Incorporating Road Safety into Pavement Management: Maximizing Surface Friction for Road Safety Improvements

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16. Abstract
This research explored the relationship between asphalt mix design, skid friction, and roadway safety. Initial tasks attempted to find a relationship between pavement skid resistance (friction) and crash frequency, particularly wet weather crashes. Friction and crash data collected over 10 years at six study sites in Wisconsin were analyzed. The results of the analysis did not indicate a relationship between crash frequency and pavement skid friction. Although some evidence suggests that the number of wet pavement crashes increased as the pavement life increased (and skid friction values decreased), the frequency of crashes was not sufficient to statistically support. Nevertheless, the fact that the relationship seems to behave inversely proportional, that is to say more crashes occurred at low friction numbers (FNs), is an important indication that skid resistance may indeed be a factor affecting wet weather crashes.

It was not possible to determine a skid friction threshold value that indicates the critical point where pavement maintenance would be needed. Although the data obtained in the research could not support a specific value, it is clear that friction values less than 35 are problematic from a safety standpoint. A possible indicator of friction on high-speed roadways is macrotexture. Therefore, macrotexture (measured as MTD) combined with friction data was of great interest in this research. Plots of MTD and FN values did not show a clear relationship between the two values, although it was evident that the larger FNs were concentrated in low MTD values.

Skid resistance is an important feature which should be considered while evaluating roadway safety. An effective asphalt pavement asset management approach will include an annual testing program to monitor skid friction values. FN values less than 35 should trigger a safety monitoring program and those pavements scheduled for future rehabilitation or reconstruction.

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DISCLAIMER

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EXECUTIVE SUMMARY

Traffic crashes and the associated injuries and fatalities pose a significant problem for transportation safety professionals. To overcome this problem numerous road crash investigations and statistical analyses have been done. These studies suggest that there is a distinct relationship between crash frequency and pavement surface conditions that should be taken into consideration to potentially improve safety on our highway network.

This research study endeavors to find a relationship between pavement skid resistance (friction) on asphalt pavements and crash frequency, particularly wet weather skid crashes. The hypotheses developed were that: (1) crash frequency is inversely proportional to skid friction values; and (2) a skid friction threshold exists at which safety of the roadway system decreases with values at or below this threshold. Additionally, the research explored the relationship between pavement macrotexture (large scale texture) and friction for high speed roadways.

To determine the correlation between pavement skid friction values and crash frequency, it was necessary to collect data pertaining to these two variables. A series of tasks were developed to acquire data and other information that provide guidance and support to achieve the objective of this research. Asphalt pavement skid resistance (measured as friction number at a given test speed and tire type, FN40R and FN50R) and crash data for study sites in Wisconsin from 1994 to 2004 were studied and analyzed.

Results of the data analysis did not indicate a statistically significant relationship between crash frequency and asphalt pavement skid resistance. Hence, statistical support for either hypothesis was not found. However, there was some evidence which suggested that the number of wet pavement skid crashes increased as the pavement life increased (and consequently skid friction values reduced). The frequency of crashes was not sufficiently large to support this claim statistically. Nevertheless, the data shows an inversely proportional relationship between FN values and crash frequency indicating that skid resistance on asphalt pavements is a factor affecting wet weather crashes.

Results of the study were hypothesized to show a directly proportionate relationship between macrotexture (measured as mean texture depth, MTD, use the Sand Patch Method) and skid resistance. In other words, the larger the MTD value, the larger the FN. However, the results of the data analysis for the study sections revealed the relationship was inversely proportional. There were no observable relationships shown on plots comparing MTD to FN data, although it was evident that the larger FNs were concentrated in low MTD values.

If a statistically significant relationship existed between MTD and pavement FN, then skid resistance can be predicted from volumetric measurements. Volumetric measurements could then help in the design of pavement mixes that yield high FN, consequently increasing the safety of the road by maximizing grip in the tire-pavement interaction. Pavement mixes that yield high FNs theoretically can provide roadway users with greater braking distance, a critical factor in reducing crashes. Although no statistically significant results were obtained, it has become clear that skid resistance is an important factor which should be considered when evaluating roadway

safety. Research around the globe has demonstrated that there is a skid resistance component in the incidence and frequency of many crashes.

This research has generated two key findings that can directly impact an asset management approach to asphalt pavement skid friction safety. First, although there was not a strong statistical correlation found between skid friction values and safety, the results of this research, supported by the literature, show that FN40R values less than 35 negatively contribute to safety under wet pavement conditions. Therefore, pavement sections identified with FN40R values at or near 35 should be programmed for rehabilitation or reconstruction activities. Second, it is possible to develop asphalt mix designs with properties that maintain the integrity of the design while developing a higher initial FN40R value.

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CHAPTER I

INTRODUCTION

Traffic crashes and the associated injuries and fatalities remain a significant problem for transportation professionals. In 2005, almost 2.7 million transportation-related injuries and 43,443 fatalities were reported in the United States as a result of over 6.1 million crashes (*1*). Wisconsin and the Federal Highway Administration (FHWA) Region Five States (Illinois, Indiana, Michigan, Minnesota, and Ohio) accounted for 6,125 of the 2005 fatalities, approximately 14 percent of the national total. When considering the state-by-state traffic fatality totals in FHWA Region Five, only Michigan observed a one-year decrease in total crashes. Wisconsin observed a three percent increase in crashes – the largest increase of the Region Five states.

Crashes and fatalities continue at unacceptable rates although safety-related implementations, ranging from stricter safety laws and public awareness campaigns to roadway and traffic control device improvements, continue to be employed. Much more is needed when one considers that the current fatality rate equates to one death every 12 minutes, or approximately 117 fatalities each day. Additionally, traffic-related incidents have a significant financial impact. For the year 2000, the cost of motor vehicle crashes in the United States was estimated to be $230.6 billion – money that could be used in other areas if the number of traffic-related crashes were reduced (*1*).

Traffic crashes are considered a unique, random, multi-factor event many times preceded by a situation in which one or more persons have failed to cope with their environments. Most often contributing factors are categorized under driver, vehicle, and roadway, each containing multiple sub-variables. Because of this complexity, and the difficulty identifying the human factors elements (i.e., human behavior at the time of crash), most crashes are considered to be the result of driver error. Therefore, the role of the transportation professional is to improve the roadway system such that the consequences of driver error are minimized. Roadway system improvements involve a comprehensive analysis of all prevailing factors at the time of crash, including those of the pavement surface, to determine the countermeasures (i.e., safety improvements) necessary to prevent crash reoccurrence.

A relationship between crash frequency and pavement surface texture is often overlooked as a significant contributor to roadway safety. It is well documented that a pavement with high skid resistance properties can be a significant factor in reducing the likelihood of a crash. Skid resistance is related to properties of both the vehicle tire and the pavement surface, and can affected by volume and composition of the traffic load, available tire tread depth and pattern, pavement temperature, the presence of water (rain), and other pavement surface conditions. Skid resistance can be measured by skid number (SN), also known as friction number (FN), which is the commonly used term for U.S. state highway agencies. For example, the Wisconsin Department of Transportation (WisDOT) has developed models for predicting FN from pavement material properties, age of the pavement, traffic, and climate. Although state highway agencies may utilize differing models, they are all in agreement that pavement surface characteristics are of prime importance.

The pavement surface must provide an adequate level of friction at the tire-to-pavement interface for safe operation of vehicles under normal weather conditions. Pavement surface texture is defined by two parameters: the microtexture and the macrotexture. Microtexture is the fine-scale (≤ 1mm depth) grittiness on the surfaces of the coarse aggregates. The microtexture is responsible for direct tire-to-pavement contact, thereby providing the resistance to skidding, i.e., non-rotating tire drag on pavement, on the prevailing road surface. Macrotexture is the large-scale roughness that exists at the pavement surface due to the arrangement of the aggregates and controls the drainage ability of the pavement. The combination of microtextures and macrotextures, along with the way these textures change due to traffic and climate factors, make up a particular pavement's overall resistance to skidding.

Problem Statement

Deterioration of the pavement (smoothing or polishing of the pavement surface), along with surface water accumulation in the form of rain, snow, or ice, can result in inadequate provision of skid resistance. Inadequate skid resistance can lead to higher incidences of skid-related crashes. There is currently no agreement on what standards to use for optimizing skid resistance, despite a significant amount of research conducted.

Roadway pavements deteriorate with time as a result of repeated vehicle passes, environmental conditions, and at times, poor pavement management. If this deterioration is not properly addressed, the amount of surface distress that can affect skid resistance will increase and be prejudicial to traffic. In this sense, pavement surface characteristics are a significant issue because of its influence in preserving roadway safety. Maintaining these characteristics during pavement construction or rehabilitation may mitigate or even prevent crashes and incidents related to loss of vehicle control, hydroplaning, and/or excessive skidding.

Numerous road crash investigations and statistical analysis have suggested that there is a relationship between crash frequency and pavement surface characteristics. Additionally, a recent paper has suggested that little has been done to incorporate pavement management and maintenance into roadway safety evaluations (*2*). Thus, research is needed to confirm the relationship between crash frequency and pavement surface characteristics, and consequentially develop strategies to incorporate these findings in roadway safety evaluations.

Objectives

The objective of this research was to evaluate the correlation between road safety and skid resistance of asphalt pavements. Minimum friction threshold values were considered as indicators for the need of roadway maintenance. Specific objectives included:

1. Determine the relationship between asphalt pavement skid resistance and traffic safety;
2. Develop asphalt pavement mix design strategies that consider skid resistance as a measure of effectiveness;

3. Identify existing prediction models for skid resistance, propose modifications to models, and identify minimum skid resistance ranges to trigger the need for roadway maintenance; and
4. Incorporate skid resistance and safety in a pavement asset management tool.

Research Tasks

The following tasks were used to complete this research.

Task 1 - Review the skid resistance criteria used, focusing on the FHWA Region Five States. A literature/phone review was conducted to document any published information or results of studies from national and regional research that related to this issue. An evaluation of pavement management strategies was conducted to explore the use of skid resistance as a measure of effectiveness in asset management models. Consideration of American Association of State Highway and Transportation Officials (AASHTO) stopping, skidding, and side friction models were included along with current skid number prediction models.

Task 2 - Analyze existing Department of Transportation (DOT) skid numbers as related to safety and materials used. Task 2 used the WisDOT skid trailer and existing WisDOT databases to determine skid number for selected new and existing pavement types in Wisconsin. A series of ribbed tire skid tests were completed to correlate expected skid numbers with pavement, type, and age. Traffic volume, mix design, and weather conditions were considered.

Task 3 - Define correlation between road safety and skid resistance. Task 3 used the skid resistance information obtained in Task 2, combined with multiple years of crash data over the evaluated roadways segments, to determine if a statistical correlation existed between crash frequency and skid resistance values.

Task 4 - Develop methods for measuring skid numbers/or other safety indicators for various maintenance materials and/or activities. Field measurements are mostly done with skid trailers using a locked wheel mechanism. Many state agencies maintain a significant database for their highways in terms of field measurements. There is however a lack of acceptable procedures for lab measurements of skid numbers of various pavement materials used in new construction or maintenance activities. Most pavement engineers depend on historical information or qualitative rules to characterize a material as good or poor in terms of contribution to skid resistance. The only exception in this trend is the polishing characteristics of aggregates and the mineral composition of the aggregates. Tests such as the British Pendulum, the carbonate/limestone content, the insoluble residue and the aggregate wear are examples that have been used to evaluate aggregates. It is however yet to be shown that there is a solid and reliable correlation between these basic aggregate characteristics and initial skid numbers, or change in these numbers with service years of asphalt pavements. Many agree that aggregate with higher resistance to abrasion (low wear numbers) and less content of sedimentary type rocks, the better is their resistance to polishing. It is however not clear how the different states are relating this to safety. A recent study by National Center for Asphalt Technology (NCAT) indicated that state highway agencies specify LA wear values between a low of 30 and a high of 65; a rather wide range that does not indicate a consensus on what value is acceptable for safe pavements.

It is also known that asphalt content and voids could have significant effect on skid resistance because of the bleeding and flushing phenomena that could result in asphalt migration to the surface covering aggregates and thus reducing the micro-texture and macro texture. Asphalt content and voids are a part of the routine mixture design process that is carefully monitored and specified by highway agencies. It is thus possible to explore the relationship between mixture design and skid numbers and predict the change as a result of traffic consolidation. Such ideas are totally new and looking into the literature one can find examples of attempts to link mixture design to skid numbers.

While using aggregate characteristics and mixture design to predict skid resistance is useful and practical because of the existence of the data, there is no substitute to direct measures of skid numbers of materials in the laboratory. There has been substantial work done on developing tests for direct measure of skid numbers in the laboratory. Examples include the British Pendulum tester (AASHTO T278) and the British polish wheel (AASHTO 279) which are commonly used for aggregates but could be potentially modified to test a complete and compacted mixture. There are also ideas recently introduced in Europe and Canada such as the Skid numbers from Stereo-Photographs, the Laser Micro-texture analyzer, which is an optical device, and the Tactile Sensor, which is a mechanical device, that could be used to quantify to a high degree of accuracy the surface profile and estimate skid resistance of various materials used in maintenance and preservation of pavements.

In this task, a review of existing technologies was summarized and a feasibility analysis completed conducted to select the two best potential devices to be used in studying various materials commonly used in practice in the FHWA Region 5 states. A comparison of micro and macro-texture for these materials was used in defining the guidelines for incorporating safety in selection of maintenance activities.

Task 5 - Develop guidelines for using skid numbers or other safety measures in project level management including prediction of change during service. The focus of Task 5 was on establishing the relationship between skid numbers, measured in the field or estimated from laboratory measures, and safety of motorists. An evaluation of the relationship between skid measurements and standard safety requirements, such as stopping distance and side skid vales were considered and related to design speed and other highway geometric factors.

Task 6 - Develop guidelines for selecting maintenance activities to meet skid resistance requirements in pavement management tools/programs. The main objective of this research was to develop tools to incorporate safety measures in pavement management programs. Skid resistance is considered a functional distress indicator affecting service level of pavements. In this context, there is a need for developing a guideline for selecting type of treatment required to correct skid resistance when it falls below a certain level that is considered unsafe for motorists. For example, WisDOT uses the Frictional Number and the cut off level is generally FN40R = 35. This research reviewed FN numbers and evaluated the relationship to safety. Guidelines for changes in acceptable range and method of measurement and prediction are proposed.

Task 7 - Prepare and submit final report. Task 7 involved the creation of this final report.

Scope

The scope of this research was limited to friction value performance on Hot Mix Asphalt (HMA) pavements. Portland Cement Concrete (PCC) pavements are also important, but outside the scope of this research. PCC pavements could be the target of a later study. Seal-coated asphalt pavements were also not considered.

CHAPTER II

LITERATURE REVIEW

Skid resistance of pavement is the opposing force developed at the tire-pavement contact area. In other words, skid resistance is the force that resists the tire sliding on pavement surfaces. Figure 1 depicts this relationship. Skid resistance, also referred to as skid friction, is an essential component of traffic safety as it is critical in maintaining vehicle control and reducing the stopping distance in emergency braking situations. The terms *skid resistance*, *pavement friction*, and *skid friction* (or merely *friction*) are used interchangeably in the literature and will be used in the same manner in this report.

Skid resistance has two major components: adhesion and hysteresis (*3*). Adhesion results from the shearing of molecular bonds formed when the tire rubber is pressed into close contact with pavement surface particles. Hysteresis results from energy dissipation when the tire rubber is deformed when passing across the asperities of a rough surface pavement. These two components of skid resistance are respectively related to the two key properties of asphalt pavement surfaces, that is microtexture and macrotexture as presented in Figure 2.

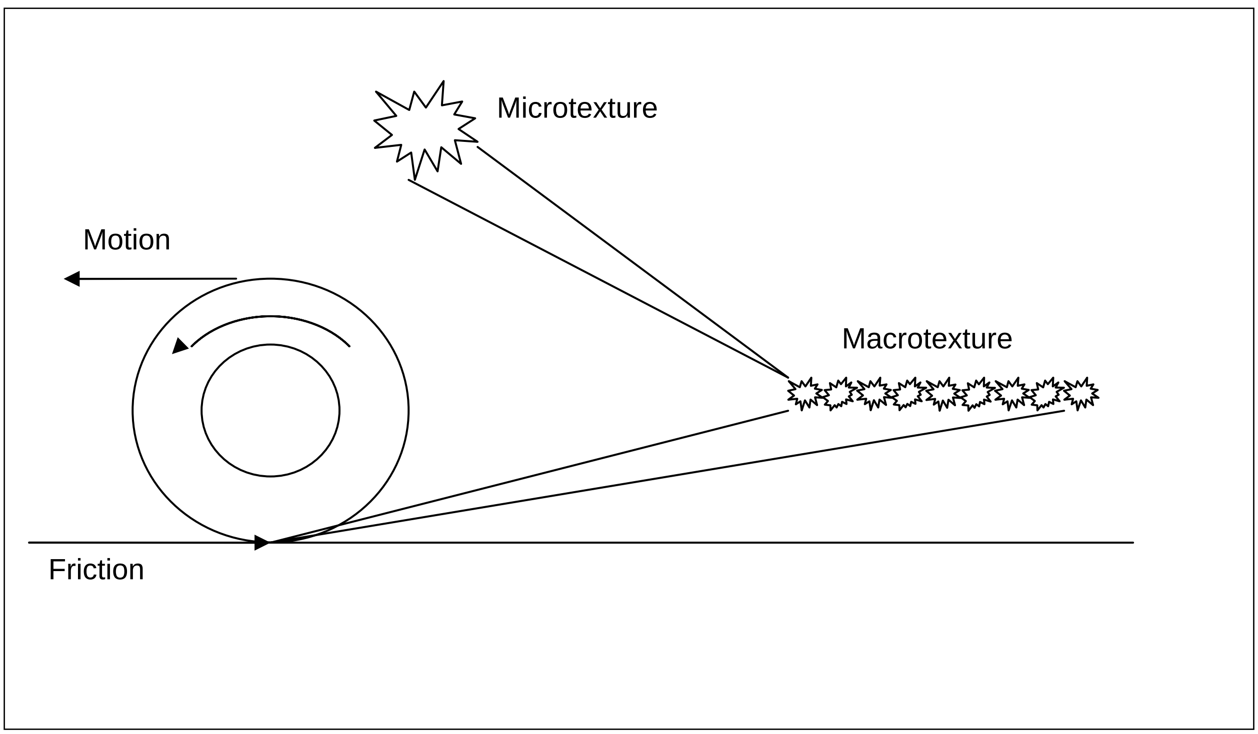


Figure 1 Friction Force and its Properties

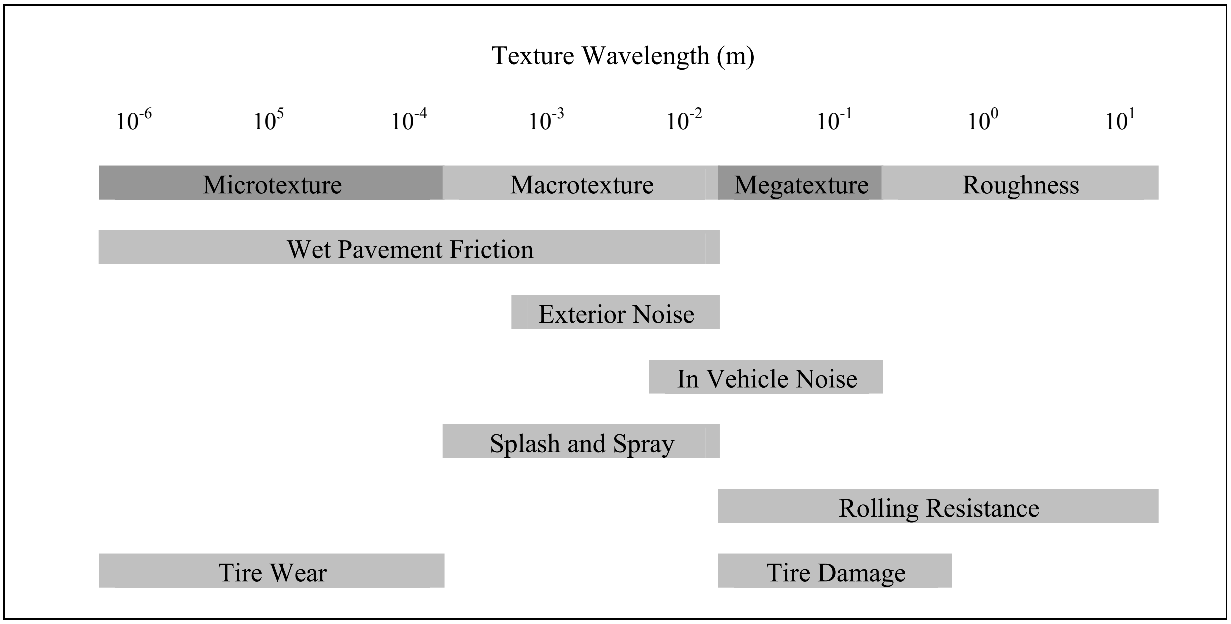


Figure 2 Texture Wavelength (m) Influence on Surface Characteristics (*8*)

Microtexture and Macrotexture

Previous studies are in agreement that both the micro- and macrotexture of a pavement surface influence the skid resistance of that pavement. Microtexture refers to irregularities in the surfaces of the stone particles (fine-scale texture) that affect adhesion. These irregularities are what make the stone particles feel smooth or rough to the touch. The magnitude of microtexture depends on initial roughness on the aggregate surface and the ability of the aggregate to retain this roughness against the polishing action of traffic (*4*). 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 using laboratory test procedures developed for this purpose. Microtexture and adhesion contributes to skid resistance at all speeds and is the prevailing influence to skid resistance at speeds less than 30 mph (*5*).

Macrotexture refers to the larger irregularities in the road surface (coarse-scale texture) that effect hysteresis. These larger irregularities are associated with voids between stone particles. The magnitude of this component will depend on several factors, including the size, shape, and gradation of coarse aggregates used in pavement construction as well as the particular construction techniques used in the placement of the pavement surface layer (*6*). Macrotexture is also essential in providing escape channels to water during the tire-surface interaction, thus reducing hydroplaning. Hydroplaning is the loss of tire traction caused by a film of water that separates the tire from the road surface. Macrotexture and hysteresis are less critical at low speeds; however, as speeds increase a coarse macrotexture is very desirable for safe wet-weather travel.

Two other road surface texture properties, namely megatexture and roughness (unevenness), are less significant than micro- and macrotexture in the generation of skid resistance. However, they are key components in the overall quality of the pavement surface. Megatexture describes irregularities that can result from rutting, potholes, patching, surface stone loss, and major joints and cracks (*7*). The megatexture properties of a pavement affect noise levels and the capacity of a tire to rotate (rolling resistance) more than it affects skid resistance. Pavement roughness refers to surface irregularities larger than megatexture that affect rolling resistance, ride quality, and vehicle operating costs. Pavement roughness provides a good overall measure of the pavement condition and is usually computed through the International Roughness Index (IRI).

Microtexture, macrotexture, megatexture, and roughness are the features of the road surface that ultimately influence most tire-road interactions including wet pavement skid resistance, noise, splash and spray, rolling resistance, and tire wear. At the 18th World Road Congress, the Committee on Surface Characteristics of the World Road Association (PIARC) proposed the texture wavelength range for each of the categories as shown in Figure 2 (*8*). Sandberg listed their influence in more detail in Table 1 (*9*).

Table 1 Influence of Texture on Some Variables (*9*)

Effect on Vehicle, Driver or Environment	Road Surface Characteristic of Importance	Magnitude of the Influence
Skid Resistance (Friction)	Macrotexture Megatexture Microtexture	High Moderate Very high
Rolling Resistance/ Fuel Consumption/ Air Pollution	Macrotexture Megatexture Unevenness	High Very high High
Tire Wear	Macrotexture Microtexture	Moderate Very high
Exterior Noise	Macrotexture Megatexture	Very high Very high
Water Runoff	Macrotexture	High
Splash and Spray	Macrotexture	High
Light Reflection	Macrotexture Microtexture	High Little known
Interior Noise	Macrotexture Megatexture Unevenness	High Very high High

Measuring Skid Resistance

The primary purpose of measuring skid resistance is for quality control during construction and for asset management thereafter. Pavement friction values are used in network surveys for pavement management, evaluation of surface restoration, specifications for new construction, crash investigations, and winter maintenance on highways, amongst other purposes (*10*). Pavement friction is also used at airports for evaluating runway conditions and determining the need for pilot advisories and maintenance activities. Recent developments for evaluating available friction on highways during winter weather maintenance activities include research in Iowa, Michigan, and Minnesota where the incorporation of friction measuring instrumentation (SALTAR) has been used with snow plows to determine the necessary rate of salt application needed to increase skid resistance and maintain vehicle control. Skid resistance values are also taken when a site reveals potential road safety problems (aging, bleeding, water accumulation, and surface contamination) and when the road surface has recently been treated to correct friction problems (*11*).

Skid Resistance Measuring Devices

Skid resistance is generally measured by the force generated when a locked tire slides along a wet pavement surface (*3*). A locked tire enables detection of skidding at the point of contact between the tire and the pavement. Methods of measuring skid resistance vary and often prevent direct comparisons of values between different testing organizations. In 2000, Henry listed 23 devices currently in use for field pavement friction testing purposes (*10*). These devices can be grouped in four categories:

- Locked wheel testers;
- Side force devices;
- Fixed slip devices; and
- Variable slip devices.

Variation in measurement of pavement friction can be attributed to measurement device, test tire, and seasonal effects. Two consecutive friction measurements on a given road surface using one device can vary up to five percent and is further exacerbated when using two different types of devices. Pavement friction is also sensitive to the test tire (ribbed or smooth) and measurements can vary between two tires of the same type. Additional variations also arise because of seasonal effects. Friction levels in spring are expected to be higher than fall because of asperities in surface caused by snow removal activities (*12*). In winter, pavement friction tends to be higher than summer since the binder in the pavement mixture is “absorbed” causing more exposure of the aggregates. However, this effect is not evident when ice or slushy conditions are present.

Locked wheel testers (American Society for Testing and Materials, ASTM E 274) are the most commonly used device in the United States (U.S.) (*10*). In this method, the relative speed between the surface of the tire and the pavement surface is equal to the vehicle speed. For computing a correspondent friction value, the operator applies the brakes and measures the torque for one second after the tire is fully locked. Usually, the left wheel path in the travel lane is tested. A ribbed tire (ASTM E 501) has been predominantly used in the U.S., but the use of the smooth tire (ASTM E 524) has increased in recent years.

Henry and Wambold recommended the use of the smooth tire for field skid testing when only one type of tire is employed, although testing with both tires yields the most pavement information (*13*). The reason behind using the smooth tire is that it is more sensitive to macrotexture and microtexture whereas the ribbed tire responds primarily to microtexture. Research completed by the Florida Department of Transportation (FDOT) indicates that the smooth tire provides skid measurements that are better indicators of safety than measurements with ribbed tires (*10*).

Henry noted a study in Pennsylvania that showed the ability of the ribbed tire to mask the benefits of macrotexture (*13*, *14*). Four PCC pavement sites along Interstate 80 were skid tested on the same day to compare ribbed tire and smooth tire measurements. Sites were tested at 40 mph and included sections with and without grooves. Table 2 shows the results of the study which proved the ability of a smooth tire to distinguish between sections with and without grooves. Friction changes with the ribbed tire are not as noticeable as the smooth tire. In other words, macrotexture can be better detected when using a smooth tire rather than a ribbed tire. However, many agencies prefer the ribbed tire over the smooth tire. The two primary reasons for such preference are that the use of a smooth tire makes friction numbers lower, and does not allowed friction measurements to be compared to the historical frictional performance of the pavement, whose values were taken with a ribbed tire. Ribbed tires are also preferred by some because they are less sensitive to water film thickness than smooth tire. Reasonable care should be applied to control the water flow rates of a skid tester when using a smooth tire. Currently, both tires have equal standing for testing under ASTM.

For laboratory measures of pavement friction, Henry described the two main devices currently in use as the British Portable Tester (BPT, ASTM E 303) and the Japanese Dynamic Friction Tester (DFTester, ASTM E 1890). The BPT functions by measuring the loss in kinetic energy of a pendulum with a rubber slider at its edge that has been released over a sample. Contact speed with this method is low and therefore microtexture tends to dominate the readings since the macrotexture effect is not properly registered. The DFTester has three rubber sliders mounted on a disk that is driven by a motor above the pavement surface. Friction is measured by a transducer as the disk spins into the sample. This method is credited with the advantage of measuring friction as a function of speed.

Table 2 Comparison of Friction Testing between Smooth and Ribbed Test Tires (*14*)

Test Site	Friction With Grooves		Friction Without Grooves	
	Smooth Tire	Ribbed Tire	Smooth Tire	Ribbed Tire
A	38	42	19	38
B	40	42	17	39
C	39	40	19	38
D	38	42	21	40

Because of the variability between testing methods, the PIARC 'International Experiment to Compare and Harmonize Texture and Skid Resistance Measurements' proposed the harmonization of friction measurements by different devices. The concept, called the International Friction Index (IFI), is now a standard (ASTM E 1960) and can be calculated from the measurement of pavement macrotexture and wet pavement friction. The IFI model consists of two numbers that describe the skid resistance of a pavement: the speed constant and the friction number.

Two major implications of standardizing friction measures are observed (*3*). The first is that there is now the possibility of using a common metric for skid resistance measurements. A second major implication is that the thrust of skid resistance measurements would seem to be moving away from friction based measurement and more to macrotexture measurements.

A similar friction measurement program was being considered by the Joint Winter Runway Friction Measurement Program in a parallel effort related to pavements used for aircraft. The International Runway Friction Index (IRFI) is intended to provide reports of runway friction for pilot advisories during operations in winter conditions (*10*). Friction data have been collected since 1995 to try to harmonize friction measurements with different devices in runways. Currently, pilots unfamiliar with the local runway friction reporting procedure find it difficult to judge the aircraft stopping distance either for landing or for a rejected take off when adverse conditions are present.

Regardless of the methodology used, the numerical skid resistance value associated with a specific pavement is usually presented as a two-digit constant, determined by multiplying the measured friction coefficient by 100 (though sometimes the number is left as a decimal). This number is described as the friction number (FN) or skid number (SN), note that FN rather than SN is the preferred abbreviation (*15*). FN is usually followed by the speed value at which the friction measurement was taken and the type of tire (i.e., FN50S represents the friction measurement taken at 50 mph with a smooth tire). FN40R describes the friction number measured at 40 mph with a ribbed tire. Representative values for friction numbers obtained with a skid trailer, and the associated recommendations for each value, are depicted in Table 3 (*4*, *16*).

Table 3 Typical Skid Resistance Value Ranges (16)

Skid Number	Recommendations
< 30	Take action to correct pavement
≥ 30	Acceptable for low volume roads
31 – 34	Monitor pavement frequently
≥ 35	Acceptable for heavily traveled roads

Different numerical values of skid resistance are used outside of the U.S. For example, in Sweden, wet-road surface friction is measured with fixed slip devices (Skiddometer BV-11 or Saab Friction Tester, SFT). With this procedure, friction values of 0.5 are desirable. Finland established levels of acceptable pavement friction as a function of speed as shown in Table 4 (*12*, *15*). Values were obtained following Finnish standards for testing (PANK 5201 or TIE 475).

The United Kingdom (UK) has developed a policy to establish acceptable friction levels for different road and traffic situations. The UK uses the term “investigatory levels”, as opposed to “intervention levels”, to denote conditions that warrant further investigation or surface treatments due to a deficiency in pavement friction. Table 5 summarizes the values taken with the SCRIM device (Side force Coefficient Road Inventory Machine) (*17*). The UK 2004 ‘Road Maintenance Condition Report’ showed that 17 percent of principal roads fell below the investigatory levels.

Models Developed

Another way to calculate pavement friction is through equation models developed to predict this number according to selected road characteristics. For example, Russell developed a model to forecast FN from asphalt material properties, age of pavement, traffic conditions, and climate (*18*):

$$FN = 41.4 - 0.00075D^2 - 1.45\ln(LAVP) + 0.245LAWEAR \quad (1)$$

where,

FN: friction number calculated at 40 mph
D: % dolomite in the mix
LAVP: Lane Accumulated Vehicle Passes
LAWEAR: Los Angeles Wear

The model states a desirable minimum predicted FN of 35. The Wisconsin Department of Transportation (WisDOT) currently employs this model to estimate FN. The Russell Model does not differentiate between factors such as microtexture and macrotexture, as other models have done. Russell concluded that a new system of measuring pavement friction values should be developed which includes these elements. He recommended that future research be conducted for two reasons: 1) To improve the quality of pavement mix designs through the incorporations of friction criteria; and 2) To get pavement friction criteria incorporated into pavement management systems that determine maintenance schedules and resurfacing.

Table 4 Typical Skid Resistance Values in Finland (*15*)

Speed (km/h)	Speed (mph)	Acceptable Friction
≤ 80	≤ 50	0.4
≤ 100	≤ 60	0.5
≤ 120	≤ 75	≥ 0.6

Table 5 U.K.'s Investigatory Skid Resistance Values (17)

Site Category and Definition		Investigatory level (at 50km/h)	
		HD28/94 (preceding)	**HD28/04 (current)**
A	Motorway	0.35	0.35
B	Dual carriageway non-event	0.35	0.35-0.40
C	Single carriageway non-event	0.40	0.40-0.45
Q	Dual Carriageway (all purpose) - minor junctions	0.40	0.45-0.55
	Single Carriageway minor junctions & approaches to and across major junctions (all limbs)	0.45	
	Approach to roundabout	0.55	
K	Approaches to pedestrian crossings and other high risk situations	0.45	0.50-0.55
R	Roundabout	0.45*	0.45-0.50
G1	Gradient 5-10% longer than 50m	0.45	0.45-0.50
G2	Gradient ≥10% longer than 50m	0.50	0.50-0.55
S1	Bend radius <500m – dual carriageway	0.45-0.50*	0.45-0.50
S2	Bend radius <500m – single carriageway		0.50-0.55

Table notes:

1. Category R and some sites in new categories S1 and S2 were previously tested at 20km/h.

2. A reduction in Investigatory Level of 0.05 is permitted for categories A, B, C, G2 and S2 in low risk situations, such as low traffic levels or where the risks presented are well mitigated and a low incidence of crashes has been observed.

Some other commonly used models to predict pavement friction include the Penn State Model and the Rado Model (*10*). The Penn State Model describes the relationship of friction to slip speed by an exponential function. In this model, the speed constant was found to be linearly related to macrotexture measurements. The Penn State Model was adopted by PIARC in the following form (with the intercept at 60 km/h instead of at a speed of zero):

$$F(S) = F60\; e^{\left(\frac{60-S}{Sp}\right)} \qquad (2)$$

where,

F(S): friction coefficient at slip speed
S: slip speed (km/h)
F60: friction coefficient at 60 km/h (36 mph)
S_P: speed constant

The Rado Model is derived from the tire and pavement properties that model the behavior of friction when a tire moves from the free rolling condition to the locked wheel condition. The Rado Model is in the form:

$$\mu(S) = \mu_{peak}\, e^{-\left[\frac{\ln(S/S_{peak})}{C}\right]^2} \tag{3}$$

where,

$\mu(S)$: friction coefficient at slip speed
S: slip speed (km/h)
μ_{peak}: peak friction level
S_{peak}: slip speed at the peak
C: shape factor related to harshness of the texture

The Rado and Penn State Models can be related to actual vehicle braking in emergency situations (*10*). Friction follows the Rado Model when the brake is first applied until the wheels are fully locked. If braking continues after the locked wheel condition is reached, the vehicle slip speed decreases and the friction follows the Penn State model until the vehicle stops.

Other models developed to predict pavement skid resistance from microtexture and macrotexture parameters are the Lees and Katedka Method, the Hankins and Underwood Design Procedure, and the Mullen Method. These methods were described by Galambos et al. in a report published in 1977 (*5*).

Microtexture Measurement

Currently, there is no system capable of measuring microtexture profiles at highway speeds. A profile of the microtexture of a pavement surface in service could be misleading because the portions of the pavement surface that contact the tires are polished by traffic and it is the microtexture of the exposed aggregate surface contacting with the tire that influences the pavement friction. Because of the difficulty in measuring microtexture profiles, a surrogate for measuring microtexture is generally preferred.

In research completed at Pennsylvania State University (Penn State), it was found that a high correlation exists between the parameter μ_0 of the Penn State model and the root mean square (RMS) of the microtexture profile height (*19*). The parameter μ_0 is the zero speed intercept of the friction-speed curve and characterizes the friction at low slip speeds. Also, it was found that the British Pendulum Numbers (BPNs) were highly correlated with the parameter μ_0. Therefore, the BPN values could be considered as the surrogate for microtexture.

In the UK, the SCRIM values are synonymous with microtexture values. The SCRIM is the device used for measuring side force coefficients. Therefore, the sliding speed of the test tire is relatively low. Because of the low slip speed, it also serves as a surrogate for measuring microtexture. In the PIARC IFI model, the speed constant is linearly related to the result of a macrotexture measurement. Therefore, if a macrotexture measurement is available, IFI has no need for measuring microtexture.

Macrotexture Measurement

Volumetric Method - Historically, macrotexture measurement has been obtained by a volumetric technique (*20*). This basic method consists of spreading a known volume of material (sand, glass beads, or grease) on the pavement and measuring the area covered. Dividing the volume by the area provides the macrotexture depth (mean texture depth, MTD). Variations of these methods are referred to as the sand patch, sand track, or grease patch methods and are described in the following paragraphs.

Originally, the sand patch method required spreading a specified volume of Ottawa sand that passed a No. 50 sieve and that was retained on a No. 100 sieve. In this method sand is spread on the pavement with a spreading tool in a circular motion. A hockey puck is commonly the recommended spreading tool. Calculation of the area of circular sand patch is done by averaging four equally spaced diameter measurements. The larger the amount of sand and the bigger the area covered the better for exact measurement of MTD. Nonetheless the MTD value can be obtained from equations whether the sand amount is small or large.

The current ASTM standard (ASTM E 965) for measuring macrotexture requires the use of glass spheres instead of sand. The material was changed for two reasons: glass spheres spread more uniformly than sand which has an irregular shape, and very low-yields are usually obtained when bags of sand are sieved, whereas glass spheres that meet the size specification are commercially available and the need to sieve the material is avoided.

A volumetric method that also employs glass spheres is used in Japan. However, instead of placing them in a circular pattern the glass spheres are spread in a linear track with a spreader that is maintained at a small fixed distance above the surface in a fixture of constant width. The length of the track on a surface relative to the length of a track on a glass plate (zero texture depth) allows the texture depth (TD) to be calculated from Equation 4.

$$TD = \frac{V(L_g - L_s)}{aL_g L_s} \tag{4}$$

where,

V: volume of the glass spheres used
a: width of the fixture
L_g, L_s: length of the track on the glass plate and on the surface

The last variation of the volumetric method is the grease patch method used by the National Aeronautics and Space Administration (NASA) (*21*). Similar to the sand patch and glass spheres methods, a known volume of grease contained in a tube with a plunger is placed on the pavement between two parallel strips of masking tape. Grease is then worked into the voids using a squeegee with a rubber face similar in hardness to an automobile tire. Relatively weak correlations have been found between average texture depths determined by the grease patch method and pavement friction coefficients.

Outflow Method - The outflow meter (ASTM STP 583) used in this method is a transparent vertical cylinder that rests on a rubber annulus placed on the pavement (*22*). A valve at the bottom of the cylinder is closed and the cylinder is filled with water. The valve is then opened, and the time that it takes the water level to fall by a fixed amount is measured. In the original outflow meter, the time (in seconds) that it took the level to pass two marks inscribed on the cylinder was measured with a stopwatch and was reported as the outflow time (OFT). A major improvement in this method is the incorporation of an electronic timer that measures the time that it takes for the level to fall from an upper electrode to a lower electrode in the water; note that OFT is highly correlated with the mean profile depth (MPD) and the MTD for nonporous pavements. Comparison of OFT and MTD is a potential method for the assessment of the effectiveness of porous pavement surfaces.

MPD, MTD, and IFI - Systems that can measure macrotexture at traffic speeds are now available due to recent advances in laser technology, and in the computational powers and speeds of small computers. The profiles produced by these devices can be used to compute various profiles statistics such as MPD, the overall RMS of the profile height, and other parameters that reduce the profile to a single parameter (*23*, *24*).

MPD has proven to yield the best results when used to determine the speed constant (S_p) of the IFI, according to a PIARC international experiment (*25*, *26*). Volumetric methods also produced good results in predicting S_p in the experiment. Results for both predictions are given in the American Society for Engineering Management (ASEM) standard practice for calculating the IFI (ASTM E 1960) (*27*):

$$S_P = 89.7MPD + 14.2 \tag{5}$$

$$S_P = 113.6MTD - 11.6 \tag{6}$$

where,

MPD and MTD are in mm

S_p is in km/hr

By combining the two equations, the following equation is yielded:

$$MTD = 0.79MPD + 0.23 \tag{7}$$

The PIARC experiment found that the parameter that best describes macrotexture for the prediction of pavement friction is MPD, as defined by ASTM and the International Organization for Standardization (ISO) (*28*). The MPD is calculated using the procedures in ASTM E 1845.

Circular Texture Meter - A new device for the measurement of MPD that can be used in the laboratory and the field is called the Circular Texture meter (CT meter) (*29*). This device was first introduced in 1998. The CT meter uses a laser to measure the profile of a circle 892 mm (35 in) in circumference. The mean depth of each 100 mm segment or arc of the circle is computed according to the ASTM standard practice and ISO. The CT meter is controlled by a notebook computer, which also performs the calculations and stores the mean depth of each segment.

The CT meter measures the profile of a circle that is divided into eight equal segments (arcs). Two of these are approximately parallel to the direction of travel, and two are approximately perpendicular to the direction of travel. For estimating the MTD it has been found that the best results are obtained when all eight segments depths are averaged.

Simultaneous measurements of MPD with the CT meter and of MTD by the volumetric method have been conducted on the three occasions: at the NASA Wallops Flight Facility in 1998 and 1999, and at a test track at Sperenberg, Germany in 2000 (*30*). The 1999 and 2000 test data included OFTs. Equation 8 and Equation 9 shows that the MPD produced by the CT meter is highly correlated with both MTD and OFT.

$$MTD = 1.03MPD + 0.15 \tag{8}$$

$$OFT = 0.27MPD + 0.09 \tag{9}$$

Linear relationships can be used to predict MTD and OFT from MPD. The ASTM procedure for determination of MPD is used to calculate MPD as the average for eight segments of the circular track obtained with the CT meter.

The prediction of MTD from MPD is not valid for highly porous pavement surfaces, as the glass spheres or sand flows into the pores, producing high values for MTD. Although it might appear that the higher values of MTD predicted in this case would be more appropriate for use for prediction of pavement friction, the prediction of OFT from MTD was very good for the three highly porous pavement surfaces tested at the Sperenberg test track.

Surface Texture for Good Skid Resistance

Good skid resistance on asphalt pavement surfaces could be achieved by controlling both microtexture and macrotexture. In the case of pavement microtexture, it depends largely upon the composition of the aggregates. Good skid resistance must be accomplished through the selection of quality, polish-resistant aggregates. An ideal aggregate for asphalt pavement should be composed of hard, coarse, angular, minerals well bonded into a softer matrix so that gradual differential wear will occur.

Pavement macrotexture will depend on the specific type of mix, the aggregate gradation (i.e., dense-graded or open-graded mix), and the stability of the mix. There are two types of pavement surfaces that can be used to achieve a high level of macrotexture in new pavement: open-graded asphalt surface for bituminous pavements, and textured surface for Portland Cement Concrete pavements (PCC). However, retexturing of existing pavement can be accomplished through open graded asphalt overlays, pavement grooving, cold milling, and seal coats (*6*).

Elements Affecting Skid Resistance

Skid Resistance and Weather

Skid resistance is generally considered a wet pavement concern because dry pavements are believed to provide sufficient skid resistance for stopping vehicles. Therefore, testing procedures and previous skid related safety studies have focused on wet pavement conditions. It is known that vehicles operating at low speeds on wet pavements develop full hysteretic friction force with the surface. At low speeds surface water is squeezed out from under the vehicle tire keeping it in full contact with the surface. However, skid resistance properties deteriorate when vehicle speeds increase. In this scenario, the hysteretic friction is reduced because a water film is developed between the vehicle tire and the pavement surface thus decreasing skid resistance, and potentially causing hydroplaning even when friction levels are adequate.

Goodwin analyzed crashes on U.S. highways in poor road weather conditions (31). Results of injuries and fatalities caused by wet surface and other slippery surfaces are shown in Figures 3 and 4. These results represent roughly 22 percent of the total number of injuries and fatalities for the same period. Wet pavements are more prevalent throughout the U.S. than snow and ice, which may be the primary reason why wet surface crashes out number other crashes in slippery conditions.

In Wisconsin, ongoing research on weather-related crashes is shedding light on the proportion of crashes that occur on wet, dry, and other surface conditions (e.g., ice, snow) (*32*). Table 6 shows the number of crashes that occurred on wet, dry, and other pavement surface conditions for the Wisconsin State Trunk Network System (STN) from 1998 through 2002. The wet-to-dry ratio is also provided. Differences between wet and dry crash frequencies are shown in Figure 5 (1998 – 2002). Figure 6 shows the trends of wet-to-dry ratios for the same period. Average distribution of crashes in Wisconsin based on pavement conditions is shown in Figure 7.

