A COMPREHENSIVE METHODOLOGY FOR PREDICTING FIELD SKID RESISTANCE OF BITUMINOUS AGGREGATES BASED ON LABORATORY TEST DATA AS WELL AS THEIR PAST SKID PERFORMANCE

by

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16. Abstract:

A comprehensive laboratory and field test program was undertaken with the objective of developing an improved procedure for predicting field skid resistance performance of bituminous aggregates. The field test program included monitoring of 55 test pavement sections that were located in various climatic zones within the state of Texas over the 3-year study period. As a part of this monitoring program skid resistance of the pavement at 64 kmph and other lower speeds, British pendulum number, and pavement macrotexture were measured. The laboratory test program consisted of complete characterization of the pavement coarse aggregates using the following test methods: polish value test, Magnesium sulfate soundness test, LA abrasion test, acid insoluble residue test and petrographic analysis. The skid resistance data collected over the 3-year study period were than used to develop a "skid performance rating" for each pavement section. Subsequently, appropriate statistical analyses were conducted to develop regression models that relate skid performance rating to various laboratory test parameters. The findings revealed that much better correlations are obtained when aggregates are categorized into sub-groups that contain aggregate with similar mineralogical make up. Accordingly aggregates were categorized based on percent carbonate minerals and the acid insoluble residue. Statistical regression models were then developed for each aggregate category. A methodology was also developed to predict skid performance rating of the aggregate based on past skid performance data. This approach utilizes skid resistance measurements on in-service pavement that have been constructed previously using the aggregate to interest. A windows based computer software named SKIDRATE was developed to implement the proposed methodology.

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IMPLEMENTATION STATEMENT

A Windows based computer software named SKIDRATE was developed to facilitate the implementation of the aggregate source rating procedure that was developed in this research. The necessary documentation on the SKIDRATE software is found in a separate companion report. SKIDRATE uses Visual Basic v 5.0 as the front end of ACCESS v 8.0 as the back end on a Windows NT v 4.0 platform. The software stores laboratory test data on all the aggregate sources used in TxDOT pavement construction projects. It also stores skid resistance data on pavements that have been constructed using each aggregate source. Subsequently, the above database can be used to predict the field skid performance rating of any given aggregate source based on (a) laboratory test data or (b past skid performance data. It allows the user to specify a desired level of reliability.

Prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view of policies of the Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Not intended for construction, bidding, or permit purposes. The engineer in charge of the research study was Priyantha W. Jayawickrama.

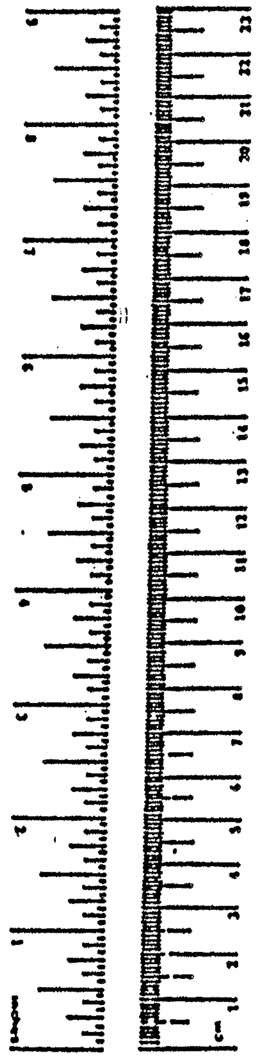
TRADE NAMES AND MANUFACTURERS' NAMES

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

METRIC CONVERSION FACTORS

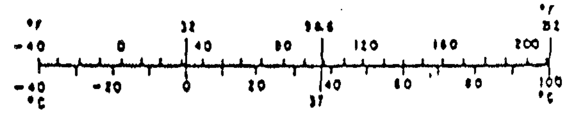
Approximate Conversions to Metric Measures

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



Approximate Conversions from Metric Measures

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



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CHAPTER 1
INTRODUCTION

1.1 GENERAL BACKGROUND

A hot mix asphalt concrete (HMAC) pavement should be designed so that it would be stable under the acceleration, deceleration and vertical loads from vehicles. In addition, it must provide adequate resistance against sliding to permit normal vehicle turning and braking movements. The frictional resistance offered by the pavement to sliding is commonly expressed in terms of the parameter known as *Skid Number (SN)*. The skid number is defined as the ratio between the frictional resistance acting along the plane of sliding and the load perpendicular to this plane. It is an important safety related property of the pavement surface that must be accounted for through the proper selection of materials, design and construction.

It is generally agreed that the skid resistance of a pavement surface is influenced by both its *microtexture* and its *macrotexture* (*1*). The term microtexture is used to describe the fine scale grittiness that is present on the surface of coarse aggregates used in the mix. The magnitude of this component will be determined by two factors: first, the initial roughness on the aggregate surface and secondly, the aggregates' ability to retain this roughness against the polishing action of traffic. Accordingly, microtexture is an aggregate related property that can be controlled through the selection of aggregates with desirable polish-resistant characteristics. The evaluation of the aggregates with respect to their polishing behavior can be accomplished by using a laboratory test procedure that has been developed for this purpose.

The term macrotexture, on the other hand, is used to describe the large scale roughness that is present on the pavement surface due to the arrangement of aggregate particles. The magnitude of this component will depend on several factors. The initial macrotexture on a pavement surface will be determined by the size, shape and gradation of coarse aggregates used in pavement construction as well as the particular construction technique used in the placement of the pavement surface layer. The properties of the bituminous mix and environmental factors such as service temperature will then determine how well the macrotexture will be preserved under traffic action.

It may be inferred from the foregoing information that good skid resistance on an HMAC pavement surface could be achieved by controlling its microtexture and macrotexture. The first of these, as explained above, must be accomplished through the selection of good quality, polish resistant aggregates. The second will depend on the type of mix specifically, the aggregate gradation (i.e. dense-graded mix or open-graded) and the stability of the mix. Although the fundamental approach used in the design of HMAC pavements for skid resistance can be summed up in this manner, the actual procedures used in aggregate evaluation and the standards adopted to define acceptable level of performance vary significantly from one state agency to another.

1.2 TXDOT SKID ACCIDENT REDUCTION PROGRAM

Texas Department of Transportation (TxDOT)'s wet weather skid accident reduction program was first established in 1974 and has been in existence since that time. A bituminous aggregate rating procedure known as *Rated Sources Polish Value* (*RSPV*) serves as the primary basis for this program. In addition, TxDOT allows aggregate qualification based on its skid performance history as a secondary and alternative method. In this section, these two methods are described in detail.

1.2.1 Rated Source Polish Value (RSPV) Procedure

An *RSPV* is required to be established only for those sources that produce material for bituminous pavement *surface* course construction. As a first step, the candidate source must be included in the department's quality monitoring (*QM*) program. All aggregate sources that are included in the *QM* program are sampled by a department representative on a regular basis. The samples are then tested in the TxDOT Materials and Tests Division laboratories to determine their polish value. All polish value samples are prepared and tested in accordance with Test Method Tex-438-A, "Accelerated Polish Test for Coarse Aggregates" (*2*). The *RSPV* for the aggregate source will be calculated based on the five most recent *QM* polish value test results. The *RSPV* for a given aggregate source represents the lower statistical limit of the PV values above which 90 percent of the aggregate sample population from that source should fall.

The relationship used in the above calculation is shown as Equation (1.1) below (*3*):

$$\text{RSPV} = \bar{x} - 1.533 \left(\sqrt{\frac{\text{MS}}{5}} \right) \quad \text{(1.1)}$$

where x = average of the five most recent QM polish values
MS = variance of the five most recent QM polish values

The sampling frequency for a given aggregate source will depend on two factors: (a) volume of material supplied to the department annually and (b) variability in the polish values measured in previous tests. If the variance of the five samples used to calculate RSPV does not exceed 3.5 *and* the volume of material supplied is less than 100,000 tons/year then the sampling frequency for aggregate source will be once every six months. But if the volume of material supplied annually is more than 100,000 tons then the sampling frequency is increased to once every three months. However, if the variance of the five samples used to calculate the *RSPV* is 3.5 or greater then a suitable sampling frequency for that specific source will be determined after evaluation of the polish value data.

The above procedure for establishing an *RSPV* is applicable only to aggregate sources that have maintained *active status* within the department's *QM* Program. Such aggregate sources may supply materials to pavement construction projects provided that their *RSPV* satisfies the minimum PV requirement for the given project. The minimum PV requirement for the project depends on the traffic volume expected on the roadway as shown in Table 1.1. Other sources that do not have an *RSPV* - called *informational sources* - are required to qualify their material on a project by project basis.

Table 1.1. Texas Department of Transportation PV Requirements

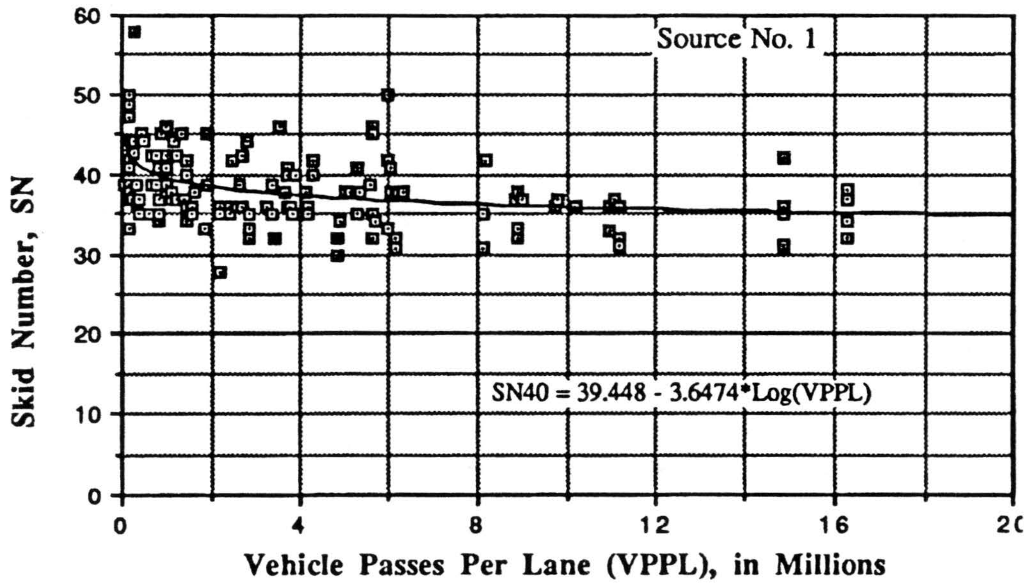
ADT	Minimum PV
Interstate Highways	32
Greater than 5,000	32
2,000 to 5,000	30
750 to 2,000	28
Less than 750	No requirement

1.2.2 Use of Skid Performance History for Aggregate Evaluation

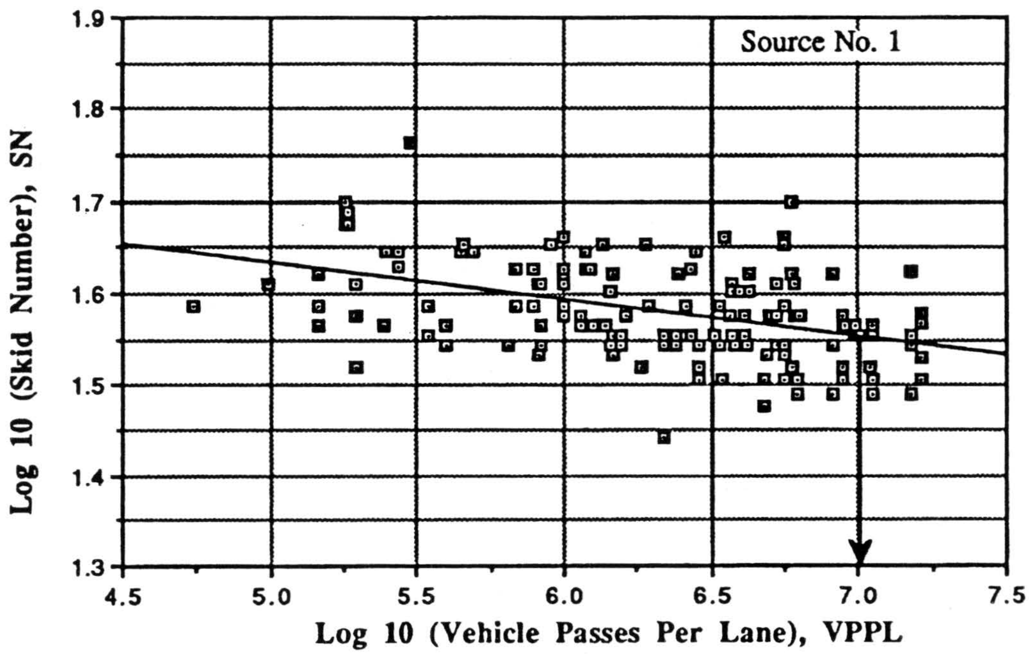
The above method for the evaluation of aggregate frictional characteristics relies on the results of the aggregate polish value (PV) as determined by test method Tex-438-A. However, more recent data suggests that some aggregates provide good skid performance in the field although they performed poorly in the laboratory polish value test.

The skid performance history for a given aggregate source is developed from skid numbers (SN_{40}) measured on pavements which have been constructed using aggregates of that type and from that source. A single data point would typically represent the average of a number of measurements made on a given test section of the roadway. For each of these data points, the cumulative number of vehicle passes corresponding to the lane on which the skid measurements were made is estimated and recorded. From this data plots of SN_{40} versus cumulative vehicle passes per lane (VPPL) can be prepared. Figure 1.1(a) and 1.2(a) are examples of such plots that have been obtained for two separate aggregate sources in Texas. These plots use linear scale and show the deterioration of skid performance with accumulation of traffic. For the analysis, however, the data must be plotted on logarithmic scale. The logarithmic plots for the same aggregate sources are shown in Figures 1.1(b) and 1.2(b). The bold lines represent the best-fit linear regression models. This linear relationship between $\log_{10}(SN_{40})$ and $\log_{10}(VPPL)$ now represents the skid performance history of the aggregate source. This model will provide the basis for aggregate qualification based on past skid performance. The qualification of the aggregate based on skid history must be performed on a project by project basis.

The procedure used in aggregate qualification can be best explained using the following example. Consider a 6-lane (3-lanes in each direction) roadway with an ADT of 8,000; design life of HMAC surface course = 8 years; traffic speed of 60 mph. Based on the information provided in Table 1.2 (*4*), a minimum skid number of 35 should be maintained on this pavement during its service life. Assuming that aggregate source No.1 is to be used in the construction of this pavement, we enter the graph shown in Figure 1.1(b) with logarithm of the desired SN_{40} i.e. $\log_{10} 35 = 1.55$, and read-off $\log_{10}(VPPL) = 7$ from the x-axis. Taking anti-logarithm VPPL = 10 million. Accordingly, the pavement surface can sustain 10 million vehicle passes on the most heavily traveled lane before the skid



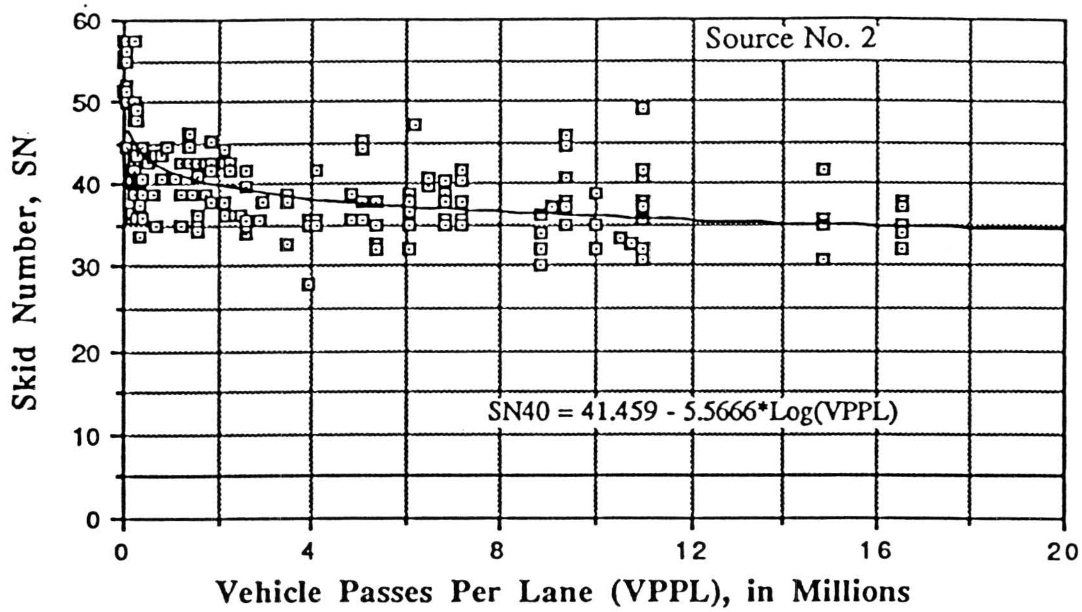
(a)



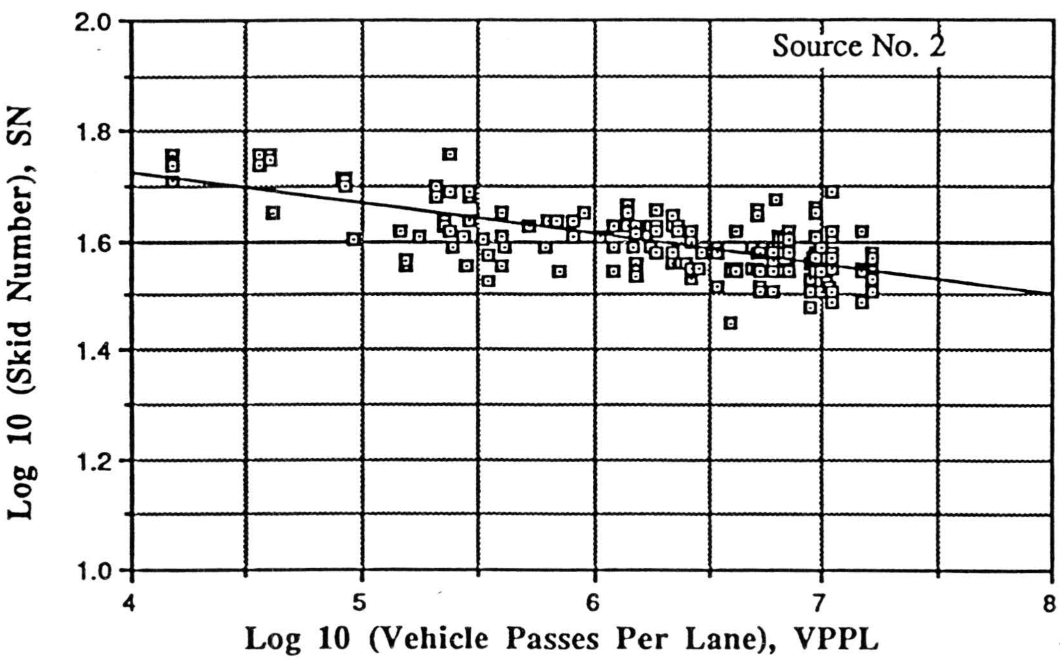
(b)

Figure 1.1. Skid Performance History for Texas Aggregate Source No.1

(a) Linear Scale, (b) Logarithmic Scale



(a)



(b)

Figure 1.2. Skid Performance History for Texas Aggregate Source No.2
(b) Linear Scale, (b) Logarithmic Scale

number falls below the desired value of 35. Now, using a directional distribution factor of 0.5 and a lane distribution factor of 0.7 the corresponding useful service life for a roadway with ADT = 8,000 will be 9.78 years. Since this number is larger than the design life of the pavement surface course (i.e. 8 years) the aggregate qualifies for use in this project.

Table 1.2. Design Guidelines for Minimum Acceptable SN_{40} for Given Traffic Speeds and Surface Types (*4*)

Mean	SN_{40}	
	STR	ACP
40 mph	33	33
50 mph	33	34
60 mph	33	35

STR= Surface Treatments
ACP=Asphalt Concrete Pavements

The above approach has been used with good success in several Texas districts. The method has been used to qualify many aggregate sources that do not meet the desired PV requirement but have provided satisfactory performance in the past. These aggregates continue to produce good in-service performance. This approach, therefore, deserves further investigation to determine its potential to be incorporated into the skid accident reduction program.

1.3 LIMITATIONS AND SHORTCOMINGS IN THE CURRENT TXDOT AGGREGATE QUALIFICATION PROCEDURES

1.3.1 Limitations and Shortcomings of the RSPV Procedure

The rated source polish value program that was described in Section 1.2.1 was originally introduced in 1974 based on the findings and recommendations from TxDOT research study 126-2. In the research project report 126-2, a British Pendulum Number of 28 was considered to be equivalent to a skid number of 32 in the field. After implementation of the proposed specification in Bryan District on an experimental basis, the researcher observed that there was significant variation of the polish value of the aggregate that was supplied by the same source but at different times. Therefore, an average polish value of 28 for a given source may not necessarily provide satisfactory performance of the aggregate in the field. The typical standard deviation corresponding to the above variation was found to be about 2. Therefore, the polish value specification was revised and the acceptable PV limit raised by 3 standard deviations. Accordingly, a PV limit of 34 (i.e. the standard deviation, 2 multiplied by 3 and added to the mean PV value of 28) was recommended for the specification. By allowing three standard deviations above the mean polish value the specifications ensure that there is a 99.7% probability that the actual PV will be greater than or equal to 28. Soon after, FHWA required documentation showing that all material used in highway surface course construction to be skid resistant. In response to that requirement the following polish value categories were established and introduced by TxDOT through Administrative Circular 22-74.

The above specifications were later revised in 1982 based on a review performed by the Texas Transportation Institute (TTI). TTI recommended the use of 2 standard deviations, instead of the 3 used earlier. The use of two standard deviations will have a probability of including 95% of the source variation. The revised specification was introduced by TxDOT Administrative Circular 28-83 and is shown in Table 1.1. This specification forms the basis for TxDOT skid accident reduction program today.

Table 1.3. Original PV Requirements Introduced by AC. 22-74

ADT	Minimum PV
Interstate Highways	35
Greater than 5,000	35
2,000 to 5,000	33
750 to 2,000	30
Less than 750	No requirement

Recalling that RSPV value itself is calculated after allowing for the source variation, it is obvious that, when the PV specification shown in Table 1.1 is used in conjunction with RSPV, there are two separate layers of statistical confidence built into the aggregate qualification procedure. Therefore, it is logical to expect a large margin of conservatism in this procedure. On the other hand, the traffic volumes and vehicle speeds on Texas highways have changed dramatically since the time the specifications were developed. The rapid growth of vehicular traffic, therefore, may have offset some of this conservatism. The applicability of the PV specification to the present day conditions can only be established through a comprehensive evaluation.

Furthermore, there have been a number of criticisms regarding the use of the laboratory PV test to evaluate aggregate performance in the field. Many of these criticisms stem from the poor correlation that has been observed between the laboratory polish values and the field skid measurements (SNs). In other words, some aggregate sources with low RSPV have demonstrated good skid performance on in-service pavements and vice versa. There are several reasons that may lead to poor correlation between aggregate source RSPV value and aggregate field skid measurements. They are as follows.

(a) Pavement skid resistance may depend on factors other than the frictional properties of the coarse aggregate used in its construction. Examples of such factors include pavement macrotexture and fine aggregates used in the mix.

(b) Poor reproducibility of polish value measurements

(c) Poor reproducibility of field skid measurements

Therefore, each of these factors must be carefully investigated in a comprehensive research study.

1.3.2 Limitations and Shortcomings of the Skid Performance History Approach

One of the major limitations in the skid performance history approach is the time needed for the development of the performance history for a given aggregate source. In other words, you must have historical data for the particular aggregate source you want to evaluate. Therefore, obviously this approach cannot be used for new aggregate sources. Secondly, it is generally agreed that the quality of the material that is produced from a given source can vary significantly over time. Therefore, it is questionable whether the performance of aggregates produced from a source in the past can be used as the basis for evaluation of the aggregates produced by that source at the present time.

Another major difficulty in the application of the skid performance history approach on a routine basis is the *variability* (or *lack of reproducibility*) associated with skid measurements. As explained earlier, the skid performance histories such as those shown in Figures 1.1 and 1.2 are developed from skid measurements made on different pavement sections built with aggregates of the same type and source. A number of possible reasons for the data scatter seen in these plots can be identified. First of all, it must be recognized that although all skid measurements correspond to the same aggregate source, there can be inherent differences in the pavement sections from which the data have been collected. Such differences may include variations in pavement macrotexture (e.g. open-graded versus dense graded mix), traffic characteristics (e.g. percent trucks). In addition to these, a number of other factors cause variability in skid measurements even if all the measurements were taken on the same pavement section. Such factors include: seasonal changes, variability in the test surface, pavement distress such as flushing or raveling, improper calibration of equipment and operator error. It is significant to note that, even though the large variability in skid data is quite evident, the present skid performance history procedure does not allow this variability to be taken into consideration. This may be demonstrated with the aid of Figure 1.3. In this figure, the skid data for the aggregate source No.1 has been reproduced at a larger scale. Also in this figure, the 60%, 80% and 90% confidence limits for the data are shown in addition to the best-fit linear regression model. In the example discussed previously, it was demonstrated that the regression model predicts that the pavement can sustain a maximum VPPL of 10 million during its useful service life. This means that, on average, the pavement will be able to carry a VPPL of 10 million before its SN deteriorates to a value of 35. In other words, there is a probability of 50% that the actual VPPL for the SN to reach 35 is less than that predicted by the model. Thus the reliability of prediction is only 50%. The confidence limits shown on the figure allow us to calculate the actual VPPLs (or pavement useful service lives) associated with higher reliabilities. For example if we consider the 60% confidence limits, we find that there is a 20% probability (i.e. half of 40% outside confidence limits) that the actual VPPL is less than 2.0 million (i.e. antilog of 6.3; See Figure 1.3). In other words, one out five times the actual service life of the surface course can be as low as one-fifth of the life predicted based on historical performance data. Thus, at 60% reliability the predicted service life of the pavement is 20 years. This example clearly illustrates the poor reliability that is inherent in the current skid performance history procedure.

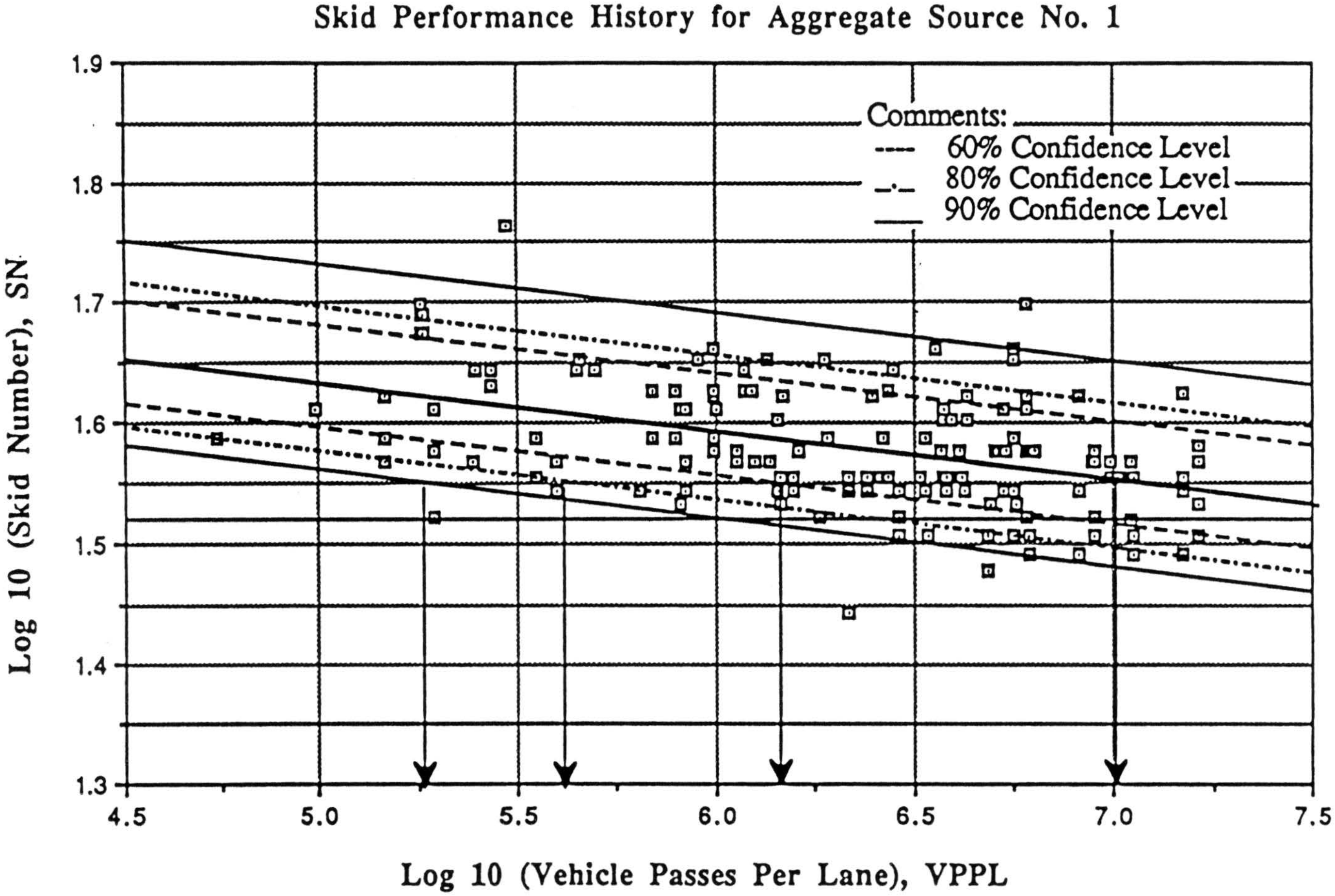


Figure 1.3 Predicted Pavement Service Life and the Corresponding Reliabilities

1.4 OBJECTIVES AND SCOPE OF THE RESEARCH STUDY

This research project was initiated with the primary objective of conducting a comprehensive evaluation of the current TxDOT aggregate qualification procedures and making recommendations for improvement. The work plan to achieve this objective consisted of a number of tasks. The specific objectives of these tasks are as follows:

(a) To conduct a nationwide survey to document the procedures used by other state DOTs to ensure adequate skid resistance on bituminous pavements.

(b) To select 50-60 test pavement sections that represent all major aggregate types, mix designs and climatic regions within the state of Texas and monitor the skid resistance of these pavements throughout the study duration. To identify a few test sections in each climatic region for more frequent monitoring so that the significance of the seasonal variations in skid numbers can be estimated.

(c) To evaluate the frictional properties of the coarse aggregates used in the construction of the selected test pavement sections through appropriate laboratory procedures.

(d) To perform necessary statistical data analysis and use the results obtained to make conclusions with regard to the following. (a) relative significance of *microtexture* versus *macrotexture*, (b) laboratory test procedures that provide better correlation with aggregate field performance, (c) significance of seasonal factors in terms of the variability of skid number measurements.

1.5 ORGANIZATION OF THE REPORT

The subsequent chapters of this report are organized in the following manner. Chapter 2 provides a detailed description of the survey of other state DOT practices to control skid resistance of hot mix asphalt concrete pavements. The chapter summarizes the procedures that were used in collecting and documenting information. It also presents the findings from the survey with respect to alternative laboratory test methods for evaluating frictional properties of pavement aggregates as well as procedures that are used for incorporating the available field performance data. Appendix A provides more detailed descriptions of the procedures used by each state DOT. Chapter 3 presents the research methodology. This chapter describes the laboratory test methods used as well as the field testing procedures and documents the data collected. Chapter 4 deals with the topic of aggregate source rating based on laboratory test data. It provides a detailed description of the analyses performed and presents regressions models that may be used for rating aggregate sources based on laboratory data. Chapter 5 presents a detailed discussion on aggregate source rating based on their historical skid performance. It provides a critical review of the current TxDOT procedure and presents an alternative approach that may be used for this purpose. The final chapter, Chapter 6 presents the conclusions reached from this research and the recommendations for implementation

CHAPTER 2

SURVEY OF STATE DOT PRACTICES TO CONTROL SKID RESISTANCE

2.1 OBJECTIVE OF THE SURVEY

Task 2 of the research project consisted of a nationwide survey to identify the current specifications and guidelines used by different state departments of transportation to ensure adequate skid resistance of Hot Mix Asphalt Concrete (HMAC) pavements. The information collected from this survey was subsequently used to improve the research plan of the research study.

2.2 SURVEY PROCEDURE

At the outset, a representative (or representatives) from each State DOT who has direct involvement in the state's skid control program was identified. This individual was then contacted by phone and interviewed to collect the essential information related to skid control practices used by that agency. Secondly, a questionnaire was developed based on the responses received during the above preliminary telephone survey, and sent to each of the contact persons to collect more detailed and specific information. Additionally, the survey participants were encouraged to attach any additional material which described their skid control strategies more completely.

The overall response rate to the questionnaire was 74%. Among those state DOTs which responded to the questionnaire, many included additional information such as design specifications and guidelines, laboratory test procedures, research reports etc. In those cases where a response to the questionnaire was not received or when the information provided was not complete, an attempt was made to collect the necessary information through a second round of telephone calls. In some cases, no information could be collected using the procedures described above. In these instances, the necessary information was obtained from published data.

A table consisting of the names of contact persons in each state DOT and their addresses and phone numbers is presented in Appendix A. The questionnaire used in the survey for data collection is given in Appendix B.

2.3 A GENERAL OVERVIEW OF SURVEY RESPONSES

The primary focus in the above survey was on HMAC pavement design procedures to achieve good skid resistance. However, other related issues such as methods and equipment used in the measurement of skid resistance in the field, threshold values used in the determination of acceptable level of skid performance etc. were also addressed in this survey. The following is a general overview of the findings from the survey. More specific information on HMAC design methods to achieve good skid resistance will be discussed in subsequent sections.

From the findings of this survey, it is evident that the locked-wheel skid test procedure performed according to ASTM E 274 is the most widely used method for field skid resistance measurement. Consequently, the lower limits of acceptable skid performance of pavements are defined in terms of skid numbers measured according to ASTM E 274. Generally, a skid number of 30 and above in low traffic volume roads and skid numbers of 35-38 and above in high traffic volume roads are considered acceptable by most of the DOTs. However, some states such as Maryland, Minnesota, and Pennsylvania use cutoff values as high as 40. Pavement sections with skid numbers higher than these values are considered safe and hence no action is taken, except for routine skid measurements which are performed for inventory purposes. When the skid numbers are between 31 and 34 the pavement sections are kept under surveillance and certain precautionary measures are taken. These include measures such as more frequent testing, posting of warning signs to inform traffic of potential skid problems etc. Most DOTs consider skid numbers below 30 as unacceptable. At this level, corrective and remedial actions are taken since pavement sections exhibiting such low skid numbers have potential for skid related accidents.

A comparison of the various state DOT design procedures for HMAC pavements reveal that the emphasis placed on the skid resistance aspects varies considerably from one state agency to another. Some of these design procedures do not include any guidelines that specifically address pavement skid resistance. In fact, 21 out of 48 states on which data was collected during the survey belonged to this category. At the same time, however, other state DOTs use very elaborate procedures for achieving satisfactory pavement skid resistance.

Secondly, it was observed that among those state DOTs which address skid resistance in their design, there were significant differences in the actual design procedures used. Interestingly, however, the underlying design philosophy for the majority of these procedures is the same. In other words, nearly all of these procedures are based on the concept that satisfactory field skid resistance may be obtained by controlling the quality of the aggregates used in the construction of the surface mix. Accordingly, these design guidelines are aimed at the identification of aggregates with appropriate polishing characteristics. The differences lie in the exact procedure used in screening the aggregates. These methods are discussed in detail in a subsequent section under the heading State DOT Design Practices to Control Pavement Skid Resistance.

Many research studies have identified surface macrotexture as a major contributory factor controlling skid resistance of the pavement. However, findings of this survey indicate that most state DOT design procedures do not have the capability to account for the differences in pavement macrotexture. One exception to this is the design procedure that was developed by the Wisconsin DOT. This design procedure relies on a statistical regression model to predict the friction number of pavement surfaces after it has been subjected to a given number of vehicle passes. The parameters used in this model include: target lane accumulated vehicle passes (LAVP), percent dolomite or limestone in coarse aggregate, percent wear in LA Abrasion test, age of pavement in years, correction factor to account for the type of aggregate used and a "texture related factor." The texture related factor is calculated based on the LAVP on the proposed roadway based on previously established statistical correlation instead of directly measuring the pavement macrotexture.

2.4 STATE DOT DESIGN PRACTICES TO CONTROL PAVEMENT SKID RESISTANCE

As explained in the preceding section, the design guidelines developed by almost all of the state highway agencies are aimed at the proper identification of good quality coarse aggregates for the surface mix. The exact procedures for doing so varies from one state to another. Hence, it would not be practical to discuss each of these procedures individually. Therefore, for the purpose of this discussion, the design procedures reviewed are broadly classified into five different categories. Table 2.1 below identifies the five categories of state DOT skid control practices.

Table 2.1 Description of DOT Categories

Category	Description
I	No specific guidelines to address skid resistance
II	Skid resistance is accounted for through mix design
III	General aggregate classification procedures are used
IV	Evaluate aggregate frictional properties using laboratory test procedures
V	Incorporates field performance in aggregate qualification

Table 2.2 categorizes the skid control practices used in each state according to the guidelines given above. As can be seen in Figure 2.1 this categorization has little relation to the geographical location of the states. The following section contains a brief description of each category and the rationale behind the categorization.

2.4.1 Category I

DOTs that do not consider friction of surface courses in the design of new pavements have been placed in this category. The past experience of these highway agencies indicate that no prior qualification of aggregates is necessary and as such, no special procedure is followed to ascertain that the frictional characteristics of the aggregates used are satisfactory. Frequent field skid testing of pavement surfaces is performed to ensure that adequate resistance to skid is maintained. Whenever the pavement surface exhibits skid numbers less than the acceptable levels, steps are taken to improve the skid resistance of that pavement surface.

The primary reason cited for such a skid control policy is the availability of good quality aggregates in these states. The experience of these DOTs has shown that aggregates used in HMAC surfaces have been performing quite satisfactorily in providing adequate

Table 2. 2 Identification of State DOT's with Respect to their Categories

State DOT	Category I	Category II	Category III	Category IV	Category V
Alabama				X	
Arizona					
Arkansas			X		
California	X				
Colorado					
Connecticut	X				
Delaware					
Florida				X	X
Georgia			X		
Idaho	X				
Illinois			X		
Indiana				X	
Iowa				X	
Kansas					
Kentucky				X	X
Louisiana				X	
Maine		X			
Maryland	X				
Massachusetts	X				
Michigan				X	
Minnesota				X	
Mississippi				X	
Missouri	X				
Montana		X			
Nebraska		X			
Nevada	X				
New Hampshire		X			

Table 2. 2 (continued): Identification of State DOT's with Respect to their Categories

State DOT	Category I	Category II	Category III	Category IV	Category V
New Jersey				X	X
New Mexico	X				
New Mexico	X				
New York				X	X
North Carolina	X				
North Dakota	X				
Ohio	X				
Oklahoma				X	
Oregon		X			
Pennsylvania				X	X
Rhode Island	X				
South Carolina				X	
South Dakota		X			
Tennessee				X	
Texas				X	X
Utah				X	
Vermont		X			
Virginia			X		
Washington		X			
West Virginia			X		
Wisconsin					X
Wyoming	X				

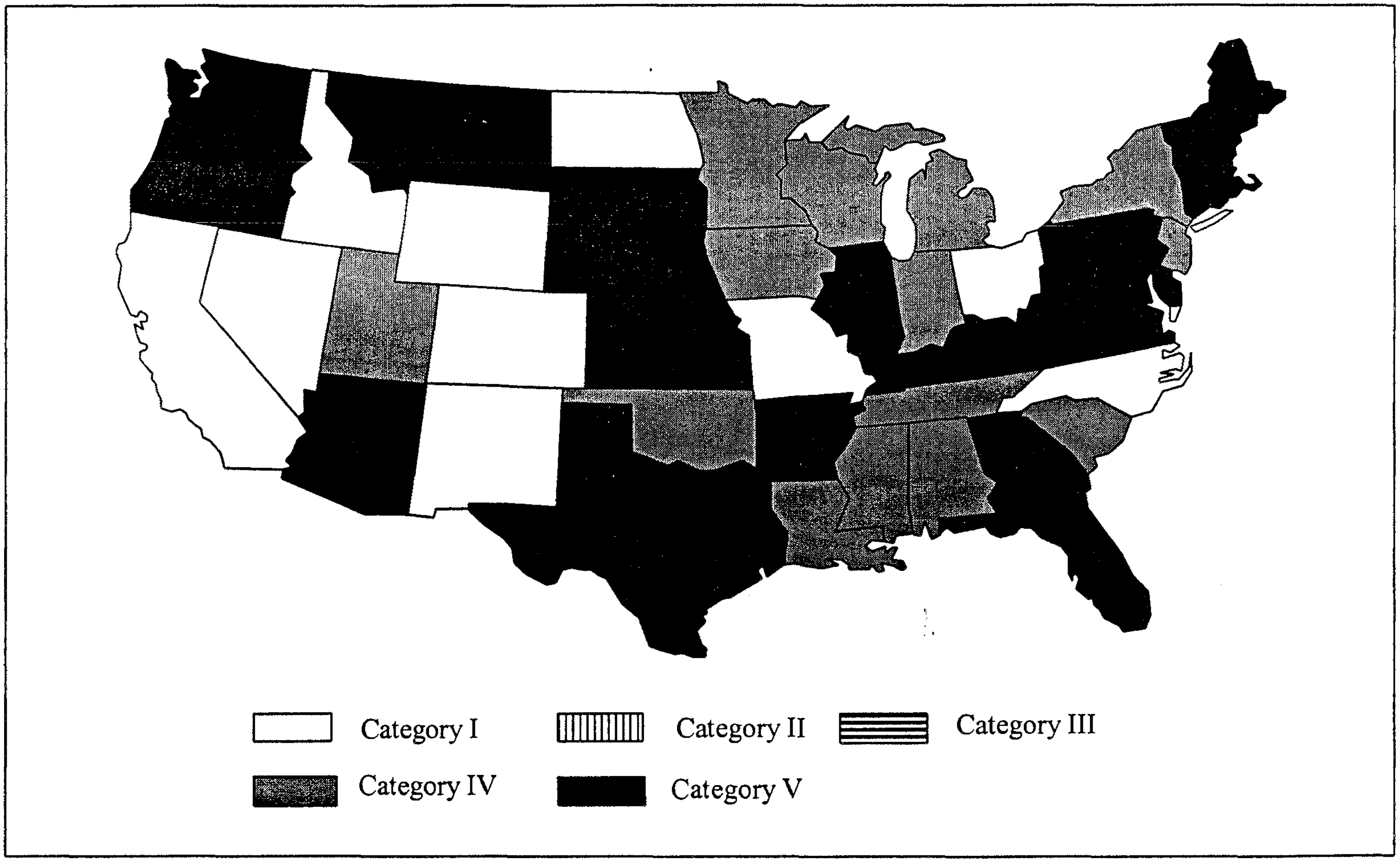


Figure 2.1. Different Categories of State DOTs

resistance to skid. DOTs that do not use any specific guidelines to control skid resistance in the design of new pavements are shown in Figure 2.1.

2.4.2 Category II

The state DOTs that were classified as category II also do not use any procedure to evaluate their aggregates with respect to frictional properties. Instead, these DOTs have based their skid control policies on the concept that adequate skid resistance characteristics of a HMAC pavement surface may be achieved through proper mix design. Once again, the experience in these states has shown that they do not have any major problems related to pavement skid resistance. Problems, if any, have been attributed to shortcomings in the mix design. Frequent field skid testing is done to ensure that sufficient resistance to skid on pavement surfaces is maintained.

2.4.3 Category III

DOTs included in this category consider friction of surface courses in the design of new pavements. Sufficient resistance to skid is obtained by controlling the quality of aggregate used in the construction of pavement surface courses. Quality of the aggregates used is controlled by specifying the type and the allowable percentages of a particular type of aggregate in the HMAC mix. Limestone aggregates have been generally considered to be more susceptible to polishing under traffic action and hence have been identified as poor quality aggregates. The specifications of DOTs in category III, limit the percentage of limestones in surface courses. These specifications have been developed based on the past experience of the individual DOTs. However, field skid testing is performed to ensure that pavement surfaces maintain satisfactory skid resistance.

2.4.4 Category IV

DOTs that use laboratory test procedures to evaluate aggregates with respect to their frictional properties are classified in category IV. Among the laboratory procedures used, Acid Insoluble Residue (AIR) Test and Polish Value (PV) Test are the two most widely used procedures. These laboratory tests are conducted in accordance with ASTM D 3042-86 and

ASTM D 3319-90 respectively. However, based on experience, some of the state DOTs have modified the ASTM test procedures.

