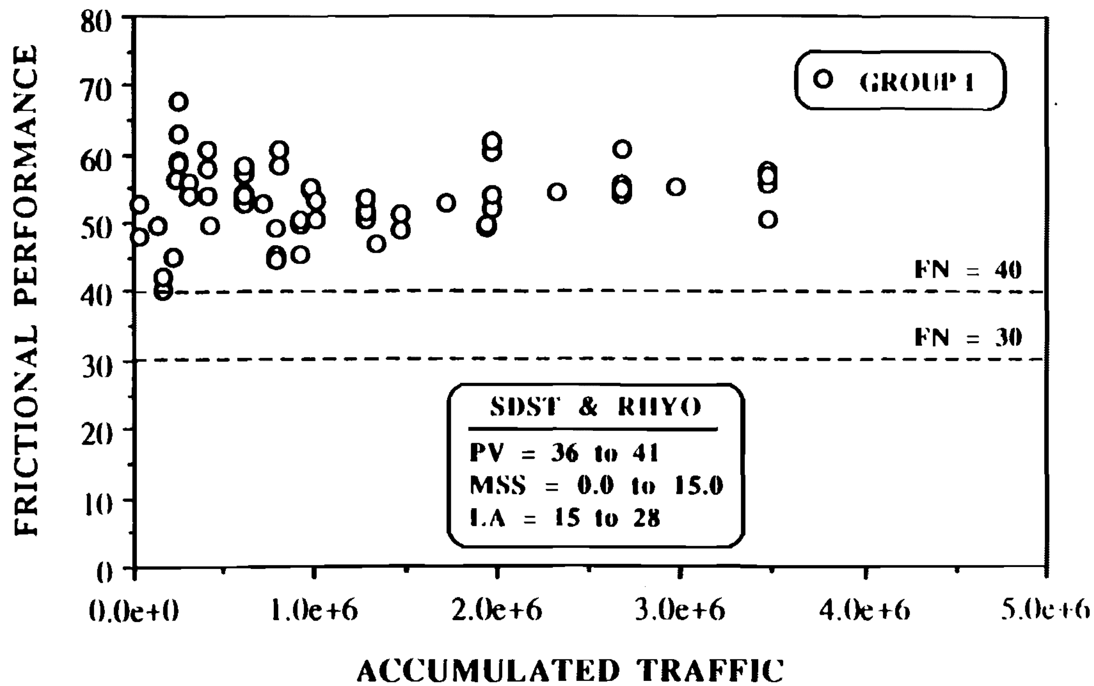


Fig. 7.85 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 36 to 41, and Low MSS and LA Losses

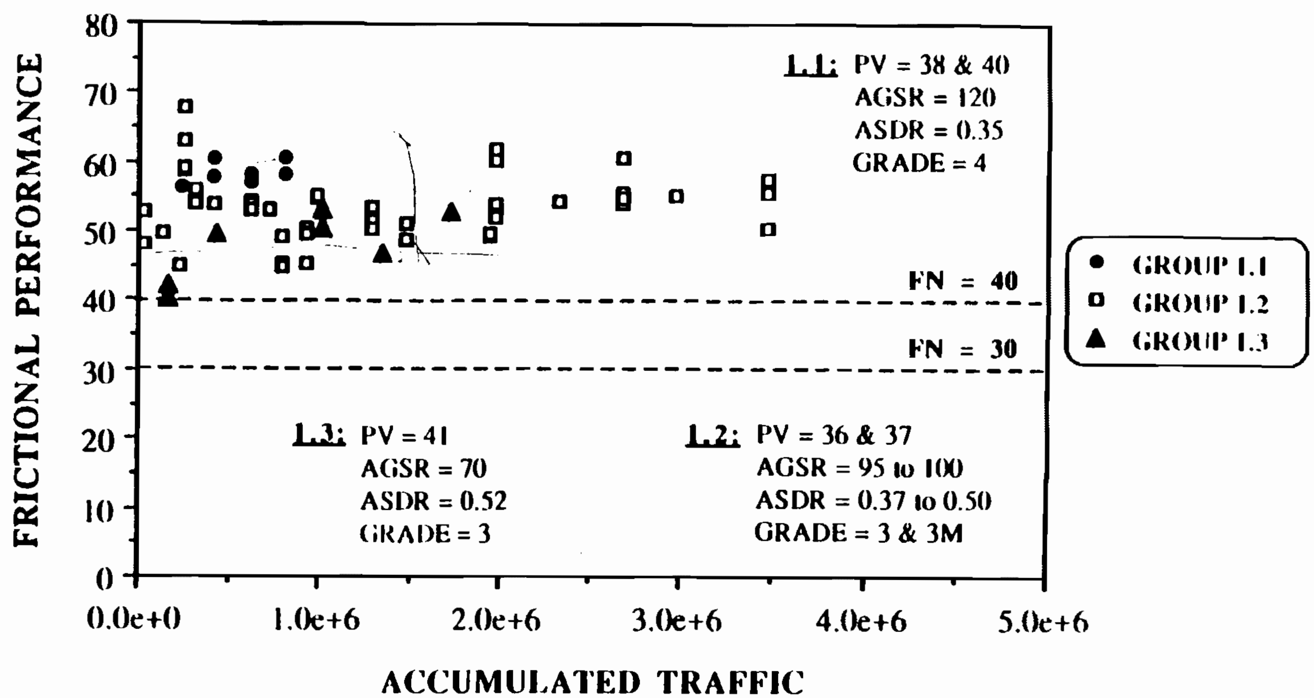
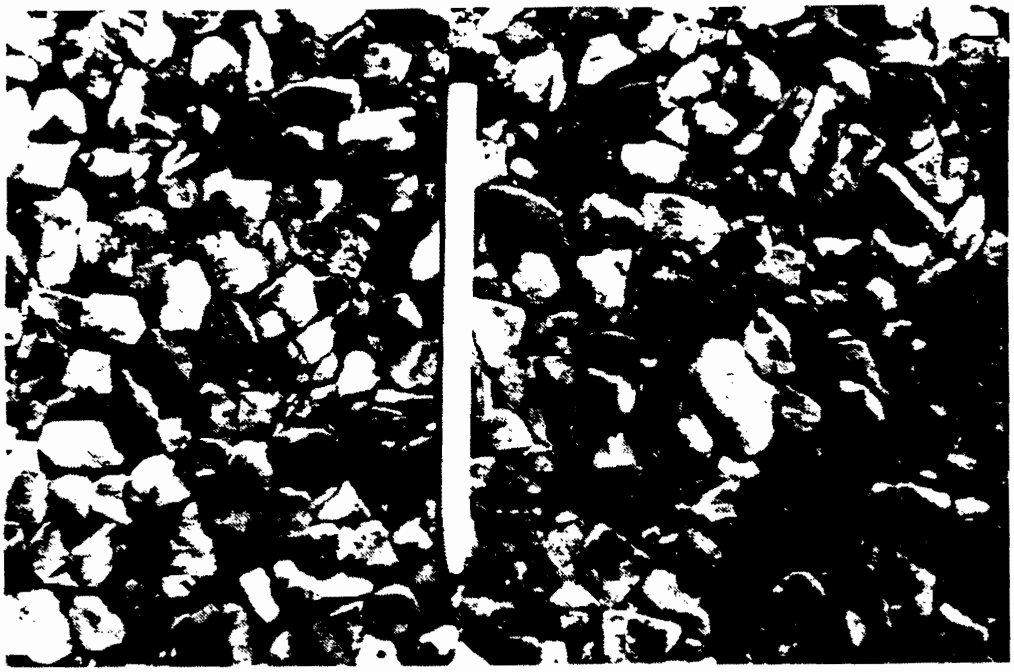
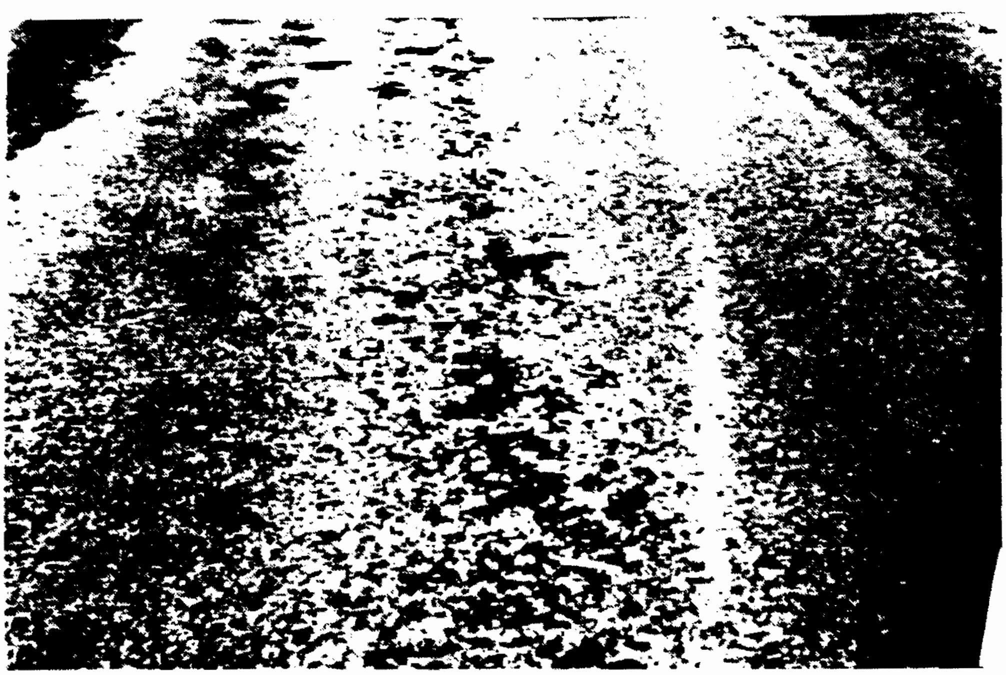


Fig. 7.86 Frictional Performance of Test Sections Constructed with Natural Aggregates Having a PV in the Range of 36 to 41 with Low MSS and LA Losses -- Grouped According to Construction Design Variables

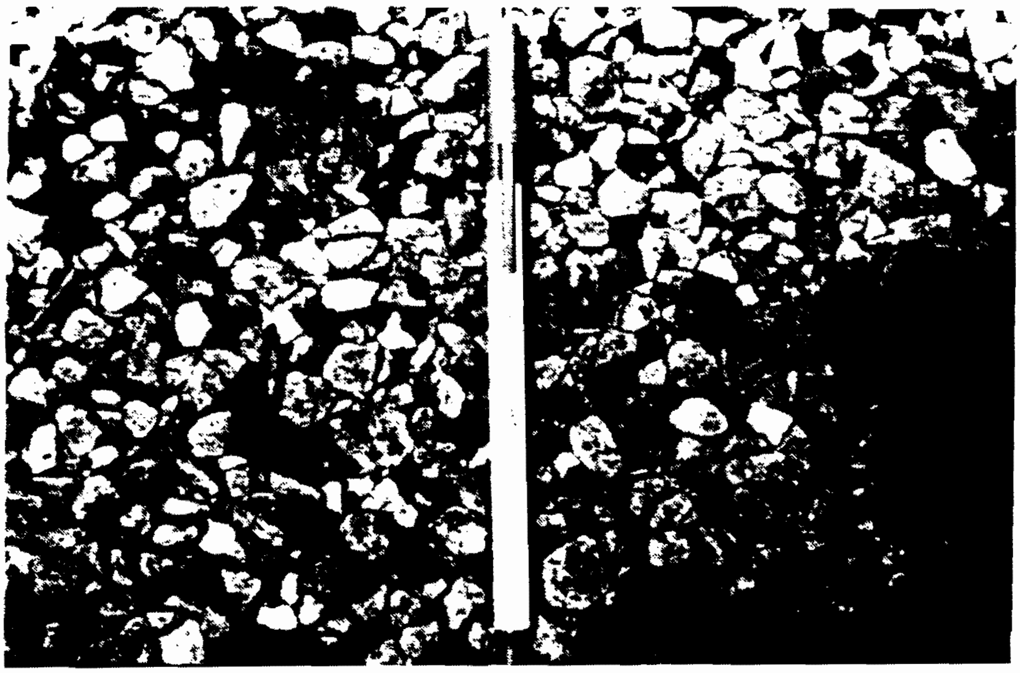


(a) AGSR = 70 sq.yd/cu.yd

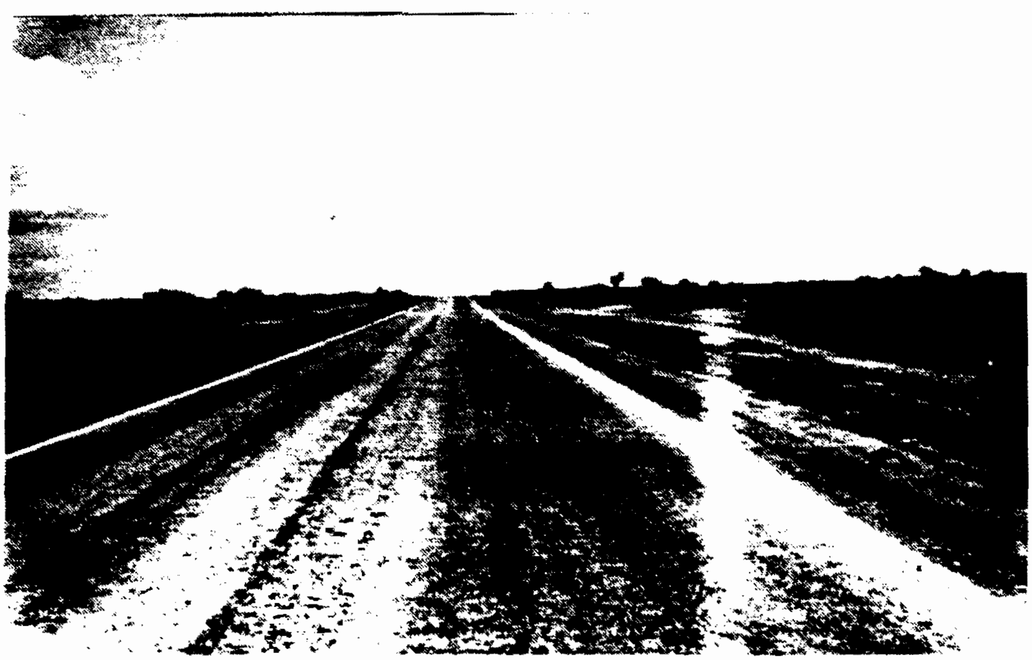


(b) bleeding and aggregate loss distresses

Fig. 7.87 Condition of section 35



(a) AGSR = 120 sq.yd/cu.yd



(b) no bleeding distress

Fig. 7.88 Condition of section 52

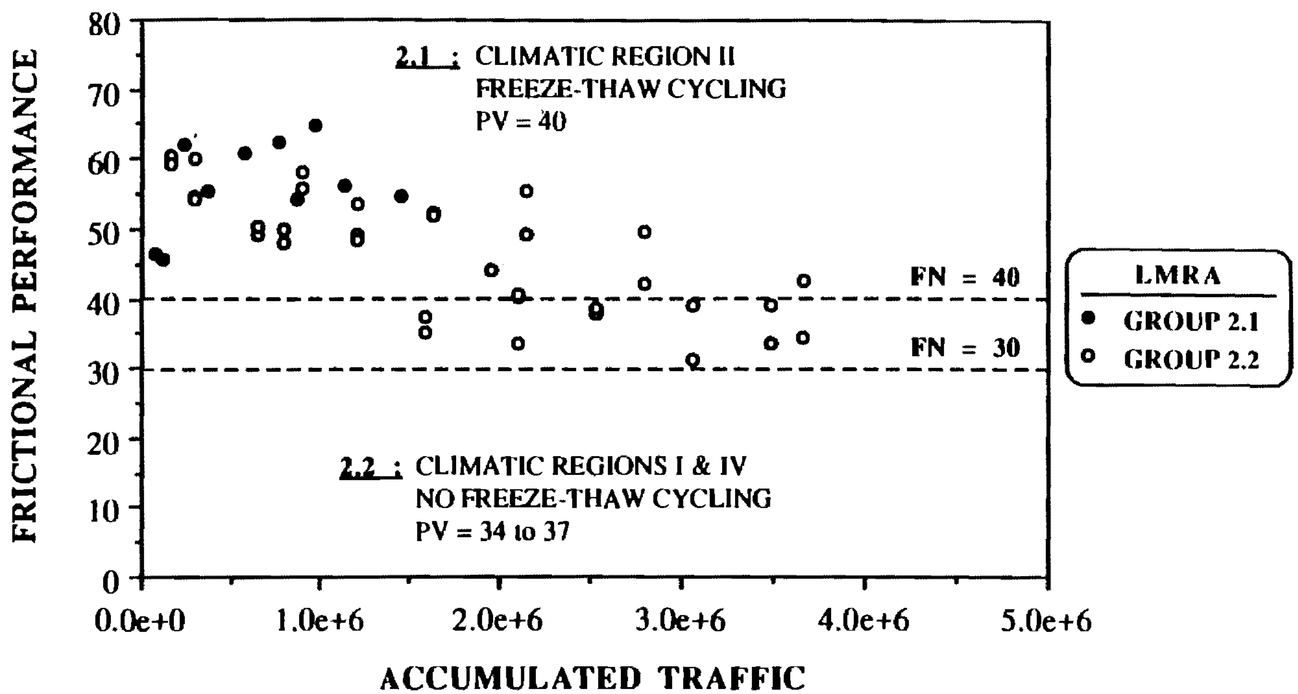


Fig. 7.89 Frictional Performance of Test Sections Constructed with LMRA Aggregates Having a PV in the Range of 34 to 40 -- Grouped According to Climatic Regions

amount of rainfall and temperature freeze-thaw cycling characterizing Region II. A further grouping of Group 2.2 in Fig. 7.90 revealed again the effect of AGSR on frictional performance. Group 2.2, consisting of sections 42 and 43, with an AGSR of 130 sq.yd/cu.yd showed to have maintained acceptable FNs up to about three and a half traffic passes. The other two groups with lower AGSRs withstood only about two million traffic passes before reaching the zone of minimum friction. The logarithmic relationship between performance and traffic was found to be stronger in Groups 2.2.2 and 2.2.3 than in Group 2.2.1. Different levels of bleeding were seen in all groups, but the most serious was with Group 2.2.3 consisting of sections 45 and 47. The sand patch readings in Fig. 7.91 showed to a little extent the lower macrotexture of sections 45 and 47. Figure 7.92 shows the wider spacing between aggregate particles and the less asphalt displacement in section 42 in comparison with the surface of section 15 of Group 2.2.2.

The performance of Group 3 is shown in Fig. 7.93. The data were first divided in two subgroups, 3.1 and 3.2, based on MSS and LA losses, as seen in Fig. 7.94. The data of each subgroup were found to also show a considerable variability. A further subdivision was then made that resulted in the grouping shown in Figs. 7.95 and 7.96. The only factor found to explain the variability in Group 3.1, with aggregates having low MSS and LA losses, was ADT. Group 3.1.1 consisting of sections with low traffic volume showed a higher level of performance. These sections can probably be grouped with the SDST and RHYO aggregates of Group 1. It may have been that the much higher ADT in Groups 3.1.2 and 3.1.3 resulted in more polishing and in reduced macrotextures in the sections constituting these groups. In Group 3.2, the factors contributing to the performance difference between Groups 3.2.2 and 3.2.3 were discussed in Section 7.4.4. The performance difference between Groups 3.2.1 and 3.2.3 may be attributed either to the fact that dolomite constituted a large percentage of the aggregates in Group 3.2.1 or to the higher AGSR of 120 sq.yd/cu.yd used in the sections of this group. The logarithmic fits projecting the performances of Groups 3.2.1 and 3.2.3 had R^2 of 0.28 and 0.54, respectively.

Group 4 consisted of the test sections built with aggregates of the poorest polish qualities. As seen in Fig. 7.97, most of its friction data fell within the zone of minimum friction. The data were subgrouped in Figs. 7.98 mainly according to climatic regions. Group 4.1 had the best performance due to the low LA loss

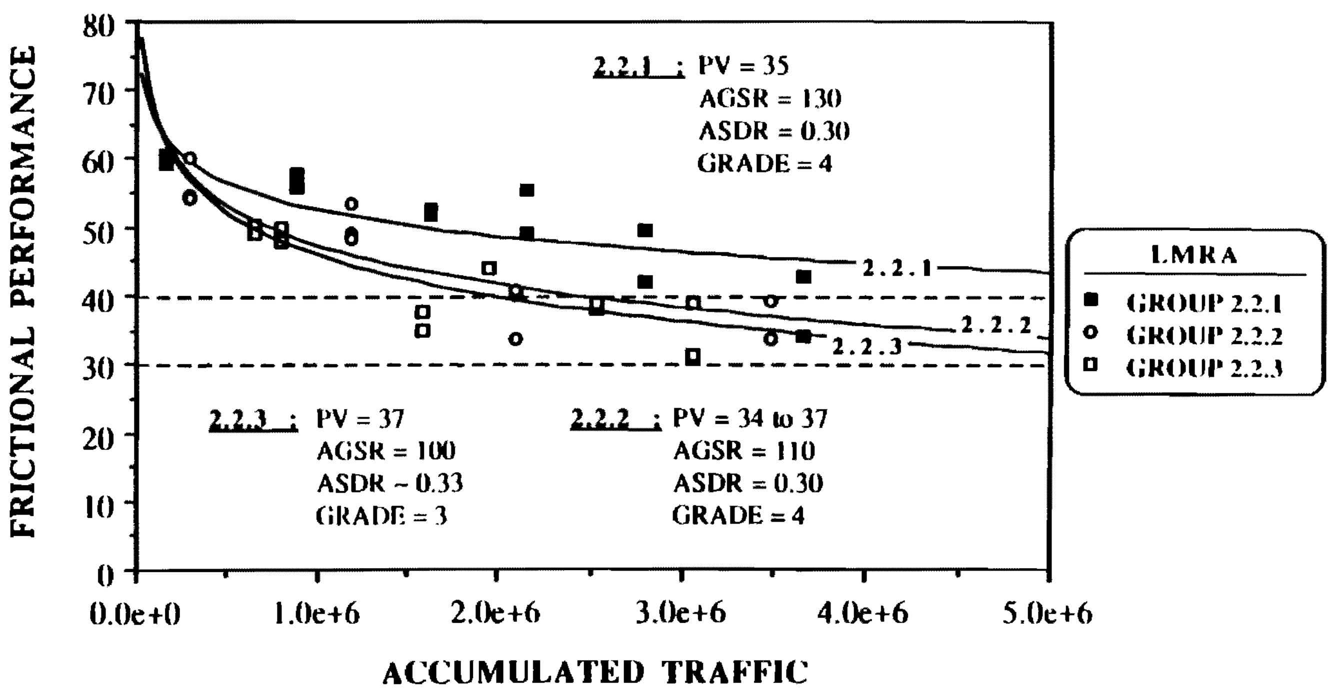


Fig. 7.90 Frictional Performance of Test Sections Constructed with LMRA Aggregates Having a PV in the Range of 34 to 37 -- Grouped According to Construction Design Variables

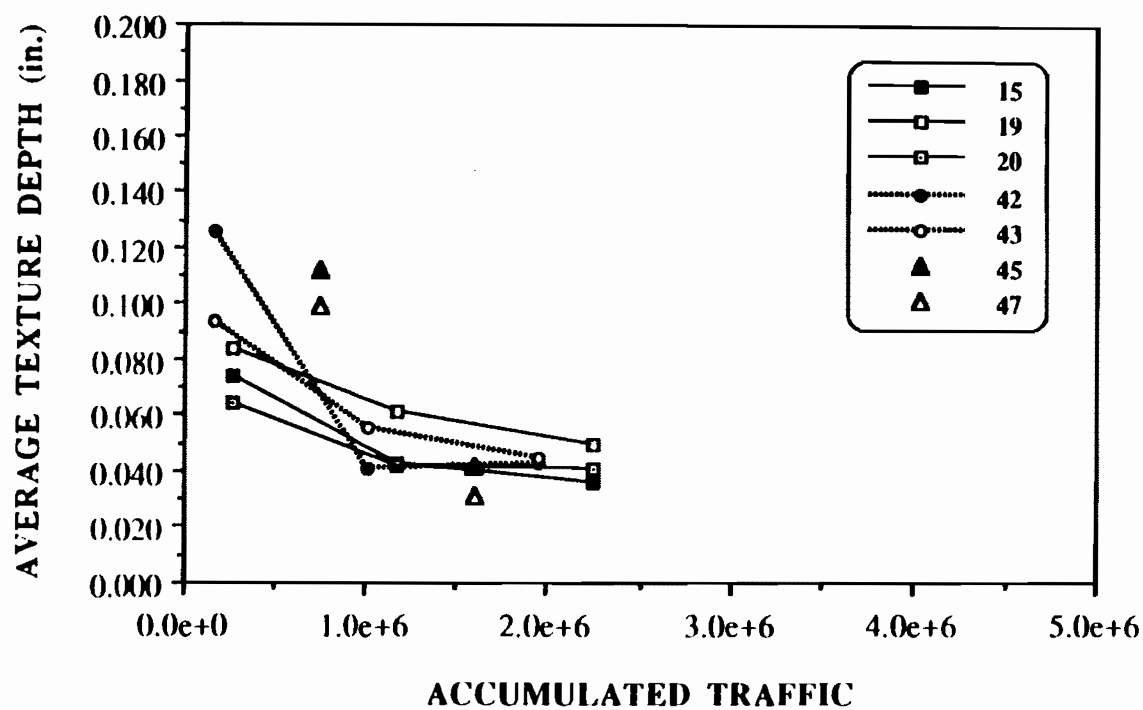
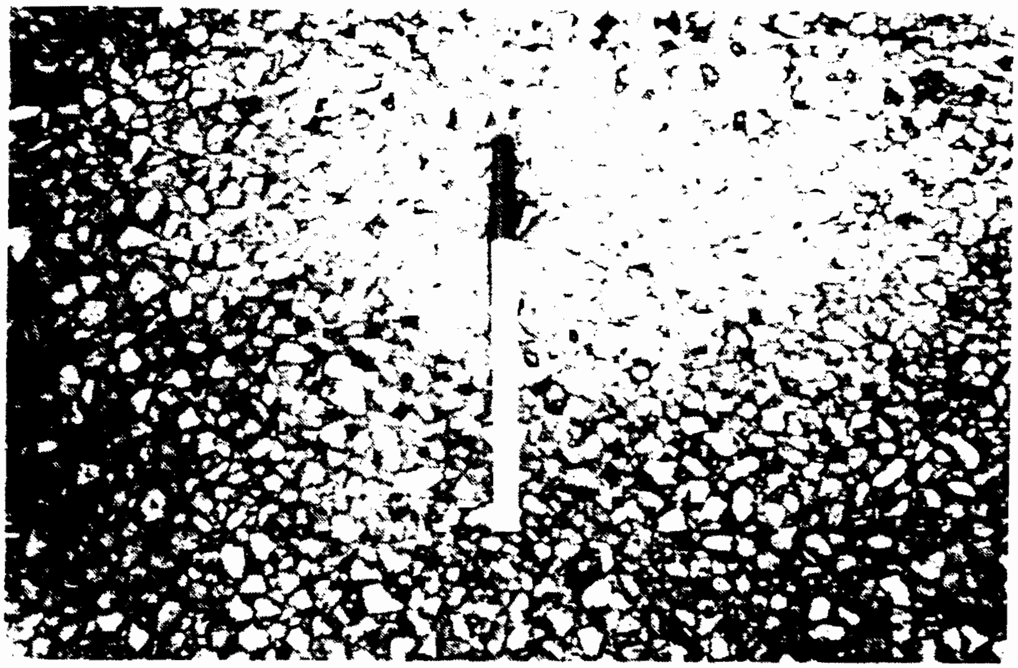
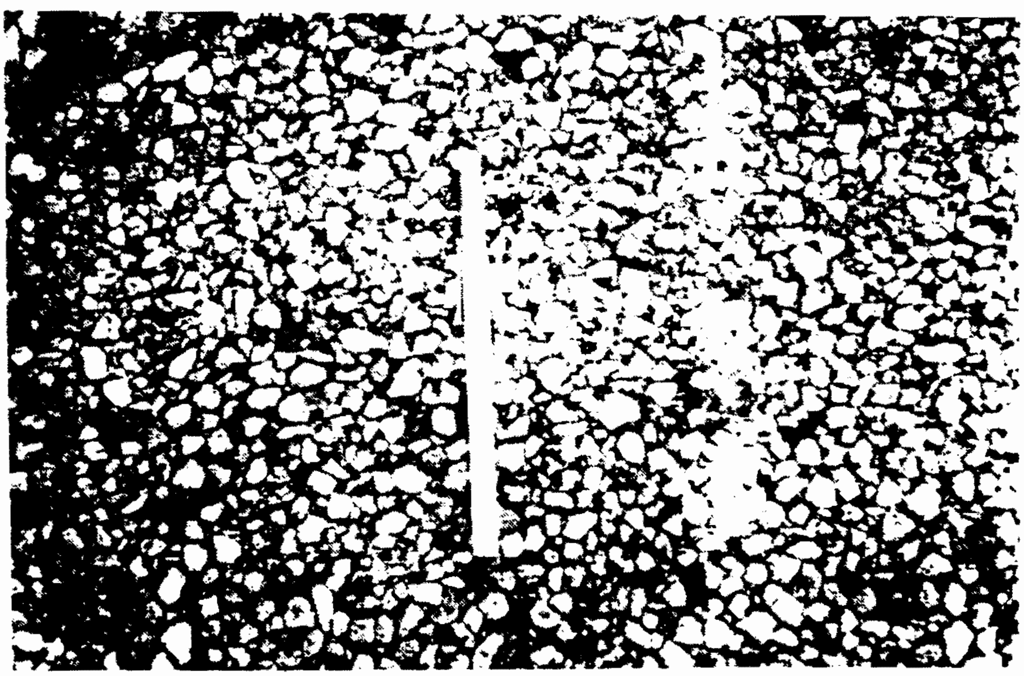


Fig. 7.91 Sand Patch Readings Taken from Sections 15, 19, 20, 42, 43, 45, and 47



(a) AGSR = 110 sq.yd/cu.yd in Section 15



(b) AGSR = 130 sq.yd/cu.yd in Section 42

Fig. 7.92 Different AGSRs in Sections 15 and 42

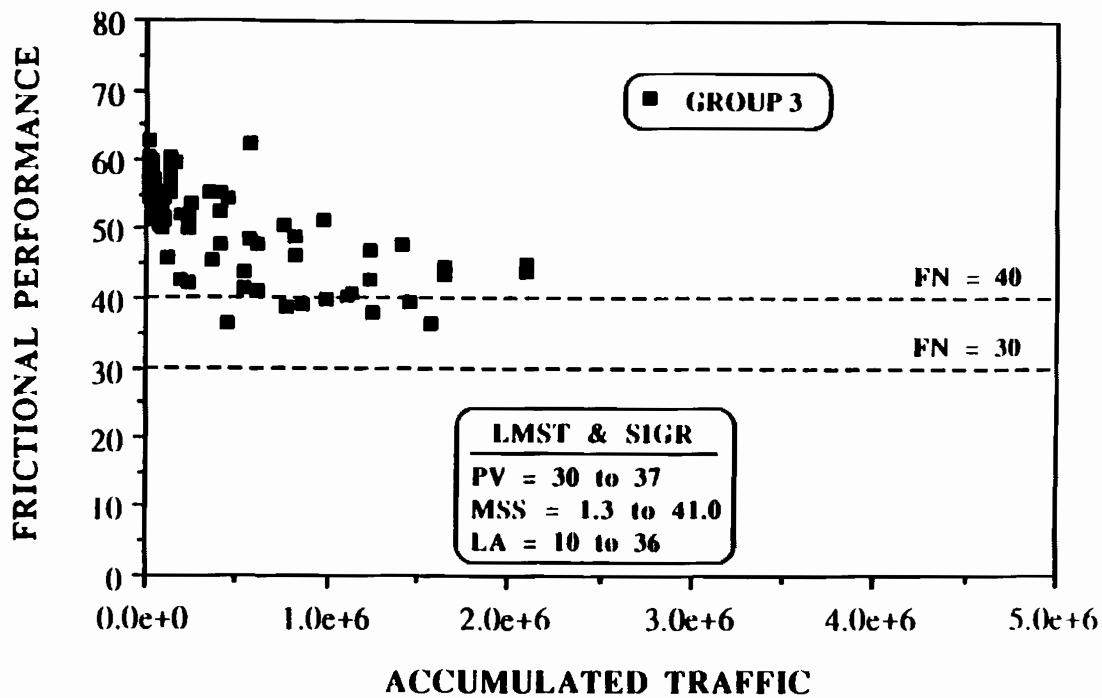


Fig. 7.93 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 30 to 37 and a Wide Range of MSS and LA Losses

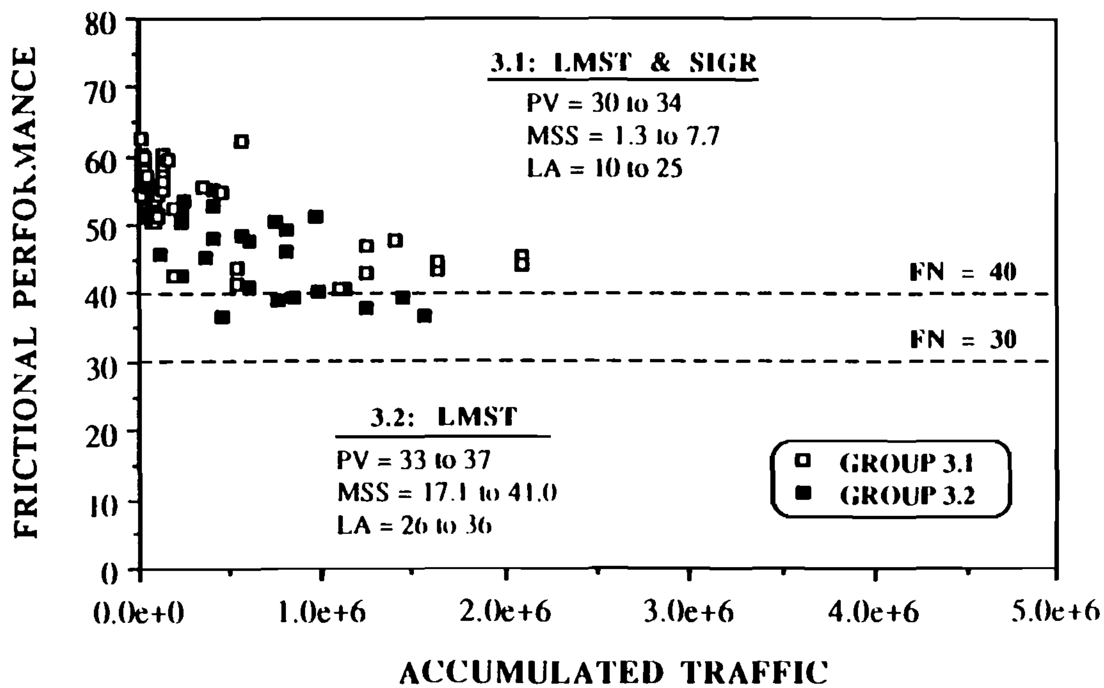


Fig. 7.94 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 30 to 37 -- Grouped According to MSS and LA Losses

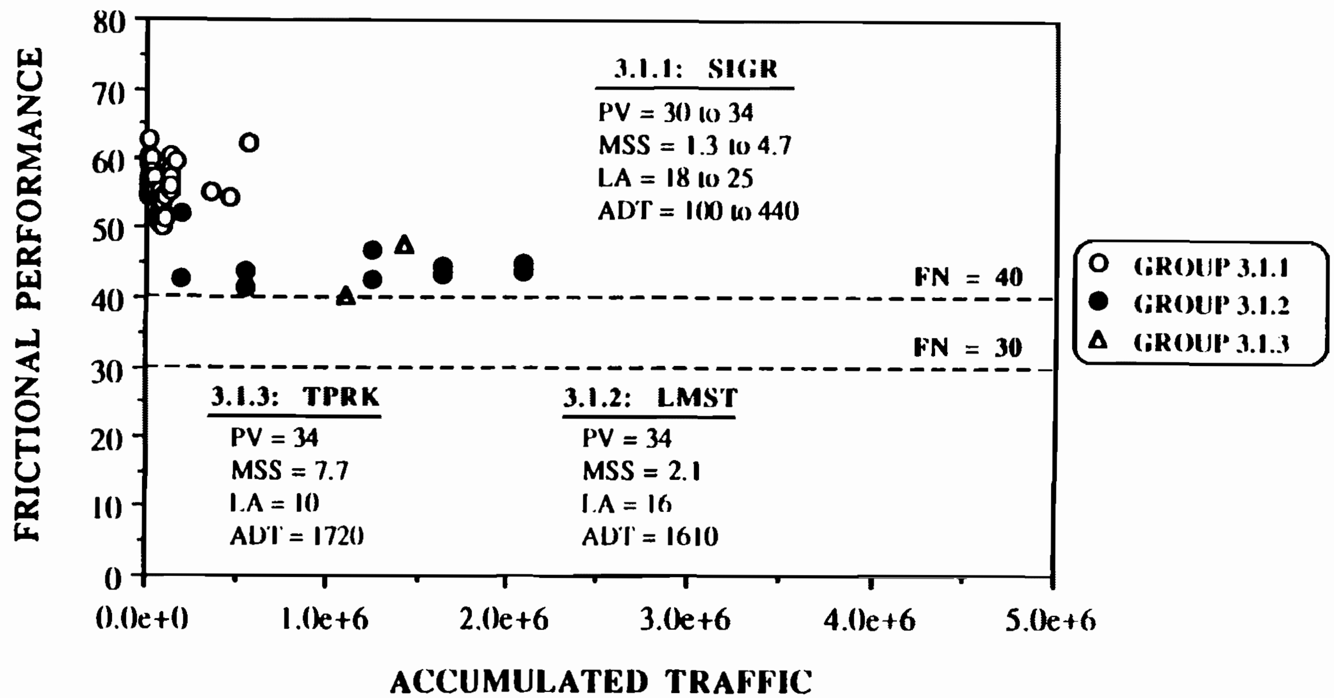


Fig. 7.95 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 30 to 34 and Relatively Low MSS and LA Losses -- Grouped According to AGMT and ADT

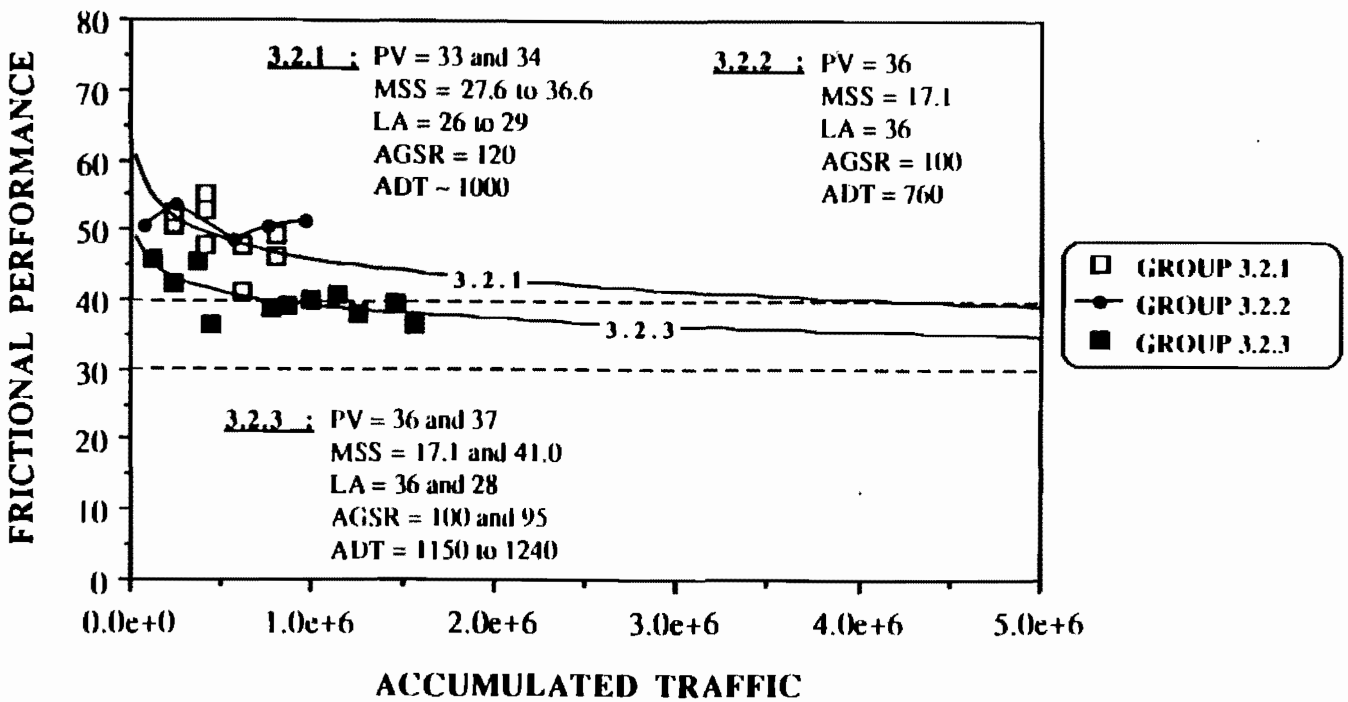


Fig. 7.96 Frictional Performance of Test Sections Constructed with LMST Aggregates Having a PV in the Range of 33 to 37 and High MSS and LA Losses -- Grouped According to AGSR and ADT