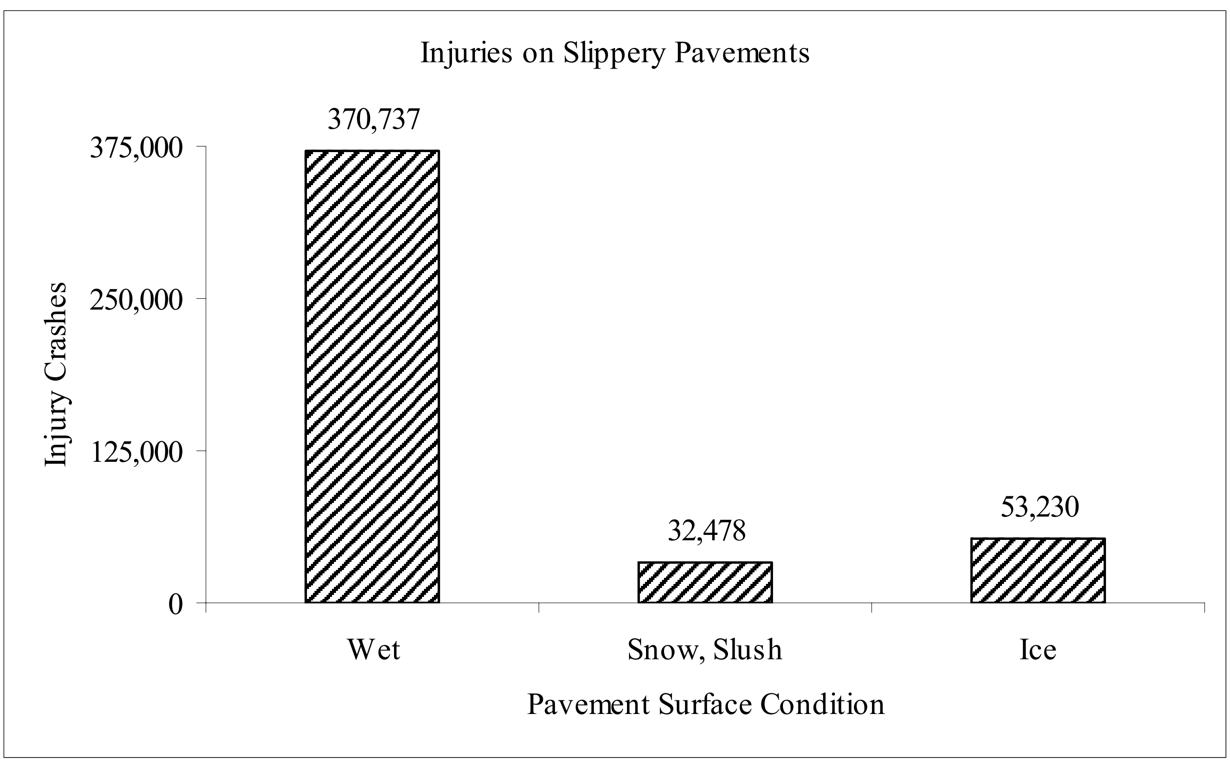


Figure 3 Average Injury Crashes on Slippery Pavements, 1995-2001 (*31*)

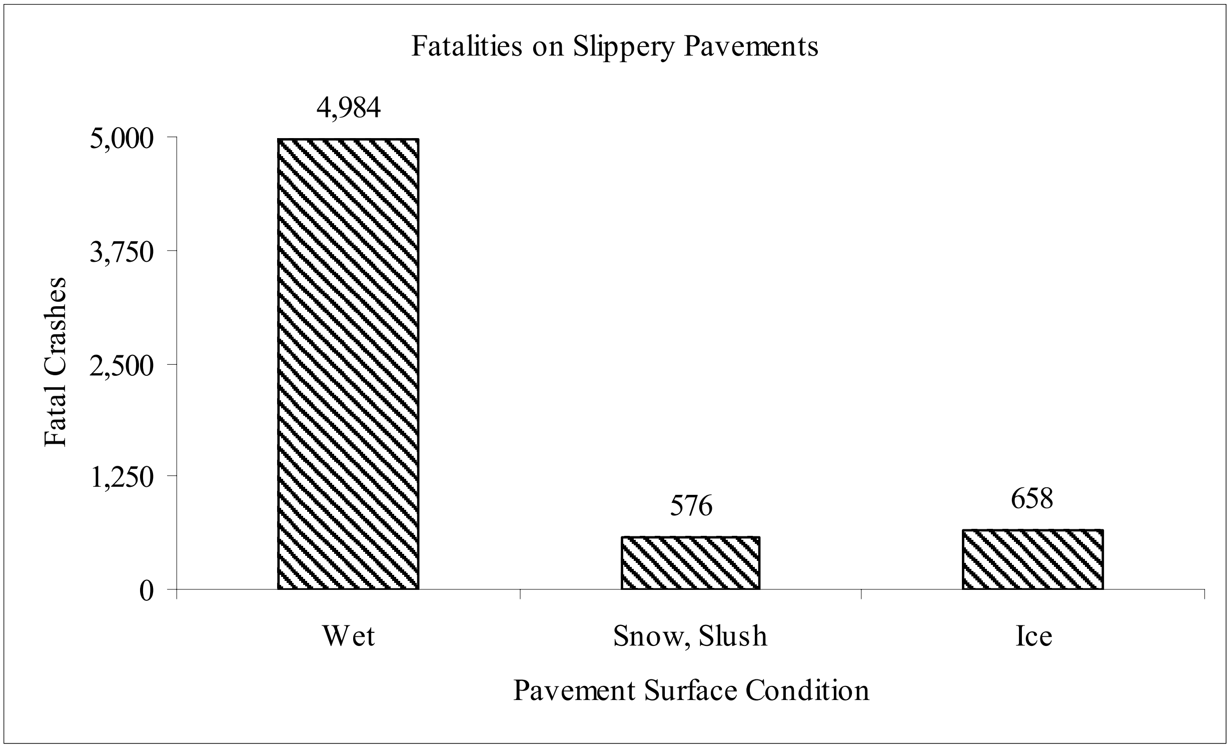


Figure 4 Average Fatal Crashes on Slippery Pavements, 1995 – 2001 (*31*)

Table 6 Wisconsin Wet/Dry Crash Ratio (*32*)

Crash Type	**1998**	**1999**	**2000**	**2001**	**2002**
Wet	8,212	6,737	7,912	8,199	6,792
Dry	43,707	42,790	39,914	43,783	43,009
Other	6,117	7,863	11,346	4,901	6,216
Wet/Dry Ratio	0.19	0.16	0.20	0.19	0.16

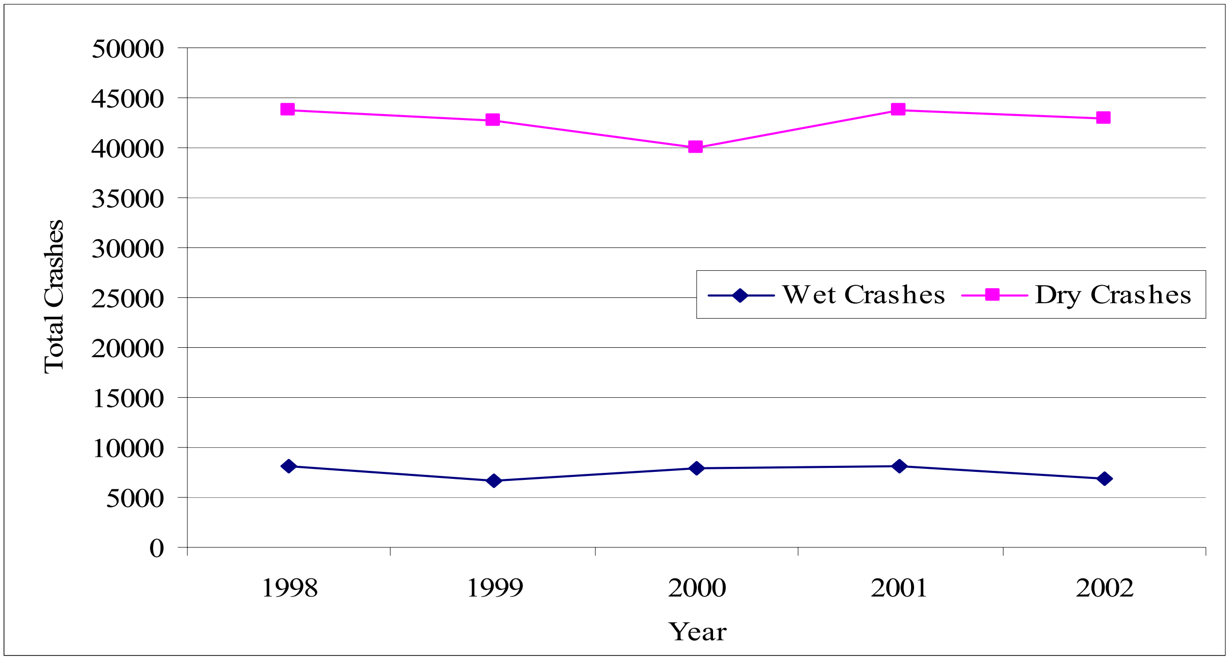


Figure 5 Wet and Dry Crashes Trends for Wisconsin, 1998 – 2002 (*32*)

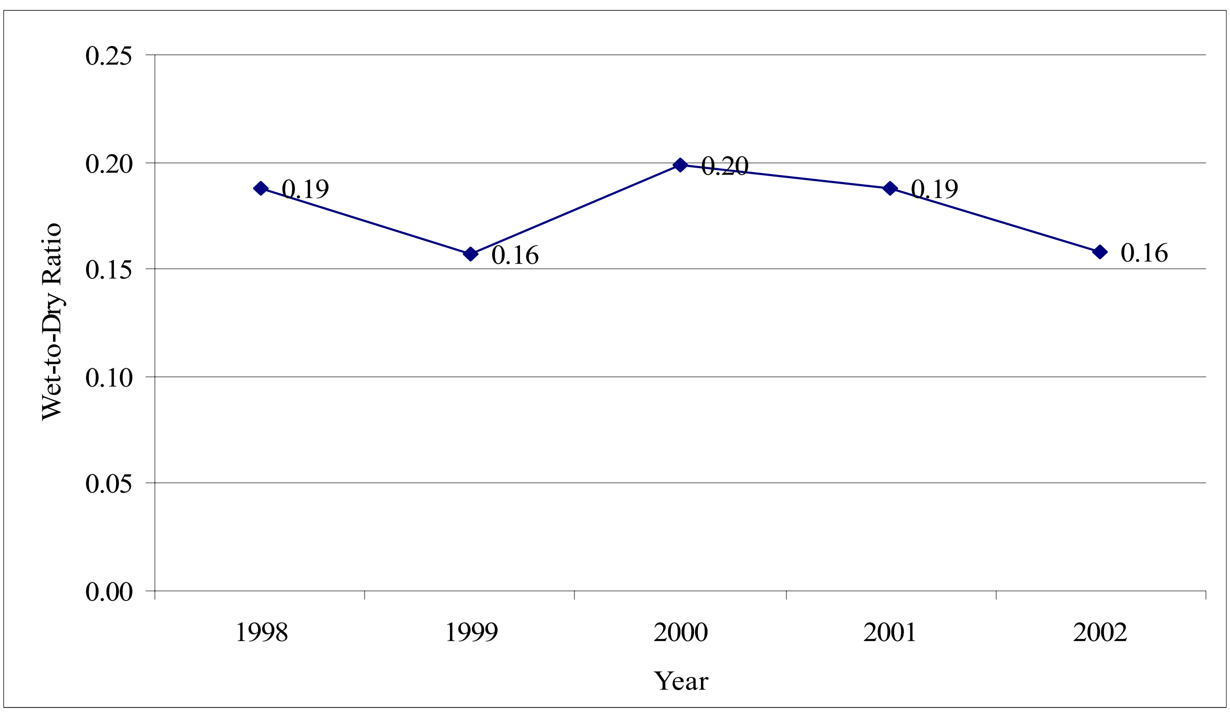


Figure 6 Wet-to-Dry Ratio for Wisconsin, 1998 – 2002 (*32*)

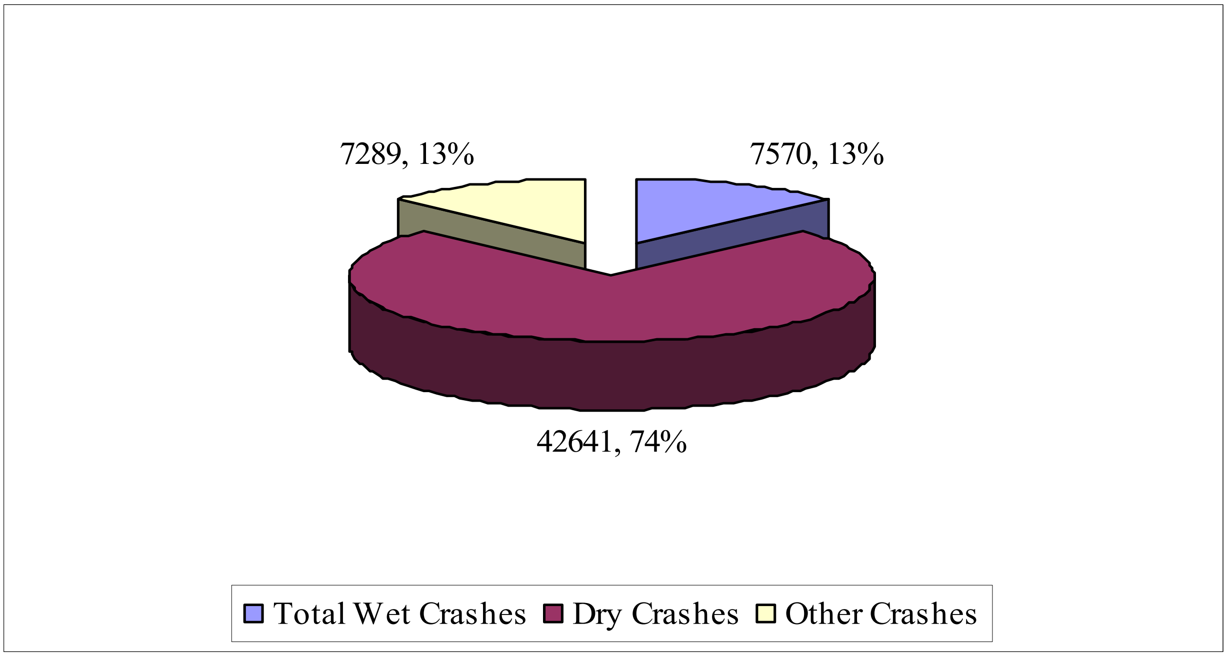


Figure 7 Annual Average Distributions of Crashes Based on Pavement Condition for Wisconsin, 1998 – 2002 (*32*)

As shown in Figure 7, wet crash frequency slightly exceeds that of ice and snow (i.e., 'other') crashes combined. Still, dry pavement crashes are most prevalent amongst all other pavement surface conditions. Increasing macrotexture is one method to help decrease crash occurrence in wet weather pavement conditions. Moreover, a recent European study reports that increased macrotexture reduces total crashes under both wet and dry pavement conditions (26). This study also shows that increased macrotexture reduces crashes at lower speeds than previously believed.

Skid Resistance and Water Film Thickness

Water Film Thickness - As previously mentioned, maintaining good skid resistance is very important in wet conditions for traffic safety. Research efforts to identify improved methods for draining rainwater from the surface of multi-lane pavements and to develop guidelines for their implementation have been conducted. Drainage plays an important role in the mitigation of hydroplaning and splash and spray, since these two problems tend to depend on the water film thickness (WFT) on the pavement. Figure 8 provides a definition of the WFT as it flows across the pavement surface (*33*).

In Figure 8, the thickness of the water film that contributes to hydroplaning is the MTD plus the thickness of the water film above the top of the surface asperities. Water below the MTD is trapped in the surface and does not contribute to the drainage of the pavement. Drainage or flow occurs in the total flow layer, *y*, which is the water film thickness plus the mean texture depth. Increasing the depth (macrotexture) is important because it allows a reservoir for water (depth below the MTD) and enhances the water flow (depth above the MTD).

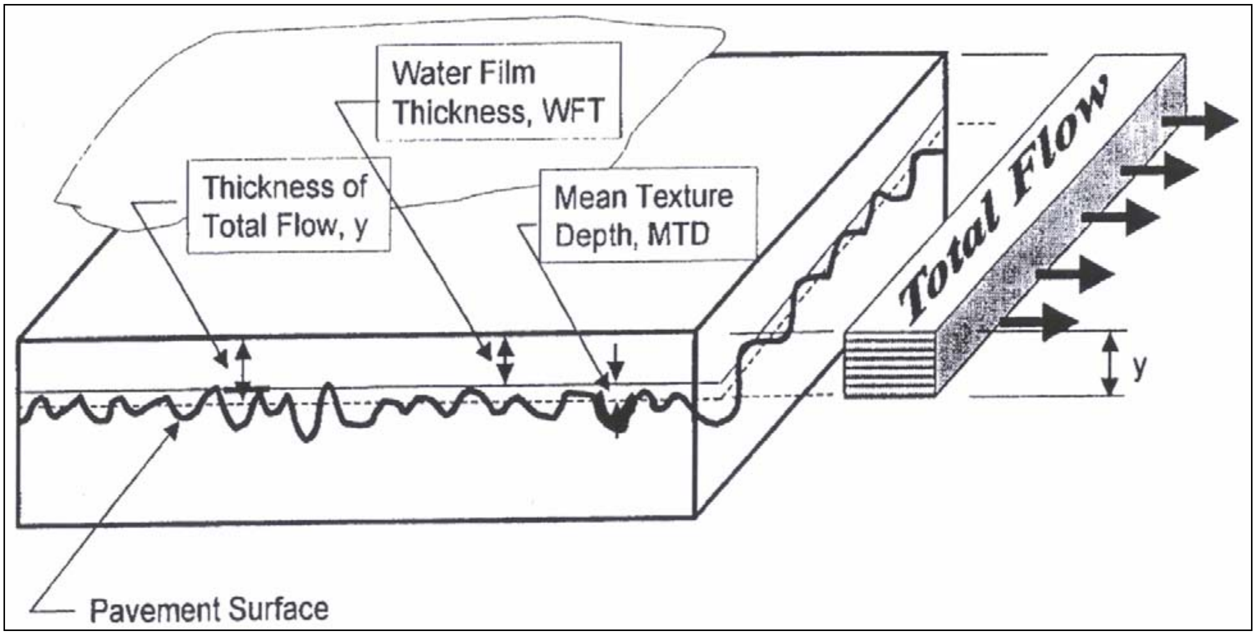


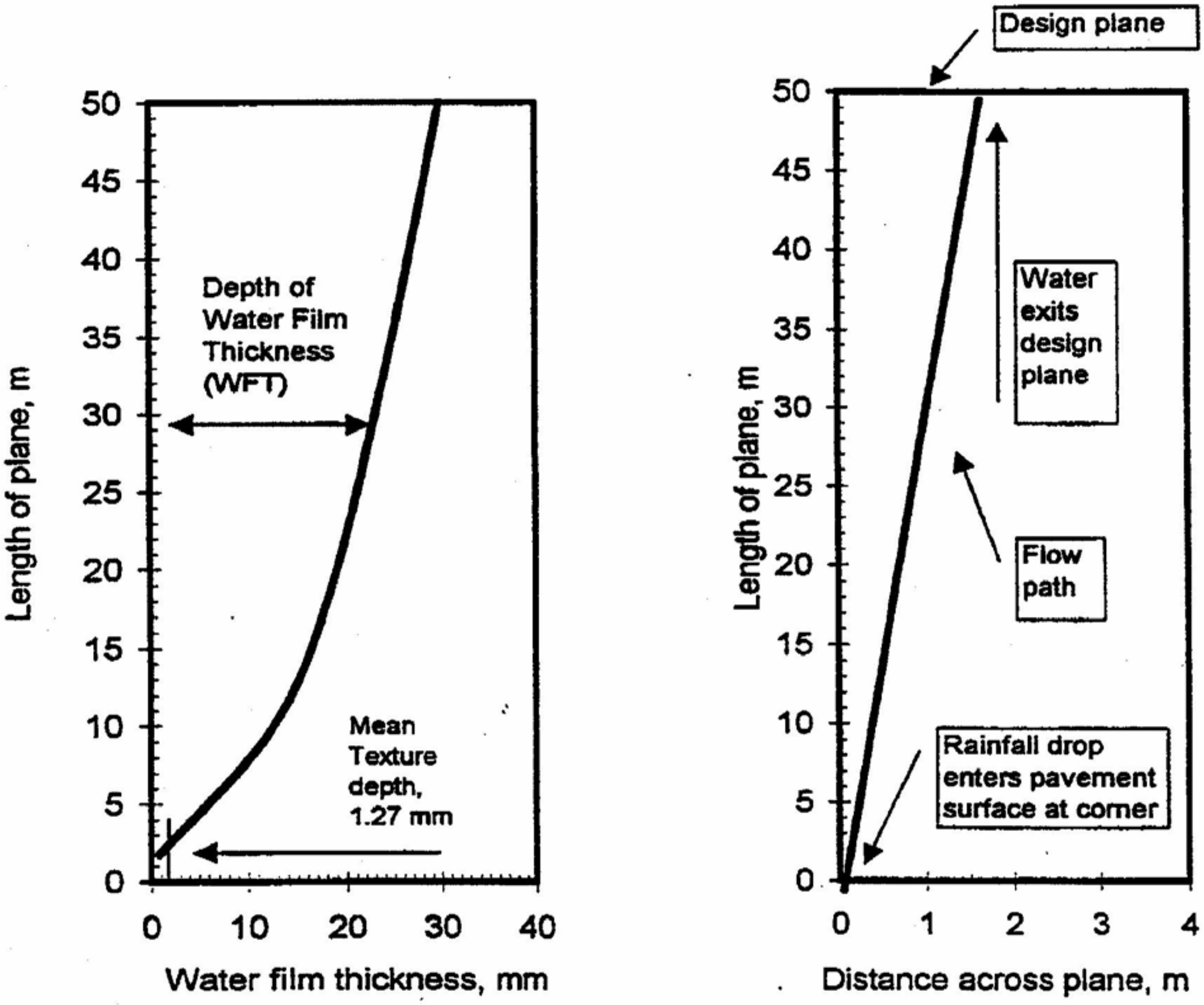
Figure 8 Definition of WFT, MTD, and Total Flow (*33*)

The flow path for a particle of water falling on a pavement surface is simply defined as the line determined by the slope along the pavement surface. Thus, the maximum flow path for a pavement section is the longest flow path for the section. This is the maximum distance that a rainfall droplet can flow between the point of contact with the water film and its point of exit from the pavement. A description of the process is presented in Figure 9 (*33*). For a given quantity of rainfall per unit area of pavement, reducing the flow path will result in a more shallow depth of flow and a concomitant reduction in the propensity for hydroplaning or excessive splash and spray.

Methods for Reducing Water Film Thickness - Reducing WFT is important for the safety of motorists avoiding hazardous situations, particularly hydroplaning. Some of the techniques developed to reduce WFT are:

- Alteration of surface geometry;
- Installation of drainage appurtenance;
- Use of permeable or porous asphalt paving mixtures;
- Grooving in Portland cement concrete; and
- Enhancement of surface texture through mixture selection and design.

Surface geometry elements, such as cross-slope and super elevation, have traditionally been employed to remove water from the pavement surface. However, pavement geometry must be designed in accordance with the American Association of State Highway and Transportation Officials (AASHTO) design guidelines, limiting the degree to which surface geometry can be used to minimize WFT (*34*). Therefore, other approaches have been developed in addition to the modification of surface geometry.



Note:
The figures above show a plan view of a design plane (right) and a profile of the water film thickness (WFT) as water flows along flow path (left). Water falling on the pavement first fills the macrotexture (lower left, here 1.27 mm deep) at which point it reaches tops of the asperities of the coarse aggregate particles. At this point, the depth of the water film increases (from zero) until the water exits either an edge of the pavement plane (case here) or a drainage appurtenance.

A plane is defined as a section of pavement that has the same geometric characteristics. In the drainage model used in this study, the drainage across the pavement is modeled by linking adjacent design planes.

Figure 9 Definition of Flow Path and Design Plane (*33*)

Appurtenances, such as grate inlets and slotted drains, are means for removing surface water from the pavement. Other means for reducing the flow of surface water across the pavement are permeable asphalt concrete pavements such as the open-graded friction course (OGAFC) used in the United States, and other porous asphalt used in many parts of Europe. These surfaces also provide a means for draining water from beneath the tire, thereby reducing hydroplaning potential. Finally, texture modification, as typified by the recent developments in the texturing of concrete pavements, and the grooving of asphalt and PCC pavements also provide a means for reducing WFT.

Controlling WFT with 'internally draining asphalt surfaces', another technique for reducing WFT on roadway surfaces, is achieved through the use of open-graded asphalt mixes. This internally draining pavement has the potential to minimize WFT and hydroplaning. Open-graded asphalt surface mixtures can reduce the water film thickness by:

- Allowing internal drainage, which effectively reduces the amount of water that must be drained across the surfaces of the pavement; and
- Increasing the MTD.

Most research reports emphasize the internal drainage aspects of the open-grade asphalt mixtures. Nonetheless, the enhanced surface texture that they afford may be of equal or more importance than the internal drainage that they provide.

The State of Oregon used porous or permeable surface layers for the first time in U.S. history in the early 1930's (*35*). Pavement consisted of a surface treatment that was placed on an impermeable base layer. The permeable surface layer increased the skid resistance of the surface, but the pavement was short-lived during periods of heavy traffic load. From this early work, OGAFC was developed. OGAFC mixes typically contain 10 to 13 percent air voids and are hot-laid with a paving machine to a depth of approximately 19 mm (*35*). Maximum aggregate size ranges from 13 to 19 mm. Asphalt content is selected as the maximum amount of asphalt that the hot mix can retain without appreciable drainage when the mixture is still hot. Mixes with differing asphalt contents are placed on a plate inside an oven and then the amount of asphalt that drains from the mix is measured. OGAFC mixes offer increased skid resistance and allow internal drainage of surface water from the pavement surface (*36*).

Open-graded mixtures with larger air-void content, referred to as porous asphalt, drainage asphalt, or permeable asphalt, have evolved from the early use of OGAFC. Porous mixtures have been used extensively in Europe. These mixes are usually placed in thicker layers than OGAFC (usually greater than 25 mm thick) with binders that are modified with fibers or polymers. These mixtures contain approximately 20 percent air voids, which is significantly higher than the OGAFC surface mixtures used in the U. S. Porous asphalt surfaces also offer high values of skid resistance and contribute to the removal of water from the pavement surface. A summary of the mixture characteristics for different porous pavements as used in the U. S. and Europe is provided in Table 7 (*36 - 42*).

Table 7 Gradiation Used for Internally Draining Asphalt Mixes

Size	Percent Passing				
	Oregon (U.S.)	**Europe**	**Switzerland**	**Belgium**	**France**
25.0 mm	99 - 100	-	-	-	-
19.0	85 - 96	100	-	-	-
14.0	-	-	-	100	100
12.5	60 - 71	-	-	-	-
11.2	-	90 - 95	-	-	-
10.0	-	-	100	-	55
9.5	-	-	-	-	-
8.0	-	28 - 40	-	-	-
6.3	17 - 31	-	-	-	23
5.0	-	18 - 23	-	-	-
4.75	-	-	-	-	-
2.75	-	-	-	-	-
2.36	-	-	-	-	-
2.0	7 - 19	10 - 12	-	-	14
710 μ	-	6 - 8	-	-	-
250	-	4 - 6	-	-	-
90	-	2 - 4	-	-	-
74	1 - 6	-	-	-	-
Air Voids (%)	5.7 - 10	17 - 22	14 - 20	16 - 28	24
Thickness (mm)	1.5 - 2.0	40 - 50	40 - 50	40	42
Permeability (1/s)	-	0.06 - 0.12	0.06 - 0.12	0.008 - 0.023	0.02

The effectiveness of porous asphalt can be enhanced if drains are installed internally within the pavement layers. Continuous fabric drains that can be placed either transverse to or longitudinally with the direction of traffic have been used successfully for a number of years. Rectangular cross-sectioned drains can be laid flat and may be placed with a new porous asphalt layer when the pavement is overlaid or during new construction (*43*).

Although porous asphalt pavements are generally accepted as useful with respect to reducing hydroplaning, their performance has been unsatisfactory in many states to the extent that several states have eliminated their use entirely. In contrast, they are used extensively on the motorways in Europe, especially in France and the Netherlands (*44*).

Advantages for using porous asphalt pavements include:

- Hydroplaning: Porous asphalt pavements reduce the thickness of the water film on the surface of the pavement, thus greatly reducing the hydroplaning potential of the pavement.
- Skid Resistance: The skid resistance for porous asphalt pavement is generally considered to be equal to that of traditional pavements. Testing performed by van der Zwan et al. showed that at higher vehicle speeds, where aggregate macrotexture has a greater effect on skid resistance, porous pavement actually gives a higher skid resistance than conventional pavement (*38*).
- Splash and Spray: Surface water can quickly infiltrate into porous asphalt, greatly reducing the amount of free surface water, which can cause splash and spray from the vehicle tires. This reduction in splash and spray provides greater visibility, resulting in safer roadway conditions than with PCC pavements or conventional dense-graded asphalt pavements (*45, 46, 47*).
- Headlight Reflection: With the surface water infiltrating into the pavement, the reflections of vehicle headlights are greatly reduced and the visibility of roadway markings is increased.

Porous asphalt surfaces offer a significant increase in surface texture over conventional dense-graded surfaces. However, there are a number of disadvantages associated with these surfaces:

- Skid Resistance: At lower speeds, the skid resistance of porous asphalt is lower than for conventional asphalt surfaces, because there is less aggregate surface at the tire-pavement interface for porous asphalt mixes. Microtexture of these surfaces is generated primarily by the coarse-sized aggregate particles. This is not considered a serious disadvantage because on high-speed motorways, skid resistance is critical at high speeds, not low speeds.
- Plugging: There is a tendency for the voids in porous asphalt surfaces to become plugged and filled with anti-skid material and other roadway debris such as sediment runoff and material spilled on the road surface. During their first year of use, approximately one-third of the permeability of porous pavements is lost as a result of plugging (48). In France, it has been concluded that a level of approximately 20 percent voids is needed for porous pavements to perform effectively. Therefore, current design practice in France requires initial void contents of 27 to 30 percent (*39*).
- Deicing Performance: Road salts tend to infiltrate into the surface voids reducing the effectiveness of the salt or requiring larger application rates than for conventional surfaces. It takes three times the amount of salt on porous pavements as on traditional pavement types to produce the same deicing effects (*49*).

- Black Ice: Porous asphalt surfaces have a tendency to develop black ice more quickly than conventional dense-graded pavements. Black ice can occur suddenly at the onset of a light rainfall when the internal pavement temperature is near or above freezing, and the air temperature is at or below freezing. Because porous asphalt conducts heat less readily than dense-graded mixtures, the water on the pavement surface freezes more rapidly. The formation of black ice is a serious safety concern and has caused French authorities to discontinue the use of porous asphalt surfacing in the Alps where the conditions for the formation of black ice are common.
- Raveling: Raveling and loss of adhesion between porous asphalt layers and the underlying layers are the most frequently cited performance problems in the U. S. However, the raveling problem may be alleviated by carefully selecting proper modifiers or the amount and type of asphalt binder in the mix (*50*).

Tappeiner cites a European report that states that there were 20 percent fewer fatalities and injuries by motorists while traveling on porous asphalt pavements during wet weather conditions (*51*). A similar reduction was also reported in the United States. These claims for improved safety must be considered within the context of the French experience where problems with black ice formation have been observed.

When reviewing the advantages and disadvantages of porous asphalt surfaces, it can be seen that this type of pavement has many positive attributes if careful attention is given to mix proportions, materials selection, and construction details. Porous asphalt surfaces, especially newer mixture designs with special binders, warrant greater use in the United States although their disadvantages must also be carefully considered.

Controlling Water Film Thickness with Surface Texture - Another method for controlling WFT is by maximizing the texture of the pavement surface. WFT is reduced in direct proportion to the increase in macrotexture (total macrotexture volume, not MTD). Since porous asphalt surfaces are typically prepared from relatively coarse aggregates to gradations with a minimal quantity of sand-sized material, they generally yield large levels of macrotexture. Macrotexture in other asphalt surfaces is also controlled by the gradation of the aggregate. The importance of macrotexture is recognized in French practice where microsurfacing techniques are now widely used. Microsurfacing utilizes thin layers of hot-mix asphalt concrete graded to maximize surface texture. This procedure has replaced porous asphalt in areas where the performance of porous asphalt has been suspect.

Other Elements Affecting Skid Resistance

In addition to climate and water in the pavement, Harwood et al. stated that the potential for a skidding crash depends mainly on the speed of the vehicle (frictional demand increases with the square of the speed), the cornering path, the magnitude of acceleration or braking, the condition of the vehicle tires, and also the characteristics of the pavement surface (*52*). On wet pavements, speed is the most significant parameter because the skid resistance at the tire-pavement interface decreases with increasing speed.

Generally, it appears that adequate macrotexture depth helps in providing a higher level of friction; however, the available friction is reduced as vehicle speed increases, as previously mentioned. It should be kept in mind that the need for adequate macrotexture does not diminish the need for high microtexture in the surface of pavements. In addition, poor microtexture and increasing water film thickness are detrimental in the skid resistance provision. Therefore both microtexture and macrotexture contribute to the magnitude of wet pavement high-speed skid resistance as seen in the trends shown in Figures 10 and 11 (*7*).

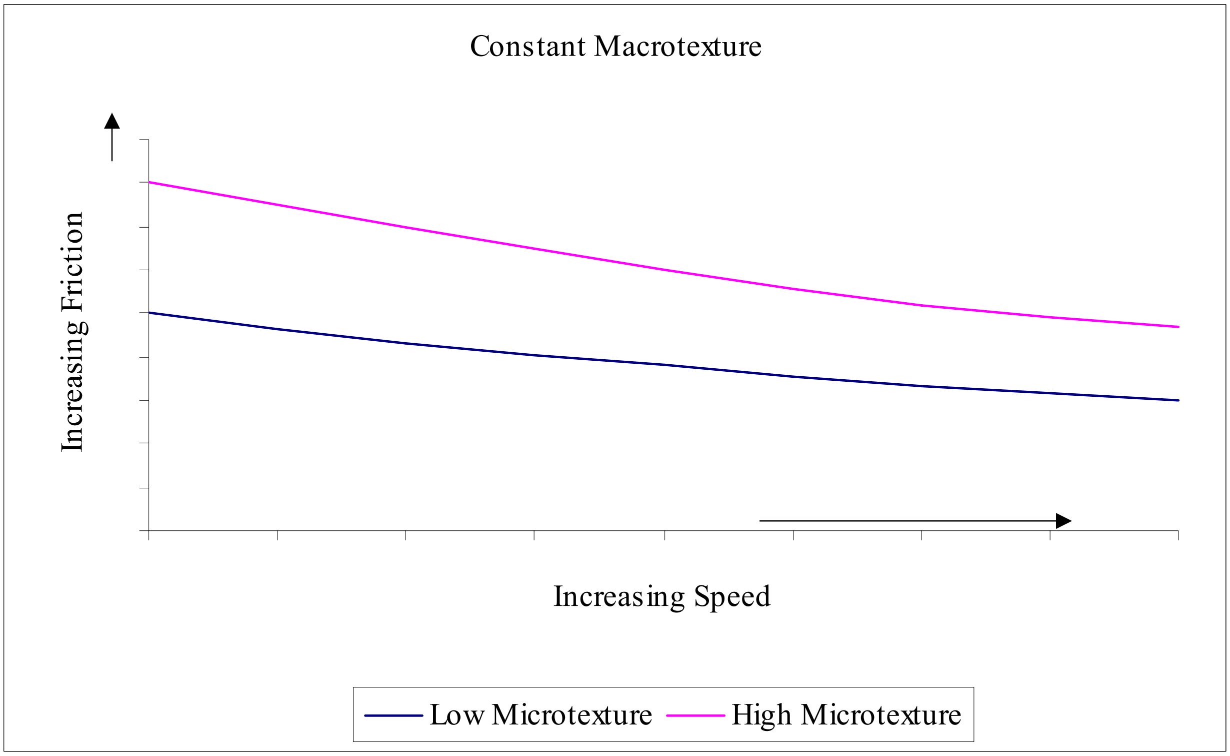


Figure 10 Effect of Microtexture on Wet Pavement Skid Friction (*7*)

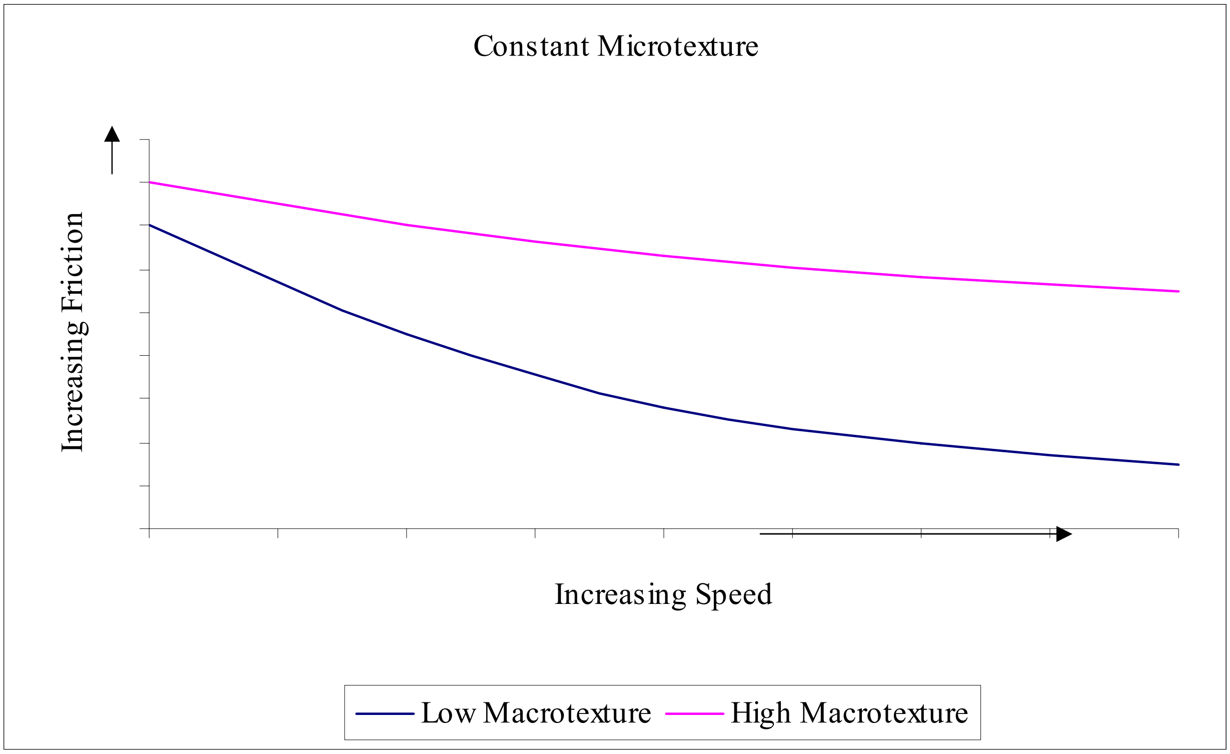


Figure 11 Effect of Macrotexture on Wet Pavement Skid Friction (*7*)

Other elements affecting the friction coefficient associated with skid resistance are the vehicle tire air pressure, tire temperature, material composition, treads pattern, and tread depth. These factors contribute to the level of strength in the interaction generated between the tire and the surface. Additionally, the pavement composition and condition of the pavement surface along with the presence or absence of moisture, mud, snow, ice, oils, salt, sand, or other surface-altering agents are also significant in determining skid resistance. These agents can reduce or increase the grip in the tire-surface interaction making the pavement friction coefficient lower or higher. Table 8 summarizes factors affecting pavement friction in the tire-surface interaction (*12*).

Bazlamit and Reza performed a study that indicated that weather temperature has a strong effect on skid resistance (*53*). The study found that the hysteresis component of friction decreases with increased temperature regardless of surface texture state. However, the adhesion component of friction showed a decrease with increased temperature for polished pavement surfaces. The study implied that compensation should be made to account for temperature effects when comparing skid numbers obtained at different temperatures.

In 2005, Austroads published a report on pavement friction and noted other elements that influence skid resistance. Large pavement markings, such as STOP bars, large arrows, school zone marking, box junctions, and other large marking are detrimental to the skid resistance provision particularly in the approach of roundabouts or intersections, where braking usually occurs (*54*). Therefore, the selection of appropriate pavement marking material can also be important when considering pavement skid resistance.

Table 8 Factors Influence Road Surface Friction (*12*)

Road	Contaminant	Tire
Macrotexture	Chemistry structure	Tread pattern design
Microtexture	Viscosity	Rubber composition
Unevenness/Megatexture	Density	Inflation pressure
Chemistry of materials	Temperature	Rubber hardness
Temperature	Thermal conductivity	Load
Thermal conductivity	Specific heat	Sliding velocity
Specific heat	Film thickness	Temperature
		Thermal conductivity
		Specific heat

Skid Resistance and Traffic Safety

Skid Resistance and Drivers Behavior

Maintaining acceptable skid resistance levels on the roadways is very important for traffic safety. Yet it is difficult to correlate the effects of pavement friction values on crash risk. Usually, drivers adjust their driving behavior (namely speed) based on the appearance of the road environment and the current weather conditions. Certainly, it can be argued that some drivers change their behavior as a result of changes in pavement friction. Table 9 shows how friction values changes for different weather-related pavement conditions in Wisconsin (*55*).

Table 9 Wisconsin Friction Measurement Data (Winter 1994-1995) (*55*)

Pavement Condition	Friction Value		
	Median	25^{th} percentile	75^{th} percentile
Dry	47	44	51
Wet	45	42	47
Slush	30	26	34
Loose snow	26	23	28
Packed snow	21	19	22
Black ice	22	19	27

Table 10 shows typical values during braking maneuvers for different weather conditions (56). It is important to note that not only different weather conditions lead to different pavement friction values - the same pavement can yield variable friction levels under the identical weather condition. Clearly this can be misleading to drivers by violating their expectancy. The variability in friction values at any given time provides little meaningful way for drivers to base their behavioral choices on the actual pavement friction values.

A study from the Finnish National Road Administration examined the extent to which drivers take pavement slipperiness into consideration (*12*, *57*). Drivers were asked to evaluate the roadway slipperiness on a scale measured by four categories of friction coefficients (f):

- Good grip ($f > 0.45$);
- Fairly good grip ($0.35 < f < 0.45$);
- Fairly slippery ($0.25 < f < 0.35$); and
- Slippery ($f < 0.25$).

The results of the Finnish study showed that drivers did a poor job of evaluating actual road conditions. Less than 30 percent of the evaluations coincided with the measured values, and more than 27 percent differed by two to three of the categories listed above. According to the study, as friction values decreased, the variation between drivers' estimates of skid resistance and actual conditions increased. Consequently, the skid resistance of the pavement did not have significant influence on driving speed.

The Finnish report describes studies completed by Wallman and Oberg considering the reaction of drivers to the roadway friction (*58*, *59*). Wallman performed an experiment in the Swedish National Road and Transport Research Institute (VTI) fixed-based driving simulator, where drivers were requested to drive along a road under summer and winter conditions. For the summer conditions a friction coefficient of 0.8 was established; for the winter environment friction coefficients of 0.8, 0.4, and 0.25 were established. Two test designs were developed with different friction distributions along the road in the winter scenarios. In a simulated environment, friction values were implemented by the reaction to braking/skidding actions.

Table 10 Friction Coefficients During Braking (*56*)

Snow or Ice Conditions	Friction Coefficient
Ice	0.1 – 0.2
New Snow	0.2 – 0.25
Old Snow	0.25 – 0.30
Refrozen snow	0.30 – 0.40
Chloride-Treated Snow	0.35 – 0.45
Sand-Treated Snow	0.30 – 0.40
Chloride-Sand Mix	0.30 – 0.50

The Wallman and Oberg study found that mean speed differences between summer and winter scenarios were 11 to 12 km/h (7.0 to 7.5 mph) and 16 to 17 km/h (10.0 to 10.5 mph). Differences between winter scenarios were only about one km/h (0.6 mph), independent of the pavement surface friction. The conclusion was that actual friction values had little to do with the driver's choice of speed; visual information along the road appeared to be much more relevant.

Oberg also compiled measurements of driver choices of speed under different road surface conditions and their effect on stopping distance. For dry conditions and roadways with a posted speed of 90 km/h (55 mph), vehicles speeds ranged from 85 (53 mph) to 95 km/h (59 mph). During winter conditions, speeds were reduced by only six to 10 km/h (four to six mph) because of ice and hard-packed snow on the road surface. The study exemplified that if a driver's reaction time is assumed to be one second and the friction coefficients vary from 0.8 (dry) and 0.25 (ice/snow), then potential stopping distance would vary from 65 meters (213 feet) to 129 m (423 ft). This means that the driver's choice of speed reduction for winter conditions is usually far from the necessary level and not enough to account for unexpected maneuvers requiring braking and/or stopping. If drivers had the ability to understand surface friction conditions, the operating speed should have been reduced to approximately 56 km/h (35 mph).

In another study, Oberg showed how stopping distances for passenger vehicles are correlated to actual pavement friction coefficients on five rural roads (*60*). The average stopping distance on road surfaces of varying degrees of slipperiness varies between 67 and 121 m for passenger cars with ribbed tires and between 68 and 155 m for passenger cars without ribbed tires. The longest stopping distances occurred when the road was slushy and the shortest stopping distances occurred on loose snow. Results in Figure 12 suggested that required stopping distance increases exponentially at friction coefficient values less that 0.3 (*60*).

A Policy on Geometric Design of Highways and Streets (Green Book), published by AASHTO, states that the stopping distance provided in a roadway "should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path" (*34*). To accomplish this, the level of skid resistance in the roadway should be enough to accommodate the braking and steering maneuvers needed. Table 11 provides AASHTO's design stopping distance values for different speeds at deceleration rates based on friction criteria described in the AASHTO's *Guidelines for Skid Resistant Pavement Design* (*61*).