The AIR Test estimates the percentage of non-carbonate insoluble residue in carbonate aggregates using dilute hydrochloric acid. The acid reacts and dissolves the carbonate portion in an aggregate and chemically separates the non-carbonates from the carbonates. The aim of the AIR Test is to establish a relationship between the physical properties determined by this test and the frictional properties exhibited by carbonate aggregates. The results of the AIR Test aid in identifying and eliminating carbonate aggregates which are prone to excessive polishing.

The PV Test involves subjecting the aggregate samples to 9 hours of accelerated polishing using a British Wheel in accordance with ASTM D 3319-90 procedure. At the end of 9 hours, the Polish Value of the aggregate is determined using a British Pendulum Tester (BPT) in accordance with ASTM E 303-83. The philosophy behind this test is that the PV of an aggregate measured after 9 hours of accelerated polishing is a quantitative representation of the terminal frictional characteristics of the aggregate when used in a pavement surface course.

Petrographic analysis is performed in accordance with ASTM C 295-90 to identify the mineral constituents present in aggregates used for surface courses. Petrographic analysis is a qualitative aggregate evaluation procedure that is more effective when used in conjunction with other frictional property tests. The use of petrographic analysis in routine designs is limited because of the fact that it requires personnel with specialized training and involves the use of sophisticated equipment. The primary objective of this procedure is to recognize the mineral constituents of an aggregate sample, the properties of which may be expected to influence the overall behavior of the aggregate.

Another test procedures which has been used in the evaluation of aggregate frictional characteristics involves the use of Moh's Hardness Test. Aggregates with higher hardness numbers on a scale of 0 to 10, are considered to have better potential to provide good frictional performance in the field. Based on the Moh's Hardness Number, aggregates are recommended for use in either high traffic or low traffic volume roads.

Mississippi and South Carolina evaluate aggregates by determining the number of freshly fractured faces in mechanically crushed aggregates. More often compliance is based on subjectively separating particles into crushed and uncrushed categories by visual inspection. Personal judgment plays an important role in determining the number of fractured faces. However, the procedure of counting the number of fractured faces can be used in conjunction with other test procedures to obtain an overall assessment of aggregate frictional characteristics.

Indiana DOT adopted the determination of elemental Magnesium content for the acceptance of dolomite aggregates. Dolomite aggregates are specified for use in certain medium to high traffic conditions to obtain high friction skid-resistant bituminous surface courses.

Michigan DOT has a unique aggregate evaluation procedure in which an Aggregate Wear Index (AWI) is assigned to the aggregate based on results from Rapid Petrographic Analysis (MTM 112-94) and Wear Track Tests (MTM 111-94). All aggregates are rated for polishing resistance and each aggregate sample is assigned an AWI. Only those aggregate and aggregate blends that meet the AWI requirements are permitted for use in surface courses.

In general, DOTs included in this category (Category IV), rate aggregate sources based on the evaluation of aggregate frictional properties in the laboratory. Only those aggregates performing satisfactorily are recommended for use in construction of HMAC surfaces. In addition to the specifications for frictional properties, aggregates may be required to meet other specifications based on their performance in tests such as LA Abrasion, Magnesium Sulfate Soundness test etc.

A summary of laboratory test procedures used by different state highway agencies for the evaluation of frictional properties of aggregates is provided in Table 2.3.

2.4.5 Category V

One of the major shortcomings in the use of laboratory test procedures for the evaluation of aggregate frictional performance is the poor correlation between an aggregate's laboratory performance and its field performance. DOTs in Category V have attempted to

Table 2.3 Laboratory Aggregate Evaluation Procedures Adopted by State DOTs

State DOT's	Polish Value Test	Acid Insoluble Residue	Petrographic Analysis	Moh's Hardness Number	No. of Fractured Faces	Other Test Methods
Alabama	X					
Florida		X				
Indiana						Elemental Mg Content
Iowa				X		
Kentucky		X				
Louisiana	X					
Michigan		X	X			Aggregate Wear Index
Minneasota		X				
Mississippi			X		X	
New Jersey	X					
New York		X				
Oklahoma		X				
Pennsylvania	X	X	X			
South Carolina					X	
Tennessee	X	X				
Texas	X					
Utah	X					
Wisconsin						Regression Model

overcome this problem by incorporating field skid performance in their aggregate qualification procedures. However, none of the state DOTs contacted use field skid performance of the aggregate as the sole basis for its qualification. Instead, these DOTs use field performance results in conjunction with laboratory test or as an alternative means of aggregate qualification. DOTs belonging to category V are briefly discussed in Appendix C.

Florida DOT: In addition to the AIR Test results, Florida uses field skid testing as a means for evaluating candidate aggregates for use in pavement surface courses (*5*). After a candidate aggregate meets the requirements of AIR Test, a trial pavement section is constructed using the candidate aggregate. Friction characteristics of the trial section are determined using the Locked Wheel Trailer Methods in accordance with ASTM E 274 Test procedure. No minimum traffic volume is required for approval of the trial section. If the test results are found to be satisfactory, then a test section that has a minimum speed limit of 80 kmph(50mph) and a minimum Average Daily Traffic (ADT) of 14,000 is constructed using the candidate aggregate. At the same time a control section meeting the test section criteria and adjoining the test section is constructed with an already approved aggregate. Friction tests are performed on the test section immediately after construction, then monthly for two months and thereafter at intervals of two months until the accumulated traffic reaches six million vehicles and/or until the skid numbers stabilize. Frictional tests are conducted on the test and control sections at a speed of 64 kmph (40mph) in accordance with ASTM E 274 using both Ribbed and Blank test tires. If it is found to be necessary, additional testing is done at a speed of 96 kmph (60mph). Candidate aggregates are approved for use in highway projects only if the test section exhibits friction numbers above 30 and compares favorably with the control section.

Kentucky DOT: After a candidate aggregate meets the requirement of AIR specifications, a pavement test section is constructed using the candidate aggregate. The test section is required to have an ADT between 6,000 and 15,000. Field skid testing is performed on the test section once a year during the fall season and is continued until the test section has accumulated 6-10 million vehicle passes. If the results of the field skid tests are found to be satisfactory, then the candidate aggregate is given the final approval.

Pennsylvania DOT: Petrographic Analysis is the primary mechanism of the Penn DOT aggregate evaluation procedure. Results of AIR and PV tests are used to supplement the data obtained from petrographic analysis. Based on petrographic analysis a Skid Resistance Level (SRL) is assigned to a candidate aggregate (*6*). SRL classifies aggregates into five types relating them to the ADT of the pavement sections where they can be used. However, the SRL system was adopted in 1975 and is based on linear regression models relating the skid numbers at 64 kmph (40mph) to the corresponding ADT values. If the petrographic examination of aggregate from a new source shows a close similarity to an existing source, it is given the same SRL rating. When an aggregate type not previously rated is submitted, results of the polish value test are considered along with petrographic examination. The results are then compared with similar information from previously rated sources and a tentative SRL is assigned to the aggregate. However, Penn DOT also makes use of past performance history for the final approval or upgrading an aggregate source. Field skid tests are performed at 64 kmph (40 mph) on test sections in accordance with ASTM E 274 using a ribbed tire. A smooth tire is used at times for comparison purposes. Performance history of aggregates from at least 10 projects and spanning over 2 years is used for this purpose. The SRL rating is changed based on the results from the performance history and the aggregate is recommended for use in highway projects.

Texas DOT: TxDOT field performance approach involves the development of skid performance history for a given aggregate source. A detailed description of the skid performance history approach used by TxDOT was provided in Section 1.2.2. The linear relationship of logo (SN40) and logo (VPPL), as shown in Figures 1.1(b) and 1.2(b), represents the historical skid performance of the aggregate source. A candidate aggregate is selected for use in a pavement surface to provide a target skid number throughout the design life of the pavement at the corresponding ADT. The target skid number is used to determine the VPPL from the logarithmic plot as described in section 1.2.2. The VPPL obtained from the plot divided by the ADT of the roadway gives the life of the pavement surface during which the skid numbers can be expected to be maintained above the target skid values. If the life of the pavement as determined above, exceeds the design life of the pavement, then the aggregate is approved for construction. The use of past field performance history has met with good success in several TxDOT districts. This method has been used to approve aggregates from sources which have failed to meet the requirements of the Polish Value test, but have provided adequate performance in the field.

2.4 CONCLUSIONS OF THE SURVEY ON DOT PRACTICES

The results of a nationwide survey on state highway agencies showed that there is considerable difference in the emphasis placed on skid resistance aspects in the design methods used by different state highway agencies. It was interesting to note that a large number of state DOTs do not have any guidelines in their design procedures which specifically address skid resistance of HMAC pavements. The general approach used in these states is to monitor the skid performance of pavements frequently so that any necessary remedial action could be taken based on the data collected. This approach appears to work well for these states because they have not experienced any particular problems with regard to the skid performance of their pavements. Other states which consider skid resistance in their design procedures, place the emphasis on controlling the quality of the coarse aggregate used in the construction of the pavement surface course. The underlying assumption in this approach is that the pavement skid resistance is primarily controlled by the polishing characteristics of the aggregates used in the surface mix. The procedures used in the qualification of aggregates, however, vary significantly among different states. Some procedures are simply based on general classification of aggregates whereas others involve detailed laboratory evaluation. Polish Value Test, Acid Insoluble Residue Test and Petrographic Analysis were found to be the most commonly used laboratory test methods. In addition to the laboratory testing, some states such as Florida, Kentucky, Pennsylvania and Texas use procedures that incorporate the field skid performance of the aggregates. However, none of the state DOTs rely on field skid testing as their primary mechanism for aggregate evaluation. From a technical point of view, it can be argued that aggregate qualification based on their field skid performance would be preferable to aggregate evaluation based on laboratory performance. However, there are a number of practical difficulties involved in the implementation of such field qualification procedures. First of all, it is necessary to collect field skid data over several years to develop an adequate field skid performance history for a given aggregate source. Therefore, obviously this approach cannot be used with new or recent aggregate sources. Secondly, the quality of the aggregates obtained from a given source can vary considerably over the period of time in which the data is collected. Therefore the historical performance may not necessarily represent the aggregate which is currently being produced. A second major difficulty arises due to the large variability associated with field skid measurements. It is known that the field skid numbers, even when performed according to ASTM E 274 standards, are sensitive to extraneous factors such as: seasonal effects (rainfall and temperature), variations in test speed, test position (both longitudinally and laterally), distresses (eg. flushing) on the road surface and the operator. High variability in the skid measurements affect the reliability of the field performance history, and hence the reliability of the aggregate qualification procedure. Finally, it must be also noted that field performance approach has a major disadvantage because of the greater demands it places on manpower, field test equipment, time and money.

Finally, it is worthwhile noting that the vast majority of the state DOT design procedures focus on controlling aggregate quality and hence pavement microtexture. However, the findings from a number of research studies demonstrate that other factors, specially the pavement macrotexture plays a significant role in determining pavement skid behavior. Ideally, a design procedure should be capable of accounting for the differences in both micro- and macro- texture components. However, no such procedure was identified during this survey. Once again, the primary reasons for this is the difficulty in implementation. A number of field devices have been developed for the purpose of measurement of pavement macrotexture. Some of these such as sand patch method, silly putty method and volumetric methods are cumbersome to use in routine testing. However, some of the more recent developments such as Mini-Texture-Meter developed by British Transport and Road Research Laboratory *(7),* Selcom Laser System developed by researchers at University of Texas at Arlington (*8*) and the noncontact high speed optical scanning technique developed by the researchers at Pennsylvania State

University (*9*), show good promise in being able to make accurate and efficient measurement of pavement macrotexture. The first two of these devices use a laser beam to scan the pavement surface and hence estimate pavement texture depth. The third device makes use of a strobed band of light with high infrared content to generate shadowgraphs. It has the capability to collect data from a vehicle moving at normal highway speeds. With the availability of these equipment which make use of most up-to-date technology, the development of a comprehensive design methodology which gives due consideration to both micro- and macro- texture components in skid resistance may be within our reach.

CHAPTER 3
RESEARCH METHODOLOGY

3.1. OVERVIEW

As mentioned in chapter 1, this research study involved two separate test programs: first, a detailed laboratory test program to characterize the selected aggregate sources and second, a field test program to determine the performance of these aggregates on in-service pavements.

Findings of the survey of other state Departments of Transportation (DOTs) indicated that several standardized laboratory test methods are used to evaluate frictional characteristics of pavement aggregates. Among these, the most commonly used laboratory test methods are:

(a) Polish Value Test (PVT)

(b) Acid Insoluble Residue Test (AIRT) and

(c) Petrography Analysis (PA).

Aggregate performance in other physical property tests, such as LA Abrasion Test (LAAT) and Magnesium Sulfate Soundness Test (MSST) are also considered in determining the suitability of aggregates for use in pavement surface courses. Hence, the laboratory test program in this research involved the evaluation of aggregate performance in the above mentioned laboratory test procedures.

The field test program involved monitoring of 54 selected pavement test sections over the entire three years of study duration. Field skid tests were conducted on the pavement surface once every year during the test program. A mini-texture-meter was used to record the pavement surface macrotexture at the time of conducting the field skid tests. British Pendulum Numbers (BPN) were measured at a minimum of three different points along the length of the pavement test section using a British Pendulum Tester (BPT). In order to study the effect of seasonal variations on field skid numbers, 6 pavement test sections were identified. These six pavement test sections were monitored at more frequent intervals to record the variations in skid numbers. The following sections describe the laboratory and field test programs in greater detail.

3.2 LABORATORY TEST PROGRAM

The laboratory tests performed on coarse aggregate were divided into two categories. Category I consists of AIRT and PA, which were performed at the Department of Geosciences at Texas Tech University. It must be noted that in this research, the AIR test was conducted according to the ASTM procedure and not according to TxDOT procedure. The TxDOT method for AIRT is listed as applicable to fine aggregates and it is performed on a small aggregate sample (100g). The ASTM procedure for AIRT is performed in triplicate and on a larger (500g) aggregate sample. Hence, it was decided to perform the AIRT according to the more rigorous ASTM test procedure. Petrography Analysis of coarse aggregate samples was also performed according to the ASTM procedure.

The second category of laboratory tests consisted of LAAT, MSST, and PVT tests performed on the coarse aggregate by TxDOT. Since, TxDOT performs the Category II tests routinely as a part of their aggregate quality monitoring program, the necessary test data was obtained from TxDOT records. Table 3.1 shows the laboratory tests and the corresponding ASTM and TxDOT test specifications.

Table 3.1 Laboratory Tests Performed on Coarse Aggregate

Category	Laboratory Test	ASTM Method	TxDOT Method
Category I	AIRT	D-3042	Tex-612-J
	PA	C-295	-
Category II	LAAT	C-131	Tex-410-A
	MSST	C-88	Tex-411-A
	PVT	D-3319 and E-303	Tex-438-A

3.2.1 Recovery of Aggregate Samples

In order to conduct the group one tests, it was decided to obtain samples of aggregates from the same aggregate sources which had supplied aggregate for construction of selected pavement test sections. However, it was realized that the properties of aggregate samples obtained from these sources may be quite different from the properties of aggregates used in construction of the test sections. Properties of the aggregate samples change with time as new and different layers of rock are excavated at the aggregate source. The geological formation of these layers of rock might be different leading to the excavation of aggregates with different properties over a period of time. Hence, it was decided to core the pavement test section to obtain the required aggregate samples.

Pavements were cored at five different points along the length of the test section using a coring rig supplied by TxDOT. A cylindrical core was extruded from the pavement and using a circular rotating saw, the top layer was separated from the rest of the core. The separated pieces of the top layer were then shipped to the Materials and Test Division of TxDOT, where the asphalt binder was chemically separated to recover the aggregate particles. These recovered aggregate particles were used to conduct tests belonging to catgeory I at Texas Tech University. Figures 3.1 and 3.2 show the different stages of obtaining the aggregate samples from the pavement test section.

3.2.2 Laboratory Tests on Aggregates Performed at Texas Tech

a) Acid Insoluble Residue Test

AIRT provides an estimate of the non-carbonate insoluble material in an aggregate. The procedure involves dissolving a known weight of aggregate sample in dilute Hydrochloric (HCL) acid. The reaction is continued until the carbonate portion present in the aggregate dissolves completely, leaving behind the non-reactive, non-carbonate portion in the form of a residue. The insouluble residue typically consists of hard minerals such as quartz, feldspar, iron oxides etc. After neutralizing the non-carbonate portion, its total weight is used to determine the Acid Insoluble Residue (AIR) as a percentage of the weight of aggregate used for the test. The concept used here is that the frictional properties of aggregates is dependent upon the differential hardness of the aggregate mineral constituents. The aim of AIRT is to determine a relationship between the physical properties determined by this test and frictional properties exhibited by the aggregates. The results of AIR tests performed on aggregates from different test sections are shown in table 3.2. Each pavement test section is identified using a section_id number, the details of which are provided later in this chapter.

Figure 3.1 Coring of Pavement Test Section for Aggregate Extraction

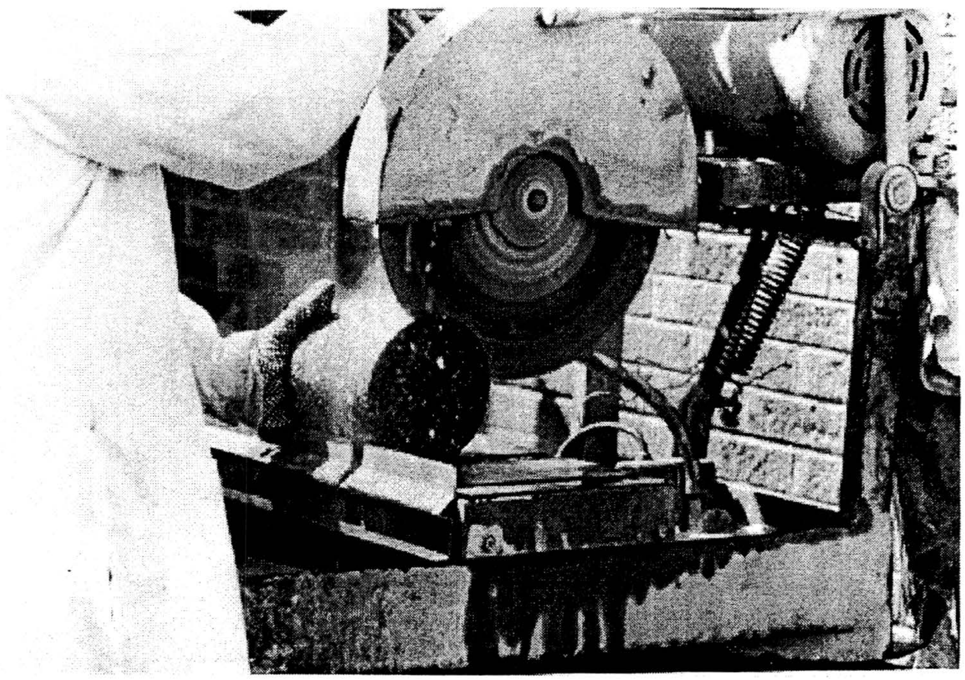


Figure 3.2 Separation of Pavement Surface Course from the Core

Table 3.2 Laboratory test results and Rated Source values for aggregates used in construction of pavement test sections.

	Rated Source Values			Laboratory Test Results			
Sec_ID	RSPV	RSSM	RSLA	PV	MSS	LAA	AIR
04IH00401	31	12	32	32	9	31	96.5
04SH01361	35	10	31	34	12	31	90.4
04SH01521	32	10	30	33	15	35	91.7
05FM22551	36	11	15	40	2	13	97.2
05US00841	31	10	27	23	13.4	23	83.4
06IH00201	37	15	20	42	10	20	96.1
06LP02501	40	25	26	42	30	27	16.4
08IH00201	33	41	32	37	32	33	2.6
08IH00202	33	41	32	37	32	33	2
08IH00203	33	32	28	34	20	27	3.51
10IH00201	25	6	29	34	3	28	97.8
10LP03231	37	15	30	36	11	29	60.1
10US00691	33	39	33	31	25	32	15.1
10US00791	30	25	29	30	11	29	2.2
11FM12751	49	2	19	51	3.9	19	79.8
11US00591	30	17	30	34	24	29	3.8
11US00592	.	.	.	36	5.9	18	90.6
11US00593	37	12	27	36	11	26	54.1

Table 3.2 (Continued): Laboratory test results and Rated Source values for aggregates used in construction of pavement test sections.

	Rated Source Values			Laboratory Test Results			
Sec_ID	RSPV	RSSM	RSLA	PV	MSS	LAA	AIR
11US00594	49	2	19	51	1	19	99.1
12FM13011	32	8	31	33	11	31	3.4
12FM20041	36	29	35	39	20	35	4.5
12FM30051	44	37	29	.	.	.	39.5
12LP01971	35	13	33	38	14	34	5.8
12SH00361	32	20	34	33	16	33	2.7
14US02901	.	.	.	.	.	.	20.3
14US02902	44	18	26	43	12	24	17.8
15LP00131	31	6	29	31	7	29	8.5
15LP00132	32	27	34	34	14	26	9.6
15LP16041	26	1	20	27	1.6	19	2.8
15US02811	33	6	10	33	3.2	9	19.9
15US02812	33	9.9	31	35	18	34	88.3
16SH03591	28	2	17	26	1	15	80.6
16US01811	32	13	33	.	.	.	80.1
16US01812	32	20	34	36	11	32	4.2
16US02811	26	2	17	29	1	15	96.9
18IH00451	41	11	31	43	5	27	19.5

Table 3.2 (Continued): Laboratory test results and Rated Source values for aggregates used in construction of pavement test sections.

	Rated Source Values			Laboratory Test Results			
Sec_ID	RSPV	RSSM	RSLA	PV	MSS	LAA	AIR
18IH035E1	49	3	25	49	4	25	11.3
18US01751	39	2	21	39	2	25	94.3
19SH00081	34	8	22	34	5	22	99.4
19US00591	34	8	22	34	5	22	99.2
19US02711	36	4	24	36	11	20	100
19US02712	37	11	28	40	3	28	99.6
20FM01051	35	29	35	37	7	32	3.8
20FM03651	35	22	35	38	14	34	5.5
20SH00871	35	29	35	38	14	34	4.7
20SH03211	32	20	34	33	16	33	2.1
20US00901	36	29	35	36	16	34	4.6
21SH00041	28	16	25	35	9	21	87.1
21SH01001	26	6	18	28	8	21	90.8
21SP04871	30	21	25	35	19	19	89.5
21US02811	30	21	25	35	19	19	86.2
24FM06591	29	5	31	33	11	25	15.7
24LP03751	28	3	21	29	13	24	13.7
24SH00201	33	4	22	34	11	30	71.3

b) Petrographic Analysis (PA)

Petrographic analysis is a qualitative aggregate evaluation procedure that is more effective when used in conjunction with other frictional property tests. The primary objective of this procedure is to identify the mineral constituents in an aggregate sample, the properties of which may be expected to influence the overall behavior of the aggregate.

In the current research study, PA was conducted on thin sections of recovered coarse aggregate particles retained over the No. 8 sieve. In the first step of the analysis, aggregate particles of different lithology were identified. Different aggregate lithologies such as, dolostone, limestone, dolomitic limestone, limey dolostone, anhydrites, sandstone, chert, marl, and quartz were considered in this study. In the second step, the different percentages of minerals present in each lithological group were determined. Mineralogical constituents like calcite, dolomite, silica, iron oxides, pyrites, anhydrites, organics, clay and cement binding the grains in aggregate particles were identified. The percentages of different lithologies and minerals present can be used to determine the carbonate and non-carbonate content in a given aggregate sample. In the third step, the various types of aggregate textures, such as grain supported, matrix supported and crystalline, were determined. Lithological and mineralogical results of petrography performed on aggregate samples are provided in Tables 3.3 and 3.4.

3.2.3 Laboratory Tests on Aggregates Performed by TxDOT

The laboratory tests described in this section are performed by TxDOT on a regular basis as a part of its comprehensive aggregate quality monitoring program. In addition, TxDOT uses these laboratory tests results to derive the rated source laboratory test values for each aggregate source. This section briefly describes the tests performed by TxDOT.

a) Polish Value Test (PVT)

Polish Value Test is the primary laboratory test method that TxDOT uses in their present aggregate evaluation procedure. In this test, coarse aggregate particles are embedded

Table 3.3: Lithological Composition of Aggregates Determined from Petrography

SEC-ID	AGGR. TYPE	MIX-DESIGN	PERCENT LITHOLOGICAL GROUPS												Lithological	
			Dolo-Stone	Anhydrite	Sand-stone	Lime-stone	Dolomiti LMST	Limey DLST	Chert	Marl	Quartz	Igneou	Shale	Misc	Carbnate	N. Carbonate
04IH00401	Gravel	CMHB-C	2	0	10	0	0	0	0	0	52	34	2	0	2	98
04SH01361	Gravel	ACP-D	10	0	30	5	0	0	5	0	0	50	0	0	15	85
04SH01521	Gravel	ACP-D	5	0	32	15	0	0	0	0	0	43	5	0	20	80
05FM22551	Gravel	CMHB-C	0	0	0	62	0	0	1	0	0	37	0	0	62	38
05US00841	Gravel	ACP-C	2	0	16	33	0	9	8	0	22	8	2	0	44	56
06IH00201	Gravel	CMHB-F	*	*	*	*	*	*	*	*	*	*	*	*	*	*
06LP02501	Caliche	CMHB-C	0	0	6	94	0	0	0	0	0	0	0	0	94	6
08IH00201	Carbonate	CMHB-C	0	0	0	100	0	0	0	0	0	0	0	0	100	0
08IH00202	Carbonate	ACP-D	0	0	0	100	0	0	0	0	0	0	0	0	100	0
08IH00203	Carbonate	ACP-C	0	0	0	100	0	0	0	0	0	0	0	0	100	0
10IH00201	Carbonate	ACP-C	0	0	0	2	0	0	2	0	0	96	0	0	2	98
10LP03231	Sandstone	SMA-C	0	10	10	74	0	0	1	0	0	0	0	5	74	26
10US00691	Carbonate	ACP-C	3	0	0	95	0	0	0	0	0	0	2	0	98	2
10US00791	Carbonate	ACP-C	0	0	0	100	0	0	0	0	0	0	0	0	100	0
11FM12751	LTWT	OGFC	0	0	0	1	0	0	1	0	0	0	0	98	1	99
11US00591	Carbonate	ACP-D	40	0	0	40	0	20	0	0	0	0	0	0	100	0
11US00592	Gravel	ACP-D	0	0	1	32	3	0	65	0	0	0	0	0	35	66
11US00593	Carbonate	ACP-D	0	0	35	64	0	0	1	0	0	0	0	0	64	36
11US00594	LTWT	OGFC	*	*	*	*	*	*	*	*	*	*	*	*	*	*
12FM13011	Carbonate	CMHB-F	2	10	0	87	0	0	0	0	2	0	0	0	89	12
12FM20041	Carbonate	ACP-D	6	0	4	40	25	15	10	0	0	0	0	0	86	14
12FM30051	Carbonate	ACP-D	4	0	0	34	35	25	2	0	0	0	0	0	98	2
12LP01971	Carbonate	ACP-D	20	0	0	80	0	0	0	0	0	0	0	0	100	0
12SH00361	Carbonate	ACP-D	0	0	2	95	0	0	1	0	0	0	0	2	95	5
14US02901	Cabonate	ACP-C	60	0	0	25	0	0	0	0	0	0	0	15	85	15
14US02902	Carbonate	CMHB-C	63	0	0	27	0	0	0	0	0	0	0	10	90	10
15LP00131	Carbonate	ACP-C	0	0	7	90	0	0	2	0	0	1	0	0	90	10

Table 3.3(Continued...) Lithological Composition of Aggregates Determined from Petrography

SEC-ID	AGGR. TYPE	MIX-DESIGN	PERCENT LITHOLOGICAL GROUPS												Lithological	
			Dolo-Stone	Anhydrite	Sand-stone	Lime-stone	Dolomiti LMST	Limey DLST	Chert	Marl	Quartz	Igneou	Shale	Misc	Carbnate	N. Carbonate
15LP00132	Carbonate	CMHB-F	0	0	2	96	0	0	2	0	0	0	0	0	96	4
15LP16041	Flint Rock	ACP-D	0	0	0	99	1	0	0	0	0	0	0	0	100	0
15US02811	Carbonate	ACP-D	0	0	0	99	0	0	1	0	0	0	0	0	99	1
15US02812	Trap Rock	Novachip	0	0	0	44	0	0	1	0	0	55	0	0	44	56
16SH03591	Gravel	ACP-D	0	0	2	39	0	0	54	3	2	0	0	0	39	61
16US01811	Gravel	ACP-D	1	0	53	24	0	0	17	0	5	0	0	0	25	75
16US01812	Carbonate	ACP-D	0	0	1	98	0	0	1	0	0	0	0	0	98	2
16US02811	Gravel	ACP-D	0	0	0	27	0	0	73	0	0	0	0	0	27	73
18IH00451	Carbonate	ACP-C	50	0	4	5	0	30	2	0	3	1	5	0	85	15
18IH035E1	LTWT-RA	ACP-C	50	0	0	15	0	30	0	0	0	0	0	5	95	5
18US01751	Carbonate	ACP-C	0	0	0	0	0	0	0	0	0	100	0	0	0	100
19SH00081	Gravel	CMHB-C	0	0	40	0	0	0	45	0	15	0	0	0	0	100
19US00591	Gravel	CMHB-F	0	0	17	0	0	0	54	0	0	24	5	0	0	100
19US02711	Sandstone	ACP-C	0	0	100	0	0	0	0	0	0	0	0	0	0	100
19US02712	Sandstone	ACP-C	0	0	100	0	0	0	0	0	0	0	0	0	0	100
20FM01051	Carbonate	ACP-D	0	0	0	60	35	5	0	0	0	0	0	0	100	0
20FM03651	Carbonate	ACP-C	0	0	0	67	21	0	10	0	2	0	0	0	88	12
20SH00871	Carbonate	ACP-C	0	0	0	60	40	0	0	0	0	0	0	0	100	0
20SH03211	Carbonate	ACP-C	0	0	0	98	0	0	2	0	0	0	0	0	98	2
20US00901	Carbonate	ACP-C	20	0	0	50	30	0	0	0	0	0	0	0	100	0
21SH00041	Gravel	ACP-D	2	0	2	50	0	0	45	0	0	1	0	0	52	48
21SH01001	Gravel	ACP-D	0	0	29	25	0	0	41	0	5	0	0	0	25	75
21SP04871	Gravel	ACP-D	1	0	4	6	0	0	82	0	0	7	0	0	7	93
21US02811	Gravel	CMHB-C	1	0	8	43	1	0	46	1	0	0	0	0	45	55
24FM06591	Carbonate	CMHB-C	10	0	2	78	0	5	5	0	0	0	0	0	93	7
24LP03751	Carbonate	ACP-D	34.95	0	0.97	61.16	0	0.97	0	0	0	0	0	1.94	97.1	2.91
24SH00201	Granite-Ca	ACP-D	8	0	4	63	0	0	21	0	0	0	0	2	71	27

Table 3.4 **Mineralogical Composition of Aggregates Determined from Petrography**

SEC-ID	PERCENT MINERALOGICAL COMPOSITION										MINERALOGICAL		
	CAL-CITE	DOLO-MITE	ORGA-NICS	SILICA	QUA-RTZ	FELD-SPARS	PYRI-TES	MISC.	IRON-OXIDES	CLAY	ANHY-DRITES	% Carbonate	% Non Carbonate
04IH00401	0.4	2	0	49	17.2	6.5	0.67	15.38	0.32	8.13	0	2.4	97.2
04SH01361	7.95	9.35	0.25	45.4	25.5	6.3	0	2.6	0	2.65	0	17.3	82.7
04SH01521	15.94	3.25	0	0.6	68.41	3.4	0.05	1.5	0.25	1.85	4.75	19.19	80.81
05FM2255	59.64	0	0.02	0.98	2.36	0	0	36.26	0.74	0	0	59.64	40.36
05US00841	36.72	5.72	0	8.79	40.05	1.8	0.12	0.14	1.06	3.67	0	42.44	55.63
06IH00201	*	*	*	*	*	*	*	*	*	*	*	*	*
06LP02501	75.95	0	0	0	24.05	0	0	0	0	0	0	75.95	24.05
08IH00201	100	0	0	0	0	0	0	0	0	0	0	100	0
08IH00202	100	0	0	0	0	0	0	0	0	0	0	100	0
08IH00203	95.5	0	0	0	4.5	0	0	0	0	0	0	95.5	4.5
10IH00201	2	1.92	0	1.98	0	0	0	94.08	0.02	0	0	3.92	96.08
10LP03231	77.96	6.04	0.01	4.5	9.99	0	0	0	0	1.5	0	84	16
10US00691	94.14	2.1	0	0	0.61	0.8	0.09	1	0.1	1.16	0	96.24	3.76
10US00791	99	0	0.6	0	0	0	0	0	0	0	0	99	0.6
11FM1275	1	0	0	69.6	0	0	0	0	0	29.4	0	1	99
11US00591	71	27	0	2	0	0	0	0	0	0	0	98	2
11US00592	33.45	2.49	0.22	63.03	1.05	0	0.04	0	0	0.72	0	35.94	65.06
11US00593	63.7	1.95	0.35	0.4	31.5	0.7	0	0	0	1.4	0	65.65	34.35
11US00594	1	0	0	69.6	0	0	0	0	0	29.4	0	1	99
12FM1301	93	2	0	0.2	5	0	0	0	0	0.8	0	95	6
12FM2004	55.66	29.91	0.42	10.3	0.84	0.02	0.14	0	0	1.19	0	85.57	12.91
12FM3005	49.61	35.15	0	4.5	9.95	0	0	0	0.19	0.6	0	84.76	15.24
12LP01971	77.6	13.15	0.05	7.4	1.2	0	0	0	0.4	0.2	0	90.75	9.25
12SH00361	96.6	0	0	2.48	0	0	0	0	0	0.4	0	96.6	2.88
14US02901	40	46.95	0	0	0	0	0	0	0	11.8	1.35	86.95	13.05
14US02902	46.7	51.94	0	0	0	0	0	0	0	1.36	0	98.64	1.36
15LP00131	89.8	0	0.07	1.74	3.6	0	0.01	0.9	0.02	3.86	0	89.8	10.2
15LP00132	94.82	0	0	5.15	0	0	0	0	0	0.03	0	94.82	5.18

Table 3.4(Continued) Mineralogical Composition of Aggregates Determined from Petrography

SEC-ID	PERCENT MINERALOGICAL COMPOSITION										MINERALOGICAL		
	CAL-CITE	DOLO-MITE	ORGA-NICS	SILICA	QUA-RTZ	FELD-SPARS	PYRI-TES	MISC.	IRON-OXIDES	CLAY	ANHY-DRITES	% Carbonate	% Non Carbonate
15LP16041	99	0.4	0.01	0.42	0	0	0	0	0	0.01	0	99.4	0.44
15US02811	98.7	0	0	1.3	0	0	0	0	0	0	0	98.7	1.3
15US02812	44.45	0	0	0.55	0	0	0	55.55	0	0	0	44.45	56.1
16SH03591	41.6	1.1	0.2	53.45	3.4	0.01	0.02	0.05	0	0.17	0	42.7	57.3
16US01811	24.34	0.11	0.51	69.08	5.96	0	0	0	0	0	0	24.45	75.55
16US01812	97.8	0	0.01	1	0.2	0	0	0	0	0.69	0	97.8	1.9
16US02811	26.6	0.03	0.59	72.23	0.3	0.15	0	0	0	0.1	0	26.63	73.37
18IH00451	12.22	58	0	4.96	15.4	0	1.16	1.22	0.02	7.02	0	70.22	29.78
18IH035E1	18.5	58	0	5	14.85	0	1.1	0	0	2.55	0	76.5	23.5
18US01751	0	0	0	0	0	20	0	75	5	0	0	0	100
19SH00081	0	0.25	0	58.75	28.9	0	0	0	0	7.1	0	0.25	94.75
19US00591	0	0.7	0	67.18	21.76	0	0	0	0	10.35	0	0.7	99.29
19US02711	0	1.9	0	4.76	76.19	2.86	0	9.52	0	4.76	0	1.9	98.09
19US02712	0	0	1	0	96	0	0	0	3	0	0	0	100
20FM0105	85.25	14.75	0	0	0	0	0	0	0	0	0	100	0
20FM0365	85.85	0.5	0	12	1.5	0	0	0	0	0.15	0	86.35	13.65
20SH00871	80.8	16	0	1.2	2	0	0	0	0	0	0	96.8	3.2
20SH03211	99.49	0	0	0	0.51	0	0	0	0	0	0	99.49	0.51
20US00901	75.6	22.5	0	0	1.9	0	0	0	0	0	0	98.1	1.9
21SH00041	46.17	0.7	1.26	47.76	2.4	0.3	0.55	0.25	0.02	0.21	0	46.87	52.75
21SH01001	26.75	0	0	37.94	30.69	0.9	0	0	1.25	2.37	0	26.75	73.15
21SP04871	10.75	1.15	0.52	77.64	1.83	0.25	0.02	0.02	0.03	0.29	0	11.9	80.6
21US02811	42.07	2.24	0	50.28	3.26	0.01	0.19	0.03	0.02	1.9	0	44.31	55.69
24FM0659	74.45	18.25	0	5.3	1	0	0	0	0	1	0	92.7	7.3
24LP03751	59.27	36.69	0	0.14	1.55	1.94	0	0	0	0.38	0	95.96	4.01
24SH00201	63	8.6	0.2	19.4	3.4	12	0	0	0.9	0.5	0	71.6	36.4

in a resin and are cast in metal coupons. These coupons containing aggregate particles are mounted on a steel wheel 40.6 cm (16 inches) in diameter, forming a continuous coarse aggregate strip 4.5 cm (1.75 inches) wide. A smooth rubber tire is brought in contact with the aggregate strip and the aggregate particles are polished for nine hours. A British Pendulum Tester is then used in accordance with ASTM E-303 to determine the BPN of the polished aggregate surface. The BPN measured after nine hours of accelerated aggregate polishing is believed to represent the terminal frictional characteristics of the aggregate after being subjected to polishing action due to traffic for a long time. Table 3.2 provides the results of polish value tests conducted in the laboratory and the rated source polish values for all aggregate sources considered in this research

b) Magnesium Sulfate Soundness Test (MSST)

MSST is widely used as an index of general aggregate quality, and is intended to estimate the resistance of aggregate to weathering actions that occur in nature. The test involves alternate soaking and drying of the aggregate samples in a magnesium sulfate solution. The alternate soaking and drying process is repeated five times. The magnesium sulfate solution penetrates the aggregate surface. Minute salt crystals grow in small pores present in the aggregate and the pressure from the crystal growth causes disintegration of the aggregates during the alternate soaking and drying process. The degradation of an aggregate sample due to the development of salt crystals is a simulation of the expansion of water upon freezing within the aggregate pores. Table 3.2 provides the results of MSS tests conducted in the laboratory and the rated source soundness values for all aggregate sources considered in this research

c) Los Angeles Abrasion Test (LAAT)

The principle of LAAT is to measure the percentage wear of aggregate degradation resulting from a combination of actions, such as abrasion, crushing, impact, and grinding. The test involves placing a measured quantity of aggregate sample and a specified number of steel spheres in a horizontally rotating steel drum. The steel spheres are used as an abrasive charge. The aggregate and steel spheres roll within the drum with an abrading and grinding action for a prescribed number of revolutions. The aggregate is removed from the drum and then sieved to measure the aggregate degradation as a percent loss. In this research study, results of the above mentioned laboratory tests at the time of pavement construction and their corresponding rated source values were used for data analysis. Table 3.2 provides the results of LAA tests conducted in the laboratory and the rated source LAA values for all aggregate sources considered in this research

3.3 FIELD TEST PROGRAM

3.3.1 Selection Criteria for for Field Test Sections

Initially 60 HMAC test pavement sections were selected on the criteria described in this below. However, testing was actually carried out in fifty-four as some of the candidate sections had not yet been constructed by the time the first cycle of tests were conducted in the summer of 1995. Testing in one of the sections was discontinued after 1995 as it was reconstructed due to a base failure.

The criteria used in the selection of test pavement sections were as follows.

(1) The pavement sections must represent all four of the climatic regions within the State of Texas (see Figure 3.3). The selection of test sections in different climatic zones facilitates comparison of the performance of similar aggregates under different climatic conditions.

(2) The aggregate types which are predominantly used within each region must be represented in the selected test pavement sections. Experience from project 490 suggests that it is not practically feasible to try to achieve balance with respect to aggregate type. Therefore, such a constraint was not imposed. Instead aggregate type was selected so that they were representative of the material that is most commonly used within each climatic zone. Table 3.5 shows the different aggregate types that are commonly used in Texas.

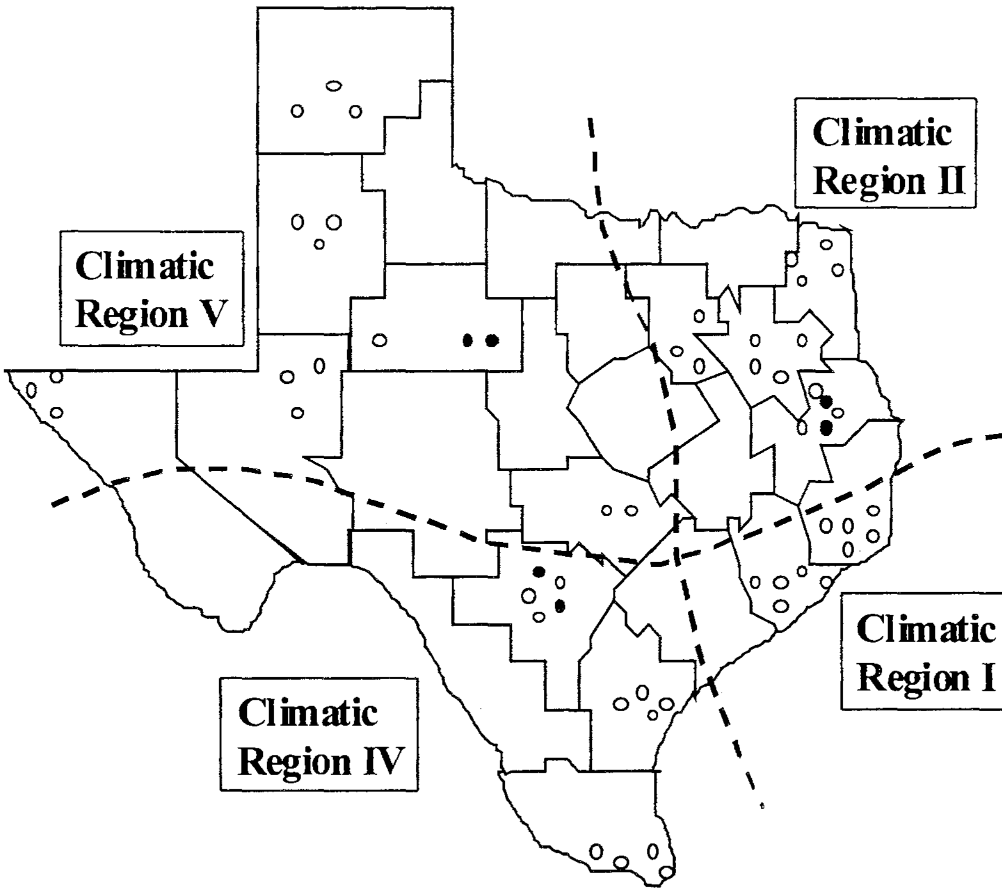


Figure 3.3 Different Climatic Zones of Texas

Table 3.5 Aggregate Types Used in the Study

Aggregate Type	Test Sections
Carbonate	30
Sandstone	3
Siliceous Gravel	16
Other (Igneous, Traprock, Lightweight etc.)	5

(3) Pavement sections must also be selected so that they represent different mix designs. The majority of the pavements commonly used in pavements are Type C and Type D. Therefore 39 of the total 54 sections selected were of these mixes. In addition several other mixes were selected in order to represent mixes with higher macrotextures. A detailed breakdown of the number of pavements in each mix design is shown in Table 3.6

Table 3.6 - Mixed Designs of Pavements Used in Study

Mix Design	Test Sections
OGFC	3
CMHB-Coarse	9
CMHB-Fine	5
Type-C	16
Type-D	22

(4) To ensure that adequate skid performance data was compiled within the 3-year project duration an adequate number of test pavement sections was selected from new construction projects (37 of the selected 54 pavement sections were construction projects completed either in 1994 or 1995.)

(5) The selected pavement sections must be distress free as far as possible so that skid numbers were not skewed by cracks, crack seals or raveling.

(6) They were also selected such that they were straight with no sharp turns and with minimal exit/entry ramps and intersections. This precaution was taken as a safety measure to reduce the probability of an accident during skid testing.