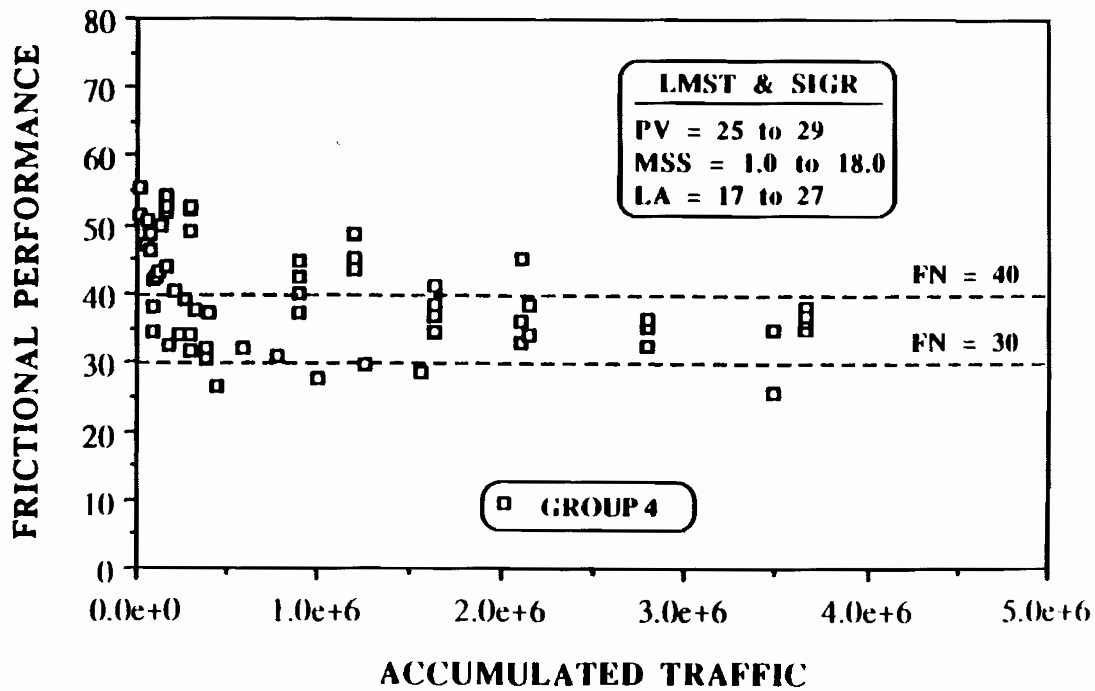


Fig. 7.97 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 25 to 29

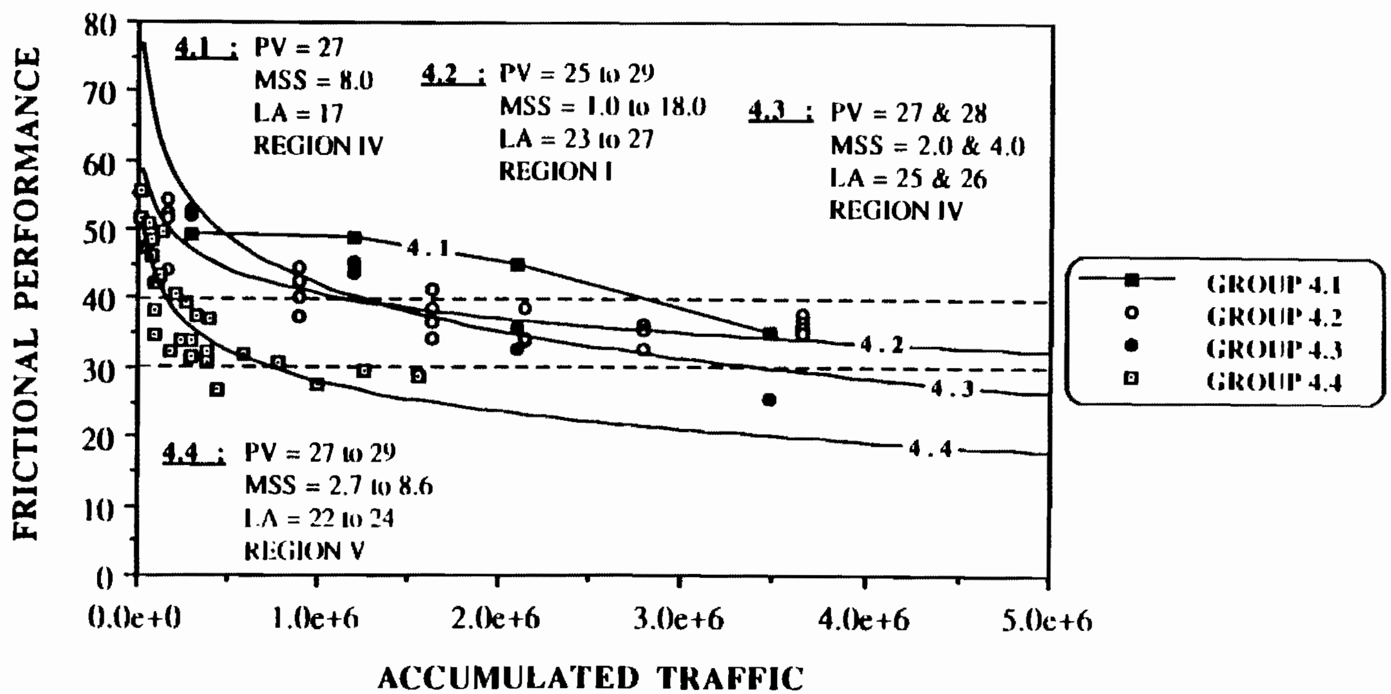


Fig. 7.98 Frictional Performance of Test Sections Constructed with Aggregates Having a PV in the Range of 25 to 29 -- Grouped According to Laboratory Properties and Climatic Regions

experienced by the cherty aggregate of that group. Groups 4.2 and 4.3 of Regions I and IV performed much better than Group 4.4 of Region V. If this performance difference was truly due to climatic regions, the relevant weather components would be moisture and temperature. That is because, in general, chemical decay of minerals is fostered by warm, moist climates (3). The test sections of Group 4.2 are located about 40 miles away from the Gulf of Mexico while the sections of Group 4.3 were located on the gulf coast itself. Region V, where the test sections of Group 4.4 are located, is characterized as dry with temperature freeze-thaw cycling.

The AGSR for Group 4.2 was 130 sq.yd/cu.yd while it was 110 sq.yd/cu.yd for Group 4.3. This factor might have caused the slight performance difference observed between these two groups. The AGSR of sections 1 and 2 of Group 4.4 was 125 sq.yd/cu.yd, eliminating the possibility that this construction factor had an effect on the performance of Group 4.4.

Figure 7.99 illustrates the similarly in the performances of Group 3.2.3 and Group 4.2, indicating clearly that the PV by itself is not an indication of the level of field frictional performance.

7.6 Grouping of Friction Data According to Mineralogical and Petrographic Properties

In this section, the friction data of the test sections and replicates built with the twenty aggregates examined petrographically were used. With the background earned from the previous section in mind, the data were grouped according to the major variables describing aggregates texture and mineralogy.

First, the data of the twenty aggregates were divided into two groups based on percentage of particles with a grain-supported texture, as shown in Fig. 7.100. It was found that most of the aggregates with more than 50 percent GST particles had performance levels above the zone of minimum friction. The data of the group with less than 50 percent GST particles were widely scattered, but generally showing a decrease in frictional resistance. A further grouping according to hard mineral content revealed more explanations of the variations encountered. In Fig. 7.101, the aggregates with more than 50 percent GST particles but having less than 10 percent hard minerals were shown to have experienced a decrease in frictional resistance.

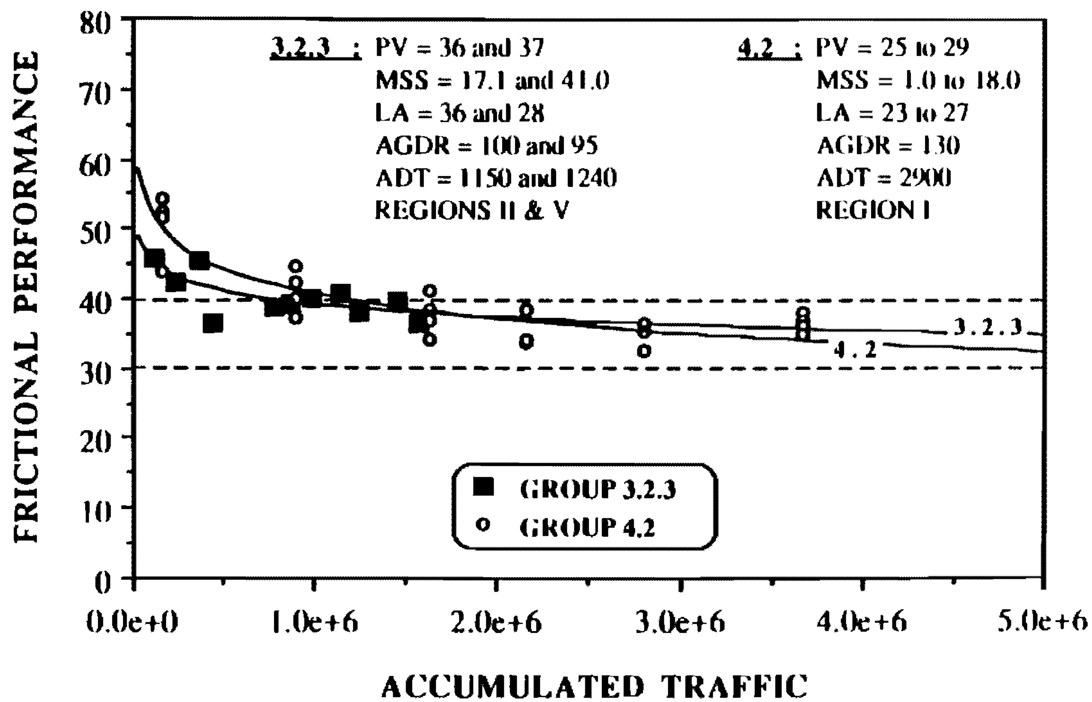


Fig 7.99 Similarities in the Performances of Two Subgroups of Test Sections with Different Aggregate Laboratory Properties, AGDR, and ADT

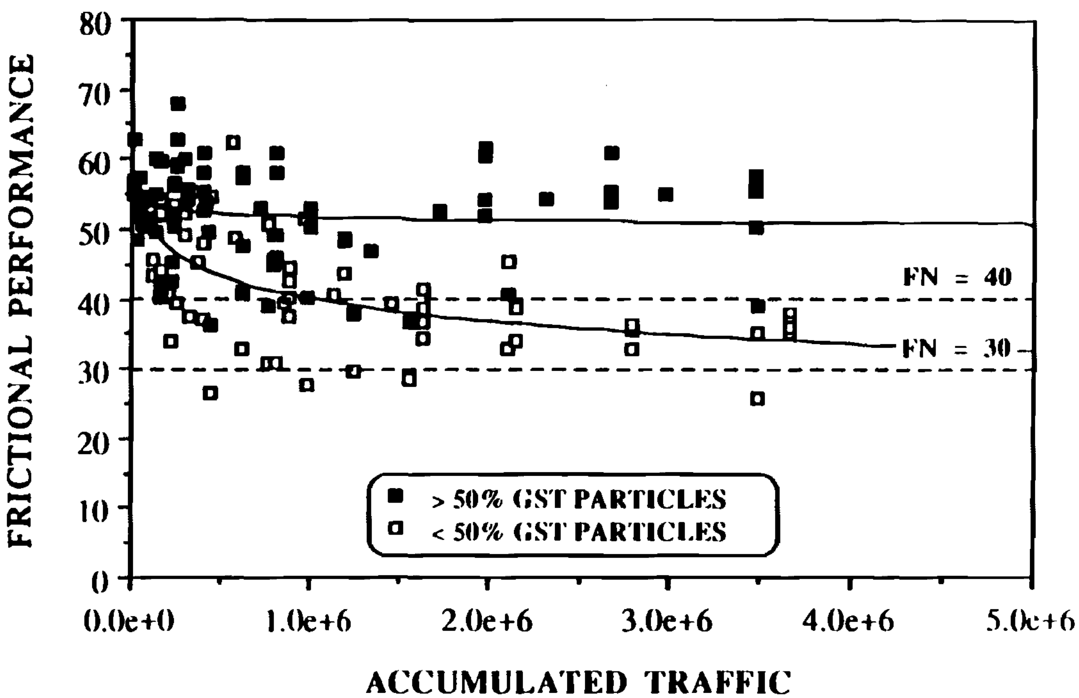


Fig. 7.100 Frictional Performance of the Aggregates Considered in the Petrographic Examination -- Grouped According to their Textural Classification

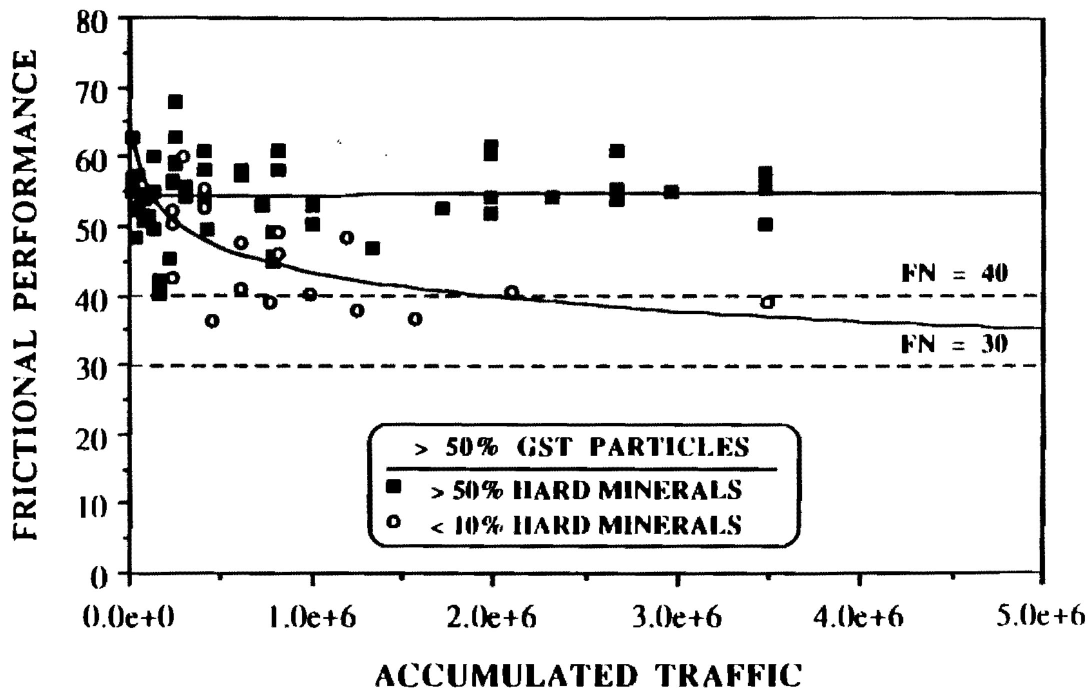


Fig. 7.101 Frictional Performance of the Aggregates with more than 50% GST Particles -- Grouped According to Percentage of Hard Minerals

In Fig. 7.102, the aggregates with less than 50 percent GST particles but having more than 90 percent hard minerals were the top of that data.

Second, the data of the twenty aggregates were grouped according to percentage of hard minerals, as shown in Fig. 7.103. The group with less than 30 percent hard minerals, which showed a general decrease in performance, was then divided in Fig. 7.104 into two subgroups according to percentage of GST particles. It was found that aggregates with more than 30 percent GST particles varied very widely in performance. As will be seen in the last figure of this section, this variation is largely explained by the ADT, void content, and the mineralogy of the grains.

Third, the friction data of the SIGR aggregates were examined in Fig. 7.105, in view of the large variability encountered in it. Aggregates 6, 39, and 16, had either zero or very little void content. Their levels of performance seemed to be stacked according to the amount of non-carbonate content. Aggregate 22, consisting largely of the same chert mineral found in aggregate 16 but with 5 percent voids, performed better than aggregate 16. The 35 percent GST of SDST material in aggregate 22 could have also contributed to the better performance.

Lastly, all aggregates with a considerable amount of carbonate minerals were grouped according to their petrographic properties. The grouping is summarized in Fig. 7.106 where two major observation were made. First, the level performance of aggregates with a large amount of MST particles was found to have improved as the percentage of voids increased. Second, the level of performance of aggregates with a considerable amount of GST particles was observed to be associated with the level of traffic, and the hardness of the grains. The dolomitic grains gave a level of performance higher than that given by the calcite grain. Aggregates with GST particles consisting of quartz grains cemented by carbonate minerals gave the highest levels of performance.

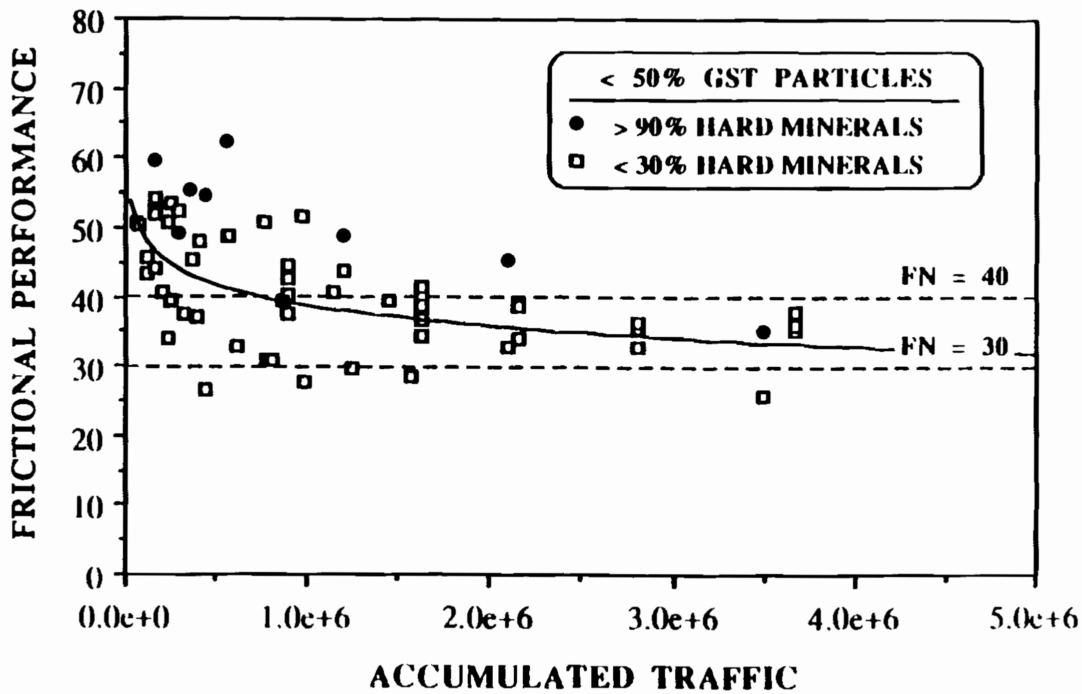


Fig. 7.102 Frictional Performance of the Aggregates with less than 50% GST Particles -- Grouped According to Percentage of Hard Minerals

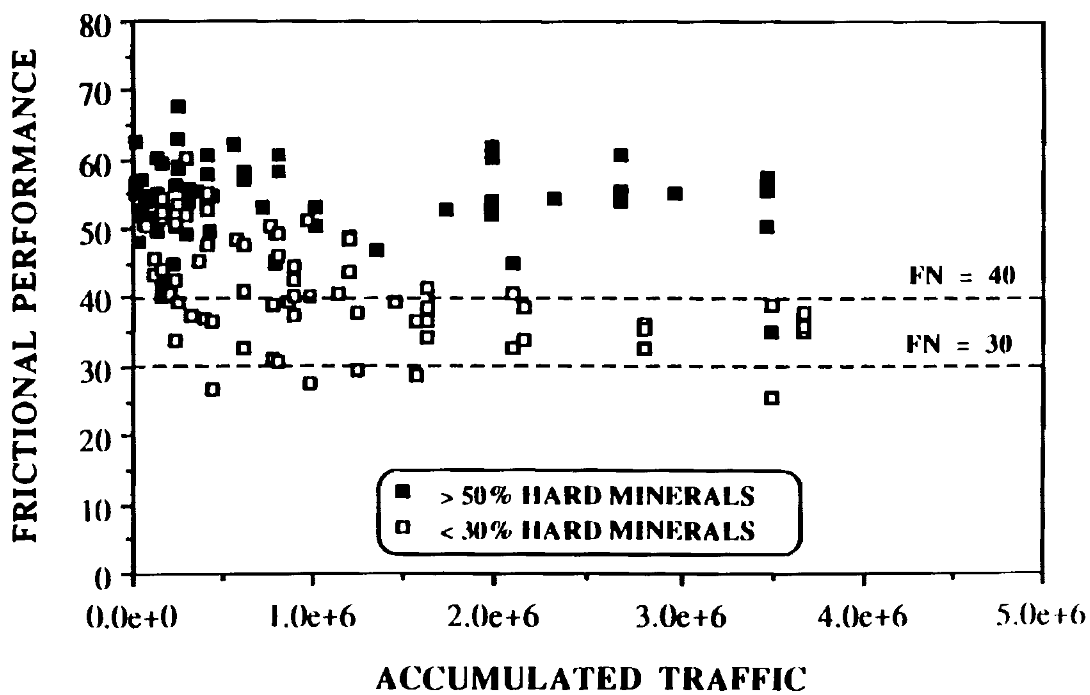


Fig. 7.103 Frictional Performance of the Aggregates Considered in the Petrographic Examination -- Grouped According to Percentage of Hard Minerals

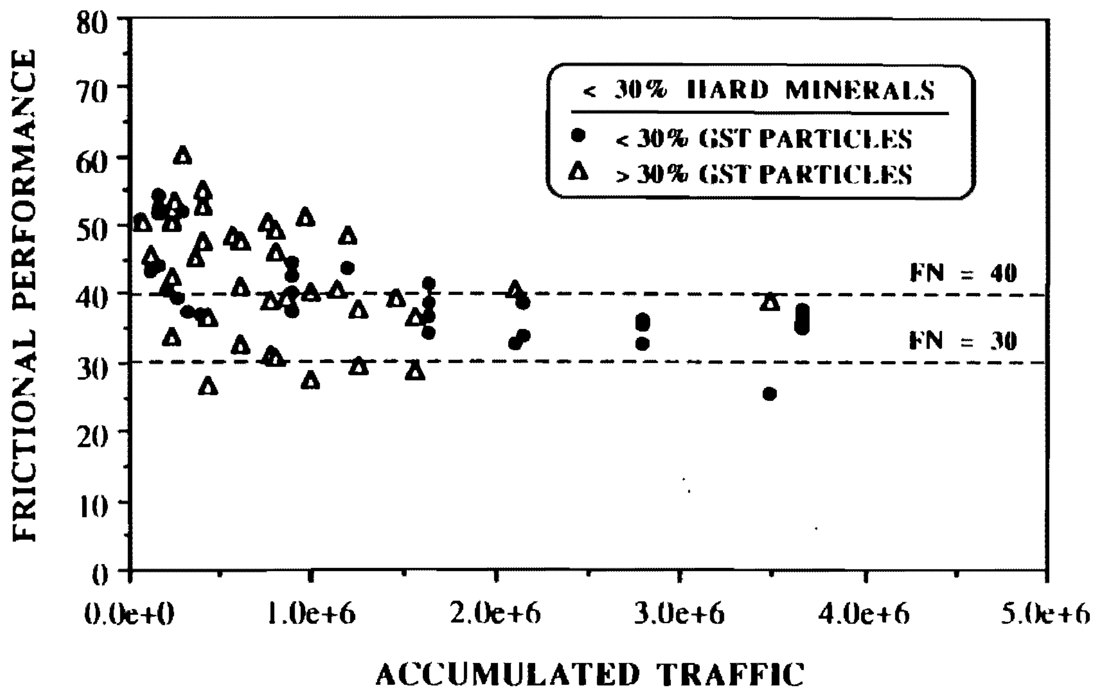


Fig. 7.104 Frictional Performance of the Aggregates with less than 30% Hard Minerals -- Grouped According to their Textural Classification

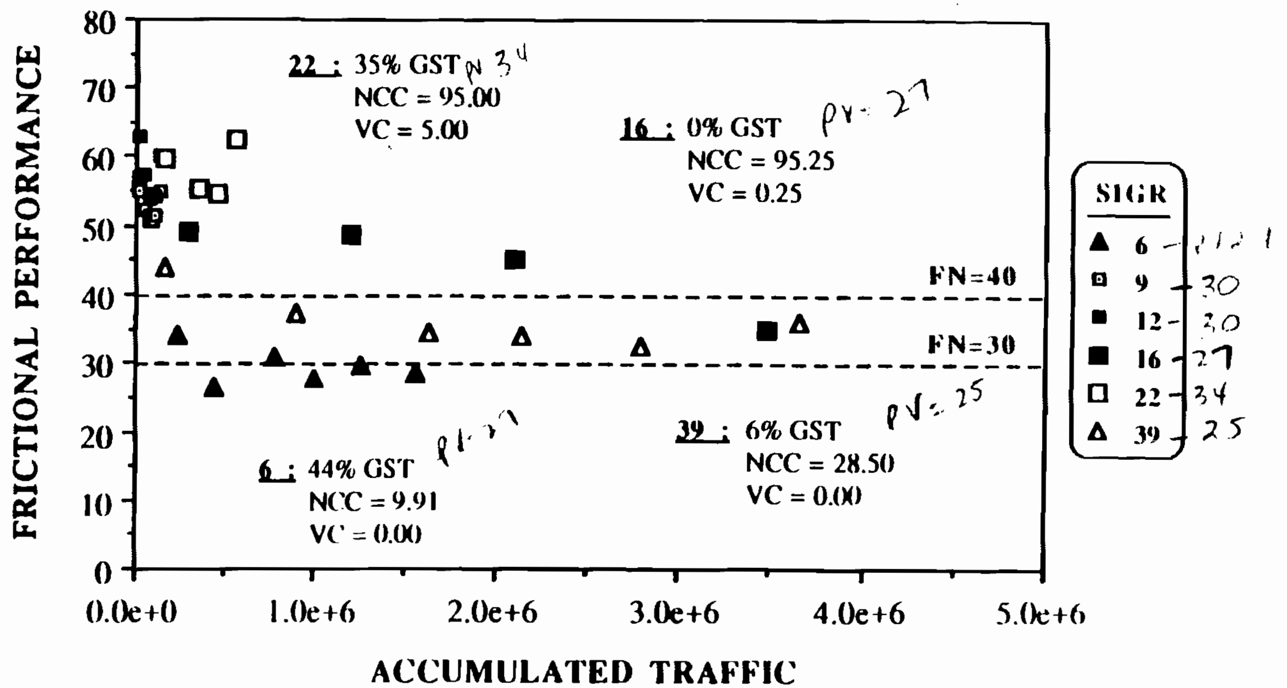


Fig. 7.105 Frictional Performances of the SIGR Aggregates Considered in the Petrographic Examination

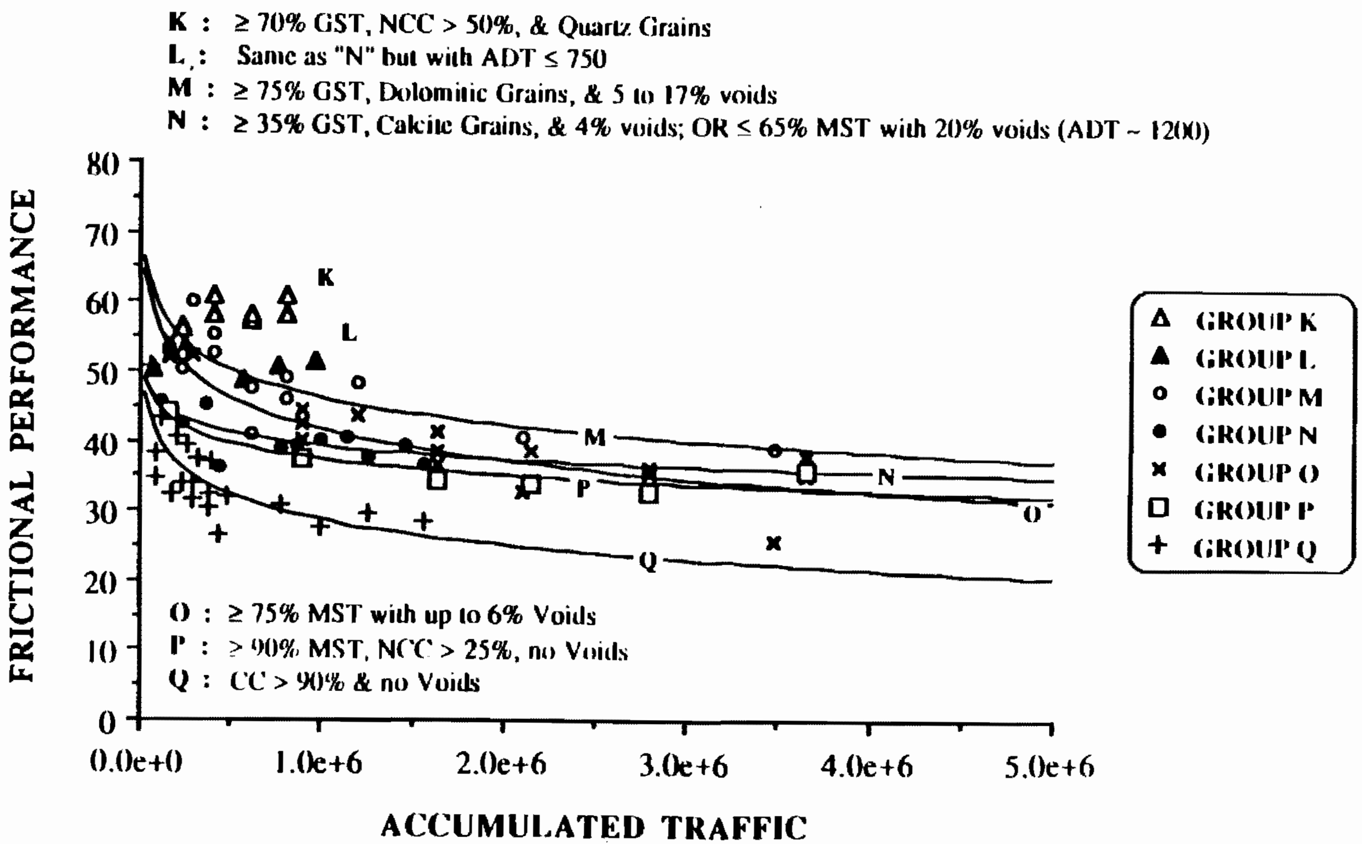


Fig. 7.106 Frictional Performance of Aggregates with Considerable Amount of Carbonate Minerals

CHAPTER 8
STATISTICAL MODELING OF SEAL COAT PERFORMANCE

8.1 Data Base and Statistical Methods

The Statistical Analysis System (SAS) program was used on the IBM mainframe to build a data base which embodied the bulk of data collected. Originally, each group of data was stored in a separate data file, and the data files were then manipulated and merged to form a data base containing all of the involved variables. The data were stored in a way that allowed the update of each data file at any time and the remerging of data files into the larger data base at any later time. The data base FCWEP, which resulted from the merging of the friction, construction, weather, engineering laboratory, and petrographic data files, was used extensively in the statistical modeling. The statistical analysis basically involved employing (1) the general linear model (GLM) procedure, suitable for unbalanced experimental designs; (2) the analysis of covariance (ANCOVA), which combines the analysis of variance (ANOVA) and regression analysis; and (3) multivariable regression analysis.

8.2 Variables Involved

There were two types of variables involved, dependent and independent. While the dependent variable, FN, was quantitative, the independent variables that contributed information for the prediction of the dependent variable were of two types: quantitative and qualitative. The former, as the name implies were those variables that could be measured. Qualitative variables could not be measured; they could only be described and thus assumed few discrete values. As will be seen subsequently, the way an independent variable was entered into a prediction equation depended on its type.

8.2.1 Dependent Variable

The friction measurement or performance response of test sections was the dependent variable -- criterion variable. Performance was measured by the FN which is quantitative in nature. Eight sets of friction measurements, spanned over about five years, have been obtained for most test sections. All of the performance measurements obtained for a particular test section and their associated accumulative traffic volumes were used each time a certain analysis was performed. This was done in order to estimate the effect of traffic on the changes in friction with time.

8.2.2 Independent Variables

A wide range of factors affecting frictional resistance of seal coats was identified for inclusion in the analysis. These factors were viewed as independent variables which were used to test the predictability of pavement friction. These were as follows:

8.2.2.1 *ADT and Accumulated Traffic.* Average daily traffic (ADT) is a quantitative variable controlling the level of aggregate embedment in the early stage of a test section life. Accumulated traffic (CUTR) was also a quantitative variable which reflected the accumulation of traffic from the day of construction until day of field testing.

8.2.2.2 *Construction Variables.* Qualitative construction variables included aggregate type and coating, percent binder, and admixture of asphalt. The distribution rate of asphalt and the spreading rate of aggregate were quantitative variables.

8.2.2.3 *Laboratory Variables.* Laboratory tests and petrographic examinations gave relative measures of the extent to which aggregates might degrade, disintegrate, wear, and polish under the exposure to traffic. Accordingly, they were thought to have a considerable influence on the significance of the prediction models being investigated.

All laboratory tests were quantitative in nature, while the majority of the petrographic variables were originally descriptive and thus subjectively evaluated. However, as discussed in Chapter 5, the petrographic evaluation was made as nearly objective as possible through the qualitatively weighted results shown in Table 5.6.

8.2.2.4 *Weather Variables.* Climatic region was a qualitative variable which could assume only four discrete values--four climatic regions. However, according to the findings in Chapter 7, only the temperature freeze-thaw division of the climatic regions was considered in the analysis. Thus, this variable was regarded as a two-level qualitative or class variable (cold and warm) in the GLM model, and was coded as a dummy variable in the ANCOVA analyses. The season of the year during which field testing was performed was also a qualitative variable that could be described as a wet or dry season. On the other hand, the length of the last rainfall period (LRP), the number of days between the last rainfall that occurred in that period and the day of field testing (DLR), and the total inches (TPP) that fell in that period were all quantitative variables.

8.3 Statistical Assumptions

There were certain assumptions that should have been approximately satisfied before the statistical analysis techniques could be employed. These assumptions were as follows (5, 51, 95):

1. The response, y, could be represented by the probabilistic model

$$y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \_\_\_\_\_ + \varepsilon$$

   in which no exact linear relationship existed between two or more of the independent variables (for multivariable regression).

2. For each value of a predictor variable, there was a normally probabilistic distribution of independent values of the criterion variable y. From each of these y distributions one or more values were sampled at random.

3. The variances of the response, y, for various experimental conditions were equivalent, a condition referred to as homoscedasticity or homogeneity of variances.

4. The error term (ε) possesses a normal probability distribution, had an expected value of zero, and was random (did not exhibit any systematic trends).

To test for these assumptions the residuals of each model were examined against the predicted FN response and the independent variables. Unusual trends in the residuals would have necessitated transformation of the variables. That is, instead of using the original measurements, their square roots, logarithms, or some other function could have been used.

8.4 Design of the Experiment for the Main Effects

The design of this experiment was aimed towards better understanding of the effects of aggregate properties, aggregate construction spreading rates, and environments on the performance of seal coat overlays. These variables were considered in this design after they had been found to be helpful in explaining the scatter in the friction data as it was shown in Chapter 7. As illustrated in the Table 8.1, the design included the different aggregate groups considered (AGGR) and the ranges of the results of the four laboratory tests obtained for most of the aggregates. Two siliceous gravel (SIGR) aggregates with high percentages of carbonate contents were grouped with the limestone (LMST) aggregates of group Limestone 2 (E). Since the obtained asphalt and aggregate spreading rates were the designed rates included in the specifications, and, therefore, to account for the slight expected variations in the actual construction rates, the aggregate spreading rate variable was introduced to this design

Table 8.1 Design of the Experiment for the Main Effects

AGGREGATE GROUPS (AGGR)	AGGREGATE PROPERTIES	LEVELS OF CONSTRUCTION (AGSR)	REGION (RGT)	
			II & V (COLD)	I & IV (WARM)
LIGHTWEIGHT (A)	PV = 48 to 51 FRTH = 3.00 to 9.53 LA = 18 to 25	70 (LOW)	Group B.1.1 (1)	---
		95 to 110 (MED)	Group B.1.2 (3)	Group A.2 (2)
		120 to 130 (HIG)	---	Group A.1 (4)
SANDSTONE AND RHYOLITE (B)	PV = 36 to 41 MSS = 0.0 to 15.0 LA = 15 to 28 INRD=55.00 to 100.00	70	Group 1.3 (5)	---
		95 to 110	Group 1.2 (6)	---
		120 to 130	Group 1.1 (7)	---
LIMESTONE ROCK ASPHALT (C)	PV = 34 to 40 MSS = 7.0 to 14.0 LA = 28 to 34 INRD=1.28 to 14.71	70	---	---
		95 to 110	Group 2.1 (8)	Groups 2.2.2 & 2.2.3 (9)
		120 to 130	---	Group 2.2.1 (10)
LIMESTONE 1 (D)	PV = 33 to 37 MSS = 17.1 to 41.0 LA = 26 to 36 INRD=0.41 to 4.24	70	---	---
		95 to 110	Group 3.2.3 (11)	---
		120 to 130	Group 3.2.1 (12)	---
LIMESTONE 2 (E)	PV = 25 to 29 MSS = 2.7 to 8.6 LA = 22 to 24 INRD=0.45 to 20.23	70	---	---
		95 to 110	Sections 1&2 of Groups 4.4 (13)	Group 4.3 (15)
		120 to 130	Sections 6&10 of Groups 4.4 (14)	Group 4.2 (16)

as a three-level variable (LAGSR), the levels being low, medium, and high. The design also included the two classes of regions (RGT), cold and warm. The groups of sections with similar performance patterns, shown graphically in Chapter 7, were placed in the appropriate cells of the design. This design represented about three-fourths the total number of friction observations available. Because of the different number of observations in each cell and because of the presence of empty cells, the design was regarded as unbalanced. Adding more levels of the considered variables would have only augmented the problems that are usually encountered with such designs. Shown between parentheses are the abbreviations used for these variables in the statistical analyses.

Because of the unbalance of this design, the GLM procedure was used for testing the significance of the main three variables. The first model generated had the FN as the dependent variable and the three variables, shown in the above design, and their interaction terms as the main predictors. In addition, accumulated traffic, referred to as CUTR in this chapter, and ADT levels were used as covariates in the model. Since the performances of some of the groups were seen to have exhibited logarithmic relationships with traffic, the natural logarithmic function of the CUTR (LGCUTR) was used in this analysis. This model, based on 355 observations, had the following form:

$$\begin{aligned} FN \quad = \quad & AGGR + (AGGR * LGCUTR) + (AGGR * RGT) + \\ & (LGCUTR * AGGR * RGT) + LAGSR + (AGGR * LAGSR) + \\ & ADT + (ADT * AGGR) \end{aligned}$$

The results are shown in Table D.1 of Appendix D. The model had an excellent R^2 of about 0.84 with each of the considered term showing significance at the individual level using Type III sum of squares and their associated F values and probabilities. The R^2 increased very slightly when the DLR weather variable was included in spite of the fact the variable itself showed significance at the individual level. However, since this variable does not serve any design purposes, it was decided to omit it from the model. The GLM solution indicated that aggregate groups A and B had friction means that were higher than those of group C, D, and E. It also indicated that groups A, B, and D had different slopes (with traffic) compared to those of groups C and E. The effect of region was seen to be significant in group A and C, particularly in interaction with traffic. In addition, the LAGSR variable was observed to explain some of the variation in groups A, B, C, and D. Lastly, the ADT effects was shown to be significant in groups B, C, and D. The scatter of the residuals against the predicted response was seen to be normal requiring no transformation of the response be done.