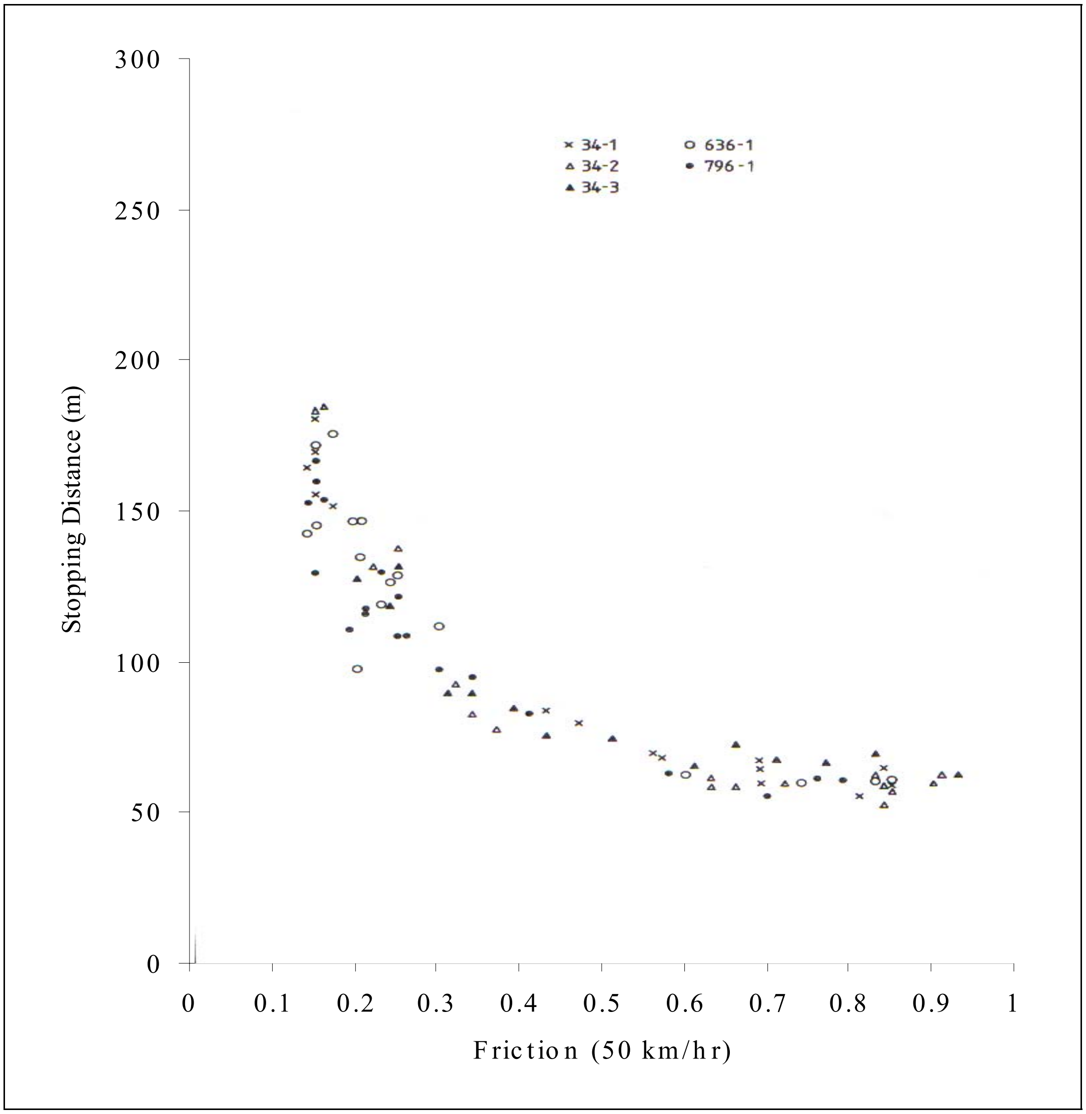


Figure 12 Stopping Distance versus Friction Coefficients (*12*)

Table 11 Stopping Sight Distance (*34*)

Metric					U.S. Customary				
Design Speed	Brake Reaction Distance	Braking Distance on Level	Stopping Sight Distance		Design Speed	Brake Reaction Distance	Braking Distance on Level	Stopping Sight Distance	
			Calc.	Design				Calc.	Design
(km/h)	(m)	(m)	(m)	(m)	(mph)	(ft)	(ft)	(ft)	(ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60.0	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83.0	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147.0	153.6	300.6	305
80	55.6	73.4	129.0	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240.0	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.5	566.0	570
120	83.4	165.2	248.6	250	65	238.9	405.5	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	820
					80	294.0	614.3	908.3	910

Note: Brake reaction distance predicted on a time of 2.5s; deceleration rate of 3.4 m/s^2 [11.2 ft/s^2] used to determine calculated sight distance.

AASHTO's guideline was published in 1976 and provides considerations for properties of mix design that produces adequate skid resistance in the pavement surface. No specific friction value is stated as adequate. Research aimed at updating these guidelines was recently completed, and is awaiting publication. The new guide will address frictional characteristics and performance of pavement surfaces and consider related tire-pavement noise and other relevant issues (NCHRP Project 1-43) (*62*).

The major considerations for hot mix asphalt (HMA) pavements in the current guidelines are (*63*):

- Aggregate used in the top layer of future pavements should be capable of providing adequate skid resistance properties when incorporated in the particular mix and the mix should be capable of providing sufficient stability to ensure the durability of the skid resistance.
- Non-polishing aggregates should be used and the mix design should allow good exposure of the aggregates. Hence, the pavement surface mixture should be designed to provide as much coarse aggregate at the tire-pavement interface as possible.
- The open graded asphalt friction course (OGAFC), with a large proportion of one size aggregate, is recommended because they provide excellent coarse texture and exposes a large area of coarse aggregate.

Values portrayed in Table 11 are a function of the brake reaction time, the design speed and the deceleration rate measured when adequate friction is present (*34*). Equation 10 shows the model used in the Green Book to calculate braking distance:

$$d = 1.075\frac{V^2}{a} \tag{10}$$

where,

d: braking distance, ft
V: design speed, mph
a: deceleration rate, ft/s^2

Equation 10 assumes a deceleration rate of 11.2 ft/s^2. Using this deceleration rate to calculate stopping distance indirectly infers that the pavement has a specific pavement friction value. A minimum friction value is obtained from Equation 11.

$$a = fg \tag{11}$$

where,

a: acceleration, ft/s
f: friction coefficient
g: acceleration of gravity, 32.2 ft/s^2

Solving for *f*, the yielded FN value is 34.8 which can be approximated as 35. If friction becomes less than 35, the AASHTO principles in Equation 10 may not apply since the needed stopping distance will be greater than what AASHTO recommends.

Adequate pavement friction is expected to allow for a safe vehicle braking maneuver. Problems occur when this adequate friction is not present. Figure 13 shows how AASHTO's braking distance changes with friction for three different speeds. From Figure 13, it can be seen that when pavement friction decreases to a level lower that 35 (non-adequate level according to AASHTO), the braking distance required for a vehicle to stop increases considerably; this trend is particularly evident as speed increases. A braking distance of 340 feet at a speed of 60 mph and a friction value of 35 could increase to 480 feet for a friction value of 20.

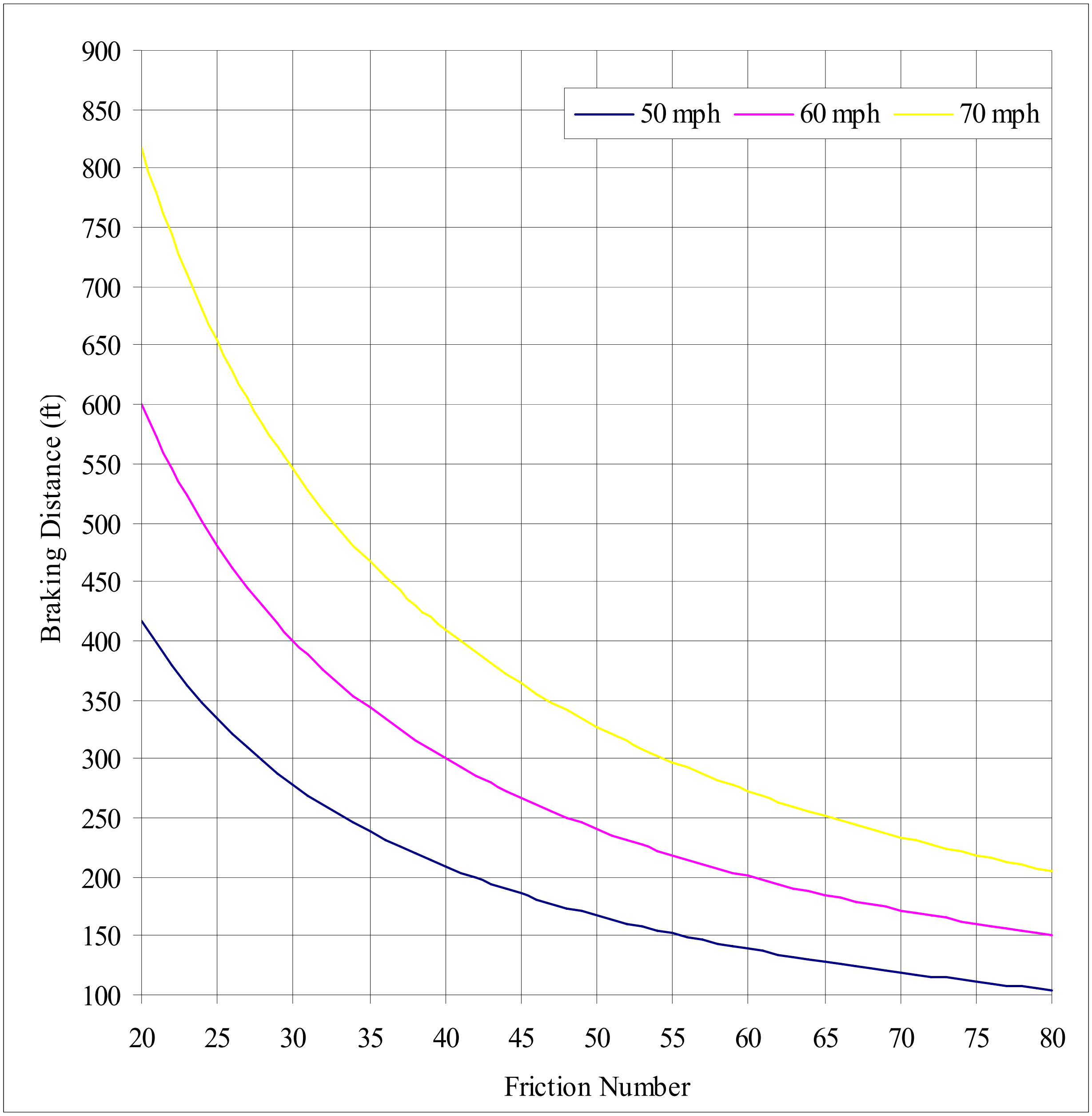


Figure 13 Variation of AASHTO's Braking Distance with Pavement Friction

ASTM establishes some limitations to the relation between stopping distances and pavement friction values measured with locked wheel testers (*10*). The friction values obtained with this device can be used to evaluate skid properties of the pavements but should not be used to quantify vehicle stopping distance since they do not resemble the specific and particular characteristics of a vehicle at the time of a crash. Harwood et al. added that friction number is measured for one particular tire design and one particular water depth, whereas a variety of tire design and water depths are found in the real road environment (*52*). The friction number by itself is a measure of available skid resistance only and does not reflect the level of friction demand for the maneuvers that are required of drivers at any particular site. In addition, the friction number measured at one particular speed is not adequate to completely define the frictional properties of a pavement surface. Two pavement surfaces can have similar friction values at 40 mph, but this can differ at higher speeds which are typical of operating conditions. Figure 14 shows the friction number and speed relationships for five different pavement surfaces. Relationships in Figure 14 are not linear but they are frequently approximated as linear. Both the friction number and the friction number-to-speed gradient, which indicates the rate at which the friction number decreases with increasing speed, must be considered in evaluating pavements.

Skid Resistance and Crash Risk

As previously indicated, the relationship between pavement surface friction and crash risk is difficult to quantify. Early attempts to relate these two factors were unsuccessful (*10*, *15*). Figure 15 shows the ratio of wet-to-dry crash frequency against skid number taken with a ribbed tire from sites in Kentucky. There is no direct correlation between wet pavement safety and the skid number. Nevertheless, many agree that if the roadway pavement is wet the risk of surface friction related crashes increase.

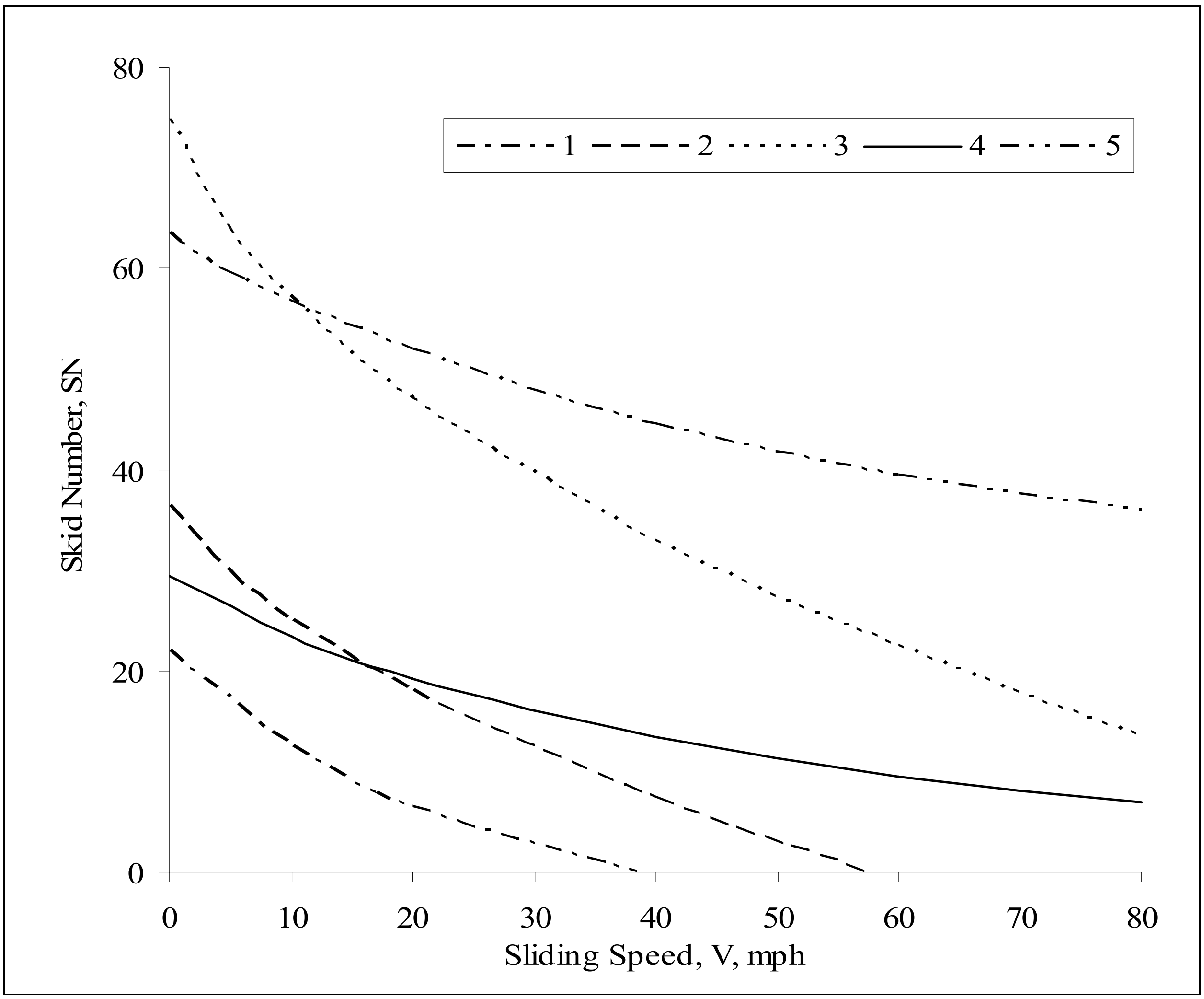


Figure 14 Friction Number-Speed Relationships for Different Pavement Surfaces (*52*)

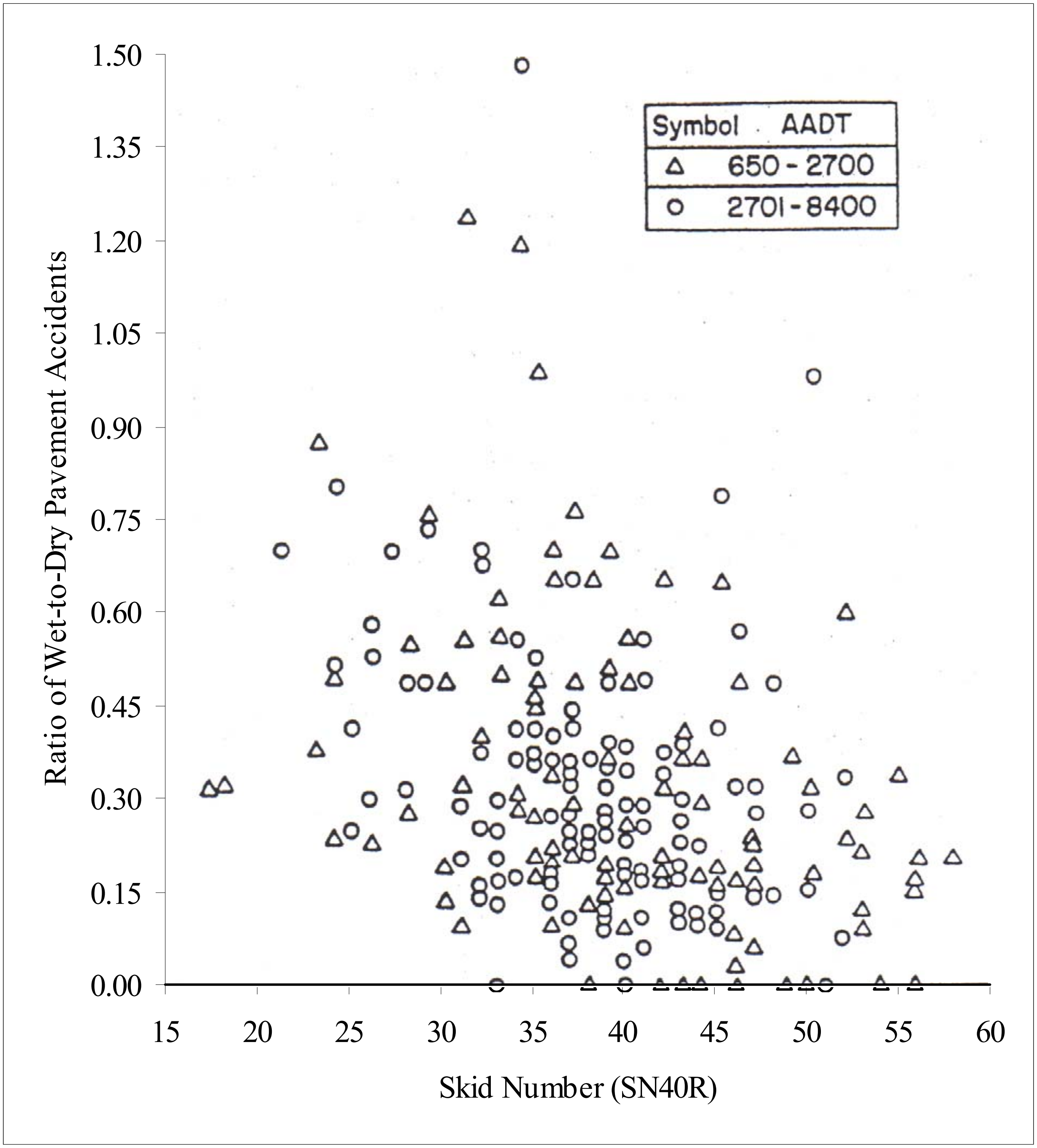


Figure 15 Ratio of Wet-to-dry Pavement Crashes versus Skid Number (*10*)

Initiatives like the Skid Accident Reduction Program (*63*) in the U.S. were developed to address the risk of surface-friction related crashes. The purpose of this program is to minimize wet weather skidding crashes through the identification and correction of roadway sections with high or potentially high skid crash history. In addition, the program addresses new surfaces by ensuring they have adequate and durable surface friction properties. The main problem herein is finding the friction threshold value that defines the breakpoint in crash frequencies.

Several transportation agencies have developed specified road friction threshold values that define the lowest acceptable road friction condition after which surface restoration will take place. For example, Maine, Washington, and Wisconsin use 35, 30, and 38, respectively, as their cutoff value (*10*). Meanwhile, Minnesota uses 45 as their acceptability level. The variability amongst these four states demonstrates the inconsistency between transportation agencies related to the minimum acceptable friction value. If the friction level is determined to be below the minimum designated threshold, it is assumed that the risk of crashes increases and therefore pavement maintenance is required by the agency.

A 2000 Missouri court case demonstrated the need for acceptable friction levels for pavement maintenance; two vehicles collided because of poor skid resistance (*64*). In this particular crash, one person died and another was seriously injured. The court ruled that it is the responsibility of the state Department of Transportation (DOT) to improve skid resistance and/or warn the motorist when highway is slippery to prevent crashes of this nature. Nonetheless, the Federal Highway Administration (FHWA) has resisted specifying a minimum friction level (*15*). Each state is best qualified to determine the conditions most appropriate for their particular situation. Austroads recently stated that no straightforward method exists for defining a skid resistance value at which a site automatically transforms from being “safe” to “hazardous” (*54*).

Giles, Sabey, and Cardew, as reviewed by Cairney, found that the risk of a skid-related crash was small for friction values above 60 but increases rapidly for skid resistance values below 50, as shown in Figure 16 (*3*, *65*). Their study compared skid resistance at 120 sites where a skid-related crash had occurred with 100 randomly chosen control sites on roadways of similar functional class and traffic volumes. The relative risk of a site being a skid-related crash site was computed by dividing the proportion of skid-related crash sites by the proportion of control sites for different skid-resistance categories.

McCullough and Hankins recommended a minimum desirable friction coefficient of 0.4 (FN = 40) measured at 50 km/h (30 mph) from a study of 571 sites in Texas (*3*, *66*). The study examined the relationship between skid resistance and crashes and found that a large proportion of crashes occurred with low skid resistance and relatively few occurred with high skid resistance. The recommended value was obtained as a convenient value close to the point where the slope of the resultant curves decreased.

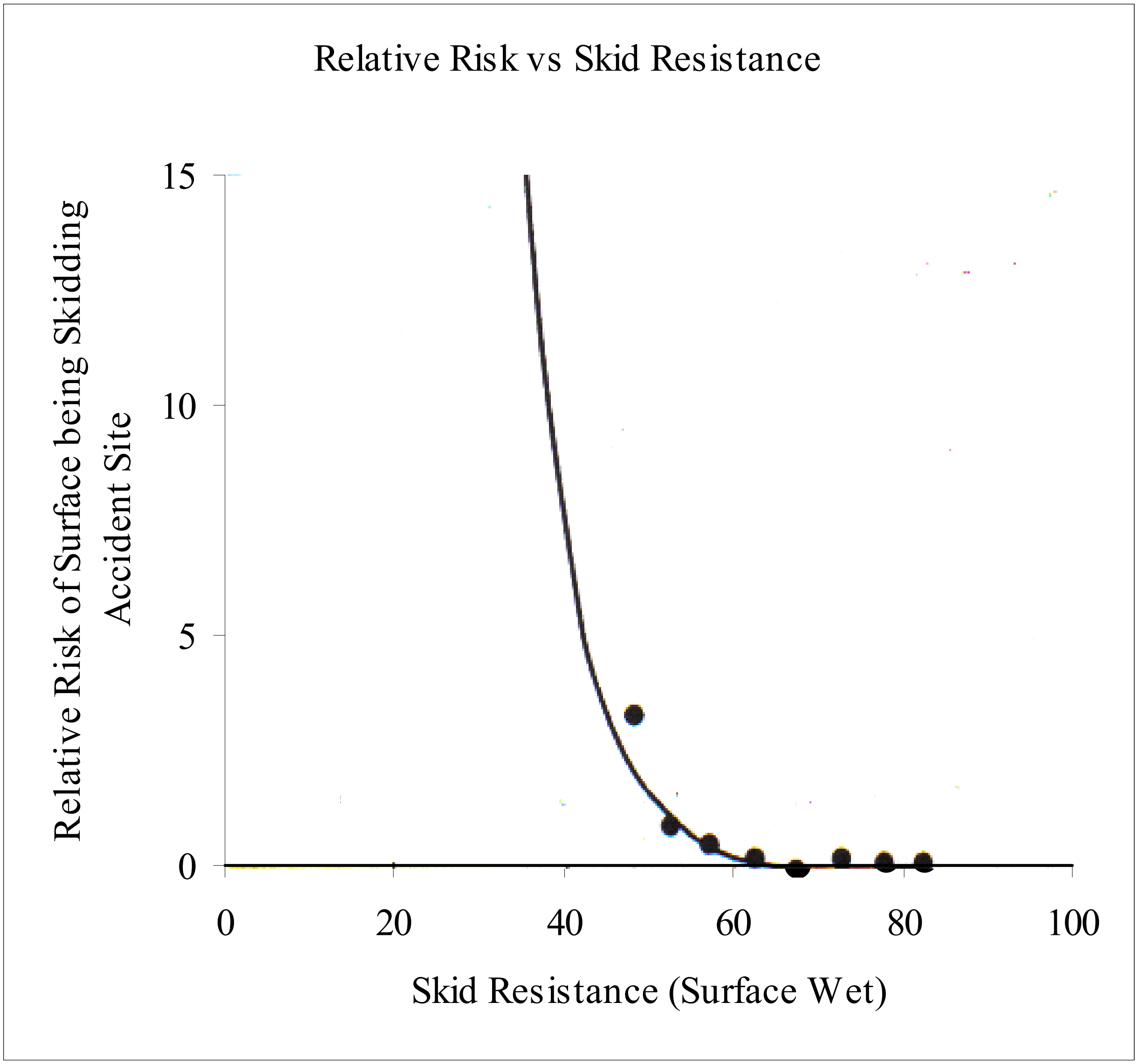


Figure 16 Relative Risk of a Surface Being Accident Prone as Function of Friction (*3*)

Before-and-after resurfacing studies had been frequently performed with the expectation that increased friction will yield a fewer number of crashes. Crash records are analyzed for similar time periods and on the same sections to evaluate if pavement surface improvement effectively reduces crashes. In this fashion, Cairney reported that Miller and Johnson carried out a before-and-after study on the M4 Motorway in England where resurfacing increased the average friction from 0.40 to 0.55 (measured at 50 mph) (*67*). Crashes were recorded for two years before and two years after resurfacing for a total of over 500 incidents. The data obtained reflected that pavement resurfacing led to a 28 percent reduction in dry pavement crashes and a 63 percent reduction in wet pavement crashes. Total crashes for the study area were reduced by 45 percent.

Kamel and Gartshore discovered similar results at selected hazardous spots in Ontario, Canada (*68*). Rehabilitation of pavement that had low friction levels, and had experienced a high rate of wet pavement crashes resulted in substantial reductions in crashes. Resurfacing treatments at intersections reduced crashes by 46 percent overall; 21 percent in dry conditions and 71 percent in wet conditions. In addition, the rehabilitation of eight freeway sites reduced total crashes by 29 percent; 16 percent in dry conditions and 54 percent in wet conditions. The improved mix design maintained better surface textures and provided longer lasting skid resistance characteristics.

Research by Gothie in the early 1990's, consisting of three separate studies, was aimed at defining the cause and effect relationship between road surface properties and risk of crashes in France (*15*). These studies involved the Bordeaux Ring Route, four trunk roads carrying 10,000 vehicles per day in southeast France, and a study of a trunk road section at a difficult site (two lane winding road) in central France. Crash rates on wet pavement increased in Bordeaux by at least 50 percent when moving from a section with an SFC (Sideway Force Coefficient) greater than 0.60 to a section with an SFC less than 0.50. Conclusions from the study were that every reduction in SFC of 0.05 increases the severity of crashes and increases the cost to society by approximately 50 percent. Severity of crashes was also found to be higher for shorter curve radii.

More recently, the New York DOT's Skid Accident Reduction Program (SKARP) identified sections of pavement experiencing unusually high proportions of wet road crashes (*69*). Before-and-after crash analyses have shown that each year more than 740 recurring crashes, from which 540 are wet-surface crashes, are being reduced as a consequence of treatments undertaken at 40 sites between 1995 and 1997 on Long Island alone. Treatment consisted of HMA resurfacing (1 ½ inches) or a thin cold emulsion microsurfacing, both using non-carbonate aggregates. Table 12 shows the detail results for crash reduction. The SKARP research assumed that a FN of 32 (FN40R) is adequate for sections of roadways with high operating demands, but others debate these results (*70*).

Contrasting results are indicated by McLean in a review of studies performed by the UK Road Research Laboratory (RRL), the U.S. Resurfacing, Restoration, and Rehabilitation (3R) Program and the Nordic Traffic Safety and Road Surface Properties Project (TOVE) (*71*). Crash rates on rural asphalt roadways were found to increase after resurfacing projects; meaning that surface improvements alone do not necessarily improve safety. Early RRL before-and-after studies found that when resurfacing of pavements improved ride quality, travel speeds tended to increase which may be why corresponding crash rates also increased. In a study of the 3R Program commissioned by the National Academy of Sciences (NAS), it was considered that both increased speeds and reduced driver attention contributed to the increase in crashes on resurfaced rural roads. The effect on crashes for the 3R Program is shown in Table 13 (*71*).

Table 12 SKARP Crash Reductions (*69*)

Study Period (months)	Number of Sites	WRA Before Treatment		WRA After Treatment		WRA Reduced		WRA % Reduction	
		T	F/I	T	F/I	T	F/I	T	F/I
7	5	22	15	4	2	18	13	82%	84%
19	13	346	224	91	40	255	184	74%	82%
31	6	72	56	28	14	44	42	61%	75%
36	16	348	280	124	76	224	204	64%	73%

Table 13 Effects of Crashes for the 3R Program (*71*)

Type of Crash	Percentage Change in Crash Rate		
	First Year After Resurfacing	Final Year of Project	Average Over Project Life
Wet Road	- 15	0	- 7
Dry Road	+ 10	0	+ 6
All Crashes	+ 5	0	+ 3

According to the 3R study, although wet pavement crashes were reduced, the increase in dry crashes resulted in an overall increase in crash rate. The TOVE Project provided three similar conclusions:

- Wet weather crash rates for slightly worn pavements are lower than for considerably worn pavements.
- Dry weather crash rates for slightly worn pavements are higher than for considerably worn pavements.
- Overall, crash rates for slightly worn pavements are higher than for considerably worn pavements.

In Sweden, similar research showed that the mean speed increased by slightly more than one km/h as a result of repaving (*15*). However, the TOVE Project did not consider speed differences as a major factor for the incidence of crashes. Instead, other contributing factors were suggested for the apparent tradeoff between wet weather and dry weather crashes, such as pavement ruts on old pavements reducing the amount of lateral variation in vehicle position.

It appears that crashes may also be in part a result of the practice of placing asphalt overlays on traffic lanes without associated shoulder work. This early practice of the 3R program increased the hazard presented by pavement edge drop off. McLean and Foley added that a study on 3R practices found an initial large increase in non-intersection crashes for resurfacing projects without shoulder work, but no change for projects which included shoulder work (*7*).

In 1984, the international Scientific Expert Group on Optimizing Road Surface Characteristics of the Organization for Economic Co-Operation and Development (OECD) indicated that in the U.S. any reduction in friction was associated with a steady increase in crashes (*72*). Detailed analyses revealed a linear crash-skid resistance relationship, as depicted in Figure 17, as the proper function for interpreting the data (*72*). This behavioral function conflicts with other relationships obtained from Europe. A study of high-speed rural roads in Germany suggested a non-linear relation as shown in Figure 18, with a higher slope for low friction values than for high friction values (*7*).

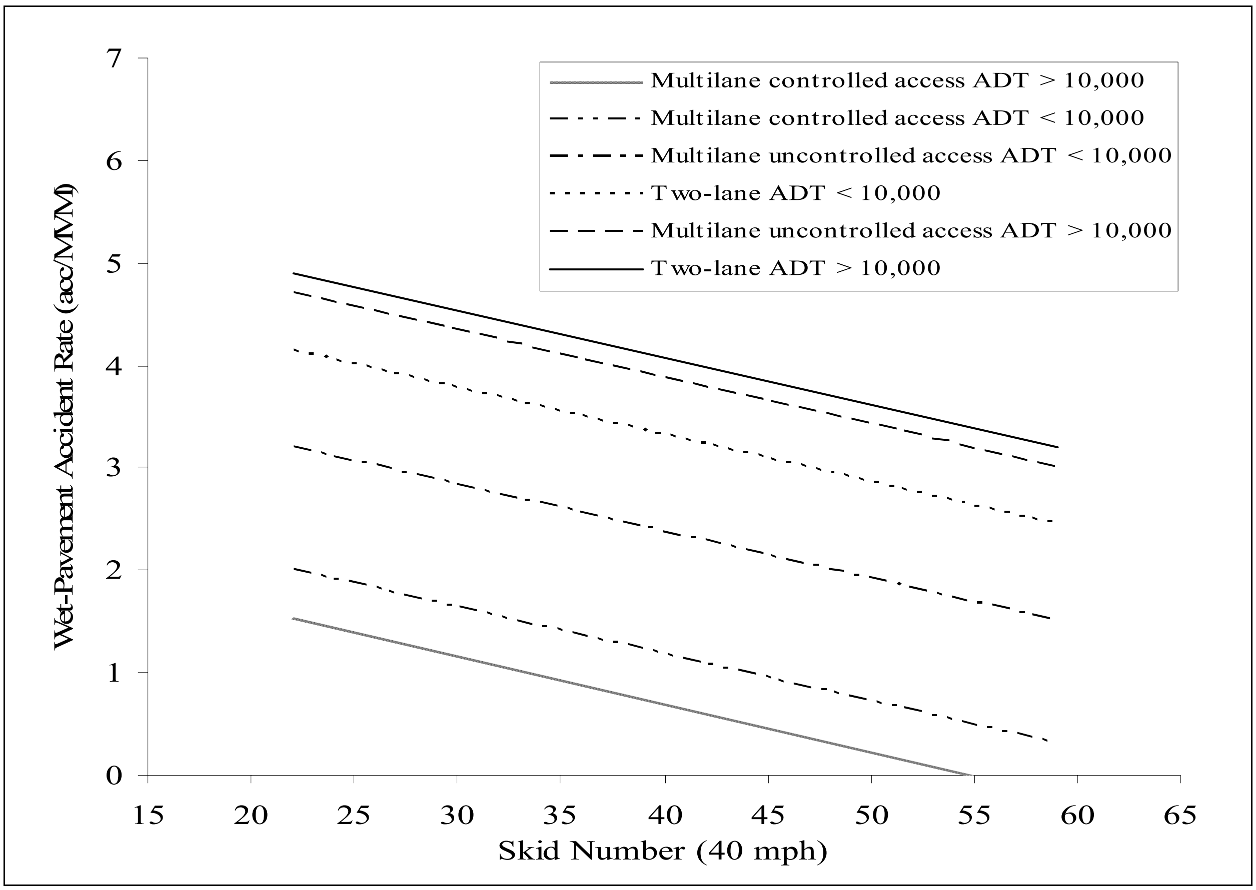


Figure 17 Relationship between Wet-Pavement Crashes and Friction (*72*)

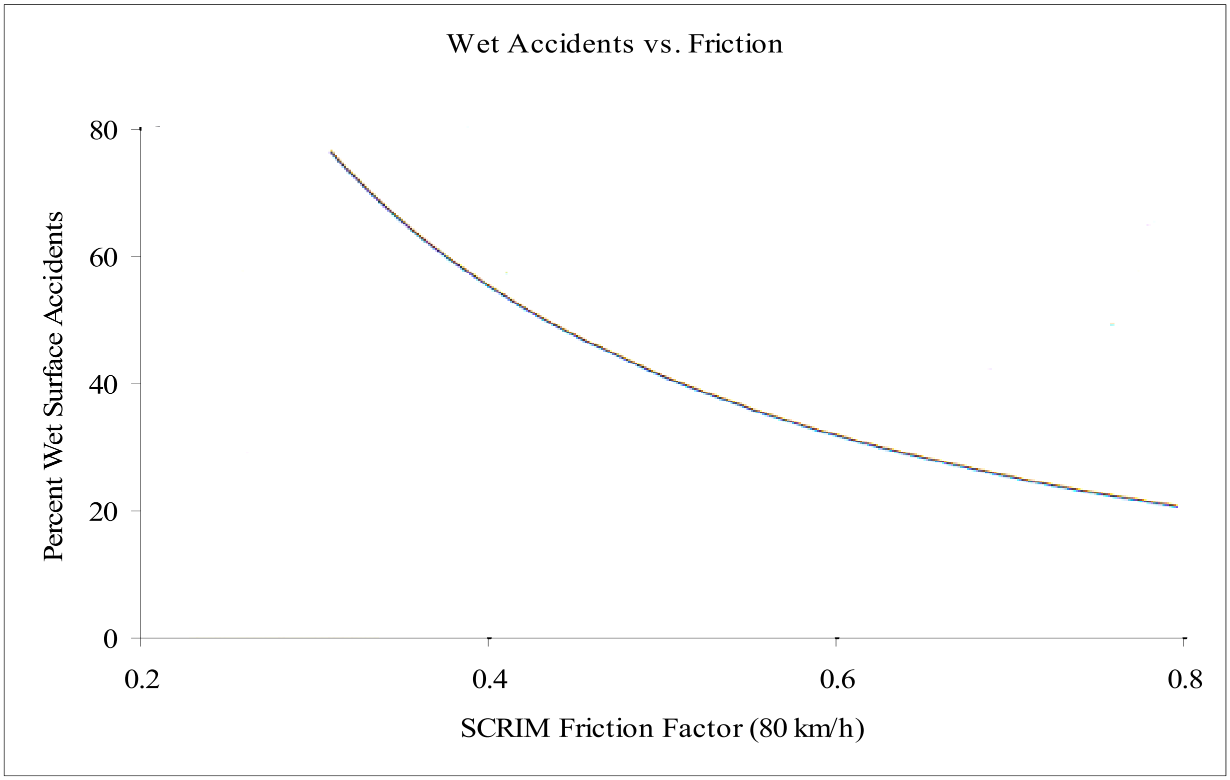


Figure 18 Non-Linear Relationship between Wet-Pavement Crashes and Friction (*7*)

Wallman and Astrom also reported a similar regression analysis in Germany by Schulze (*12*, *73*). This regression between friction numbers and crashes was not directly based on the frequency of crashes instead it was based on the proportion of crashes that occurred under wet pavement conditions. This proportion of wet pavement crashes varied between zero percent and approximately 50 percent on most of the road sections studied. If on any particular section of road the proportion of wet pavement crashes significantly exceeds this range of percentages, then this can be taken as an indication of reduced traffic safety under wet conditions. Figure 19 shows the general trend of the increasing percentage of wet surface crashes with the decreasing friction level (*12*). Henry pointed out that since many other factors contribute to crashes, including pavement conditions, prevailing speed, and traffic conditions, one should not expect to be able to predict crash frequency from skid resistance data alone (*10*).

Another study described by Wallman and Astrom with similar behavior is the Norwegian Road-Grip Project. In this study, comprehensive friction measurements and roadway observations were completed resulting in the assessment of crash rates for different friction intervals as summarized in Tables 14 and 15 (*12*).

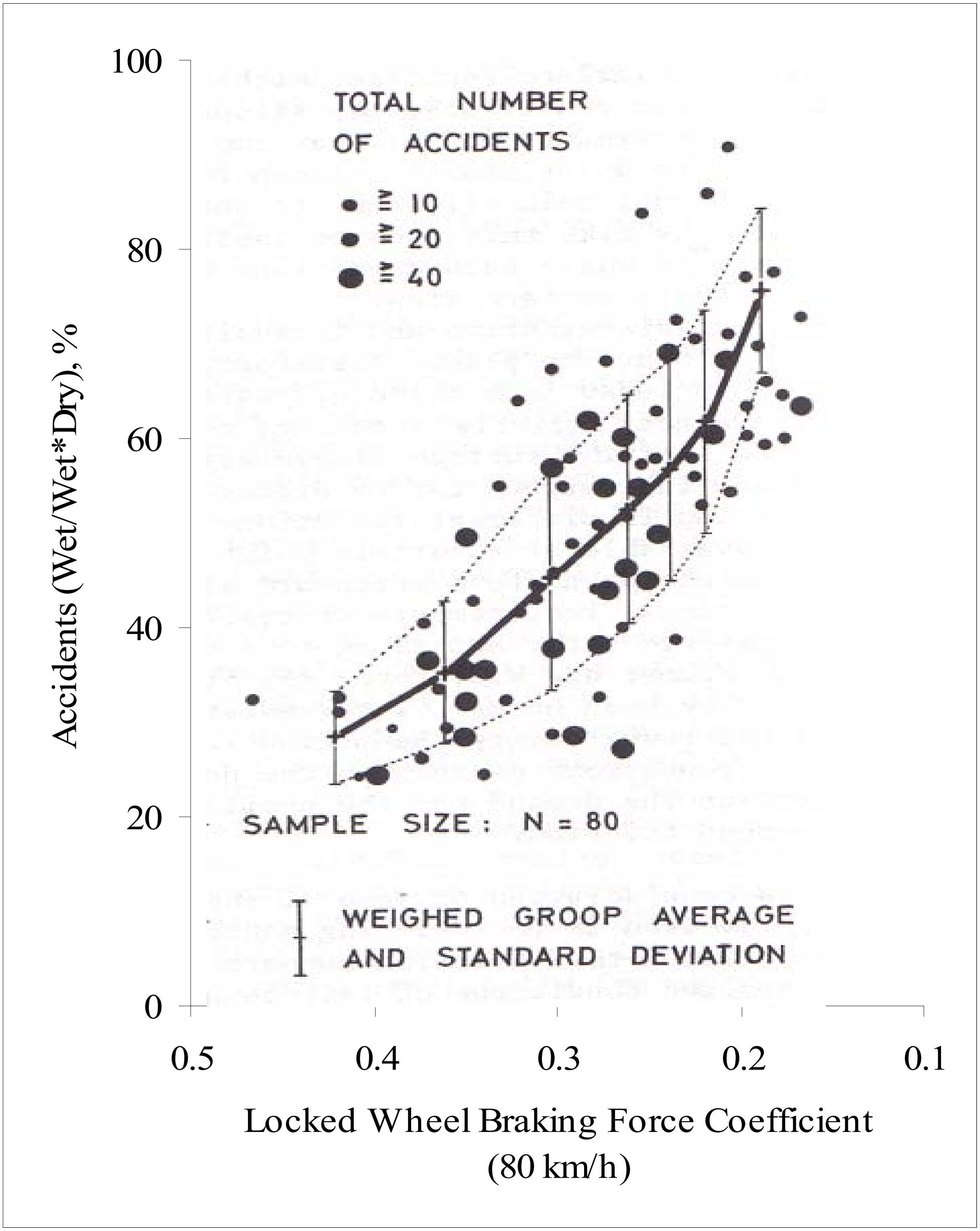


Figure 19 Percentage of Wet-Pavement Accidents and Friction (*12*)

Table 14 Crash Rates for Different Friction Intervals (*12*)

Friction Interval	Crash Rate (personal injuries per million vehicle kilometers)
< 0.15	0.80
0.15 – 0.24	0.55
0.25 – 0.34	0.25
0.35 – 0.44	0.20

Table 15 Crash Rates at Different Roadway Conditions (*12*)

Roadway Condition	Crash Rate (personal injuries per million vehicle kilometers)
Dry bare roadway, winter	0.12
Wet bare roadway, winter	0.16
Slush	0.18
Loose snow	0.30
Ice	0.53
Hoarfrost	0.53
Packed snow	0.31
Bare ruts	0.12
Black ice in ruts	0.30
Dry bare roadway, summer	0.14
Wet bare roadway, summer	0.18

The Nordic TOVE project provided similar results for two lane highways in Denmark, shown in Figure 20, for friction values obtained with a Stradograph, a side force device (*12*). In another study involving accident per million vehicle kilometers traveled (VKT); Schlosser examined their relationship with skid resistance (*3*, *74*). The main findings were that wet accidents per VKT reduce as skid resistance increases up to a value of six or seven, with little reduction after that. At these higher levels of skid resistance, the accident rate is very low.