(7) Details of each pavement test section and important attributes that are helpful in identifying its location and aggregate sources are shown in Tables 3.7(a), 3.7(b) and 3.7(c).

3.3.2 Skid Number Measurement at 64 kmph (40 mph)

Skid numbers at 64 kmph were taken at five locations, each spaced about 100 m apart, on each test section. Each test section was approximately 500m long. On sections where there were two or more lanes in one direction the outside lane was used for testing.

Five skid measurements were taken in order to minimize the variability in skid numbers due to non-uniformity in the test surface in the longitudinal direction. Taking sufficient number of skid measurements may minimize possible error due to this type of random variability. The arithmetic mean of a sufficient number of readings can be assumed to be representative of the average condition of the pavement section. The frictional resistance of the pavement surface also varies in the lateral direction across the travel lane. The skid resistance is a minimum along the wheel path and the measured skid numbers tend to vary depending on the lateral position of the test trailer. Technically, all skid measurements should be performed along the centerline of the left wheel path. Therefore, to reduce the effect of non-uniformity in the lateral direction skiding was made on the left wheel path. Figure 3.4 depicts the five locations where a skid measurement was made on a test section. The average skid number measured on each test section and the corresponding accumulated vehicle passes per lane(AVPPL) at the time field skid testing are shown in Table 3.8. This table provides the average field skid numbers measured at 64Kmph during the three years of the research study.

Table 3.7(a) Information on Pavement Test Site Location and Mix Design

Section ID	District	County	Highway	Mix Type
04IH00401	Amarillo	Gray	IH 40E	CMHB-C
04SH01361	Amarillo	Hutchinson	SH136E	ACP-D
04SH01521	Amarillo	Hutchinson	SH152	ACP-D
05FM22551	Lubbock	Lubbock	FM2255W	CMHB-C
05US00841	Lubbock	Lubbock	US 84W	ACP-C
06IH00201	Odessa	Ward	IH 20W	CMHB-F
06LP02501	Odessa	Midland	LP 250	CMHB-C
06US03851	Odessa	Crane	US 385S	CMHB-C
08IH00201	Abilene	Callahan	IH 20E	CMHB-C
08IH00202	Abilene	Callahan	IH 20E	ACP-D
08IH00203	Abilene	Mitchell	IH 20W	ACP-C
10IH00201	Tyler	Gregg	IH 20E	ACP-C
10LP03231	Tyler	Smith	LP 323	SMA-C
10US00691	Tyler	Smith	US 69S	ACP-C
10US00791	Tyler	Rusk	US 79N	ACP-C
11FM12751	Lufkin	Nacogdoches	FM 1275N	OGFC
11US00591	Lufkin	Angelina	US 59S	ACP-D
11US00592	Lufkin	Angelina	US 59S	ACP-D
11US00593	Lufkin	Angelina	US 59S	ACP-D
11US00594	Lufkin	Nacogdoches	US 59S	OGFC
12FM13011	Houston	Bazoria	FM 1301W	CMHB-F

Table 3.7(a)(Continued) Information on Pavement Test Site Location and Mix Design

Section ID	District	County	Highway	Mix Type
12FM20041	Houston	Bazoria	FM 2004N	ACP-D
12FM20041	Houston	Bazoria	FM 2004N	ACP-D
12FM30051	Houston	Galveston	FM 3005E	ACP-D
12LP01971	Houston	Galveston	LP 197N	ACP-D
12SH00361	Houston	Bazoria	SH 36S	ACP-D
14US02901	Austin	Travis	US 290W	ACP-C
14US02902	Ausitn	Travis	US 290W	CMHB-C
15LP00131	San Antonio	Bexar	LP 13S	ACP-C
15LP00132	San Antonio	Bexar	LP 13S	CMHB-F
15LP16041	San Antonio	Bexar	LP 1604W	ACP-D
15US02811	San Antonio	Bexar	US 281N	ACP-D
15US02812	San Antonio	Bexar	US 281N	Novachip
16SH03591	Corpus Christi	San Patricio	SH 359E	ACP-D
16US01811	Corpus Christi	Bee	US 181S	ACP-D
16US01812	Corpus Christi	Bee	US 181N	ACP-D
16US02811	Corpus Christi	Jim Wells	US 281N	ACP-D
18IH00451	Dallas	Ellis	IH 45S	ACP-C
18IH035E1	Dallas	Ellis	IH 35ES	ACP-C
18US01751	Dallas	Kaufman	US175W	ACP-C
19SH00081	Atlanta	Bowie	SH 8N	CMHB-C
19US00591	Atlanta	Bowie	US 59 N	CMHB-F

Table 3.7(a)(Continued) Information on Pavement Test Site Location and Mix Design

Section ID	District	County	Highway	Mix Type
19US02711	Atlanta	Titus	US 271S	ACP-C
19US02712	Atlanta	Titus	US 271N	ACP-C
20FM01051	Beaumont	Jasper	FM 105N	ACP-D
20FM03651	Beaumont	Jefferson	FM 365E	ACP-C
20SH00871	Beaumont	Orange	SH 87S	ACP-C
20SH03211	Beaumont	Liberty	SH 321S	ACP-C
20US00901	Beaumont	Liberty	US 90E	ACP-C
21SH00041	Pharr	Cameron	SH 4E	ACP-D
21SH01001	Pharr	Cameron	SH 100E	ACP-D
21SP04871	Pharr	Hidalgo	SPUR 487	ACP-D
21US02811	Pharr	Hidalgo	US 281N	CMHB-C
24FM06591	El Paso	El Paso	FM 659N	CMHB-C
24LP03751	El Paso	El Paso	LP 375N	ACP-D
24SH00201	El Paso	El Paso	SH 20E	ACP-D

Table 3.7(b) Pavement Test Site Reference Marker Information

Section ID	Reference Makers	Description of Test Site Location
04IH00401	0.45(127)/0.50(126)	East of SH 70
04SH01361	1.20(086)/0.80(084)	East of Fritch
04SH01521	2.60(336)/1.30(334)	Between city of Borger & Carson CL
05FM22551	0.50(314)/1.50(312)	From Loop 289 west to Shallowater
05US00841	0.20(392)/0.20(392)	Just west of Health Science Center
06IH00201	0.10(064)/0.80(063)	Between Reeves CL and 0.5 miles west of Pyote
06LP02501	1.90(278)/1.90(278)	Just west of bridge and east of exit
06US03851	1.10(388)/0.80(390)	Between south of Crane and intersection of US 67 in McCamey
08IH00201	0.70(296)/1.30(298)	2.5 miles east of Taylor CL to west FM-604 in Clyde
08IH00202	0.00(294)/0.00(294)	Just west of section 1
08IH00203	0.10(220)/0.95(219)	W. of Junction 1899 on IH-20W
10IH00201	0.00(590)/1.00(591)	E. of SH 42 & W. of SH 31
10LP03231	N/A	0.2 miles east of US 69 in Tyler E. to 0.2 miles E of SH 110
10US00691	1.00(340)/1.00(342)	Section starts 1.25 miles south of FM 2813 near Pineridge ranch
10US00791	0.00(348)/2.00(346)	Section starts at RM 348 heading N.
11FM12751	0.18(338)/0.18(338)	Section starts right after traffic lights of northbound East College .
11US00591	0.20(380)/0.20(380)	South of Angelina river bridge
11US00592	0.30(384)/0.30(386)	1600 ft. to Mill Creek
11US00593	1.25(388)/1.25(388)	Near Martin School
11US00594	0.75(338)/0.75(338)	South corner of Burrows St.
12FM13011	1.70(670)/2.30(666)	West of SH36

Table 3.7(b)(Continued) Pavement Test Site Reference Marker Information

Section ID	Reference Makers	Description of Test Site Location
12FM20041	0.07(528)/1.93(530)	Between FM 523 & SH 288
12FM30051	1.70(504)/2.30(508)	Between RM 506 & 7 mile Rd.
12LP01971	0.30(502)/1.70(504)	East of I-45 just N. of first culvert
12SH00361	3.60(692)/0.40(688)	Just S. of culvert S. of city of Jones Creek
14US02901	3.50(608)/3.25(608)	Between Manor and Elgin; between Klaus Ln. & Albert Voelker
14US02902	0.55(602)/0.55(602)	Between Albert Voelker rd. & Ballerstedt
15LP00131	1.60(504)/2.20(502)	Between Holmgreen & Seabreeze rd.
15LP00132	0.80(496)/1.20(498)	Between Indian HIlls and S. New Braunfields
15LP16041	0.60(524)/1.40(526)	Section starts at bridge over Salado Creek in north loop
15US02811	1.70(526)/0.20(528)	After Bitters Rd. exit & just north of Salado Creek culvert
15US02812	0.85(520)/1.05(518)	About 5 miles north of Loop 1604; north of Marshall Rd.
16SH03591	1.90(572)/2.00(576)	In city of Mathis
16US01811	1.80(592)/3.80(596)	Between SH202 & Bus 181
16US01812	0.00(582)/2.00(580)	North of city of Beeville
16US02811	1.50(668)/0.40(666)	North of intersection of US281 & FM 2044
18IH00451	0.10(246)/0.90(245)	From Navarro county line to US 287
18IH035E1	0.90(397)/0.10(396)	From US 77 south of Waxahachie to Hill-Ellis county line.
18US01751	1.10(616)/0.90(614)	From Fm 148 East to FM 1390 (westbound lanes)
19SH00081	0.00(212)/2.00(210)	South of US 82 & North of FM 2149
19US00591	0.50(224)/1.50(222)	North of Sulphur River bridge
19US02711	3.10(252)/3.10(252)	Between SH49 & FM 899 - Site B

Table 3.7(b)(Continued) Pavement Test Site Reference Marker Information

Section ID	Reference Makers	Description of Test Site Location
19US02712	0.90(250)/3.10(246)	North of bridge - Site A
20FM01051	0.90(430)/1.10(428	North of Orange-Japser county line
20FM03651	0.35(766)/0.00(766)	Section starts @ 4th light pole after bridge (E)
20SH00871	1.20(488)/0.00(488)	Just south of Bridge city
20SH03211	1.55(452)/0.00(452)	From north end of 4 lane section southward.
20US00901	0.35(886)/0.00(866)	Just east of Raywood section. Starts after bridge
21SH00041	0.85(562)/3.00(566)	North end of Brownsville airport along fence
21SH01001	0.30(732)/3.80(556)	Just east of US 83/77 & SH 100 intersection
21SP04871	0.80(724)/0.80(724)	North of intersection of FM 2220 & US 83
21US02811	2.20(788)/2.20(788)	Just north of USA/Mexico border
24FM06591	0.00(324)/0.00(324)	Between Loop 375 & US 62/180
24LP03751	0.00(032)/0.95(031)	Between US 62/180 & Railroad Dr
24SH00201	1.40(346)/0.50(348)	Between Loop 375 & Clint

Table 3.7(c) Aggregate Source Used in Construction of Pavement Test Section

Section ID	Aggregate Source	Source Code
04IH00401	Gilvin&Terrel Inc.,Roach Pit	N/A
04SH01361	Milligan J. Lee Inc., Coon Pit	11808
04SH01521	E.D. Baker Johnson Pit	11807
05FM22551	Western Rock, Pedernal Pit, Encino,NM	50309
05US00841	Appian Corp., W. Campbell Pit	2517307
06IH00201	Trans Pecos, Hoban Pit	619502
06LP02501	Jones 191 Pit	N/A
06US03851	Jones Dusek Pit	N/A
08IH00201	Vulcan Materials, Black Lease Pit	822107
08IH00202	Vulcan Materials, Black Lease Pit	822107
08IH00203	Price Construction, Clements Pit	708802
10IH00201	Granite Mountain, Sweet Home	50106
10LP03231	Meridian, Apple Pit	50437
10US00691	Meridian Richland Pit, Boorheim	1817504
10US00791	Gifford Hills, New Braunfels Pit	1504603
11FM12751	TXI Streetman, Superrock	1817502
11US00591	Redland Beckman Pit	1501503
11US00592	Gifford Hill, Eagle Mills	50119
11US00593	Boorheim fields, Apple Pit	50437
11US00594	TXI Streetman, Superrock	1817502
12FM13011	Redland, Beckman Pit	1501503

Table 3.7(c)(Continued) Aggregate Source Used in Construction of Pavement Test Section

Section ID	Aggregate Source	Source Code
12FM20041	Luhr Bros', Tower Rock Quarry, MO	50601
12FM30051	Delta Materials, Marble Falls, Sandstone	N/A
12LP01971	Luhr Bros', Tower Rock Quarry, MO	50601
12SH00361	Redland Beckman Pit	1501503
14US02901	N/A	N/A
14US02902	Delta Materials, Brownlee Pit	1402704
15LP00131	Vulcan, Loop 1604	1501506
15LP00132	Vulcan Materials, Helotes Pit	1501514
15LP16041	Capitol S&G, West Pit, Flintrock	01518
15US02811	Vulcan Knippa	1523206
15US02812	Redland Beckman	1501503
16SH03591	Wright Bro's Realitos Pit	2106701
16US01811	Bay Sweet 16/Redland	2106706
16US01812	Redland Beckman	1501503
16US02811	Wright Bros' Realitos Pit	2106701
18IH00451	Smith Crushed Stone, Bullard Pit	14708
18IH035E1	TXI Streetman, Rap	1817502
18US01751	Meridian Mill Creek OK	50438
19SH00081	Gifford Hill, Little River	0050114
19US00591	Gifford Hill, Little River	0050114

Table 3.7(c)(Continued) Aggregate Source Used in Construction of Pavement Test Section

Section ID	Aggregate Source	Source Code
19US02711	Meridian, Apple Pit	50437
19US02712	Boorheim fields, Apple Pit	50437
20FM01051	Luhr Bros' Tower Quarry, St. Gene	50601
20FM03651	Luhr Bro's Tower Quarry, St. Gene	50601
20SH00871	Luhr Bro's Tower Quarry, St. Gene	50601
20SH03211	Redland Stone, Beckman Pit	1501503
20US00901	Luhr Bro's Tower Quarry, st. Gene	50601
21SH00041	Fordyce Co., Showers Pit	2110904
21SH01001	Fordyce Co., Showers Pit	2110904
21SP04871	Upper Valley Materials, D-garcia Pit	2110905
21US02811	Upper Valley Materials, D Garcia Pit	2110905
24FM06591	Jobe Concrete, Mckelligon Pit, Dolomite	2407201
24LP03751	Jobe Concrete, Mckelligon Pit, Dolomite	2407201
24SH00201	Jobe Concrete, Mckelligon Pit, Granite	2407206

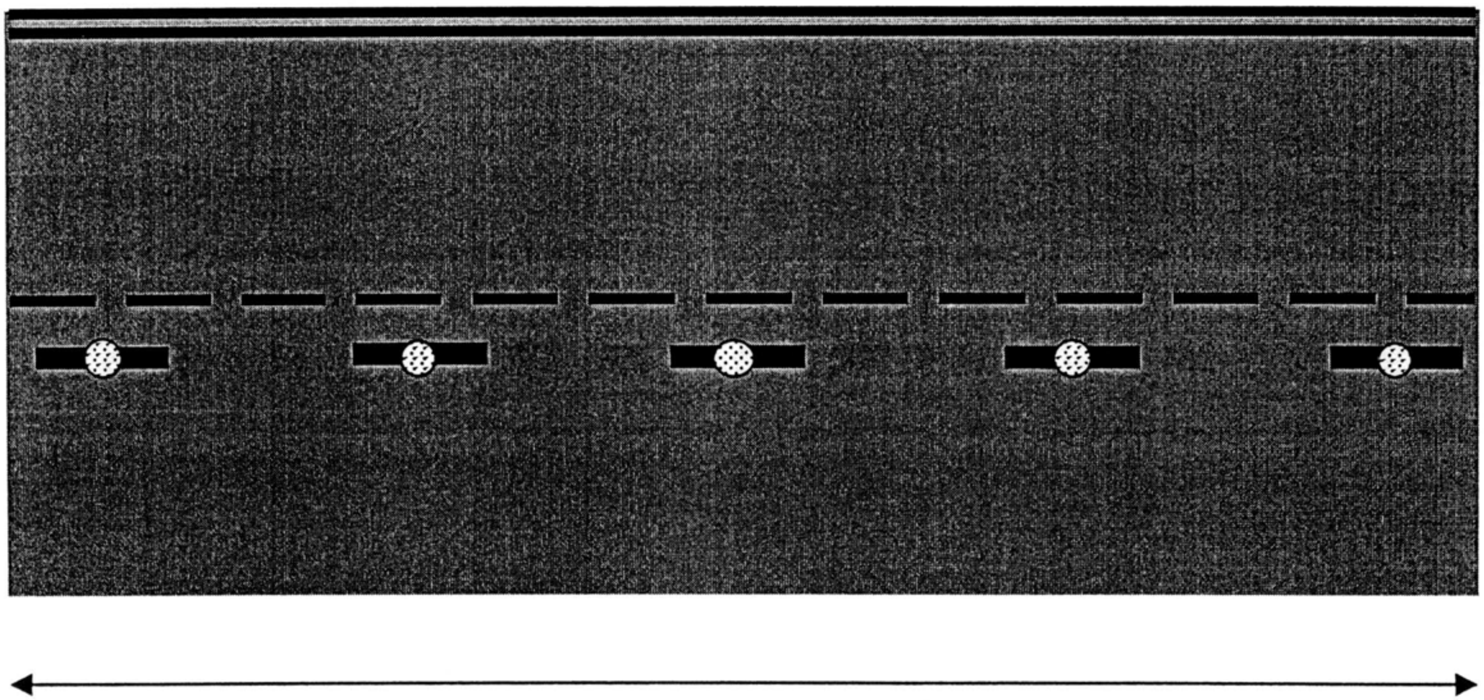


Figure 3.4 Typical Layout of a Pavement Test Section for Field Skid Test

Table 3.8 Average Field Skid Numbers and AVPPL for All Test Sections

Section_ID	AVGSN95	VPPL95	AVGSN96	VPPL96	AVGSN97	VPPL97
04IH00401	48.8	445454	47.4	1555520	48	2740690
04SH01361	47.6	249084	48.6	636522	47.7	1050129
04SH01521	55.2	195228	52	498916	51.2	823140
05FM22551	40.2	480974	46.8	1672806	43	2925912
05US00841	50.6	16331597	45.4	17298047	45.8	18314196
06IH00201	54.2	121723	51	509538	52.5	931795
06LP02501	45	3909924	42.5	5296273	41	6805834
08IH00201	37.4	794763	24.2	3031921	25.6	5011661
08IH00202	35.6	794763	21.8	3031921	26.2	5011661
08IH00203	40.2	460900	30.4	1758245	26.6	2906268
10IH00201	46.2	1016759	36.2	2795802	38.5	5919398
10LP03231	47.5	413100	46.6	1979055	47.6	4728669
10US00691	38.2	478012	43.3	1321537	42.6	2802652
10US00791	58.4	217708	50	603757	43.4	1278217
11FM12751	57	3533663	55	4692295	57.4	6669294
11US00591	36.8	4440748	32.2	6031372	29.2	8752736
11US00592	41.4	11611072	41.8	13424220	37.4	16526292
11US00593	40	3723090	39.3	6195600	38.2	10425877
11US00594	54.6	2174726	54.4	2858835	55	4026121
12FM13011	44.8	319785	40	488385	37.8	751891
12FM20041	62.6	797742	54	1539704	54.8	2377356
12FM30051	56.5	7279629	52.2	8673808	58.6	10853120
12LP01971	52.4	1063488	49.8	1585168	52.2	2400600
12SH00361	52.8	588744	44.6	1465647	40.2	2836419
14US02901	40.8	4100417	40	5759652	42.4	8373862
14US02902	41.2	4495773	37	5899738	39.7	8111750
15LP00131	29.8	5098940	28.8	6165354	29.8	7994996

Table 3.8(Continued) Average Field Skid Numbers and AVPPL for All Test Sections

15LP00132	38	237048	37.8	1580143	44.8	3851653
15LP16041	21.6	15128295	22.4	16682344	21.6	19614891
15US02811	28.2	48310830	29	53399365	31.4	63001692
15US02812	42.6	11664495	46	14239710	41.8	19099232
16SH03591	N/A	N/A	40.5	612949	43	1231660
16US01811	N/A	N/A	60.6	972348	50.8	1953893
16US01812	N/A	N/A	45.2	620286	39.2	1246380
16US02811	52.8	1259897	46.8	1795427	47.6	2511017
18IH00451	44.4	2626905	38.4	4468217	39.5	7729489
18IH035E1	38.2	4435665	35.4	6471187	36	10076478
18US01751	46.2	6704334	44.8	8068795	46.8	10485461
19SH00081	48.4	222666	45.2	608977	47.6	1281262
19US00591	51	683644	45.8	1869788	44.9	3934203
19US02711	50.5	1073556	45.2	1537473	47.6	2334663
19US02712	60.4	182590	60.8	505257	57.6	1059771
20FM01051	50	3488661	45.2	3788881	52	4411696
20FM03651	59.4	874836	44.6	1240861	47.6	2006277
20SH00871	49.8	1583799	38.8	2353739	39	3951093
20SH03211	45	270070	34.4	849342	30.8	2047958
20US00901	49.8	240567	41.8	756564	46.2	1824279
21SH00041	50.2	94064	39.8	542012	31.4	1140555
21SH01001	44.2	2320904	42.4	3082904	39.4	4101040
21SP04871	41.8	225950	40.2	1326999	34.9	2792439
21US02811	53	177021	45.4	1039632	38.1	2187718
24FM06591	33	1028200	30	1539668	27.8	2098818
24LP03751	42.8	1835064	39.6	2510142	34	3248137
24SH00201	33	1354213	33.8	1873327	25.4	2440804

3.3.3 Skid Number versus Test Speed Relationship

The skid number (SN) and the speed at which it is measured (V) are inversely related. In other words, skid number decreases as the speed increases. Figures 3.5(a) – 3.5(f) depict the above inverse relationship between SN and V. The slope of the line is defined as the Percent Normalized Gradient (PNG). Skid number can be mathematically represented as:

$$SN = SN_0 \, e^{-(PNG/100)V} \tag{3.1}$$

$$\ln(SN) = \ln(SN_0) - (PNG/100).V$$

where SN_0 = skid number at 0 kmph

The PNG is an indicator of the rate of drop in skid number as speed increases. A lower normalized gradient indicates that it will retain its skid resistance at higher speeds than a pavement with a higher normalized gradient.

SN-Speed relationship is important for two special reasons:

(a) The SN-Speed relation for a given pavement surface serves as the fundamental mechanism for separating the contributions from the microtexture and the macrotexture to the pavement frictional resistance.

(b) The standard test speed for skid measurements, according to ASTM-274 is 64 kmph (40 mph.) However, in routine skid resistance tests, deviations from the standard test speed occur.

Speed relations provide a mechanism for "normalizing" these SN values to the standard SN40 values.

The inverse relationship between the skid number and the test speed is explained as follows. The skid numbers are measured under wet pavement conditions and hence they depend on the thickness of the water film between the tire and the pavement surface. As the tire moves over the wet pavement surface, water is displaced from the underneath the tire. At low speeds, the rate of expulsion of the water is low and the flow rate could be easily accommodated by the grooves in the tire and the macrotexture in the pavement. At these speeds the measured skid number depends primarily on the microtexture of the pavement and

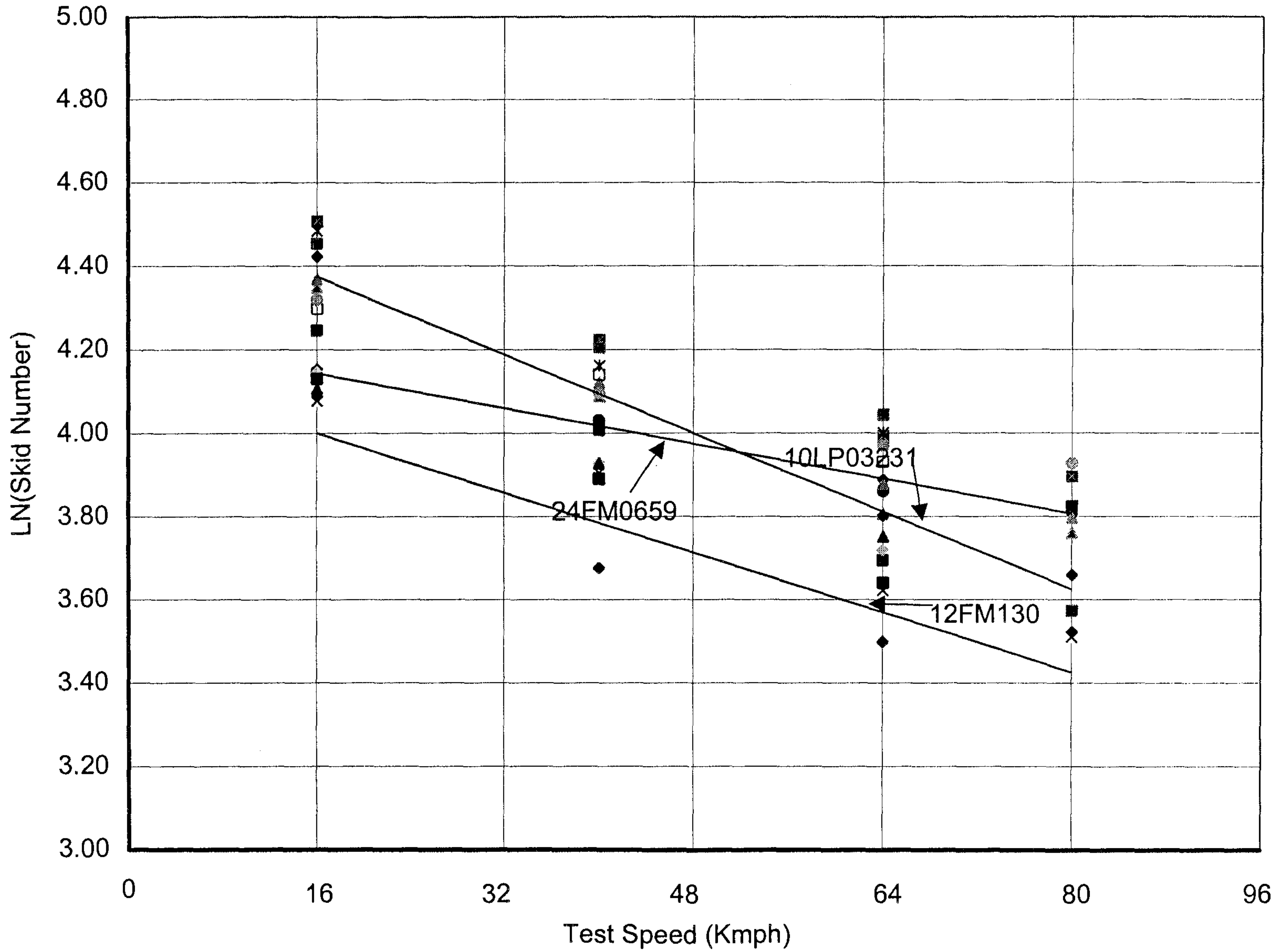


Figure 3.5(a) Effect of Speed on Skid Number on Open Textured Surfaces - 1995

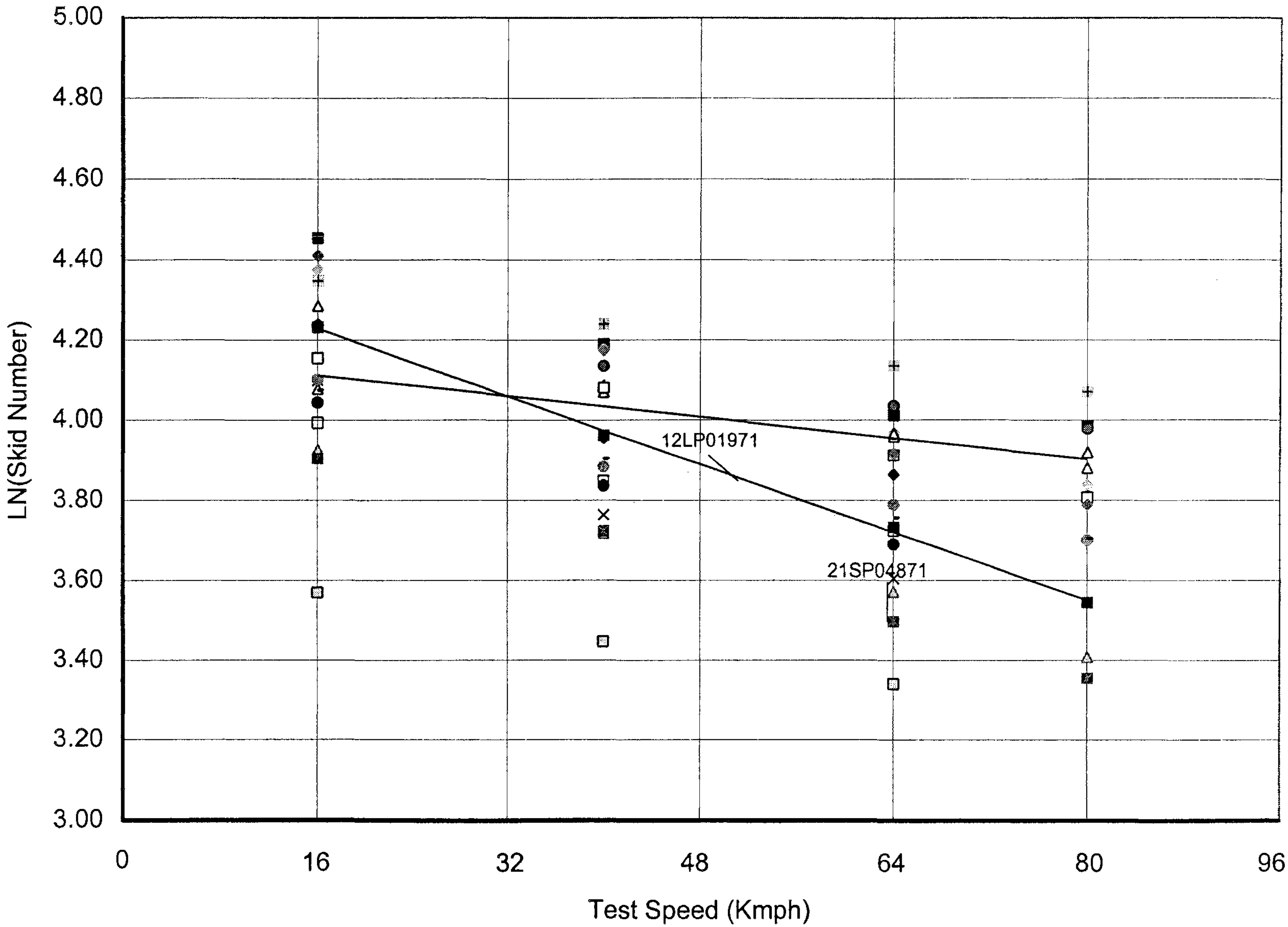


Figure 3.5(b) Effect of Speed on Skid Number on Type D Surfaces - 1995

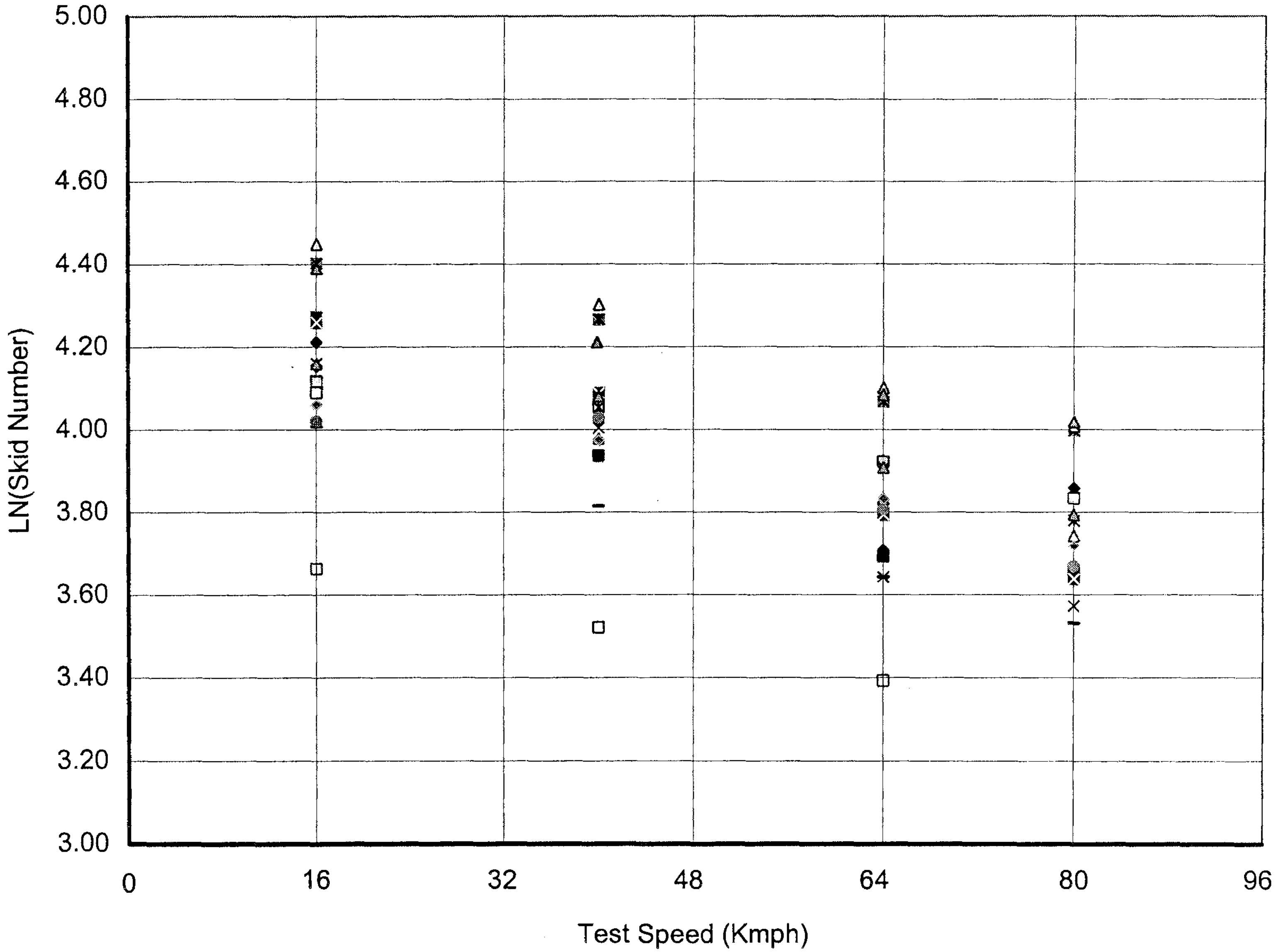


Figure 3.5(c) Effect of Speed on Skid Number on Type C Surfaces - 1995

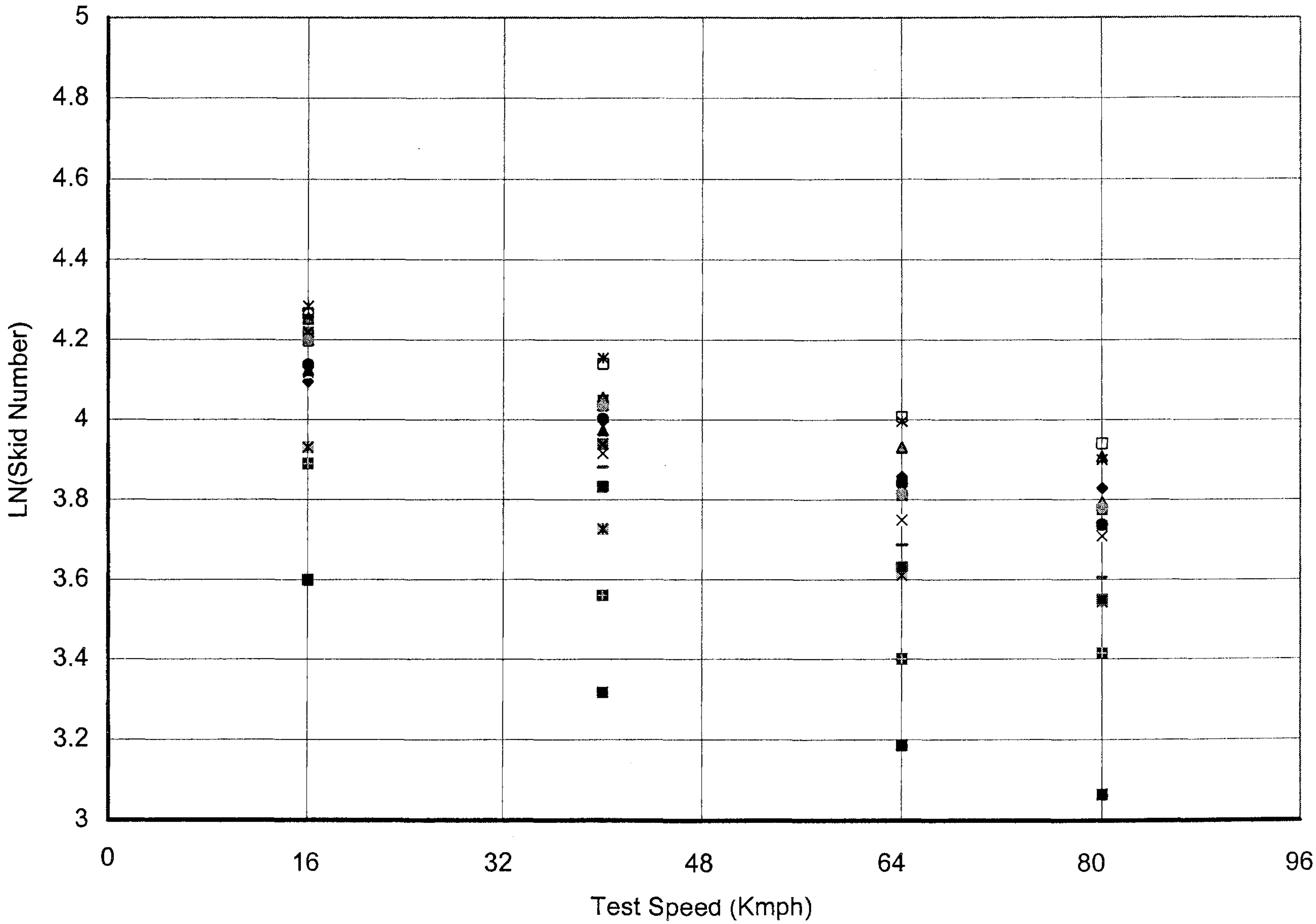


Figure 3.5(d) Effect of Speed on Skid Number on Open Graded Surfaces - 1996

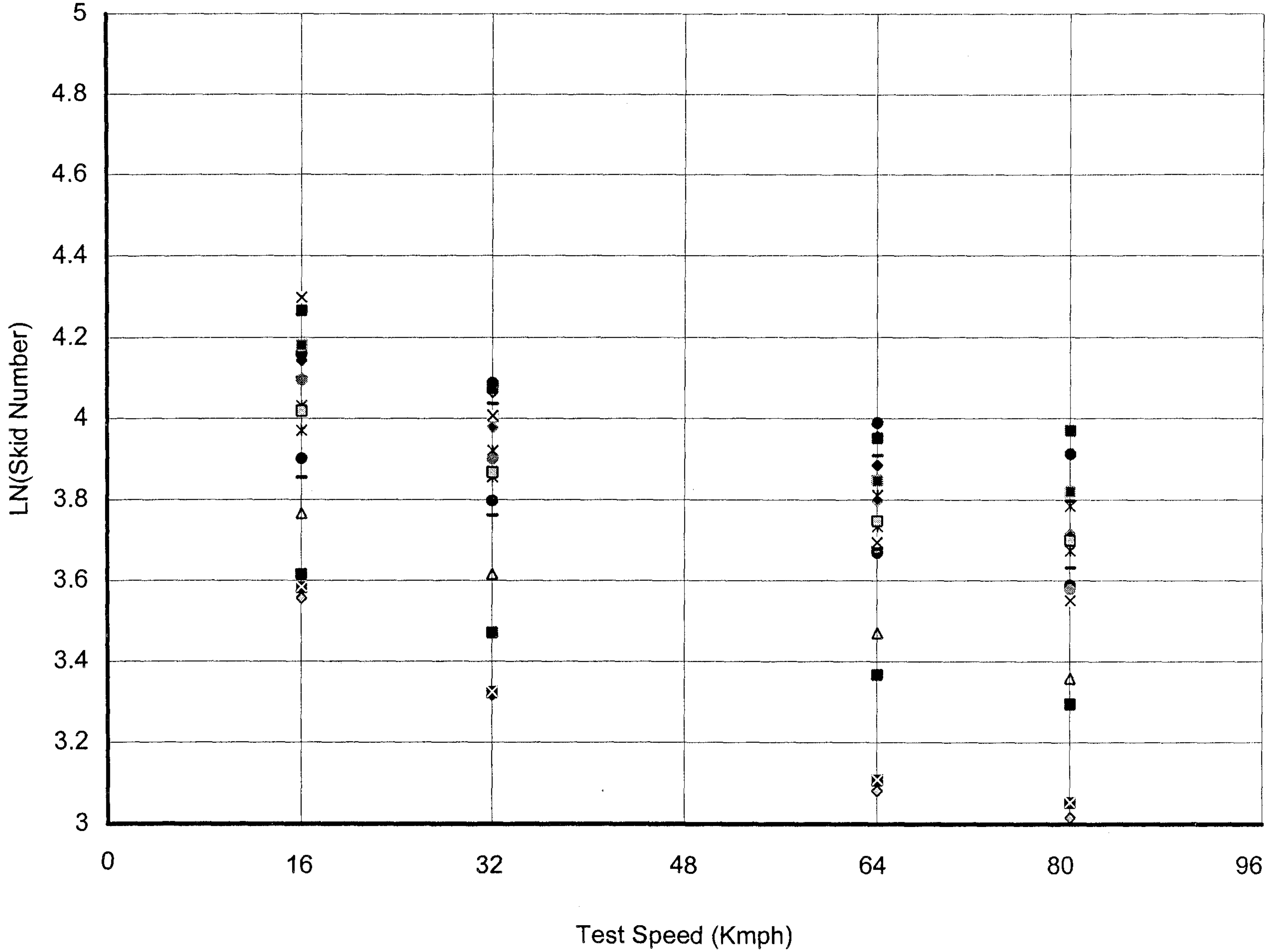


Figure 3.5(e) Effect of Speed on Skid Number on Type D Srufaces - 1996

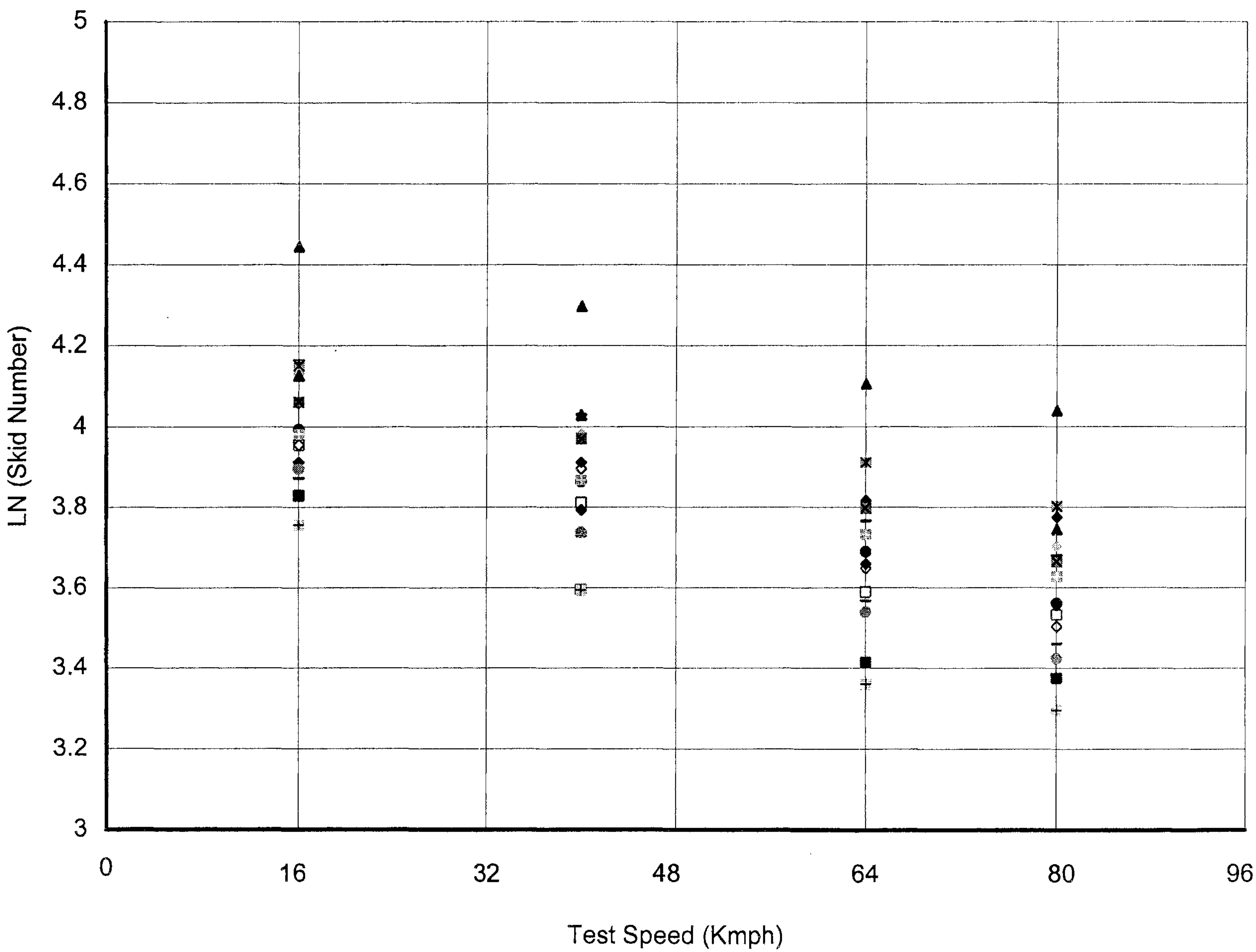


Figure 3.5(f) Effect of Speed on Skid Number on Type C Surfaces - 1996

is independent of the macrotexture. However, as the speed increases, the rate of water expulsion from underneath the tire increases as well. If the pavement has a smooth macrotexture, the channels on the pavement surface will now be unable to handle the larger flow rate. As a result, the thickness of the water film underneath the tire increases and the measured skid number decreases rapidly. A heavily textured pavement, however, will have better drainage capability and therefore will be able to maintain the skid number at higher speeds. Therefore, in a pavement with good macrotexture, the skid number does not decrease with speed as much as it does on a pavement with poor macrotexture. This theory for explaining SN-V relationship has been confirmed by observed behavior.