The letter B printed following each estimate in the solution was to indicate that the estimates were biased. To make sure that the bias was not wholly the results of empty cells being present in the design (as those affect the number of degrees of freedom (DF) of the interaction terms), another general linear model was generated in which the groups filling sixteen of the cells were considered as sixteen levels of a one-way ANOVA. In order to avoid overspecification in the model, the ADT variable was left out after having observed the small reduction in R^2 which resulted from this action (R^2 decreased to about 0.825). The second model had the form of:

$$FN = LGCUTR + GROUP + (LGCUTR * GROUP)$$

and an R^2 of about 0.825, the same as that of the first model after dropping ADT. The letter B following the estimates in the solution of the second model was a clear indication that the bias was primarily the result of unequal numbers of observations in the filled cells.

To get the prediction equation, a regression analysis was performed on the sixteen groups. The resulting estimates for the groups and their interactions with LGCUTR are summarized in Table D.3. Since the GROUP variable was dummy coded, the estimates of the intercept and LGCUTR applies directly to group 16. As can be seen, the intercept and slope estimates of group 10 through 14 were shown not to be statistically significant from that of group 16. This affirmed the observation made from Fig. 7.99 where a similarity in performance was depicted by groups 3.2.3 and 4.2, equivalent to groups 11 and 16 of the prediction model, which had PVs of about 36 and 27, respectively. It should be emphasized at this stage that although the AGGR variable was found along with the other two to be statistically significant in explaining the variation in frictional response, the listing of the ranges of aggregate laboratory properties in Table 8.1 was not intended for purposes of making speculations as to which test or a group of tests contributed to the significance of the models. The laboratory tests will be dealt with individually or in combination or interaction with other tests and variables in the regression models presented in the following section.

8.5 Multivariable Regression Analysis

In general, this analysis was intended to find the best general linear regression models of the type

$$FN = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \_\_\_\_ + \beta_k X_k + \varepsilon$$

to describe the relationship between the frictional performance of seal coat surfaces (FN) and all of the independent variables involved (X_1, X_2, _ _ _, X_k). Since an explanation of causal effects of each individual independent variable was the primary thrust of this investigation, the stepwise regression procedure, an available option in the SAS program, was used for screening the variables to be included in each of the different prediction models considered. The stepwise regression procedure was a selection and an elimination algorithm which allowed for a two-stage review of variables at each step of the analysis. That is, at the first step, a simple linear model was formed by regressing the frictional performance against the variable with which it is most correlated. Then, the explanatory ability of each remaining variable was examined (through its partial F statistic), and a new model was formed by adding the variables that possess the greatest marginal explanatory ability. In addition to the selection of a new variable to enter into the model, there was a reexamination of all variables previously entered. If the partial F test value of any variable within the model is found to be insignificant, the associated variable was removed from the analysis. This procedure was continued until all remaining variables were judged to be insignificant in their ability to contribute information for the prediction of the frictional performance, that is, where all partial F statistics for variables not yet entered into the model were small.

A qualitative independent variable, such as region (RGT), was entered into the model by using dummy variables, with the number of dummy variables always being one less than the number of levels (categories) associated with it. For example, if the frictional performance depended on the variable RGT, the first two terms of the model would be

$$y = \beta_0 + \beta_1 RGT + \text{———} + \varepsilon$$

where

RGT = 1 if the response was for the cold region or RGT = 0 if not.

Multicollinearity was a problem that arose when two or more of the independent variables were found to be highly correlated with each other. When such a problem was encountered, the respective individual contribution of the correlated variables to the reduction in the error sum of squares could not be determined. Therefore, the contribution of information by a particular independent variable to the prediction of FN depended on the other independent variables included in the model. If two variables contributed overlapping information, the first β_1 parameter might have been overestimated while β_2 tended to be underestimated (82). In fact, multicollinearity could have even caused the algebric sign of one or more regression parameter estimates to be contrary to logic. Thus, if a multicollinearity problem was faced, the explanation of the causal effects of individual variables on the frictional performance was undertaken with great caution.

To tell whether multicollinearity was causing problems, the standard errors of the coefficients were examined. If several coefficients had high standard errors and dropping one or more variables from the equation lowered the standard errors of the remaining variables, multicollinearity was usually the source of the problem (95). In addition, the COLLIN option of the SAS regression procedure was always used which readily pointed to the variables causing multicollinearity problems. The approach to dealing with multicollinearity, found applicable to the types of variables involved, was to drop one of the two correlated variables from the equation and to reestimate it. This could have caused bias in the reestimated model, but it might be justified if the bias could be argued to be small.

The term R^2, the multiple coefficient of correlation, provided a measure of the fit of the multivariable regression models (82). That is, R^2 gave the proportion of the total sum of squares that was explained by the predictor variables. The remainder could be explained by the omission of important information-contributing variables from the model and experimental error. A small value of R^2 meant that the predictor variables contributed very little information for the prediction of frictional performance; a high value of R^2 meant that the predictor variables provided most of the information necessary for the prediction of frictional performance.

Interaction terms were considered in the analysis to evaluate their ability to contribute information for the prediction of frictional performance, and residual analysis was performed in an effort to identify new variables that could have improved the fit of a model. This analysis involved plotting the residuals against each independent variable, and the pattern of the variation in the residuals was examined. Systematic patterns would have prompted the inclusion of a certain function of a predictor variable. As in the case of the general linear model, the residuals were also plotted against the predicted FNs to see if transformation of the measured FNs was needed.

The regression analyses were performed on three sets of data. First, all friction measurements were used to formulate a general multivariable regression model. Since the SAS program excludes friction observations associated with missing values of independent variables, only the polish value (PV), soundness (MSS), Los Angeles abrasion (LA), and insoluble residue (INRD) tests along with other types of variables were considered for inclusion in this model. Each aggregate material was then dealt with individually, and a prediction model was generated that described the performance of that material and the significant variables. Second, the friction data associated with the uncoated aggregates were handled separately in order that the rest of the laboratory tests could be considered for inclusion in the model. Lastly, the friction data associated with the aggregates that were petrographically examined were used in testing for the significance of the petrographic variables in explaining the performance variation. Sections 21, 50, and 58, and the south bound of sections 29 and 30 were not included in the analysis because of the untypical performance variations observed in them.

8.5.1 Statistical Modeling Using All Friction Measurements

8.5.1.1 *General Multivariable Regression Model.* In this model the PV, LA, MSS, and INRD tests along with the interactions among them and their interaction with traffic were included in the stepwise run. Other construction and weather variables were also considered. Because the lightweight (LTWT) aggregates had missing values for the INRD test, they were automatically not considered in this model. The stepwise procedure narrowed down the number of variables to about eight, among which multicollinearity problems were evident. After eliminating the trouble variables, the prediction model, based on 335 observations, constituted the following variables:

$$FN = 47.46 - 2.976\,(LGCUTR) + 0.02\,(PV * LA) + 3.6 \times 10^{-4}\,(PV * INRD * LGCUTR) + 0.146\,(AGSR)$$

The model had an adjusted R^2 of about 0.55 and a significant F value. All the variables constituting the model were shown to be significant at the individual level. The MSS test was not part of the model because it failed to discriminate between the sandstone (SDST) and the sound LMST aggregates. The predicted FN is found to generally decrease with LGCUTR, but the intercept of the prediction curve increases as either PV or LA increases. In addition, the slope of the curve shifts upward for those aggregate with high PVs and high INRDs. The intercept also increases as the AGSR increases. A summary of the regression solution is presented in Table D.4.

8.5.1.2 *Prediction Model for the Limestone Rock Asphalt Aggregates.* Table D.5 shows the strong prediction model obtained for this group of aggregates (adjusted $R^2 = 0.83$). Since the MSS and LA values were similar for all of the LMRA aggregates, it was expected that only the PV and INRD might show up in the model. Instead, the performance of these aggregates was found to be dependent on the CUTR, the interaction between CUTR and region, and the AGSR construction variable. Because of the different trends observed in the FN response of these aggregates, the natural logarithmic function of FN (LGFN) was used in this model in order to linearize it. The performance was better explained by CUTR as opposed to LGCUTR because the logarithmic fit was not as strong as the linear fit. The prediction model, based on 55 observations, has the following form:

$$LGFN = 3.17 - 1.39 \times 10^{-7}\,(CUTR) + 2.13 \times 10^{-7}\,(REGION * CUTR) + 7.98 \times 10^{-3}\,(AGSR)$$

8.5.1.3 *Prediction Model for the Limestone Aggregates.* An overspecified model, presented in Table D.6, which included many variables and had an adjusted R^2 of about 0.74 was generated for these aggregates. All of the constituting variables were shown to be significant. However, a major multicollinearity problem was seen in this model due to many of the variables being highly correlated. Before correcting for this problem, the FN was found to decrease with traffic (LGCUTR), and this relationship was found to shift downward for the cold region. Also in this region, an increase in PV was seen to positively change that slope. An increase in MSS was observed to positively affect the FN with its effect being counteracted by the interaction between PV and MSS. The ADT, in interaction with region and with PV and MSS was seen to cause a decrease in the FN. The correlation matrix in Table D.6 shows that most of these variables were highly correlated. After correcting for the multicollinearity problem, only the effects of two variables stayed in the model; the effect of region and MSS in the cold region. The corrected model, based on 87 observations, has the following form:

$$FN = 94.46 - 3.87\,(LGCUTR) - 0.49\,(REGION * LGCUTR) + 0.26\,(MSS * REGION)$$

The adjusted R^2 was only about 0.37, much lower than the inflated one of the uncorrected model.

8.5.1.4 *Prediction Model for the Lightweight Aggregates.* Since the performance of most of these aggregates was related linearly with traffic with only the performance of one group of sections showing a non-linear fit, a better prediction model resulted when using LGFN against CUTR. The model, shown in Table D.7, was statistically feasible with a moderate adjusted R^2 of about 0.57. The predicted LGFN is shown to be dependent on the region, the interaction between region and CUTR, the AGSR construction variable, and the percent particles retained on the 1/2-in. sieve (SV1). The addition of either the PV or the freeze-thaw test to the model increased the R^2 only slightly (from about 0.59 to about 0.61). In the cold region (RGT = 1), the predicted FN is expected to increase with traffic. The predicted performance is also expected to increase as the aggregate spreading rate (AGSR) and gradation increase. The regression equation, based on 129 observations, has the following form:

$$LG\ FN = 3.08 - 0.14\ (REGION) + 6.28 \times 10^{-8}(REGION * CUTR) + 8.35 \times 10^{-3}\ (AGSR) + 0.011\ (SV1)$$

8.5.1.5 *Prediction Model for the Sandstone Aggregates.* The only variable found to explain some of the variation in the performance for these aggregate was the construction AGSR variable. The adjusted R^2 was only about 0.17 with both the model and AGSR showing significance at the 0.05 probability level. The model, based on 63 observations and shown in Table D.8, has the following form:

$$FN = 34.32 + 0.19\ (AGSR)$$

The fact that many replications, both bound and construction replications, were constructed of this aggregate material dramatically augmented the error term of this model.

8.5.1.6 *Prediction Model for the Siliceous Aggregates.* An excellent adjusted R^2 of about 0.90 was obtained for these aggregates, with the INRD test and LGCUTR accounted for about 86 percent of the explained variation. The LGFN was used in this model for the same reason as in the case of the LMRA aggregates. The prediction model, presented in Table D.9, also showed the number of particles with at least two crushed faces (CF2) to be a significant variable. The prediction model, based on 79 observations, is as follows:

$$LGFN = 4.11 - 0.055\ (LGCUTR) + 5.2 \times 10^{-3}\ (INRD) + 9.25 \times 10^{-4}\ (CF2)$$

The predicted LGFN is found to decrease with traffic, while it is expected to increase as the INRD and CF2 increase.

8.5.2 Statistical Modeling Using the Uncoated Aggregates Data

When only the friction data of the uncoated aggregates were used, and when all of the laboratory tests along with other types of variables were considered, a regression model similar to that obtained using all friction data resulted. The model, with an adjusted R^2 of 0.56 and 187 observations, included LGCUTR, the interaction between PV and INRD with traffic, and the construction AGSR variable. The model and all of the constituting variables were found to be significant. Table D.10 gives the solution of the following model:

$$FN = 43.0 - 2.4\,(LGCUTR) + 0.018\,(PV * LA) + 3.63 \times 10^{-4}\,(PV * INRD * LGCUTR) + 0.125\,(AGSR)$$

Because of the variability known to be associated with the LA test, an attempt was made to replace this test with either the ADI or the TDT test, both from the same group of tests (Chapter 5). The adjusted R^2 decreased to 0.47 in both cases although each of them was shown to be significant.

Another stepwise run was performed in which the PV and LA tests variables were left out of the model statement, to see if a model with the same significance could be generated with the PV and LA variables not being part of it. The resulting model, shown in Table D.11, had an adjusted R^2 of about 0.566, but this time with traffic, the absorption (ABSP) test, the aggregate durability index (ADI) test, the interaction between INRD and traffic, and the construction AGSR variable explaining about the same amount of the experienced variation. However, there was a multicollinearity problem caused by the ADI and AGSR variables which was corrected after dropping ADI from the model. The R^2 decreased to about 0.50. The corrected model, based on 187 observations, is as follows:

$$FN = 61.55 - 3.0\,(LGCUTR) + 3.34\,(ABSP) + 0.0134\,(INRD * LGCUTR) + 0.1\,(AGSR)$$

It is believed that the effect of the ABSP test was significant because its results varied in the same manner the results of the PV and MSS tests did in the case of the LMST aggregates.

8.5.3 Statistical Modeling Using the Petrographically-Examined Aggregates

When the friction data of the petrographically-examined aggregates were used, a regression model with a good adjusted R^2 of about 0.65 resulted. The model, based on 178 observations, has the following form, with all of the variables being significant:

$$FN = 54.13 - 2.57\,(LGCUTR) + 0.0123\,(GST * LGCUTR) - 0.018\,(CG * LGCUTR) + 0.168\,(DG) + 0.11\,(NCM) + 0.45\,(VC) + 0.16\,(AGSR)$$

While the FN was found to decrease with traffic, the slope of this relationship was seen to change upwardly depending on the percentage of grain-supported texture (GST) particles in an aggregate. However, for those grains which were carbonate (as opposed to hard), that slope was observed to adjust for this property. This indicated that although the texture was good (GST), it did not greatly enhance the performance because the grains were carbonate. When the grains were dolomite, the FN was found to also increase. In addition, the percent of non-carbonate matrix (NCM) was seen to positively affect performance in spite of the lack in non-carbonate grains. This explained why aggregate 16, which had a low PV but a high percentage of NCM, exhibited a better performance than the carbonate aggregates with low PVs. Moreover, the void content (VC) was found to have a positive effect on performance, particularly on that of some of the carbonate aggregates with considerable amount of matrix-supported texture (MST) particles. Although region did not come out as an important variable in this model, it is believed that the fact that most of the somewhat porous, MST carbonate aggregates were located in the warm, moist region contributed the VC variable showing significance in this model. Finally, as in most of the other models, the construction AGSR variable was found to be significant.

8.6 Application of the Prediction Models

The prediction model can be used in two ways:

1. It can be used to estimate the mean value of frictional performance for given values of the predictor variables.

2. It can be used to predict some future individual value of frictional performance for given values of the predictor variables.

Estimates of the mean value or predictions of specific values of frictional performance (to be observed in the future) for given values of the predictor variables can be obtained by substituting values of the predictor variables into a prediction model. A confidence interval for the mean value and a prediction interval for the specific value of frictional performance can then be constructed. For the same confidence level and the same values of predictor variables, the prediction interval for the specific value is wider than the confidence interval for the mean value. This is a reflection of the fact that the variance of the error of predicting a particular value of frictional performance exceeds the variance of the error of estimating the mean value (82). A confidence interval for a mean FN value can be constructed using the following equation:

$$\mu_{FN / X_1, X_2, ..} = \overline{FN} + \widehat{\beta}_1 (X_{01} - \overline{X_1})^2 \mp$$

$$t_{[n - (k + 1)], 1 - \frac{\alpha}{2}} S_{FN / X_1, X_2 ..} \sqrt{\frac{1}{n} + \frac{(X_{01} - \overline{X}_1)^2}{(n - 1) S^2_{x_1}} + ...}$$

where,

$\mu_{FN / X1, X2, ..}$ = expected mean FN

$\overline{FN}$	=	observed mean FN (referred to as DEP MEAN in the computer outputs)
$\widehat{\beta}_1$	=	predicted parameter estimate of an independent variable X_1,
X_{01}	=	a future value of an independent variable X_1,
$\overline{X_1}$	=	observed mean value of an independent variable X_1 (included in the computer outputs),
$t_{[n - (k + 1)], 1 - \alpha / 2}$	=	the student's t value corresponding to $[n - (k + 1)]$ degrees of freedom and $1 - \alpha / 2$ percent confidence,
n	=	number of observations used in each model,
k	=	number of independent variables in each model,
α	=	significance level (the probability of making Type I error),
$S_{FN / X1, X2, ..}$	=	standard deviation of FN (referred to as ROOT MSE in the computer outputs), and
$S^2_{x_1}$	=	observed variance of an independent variable X_1 (included in the computer outputs)

The prediction interval for a specific FN value can be constructed using the above equation but with one added to the term under the square root symbol.

The regression equation generated for the experimental design presented in Table 8.1 can be used most appropriately for predicting the performance of aggregates that are, based on history, to fall in any of the sixteen groups constituting that design. An example showing the predicted versus observed FN along with the 95 percent confidence interval (lower mean (LM) and upper mean (UM) values) for limestone aggregate 37 of group 16 is illustrated in Fig. 8.1.

The regression model generated for the lightweight aggregates can be used to predict the performance of these aggregates. In addition to traffic, the designer needs only to know the region (warm or cold), construction spreading rate of aggregate, and percent particles retained on a 1/2-in. sieve.

The performance of the limestone rock asphalt aggregates can be predicted using the model generated specifically for these aggregates. Only traffic, region, and construction spreading rate of aggregate need to be known in order that a prediction can be made. Figure 8.2 shows the observed versus predicted performance of limestone rock asphalt aggregate 42.

For the siliceous gravel aggregates, the amount of insoluble residue, number of particles with at least two crushed faces, and traffic are necessary to know in order that performance can be predicted using the regression equation formulated specifically for these aggregates. Figure 8.3 shows the observed versus predicted performance of siliceous gravel aggregate 39.

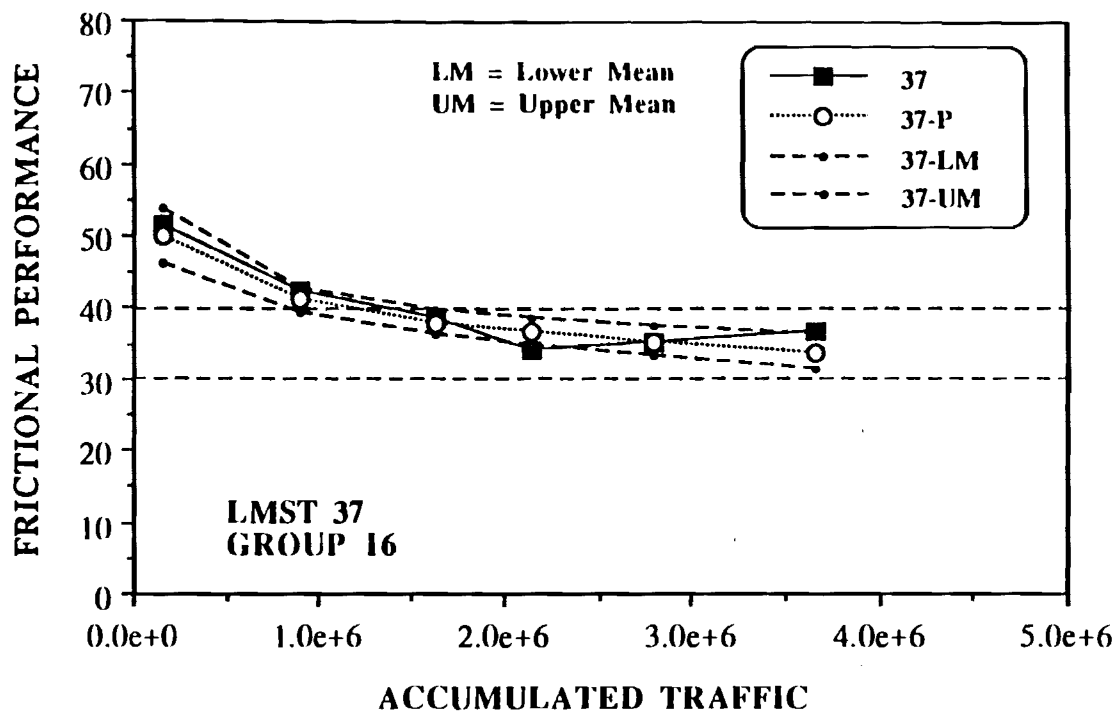


Fig. 8.1 Predicted versus Observed Performance for LMST Aggregate 37 -- GLM Regression Equation

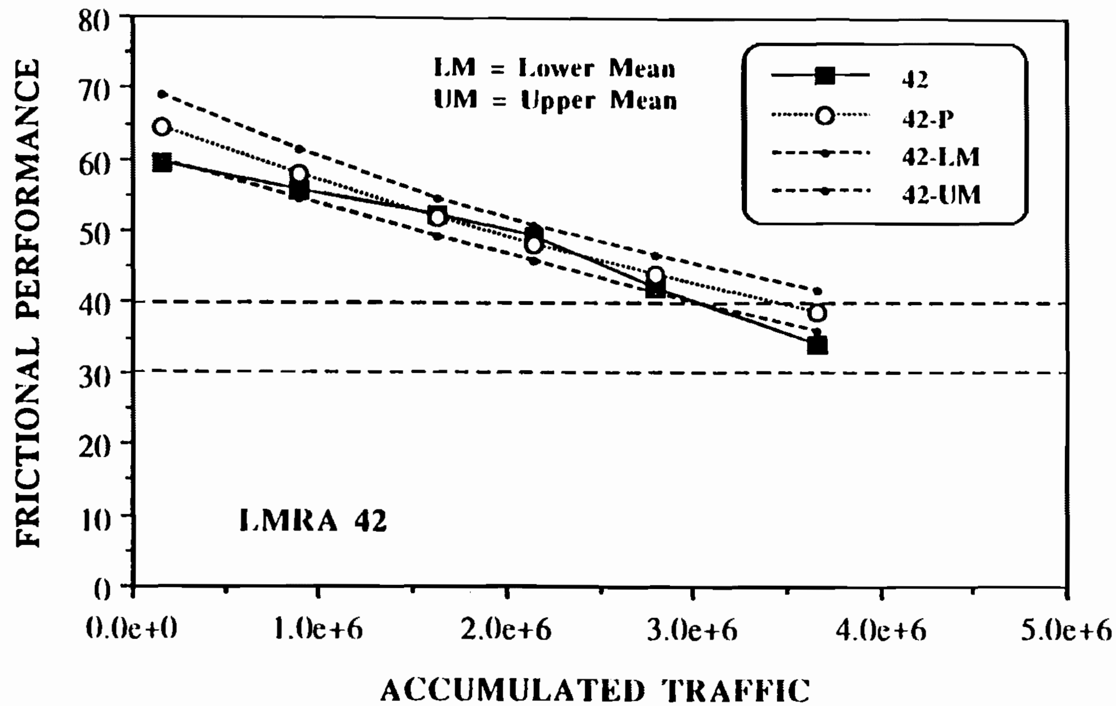


Fig. 8.2 Predicted versus Observed Performance for LMRA Aggregate 42 -- LMRA Regression Equation

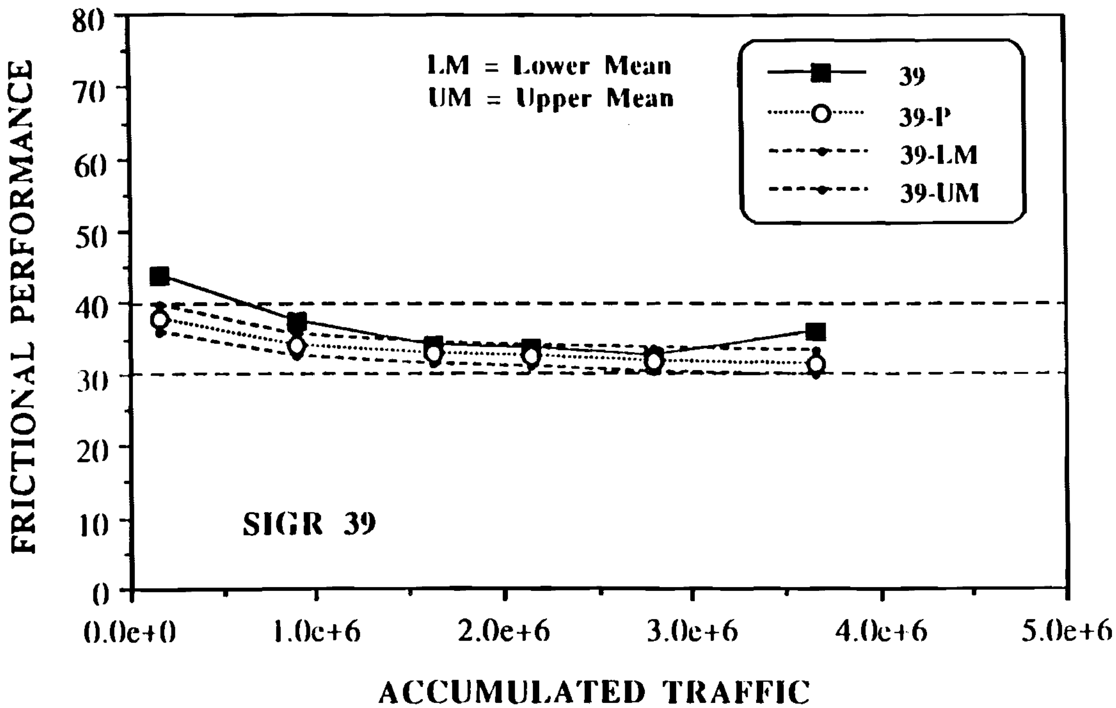


Fig. 8.3 Predicted versus Observed Performance for SIGR Aggregate 39 -- SIGR Regression Equation

The sandstone aggregates incorporated in this study were all constructed in the cold region and all maintained somewhat constant levels of performance with the level increasing as the aggregate spreading rate increased. The use of the sandstone regression model, which is totally dependent on the aggregate spreading rate, gives the designer an idea on the level of performance to be expected. Also, the model formulated using the data of the uncoated aggregates can be used. However, this model is not recommended for predicting the performance of calcareous sandstones because the model tends to underestimate the performance of these aggregates.

The performance of the limestone aggregates seems to be best predicted using the regression model formulated using the data of the petrographically-examined aggregates. A comparison is made in Figs. 8.4 and 8.5 that shows the closer prediction obtained using the petrographic model as opposed to the underestimated prediction given by the model of the uncoated aggregates data. Also, the petrographic model better differentiates between the sandstone and grain-supported texture limestone aggregates by distinguishing between hard and carbonate grains. Moreover, it gives levels of predicted performance for the high insoluble residue, grain-supported sandstone that are higher than those given for the high insoluble residue, matrix-supported siliceous gravel aggregates.

The various prediction models can be incorporated in a pavement management system as illustrated in the example shown in Fig. 8.6. Suppose that a seal coat overlay is to be placed on a highway, located in the warm region, with an expected ADT of 2740 vehicle per lane, and no major rehabilitation project is scheduled for this highway within the next eight years. The predicted performance of a locally available limestone rock asphalt aggregate, spread at 100 sq.yd/cu.yd, reveals a decrease in the FN to 30 at about four million traffic passes, equivalent to four years of service. Therefore, another seal coat overlay with this local material is needed if an acceptable level of frictional resistance is to be maintained throughout the second four-year period. On the other hand, the predicted performance of a sandstone aggregate with a 100 percent insoluble residue and an absorption value of 1.4, spread at the same rate as that of the limestone rock asphalt aggregate, shows that this aggregate will maintain a high level of performance throughout the whole eight -year period, thus eliminating the need

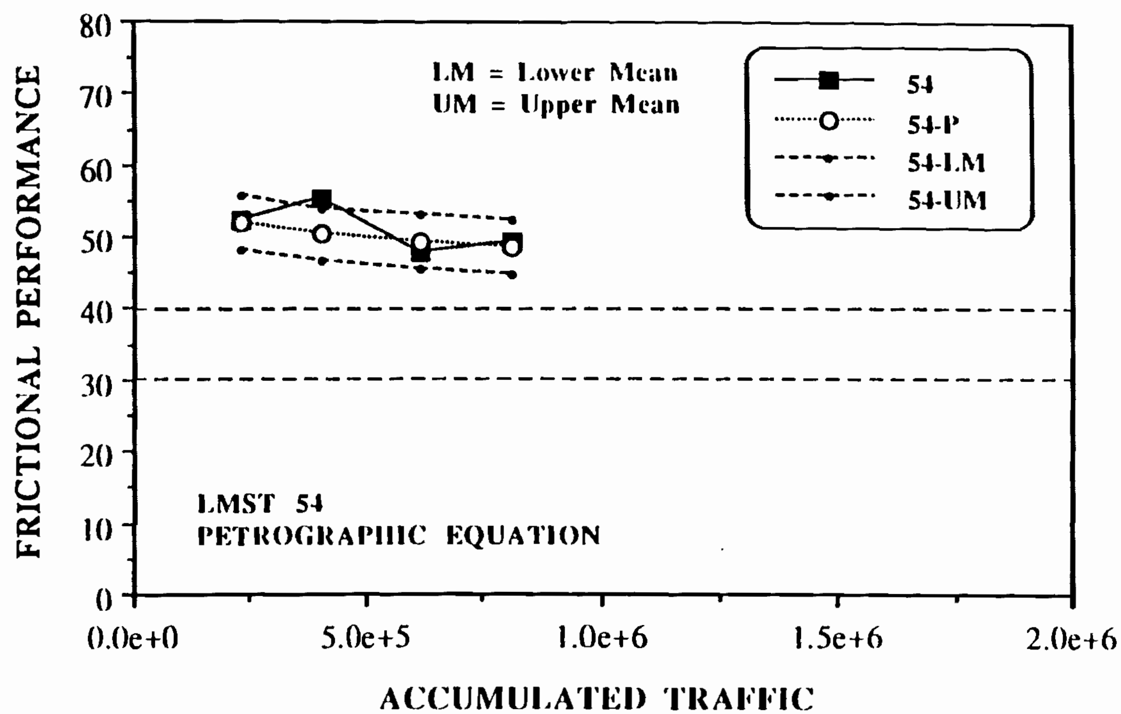


Fig. 8.4 Predicted versus Observed Performance for LMST Aggregate 54 -- Petrographic Regression Equation

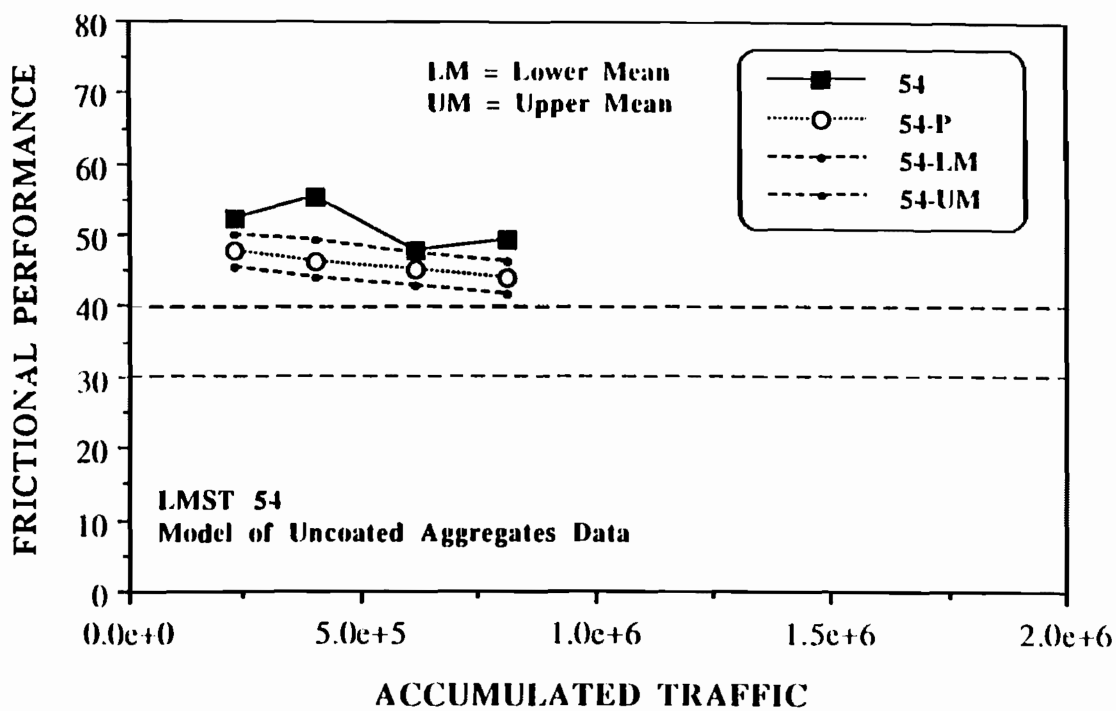


Fig. 8.5 Predicted versus Observed Performance for LMST Aggregate 54 -- Regression Equation of Uncoated Aggregates Data

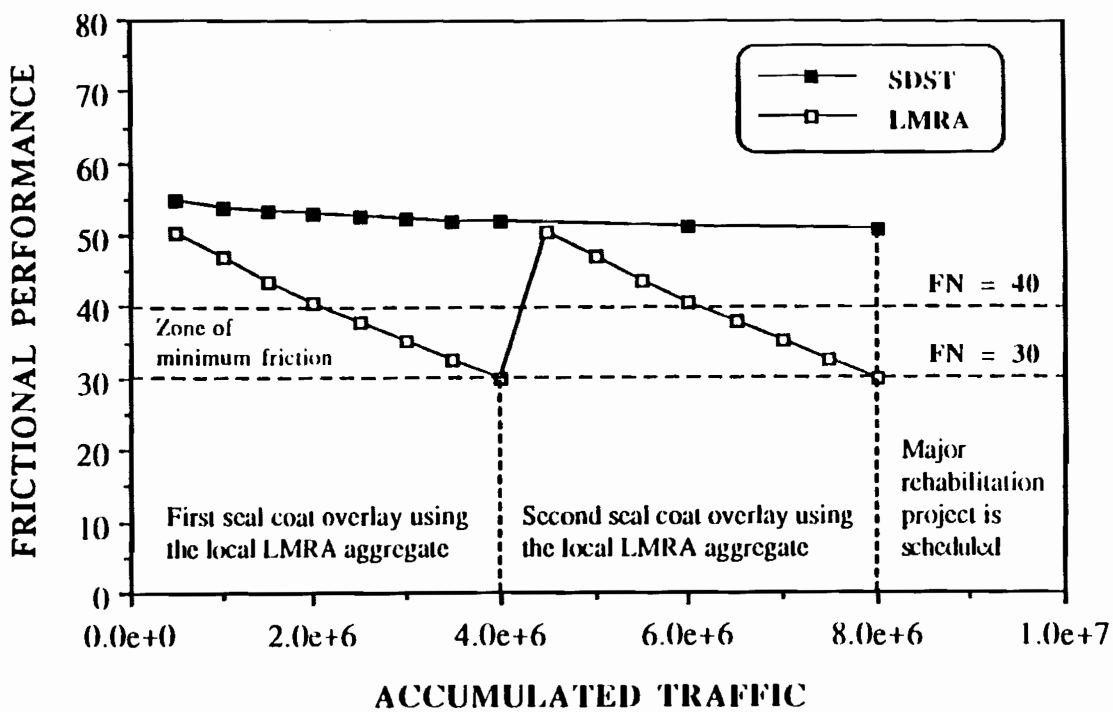


Fig. 8.6 An Example on How the Prediction Models Can Be Incorporated in a Pavement Management System

for the second seal coat overlay that would otherwise be needed in the middle of this period. If this sandstone aggregate is not available locally, the extra cost incurred from hauling this good quality aggregate should be weighted against the cost of a second overlay built with the local material.

CHAPTER 9
CONCLUSIONS

9.1 Summary

The ultimate aim of this research was to formulate statistical models that can be used for predicting the frictional performance of seal coat pavement overlays. The methodology involved establishing fifty-nine seal coat test sections in many parts of the State of Texas, including all four environmental regions, and monitoring their performances over time. Many factors, believed to have an influence on performance level and identified in the literature and Texas districts surveys, were considered in this study. These included aggregate properties, construction variables, traffic, and environment and weather variables.

Frictional performance was measured by a skid trailer and expressed as a friction number (FN). Eight sets of FN measurements, spanned over about five years, have been obtained which were used in the analysis. In addition, three rounds of British pendulum and sand patch testing were performed on most of the test sections. The data were used for correlation purposes with the FN and in the interpretation of the trends in frictional performance. Weather data relevant to the period prior to field testing were also collected.

Construction data were gathered which basically dealt with construction application rates and types of aggregates and asphalt. Aggregate samples obtained from construction sites were tested in the laboratory for their basic properties, polish susceptibility, resistance to weathering action, and resistance to abrasion and impact actions. Twenty of the samples were also examined for their mineralogical and petrographical properties. In this examination, the mineralogical constituents were estimated, and the textural characteristics were evaluated.

Correlations among the laboratory tests and among the field tests were studied. The performance data were graphed to detect the sources of variations and were grouped according to the different considered variables. The grouping gave insights into which variables controlled the observed differences in frictional performance. The grouping was followed by extensive statistical modeling which pinpointed the significant variables.

9.2 Conclusions

9.2.1 Laboratory Studies

The correlation studies revealed no correlations between the polish value test and the soundness, Los Angeles abrasion, and insoluble residue tests, on which data were available for most of the aggregates. However, good correlations were found between the polish value test and each of the soundness and Los Angeles abrasion tests for the limestone group. The correlation between the Los Angeles abrasion and soundness was found to have followed a nearly exponential path, the shape of which was contributed to by the limestone group. In addition, the Los Angeles abrasion test was seen to better distinguish among the natural aggregates, particularly between the hard aggregates and the limestones with low polish values.

In the case of uncoated aggregates, poor overall correlations were noticed between the polish value test and all of the other tests. However, excellent correlations were observed between the polish value test and the specific gravity and absorption tests, whereas a moderate correlation coefficient was obtained for the correlation between the polish value and Texas degradation tests.

The correlations between each of the Los Angeles abrasion and soundness tests and the other tests in their respective test groups were studied. The aggregate durability index test was found to have no correlation with the Los Angeles abrasion test, while a moderate overall correlation was observed between the Los Angeles abrasion and Texas degradation tests. The overall correlation of the freeze-thaw test with the soundness test revealed a high coefficient of 0.81. Furthermore, the absorption and soundness tests were observed to be highly correlated. Much of the strength of that correlation was attributed to the limestone group.

Several observations were made from the petrographic study. First, low polish values and high polish values were found to be associated with aggregates having high percentages of matrix-supported texture and grain-supported texture particles, respectively. Second, the polish value test failed to distinguish between aggregates exhibiting similar texture properties but different percentages of void contents. This was particularly evident for the aggregates with a considerable amount of matrix-supported texture particles. Third, the polish value test seemed not to reflect the effect of the hardness of the minerals constituting an aggregates. Fourth, it was also understood that the presence of a high amount of voids in grain-supported texture particles, indicating a loose compaction of grains or the presence of high levels of porosity in matrix-supported texture particles, could have contributed to high soundness losses. Lastly, the presence of a large amount of tightly compacted carbonate grains in grain-supported texture particles was found not to produce a high polish value.

9.2.2 Field Studies

To determine whether the skid test did indeed measure both the microtexture and macrotexture properties of seal coat surfaces, the correlation between FN and British pendulum number (BPN) was established, and the dependency of the relationship between FN and average texture depth (ATD) on BPN was studied. On one hand, the overall correlation between FN and BPN was good, indicating that the two were directly related. Moreover, the overall correlation was found to be stronger than most of the correlations for the individual aggregate groups. Much of the strength was believed to have been attributed to the siliceous gravel group and to the fact that the other aggregate groups, when all combined, represented all levels of microtexture. On the other hand, the FN was found to be somewhat related to the ATD for two medium levels of BPN, with the relationship shifting upward for the higher level of BPN. In addition, the FN was found to be independent of ATD for a high level of BPN, such as that possessed by the lightweight aggregates. Lastly, a rational correlation between FN and ATD could not be detected for low levels of BPN.