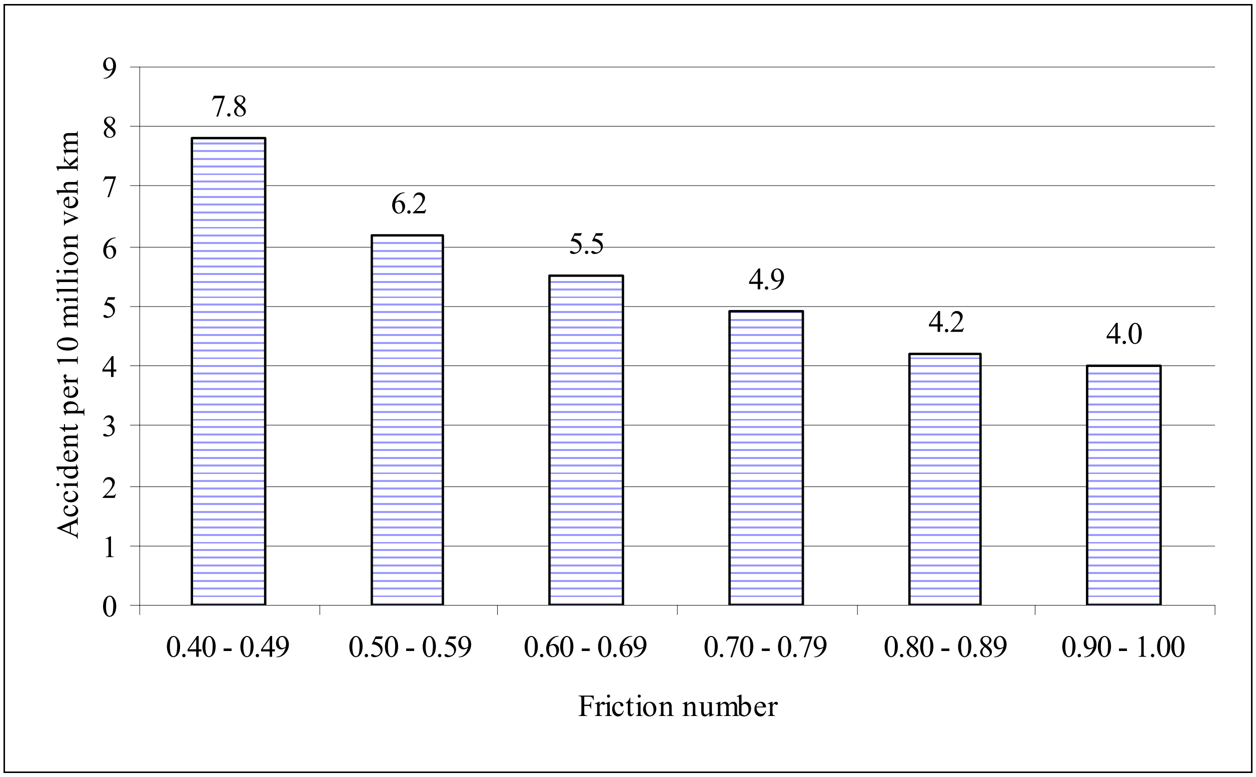


Figure 20 Crash Rates as a Function of Friction (*12*)

Cairney mentioned that all studies that relate skid resistance to crashes, conducted to date, suffer from data limitations to different extents; skid resistance data itself being a major deficiency (*3*). This opinion is particularly true when seasonal and lane variations of friction are not taken into account. On multi-lane roads, it appears to be usual practice to measure skid resistance in only one lane, and then apply this to all crashes. It would be expected however that skid resistance would vary from lane to lane and from wheel path to wheel path, since there are different traffic flows on each lane, particularly different flows of heavy vehicles.

Other data limitations consist of not accounting for vehicle speeds, posted or measured, or geometric factors like gradient. Higher friction values are needed for higher speeds, and are more critical on down grades to account for the additional stopping distance required. In terms of crash data, few investigators appear to be worried about the accuracy with which crash locations are described on official records, with the consequent possibility of attributing an inaccurate skid resistance to the crash site. In addition, crash data imposes constraints to investigators and it appears that their interest in the type of crashes reduced with improved friction is not a priority.

Kinnear et al. carried out one of the few studies that differentiate between different types of crashes (*3*, *75*). In this study, three high skid resistance treatments, calcined bauxite epoxy resin, rhyolite asphaltic concrete, and sand tar mix were compared with the usual river gravel asphaltic concrete and with control sites at which nothing was done. The sites were characterized as follows: control sites had an average SFC of just over 0.50; the river gravel asphalt sites and the sand tar sites had an average SFC of 0.63; the rhyolite asphalt sites had an average of 0.72; and the calcined epoxy resin sites had an average of SFC 0.95. The sand tar sites and the calcined epoxy resin sites retained their value throughout the study period, all the rest decreased from their original SFC. Table 16 shows the crash reductions in addition to an economic analysis of the benefit cost ratios associated with the different treatments.

Yager describes the considerable interest that currently exists in the interactions between vehicle and pavement surface as they affect safety, comfort, convenience, and economics, including user costs (*76*). He recommended focus on the evaluation, modeling, and understanding of these interactions and the studies that identify, quantify, measure, and model the factors influencing these interactions. Yager suggests that long-term improvements be sought in many areas including methods, apparatuses, and procedures for measuring friction of roadway surfaces during all conditions, especially winter conditions. Appropriate and accurate friction information can be used to determine roadway surface roughness correlation and improved methods for evaluating wet weather vehicular crash sites.

Lindenmann affirmed that although it is undeniable that correlations exist between skid resistance and crash occurrence; quantifying these correlations is only possible – if at all – for individual situations and not in general (*77*). His study of the influence of pavement skid resistance on wet crash occurrence in freeways covered the whole Swiss National Highway Network. The fundamental finding of the Lindenmann study is that no quantifiable and analytically describable correlation could be found between skid resistance and crash occurrence. The results were not applicable to mixed-traffic rural and urban highways.

Table 16 Crash Reductions, Cost of Treatments, and Benefit/Cost Ratios (*3*)

Crash reductions	Rhyolite	Sand/tar	Cal Bauxite	River Gravel	Control
Head on*	4	4	2 [1]	1 [1]	8 [2]
All angular	25 [1]	32 [1]	44 [1]	NS	NS
Rear end	31 [2]	46 [2]	25 [2]	24 [1]	NS
Other	47 [2]	45 [2]	6 NS	3 NS	NS
Total value of crash reductions	2,723,452	3,085,252	1,634,391	393,912	851,738
Adjusted annual value[3]	1,311,114	1,496,454	524,378	-134,871	N/A
Cost of treatments[3]	106,018	120,078	244,108	36,165	0
B/C ratio	84	60	15	Negative	N/A

[1] Statistically significant at the 0.05 level

[2] Statistically significant at the 0.01 level

[3] Currency units were not provided. May be Australian Dollar.

Skid Resistance and Surface Texture

As mentioned previously, wet surface friction decreases with increasing speed, and the rate of this decrease is affected by macrotexture. However, many standard measures of skid resistance are conducted at relatively low speed and primarily measure the microtexture contribution to surface friction. This procedure means that for roads that operate at high speeds, a measure of low-speed skid resistance alone may not be an adequate indicator of skidding crash risk. The IFI addresses this issue by allowing vehicles to operate at any safe speed without an impact in the friction number (*10*).

Concentration on microtexture properties only can be misleading. McLean and Foley cited research by Roe, Webster, and West where they investigated the effects of both low speed skid resistance and macrotexture on crash frequency for rural roads in Great Britain (*7*, *78*). The study concluded that macrotexture had a greater influence on crash frequency than low-speed skid resistance. Both wet and dry weather crashes were found to increase for a texture depth of less than 0.7 mm measured with laser (sand patch texture depth of about one mm). Roe, Webster, and West developed an inter-relationship between texture, skidding resistance, and crashes (*15*, *78*). One approach to estimate suitable level of texture was to determine the texture level at which the proportion of crashes exceeds the proportion of textures on the road network. The level was found to fall between 0.6 mm and 0.8 mm, indicating that the risk of crashes is greater for roads with an average Sensor Measured Texture Depth (SMTD) below 0.7 mm than those above.

Similarly, Gothie in a study of crashes and surface characteristics on open roads in France found that wet-weather crashes increased markedly for sand-patch texture depth less than about 0.5 mm (*79*). These results are reproduced in Figure 21 (*7*).

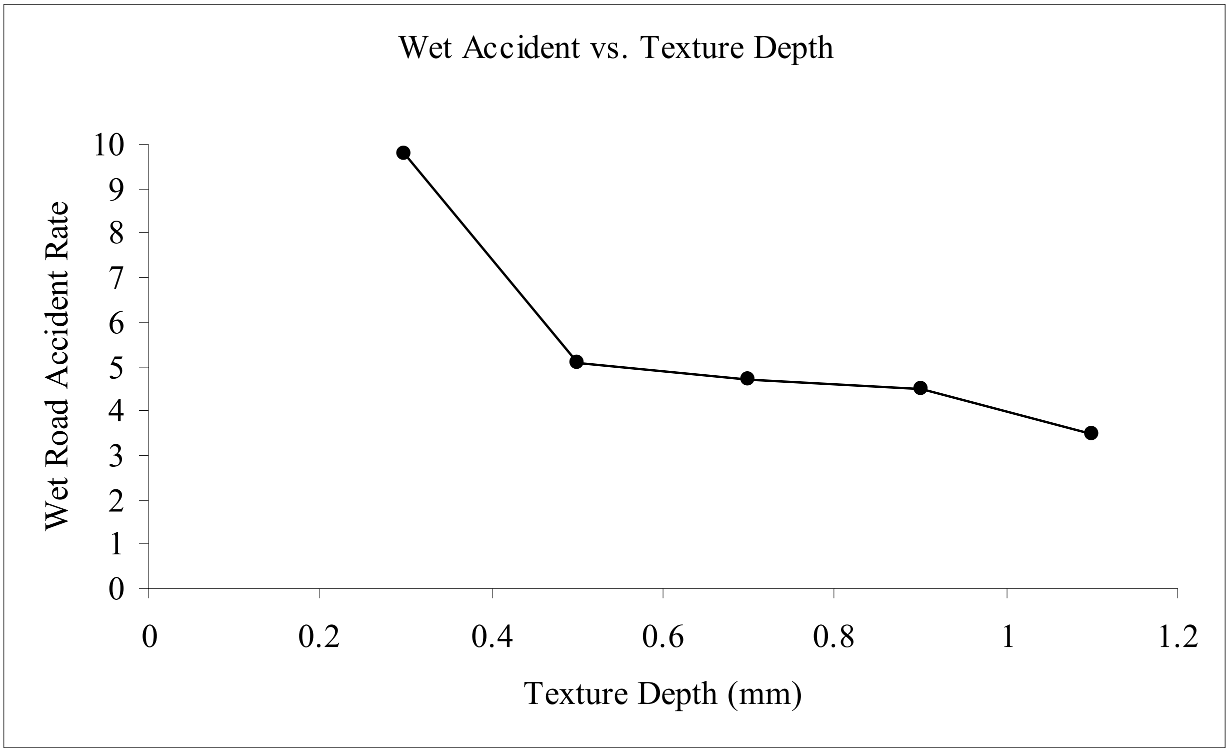


Figure 21 Relationship between Wet-Accident and Surface Texture Depth (*7*)

In order to optimize texture and frictional properties over the life of a pavement, the materials used to create the pavement should ensure the greatest initial friction. A study performed by Goodman et al. was conducted to determine the difference (if any) of samples prepared in the laboratory versus the field (*85*). The surfaces resulting from Superpave gyratory compaction was tested to determine if the macrotexture and frictional properties could indicate the properties of the final pavement surface and thus being able to change the mix design prior to construction. Eight different paving projects were selected from the City of Ottawa's annual resurfacing program. Superpave gyratory compaction was used to create samples of the paving specifications for the eight sites. The gyratory specimens were cut so that top and bottom surfaces could be tested separately. According to ASTM E E965, sand patch testing was conducted on both the top and bottom gyratory surfaces as well as the field specimens. The British Pendulum Number (BPN) and Mean Texture Depth (MTD) were also determined.

The BPN for the top gyratory surfaces were generally greater than those for the bottom surfaces, but this was not statistical relationship was found. The BPN values recorded for the bottom surfaces best matched the field pavement with the exception of the Stone Mastic Asphalt. This was opposite of the macrotexture investigation where the MTD values for the top gyratory specimens were better correlated with the field specimens. A preliminary relationship between BPN and mix properties (fineness modulus, bulk relative density, percent passing the 4.75 mm sieve and asphalt cement content) was developed to aid friction optimization at the mix design stage. The following equation shows the relationship:

$$BPN_{InitialField} = 42.32 + 2.95(\frac{P4.75 * BRD}{FM * AC}) \qquad (12)$$

where,

$BPN_{InitialField}$ = Initial field British Pendulum Number;
P4.75 = Percent of aggregate passing the 4.75 mm sieve;
BRD = Bulk relative density;
FM = Fineness modulus; and
AC = Asphalt cement content.

The results also suggest that the gyratory compactor orients aggregate particles in a different manner than the field compaction equipment.

Research by Cairney and Styles (*80*) concluded that crash risk is greater at sites with low macrotexture, in agreement with previous studies. However, there is no agreement about the precise value of macrotexture at which crash risk begins to rise. In their study it was stated that before a surface management process can be developed based on macrotexture, further study is required to examine the relationship between crash risk and macrotexture in more detail. It was recommended that further study should take into account intersections, road geometry, and road surfacing materials. In addition, they suggested a study of vehicle braking distances on surfaces with different macrotexture to demonstrate their effect in situations where increased macrotexture might be required.

Macrotexture impact on safety is not well understood by transportation professionals. Efforts to better understand macrotexture as it effects skid resistance, and how macrotexture can be incorporated in the pavement safety equation appear to be promising directions for research. There have been five International Symposiums on Surface Characteristics of Roads and Airfields (State College, PA in 1988; Berlin, Germany in 1992; and Christchurch, New Zealand in 1996, Nantes, France in 2000, and recently Toronto, Canada in 2004) designed to gather information on the latest progress in these areas. Relevant information for these conferences has been included in this chapter.

Friction and Pavement Management Integration

Pavement management usually overlooks macrotexture and focuses more on microtexture; a greater problem is that sometimes skid resistance is not considered to be a factor in roadway safety and is not incorporated in the pavement management process. End-result specifications for texture and friction levels are also needed to ensure desired levels are obtained during construction (*81*). Jayawickrama (*4*) conducted a study of state practices to control skid resistance on asphalt roadways. Five categories of practice were considered:

- 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; and
- V – Incorporates field performance in aggregate qualification.

Table 17 summarizes the results (*4*). Note that 14 states reported that friction/skid resistance was not considered in the design of new pavements. Ohio was the only FHWA Region Five state in this category. Most states use laboratory tests to evaluate frictional properties of aggregates. Only four states attempted to use field performance of aggregates in their aggregate evaluation. Zero states used field skid performance as the sole basis for aggregate qualification. Although many research studies have identified macrotexture as a major contributor to friction most state DOTs do not account for it. The vast majority of states design procedures focus on microtexture by controlling aggregate quality. This was also confirmed by another more recent survey that showed that only five state agencies measure macrotexture and only three of these states measure it routinely (*10*).

Song et al. conducted a network level analysis of pavement skid resistance for the State of Maryland using the Maryland State Highway Administration (MDSHA) Pavement Management System (PMS) (*86*). Approximately 15,000 lane-miles of pavement are covered by the MDSHA PMS, with 61 percent flexible pavements, two percent rigid pavements, and 37 percent composite pavements (rigid pavement base with flexible HMA overlay). The MDSHA annually checks pavements within the network using a locked-wheel skid trailer with ribbed tires to find calculate friction numbers (FN) on the outside lane of all sections at 0.3 mile intervals. Testing was performed at approximately 40 mph, with 81 percent of all friction numbers obtained at speeds between 38 and 41mph. This testing has shown that skid resistance reaches a steady state in about one year after resurfacing.

Thirty-one percent of test sections were in urban areas, and 69 percent were in rural areas. Findings showed that the average friction number of rural roads and urban roads are 48.5 and 42 respectively - about 6.5 FNs higher on rural roads. One year after new surfacing, skid resistance continues to deteriorate, although the deterioration rates are relatively slow. Regression analysis shows that on average the deterioration rates of rural pavements and urban pavements are 0.22 and 0.26 FNs per year respectively, and FNs of rural roads are between six and eight units higher than those of urban roads. The regression analysis also indicate that the difference in friction between rural and urban roads appears one year after resurfacing and after two years the difference does not change significantly.

Aggregates were also reviewed for friction changes by different Superpave aggregate gradations: 4.75 mm, 9.5 mm, 12.5 mm and 19.0 mm. It was found that 4.75 mm aggregates are mostly used for comparatively low traffic and 19.0 mm aggregates are used for comparatively high traffic. Aggregates in categories 9.5 mm and 12.5 mm are used in a relatively wide traffic intensity range. Regression analysis indicated that 9.5 mm provides a higher friction number than 12.5 mm. This difference gradually decreases with increasing traffic intensities.

Table 17 Categories of Design Procedures Used by State DOTs (*4*)

State DOT	Category					State DOT	Category				
	I	II	III	IV	V		I	II	III	IV	V
Alabama				X		Nebraska		X			
Arizona		X				Nevada	X				
Arkansas			X			New Hampshire		X			
California	X					New Jersey				X	
Colorado	X					New Mexico	X				
Connecticut	X					New York				X	
Delaware			X			North Carolina	X				
Florida				X	X	North Dakota	X				
Georgia			X			Ohio	X				
Idaho	X					Oklahoma				X	
Illinois			X			Oregon		X			
Indiana				X		Pennsylvania				X	X
Iowa				X		Rhode Island	X				
Kansas			X			South Carolina				X	
Kentucky				X	X	South Dakota		X			
Louisiana				X		Tennessee				X	
Maine		X				Texas				X	X
Maryland	X					Utah				X	
Massachusetts	X					Vermont		X			
Michigan				X		Virginia			X		
Minnesota				X		Washington		X			
Mississippi				X		West Virginia			X		
Missouri	X					Wisconsin				X	
Montana		X				Wyoming	X				
Nebraska		X									

The authors also looked at the correlation between network average FN and average temperatures, and found that a strong linear correlation existed between the average daily temperature and the average FN: A one degree (˚*F*) increase of temperature leads to one unit decrease of FN. There is a similar (but not as strong) correlation between average daily rainfall and the average FN: a 0.1 inch increase in daily average rainfall leads to a 1.26 increase in FN. The authors conclude that climate related influences should be considered before evaluating the pavement network friction between years, and that climate information should be incorporated into the PMS and pavement skid resistance analysis to help decision makers make a well-informed judgment of engineering efforts.

Pavement and skid resistance management is important enough that the Federal Aviation Agency (FAA) developed the Advisory Circular: Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces as the guideline for the design, construction, evaluation, and maintenance of high skid-resistant runway pavements (*82*). Runway surface performance has been closely monitored for the safety of the people traveling on aircrafts. According to the Advisory Circular, runway friction characteristics depend on type and frequency of aircraft activity, weather, and environmental issues, among other factors. Also contaminants such as rubber deposits, dust particles, jet fuel, oil spillage, water, snow, ice, and slush all cause loss of friction properties on runway pavement surfaces. To achieve safety operations on runways, the FAA has been working extensively with the National Aeronautics and Space Administration (NASA) and United States Air Force (USAF). Emphasis has been given to pavement surface design, evaluation, and maintenance techniques to detect deterioration of skid resistance.

The FAA has also developed minimum frequencies for runway friction surveys as shown in Table 18. This helps airport operators to detect abnormalities and take corrective actions in a timely fashion. According to the need, friction is measured and compared to the friction values obtained with the Continuous Friction Measuring Equipment (CFME) as shown in Table 19. These values are guidelines for evaluating the surface friction deterioration of runway pavements and for identifying appropriate corrective actions required for safe aircraft operations.

Table 18 Friction Survey Frequency (*82*)

Number of Daily Minimum Turbojet Aircraft Landings Per Runway End	**Minimum Friction Survey Frequency**
< 15	1 year
16 – 30	6 months
31 – 90	3 months
91 – 150	1 month
151 – 210	2 weeks
> 210	1 week

Table 19 Friction Level Classification for Runway Pavement Surfaces (*82*)

Method	40 mph			60 mph		
	Minimum	Maintenance Planning	New Design/ Construction	Minimum	Maintenance Planning	New Design/ Construction
Mu Meter	.42	.52	.72	.26	.38	.66
Runway Friction Tester	.50	.60	.82	.41	.54	.72
Skiddometer	.50	.60	.82	.34	.47	.74
Airport Surface Friction Tester	.50	.60	.82	.34	.47	.74
Safegate Friction Tester	.50	.60	.82	.34	.47	.74
Friction Meter	.43	.53	.74	.24	.36	.64
Tatra Friction Tester	.48	.57	.76	.42	.52	.67
Norsemeter RUNAR (at fixed 16% slip)	.45	.52	.69	.32	.42	.63

Incorporating skid resistance and providing uniformly high levels of skid resistance across an entire road network as has been done for runways would be extremely difficult and expensive. For this reason, Austroads has developed *Guidelines for the Management of Road Surface Skid Resistance* which presents cost-effective strategies to manage a network that provides an appropriate level of skid resistance (*54*). These guidelines provide information necessary for a road authority to develop and implement a local strategy to manage skid resistance on its road network based on a framework containing 16 key elements as shown in Table 20.

Table 20 Key Elements for the Development of Strategies for the Management of Skid Resistance (54)

Policy and Management
1. Defining objectives
2. Defining responsibilities
3. Policy on the release of skid resistance data to the public, media and legal profession
4. Dissemination and training
Network Standards
5. Materials standards
6. Policy and process for selection of surfacings
7. Standards for construction and maintenance activities
8. Setting levels for skid resistance and surface texture
Testing
9. Selection of test equipment and setting a testing regime
10. Calibration and maintenance regime
11. Collection, processing and display of data
Analysis and Action
12. Use of data to identify and prioritize sites for investigation and/or treatment
13. Site investigation process
14. Policy on the use of warning signs
15. Selection of remedial actions
Review / Improvement
16. Monitoring, review, and improvement

Strategies to be developed by a road authority are presented by Austroads as:

- Proactive – To measure skid resistance and/or surface texture, routinely or on a less frequent basis, on all or specific parts (based on local criteria) of the network, to assist in the identification, prioritization, and programming of sites where remedial action may be required in advance of crash rates increasing;
- Reactive – To measure skid resistance and/or surface texture at sites where a number of crashes have already occurred (such as identified crash black spots), or a recent incident or crash is required to be investigated, to determine any possible contribution of the road surface to the incident/s or crashes;
- Research – To measure skid resistance and/or surface texture as part of research into the in-service performance of materials or maintenance processes; and
- No Testing – Not to measure skid resistance, and to rely on road surface construction and maintenance regimes, visual inspections of the network, and/or the measurement of surface texture.

A weakness of some skid-resistance strategies is that they tend to promote resealing or resurfacing as the only real option to a potential skid resistance issue at a location. The multi-disciplinary approach advocated by the Austroads guidelines requires an awareness that road safety engineering countermeasures can be used alongside more traditional road surface maintenance actions in such situations. Rather than reseal or resurface, other features may be discussed and be found to be potentially more effective such as: additional warning signs, the introduction of a speed advisory sign, speed limit, or a subtle change to the road markings.

Austroads also recommends that road authorities mitigate the incidence of crashes involving skidding by:

- Designing and managing a road network so that maneuvers demanding high levels of skid resistance are avoided as far as possible;
- Ensuring that a road surface has performed adequately in service, through the disciplines of inspection and routine, programmed, and reactive maintenance activities;
- Being aware of highway management and maintenance activities that have the potential to impact the level of skid resistance generated at a location; and
- Encouraging and educating road users to maintain that their vehicles (particularly with respect to tires, brakes, and suspension) and to moderate their speed in wet-road conditions.

Austroads Guidelines state that the key to successfully maintaining or improving road-network user safety, and mitigating the risk of claims, is to set relevant unambiguous strategies, systems, policies, standards, and practices which can consistently and demonstrably be achieved within the available resources. Failure to meet over-ambitious policies and standards (albeit set with the best of intent) has been found to dramatically increase the vulnerability of road authorities.

The UK has also developed a process that allows surveys and treatments to be carried out to bring the pavement surface up to the required level (*15*). Figure 22 shows the recommended steps for the management of pavement skid resistance. Investigatory levels of friction were previously mentioned in Table 5.

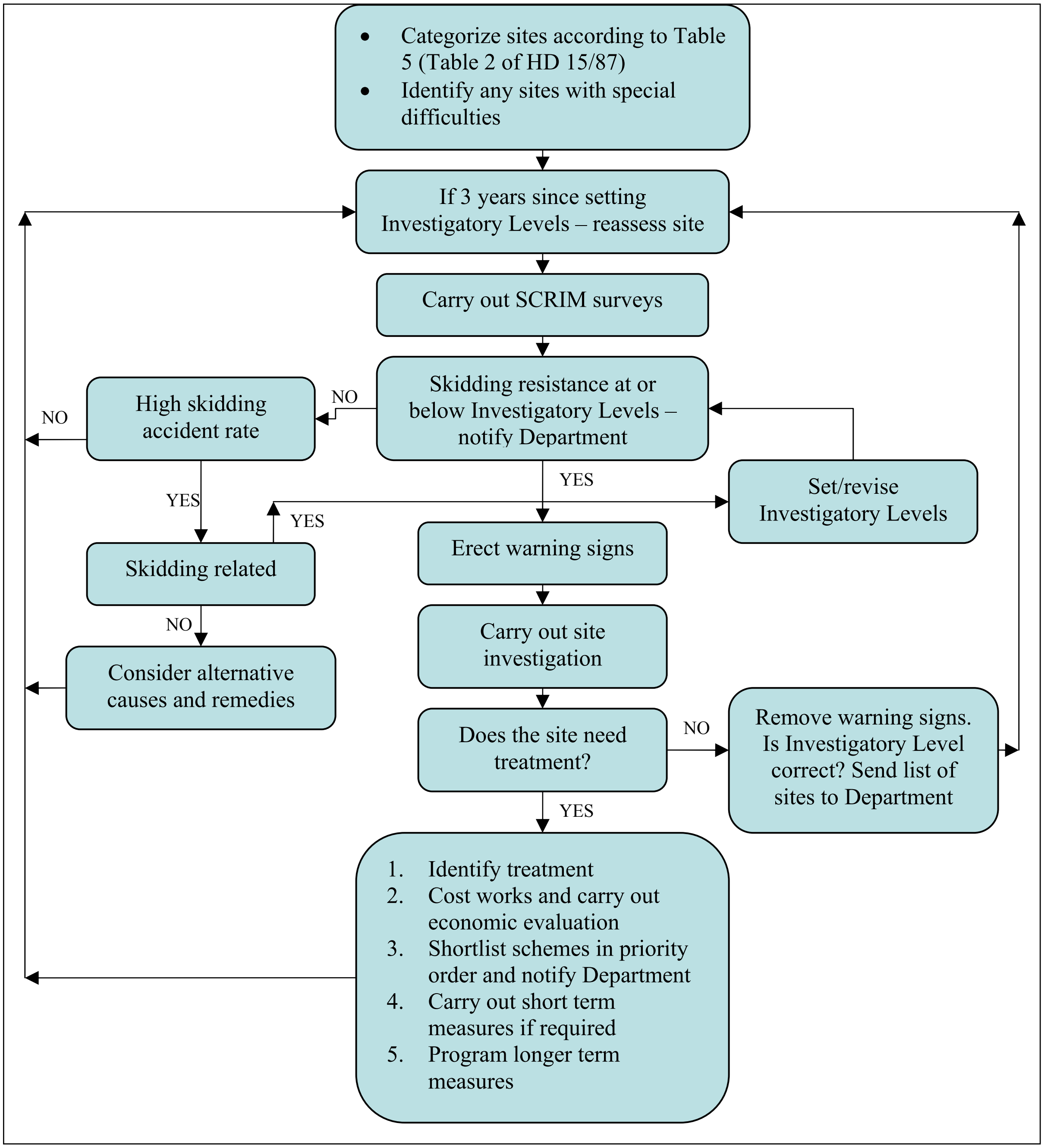


Figure 22 Procedure to Identify and Treat Sites with Skidding Problems (*15*)

Small and Swisher developed a proposed framework for integrating asset management and pavement management systems (*83*). The framework for integration was based on the following seven significant objective factors determined to be important to transportation decision-makers:

- Agency cost;
- User cost;
- Safety;

- Mobility;
- Environment;
- Quality of ride; and
- Relative use of the system.

The integration is performed to provide a mechanism for the assessment of network performance. This integration is an example of the need to incorporate safety elements into pavement processes, namely skid resistance and its constituents. In addition, it will help to develop guidelines for pavement management and for deciding when maintenance activities are justified solely for skid friction safety improvements.

Summary of Findings

Skid resistance is an important consideration in highway safety. There is evidence to suggest that low skid resistance results in increased frequency of wet-weather crashes. Some studies have found that wet-weather crash rates increase linearly with respect to decreasing skid resistance. Other studies suggest that the relationship may be non-linear; the slope increasing while skid resistance decreases. The common point is that a decrease in pavement skid resistance will likely result in an increase in crash risk. Pavement management strategies need to be developed to integrate skid resistance in the mix design and safety considerations. Maintaining high levels of skid resistance is important especially where there is frequent braking in response to unexpected events, such as intersection approaches.

There exist sufficient studies to indicate that microtexture and macrotexture affect the provision of skid resistance. Nonetheless it has been indicated in many studies that skid resistance coefficient measurements correlate mostly with microtexture but not macrotexture. The role of each one in providing sufficient pavement friction varies depending on the speed, aggregates, and other characteristics.

It is well recognized that microtexture is a function of the initial roughness on the aggregate surfaces and the ability of aggregates to resist polishing. Selection of aggregate mineralogy and measuring its polishing resistance has been used widely as a measure of potential microtexture. Microtexture is considered a controlling factor for skid resistance at low speeds but not high speeds, although it contributes to high speed skid resistance. Because of the difficulty of quantifying aggregate roughness and polishing resistance, microtexture is best measured using surrogate tests that allow measuring pavement friction at low speeds using small scale devices such as the British Pendulum and the Japanese DFTester.

Macrotexture is mainly a function of surface texture, the large-scale roughness that is present due to arrangement of aggregate particles or the grooving created intentionally on a roadway surface. The importance of this surface roughness is that it promotes reduction of water film thickness during wet conditions and thus reduces the possibility for hydroplaning. Macrotexture can be measured by a volumetric method using sand, grease, or glass beads, or by using a laser profiler. The laser method can be done at reasonably high travel speeds and thus is suitable for continuous pavement network monitoring.

A common finding is that not much has been done in terms of correlating macrotexture with skid resistance. Although to provide for sufficient pavement friction it is important to consider both microtexture and macrotexture in design.

CHAPTER III

EXPERIMENTAL DESIGN

To determine the correlation between pavement skid friction values and crash frequency, consistent with research tasks 1 through 3, it was necessary to collect data pertaining to the two variables. A series of subtasks were developed to acquire data and other information that provide guidance and support to the intention of this research. This chapter provides a detailed description of the experimental design.

Research Methodology

Six subtasks were developed to identify the levels of skid resistance of pavements and their relationship with crashes. Each subtask is described below.

Subtask 1: Literature Review
A literature review was conducted to document any published and unpublished information or results of studies from international, national, and regional research relevant to pavement skid resistance. Information that portrayed a relationship between skid resistance and crashes was primarily targeted. All elements of the literature review were presented in Chapter II.

Subtask 2: Agency 'Request for Information'
A request for information (survey) was developed to determine selected state's practices related to the current use of skid resistance in asphalt pavement design, construction, maintenance, and safety. FHWA Region Five States (Minnesota, Wisconsin, Illinois, Michigan, Indiana, and Ohio) were targeted along with other selected states which included Iowa, Missouri, Nebraska, Pennsylvania, and Washington. All survey questions were beta tested by the research team before submitting. A text-based survey was developed and sent as an attachment via electronic mail. Responses were received by electronic mail, regular mail, or fax.

The final version of the survey included a total of 26 questions. Questions were phrased in multiple-choice format to minimize the time it took agencies to complete the survey and to simplify the process of compiling the results. Additional space was provided in selected questions for the respondent to include specific supplementary information.

The survey was designed to evaluate the state's pavement management strategies to determine if skid resistance elements have been incorporated as a measure of effectiveness in asset management models. The consideration of skid resistance requirements in asphalt mix design specifications and the use of microtexture and macrotexture for the skid resistance design were particularly targeted. Agencies were asked if they measure asphalt mixture or aggregate properties related to microtexture and macrotexture in the lab and/or in the field to assure good skid-resistant pavement. The lab and field methods used to monitor and measure asphalt pavement texture was also requested, in addition to specific criteria and frequency of their use. The survey also queried agencies about how they correlate skid numbers to crash/safety data. There was particular interest in knowing if states have established friction threshold value requirements for decision making regarding maintenance activities. In addition it was requested whether or not agencies have developed models to estimate friction number (FN).

After the responses were received, questions were tabulated in a descriptive and detailed format. The descriptive section generalized the response rate to each question with the detailed section providing additional details on each response. When applicable, the specific answer of each state was shown. All elements of the survey are presented in Chapter IV. A copy of the survey is included in Appendix A with the detailed results in Appendix B.

Subtask 3: Selection of Test Sites

This subtask involved use of the Wisconsin Department of Transportation (WisDOT) friction database to locate appropriate sites for testing. WisDOT friction database has an inventory of friction number samples taken throughout the state since the late 1980's.

Test sites were necessary to obtain skid friction values and observe how surface conditions changed throughout time. The change of friction with time was compared with the number of crashes for the same period of time. Selected sites were hot mix asphalt (HMA) pavements to meet the objective of this research study. HMA sites selected were not resurfaced, seal coated, or post treated since the last WisDOT skid-friction evaluation. To minimize travel and cost of data collection, study sites in the Madison area were of primary interest. Nevertheless, study sites throughout the state were considered. Sites included two-lane undivided highways and four-lane divided highways with posted speeds of 55 and 65 mph, respectively.

WisDOT's pavement information files (PIF) were used to verify that sites complied with the requirements mentioned before. PIF files for WisDOT's eight transportation districts contain the following information: site location, pavement placing year, pavement type (HMA, PCC, HMA over PCC, etc), functional classification, and other related details. Pavement mix design was not included in the PIF. A site was eliminated from the study if it was resurfaced after the last recorded skid test was conducted.

More than 50 sites were found to meet the research criteria, although many of them had one or more data element voids. Six sites with good historical data were selected for this research. A description of the sites is shown in Table 21 and their map location is illustrated in Figure 23. Historical friction information for each site covered a period from the early to late 1990's. Multiple asphalt mixes are present within all test segments, and the approximate distribution of the mix was documented in WisDOT databases. The purpose for selecting sites with different subsection asphalt mixtures was to evaluate how variations of characteristics in their surface (namely macrotexture and microtexture) impact roadway safety.

Table 22 shows general pavement characteristics and AADT for the sections evaluated. Figures 24 through 29 include photos taken at sites.

Table 21 Selected Sites for Study in Wisconsin

Site	Highway	County	Location	Highway Type	Section Length (Miles)
1	IH 43	Walworth	USH 12 to STH 120	Four-Lane Divided	7.3
2	IH 43	Waukesha	STH 164 to CTH Y	Four-Lane Divided	3.2
3	IH 94	Monroe	CTH E to STH 21	Four-Lane Divided	5.2
4	IH 94	Waukesha	STH 67 to CTH BB	Four-Lane Divided	1.8
5	STH 21	Juneau	7^{th} Ave to 2^{nd} Ave	Two-Lane Undivided	5.3
6	USH 151	Grant/ Lafayette	STH 126 to East Side Rd	Two-Lane Undivided	5.7

Notes: IH: Interstate Highway, STH: State Trunk Highway, USH: United States Highway

Table 22 Pavement Characteristics

Site	Highway	Pavement Year	Pavement Type[1]	Number of Pavement Subsections (Mix Types)	AADT
1	IH 43	1993	3 - AC/RB	8	8,600 (NB)[2] 8,700 (SB)[2]
2	IH 43	1992	3 - AC/RB	9	20,500 (NB) 20,300 (SB)[3]
3	IH 94	1992	3 - AC/RB	6	11,000 (EB)[3]
4	IH 94	1991	3 - AC/RB	3	19,500 (WB)[3]
5	STH 21	1994	1 - AC/FB	7	4,600[4]
6	USH 151	1993	3 - AC/RB	7	7,800[2]

[1] TYPE 1 - AC/FB (Asphaltic Concrete pavement over Flexible Base); TYPE 3 - AC/RB (Asphaltic Concrete Pavement over Portland Cement Concrete)
[2] 2002 Data
[3] 2003 Data
[4] 2001 Data

Figure 23 Geographic Location of Sites in Wisconsin

Figure 24 Site 1 – IH 43 Walworth County, Wisconsin (South from Southbound Lanes)

Figure 25 Site 2 - IH 43 Waukesha County, Wisconsin (North from Northbound Lanes)

Figure 26 Site 3 - IH 94 Monroe County, Wisconsin (West from Eastbound Lanes)

Figure 27 Site 4 - IH 94 Waukesha County, Wisconsin (West from Westbound Lanes)

Figure 28 Site 5 – STH 21 Juneau County, Wisconsin (East from Westbound Lane)

Figure 29 Site 6 – USH 151 Grant & Lafayette Counties, Wisconsin (West from Westbound Lane)

Based on available FN data, Sites 1, 2, and 6 were selected for testing in both traffic directions. Site 3 only included the eastbound traffic direction; Sites 4 and 5 only included the westbound traffic direction. WisDOT pavement friction information pertained only to these directions.

For the sites selected, historical friction information was compiled. Friction data from the WisDOT database exists through the late 1990's, when WisDOT regularly monitored friction on their highway system. Currently, WisDOT no longer conducts a comprehensive pavement friction testing program but still test pavements on special request.

Subtask 4: Field Data Collection

In the summer of 2004, a series of skid tests were completed to complement historical friction information obtained from the friction database. The new skid tests were used to determine how skid friction has changed over time throughout the study sections. This task involved the use of the WisDOT skid trailer to collect existing friction numbers for the pavement sections under study.

Prior to the testing, a required Request for Pavement Evaluation (Friction Data Sheet, Appendix C) was sent to WisDOT containing all the information the skid trailer operator needed when testing. Included in the request form are site location, starting and finishing test points, speed, and braking intervals for skid tests. One form was necessary for every site tested. Forms were completed to match the original tests WisDOT performed on these sections for comparison consistency (same speed, type of tire, and same braking intervals when measuring friction). Original tests were performed by WisDOT conforming to current standards. Before testing had begun, field visits were conducted to verify that the pavement was in good condition for testing and no overlays had been applied within the study period.

All tests were performed during late summer of 2004, based on availability of the WisDOT skid trailer, as shown in Table 23. A KJ Law 1290 skid trailer model, as shown in Figure 30, was used for all tests. Testing conditions for the new friction tests are shown in Table 24. Braking intervals shown in the Table 24 were less than the 0.5 mile (1 km) ASTM criteria.

Table 23 Skid Friction Test Dates

Site	Highway	Friction Test Dates
1	IH 43	9/3/2004
2	IH 43	9/16/2004
3	IH 94	9/30/2004
4	IH 94	8/20/2004
5	STH 21	9/30/2004
6	USH 151	8/17/2004

Figure 30 WisDOT's KJ Law 1290 Skid Trailer

Table 24 Skid Friction Testing Conditions

Site	Posted Speed (mph)	Test Speed (mph)	Direction Tested	Braking Intervals(ft) @ 40 mph	Braking Intervals(ft) @ 50 mph
1	65	40/50	N/S	400/500	400/500
2	65	40/50	N/S	300	350
3	65	40/50	E	350	350
4	65	40/50	W	300	500
5	55	40/50	W	530	530
6	55	40/50	E/W	440	528

The standards followed in the field were the ASTM E-274 (Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire) and E-501 (Standard Specification for Standard Rib Tire for Pavement Skid-Resistance Tests) specifications (*84*). ASTM E-274 explains the characteristics required for the test apparatus and test procedures. The apparatus ASTM described consisted of a vehicle with test wheels incorporated into it or a trailer towed by a vehicle. KJ Law 1290 skid trailer complied with the latter description.

The skid test procedure described in ASTM E-274 Section 8 is fairly simple and is quoted as follows:

> *8.1 Bring the apparatus to the desired speed and deliver water to the pavement ahead of the test tire. Approximately 0.5 s after beginning of the water delivery, apply the test wheel brake so as to lock the wheel completely. The wheel shall remain locked for the duration of the data averaging interval.*
>
> *8.2 Water delivery may be terminated as soon as the brake is released.*

The water sprayed before locking up the tire is delivered through a water nozzle positioned at a height of two to four inches, as shown in Figure 31. The quantity of water applied at 40 mph is 4.0 gal ± 10%/min-in, and it is spread at least one inch wider that the test tire. Water is kept inside a water tank located in the main unit.

During the data averaging interval, the friction for a specific point was calculated. The skid number for a section was computed as the average of all the friction measurements along a section. Values are computed by the tractive force (horizontal force applied to the test tire) divided by the dynamic vertical load on the test wheel.

Standard test speed was 40 mph (65 km/h), but the vehicle could maintain speeds of 40 to 60 mph (65 to 100 km/h) within ± 1 mph (1.5 km/h) for testing. As shown in Table 24, speeds used in the present study were 40 and 50 mph using a ribbed tire (ASTM E-501). Note that a ribbed tire was used for consistency with previous testing operations. During testing, it was necessary to stop and allow some time for the tire to cool down so that the temperature did not affect the measurements. The tests were performed along the driving lane (right lane) with tire braking on the left wheel path. A computer inside the WisDOT skid trailer (Figure 32) monitored and recorded the friction measurements, speed at which the value was taken, temperature, water flow, time, and percent slip. All measurements taken within a margin of ± 1 mph of the test speed were kept; values outside this range were eliminated as established by ASTM.

All skid test results were provided in electronic format. As mentioned, all measurements were taken for wet conditions.

Figure 31 KJ Law 1290 Spraying Water Nozzle

Figure 32 KJ Law 1290 Recording Computer

Subtask 5: Crash Data Collection

Crash data for the present study were obtained through the WisDOT MV4000 motor vehicle crash databases and covered an 11-year period (1994 – 2004) for each site. Each analysis period began from the year the pavement was initially put into place at every section. Crash types included in the evaluation involved those that could be correlated to pavement friction; specifically, those that involved hard braking, skidding, or loss of vehicle control. Note that 2004 crash data only included crashes that occurred through the month of June.

WisDOT crash database contained information on how, when, and where a crash occurred. Information about how or why a crash occurred was not always as clear as desired, since there was not a narrative detailing exactly what happened at or leading up to the crash site. Information related to crash causes included crash type, driver's action at time of crash, information about object struck, if crash is with fixed object, number of vehicles involved, and severity of crash. Police narrative and related information were also taken into consideration, specifically the hypothesized cause of the crash (too fast for conditions, speeding, inattentive driving, mechanical problem, etc). Additional relevant information for this research, contained in the database, was the condition of the pavement at the moment of the crash resulting from the effects of weather.

Crashes of particular relevance were those where the contributing circumstances were described in the database as failure to keep vehicle under control (FVC) and too fast for conditions (TFC). In addition, the primary crash type that was analyzed were those categorized as NO C (no collision with another vehicle), which in many cases means that the vehicle went off the road. These types of crashes could suggest that a problem with the pavement is a factor of its occurrence. These criteria do not mean that other collisions and circumstances were not reviewed; all crashes were verified on a case by case basis.

Crashes included those that happened on wet, icy, snowy, and dry pavements. Nevertheless, other crashes categorized as rear end (REAR), angle (ANGL), and sideswipes in the same (SSS) or opposite direction (SSO) were selected. Rear type crashes where the highway patrol ruled out vehicle following to close (FTC) were considered friction related.

Crashes that were first eliminated from the original dataset were those described as deer related crashes. Other crashes in the dataset categorized as caused by inattentive driving, improper overtake, failure to yield, following to close, etc. were also eliminated. Those with an alcohol flag were removed since human factors likely contributed to the crash.

Subtask 6: Correlation between Skid Resistance and Road Safety

This final subtask focused on establishing the relationships between friction numbers and the safety of motorists. Crash data and FN data were combined to determine a statistical correlation between crash frequency and skid resistance values. Additional relationships were evaluated, including those between skid resistance and macrotexture.

CHAPTER IV

STATE PRACTICES – REQUEST FOR INFORMATION

A request for information on the current use of skid resistance in hot mix asphalt (HMA) pavement design, construction, maintenance, and safety was conducted. The targeted Departments of Transportation (DOTs) were the FHWA Region Five States (Illinois, Indiana, Michigan, Minnesota, Ohio, and Wisconsin) and five selected others (Iowa, Missouri, Nebraska, Pennsylvania, and Washington). Completed surveys were received from ten states. Minnesota data were not received in full. A copy of the "request for information" document is included in Appendix A. Detailed results are shown in Appendix B. The following is an executive summary of the results. Please refer to Appendix B for supporting data tables and specific comments received.

Michigan, Ohio, Wisconsin, Nebraska, and Washington reported that they did not directly consider skid resistance requirements in HMA design specifications, as shown in Table 25. Among the states that did consider skid resistance in their HMA design specifications, only Iowa and Missouri consider microtexture and macrotexture.

To maintain adequate microtexture, Illinois specifies aggregates for HMA mixes based upon average daily traffic volumes (ADT). The coarse aggregate specifications are described in the Supplemental Specifications for Section 1004 (Appendix D). Pennsylvania also established SRL (State Resistance Level) requirements based on ADT. Similar but more elaborated specifications are in use in Iowa for maintaining adequate microtexture and macrotexture. In addition to ADT, Iowa also incorporated speed to the friction criteria for HMA mix design. Iowa specifications 2303.02 (Appendix E) indicate the amount and quality of friction aggregate in the coarse fraction of the gradation blend (Appendix F). Missouri (Specifications 403.3.5, Appendix G) provided specifications for minimum amount of non-carbonated aggregates in mixtures containing limestone.

Table 25 Surveyed States' Requirements for Skid Resistance in HMA Design

States	Skid Resistance Not Considered	Skid Resistance Elements Considered		
		Microtexture	Microtexture and Macrotexture	Other Elements
FHWA Region Five	Michigan, Ohio[1], and Wisconsin	Illinois	None	Indiana[2]
Selected Others	Nebraska and Washington	Pennsylvania	Iowa and Missouri	None

[1]Ohio considers skid resistance on a case-by-case basis
[2]Indiana does not use microtexture or macrotexture to determine skid resistance

Testing Methods

As shown in Table 26, only four states responded affirmatively when asked if their agencies use laboratory testing methods for measuring asphalt mixture or aggregate properties related to microtexture and macrotexture. Michigan and Pennsylvania have laboratory procedures to measure only microtexture. Michigan measured the aggregate wear index (AWI) by wear track polishing tests to verify that the traffic-polished aggregates would not contribute to the wet road slipperiness (Standard MTM 111, Appendix H). Michigan also verified the polishing of aggregates from sample petrographic compositions (Standard MTM 112, Appendix I). Pennsylvania also performed petrographic examinations, and measured Polished Stone Value (PSV) using the British pendulum and the British wheel. Indiana and Iowa said they had laboratory procedures for measuring both microtexture and macrotexture. The Indiana procedure consisted of the British Pendulum (only for unspecified specialty projects) and Iowa did not have any specific laboratory procedure except for verifying specifications compliance for the HMA mix designs. It is interesting to note that Michigan and Pennsylvania categorized the British Pendulum as a means for measuring only microtexture, but for Indiana it was used to measure both microtexture and macrotexture.