The current state of-the-knowledge on SN-Speed relations is as follows.: SN versus Speed, V can be described by Equation 3.1.

$$SN = SN_0 \ e^{-(PNG/100).V} \qquad (3.1)$$

where SN_0 is the intercept of the curve with y-axis and represents the fictitious SN at speed zero. The magnitude of SN_0 is a function of the microtexture of the pavement. PNG (Percent Normalized Gradient) represents the slope of the curve and is a function of the macrotexture of the pavement. PNG and macrotexture are inversely related. In other words the greater the macrotexture in the pavement, the smaller the magnitude of the slope (PNG) would be.

The above procedure provides a mechanism for separating the effects of micro- and macro- textures. Therefore, statistical regression should be performed between SN_0 (and not SN_{40}) and the laboratory tests data concerning aggregate polishing characteristics, such as PV-value. Similarly, regression equations can be developed to relate PNG and parameters that control macrotexture such as aggregate gradation

The above approach which allows separation of the micro- and macro- texture components in the skid numbers, showed considerable promise and was investigated in this study. During the first two years of the study skid numbers for speeds of 10 mph, 25 mph , 40 mph, and in many cases 50 mph were determined. The results are summarized in Figures 3.5(a) -3.5(f) and the skid numbers for a particular speed are the average of five skid results as described in Section 3.3.2. Tables 3.9(a) and (b) shows the SN_0 and PNG values for all 54 sections.

3.3.4 Macrotexture Measurement

Macrotexture is the large scale asperities present on the pavement surface. Macrotexture represents the overall topography of the pavement surface which is a function of the size, shape and gradation of the coarse aggregates.

In the second and third year of the study macrotexture was measured using the Mini Texture Meter (MTM). The Mini Texture Meter, developed by the The Transport and Road Research Laboratory, England, is a hand operated laser device that can measure surface texture up to a resolution of 0.01mm. Figure 3.6 shows the calibration procedure of the MTMT using a calibration mat provided by the manufacturer. Figure 3.7 shows the usage of the MTM in the field.

In the first year of this study, the MTM was not available for use and thus the sand patch method was used to measure macrotexture. Since a satisfactory correlation between the two tests could not be established, only the MTM macrotexture measurements, also referred to as Sensor Measured Texture Depth (SMTD), was used in all subsequent analyses. The results of the last two years SMTD measurements are summarized in Table 3.10.

Table 3.9 (a) SN_o Values for all Pavement Test Sections

SECTION ID	SNo '96	SNo '97
04IH00401	69	64
04SH01361	73	65
04SH01521	93	72
05FM22551	83	80
05US00841	96	61
06IH00201	103	75
06LP002501	89	67
08IH00201	69	40
08IH00202	57	38
08IH00203	80	50
10IH00201	67	59
10LP03231	69	70
10US00691	77	74
10US00791	91	69
11FM12751	97	77
11US00591	69	47
11US00592	59	56
11US00593	64	53
11US00594	102	80
12FM13011	96	136
12FM20041	83	68
12FM30051	73	61
12LP01971	64	65
12SH00361	91	65
14US02901	76	61
14US02902	73	55
15LP00131	43	49

Table 3.9 (a) (Continued) SN_o Values for all Pavement Test Sections

SECTION ID	SNo '96	SNo '97
15LP00132	73	75
15LP16041	*	38
15US02811	38	39
15US02812	68	74
16US02811	77	68
18IH00451	85	61
18IH035E1	62	53
18US01751	64	70
19SH00081	90	72
19US00591	84	81
19US02711	67	70
19US02712	96	95
20FM01051	71	58
20FM03651	87	66
20SH00871	67	55
20SH03211	65	56
20US00901	72	59
21SH00041	96	66
21SH01001	65	58
21SP04871	81	84
21US02811	80	74
24FM06591	63	54
24LP03751	64	49
24SH00201	58	55

Table 3.9 (b) PNG Values for all Pavement Test Sections

SECTION ID	PNG '96	PNG '97
04IH00401	0.86	0.7
04SH01361	1.04	0.7
04SH01521	1.2	0.7
05FM22551	1.74	1.3
05US00841	1.13	0.7
06IH00201	1.55	0.9
06LP002501	1.54	1.1
08IH00201	1.48	1.3
08IH00202	1.25	1.3
08IH00203	1.58	1.2
10IH00201	0.92	1.1
10LP03231	0.84	1.0
10US00691	1.58	1.3
10US00791	1.05	0.9
11FM12751	1.37	0.8
11US00591	1.65	1.0
11US00592	0.9	0.7
11US00593	1.18	0.8
11US00594	1.62	1.0
12FM13011	1.88	1.5
12FM20041	0.69	0.6
12FM30051	0.64	0.6
12LP01971	0.52	0.7
12SH00361	1.35	1.0
14US02901	1.48	1.1
14US02902	1.43	1.0
15LP00131	0.9	1.2
15LP00132	1.63	1.6

Table 3.9 (b) (Continued) PNG Values for all Pavement Test Sections

SECTION ID	PNG '96	PNG '97
15LP16041	*	1.3
15US02811	0.77	0.8
15US02812	1.18	0.9
16US02811	0.9	0.9
18IH00451	1.6	1.2
18IH035E1	1.19	1.0
18US01751	0.86	1.1
19SH00081	1.46	1.1
19US00591	1.21	1.1
19US02711	0.72	1.0
19US02712	1.13	1.1
20FM01051	0.89	0.6
20FM03651	0.93	1.0
20SH00871	0.78	0.9
20SH03211	0.92	1.2
20US00901	0.93	0.9
21SH00041	1.58	1.3
21SH01001	0.97	0.8
21SP04871	1.69	1.8
21US02811	0.98	1.1
24FM06591	1.44	1.2
24LP03751	0.94	0.5
24SH00201	1.39	1.4

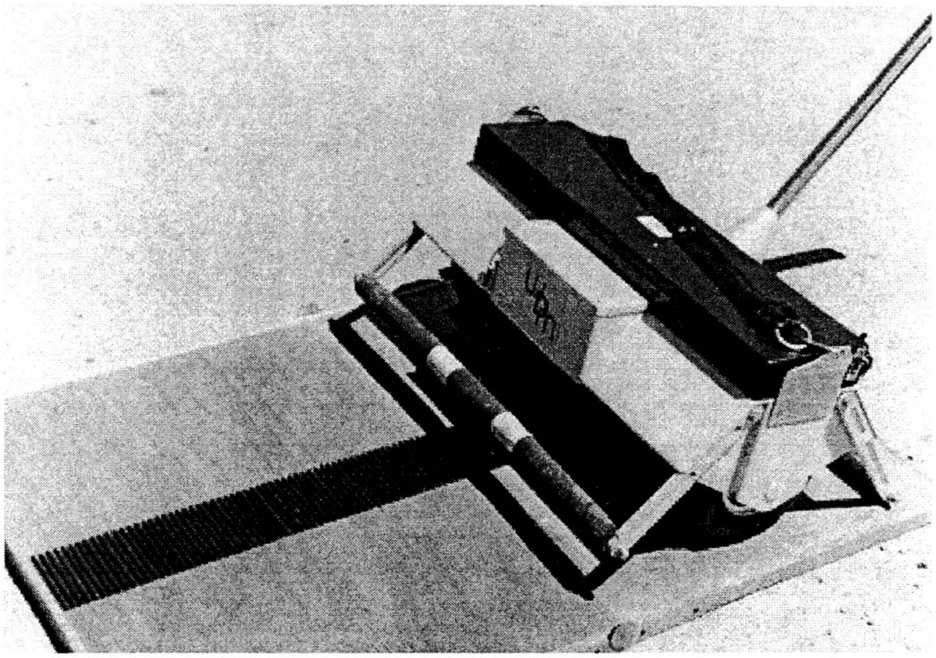


Figure 3.6 Calibration Procedure of Mini Texture Meter

Figure 3.7 Pavement Macrotexture Measurement Using a Mini Texture Meter

Table 3.10 Average Macrotexture Measurement on Pavement Test Sections

SEC_ID	AVG_MTM 96 (mm)	AVG_MTM 97 (mm)
04IH00401	29.8	37
04SH01361	30.2	31
04SH01521	26.6	32
05FM22551	36.8	44
05US00841	32	34
06IH00201	46.4	49
06LP02501	46.8	40
08IH00201	31.6	34
08IH00202	21	21
08IH00203	31.6	24
10IH00201	30.4	29
10LP03231	48	46
10US00691	29.4	32
10US00791	29.2	31
11FM12751	64.2	71
11US00591	28.8	33
11US00592	28	32
11US00593	19.6	27
11US00594	48.4	59
12FM13011	21	20
12FM20041	28.8	31
12FM30051	40.8	46
12LP01971	26.4	31
12SH00361	23.2	24
14US02901	27.4	26
14US02902	36	41
15LP00131	21	19

Table 3.10(Continued) Average Macrotexture Measurement on Pavement Test Sections

SEC_ID	AVG_MTM 96 (mm)	AVG_MTM 97 (mm)
15LP00132	41.4	49
15LP16041	25	31
15US02811	21	28
15US02812	33	40
16SH03591	*	32
16US01811	*	30
16US01812	*	29
16US02811	27.8	28
18IH00451	35.2	34
18IH035E1	33.4	29
18US01751	33.2	32
19SH00081	45.6	43
19US00591	33.8	43
19US02711	29.2	26
19US02712	24.2	29
20FM01051	35.2	35
20SH00871	28	30
20SH03211	25	28
20SH03651	28.4	29
20US00901	27.8	27
21SH00041	25	25
21SH01001	31	35
21SP04871	38.6	23
21US02811	50.4	55
24FM06591	34.6	41
24LP03751	27.8	36
24SH00201	24.2	24

3.3.5 British Pendulum Number (BPN) measurement

British Pendulum Tester (BPT), shown in Figure 3.8, was also used in accordance with ASTM E-303 to determine the British Pendulum Number (BPN). The BPN was measured at three different points on the left wheel path of the outside lane in the longitudinal direction of the test section. At each point the pavement was brushed with water to remove surface dust. Care was taken to calibrate the pendulum and adjust its swing such that the contact length of the sliding rubber was exactly $5^1/_2$ inches. Figure 3.9 shows the measurement of contact length after adjusting the contact height of BPN in the field. The first reading from the first swing of the pendulum was ignored and the next five readings were recorded. The average of these five swings were taken as the BPN of that point. This procedure was repeated in the next two points. The average of the resulting 15 points was taken as the average BPN of the test section. These averages are shown in Table 3.11 for the years 1995, 1996, and 1997.

3.3.6 Documentation of pavement condition

Photographs of each point where a British pendulum test was conducted was taken. This was done to compare any visible changes in the pavement surface condition occured from year to year. This documentation may have become useful to explain any anomalous change in BPN between two readings on the same location. A sample of these photographs are shown in figures 3.10 (a) and (b).

3.4 MONITORING OF PAVEMENTS FOR SEASONAL VARIATION

3.4.1 General Description of Pavements Selected for Climatic Influence

On the basis of climatic conditions, Texas is divided into four regions as indicated by the dotted lines in Figure 3.3. These regions are: Region I: wet, non freeze-thaw, Region II: wet, freeze-thaw, Region IV: dry, non freeze-thaw, Region V: dry, freeze-thaw. Six sections, two each from climatic regions II, IV, and V, were chosen for this study to examine the seasonal variations in skid number. All six pavement sections were hot mix asphalt concrete pavements although they were not similar in mix design or the type of aggregate used in their

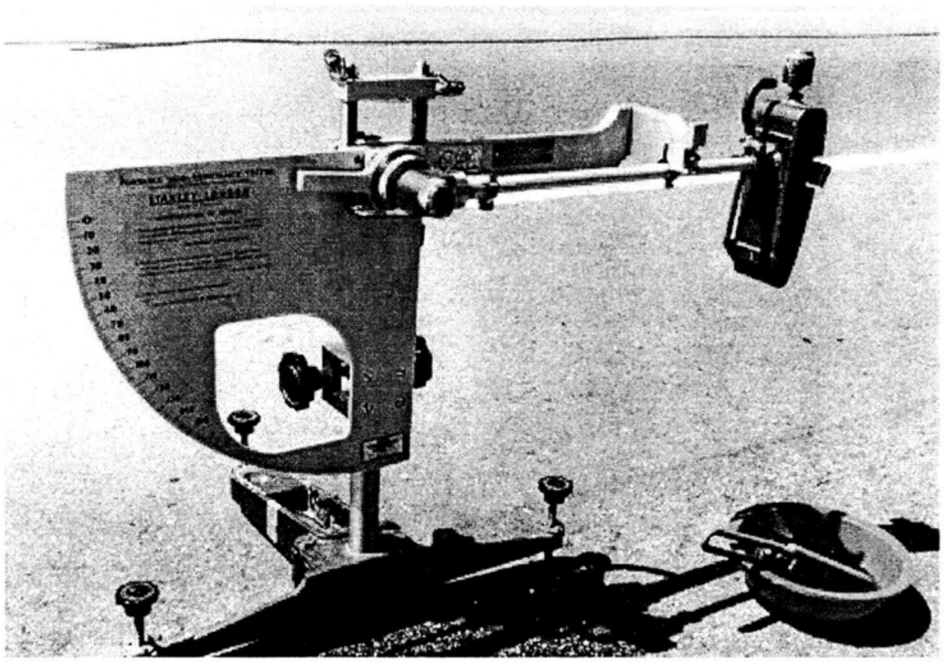


Figure 3.8 British Pendulum Tester

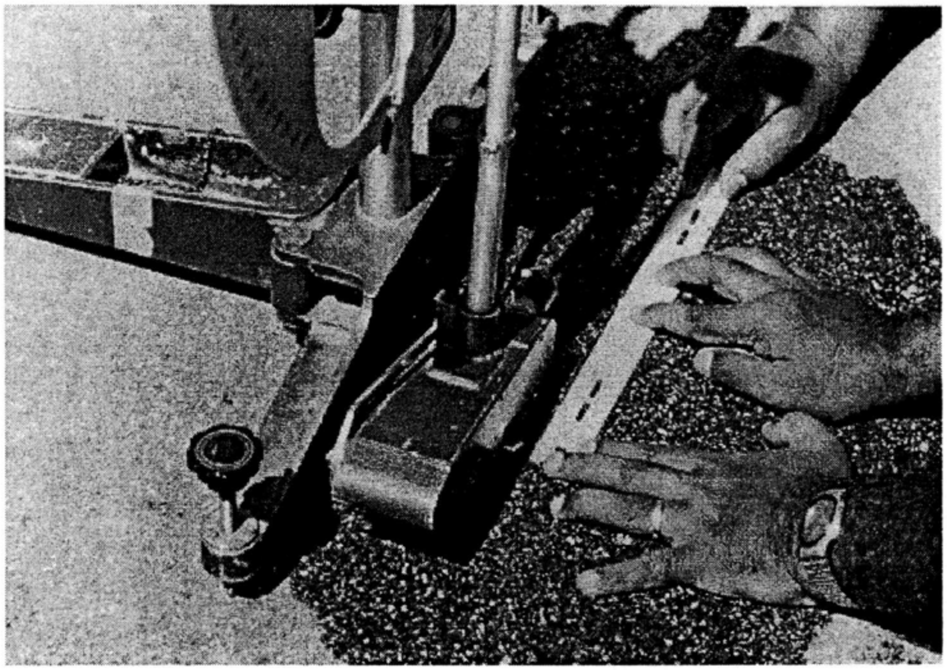


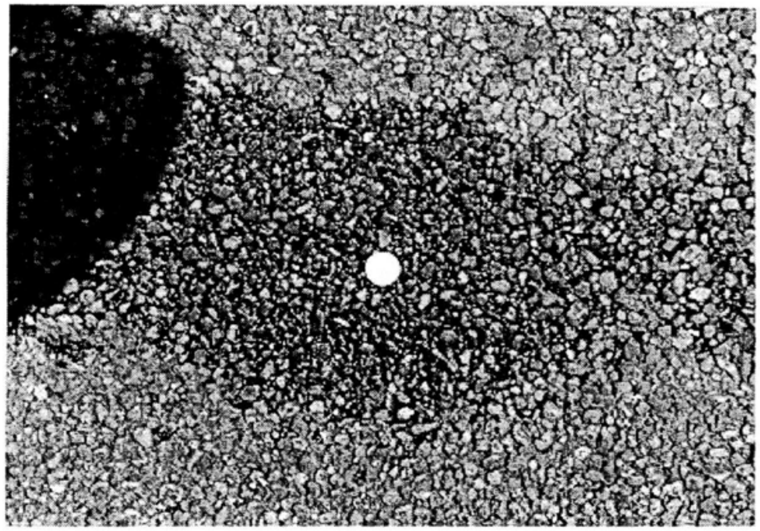
Figure 3.9 Measurement of Contact Length of BPT in the Field

Table 3.11 Field Measured BPN Values on Pavement test Sections

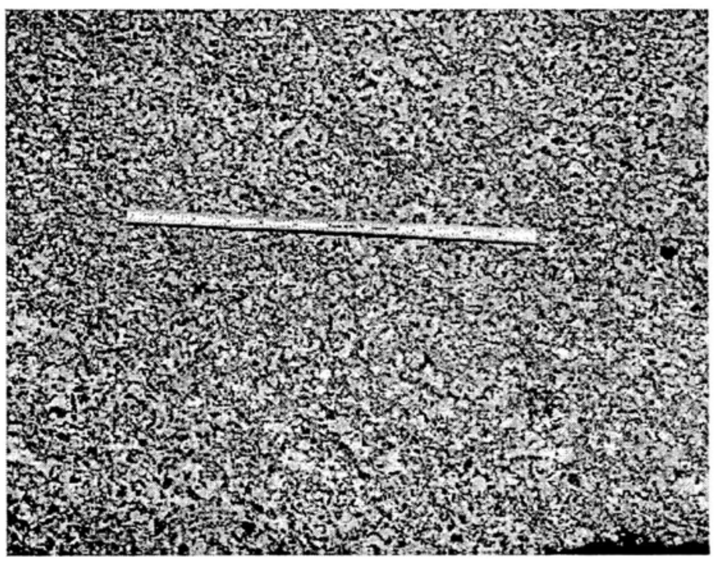
Sec_ID	BPN 95	BPN 96	BPN 97
04IH00401	74.1	63.2	64.6
04SH01361	78.1	65.3	71.9
04SH01521	75.1	71.7	70.0
05FM22551	71.7	77.3	73.7
05US00841	68.8	61.7	72.2
06IH00201	79.1	73.2	75.0
06LP02501	77.2	69.3	65.3
08IH00201	61.8	45.3	49.1
08IH00202	64.7	49.4	52.1
08IH00203	72.0	55.7	48.8
10IH00201	62.5	55.4	48.9
10LP03231	69.7	59.3	69.6
10US00691	71.3	66.7	67.1
10US00791	74.1	69.4	58.3
11FM12751	75.3	80.1	77.6
11US00591	58.1	56.4	60.0
11US00592	57.0	60.4	61.1
11US00593	55.8	61.1	61.0
11US00594	69.0	80.4	77.4
12FM13011	61.8	64.7	61.3
12FM20041	71.1	65.9	66.3
12FM30051	67.4	62.1	75.5
12LP01971	62.1	62.3	70.6
12SH00361	64.9	57.2	58.9
14US02901	60.1	66.8	69.6
14US02902	56.7	64.6	67.3
15LP00131	52.2	64.7	64.6
15LP00132	69.8	66.9	76.9

Table 3.11 (Continued) Field Measured BPN Values on Pavement test Sections

Sec_ID	BPN 95	BPN 96	BPN 97
15LP16041	*	40.3	37.8
15US02811	47.3	54.6	44.0
15US02812	58.0	66.7	57.7
16SH03591	*	*	73.1
16SH01811	*	*	73.7
16SH01812	*	*	61.7
16US02811	60.5	59.3	70.5
18IH00451	69.5	56.3	58.2
18IH035E1	55.8	55.5	49.9
18US01751	70.0	64.0	54.5
19SH00081	66.8	65.9	75.4
19US00591	64.7	65.6	63.8
19US02711	62.4	59.7	73.7
19US02712	77.8	84.1	83.2
20FM01051	67.7	61.6	57.5
20FM03651	71.7	64.5	57.7
20SH00871	62.2	57.4	49.2
20SH03211	65.8	58.5	60.3
20US00901	60.6	61.0	62.9
21SH00041	63.7	56.3	60.9
21SH01001	54.7	63.4	57.7
21SP04871	65.2	69.3	69.5
21US02811	65.1	58.7	66.0
24FM06591	58.0	61.3	50.8
24LP03751	69.3	61.1	55.4
24SH00201	69.7	65.5	55.7



(a) Pavement Surface with Rough Macrotexture



(b) Pavement Surface with Smooth Macrotexture

Figure 3.10 Documentation of Pavement Macrotexture construction. The approximate locations of these sections are shown by the bold circles in Figure 3.3, while the open circles represented other pavement sections that were included in the study but were not monitored for seasonal variations. In Region II, the two sections are located in Angelina county (TxDOT district Lufkin), the two sections in Region IV are in Bexar county (TxDOT district San Antonio) and in Region V the sections are in Callahan county (TxDOT district Abilene). The number of sections and their locations were selected based on the availability of skid trailers and manpower in the TxDOT districts.

The testing frequency also depended on these factors. Another important criterion considered in the test section location is the proximity to a Climatic Data Center. All test pavement sections were selected such that they were located as close as possible to a National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center. Table 3.12 presents the relevant attributes of each test pavement section.

3.4.2 Mechanisms Controlling Seasonal Variation of Skid Number

At the present time, there is no understanding of the fundamental mechanisms that control seasonal variations in skid number. However, some empirical understanding of the processes involved exists. The general hypothesis presented by previous researchers to explain the observed phenomenon is as follows: in summer, there are prolonged periods of dry weather which allows the fine particles that are polished off the pavement surface to accumulate resulting in loss of microtexture and macrotexture. This action together with contamination from vehicles, such as oil drippings and grease lead to lower skid resistance during summer. In winter deicing salts cause surface wear and the aggregate surface rejuvenates exposing new particles. Due to heavy precipitation in the spring the fine grit is flushed out leaving coarser surface on the aggregate surface. Rainfall also flushes out the drainage channels between aggregates and thus increases the macrotexture of the pavement. The coarser aggregate surface and the increased macrotexture in turn leads to increase in the skid resistance of the pavement in early spring. In addition, it is believed that the polishing action of the aggregate is reduced in winter as pavements remain wet for longer periods than

Table 3.12. General Information Pertaining to Test Pavement Sections

Hwy Section	Location starts (km[1] from RM[2])	Region	Mix Design	MTM Texture[3] (mm)	Aggregate Type	ADT (Thousands)	No. of Lanes	Cumulative VVPL[4] (millions)
US 59-1	0.2 south of 380	II	Type D[4]	0.31	Limestone	19.3	4	8.75
US 59-2	0.3 south of 384	II	Type D	0.30	Siliceous	22.0	4	16.5
US 281-1	0.2 north of 528	IV	Type D	0.25	Limestone	88.8	6	63.0
US 281-2	0.9 north of 520	IV	Novachip[4]	0.37	Traprock	28.1	4	19.1
IH 20-1	0.7 east of 296	V	Type D	0.33	Limestone	18.9	4	5.0
IH 20-2	0.0 east of 297	V	CMHB-C[4]	0.21	Limestone	18.9	4	5.0

[1]1 km = 0.6 mi.

[2]RM = Refference Marker

[3]The texture measurement shown is the average of the macrotexture measurements made in the past two years with a Mini Texture Meter that uses a laser diffraction method.

[4]Cumulative VPPL = approximate cumulative vehicle passes per lane from the date of construction to the date of skid measurement in 1997.
[5]Type D is a mix design which provides a smoother or denser mix in comparison to Novachip or CMHB-C mix.

in summer. In wetter periods the water film covering the pavement act as lubricants and reduce the polishing effect of vehicles on the surface aggregates.

Temperature changes do not have any direct effect on the skid resistance of the pavement surface. Nevertheless, they affect the properties of the rubber tire used in locked wheel skid trailer. The mechanism involved in variation due to temperature changes is attributed to hysteresis of the rubber tire. Hysteresis is the energy lost upon elastic recovery, in the form of heat, when the rubber tire is compressed as it slides over the pavement. It follows that at higher temperatures rubber becomes more flexible leading to more energy loss. Higher temperatures thus lead to a decrease in the measured skid resistance. However, no conclusive proof of this is available in literature as some studies claim the effect of temperature to be non-existent or very insignificant (10). On the other hand many other studies indicate that temperature is a significant factor (11, 12, 13, 14, 15, 16, 17, 18, 19) One such study even suggests that the temperature has a greater influence on skid resistance than rainfall (19).

3.4.3 Data Collection Procedures

All skid measurements were performed according to ASTM Specification E274 (20). For each of the six sections the average SN_{64}, rainfall, and average hourly ambient temperature throughout the study duration were collected. The average SN_{64} was the average of five skid measurements made at 64 km/hr (40 m/hr) at five locations longitudinally along the inside wheel path of the outside travel lane. The distance between the two consecutive longitudinal locations was approximately 100m. Care was taken to ensure that these locations were free of flushing, raveling, cracking or other types of distress. Skid measurements for the four test pavement sections in Regions II and IV were made at approximately two week intervals. In Region V, the measurements were taken at monthly intervals. However, due to equipment failure and time required for repair, data does not exist for every bi-weekly or monthly period. In spite of this setback a total of 107 sets of SN_{64} readings were collected for the six sections over a period of more than 18 months.

Rainfall and temperature data was collected from the weather station nearest to the test location provided by the National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center. The following weather stations were used: Lufkin NW7 for Region II test sections, San Antonio Airport for Region IV test sections, and WSO Abilene for Region V test sections. For the purposes of analysis only significant rainfall measurements were used, where significant rainfall was taken as rainfall 2.5mm (0.1 inches) or greater.

3.4.4 Significance of Rainfall and Temperature on Skid Number

It is evident from Figure 3.11 that significant variations do occur in skid numbers measured on the same pavement at different times. There appears to be a general long-term trend in skid number variations. As the temperature rises the skid numbers decrease in magnitude and as the temperature falls the skid numbers increase in magnitude. It follows a cyclical pattern with the lowest skid numbers in the summer months and the highest skid numbers in winter or early spring. This pattern is in agreement with what other research studies have reported in the past (11, 13, 14, 15, 16, 17, 21, and 22). Another significant observation is that both pavements follow a very similar skid variation pattern. This observation suggests that the variations observed are not random but occurred in response to some common factor that influenced both pavements. Furthermore, the sudden increases (or peaks) in skid number (such as those identified by the arrows) appear to be closely associated with significant rainfall events especially when they occur after extended periods of dry weather. The only exception is the second peak seen in the skid pattern. Rainfall in the summer months, however, is not associated with high skid numbers. This is possibly due to the counteracting effects of higher temperatures and other factors, such as heavier traffic in summer.

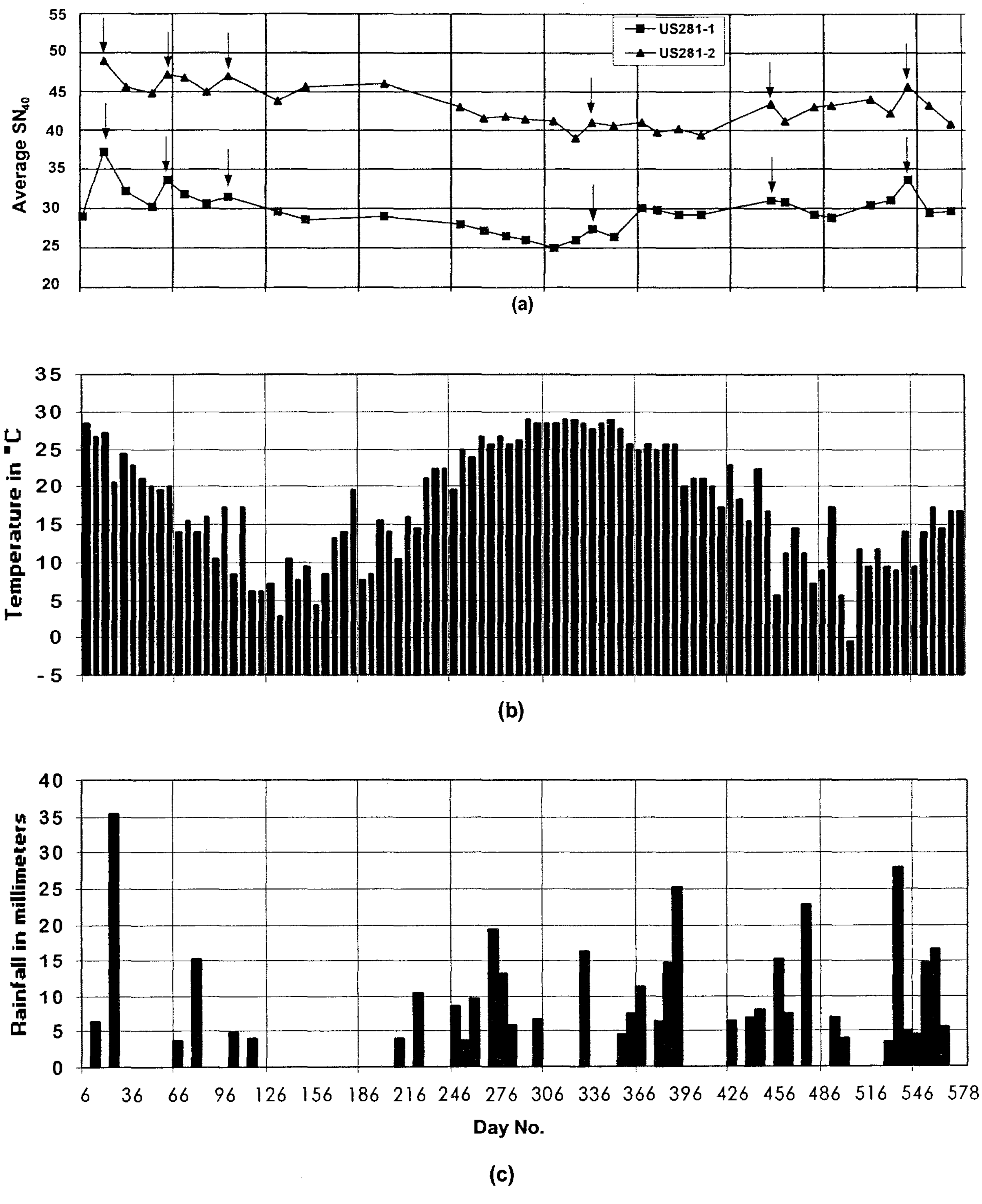


Figure 3.11. (a) Skid Number Measured on US 281-1 and US 281-2, (b) Temperature and Cumulative 5-Day Rainfall

CHAPTER 4
RATING OF AGGREGATES BASED ON LABORATORY TEST DATA

INTRODUCTION

The current chapter discusses the development of a procedure for evaluating the field skid performance rating of a aggregate based on the results of various laboratory tests performed on the aggregate. This evaluation procedure makes use of a skid performance rating (SPR) number, instead of actual field measured skid numbers, to predict the field skid performance of aggregates to be used in pavement surface courses. The technique of determining the SPR number for a pavement surface and its prediction are discussed below.

4.1 DESCRIPTION OF SKID PERFORMANCE RATING (SPR) PROCEDURE.

The aim of developing a SPR procedure was to develop reliable regression equations between the field skid performance of the pavement and the laboratory properties of the pavement aggregate. Among the laboratory procedures included in this analysis, the polish value test which forms the basis for TxDOT's current aggregate qualification procedure was of special interest. In this test, the British Pendulum Number after 9 hours of polishing (BPN_9) represents the "terminal" frictional condition of the aggregate after it has been exposed to millions of vehicle passes. Therefore, we should seek a correlation between BPN_9 and the terminal skid resistance of the aggregate. Unfortunately, many of the pavements sections monitored in this research study have not sustained large enough cumulative traffic volumes to enable direct measurement of terminal skid numbers. Therefore, the terminal skid number corresponding to each pavement must be established by continuing skid resistance measurements beyond the 3-year research period. Since the terminal skid resistance could not be determined for all the test pavement sections, an alternative measure of field performance of the aggregate called the "Skid Performance Rating (SPR)" was developed. The procedure used in the determination of the SPR for each pavement can be explained using Figure 4.1

Using the average daily traffic (ADT) data provided by TxDOT, the number of accumulated vehicle passes per lane (VPPL) from the date of construction to the date of field skid testing was determined for each test section. A combined plot of field skid numbers at 64 km/h (40 mph) and VPPL for all test sections was prepared using the data collected during the three years of the research study. Figure 4.1 is a schematic example of such a combined plot of field measured skid numbers and VPPL.

In Figure 4.1, several shapes for the data points have been used to differentiate the observations made on various pavement test sections. For example, data points plotted using gray circles belong to the same pavement test section and the line joining these gray circles represents the variation of field skid history for that pavement test section with accumulating VPPL. It can be seen from Figure 4.1, that field skid numbers decline from one test cycle to the other and with accumulating VPPL. Subsequently, for the purpose of analysis, the SN versus VPPL plot was divided into a number of zones, each zone representing "excellent," "very good," "good," "fair," or "poor" field performance.

Note that the dotted lines that form the boundaries of performance zones have been drawn such that they follow the same general SN_{64} versus vehicle passes trend. These dotted lines, which are labeled by SPR numbers, decline sharply in the initial stages and gradually stabilize towards a terminal field skid number with accumulating VPPL. The terminal field skid number corresponding to the labeled dotted lines are shown in Table 4.1

Table 4.1 SPR Numbers and Corresponding Terminal Field Skid Numbers

SPR Numbers	Terminal Field Skid Numbers
1	28
2	32
3	38
4	43
5	47

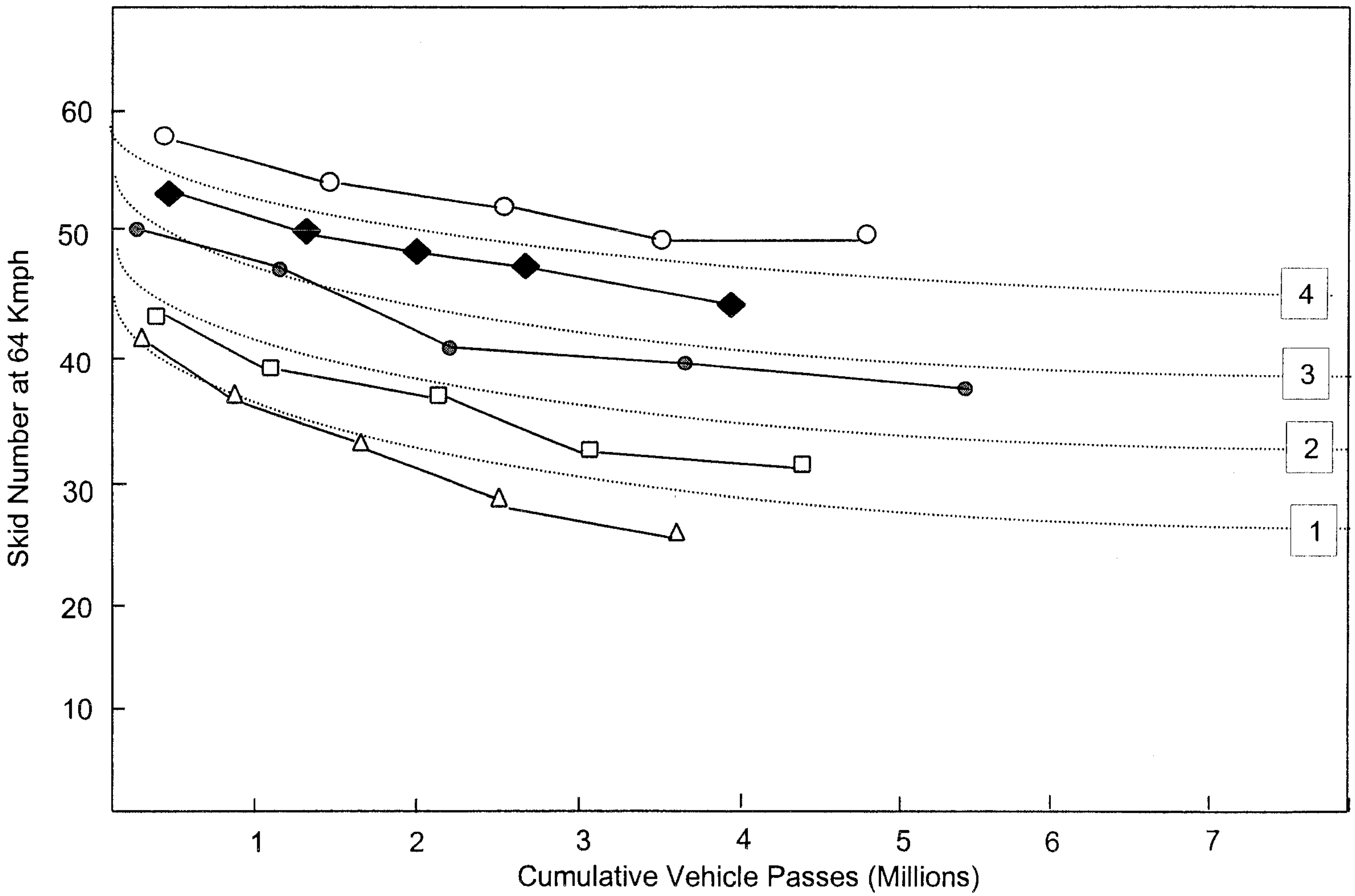


Figure 4.1 Determination of Field Skid Performance Ratings

SPR for a given test section is then calculated in the following manner. SPR of each skid measurement of a pavement test section is determined by linear interpolation based on its location between the dotted lines. Subsequently, the SPR for the given pavement test section is calculated as the average of all such linearly interpolated SPR numbers. For example if you consider the test pavement section represented by the gray circles, starting from the left hand side of the plot, SPR numbers that can be assigned to each data point are 2.5, 3.1, 2.2, 2.8, and 2.4. Hence, an average SPR number for this pavement test section will be 2.6. It is anticipated that the average SPR calculated in this manner will be a fairly good representation of the terminal skid resistance of the pavement.

Figure 4.2 shows a plot of the field measured skid numbers (SN_{64}) and the corresponding VPPL for all the pavement test sections used in the current research study. Review of data shown in Figure 4.2 reveals that the manner in which the skid numbers decrease with increasing VPPL, changes significantly from one pavement section to another. Some pavements maintain the skid numbers at high values even after sustaining several million vehicle passes whereas others show very rapid deterioration in skid numbers. Therefore, it was clear that general trendlines such as those shown in Figure 4.1 could not be drawn on the combined plot. However, upon further review of the skid performance curves for different types of aggregates, it became apparent that the rate of deterioration of the field measured skid number is closely related to the mineralogical makeup of the aggregates used in the pavement surface course. Specifically it was noted that the carbonate aggregates with a low percentage of hard minerals demonstrated a very rapid decrease in the skid numbers measured. On the other hand, carbonate aggregates with significant hard mineral content and nOn-carbonate aggregates maintained better skid numbers. Therefore, it was decided to group the field data based on the mineralogical composition of aggregates and then redraw the SN_{64} versus VPPL plots to determine the proper SPR numbers for each pavement test section. The procedure of categorizing the aggregates based on their mineralogical composition is briefly discussed in the next section.