9.2.3 Observed Frictional Performance Variation

The variations in performance were observed to be minimal in most of the bond and construction replications. In most of the lane replications, the inner lanes were observed to have maintained a higher level of frictional performance than those exhibited by the outer lanes. This indicated that the level of average daily traffic was truly lower on these lanes, and the variations in most of these groups could no longer be considered random.

The grouping of the entire friction data according to the various aggregate materials was found to explain, in broad terms, much of the scatter observed in the data. Superior performance was observed in the lightweight and sandstone groups, and a slight decrease in performance was seen in the rhyolite group. The limestone rock asphalt and limestone groups experienced different, noticeable rates of decrease in frictional performance. Lastly, different levels of performance were encountered in the siliceous gravel group.

Several observations were made from the test sections constructed end-to-end and from the grouping of sections with similar aspects which experienced similar trends in frictional performance. These were as follows:

1. The performance of aggregates with high polish values and high soundness losses (carbonate aggregates) was inferior to that of aggregates with high polish values but low soundness losses (non-carbonate aggregates). The level of performance for the former was found to be dependent on the level of average daily traffic as the polishing action of this variable interacted with the amounts of rejuvenation by weathering action.
2. Porous aggregates, particularly some lightweight and limestone rock asphalt, were seen to maintain excellent rejuvenated surfaces in the area characterized by temperature freeze-thaw cycling.
3. The level of construction aggregate spreading rate was shown to explain much of the variations in many of the groups identified.
4. The performance of low polish value aggregates which possessed some porosity or had high contents of non-carbonate minerals was better than that of the other low polish value aggregates.

9.2.4 Establishment and Monitoring of Test Sections

The following conclusions and recommendations pertain to service conditions that may need to be warranted if useful, historical frictional performance is to be obtained for some aggregate sources. These are:

1. The 1000 feet in length for test sections in most cases for allowing five friction values to be measured.
2. Although the long-term, seasonal variation factor did not come out to be a statistically significant factor, it is still believed that more than one friction measurement may need to be collected in any year so that expected fluctuations, similar to those observed in the two collected friction measurements can be represented in the performance history of a source. The timing of friction inventory testing can be flexible especially that it is understood by now that there is no clear distinction between dry and wet seasons in most of the climate subdivisions in Texas. However, because some trends have been observed as to the effect of dry-versus-wet season on the measured frictional performance of test sections constructed in the Edwards Plateau climatic subdivision, it is recommended that, for those subdivisions possessing more distinctive dry-versus-wet seasons, testing be performed in both dry and wet seasons. These subdivisions also include the High Plains, Low Rolling Plains, and Trans-Pecos subdivisions.
3. Sections from the same aggregate source need to be established on roadways representing all ranges of average daily traffic as this variable was observed (1) to have initial aggregate particles influenced the level of embedment, as measured by the sand patch macrotexture test, and (2) to have governed the rate of polish as opposed to that of surface rejuvenation by weathering action, particularly in the case of soft limestone aggregates placed in the wet regions.

9.2.5 Significant Variables in the Prediction Models

The grouping of aggregates along with the aggregate spreading rate and region variables were found to explain about 85 percent of the variation in performance. A prediction equation was formulated that described the performance of the sixteen group, included in the experimental design.

A general multivariable regression model was formulated using all friction measurements. The interactions between the polish value and Los Angeles abrasion test and that between the polish value and insoluble residue tests with traffic along with the natural logarithmic function of accumulated traffic (LGCUTR) and aggregate spreading rate explained about 56 percent of the variations.

A prediction model was generated for each aggregate material group. For the limestone rock asphalt, traffic, the interaction between traffic and region, and aggregate spreading rate explained about 80 percent of the variations. In the limestone group, only region, the interaction between soundness and region, and traffic stayed in this model after correcting for multicollinearity problems. Region, the interaction between region and traffic, aggregate spreading rate, and percentage of particles retained on the 1/2-in. sieve contributed to explaining about 62 percent of the performance variation in the lightweight group. Only one factor, aggregate spreading rate, was found to explain some of the variation in the sandstone group, whereas traffic, the insoluble residue test and the level of particle crushing were found to explain about 90 percent of the variation experienced in the siliceous gravel group.

When the friction data of the uncoated aggregates were used, the insoluble residue and absorption tests were found to replace the polish value, Los Angeles abrasion, and insoluble residue interaction terms and still give a somewhat similar fit in the regression model. The construction aggregate spreading rate was also shown to be significant.

Many petrographic properties were found to be significant in explaining frictional performance variation. These included the percentage of grain-supported texture particles, percentage of carbonate and dolomitic grains, percentage of non-carbonate matrix, and void content. These variables along with the aggregate spreading rate variable explained about 67 percent of the variation experienced with the twenty aggregates considered in the petrographic examinations.

9.3 Contributions and Significance of Findings

The prediction models formulated in this study will provide highway engineers with tools by which the following may be made possible:

1. An optimization of aggregate use with regard to economics and conservation of resources through an improved procedure for selecting aggregates for a given class of use, in a given environment of service, and for a given level of performance.
2. Prediction, at the design phase, of the life of seal coat overlays, and, consequently, more effective planning for seal coat rehabilitation projects.
3. A better understanding of the construction practices required for maintaining a desirable level of frictional performance.

9.4 Recommendations

It is recommended that:

1. TxDOT replace the polish value test, as it is now performed, as the primary method for evaluating aggregates for surface friction.
2. The prediction models based upon aggregate properties for each aggregate material group should be implemented. A very substantial data base exists which provides a sound basis for the equations.
3. Other research should continue to determine the best methods to predict skid number of other types of materials including aggregate blends and hot mix asphalt.

APPENDIX A

RESULTS OF THE PETROGRAPHIC EXAMINATION

Table A.1 Summary of Mineralogical and Petrographic Properties for Aggregate No. 1

Aggregate Number: 1	**Color:** Tan-gray to gray	**Producer Pit**	**PV**
Group: 1 of 3	**Sphericity:** Low	Whites mines	
Percentage: 55 %	**Roundness:** Angular	Brownwood	28
Name: Crushed mudstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar 3 % Dolomite 1 %	Pyrite
0 %	Total= 95 %		Total= 4 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine to medium	Angular to rounded	Low to high

Table A.1 Summary of Mineralogical and Petrographic Properties for Aggregate No. 1 (Continued)

Aggregate Number: 1
Group: 2 of 3
Percentage: 40 %
Name: Crushed wackestone limestone

Color: Tan to gray
Sphericity: Medium
Roundness: Angular

Producer Pit	**PV**
Whites mines Brownwood	28

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar	Pyrite
0 %	Total= 80 %		Total= 17 %	Total= 3 %

Grain size	**Roundness**	**Sphericity**
Fine to medium	Angular to rounded	Low to high

Table A.1 Summary of Mineralogical and Petrographic Properties for Aggregate No. 1 (Continued)

Aggregate Number: 1	**Color:** Tan to gray	**Producer Pit**	**PV**
Group: 3 of 3	**Sphericity:** Medium	Whites mines	
Percentage: 5 %	**Roundness:** Angular	Brownwood	28
Name: Crushed packstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Spar		Mud Spar	
5 %	Total= 15 %		Total= 80 %	

Grain size	Roundness	Sphericity
Fine to medium	Angular to rounded	Low to high

Table A.2 Summary of Mineralogical and Petrographic Properties for Aggregate No. 6

Aggregate Number: 6	**Color:** Tan to gray	**Producer Pit**	**PV**
Group: 1 of 4	**Sphericity:** Medium	James Gravel	
Percentage: 50 %	**Roundness:** Well rounded to angular	Blackburn	27
Name: Partially crushed mudstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Ultra fine mud		Ultra fine spar Mud	Dendritic pyrolusite
0 %	Total= 95 %		Total= 4 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.2 Summary of Mineralogical and Petrographic Properties for Aggregate No. 6 (Continued)

Aggregate Number: 6
Group: 2 of 4
Percentage: 41 %
Name: Partially crushed packstone limestone

Color: Tan
Sphericity: Medium
Roundness: Well rounded to subangular

Producer Pit	**PV**
James Gravel Blackburn	27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Spar		Ultra fine Spar Mud	Dendritic pyrolusite
0 %	Total= 25 %		Total= 74 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.2 Summary of Mineralogical and Petrographic Properties for Aggregate No. 6 (Continued)

Aggregate Number: 6 **Group:** 3 of 4 **Percentage:** 6 % **Name:** Partially crushed mixed chert and quartz	**Color:** **Sphericity:** **Roundness:**	**Producer Pit** James Gravel Blackburn	**PV** 27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		50% Chert particles 50% Quartz particles		
0 %		Total= 100 %		

Grain size	Roundness	Sphericity

Table A.2 Summary of Mineralogical and Petrographic Properties for Aggregate No. 6 (Continued)

Aggregate Number: 6	**Color:**	**Producer Pit**	**PV**
Group: 4 of 4	**Sphericity:**	James Gravel	
Percentage: 3 %	**Roundness:**	Blackburn	27
Name: Partially crushed quartzite			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	**Lacks matrix**

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
				Quartzite
0 %				Total= 100 %

Grain size	**Roundness**	**Sphericity**

Table A.3 Summary of Mineralogical and Petrographic Properties for Aggregate No. 7

Aggregate Number: 7	**Color:** Tan, minor tan to gray	**Producer Pit**	**PV**
Group: 1 of 3	**Sphericity:** Low	Whites Mines	
Percentage: 20 %	**Roundness:** Angular	Massey	37
Name: Crushed mudstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Spar	1 chert particle
0 %	Total= 94 %		Total= 5 %	Total= 1 %

Grain size	Roundness	Sphericity
Very fine to coarse	Very angular to rounded	Low to high

Table A.3 Summary of Mineralogical and Petrographic Properties for Aggregate No. 7 (Continued)

Aggregate Number: 7
Group: 2 to 3
Percentage: 15 %
Name: Crushed wackestone limestone

Color: Tan to gray
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
Whites Mines	
Massey	37

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Minor dolomite 5% Spar & mud 24%	Minor pyrite
5 %	Total= 65 %		Total= 29 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Very fine to coarse	Angular to rounded	Low to high

Table A.3 Summary of Mineralogical and Petrographic Properties for Aggregate No. 7 (Continued)

Aggregate Number: 7	**Color:** Tan	**Producer Pit**	**PV**
Group: 3 of 3	**Sphericity:** Low	Whites Mines	
Percentage: 65 %	**Roundness:** Angular	Massey	37
Name: Crushed packstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Minor dolomite 5 % Spar & mud 54%	Minor pyrite
5 %	Total= 35 %		Total= 59 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Very fine to coarse	Angular to rounded	Low to high

Table A.4 Summary of Mineralogical and Petrographic Properties for Aggregate No. 9

Aggregate Number: 9	**Color:** Cream	**Producer Pit**	**PV**
Group: 1 of 3	**Sphericity:** Medium	Panhandle Gravel	
Percentage: 10 %	**Roundness:** Subrounded	Box Canyon	30
Name: Partially crushed calcareous sandstone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Limestone	Quartz Chert Feldspar
5 %	Total= 20 %		Total= 5 %	Total= 70 %

Grain size	Roundness	Sphericity
Fine	Subangular to rounded	High

Table A.4 Summary of Mineralogical and Petrographic Properties for Aggregate No. 9 (Continued)

Aggregate Number: 9
Group: 2 of 3
Percentage: 10 %
Name: Partially crushed wackestone limestone

Color: Tan
Sphericity: Low
Roundness: Subrounded to subangular

Producer Pit	**PV**
Panhandle Gravel	
Box Canyon	30

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Spar 90% Dolomite 10 %	Lignite
5 %	Total= 70 %		Total= 24 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine to coarse	Angular	Low

Table A.4 Summary of Mineralogical and Petrographic Properties for Aggregate No. 9 (Continued)

Aggregate Number: 9
Group: 3 of 3
Percentage: 80 %
Name: Partially crushed granite gneiss

Color: Clear to black
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
Panhandle Gravel	
Box Canyon	30

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	**Lacks matrix**

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
				Quartz 95% Feldspar 2% Mica 3%
0 %				Total= 100 %

Grain size	Roundness	Sphericity
Fine to coarse	Angular	Low to medium

Table A.5 Summary of Mineralogical and Petrographic Properties for Aggregate No. 12

Aggregate Number: 12	**Color:** Brown tan	**Producer Pit**	**PV**
Group: 1 of 4	**Sphericity:** Low	Tx S&G	
Percentage: 15 %	**Roundness:** Angular	Tascosa	30
Name: Partially crushed calcareous sandstone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud			Quartz 72% Clay minerals 8 %
0 %	Total= 20 %			Total= 80 %

Grain size	**Roundness**	**Sphericity**
Very fine to fine	Angular	High

Table A.5 Summary of Mineralogical and Petrographic Properties for Aggregate No. 12 (Continued)

Aggregate Number: 12
Group: 2 of 4
Percentage: 35 %
Name: Pratially crushed sandstone

Color: Brown
Sphericity: Medium
Roundness: Angular

Producer Pit	**PV**
Tx S&G Tascosa	30

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
		Silica		Quartz 64% Feld spar 8% Clay mrl. 8%
0 %		Total= 20 %		Total= 80 %

Grain size	**Roundness**	**Sphericity**
Very fine to fine	Angular to subrounded	Low to high

Table A.5 Summary of Mineralogical and Petrographic Properties for Aggregate No. 12 (Continued)

Aggregate Number: 12	**Color:** Red to brown	**Producer Pit**	**PV**
Group: 3 of 4	**Sphericity:** Low	Tx S&G	
Percentage: 40 %	**Roundness:** Angular	Tascosa	30
Name: Partially crushed granite gneiss			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	**Lacks matrix**

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
				Quartz 80% Feld spar 10% Mica 10%
0 %				Total= 100 %

Grain size	**Roundness**	**Sphericity**
Very fine to coarse	Low	Moderate to Low

Table A.5 Summary of Mineralogical and Petrographic Properties for Aggregate No. 12 (Continued)

Aggregate Number: 12
Group: 4 of 4
Percentage: 10 %
Name: Partially crushed quartz

Color: Clear to red tint
Sphericity: Medium
Roundness: Angular to subrounded

Producer Pit	**PV**
Tx S&G	
Tascosa	30

TEXTURE			
Matrix-supported texture		Grain-supported texture	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Quartz		
0 %		Total= 100 %		

Grain size	Roundness	Sphericity

Table A.6 Summary of Mineralogical and Petrographic Properties for Aggregate No. 15

Aggregate Number: 15
Group: 1 of 2
Percentage: 60 %
Name: Crushed packestone limestone

Color: Light gray
Sphericity: Medium
Roundness: Subangular

Producer Pit	**PV**
Whites Mines Uvalde	34

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar 24 % Mud 24 % Dolomite 12 %	Pyrite 2.0% Limonite .5% Silica 2.5%
25 %	Total= 10 %		Total= 60 %	Total= 5 %

Grain size	**Roundness**	**Sphericity**
Relatively coarse	Well rounded to angular	Low to high

Table A.6 Summary of Mineralogical and Petrographic Properties for Aggregate No. 15 (Continued)

Aggregate Number: 15	**Color:** Light to Medium gray	**Producer Pit**	**PV**
Group: 2 of 2	**Sphericity:** Low	Whites Mines	
Percentage: 40 %	**Roundness:** Angular	Uvalde	34
Name: Crushed packestone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud 50 % Spar 50 %		Spar 30 % Mud 20 % Dolomite 15 %	Pyrite 4% Limonite 3% Silica 3%
5 %	Total= 20 %		Total= 65 %	Total=10 %

Grain size	Roundness	Sphericity
Fine to coarse	Rounded to angular	Low to high

Table A.7 Summary of Mineralogical and Petrographic Properties for Aggregate No. 16

Aggregate Number: 16	**Color:** Dark brown	**Producer Pit**	**PV**
Group: 1 of 3	**Sphericity:** Low	Bay Inc	
Percentage: 90 %	**Roundness:** Angular to rounded	Perez	27
Name: Partially crushed chert			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Chert 94% Organic impurities 1%		Quartz
0 %		Total= 95 %		Total= 5 %

Grain size	Roundness	Sphericity
Very fine	Subangular	High

Table A.7 Summary of Mineralogical and Petrographic Properties for Aggregate No. 16 (Continued)

Aggregate Number: 16	**Color:**	**Producer Pit**	**PV**
Group: 3 of 3	**Sphericity:**	Bay Inc	
Percentage: 5 %	**Roundness:**	Perez	27
Name: Partially crushed quartz			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Quartz		
0 %		Total= 100 %		

Grain size	Roundness	Sphericity

341

Table A.8 Summary of Mineralogical and Petrographic Properties for Aggregate No. 18

Aggregate Number: 18
Group: 1 of 3
Percentage: 55 %
Name: Crushed mudstone limestone

Color: Tan
Sphericity: Low
Roundness: Angular to subrounded

Producer Pit	**PV**
Gifford Hill	
Ogden	27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Minor spar		Spar	Limonite
5 %	Total= 90 %		Total= 4 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular	Low

Table A.8 Summary of Mineralogical and Petrographic Properties for Aggregate No. 18 (Continued)

Aggregate Number: 18	**Color:** Tan	**Producer Pit**	**PV**
Percentage: 40 %	**Sphericity:** Medium	Gifford Hill	
Name: Crushed wackestone limestone	**Roundness:** Angular	Ogden	27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Some spar		Spar	Limonite
5 %	Total= 75 %		Total= 19 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine	Angular to rounded	Low to high

Table A.8 Summary of Mineralogical and Petrographic Properties for Aggregate No. 18 (Continued)

Aggregate Number: 18
Group: 3 of 3
Percentage: 5 %
Name: Crushed packstone limestone

Color: Tan
Sphericity: Medium
Roundness: Subangular

Producer Pit	**PV**
Gifford Hill	
Ogden	27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar and mud	Pyrite
5 %	Total= 20 %		Total= 74 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine to medium	Angular to rounded	Low to high

Table A.9 Summary of Mineralogical and Petrographic Properties for Aggregate No. 22

Aggregate Number: 22	**Color:** Tan to black	**Producer Pit**	**PV**
Group: 1 of 2	**Sphericity:** Medium	Gifford Hill	
Percentage: 65 %	**Roundness:** Angular to rounded	Little River	34
Name: Partially crushed chert			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
		Chert 94% Hematite 1% Quartz 5%		
5 %		Total= 95 %		

Grain size	**Roundness**	**Sphericity**
Fine	Angular	Low

Table A.9 Summary of Mineralogical and Petrographic Properties for Aggregate No. 22 (Continued)

Aggregate Number: 22
Group: 2 of 2
Percentage: 35 %
Name: Partially crushed sandstone

Color: Tan, light to dark
Sphericity: Medium
Roundness: Subrounded

Producer Pit	**PV**
Gifford Hill	
Little River	34

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
		Silica 19.5% Hematite 0.5%		Quartz
5 %		Total= 20 %		Total= 75 %

Grain size	**Roundness**	**Sphericity**
Very fine to fine	Subangular	High

Table A.10 Summary of Mineralogical and Petrographic Properties for Aggregate No. 25

Aggregate Number: 25	**Color:** Green to red tinted	**Producer Pit**	**PV**
Group: 1 of 1	**Sphericity:** Low	Boorhen Fields	
Percentage: 100 %	**Roundness:** Angular	Apple, OK	36
Name: Crushed sandstone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Silica 24.5% Hematite 0.5%		Quartz 73.5% Pyrite 1.5%
0 %		Total= 25 %		Total= 75 %

Grain size	Roundness	Sphericity
Fine	Subangular to subrounded	High

Table A.11 Summary of Mineralogical and Petrographic Properties for Aggregate No. 31

Aggregate Number: 31
Group: 1 of 1
Percentage: 100 %
Name: Crushed sandstone

Color: Light tan to red brown
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
HMB	
Dequeen, AK	41

TEXTURE			
Matrix-supported texture		Grain-supported texture	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Silica 24.75% Hematite 0.25%		Quartz 67.5% Feldspar 7.5%
0 %		Total=		Total= 75 %

Grain size	Roundness	Sphericity
Fine	Subangular	High

Table A.12 Summary of Mineralogical and Petrographic Properties for Aggregate No. 32

Aggregate Number: 32
Group: 1 of 3
Percentage: 55 %
Name: Crushed mudstone limestone

Color: Light tan to light gray
Sphericity: Medium
Roundness: Angular to subrounded

Producer Pit	**PV**
Texas Crushed Stone	
Feld	36

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar	Chert Limonite
20 %	Total= 75 %		Total= 4 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.12 Summary of Mineralogical and Petrographic Properties for Aggregate No. 32 (Continued)

Aggregate Number: 32
Group: 2 of 3
Percentage: 35 %
Name: Crushed packstone limestone

Color: Dark tan
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
TX Crushed Stone Feld	36

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Spar		Spar	Limonite
25 %	Total= 40 %		Total= 34 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine to coarse	Angular	Low

Table A.12 Summary of Mineralogical and Petrographic Properties for Aggregate No. 32 (Continued)

Aggregate Number: 32
Group: 3 of 3
Percentage: 10 %
Name: Crushed dolomitic wackestone limestone

Color: Earthy light gray
Sphericity: Low
Roundness: Subrounded to angular

Producer Pit TX Crushed Stone Feld — **PV** 36

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud Minor spar		Minor spar and mud 1 % Dolomite 8 %	Limonite
10 %	Total= 80 %		Total= 9 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine	Angular	Low

Table A.13 Summary of Mineralogical and Petrographic Properties for Aggregate No. 36

Aggregate Number: 36
Group: 1 of 3
Percentage: 55 %
Name: Crushed mudstone limestone

Color: Tan to gray
Sphericity: Low
Roundness: Angular to subrounded

Producer Pit	**PV**
Colorado Materials Hunter	28

TEXTURE			
Matrix-supported texture		Grain-supported texture	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Ultra fine mud		Mud Spar	
0 %	Total= 95 %		Total= 5 %	

Grain size	Roundness	Sphericity
Fine	Angular to rounded	Low

Table A.13 Summary of Mineralogical and Petrographic Properties for Aggregate No. 36 (Continued)

Aggregate Number: 36	**Color:** Tan	**Producer Pit** PV
Group: 2 of 3	**Sphericity:** Low	Colorado Materials
Percentage: 20 %	**Roundness:** Angular	Hunter 28
Name: Crushed wackestone limestone		

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar Mud	Limonite
0 %	Total= 80 %		Total= 19 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine to medium	Angular to rounded	Low

Table A.13 Summary of Mineralogical and Petrographic Properties for Aggregate No. 36 (Continued)

Aggregate Number: 36	**Color:** Tan	**Producer Pit** **PV**
Group: 3 of 3	**Sphericity:** Low	Colorado Materials
Percentage: 25 %	**Roundness:** angular	Hunter 28
Name: Crushed packstone limestone		

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Spar Minor mud		Mud spar	
5 %	Total= 20 %		Total= 75 %	

Grain size	Roundness	Sphericity
Fine to medium	Subrounded to angular	High to low

Table A.14 Summary of Mineralogical and Petrographic Properties for Aggregate No. 37

Aggregate Number: 37	**Color:** Light tan	**Producer Pit**	**PV**
Group: 1 of 3	**Sphericity:** Medium	Gifford Hill	
Percentage: 80 %	**Roundness:** Angular	Ogden	29
Name: Crushed mudstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud Minor spar		Spar	1 chert particle
5 %	Total= 90 %		Total= 4 %	Total= 1%

Grain size	Roundness	Sphericity
Fine	Angular	Low

Table A.14 Summary of Mineralogical and Petrographic Properties for Aggregate No. 37 (Continued)

Aggregate Number: 37	**Color:** Tan	**Producer Pit**	**PV**
Group: 2 of 3	**Sphericity:** Low	Gifford Hill	
Percentage: 15 %	**Roundness:** Angular	Ogden	29
Name: Crushed wackestone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Ultra fine grained spar	
10 %	Total= 70 %		Total= 20 %	

Grain size	Roundness	Sphericity
Fine	Angular to rounded	Low to high

Table A.14 Summary of Mineralogical and Petrographic Properties for Aggregate No. 37 (Continued)

Aggregate Number: 37
Group: 3 of 3
Percentage: 5 %
Name: Crushed packstone limestone

Color: Tan
Sphericity: Medium
Roundness: Angular

Producer Pit	**PV**
Gifford Hill	
Ogden	29

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Minor partially recryst. mud		Ultra fine grained spar	
10 %	Total= 30 %		Total= 60 %	

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.15 Summary of Mineralogical and Petrographic Properties for Aggregate No. 38

Aggregate Number: 38
Group: 1 of 3
Percentage: 75 %
Name: Crushed mudstone limestone

Color: Tan
Sphericity: Low
Roundness: Angular to subrounded

Producer Pit	**PV**
Redlad Worth	
Beckman	27

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Minor spar		Spar	
2 %	Total= 95 %		Total= 3 %	

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.15 Summary of Mineralogical and Petrographic Properties for Aggregate No. 38 (Continued)

Aggregate Number: 38	**Color:** Tan	**Producer Pit**	**PV**
Group: 2 to 3	**Sphericity:** Low	Redlad Worth	
Percentage: 15 %	**Roundness:** Angular to subrounded	Beckman	27
Name: Crushed wackestone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Minor spar		Mud Spar	
5 %	Total= 75 %		Total= 20 %	

Grain size	Roundness	Sphericity
Fine	Angular to rounded	Low to high

Table A.15 Summary of Mineralogical and Petrographic Properties for Aggregate No. 38 (Continued)

Aggregate Number: 38	**Color:** Tan	**Producer Pit**	**PV**
Group: 3 of 3	**Sphericity:** Low	Redlad Worth	
Percentage: 10 %	**Roundness:** Angular to subrounded	Beckman	27
Name: Crushed packstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud Minor spar		Spar Mud	
10 %	Total= 20 %		Total= 70 %	

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low to high

Table A.16 Summary of Mineralogical and Petrographic Properties for Aggregate No. 39

Aggregate Number: 39
Group: 1 of 3
Percentage: 73 %
Name: Partially crushed mudstone limestone

Color: Tan
Sphericity: Medium
Roundness: Angular to rounded

Producer Pit PV
South Texas Aggregates
Knippa 25

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Fossils 1% Spar 2% Dolomite 2%	
0 %	Total= 95 %		Total= 5 %	

Grain size	Roundness	Sphericity
Fine	Angular to rounded	Low to high

Table A.16 Summary of Mineralogical and Petrographic Properties for Aggregate No. 39 (Continued)

Aggregate Number: 39	**Color:** Tan	**Producer Pit** **PV**
Group: 2 of 3	**Sphericity:** Low	South Texas Aggregates
Percentage: 24 %	**Roundness:** Angular	Knippa 25
Name: Partially crushed chert		

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
		Chert		
0 %		Total= 100 %		

Grain size	Roundness	Sphericity

Table A.7 Summary of Mineralogical and Petrographic Properties for Aggregate No. 16 (Continued)

Aggregate Number: 16	**Color:** Tan	**Producer Pit**	**PV**
Group: 2 of 3	**Sphericity:** Medium	Bay Inc	
Percentage: 5 %	**Roundness:** Rounded	Perez	27
Name: Partially crushed sandy and dolomitic wackestone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Dolomite	Quartz
5 %	Total= 75 %		Total= 15 %	Total= 5 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular to subrounded	Low to high

Table A.16 Summary of Mineralogical and Petrographic Properties for Aggregate No. 39 (Continued)

Aggregate Number: 39
Group: 3 of 3
Percentage: 3 %
Name: Partially crushed sandstone

Color: Dark tan to gray
Sphericity: Low
Roundness: Angular

Producer Pit **PV**
South Texas Aggregates
Knippa 25

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
		Silica		Quartz
10 %		Total= 20 %		Total= 70 %

Grain size	Roundness	Sphericity
Fine	Subangular	High

Table A.17 Summary of Mineralogical and Petrographic Properties for Aggregate No. 50

Aggregate Number: 50
Group: 1 of 4
Percentage: 10 %
Name: Crushed mudstone limestone

Color: Light tan to tan
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
White's Mines	
Massey	34

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar	Limonite Pyrite
10 %	Total= 88 %		Total= 1 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine	Angular to rounded	Low

Table A.17 Summary of Mineralogical and Petrographic Properties for Aggregate No. 50 (Continued)

Aggregate Number: 50
Group: 2 of 4
Percentage: 45 %
Name: Crushed wackestone limestone

Color: Light tan to light gray
Sphericity: Medium
Roundness: Angular

Producer Pit	**PV**
White's Mines	
Massey	34

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Spar Fossils	Stylolites Pyrite Limonite
15 %	Total= 65 %		Total= 18 %	Total= 2 %

Grain size	Roundness	Sphericity
Fine to medium	Angular to rounded	Low to high

Table A.17 Summary of Mineralogical and Petrographic Properties for Aggregate No. 50 (Continued)

Aggregate Number: 50	**Color:** Light tan to light gray	**Producer Pit**	**PV**
Group: 3 of 4	**Sphericity:** Low	White's Mines	
Percentage: 35 %	**Roundness:** Angular	Massey	34
Name: Crushed packstone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Spar Mud	Pyrite Stylolites Limonite
10 %	Total= 35 %		Total= 54 %	Total= 1 %

Grain size	**Roundness**	**Sphericity**
Fine to medium	Angular to rounded	Low to high

Table A.17 Summary of Mineralogical and Petrographic Properties for Aggregate No. 50 (Continued)

Aggregate Number: 50	**Color:** Earthy tan to gray	**Producer Pit**	**PV**
Group: 4 of 4	**Sphericity:** Medium	White's Mines	
Percentage: 10 %	**Roundness:** Angular	Massey	34
Name: Crushed dolomitic wackestone limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Dolomite 15 % Spar 5 %	
5 %	Total= 75 %		Total= 20 %	

Grain size	**Roundness**	**Sphericity**
Fine	Angular to subrounded	Low to medium

Table A.18 Summary of Mineralogical and Petrographic Properties for Aggregate No. 51

Aggregate Number: 51	**Color:** Dark gray to dark tan	**Producer Pit**	**PV**
Group: 1 of 2	**Sphericity:** Low	Delta Materials	
Percentage: 70 %	**Roundness:** Angular	Marble Falls	38
Name: Crushed calcareous sandstone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Dolomite	Quartz
0 %	Total= 20 %		Total= 10 %	Total= 70 %

Grain size	Roundness	Sphericity
Fine	Subangular to subrounded	High

Table A.18 Summary of Mineralogical and Petrographic Properties for Aggregate No. 51 (Continued)

Aggregate Number: 51	**Color:** Dark gray to dark tan	**Producer Pit**	**PV**
Group: 2 of 2	**Sphericity:** Low	Delta Materials	
Percentage: 30 %	**Roundness:** Angular	Marble Falls	38
Name: Crushed sandy limestone			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	**More than 10% grains**	Matrix present between grains	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Mud		Dolomite	Quartz
0 %	Total= 80 %		Total= 5 %	Total= 15 %

Grain size	**Roundness**	**Sphericity**
Fine	Subangular to subrounded	High

Table A.19 Summary of Mineralogical and Petrographic Properties for Aggregate No. 52

Aggregate Number: 52
Group: 1 of 1
Percentage: 100 %
Name: Crushed calcareous sandstone

Color: Medium gray
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
East Texas Stone	
Blue Mountian	40

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	**Matrix**		**Grains**	
	Carbonate	**Non-carb.**	**Carbonate**	**Non-carb.**
	Spar	Pyrite	Limestone	Quartz 65% Pyrite 2% Lignite 1%
0 %	Total= 29.7 %	Total= 0.3 %	Total= 2 %	Total= 68 %

Grain size	**Roundness**	**Sphericity**
Fine	Subangular	High

Table A.20 Summary of Mineralogical and Petrographic Properties for Aggregate No. 53

Aggregate Number: 53
Group: 1 to 2
Percentage: 25 %
Name: Crushed dolomitic mudstone limestone

Color: Light tan\
Sphericity: Low
Roundness: Angular

Producer Pit	**PV**
Reese Albert	
Willeke	33

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	Matrix present between grains	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Dolomite 5 % Spar 4 %	Limonite
5 %	Total= 85 %		Total= 9 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine	Angular	Low

Table A.20 Summary of Mineralogical and Petrographic Properties for Aggregate No. 53 (Continued)

Aggregate Number: 53	**Color:** Light tan to dark gray	**Producer Pit**	**PV**
Group: 2 of 2	**Sphericity:** Low	Reese Albert	
Percentage: 75 %	**Roundness:** Angular	Willeke	33
Name: Crushed packstone dolomite			

TEXTURE			
Matrix-supported texture		**Grain-supported texture**	
Less than 10% grains	More than 10% grains	**Matrix present between grains**	Lacks matrix

Voids	Matrix		Grains	
	Carbonate	Non-carb.	Carbonate	Non-carb.
	Mud		Dolomite	Limonite
5 %	Total= 20 %		Total= 74 %	Total= 1 %

Grain size	Roundness	Sphericity
Fine	Angular	Low

APPENDIX B
FIELD TESTING DATA

Table B.1 Skid Resistance Test Data

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
1	E	.	1	29-Jan-87	56	50	53	49	56	51	.	10	.	199	63970
1	E	.	2	22-Jul-87	47	42	48	46	40	43	9	10	40	373	121390
1	E	.	3	7-Apr-88	41	44	41	45	37	41	9	10	60	632	203950
1	E	.	4	29-Sep-88	40	41	39	41	41	39	.	10	.	807	256450
1	E	.	5	2-May-89	36	37	39	37	39	38	.	9	.	1022	320950
1	E	.	6	8-Jan-90	44	26	42	37	37	37	.	9	.	1273	396250
2	N	.	1	29-Jan-87	33	35	37	36	35	35	.	.	.	199	94525
2	N	.	2	22-Jul-87	.	.	.	.	.	.	.	.	.	373	177175
2	N	.	3	7-Apr-88	33	31	32	31	32	32	.	.	.	632	297290
2	N	.	4	29-Sep-88	31	30	32	30	31	31	.	.	.	807	375165
2	N	.	6	8-Jan-90	32	33	29	34	32	32	.	9	.	1273	582535
2	S	.	1	29-Jan-87	37	37	39	39	43	38	.	.	.	199	94525
2	S	.	2	22-Jul-87	32	32	33	34	33	32	10	10	50	373	177175
2	S	.	3	7-Apr-88	34	37	33	34	34	34	10	10	60	632	297290
2	S	.	4	29-Sep-88	33	29	32	33	36	32	.	10	.	807	375165
3	N	.	1	29-Jan-87	.	.	.	.	.	.	.	.	.	206	84795
3	N	.	2	22-Jul-87	52	56	54	54	56	52	10	10	55	380	142215
3	N	.	3	7-Apr-88	60	64	57	59	63	58	10	9	70	639	228655
3	N	.	4	29-Sep-88	60	61	60	62	61	58	.	9	.	814	288155
3	N	.	5	2-May-89	54	49	55	57	54	54	.	8	.	1029	361255
3	N	.	6	8-Jan-90	63	58	66	64	66	63	.	8	.	1280	446595
3	N	.	7	22-Oct-90	61	59	63	58	62	61	.	.	.	1508	524115

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
3	N	.	8	30-Apr-91	70	67	68	70	61	67	.	.	.	1698	588715
3	S	.	3	7-Apr-88	59	59	60	61	61	57	.	.	.	639	228655
3	S	.	4	29-Sep-88	62	62	58	61	64	59	.	.	.	814	288155
4	E	.	1	29-Jan-87	61	60	62	61	62	59	.	.	.	223	278750
4	E	.	2	22-Jul-87	57	57	59	56	58	55	9	10	45	397	496250
4	E	.	3	7-Apr-88	61	63	61	54	63	58	9	9	55	656	815150
4	E	.	4	29-Sep-88	60	59	55	61	62	57	.	9	.	831	1025150
4	E	.	5	28-Apr-89	59	58	62	61	62	60	.	8	.	1042	1278350
4	E	.	7	22-Oct-90	64	60	62	63	59	62	.	.	.	1584	1928750
4	E	.	8	30-Apr-91	66	69	68	66	66	67	.	.	.	1774	2072750
4	W	.	1	29-Jan-87	59	57	55	56	59	55	.	.	.	223	278750
4	W	.	3	7-Apr-88	61	61	62	60	61	58	.	.	.	656	815150
4	W	.	4	29-Sep-88	61	59	61	62	62	58	.	.	.	831	1025150
4	W	.	6	8-Jan-90	61	62	64	60	57	61	.	8	.	1297	1584350
5	W	.	1	29-Jan-87	64	68	65	64	65	62	10	10	40	189	228250
5	W	.	2	22-Jul-87	60	62	63	60	62	59	10	10	50	363	445750
5	W	.	3	8-Apr-88	66	65	66	66	64	62	10	9	75	623	770750
5	W	.	4	29-Sep-88	64	64	60	66	61	60	.	9	.	797	988250
5	W	.	5	28-Apr-89	64	64	65	65	65	65	.	8	.	1008	1252000
5	W	.	6	8-Jan-90	70	70	69	67	67	69	.	8	.	1263	1570750
5	W	.	7	18-Oct-90	70	64	68	67	67	67	.	.	.	1546	1924500

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
5	W	.	8	12-Jun-91	63	64	64	65	68	65	.	.	.	1783	2220750
6	W	.	1	29-Jan-87	33	35	35	35	34	34	9	10	40	189	228250
6	W	.	2	22-Jul-87	26	26	27	27	27	27	9	10	40	363	445750
6	W	.	3	8-Apr-88	31	33	32	30	30	31	9	10	55	623	770750
6	W	.	4	29-Sep-88	28	27	29	28	26	28	.	10	.	797	988250
6	W	.	5	28-Apr-89	29	29	28	29	33	30	.	10	.	1008	1252000
6	W	.	6	8-Jan-90	28	29	29	28	29	29	.	9	.	1263	1570750
6	W	.	7	18-Oct-90	30	30	33	32	32	31	.	.	.	1546	1924500
6	W	.	8	12-Jun-91	26	29	29	26	27	27	.	.	.	1783	2220750
7	W	.	1	29-Jan-87	48	43	40	42	46	43	9	10	40	189	228250
7	W	.	2	22-Jul-87	38	38	35	38	37	36	9	10	45	363	445750
7	W	.	3	8-Apr-88	37	41	37	43	42	39	9	9	55	623	770750
7	W	.	4	29-Sep-88	40	40	39	43	44	40	.	9	.	797	988250
7	W	.	5	28-Apr-89	37	37	38	39	39	38	.	9	.	1008	1252000
7	W	.	6	8-Jan-90	39	34	37	37	36	37	.	9	.	1263	1570750
7	W	.	7	18-Oct-90	39	42	32	41	39	39	.	.	.	1546	1924500
7	W	.	8	12-Jun-91	42	44	42	42	39	42	.	.	.	1783	2220750
8	N	.	1	28-Jan-87	61	61	65	64	65	60	10	10	40	188	21340
8	N	.	2	21-Jul-87	57	60	58	56	58	55	10	10	50	363	39715
8	N	.	3	6-Apr-88	57	57	57	58	58	55	9	10	65	621	66325
8	N	.	4	28-Sep-88	58	58	57	55	56	55	.	10	.	796	83825
8	N	.	5	27-Apr-89	55	55	56	54	53	55	.	10	.	1007	104925

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
8	N	.	6	8-Feb-90	59	57	55	57	58	57	.	10	.	1294	133625
8	N	.	7	16-Oct-90	54	53	55	53	53	54	.	.	.	1544	158625
8	N	.	8	11-Jun-91	51	51	50	51	51	51	.	.	.	1782	182425
8	S	.	1	28-Jan-87	59	58	64	59	58	57	.	.	.	188	21340
8	S	.	3	6-Apr-88	58	56	56	56	58	55	.	.	.	621	66325
9	N	.	1	28-Jan-87	58	61	58	58	58	56	9	10	60	188	21340
9	N	.	2	21-Jul-87	52	57	55	55	53	52	9	10	60	363	39715
9	N	.	3	6-Apr-88	54	54	53	52	54	51	9	10	60	621	66325
9	N	.	4	28-Sep-88	54	53	53	53	51	51	.	10	.	796	83825
9	N	.	5	27-Apr-89	51	51	51	52	53	52	.	10	.	1007	104925
9	N	.	6	8-Feb-90	55	57	52	57	55	55	.	10	.	1294	133625
9	N	.	7	16-Oct-90	53	52	52	50	52	52	.	.	.	1544	158625
9	N	.	8	11-Jun-91	50	54	49	49	49	50	.	.	.	1782	182425
9	S	.	1	28-Jan-87	58	56	58	59	56	55	.	.	.	188	21340
9	S	.	3	6-Apr-88	56	49	56	51	51	51	.	.	.	621	66325
10	N	.	1	28-Jan-87	57	61	58	56	57	55	10	10	65	188	21340
10	N	.	2	21-Jul-87	48	50	48	51	48	47	10	10	65	363	39715
10	N	.	3	6-Apr-88	48	49	50	44	48	46	10	10	65	621	66325
10	N	.	4	28-Sep-88	43	45	45	41	42	42	.	10	.	796	83825
10	N	.	5	27-Apr-89	42	42	42	43	42	42	.	10	.	1007	104925
10	N	.	6	8-Feb-90	50	48	54	46	51	50	.	10	.	1294	133625