All states surveyed use skid trailers (ASTM E 274, AASHTO T 242) as field testing methods to measure asphalt pavement microtexture and macrotexture. Ohio was the only state that also experimented with other equipment; namely a side force wheel mounted at a slight skew. The frequency for skid trailer use varies from state to state. Some states have regular friction measurement period intervals and others only measure when requested. Table 27 summarizes the locations and frequency of pavement surface testing performed in the field for all states surveyed.

Table 26 Laboratory Testing Methods for Measuring Asphalt Mixture or Aggregate Properties

State	Measurement	Test Used	Applicable Standards/ Reference
Michigan	Microtexture	Wear Track Polishing Tests Petrographic Compositions	Std. MTM 111 / Appendix H Std. MTM 112 / Appendix I
Pennsylvania	Microtexture	Petrographic Compositions Polished Stone Value (PSV) using the British Pendulum and the British Wheel	Std. MTM 112 / Appendix I
Indiana	Microtexture and Macrotexture	British Pendulum	
Iowa	Microtexture and Macrotexture	Unspecified	

Table 27 Pavement Surface Field Testing Frequency by State

State	Location	Testing Frequency
Indiana	Interstate system Remainder of system	Annually 3-year cycle
Michigan	Entire state-maintained network	3-year cycle
Nebraska	Entire state-maintained network New pavements	3-year cycle Once within 2-3 months of placement
Washington	Entire state-maintained network	2-year cycle
Iowa	Interstate system	2-year cycle
Illinois, Missouri, Ohio[1], Pennsylvania, and Wisconsin[2]	Select locations throughout state	By request or as needed

[1] Ohio has a regular testing cycle for research projects only.
[2] Wisconsin testing occurs on ten specified days per year and is usually limited to experimental pavements.

States' Use of Performance Measures

Six states reported that they use the friction numbers (FN) collected as a measure for making maintenance activity decisions. Iowa and Ohio did not specify the threshold values implemented. The remaining four states that mentioned specific criteria in terms of friction numbers pursued are Indiana, Michigan, Pennsylvania and Washington. Indiana indicated friction flag values of 20 and 35 (smooth tire, 40 mph) for inventory friction testing and warranty projects, respectively. Michigan takes action on pavement sections with FN of 30 and a high crash history. In Pennsylvania, corrective action is taken when all of the following occurs:

- Site is on the wet pavement crash cluster list or a known skid friction problem exists;
- One or more high friction needs exists within the cluster area (such as curves with low design speed, or short stopping sight distance for vertical curves); and
- Either the FN is less than 35 (ribbed tire), or less than 20 (smooth tire).

Washington specified that locations with skid numbers at or below 30 shall be retested, but solutions are implemented only for locations with skid numbers below 26. This FN threshold was based on published data showing a correlation between wet-weather crash rates and skid numbers less that 26. In locations with FN values below 30, Washington follows the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) for determining if such locations should be signed “Slippery When Wet.” Pennsylvania also employs MUTCD recommendations for low skid resistance.

Indiana has other requirements for pavement performance indicators that include the consideration of International Roughness Index (IRI) and Pavement Condition Rating (PCR). Wisconsin and Iowa also reported the development of models to estimate FN. Wisconsin has developed an equation that determines friction numbers as a function of: accumulated vehicle passes, percent dolomite in the coarse aggregate, age of pavement, LA Wear test results, and percent of heavy vehicles. The model was developed for HMA mixes and Portland Cement Concrete (PCC) as shown in Equation 12 (previously introduced as Equation 1 in Chapter II) and Equation 13.

HMA:

$$FN = 41.4 - 0.00075\,D^2 - 1.45\ln(LAVP) + 0.245\,LAWEAR \tag{13}$$

PCC:

$$\ln(FN) = 3.99 - 0.0419\ln(LAVP) - 0.00129D + 0.00474HV \tag{14}$$

where,

FN = friction number calculated at 40 mph
D = % dolomite in the mix
LAVP = Lane Accumulated Vehicle Passes
LAWEAR = Los Angeles Wear
HV = % heavy vehicles in the design lane

The model used in Iowa is simply a regression equation built through their Pavement Management System (PMS) database, and is not done on a regular basis except for system-wide analysis. The model was not provided.

Illinois, Iowa, Michigan, Missouri and Ohio also correlate FN to crash data. The process includes monitoring high crash frequency sites and measuring available pavement friction. Although Pennsylvania responded that no correlation between FN and crash data is done in the state, they made it clear that crash data and skid resistance data are both used to determine potential wet pavement crash cluster sites. Missouri was the only state that correlates wet and dry crashes to pavement friction.

State Agency Testing

Studies have been performed by several agencies regarding skid resistance. Only Illinois reported not to have conducted any studies related to skid resistance. Michigan specified that although they have not performed any recent studies, work was done in the agency in the 1960’s tracking different surface types. In Indiana, a study titled *Upgrading the INDOT Pavement Friction Testing Program* was conducted to upgrade the agency’s friction testing program with state-of-the-art technologies, evaluate network pavement friction performance, and examine the agency's state-of-the-practice in friction testing. The study also investigated the primary frictional variations such as system variations, seasonal variations, spatial variations, and temporal variations. Based on the results, the department established test frequencies, test tire, test speed, friction correction, and friction requirement for its network pavement inventory friction testing program.

Iowa was working through Iowa State University and the North Central Superpave Center to develop a laboratory procedure to measure HMA mixture surface friction characteristics. A list of other research projects sponsored by the agency is provided in Appendix J.

Ohio had finished a study on polishing and laboratory testing, and is starting a new study to look at a quicker approach, titled *Blending Proportions of High Skid and Low Skid Aggregate*. They had also conducted a limited study of the effects of a certain non-specified pavement treatment on skid resistance over an extended period of time. Additional studies were currently being conducted by the agency including correlating data between the standard ribbed and smooth tires, and evaluation of the long-term performance of Novachip and Smoothseal pavements. Studies in Pennsylvania established the SRL requirements mentioned before and also established the SRL Evaluation program.

Ohio reported that detailed skid resistance research has sharply declined as agency liability issues rose in the 1980's. Ohio's current research efforts are focused on maintaining a good pavement friction without specifying minimum levels. They are also considering reinstating full network level friction testing on an annual basis. The problem is that this would require a large increase in personnel and a rigorous policy regarding friction number values and any treatments or pavement decisions they would trigger.

CHAPTER V

CRASH ANALYSIS RESEARCH RESULTS

Pavement friction values and crash data for six study sites in Wisconsin were analyzed in the effort to evaluate the correlation between road safety and pavement skid resistance. As discussed in Chapter III, the research intent was to determine if crash frequency is inversely proportional to skid resistance, and if so, identify the pavement friction value threshold where the safety of the roadway system becomes critical. Establishing a minimum friction threshold would be considered an indicator in an asset management plan triggering the need for roadway maintenance. Results of the field investigation and crash analysis are presented in this chapter.

Pavement Data

In Chapter III, it was established that each study section was subdivided into an array of different pavement mixes. Having these Hot Mix Asphalt (HMA) mixes along the roadway sections allowed for the evaluation of a variety of pavement surface textures. Findings from WisDOT archives revealed that the sections studied in this research were previously part of a study comparing the performance of Stone Matrix Asphalts (SMA) and standard asphaltic concrete pavements during the last decade. Table 28 shows the description of the various mix types in place in one or more of the study sections. The pavement section codes presented in Table 28 are used throughout the remainder of this chapter.

Historical Friction Data

Skid resistance data were obtained annually for each study site from approximately 1991 through 1998 as part of WisDOT's previously mentioned study and earlier pavement friction testing programs. Obtaining this historical friction data were important for establishing temporal trends of skid resistance along the study sites. Table 29 presents the historical average friction data for the study sites. Results are reported following the appropriate skid friction nomenclature. Friction measurements taken at 40 and 50 miles per hour with a ribbed tire are denoted as FN40R and FN50R, respectively. Note that all historical tests were conducted with ribbed tires.

For undetermined reasons, WisDOT did not complete 50 mph tests in the first year of service life for pavements in sites 1, 3, 5, and 6. Nonetheless, there are no missing pavement friction data for 40 mph, which is the standard test speed for friction measurements.

Table 28 Pavement Sub Sections Mix Descriptions

Pavement Sub Section	Description
CT (Control)	Dense Graded Asphalt Mix
WI (Control)	Wisconsin's A1 Mix
F1	SMA w/ Cellulose Fiber Stabilizer
F2	SMA w/Mineral Fiber Stabilizer
P1	SMA w/Polymer (Thermoplastic) Stabilizer (Lo %)
P2	SMA w/Polymer (Thermoplastic) Stabilizer (Hi %)
E1	SMA w/Polymer (Elastomeric) Stabilizer (Lo %)
E2	SMA w/Polymer (Elastomeric) Stabilizer (Hi %)
SH	Strategic Highway Research Project (SHRP) Mix
SX	SHRP Mix Gradation w/Wisconsin 85-100
SP	SMA w/Polymer (EVA)
SS	SMA Surface w/Vestoplast over Wisconsin A1 Binder
SF	SMA Full Depth w/Vestoplast
SM	SMA over SMA
SA (Control)	SMA over A Mix
SD	SMA w/Polymer (Vestoplast) over A Mix
HV	HV Asphalt w/ MAC 10

Table 29 Historical Friction Data

Site		1991	1992	1993	1994	1995	1996	1997	1998
1: IH 43 Walworth City									
Northbound	FN40R	-	-	42.7	47.1	46.2	40.1	42.9	45.3
	FN50R	-	-	-	44.9	42.2	36.9	40.8	41.6
Southbound	FN40R	-	-	46.3	47.8	45.9	42.9	45	47.6
	FN50R	-	-	-	44.6	42.9	38.7	42	42.7
2: IH 43 Waukesha City									
Northbound	FN40R	-	42.7	45.2	42.3	40.2	43.4	42	-
	FN50R	-	39.2	41.4	39.5	37.8	36.2	36.1	-
Southbound	FN40R	-	45.9	44.6	40.5	41.1	40.7	40.8	-
	FN50R	-	41.5	42.2	36.8	38.1	38.7	38	-
3: IH 94 Monroe City									
Eastbound	FN40R	-	-	49.1	48.8	46.8	41.7	46.2	44.1
	FN50R	-	-	-	46.3	42.9	38.5	42.5	41.7
4: IH 94 Waukesha City									
Westbound	FN40R	54.3	43.7	-	43.7	39.1	-	41.6	41.1
	FN50R	46.1	40.7	-	40.7	35.2	-	38.1	37.4
5: STH 21 Juneau County									
Westbound	FN40R	-	-	-	50.1	50.1	45	49.8	54.6
	FN50R	-	-	-	45.6	47.3	41.8	45.5	49
6: USH 151 Grant/Lafayette Counties									
Northbound	FN40R	-	-	41.3	47.4	43.9	40.8	-	42.2
	FN50R	-	-	-	42.9	40.9	36.7	-	37.6
Southbound	FN40R	-	-	40.3	47.5	44.1	42.3	-	42.6
	FN50R	-	-	-	41.9	40.7	38.8	-	40.7

Field Friction Data

Obtaining a robust pavement skid friction database for the study sites was critical in developing the basis for which crash data could be compared. Therefore, new surface skid tests were performed at every site to complement the skid friction database information obtained from WisDOT.

Table 30 shows the average friction numbers (FN) measured with the WisDOT skid trailer at every study site section for the year 2004. Friction data for the study sites between approximately 1998 and 2004 were not available from WisDOT. Recall from Chapter III that sites 1, 2, and 6 were tested in both directions, site 3 was only tested in the eastbound direction, and site 4 and 5 were only tested in the westbound direction. Hence, FNs for all traffic directions are not provided in Table 31.

Both the FN and the speed gradient are considered in evaluating pavements. Speed gradient indicates the rate at which skid friction decreases with increasing speed. Table 31 shows the speed gradient for the 2004 average friction data and the dispersion parameters at study sites.

Figures 33 through 41 show the temporal trends of average pavement friction data. The average includes historical and 2004 data. Each highway direction of a study site is composed of multiple subsections of different mix designs. Therefore, it must be kept in mind that values presented here are averages of the friction values obtained for different sub-section mix designs.

Table 30 2004 Average Friction Values at Study Sites

Study Site	Roadway	Location (County)	Date	Northbound		Southbound	
				FN40R	FN50R	FN40R	FN50R
1	IH 43	Walworth	9/3/2004	35.6	35.2	39.1	37.3
2	IH 43	Waukesha	9/16/2004	35.7	33.1	37.3	35.4
3	IH 94	Monroe	9/30/2004	-	-	40.5[1]	37.7[1]
4	IH 94	Waukesha	8/20/2004	34.5[2]	32.0[2]	-	-
5	STH 21	Juneau	9/30/2004	47.3[2]	44.0[2]	-	-
6	USH 151	Grant/Lafayette	8/17/2004	37.4	34.7	36.1	33.6

[1] Eastbound direction

[2] Westbound direction

Table 31 Speed Gradients and Dispersion Parameters at Study Sites

Site	Highway Direction	Average FN		Speed Gradient	Lock-ups	Min	Max	Range	SD	Var.
1	NB	FN40R	35.6	0.04	50	29.1	44.4	15.3	4.0	16.3
		FN50R	35.2		48	29.9	42.9	13.0	3.4	11.6
	SB	FN40R	39.1	0.18	38	30.9	45.5	14.6	3.8	14.1
		FN50R	37.3		37	32.2	43.5	11.3	3.7	13.4
2	NB	FN40R	35.7	0.26	49	32.0	44.4	12.4	3.3	10.7
		FN50R	33.1		45	28.9	43.8	14.9	3.4	11.3
	SB	FN40R	37.3	0.19	52	32.1	49.4	17.3	4.3	18.3
		FN50R	35.4		46	31.1	46.3	15.2	3.7	13.9
3	EB	FN40R	40.5	0.28	76	35.5	47.7	12.2	3.6	12.9
		FN50R	37.7		76	33.0	43.7	10.7	3.2	10.2
4	WB	FN40R	34.5	0.25	31	27.9	41.9	14.0	4.6	21.4
		FN50R	32.0		18	26.3	38.4	12.1	4.2	17.8
5	WB	FN40R	47.3	0.33	52	42.3	54.3	12.0	3.0	8.8
		FN50R	44.0		53	40.2	49.3	9.1	2.3	5.1
6	NB	FN40R	37.4	0.27	61	32.0	43.5	11.5	2.4	5.5
		FN50R	34.7		51	25.3	42.0	16.7	2.4	5.9
	SB	FN40R	36.1	0.25	81	30.7	42.9	12.2	2.4	5.7
		FN50R	33.6		45	29.0	36.9	7.9	2.1	4.4

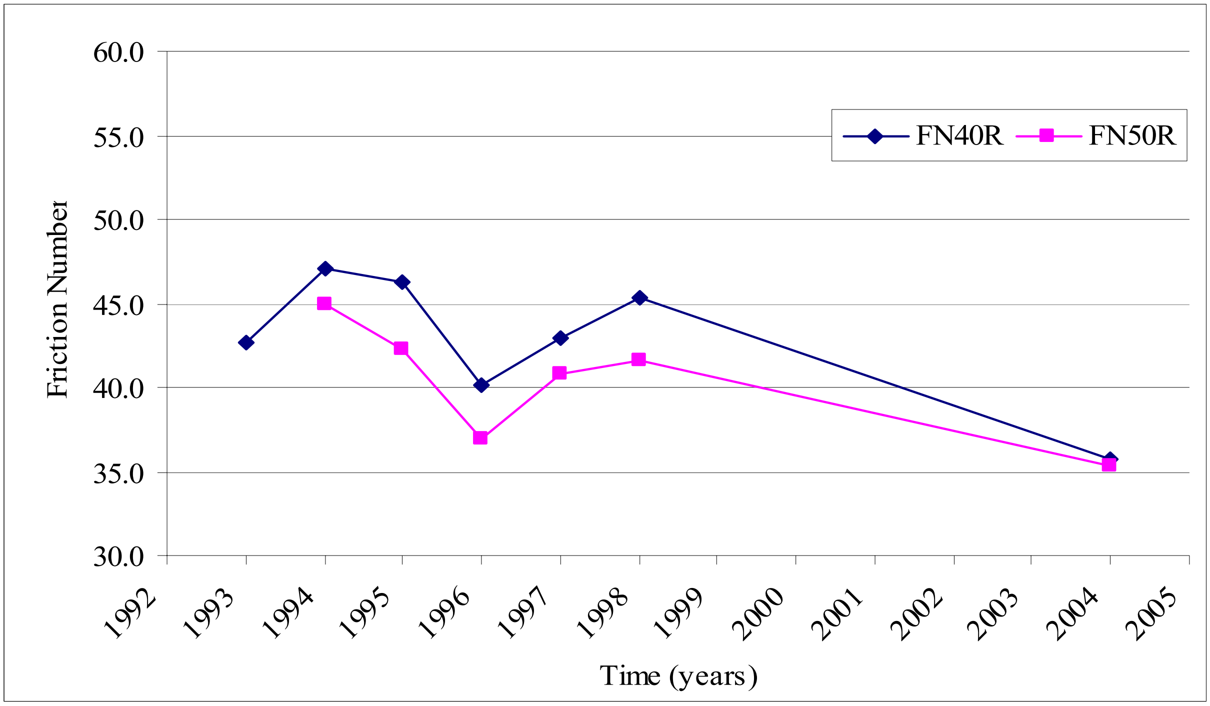


Figure 33 Site 1 (IH 43 Walworth) NB Historical Friction Trends

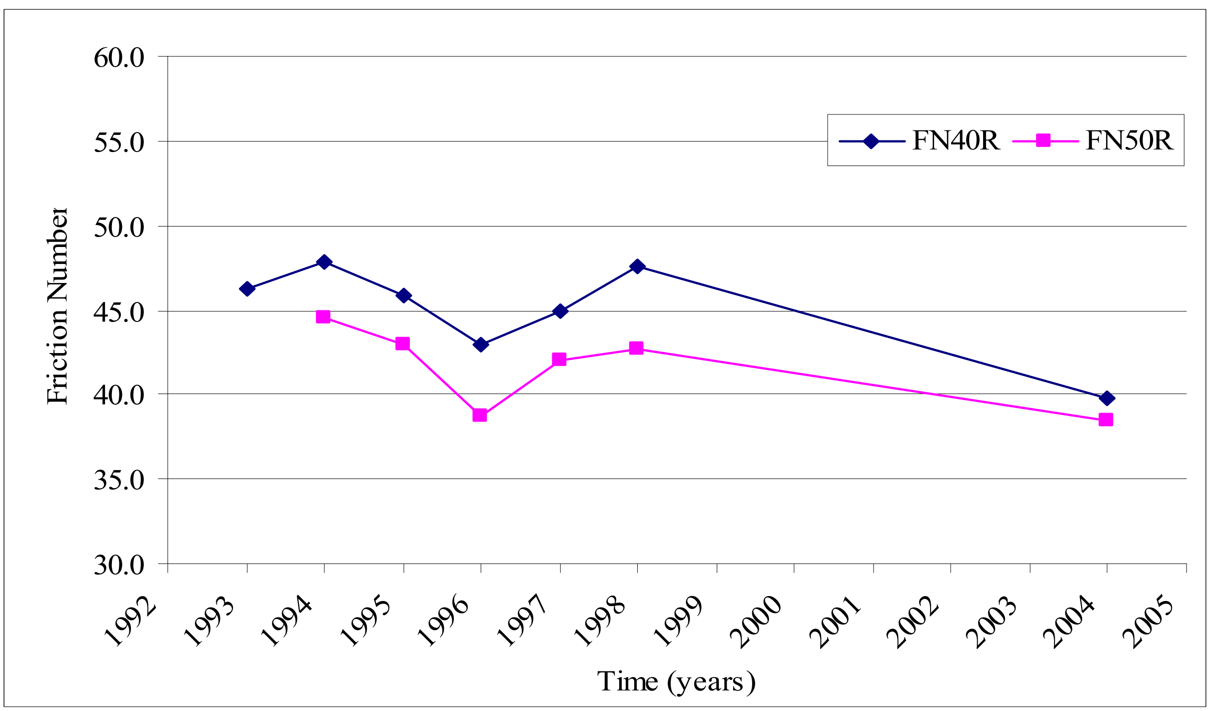


Figure 34 Site 1 (IH 43 Walworth) SB Historical Friction Trends

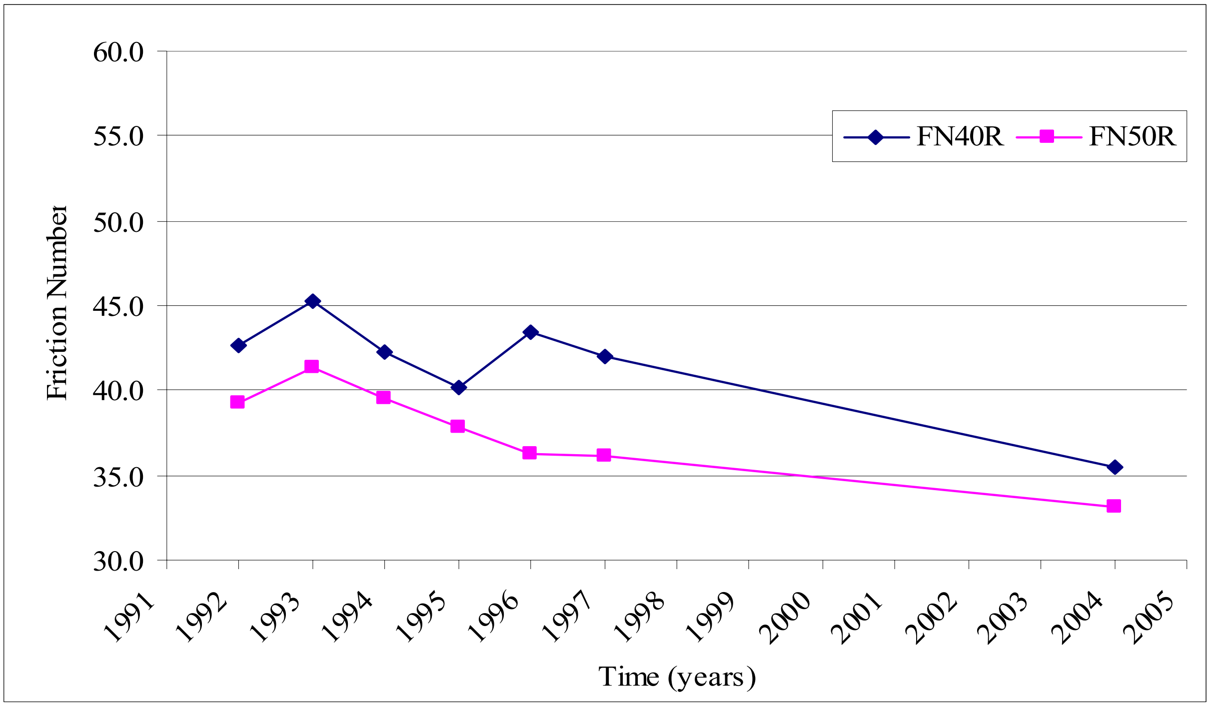


Figure 35 Site 2 (IH 43 Waukesha) NB Historical Friction Trends

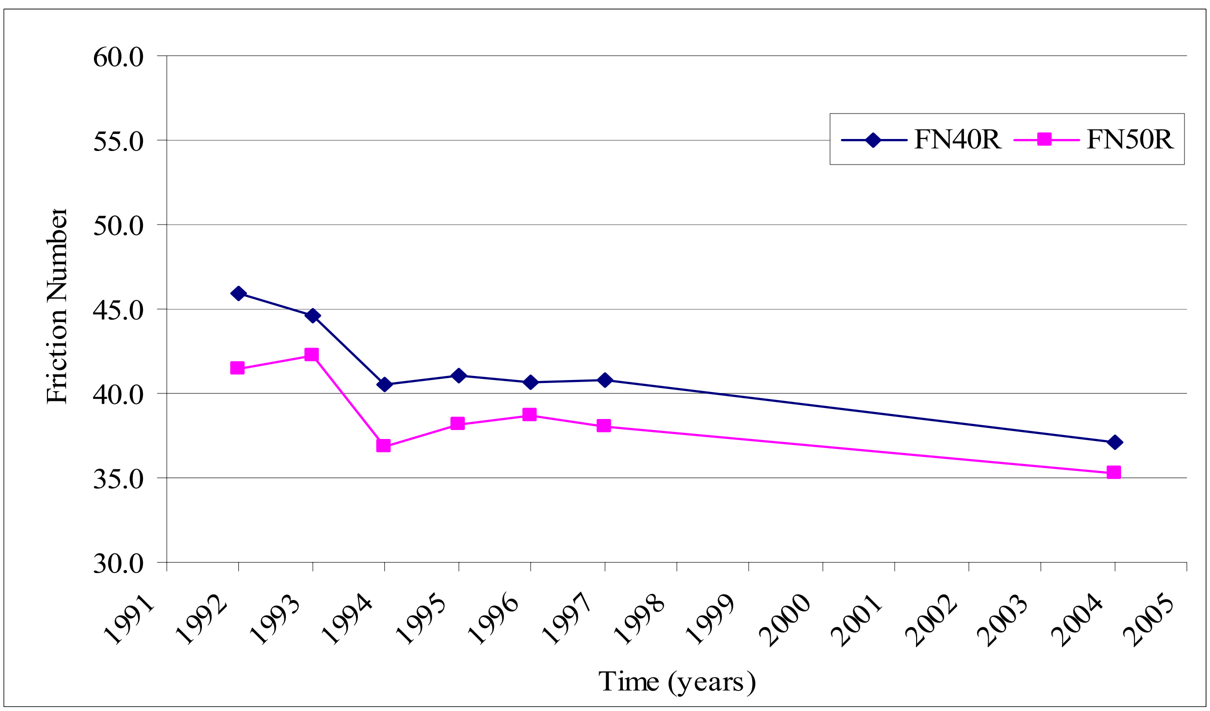


Figure 36 Site 2 (IH 43 Waukesha) SB Historical Friction Trends

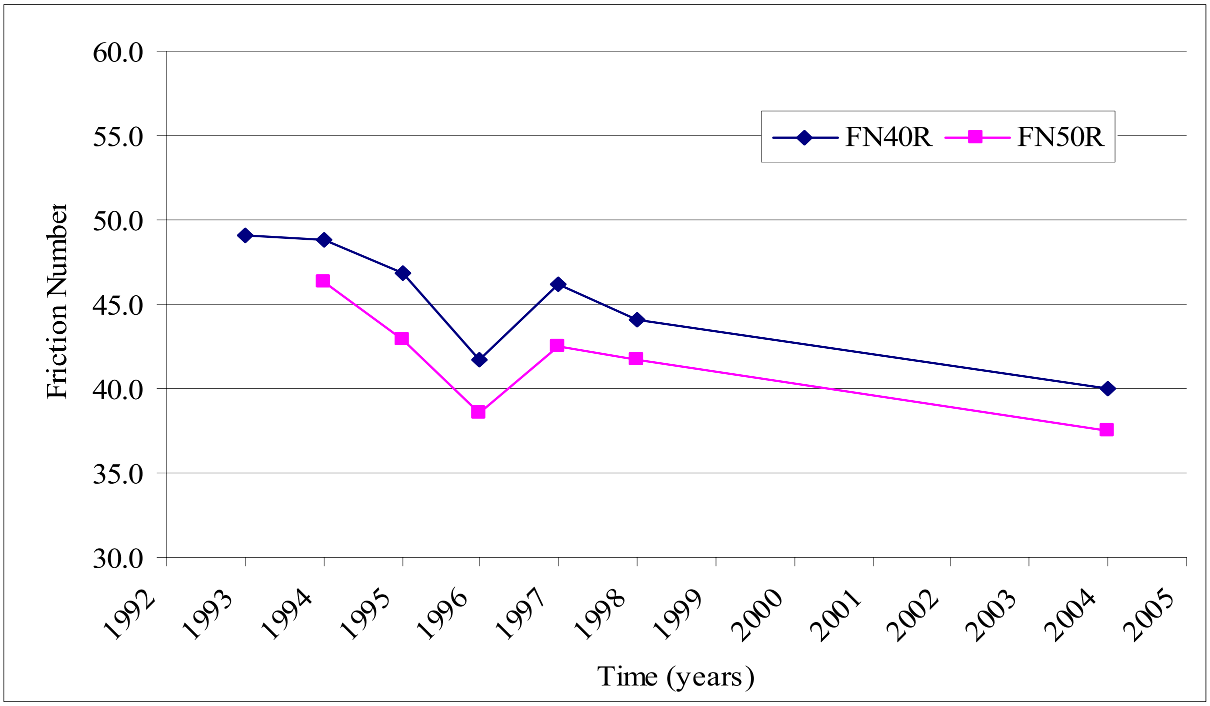


Figure 37 Site 3 (IH 94 Monroe) EB Historical Friction Trends

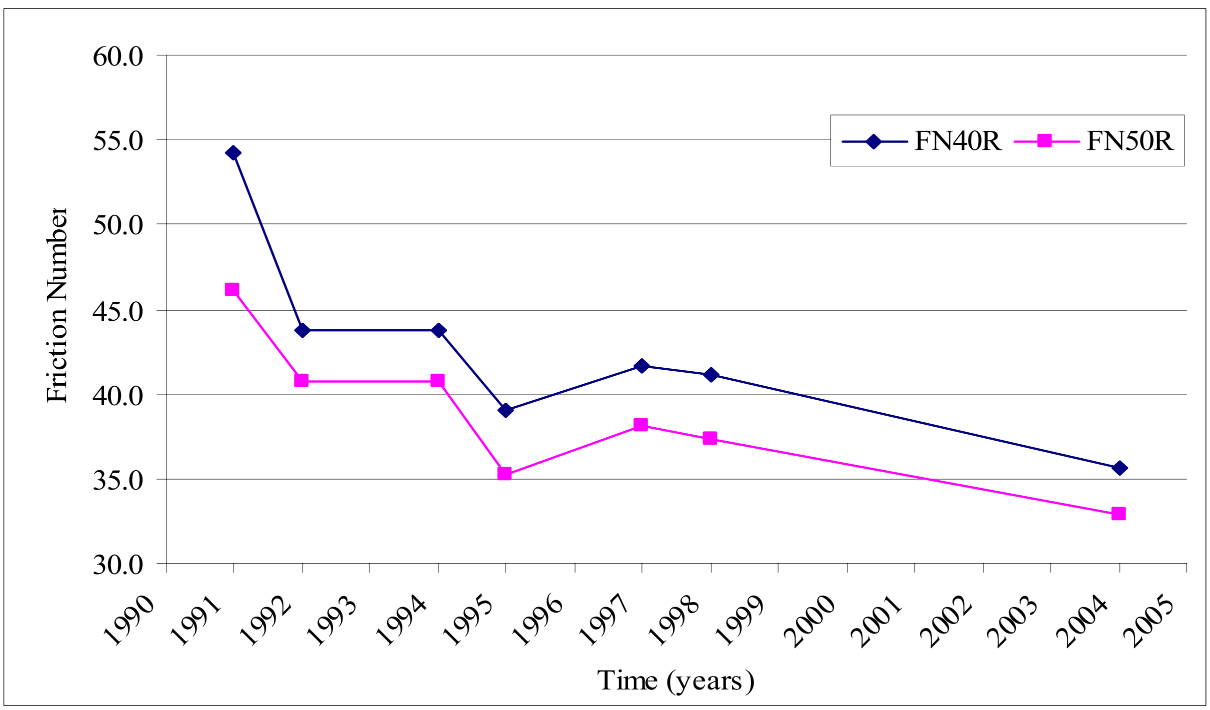


Figure 38 Site 4 (IH 94 Waukesha) WB Historical Friction Trends

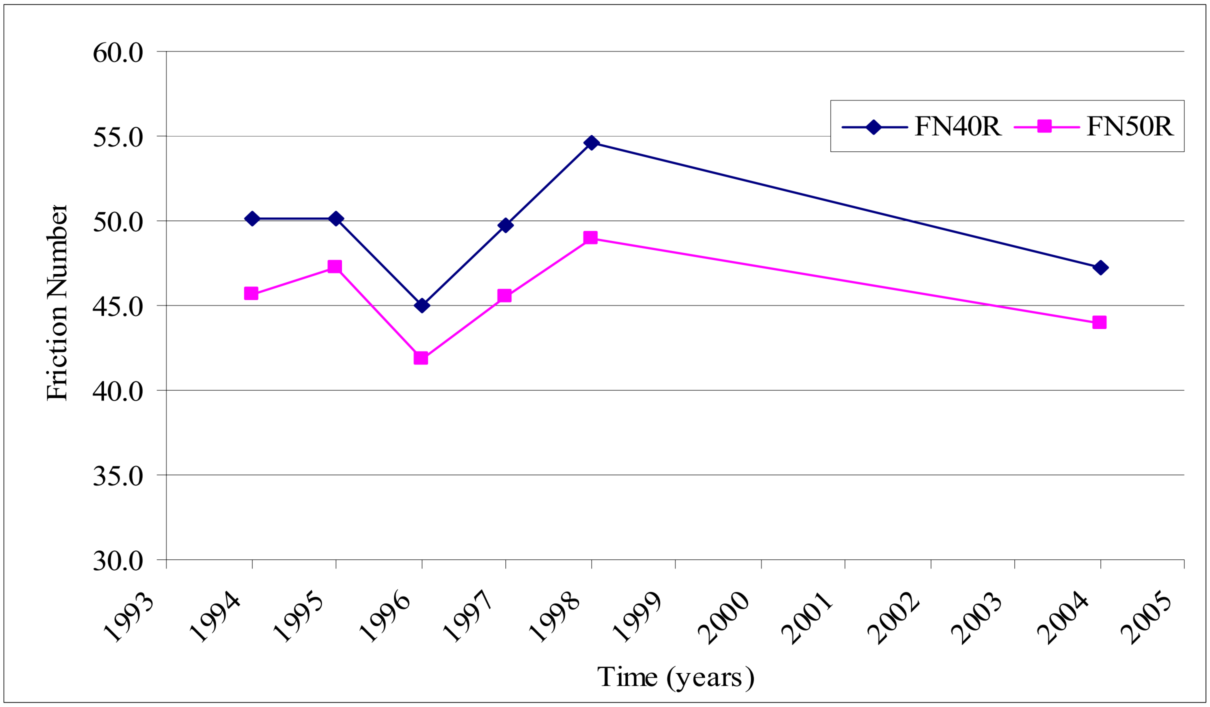


Figure 39 Site 5 (STH 21 Juneau) WB Historical Friction Trends

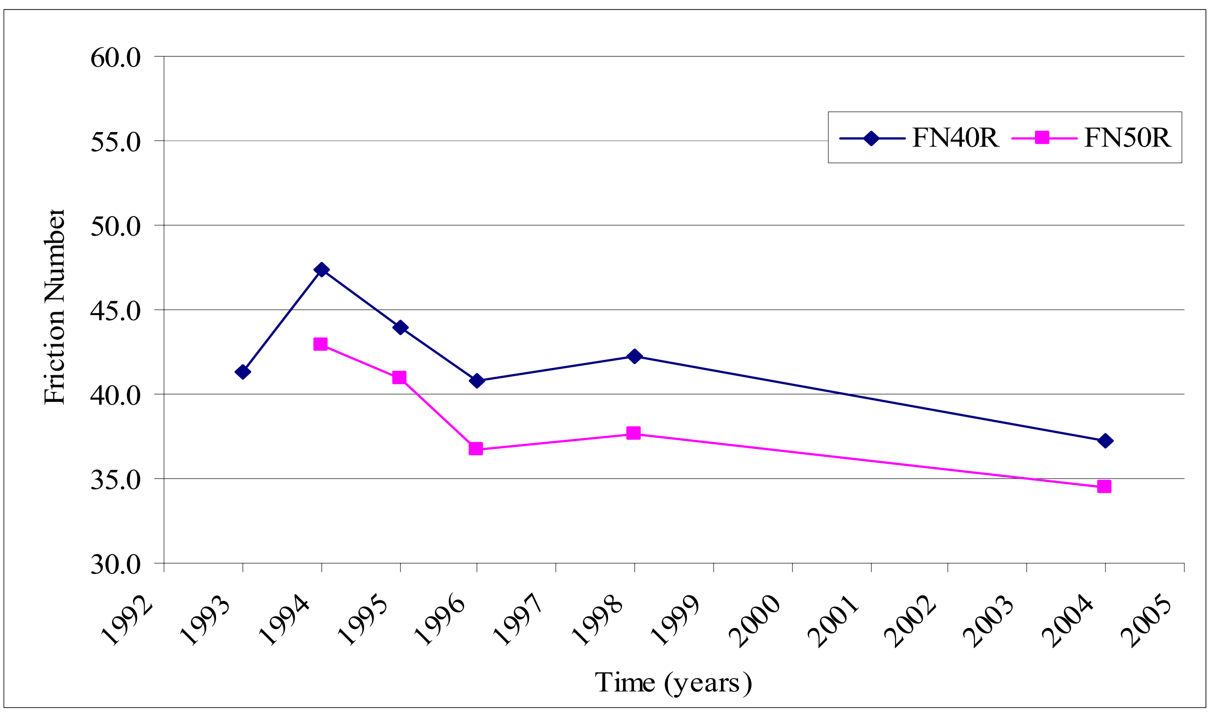


Figure 40 Site 6 (USH 151 Grant/Lafayette) NB Historical Friction Trends

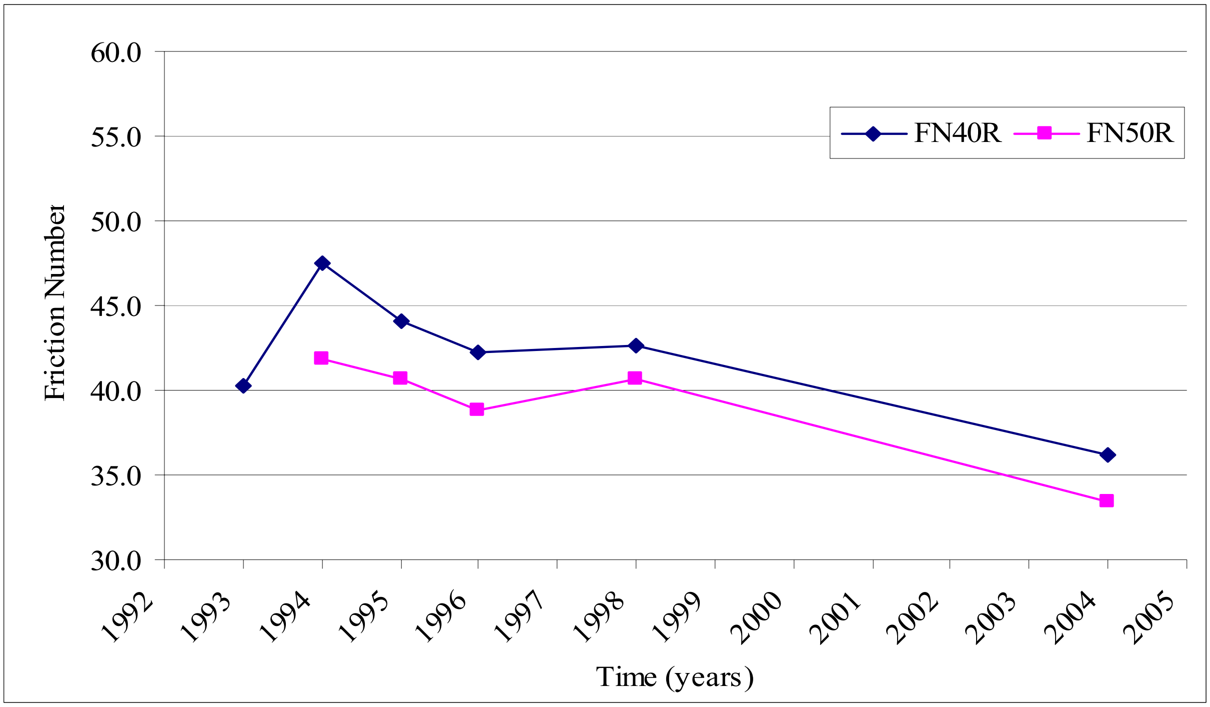


Figure 41 Site 6 (USH 151 Grant/Lafayette) SB Historical Friction Trends

Figures 33 through 41 show that the average friction values have decreased since 1998, as expected. Since each one of the previously shown sections was subdivided according to different pavement mix designs, uniform FNs for each pavement mix design are necessary. Therefore, to further explore the pavement friction values, the friction results were subdivided uniformly by corresponding pavement mix section.

Historical friction data for each subsection of the selected study sites are shown in Tables 32 through 46. Graphical depictions of the historical data are illustrated in Figures 42 through 59. Refer to Table 22 for the subsection mix design description and to Appendix K for the speed gradients and dispersion parameters.