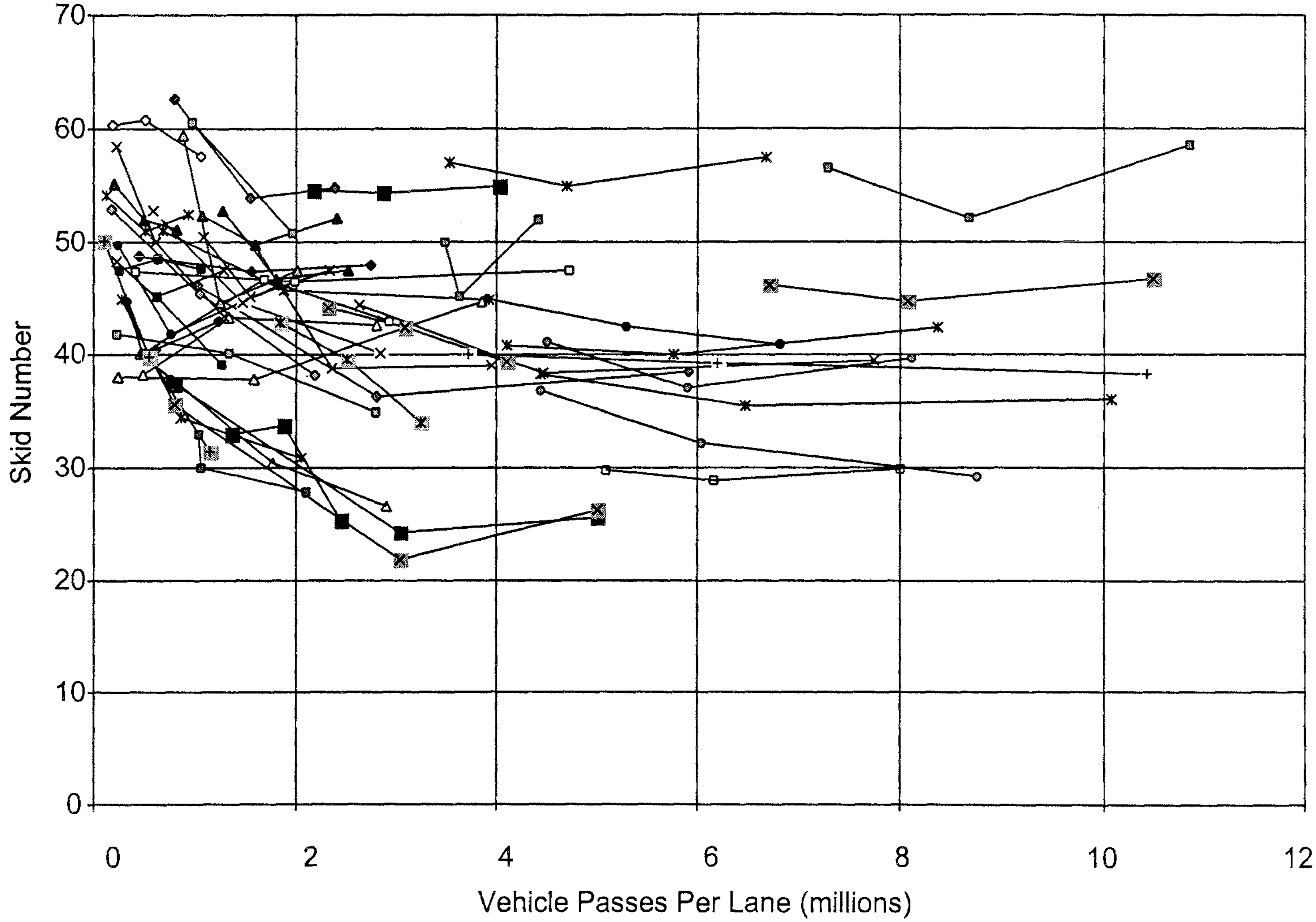


Figure 4.2. Skid Number Vs VPPL for All Test Sections

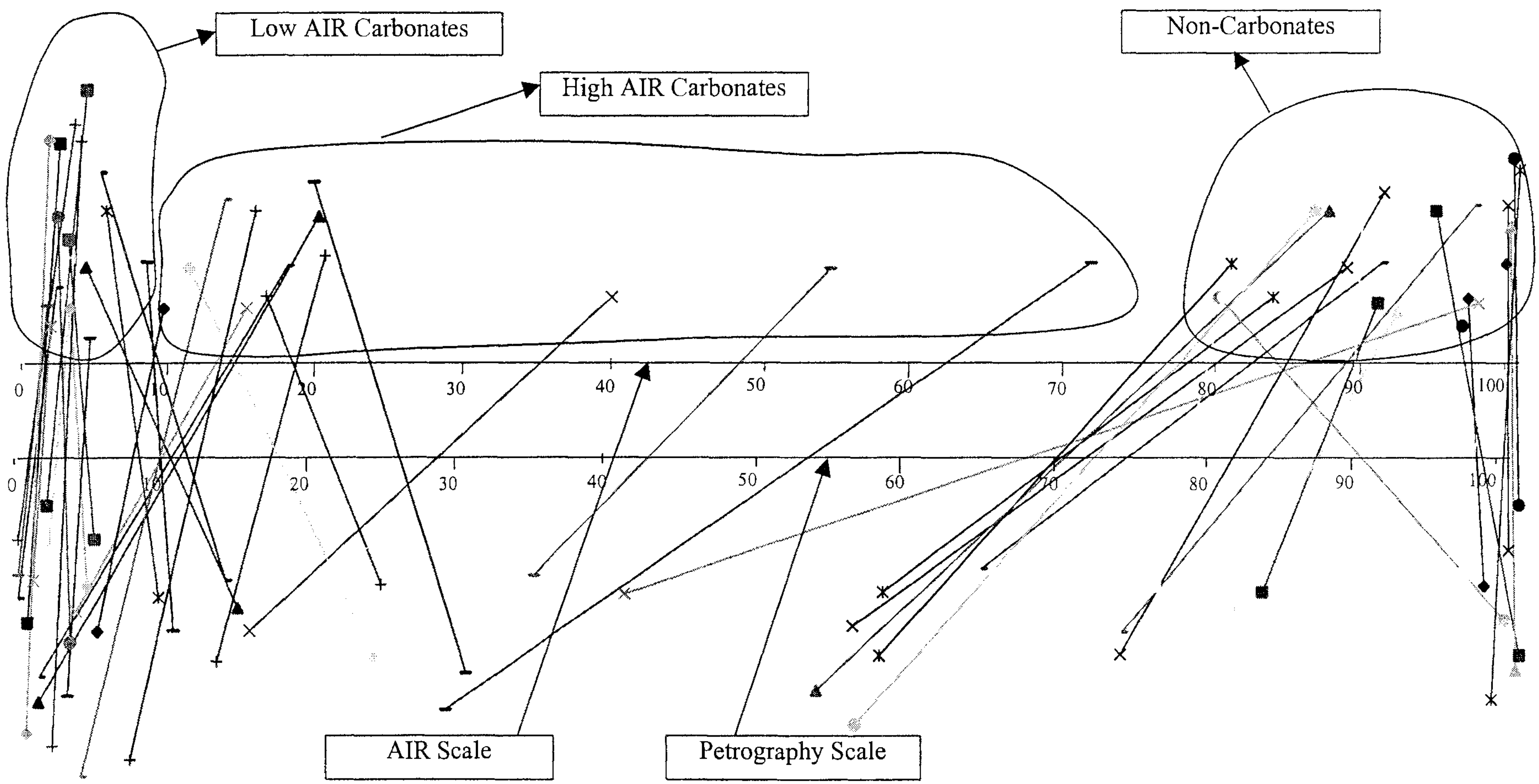


Figure 4.3 Plot of AIR and Non-Carbonate Mineral Content Used for Aggregate Classification

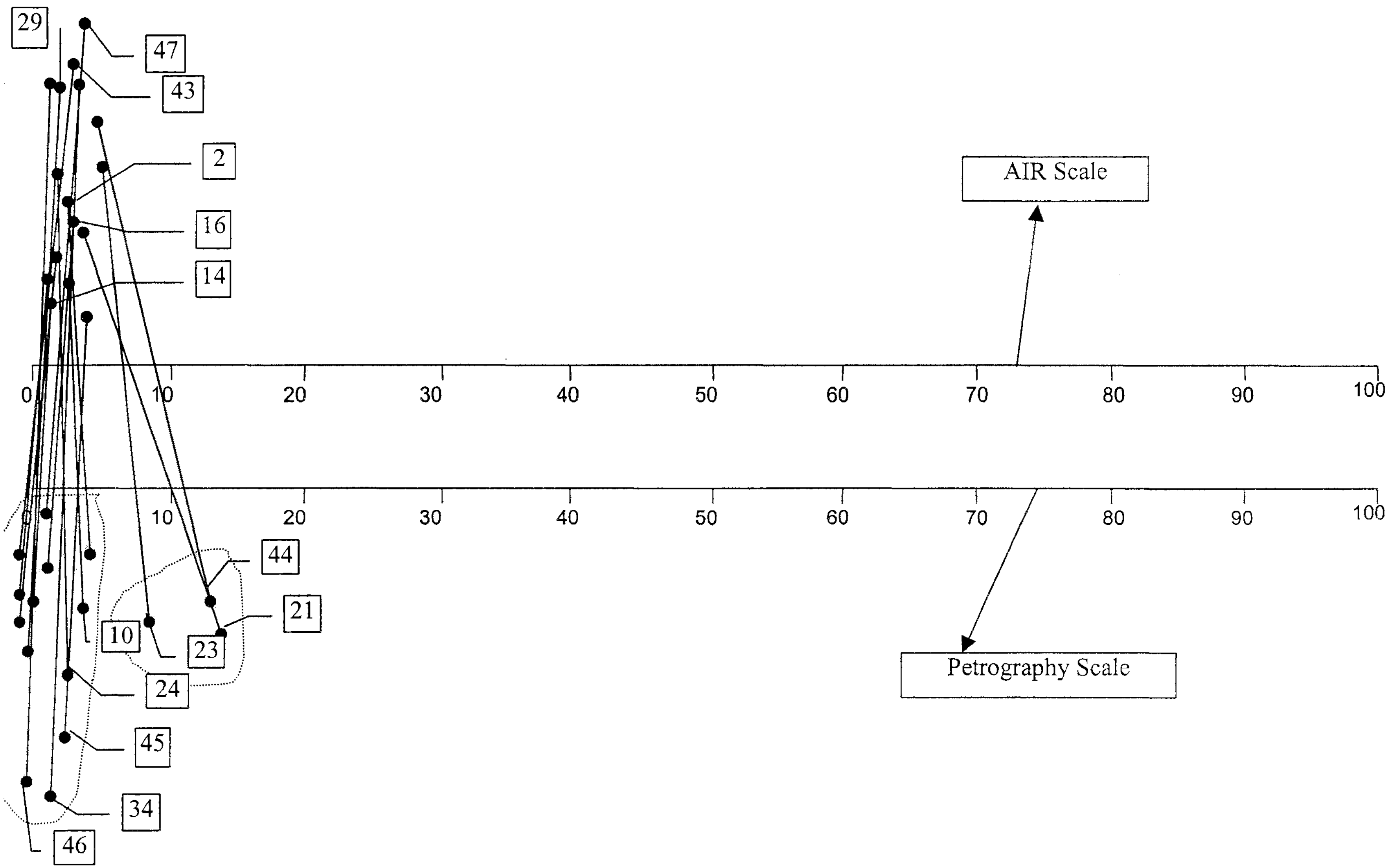


Figure 4.3(a) Plot of Low AIR Carbonates Along AIR and Petrography Scales

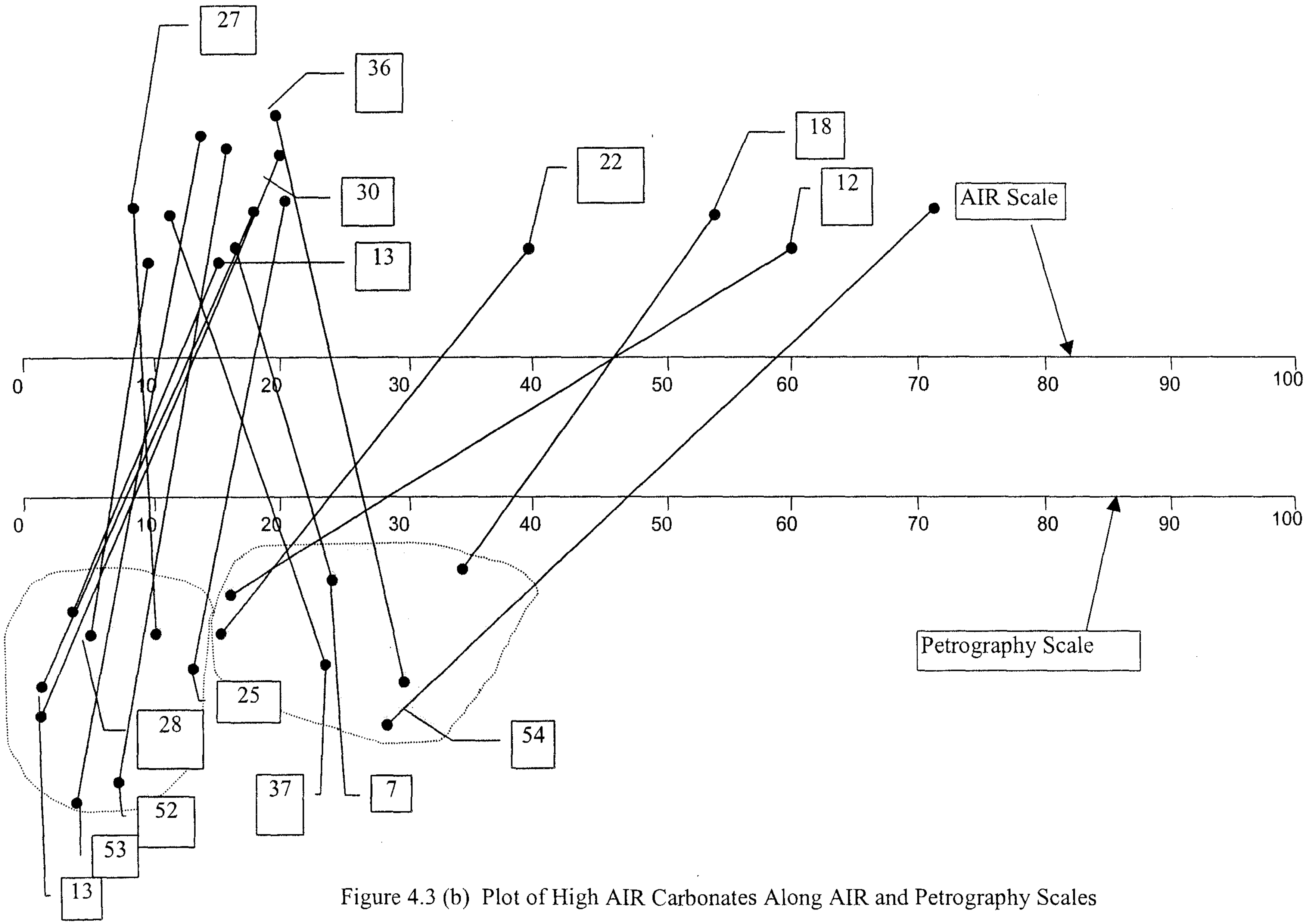


Figure 4.3 (b) Plot of High AIR Carbonates Along AIR and Petrography Scales

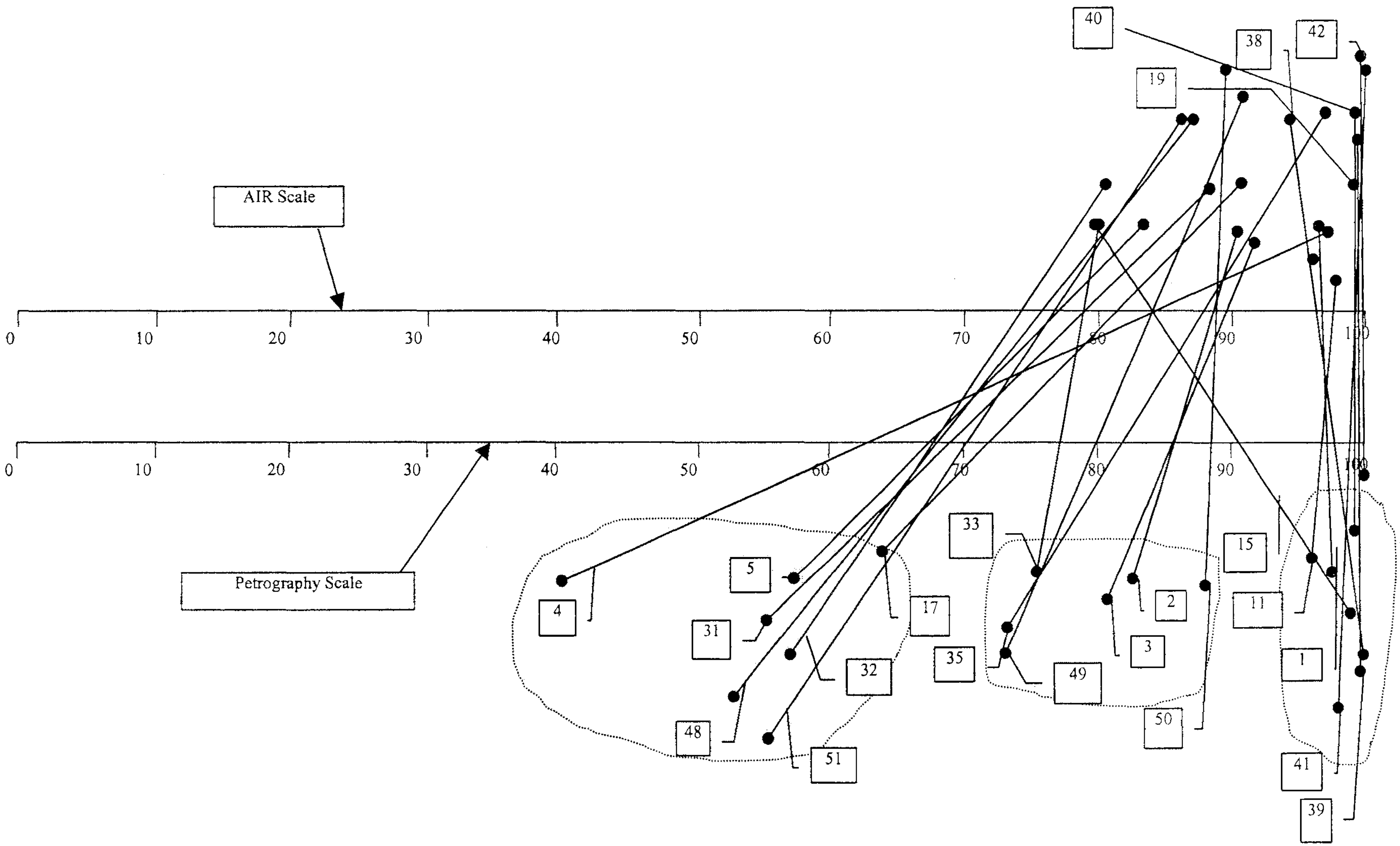


Figure 4.3(c) Plot of Non-Carbonates Along AIR and Petrography Scales

Table 4.2 Identification Labels Used for Pavement Test Section ID's in Figure 4.3

Label Number	Section ID
1	04IH00401
2	04SH01361
3	04SH01521
4	05FM22551
5	05US00841
6	06IH00201
7	06LP02501
8	08IH00201
9	08IH00202
10	08IH00203
11	10IH00201
12	10LP03231
13	10US00691
14	10US00791
15	11FM12751
16	11US00591
17	11US00592
18	11US00593
19	11US00594
20	12FM13011
21	12FM20041
22	12FM30051
23	12LP01971
24	12SH00361
25	14US02901
26	14US02902
27	15LP00131

Label Number	Section ID
28	15LP00132
29	15LP16041
30	15US02811
31	15US02812
32	16SH03591
33	16US01811
34	16US01812
35	16US02811
36	18IH00451
37	18IH035E1
38	18US01751
39	19SH00081
40	19US00591
41	19US02711
42	19US02712
43	20FM01051
44	20FM03651
45	20SH00871
46	20SH03211
47	20US00901
48	21SH00041
49	21SH01001
50	21SP04871
51	21US02811
52	24FM06591
53	24LP03751
54	24SH00201

4.2.1 Low AIR Carbonates

The aggregates which fall into this category have the following label numbers: 8, 9, 10, 14, 16, 20, 21, 23, 24, 29, 34, 43, 44, 45, 46, and 47. The insoluble AIR portion of all these aggregates is below 8% and their position along the Petrography scale also indicates that they have a very low percentage of non-carbonate mineral content. The low AIR and non-carbonate minerals content indicate that aggregates in this group are pure carbonates. However, it can also be observed that aggregates 21,23 and 44 have slightly greater amounts of non-carbonate mineral content than the rest of the aggregates in the group. Nevertheless, these three aggregates are essentially carbonates with a low AIR value. In order to incorporate the difference due to varying amounts of non-carbonate mineral content, the carbonate aggregates of this group were sub-divided into two smaller groups. The first group consists of carbonate aggregates with AIR and non-carbonate mineral content values below 8%. The second group consists of carbonate aggregates with AIR values less than 8% but the non-carbonate mineral content values were slightly greater than 8%. The difference between these two sub-groups was later incorporated in statistical regression analysis by using representative dummy variables.

4.2.2 High AIR Carbonates

The aggregates which fall into this category have the following label numbers: 7, 12, 13, 18, 22, 25, 26, 27, 28, 30, 36, 37, 52, 53,and 54. The insoluble AIR portion of all these aggregates is above 8% and their position along the Petrography scale clearly indicates that they have a low percentage of non-carbonate mineral content. Indicating that aggregates in this group are carbonates with slightly higher AIR values, than the aggregates of the previous group. However, it can also be observed that aggregates of this group have varying amounts of non-carbonate mineral content. The non-carbonate mineral content of these aggregates varies from below 8% to 35%. This varying non-carbonate mineral content could be due to the presence of non-carbonate minerals such as silica and quartz. However, this variation can be advantageously used in further subdivision of aggregates in this category. Further inspection of Figure 4.3(b) suggests that the High AIR Carbonates can be divided into three subgroups based on the non-carbonate mineral content. The first sub-group consists of aggregates with AIR values greater than 8%, but the non-carbonate mineral content is less than 8%. The second sub-group consists of aggregates with AIR greater than 8% and the non-carbonate mineral content ranges from 8 to 20%. The third sub-group consists of aggregates with AIR values greater than 8% and the non-carbonate mineral content ranges between 20 to 40%. The difference between the three sub-groups was incorporated into statistical regression analysis using representative dummy variables.

4.2.3 Non-Carbonates

The aggregates which fall into this category have the following label numbers: 1, 2, 3, 4, 5, 6, 11, 15, 17, 19, 31, 32, 33, 35, 38, 39, 40, 41, 42, 48, 49, 50, and 51. The insoluble AIR portion of all these aggregates is well above 80% and their position along the Petrography scale clearly indicates that they have a high percentage of non-carbonate mineral content. Indicating that aggregates in this group are non-carbonates with very high AIR values. However, it can also be observed that aggregates of this group have varying amounts of non-carbonate mineral content. The non-carbonate mineral content of these aggregates varies from below 40% to 100%. This varying non-carbonate mineral content could be due to the presence of carbonate minerals such as calcite and dolomite. However, this variation can be advantageously used in further subdivision of aggregates in this category. Upon further inspection of Figure 4.3(c), the non-carbonate aggregates were sub-divided into three smaller groups. The first groups consists of aggregates with AIR values above 80%, but the non-carbonate mineral content ranges between 40 to 70% in them. The second group consists of aggregates with AIR value above 80% and the non-carbonate mineral content ranges from 70 to 95%. The third group consists of aggregates with very high AIR values above 90%, with the non-carbonate mineral content ranging between 95 and 100%. Again, representative dummy variables were used to incorporate the difference between the three sub-groups in statistical regression analysis.

After the classification of aggregates into the three main categories discussed in this section, the SN_{64} versus VPPL plots were redrawn for the pavement test sections. The purpose of redrawing the SN_{64} versus VPPL plots was to assign the proper SPR numbers to the aggregates used in the construction of these pavement test sections.

4.3 SPR EVALUATION OF PAVEMENT TEST SECTION FOR DIFFERENT AGGREGATE TYPES

SPR numbers were evaluated for pavement test sections from SN_{64} versus VPPL plots which were redrawn after the aggregates were categorized as discussed in section 4.2. Figures 4.4, 4.5, and 4.6 show the SN_{64} and VPPL plots for the pavement test sections. It can be seen from the SN64 versus VPPL plots that the SPR curves have different pattern from one plot to the other. In Figure 4.4, the SPR curves have steep initial drops as the low AIR carbonate aggregates exhibit a faster drop in field skid number with increasing VPPL. The drop in the initial portion of the SPR curves is not so steep for the high AIR carbonates. The non-carbonate aggregate maintain higher field skid numbers as compared to the aggregates of the other two groups, hence, the SPR curves for the non-carbonates are much shallower. The SPR numbers which were calculated from the above mentioned plots and the corresponding results of laboratory tests on aggregates are provided in Tables 4.3, 4.4, and 4.5. The statistical regression analysis was performed using the data provided in the above mentioned tables.

4.4 STATISTICAL REGRESSION ANALYSIS

The statistical regression techniques used to analyze the data are described in this section. The results obtained from these regression are also presented individually for the three groups of aggregates described earlier.

4.4.1 Statistical Methods

The Statistical Analysis System (SAS v 6.1.2) available on the VAX/VMS mainframe system of Texas Tech University was used to successfully complete the statistical analysis involved in this research study. In order to perform the regression analysis, the SPR values assigned to each pavement test section in section 4.3 were

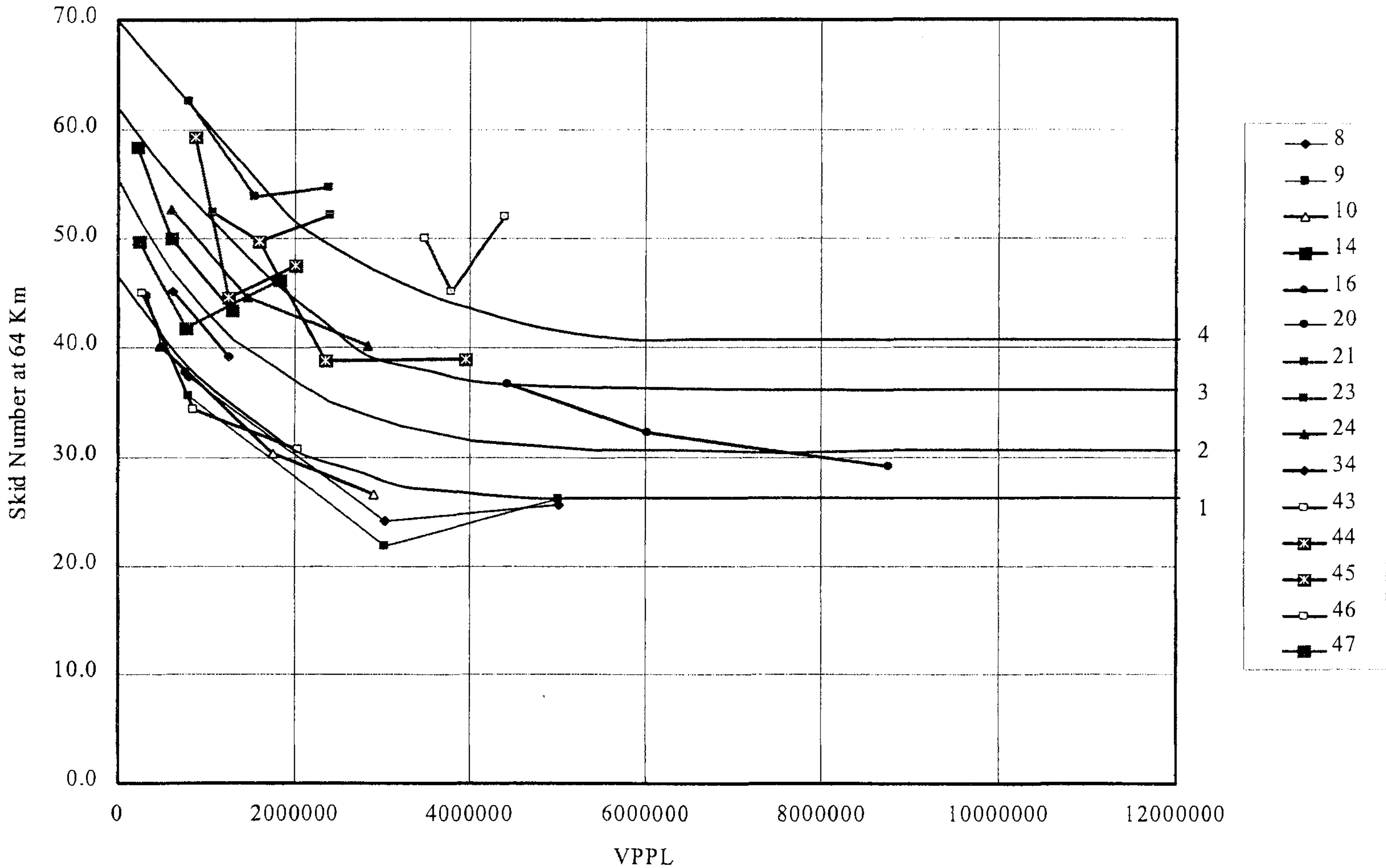


Figure 4.4 Skid Number at 64 Kmph vs VPPL for Low AIR Carbonates

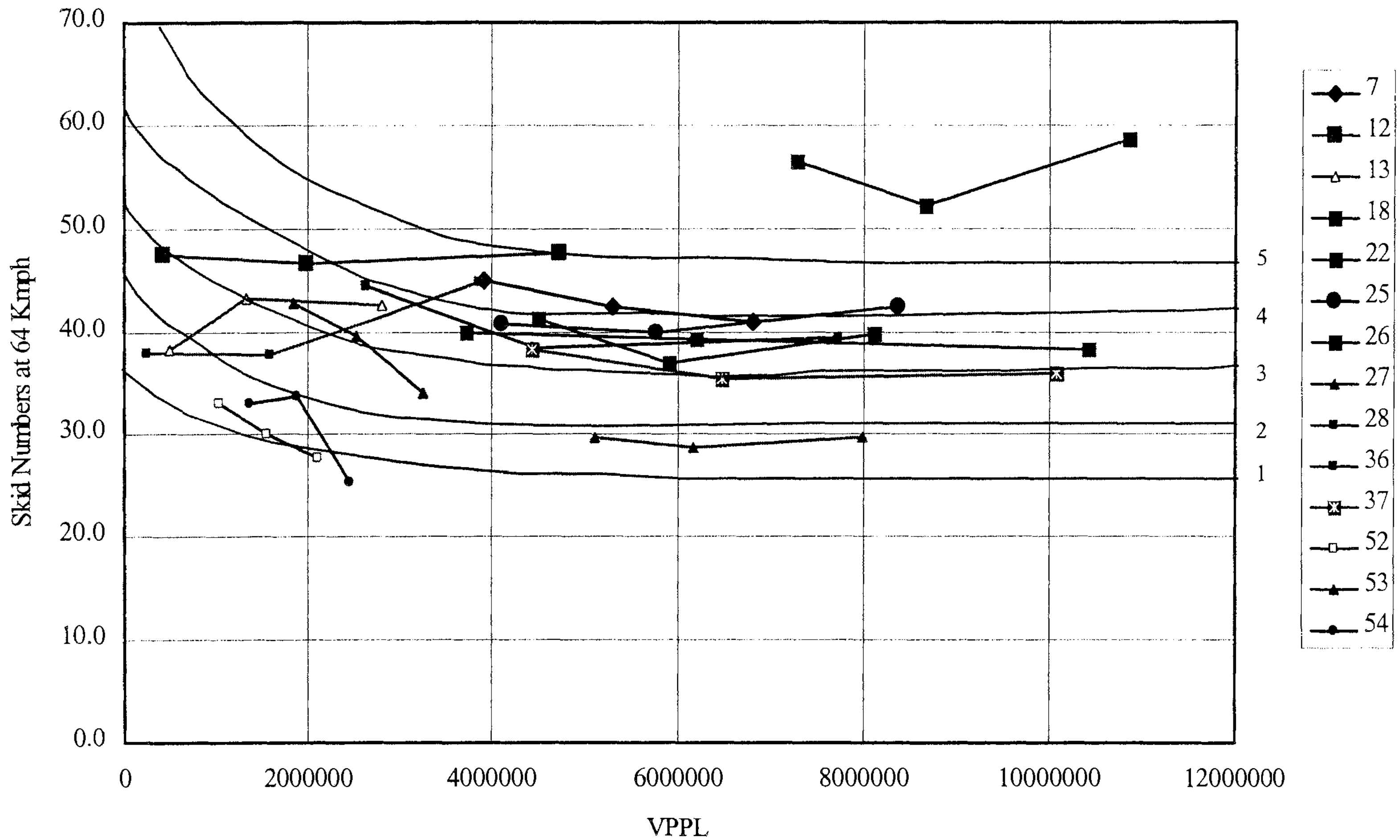


Figure 4.5 Skid Number at 64 Kmph vs VPPL for High Air Carbonates

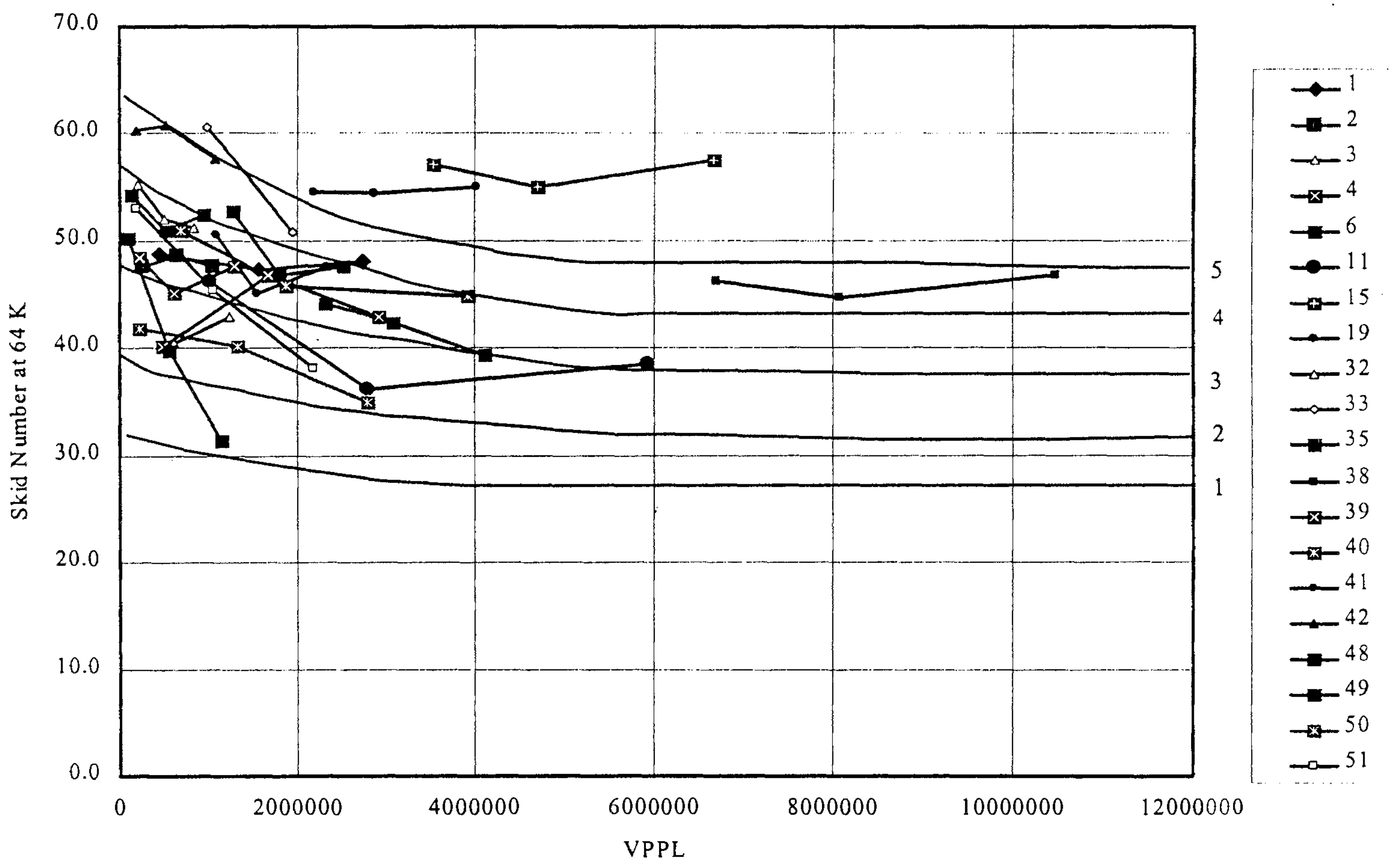


Figure 4.6 Skid Number at 64 Kmph vs VPPL for Non-Carbonates

Table 4.3 Data of Low AIR Carbonates used for Statistical Analysis

Label Number	Section ID	SPR	NCRB	AIR	Dummy Variable	RSPV	RSSM	RSLA	Macrotexture	Rainfall
8	08IH00201	0.7	0	2.6	0	33	41	32	32.8	24.4
9	08IH00202	0.533	0	2	0	33	41	32	21	24.4
10	08IH00203	0.733	4.5	3.51	0	33	32	28	27.8	24.4
14	10US00791	2.43	1	2.22	0	30	25	29	30.1	39.94
16	11US00591	1.95	2	3.8	0	30	17	30	30.9	42.4
20	12FM13011	0.8	5	3.4	0	32	8	31	20.5	46.07
21	12FM20041	4	14.43	4.5	1	36	29	35	29.9	46.07
23	12LP01971	3.5	9.25	5.86	1	35	13	33	28.7	42.28
24	12SH00361	2.72	3.4	2.7	0	32	20	34	23.6	46.07
29	15LP16041	0.5	0.6	2.85	0	26	1	20	28	30.98
34	16US01812	1.75	2.2	4.22	0	32	20	34	14.5	30.13
43	20FM01051	4.32	0	3.82	0	35	29	35	35.1	57
44	20FM03651	3.13	13.65	5.5	1	35	22	35	29	57
45	20SH00871	2.87	3.2	4.77	0	35	29	35	26.5	58.93
46	20SH03211	0.783	0.51	2.15	0	32	20	34	28.7	54.11
47	20US00901	2.02	1.9	4.6	0	36	29	35	27.4	54.11

Table 4.4 Data of High AIR Carbonates used for Statistical Analysis

Label Number	Section ID	SPR	NCRB	AIR	Dummy Variables		RSPV	RSSM	RSLA	Macrotexture	Rainfall
					DV1	DV2					
7	06LP02501	3.85	24.05	16.4	0	1	40	25	26	43.4	14.66
12	10LP03231	3.65	16	60.1	1	0	37	15	30	47	39.94
13	10US00691	2.38	3.76	15.1	0	0	33	39	33	30.7	39.94
18	11US00593	3.12	34.35	54.09	0	1	37	12	27	23.3	42.4
22	12FM30051	5.63	15.24	39.52	1	0	44	37	29	43.4	42.28
25	14US02901	3.57	13.05	20.32	1	0	.	.	.	26.7	31.88
26	14US02902	3.32	1.36	17.86	0	0	44	18	26	38.5	31.88
27	15LP00131	1.3	10.2	8.53	1	0	31	6	29	20	30.98
28	15LP00132	2.4	5.18	9.65	0	0	32	27	34	45.2	30.98
30	15US02811	2.57	1.3	19.9	0	0	33	6	10	24.5	30.98
36	18IH00451	3.25	29.78	19.57	0	1	41	11	31	34.6	36.08
37	18IH035E1	2.82	23.5	11.33	0	1	49	3	25	31.2	36.08
52	24FM06591	0.53	7.3	15.7	0	0	29	5	31	37.8	10.87
53	24LP03751	2.4	4.04	13.7	0	0	28	3	21	31.9	10.87
54	24SH00201	0.5	28.4	71.3	0	1	33	4	22	24.1	10.87

Table 4.5 Data of Non-Carbonates used for Statistical Analysis

Label Number	Section ID	SPR	NCRB	AIR	Dummy Variables		RSPV	RSSM	RSLA	Macrotexture	Rainfall
					DV1	DV2					
1	04IH00401	3.93	97.6	96.5	0	1	31	12	32	33.4	19.56
2	04SH01361	3.6	82.7	90.4	1	0	35	10	31	30.6	19.56
3	04SH01521	4.03	80.81	91.7	1	0	32	10	30	29.3	19.56
4	05FM22551	3.25	40.36	97.2	0	0	36	11	15	40.4	18.65
5	05US00841	4.67	57.56	83.4	0	0	31	10	27	33	18.65
6	06IH00201	4.07	.	96.1	0	1	37	15	20	47.7	14.66
11	10IH00201	2.93	96.08	97.8	0	1	25	6	29	29.7	39.94
15	11FM12751	5.83	99	79.8	0	1	49	2	19	67.6	41.8
17	11US00592	3.42	64.06	90.69	0	0	.	.	.	30	42.4
19	11US00594	5.3	.	99.1	0	1	49	2	19	53.7	41.8
31	15US02812	4.1	55.55	88.32	0	0	33	9.9	31	36.5	30.98
32	16SH03591	2.78	57.3	80.6	0	0	28	2	17	16	30.13
33	16US01811	5	75.55	80.1	1	0	32	13	33	15	30.13
35	16US02811	4.23	73.37	96.97	1	0	26	2	17	27.9	30.13
38	18US01751	4.56	100	94.31	0	1	39	2	21	32.6	30.14

Table 4.5(Continued) Data of Non-Carbonates used for Statistical Analysis

Label Number	Section ID	SPR	NCRB	AIR	Dummy Variables		RSPV	RSSM	RSLA	Macrotexture	Rainfall
					DV1	DV2					
39	19SH00081	3.48	99.75	99.4	0	1	34	8	22	44.3	47.01
40	19US00591	3.95	99.3	99.2	0	1	34	8	22	38.4	47.01
41	19US02711	4.13	98.1	100	0	1	36	4	24	27.6	47.01
42	19US02712	4.95	100	99.6	0	1	37	11	28	26.6	47.01
48	21SH00041	2.33	53.13	87.1	0	0	28	16	25	25	26.61
49	21SH01001	3.33	73.25	90.8	1	0	26	6	18	33	26.61
50	21SP04871	2.43	88.1	89.51	1	0	30	21	25	30.8	23.4
51	21US02811	3.3	55.69	86.2	0	0	30	21	25	52.7	23.4

considered as the dependent variable. The aim of the statistical analysis was to come up with a regression expression that utilizes the results of laboratory tests conducted on aggregates to reliably predict the SPR value of the aggregate. In such a case, the results of laboratory test on aggregates can be considered as independent variables.

Of the several procedures available for regression analysis, PROC REG (PRG) and PROC STEPWISE (PRS) procedures available in SAS were used in this analysis. The first procedure, PRG, can be used to perform a regression on the dependent variable using a specified list of independent variables. In other words, the independent variables to be used in a regression expression have to be specified in the SAS analysis. In order to obtain a regression model with good predictive capabilities, it requires that the PRG procedure has to be used repeatedly for a large number of combinations of the independent variables. The number of independent variables to be included in a regression model becomes very complex when it is suspected that there exists an interaction between the independent variables. Hence, PRG procedure was used only in the initial stages of the statistical analysis.

The second procedure, PROC STEPWISE (PRS), can be used with two different options to perform statistical analysis. The first option is to use a FORWARD SUBSTITUTION (PRSF) procedure and the second option is to use a BACKWARD ELIMINATION (PRSB) procedure.

In the PRSF option, the stepwise regression procedure starts by scanning the list of independent variables and finding the variable that has the highest simple correlation with the dependent variable. The list of independent variables can include the original independent variables and any interaction terms among the independent variables. The independent variable with the highest simple correlation at the specified level of significance is included in regression model in the first step. Thereafter, another independent variable with the next highest simple correlation is concatenated to the regression model obtained in the first step. The procedure of including independent variables is continued until no other independent variable is found significant enough to be included in the regression model.

In the PRSB option, the stepwise procedure starts by incorporating all the specified independent variables in the regression model. The independent variable that has the least simple correlation with the dependent variable at the prescribed level of significance is removed from the regression model in the first step. The process of removing the least correlated independent variables one by one, from the regression model is continued until a stage where the independent variables that remain in the regression model are found to be highly significant.

Both the PRSF and PRSB procedure represent methods used by statisticians to find a subset of independent variables that are useful in reliably predicting the dependent variable. Since, the two procedures are different, they cannot be relied on to produce identical subsets of independent variables in the regression models (Conover, 1989). However, there will generally be a good agreement between the two subsets. Although, both of these procedures are attempting to identify only significant variables, there is no assurance that all important variables have been identified or that the models contain only important variables. In such situations, careful inspection of the regression models was performed, which resulted in either the acceptance or rejection of the regression model. The following section presents the results of the statistical regression analysis performed on the three different data sets.

4.4.2 Results of Statistical Regression Analysis on All Aggregates

The regression analysis was performed in two stages. In the first stage, the analysis was performed using the SPR numbers as the dependent variable and the results of the individual laboratory tests as the independent variable. The aim of this preliminary analysis is to determine the capability of each laboratory test to successfully predict the SPR numbers. The regression model used in this preliminary stage is given by Equation (4.1).

$$SPR = \alpha + \beta(\text{Independent Variable}) \tag{4.1}$$

where

α = Intercept

β = Coefficient for the independent variable

The independent variables used in this stage of regression analysis are RSPV, RSSM, RSLA, AIR, percentage non-carbonate content, macrotexture, and the average annual rainfall of the location where the pavement test section is constructed. Three different statistical procedures, namely, PRG, PRSF and PRSB were used to investigate the predictive capability of the laboratory tests at a level of significance of 0.05. The results of the three procedure are provided in Table 4.6.

Table 4.6 Preliminary Results of Statistical Regression Analysis

Independent Variable	R^2	P-value of the variable
RSPV	0.173	0.0026
RSSM	0.029	0.2322
RSLA	0.015	0.3871
AIR	0.298	0.0001
Non-Carbonate Content (NCRB)	0.346	0.0001
Macrotexture	0.163	0.0036
Ave. Annual Rainfall	0.035	0.1923
Results of PSRF		
Model R^2 = 0.5236, SPR = -1.648 + 0.112(RSPV) + 0.021(NCRB), P-values of the two independent variables was found to be 0.0001. Overall model P-value = 0.0001		
Results of PSRB		
Model R^2 = 0.5781, SPR = -2.743 + 0.112(RSPV) + 0.022(AIR) + 0.026(RAIN), P-values were found to be 0.0001 for RSPV and AIR, and 0.021 for the RAIN.		

From the information provided in Table 4.6 it can be inferred that even though the P-value is very less for some variables, the predictive capability of the independent variables is very poor. In PSRF procedure, RSPV and NCRB variables are found to be very significant (P-value = 0.0001) and were effectively entered into the regression model. No other independent variables were found to be significant enough to be included in the model. However, it must be pointed out that the P-value of the overall model is also very good, but the predictive capability as indicated by the low R^2 is very poor. In a similar way, the results of the PSRB procedure indicate that the predictive capability of the model is poor and it retains a different set of variables than the PSRF procedure. Due to the low predictive capability of the model shown above, it was decided not to accept the regression models. In order to pursue the regression analysis for better predictive models, it was decided to incorporate the differences between the aggregates into the analysis procedure and it forms the second stage of regression analysis.

4.4.3 Results of Statistical Regression Analysis on Grouped Aggregates

In this stage, the differences between the aggregates groups were incorporated by performing the regression analysis on the three individual groups of aggregates separately. The various sub-groups based on the petrography results were incorporated into the statistical analysis by making use of indicator dummy variables. These dummy variables take values of '0' or '1'. A value of 1 for a dummy variable indicates the occurrence of given aggregate in that particular sub-group represented by the dummy variable. Interaction between the independent variables was also incorporated into the analysis. As a first step, the analysis was performed using the SPR of the grouped aggregates and the RPSV values. Since, TxDOT uses RPSV values to qualify aggregates for use in pavement surface course, it was decided to use RSPV as the initial step towards better regression models. The regression model used is given by Equation 4.2.

$$SPR = \alpha + \beta(RSPV) \tag{4.2}$$

where

α = intercept

β = regression coefficient for the independent variable

The results of the above regression equation for the three different groups of aggregates are shown in Table 4.7.