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
10	N	.	7	16-Oct-90	47	44	53	48	45	47	.	.	.	1544	158625
10	N	.	8	11-Jun-91	38	40	41	36	39	39	.	.	.	1782	182425
10	S	.	1	28-Jan-87	51	55	49	58	55	52	.	.	.	188	21340
10	S	.	3	6-Apr-88	54	54	48	52	44	49	.	.	.	621	66325
11	N	.	1	28-Jan-87	64	62	62	62	60	59	9	10	65	188	21340
11	N	.	2	21-Jul-87	61	58	60	56	54	55	9	10	65	363	39715
11	N	.	3	6-Apr-88	56	54	55	54	54	52	9	10	65	621	66325
11	N	.	4	28-Sep-88	57	55	55	56	56	54	.	10	.	796	83825
11	N	.	5	27-Apr-89	54	55	52	56	57	55	.	10	.	1007	104925
11	N	.	6	8-Feb-90	57	59	58	60	58	58	.	10	.	1294	133625
11	N	.	7	16-Oct-90	58	54	54	54	56	55	.	.	.	1544	158625
11	N	.	8	11-Jun-91	51	50	50	50	50	50	.	.	.	1782	182425
11	S	.	1	28-Jan-87	57	60	55	56	56	55	.	.	.	188	21340
11	S	.	3	6-Apr-88	56	53	54	54	52	52	.	.	.	621	66325
12	N	.	1	28-Jan-87	67	67	64	63	68	63	9	10	65	188	21340
12	N	.	2	21-Jul-87	59	60	59	61	60	57	9	10	65	363	39715
12	N	.	3	6-Apr-88	57	57	58	57	56	55	9	10	65	621	66325
12	N	.	4	28-Sep-88	58	57	57	55	57	55	.	10	.	796	83825
12	N	.	5	27-Apr-89	55	53	56	55	53	54	.	10	.	1007	104925
12	N	.	6	8-Feb-90	62	59	60	62	58	60	.	10	.	1294	133625
12	N	.	7	16-Oct-90	55	54	55	52	55	54	.	.	.	1544	158625

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
12	N	.	8	11-Jun-91	51	48	50	48	50	49	.	.	.	1782	182425
12	S	.	1	28-Jan-87	51	57	58	63	68	57	.	.	.	188	21340
12	S	.	3	6-Apr-88	56	57	57	56	55	54	.	.	.	621	66325
13	E	.	1	28-Jan-87	57	61	60	59	61	57	9	10	55	188	24540
13	E	.	2	21-Jul-87	53	54	55	58	55	53	9	10	65	363	42915
13	E	.	3	6-Apr-88	58	55	58	57	59	55	.	.	.	621	70485
13	W	.	1	28-Jan-87	61	61	61	61	59	58	.	.	.	188	24540
13	W	.	3	6-Apr-88	57	59	57	59	57	55	.	.	.	621	70485
14	E	.	1	28-Jan-87	60	63	64	59	60	59	9	10	65	188	23740
14	E	.	2	21-Jul-87	52	53	54	55	52	51	9	10	65	363	42115
14	E	.	3	6-Apr-88	54	52	51	55	53	51	9	9	65	621	68725
14	E	.	4	28-Sep-88	53	55	51	51	51	50	.	9	.	796	86225
14	E	.	5	27-Apr-89	51	51	50	51	54	51	.	9	.	1007	107325
14	E	.	6	8-Feb-90	58	53	58	56	56	56	.	9	.	1294	136025
14	E	.	7	16-Oct-90	51	53	53	51	53	52	.	.	.	1544	161025
14	E	.	8	11-Jun-91	50	49	48	49	50	49	.	.	.	1782	184825
14	W	.	1	28-Jan-87	62	61	65	65	61	60	.	.	.	188	23740
14	W	.	3	6-Apr-88	53	55	53	52	55	52	.	.	.	621	68725
15	S	I	1	22-Oct-86	64	65	63	63	60	60	.	.	.	84	291900

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
15	S	I	2	8-Jul-87	52	49	50	51	49	48	10	9	70	343	1201375
15	S	I	3	21-Mar-88	40	43	42	41	43	41	10	8	75	599	2101775
15	S	I	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
15	S	I	5	19-Apr-89	39	39	41	38	39	39	10	8	.	978	3484275
16	S	I	1	22-Oct-86	52	54	49	50	50	49	.	.	.	84	291900
16	S	I	2	8-Jul-87	48	50	54	52	49	49	10	10	45	343	1201375
16	S	I	3	21-Mar-88	45	44	50	45	49	45	10	10	60	599	2101775
16	S	I	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
16	S	I	5	19-Apr-89	34	35	35	35	36	35	10	10	.	978	3484275
17	S	I	1	22-Oct-86	51	56	56	55	56	53	.	.	.	84	291900
17	S	I	2	8-Jul-87	46	45	46	47	50	45	10	10	45	343	1201375
17	S	I	3	21-Mar-88	38	39	35	34	37	36	10	10	65	599	2101775
17	S	I	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
17	S	I	5	19-Apr-89	.	.	.	.	.	.	.	.	.	978	3484275
18	S	I	1	22-Oct-86	52	57	53	57	52	52	.	.	.	84	291900
18	S	I	2	8-Jul-87	48	45	44	45	43	44	10	10	35	343	1201775
18	S	I	3	21-Mar-88	32	34	34	34	32	33	10	10	45	599	2101775
18	S	I	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
18	S	I	5	19-Apr-89	24	26	25	28	25	26	10	10	.	978	3484275
19	S	O	1	22-Oct-86	57	58	58	59	53	55	.	.	.	84	291900
19	S	O	2	8-Jul-87	51	45	54	51	53	49	10	8	55	343	1201375

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
19	S	O	3	21-Mar-88	35	32	35	35	34	34	10	6	75	599	2101775
19	S	O	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
19	S	O	5	19-Apr-89	.	.	.	.	.	.	.	.	.	978	3484275
20	S	O	1	22-Oct-86	61	60	55	59	47	54	.	.	.	84	291900
20	S	O	2	8-Jul-87	57	59	58	59	46	54	10	9	70	343	1201375
20	S	O	3	21-Mar-88	39	40	43	41	43	40	10	9	75	599	2101775
20	S	O	4	12-Sep-88	.	.	.	.	.	.	.	.	.	774	2714275
20	S	O	5	19-Apr-89	32	33	34	35	34	34	10	9	.	978	3484275
21	N	O	1	22-Oct-86	39	44	43	46	45	42	.	8	.	88	748000
21	N	O	2	8-Jul-87	30	31	28	29	29	29	10	4	85	347	3044000
21	N	O	3	21-Mar-88	19	20	19	22	18	20	10	2	90	603	5328000
21	N	O	4	12-Sep-88	22	15	10	18	16	17	10	2	90	778	6859250
21	N	O	5	19-Apr-89	14	15	15	16	17	15	10	2	.	982	8644250
22	E	.	2	30-Jun-87	62	63	64	63	60	60	10	10	50	335	155505
22	E	.	4	7-Sep-88	53	69	60	53	54	55	10	9	.	769	345475
22	E	.	5	4-May-89	51	56	57	54	55	55	.	8	.	1007	446575
22	E	.	6	31-Jan-90	61	61	63	63	64	62	.	8	.	1279	562175
23	S	I	1	19-Nov-86	50	46	42	40	41	43	.	.	.	111	153180
23	S	I	2	1-Jul-87	59	63	59	60	60	58	10	10	45	336	473220
23	S	I	4	8-Sep-88	65	68	70	67	71	65	.	9	.	770	1108220
23	S	I	5	4-May-89	64	61	65	61	62	63	.	8	.	1007	1460460

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
23	S	I	6	30-Jan-90	63	64	65	64	64	64	.	8	.	1278	1861540
23	S	I	8	26-Mar-91	71	68	68	70	68	69	.	.	.	1698	2483140
23	S	O	1	19-Nov-86	40	50	47	44	40	43	.	.	.	111	229770
23	S	O	2	1-Jul-87	56	48	52	54	54	51	10	8	65	336	709830
23	S	O	4	8-Sep-88	63	64	60	62	64	60	10	8	.	770	1662330
23	S	O	5	4-May-89	60	60	66	66	60	62	.	8	.	1007	2190690
23	S	O	6	30-Jan-90	63	60	62	61	61	61	.	6	.	1278	2792310
23	S	O	7	3-Oct-90	60	54	56	55	56	56	.	.	.	1524	3338430
23	S	O	8	26-Mar-91	63	57	64	62	66	62	.	.	.	1698	3724710
24	S	I	1	19-Nov-86	39	40	43	46	48	42	.	.	.	111	153180
24	S	I	2	1-Jul-87	59	66	61	58	60	58	10	10	45	336	473220
24	S	I	4	8-Sep-88	68	71	72	69	69	66	.	9	.	770	1108220
24	S	I	5	4-May-89	64	65	64	65	65	65	.	8	.	1007	1460460
24	S	1	6	30-Jan-90	67	67	65	66	64	66	.	8	.	1278	1861540
24	S	I	8	26-Mar-91	69	69	66	70	68	68	.	.	.	1698	2483140
24	S	O	1	19-Nov-86	42	41	41	42	45	41	.	.	.	111	229770
24	S	O	2	1-Jul-87	58	56	58	54	54	54	10	8	70	336	709830
24	S	O	4	8-Sep-88	64	50	65	65	63	59	8	8	.	770	1662330
24	S	O	5	4-May-89	63	65	66	69	64	65	.	8	.	1007	2190690
24	S	O	6	30-Jan-90	62	63	63	63	62	63	.	6	.	1278	2792310
24	S	O	7	3-Oct-90	62	62	62	62	61	62	.	.	.	1524	3338430
24	S	O	8	26-Mar-91	64	64	64	63	59	63	.	.	.	1698	3724710

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
25	SW	.	1	19-Nov-86	52	49	46	44	42	45	.	.	.	114	222300
25	SW	.	2	1-Jul-87	58	56	54	55	53	53	9	10	45	338	722800
25	SW	.	5	4-May-89	54	55	55	53	54	54	9	9	.	1010	2317300
25	SW	.	6	29-Jan-90	55	54	55	55	56	55	.	9	.	1280	2965300
25	SW	.	7	3-Oct-90	55	50	56	54	53	54	.	.	.	1527	3558100
25	SW	.	8	26-Mar-91	57	54	50	57	57	55	.	.	.	1701	3975700
26	E	.	1	19-Nov-86	51	53	55	57	58	53	.	.	.	106	34450
26	E	.	2	29-Jun-87	51	54	54	51	48	50	10	9	35	328	128200
26	E	.	4	7-Sep-88	57	56	56	59	54	54	10	9	.	763	311775
26	E	.	5	4-May-89	55	56	55	52	51	54	10	9	.	1002	408570
26	E	.	7	4-Oct-90	44	43	42	45	41	43	.	.	.	1520	618360
26	E	.	8	27-Mar-91	54	54	55	55	55	55	.	.	.	1841	748365
26	W	.	1	19-Nov-86	53	50	48	48	51	48	.	.	.	106	34450
26	W	.	2	29-Jun-87	54	49	53	50	51	50	10	9	35	328	128200
26	W	.	4	7-Sep-88	60	58	59	59	56	56	10	9	.	763	311775
26	W	.	5	4-May-89	55	54	53	53	55	54	10	9	.	1002	408570
26	W	.	7	4-Oct-90	38	38	41	43	39	40	.	.	.	1520	618360
26	W	.	8	27-Mar-91	51	51	53	55	57	53	.	.	.	1841	748365
27	N	.	1	19-Nov-86	67	63	62	65	73	63	.	.	.	106	243800
27	N	.	2	29-Jun-87	49	50	49	45	42	45	10	9	50	328	790400
27	N	.	4	7-Sep-88	64	61	60	66	65	60	9	6	90	763	1977900
27	N	.	5	4-May-89	61	60	61	61	61	61	9	6	.	1002	2671000

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
27	N	.	6	1-Feb-90	57	59	56	56	56	57	.	6	.	1275	3462700
27	N	.	7	4-Oct-90	43	48	40	44	37	42	.	.	.	1520	4173200
27	N	.	8	27-Mar-91	59	61	63	53	59	59	.	.	.	1841	5104100
27	S	.	1	19-Nov-86	69	73	74	73	68	68	.	.	.	106	243800
27	S	.	2	29-Jun-87	42	47	44	50	48	45	10	9	50	328	790400
27	S	.	4	7-Sep-88	64	63	67	66	63	62	9	6	90	763	1977900
27	S	.	5	4-May-89	54	56	56	53	55	55	9	6	.	1002	2671000
27	S	.	6	1-Feb-90	51	50	51	49	51	50	.	6	.	1275	3462700
27	S	.	7	4-Oct-90	34	30	45	41	43	39	.	.	.	1520	4173200
27	S	.	8	27-Mar-91	59	61	63	53	59	59	.	.	.	1841	5104100
28	N	.	1	19-Nov-86	63	64	62	61	60	59	.	.	.	106	243800
28	N	.	2	29-Jun-87	51	51	51	52	51	49	10	10	50	328	790400
28	N	.	4	7-Sep-88	53	56	55	59	59	54	10	8	80	763	1977900
28	N	.	5	4-May-89	53	53	54	54	55	54	10	8	.	1002	2671000
28	N	.	6	1-Feb-90	54	55	57	55	56	55	.	8	.	1275	3462700
28	N	.	7	4-Oct-90	48	48	49	46	45	47	.	.	.	1520	4173200
28	N	.	8	27-Mar-91	61	59	60	58	57	59	.	.	.	1841	5104100
28	S	.	1	19-Nov-86	59	61	66	61	60	59	.	.	.	106	243800
28	S	.	2	29-Jun-87	51	50	51	51	53	49	10	10	50	328	790400
28	S	.	4	7-Sep-88	49	56	56	55	54	52	10	8	80	763	1977900
28	S	.	5	4-May-89	56	56	55	56	55	56	10	8	.	1002	2671000
28	S	.	6	1-Feb-90	55	58	59	57	59	58	.	8	.	1275	3462700

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
28	S	.	7	4-Oct-90	47	51	51	48	49	49	.	.	.	1520	4173200
28	S	.	8	27-Mar-91	56	53	57	60	57	57	.	.	.	1841	5104100
29	N	.	1	18-Nov-86	40	41	43	46	50	43	.	.	.	130	188500
29	N	.	2	29-Jun-87	47	43	39	38	46	41	9	8	75	353	538850
29	N	.	4	6-Sep-88	53	53	47	36	32	43	9	5	90	787	1245700
29	N	.	5	3-May-89	45	47	39	43	43	43	9	5	.	1026	1640050
29	N	.	6	1-Feb-90	47	42	43	44	44	44	.	5	.	1300	2092150
29	N	.	7	23-Oct-90	39	29	36	42	42	38	.	.	.	1564	2527750
29	N	.	8	27-Mar-91	49	45	41	46	45	45	.	.	.	1719	2783500
29	S	.	1	18-Nov-86	50	45	41	40	42	42	.	.	.	130	188500
29	S	.	2	29-Jun-87	36	32	34	40	37	35	9	8	75	353	538850
29	S	.	4	6-Sep-88	39	44	34	48	40	40	9	5	90	787	1245700
29	S	.	5	3-May-89	38	35	34	39	38	37	9	5	.	1026	1640050
29	S	.	6	1-Feb-90	35	39	38	33	44	38	.	5	.	1300	2092150
29	S	.	7	23-Oct-90	33	34	28	42	34	34	.	.	.	1564	2527750
29	S	.	8	27-Mar-91	36	30	33	35	38	34	.	.	.	1719	2783500
30	N	.	1	18-Nov-86	54	54	55	55	54	52	.	.	.	130	188500
30	N	.	2	29-Jun-87	44	47	44	47	44	44	10	9	50	353	538850
30	N	.	4	6-Sep-88	46	47	57	51	42	47	10	6	75	787	1245700
30	N	.	5	3-May-89	45	45	46	43	44	45	10	5	.	1026	1640050
30	N	.	6	1-Feb-90	46	43	43	46	48	45	.	5	.	1300	2092150
30	N	.	7	23-Oct-90	43	37	44	42	46	42	.	.	.	1564	2527750

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
30	N	.	8	27-Mar-91	48	47	42	48	44	46	.	.	.	1719	2783500
30	S	.	1	18-Nov-86	43	47	50	54	56	48	.	.	.	130	188500
30	S	.	2	29-Jun-87	36	33	32	28	30	32	10	6	70	353	538850
30	S	.	4	6-Sep-88	44	36	33	43	34	37	10	4	90	787	1245700
30	S	.	5	3-May-89	42	42	37	41	36	40	10	2	.	1026	1640050
30	S	.	6	1-Feb-90	44	43	36	40	42	41	.	2	.	1300	2092150
30	S	.	8	27-Mar-91	39	44	40	39	32	39	.	.	.	1719	2783500
31	E	.	1	19-Nov-86	50	45	43	40	39	42	.	.	.	141	155100
31	E	.	2	30-Jun-87	51	55	50	50	52	50	9	9	50	364	427550
31	E	.	4	8-Sep-88	49	54	50	50	59	50	6	8	80	799	1008950
31	E	.	5	4-May-89	48	46	46	47	47	47	6	6	.	1037	1342150
31	E	.	6	31-Jan-90	53	51	53	53	54	53	.	6	.	1309	1722950
31	E	.	7	5-Oct-90	66	51	45	39	60	52	.	.	.	1556	2068750
31	W	.	1	19-Nov-86	42	41	40	41	42	40	.	.	.	141	155100
31	W	.	4	8-Sep-88	53	59	57	55	53	53	.	.	.	799	1008950
32	N	I	1	20-Nov-86	47	52	54	54	55	50	.	10	.	104	76960
32	N	I	2	1-Jul-87	56	56	54	56	57	54	8	10	40	327	241980
32	N	I	4	9-Sep-88	50	54	43	52	53	49	8	10	60	762	573960
32	N	I	5	5-May-89	49	51	51	50	52	51	8	10	.	1000	759600
32	N	I	6	29-Jan-90	51	50	52	52	52	51	.	10	.	1309	970200
32	N	I	7	3-Oct-90	54	53	47	51	54	52	.	.	.	1517	1162860

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
32	N	I	8	26-Mar-91	52	49	51	53	54	52	.	.	.	1691	1298580
32	N	O	1	20-Nov-86	52	52	47	43	43	46	.	10	.	104	115440
32	N	O	2	1-Jul-87	47	47	47	46	47	45	6	10	60	327	362970
32	N	O	4	9-Sep-88	42	39	41	40	40	39	6	8	75	762	860940
32	N	O	5	5-May-89	42	43	39	40	39	41	6	8	.	1000	1139400
32	N	O	6	29-Jan-90	39	37	41	40	41	40	.	8	.	1309	1455300
32	N	O	7	3-Oct-90	38	41	36	36	41	38	.	.	.	1517	1744290
32	N	O	8	26-Mar-91	36	37	38	33	34	36	.	.	.	1691	1947870
33	N	I	1	20-Nov-86	54	54	53	53	52	51	.	10	.	104	76960
33	N	I	2	1-Jul-87	61	61	58	60	60	57	10	10	40	327	241980
33	N	I	4	9-Sep-88	62	62	62	62	62	59	10	10	65	762	573960
33	N	I	5	5-May-89	61	57	58	57	56	58	10	10	.	1000	759600
33	N	I	6	29-Jan-90	64	65	64	62	63	64	.	10	.	1270	970200
33	N	I	7	3-Oct-90	62	61	65	66	63	63	.	.	.	1517	1162860
33	N	I	8	26-Mar-91	65	63	62	63	62	63	.	.	.	1691	1298580
33	N	O	1	20-Nov-86	50	52	54	54	48	50	.	10	.	104	115440
33	N	O	2	1-Jul-87	57	60	62	58	63	57	7	10	50	327	362970
33	N	O	4	9-Sep-88	66	65	63	66	65	62	7	9	65	762	860940
33	N	O	5	5-May-89	66	62	63	63	62	63	7	8	.	1000	1139400
33	N	O	6	29-Jan-90	65	67	66	68	67	67	.	8	.	1270	1455300
33	N	O	7	3-Oct-90	65	67	66	68	67	67	.	.	.	1517	1744290
33	N	O	8	26-Mar-91	70	68	69	70	69	69	.	.	.	1691	1947870

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
34	N	I	1	20-Nov-86	51	49	48	47	46	47	.	.	.	104	76960
34	N	I	2	1-Jul-87	64	66	64	64	66	62	7	10	40	327	241980
34	N	I	4	9-Sep-88	66	63	62	64	64	61	3	9	65	762	573960
34	N	I	5	5-May-89	63	63	61	63	62	62	3	9	.	1000	759600
34	N	I	6	29-Jan-90	64	65	65	64	65	65	.	9	.	1270	970200
34	N	I	7	3-Oct-90	65	66	65	64	61	64	.	.	.	1517	1162860
34	N	I	8	26-Mar-91	67	70	65	68	66	67	.	.	.	1691	1298580
34	N	O	1	20-Nov-86	52	51	47	44	43	46	.	.	.	104	115440
34	N	O	2	1-Jul-87	60	57	57	58	58	56	7	10	50	327	362970
34	N	O	4	9-Sep-88	59	58	52	58	56	54	5	9	65	762	860940
34	N	O	5	5-May-89	58	54	57	57	55	56	5	8	.	1000	1139400
34	N	O	6	29-Jan-90	55	57	54	54	53	55	.	8	.	1270	1455300
34	N	O	7	3-Oct-90	57	55	56	54	56	56	.	.	.	1517	1744290
34	N	O	8	26-Mar-91	52	55	51	53	54	53	.	.	.	1691	1947870
35	N	O	1	19-Nov-86	46	43	42	42	41	42	.	.	.	118	247800
35	N	O	2	30-Jun-87	51	50	52	49	53	49	10	4	85	341	743250
35	N	O	4	7-Sep-88	50	43	37	48	45	43	10	2	90	775	1712250
35	N	O	5	4-May-89	49	45	54	46	43	47	10	2	.	1014	2242830
35	N	O	7	3-Oct-90	40	42	30	29	43	37	.	.	.	1531	3390570
35	N	O	8	27-Mar-91	40	39	52	46	33	42	.	.	.	1706	3779070
35	S	O	1	19-Nov-86	41	41	41	42	41	40	.	.	.	118	247800
35	S	O	2	30-Jun-87	51	51	46	52	51	48	.	.	.	341	743250

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
35	S	O	4	7-Sep-88	42	49	32	33	45	39	.	.	.	775	1712250
35	S	O	5	4-May-89	53	56	55	55	52	54	10	5	.	1014	2242830
35	S	O	6	31-Jan-90	58	42	58	57	48	53	.	5	.	1286	2846670
35	S	O	7	3-Oct-90	47	46	43	51	48	47	.	.	.	1531	3390570
35	S	O	8	27-Mar-91	51	44	57	50	54	51	.	.	.	1706	3779070
36	S	.	1	21-Oct-86	60	57	58	57	51	54	.	10	.	56	156800
36	S	.	2	7-Jul-87	41	41	41	42	41	40	4	10	55	315	891400
36	S	.	3	22-Mar-88	36	39	37	37	39	37	4	9	60	573	1634800
36	S	.	4	13-Sep-88	36	38	33	33	33	34	4	8	65	748	2151050
36	S	.	5	19-Apr-89	33	34	31	33	32	33	4	6	70	966	2794150
36	S	.	6	6-Feb-90	33	37	34	36	35	35	.	6	.	1259	3658500
36	S	.	7	4-Dec-90	30	33	35	36	35	34	.	.	.	1560	4546450
36	S	.	8	14-Mar-91	32	32	33	34	32	33	.	.	.	1660	4841450
37	S	.	1	21-Oct-86	55	55	54	51	54	52	.	.	.	56	156800
37	S	.	2	7-Jul-87	43	44	43	45	44	43	6	9	65	315	891400
37	S	.	3	22-Mar-88	37	41	39	38	42	38	6	8	70	573	1634800
37	S	.	4	13-Sep-88	35	34	31	35	38	34	6	8	70	748	2151050
37	S	.	5	19-Apr-89	35	37	34	35	36	35	6	8	70	966	2794150
37	S	.	6	6-Feb-90	37	36	35	36	40	37	.	8	.	1259	3658500
37	S	.	7	4-Dec-90	31	31	39	42	39	36	.	.	.	1560	4546450
37	S	.	8	14-Mar-91	34	30	35	31	33	33	.	.	.	1660	4841450
38	S	.	1	21-Oct-86	58	53	56	52	54	52	.	.	.	56	156800

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
38	S	.	2	7-Jul-87	43	46	49	46	46	45	7	9	65	315	891400
38	S	.	3	22-Mar-88	42	43	40	44	43	41	7	9	65	573	1634800
38	S	.	4	13-Sep-88	41	40	39	37	40	38	7	9	70	748	2151050
38	S	.	5	19-Apr-89	35	37	38	35	37	36	7	9	70	966	2794150
38	S	.	6	6-Feb-90	38	32	37	42	41	38	.	9	.	1259	3658500
38	S	.	7	4-Dec-90	35	39	39	41	40	39	.	.	.	1560	4546450
38	S	.	8	14-Mar-91	33	36	36	33	35	35	.	.	.	1660	4841450
39	S	.	1	21-Oct-86	46	43	48	47	43	44	.	.	.	56	156800
39	S	.	2	7-Jul-87	37	39	38	37	40	37	9	10	55	315	891400
39	S	.	3	22-Mar-88	33	34	37	35	35	34	9	10	60	573	1634800
39	S	.	4	13-Sep-88	34	35	34	35	34	34	7	10	60	748	2151050
39	S	.	5	19-Apr-89	34	34	29	33	33	33	7	10	60	966	2794150
39	S	.	6	6-Feb-90	34	37	35	35	39	36	.	10	.	1259	3658500
39	S	.	7	4-Dec-90	33	34	36	38	40	36	.	.	.	1560	4546450
39	S	.	8	14-Mar-91	33	30	31	31	32	31	.	.	.	1660	4841450
40	S	.	1	21-Oct-86	66	66	66	65	72	64	.	.	.	56	156800
40	S	.	2	7-Jul-87	64	68	69	67	70	64	10	10	50	315	891400
40	S	.	3	22-Mar-88	69	72	70	68	73	67	10	9	60	573	1634800
40	S	.	4	13-Sep-88	67	68	66	74	72	66	10	9	65	748	2151050
40	S	.	5	19-Apr-89	66	66	65	69	65	66	10	8	80	966	2794150
40	S	.	6	6-Feb-90	64	67	67	69	63	66	.	8	.	1259	3658500
40	S	.	7	4-Dec-90	66	71	66	77	76	71	.	.	.	1560	4546450
40	S	.	8	14-Mar-91	65	67	70	66	67	67	.	.	.	1660	4841450

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
41	S	.	1	21-Oct-86	61	56	62	63	60	58	.	.	.	56	156800
41	S	.	2	7-Jul-87	64	61	60	63	60	59	10	10	50	315	891400
41	S	.	3	22-Mar-88	66	62	65	65	63	61	10	9	60	573	1634800
41	S	.	4	13-Sep-88	64	67	68	67	65	63	10	9	70	748	2151050
41	S	.	5	19-Apr-89	64	63	63	62	62	63	10	9	80	966	2794150
41	S	.	6	6-Feb-90	64	67	67	68	65	66	.	9	.	1259	3658500
41	S	.	7	4-Dec-90	71	72	72	70	70	71	.	.	.	1560	4546450
41	S	.	8	14-Mar-91	68	64	69	68	67	67	.	.	.	1660	4841450
42	S	.	1	21-Oct-86	57	64	63	64	63	59	.	.	.	56	156800
42	S	.	2	7-Jul-87	57	65	59	56	54	56	10	10	50	315	891400
42	S	.	3	22-Mar-88	56	54	49	59	55	52	10	9	60	573	1634800
42	S	.	4	13-Sep-88	59	51	45	48	52	49	10	9	70	748	2151050
42	S	.	5	19-Apr-89	43	39	45	44	39	42	10	9	80	966	2794150
42	S	.	6	6-Feb-90	38	38	33	31	31	34	.	5	.	1295	3658500
42	S	.	7	4-Dec-90	44	29	37	39	29	36	.	.	.	1560	4546450
42	S	.	8	14-Mar-91	37	40	38	36	37	38	.	.	.	1660	4841450
43	S	.	1	21-Oct-86	65	60	64	64	64	61	.	.	.	56	156800
43	S	.	2	7-Jul-87	59	58	62	64	60	58	8	10	50	315	891400
43	S	.	3	22-Mar-88	52	54	55	57	52	52	8	9	60	573	1634800
43	S	.	4	13-Sep-88	64	62	52	59	53	56	8	8	70	748	2151050
43	S	.	5	19-Apr-89	47	51	49	51	49	49	8	7	80	966	2794150
43	S	.	6	6-Feb-90	47	37	42	43	44	43	.	7	.	1259	3658500
43	S	.	7	4-Dec-90	37	46	34	37	42	39	.	.	.	1560	4546450

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
43	S	.	8	14-Mar-91	42	44	36	47	48	43	.	.	.	1660	4841450
44	W	.	1	14-May-87	61	61	61	61	59	58	10	10	.	241	649400
44	W	.	2	6-Jul-87	60	63	52	58	61	56	10	10	.	294	800450
44	W	.	3	22-Mar-88	61	65	63	60	62	59	10	10	.	584	1587750
44	W	.	4	14-Sep-88	64	64	60	62	62	60	10	10	60	729	1950250
44	W	.	5	5-May-89	57	60	60	60	61	60	10	8	.	962	2532750
44	W	.	6	30-Nov-89	54	55	54	55	48	53	.	8	.	1171	3055250
44	W	.	7	2-Oct-90	66	66	63	62	64	64	.	.	.	1477	3820250
44	W	.	8	15-Mar-91	63	60	63	60	62	62	.	.	.	1641	4230250
45	W	.	1	14-May-87	54	52	55	49	51	50	9	9	.	241	649400
45	W	.	2	6-Jul-87	54	49	51	52	54	50	9	9	.	294	800450
45	W	.	3	22-Mar-88	39	37	24	42	36	35	8	4	.	584	1587750
45	W	.	4	14-Sep-88	37	51	55	46	38	44	8	4	85	729	1950250
45	W	.	5	5-May-89	43	34	43	41	29	38	8	4	.	962	2532750
45	W	.	6	30-Nov-89	42	37	25	37	54	39	.	4	.	1171	3055250
45	W	.	7	2-Oct-90	21	22	37	34	30	29	.	.	.	1477	3820250
45	W	.	8	15-Mar-91	31	21	33	19	33	27	.	.	.	1641	4230250
46	W	.	1	14-May-87	60	61	60	58	59	57	10	.	.	241	649400
46	W	.	2	6-Jul-87	60	60	61	59	57	57	10	9	.	294	800450
46	W	.	3	22-Mar-88	57	58	60	59	60	56	9	8	.	584	1587750
46	W	.	4	14-Sep-88	63	62	63	63	61	60	8	7	70	729	1950250
46	W	.	5	5-May-89	61	60	57	56	59	59	8	7	.	962	2532750

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
46	W	.	6	30-Nov-89	49	53	49	50	54	51	.	7	.	1171	3055250
46	W	.	7	2-Oct-90	60	62	57	59	61	60	.	.	.	1477	3820250
46	W	.	8	15-Mar-91	59	60	57	59	62	59	.	.	.	1641	4230250
47	W	.	1	14-May-87	52	52	49	52	50	49	.	.	.	241	649400
47	W	.	2	6-Jul-87	52	51	51	44	51	48	6	6	.	294	800450
47	W	.	3	22-Mar-88	39	39	37	40	37	38	6	4	.	584	1587750
47	W	.	4	14-Sep-88	40	50	42	50	45	44	6	4	80	729	1950250
47	W	.	5	5-May-89	43	45	32	42	32	39	6	4	.	962	2532750
47	W	.	6	30-Nov-89	37	28	34	24	34	31	.	4	.	1171	3055250
47	W	.	7	2-Oct-90	29	27	38	23	20	27	.	.	.	1477	3820250
47	W	.	8	15-Mar-91	32	27	31	31	21	28	.	.	.	1641	4230250
48	E	.	1	14-May-87	59	60	61	59	59	57	10	.	.	240	646900
48	E	.	2	6-Jul-87	61	55	58	61	60	57	10	6	.	293	797950
48	E	.	3	22-Mar-88	58	60	53	61	40	52	10	4	.	583	1585250
48	E	.	4	14-Sep-88	57	61	55	52	59	55	10	4	90	728	1947750
48	E	.	5	5-May-89	53	57	60	57	57	57	10	4	.	961	2530250
48	E	.	6	30-Nov-89	49	50	54	51	56	52	.	4	.	1170	3052750
48	E	.	7	2-Oct-90	53	54	59	49	56	54	.	.	.	1476	3817750
48	E	.	8	15-Mar-91	45	47	44	50	55	48	.	.	.	1640	4227750
49	E	.	1	14-May-87	62	61	59	61	61	58	10	.	.	240	646900
49	E	.	2	6-Jul-87	58	58	58	60	51	55	10	8	.	293	797950
49	E	.	3	22-Mar-88	56	58	61	60	59	56	9	6	.	583	1585250

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
49	E	.	4	14-Sep-88	43	48	60	61	60	52	9	4	80	728	1947750
49	E	.	5	5-May-89	33	56	56	59	59	53	.	4	.	961	2530250
49	E	.	6	30-Nov-89	49	46	50	49	50	49	.	4	.	1170	3052750
49	E	.	7	2-Oct-90	49	47	44	45	52	47	.	.	.	1476	3817750
49	E	.	8	15-Mar-91	59	59	63	61	60	60	.	.	.	1640	4227750
50	N	.	3	4-Apr-88	59	63	50	46	46	51	.	.	.	240	229050
50	N	.	4	26-Sep-88	46	57	46	44	55	48	10	4	90	415	404050
50	N	.	5	24-Apr-89	31	31	32	30	39	33	.	2	.	625	614050
50	N	.	6	6-Nov-89	27	41	27	32	27	31	.	2	.	821	810050
50	N	.	7	8-Oct-90	26	33	35	28	37	32	.	.	.	1157	1146050
51	N	.	3	4-Apr-88	62	59	59	57	58	57	.	.	.	240	229050
51	N	.	4	26-Sep-88	56	62	60	62	63	58	10	7	70	415	404050
51	N	.	5	24-Apr-89	54	59	57	60	56	57	.	7	.	625	614050
51	N	.	6	6-Nov-89	58	58	55	60	60	58	.	7	.	821	810050
51	N	.	7	8-Oct-90	60	56	57	49	55	55	.	.	.	1157	1146050
52	N	.	3	4-Apr-88	58	60	56	58	62	56	.	.	.	240	229050
52	N	.	4	26-Sep-88	63	62	63	66	64	61	10	8	70	415	404050
52	N	.	5	24-Apr-89	56	60	59	56	60	58	.	8	.	625	614050
52	N	.	6	6-Nov-89	65	60	61	58	60	61	.	8	.	821	810050
52	N	.	7	8-Oct-90	60	65	62	53	63	61	.	.	.	1157	1146050
53	N	.	3	4-Apr-88	53	52	54	53	49	50	.	.	.	240	229050

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
53	N	.	4	26-Sep-88	53	56	52	58	55	53	10	4	80	415	404050
53	N	.	5	24-Apr-89	39	39	41	43	43	41	.	4	.	625	614050
53	N	.	6	6-Nov-89	45	46	46	48	46	46	.	4	.	821	810050
53	N	.	7	8-Oct-90	43	45	43	45	43	44	.	.	.	1157	1146050
54	N	.	7	8-Oct-90	50	45	45	46	46	46	.	.	.	1157	1146050
54	S	.	3	4-Apr-88	56	51	54	53	58	52	.	.	.	240	229050
54	S	.	4	26-Sep-88	58	57	58	58	57	55	.	8	.	415	404050
54	S	.	5	24-Apr-89	46	49	51	43	50	48	.	8	.	625	614050
54	S	.	6	6-Nov-89	48	48	51	49	50	49	.	8	.	821	810050
55	S	O	5	24-Apr-89	38	41	42	37	44	40	.	.	.	628	1101650
55	S	O	6	6-Nov-89	47	50	44	48	50	48	.	.	.	824	1415250
55	S	O	7	8-Oct-90	38	41	47	42	42	42	.	.	.	1160	1684050
55	S	O	8	21-May-91	36	38	45	47	43	42	.	.	.	1385	1864050
56	E	I	4	27-Sep-88	56	57	56	57	56	54	.	10	.	412	609200
56	E	I	5	25-Apr-89	55	53	56	56	53	55	.	10	.	652	982700
56	E	I	6	7-Nov-89	54	51	49	50	54	52	.	10	.	848	1292380
56	E	I	7	110CT90	53	50	54	52	50	52	.	.	.	1186	1826420
56	E	I	8	21-May-91	64	61	57	58	53	59	.	.	.	1408	2177180
56	E	O	4	27-Sep-88	53	52	50	52	50	50	.	10	.	412	913800
56	E	O	5	25-Apr-89	48	50	49	48	50	49	.	10	.	652	1474050

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
56	E	O	6	7-Nov-89	48	49	49	49	53	50	.	10	.	848	1938570
56	E	O	7	11OCT90	57	50	54	51	56	54	.	.	.	1186	2739630
56	E	O	8	21-May-91	53	54	53	57	55	54	.	.	.	1408	3265770
57	E	I	4	27-Sep-88	56	56	55	54	55	53	.	10	.	412	609200
57	E	I	5	25-Apr-89	56	56	54	54	55	55	.	10	.	652	982700
57	E	I	6	7-Nov-89	46	49	54	54	49	50	.	10	.	848	1292380
57	E	I	7	11OCT90	44	56	43	55	50	50	.	.	.	1186	1826420
57	E	I	8	21-May-91	54	60	57	60	59	58	.	.	.	1408	2177180
57	E	O	4	27-Sep-88	53	53	51	53	52	50	.	10	.	412	913800
57	E	O	5	25-Apr-89	46	50	48	50	50	49	.	9	.	652	1474050
57	E	O	6	7-Nov-89	50	49	52	46	49	49	.	9	.	848	1938570
57	E	O	7	11OCT90	55	52	54	55	54	54	.	.	.	1186	2739630
57	E	O	8	21-May-91	59	49	56	53	52	54	.	.	.	1408	3265770
58	E	I	4	27-Sep-88	49	47	38	34	38	40	.	.	.	412	609200
58	E	I	5	25-Apr-89	46	47	38	32	40	41	.	8	.	652	982700
58	E	I	6	7-Nov-89	45	36	47	41	42	42	.	8	.	848	1292380
58	E	I	7	11OCT90	52	53	51	52	51	52	.	.	.	1186	1826420
58	E	I	8	21-May-91	52	49	51	46	45	49	.	.	.	1408	2177180
58	E	O	4	27-Sep-88	38	40	37	38	38	37	.	.	.	412	913800
58	E	O	5	25-Apr-89	34	37	35	37	37	36	.	8	.	652	1474050
58	E	O	6	7-Nov-89	47	41	41	48	46	45	.	8	.	848	1938570

Table B.1 Skid Resistance Test Data (Continued)

Sec.	B	L	R	Testing Date	FN1	FN2	FN3	FN4	FN5	FN	AGRT	BLG	AGEB	AGE	Acc. Traffic
58	E	O	7	11OCT90	55	56	59	53	59	56	.	.	.	1186	2739630
58	E	O	8	21-May-91	51	48	42	48	42	46	.	.	.	1408	3265770
59	E	I	4	27-Sep-88	57	59	56	56	56	55	.	.	.	412	609200
59	E	I	5	25-Apr-89	57	55	55	55	54	55	.	10	.	652	982700
59	E	I	6	7-Nov-89	55	47	57	51	57	53	.	10	.	848	1292380
59	E	I	7	11OCT90	50	53	49	53	57	52	.	.	.	1186	1826420
59	E	I	8	21-May-91	54	61	58	60	56	58	.	.	.	1408	2177180
59	E	O	4	27-Sep-88	47	52	45	45	45	45	.	.	.	412	913800
59	E	O	5	25-Apr-89	52	53	50	48	53	51	.	7	.	652	1474050
59	E	O	6	7-Nov-89	45	48	52	50	54	50	.	7	.	848	1938570
59	E	O	7	11OCT90	56	48	55	48	55	52	.	.	.	1186	2739630
59	E	O	8	21-May-91	43	55	45	59	52	51	.	.	.	1408	3265770