Table 32 Site 1 NB Pavement Subsection Friction Numbers

I-43 Walworth County										
Year	Northbound Subsection Mix Design									
	CT		F1		F2		P1		P2	
	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	48.2	-	41.7	-	37.7	-	43.0	-	42.7	-
1994	48.8	45.9	42.9	41.3	46.6	44.6	48.1	46.3	48.9	46.2
1995	49.1	43.5	44.2	40.2	45.6	41.4	46.7	43.1	45.5	42.7
1996	45.8	41.0	36.2	33.7	39.0	35.4	39.1	37.0	40.3	37.5
1997	49.1	45.1	39.5	37.6	41.5	39.6	41.6	40.6	42.7	41.0
1998	52.5	47.7	42.3	38.5	43.6	39.8	43.9	40.4	44.4	41.7
2004	42.9	41.4	32.3	32.0	35.4	33.9	33.7	34.4	34.7	34.9

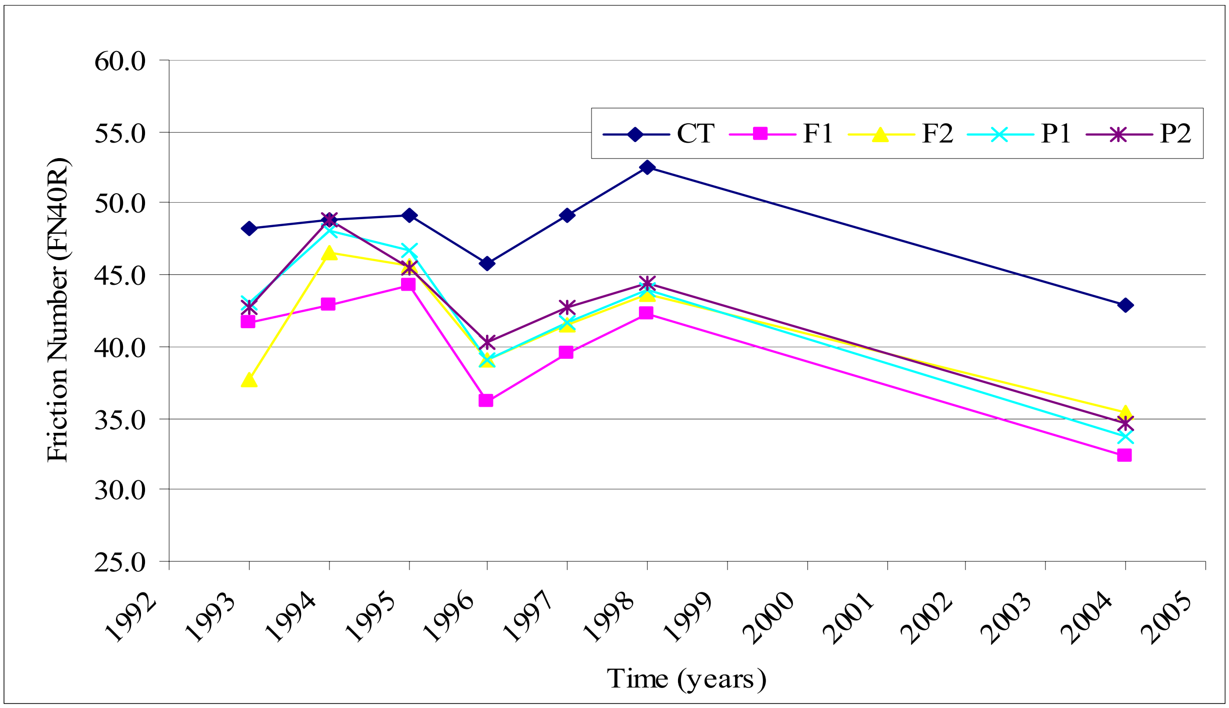


Figure 42 Site 1 NB Subsection (IH 43 Walworth) Historical FN40R Trends

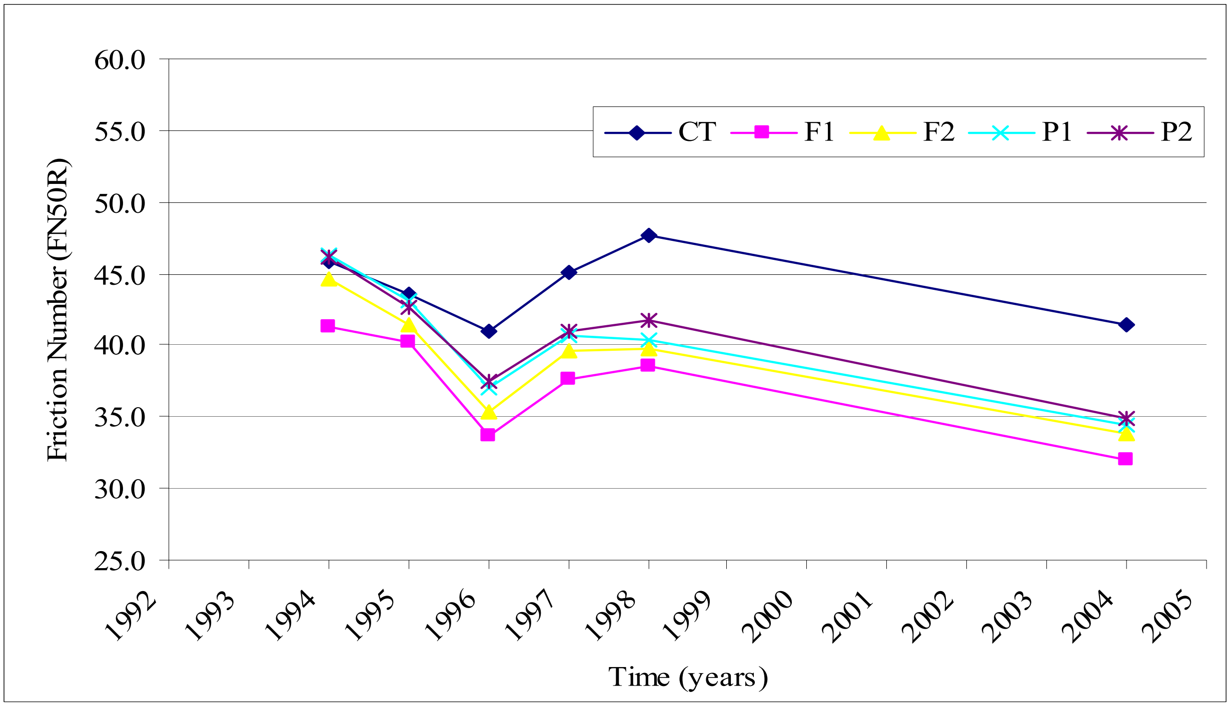


Figure 43 Site 1 NB Subsection (IH 43 Walworth) Historical FN50R Trends

Table 33 Site 1 SB Pavement Subsection Friction Numbers

I 43 Walworth County								
	Southbound Subsection Mix Design							
	CT		E1		E2		HV	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	51.6	-	41.4	-	39.2	-	53.0	-
1994	50.2	46.6	45.9	43.4	45.4	42.5	49.8	45.7
1995	48.2	44.3	44.2	41.3	43.5	41.5	47.6	44.6
1996	46.7	42.0	39.4	35.8	39.7	35.5	45.9	41.4
1997	49.6	45.3	41.6	39.3	41.2	39.5	47.7	43.7
1998	51.6	46.3	44.4	39.4	44.0	39.3	50.4	45.7
2004	43.0	41.1	36.1	34.7	37.0	35.3	43.0	42.4

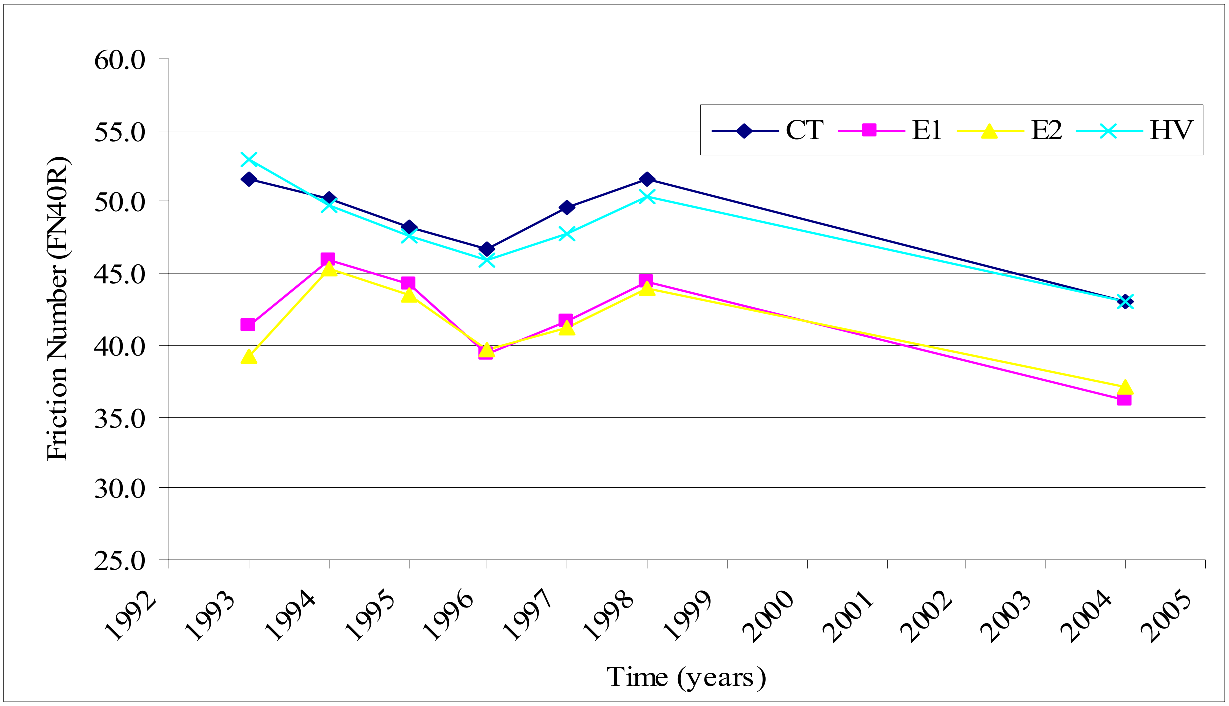


Figure 44 Site 1 SB Subsection (IH 43 Walworth) Historical FN40R Trends

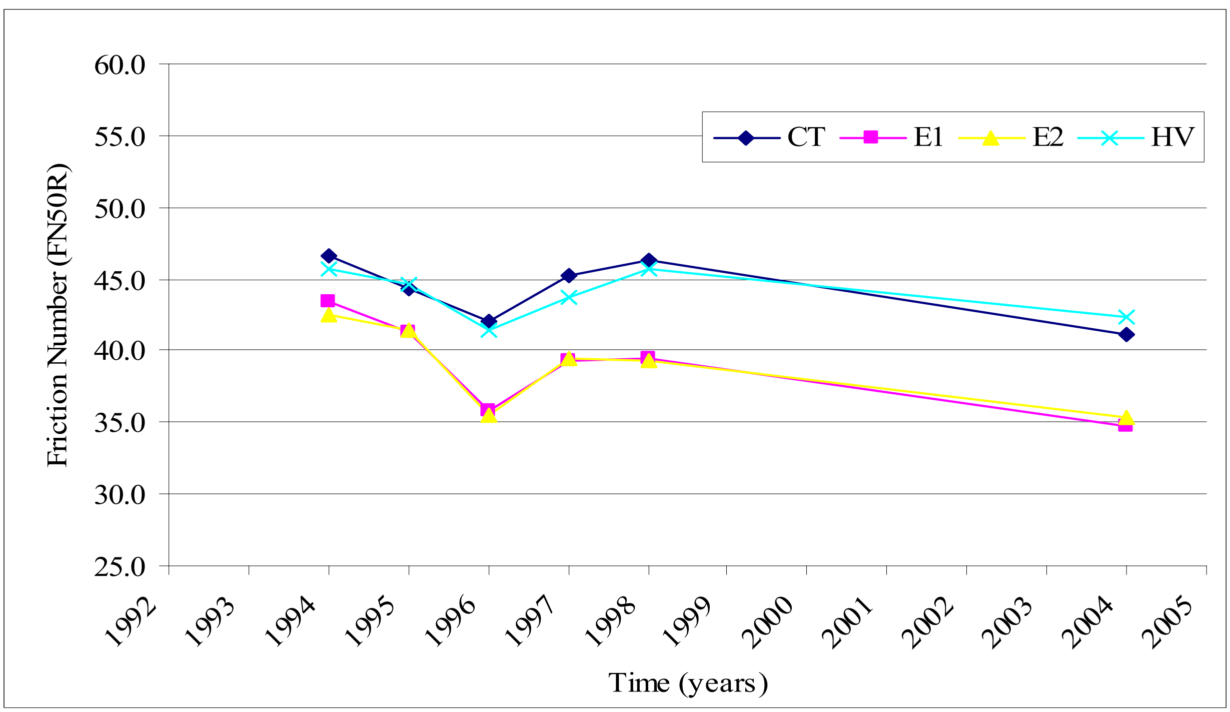


Figure 45 Site 1 SB Subsection (IH 43 Walworth) Historical FN50R Trends

Table 34 Site 2 NB Pavement Subsection Friction Numbers

I 43 Waukesha County						
	Northbound Subsection Mix Design					
	CT		F1		F2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1992	51.5	46.9	36.1	33.9	32.7	31.1
1993	47.6	45.7	43.4	39.6	43.7	40.0
1994	46.6	42.4	39.9	38.1	42.1	39.8
1995	43.2	39.9	38.4	36.8	40.1	38.2
1996	49.9	40.2	40.8	34.5	41.3	35.6
1997	48.5	40.9	39.0	33.9	40.1	35.2
2004	40.9	37.9	34.3	31.4	33.3	32.5

Table 35 Site 2 NB Pavement Subsection Friction Number (Continued)

I 43 Waukesha County						
	Northbound Subsection Mix Design					
	P1		SH		SX	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1992	37.2	35.3	49.6	44.3	49.0	43.6
1993	44.1	40.4	44.3	40.3	48.2	42.6
1994	41.1	39.3	41.1	37.2	43.1	40.4
1995	39.6	37.5	38.1	35.3	42.0	38.9
1996	41.2	34.9	42.7	35.2	44.5	36.6
1997	40.1	35.0	40.8	34.3	43.3	37.4
2004	34.1	32.3	33.8	31.0	36.6	33.3

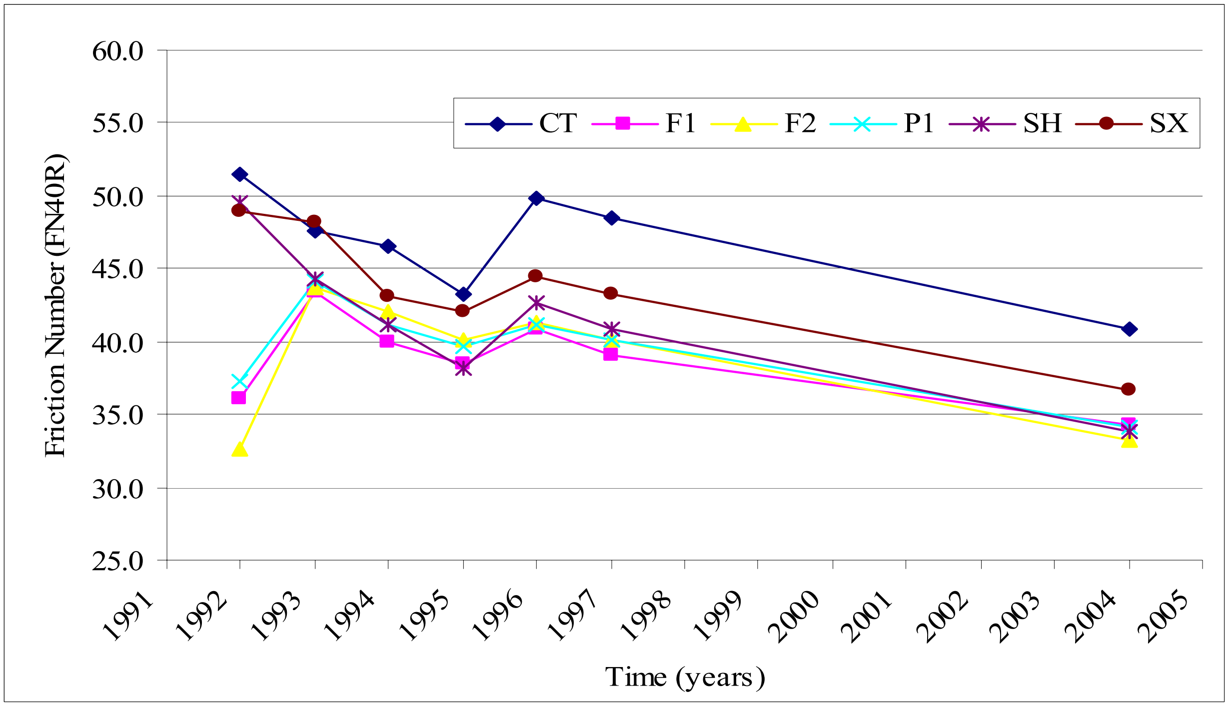


Figure 46 Site 2 NB Subsection (IH 43 Waukesha) Historical FN40R Trends

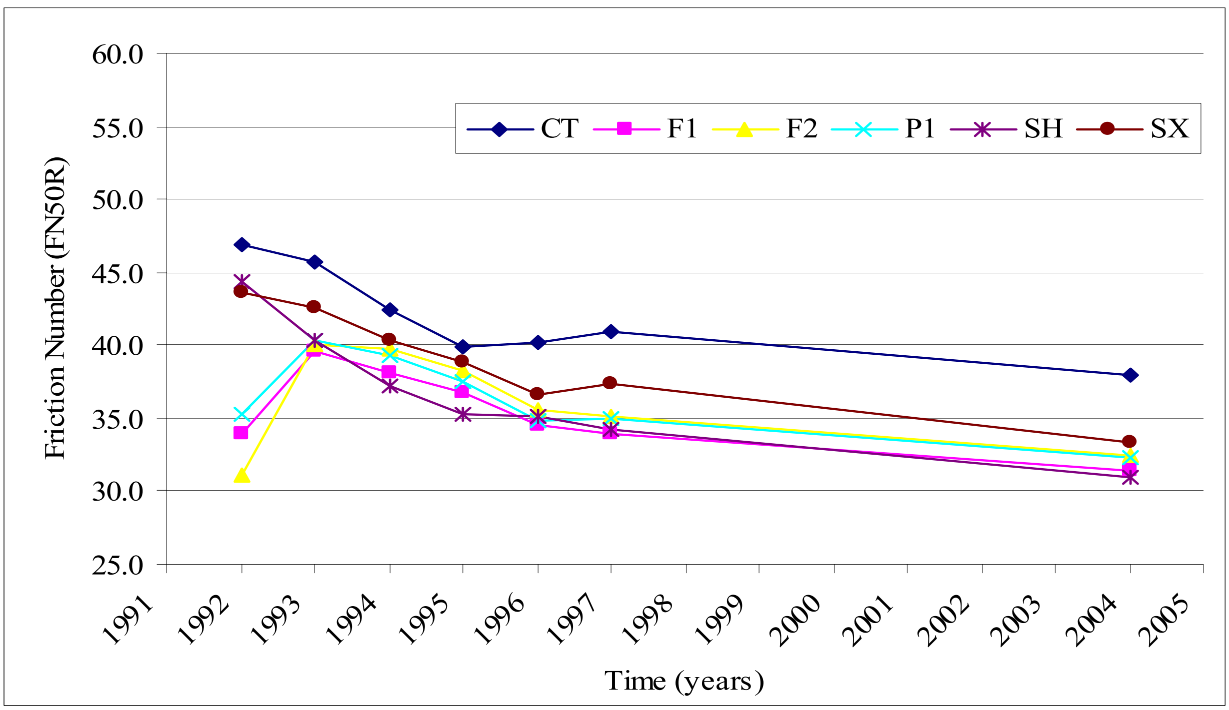


Figure 47 Site 2 NB Subsection (IH 43 Waukesha) Historical FN50R Trends

Table 36 Site 2 SB Pavement Subsection Friction Numbers

I 43 Waukesha County, Wisconsin						
	Southbound Subsection Mix Design					
	CT		E1		E2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1992	51.0	47.4	40.2	35.3	39.0	35.2
1993	49.3	45.3	42.4	41.0	43.4	41.2
1994	45.8	40.2	38.6	36.1	39.2	36.5
1995	44.8	41.1	39.5	37.2	40.0	37.6
1996	47.5	44.4	38.8	36.9	38.2	37.1
1997	47.6	44.4	38.1	35.8	38.7	36.4
2004	45.2	42.0	35.4	33.6	35.1	33.6

Table 37 Site 2 SB Pavement Subsection Friction Number (Continued)

I 43 Waukesha County						
	Southbound Subsection Mix Design					
	P2		SH		SX	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1992	45.3	40.3	47.8	43.3	51.9	47.2
1993	42.1	40.9	45.9	42.0	44.5	43.0
1994	38.3	35.8	40.7	35.5	40.2	36.9
1995	40.2	37.6	41.4	36.5	40.9	38.3
1996	38.5	37.2	40.8	38.1	40.3	38.3
1997	37.8	36.4	41.3	38.0	41.1	37.1
2004	35.0	34.0	36.3	34.3	35.3	33.8

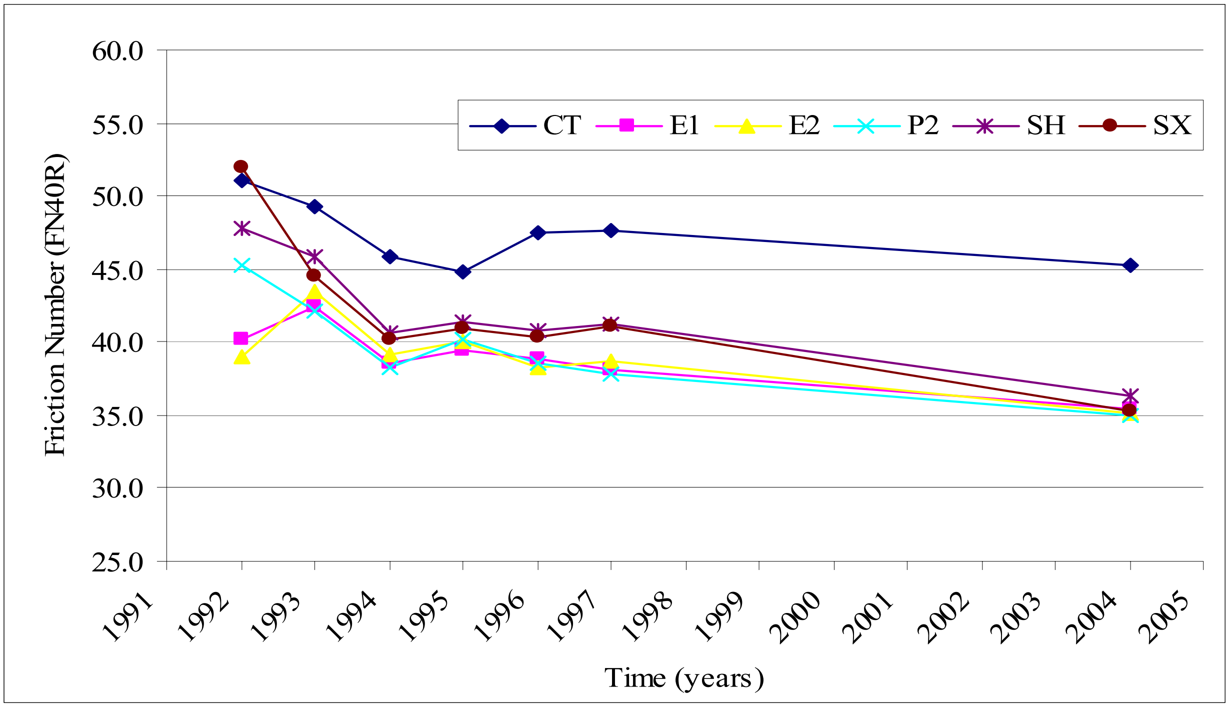


Figure 48 Site 2 SB Subsection (IH 43 Waukesha) Historical FN40R Trends

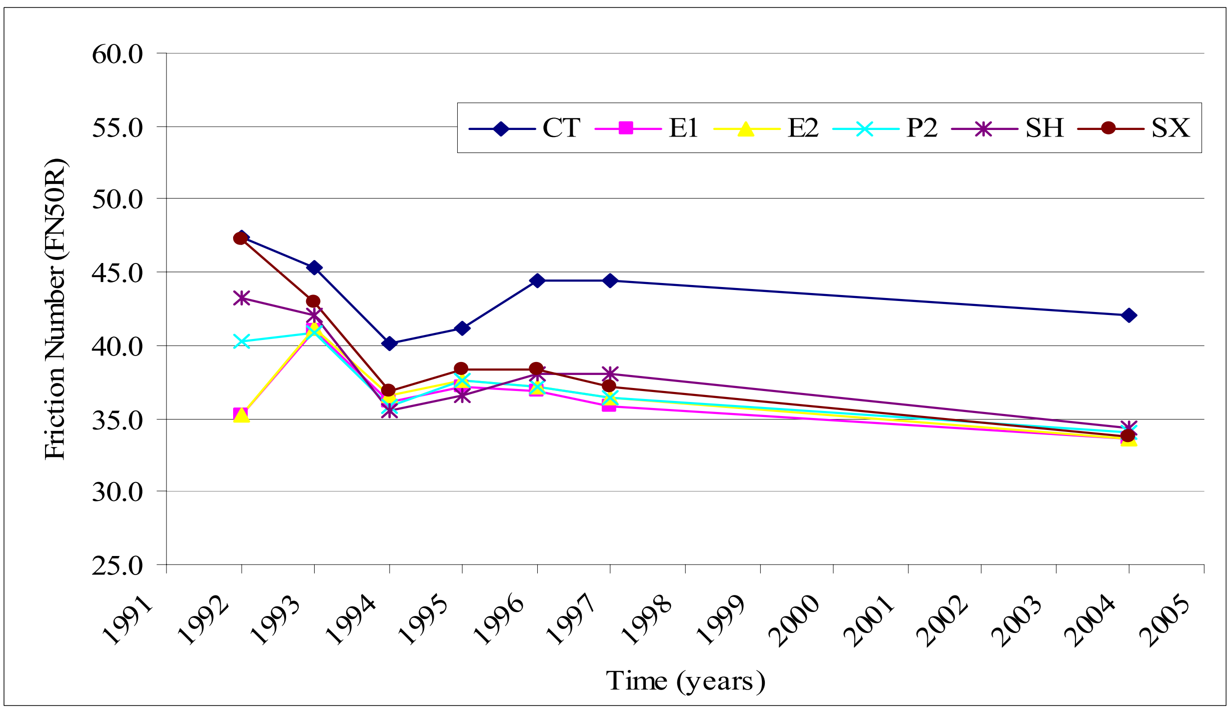


Figure 49 Site 2 SB Subsection (IH 43 Waukesha) Historical FN50R Trends

Table 38 Site 3 EB Pavement Subsection Friction Numbers

I 43 Monroe County						
	Eastbound Subsection Mix Design					
	WI		SS		SF	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	50.4	-	48.5	-	48.5	-
1994	51.8	46.7	47.5	45.4	48.5	45.4
1995	48.5	42.9	45.0	42.2	45.4	42.7
1996	46.5	40.9	39.4	37.3	40.6	37.6
1997	48.3	43.5	45.3	42.0	45.5	41.8
1998	48.2	44.6	43.2	40.3	42.4	40.7
2004	45.4	41.7	37.4	35.0	37.3	34.5

Table 39 Site 3 EB Pavement Subsection Friction Number (Continued)

I 43 Monroe County						
	Eastbound Subsection Mix Design					
	SP		SH		SX	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	48.4	-	50.2	-	48.8	-
1994	47.9	46.1	48.7	47.2	48.4	46.7
1995	45.8	42.3	52.6	46.7	43.4	40.8
1996	40.3	38.8	43.1	39.5	40.1	37.0
1997	46.3	42.5	46.4	43.5	45.1	41.4
1998	43.7	41.8	44.1	42.2	42.8	40.7
2004	38.3	35.8	41.5	39.1	40.2	38.8

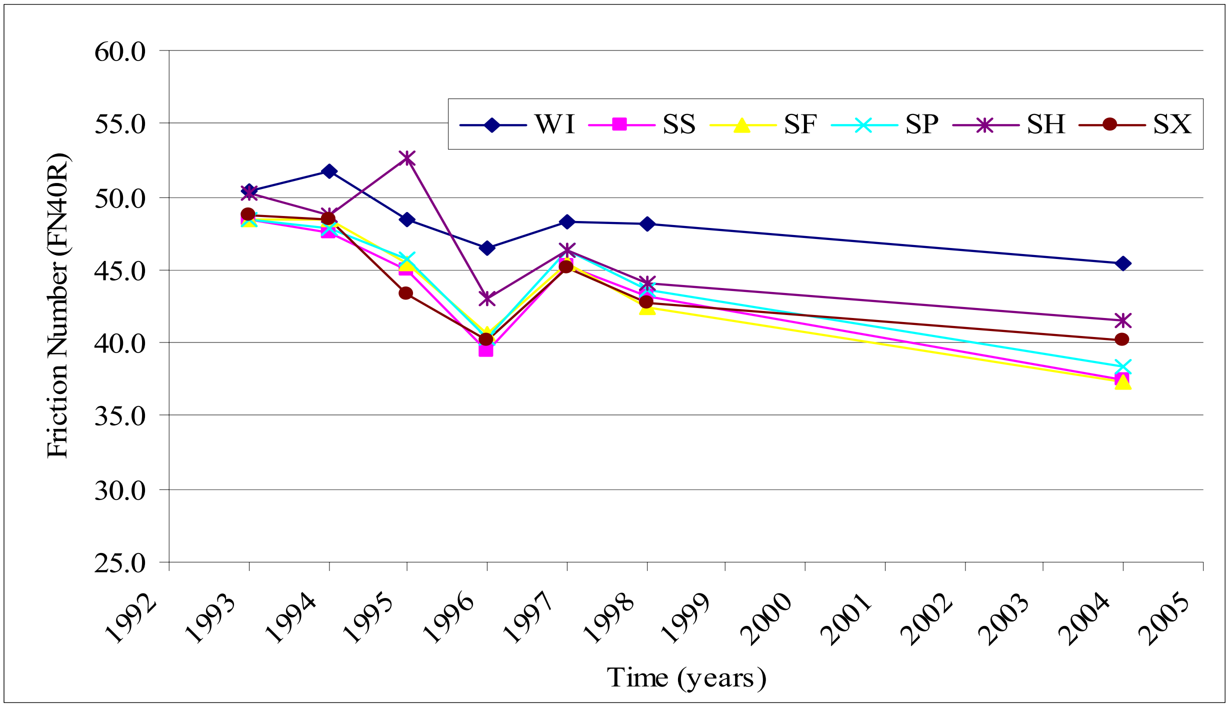


Figure 50 Site 3 EB Subsection (IH 94 Monroe) Historical FN40R Trends

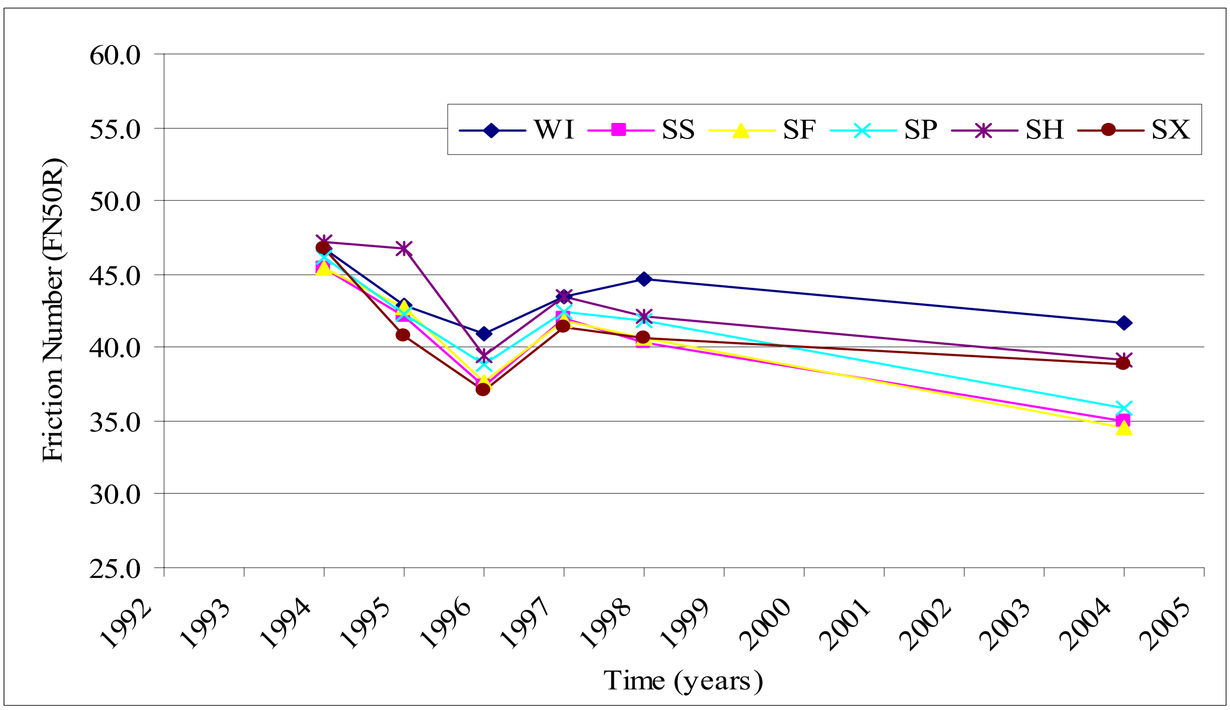


Figure 51 Site 3 EB Subsection (IH 94 Monroe) Historical FN50R Trends

Table 40 Site 4 WB Pavement Subsection Friction Numbers

I 94 Waukesha County						
	Westbound Subsection Mix Design					
	SM		SD		SA	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1991	46.3	39.2	58.6	46.6	58.1	52.5
1992	39.9	36.3	44.7	42.7	46.6	43.0
1994	37.9	35.6	45.8	43.3	47.3	43.1
1995	32.7	30.0	41.0	37.3	43.6	38.4
1997	34.5	31.5	44.7	41.6	45.5	41.3
1998	32.3	29.8	44.6	40.2	46.5	42.3
2004	29.9	27.8	37.7	35.0	39.1	36.0

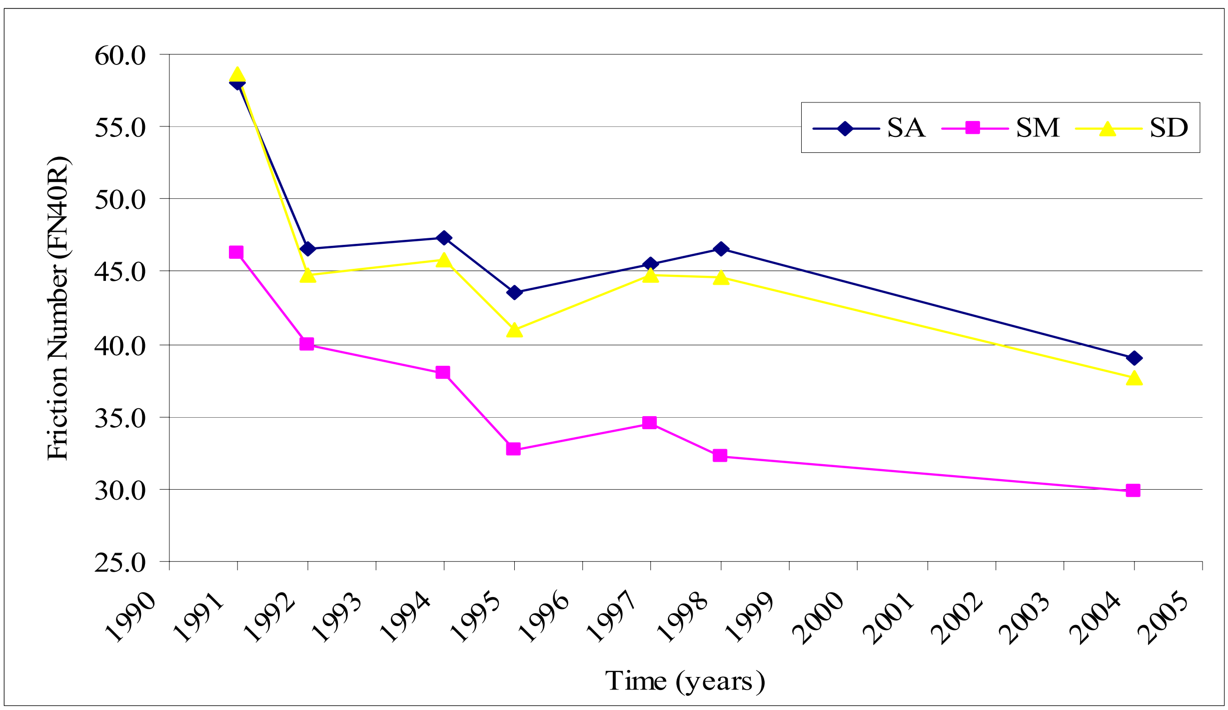


Figure 52 Site 4 WB Subsection (IH 94 Waukesha) Historical FN40R Trends

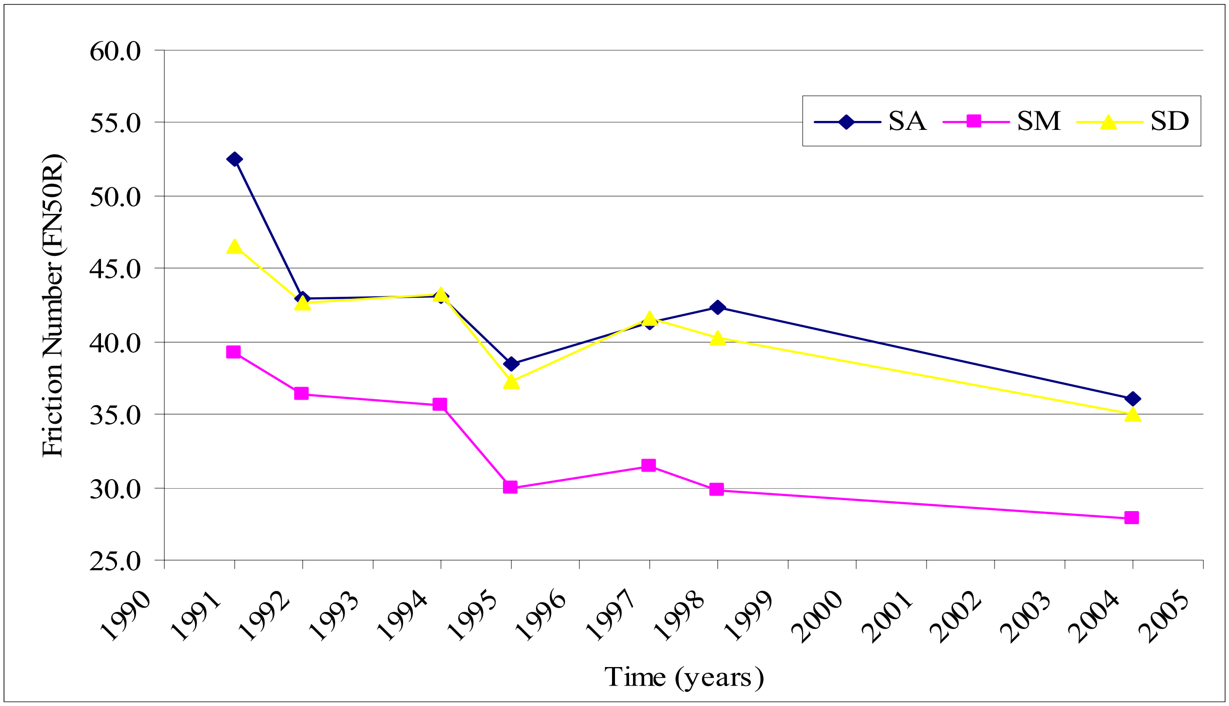


Figure 53 Site 4 WB Subsection (IH 94 Waukesha) Historical FN50R Trends

Table 41 Site 5 WB Pavement Subsections Friction Number

STH 21 Juneau County						
	Westbound Subsection Mix Design					
	CT		E1		E2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1994	47.7	43.5	52.5	47.9	51.1	46.3
1995	51.2	47.5	50.5	48.3	50.0	47.5
1996	45.0	43.7	44.8	42.0	45.0	42.1
1997	53.5	48.2	50.9	47.2	49.2	45.0
1998	58.7	53.1	53.8	50.0	53.0	49.1
2004	52.7	47.9	46.7	44.6	45.8	42.5

Table 42 Site 5 WB Pavement Subsection Friction Number (Continued)

STH 21 Juneau County								
	Westbound Subsection Mix Design							
	E1		E2		P1		P2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1994	50.6	47.0	49.6	44.0	49.3	45.4	49.8	44.9
1995	48.8	46.5	50.3	46.9	49.8	47.2	50.0	47.1
1996	44.8	38.6	45.4	42.6	46.1	42.1	43.6	41.2
1997	47.2	43.0	50.0	44.8	48.6	44.9	49.5	45.4
1998	55.1	47.5	55.8	48.5	52.3	47.3	53.3	47.8
2004	46.3	42.9	47.4	43.8	46.0	43.4	45.9	42.9

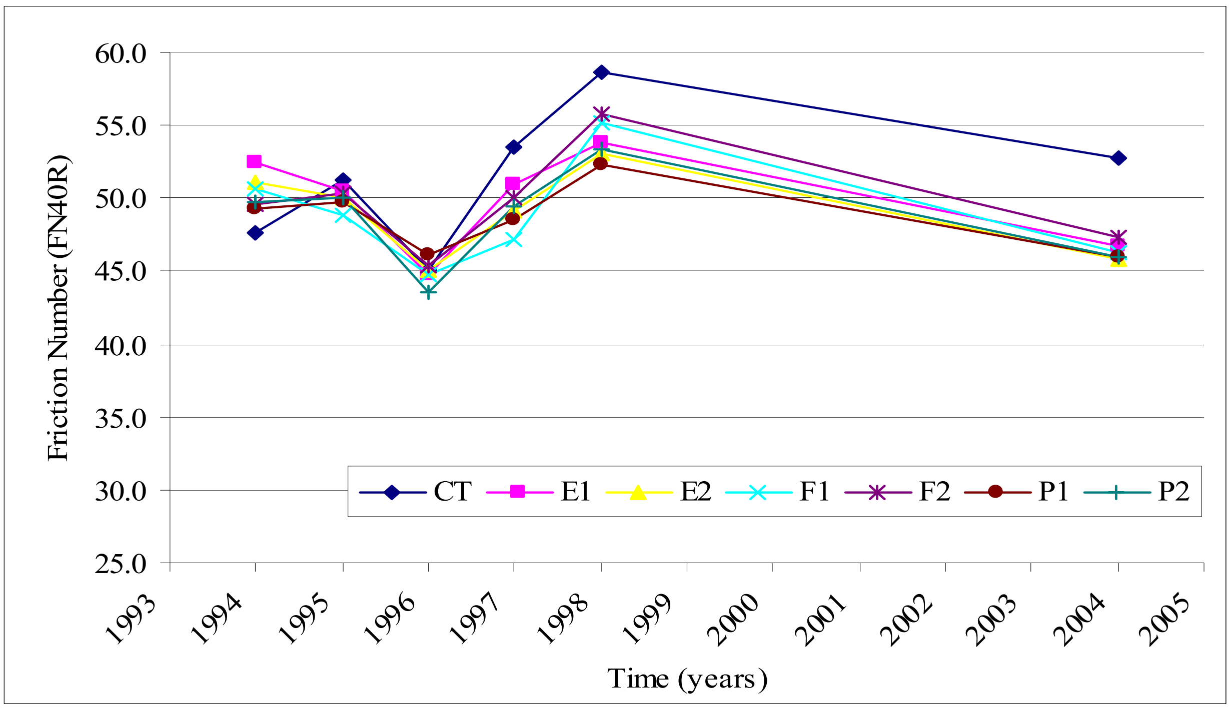


Figure 54 Site 5 WB Subsection (STH 21 Juneau) Historical FN40R Trends

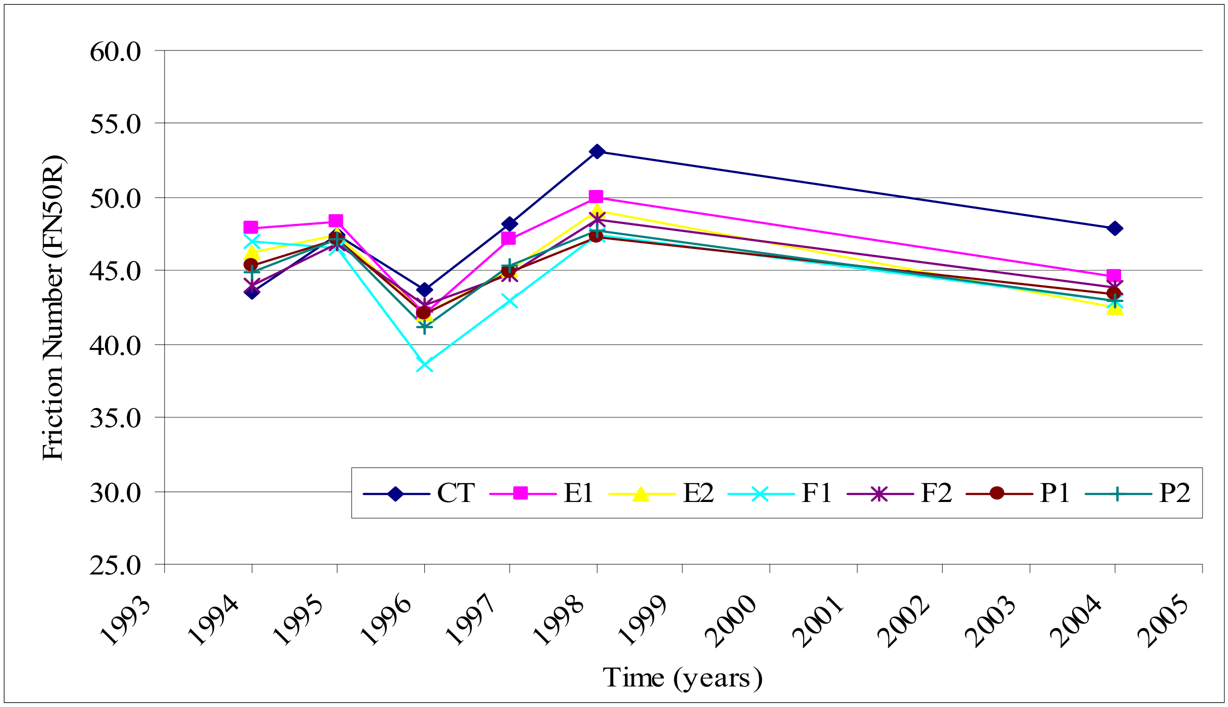


Figure 55 Site 5 WB Subsection (STH 21 Juneau) Historical FN50R Trends

Table 43 Site 6 NB Pavement Subsection Friction Numbers

USH 151 Grant/Lafayette Counties						
	Northbound Subsection Mix Design					
	CT		**E1**		**E2**	
Year	**FN40R**	**FN50R**	**FN40R**	**FN50R**	**FN40R**	**FN50R**
1993	45.8	-	37.2	-	40.0	-
1994	45.7	39.7	49.0	44.7	47.0	43.3
1995	41.0	36.6	44.7	41.3	45.1	43.3
1996	39.8	32.8	41.6	38.2	41.1	37.6
1998	39.9	35.5	45.4	39.4	42.9	37.4
2004	37.0	32.0	39.5	36.5	37.1	34.7

Table 44 Site 6 NB Pavement Subsection Friction Number (Continued)

USH 151 Grant/Lafayette Counties								
	Northbound Subsection Mix Design							
	F1		F2		P1		P2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	43.2	-	38.8	-	40.6	-	43.2	-
1994	45.4	40.5	45.9	41.7	49.2	44.4	49.7	45.8
1995	42.7	39.3	42.3	39.8	45.5	43.1	45.9	43.2
1996	38.3	34.8	39.8	35.6	41.6	38.0	43.4	40.1
1998	39.4	35.8	39.6	36.0	42.2	38.3	46.2	40.6
2004	35.2	33.6	36.7	34.3	36.6	34.7	39.0	35.9