Table 4.7 Regression Results for Grouped Aggregates Using Equation 4.2.

Aggregate Group	R^2	Equation 2 - SPR	P-Value	Overall P-Value
Low Air Carbonates	0.3251	= -7.110 + 0.279(RSPV)	0.0211	0.0211
High AIR Carbonates	0.4318	= -2.416 + 0.140(RSPV)	0.0107	0.0107
Non-Carbonates	0.5051	= 0.474 + 0.102(RSPV)	0.0002	0.0002

From the results provided in Table 4.7, it can be seen that the division of aggregates into three separate groups yields a better correlation as indicated by the R^2 values. In addition, the P-values of the independent variables and model are highly significant. However, the R^2 values have a low predictive capabilitiy and the use of RSPV alone is not able to explain the SPR values completely. The plots of actual versus predicted SPR using Equation 4.2 are shown in Figures 4.7, 4.8 and 4.9.

In order, to achieve a better correlation with the SPR values, it was decided to include differences between the sub-groups of aggregates as discussed in the previous section. Dummy variables were used to represent the various sub-groups of aggregates. PRSF and PRSB procedures were then used to regress the SPR values with the independent variables, the results of which are shown in Table 4.8.

The predicted values of SPR using the regression equation shown in Table 4.8 for the three groups of aggregates are plotted against the computed values of SPR and are shown in Figures 4.10, 4.11, 4.12. These plots show that equations in Table 4.8 show a slightly better predictive capabilities than using Equation 4.2 to predict the SPR numbers.

The next stage of the regression analysis involved the improvement of results obtained in Table 4.8. Other independent variables such as RSSM, RSLA, AIR, NCRB, Macrotexture, and Rainfall were included in the regression model at this stage. Dummy variables were also included in the list of independent variables to account for the sub-groups in the three major groups of aggregates. Appropriate interaction terms between

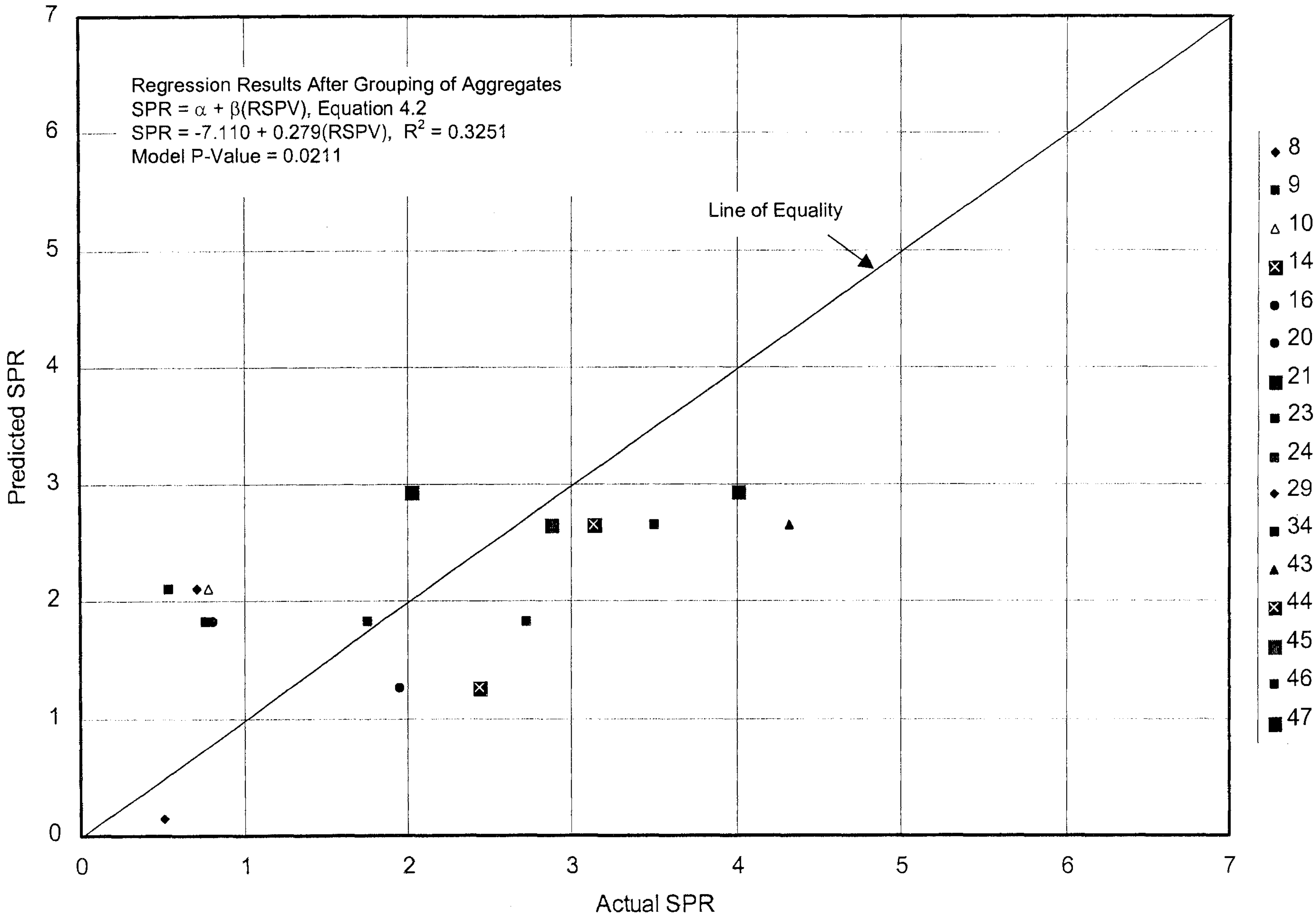


Figure 4.7 Predicted vs Actual SPR for Low Air Carbonates

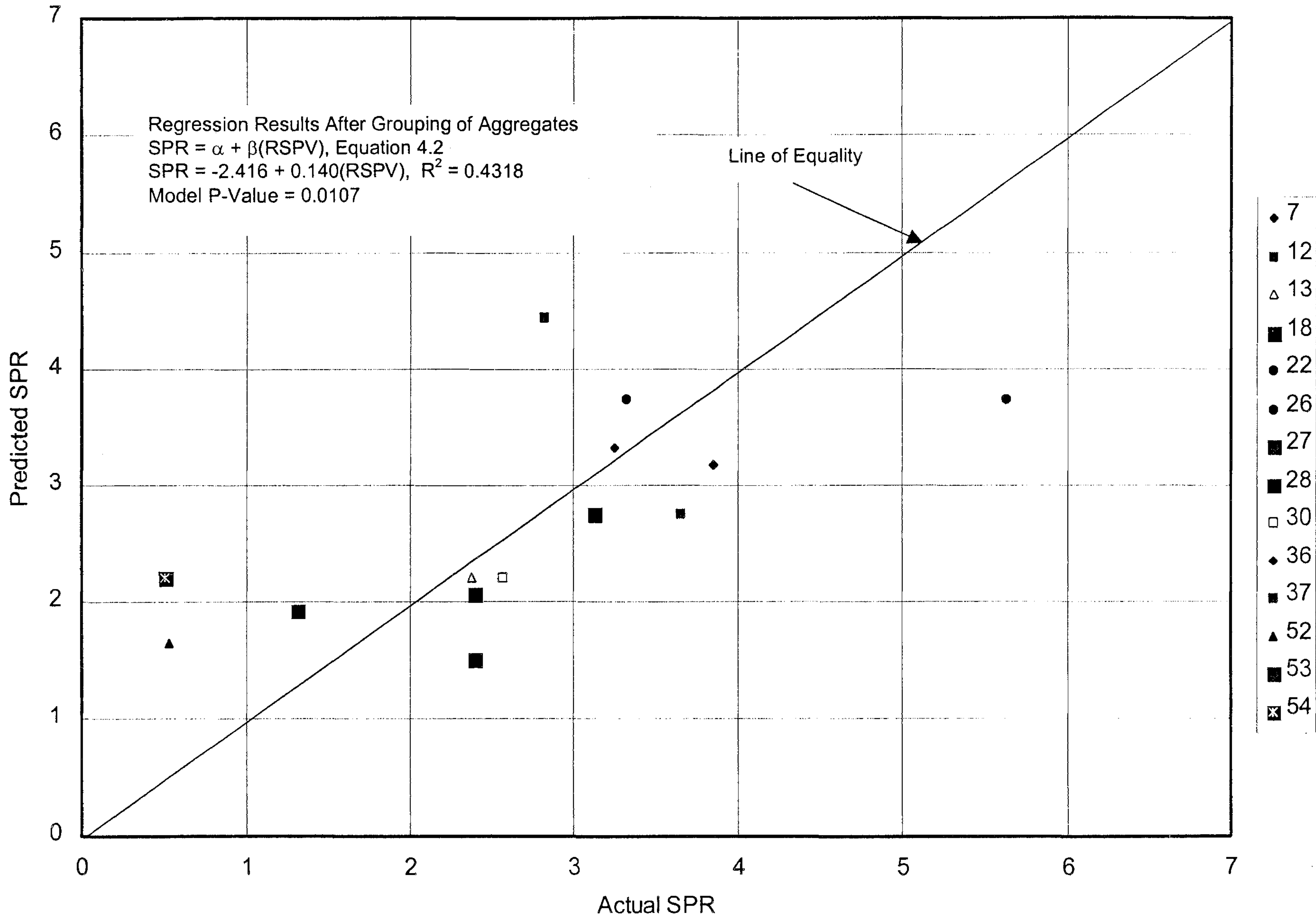


Figure 4.8 Predicted vs Actual SPR for High Air Carbonates

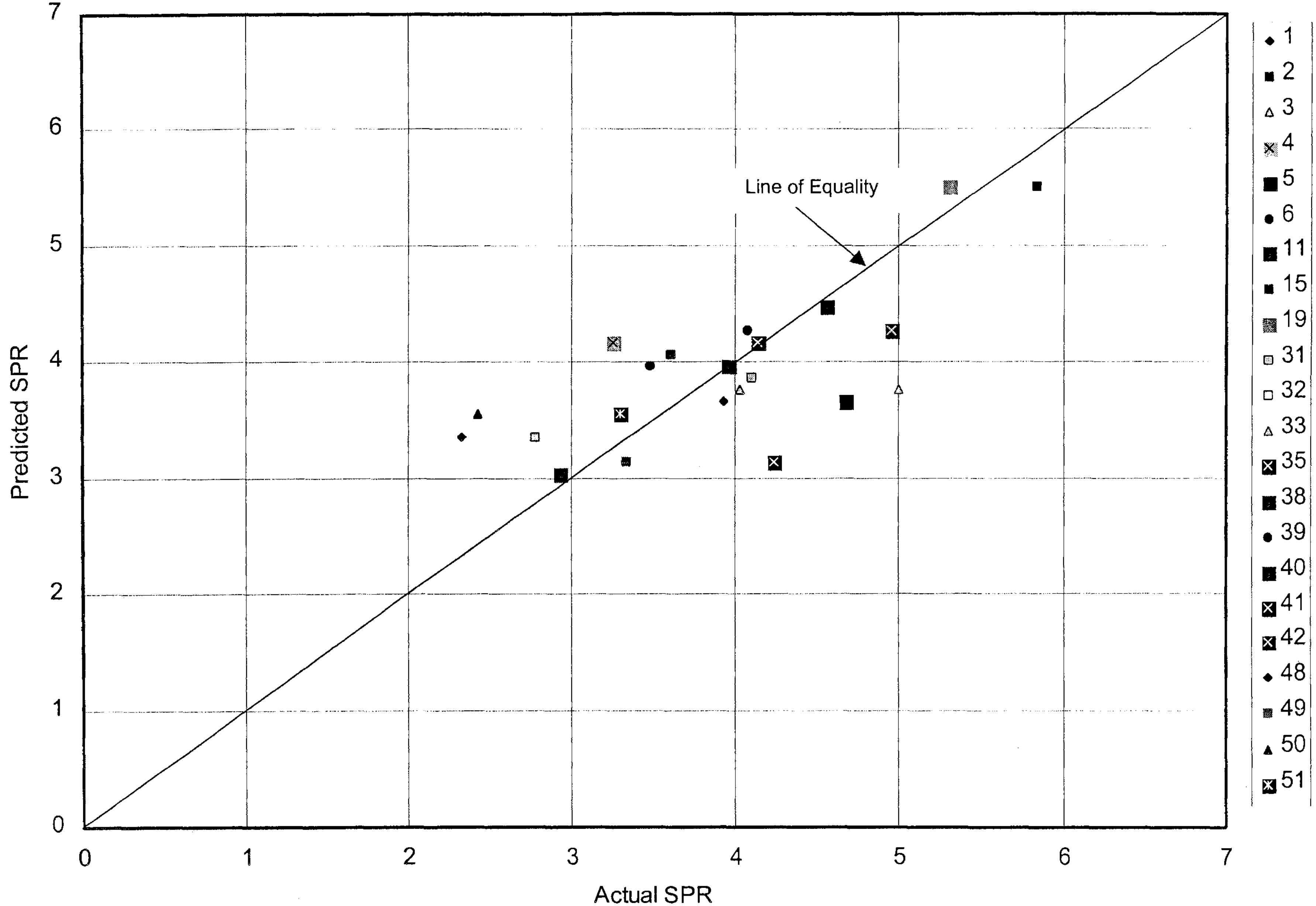


Figure 4.9 Predicted vs Actual SPR for Non Carbonates

Table 4.8 Results of Regression Analysis After Inclusion of Dummy Variables

Low AIR Carbonates

SPR = -4.226 + 0.184(RSPV) - 16.439(DV) + 0.501(RSPV * DV), with a $R^2 = 0.4516$,

P-values of

Overall model	=	0.0579
Intercept	=	0.2953
RSPV	=	0.1496
Dummy Variable (DV)	=	0.7295
Interaction term: RSPV * DV	=	0.7097

High AIR Carbonates

SPR = -1.345 + 0.109(RSPV) - 7.511(DV_1) - 0.452(DV_2) + 0.222(RSPV * DV_1) + 0.003(RSPV * DV_2) with a $R^2 = 0.4954$

P-values of

Overall Model	=	0.0548
Intercept	=	0.6069
RSPV	=	0.1837
Dummy Variable DV_1	=	0.1450
Dummy Variable DV2	=	0.9151
Interaction term, RSPV * DV1	=	0.1198
Interaction term, RSPV * DV2	=	0.9737

Non-Carbonate Aggregates

SPR = -0.3537 + 0.1212(RSPV) + 3.1699(DV_1) + 0.645(DV_2) - 0.0896(RSPV * DV_1) -0.0128(RSPV * DV_2) with a $R^2 = 0.5616$

P- values of

Overall Model	=	0.0138
Intercept	=	0.9108

Table 4.8 (Continued) Results of Regression Analysis After Inclusion of Dummy Variables

RSPV	=	0.2422
Dummy Variable DV_1	=	0.4460
Dummy Variable DV_2	=	0.8485
Interaction term, RSPV * DV1	=	0.5060
Interaction term, RSPV * DV2	=	0.9037

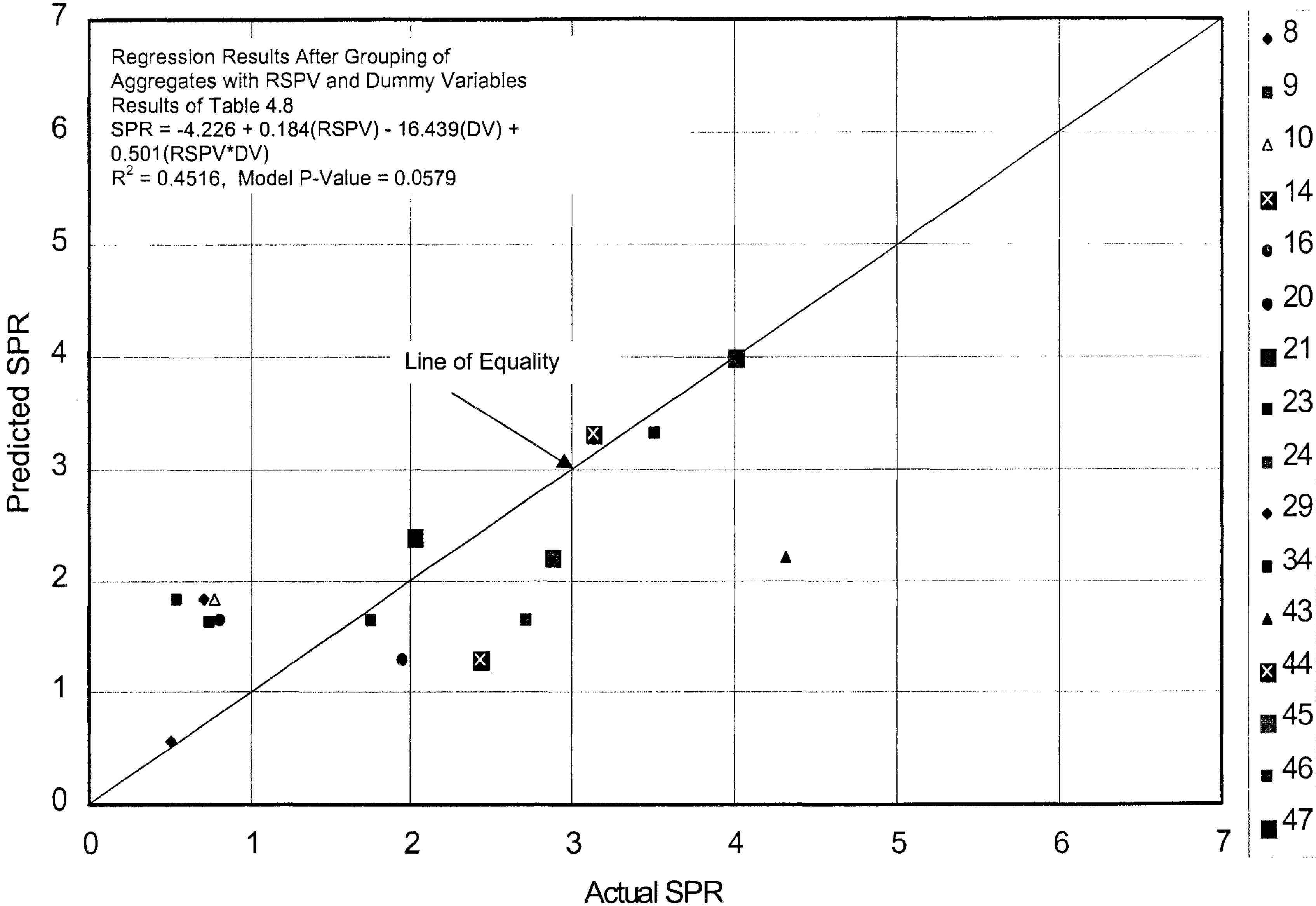


Figure 4.10 Predicted versus Actual SPR for Low AIR Carbonates

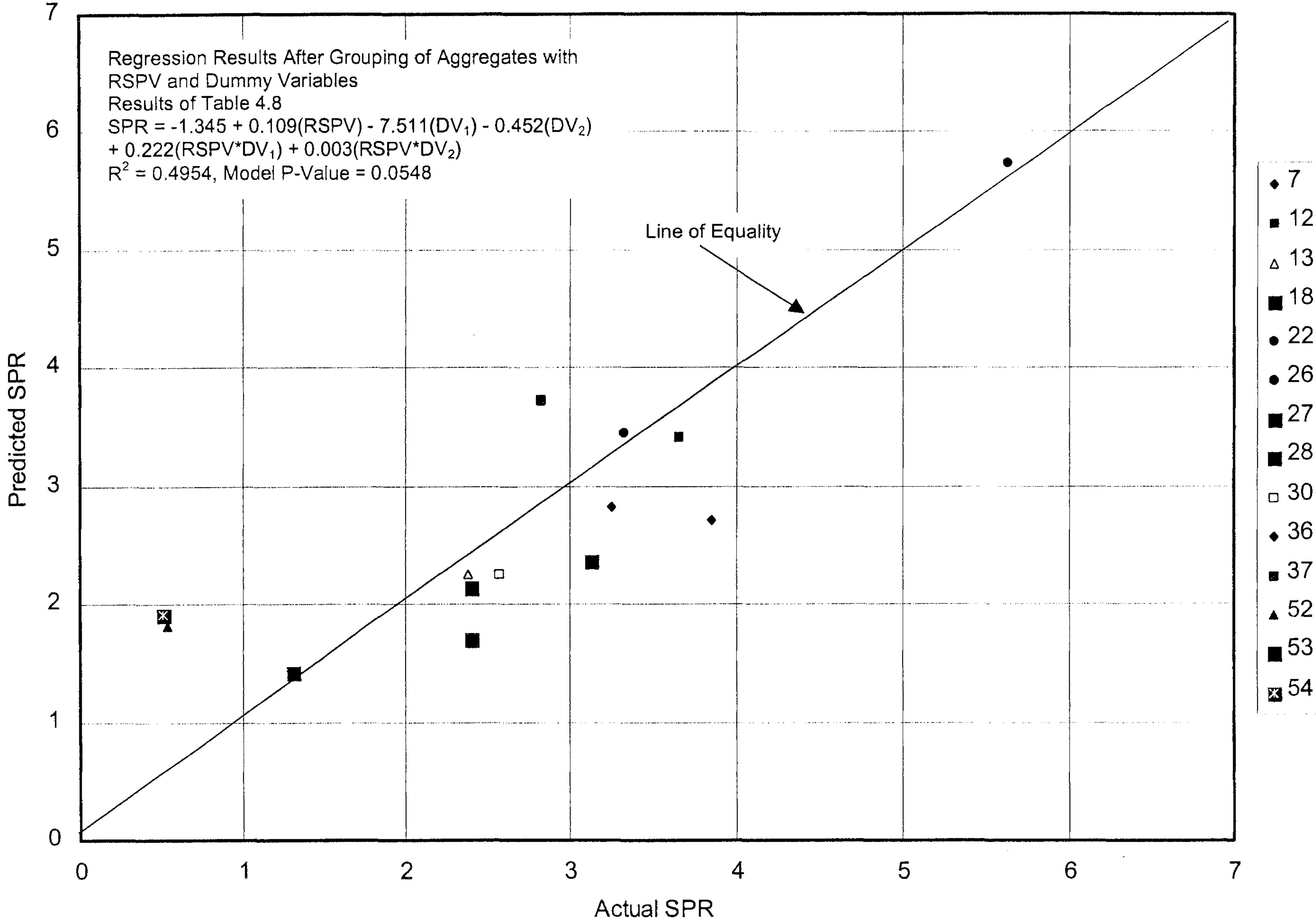


Figure 4.11 Predicted vs Actual SPR for High Air Carbonates

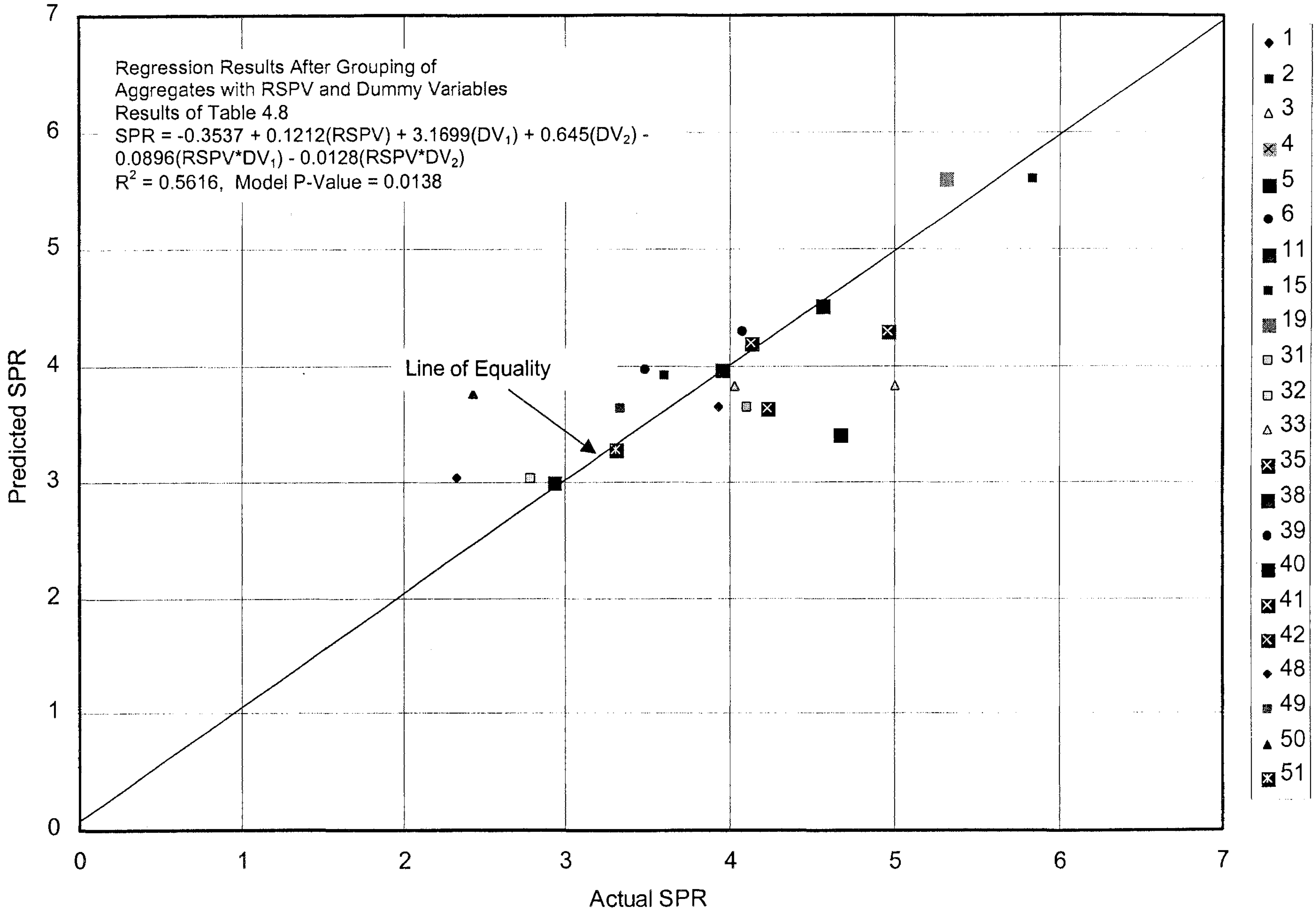


Figure 4.12 Predicted vs Actual SPR for Non Carbonates

the different laboratory test results and the dummy variables were also included. PSRF and PSRB procedure were then used to regress the SPR values and the independent variables, the results of which are shown in Table 4.9. The regression models presented in Table 4.9 show that the variables included in the model are significant in the case of High Air carbonates and the non-carbonates. The level of significance of the overall model shows a very good P-value. However, in the case of Low AIR carbonates a regression model better than the one shown in Table 4.9 was not possible. Figures 4.13, 4.14 and 4.15 show the plots of SPR values predicted using equations of Table 4.9 and the calculated SPR values in the previous section.

4.5 PREDICTION RELIABILITY

The reliability of the SPR predictions that we make using the regression models presented above depends on the coefficient of determination, R^2 of each model. If the model does not provide a close fit to the observed data, then the scatter in data around the *line of equality* will be larger, and the R^2 of the model will be lower. When such a model is used to make a prediction for a new aggregate source, there is greater probability (or likelihood) to get large deviations between the actual and predicted performances. According to information presented in Table 4.9, the R^2-values for the final regression models for low AIR carbonates, high AIR carbonates and non-carbonates are 0.577, 0.790 and 0.836 respectively. Accordingly, we will have greater confidence in the SPR predictions that we make for non-carbonates than for low AIR carbonates. However, SPR^{PRED} values alone (where SPR^{PRED}= SPR value predicted using the regression model) do not tell the user that one SPR^{PRED} is more reliable than the other. In addition, the user may desire to make an alternative SPR prediction based on the aggregate's historical skid performance. That prediction will also have some uncertainty associated with that depending on the degree of data scatter found in skid resistance vs. VPPL plot. If the reliabilities associated with the SPR prediction using past performance approach is very different from the SPR prediction made using lab data, no direct comparison between SPR values can be made.

To illustrate the need for a reliability based approach, let us assume that a low AIR carbonate aggregate and a non-carbonate are being considered for a particular pavement

Table 4.9 Regression Results Including RSPV and Other Laboratory Test Results

Low AIR Carbonates

$SPR = - 0.4941 + 0.0410(RSPV * DV) + 0.0535(RAIN)$ with a $R^2 = 0.5774$

P-values of

Overall Model	=	0.0037
Intercept	=	0.5655
Interaction term of RSPV and DV	=	0.0300
RAIN	=	0.0167

High AIR Carbonates

$SPR = 2.3404 - 0.3117(AIR) + 0.0088(RSPV * AIR)$ with a $R^2 = 0.79005$

P-values of overall model =0.0002, all the variables included in the regression model had a P-value of 0.0001, Suggesting that the model and the variables included in it are highly significant.

Non-Carbonates

$SPR = -11.1026 + 1.0624(RSPV) - 136.8592(DV_1) + 11.6609(DV_2) + 4.0443(RSPV * DV_1) - 0.9240(RSPV * DV_2) + 1.6443(AIR * DV_1) - 0.0068(RSPV * AIR) - 0.0487(RSPV * AIR * DV_1) + 0.0064(RSPV * AIR * DV_2)$ with a $R^2 = 0.8361$

P-values of

Intercept	=	0.0173
RSPV	=	0.0044
Dummy Variable DV_1	=	0.0313
Dummy Variable DV_2	=	0.0154
Interaction of RSPV and DV_1	=	0.0483
Interaction of RSPV and DV_2	=	0.0112
Interaction of AIR and DV_1	=	0.0187
Interaction of RSPV and AIR	=	0.0078
Interaction of RSPV, AIR and DV_1	=	0.0300
Interaction of RSPV, AIR and DV_2	=	0.0141

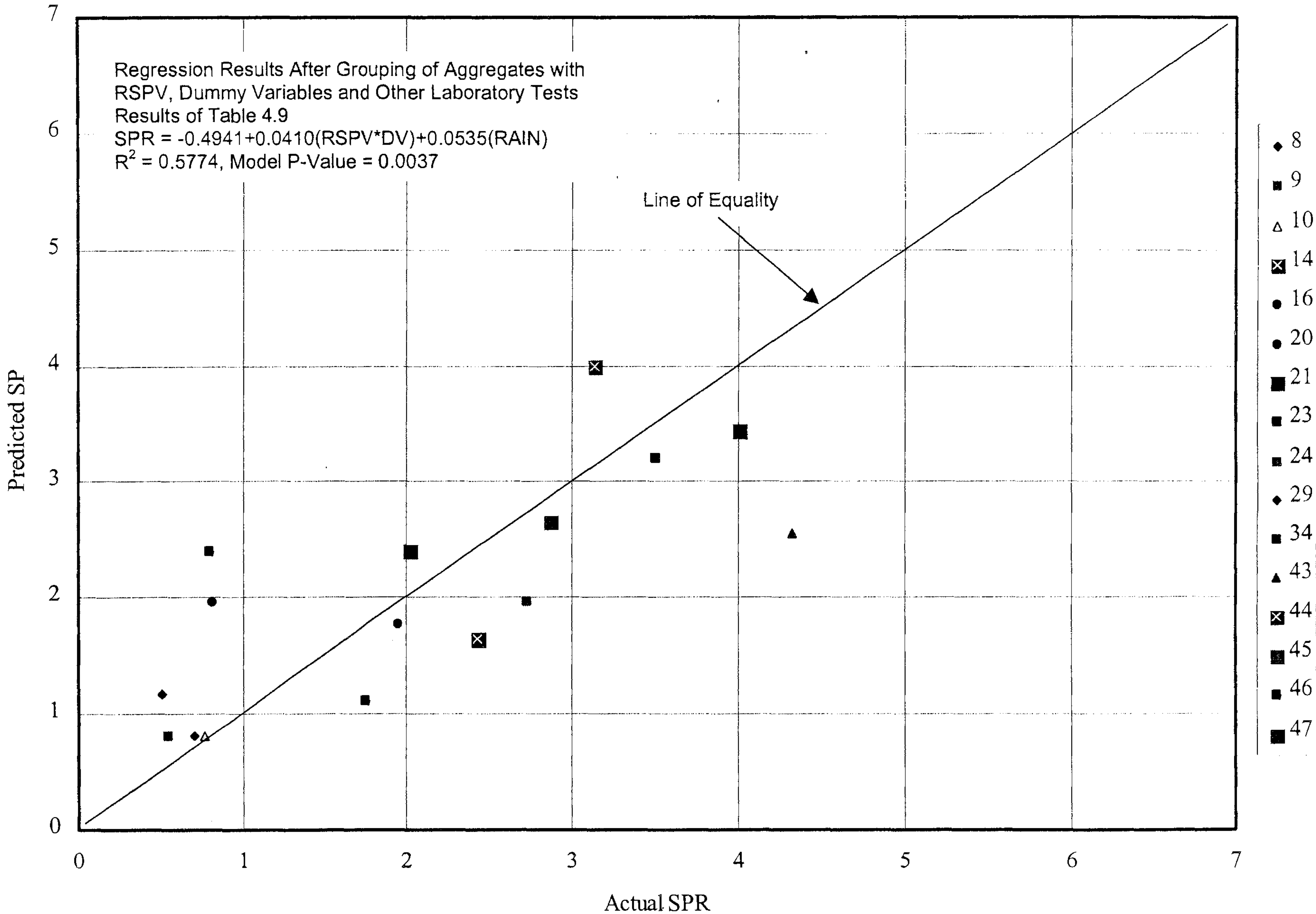


Figure 4.13 Predicted versus Actual SPR for Low AIR Carbonates

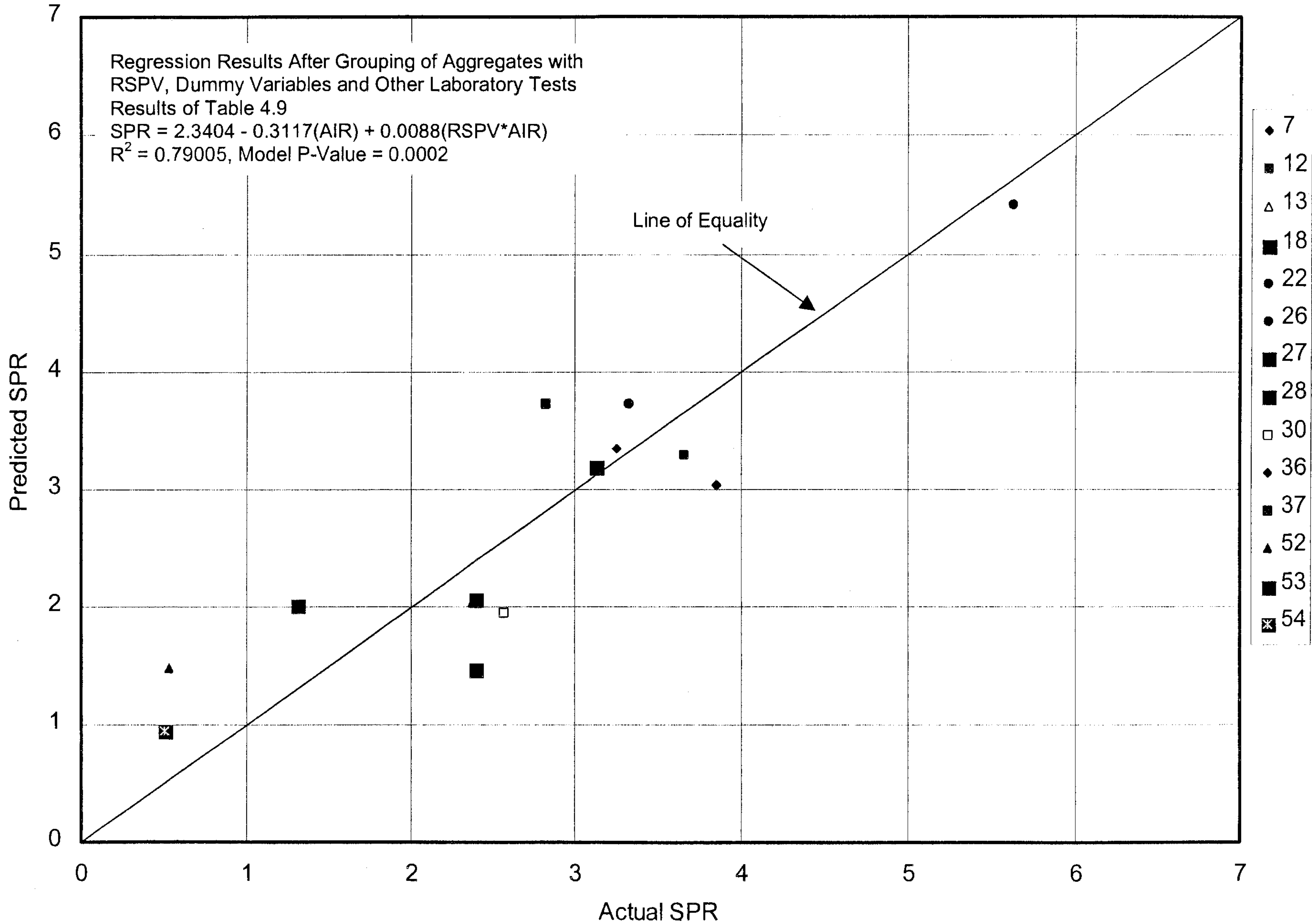


Figure 4.14 Predicted vs Actual SPR for High AIR Carbonates

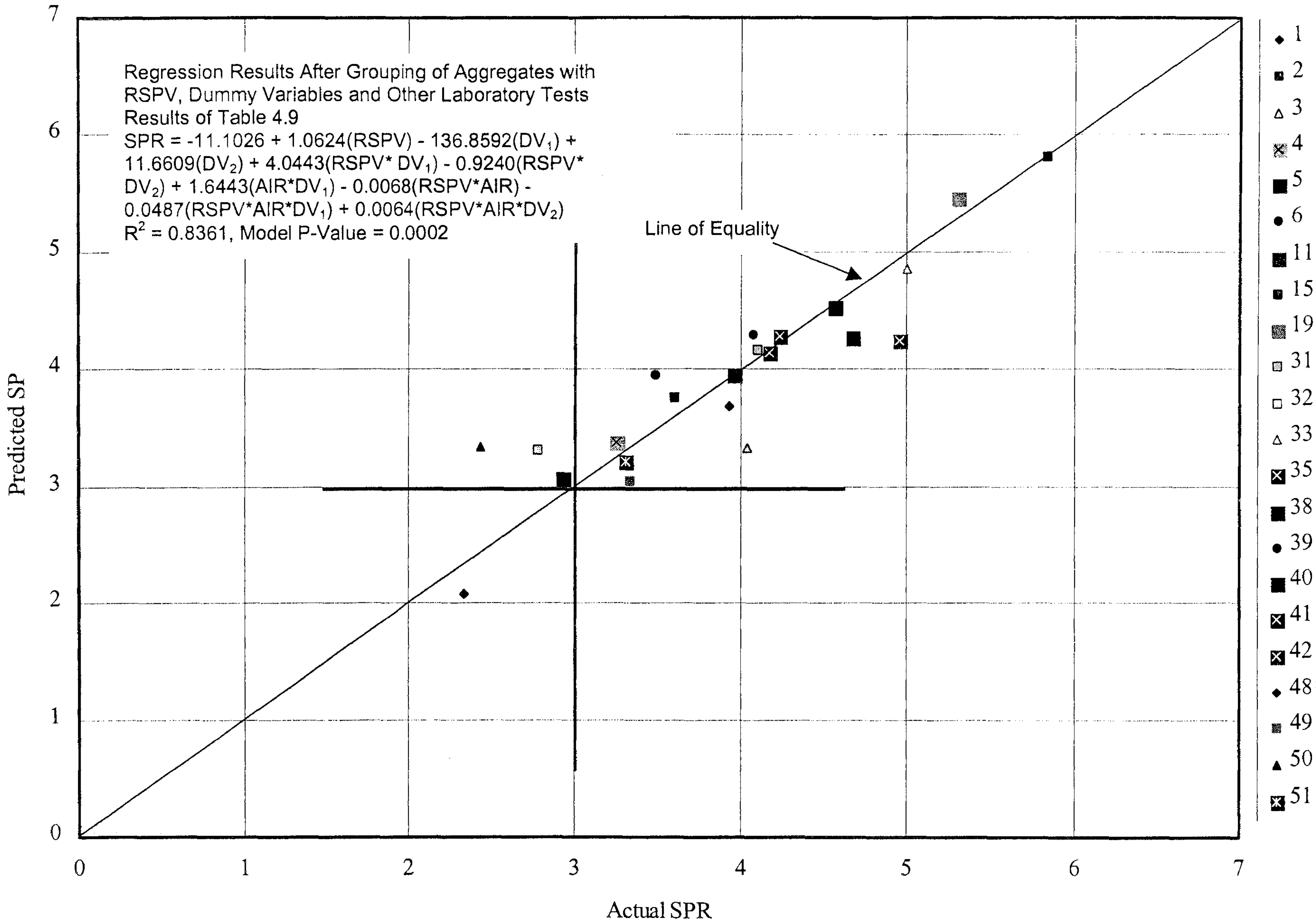


Figure 4.15 Predicted vs Actual SPR for Non Carbonates

construction project. On each aggregate, the necessary laboratory data as well as past skid performance data are available. Therefore, the expected skid performance rating of each of the two aggregates can be determined using two different methods: (a) based on laboratory data, (b) based on past skid resistance data. To account for the uncertainties associated with SPR predictions, the following approach is proposed in this research uses.

The SPR^{PRED} (i.e. SPR obtained from the regression model) has a 50% reliability. In other words, there is a 50% probability that the actual skid performance will be better than predicted. Also there is a 50% probability that the actual skid performance will be worse than predicted. As an alternative, we can determine the SPR value corresponding to a higher level of reliability desired by the engineer, say 90%. This value, denoted by SPR_{90}^{PRED}, will be smaller than SPR^{PRED} in magnitude. The probability that the actual performance will be better than SPR_{90}^{PRED} is now 90% and the probability that the actual performance will be worse than SPR_{90}^{PRED} is 10%. If the data scatter in the model is high then SPR_{90}^{PRED} will be much smaller than SPR^{PRED}. If the data scatter is small then SPR_{90}^{PRED} will still be smaller than SPR^{PRED} but will be close to it. If the model was a perfect fit, then $SPR_{90}^{PRED} = SPR^{PRED}$. Thus, SPR_{90}^{PRED} provides a more rational basis for comparison between different aggregate sources when they belong to different aggregate categories. It is also a better parameter to use when the same aggregate is evaluated based on two different approached, namely laboratory data approach and past skid performance data approach.

Figures 4.15 and 4.16 illustrate how this concept applies to SPR prediction based on lab data. Let us assume that we are going to use the non-carbonate regression model to identify those aggregates that provide an SPR of 3.0 or better. Based on Figure 4.15, only one aggregate source fail to qualify under $SPR^{PRED} > 3.0$ criterion (i.e. points above the horizontal bold line). However, based on actual performance there are a total of 4 aggregate sources (i.e. points to the left of the vertical bold line) that do not meet the minimum SPR=3.0 criterion. In other words, the three points lying in the upper, left hand quadrant were not correctly rated by the model. Subsequently, we calculate the SPR corresponding to 90% reliability (or 90% confidence level). These are shown in Figure. Figure 4.16.