Table B.2 British Pendulum Test Data

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
1	E	.	2	01JUL87	56	60	49	53	61	55.8	9	10	40	352	114460
1	E	.	3	15JUN88	56	50	58	64	55	56.6	9	10	60	701	224650
2	S	.	2	01JUL87	42	53	47	47	45	46.8	10	10	50	352	167200
2	S	.	3	15JUN88	45	42	46	45	43	44.2	10	10	60	701	327995
3	N	.	2	01JUL87	63	75	71	85	67	72.2	10	10	55	359	135285
3	N	.	3	15JUN88	79	78	73	67	72	73.8	10	9	70	708	252115
4	E	.	2	01JUL87	74	72	82	78	76	76.4	9	10	45	376	470000
4	E	.	3	15JUN88	76	72	80	80	78	77.2	9	9	55	725	897950
5	W	.	1	19MAR87	91	85	82	87	100	89.0	10	10	40	238	289500
5	W	.	2	25AUG87	84	84	92	85	80	85.0	10	10	50	397	488250
5	W	.	3	16JUN88	72	75	77	78	82	76.8	10	9	75	692	857000
6	W	.	1	19MAR87	53	59	52	62	51	55.4	9	10	40	238	289500
6	W	.	2	25AUG87	45	51	46	51	48	48.2	9	10	40	397	488250
6	W	.	3	16JUN88	48	40	38	38	41	41.0	9	10	55	692	857000
7	W	.	1	19MAR87	66	62	67	59	68	64.4	9	10	40	238	289500
7	W	.	2	25AUG87	62	49	65	54	57	57.4	9	10	45	397	488250
7	W	.	3	16JUN88	48	46	57	52	48	50.2	9	9	55	692	857000
8	N	.	1	19MAR87	81	70	88	91	72	80.4	10	10	40	238	26590
8	N	.	2	26AUG87	70	69	82	71	71	72.6	10	10	50	398	43390
8	N	.	3	29JUN88	76	97	83	87	90	86.6	10	10	65	705	74725

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
9	N	.	1	19MAR87	79	74	76	79	79	77.4	9	10	60	238	26590
9	N	.	2	26AUG87	64	69	71	69	71	68.8	9	10	60	398	43390
9	N	.	3	29JUN88	89	98	81	84	89	88.2	9	10	60	705	74725
10	N	.	1	19MAR87	75	70	76	86	70	75.4	10	10	65	238	26590
10	N	.	2	26AUG87	51	55	62	53	56	55.4	10	10	65	398	43390
10	N	.	3	29JUN88	72	67	70	78	66	70.6	10	10	65	705	74725
11	N	.	1	19MAR87	105	83	96	98	91	94.6	9	10	65	238	26590
11	N	.	2	26AUG87	71	70	77	75	85	75.6	9	10	65	398	43390
11	N	.	3	29JUN88	90	101	88	74	92	89.0	9	10	65	705	74725
12	N	.	1	19MAR87	84	86	69	81	91	82.2	9	10	65	238	26590
12	N	.	2	26AUG87	57	64	77	57	57	62.4	9	10	65	398	43390
12	N	.	3	29JUN88	91	78	93	93	98	90.6	9	10	65	705	74725
13	E	.	1	19MAR87	77	81	70	84	70	76.4	9	10	55	238	29790
13	E	.	2	26AUG87	75	69	63	75	75	71.4	9	10	65	398	46590
14	E	.	1	19MAR87	74	86	92	86	91	85.8	9	10	65	238	28990
14	E	.	2	26AUG87	71	72	64	65	71	68.6	9	10	65	398	45790
14	E	.	3	29JUN88	57	78	89	93	79	79.2	9	9	65	705	77125

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
15	S	I	1	22OCT86	72	70	79	72	66	71.8	.	.	.	84	291900
15	S	I	2	18AUG87	64	48	65	52	60	57.8	10	9	70	374	1310650
15	S	I	3	12JUL88	55	66	63	60	64	61.6	10	8	75	712	2497275
16	S	I	1	22OCT86	60	62	64	70	66	64.4	.	.	.	84	291900
16	S	I	2	18AUG87	44	47	51	52	48	48.4	10	10	45	374	1310650
16	S	I	3	12JUL88	70	70	70	70	69	69.8	10	10	60	712	2497275
17	S	I	1	22OCT86	66	61	56	62	56	60.2	.	.	.	84	291900
17	S	I	2	18AUG87	44	42	46	43	42	43.4	10	10	45	374	1310650
17	S	I	3	12JUL88	60	64	50	50	52	55.2	10	10	65	712	2497275
18	S	I	1	22OCT86	67	48	55	54	48	54.4	.	.	.	84	291900
18	S	I	2	18AUG87	45	38	45	44	49	44.2	10	10	35	374	1310650
18	S	I	3	12JUL88	54	43	45	50	47	47.8	10	10	45	712	2497275
19	S	O	1	22OCT86	64	62	64	67	65	64.4	.	.	.	84	291900
19	S	O	2	18AUG87	55	70	64	62	53	60.8	10	8	55	374	1310650
19	S	O	3	12JUL88	50	63	54	48	54	53.8	10	6	75	712	2497275
20	S	O	1	22OCT86	70	74	90	86	75	79.0	.	.	.	84	291900
20	S	O	2	18AUG87	51	56	56	51	51	53.0	10	9	70	374	1310650
20	S	O	3	12JUL88	44	50	52	54	51	50.2	10	9	75	712	2497275
21	N	O	1	22OCT86	60	60	62	63	61	61.2	.	8	.	88	748000
22	E	.	4	17AUG88	85	90	89	82	86	86.4	10	9	.	747	336075

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
23	S	I	2	28JUL87	80	87	79	67	80	78.6	10	10	45	362	512100
23	S	O	2	28JUL87	81	86	83	91	63	80.8	10	8	65	362	768150
23	S	O	4	17AUG88	81	89	84	82	81	83.4	10	8	.	747	1613490
24	S	I	2	28JUL87	82	87	92	86	78	85.0	10	10	45	362	512100
24	S	O	2	28JUL87	81	85	74	75	75	78.0	10	8	70	362	768150
24	S	O	4	17AUG88	86	82	83	80	82	82.6	8	8	.	747	1613490
25	SW	.	2	28JUL87	64	72	65	72	58	66.2	9	10	45	365	748900
26	W	.	2	07JUL87	63	69	69	54	67	64.4	10	9	35	336	131760
26	W	.	4	16AUG88							10	9	.	741	302865
27	N	.	2	07JUL87	65	75	51	60	73	64.8	10	9	50	336	810400
27	N	.	4	16AUG88	67	65	74	68	66	68.0	9	6	90	741	1914100
28	N	.	2	07JUL87	66	71	72	72	69	70.0	10	10	50	336	810400
28	N	.	4	16AUG88	79	85	84	85	85	83.6	10	8	80	741	1914100
29	N	.	2	06JUL87	70	50	60	60	51	58.2	9	8	75	360	550050
29	N	.	4	16AUG88	66	77	79	74	81	75.4	9	5	90	766	1211050
30	N	.	2	06JUL87	55	55	65	62	62	59.8	10	9	50	360	550050
30	S	.	4	16AUG88	77	70	79	81	75	76.4	10	4	90	766	1211050

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
31	E	.	2	27JUL87	45	64	67	57	52	57.0	9	9	50	391	461300
31	E	.	4	17AUG88	66	83	89	83	91	82.4	6	8	80	777	978150
32	N	I	1	07JAN87	85	64	72	72	69	72.4	.	10	.	152	112480
32	N	I	2	05AUG87	66	65	59	64	61	63.0	8	10	40	362	267880
32	N	I	4	18JUL88	77	72	76	70	71	73.2	8	10	60	709	532620
32	N	O	1	07JAN87	75	68	69	56	72	68.0	.	10	.	152	168720
32	N	O	2	05AUG87	64	70	64	75	72	69.0	6	10	60	362	401820
32	N	O	4	18JUL88	56	62	65	57	67	61.4	6	8	75	709	798930
33	N	I	1	07JAN87	75	92	86	72	79	80.8	10	10	.	152	112480
33	N	I	2	05AUG87	89	90	84	83	91	87.4	10	10	40	362	267880
33	N	I	4	18JUL88	94	94	98	93	91	94.0	10	10	65	709	532620
33	N	O	1	07JAN87	89	90	96	89	90	90.8	.	10	.	152	168720
33	N	O	2	05AUG87	87	72	79	81	76	79.0	7	10	50	362	401820
33	N	O	4	18JUL88	97	83	89	90	95	90.8	7	9	65	709	798930
34	N	I	1	07JAN87	78	90	84	81	82	83.0	.	10	.	152	112480
34	N	I	2	05AUG87	72	75	83	77	69	75.2	7	10	40	362	267880
34	N	I	4	18JUL88	95	88	94	91	89	91.4	3	9	65	709	532620
34	N	O	1	07JAN87	80	76	80	80	95	82.2	.	10	.	152	168720
34	N	O	2	05AUG87	67	70	66	59	66	65.6	7	10	50	362	401820
34	N	O	4	18JUL88	85	72	76	78	79	78.0	5	9	65	709	798930

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
35	N	O	2	27JUL87	75	64	75	57	66	67.4	10	4	85	368	804000
35	N	O	3	17AUG88	80	84	68	75	70	75.4	10	2	90	754	1665630
36	S	.	1	21OCT86	44	60	67	70	64	61.0	.	10	.	56	156800
36	S	.	2	17AUG87	40	48	57	54	46	49.0	4	10	55	356	1013238
36	S	.	4	06JUL88	53	56	52	59	54	54.8	4	8	65	679	1947500
37	S	.	1	21OCT86	60	69	65	59	61	62.8	.	.	.	56	156800
37	S	.	2	17AUG87	48	44	46	50	39	45.4	6	9	65	356	1013238
37	S	.	4	06JUL88	58	56	53	58	62	57.4	6	8	70	679	1947500
38	S	.	1	21OCT86	56	60	62	63	67	61.6	.	.	.	56	156800
38	S	.	2	17AUG87	51	44	51	41	48	47.0	7	9	65	356	1013238
38	S	.	4	06JUL88	56	55	58	53	53	55.0	7	9	70	679	1947500
39	S	.	1	21OCT86	56	55	54	41	61	53.4	.	10	.	56	156800
39	S	.	2	17AUG87	39	38	39	49	49	42.8	9	10	55	356	1013238
39	S	.	4	06JUL88	55	52	51	61	54	54.6	7	10	60	679	1947500
40	S	.	1	21OCT86	92	106	101	80	97	95.2	10	10	.	56	156800
40	S	.	2	17AUG87	85	82	80	85	84	83.2	10	10	50	356	1013238
40	S	.	4	06JUL88	91	85	84	88	95	88.6	10	9	65	679	1947500
41	S	.	1	21OCT86	86	78	78	87	74	80.6	10	10	.	56	156800
41	S	.	2	17AUG87	87	68	66	68	81	74.0	10	10	50	356	1013238
41	S	.	4	06JUL88	86	90	87	85	89	87.4	10	9	70	679	1947500

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
42	S	.	1	21OCT86	78	71	78	65	66	71.6	.	10	.	56	156800
42	S	.	2	17AUG87	58	65	70	58	61	62.4	8	10	50	356	1013238
42	S	.	4	06JUL88	74	79	70	74	70	73.4	6	8	75	679	1947500
43	S	.	1	21OCT86	85	86	91	82	88	86.4	.	10	.	56	156800
43	S	.	2	17AUG87	67	63	74	68	67	67.8	8	10	50	356	1013238
43	S	.	4	06JUL88	75	77	74	78	75	75.8	8	8	70	679	1947500
44	W	.	2	17JUN87	84	82	97	77	82	84.4	10	10	.	275	746300
44	W	.	4	26JUL88	74	77	65	75	75	73.2	10	10	60	679	1600250
45	W	.	2	17JUN87	76	71	73	70	69	71.8	9	9	.	275	746300
45	W	.	4	26JUL88	52	52	42	61	55	52.4	8	4	85	679	1600250
46	W	.	2	17JUN87	82	86	99	90	99	91.2	10	9	.	275	746300
46	W	.	4	26JUL88	75	70	74	72	75	73.2	8	7	70	679	1600250
47	W	.	2	17JUN87	65	72	69	72	59	67.4	6	6	.	275	746300
47	W	.	4	26JUL88	60	54	49	53	45	52.2	6	4	.	679	1600250
48	E	.	2	17JUN87	85	76	98	82	75	83.2	10	6		275	746300
48	E	.	4	26JUL88	64	54	58	62	70	61.6	10	4	90	679	1600250
49	E	.	2	17JUN87	77	69	75	64	71	71.2	10	8	.	275	746300
49	E	.	4	26JUL88	72	70	65	75	77	71.8	9	4	80	679	1600250

(continued)

Table B.2 British Pendulum Test Data (Continued)

Sec.	B	L	R	Testing Date	BPN1	BPN2	BPN3	BPN4	BPN5	BPN	AGRT	BLG	AGEB	Age	Acc. Traffic
50	N	.	4	24AUG88	72	66	78	68	71	71.0	10	4	90	382	371050
51	N	.	4	24AUG88	89	85	94	88	96	90.4	10	6	70	382	371050
52	N	.	4	24AUG88	92	100	87	88	95	92.4	10	8	70	382	371050
53	N	.	4	24AUG88	79	72	80	75	76	76.4	10	4	80	382	371050

Table B.3 Sand Patch Test Data

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
1	E	.	2	01JUL87	0.094	0.086	0.098	0.113	0.098	0.098	9	10	40	352	114460
1	E	.	3	15JUN88	0.071	0.074	0.077	0.065	0.071	0.072	9	10	60	701	224650
2	S	.	2	01JUL87	0.083	0.083	0.077	0.083	0.083	0.082	10	10	50	352	167200
2	S	.	3	15JUN88	0.083	0.061	0.055	0.077	0.077	0.071	10	10	60	701	327995
3	N	.	2	01JUL87	0.074	0.068	0.061	0.071	0.068	0.068	10	10	55	359	135285
3	N	.	3	15JUN88	0.045	0.050	0.044	0.047	0.044	0.046	10	9	70	708	252115
4	E	.	2	01JUL87	0.077	0.080	0.065	0.083	0.083	0.078	9	10	45	376	470000
4	E	.	3	15JUN88	0.071	0.074	0.077	0.065	0.071	0.072	9	9	55	725	897950
5	W	.	1	19MAR87	0.083	0.086	0.077	0.108	0.094	0.090	10	10	40	238	289500
5	W	.	2	25AUG87	0.059	0.068	0.080	0.071	0.098	0.075	10	10	50	397	488250
5	W	.	3	16JUN88	0.065	0.061	0.074	0.055	0.059	0.063	10	9	75	692	857000
6	W	.	1	19MAR87	0.126	0.113	0.119	0.103	0.119	0.116	9	10	40	238	289500
6	W	.	2	25AUG87	0.086	0.094	0.094	0.080	0.108	0.092	9	10	40	397	488250
6	W	.	3	16JUN88	0.103	0.103	0.083	0.098	0.108	0.099	9	10	55	692	857000
7	W	.	1	19MAR87	0.108	0.119	0.108	0.132	0.139	0.121	9	10	40	238	289500
7	W	.	2	25AUG87	0.103	0.098	0.086	0.094	0.080	0.092	9	10	45	397	488250
7	W	.	3	16JUN88	0.086	0.090	0.080	0.074	0.086	0.083	9	9	55	692	857000

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
8	N	.	1	19MAR87	0.083	0.090	0.094	0.098	0.094	0.092	10	10	40	238	26590
8	N	.	2	26AUG87	0.086	0.077	0.077	0.071	0.083	0.079	10	10	50	398	43390
8	N	.	3	29JUN88	0.103	0.108	0.108	0.108	0.119	0.109	10	10	65	705	74725
9	N	.	1	19MAR87	0.103	0.094	0.108	0.080	0.083	0.094	9	10	60	238	26590
9	N	.	2	26AUG87	0.074	0.080	0.083	0.071	0.103	0.082	9	10	60	398	43390
9	N	.	3	29JUN88	0.119	0.113	0.103	0.125	0.094	0.111	9	10	60	705	74725
10	N	.	1	19MAR87	0.098	0.086	0.113	0.086	0.083	0.093	10	10	65	238	26590
10	N	.	2	26AUG87	0.068	0.077	0.080	0.077	0.061	0.073	10	10	65	398	43390
10	N	.	3	29JUN88	0.090	0.094	0.083	0.077	0.077	0.084	10	10	65	705	74725
11	N	.	1	19MAR87	0.086	0.077	0.074	0.077	0.103	0.083	9	10	65	238	26590
11	N	.	2	26AUG87	0.077	0.080	0.083	0.080	0.083	0.081	9	10	65	398	43390
11	N	.	3	29JUN88	0.077	0.083	0.083	0.094	0.094	0.086	9	10	65	705	74725
12	N	.	1	19MAR87	0.074	0.077	0.061	0.068	0.077	0.071	9	10	65	238	26590
12	N	.	2	26AUG87	0.065	0.090	0.053	0.068	0.077	0.071	9	10	65	398	43390
12	N	.	3	29JUN88	0.080	0.094	0.090	0.077	0.090	0.086	9	10	65	705	74725
13	E	.	1	19MAR87	0.086	0.086	0.119	0.103	0.113	0.101	9	10	55	238	29790
13	E	.	2	26AUG87	0.071	0.077	0.086	0.090	0.094	0.084	9	10	65	398	46590

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
14	E	.	1	19MAR87	0.113	0.108	0.086	0.086	0.090	0.097	9	10	65	238	28990
14	E	.	2	26AUG87	0.077	0.068	0.071	0.065	0.080	0.072	9	10	65	398	45790
14	E	.	3	29JUN88	0.090	0.083	0.083	0.094	0.094	0.089	9	9	65	705	77125
15	S	I	1	22OCT86	0.077	0.077	0.083	0.071	0.063	0.074	.	.	.	84	291900
15	S	I	2	18AUG87	0.039	0.039	0.050	0.043	0.043	0.043	10	9	70	374	1310650
15	S	I	3	12JUL88	0.033	0.036	0.039	0.035	0.038	0.036	10	8	75	712	2497275
16	S	I	1	22OCT86	0.094	0.083	0.083	0.094	0.080	0.087	.	.	.	84	291900
16	S	I	2	18AUG87	0.057	0.074	0.063	0.071	0.059	0.065	10	10	45	374	1310650
16	S	I	3	12JUL88	0.077	0.080	0.083	0.077	0.071	0.078	10	10	60	712	2497275
17	S	I	1	22OCT86	0.086	0.094	0.086	0.086	0.090	0.088	.	.	.	84	291900
17	S	I	2	18AUG87	0.048	0.068	0.050	0.080	0.061	0.061	10	10	45	374	1310650
17	S	I	3	12JUL88	0.077	0.063	0.074	0.071	0.065	0.070	10	10	65	712	2497275
18	S	I	1	22OCT86	0.086	0.086	0.090	0.077	0.080	0.084	.	.	.	84	291900
18	S	I	2	18AUG87	0.080	0.083	0.074	0.068	0.090	0.079	10	10	35	374	1310650
18	S	I	3	12JUL88	0.125	0.094	0.108	0.113	0.094	0.107	10	10	45	712	2497275
19	S	O	1	22OCT86	0.090	0.086	0.083	0.077	0.083	0.084	.	.	.	84	291900
19	S	O	2	18AUG87	0.065	0.065	0.059	0.057	0.059	0.061	10	8	55	374	1310650
19	S	O	3	12JUL88	0.053	0.057	0.045	0.051	0.043	0.050	10	6	75	712	2497275

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
20	S	O	1	22OCT86	0.065	0.059	0.063	0.065	0.068	0.064	.	.	.	84	291900
20	S	O	2	18AUG87	0.038	0.043	0.045	0.044	0.038	0.042	10	9	70	374	1310650
20	S	O	3	12JUL88	0.041	0.039	0.043	0.040	0.040	0.041	10	9	75	712	2497275
21	N	O	1	22OCT86	0.039	0.041	0.045	0.045	0.047	0.043	.	8	.	88	748000
22	E	.	4	17AUG88	0.080	0.113	0.090	0.074	0.098	0.091	10	9	.	747	336075
23	S	I	2	28JUL87	0.080	0.077	0.090	0.094	0.083	0.085	10	10	45	362	512100
23	S	O	2	28JUL87	0.059	0.065	0.053	0.047	0.047	0.054	10	8	65	362	768150
23	S	O	4	17AUG88	0.065	0.059	0.064	0.041	0.041	0.054	10	8	.	747	1613490
24	S	I	2	28JUL87	0.077	0.080	0.071	0.086	0.094	0.082	10	10	45	362	512100
24	S	O	2	28JUL87	0.051	0.047	0.059	0.055	0.061	0.055	10	8	70	362	768150
24	S	O	4	17AUG88	0.063	0.048	0.063	0.063	0.077	0.063	8	8	.	747	1613490
25	SW	.	2	28JUL87	0.080	0.090	0.083	0.080	0.098	0.086	9	10	45	365	748900
26	W	.	2	07JUL87	0.098	0.098	0.077	0.119	0.094	0.097	10	9	35	336	131760
27	N	.	2	07JUL87	0.063	0.083	0.083	0.077	0.074	0.076	10	9	50	336	810400
27	N	.	4	16AUG88	0.030	0.023	0.030	0.024	0.033	0.028	9	6	90	741	1914100

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
28	N	.	2	07JUL87	0.090	0.090	0.103	0.119	0.098	0.100	10	10	50	336	810400
28	N	.	4	16AUG88	0.098	0.113	0.119	0.098	0.103	0.106	10	8	80	741	1914100
29	N	.	2	06JUL87	0.108	0.074	0.098	0.086	0.083	0.090	9	8	75	360	550050
29	N	.	4	16AUG88	0.061	0.039	0.063	0.050	0.057	0.054	9	5	90	766	1211050
30	N	.	2	06JUL87	0.086	0.113	0.113	0.113	0.108	0.107	10	9	50	360	550050
30	S	.	4	16AUG88	0.051	0.048	0.048	0.053	0.061	0.052	10	4	90	766	1211050
31	E	.	2	27JUL87	0.086	0.077	0.080	0.077	0.083	0.081	9	9	50	391	461300
31	E	.	4	17AUG88	0.108	0.098	0.113	0.098	0.119	0.107	6	8	80	777	978150
32	N	I	1	07JAN87	0.157	0.132	0.156	0.139	0.175	0.152	.	10	.	152	112480
32	N	I	2	05AUG87	0.086	0.077	0.090	0.068	0.094	0.083	8	10	40	362	267880
32	N	I	4	18JUL88	0.094	0.108	0.094	0.108	0.094	0.100	8	10	60	709	532620
32	N	O	1	07JAN87	0.094	0.094	0.139	0.113	0.125	0.113	.	10	.	152	168720
32	N	O	2	05AUG87	0.080	0.086	0.077	0.080	0.080	0.081	6	10	60	362	401820
32	N	O	4	18JUL88	0.074	0.077	0.080	0.071	0.077	0.076	6	8	75	709	798930
33	N	I	1	07JAN87	0.147	0.156	0.156	0.175	0.156	0.158	10	10	.	152	112480
33	N	I	2	05AUG87	0.103	0.083	0.083	0.090	0.086	0.089	10	10	40	362	267880
33	N	I	4	18JUL88	0.094	0.113	0.108	0.108	0.098	0.104	10	10	65	709	532620

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
33	N	O	1	07JAN87	0.090	0.119	0.113	0.113	0.125	0.112	.	10	.	152	168720
33	N	O	2	05AUG87	0.055	0.068	0.077	0.063	0.083	0.069	7	10	50	362	401820
33	N	O	4	18JUL88	0.077	0.077	0.080	0.086	0.083	0.081	7	9	65	709	798930
34	N	I	1	07JAN87	0.147	0.165	0.186	0.175	0.156	0.166	.	10	.	152	112480
34	N	I	2	05AUG87	0.090	0.080	0.077	0.083	0.080	0.082	7	10	40	362	267880
34	N	I	4	18JUL88	0.080	0.086	0.077	0.080	0.094	0.083	3	9	65	709	532620
34	N	O	1	07JAN87	0.147	0.132	0.119	0.139	0.132	0.134	.	10	.	152	168720
34	N	O	2	05AUG87	0.065	0.077	0.059	0.061	0.068	0.066	7	10	50	362	401820
34	N	O	4	18JUL88	0.074	0.065	0.074	0.068	0.077	0.072	5	9	65	709	798930
35	N	O	2	27JUL87	0.045	0.051	0.057	0.061	0.059	0.055	10	4	85	368	804000
35	N	O	3	17AUG88	0.021	0.063	0.034	0.041	0.040	0.040	10	2	90	754	1665630
36	S	.	1	21OCT86	0.132	0.147	0.126	0.140	0.132	0.135	.	10	.	56	156800
36	S	.	2	17AUG87	0.068	0.063	0.074	0.071	0.057	0.067	4	10	55	356	1013238
36	S	.	4	06JUL88	0.063	0.063	0.068	0.063	0.057	0.063	4	8	65	679	1947500
37	S	.	1	21OCT86	0.119	0.119	0.108	0.113	0.119	0.116				56	156800
37	S	.	2	17AUG87	0.080	0.047	0.053	0.077	0.047	0.061	6	9	65	356	1013238
37	S	.	4	06JUL88	0.063	0.059	0.055	0.057	0.063	0.059	6	8	70	679	1947500

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
38	S	.	1	21OCT86	0.165	0.140	0.113	0.119	0.113	0.130	.	.	.	56	156800
38	S	.	2	17AUG87	0.047	0.071	0.061	0.080	0.074	0.067	7	9	65	356	1013238
38	S	.	4	06JUL88	0.063	0.077	0.059	0.065	0.074	0.068	7	9	70	679	1947500
39	S	.	1	21OCT86	0.140	0.132	0.199	0.165	0.156	0.158	.	10	.	56	156800
39	S	.	2	17AUG87	0.063	0.071	0.074	0.068	0.061	0.067	9	10	55	356	1013238
39	S	.	4	06JUL88	0.071	0.077	0.090	0.083	0.080	0.080	7	10	60	679	1947500
40	S	.	1	21OCT86	0.147	0.126	0.119	0.103	0.126	0.124	10	10	.	56	156800
40	S	.	2	17AUG87	0.045	0.038	0.043	0.057	0.044	0.045	10	10	50	356	1013238
40	S	.	4	06JUL88	0.053	0.057	0.053	0.044	0.050	0.051	10	9	65	679	1947500
41	S	.	1	21OCT86	0.132	0.132	0.132	0.126	0.156	0.136	10	10	.	56	156800
41	S	.	2	17AUG87	0.041	0.041	0.047	0.041	0.045	0.043	10	10	50	356	1013238
41	S	.	4	06JUL88	0.063	0.055	0.051	0.051	0.055	0.055	10	9	70	679	1947500
42	S	.	1	21OCT86	0.140	0.098	0.140	0.112	0.140	0.126	.	10	.	56	156800
42	S	.	2	17AUG87	0.045	0.038	0.053	0.040	0.031	0.041	8	10	50	356	1013238
42	S	.	4	06JUL88	0.044	0.048	0.040	0.040	0.044	0.043	6	8	75	679	1947500
43	S	.	1	21OCT86	0.090	0.103	0.094	0.094	0.086	0.093	.	10	.	56	156800
43	S	.	2	17AUG87	0.059	0.074	0.043	0.043	0.057	0.055	8	10	50	356	1013238
43	S	.	4	06JUL88	0.041	0.051	0.044	0.041	0.048	0.045	8	8	70	679	1947500

(continued)

Table B.3 Sand Patch Test Data (Continued)

Sec.	B	L	R	Testing Date	ATD1	ATD2	ATD3	ATD4	ATD5	ATD	AGRT	BLG	AGEB	Age	Acc. Traffic
44	W	.	2	17JUN87	0.119	0.083	0.094	0.080	0.103	0.096	10	10	.	275	746300
44	W	.	4	26JUL88	0.068	0.077	0.071	0.057	0.077	0.070	10	10	60	679	1600250
45	W	.	2	17JUN87	0.119	0.098	0.086	0.147	0.113	0.113	9	9	.	275	746300
45	W	.	4	26JUL88	0.068	0.059	0.024	0.035	0.026	0.042	8	4	85	679	1600250
46	W	.	2	17JUN87	0.094	0.094	0.086	0.098	0.119	0.098	10	9	.	275	746300
46	W	.	4	26JUL88	0.045	0.077	0.039	0.053	0.077	0.058	8	7	70	679	1600250
47	W	.	2	17JUN87	0.044	0.094	0.125	0.125	0.108	0.099	6	6	.	275	746300
47	W	.	4	26JUL88	0.039	0.039	0.024	0.028	0.028	0.032	6	4	.	679	1600250
48	E	.	2	17JUN87	0.047	0.068	0.083	0.071	0.068	0.067	10	6	.	275	746300
48	E	.	4	26JUL88	0.041	0.047	0.044	0.045	0.045	0.044	10	4	90	679	1600250
49	E	.	2	17JUN87	0.083	0.086	0.059	0.098	0.080	0.081	10	8	.	275	746300
49	E	.	4	26JUL88	0.047	0.045	0.043	0.044	0.036	0.043	9	4	80	679	1600250
50	N	.	4	24AUG88	0.048	0.047	0.050	0.053	0.049	0.049	10	4	90	382	371050
51	N	.	4	24AUG88	0.071	0.063	0.057	0.063	0.061	0.063	10	6	70	382	371050
52	N	.	4	24AUG88	0.063	0.068	0.077	0.077	0.068	0.071	10	8	70	382	371050
53	N	.	4	24AUG88	0.045	0.047	0.057	0.055	0.053	0.051	10	4	80	382	371050

APPENDIX C
WEATHER DATA

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
1	5	NC	BRECKENR	29-Jan-87	D	13	1	0.31	15.31	22
1	5	NC	BRECKENR	22-Jul-87	D	6	3	1.44	34.89	91
1	5	NC	BRECKENR	7-Apr-88	D	7	1	0	43.7	182
1	5	NC	BRECKENR	29-Sep-88	W	1	1	1.08	55.73	182
1	5	NC	BRECKENR	2-May-89	W	1	1	0.45	65.93	239
1	5	NC	BRECKENR	8-Jan-90	D	9	1	0.33	87.85	269
2	5	NC	BRECKENR	29-Jan-87	D	13	1	0.31	15.31	22
2	5	NC	BRECKENR	22-Jul-87	D	6	3	1.44	34.89	91
2	5	NC	BRECKENR	7-Apr-88	D	7	1	0	43.7	182
2	5	NC	BRECKENR	29-Sep-88	W	1	1	1.08	55.73	182
2	5	NC	BRECKENR	8-Jan-90	D	9	1	0.33	87.85	269
3	5	NC	BRECKENR	29-Jan-87	D	13	1	0.31	15.31	22
3	5	NC	BRECKENR	22-Jul-87	D	6	3	1.44	34.89	91
3	5	NC	BRECKENR	7-Apr-88	D	7	1	0	43.7	182
3	5	NC	BRECKENR	29-Sep-88	W	1	1	1.08	55.73	182
3	5	NC	BRECKENR	2-May-89	W	1	1	0.45	65.93	239
3	5	NC	BRECKENR	8-Jan-90	D	9	1	0.33	87.85	269
3	5	NC	.	22-Oct-90	W	.	.	.	.	.
3	5	NC	.	30-Apr-91	W	.	.	.	.	.
4	5	NC	RISINGST	29-Jan-87	D	10	4	0.42	17.05	42
4	5	NC	RISINGST	22-Jul-87	D	5	3	0.7	41.69	58
4	5	NC	RISINGST	7-Apr-88	D	7	1	0	53.54	135

(continued)

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
4	5	NC	RISINGST	29-Sep-88	W	0	6	1.3	65.41	135
4	5	NC	RISINGST	28-Apr-89	W	7	3	2.18	79.21	185
4	5	NC	RISINGST	8-Jan-90	D	8	1	0.11	99.37	214
4	5	NC	.	22-Oct-90	W	.	.	.	.	.
4	5	NC	.	30-Apr-91	W	.	.	.	.	.
5	5	LR	ABILENE	29-Jan-87	D	11	7	0.45	17.45	14
5	5	LR	ABILENE	22-Jul-87	D	6	3	1.91	31.77	21
5	5	LR	ABILENE	8-Apr-88	D	7	1	0	44.94	83
5	5	LR	ABILENE	29-Sep-88	W	0	6	1.52	58.38	83
5	5	LR	ABILENE	28-Apr-89	W	8	4	0.59	66.04	121
5	5	LR	ABILENE	8-Jan-90	D	9	1	0.06	78.34	156
5	5	LR	.	18-Oct-90	W	.	.	.	.	.
5	5	LR	.	12-Jun-91	W	.	.	.	.	.
6	5	LR	ABILENE	29-Jan-87	D	11	7	0.45	17.45	14
6	5	LR	ABILENE	22-Jul-87	D	6	3	1.91	31.77	21
6	5	LR	ABILENE	8-Apr-88	D	7	1	0	44.94	83
6	5	LR	ABILENE	29-Sep-88	W	0	6	1.52	58.38	83
6	5	LR	ABILENE	28-Apr-89	W	8	4	0.59	66.04	121
6	5	LR	ABILENE	8-Jan-90	D	9	1	0.06	78.34	156
6	5	LR	.	18-Oct-90	W	.	.	.	.	.
6	5	LR	.	12-Jun-91	W	.	.	.	.	.
7	5	LR	ABILENE	29-Jan-87	D	11	7	0.45	17.45	14

(continued)

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
7	5	LR	ABILENE	22-Jul-87	D	6	3	1.91	31.77	21
7	5	LR	ABILENE	8-Apr-88	D	7	1	0	44.94	83
7	5	LR	ABILENE	29-Sep-88	W	0	6	1.52	58.38	83
7	5	LR	ABILENE	28-Apr-89	W	8	4	0.59	66.04	121
7	5	LR	ABILENE	8-Jan-90	D	9	1	0.66	78.34	156
7	5	LR	.	18-Oct-90	W	.	.	.	.	.
7	5	LR	.	12-Jun-91	W	.	.	.	.	.
8	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
8	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
8	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
8	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
8	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
8	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.31	339
8	5	HP	.	16-Oct-90	W	.	.	.	.	.
8	5	HP	.	11-Jun-91	W	.	.	.	.	.
9	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
9	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
9	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
9	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
9	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
9	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.31	339
9	5	HP	.	16-Oct-90	W	.	.	.	.	.
9	5	HP	.	11-Jun-91	W	.	.	.	.	.