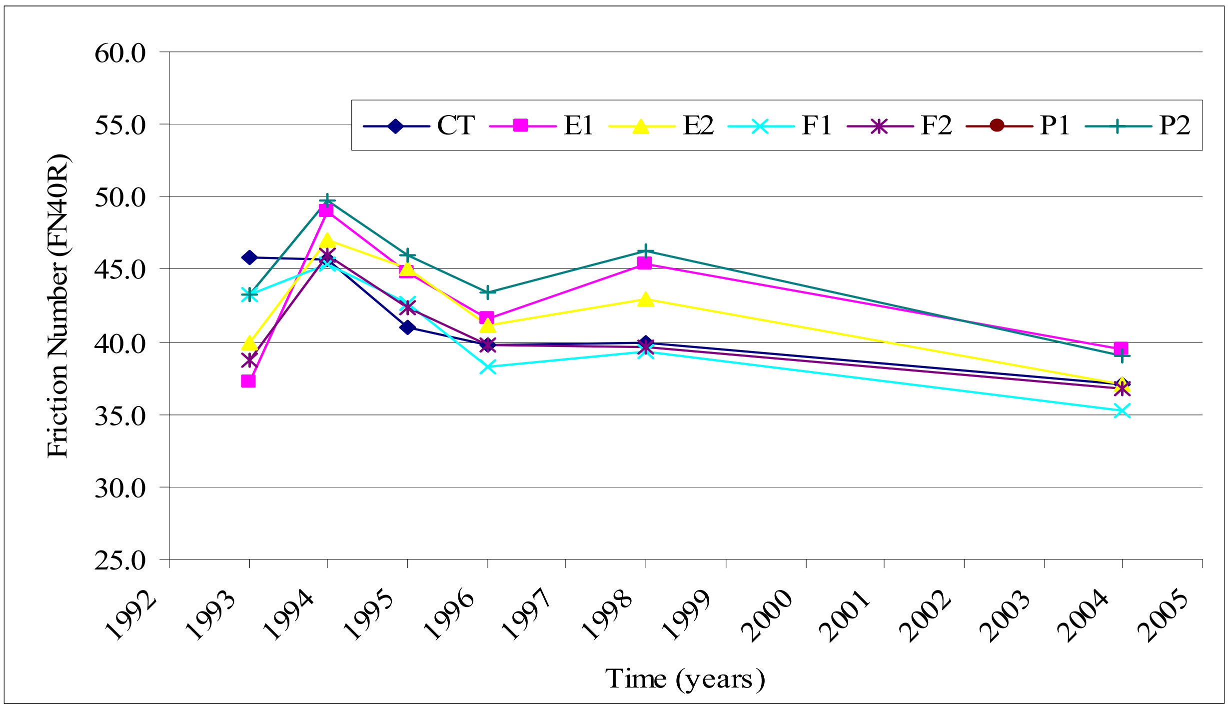


Figure 56 Site 6 NB Subsection (USH 151 Grant/Lafa.) Historical FN40R Trends

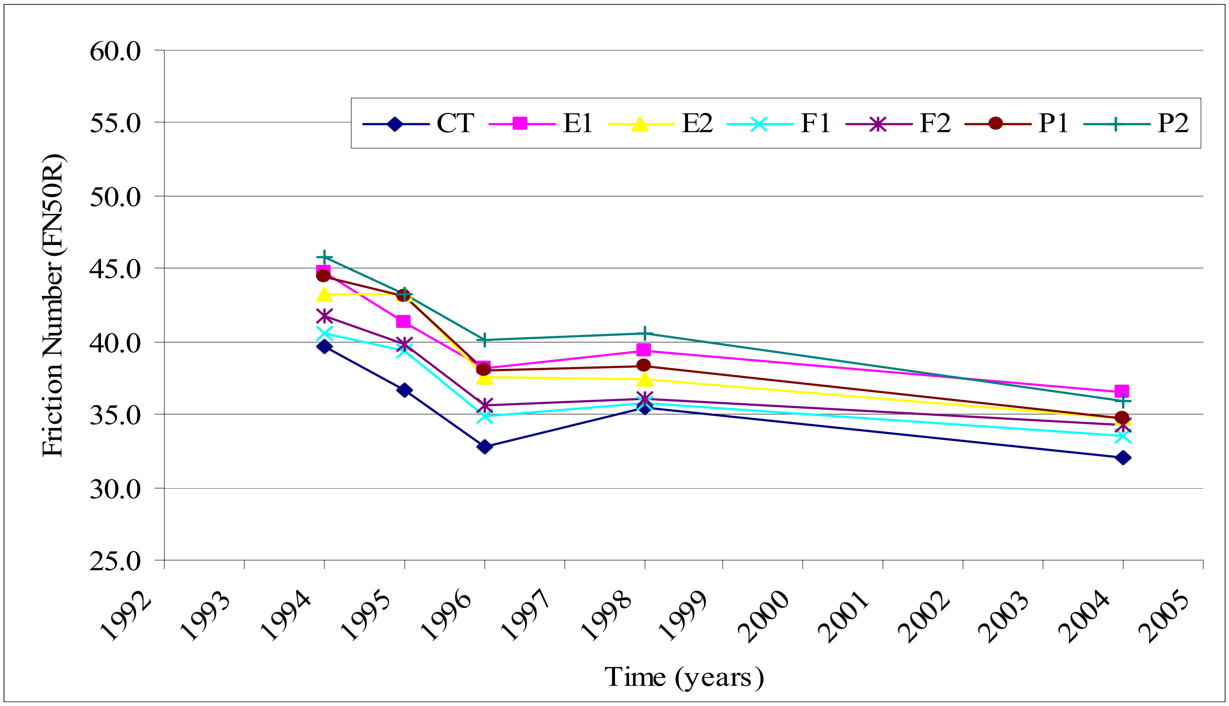


Figure 57 Site 6 NB Subsection (USH 151 Grant/Lafa.) Historical FN50R Trends

Table 45 Site 6 SB Pavement Subsection Friction Numbers

USH 151 Grant/Lafayette Counties						
	Southbound Subsection Mix Design					
	CT		E1		E2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	44.0	-	39.1	-	39.8	-
1994	45.8	36.6	48.1	43.1	45.2	41.5
1995	41.8	36.8	43.9	41.0	42.0	40.3
1996	41.7	37.5	41.1	37.9	39.4	36.1
1998	40.4	38.2	41.0	40.6	40.1	38.2
2004	36.7	33.9	35.1	33.4	34.2	32.0

Table 46 Site 6 SB Pavement Subsection Friction Numbers (Continued)

USH 151 Grant/Lafayette Counties								
	Southbound Subsection Mix Design							
	F1		F2		P1		P2	
Year	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R	FN40R	FN50R
1993	36.0	-	37.9	-	42.1	-	43.4	-
1994	47.0	41.1	47.9	42.6	50.1	44.3	48.6	43.9
1995	44.5	40.8	44.7	39.9	45.8	42.5	45.7	43.3
1996	41.7	39.1	43.3	39.5	45.0	40.3	44.0	41.1
1998	43.1	40.3	44.6	41.0	44.9	42.9	44.2	43.4
2004	37.5	30.4	35.0	33.4	37.3	34.9	37.7	35.5

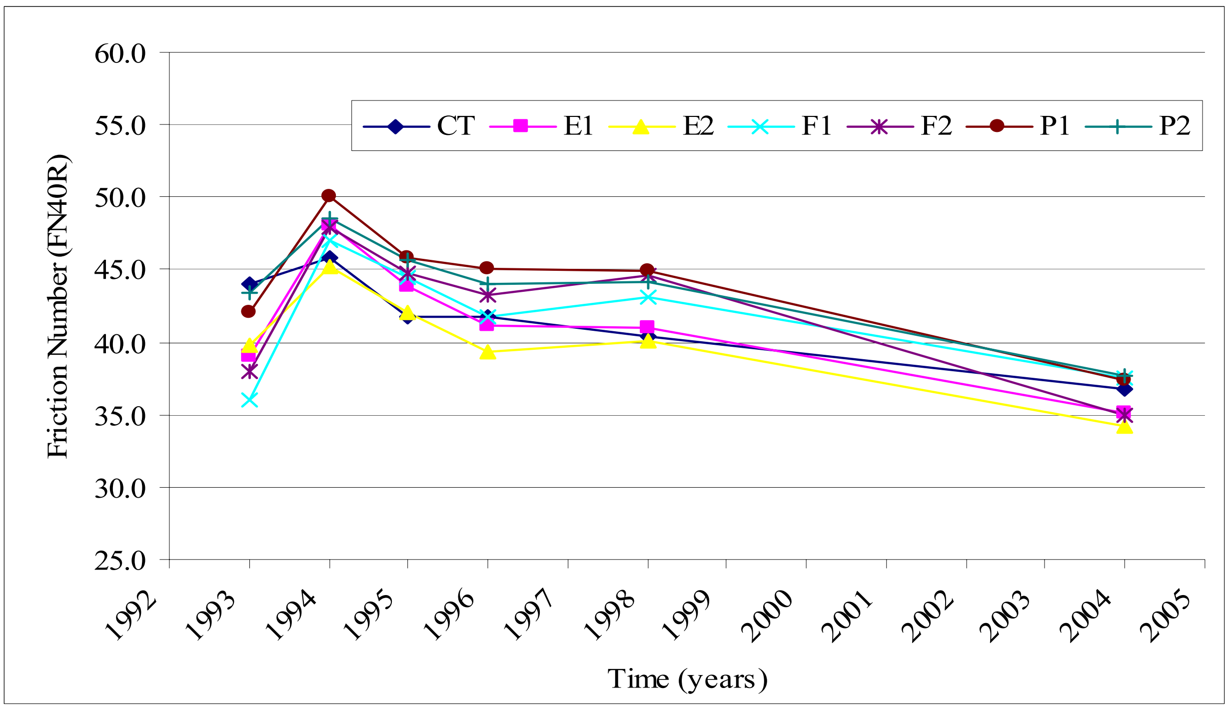


Figure 58 Site 6 SB Subsection (USH 151 Grant/Lafa.) Historical FN40R Trends

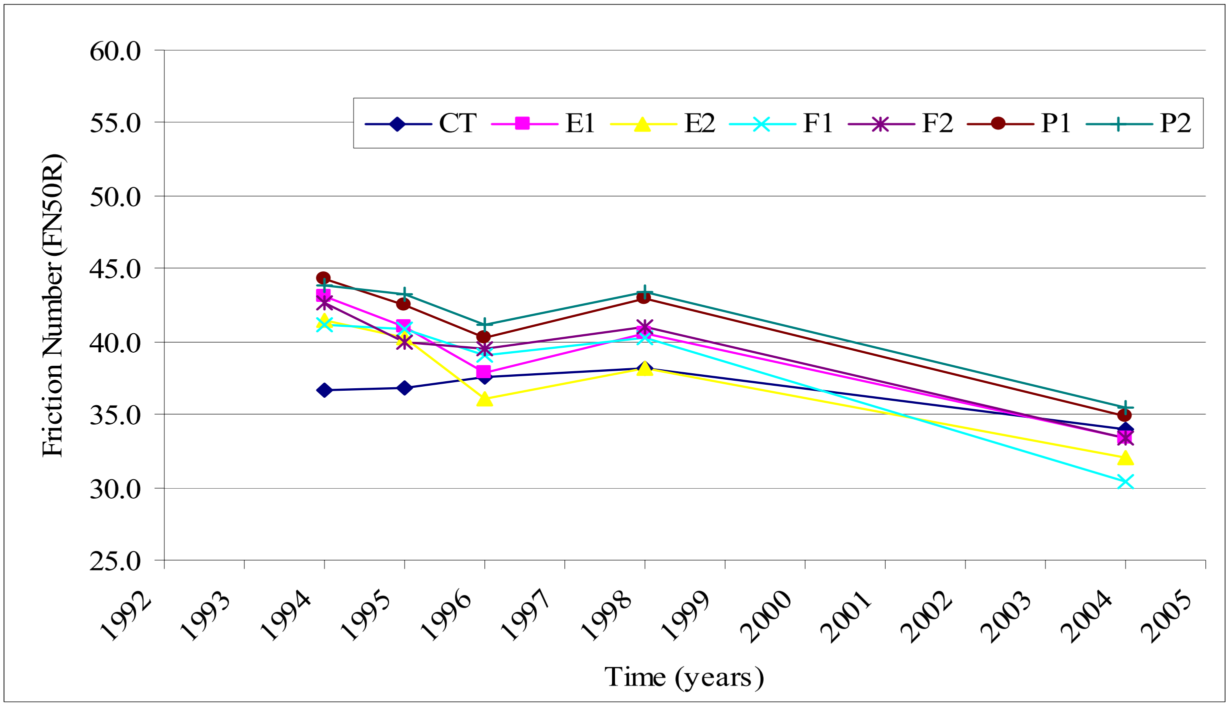


Figure 59 Site 6 SB Subsection (USH 151 Grant/Lafa.) Historical FN50R Trends

Crash Data Summary

Crash data for each study site were collected and analyzed, covering a period from January 1994 to June 2004. Table 47 shows the number of all crashes for each site during this period. Figures 60 through 67 show annual crash frequency at each study site, along with an average trend line. As shown in Figures 60 through 67, there are no consistent trends in total crash frequency over the analysis period. More specifically, no trend of crash increases with decreasing skid friction values was observed.

Table 47 Total Number of All Crash Types for Study Sites (Period 1994 - 2004)

Site	Highway	County	Number of Crashes
1	IH 43 NB	Walworth	127
	IH 43 SB	Walworth	49
2	IH 43 NB	Waukesha	114
	IH 43 SB	Waukesha	124
3	IH 94 EB	Monroe	137
4	IH 94 WB	Waukesha	22
5	STH 21	Juneau	58
6	USH 151	Grant/Lafayette	84

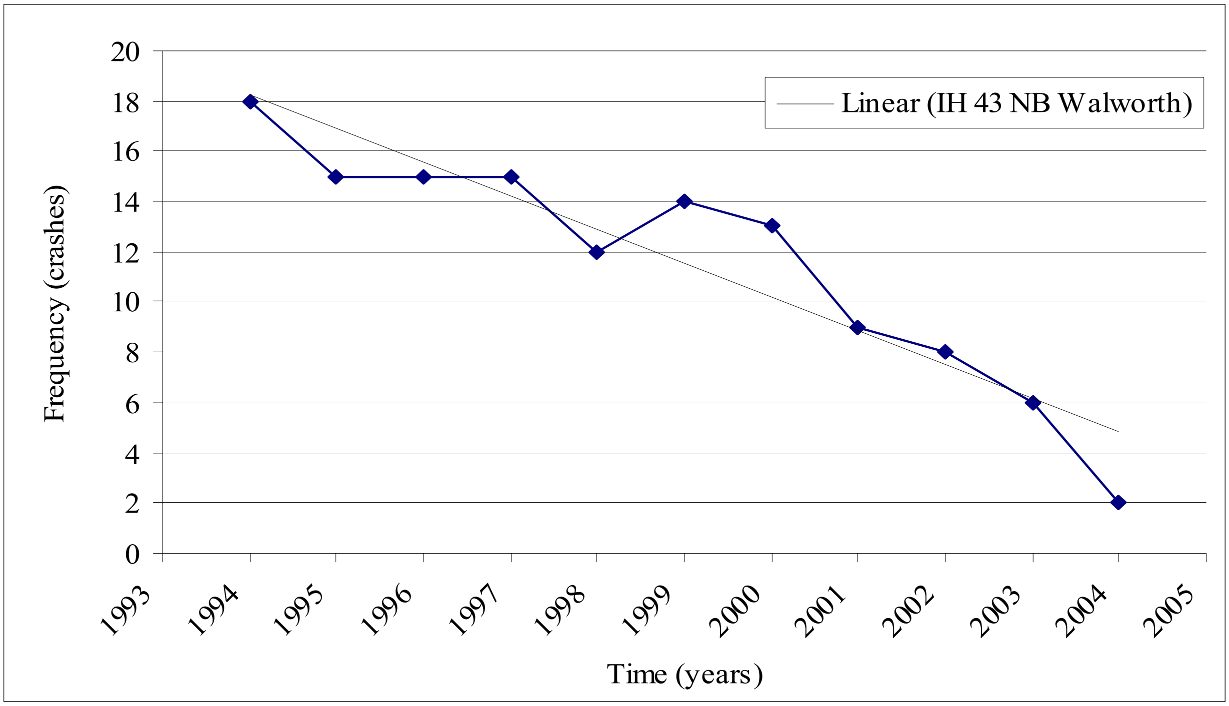


Figure 60 All Crashes at Site 1 NB (IH 43 Walworth)

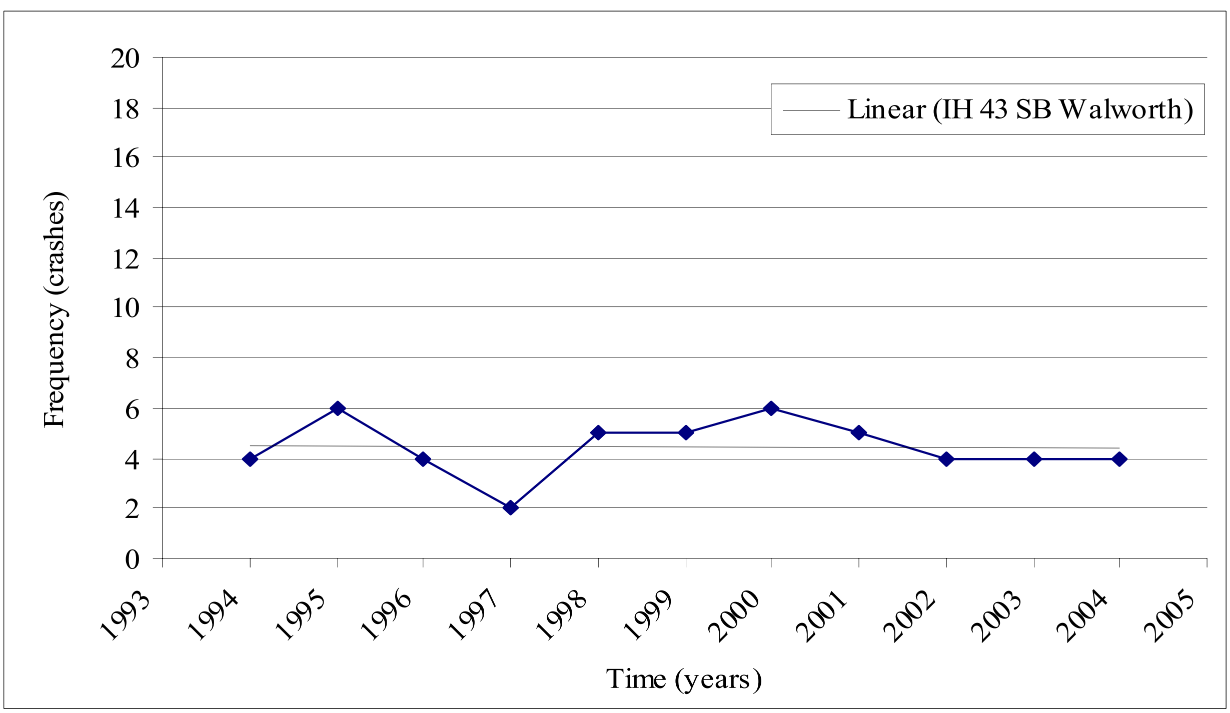


Figure 61 All Crashes at Site 1 SB (IH 43 Walworth)

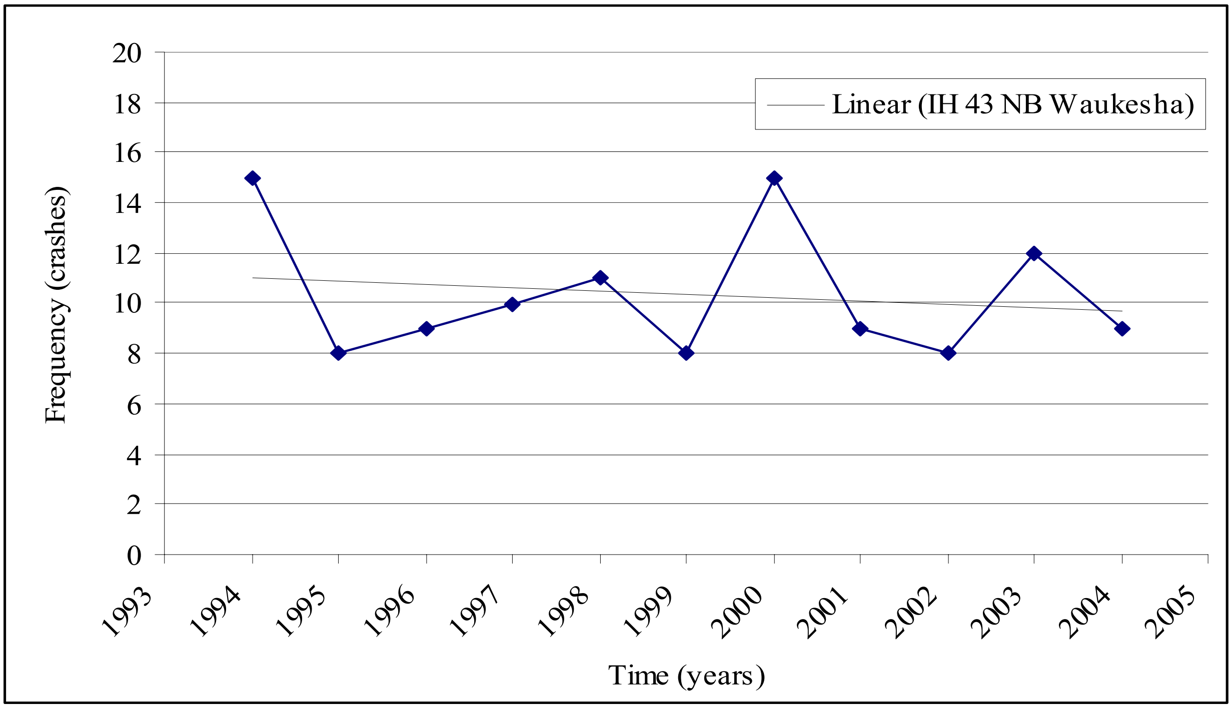


Figure 62 All Crashes at Site 2 NB (IH 43 Waukesha)

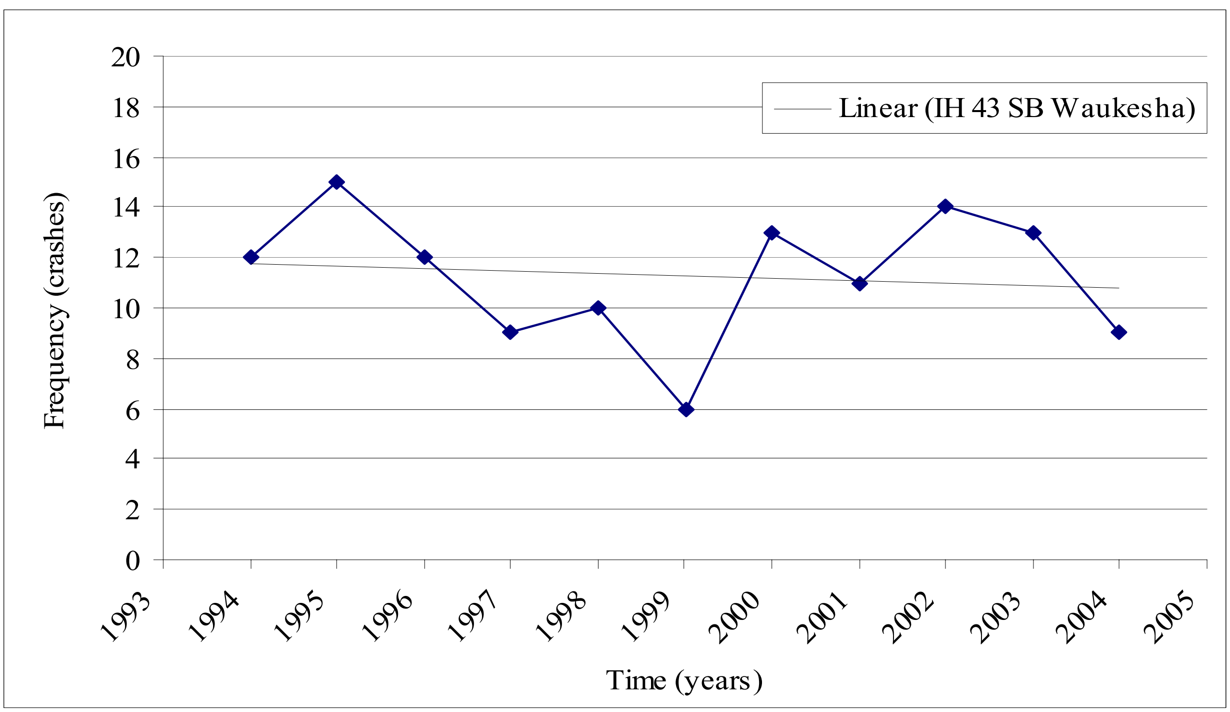


Figure 63 All Crashes at Site 2 SB (IH 43 Waukesha)

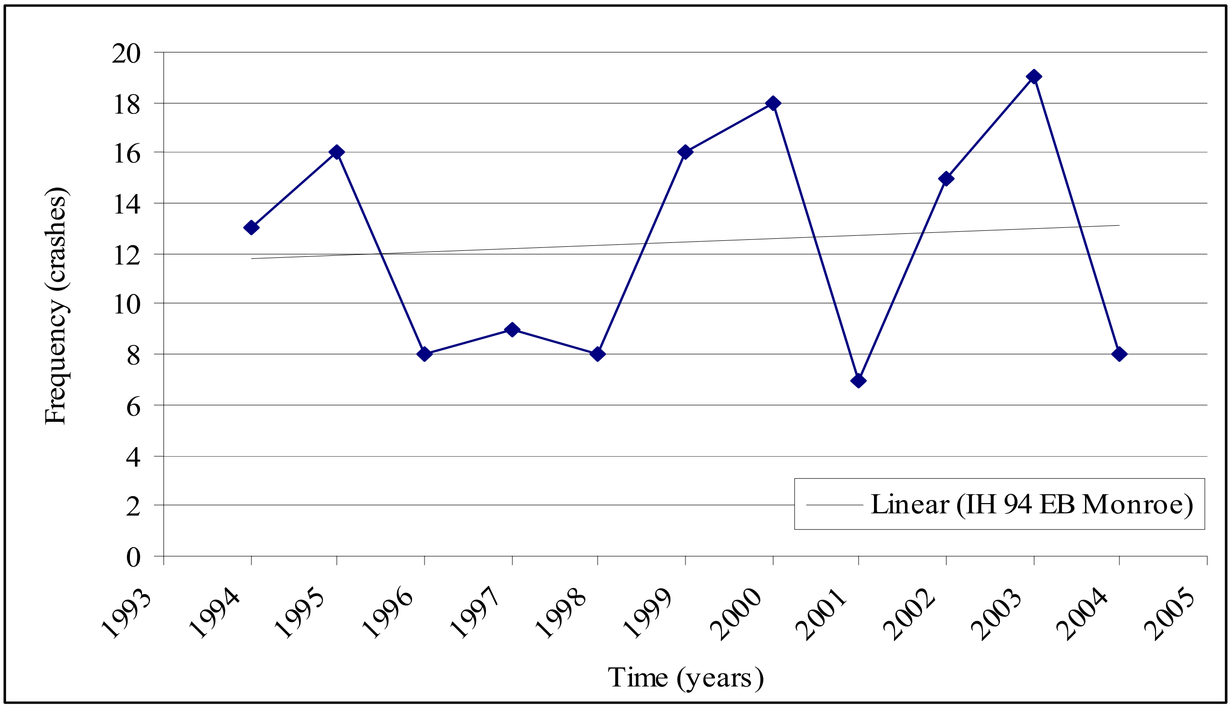


Figure 64 All Crashes at Site 3 EB (IH 94 Monroe)

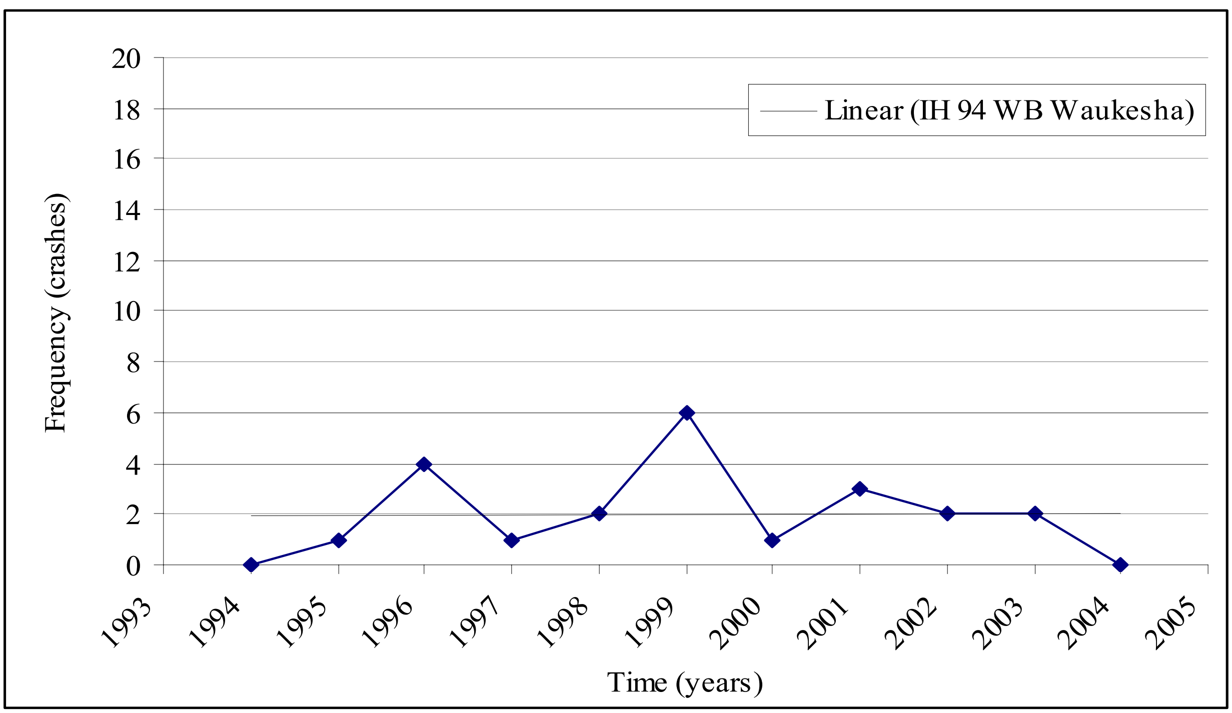


Figure 65 All Crashes at Site 4 WB (IH 94 Waukesha)

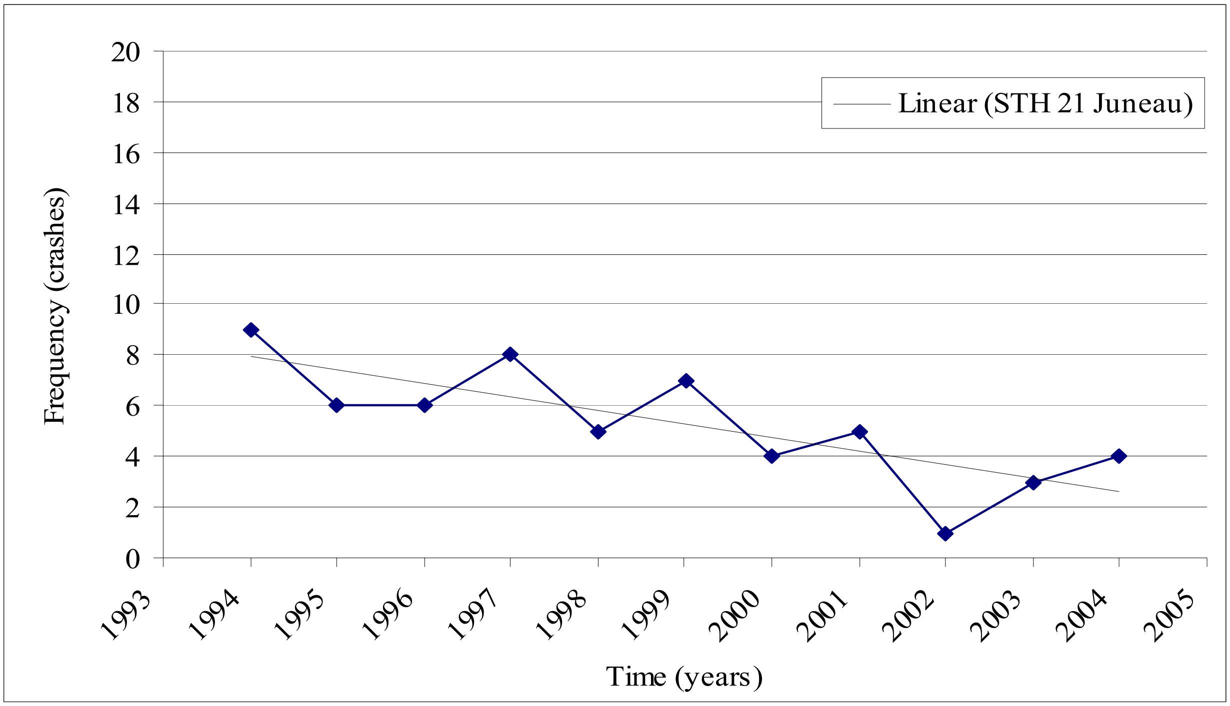


Figure 66 All Crashes at Site 5 (STH 21 Juneau)

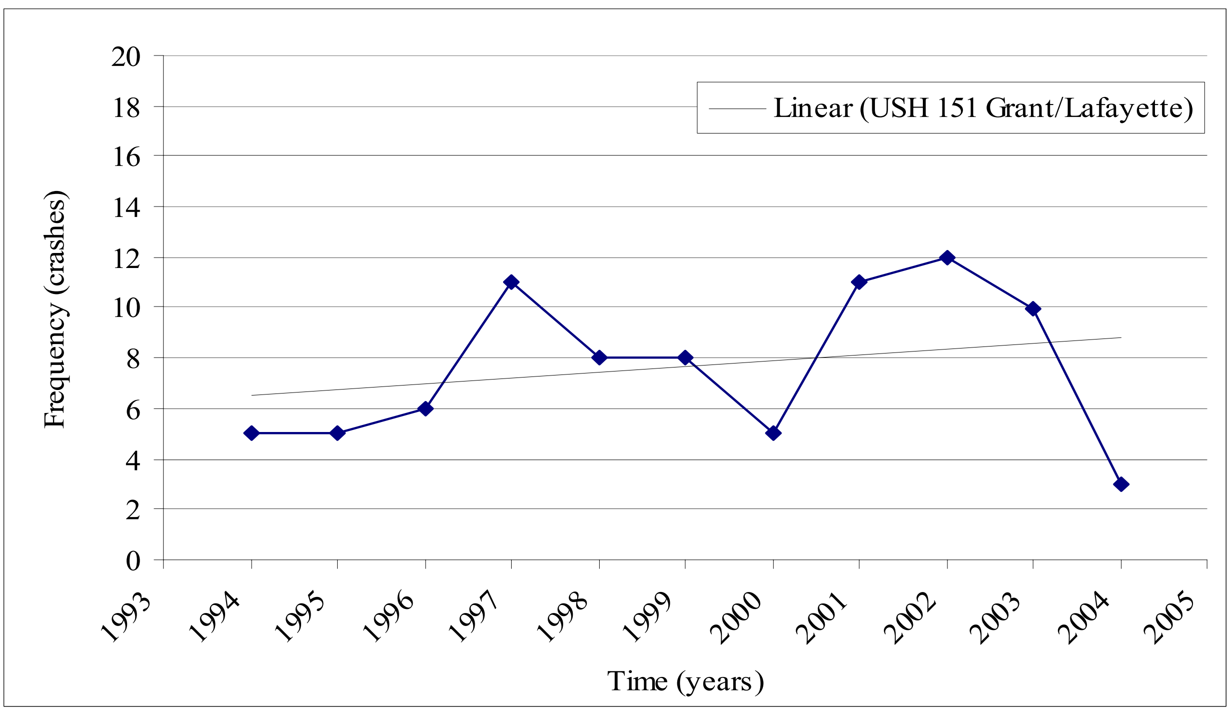


Figure 67 All Crashes at Site 6 (USH 151 Grant/Lafayette)

Crash data were further analyzed to select only skid-related crashes. Crash types evaluated included those that involved heavy braking, skidding, or loss of vehicle control. Table 48 shows the crashes selected as potentially skid-related. Selected skid-related crashes in Table 48 were validated using the original crash report completed by the law enforcement officer at the scene. This validation was completed by reviewing the narrative and crash diagrams in each report and correlating the key facts of the crash. Several of the previously selected skid crashes were eliminated via this process since no contributing factors could be found between the crash and the associated pavement. Table 49 shows the validated crashes selected for study. All of the crashes in Table 49 are considered to have pavement as a contributing factor. All skid crashes selected for study are summarized in greater detail in Tables 50 through 56. Descriptions were summarized from the narrative included in each crash report.

Table 48 Selected Friction-Related Crashes for Study Sites (Period 1994 - 2004)

Site	Highway	Wisconsin County	Number of Crashes
1	IH 43 NB	Walworth	24
	IH 43 SB	Walworth	30
2	IH 43 NB	Waukesha	45
	IH 43 SB	Waukesha	42
3	IH 94	Monroe	50
4	IH 94	Waukesha	11
5	STH 21	Juneau	10
6	USH 151	Grant/Lafayette	44

Table 49 Friction-Related Crashes Selected for Study (Period 1994 - 2004)

Site	Highway	Wisconsin County	No. Crashes
1	IH 43 NB	Walworth	7
	IH 43 SB	Walworth	10
2	IH 43 NB	Waukesha	22
	IH 43 SB	Waukesha	22
3	IH 94	Monroe	23
4	IH 94	Waukesha	5
5	STH 21	Juneau	3
6	USH 151	Grant/Lafayette	25

Table 50 Crash Descriptions for Site 1: IH 43, Walworth County, WI

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Northbound			
94080350506	01/25/1994	Ice	Lost control, struck parked vehicle along the road.
94080350602	01/25/1994	Ice	Skid, run off the road, rolled over twice.
95130671453	02/26/1995	Ice	Run off the road, struck speed limit sign.
95382220189	07/26/1995	Wet	Hydroplane, lost control, went into ditch.
98221840617	06/23/1998	Dry	Lost control, struck guardrail.
99261942369	06/28/1999	Dry	Lost control, overcorrected, skid into median ditch.
00040191548	01/03/2000	Ice	Lost control, partially overturn into median.
Southbound			
95060270516	01/17/1995	Ice	Skid, lost control, hit by another vehicle, both run off road.
95130671455	02/26/1995	Ice	Skid, struck bridge rail, struck head on by other vehicle
95150810690	02/27/1995	Ice	Skid, run off the road, struck tree.
96191230370	04/15/1996	Wet	Skid, lost control, roll over, ended up in median.
97140900895	03/14/1997	Ice	Lost control, spun, hit guardrail.
99070350235	01/18/1999	Ice	Lost control, hit guardrail.
00120730713	02/18/2000	Snow	Skid, struck rear of vehicle in front.
01020082002	12/26/2000	Snow	Lost control, skid, overturn twice.
02140650554	01/14/2002	Ice	Hit slick spot on road, rolled over several times.
04180780218	02/26/2004	Dry	Lost control, rolled over one time.

Table 51 Crash Descriptions for Site 2: IH 43, Waukesha County, WI (Northbound)

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Northbound			
94100450384	02/08/1994	Snow	Lost control, hit another vehicle.
94100460702	02/09/1994	Snow	Lost control, run off road, roll over.
94110491181	02/08/1994	Ice	Lost control, struck guardrail.
94140671213	02/25/1994	Snow	Lost control, spun, struck another vehicle.
94140671215	02/25/1994	Snow	Fishtail, struck another vehicle.
95050240663	01/19/1995	Ice	Lost control, hit by another car, ended up in median.
96070321458	01/29/1996	Snow	Lost control, hit sign post.
97040223095	01/15/1997	Snow	Unable to stop, went into the median, hit another car.
98030141320	01/08/1998	Ice	Lost control, spun, hit by another vehicle.
99030191262	01/08/1999	Snow	Lost control, left road, overturned.
00161040267	04/07/2000	Ice	Skid, jackknife.
00171110646	04/07/2000	Snow	Lost control, skid, hit by another vehicle.
00171110654	04/07/2000	Snow	Vehicle pulling trailer, lost control, overturned trailer.
00543561572	12/13/2000	Snow	Lost control and sideswiped another vehicle.
01100380219	01/29/2001	Wet	Lost control, entered median, rolled over.
02643400684	11/28/2002	Dry	Lost control, left road, struck post.
03181000460	04/07/2003	Snow	Lost control, hit guardrail face.
03181000498	04/07/2003	Snow	Truck unable to stop, went into ditch, hit sign.
03673460231	12/05/2003	Wet	Lost control, stuck another vehicle.
04210890866	03/19/2004	Ice	Lost control, struck wire fence.
04210890882	03/19/2004	Ice	Lost control, spun, hit guardrail.
04230980510	03/28/2004	Wet	Lost control, struck a sign.

Table 52 Crash Descriptions for Site 2: IH 43, Waukesha County, WI (Southbound)

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Southbound			
94130620173	02/25/1994	Ice	Unable to stop, skid hit another vehicle.
94130620193	02/25/1994	Ice	Unable to stop, skid hit another vehicle.
95503190787	11/10/1995	Snow	Lost control, left lane, hit by another vehicle.
95543351268	11/27/1995	Snow	Lost control, spun, struck delineator posts.
95553390159	11/29/1995	Snow	Skid on ice, cross over snow bank, roll over.
96040190770	01/16/1996	Wet	Skid, went into ditch.
96453310931	11/20/1996	Snow	Lost control, skid, struck other vehicle.
96463372121	11/20/1996	Snow	Skid, run off road, overturned.
98262161766	07/24/1998	Dry	Motorcycle skid, flips on the road.
99060331915	01/23/1999	Wet	Changing lane, skid, hit traffic sign.
00221440631	05/17/2000	Wet	Left lane, sideswipe another car and went into embankment.
00241601326	05/21/2000	Dry	Run off road, went into median, hit by two vehicles.
00271791563	06/13/2000	Wet	Lost control, run off road, hit hillside embankment.
00533480679	12/07/2000	Snow	Lost control, went of road, hit a tree.
01010032402	12/20/2000	Snow	Changing lane, lost control, hit guardrail.
01743530980	12/12/2001	Wet	Left lane, hit parked vehicle.
01743530984	12/12/2001	Wet	Left lane, hit parked vehicle.
02050181149	01/14/2002	Snow	Lost control, run off road, went into ditch.
02542810750	09/27/2002	Dry	Lost control and went into ditch.
02663502627	12/02/2002	Snow	Lost control, hit by another vehicle, both hit guardrail.
03120700074	03/04/2003	Snow	Lost control, skid, run off road.
03613211839	11/04/2003	Wet	Vehicle slowing, hit by vehicle behind.

Table 53 Crash Descriptions for Site 3: IH 94, Monroe County, WI

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Eastbound			
94080340216	01/06/1994	Ice	Lost control, spun four times, hit guardrail.
94231290866	04/24/1994	Wet	Lost control, spun, run off road.
95452760721	09/14/1995	Dry	Left lane, overcorrected, skid, run off road.
96453312431	11/15/1996	Ice	Lost control, skid, hit guardrail face.
97140900243	03/13/1997	Snow	Slowing, rear ended by another vehicle.
99060270812	01/02/1999	Snow	Lost control on slippery road, hit guardrail.
99080500311	01/14/1999	Ice	Lost control, jackknifed in the median.
99080530342	01/22/1999	Ice	Spun out of control, struck guardrail.
99080530359	01/23/1999	Ice	Overtaking on left, lost control, overturned.
99080530361	01/23/1999	Ice	Lost control, overturned.
99282110958	07/20/1999	Wet	Left lane, lost control, overturned.
99292210343	07/18/1999	Wet	Lost control went into ditch, flipped/rolled.
00453050638	10/24/2000	Unknown	Hit concrete barrier, delineator post, went into ditch.
01040160511	12/23/2000	Ice	Lost control, skid into median, overturned.
02040110825	12/23/2001	Snow	Lost control, hit vehicle, spun, hit by another vehicle.
02190870424	03/02/2002	Ice	Lost control, hit bridge rail.
02241140469	04/14/2002	Wet	Hydroplane, went under flatbed trailer.
02241140475	04/11/2002	Wet	Hydroplane, struck semi trailer.
02402070046	07/20/2002	Wet	Hydroplane, spun, hit delineator post, went into ditch.
03060360711	01/28/2003	Snow	Sideswipe truck, lost control and skid into median.
03130730472	03/03/2003	Ice	Lost control, run off road, hit guardrail.
04120540252	02/06/2004	Ice	Lost control, hit delineator post.
04210910700	03/17/2004	Snow	Lost control went into the median, rolled over.

Table 54 Crash Descriptions for Site 4: IH 94, Waukesha County, WI

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Westbound			
95180970342	03/27/1995	Snow	Lost control, spun into ditch, hit post.
96191240395	04/15/1996	Snow	Lost control, spun into ditch, hit trees.
96463391240	11/28/1996	Ice	Skid, run off road down embankment.
98272290403	08/06/1998	Wet	Fishtailed, went into median.
99211620440	06/02/1999	Dry	Drift onto shoulder, overcorrected, skid, went into ditch.

Table 55 Crash Descriptions for Site 5: STH 21, Juneau County, WI

Crash Microfilm Number	Crash Date	Road Condition	Crash Description
Westbound			
96463340643	11/09/1996	Dry	Left lane, lost control, overturned.
02010031714	12/22/2001	Ice	Skid, run off the road, ended up in ditch.

Table 56 USH 151, Grant/Lafayette Counties, WI

Crash Micro. Num.	Crash Date	Road Condition	Crash Description
Northbound			
95110590454	02/10/1995	Ice	Lost control, run off road, struck snow bank.
95110590458	02/10/1995	Ice	Lost control on ice patch, run off road.
97120710940	02/14/1997	Ice	Lost control, skid into ditch continuing thru a fence.
97191320072	05/01/1997	Dry	Lost control, skid, struck mailbox.
97433230781	11/09/1997	Wet	Slowing, hit stopped vehicle.
97483530730	12/03/1997	Snow	Lost control, skid, struck oncoming car and truck.
98050350203	01/04/1998	Ice	Lost control, skid, struck ditch.
99110740868	03/05/1999	Ice	Skid, rolled twice into ditch.
99110740873	03/06/1999	Snow	Lost control went into ditch continuing thru a fence.
00563640040	12/07/2000	Snow	Skid, run off road, struck ditch.
01251170248	04/15/2001	Wet	Lost control, skid, spun into ditch.
01462080542	07/18/2001	Wet	Hydroplane, lost control, struck embankment.
02462382377	08/21/2002	Wet	Lost control, spun, skid into ditch.
03070421674	01/28/2003	Ice	Went into curve, struck parked vehicle in the rear.
03593150829	10/29/2003	Wet	Lost control went into ditch, struck retaining wall.
04180760192	03/09/2004	Ice	Lost control went into embankment, rolled over.
Southbound			
97020072717	12/22/1996	Ice	Lost control went into ditch and driveway.
97120780469	02/21/1997	Snow	Lost control went into ditch.
98100832167	03/09/1998	Ice	Lost control went into ditch, overturned.
99110740870	03/05/1999	Ice	Skid out of control, struck parked squad car.
99463490901	12/08/1999	Ice	Struck icy spot, lost control, and skid into embankment.
01120460885	01/26/2001	Wet	Lost control, skid into snow bank and overturned.
02010031571	12/22/2001	Wet	Skid into ditch, hit embankment, overturn.
03030170536	01/07/2003	Dry	Run over ice, spin, struck mailbox.
04010021944	12/05/2003	Snow	Lost control slid into ditch and overturned.

Figures 68 through 76 provide a graphical representation of the annual incidence of friction-related crashes.