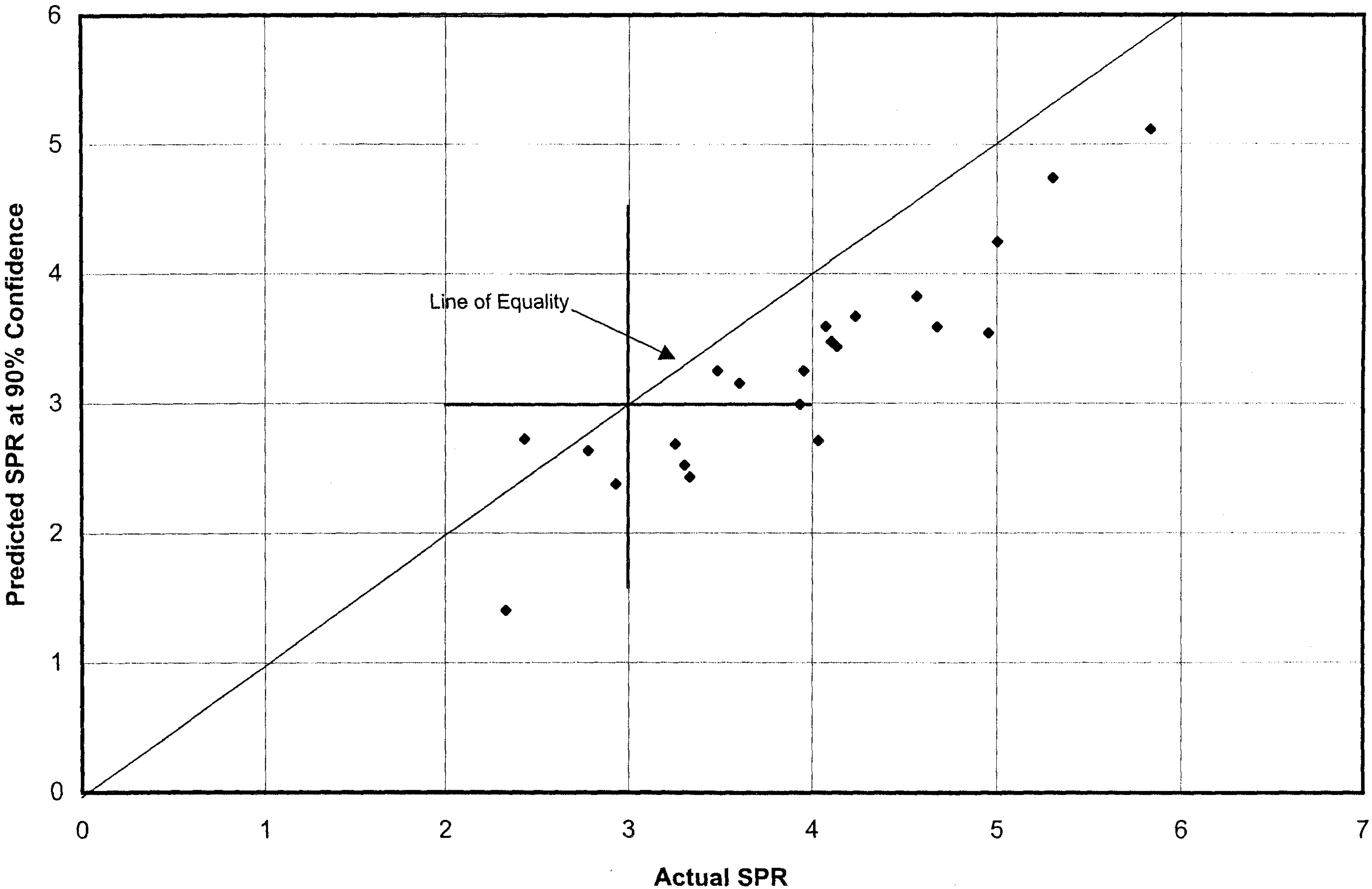


Figure 4.16 Predicted SPR vs Actual SPR for Non Carbonate Aggregates

The equation that is used to calculate the SPR_{90}^{PRED} values is as follows.

$$SPR_{90}^{PRED} = SPR^{PRED} - t_{1-\frac{\alpha}{2},\, n-k-1} \,.\, S_{SPR} \quad \text{...} \quad (4.3)$$

where:

t	=	appropriate quantile from student t-distribution table
1-α	=	desired level of confidence
n	=	no of data sets used in developing the regression model
k	=	number of variables in the model

On Figure 4.16 all the points are shifted vertically down. As it can be seen, by increasing reliability (50% to 90%), the probability that an aggregate source will provide poorer performance than predicted has been decreased.

CHAPTER 5

RATING OF AGGREGATES BASED ON HISTORICAL FIELD PERFORMANCE

INTRODUCTION

Most aggregate sources that supply coarse aggregate for TxDOT bituminous pavement construction projects have historical skid performance that may be used as the basis for an alternative approach for rating aggregate sources. In fact, TxDOT already uses such an aggregate qualification procedure. A detailed description of this procedure was given in Chapter 1. In this research, a critical review was conducted to evaluate the above procedure and hence make recommendations for its improvements. This chapter presents the findings from the above review.

5.1 AGGREGATE EVALUATION BASED ON HISTORICAL PERFORMANCE

Evaluation of aggregate based on past skid performance data is a logical approach to use especially when good relationship between the aggregate's laboratory properties and its field performance is not found. Nevertheless, there are a number of shortcomings in this approach that must be identified at the outset.

(a) First of all, this approach can only be used for aggregate sources for which adequate historical performance data is available. In other words, this option cannot be used for the evaluation of a new source and therefore, the procedure cannot completely replace aggregate rating methods that rely on laboratory data.

(b) Secondly, this approach requires field skid measurements on in-service pavements and thus demands more manpower as well as other resources such as time, field skid measurement equipment etc.

(c) Thirdly, development of a skid resistance history requires total accumulated vehicle passes (i.e. accumulated VPPL) to be known for every skid measurement. Unfortunately, it is ver difficult to obtain reliable estimate of AVPPLs.

(d) Also, it is important to realize that the mineralogical composition and the quality of the aggregate obtained from the same source (i.e. quarry) can vary

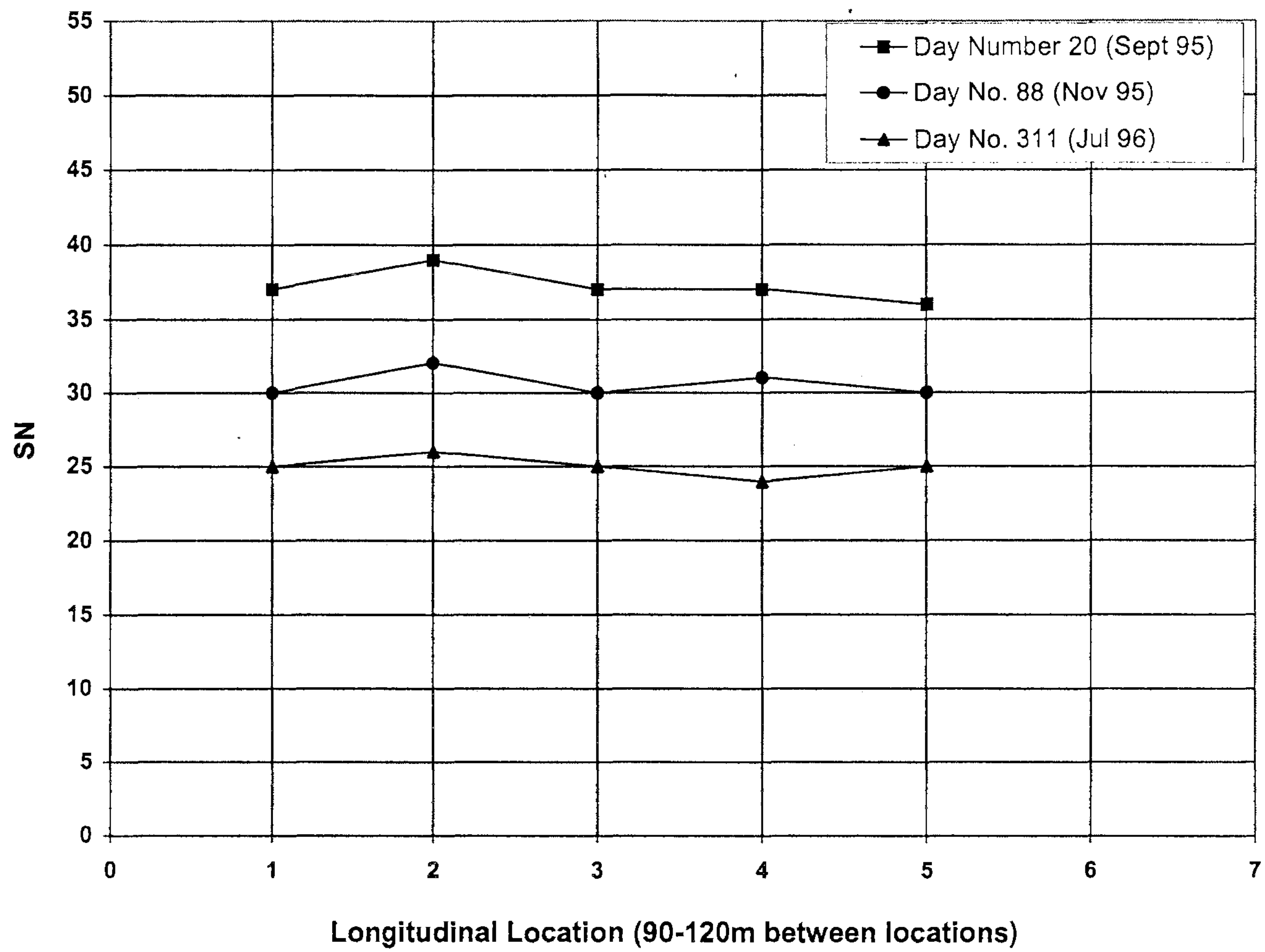


Figure 5.1. Longitudinal Variation of Skid Numbers on US 281-1

measured on different days were used to quantify seasonal variation. The standard deviation corresponding to that variation is called *between day standard deviation*. Table 5.1 compares the two types of standard deviations.

The longitudinal variation can be significantly reduced by taking the average of five measurements as was done in this study and recommended by ASTM 274.

Table 5.1. Comparison of Within Day and Between Day Standard Deviations

Highway Section	No. of Observations	Std. Deviation of SN_{64} within a day	Std. deviation of SN_{64} between days
US 281-1	31	0.91	2.97
US 281-2	32	1.56	2.19
AVERAGE		1.24	2.58

The influence of variation in skid number caused by variation in instrumentation and operator was not specifically addressed in this research. However published data indicate that their influence is less than variation due to weather, provided that the instrument is calibrated in accordance with ASTM 274 and operators are adequately trained. One such study *(16)* reports a range of standard deviation of 1.13-1.84 with 11 operators. ASTM 274 reports a standard deviation of 2 SN from numerous tests that varied in speeds, surfaces, and skid trailers.

5.3 NORMALIZATION OF SKID NUMBER MEASUREMENTS TO ELIMINATE THE INFLUENCE OF CLIMATIC VARIATIONS

The data presented in Chapter 3 clearly demonstrates that field skid measurements are very sensitive to the climatic conditions at the test location prior to and at the time of measurement. Therefore, it is necessary to apply a correction to the measured skid number and hence obtain a "normalized skid number" that is independent of the climatic conditions. Accordingly, this research investigated the possibility of developing a model that can be used for such normalization of skid data. This analysis was conducted using the data described in Chapter 3, Section 3.4. Previous researchers have attempted a number of different methods for normalizing skid measurements for seasonal effects *(14, 17 and 18)*.

(a) Models based on Climatic and Other Variables

One method by which the normalization of skid numbers can be achieved is to use a mathematical relationship between the measured skid number and the variables that cause seasonal fluctuations in it. As described previously, many researchers have attempted to develop such relationships by using mechanistic methods or statistical regression techniques.

In the present study, the researchers followed a similar statistical approach to develop an equation that can be used to normalize the skid measurements. For this purpose data collected from all 6 pavement sections were combined and then analyzed using multiple regression techniques. The form of the statistical regression model used in this analysis is represented by Eq. (5.1) below.

$$SN_{64} = A_0 + A_1\,(TEMP) + A_2\,(RF) + A_3\,(DD) + A_4\,Log_{10}(VPPL) + A_5\,Sin\,\{(2\pi/365)\,JD + A_6\} + \Sigma B_r\,I_r \quad \ldots\ldots.(5.1)$$

where:

SN_{64}	=	skid number at 64 km/hr (40 mph)
TEMP	=	a variable representing temperature condition preceding or at the time of skid measurement
RF	=	a variable representing amount of rainfall received at the location of the test pavement section prior to the skid measurement
DD	=	a variable representing the number of dry days preceding measurement
VPPL	=	cumulative vehicle passes on the test lane
JD	=	Julian calendar day corresponding to the day of measurement
I_r	=	Indicator variables that identify the pavement section (r =1,2,…..5)
A_i, B_r	=	Regression Coefficients (i =1, 2, ….5, and r =1,….5)

The choice of this particular form of model was primarily based on the findings of previous research studies. In the above regression model, the first variable, TEMP accounts for possible influence of temperature on the pavement skid resistance as well as on the measuring system. Since it has generally been observed that skid numbers fall with rising temperature, the coefficient, A_1 is expected to be negative. In this analysis, a number of parameter combinations were used to represent the variable, TEMP. They were: (1) air temperature at the time of measurement, (2) average temperature over a 24-hr period prior to the measurement, (3) average of daily temperatures for 5 days preceding the measurement. The parameter, RF accounts for the influence of the antecedent rainfall on SN_{64}. Once again, more than one parameter was used to represent RF. The parameter, DD in the model accounts for the decrease in SN_{64} with increasing number of dry days prior to skid measurement. A number of different parameters including (1) number of dry days since last significant (> 2.5mm or 0.1 inch) rainfall event, (2) dry spell factor, log (dry days +1) were used in the equation for variable, DD. The Log_{10}(VPPL) term accounts for possible decrease in SN_{64} with accumulating traffic. However, the accumulated traffic on pavement sections that were monitored in this research ranged from 5 – 63 million (See Table 3. 12) and therefore, it was reasonable to expect that any further decrease in SN_{64} with increasing traffic would be minimal. The sinusoidal term in the model represents cyclic fluctuations that may occur in skid numbers that are caused by factors other than the environmental variables discussed above. These may include seasonal changes in ADT (i.e. more traffic in summer than in winter), highway management practices that vary depending on the season (e.g. deicing chemicals and other abrasive material applied on icy roads to minimize potential skid problems during winter). The terms I_1 through I_5 are indicator variables that account for the differences in the mean skid number from one test pavement to another.

The final regression model was selected based on the coefficient of determination, R^2 as well as level of significance. The selected model is shown below. In this final model only those terms that were significant at the 0.05 level were retained. As it can be seen, the coefficients A_3, A_4, and A_6 were not found to be significant and therefore, variables DD and VPPL do not appear in the final regression model.

$$SN_{64} = a_0 + a_1(TEMP_5) + a_2(RF_5) + a_3\mathrm{Sin}\{(2\pi/365)\,JD\} + b_1 I_1 + b_2 I_2 + b_3 I_3 + b_4 I_4 + b_5 I_5 \quad \ldots\ldots(5.2)$$

Coefficient of Determination, $R^2 = 0.917$
Adjusted $R^2 = 0.909$

In the above equation, $TEMP_5$ represents the average of daily temperatures for the 5 days prior to the measurement. RF_5 represents the cumulative rainfall over the 5-day period preceding the measurement. The magnitudes of a and b coefficients and their respective p-values are shown in the table below.

Table 5.2. Regression Coefficients and Corresponding p -Values

	a_0	a_1	a_2	a_3	b_1	b_2	b_3	b_4	b_5
magnitude	32.28	-0.14	0.031	-0.66	13.53	-3.12	-2.78	9.52	7.43
p-value	.0001	.0001	.0079	.0309	.0001	.0001	.0004	.0001	.0001

The model presented above can be examined to determine the influence that each of the variables will have on the measured skid number. To make this comparison, the range (i.e. maximum – minimum) corresponding to each of the two environmental variables were obtained from the original data set. The range multiplied by the appropriate regression coefficient provides an indication of the change in SN_{64} associated with the change in each variable. Similar calculations were also performed using the 90% confidence interval (i.e. 95^{th} percentile – 5^{th} percentile) of the two variables as well. The results are shown in the table below.

A comparison of the numbers in the two $\Delta(SN_{64})$ columns of this table suggests that both the temperature and rainfall have equally significant influence on the measured SN_{64}. There is a noticeable difference between the range and the 90% confidence interval for RF_5. This difference is due to the presence of a few large values in the upper

Table 5.3. Influence of Temperature and Rainfall on Measured Skid Number

Variable		Based on Range		Based on 90% C.I.	
	a	Range	$\Delta(SN_{64})$	90% C.I.	$\Delta(SN_{64})$
$TEMP_5$ (°C)	0.144	23.9	3.44	22.8	3.28
RF_5 (mm)	0.031	106.2	3.30	45.2	1.41

end of RF_5 spectrum. Furthermore, based on the above regression model, the influence of rainfall and temperature on measured SN_{64} is much larger than that due to non-environmental factors. The latter contributes to a maximum seasonal variation of only 0.66.

Regression models such as that described above can be useful in identifying variables that contribute to variations in skid numbers and determining their relative significance. However, as far as the normalization of skid measurements to obtain true mean skid resistance is concerned, their usefulness is very limited. This is primarily because, in most routine testing, the necessary data on rainfall, temperature and other variables corresponding to the specific test location will not be available.

(b) Models Based on Julian Calendar Day

An alternative approach that has been used by some researchers to overcome the difficulty described above is to use a model that performs normalization based on the Julian Calendar day only. This approach has been used in studies conducted in Kansas and New Jersey *(22)*. Equation (5.3) below represents the general form of the model used in this approach.

$$SN_{64} = A_0 + A_1 \, \mathrm{Sin}\{(2\pi/365)\, JD + A_2\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.3)$$

The single sinusoidal term in this equation represents the influence of all variables that exhibit cyclic variation with season. These include the rainfall and temperature. From the ease of implementation standpoint, the above model has an obvious advantage over the type of model described in the previous section. However, it must be noted that this form of the equation can only account for the long-term variation in skid numbers. It cannot account for short-term variations such as those that occur due to individual rainfall events. An example can be seen in Figure 5.2. The dotted lines in the figure represent the best-fit models of the form described by Eq. (3) for the data recorded for the two pavement sections in San Antonio District. These models were obtained by using a non-linear inverse parameter estimation procedure called Levenberg-Marquard Procedure. The models obtained from this analysis and the corresponding R^2 values are shown in Table 5.4.

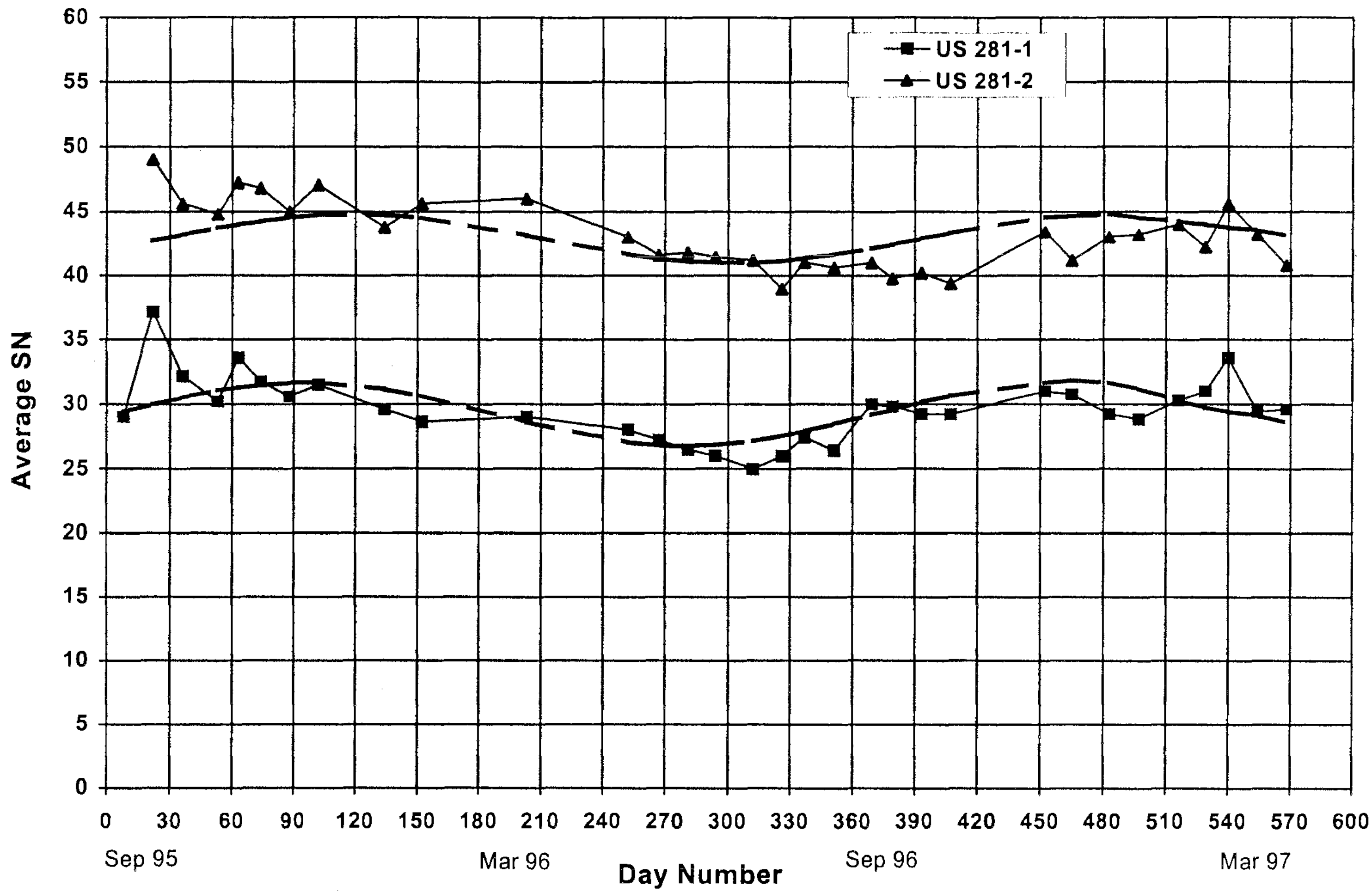


Figure 5.2. The Best Fit Sinusoidal Models Based on Julian Day for US 281-1 and US 281-2 Data

Table 5.4. Results from Levenberg-Marquard Non-linear Parameter Estimation Procedure

Section No.	A_0	A_1	A_2	R^2
US 281-1	29.22	2.47	2.05	0.434
US 281-2	42.89	1.93	1.65	0.293

From Figure 5.2 it is quite obvious that short-term variations constitute a significant part of the total variation in skid number. Table 5.5 below compares the magnitudes of long-term variation that is explained by the model and the short-term variation for the two Region IV pavement sections.

Table 5.5. Comparison of Variances for Long-term and Short-term Variations

	US 281-1	US 281-2
Long-term Variance	2.739	1.785
Short-term Variance	3.567	4.901
Total Variance	6.306	6.686

In addition to the above, a study in which data was collected over several complete cycles (i.e. several years) concluded that that the long-term seasonal pattern showed variations from one cycle to another *(14)*. This observation tends to raise further questions with regard to the validity of the use of Julian Calendar day to represent cyclic changes in measured skid numbers.

(c) Use of Multiple Skid Measurements

One obvious method that can be used to minimize possible error due to long- and short-term seasonal variations in skid number is to make multiple skid measurements at different times of the year and then use the average of all such skid measurements. This section examines the effectiveness of this approach. The average of the six between day standard deviations for each test sections is 2.580. Thus, if the true mean skid number for a given pavement is μ_{SN}, then the 90% confidence interval will be $\mu_{SN} \pm 1.645 \times 2.580$, or $\mu_{SN} \pm 4.244$. In other words, there is a 90% probability that any given single skid measurement will be within ± 4.244 of the true mean. Had we obtained a sample of *n* number of skid measurements and used the sample average, then the standard deviation of the average of n measurements will be $2.580/\sqrt{n}$. For n = 4, the standard deviation will be 1.290 and therefore, there is a 90% probability that average of 4 skid numbers will be within ± 2.122 of the true mean. Similarly, the 90% confidence limits for n = 8 and 16 can be shown to be $\mu_{SN} \pm 1.500$ and $\mu_{SN} \pm 1.061$, respectively. However, since the variation of skid numbers are not completely random but follow a systematic pattern based on the season, it will be necessary for the measurements to be distributed evenly between different seasons to avoid possible bias.

In summary, the analysis described in this section leads to following conclusions. Three different methods for the normalization of skid data to eliminate the seasonal variations were examined. The first of these, which was a regression model based on the average of mean daily temperatures and the cumulative rainfall for the 5 days preceding measurement, was able to explain much of the variation observed in skid numbers. Such a model can be used to identify the significant variables and to determine their relative contribution. However, this type of model may have limited usefulness in normalizing routine skid measurements because many of the necessary climatic data are not collected during such testing.

The second model, a regression model based on Julian calendar day, is more convenient for routine use but has a major limitation of not being able to account for short-term variations in skid numbers. This is a major drawback because the analysis of data collected in this study shows that short-term variations constitute the larger portion of total variation.

The third approach that was examined involves the use of the average of more than one measurement to minimize the influence of seasonal variations and thus arrive at a better estimate of the true mean skid number of a given pavement. Based on the seasonal variations that were observed, the number of skid measurements needed to come within an acceptable margin of error was determined.

5.4 Review of TxDOT's Current Procedure for Aggregate Qualification Based on Historical Skid Performance

Section 5.1 examined the general approach of aggregate qualification based on past performance and discussed some of its limitations and shortcomings. Sections 5.2 and 5.3 focused on one of the major limitations in this approach, namely sensitivity of skid numbers to climatic variables and how that difficulty may be overcome. This section reviews the specific procedure that is currently used by TxDOT. A detailed description of the above procedure was provided in Chapter 1. The review conducted in this research identified the following deficiencies in the TxDOT procedure.

(a) Incorrect Form of Equation

TxDOT procedure involves fitting a straight line to the Skid Number (SN_{64}) vs. Cumulative Vehicle Passes data. Accordingly, this line will have the following form of

$$\mathrm{Log}_{10}(SN_{64}) = A - B\ \mathrm{Log}(VPPL) \qquad (5.4)$$

equation.

where:

SN_{64} = Skid Number at 64 kmph

VPPL = Cumulative Vehicle Passes

A, B = Positive Regression Coefficients

The same equation can be reorganized to yield the following form.

$$SN_{64} = \frac{C}{VPPL^{B}} \qquad (5.5)$$

where:

$C = 10^{A}$

According to this equation, SN_{64} approaches zero as the VPPL increases the skid number. This contradicts the observed behavior. The general trend that is commonly assumed is that the skid number approaches some "terminal value" as VPPL increases. In fact, that same assumption is inherent in the laboratory evaluation of aggregates based on RSPV. This contradiction between the observed behavior and the form of the equation that is used has adverse effect on the reliability of the procedure. Since, the actual observations follow a trend that asymptotically approaches a constant "terminal skid number," the slope, B back calculated from the above equation is very small. As a result, the useful pavement service life that corresponds to a given minimum acceptable skid number is very sensitive to that value of slope, B. Addition or deletion of a few data points can change the slope, B and the corresponding change in the useful pavement service life can be extremely large. A more appropriate form of the equation that can be used in this situation is as follows.

$$SN_{64} = SN_T + (SN_I - SN_T)\exp(-k.VPPL) \quad \text{......} \quad (5.6)$$

where:

SN_T = Terminal Skid Number

SN_I = Initial Skid Number (SN_{64} on new pavement)

k = a constant that defines rate of skid number deterioration

Figure 5.3 shows how this form of equation can be used to curve-fit the skid resistance data. SN_T parameter that is obtained from such curve-fit will be used in ranking the aggregate source. The SN_T parameter that is determined in this manner will be analogous to the aggregate polish value that is determined after 9 hours of polishing in the laboratory.

(b) Inability to Account for Scatter (or Variability) in Data

Another noteworthy shortcoming in the current field qualification procedure used by TxDOT is its inability to account for variability in the skid data. This is particularly important because large variations are common in skid resistance measurements. The significance of data variability can be easily demonstrated by using Figures 5.3 and 5.4. As it can be seen in these figures, both data sets have yielded the same best fit curve and hence same SN_T parameter from the analysis. However, data scatter shown in Figure 5.4 is much larger and as a result, the confidence associated with the SN_T parameter obtained from this data is much lower. The current TxDOT procedure does not distinguish

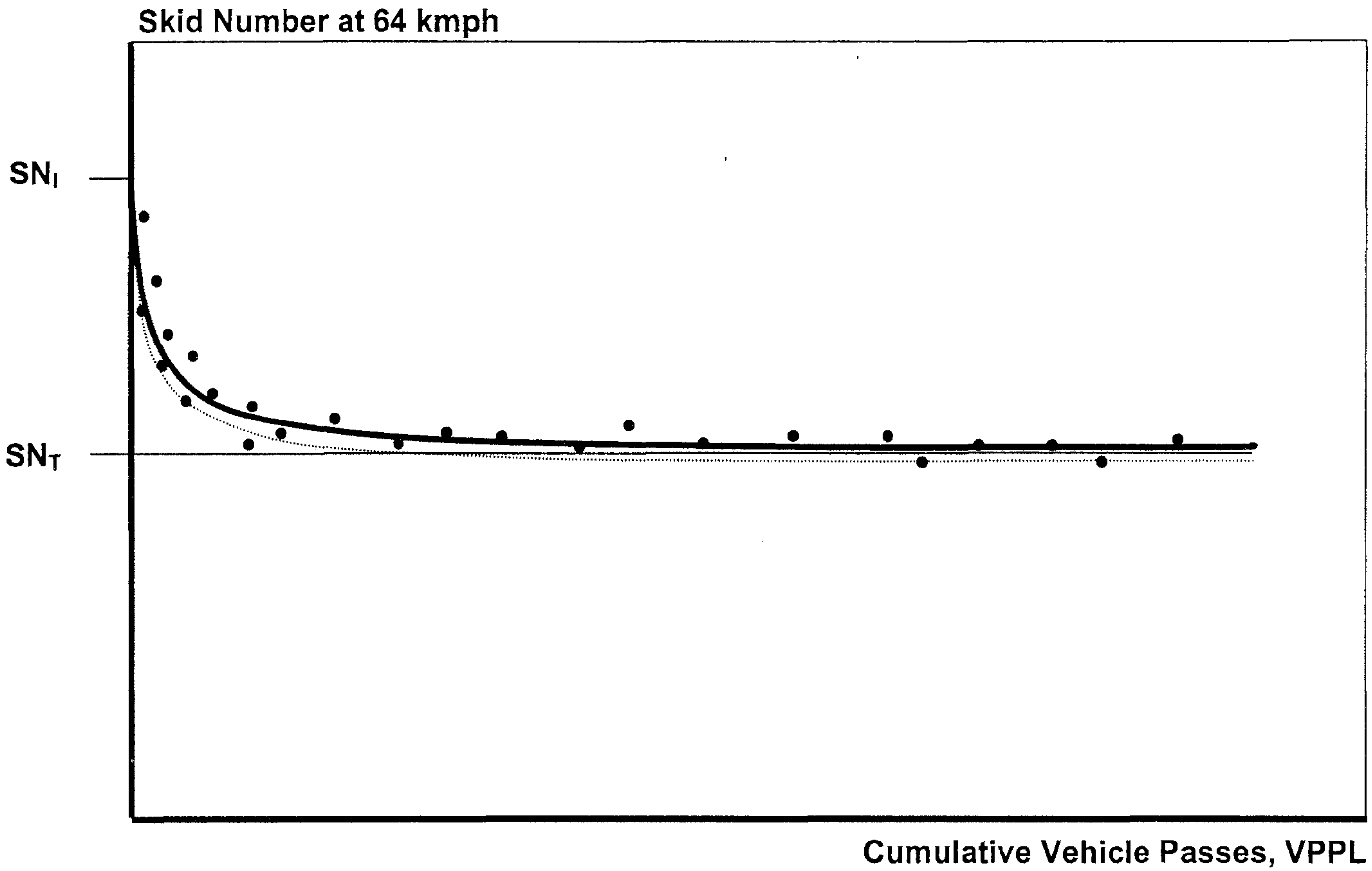


Figure 5.3 Determination of SN_T from Historical Skid Performance Data

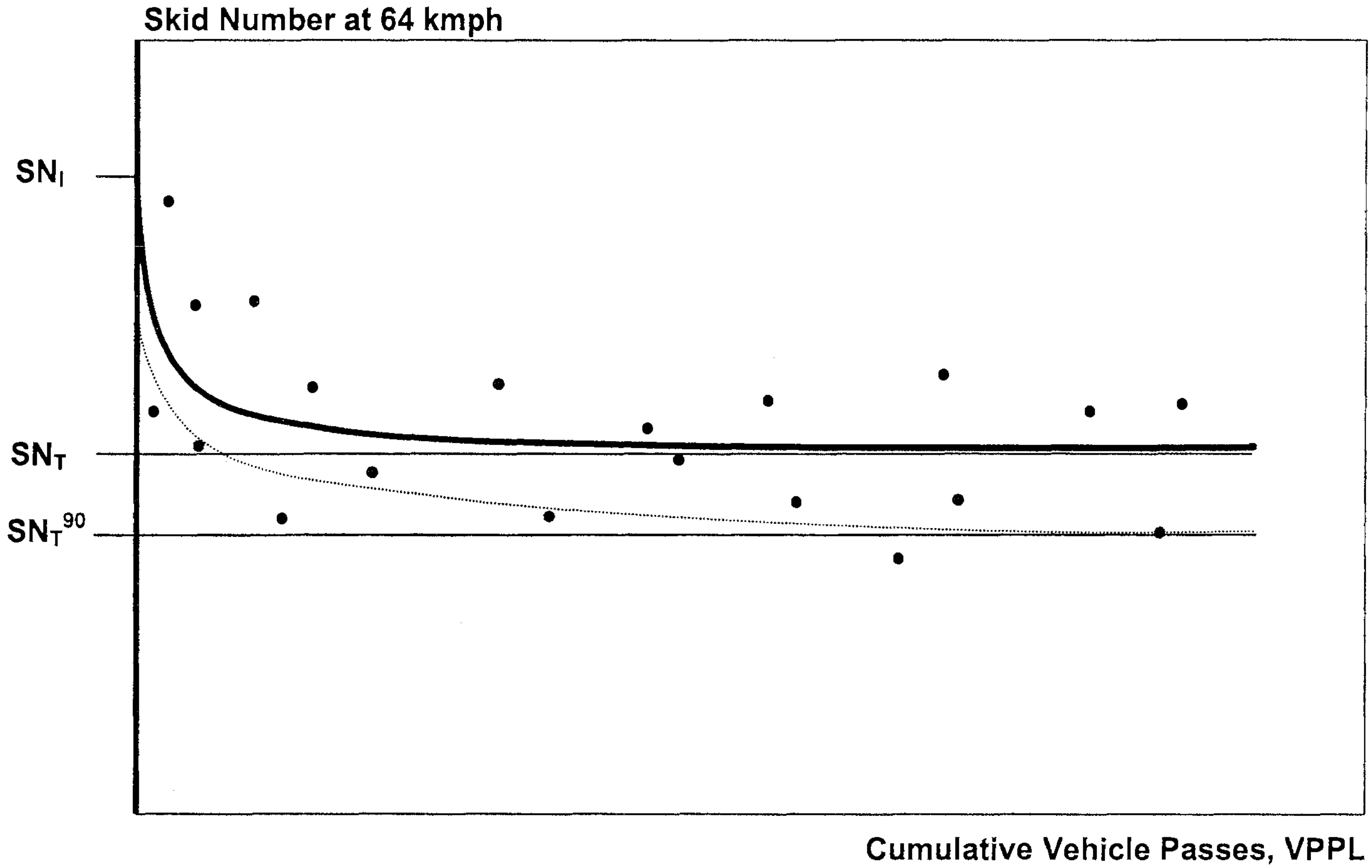


Figure 5.4 Poor Reliability in the Predicted SN_T due to Large Scatter in Data

between the two situations. Interestingly however, such variability is considered when aggregates are rated using the polish value. As discussed in Chapter 1, the RSPV value of an aggregate source, as defined by Equation (1.1), is lower whenever the variability in individual PV measurement is larger. To account for data variability, a similar approach must be used when aggregates are qualified based on historical performance data. This can be easily accomplished by using a curve associated with a higher level of confidence. The best fit curves shown in bold lines are associated with confidence level of 50%. Instead, if a curve corresponding to say, 90% confidence level is drawn, they will be similar to those shown in dotted lines. As shown in the figures, the SN_T (or SN_T^{90}) values obtained from these dotted lines do distinguish between the Case I : where data was quite consistent versus Case II: where there was large scatter.

(c) Minimum Number of Data Points and Other Constraints

The minimum number of data points required by the current TxDOT procedure for constructing a skid performance history is six *(4)*. The findings from this study show that six data points will, by no means, be adequate to construct a reliable skid performance history. As explained earlier in Section 5.3 of this chapter, the variability associated with skid measurements is high and multiple measurements of skid number are necessary to minimize possible error due to seasonal fluctuations. It was found that the average of 4-5 separate readings taken at different times of the year on a given pavement will provide a skid measurement within ± 2 skid numbers within true mean skid number of that pavement at 90% confidence level. A minimum of 16 measurements will be needed to be ± 1 skid number within the true mean skid number. In addition, it is important to remember that while the pavement skid resistance is largely controlled by the coarse aggregate in the bituminous mix, there are numerous other factors that may have secondary influence. These factors include: macrotexture of the pavement, fine aggregate used in the mix, percent truck traffic etc. For example, let us assume that higher macrotextures provide better skid resistance. If most of the data points used in the development of the skid performance history were obtained from coarser mixes (i.e. pavements with rougher textures), that will introduce undue bias into the evaluation.

Therefore, it is necessary to introduce further constraints to eliminate possible bias due to such extraneous factors.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

This research study was undertaken with the primary objective of performing a comprehensive evaluation of TxDOT's current aggregate screening procedure to achieve satisfactory skid resistance performance on bituminous pavements. This evaluation was based on data collected on 55 hot mix asphalt concrete pavement sections. The data consisted of skid resistance measurements on all 55 test pavement sections over the 3-year study period as well as detailed laboratory characterization data on the coarse aggregates used in each bituminous mix. This research study focused on the following specific issues related to aggregate rating to ensure satisfactory skid resistance on HMAC pavements:

(a) To what extent does the terminal skid resistance on a HMAC pavement depend on the quality of the coarse aggregate used in the mix? To what extent is it influenced by other factors other than coarse aggregate quality (such as macrotexture, climatic variables, fine aggregates in the mix etc.) ?

(b) Does the current TxDOT procedure based on aggregate polish value provide a reliable basis for rating aggregate sources? If not, can a more reliable procedure be developed based on an alternative laboratory test? Or else, can other laboratory test data be used in conjunction with polish value test to improve the reliability of the current RSPV procedure?

(c) Can aggregate sources be rated based on their historical skid resistance performance instead of relying on laboratory test approach?

6.1 CONCLUSIONS

This section summarizes the conclusions reached based on the findings from this research.

(a) Skid resistance performance of bituminous pavements is primarily controlled by the type and quality of the coarse aggregates used in the mix. Based on the review of skid resistance data collected on 55 test pavement sections on which the macrotextures, climatic regions, daily traffic volumes varied widely, it was concluded that certain types of aggregate consistently provide better overall skid performance than others. The aggregate type and their skid performance behavior are discussed below in more detail.

(b) There is noticeable difference in the manner in which the pavement skid resistance changes with increasing vehicle passes for different types of aggregate. In this research aggregates were separated into 3 primary categories: (i) Category I: carbonates with low hard mineral content, (ii) Category II: carbonates with high hard mineral content and (iii) Category III: non-carbonates. Among these, the aggregates belonging to the first category exhibit rapid deterioration of skid number with increasing VPPL (See Figure 4.4). Carbonates with high hard mineral content and non-carbonate aggregates, on the other hand, maintain skid number on the pavement much better (See Figures 4.5 and 4.6).

(c) Secondly, 9 (out of 16) aggregate sources belonging to Category I rated less than or equal to 2 (fair or poor) based on field performance (Table 4.3, p. 106). This number compares with 3 sources (out of 15) in Category II (Table 4.4. p. 107) and none (out of 23) in Category III (Table 4.5, pp. 108-109). This is strong indication that the majority of the aggregate sources that yield poor skid resistance performance are carbonate aggregates with low hard mineral content. Non-carbonate aggregates have provided the best skid resistance performance as a group.

(d) It does not appear that, the current RSPV approach is capable of identifying poor performance aggregate sources correctly and screening them out. For example, out of the 9 aggregate sources in Category I that yielded poor field skid performance rating (i.e SPR $\leq$ 2), only one had RSPV<30. 8 sources had RSPV $\geq$30 and 7 sources had RSPV $\geq$32 (Table 4.3, p.106). By contrast, in Category III (i.e. non-carbonates), although none of the aggregate sources yielded Skid Performance Rating $\leq$ 2.0, there were 5 aggregate sources that had RSPV<30.

(e) The inability of the RSPV to predict field skid performance of aggregates is further confirmed by the poor statistical correlation between Skid Performance Rating, SPR and RSPV. The results from the above statistical analysis are summarized in Table 4.6, p.112. In the above analysis all the aggregate sources were pooled together and statistical correlation sought between aggregate skid performance rating, SPR and each of the laboratory test parameters taken one at a time. According to the findings from the above analysis, the coefficient of determination, R^2 for SPR vs. RSPV relationship is 0.176.

(f) However, at the same time, the same statistical analysis showed that none of the other lab test parameters provided a correlation that is clearly better than that obtained for RSPV. This observation leads to following conclusions. First, a single laboratory test parameter may not serve as a predictor of the field skid performance of all types of aggregates with a widely varying mineralogical composition. Secondly, the poor correlation obtained for all of the lab parameters also suggest that the problem, to a significant extent, lies in the difficulty in obtaining a true measure of the field skid performance of the aggregate. In other words, even if there was a laboratory test that is capable of simulating aggregate field skid behavior perfectly, strong correlation may not be obtained owing to difficulties in measuring field skid resistance accurately.

(g) If the aggregates are categorized based on Acid Insoluble Residue (AIR), then the correlation between SPR and PSPV improves. The results of this analysis are found in Table 4.7, p.114. However, the R^2 values for these models may still be unsatisfactory. It is significant to note that, in each case, RSPV was found to be the best predictor of the field skid performance. In other words, among all the lab test parameters, it provided the strongest correlation to SPR.

(h) The above observation suggest that a common RSPV scale cannot be used to predict the field skid performance of low AIR carbonates, high AIR carbonates and non-carbonates. For e.g. an RSPV value of 30 will correspond to an FPR of 1.26 for low AIR carbonates, an FPR of 1.78 for low AIR carbonates and FPR of 3.534 for non-carbonates. In other words, the carbonate materials with RSPV=30 will be "fair" in terms of their field skid performance whereas the non-carbonate material with same RSPV will be "very good."

(i) Stronger correlations can be obtained by further subdivision of aggregates based on percent of non-carbonate mineral contained in the aggregate. These regression equations are shown in Table 4.8, p.118. The final regression equations with the highest correlations were obtained by introducing other parameters into the equation that has been developed for each aggregate category. These final regression equations are summarized in Table 4.9, p.124. In developing these equations only those variables that were significant at a 0.05 level (i.e. a p-values of 0.05 or lower) were retained in the equation. It is interesting to note that out of all the laboratory test parameters considered, RSPV and AIR were the only lab test parameters that were found to be significant. Other lab test parameters, namely RSSM and RSLA, were not found to be significant.

(j) It is also important to note that the statistical analysis did not identify pavement macrotexture as a significant variable. In addition, pavement macrotextures measured using sand patch method and mini-texture-meter did not show any correlation with the percent normalized gradient (PNG) in the SN versus speed plots. These two findings indicate that when the skid resistance is measured using the ribbed tire, as done in Texas, it is not influenced by the pavement macrotexture to any significant extent. Also, it indicates that the range of macrotextures found on Texas pavements are not large enough for this parameter to influence pavement skid resistance measurements.

(k) The only extraneous factor that showed significance at the 0.05 level was the average annual rainfall at the test location. Rainfall was identified as a significant variable in aggregate category I: low AIR carbonates. The positive coefficient for rainfall in the regression equation suggests that skid resistance measured at locations with higher average annual rainfall tend to be higher than those for low rainfall locations in this aggregate category.

(l) Field skid resistance measurements are very sensitive to climatic conditions prevailing at the test site prior to and at the time of measurement. Among the climatic variables that influence the measured skid resistance, rainfall and temperature are the most significant. These factors can cause large fluctuations on the skid resistance measurements on a given pavement. The only practical approach to minimize these effects is to make multiple skid number measurements and use the average. Furthermore, these multiple measurements should represent the range of climatic conditions that the pavement section may experience.

(m) The large variability that is inherent in the measurement of skid number as well as the uncertainties in the estimation of cumulative vehicle passes make aggregate source rating based on past field performance a less attractive approach. Because of this and other reasons listed in Section 5.1 of this report, it is quite clear that aggregate rating based on historical field performance cannot be used as the sole or primary basis for aggregate screening. A better approach will be to consider both laboratory and field performance data.

(n) There is an inconsistency between the TxDOT polish value approach and skid performance history approach in the manner they characterize aggregate polishing behavior. The polish value approach assumes that, as the aggregate is polished by the action of traffic, its frictional resistance approaches a minimum terminal value. The polish value measured after 9 hours of polishing in the lab is considered to be a measure of the above terminal frictional resistance. By contrast, the form of the equation that is used in the skid performance history approach assumes that the skid number continues to decrease and eventually reach zero. Chapter 5 of this research report outlines a procedure that will estimate the terminal skid resistance from historical performance data and hence eliminate the above incompatibility.