(continued)

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
10	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
10	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
10	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
10	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
10	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
10	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.31	339
10	5	HP	.	16-Oct-90	W	.	.	.	.	.
10	5	HP	.	11-Jun-91	W	.	.	.	.	.
11	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
11	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
11	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
11	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
11	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
11	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.31	339
11	5	HP	.	16-Oct-90	W	.	.	.	.	.
11	5	HP	.	11-Jun-91	W	.	.	.	.	.
12	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
12	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
12	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
12	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
12	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
12	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.31	339
12	5	HP	.	16-Oct-90	W	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
12	5	HP	.	11-Jun-91	W	.	.	.	.	.
13	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
13	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
13	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
14	5	HP	HEREFORD	28-Jan-87	D	7	10	1.11	20.25	64
14	5	HP	HEREFORD	21-Jul-87	W	4	4	0.94	31.75	111
14	5	HP	HEREFORD	6-Apr-88	D	4	3	1.08	45.79	149
14	5	HP	HEREFORD	28-Sep-88	W	4	5	2.64	61.43	153
14	5	HP	HEREFORD	27-Apr-89	D	13	5	0.2	63.67	256
14	5	HP	HEREFORD	8-Feb-90	D	22	1	0.03	78.34	156
14	5	HP	.	16-Oct-90	W	.	.	.	.	.
14	5	HP	.	11-Jun-91	W	.	.	.	.	.
15	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
15	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
15	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
15	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
15	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
16	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
16	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
16	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
16	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
16	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
17	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
17	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
17	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
17	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
17	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
18	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
18	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
18	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
18	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
18	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
19	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
19	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
19	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
19	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
19	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
20	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
20	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
20	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
20	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
20	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
21	4	SC	CORPUSCR	22-Oct-86	W	0	3	2.21	11.18	0
21	4	SC	CORPUSCR	8-Jul-87	D	0	3	1.29	40.28	3
21	4	SC	CORPUSCR	21-Mar-88	D	4	2	0.39	53.44	11
21	4	SC	CORPUSCR	12-Sep-88	D	0	1	0.11	61.01	11
21	4	SC	CORPUSCR	19-Apr-89	W	5	5	2.61	73.37	17
22	2	ET	MOUNTPLE	30-Jun-87	W	0	3	0.75	41.26	65
22	2	ET	MOUNTPLE	7-Sep-88	D	4	1	0.35	90.29	122
22	2	ET	MOUNTPLE	4-May-89	W	0	4	2.1	120.47	160
22	2	ET	MOUNTPLE	31-Jan-90	D	0	2	1.62	163.08	209
23	2	ET	CARTHAGE	19-Nov-86	D	8	1	0	16.44	3
23	2	ET	CARTHAGE	1-Jul-87	W	0	1	0.25	48.58	60
23	2	ET	CARTHAGE	8-Sep-88	D	4	2	0.63	101.27	118
23	2	ET	CARTHAGE	4-May-89	W	0	2	0.4	130.1	157
23	2	ET	CARTHAGE	30-Jan-90	D	1	9	7.04	181.35	192
23	2	ET	.	3-Oct-90	D	.	.	.	.	.
23	2	ET	.	26-Mar-91	D	.	.	.	.	.
24	2	ET	CARTHAGE	19-Nov-86	D	8	1	0	16.44	3
24	2	ET	CARTHAGE	1-Jul-87	W	0	1	0.25	48.58	60
24	2	ET	CARTHAGE	8-Sep-88	D	4	2	0.63	101.27	118
24	2	ET	CARTHAGE	4-May-89	W	0	2	0.4	130.1	157
24	2	ET	CARTHAGE	30-Jan-90	D	1	9	7.04	181.35	192
24	2	ET	.	3-Oct-90	D	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
24	2	ET	.	26-Mar-91	D	.	.	.	.	.
25	2	ET	CARTHAGE	19-Nov-86	D	8	1	0	16.44	3
25	2	ET	CARTHAGE	1-Jul-87	W	0	1	0.25	48.58	60
25	2	ET	CARTHAGE	4-May-89	W	0	2	0.4	130.1	157
25	2	ET	CARTHAGE	29-Jan-90	D	1	9	7.04	181.35	192
25	2	ET	.	3-Oct-90	D	.	.	.	.	.
25	2	ET	.	26-Mar-91	D	.	.	.	.	.
26	2	ET	COOPER	19-Nov-86	D	3	5	2.25	7.73	4
26	2	ET	COOPER	29-Jun-87	W	31	1	0	26.56	47
26	2	ET	COOPER	7-Sep-88	D	5	2	2.47	74.33	101
26	2	ET	COOPER	4-May-89	W	0	3	0.96	93.46	139
26	2	ET	.	4-Oct-90	D	.	.	.	.	.
26	2	ET	.	27-Mar-91	D	.	.	.	.	.
27	2	ET	SULFURSP	19-Nov-86	D	3	8	2.4	11.35	3
27	2	ET	SULFURSP	29-Jun-87	W	4	12	8.06	43.99	54
27	2	ET	SULFURSP	7-Sep-88	D	4	2	1.32	78.81	112
27	2	ET	SULFURSP	4-May-89	W	0	0	1.32	118.99	147
27	2	ET	SULFURSP	1-Feb-90	D	0	2	2.46	158.48	176
27	2	ET	.	4-Oct-90	D	.	.	.	.	.
27	2	ET	.	27-Mar-91	D	.	.	.	.	.
28	2	ET	SULFURSP	19-Nov-86	D	3	8	2.4	11.35	3

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
28	2	ET	SULFURSP	29-Jun-87	W	4	12	8.06	43.99	54
28	2	ET	SULFURSP	7-Sep-88	D	4	2	1.32	78.81	112
28	2	ET	SULFURSP	4-May-89	W	0	3	1.32	118.99	147
28	2	ET	SULFURSP	1-Feb-90	D	0	2	2.46	158.48	176
28	2	ET	.	4-Oct-90	D	.	.	.	.	.
28	2	ET	.	27-Mar-91	D	.	.	.	.	.
29	2	ET	SHERMAN	18-Nov-86	D	6	1	0	17.24	3
29	2	ET	SHERMAN	29-Jun-87	W	0	13	4.4	42.38	40
29	2	ET	SHERMAN	6-Sep-88	D	3	9	2.62	80.86	90
29	2	ET	SHERMAN	3-May-89	W	0	6	0.59	106.65	125
29	2	ET	SHERMAN	1-Feb-90	D	0	3	2.17	139.7	152
29	2	ET	.	23-Oct-90	W	.	.	.	.	.
29	2	ET	.	27-Mar-91	D	.	.	.	.	.
30	2	ET	SHERMAN	18-Nov-86	D	6	1	0	17.24	3
30	2	ET	SHERMAN	29-Jun-87	W	0	13	4.4	42.38	40
30	2	ET	SHERMAN	6-Sep-88	D	3	9	2.62	80.86	90
30	2	ET	SHERMAN	3-May-89	W	0	6	0.59	106.65	125
30	2	ET	SHERMAN	1-Feb-90	D	0	3	2.17	139.7	152
30	2	ET	.	23-Oct-90	W	.	.	.	.	.
30	2	ET	.	27-Mar-91	D	.	.	.	.	.
31	2	ET	TEXARKAN	19-Nov-86	D	0	3	0.23	18.9	3
31	2	ET	TEXARKAN	30-Jun-87	W	0	10	1.41	43.49	40

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
31	2	ET	TEXARKAN	8-Sep-88	D	6	1	0	96.75	91
31	2	ET	TEXARKAN	4-May-89	W	0	6	4.61	150.2	133
31	2	ET	TEXARKAN	31-Jan-90	D	0	3	1.58	189.68	164
31	2	ET	.	5-Oct-90	D	.	.	.	.	.
32	2	ET	CROCKETT	20-Nov-86	D	6	1	0	14.4	3
32	2	ET	CROCKETT	1-Jul-87	W	0	2	1.3	37.76	24
32	2	ET	CROCKETT	9-Sep-88	D	5	1	0	74.6	68
32	2	ET	CROCKETT	5-May-89	W	0	7	5.75	110.59	96
32	2	ET	CROCKETT	29-Jan-90	D	0	8	4.15	138.11	125
32	2	ET	.	3-Oct-90	D	.	.	.	.	.
32	2	ET	.	26-Mar-91	D	.	.	.	.	.
33	2	ET	CROCKETT	20-Nov-86	D	6	1	0	14.4	3
33	2	ET	CROCKETT	1-Jul-87	W	0	2	1.3	37.76	24
33	2	ET	CROCKETT	9-Sep-88	D	5	1	0	74.6	68
33	2	ET	CROCKETT	5-May-89	W	0	7	5.75	110.59	96
33	2	ET	CROCKETT	29-Jan-90	D	0	8	4.15	138.11	125
33	2	ET	.	3-Oct-90	D	.	.	.	.	.
33	2	ET	.	26-Mar-91	D	.	.	.	.	.
34	2	ET	CROCKETT	20-Nov-86	D	6	1	0	14.4	3
34	2	ET	CROCKETT	1-Jul-87	W	0	2	1.3	37.76	24
34	2	ET	CROCKETT	9-Sep-88	D	5	1	0	74.6	68
34	2	ET	CROCKETT	5-May-89	W	0	7	5.75	110.59	96

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
34	2	ET	CROCKETT	29-Jan-90	D	0	8	4.15	138.11	125
34	2	ET	.	3-Oct-90	D	.	.	.	.	.
34	2	ET	.	26-Mar-91	D	.	.	.	.	.
35	2	ET	GILMER	19-Nov-86	D	4	8	2	10.78	3
35	2	ET	GILMER	30-Jun-87	W	5	10	4.82	41.88	63
35	2	ET	GILMER	7-Sep-88	D	3	7	0.94	97.3	123
35	2	ET	GILMER	4-May-89	W	0	3	0.77	125.26	180
35	2	ET	GILMER	31-Jan-90	D	2	8	10.27	158.89	219
35	2	ET	.	3-Oct-90	D	.	.	.	.	.
35	2	ET	.	27-Mar-91	D	.	.	.	.	.
36	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
36	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
36	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
36	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
36	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
36	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
36	1	UC	.	4-Dec-90	D	.	.	.	.	.
36	1	UC	.	14-Mar-91	D	.	.	.	.	.
37	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
37	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
37	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
37	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
40	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
40	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
40	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
40	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
40	1	UC	.	4-Dec-90	D	.	.	.	.	.
40	1	UC	.	14-Mar-91	D	.	.	.	.	.
41	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
41	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
41	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
41	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
41	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
41	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
41	1	UC	.	4-Dec-90	D	.	.	.	.	.
41	1	UC	.	14-Mar-91	D	.	.	.	.	.
42	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
42	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
42	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
42	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
42	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
42	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
42	1	UC	.	4-Dec-90	D	.	.	.	.	.
42	1	UC	.	14-Mar-91	D	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
40	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
40	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
40	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
40	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
40	1	UC	.	4-Dec-90	D	.	.	.	.	.
40	1	UC	.	14-Mar-91	D	.	.	.	.	.
41	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
41	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
41	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
41	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
41	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
41	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
41	1	UC	.	4-Dec-90	D	.	.	.	.	.
41	1	UC	.	14-Mar-91	D	.	.	.	.	.
42	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
42	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
42	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
42	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
42	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
42	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
42	1	UC	.	4-Dec-90	D	.	.	.	.	.
42	1	UC	.	14-Mar-91	D	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
43	1	UC	VICTORIA	21-Oct-86	W	0	2	0.64	7.82	0
43	1	UC	VICTORIA	7-Jul-87	W	1	1	0	41.25	2
43	1	UC	VICTORIA	22-Mar-88	D	5	2	1.08	62.46	15
43	1	UC	VICTORIA	13-Sep-88	W	1	1	0.2	71.54	15
43	1	UC	VICTORIA	19-Apr-89	D	0	6	0.5	83.83	21
43	1	UC	VICTORIA	6-Feb-90	D	3	4	0.38	104.84	29
43	1	UC	.	4-Dec-90	D	.	.	.	.	.
43	1	UC	.	14-Mar-91	D	.	.	.	.	.
44	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
44	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
44	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
44	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
44	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57
44	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
44	1	UC	.	2-Oct-90	W	.	.	.	.	.
44	1	UC	.	15-Mar-91	D	.	.	.	.	.
45	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
45	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
45	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
45	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
45	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57
45	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
45	1	UC	.	2-Oct-90	W	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
45	1	UC	.	15-Mar-91	D	.	.	.	.	.
46	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
46	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
46	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
46	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
46	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57
46	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
46	1	UC	.	2-Oct-90	W	.	.	.	.	.
46	1	UC	.	15-Mar-91	D	.	.	.	.	.
47	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
47	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
47	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
47	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
47	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57
47	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
47	1	UC	.	2-Oct-90	W	.	.	.	.	.
47	1	UC	.	15-Mar-91	D	.	.	.	.	.
48	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
48	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
48	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
48	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
48	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
48	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
48	1	UC	.	2-Oct-90	W	.	.	.	.	.
48	1	UC	.	15-Mar-91	D	.	.	.	.	.
49	1	UC	LIBERTY	14-May-87	W	1	9	1.54	40.95	9
49	1	UC	LIBERTY	6-Jul-87	W	3	24	22.02	62.97	11
49	1	UC	LIBERTY	22-Mar-88	D	4	2	2.76	101.35	32
49	1	UC	LIBERTY	14-Sep-88	W	0	2	0	120.31	32
49	1	UC	LIBERTY	5-May-89	D	0	4	3.34	134.62	57
49	1	UC	LIBERTY	30-Nov-89	D	0	9	1.84	184.98	61
49	1	UC	.	2-Oct-90	W	.	.	.	.	.
49	1	UC	.	15-Mar-91	D	.	.	.	.	.
50	5	EP	SONORA	4-Apr-88	D	27	0	0	6.79	119
50	5	EP	SONORA	26-Sep-88	W	8	2	2.75	18.41	124
50	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
50	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167
50	5	EP	.	8-Oct-90	W	.	.	.	.	.
50	5	EP	.	5-Jun-91	W	.	.	.	.	.
51	5	EP	SONORA	4-Apr-88	D	27	0	0	6.79	119
51	5	EP	SONORA	26-Sep-88	W	8	2	2.75	18.41	124
51	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
51	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167
51	5	EP	.	8-Oct-90	W	.	.	.	.	.

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
51	5	EP	.	5-Jun-91	W	.	.	.	.	.
52	5	EP	SONORA	4-Apr-88	D	27	0	0	6.79	119
52	5	EP	SONORA	26-Sep-88	W	8	2	2.75	18.41	124
52	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
52	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167
52	5	EP	.	8-Oct-90	W	.	.	.	.	.
52	5	EP	.	5-Jun-91	W	.	.	.	.	.
53	5	EP	SONORA	4-Apr-88	D	27	0	0	6.79	119
53	5	EP	SONORA	26-Sep-88	W	8	2	2.75	18.41	124
53	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
53	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167
53	5	EP	.	8-Oct-90	W	.	.	.	.	.
53	5	EP	.	5-Jun-91	W	.	.	.	.	.
54	5	EP	SONORA	4-Apr-88	D	27	0	0	6.79	119
54	5	EP	SONORA	26-Sep-88	W	8	2	2.75	18.41	124
54	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
54	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167
54	5	EP	.	8-Oct-90	W	.	.	.	.	.
54	5	EP	.	5-Jun-91	W	.	.	.	.	.
55	5	EP	SONORA	24-Apr-89	W	11	2	0.12	26.68	162
55	5	EP	SONORA	6-Nov-89	W	7	2	4.55	36.43	167

Table C.1 Short-Term and Long-Term Weather Data Associated with the Skid Resistance Test (Continued)

Sections	RG	SUBV	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
55	5	EP	.	8-Oct-90	W	.	.	.	.	.
55	5	EP	.	21-May-91	W	.	.	.	.	.
56	5	TP	VANHORN	27-Sep-88	W	13	0	0	13.7	89
56	5	TP	VANHORN	25-Apr-89	D	26	1	0.07	15.43	160
56	5	TP	VANHORN	7-Nov-89	D	28	2	0.34	22.31	164
56	5	TP	.	11-Oct-90	W	.	.	.	.	.
56	5	TP	.	21-May-91	D	.	.	.	.	.
57	5	TP	VANHORN	27-Sep-88	W	13	0	0	13.7	89
57	5	TP	VANHORN	25-Apr-89	D	26	1	0.07	15.43	160
57	5	TP	VANHORN	7-Nov-89	D	28	2	0.34	22.31	164
57	5	TP	.	11-Oct-90	W	.	.	.	.	.
57	5	TP	.	21-May-91	D	.	.	.	.	.
58	5	TP	VANHORN	27-Sep-88	W	13	0	0	13.7	89
58	5	TP	VANHORN	25-Apr-89	D	26	1	0.07	15.43	160
58	5	TP	VANHORN	7-Nov-89	D	28	2	0.34	22.31	164
58	5	TP	.	11-Oct-90	W	.	.	.	.	.
58	5	TP	.	21-May-91	D	.	.	.	.	.
59	5	TP	VANHORN	27-Sep-88	W	13	0	0	13.7	89
59	5	TP	VANHORN	25-Apr-89	D	26	1	0.07	15.43	160
59	5	TP	VANHORN	7-Nov-89	D	28	2	0.34	22.31	164
59	5	TP	.	11-Oct-90	W	.	.	.	.	.
59	5	TP	.	21-May-91	D	.	.	.	.	.

Table C.2 Short-Term and Long-Term Weather Data Associated with the British Pendulum and Sand Patch Tests

Sections	RG	SUBD	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
1-3	V	NC	Breckenridge	01JUL87	D	3	1	0.23	25.69	91
	V	NC	Breckenridge	15JUN88	W	12	4	2.88	39.38	183
4	V	NC	Rising Star	01JUL87	D	0	2	0.63	40.99	58
	V	NC	Rising Star	15JUN88	W	12	6	4.87	61.54	135
5-7	V	LR	Abilene	19MAR87	D	2	5	1.01	22.01	18
	V	LR	Abilene	25AUG87	D	11	3	2.39	23.73	18
	V	LR	Abilene	16JUN88	W	0	1	0.01	52.36	83
8-14	V	HP	Hereford	19MAR87	D	1	5	0.20	21.76	93
	V	HP	Hereford	26AUG87	W	1	3	1.22	45.37	115
	V	HP	Hereford	29JUN88	W	0	4	0.73	54.14	153
15-21	IV	SC	Corpus Christi	22OCT86	W	0	3	2.21	11.18	0
	IV	SC	Corpus Christi	18AUG87	D	19	1	0.21	53.66	4
	IV	SC	Corpus Christi	12JUL88	D	2	7	0.36	57.36	11
22	II	ET	Mount Pleasant	27JUL87	D	4	1	0.08	74.01	65
	II	ET	Mount Pleasant	17AUG88	D	1	4	2.49	92.78	122
23-25	II	ET	Carthage	28JUL87	D	2	4	2.23	52.70	60
	II	ET	Carthage	17AUG88	D	5	7	6.60	100.01	118
26	II	ET	Cooper	07JUL87	W	5	2	2.03	30.36	47
	II	ET	Cooper	16AUG88	D	4	3	1.93	78.13	101

(continued)

Table C.2 Short-Term and Long-Term Weather Data Associated with the British Pendulum and Sand Patch Tests (Continued)

Sections	RG	SUBD	Station	Testing Date	Season	DLR	LRP	TPP	CUPR	CUFT
27-28	II	ET	Sulfur Springs	07JUL87	W	3	3	1.10	45.09	54
	II	ET	Sulfur Springs	16AUG88	D	3	2	0.67	77.25	112
29-30	II	NC	Sherman	06JUL87	W	3	4	1.54	43.08	40
	II	NC	Sherman	16AUG88	D	4	6	0.35	78.14	90
31	II	ET	Texarkana	27JUL87	D	0	5	1.77	46.30	40
	II	ET	Texarkana	17AUG88	D	0	8	2.41	94.95	91
32-34	II	ET	Crockett	07JAN87	D	3	2	0.51	29.60	8
	II	ET	Crockett	05AUG87	D	4	5	0.75	41.92	24
	II	ET	Crockett	18JUL88	D	5	10	3.04	71.19	68
35	II	ET	Gilmer	27JUL87	D	2	2	1.01	44.88	63
	II	ET	Gilmer	17AUG88	D	5	2	0.20	52.39	42
36-43	I	UC	Victoria	21OCT86	W	0	2	0.64	7.82	0
	I	UC	Victoria	17AUG87	W	17	10	3.77	46.23	2
	I	UC	Victoria	06JUL88	W	0	2	1.05	69.56	15
44-49	I	UC	Liberty	17JUN87	W	0	17	15.39	56.34	11
	I	UC	Liberty	26JUL88	W	0	5	1.19	111.81	32
50-53	V	EP	Sonora	24AUG88	D	4	1	0.95	12.05	57

APPENDIX D

PRODICTION MODELS COMPUTER OUTPUTS

Table D.1 Three-Variable GLM Model

SAS 20:55 TUESDAY, AUGUST 27, 1991

GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: FN

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V
MODEL	29	31150.44194538	1074.15317053	57.23	0.0	0.836233	8.711
ERROR	325	6100.46552738	18.77066316		ROOT MSE		FN MEA
CORRECTED TOTAL	354	37250.90747275			4.33251234		49.7325042

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE III SS	F VALUE	PR >
AGGR	4	17293.54896334	230.33	0.0	4	1551.68445536	20.67	0.000
LGCUTR*AGGR	5	4893.57083960	52.14	0.0001	5	2153.45228166	22.94	0.000
AGGR*RGT	4	2095.10085401	27.90	0.0001	4	1438.11287015	19.15	0.000
LGCUTR*AGGR*RGT	4	1394.20094170	18.57	0.0001	4	1704.37329012	22.70	0.000
LAGSR	2	3063.01791495	81.59	0.0001	2	2390.55288756	63.68	0.000
AGGR*LAGSR	5	1378.77114586	14.69	0.0001	5	1337.36369584	14.25	0.000
ADT	1	118.81673852	6.33	0.0124	1	639.85718132	34.09	0.000
ADT*AGGR	4	913.41454740	12.17	0.0001	4	913.41454740	12.17	0.000

PARAMETER		ESTIMATE		T FOR HO: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT		137.93147064	B	10.98	0.0001	12.56480494
AGGR	A	-78.35622347	B	-4.53	0.0001	17.29547972
	B	-140.56527578	B	-1.08	0.2809	130.14582139
	C	16.53683335	B	0.92	0.3593	18.01266898
	D	-48.46119877	B	-2.70	0.0073	17.93560772
	E	0.00000000	B	.	.	.
LGCUTR*AGGR	A	0.94060362	B	1.22	0.2237	0.77165334
	B	3.65748253	B	0.41	0.6807	8.87975286
	C	-8.26052347	B	-11.20	0.0001	0.73745092
	D	-1.54685333	B	-1.56	0.1192	0.99007353
	E	-5.25137845	B	-7.89	0.0001	0.66524342
AGGR*RGT	A COLD	-71.48454470	B	-4.79	0.0001	14.91254029
	A WARM	0.00000000	B	.	.	.
	B COLD	50.39461694	B	0.39	0.6964	129.05996924
	B WARM	0.00000000	B	.	.	.
	C COLD	-141.66172481	B	-6.93	0.0001	20.45458684
	C WARM	0.00000000	B	.	.	.
	D COLD	0.00000000	B	.	.	.
	E COLD	-47.67391596	B	-2.35	0.0194	20.29306788
	E WARM	0.00000000	B	.	.	.
LGCUTR*AGGR*RGT	A COLD	4.94815924	B	4.89	0.0001	1.01271661
	A WARM	0.00000000	B	.	.	.
	B COLD	-3.39326389	B	-0.38	0.7008	8.82458120
	B WARM	0.00000000	B	.	.	.
	C COLD	11.47008507	B	8.08	0.0001	1.41985359
	C WARM	0.00000000	B	.	.	.
	D COLD	0.00000000	B	.	.	.
	E COLD	1.45889343	B	1.23	0.2187	1.18375582
	E WARM	0.00000000	B	.	.	.
LAGSR	HIG	-4.31204279	B	-2.71	0.0066	1.57680488

Table D.1 Three-Variable GLM Model (Continued)

SAS 20:55 TUESDAY, AUGUST 27, 1991

GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: FN

PARAMETER		ESTIMATE	T FOR HO: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
	LOW	-3.80652331 B	-2.26	0.0245	1.68391329
	MED	0.00000000 B	.	.	.
AGGR*LAGSR	A HIG	14.79084313 B	7.09	0.0001	2.08492069
	A LOW	-8.32084099 B	-3.54	0.0005	2.35187154
	A MED	0.00000000 B	.	.	.
	B HIG	10.51737395 B	4.69	0.0001	2.24317167
	B LOW	0.00000000 B	.	.	.
	B MED	0.00000000 B	.	.	.
	C HIG	11.94386236 B	5.68	0.0001	2.10222992
	C MED	0.00000000 B	.	.	.
	D HIG	7.18609990 B	3.11	0.0021	2.31343758
	D MED	0.00000000 B	.	.	.
	E HIG	0.00000000 B	.	.	.
	E MED	0.00000000 B	.	.	.
ADT		-0.00690090 B	-2.55	0.0112	0.00270550
ADT*AGGR	A	0.00043185 B	0.14	0.8877	0.00305445
	B	0.00765894 B	2.69	0.0074	0.00284211
	C	0.00858120 B	2.59	0.0100	0.00331267
	D	-0.01703479 B	-3.15	0.0018	0.00540763
	E	0.00000000 B	.	.	.

NOTE: THE X'X MATRIX HAS BEEN DEEMED SINGULAR AND A GENERALIZED INVERSE HAS BEEN EMPLOYED TO SOLVE THE NORMAL EQUATIONS. THE ABOVE ESTIMATES REPRESENT ONLY ONE OF MANY POSSIBLE SOLUTIONS TO THE NORMAL EQUATIONS. ESTIMATES FOLLOWED BY THE LETTER B ARE BIASED AND DO NOT ESTIMATE THE PARAMETER BUT ARE BLUE FOR SOME LINEAR COMBINATION OF PARAMETERS (OR ARE ZERO). THE EXPECTED VALUE OF THE BIASED ESTIMATORS MAY BE OBTAINED FROM THE GENERAL FORM OF ESTIMABLE FUNCTIONS. FOR THE BIASED ESTIMATORS, THE STD ERR IS THAT OF THE BIASED ESTIMATOR AND THE T VALUE TESTS HO: E(BIASED ESTIMATOR) = 0. ESTIMATES NOT FOLLOWED BY THE LETTER B ARE BLUE FOR THE PARAMETER.

Table D.2 One-Way GLM Model

SAS 20:46 TUESDAY, AUGUST 27, 1991

GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: FN

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V
MODEL	31	30656.11604231	988.90696911	47.92	0.0	0.824122	9.14
ERROR	317	6542.41924249	20.63854651		ROOT MSE		FN ME
CORRECTED TOTAL	348	37198.53528479			4.54296671		49.688363

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE III SS	F VALUE	PR >
LGCUTR	1	0.41149891	0.02	0.8878	1	653.51917951	31.66	0.00
GROUP	15	23867.05988307	77.10	0.0	15	5345.54762448	17.27	0.00
LGCUTR*GROUP	15	6788.64466032	21.93	0.0001	15	6788.64466032	21.93	0.00

PARAMETER		ESTIMATE		T FOR HO: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT		106.70431235	B	9.77	0.0001	10.92031109
LGCUTR		-4.76603926	B	-6.30	0.0001	0.75610484
GROUP	1	-81.05338469	B	-3.66	0.0003	22.17587777
	2	-40.36643883	B	-1.92	0.0554	20.99194357
	3	-124.83677399	B	-8.52	0.0001	14.65176968
	4	-74.83777333	B	-3.96	0.0001	18.91453364
	5	-113.62928150	B	-4.56	0.0001	24.93406790
	6	-58.09867431	B	-4.53	0.0001	12.82881277
	7	-61.07945347	B	-1.71	0.0883	35.72676047
	8	-93.52443757	B	-4.66	0.0001	20.07714438
	9	79.29818981	B	4.19	0.0001	18.90973095
	10	37.42638750	B	1.98	0.0487	18.91453364
	11	-25.09143364	B	-1.38	0.1679	18.15587350
	12	4.17861945	B	0.12	0.9070	35.72676047
	13	4.07308573	B	0.29	0.7729	14.10512366
	14	0.12637974	B	0.00	0.9961	26.09687723
	15	70.87783219	B	2.47	0.0140	28.67148005
	16	0.00000000	B	.	.	.
LGCUTR*GROUP	1	6.16889986	B	3.98	0.0001	1.54990227
	2	4.05440653	B	2.80	0.0055	1.44982996
	3	10.39536665	B	10.04	0.0001	1.03493113
	4	7.06731312	B	5.40	0.0001	1.30961200
	5	8.88492997	B	4.86	0.0001	1.82716042
	6	5.07539606	B	5.67	0.0001	0.89560103
	7	5.71690077	B	2.13	0.0336	2.67840049
	8	8.06632563	B	5.50	0.0001	1.46551006
	9	-5.35136107	B	-4.06	0.0001	1.31863303
	10	-1.91230741	B	-1.46	0.1452	1.30961200
	11	1.94486095	B	1.48	0.1388	1.31039917
	12	0.04868414	B	0.02	0.9855	2.67840049
	13	-1.04797174	B	-1.00	0.3162	1.04399285
	14	-0.93420824	B	-0.45	0.6518	2.06798820
	15	-5.04477402	B	-2.46	0.0143	2.04705529
	16	0.00000000	B			

Table D.3 Regression Equation for the One-Way GLM Model

SAS

20:46 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	31	29509.43407	951.91723	39.717	0.0001
ERROR	323	7741.47340	23.96740991		
C TOTAL	354	37250.90747			

ROOT MSE	4.895652	R-SQUARE	0.7922
DEP MEAN	49.7325	ADJ R-SQ	0.7722
C.V.	9.843969		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	101.55039	11.71296926	8.670	0.0001
LGCUTR	1	-4.24198826	0.80939370	-5.241	0.0001
GROUP1	1	-75.89946709	23.87036818	-3.180	0.0016
GROUP2	1	-35.21252122	22.59299120	-1.559	0.1201
GROUP3	1	-119.68286	15.74819426	-7.600	0.0001
GROUP4	1	-69.68385572	20.35115642	-3.424	0.0007
GROUP5	1	-108.47536	26.84568586	-4.041	0.0001
GROUP6	1	-52.94475671	13.77786481	-3.843	0.0001
GROUP7	1	-55.92553587	38.48353356	-1.453	0.1471
GROUP8	1	-88.37051997	21.60586706	-4.090	0.0001
GROUP9	1	84.45210742	20.34597279	4.151	0.0001
GROUP10	1	42.58030511	20.35115642	2.092	0.0372
GROUP11	1	-19.93751604	19.53226906	-1.021	0.3081
GROUP12	1	9.33253706	38.48353356	0.243	0.8085
GROUP13	1	9.22700334	15.15751539	0.609	0.5431
GROUP14	1	5.28029734	28.09984244	0.188	0.8511
GROUP15	1	76.03174979	30.87638883	2.462	0.0143
LGGRP1	1	5.64484886	1.66759373	3.385	0.0008
LGGRP2	1	3.53035553	1.55957045	2.264	0.0243
LGGRP3	1	9.87131565	1.11132988	8.882	0.0001
LGGRP4	1	6.54326212	1.40816492	4.647	0.0001
LGGRP5	1	8.36087896	1.96677630	4.251	0.0001
LGGRP6	1	4.55134506	0.96056646	4.738	0.0001
LGGRP7	1	5.19284977	2.88481095	1.800	0.0728
LGGRP8	1	7.54227463	1.57649801	4.784	0.0001
LGGRP9	1	-5.87541208	1.41790765	-4.144	0.0001
LGGRP10	1	-2.43635841	1.40816492	-1.730	0.0846
LGGRP11	1	1.42080995	1.40901508	1.008	0.3140
LGGRP12	1	-0.47536686	2.88481095	-0.165	0.8692
LGGRP13	1	-1.57202275	1.12112947	-1.402	0.1618
LGGRP14	1	-1.45825924	2.22656057	-0.655	0.5130
LGGRP15	1	-5.56882502	2.20398239	-2.527	0.0120

Table D.4 General Multivariable Regression Model

SAS 20:59 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	4	14790.13482	3697.53370	100.833	0.0001
ERROR	330	12101.00700	36.66971817		
C TOTAL	334	26891.14181			

ROOT MSE	6.055553	R-SQUARE	0.5500
DEP MEAN	47.88782	ADJ R-SQ	0.5445
C.V.	12.64529		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	47.46596071	4.56725354	10.393	0.0001
TR	1	-2.96666189	0.23451449	-12.650	0.0001
PVLA	1	0.01957436	0.001709730	11.449	0.0001
PVINRDTR	1	0.000361770	0.000023275	15.543	0.0001
AGSR	1	0.14658706	0.03166981	4.629	0.0001

SAS

VARIABLE	MEAN	VARIANCE
TR	13.3517383	2.219
PVLA	826.3646724	54571.415
PVINRDTR	22526.9576563	435104864.215
AGSR	105.8866279	187.244

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP TR	VAR PROP PVLA	VAR PROP PVINRDTR	VAR PROP AGSR
1	4.483400	1.000000	0.0003	0.0005	0.0023	0.0065	0.0005
2	0.456635	3.133423	0.0001	0.0003	0.0116	0.3826	0.0009
3	0.047865	9.678184	0.0041	0.0119	0.7086	0.0764	0.0459
4	0.0083432	23.181308	0.0216	0.9267	0.0551	0.1448	0.2914
5	0.0037566	34.546624	0.9740	0.0607	0.2224	0.3897	0.6613

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	46.1642	0.8603	44.4718	47.8565	34.1320	58.1963	4.6568
2	43.2770	44.2667	0.7780	42.7362	45.7971	32.2562	56.2771	-0.9897
3	40.5170	42.7297	0.7274	41.2988	44.1607	30.7316	54.7279	-2.2127
4	39.4130	42.0512	0.7106	40.6534	43.4491	30.0570	54.0455	-2.6382
5	37.6000	41.3867	0.6977	40.0141	42.7593	29.3953	53.3780	-3.7867
6	37.2000	40.7624	0.6891	39.4068	42.1180	28.7730	52.7518	-3.5624
7	34.6290	45.0169	0.8077	43.4281	46.6058	32.9989	57.0349	-10.3879
8	.	43.1564	0.7393	41.7020	44.6108	31.1554	55.1574	.
9	31.5010	41.6237	0.7015	40.2438	43.0037	29.6315	53.6159	-10.1227
10	30.5810	40.9348	0.6908	39.5759	42.2936	28.9450	52.9245	-10.3538

Table D.5 Regression Model for the Limestone Rock Asphalt Aggregates

SAS 21:05 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: LGFN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	3	2.65115969	0.88371990	90.746	0.0001
ERROR	51	0.49665864	0.009738405		
C TOTAL	54	3.14781833			

ROOT MSE	0.09868336	R-SQUARE	0.8422
DEP MEAN	3.829431	ADJ R-SQ	0.8329
C.V.	2.576972		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	3.17294559	0.12711697	24.961	0.0001
CUTR	1	-1.39558E-07	1.03064E-08	-13.541	0.0001
RGTCUTR	1	2.13076E-07	2.97755E-08	7.156	0.0001
AGSR	1	0.007985820	0.001137066	7.023	0.0001

SAS

VARIABLE	MEAN	VARIANCE
CUTR	2042383.89831	1800250038288
RGTCUTR	212168.64407	231288655440
AGSR	110.67797	158

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP CUTR	VAR PROP RGTCUTR	VAR PROP AGSR
1	2.954575	1.000000	0.0012	0.0263	0.0197	0.0012
2	0.842564	1.872605	0.0001	0.0344	0.7136	0.0002
3	0.197315	3.869610	0.0094	0.9285	0.1169	0.0092
4	0.0055456	23.082091	0.9893	0.0108	0.1497	0.9894

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	4.0978	4.0106	0.0237	3.9631	4.0582	3.8069	4.2144	0.0871
2	3.8801	3.8837	0.0175	3.8486	3.9189	3.6825	4.0849	-.003625
3	3.7063	3.7581	0.0151	3.7278	3.7883	3.5577	3.9585	-0.0518
4	.	3.6726	0.0165	3.6395	3.7057	3.4717	3.8734	.
5	3.6687	3.5651	0.0210	3.5230	3.6072	3.3626	3.7677	0.1036
6	4.0016	4.0106	0.0237	3.9631	4.0582	3.8069	4.2144	-.009059
7	3.8914	3.8837	0.0175	3.8486	3.9189	3.6825	4.0849	.0077085
8	3.5178	3.7581	0.0151	3.7278	3.7883	3.5577	3.9585	-0.2403
9	.	3.6726	0.0165	3.6395	3.7057	3.4717	3.8734	.
10	.	3.5651	0.0210	3.5230	3.6072	3.3626	3.7677	.
11	3.9914	4.0106	0.0237	3.9631	4.0582	3.8069	4.2144	-0.0192
12	3.9812	3.8837	0.0175	3.8486	3.9189	3.6825	4.0849	0.0975

Table D.6 Regression Model for the Limestone Aggregates

SAS 11:00 THURSDAY, AUGUST 29, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	7	3330.91879	475.84554	35.112	0.0001
ERROR	79	1070.63994	13.55240429		
C TOTAL	86	4401.55873			

ROOT MSE	3.681359	R-SQUARE	0.7568
DEP MEAN	41.89266	ADJ R-SQ	0.7352
C.V.	8.7876		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	108.03576	5.87305379	18.395	0.0001
LGCUTR	1	-4.79356877	0.41639886	-11.512	0.0001
RGTTR	1	-7.45203438	0.72880150	-10.225	0.0001
PVRGTTR	1	0.24771901	0.02649170	9.351	0.0001
MSS	1	2.84477922	0.41053654	6.929	0.0001
PVMSS	1	-0.05220739	0.01151734	-4.533	0.0001
ADTRGT	1	-0.02475080	0.004012603	-6.168	0.0001
PVMSSADT	1	-0.000017500	.00000252765	-6.924	0.0001

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP LGCUTR	VAR PROP RGTTR	VAR PROP PVRGTTR	VAR PROP MSS	VAR PROP PVMSS	VAR PROP ADTRGT	VAR PROP PVMSSADT
1	6.636082	1.000000	0.0001	0.0001	0.0001	0.0000	0.0001	0.0001	0.0005	0.0006
2	0.717700	3.040776	0.0013	0.0011	0.0004	0.0001	0.0005	0.0006	0.0005	0.0095
3	0.578784	3.386082	0.0012	0.0018	0.0008	0.0006	0.0000	0.0000	0.0057	0.0092
4	0.044988	12.145259	0.0004	0.0069	0.0144	0.0020	0.0000	0.0003	0.2521	0.1152
5	0.016288	20.184627	0.0030	0.0000	0.0001	0.0012	0.0369	0.0383	0.2115	0.8176
6	0.0030471	46.667191	0.3324	0.3593	0.0679	0.1418	0.0877	0.1116	0.3356	0.0016
7	0.0022999	53.715574	0.5292	0.4712	0.0662	0.0335	0.2636	0.2413	0.0041	0.0184
8	0.0008107	90.473021	0.1324	0.1596	0.8501	0.8207	0.6112	0.6079	0.1900	0.0278

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	44.6327	0.8413	42.9582	46.3072	37.1162	52.1491	6.1883
2	43.2770	41.1275	0.7607	39.6135	42.6416	33.6452	48.6099	2.1495
3	40.5170	38.4440	0.7909	36.8697	40.0182	30.9492	45.9388	2.0730
4	39.4130	37.3562	0.8315	35.7011	39.0113	29.8440	44.8684	2.0568

Table D.6 Regression Model for the Limestone Aggregates (Continued)

SAS 21:08 TUESDAY, AUGUST 27, 1991

VARIABLE	N	MEAN	STD DEV	SUM	MINIMUM	MAXIMUM
LGCUTR	91	13.3	1.4	1213	9.968	15
RGTTR	91	8.0	6.3	726	0.000	15
PVRGTTR	91	256.6	209.5	23355	0.000	541
MSS	93	13.4	12.8	1243	1.700	41
PVMSS	93	445.3	460.9	41411	45.900	1517
ADTRGT	91	440.0	468.6	40044	0.000	1246
PVMSSADT	91	577750.9	588394.5	52575328	4698.826	1889443
PVRGT	93	20.3	15.8	1886	0.000	37
MSSRGT	93	10.9	13.8	1013	0.000	41

PEARSON CORRELATION COEFFICIENTS / PROB > |R| UNDER HO:RHO=0 / NUMBER OF OBSERVATIONS

	LGCUTR	RGTTR	PVRGTTR	MSS	PVMSS	ADTRGT	PVMSSADT	PVRGT	MSSRGT
LGCUTR	1.00000	-0.47143	-0.36201	0.16309	0.14753	-0.07000	0.35199	-0.46818	0.00754
	0.0000	0.0001	0.0004	0.1224	0.1629	0.5097	0.0006	0.0001	0.9434
	91	91	91	91	91	91	91	91	91
RGTTR	-0.47143	1.00000	0.98151	0.46141	0.49369	0.79774	0.10486	0.98984	0.66384
	0.0001	0.0000	0.0001	0.0001	0.0001	0.0001	0.3226	0.0001	0.0001
	91	91	91	91	91	91	91	91	91
PVRGTTR	-0.36201	0.98151	1.00000	0.57974	0.61564	0.88663	0.23278	0.99062	0.76404
	0.0004	0.0001	0.0000	0.0001	0.0001	0.0001	0.0264	0.0001	0.0001
	91	91	91	91	91	91	91	91	91
MSS	0.16309	0.46141	0.57974	1.00000	0.99470	0.81320	0.89444	0.52825	0.93085
	0.1224	0.0001	0.0001	0.0000	0.0001	0.0001	0.0001	0.0001	0.0001
	91	91	91	93	93	91	91	93	93
PVMSS	0.14753	0.49369	0.61564	0.99470	1.00000	0.84151	0.87297	0.56279	0.95202
	0.1629	0.0001	0.0001	0.0001	0.0000	0.0001	0.0001	0.0001	0.0001
	91	91	91	93	93	91	91	93	93
ADTRGT	-0.07000	0.79774	0.88663	0.81320	0.84151	1.00000	0.52929	0.84107	0.92639
	0.5097	0.0001	0.0001	0.0001	0.0001	0.0000	0.0001	0.0001	0.0001
	91	91	91	91	91	91	91	91	91
PVMSSADT	0.35199	0.10486	0.23278	0.89444	0.87297	0.52929	1.00000	0.16463	0.68340
	0.0006	0.3226	0.0264	0.0001	0.0001	0.0001	0.0000	0.1189	0.0001
	91	91	91	91	91	91	91	91	91
PVRGT	-0.46818	0.98984	0.99062	0.52825	0.56279	0.84107	0.16463	1.00000	0.71865
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.1189	0.0000	0.0001
	91	91	91	93	93	91	91	93	93
MSSRGT	0.00754	0.66384	0.76404	0.93085	0.95202	0.92639	0.68340	0.71865	1.00000
	0.9434	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001	0.0000
	91	91	91	93	93	91	91	93	93

Table D.6 Corrected Regression Model for the Limestone Aggregates (Continued)

SAS

21:08 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN

ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	3	1718.03095	572.67698	17.713	0.0001
ERROR	83	2683.52778	32.33166003		
C TOTAL	86	4401.55873			

ROOT MSE	5.686094	R-SQUARE	0.3903
DEP MEAN	41.89266	ADJ R-SQ	0.3683
C.V.	13.57301		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	94.46100942	8.20527627	11.512	0.0001
LGCUTR	1	-3.86772509	0.57548906	-6.721	0.0001
RGTTR	1	-0.48916503	0.16707832	-2.928	0.0044
MSSRGT	1	0.26231247	0.06723474	3.901	0.0002

SAS

VARIABLE	MEAN	VARIANCE
LGCUTR	13.33255084	1.85636519
RGTTR	7.97289492	39.18597674
MSSRGT	10.89354839	191.60408836

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP LGCUTR	VAR PROP RGTTR	VAR PROP MSSRGT
1	3.311944	1.000000	0.0004	0.0005	0.0099	0.0166
2	0.530820	2.497860	0.0020	0.0025	0.0184	0.2577
3	0.154323	4.632610	0.0003	0.0025	0.5351	0.4786
4	0.0029133	33.717254	0.9972	0.9944	0.4366	0.2470

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	46.9552	1.2261	44.5165	49.3938	35.3858	58.5246	3.8658
2	43.2770	44.1642	1.0911	41.9940	46.3344	32.6484	55.6800	-0.8872
3	40.5170	41.9035	1.1067	39.7024	44.1047	30.3819	53.4252	-1.3865
4	39.4130	40.9055	1.1497	38.6189	43.1922	29.3672	52.4438	-1.4925
5	37.6000	39.9281	1.2105	37.5205	42.3357	28.3652	51.4910	-2.3281
6	37.2000	39.0098	1.2822	36.4596	41.5600	27.4164	50.6032	-1.8098
7	34.6290	46.1459	1.0549	44.0477	48.2441	34.6434	57.6483	-11.5169
8	.	43.4086	0.9596	41.4999	45.3172	31.9392	54.8779	.
9	31.5010	41.1536	1.0228	39.1193	43.1878	29.6626	52.6445	-9.6526
10	30.5810	40.1399	1.0889	37.9741	42.3058	28.6249	51.6549	-9.5589
11	32.0000	38.2228	1.2615	35.7136	40.7319	26.6383	49.8072	-6.2228
12	38.1250	46.1459	1.0549	44.0477	48.2441	34.6434	57.6483	-8.0209

Table D.7 Regression Model for the Lightweight Aggregates

SAS 21:12 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: LGFN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	4	1.37259590	0.34314898	43.574	0.0001
ERROR	124	0.97650124	0.007875010		
C TOTAL	128	2.34909714			

ROOT MSE	0.08874125	R-SQUARE	0.5843
DEP MEAN	4.053303	ADJ R-SQ	0.5709
C.V.	2.189356		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	3.08417426	0.07796841	39.557	0.0001
RGT	1	-0.14457234	0.02350367	-6.151	0.0001
RGTTR	1	6.28704E-08	1.01214E-08	6.212	0.0001
AGSR	1	0.008358348	0.000646096	12.937	0.0001
SV1	1	0.01133682	0.001453932	7.797	0.0001