(o) The proposed methodologies allow the materials engineer to predict a "skid performance rating (SPR)" based lab test data as well as historical performance data. Table 4.1 shows the relationship between SPR and the terminal skid number. In the future, with data obtained from continued monitoring of the test pavement sections, regression equations can be modified to predict "terminal skid number."

6.2 RECOMMENDATIONS

The following recommendations are made for future implementation of the findings from this research study.

(a) There are two other important tasks to be completed prior to implementation of the findings of this study. The first involves verification of the methodology proposed. The need for such verification is explained in (b) below. The second task involves careful analysis to determine the impact from implementation. The above analysis must carefully examine the impact on each TxDOT district individually.

(b) Verification of the proposed methodology is necessary for the following reasons. For the majority of the pavement sections monitored in this study, the skid measurements made over the 3-year study duration were not adequate to reliably establish the terminal skid number. Therefore, an alternative measure of the field skid performance called "skid performance rating" (or SPR) was established by the researchers. To develop the SPR, aggregates that exhibit similar Skid Number versus VPPL trends were grouped and curves corresponding SPR= 1, 2, 3, and 4 were drawn. Each SPR curve has a specific terminal skid number associated with it. This procedure was developed with the expectation that pavement sections will continue to be monitored and once adequate the data have been collected, the terminal skid numbers will be directly determined from this data. The analysis will then be repeated using the actual terminal skid number in place of the SPR. In this manner the methodology proposed in this research can be verified. <u>Therefore, in order to reap the full benefits from this research, the test pavement sections must be monitored for a further period of time</u>.

(c) Rating of aggregates based on past skid performance is not recommended for use as the primary aggregate qualification procedure. The large variability associated with skid number measurements, the need for additional resources to perform field skid testing, differences that are commonly found in aggregate coming from the same source but at different times, poor reliability of traffic data are the primary reasons for the above recommendation. The current TxDOT skid history procedures has some major deficiencies. An alternative procedure that overcomes these deficiencies is proposed.

(d) The findings from this research study can be implemented at a number of different levels. The lowest level implementation will involve the regression models presented in Table 4.7. This will result in an improvement in the current RSPV procedure. However, implementation will require aggregate classification based on AIR and petrography test results. According to this model, the threshold RSPV values used by TxDOT will be modified producing different scales for different aggregate categories; namely low AIR carbonates, high AIR carbonates and non-carbonates. This will overcome some of the more obvious shortcomings of the current procedures but still, the reliability of the procedure may not be satisfactory.

(e) The approach recommended combines both laboratory data and past skid performance data. The implementation of this approach which is more rational and technically sound, requires the aid of a computer software. Consequently, a computer software named SKIDRATE was developed. SKIDRATE runs on Windows NT platform and uses the database management software, ACCESS® to store all relevant aggregate data. Such data include: aggregate source information, Polish Value, Mg Sulfate Soundness, LA Abrasion, Acid Insoluble Residue and mineralogical composition. It allows the storage of test values taken at different times and therefore, the review and analysis of stored data to determine "rated source values" (such as RSPV, RSSM etc) can be accomplished using the software. In addition to the aggregate laboratory properties, SKIDRATE also stores field skid measurements made on the pavements that have been constructed using each of the aggregate sources. Subsequently, the program can be used to predict the Skid Performance Rating for any selected aggregate source based on the laboratory test data or historical skid performance data. Furthermore, when using SKIDRATE the user can specify a desired level of reliability. The advantages in using the reliability based prediction was explained Chapters 4 and 5. More complete documentation on the SKIDRATE software can be found in a companion report.

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APPENDIX A

Table A1 Addresses of Contact Persons in Various DOT's

Sl. No.	State DOT	Name of the Contact Person	Division	Address	Phone and/ or Fax Number
1	Alabama	Billy G. Allen	Bureau of Research and Development	1409 Coliseum Boulevard Montgomery, AL. Zip - 36130-3050	Ph: 205-242-6541 Fx: 205-264-2042
2	Alaska	-	-	-	-
3	Arizona	Oscar Musawi	-	-	Ph.:602-255-7788 Fx:602-255-8413
4	Arkansas	Boon Thian	Research and Planning	P.O. Box 2261, Little Rock, AK-72203	Ph:501-569-2498
5	California	Bob Page	Pavements	-	Ph: 916-227-7299 Fx: 916-227-7075
6	Colorado	-	-	-	-
7	Connecticut	Donald A. Larson	Division of Research Office of Research and Materials	280 West Street Rocky Hill, CT Zip 06067-3522	Ph: 203-258-0301 Fx: 203-529-0323
8	Delaware	Wayne Kling	Materials and Research	P.O. Box 778 Dover, DE - 19903	Ph:302-739-4852
9	Florida	Donald L. Hewett	State Materials Office	2006 Northeast Waldo Rd. Gainesville, FL 32609	Ph:904-372-5304 Ext. 127 Fx:402-479-3975
10	Georgia	Don Watson	-	15 Kennedy Dr. Forest Park, GA-30050	Ph:404-363-7521
11	Hawaii	-	-	-	-
12	Idaho	Bob Smith	-	-	Ph: 208-334-8437
13	Illinois	David Lippert	-	-	Ph:217-782-7200 Fx:217-782-2572

Table A1 (Continued) List of Contact Persons in Various DOT's

Sl. No.	State DOT	Name of the Contact Person	Division	Address	Phone and/ or Fax Number
14	Inidana	Robert J. Rees	Division of Materials and Tests	120 S. Shortridge Rd. Indianapolis, IN Zip-46219	Ph:317-232-5280 Fx:317-356-9351
15	Iowa	Jim Myers Kevin Jones	Chief Geologist / Skid Testing Engineer		Ph:515-239-1452 Ph:515-239-1232
16	Kansas	Richard Riley			Ph:913-296-3711 Fx:913-296-6665
17	Kentucky	Wesley Glass	Division of Materials	1227 Wilkinson Blvd. Franfort, KY Zip-40622	Ph:502-564-3160
18	Lousiana	Doug Hood	Materials and Testing Section	5080 Florida Blvd. Baton Rouge, LA Zip 70806	Ph:504-929-9131 Fx:504-929-9187
19	Maine	Dale PeabodY			Ph: 207-287-3171
20	Maryland	Ann Brach	Research		Ph: 410-321-3547
21	Massachuset ts	Mat Turo	Transportation Planning and Development		Ph:617-973-7266
22	Michigan	Robert W. Muethel	Pavement Technology Unit	P.O. Box 30049 Lansing, MI 48909	Ph:517-322-1087 Fx:517-322-5664
23	Minnesota	Roger Olson	Research Section	1400 Gervais Avenue Maplewood, MN Zip-55109	Ph: 612-779-5517 Fx: 612-779-5616
24	Mississippi	Alfred B. Crawley/ Reginald Jenkins	Research Division	P.O. Box 1850 Jackson, MS 39215	Ph: 601-359-7650 Fx: 601-359-7634
25	Missouri	Gerald Manchester	Materials and Research	P.O. Box 270 .Jefferson City, Zip 65102 MO	Ph: 315-751-3849 Fx: 314-526-5636
26	Montana	Scott Barnes			Ph: 406444-6267
27	Nebraska	George Wolstrum	Materials and Tests		Ph: 402-475-4750 Fx: 402-479-3975

Table A1 (Continued) List of Contact Persons in Various DOT's

Sl. No.	State DOT	Name of the Contact Person	Division	Address	Phone and/ or Fax Number
28	Nevada	Patricia Polish	Materials and Testing	1-263 Steward St. Carson Citv NV 89712	Ph:702-687-5173 Fx:702-687-4846
29	New Hampshire				
30	New Jersey	Richard Cary	Research		Ph:609-530-3885 Fx:609-530-3036
31	New Mexico	James Hawkins	Materials and Testing		Ph:505-927-9344 Fx:505-827-9342
32	New York	William H. Skerritt			Ph518-457-1038
33	North Carolina	Jerr Y Blackwelder	Construction and Testing		Ph:919-250-4094
34	North Dakota	Tim Homer	Planning Division		Ph:701-328-4406
35	Ohio	Bill Edwards	Research and Development		Ph:614-466-2916
36	Oklahoma	Scott Hoefner	Materials Division.		Ph: 405-521-2677
37	Oregon	Jeff Gower	Pavements Unit	2950 State St. Salem, OR 97310	Ph: 503-378-2580 Fx: 503-378-8974
38	Pennsylvani a	Gaylord Cumberledge/ David Reidenour	Roadway Management/ Materials Division		Ph: 717-787-1199 Ph: 717-787-2489
39	Rhode Island	Francis J. Manning / Colin A. Franco	Research and Technology	Two Capitol Hill, RM 013 Providence, RI 02903	Ph: 401-277-4955 Fx: 401-277-6038
40	South Carolina	Mike Sanders	Research and Materials Laboratory	P.O Box 131 Columbus, SC 29372	Ph:803-737-6691 Fx:803-737-6649

Table A1 (Continued) List of Contact Persons in Various DOT's

Sl. No.	State DOT	Name of the Contact Person	Division	Address	Phone and/ or Fax Number
41	South Dakota				
42	Tennessee	Gary Head	Division of Materials and Tests		Ph:615-350-4105
43	Texas				
44	Utah	Dal Hawks / Darrel Gianetti	Materials and Research		Ph:801-965-4196/ Ph.:801-965-4205
45	Vermont	Donald Frascoia/ Dwayne Stephens			Ph:802-828-2561/ Ph.:802-828-3527
46	Virginia	Mike Bower	Materials Division		Ph:804-328-3174
47	Washington	Bob Gietz	Materials Lab	P.O. Box 167 Olympia, Wa 98507	Ph:206-753-7109 Fx:206-586-4611
48	West Virginia	Bob Kessler	Materials Control and Soils Testing		Ph:304-558-3160 Fx: 304-558-0253
49	Wisconsin	Bill Duckert / Bonnie Collins	Bureau of Highway Engineering.	Truax Center 3502 Kinsman Blvd Madison, WI 53704	Ph:608-246-5440 Fx:608-246-4669
50	Wyoming	Warren Oyler	Materials Division	5300 Bishop Blvd. Cheyenne, WY 82002	Ph:307-777-4071 Fx:307-777-4481

APPENDIX B

Questionnaire on Skid Accident Reduction Programs of Different State Departments of Transportation

Study being conducted by the Department of Civil Engineering, Texas Tech University, Lubbock, Texas.

Please fill out the requested information and mail it to the address given below. If you have any questions, please feel free to contact us.

State Agency:______________________________
Division:______________________________

Contact Person:______________________________
Address:______________________

Phone no:________________ Fax no:________________
Our Address:

P. W. Jayawickrama, Ph.D.
Assistant Professor
Department of Civil Engineering
Texas Tech University
Lubbock, TX 79409
Ph No: 806-742-3471
Fax no: 806-742-3488

Sanjaya Senadheera, Ph.D.
Research Associate
Department of Civil Engineering
Texas Tech University
Lubbock, TX 79409
Ph No: 806-742-3037
Fax no: 806-742-3488

The Department of Civil Engineering at Texas Tech University, in association with the Texas Department of Transportation, is conducting a research study on the use of historical skid performance as the basis for the evaluation of coarse aggregates used in the construction of seal coat and hot mix asphalt concrete surfaces.

This survey is being conducted as a part of the above study to collect information regarding skid reduction policy/policies used by your Transportation Department. We would like information on both laboratory and field test procedures applicable to your skid control program.

1. Do you have any restriction with regard to the type and/or the percentage of aggregates (such as carbonate rocks, siliceous gravel etc) that may be used in the construction of HMAC and Seal coat surfaces ?
If yes, please specify and provide a brief explanation.

Type of Aggregate	Maximum °/0 Allowed in Surface Course
______________________	______________________________
______________________	______________________________
______________________	______________________________

Any Further Explanation

____
____
____
____

2. Please fill out the following information concerning any laboratory test methods which you may be using for the evaluation of aggregates with respect to their frictional properties (example: polish value test). If the test procedure used depends on the type of aggregate, then please fill out a separate page for each type of aggregate.

a. Type of Aggregate:________________________________
[write 'all', if the same test procedure is used for all aggregate types.]

b. Test procedure performed to determine frictional properties of aggregate;
Test Method:____________________________
Test No:________________________________ (eg. ASTM No.)

c. Please outline your specifications for aggregate qualification based on the above test method:

____
____
____

[Attach additional sheets / Printed information, as necessary.]

This section pertains to any aggregate qualification procedures that relies on the past performance of the aggregate with respect to pavement skid resistance. If you currently use any such performance based procedures please provide the following information.

3. Method/s used in the determination of skid performance in the field. {Mark those applicable}.

a. Locked Wheel Trailer Methods ____________
b. Yaw Mode ____________
c. Portable Skid Testers ____________ Specify Type:____________
d. Automobile Methods ____________ Specify Type:____________
e. Other testing methods ____________ Specify Type:____________

4. Please describe the aggregate qualification procedure based on field skid resistance measurements. {Attach any relevant printed information).

____
____
____
____
____
____

5. Please include your comments with regard to the laboratory test procedures used for the evaluation of aggregate frictional properties.

Reproduceability of the Test Results:_______________________________
Correlation with Field Performance:_________________________________
Time Required for Testing:____
Specialized Training for Testing:____________________________________
Specific Limitations:____
Other Comments:____

6. Write down your comments based on your experience with regard to the use of aggregate qualification based on past performance.
Reliability:_______________________
Limitations:______________________
Other Comments:__________________

APPENDIX C

Alabama Department of Transportation

Classified in category IV, Alabama DOT uses British Pendulum Number(BPN-9) values to qualify and determine the allowable percentage of carbonate stone used for construction of pavements. The quality of a pavement surface is controlled by specifying the percentage of carbonate stones., used in its construction BPN 9 values are also used to rate aggregate sources.

General Procedure

The state of Alabama considers friction in the design of new pavement surfaces. A laboratory procedure has been adopted to evaluate aggregates based on their frictional properties. Alabama DOT identifies carbonate stones as problem aggregates and evaluates their frictional properties before specifying an allowable percentage of carbonate stones that can be used in surface layer construction. A British Pendulum Tester is used in accordance with ASTM D33 19 and ASTM E303 to determine the BPN 9 for aggregates from a particular source. The aggregate is subjected to 9 hours of accelerated polishing using the British Wheel and then a British Pendulum is used to determine the BPN 9 value.

According to Section 410.01 of Alabama DOT specifications, carbonate stones such as limestone, dolomite or aggregates which tend to polish under traffic are permitted only in underlying layers, shoulder paving and widening. However Section 416 as amended by Special Provision No: 1303(2), allows carbonate stones in wearing layers with a maximum percentage of the aggregate specified based on the BPN 9 value of the aggregate source.

Percentage of Carbonate Stone Allowed in Wearing Layers
(Source: Special Provision No: 1303(2) of Alabama DOT)

BPN 9 Value of Aggregate Source	Maximum Allowable Percentage of Carbonate Stone
< 25	30
26	35
27	40
28	45
29	50
30	55
31	60
32	65
33	70
34	75
>35	80

Alabama DOT allows as much as 80 percent of aggregates to be carbonate stone in a mix design when the aggregate specimens have a high BPN 9 of 35. It also states that the maximum allowable percentage values are not to be exceeded in any case.

Other types of aggregates that are used in wearing layers are siliceous aggregates such as gravel, granite, slag, sandstone or a combination of these. For a suitable mix design aggregates should meet the requirements of sections 801 and 802. These sections specie the amount of deleterious substances, percent LA wear, percent soundness, gradation etc. Of siliceous coarse and fine aggregates.

References

1. Sections 410, 416, 801, 802 and Special Provisions No 1303(2) of Alabama DOT standard Specifications.
2. Questionnaire on Skid Accident Reduction Program of Different State Departments of Transportation, Response from Contact Person in Alabama DOT.

Florida Department of Transportation

Florida DOT evaluates frictional properties of aggregates according to the FM 5 510 Test Standard. This test determines Acid Insoluble Residue(AIR) material retained on No: 200 sieve. DOT standards specify a minimum of 12% AIR. FM 5 510 is applied to Oolitic limestone aggregate. Aggregates are also qualified based on historical correlation between minimum 12% AIR material in Oolitic limestone and adequate field friction numbers. In addition to the evaluation of field frictional tests performed on test sections, FM 5-510 is used to grant approval to potential good quality aggregates.

General Procedure

FLDOT considers friction only in Open Graded Friction Courses(OGFC) when designing new pavements. The quality of aggregates is controlled by specifying the aggregate type and limiting its use in OGFC. FLDOT evaluates frictional properties of aggregates by determining the AIR in accordance with FM 5-510 Test procedure, which is specifically applied to Oolitic limestone. A minimum of 60 percent Oolitic limestone is used in OGFC. The experience of FLDOT has shown that the reproducibility of test results in FM 5-510 is good. The results of laboratory frictional tests on aggregates have been found to be in good correlation with aggregate field performance.

Aggregate Source Approval Aggregate sources are approved in accordance with Chapter 14-103, Aggregate Source Approval, Rules of the Department of Transportation. This chapter provides the procedures for approving aggregate sources which are intended to be the source of specific aggregates for use on Florida DOT projects.

Characteristics of aggregates such as color, texture, hardness, physical or chemical properties, and other properties are determined in accordance with the Manual of Florida Sampling and Testing Methods or other recognized testing procedures in accordance with ASTM C 295-85. Acid Insoluble Material retained on the No: 200 sieve is determined using a minimum of five samples in accordance with FM 5-510. Results of wear tests, hardness, crushed faces, angularity and other relevant frictional characteristics are also obtained.

Field Tests A trial section with a minimum length of 500ft is constructed using the aggregate requiring source approval. Friction characteristics of the trial section surface are determined using Locked Wheel Trailer Methods in accordance with ASTM E274 Test Method. If the test results are found to be satisfactory, then a test section which has a minimum speed limit of 50 miles per hour with a ADT of 14,000 is constructed. The test section is constructed in such a way that it has a minimum of four lanes, a length of 1000 feet, and has no intersections, ramps or curves. At the same time a control section is constructed with an already approved aggregate. Frictional tests are conducted on the test section and the control section by the State Materials Office at a speed of 40 miles per hour in accordance with ASTM E274-85, using both Rib and Blank test tires. Additional testing is done at a speed of 60 miles per hour, if found to be necessary. Friction tests are continued until the accumulated traffic reaches six million vehicles and/or till the friction number stabilizes. Results of the test section and control section are then compared. Approval to the new aggregate source is given only when the comparison is favorable.

References

1. Section 14-103.005, Supplemental Source Requirements for Alternate Open Graded Friction Course (FC-2) Aggregate, Chapter 14-103, Aggregate Source Approval, Rules of the Department of Transportation. Florida Administrative Code, Vol. 7.
2. Questionnaire on Skid Accident Reduction Programs of Different State Departments of Transportation, Response from Contact Person in Florida DOT.

Kentucky of Transportation

KYDOT classifies aggregates based on Acid Insoluble Residue Test results. It has identified aggregate sources which have demonstrated satisfactory polish resistant qualities when used in surface mixes. In addition to meeting the polish resistant requirements, sampling and testing of aggregates is performed on a individual project basis to evaluate the polish resistant qualities of aggregates.

General Procedure

Aggregates supplied for use in surface courses should meet the requirements of the KYDOT Standard Specifications sections 804 and 805, Special Notes, and restrictions contained therein. Acid Insoluble Residue (AIR) Tests(KM 64
223) are conducted to ascertain polish resistant qualities of aggregates to be used in fiction courses. Aggregates from sources demonstrating satisfactory resistance to polish are classified into classes A or B based on test results.

Class A Aggregate Sources Aggregates from a source exhibiting a minimum AIR of 50 percent or greater are considered Class A. An aggregate is also considered Class A based on satisfactory skid resistant performance in another state, laboratory tests, and field tests on experimental test sections. Blending of aggregates from different Class A sources is approved on a project by project basis upon request to the Division of Materials Central Laboratory. KYDOT has identified Class A polish resistant aggregate sources which supply crushed gravel, crushed slag, crushed quartzite, crushed siltstone, crushed sandstone, crushed granite, trap rock, dolomite and limestone. Dolomite and Limestone aggregates are permitted as a coarse aggregate for all uses, except as surface courses of interstate highways.

Class B Aggregate Sources Pending further investigation, aggregates which are restricted from being used in polish resistant bituminous surface mixes are considered Class B. Aggregates from sources listed under class B are allowed to be used only if the project bid item permits and in accordance with 'Special Note for Polish Resistant Aggregate Requirements of sections 804 and 805'. Upon satisfactory history of performance of an aggregate from class B, it can be added to the source list of class A. Aggregate sources are removed from the list of polish resistant aggregate sources if they exhibit poor performance. The following are requirements for an aggregate source to be in class B:

Limestone aggregate source Aggregates have to exhibit abreast 15% AIR content when tested in accordance with KM 64-223 or should hay) a positive indication of satisfactory resistance to polish before its use is allowed in a polish resistant portion.
Gravel Aggregate source Aggregates from these sources have to exhibit 15
50% AIR content when tested in accordance with KM 64-223.
Dolomite aggregate source Dolomite aggregates have to exhibit atleast 37% of $MgCO_3$ present in then when tested in accordance with KM 64-224 or else they should have satisfactory skid resistance. In addition, dolomite aggregates should not have absorption greater that 3% when tested in accordance with KM 64-607.

Louisiana Department of Transportation

Louisiana DOT considers friction in the design of new pavements by controlling aggregate quality. Polish Value Test has been adopted as a laboratory procedure for controlling aggregate quality. Based on the results of the Polish Value Test aggregates are assigned a frictional rating. This frictional rating of an aggregate determines the maximum allowable percentage of that aggregate to be used in a mix design.

General Procedure

The Louisiana Department of Transportation uses the Polish Value Test as a means of controlling the quality of aggregates that are used in friction courses. Aggregates to be used in friction courses should conform with Subsection 1003.01 and they can be either crushed gravel, crushed stone, crushed slag or lightweight aggregate. In addition, aggregates are assigned a friction rating according to Subsection 1 003.06(a) and the procedure of assigning friction rating is as follows:

Friction Rating	Description
I	Used for aggregates with a Polish Value of 37 or greater and which demonstrate an ability to retain acceptable friction numbers for the service life of the pavement.
II	Used for aggregates which have a Polish Value of 35 to 37 and demonstrate an ability to retain adequate friction numbers for the life of the pavement.
III	Used for aggregates having a Polish Value of 30 to 24 and which demonstrate an ability to retain adequate friction numbers for the life of the pavement.
IV	Used for aggregates with a Polish Value of 20 to 29

Based on the friction rating of aggregates they are allowed in particular types of mixes. The allowable usage of coarse aggregates shall be as follows.

Friction Rating	Allowable Usage
I	All Mixtures
II	All Mixtures
III	All Mixtures except mix Type 8F WC†
IV	All Mixtures except mix Types 3WC†, 8WC of 8F WC

† For Type 8F WC mix at least 30 percent by weight of the total aggregate by volume shall have a friction rating T or at least 50 percent by weight of the total aggregate by volume shall have a friction rating of II. An additional requirement imposed is that, not more than 10 percent of these materials shall pass the No. 10 sieve.

‡ For Type 3-WC mix when the ADT/Lane is greater than 1000 vehicles per day, a minimum of 50 percent by weight of the coarse aggregates used in the mixture should have a friction rating of I or II The aggregate used in Type 3 WC mixtures may also be used in the construction of shoulders, drives, curbs, and detours. The above mentioned aggregate requirements are in addition to those mentioned for mix types 8, 8F, and 3 in section 501.02,(c),(5) a, b and c.

References

1. Section 501, Asphaltic Concrete Mixtures,
2. Section 1003.05, Aggregates for Asphaltic Surface Treatment,
3. Section 1003.06, Aggregates for Asphaltic Mixtures.
4. Standard Code of Specifications, Louisiana Department of Transportation.

Michigan Department of Transportation

Aggregates used in the construction of surface courses of the traveled roadway must meet Aggregate Wear Index(AWI) requirements. All aggregate are rated for polishing resistance. Each aggregate is assigned an AWI based on the results of Wear Track Testing (MTM 111) and/or Petrographic Analysis (MTM 112) of representative sample of the aggregate. Only aggregates with suitable A WI members are permitted to be used. Blending of aggregate is permitted provided, the blends achieve the desired AWI requirements when used in a mix.

General Procedure

Michigan DOT employs coarse aggregates, in dense graded, and open graded mix designs. All aggregates are subjected to tests as outlined in section 8.02.02 of the Standard Specifications for Construction. The gradation and physical requirements for the aggregates employed are listed in tables 8.01-1 and 8.02-2 of the standard specifications.

Table 8.02-2 provides specifications on the minimum percentage of crushed material, maximum percentage loss by Los Angeles abrasion (MTM 102), maximum percentage of chert, maximum percentage of Freeze Thaw Dilation per 100 cycles, and maximum percentage of sum of soft particles and chert for gravel, stone, and crushed concrete. The table also provides specifications on maximum percentage of sum of soft particles and chert, and maximum percentage of Freeze Thaw Dilation per 100 cycles for slag aggregates.

In addition to the above specifications, Aggregate Wear Index (AWI) is specified for a roadway. Wear Track Testing (MTM 111) and Petrographic Analysis (MTM 112) are two test procedures employed to determine the AWI numbers. The two laboratory test procedures are briefly described below.

Wear Track Testing(MTM 111) In MTM 111, sieve analysis is performed on selected aggregate sample. Aggregate particles in the size range of 3/8 in and retained on No. 4 sieve are separated and they are placed in etch treated steel specimen molds. The molds are trapezoidal in shape, with dimensions of the parallel sides being 15-1/2" and 19-1/2", and the non parallel sides being 11" in. The depth of the mold is 1-1/2", yielding a test slab of the same thickness. Portland cement mortar is poured in the mold containing aggregate particles. Wire reinforcement is provided in the mold whenever necessary. The surfaces of the slabs are brushed and cleaned after curing them for 24 hours. The slabs are then cured for 7 days in moist air and 14 days in air. Sixteen slabs are needed to conduct a wear track test.

Initial friction values of the cured slabs are obtained using a static friction tester. The slabs are now clamped in place on a circular test bed of 7 fit diameter set on a concrete pedestal. Polishing is accomplished by two 15" smooth treaded tires (ASTM E-524) mounted on a horizontal cross arm. Each polishing wheel is spring loaded to 800 lbs to simulate the weight of a vehicle. The circular track is then subjected to half a million wheel passes. The specimens are skid tested on a static skid test device containing a 15" smooth tread test tire(ASTM E-524) mounted in a frame work containing a calibrated load cell. The specimens are wetted by a recirculating water sprayer. The static test tire is rotated to a speed of 40 mph before bringing it in contact with the test slab. A high speed oscillograph records the torque generated by the contact of the test tire and the slab. Static skid tests are done on slabs at intervals of half million wheel passes upto four million wheel passes. A set of eight static skid tests constitute a complete set of skid tests for a wear track test series. The static skid test values at half million and one million wheel pass intervals are dropped. The least square line is computed using the remaining six static skid test values. The static skid test value on the least square line corresponding to four million wheel passes is reported as the AWI for the aggregate sample used in wear track testing.

Revised Informational memorandum, #374-R, dated 26 June 1990 provides the AWI number requirements based on ADT values.

ADT	AWI (Minimum)
Less than 100 per lane	No Requirement
100 to 500 per lane	220
500 or greater per lane	260

Minnesota Department of Transportation

Friction in surface courses of pavements is considered by controlling the quality of aggregate used in pavement construction. Aggregate used in pavement construction are classified into five different classes. Acid Insoluble Residue (AIR) test has been adopted by the Minnesota DOT as a means of classifying aggregates.

General Procedure

The Minnesota DOT provides specifications for aggregates to be used in bituminous mixtures. Aggregates used in pavement construction should conform to any one of the five classes described in article 3139.2, Composition Graded Aggregates for Bituminous Mixtures. The different aggregate classes are:

Class	Aggregate Use
A	crushed quarry or mine trap rock, quartzite, granite, other igneous or metamorphic rock as approved,
B	crushed quarry or mine rock, carbonate, rhyolite, shcist,
C	natural or partly crushed natural gravel,
D	100 percent crushed natural gravel,
E	steel slag or a blend of any two or more aggregates from classes A, B and D. Steel slag is used only in wearing courses with a maximum allowable percentage of 35 by weight of the total aggregate. Minnesota DOT must approve a class E aggregate before it is used in pavement construction

Aggregates used in wearing courses shall be crushed stone conforming to classes A, D or a combination of both. Carbonate stone is not allowed for use in wearing courses.

Sampling and testing of aggregates are performed in accordance with the MNDOT Bituminous Manual. Los Angeles Rattler Loss (LA Abrasion) is performed to check quality requirements of aggregates with percentage loss on coarse aggregate fraction not exceeding 40 percent. Magnesium Sulfate tests are performed to determine the soundness of aggregates. MNDOT mentions the use of Acid Insoluble Residue(AIR) Test to determine the frictional characteristics of aggregates. The information on AIR tests provided to us is insufficient to make any comments on the AIR procedure adopted by MNDOT.

References

1. Section 3139 with sub sections 3139.1 3139.2 3139.3 Standard Code of Specifications. Minnesota State Department of Transportation.

Mississippi Department of Transportation

MSDOT considers friction in the design of new pavements by controlling aggregate quality through laboratory test. . Petrographic analysis and number of fractured faces, of crushed aggregates are two laboratory procedures adopted by MSDOT. Aggregate types and limiting percentage of crushed limestone are means of controlling aggregate quality.

General Procedure

MSDOT specifies that at least 90 percent by weight of the combined aggregates retained on # 4 sieve shall have two or more mechanically fractured faces. Crushed limestone is permitted for use in Hot Mix Asphalt Concrete MANIAC) and seal coat surfaces, provided that limestone shall not exceed 30 percent of the combined aggregate weight retained on the # 8 sieve and/or limestone shall not exceed 30 percent of the total combined aggregate by weight passing through # 8 sieve.

Standard Operating Procedures(S.O.P) No: TMD-23-01-00-000 of June 1, 1978 provides the guidelines for aggregate sampling, testing, inspecting and reporting. Guidelines are provided for the approval of good quality aggregate sources. A petrographic analysis of the aggregate deposit is performed only when it is considered necessary.

References

1. Completed questionnaire on Skid Accident Reduction Programs of Different State Departments of Transportation.
2. Standard Operating Procedures No TMD-23-01-00-000, Issued on June 1 1978, Mississippi State Department of Transportation.

New Jersey Department of Transportation

New Jersey Department of Transportation (NJDOT) considers friction in the design of new pavements by controlling the quality of aggregate used in mix design. Bureau of Research conducted studies to develop a laboratory test method for prequalifying aggregates based on polish value of aggregates. NJDOT specifies aggregates to he used in bituminous mixtures in section 901 of the Standard specifications for Road and Bridge Construction-1989.

General Procedure

NJDOT recognized the need to prequalify aggregates used in pavement construction based on their frictional properties. The Bureau of Research conducted a Skid Resistance Implementation Study and submitted a report (FHWA/NJ -94-002-7750) to NJDOT in May 1994.

The objectives of the study conducted by the Bureau of Research were to develop a laboratory procedure for qualifying aggregates based on expected terminal skid resistance. The expected terminal skid resistance of a pavement surface is the constant terminal skid resistance value after approximately two million vehicle passes over a pavement surface. Field skid tests in accordance with ASTM E-274 were performed to determine skid numbers at 40 mph. A regression model of the form shown below was used to predict the terminal skid number.

$$SN_{40} = SN_{Terminal} + B1.\ Sin[(\ B2.(JDay) + B3\ ) \qquad (1)$$

where

SN_{40}	=	Skid number measured at 40mph
$SN_{Terminal}$	=	Terminal value of skid resistance
JDAY	=	Julian Calendar Day
B1	=	estimated regression coefficient accounting for variations of the seasonal effect and between 1.3 and 3.0. NJDOT adopted a nominal of 3.0 for B1was found to range
B2	=	a constant for converting the annual seasonal cycle to 360 degrees. The value of B2 is 0.986.
B3	=	estimated regression coefficient for lateral displacement of the seasonal effect. B3 was found to be equivalent to 2 days and was ignored by NJDOT.

The model is now reduced to a simple equation which is used to determine the terminal skid resistance.

$$SN_{40} = SN_{Terminal} +3.0.Sin(0.986(Jday)) \qquad (2)$$

The terminal skid resistance data was obtained at 26 field sites using equation 2. Pavement cores were obtained from these sites. Polish values of aggregate samples obtained from the pavement cores were determined in the laboratory. A linear regression model was fitted to the terminal skid resistance and polish value data. The regression model is given by equation 3.

$$SN_{Terminal} = 0.5 + 1.37\ PV \qquad (3)$$

where -0.5 and 1.37 are regression constants and PV is the polish value of the aggregate sampled determined in the laboratory. NJDOT observed that the terminal skid resistance value of a pavement surface was greater than the polish value of the aggregates determined in the laboratory. The terminal skid resistance is representative of performance of the pavement matrix whereas polish value reflects the characteristics of the aggregate samples.

Equations 2 and 3 are used to predict the field performance of candidate aggregate samples. Assuming that equation 3 is correct, the procedure for evaluating candidate aggregates involves the selection of a desired terminal skid resistance value for a pavement surface. A slight variation is allowed to this value to account for seasonal variation. The values of terminal skid resistance are now used in equation 3 to determine the minimum expected laboratory polish value required of the candidate aggregate samples. Polish value test is conducted in the laboratory on seven specimens of the aggregate in accordance with ASTM D 3319. The polish value of the specimens are measured at 0, 1,2, and 4 hours after the test is begun using a British Pendulum Tester. A regression line is fitted to the four observed polish values. The line is of the form described below

$$Y = A + BX \qquad (4)$$

where

Y	=	average of the polish values measured in the lab
X	=	1/(t+1), t = 0, 1, 2, 4 test duration in hours
A	=	constant term obtained from regression analysis
B	=	regression coefficient

The value of the constant term A must be greater than the minimum required polish value and it is tested against the polish value of a control aggregate sample. Upon satisfactory performance the candidate aggregate samples are approved for use in pavement construction.

The procedure described above is being used by the NJDOT to approve candidate aggregate sources for the past two years. The entire procedure is based on the assumption that equation 3 is correct. It has been cited by NJDOT that the evaluation procedure is effective at 50 percent confidence interval limits. No information is provided at this time as to the inclusion of above mentioned aggregate evaluation procedure in the standard specifications of NJDOT.

References

1. Skid Resistance Implementation Study, New Jersey Department of Transportation, Bureau of Research, FHWA/NJ-94-002-7750, May 1994.

Oklahoma Department of Transportation

Classified category IV Oklahoma Department of Transportation (OKDOT) considers friction when designing and constructing new pavements. Acid Insoluble Residue (AIR) Test is used by OKDOT as a laboratory procedure to screen aggregates for surface courses. On conducting AIR test for candidate aggregate samples the percentage loss i.9 determined. For aggregates to be used in surface a maximum percentage loss of 30 percent is fixed. Candidate aggregates exhibiting percentage 10.9s greater than 30 are no, allowed to he used in surface courses.

A cut off field skid member of 35 is fixed by OKDOT. Whenever the skid number of a pavement falls below 35 rehabilitation step. are undertaken. Constant monitoring of pavement surfaces is not undertaken by but whenever possible problem areas are identified. No adjustments are specified to account for seasonal changes or for variations in speed during field skid testing.

Reference

1. Summary of conversation with contact person in Oklahoma Department of Transportation.

Pennsylvania Department of Transportation

Pennsylvania Department of Transportation (Penn DOT) classifies aggregates into five different group. This classification governs the use of aggregates for different pavement sections depending on the amount of traffic in each section. Aggregates are classified based on the result of Petrographic Analysis, Accelerated Polishing Test, and Acid Insoluble Residue(AIR) Tests. The three types of tests are a means of controlling the quality of aggregate used in pavement construction. Penn DOT also uses past field performance of aggregates as a technique for aggregate classification.

General Procedure

Aggregates for surface courses are classified based on their Skid Resistance Level (SRL) as follows

Classification based on SRL	Rock Type
L - Low	Limestone and few finely Textured Dolomites
M - Medium	Dolomites and some types of Limestone
G - Good	Siliceous Dolomite and Limestone, Gravel with over 25% Carbonate
H - High Grade	Gravel with over 10% carbonate, Quartzite, Siltstone, Argillite, Gneiss, Diabase and Blast Furnace Slag
E - Excellent	ravel and Sandstone

Aggregates passing through the general classification listed above are used in pavement construction. The use of a certain type of aggregate will be governed by the amount of traffic in each pavement section. The skid resistance level (SRL) of aggregates is related to average daily traffic volume as in the following table.

Average Daily Traffic	SRL Required
1000 and Below	E, H. G. M, L
1000 to 3000	E, H. G. M
3000 to 5000	E, H. G
5000 to 20,000	E or H
20,000 and Above	E

Three laboratory techniques namely Petrographic Analysis, Accelerated Polishing Test and Acid Insoluble Residue Test are used to determine the SRL of aggregates. However, Petrographic Analysis is used as the main indicator of SRL and it provides information on the type or rock, grain size, matrix and extent of weathering. The accelerated polishing method used by Penn DOT is similar to the one used by Texas. The results of the Polishing Test are used as a supplement to Petrographic Analysis. AIR used to evaluate aggregate frictional properties when it is difficult to evaluate frictional properties by petrographic analysis. The aggregate rating system is qualitative and is not an automated system.

Penn DOT also makes use of performance history of aggregates used in pavement construction. An aggregate is upgraded in class depending on its performance history. The performance history should be available on atleast 10 projects and for a period of atleast 2 years. Aggregates from new sources are tested in the laboratory and depending on the results they are approved for construction. Pavements constructed with aggregates from new sources are followed up with skid testing to ensure adequate performance with respect to skid resistance.

References

1. Pavement Restoration, Resurfacing and Rehabilitation, Circular Letter Dated 6/10/82 Commonwealth of Pennsylvania - Department of Transportation.
2. The Relationship of Skid Resistance to Petrography of Aggregates - Final Report, Furbush, M.A., and Styers,K.E., Bureau of Materials, Testing and Research, Commonwealth of Pennsylvania - Department of Transportation.
3. Summary of Telephonic Conversation with Contact Persons in Penn DOT.

Tennessee Department of Transportation

Tennessee Department of Transportation (TNDOT) considers friction in the design of new pavement surfaces by controlling the quality of aggregate used in pavement construction. Aggregates are classified in to three types based on skid resistance performance Polish Value Test. and Acid Insoluble Residue (AIR) Tested are procedure used in the laboratory to evaluate frictional Properties of aggregates.

General Procedure

Coarse aggregate used for pavement construction usually consists of crushed gravel, crushed granite, crushed slag, crushed quartzite, crushed calcareous sandstone and crushed gneiss. Aggregates are classified into three types and any aggregate used in pavement shall have physical, chemical and performance characteristics of either of the three types. The three types of aggregate classification are briefly discussed below.

Type I — Aggregate are considered as type T upon exhibiting a minimum of 50 percent silica dioxide content and a maximum of 32 percent calcium carbonate content. The ATR results of a coarse aggregate of type I should contain a minimum of 50 percent by weight of original sample of AIR that are coarser than the No. 100 Sieve. Aggregates of this type should have a minimum Polish Value of 33.

Type II — Type II aggregates should have a minimum 30 percent of silica dioxide content and a minimum of 35 percent by weight of original sample of ATR that is coarser than No. 100 sieve. Aggregate in type II should have a minimum polish value of 30.

Type III — Aggregates classified as type III should have a minimum 20 percent of silica dioxide content and a minimum of 25 percent by weight of original sample of ATR that is coarser than No. 100 sieve. The minimum expected polish value of type III aggregates is 25.

The use of carbonate rocks such as limestone and dolomite is not permitted in the coarse aggregate. An aggregate when combined with required amount of bitumen the resultant mixture shall have the following characteristics.

Characteristic	High Volume Roads (ADT over 1000)	Low Volume Roads (ADT 1000 or below)
Min stability (lb)	2000	1000
Void Content (%)	3-5.5	2-5
Flow	8-16	8-16
Min. VMA	14	-
Dust to Asphalt Ratio	0.6-12.	-

The dust to asphalt ratio is defined as the percent of the total aggregate sample that passes the 200 mesh sieve as determined by AASHTO T - 1 1 divided by the percent asphalt in the total mix.

When a asphalt mix designated as 'Grade E' by TNDOT, is used for traffic lanes, the mineral aggregate shall be composed of not less than 50 percent nor more than 80 percent of crushed limestone, and not more than 50 percent or not less than 20 percent natural sand, slag sand or sand manufactured from gravel. Limestone is allowed to be used in a grade E mix when constructing shoulders or other non traffic lane construction.

Several different aggregates are used in the state of Tennessee. Gravel with 3 5 percent absorption is used in western parts of the state. Mine slag and siliceous carbonate gravel with varying silica contents are used in the middle parts of the state. Gravel, slag and granite from North Carolina, West Virginia and Virginia are used in Eastern Tennessee.

References

1. Section 9030A, Revised 2-14-94, Standard Specifications of Tennessee Department of Transportation.
2. Summary of Telephonic conversation with contact person in Tennessee DOT.

Texas Department Transportation

Classified in category V, Texas Department of Transportation(TXDOT) considers friction of surface courses when designing new pavements. Friction of surface courses is taken care by controlling the quality of aggregates used in pavement construction. TXDOT uses, the Polish Value Test to evaluate the frictional quality of aggregate. Based on the results of the Polish Value Test aggregate sources are rated for rise in pavement construction. Of late TXDOT has found that Polish Value Test its not a good indicator of the frictional properties of aggregates. Past field performance of aggregates used in pavement surface is used to evaluate aggregate source.

General Procedure
Item 302 of TXDOT's Standard specifications for construction of Highways, Streets and Bridges provides specifications on aggregates for surface treatments. Aggregates used for surface treatments shall be composed of gravel, crushed gravel, crushed stone, crushed slag or natural limestone rock asphalt and should comply with specifications mentioned in section 302.2 of item 302. Some of these specifications are tabulated below.

Table for Specifications for Aggregates for Surface Courses

Test Method	Specification
Tex-217-F, Part I	Not more than 2.0 percent by weight of soft particles and other deleterious materials are allowed in aggregates.
Tex-217-F, Part II	Not more than 1.0 percent loss from fine dust, clay like particles and /or silt allowed
Tex-224-F	Flakiness index for aggregates shall not exceed 17 unless otherwise stated on plans
Tex-410-A	Percent wear shall not exceed 35 percent.
Tex-460-A, Part I	Crushed gravel shall have a minimum of 85 percent of the particles retained on No: 4 sieve with two or more mechanically induced crushed faces.
Tex-411-A	The loss from 5 cycles of magnesium sulfate soundness test shall not exceed 25 percent.
Tex-438-A	Polish Value of the aggregate shall not be less than the value shown on the plans. Polish value requirement is applicable only to aggregate used on travel lanes

Based on the results of Polish Value Test an aggregate source is either approved or rejected. The Materials and test Division of TXDOT uses the Rated Source Polish Value (RSPV) for an aggregate source to prepare a catalog of approved aggregate sources. When aggregates are supplied from sources that are not rated, Polish Value Tests are conducted on the aggregate samples in accordance with test methods Tex-400-A and Tex-438-A, Part I. Blending of aggregates to obtain the required Polish value is sometimes permitted, but it is allowed depending on the requirements of the project. Test method

Tex-38-A, Part II, Method B is used to the determine the blend percentages. However, In blends a minimum of 50 percent by volume should be non polishing aggregates.

The specifications of Polish Value Test eliminates aggregates from sources which fail to meet the Polish Values requirements. This elimination procedure classifies most of the aggregate sources in the State of Texas as failing to meet the Polish Value Test specifications. In order to overcome the elimination of aggregate sources, TXDOT adopted the use of historical field performance of aggregates in surface courses as a means of rating aggregate sources. TXDOT adopted FHWA guidelines for evaluating aggregates based on their historical field performance.

References.

1. Item 302, TXDOT's Standard specifications for construction of Highways, Streets and Bridges - 1993.

UTAH Department of Transportation

Utah Department of Transportation(DOT) considers friction of surface courses while designing new pavements. The quality of aggregate used in surface courses is controlled by Polish Value Test. The test is used to screen aggregates for skid resistance. The experience of Utah DOT has shown that Polish Value Test is not a reliable indicator of aggregate frictional properties. Sometimes aggregates with lower Polish Values have performed better in the field. Utah DOT is considering to opt for a chemical analysis procedure to determine the amount of carbonates in aggregates.

A cutoff value of 38 is used to qualify aggregates when tested with British Pendulum Tester to determine the British Pendulum Number (BPN). The Utah DOT experiencing problems regarding the rate at which the fine silica carbide grit is applied during Polish Value Testing. Hence the cutoff value of If is undergoing research which might he changed.

References

1. Summary of telephone conversation with the contact person in Utah DOT.