SAS

VARIABLE	MEAN	VARIANCE
RGT	0.630769	0
RGTTR	833423.692308	1106090869040
AGSR	105.023077	282
SV1	11.458462	64

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP RGT	VAR PROP RGTTR	VAR PROP AGSR	VAR PROP SV1
1	4.052421	1.000000	0.0005	0.0092	0.0130	0.0007	0.0074
2	0.556845	2.697677	0.0030	0.0209	0.3084	0.0058	0.0001
3	0.244624	4.070123	0.0018	0.0217	0.2649	0.0064	0.3652
4	0.140522	5.370139	0.0013	0.8629	0.2862	0.0000	0.1776
5	0.0055877	26.930278	0.9933	0.0853	0.1276	0.9871	0.4497

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	.	4.0271	0.0197	3.9882	4.0661	3.8472	4.2071	.
2	3.9569	4.0307	0.0194	3.9924	4.0691	3.8510	4.2105	-0.0739
3	4.0604	4.0362	0.0190	3.9986	4.0737	3.8566	4.2158	0.0242
4	4.0636	4.0399	0.0187	4.0029	4.0769	3.8604	4.2194	0.0236
5	3.9853	4.0445	0.0184	4.0081	4.0809	3.8651	4.2239	-0.0592
6	4.1495	4.0499	0.0181	4.0141	4.0856	3.8706	4.2291	0.0996
7	4.1043	4.0548	0.0178	4.0195	4.0900	3.8756	4.2339	0.0495
8	4.2077	4.0588	0.0176	4.0240	4.0937	3.8798	4.2379	0.1489
9	4.0508	4.0362	0.0190	3.9986	4.0737	3.8566	4.2158	0.0146
10	4.0730	4.0399	0.0187	4.0029	4.0769	3.8604	4.2194	0.0331

Table D.8 Regression Model for the Sandstone Aggregates

SAS 21:29 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	1	418.59321	418.59321	13.599	0.0005
ERROR	62	1908.38891	30.78046630		
C TOTAL	63	2326.98212			

ROOT MSE	5.548015	R-SQUARE	0.1799
DEP MEAN	53.38289	ADJ R-SQ	0.1667
C.V.	10.39287		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	34.32426216	5.21444901	6.583	0.0001
AGSR	1	0.19269387	0.05225278	3.688	0.0005

SAS

VARIABLE	MEAN	VARIANCE
AGSR	98.90625000	178.94345238

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP AGSR
1	1.991117	1.000000	0.0044	0.0044
2	0.0088835	14.971230	0.9956	0.9956

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	45.1170	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	-7.5132
2	53.0290	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	0.3988
3	54.2000	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	1.5698
4	55.0000	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	2.3698
5	53.6000	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	0.9698
6	55.0000	52.6302	0.7229	51.1851	54.0753	41.4461	63.8143	2.3698
7	52.6610	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	-0.9326
8	49.7170	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	-3.8766
9	54.1330	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	0.5394
10	53.8000	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	0.2064
11	43.0000	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	-10.5936
12	54.6000	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	1.0064
13	48.2450	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	-5.3486
14	49.5330	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	-4.0606
15	55.9730	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	2.3794
16	54.0000	53.5936	0.6959	52.2027	54.9846	42.4164	64.7709	0.4064

Table D.9 Regression Model for the Siliceous Gravel Aggregates

SAS 21:19 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: LGFN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	3	3.61973902	1.20657967	228.441	0.0001
ERROR	75	0.39613555	0.005281807		
C TOTAL	78	4.01587458			

ROOT MSE	0.07267604	R-SQUARE	0.9014
DEP MEAN	3.8826	ADJ R-SQ	0.8974
C.V.	1.871839		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	4.11365330	0.10523380	39.091	0.0001
LGCUTR	1	-0.05534051	0.007201798	-7.684	0.0001
INRD	1	0.005196017	0.000378141	13.741	0.0001
CF2	1	0.000925081	0.000365233	2.533	0.0134

SAS

VARIABLE	MEAN	VARIANCE
LGCUTR	11.98183831	2.37246644
INRD	71.99550000	827.68323519
CF2	62.03250000	512.87791772

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP LGCUTR	VAR PROP INRD	VAR PROP CF2
1	3.783699	1.000000	0.0004	0.0006	0.0049	0.0075
2	0.132857	5.336609	0.0004	0.0073	0.4236	0.1428
3	0.079863	6.883129	0.0110	0.0276	0.0174	0.8493
4	0.0035812	32.504647	0.9882	0.9645	0.5540	0.0004

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	3.5232	3.5219	0.0235	3.4751	3.5686	3.3697	3.6740	0.001347
2	3.2853	3.4848	0.0216	3.4417	3.5279	3.3338	3.6359	-0.1995
3	3.4323	3.4545	0.0208	3.4130	3.4960	3.3039	3.6051	-0.0222
4	3.3192	3.4408	0.0207	3.3995	3.4820	3.2902	3.5913	-0.1216
5	3.3878	3.4277	0.0207	3.3864	3.4690	3.2771	3.5782	-0.0399
6	3.3534	3.4151	0.0209	3.3735	3.4567	3.2645	3.5658	-0.0617
7	3.4468	3.4039	0.0211	3.3618	3.4460	3.2531	3.5547	0.0429
8	3.3105	3.3960	0.0214	3.3534	3.4385	3.2450	3.5469	-0.0854
9	4.1008	4.0336	0.0161	4.0014	4.0657	3.8853	4.1819	0.0672
10	4.0150	3.9992	0.0138	3.9716	4.0268	3.8518	4.1466	0.0157
11	4.0083	3.9708	0.0128	3.9452	3.9964	3.8238	4.1179	0.0375
12	3.9982	3.9579	0.0127	3.9325	3.9832	3.8109	4.1049	0.0403

Table D.10 General Multivariable Regression Model for the Uncoated Aggregates — PV, LA, and INRD

SAS 21:30 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	4	7213.43700	1803.35925	59.642	0.0001
ERROR	182	5503.06429	30.23661700		
C TOTAL	186	12716.50129			

ROOT MSE	5.498783	R-SQUARE	0.5673
DEP MEAN	49.30698	ADJ R-SQ	0.5577
C.V.	11.15214		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR HO: PARAMETER=0	PROB > |T|
INTERCEP	1	43.02527016	5.46547005	7.872	0.0001
TR	1	-2.40601559	0.28673289	-8.391	0.0001
PVLA	1	0.01809692	0.002142575	8.446	0.0001
PVIRTR	1	0.000362851	0.000026334	13.779	0.0001
AGSR	1	0.12481432	0.03813107	3.273	0.0013

SAS

VARIABLE	MEAN	VARIANCE
TR	12.8061045	2.257
PVLA	775.5052083	39486.859
PVIRTR	28503.9760396	388810094.290
AGSR	102.1542553	152.687

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP TR	VAR PROP PVLA	VAR PROP PVIRTR	VAR PROP AGSR
1	4.640415	1.000000	0.0002	0.0005	0.0024	0.0068	0.0005
2	0.303516	3.910098	0.0003	0.0002	0.0200	0.5119	0.0022
3	0.042371	10.465098	0.0070	0.0212	0.8923	0.0780	0.0424
4	0.0098321	21.724723	0.0037	0.7640	0.0055	0.2818	0.3669
5	0.0038659	34.645953	0.9887	0.2141	0.0798	0.1215	0.5881

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	44.2134	1.0234	42.1941	46.2327	33.1774	55.2494	6.6076
2	43.2770	42.6751	0.9766	40.7482	44.6020	31.6556	53.6945	0.6019
3	40.5170	41.4290	0.9629	39.5291	43.3290	30.4142	52.4438	-0.9120
4	39.4130	40.8790	0.9642	38.9766	42.7814	29.8638	51.8942	-1.4660
5	37.6000	40.3402	0.9697	38.4269	42.2535	29.3231	51.3573	-2.7402
6	37.2000	39.8341	0.9787	37.9029	41.7652	28.8139	50.8543	-2.6341
7	34.6290	43.2851	0.9909	41.3299	45.2403	32.2606	54.3095	-8.6561
8	.	41.7768	0.9642	39.8744	43.6792	30.7616	52.7920	.
9	31.5010	40.5343	0.9671	38.6262	42.4424	29.5181	51.5505	-9.0333
10	30.5810	39.9758	0.9757	38.0506	41.9010	28.9566	50.9950	-9.3948

Table D.11 Regression Model for the Uncoated Limestone Aggregates — ABSP and INRD

SAS 21:55 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	4	6447.11432	1611.77858	46.790	0.0001
ERROR	182	6269.38697	34.44718115		
C TOTAL	186	12716.50129			

ROOT MSE	5.869172	R-SQUARE	0.5070
DEP MEAN	49.30698	ADJ R-SQ	0.4962
C.V.	11.90333		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	61.54797391	5.52246723	11.145	0.0001
LGCUTR	1	-3.02821868	0.33404173	-9.065	0.0001
IRLGTR	1	0.01338189	0.001057629	12.653	0.0001
ABSP	1	3.34441118	0.49664793	6.734	0.0001
AGSR	1	0.10357834	0.04025270	2.573	0.0109

SAS

VARIABLE	MEAN	VARIANCE
LGCUTR	12.80610451	2.256697
IRLGTR	812.30754215	293779.882553
ABSP	1.52656250	1.070443
AGSR	102.15425532	152.687308

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP LGCUTR	VAR PROP IRLGTR	VAR PROP ABSP	VAR PROP AGSR
1	4.434105	1.000000	0.0003	0.0005	0.0065	0.0086	0.0005
2	0.414818	3.269446	0.0000	0.0000	0.2152	0.2581	0.0001
3	0.138122	5.665937	0.0049	0.0028	0.3648	0.5622	0.0163
4	0.0089139	22.303323	0.0000	0.6767	0.3810	0.1495	0.4425
5	0.0040417	33.122461	0.9947	0.3200	0.0325	0.0216	0.5405

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	45.3989	1.0501	43.3269	47.4708	33.6345	57.1633	5.4221
2	43.2770	43.4629	1.0145	41.4611	45.4646	31.7106	55.2151	-0.1859
3	40.5170	41.8947	1.0183	39.8856	43.9039	30.1413	53.6482	-1.3777
4	39.4130	41.2025	1.0292	39.1717	43.2332	29.4453	52.9597	-1.7895
5	37.6000	40.5244	1.0453	38.4620	42.5869	28.7617	52.2872	-2.9244
6	37.2000	39.8875	1.0649	37.7863	41.9887	28.1179	51.6571	-2.6875
7	34.6290	42.5589	1.1045	40.3796	44.7382	30.7751	54.3427	-7.9299
8	.	40.6608	1.1168	38.4572	42.8644	28.8725	52.4491	.
9	31.5010	39.0972	1.1558	36.8167	41.3777	27.2943	50.9001	-7.5962
10	30.5810	38.3943	1.1812	36.0637	40.7248	26.5816	50.2070	-7.8133

Table D.12 Regression Model for the Petrographically Examined Aggregates

SAS 22:00 TUESDAY, AUGUST 27, 1991

DEP VARIABLE: FN
ANALYSIS OF VARIANCE

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PROB>F
MODEL	7	10285.02900	1469.28986	48.179	0.0001
ERROR	170	5184.44231	30.49671945		
C TOTAL	177	15469.47131			

ROOT MSE	5.522383	R-SQUARE	0.6649
DEP MEAN	47.17201	ADJ R-SQ	0.6511
C.V.	11.70691		

PARAMETER ESTIMATES

VARIABLE	DF	PARAMETER ESTIMATE	STANDARD ERROR	T FOR H0: PARAMETER=0	PROB > |T|
INTERCEP	1	54.12696489	5.50628947	9.830	0.0001
TR	1	-2.57508879	0.34124896	-7.546	0.0001
PGSTTR	1	0.01229235	0.001172017	10.488	0.0001
PCGTR	1	-0.01861412	0.002974918	-6.257	0.0001
PDG	1	0.16783233	0.05048944	3.324	0.0011
PNCM	1	0.10749939	0.03193486	3.366	0.0009
WVC	1	0.45583628	0.08439002	5.402	0.0001
AGSR	1	0.16075580	0.04110433	3.911	0.0001

SAS

VARIABLE	MEAN	VARIANCE
TR	13.47889198	1.921934
PGSTTR	825.18296447	270889.111120
PCGTR	177.35765002	61104.138953
PNCG	36.45151351	1341.530747
PDG	4.65000000	196.213995
PNCM	13.41891892	362.650999
WVC	3.29324324	35.408446
AGSR	106.57458564	253.201351

COLLINEARITY DIAGNOSTICS

NUMBER	EIGENVALUE	CONDITION NUMBER	VAR PROP INTERCEP	VAR PROP TR	VAR PROP PGSTTR	VAR PROP PCGTR	VAR PROP PDG	VAR PROP PNCM	VAR PROP WVC	VAR PROP AGSR
1	4.896949	1.000000	0.0002	0.0003	0.0040	0.0045	0.0031	0.0043	0.0067	0.0003
2	1.503766	1.804565	0.0001	0.0001	0.0030	0.0369	0.0694	0.0591	0.0374	0.0000
3	0.780203	2.505296	0.0000	0.0000	0.0048	0.0020	0.1622	0.0188	0.3726	0.0000
4	0.411604	3.449237	0.0003	0.0005	0.0247	0.0034	0.0882	0.4227	0.2402	0.0009
5	0.249741	4.428102	0.0003	0.0007	0.2838	0.1709	0.0742	0.0363	0.1801	0.0022
6	0.147828	5.755528	0.0015	0.0012	0.1300	0.6346	0.3777	0.1076	0.0029	0.0091
7	0.0059424	28.706719	0.0224	0.9232	0.2106	0.1380	0.1718	0.2138	0.0553	0.4022
8	0.0039663	35.137220	0.9751	0.0740	0.3391	0.0096	0.0534	0.1375	0.1048	0.5851

OBS	ACTUAL	PREDICT VALUE	STD ERR PREDICT	LOWER95% MEAN	UPPER95% MEAN	LOWER95% PREDICT	UPPER95% PREDICT	RESIDUAL
1	50.8210	43.[illegible]337	1.1527	41.6582	46.2091	32.7973	55.0700	6.8873
2	43.2770	42.[illegible]684	1.0385	40.1183	44.2185	31.0759	53.2609	1.1086
3	40.5170	40.7386	0.9699	38.8240	42.6533	29.6704	51.8069	-0.2216
4	39.4130	40.1074	0.9478	38.2364	41.9785	29.0466	51.1682	-0.6944

Pages 453-462 intentionally left blank

(originals miscounted)

Next section, References, begins on page 463

REFERENCES

1. Adam, V. and S.C. Shah, "Evaluation of Open-Graded Plant Mix Seal Surfaces for Correction of Slippery Pavements," TRR 523, Transportation Research Board, Washington, D.C., 1974, pp. 88-96.

2. Dietrich, R.V. and B.J. Skinner, Rocks and Rock Minerals, John Wiley and Sons, New York, 1979.

3. Williams, H., F.J. turner, and C.M. Gilbert, Petrography: An Introduction to the Study of Rocks in Thin Sections, W.H. Freeman and Company, San Francisco, 1982.

4. Greensmith, J.T., Petrology of the Sedimentary Rocks, 7th Edition, Unwin Hyman, London, 1989.

5. Anderson, V.L. and R.A., McLean, Design of Experiments: A Realistic Approach, Marcel Dekker, Inc., New York, 1974.

6. ANSI/ASTM D 3042-79, Annual Book for ASTM Standards, "Standard Test Method for Insoluble Residue in Carbonate Aggregates," American Specifications for Testing and Materials, 1979.

7. ANSI/ASTM D 3744-79, Annual Book for ASTM Standards, "Standard Test Method for Aggregate Durability Index," American Specifications for Testing and Materials, 1979.

8. ANSI/ASTM E 274-79, Annual Book for ASTM Standards, "Standard Test Method for Skid Resistance of Paved Surfaces Using A Full-Scale Tire," American Specifications for Testing and Materials, 1979.

9. Aughenbaugh, N.B. and R.W. Lounsbury, "Petrographic Analysis of Northern Indiana Carbonate Aggregates," Northwestern University, Evanston, Illinois, August 1966.

10. Balmer, G.G. and R.R. Hegmon, "Recent Developments in Pavement Texture Research," TRR 788, Transportation Research Board, Washington, D.C., 1980, pp. 28-33.

11. Balmer, G.G., "Pavement Texture: Its Significance and Development," TRR 666, Transportation Research Board, Washington, D.C., 1978, pp. 1-6.

12. Beaton, J.L., "Providing Skid Resistant Pavements," Skidding Accidents: Pavement Characteristics, TRR 622, Transportation Research Board, Washington, D.C., 1976, pp. 39-50.

13. Benson, F.C., et al., "Analysis of Field Skid Resistance Performance of Different Aggregates from District 15," Research Report No. 286-3, TTI, Texas A&M University, College Station, February 1984.

14. Brillet, F., "Skid Resistance of Highway Pavements: Assessments of Fourteen Years of National Skid-Resistance Surveying," Bulletin de Liaison des Laboratoires des Ponts, December 1984.

15. Burchett, J.L., and R.L. Rizenbergs, "Seasonal Variations in the Skid Resistance of Pavements in Kentucky," TRR 788, Transportation Research Board, Washington, D.C., 1980, pp. 6-14.

16. Carr, J.T., Jr., "The Climate and Physiography of Texas," TWDB Report 53, Texas Water Development Board, Austin, March 1969.

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

18. Clifford, J.M., "Skid-Resistance Measurements Made on Several Segmental Block Pavements," RP/13/83, National Institute for Transport and Road Research, Transvaal, South Africa, May 1983.

19. Colony, D.C., "Overview of Skid Resistance on Ohio Pavements," Journal of Transportation Engineering, Vol. 111, No. 5, American Society of Civil Engineers, September 1985, pp. 546-560.

20. Dahir, S.H., and J.J. Henry, "Alternatives for the Optimization of Aggregate and Pavement Properties Related to Friction and Wear Resistance," FHWA/RD-78/209, Pennsylvania Transportation Institute, University Park, April 1978.

21. Dahir, S.H., and W.E. Meyer, "Bituminous Pavement Polishing," ARP-S66, Federal Highway Administration, Washington, D.C., November 1974.

22. Dahir, S.H., and W.E. Meyer, "Polishing Characteristics of Common Rock Types Used as Aggregate in Bituminous Pavement Surfaces," ASTM Journal of Testing and Evaluation, Vol. 6, No. 1, American Society for Testing and Materials, January 1978, pp. 52-59.

23. Dahir, S.H., and W.G. Mullen, "Factors Influencing Aggregate Skid-Resistance Properties," HRR 376, Highway Research Board, Washington, D.C., 1971, pp.136-148.

24. Dahir, S.H., et al., "Laboratory Evaluation of Pavement Surface Texture Characteristics in Relation to Skid Resistance," FHWA/RD/75-60, Federal Highway Administration, Washington, D.C., June 1972.

25. Dahir, S.H., et al, "Seasonal Skid Resistance Variations," FHWA/PA 80/75-10, Federal Highway Administration, Washington, D.C., August 1979.

26. Dahir, S.H., W.E. Meyer, and R.R. Hegmon, "Laboratory and Field Investigation of Bituminous Pavement and Aggregate Polishing," TRR 584, Transportation Research Board, Washington, D.C., 1976, pp.1-14.

27. Dahir, S.H., "Petrographic Insights into the Susceptibility of Aggregates to Wear and Polishing," TRR 695, Transportation Research Board, Washington, D.C., 1978, pp. 20-27.

28. Dahir, S.H., "Review of Aggregate Selection Criteria for Improved Wear Resistance and Skid Resistance of Bituminous Surfaces," ASTM Journal of Testing and Evaluation, Vol. 7, No. 5, ASTM, September 1979, pp. 245-253.

29. Davis, M.M., and J.A. Epps, "Skid Resistant Surfaces," Research Report No. 214-7, TTI, Texas A&M University, College Station, December 1975.

30. Dodge, Y., Analysis of Experiments with Missing Data, John Wiley & Sons, New York, 1985.

31. Dolar-Manuani, L., "Importance of Petrographic Identification of Natural Aggregates," Ontario Hydro Research Quarterly, Vol. 22, No. 2, pp. 15-19.

32. Dunlap, D.F., P.S. Francher Jr.,.R.E. Scott, C.C. MacAdam, and L. Segel, "Pavement Skid-Resistance Requirements," TRR 584, Transportation Research Board, Washington, D.C., 1976, pp. 15-21.

33. Elmore, W.E. and K.D. Hankins, "Comparing Laboratory and Field Polish Rates of Coarse Aggregates for Skid Resistance Purposes," Research Report No. 216-1F, Texas Highway Department, Austin, July 1977.

34. Elsenaar, P.M. W., et al, "Pavement Characteristics and Skid Resistance," Skidding Accidents: Pavement Characteristics, TRR 622, Transportation Research Board, Washington, D.C., pp. 1-25.

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

36. Eppensteiner, W., et al, "Assessment of Mineral Aggregates in Bituminous Pavement Construction. The Relation Between Laboratory Tests and Performance in Practice. Part 2: Long Term Observations," Ministry of Building and Technology, Strassenforschung, Austria, 1979.

37. Epps, J. A., and B. M. Gallaway, "Synthetic Aggregate Seal Coats: Current Texas Highway Department Practices," Research Report No. 83-1, Texas A&M University, College Station, May 1972.

38. Epps, J.A., B.M. Gallaway, and M.R. Brown, "Synthetic Aggregate Seal Coats," Research Report No. 83-2F, Texas A&M University, College Station, May 1974.

39. Ferrara, T.C., J.P. Mahoney, and J.A. Epps, "Skid Resistance of Different Gravel Surface in District 25," Research Report No. 214-24, Texas A&M University, College Station, July 1981.

40. Furbush, M.A., et al, "Relationship of Skid Resistance to Petrography of Aggregates," PB-220 071, Federal Highway Administration, 1 July 1972.

41. Gallaway, B.M., and J.A. Epps, "Conventional Chip Seals as Corrective Measures for Improved Skid Resistance," TRR 523, Transportation Research Board, 1974, pp. 97-109.

42. Gallaway, B.M. and W.J. Harper, "Laboratory and Field Evaluation of Lightweight Aggregates as Coverstone for Seal Coats and Surface Treatments," HRR 150, Highway Research Board, Washington, D.C., 1966, pp. 25-81.

43. Gallaway, B.M., J.A. Epps, and H. Tomita, "Effects of Pavement Surface Characteristics and Textures on Skid Resistance," Research Report No. 138-4, Texas A&M University, College Station, March 1971.

44. Gandhi, P.M. and F.C. Benson, "Design of Skid-Resistant Asphalt Pavements," Research Report No. 286-4, Texas A&M University, College Station, June 1984.

45. Gartner, W., Jr., "Engineering and Administrative Considerations in Constructing, Maintaining and Testing Skid-Resistant Pavements," TRR 523, Transportation Research Board, Washington, D.C., 1974, pp. 15-19.

46. Gee, S., W.D. King, Jr., and R.R. Hegmon, "Pavement Texture Measurement by Laser: A Feasibility Study, " Surface Texture Versus Skidding: Measurements, Frictional Aspects and Safety Features of Tire-Pavement Interactions, ASTM STP 583, American Society for Testing and Materials, 1975, pp. 29-41.

47. Gillespie, T.D., "Pavement Surface Characteristics and Their Correlation With Skid Resistance," Report No. 12, Pennsylvania State University.

48. Goodwin, W.A., "Pre-Evaluation of Pavement Materials for Skid Resistance: A Review of U.S. Techniques," HS-009 492, Highway Research Board Western Summer Meeting, Denver, August 1968.

49. Gramling, W.L., "Effect of Aggregate Mineralogy on Polishing Rate and Skid Resistance in Pennsylvania," HRR 341, Highway Research Board, Washington, D. C., 1970, pp. 18-21.

50. National Cooperative Highway Research Program, "Guidelines for Skid-Resistant Highway Pavement Surfaces," Research Results Digest 89, November 1976, pp. 1-12.

51. Hair, J.F. Jr., R.E. Anderson, and R.L. Tatham, Multivariate Data Analysis, Macmillan Publishing Company, New York, 1987.

52. Hankins, K.D., and J.P. Underwood, "Two Aids for Field Personnel in Reducing Skidding Accidents," Research Report No. 135-7F, Texas Highway Department, Austin, August 1975.

53. Hankins, K.D., et al, "The Degree of Influence of Certain Factors Pertaining to the Vehicle and the Pavement on Traffic Accidents Under Wet Conditions," Research Report No. 133-3F, Texas Highway Department, Austin, September 1970.

54. Hankins, K.D., R.T. Gregory, and W.J. Berger, "Skidding Accident Systems Model," Research Report No. 135-1, Texas Highway Department, Austin, March 1971.

55. Hankins, K.D., "How to Provide Needed Skid Resistance: State of the Art, State of the Practice," Report No. SS 11.11, Texas Highway Department, Austin, January 1975.

56. Hankins, K.D., "Pavement Surface Texture as Related to Skid Resistance," Research Report No. 45-4, Texas Highway Department, Austin, August 1967.

57. Hegmon, R.R., "Seasonal Variations in Pavement Skid Resistance--Are These Real?," Public Roads, Washington, D.C., September 1978, pp. 55- 62.

58. Hegmon, R.R., "Skid Resistance of Polished Aggregates, Field and Laboratory Investigation," Report No. 67-11, Pennsylvania Department of Highways.

59. Henry, J.J. and K. Saito, "Skid-Resistance Measurements with Blank and Ribbed Test Tires and Their Relationship to Pavement Texture," TRR 945, Transportation Research Board, Washington, D.C., 1983, pp. 38-43.

60. Henry, J.J., K. Saito, and R. Blackburn, "Predictor Model for Seasonal Variations in Skid Resistance," Vol. II, Comprehensive Report, Pennsylvania Transportation Institute, Pennsylvania State University, University Park, April 1984.

61. Henry, J.J., and S.H. Dahir, "Effects of Textures and the Aggregates That Produce Them on the Performance of Bituminous Surfaces, " TRR 712, Transportation Research Board, Washington, D.C., 1979, pp. 44-50.

62. Henry, J.J., et al, "Evaluation of Pavement Texture," Pennsylvania Transportation Institute, University Park, PA, October 1984.

63. Henry, J.J., "Use of Blank and Ribbed Test Tires for Evaluating Wet-Pavement Friction," TRR 788, Transportation Research Board, Washington, D.C., 1980, pp. 1-6.

64. Hettmansperger, T.P., Statistical Inference Based on Ranks, John Wiley & Sons, New York, 1984.

65. Hill, B.J. and J.J. Henry, "Short-Term, Weather-Related Skid Resistance Variations," TRR 836, Transportation Research Board, Washington, D.C., 1981, pp. 76-81.

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

67. Hobeda, P., "Wear and Polishing of Aggregates: A Study Using an Electron Microscope," Report No. 26, Swedish National Road & Traffic Research Institute, Stockholm, Sweden.

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

69. Horne, W.B., and F. Buhlmann, "A Method of Rating the Skid Resistance and Micro/Macrotexture Characteristics of Wet Pavements," ASTM Special Technical Publications, American Society for Testing and Materials, 1983, pp. 191-218.

70. Hosking, J.R., "Synthetic Aggregates of High Resistance to Polishing. Part 1: Gritty Aggregates," TRRL LR 350, Transport and Road Research Laboratory, Crowthorne, England, 1970.

71. Hosking, J.R., "The Effect of Aggregate on the Skidding Resistance to Bituminous Surfacings: Factors Other Than Resistance to Polishing," TRRL LR 553, Transport and Road Research Laboratory, Crowthorne, England.

72. Huang, E.Y. and T. Ebrahimzaden, "Laboratory Investigation of the Effect of Particle Shape Characteristics and Gradation of Aggregates on the Skid Resistance of Asphalt Surface Mixtures," Skid Resistance of Highway Pavements, ASTM STP 530, American Society for Testing and Materials, 1973, pp. 117-137.

73. Hyyppa, J.M., "The Polishing of Mineral Aggregate in Road Surfacings/In English and Finnish/," State Institute of Technical Research, Finland, 1969.

74. Kachigan, S.K., Statistical Analysis: An Interdisciplinary Introduction to Univariate & Multivariate Methods, Radius Press, New York, 1986.

75. Kidd, S.Q., "Skid Resistance and Wear Properties of Aggregates for Asphalt Paving Mixtures," Mississippi State Highway Department, MSHD-RD-85-55, Jackson, Mississippi, February 1985.

76. Kilmurray, J.O., "Petrographic Studies, Area Distribution, and Technical Qualities of Tufas Used as Aggregates in the Province of Buenos Aires," Pan American Highway Congresses Proceedings No. 142, May 1963, pp. 790-803.

77. Kummer, H.W. and W.E. Meyer, "Tentative Skid-Resistance Requirements for Main Rural Highways, NCHRP Synthesis 37, Highway Research Board, Washington, D.C., 1967.

78. Larkin, T.J. and G.W. Bomar, Climatic Atlas of Texas, TDWR LP-192, Texas Department of Water Resources, Austin, December 1983.

79. Mahone, D.C. and R.K. Shaffer, "Corrective Programs for Improving Skid Resistance," HRR 376, Highway Research Board, Washington, D.C., 1971, pp. 54-58.

80. McCullough, B.F. and K.D. Hankins, "A Study of Factors Affecting the Operation of a Locked Wheel Skid Trailer," Research Report No. 45-3, Texas Highway Department, Austin, August 1966.

81. McCullough, B.F. and K.D. Hankins, "Skid Resistance Guidelines for Surface Improvements on Texas Highways," Research Report No. 45-2, Texas Highway Department, Austin, 1965.

82. Mendenhall, W. and J.E. Reinmuth, Statistics for Management and Economics, Duxbury Press, Boston, 1978.

83. Meyer, W.E., "Synthesis of Frictional Requirements Research," FHWA/ RD-81/159, Federal Highway Administration, Washington, D.C., June 1982.

84. Miller, R.W. and W.P. Chamberlin, "Skid Resistance of Bituminous Pavements Built with Carbonate Aggregates," FHWA-NY-RR-80-77, Federal Highway Adminstration, Washington, D.C., April 1980.

85. Mullen, W.G., S.H. Dahir, and B.D. Barnes, "Two Laboratory Methods for Evaluating Skid-Resistance Properties of Aggregates," HRR 376, Highway Research Board, Washington, D.C., pp.123-135.

86. Mullen, W.G., S.H. Dahir, and N.F. El Madani, "Laboratory Evaluation of Aggregates, Aggregate Blends, and Bituminous Mixes for Polish Resistance," TRR 523, Transportation Research Board, Washington, D.C., 1974, pp. 56-64.

87. Mullen, W.G., "Prediction of Pavement Skid Resistance from Laboratory Tests," TRR 523, Transportation Research Board, Washington, D.C., 1974, pp. 40-55.

88. Olsen, R.E., "A Review of European Practices for Laboratory Evaluation of Aggregate Polishing Characteristics," HRR 341, Highway Research Board, Washington, D.C., 1970, pp. 15-17.

89. Papaleontiou, C., A.H. Meyer, and D.W. Fowler, "Evaluation of the 4-Cycle Magnesium Sulfate Soundness Test," Research Report 438-1F, Center for Transportation Research, The University of Texas at Austin, November 1987.

90. Patty, T.S., "Accelerated Polish Test for Coarse Aggregate," Texas Highway Department, Austin, August 1973.

91. Patty, T.S., "Petrography as Related to Potential Skid Resistance of Paving Aggregates Used on Texas Highway Projects," Texas Highway Department, Austin, March 1975.

92. Patty, T.S., "Rapid Field Method for Determining the Polish Susceptibility of Carbonate Aggregates," Texas Highway Department, Austin, August 1974.

93. Federal Highway Adminstration, "Pavement Surface Texture: Significance and Measurement," FHWA/RD-84/091, Study No. 31W2-022, Washington, D.C.

94. Pereira, O.A. and R.R. Cunha, "Determination of Skidding Resistance of Pavements by the Sand Patch Method," R & D Report No. 20, National Lab of Civil Engineering, Lisbon, Portugal, February 1972, pp. 10.

95. Pindyck, R.S. and D.L. Rubinfeld, Econometric Models and Economic Forecasts, McGraw-Hill Book Company, St. Louis, 1981.

96. Rice, J.M., "Seasonal Variations in Pavement Skid Resistance," Public Roads, Washington, D.C., March 1977, pp. 160-166.

97. Rose, J.G., et al, "Summary and Analysis of the Attributes of Methods of Surface Texture Measurement," Skid Resistance of Highway Pavements, ASTM STP 530, American Society for Testing and Materials, 1973, pp. 60-77.

98. Saito, K. and J.J. Henry, "Mechanistic Model for Predicting Seasonal Variations in Skid Resistance," TRR 946, Transportation Research Board, Washington, D.C., 1983, pp. 29-37.

99. Salt, G.F., "Research on Skid-Resistance at the Transport and Road Research Laboratory (1927-1977)," Skidding Accidents: Pavement Characteristics, TRR 622, Transportation Research Board, Washington, D.C., pp. 26-38.

100. Schonfeld, R. "Photo Interpretation of Pavement Skid Resistance in Practice," TRR 523, Transportation Research Board, Washington, D.C., 1974, pp. 65-75.

101. Highway Research Board, "Skid Resistance," NCHRP Synthesis 14, Washington, D.C., 1972.

102. Smith, H.A., "Pavement Contributions to Wet-Weather Skidding Accident Reduction," Skidding Accidents: Pavement Characteristics, TRR 622, Transportation Research Board, Washington, D.C., pp. 51-59.

103. XVIIIth World Road Congress, "Surface Characteristics," Technical Committee Report No. 1, Brussels, Belgium, September 13-19, 1987.

104. Federal Highway Administration, "Synthesis of Safety Research Related to Traffic Control and Roadway Elements--Volume 1," FHWA/TS-82/232, Washington, D.C., December 1982.

105. Test Method Tex-210-F-1986, Manual of Test Methods, "Determination of Asphalt Content of Bituminous Mixtures by Extraction," Texas State Department of Highways and Public Transportation, March 1986.

106. Test Method Tex-401-A-1983, Manual of Test Methods, "Sieve Analysis of Fine and Coarse Aggregate," Texas State Department of Highways and Public Transportation, January 1983.

107. Test Method Tex-403-A-1983, Manual of Test Methods, "Saturated Surface-Dry Specific Gravity and Absorption of Aggregates," Texas State Department of Highways and Public Transportation, January 1983.

108. Test Method Tex-406-A-1983, Manual of Test Methods, "Materials Finer Than No. 200 Sieve in Mineral Aggregates (Decantation Test for Concrete Aggregates)," Texas State Department of Highways and Public Transportation, January 1983.

109. Test Method Tex-410-A-1983, Manual of Test Methods, "Abrasion of Coarse Aggregate by Use of the Los Angeles Machine," Texas State Department of Highways and Public Transportation, January 1983.

110. Test Method Tex-411-A-1978, Manual of Test Methods, "Soundness of Aggregate by Use of Sodium Sulphate or Magnesium Sulphate," Texas State Department of Highways and Public Transportation, January 1978.

111. Test Method Tex-413-A-1983, Manual of Test Methods, "Determination of Deleterious Material and/or Crushed Particles in Mineral Aggregate," Texas State Department of Highways and Public Transportation, January 1983.

112. Test Method Tex-432-A-1983, Manual of Test Methods, "Coarse Aggregate Freeze-Thaw Test," Texas State Department of Highways and Public Transportation, January 1983.

113. Test Method Tex-433-A-1983, Manual of Test Methods, "Absorption and Dry Bulk Specific Gravity of Synthetic Coarse Aggregate," Texas State Department of Highways and Public Transportation, January 1983.

114. Test Method Tex-436-A-1983, Manual of Test Methods, "Measurement of Texture Depth by the Sand-Patch Method," Texas State Department of Highways and Public Transportation, January 1983.

115. Test Method Tex-438-A-1983, Manual of Test Methods, "Accelerated Polish Test for Coarse Aggregate," Texas State Department of Highways and Public Transportation, January 1983.

116. Ubach, A., "Study of the Susceptibility to Polishing of Aggregate Mixtures of Different Petrological Composition," Transportation and Soil Mechanics Lab, Spain, July 1970.

117. Underwood, J.P., "Field Friction Performance of Several Experimental Test Sections," Research Report No. 126-1, Texas Highway Department, Austin, March 1971.

118. Underwood, J.P., K.D. Hankins, and E. Garana, "Aggregate Polishing Characteristics: The British Wheel Test and the Insoluble Residue Test," Research Report No. 126-2, Texas Highway Department, Austin, January 1971.

119. Weisberg, S., Applied Linear Regression, John Wiley & Sons, New York, 1980.

120. Wylde, L.J., "Literature Review: Crushed Rock and Aggregate for Road Construction: Some Aspects of Performance, Test Methods and Research Needs," Australian Road Research Board, January 1976.

121. Yager, T.J. and F. Buhlmann, "Macrotexture and Drainage Measurements on a Variety of Concrete and Asphalt Surfaces," Pavement Surface Characteristics and Materials, ASTM STP 763, C. M. Hayden, Ed., American Society for Testing and Materials, 1982, pp. 16-30.

122. Epps, J.A., B.M. Gallaway, and C.H. Hughes, "Field Manual on Design and Construction of Seal Coats," Research Report 214-25, Texas Transportation Institution, The Texas A&M University System, College Station, July 1981.

123. Howerter, E.D and T.J. Rudd, "Automation of the Schonfeld Method for Highway Surface Texture Classification," TRR 602, TRB, Washington, D.C., 1976.

124. Meyer, W.E., R.R. Hegmon, and T.D. Gillespie, "Locked-Wheel Pavement Skid Tester Correlation and Calibration Techniques," NCHRP Report 151, Transportation Research Board, 1974.

125. Hankins, K.D., "Pavement Material Properties as Related to Skid Resistance," Research Report 45-5F, Texas Highway Department, August 1969.

126. Epps, J.A., et al, "Performance of Surface Courses Constructed with Limestone Rock Asphalt Materials," Research Report 214-14, Texas Transportation Institute, Texas A&M University, College Station, November 1979.

127. Gallaway, B.M., R.E. Schiller, Jr., and J.G. Rose, "The Effects of Rainfall Inensity, Pavement Cross Slope, Surface Texture, and Drainage Length on Pavement Water Depth," Research Report 138-5, Texas A&M University, College Station, May 1971.

128. Hughes, C.H., Sr. and J.A. Epps, "Sprinkle Treatment-How, Why and Where," Research Report 214-4, Texas Transportation Institute, Texas A&M University, College Station, December 1975.

129. McLeod, N.W., "A General Method of Design of Seal Coats and Surface Treatment," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969.

130. Wright, N., "Recent Development in Surface Dressing in the United Kingdom," TRRL Supplementory Report 486, Transportation and Road Research Laboratory, 1979.

131. Kersten, M.S. and E.L. Skok, Jr., "Criteria for Seal Coating Bituminous Surfaces," University of Minnesota, 1969.

132. Epps, J.A., C.W. Chaffin, and A.J. Hill, "Field Evaluation of a Seal Coat Design Method," Research Report 124-22, Texas Transportation Institute, Texas A&M University, College Station, July 1980.

133. AASHTO® Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D.C., 1986.

134. Yoder, E.J. and M.W. Witczak, Principles of Pavement Design, Second Edition, John Wiley & Son, Inc., 1975.

135. Desolminihae, H.E., W.R. Hudson, and E. Ricci, "Serviceability Ratings of Texas Highway System for Pavement Management," Research Report 400-1F, Center for Transportation Research, The University of Texas at Austin, August 1986.

136. American Specifications for Testing and Materials, 1975,.ASNI/ASTM C295-65, Annual Book for ASTM Standards, "Standard Recommended Practice for Petrographic Examination of Aggregates for Concrete,"

137. ANSI/ASTM E 303-83, Annual Book for ASTM Standards, "Standard Method for Measuring Surface Frictional Properties Using the British Pendulum Tester," American Specifications for Testing and Materials, 1983.

138. Epps, J.A., A.H. Meyer, I.E. Larrimore, and J.L. Jones, "Roadway Maintenance Evaluation User's Manual," Research Report 151-2, Texas Transportation Institute, September 1974.

139. ASTM, Committee E17 on Surface Characteristics, Minutes of the Meeting, San Antonio, June 1986.

140. Heaton, B.S., J.J. Emery, and N.A. Kamel, "Prediction of Pavement Skid Resistance Performance," Australian Road Research Board Proceedings, Vol. 9, Part 3, 1978, pp. 121-126.

141. Bass, D.A., "Use of the Skid Trailer in District 2 for the Evaluation of Asphalt Paving Aggregates", to be published in a Research Digest by the Transportation Planning Division (D-10) of the Texas SDHPT.

CTR
Center for Transportation Research
3208 Red River, Suite 200
Austin, Texas 78705