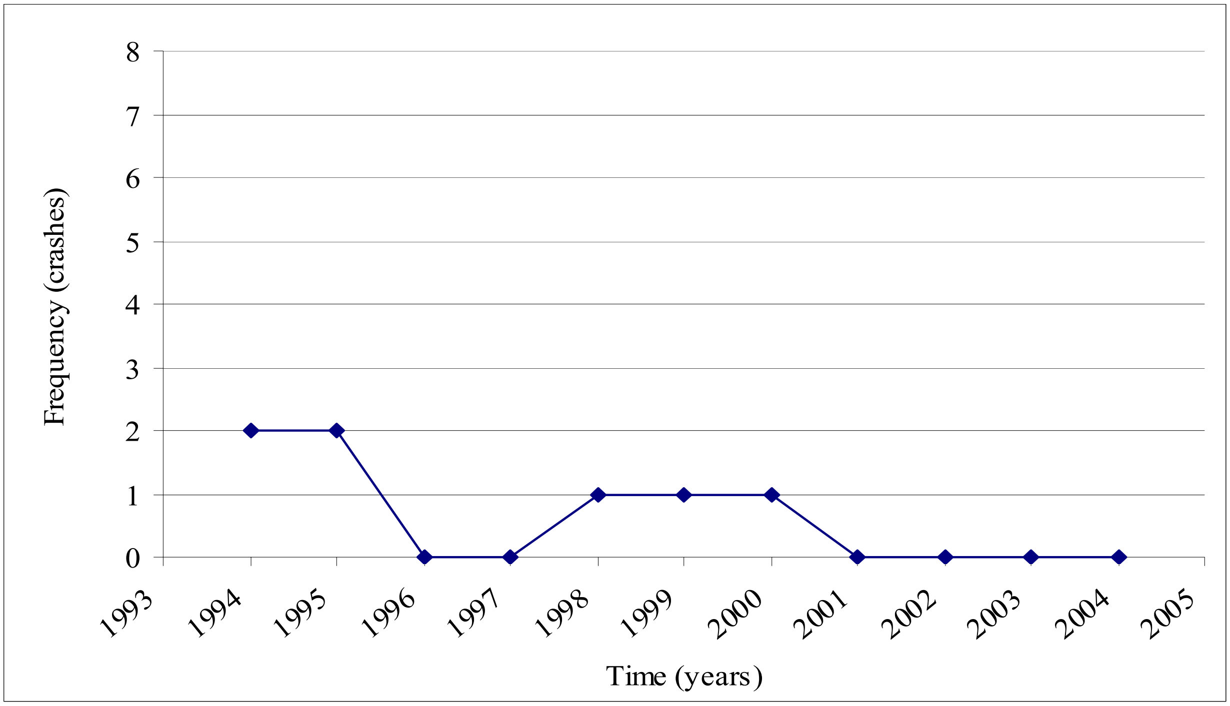


Figure 68 Friction-Related Crashes for Site 1 NB (IH 43 Walworth)

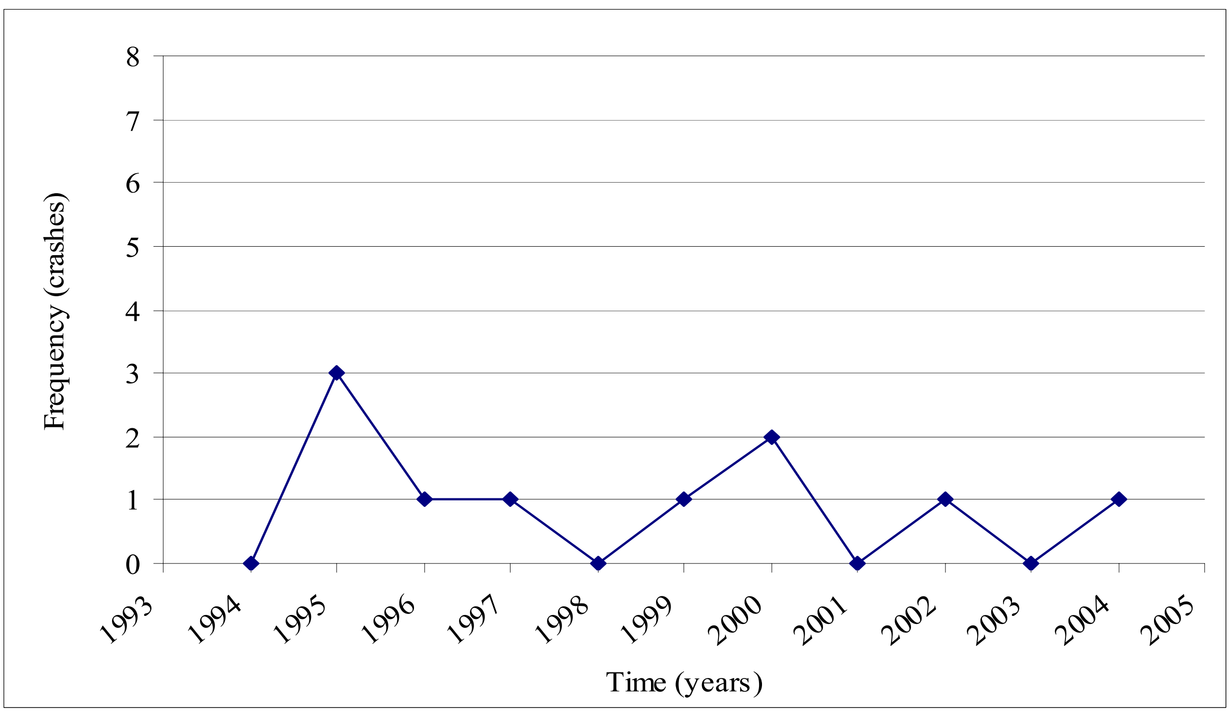


Figure 69 Friction-Related Crashes for Site 1 SB (IH 43 Walworth)

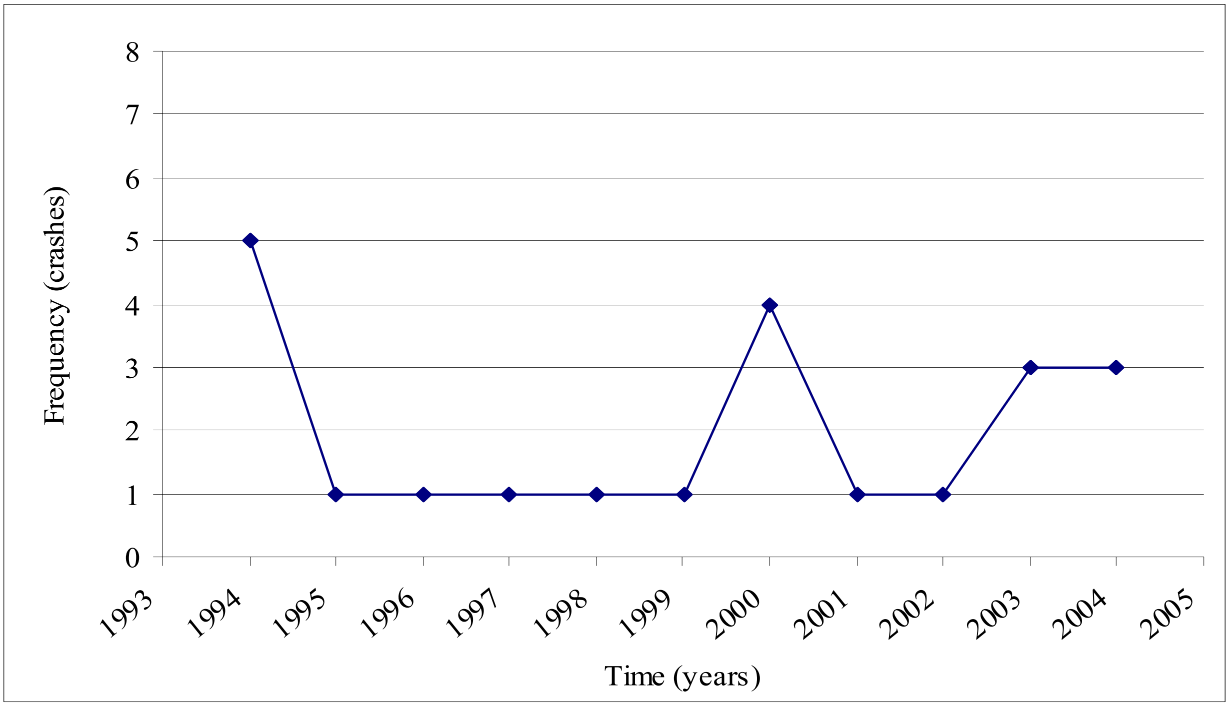


Figure 70 Friction-Related Crashes for Site 2 NB (IH 43 Walworth)

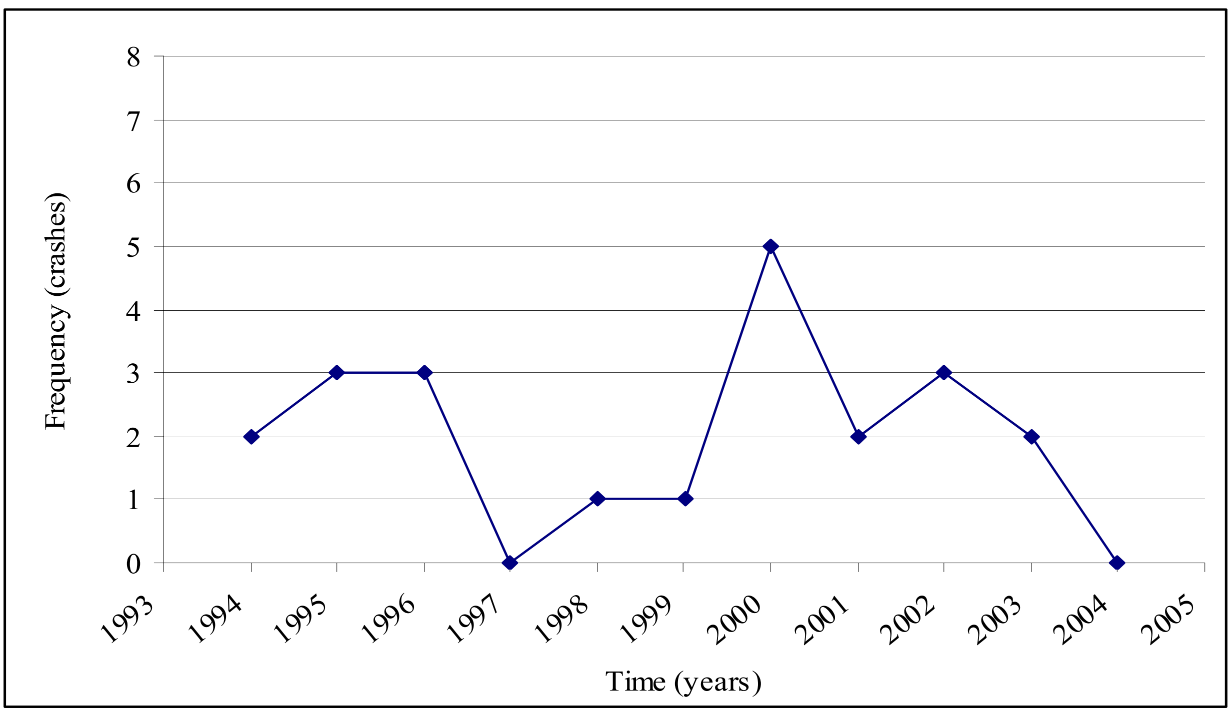


Figure 71 Friction-Related Crashes for Site 2 SB (IH 43 Walworth)

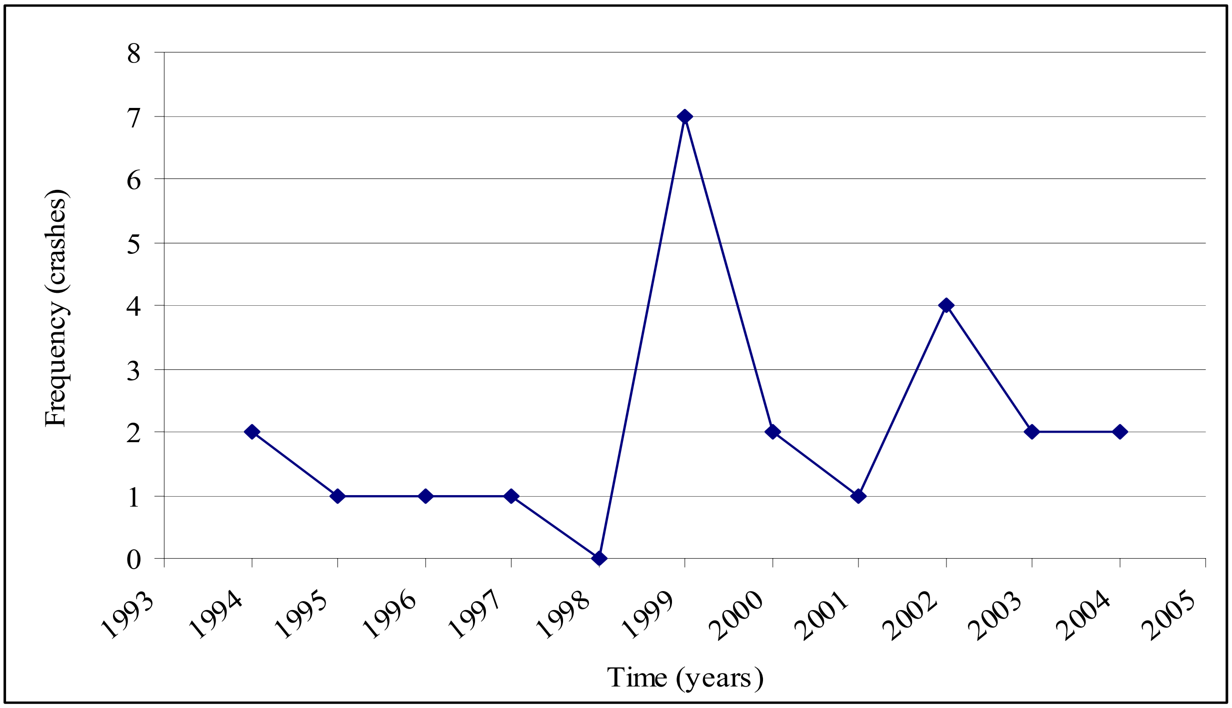


Figure 72 Friction-Related Crashes for Site 3 EB (IH 94 Monroe)

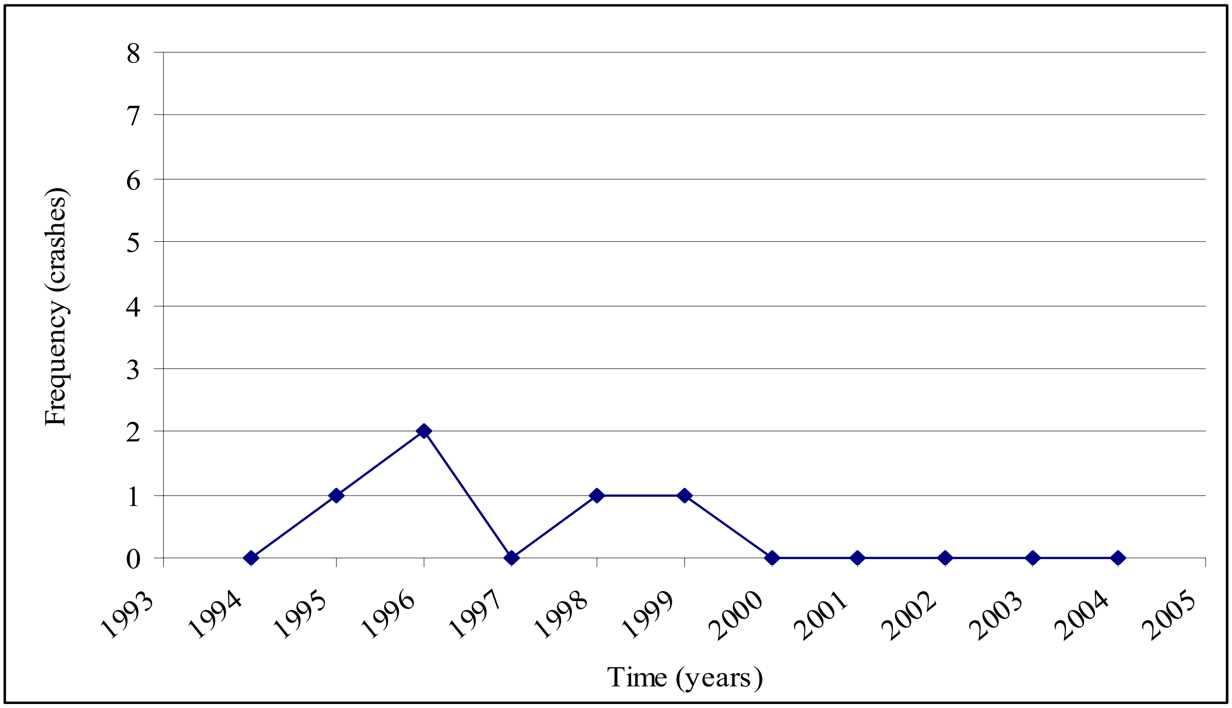


Figure 73 Friction-Related Crashes for Site 3 WB (IH 94 Waukesha)

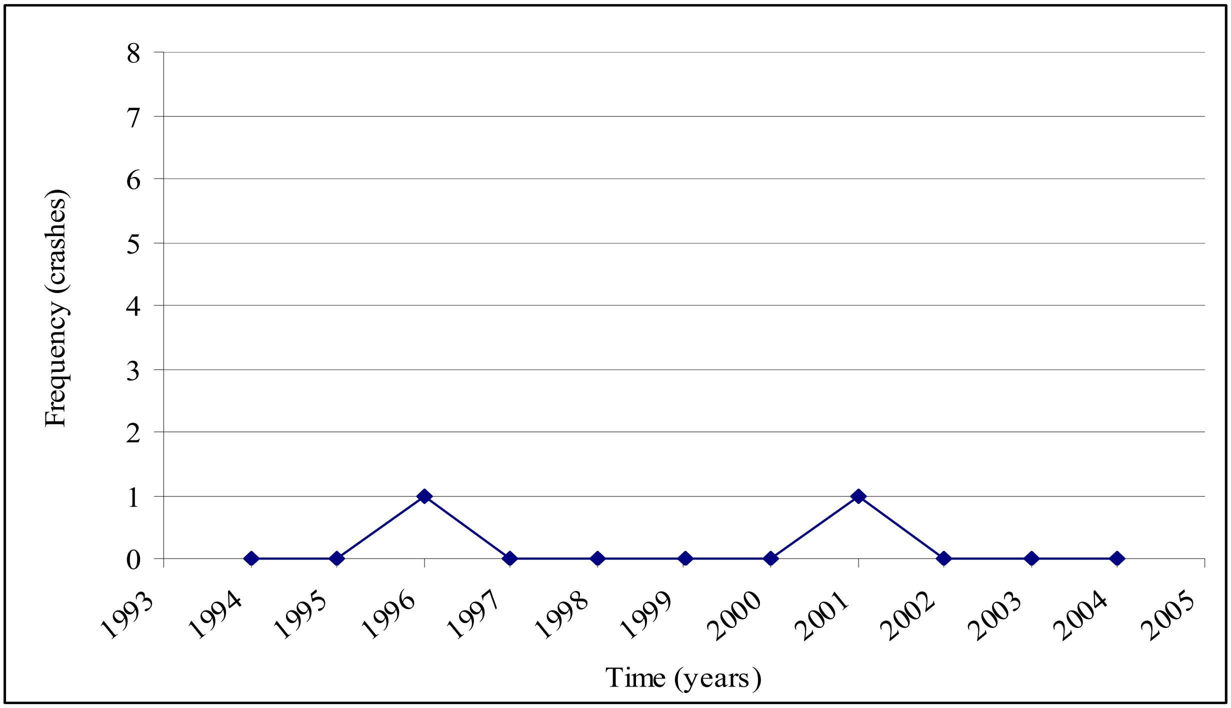


Figure 74 Friction-Related Crashes for Site 5 WB (STH 21 Juneau)

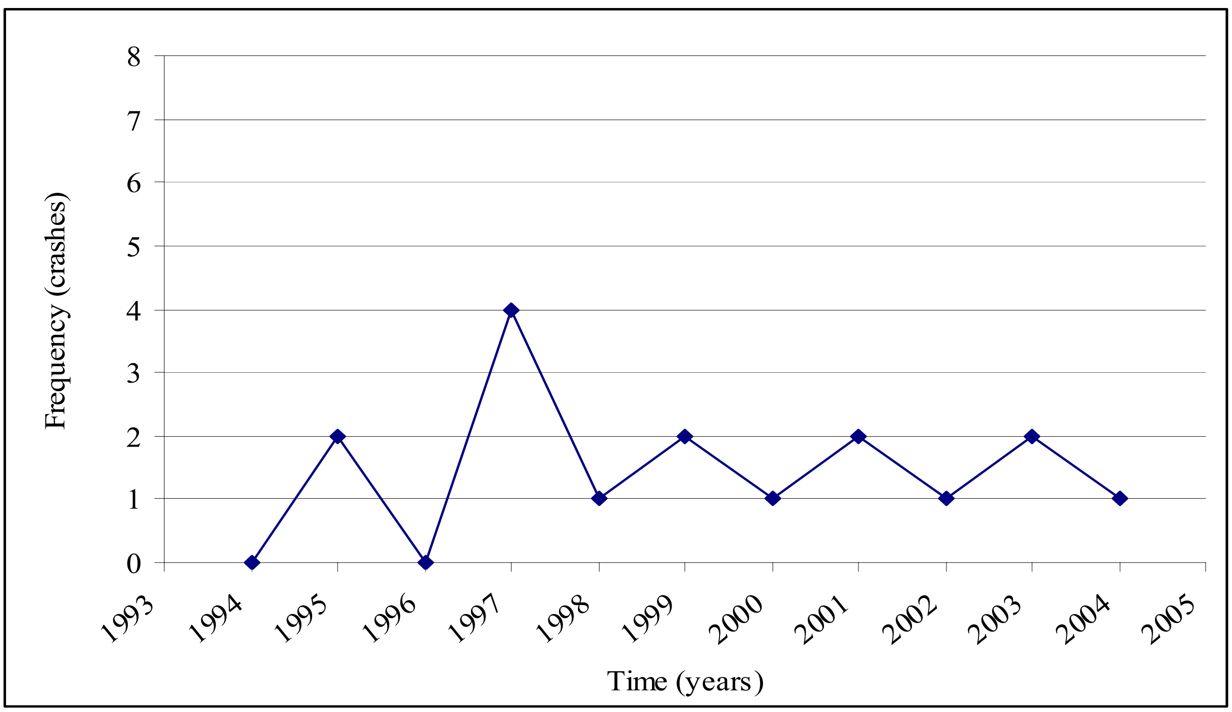


Figure 75 Friction-Related Crashes for Site 6 NB (USH 151 Grant/Lafayette)

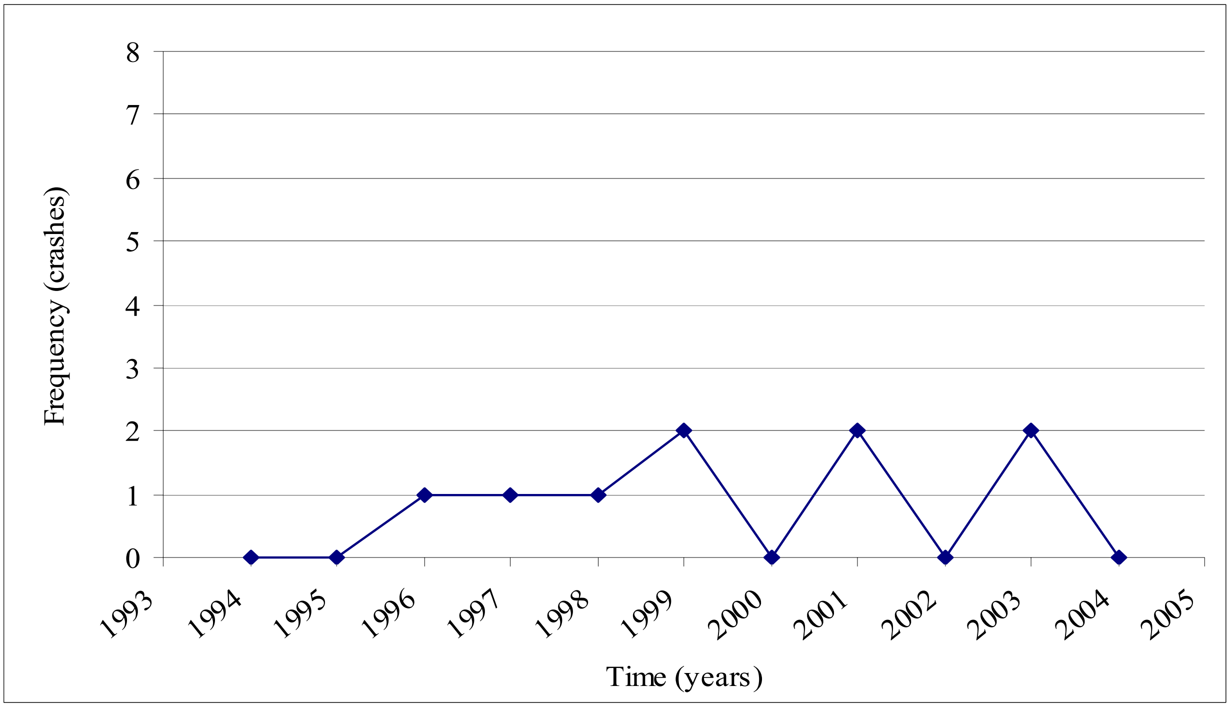


Figure 76   Friction-Related Crashes for Site 6 SB (USH 151 Grant/Lafayette)

A review of Tables 48 through 56 and Figures 68 through 76 show that there is not a consistent history of pavement friction-related crashes on any of the study sections. It must be kept in mind that these are not wet weather crashes only. Crashes above include skidding on any pavement condition (dry, wet, snow, or ice). However, since friction numbers were taken for wet conditions, a brief review of the wet pavement crashes is warranted.

Crash Data Comparison to Friction

Figures 77 through 85 show the trends for wet pavement skid-related crashes with their respective average friction value (FN40R) for each study site.

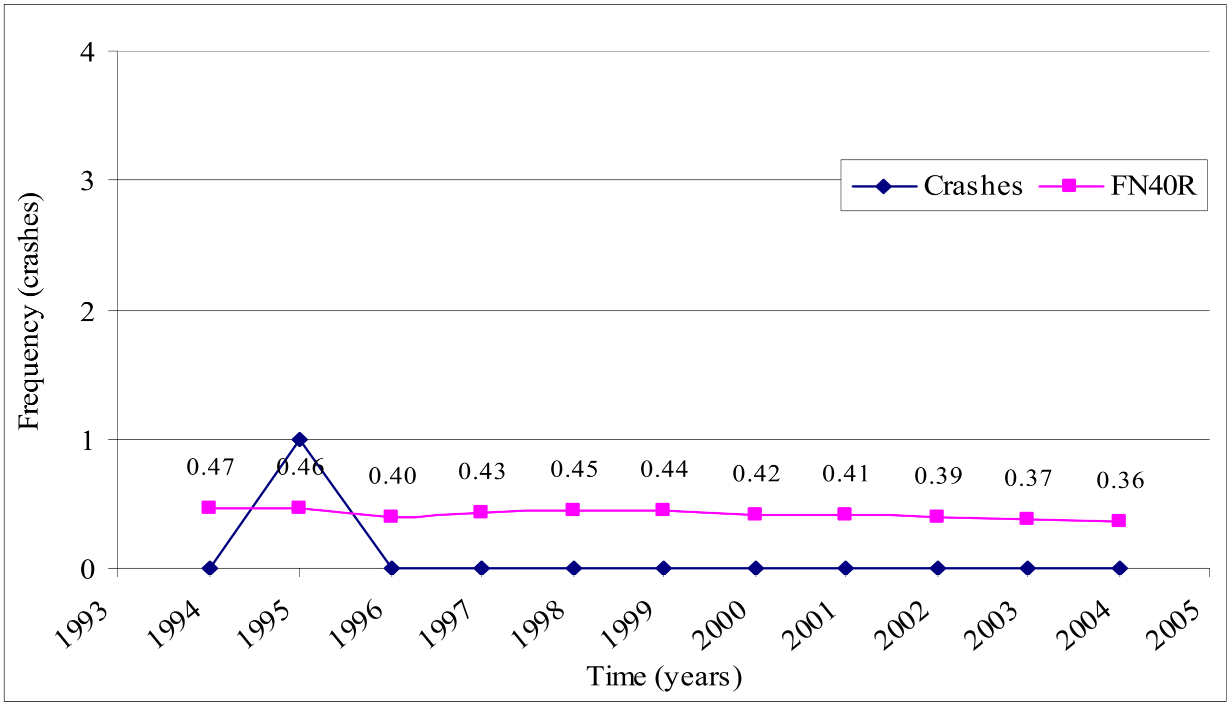


Figure 77 Wet Friction-Related Crashes for Site 1 NB (IH 43 Walworth)

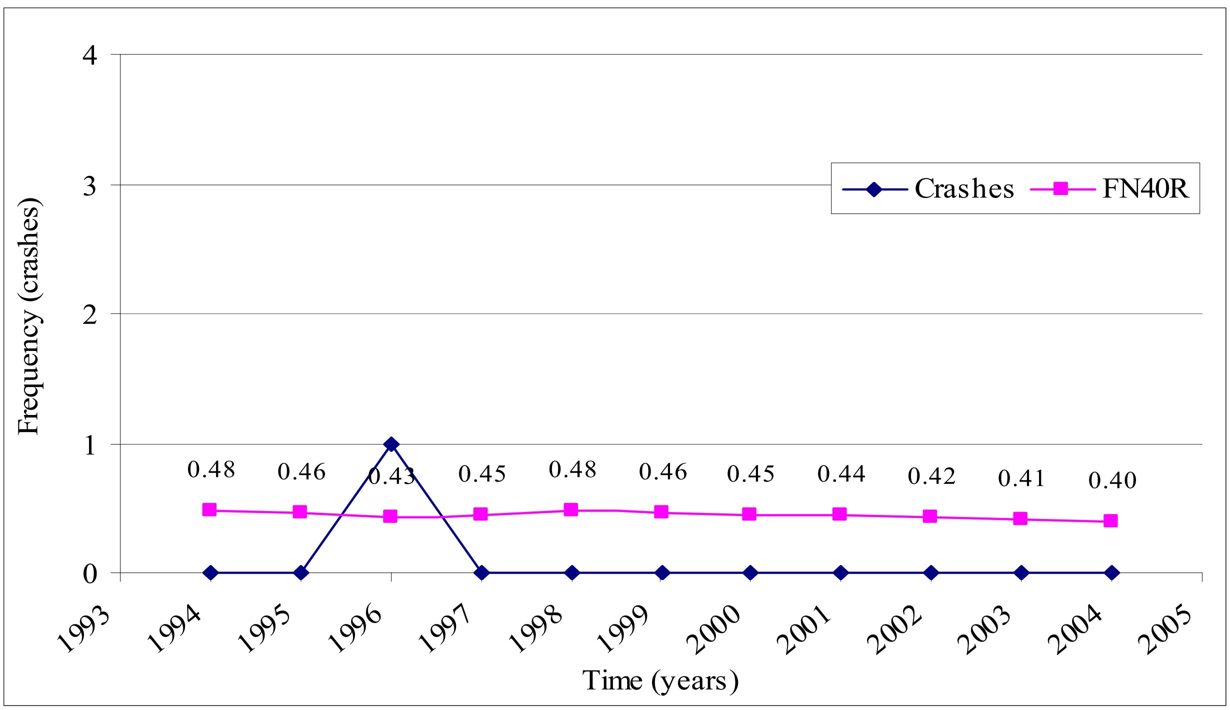


Figure 78 Wet Friction-Related Crashes for Site 1 SB (IH 43 Walworth)

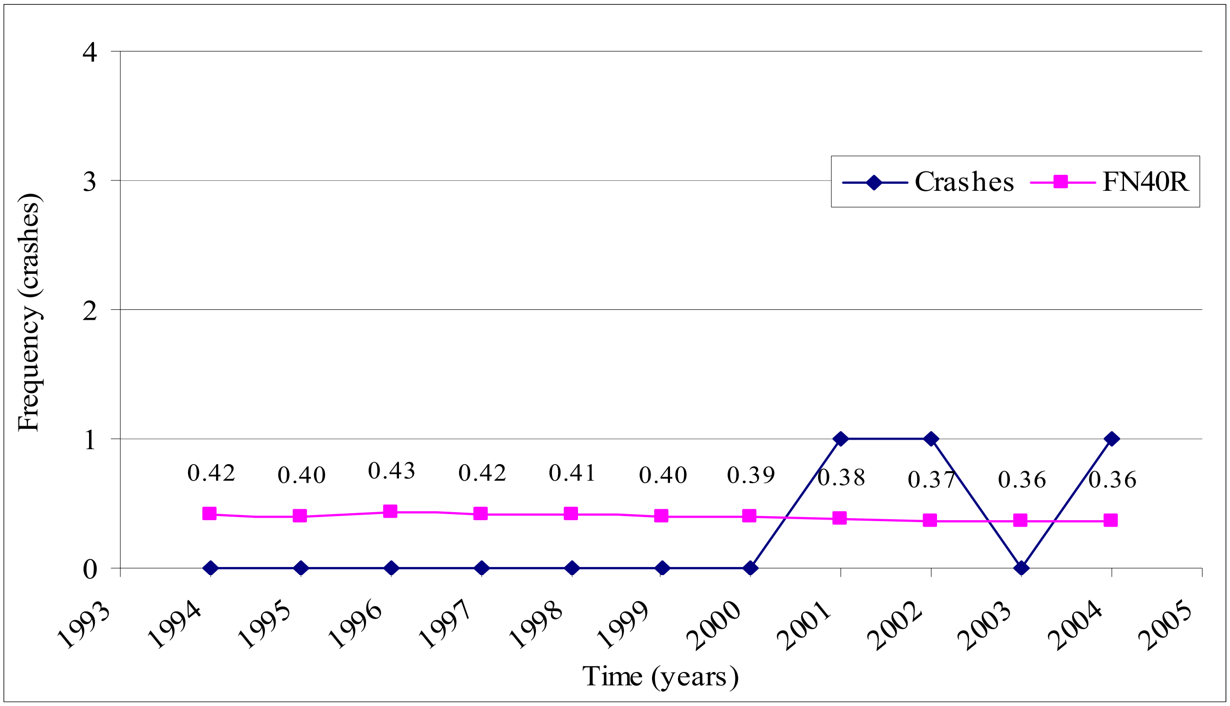


Figure 79 Wet -Related Crashes for Site 2 NB (IH 43 Waukesha)

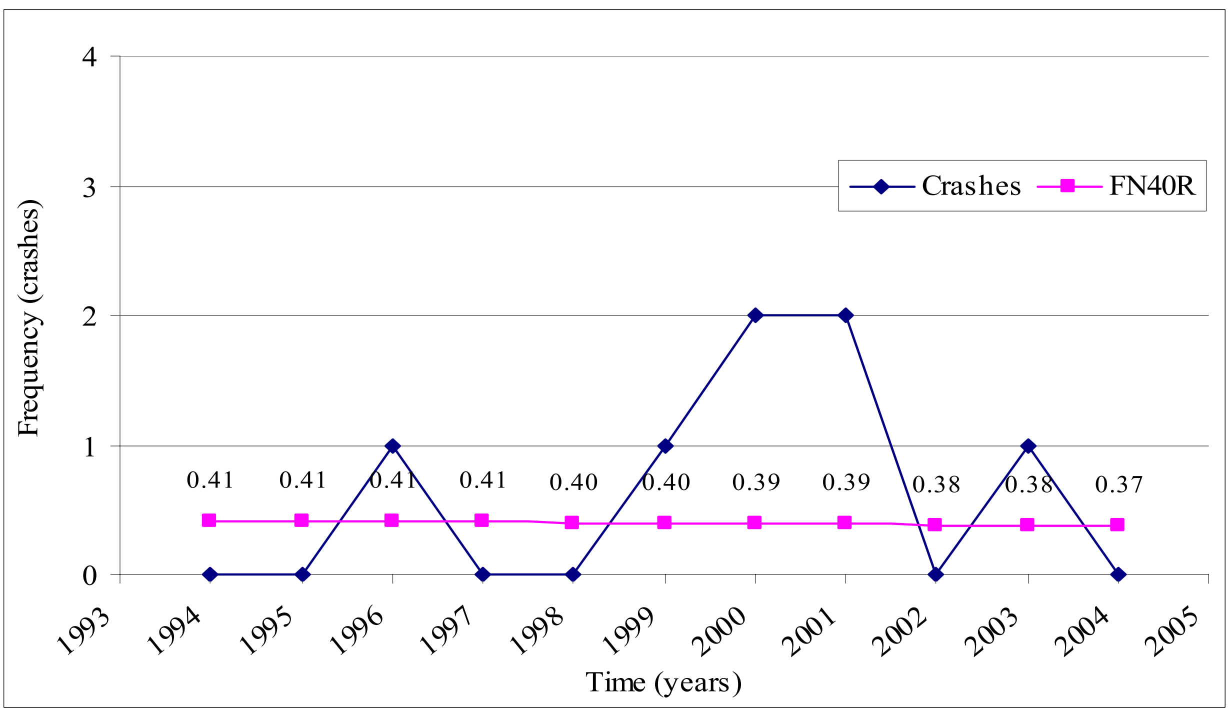


Figure 80 Wet Friction-Related Crashes for Site 2 SB (IH 43 Waukesha)

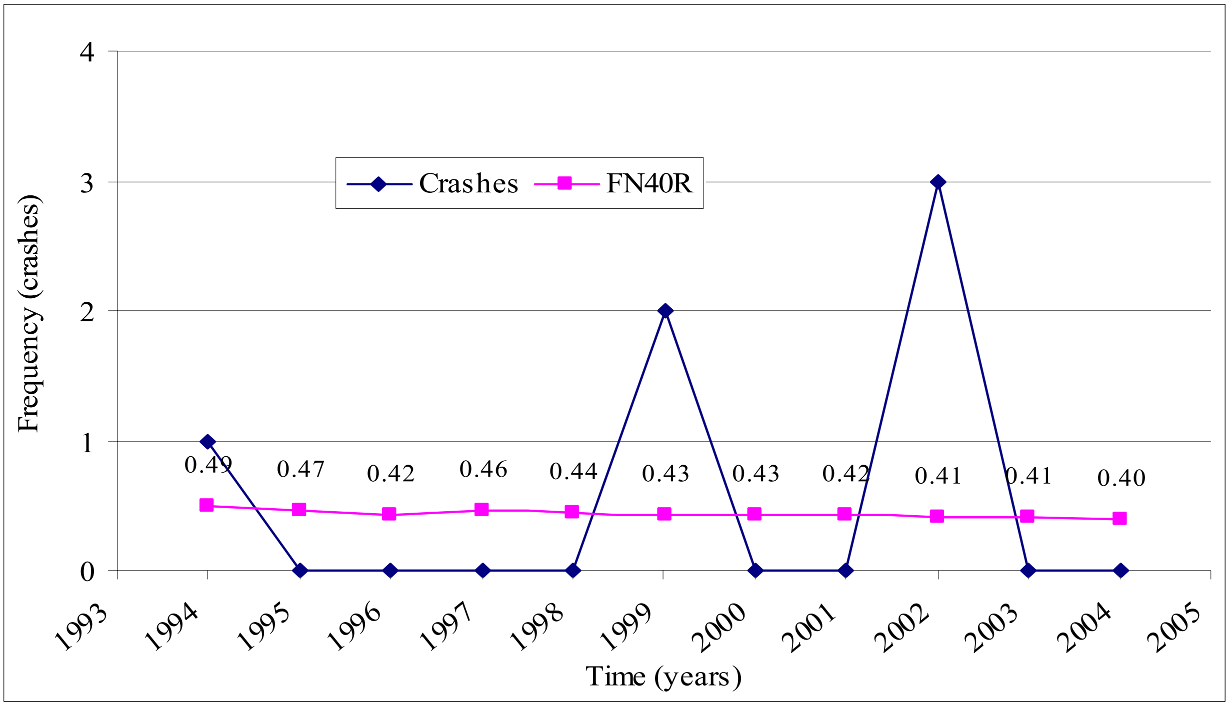


Figure 81 Wet Friction-Related Crashes for Site 3 EB (IH 94 Monroe)

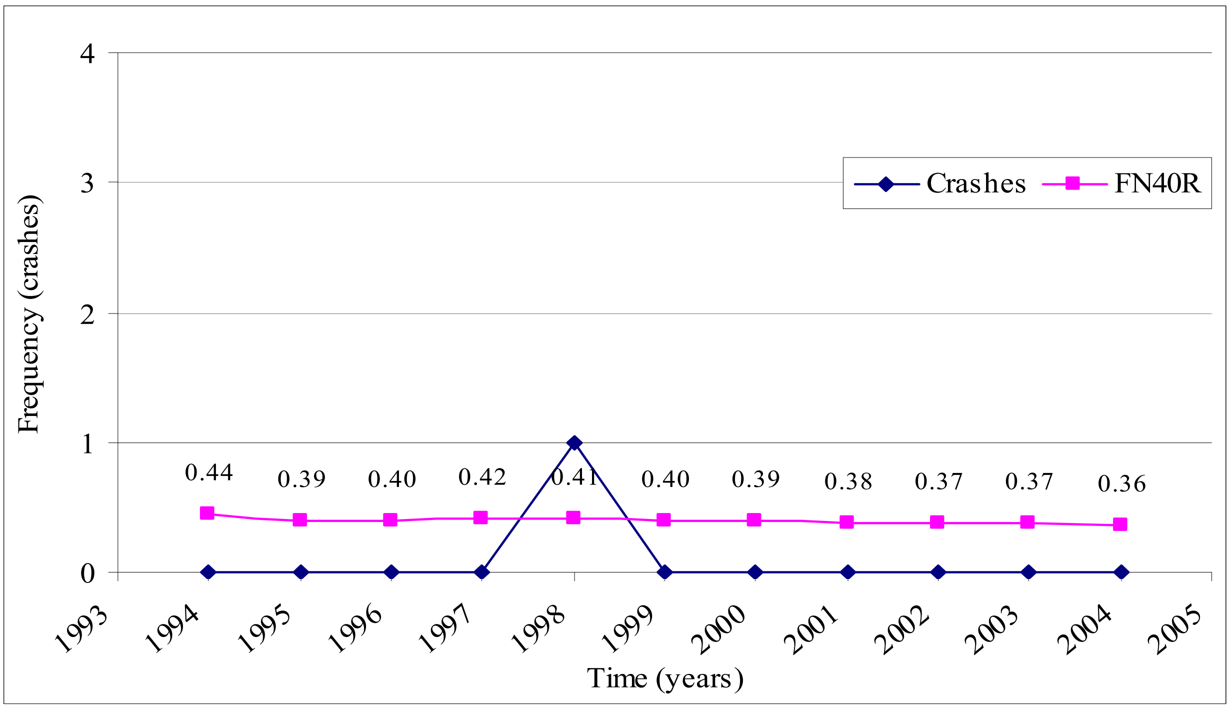


Figure 82 Wet Friction-Related Crashes for Site 4 WB (IH 94 Waukesha)

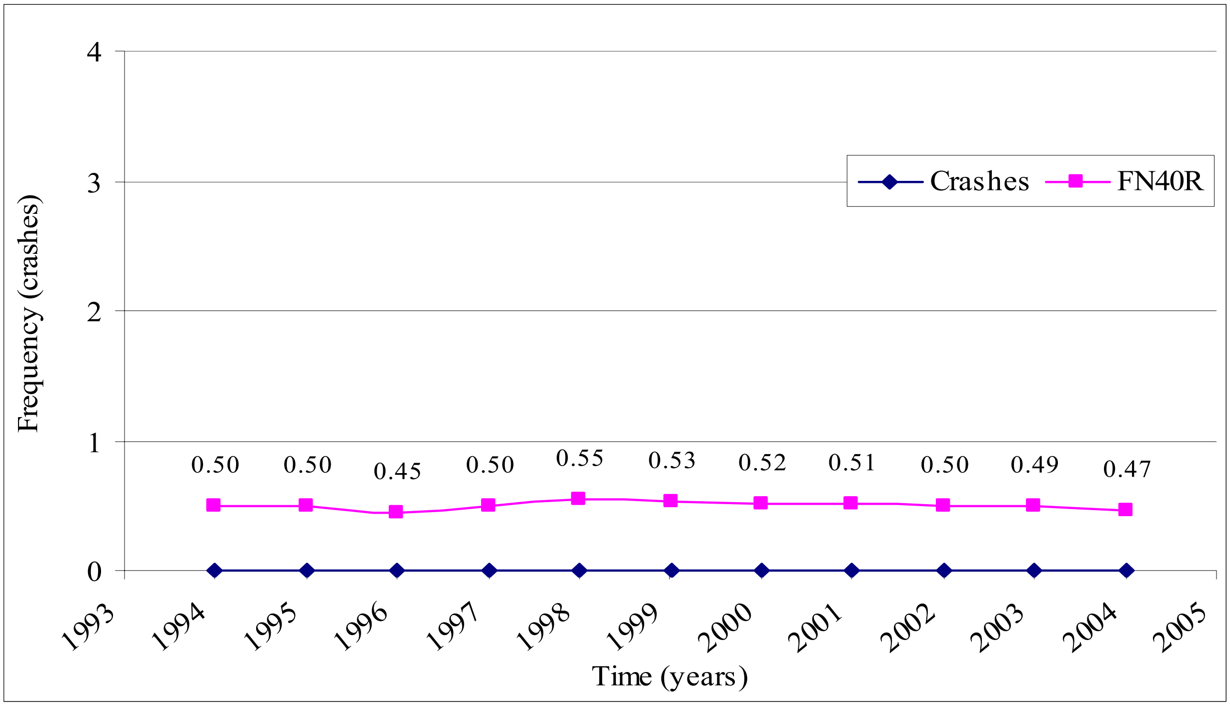


Figure 83 Wet Friction-Related Crashes for Site 5 WB (STH 21 Juneau)

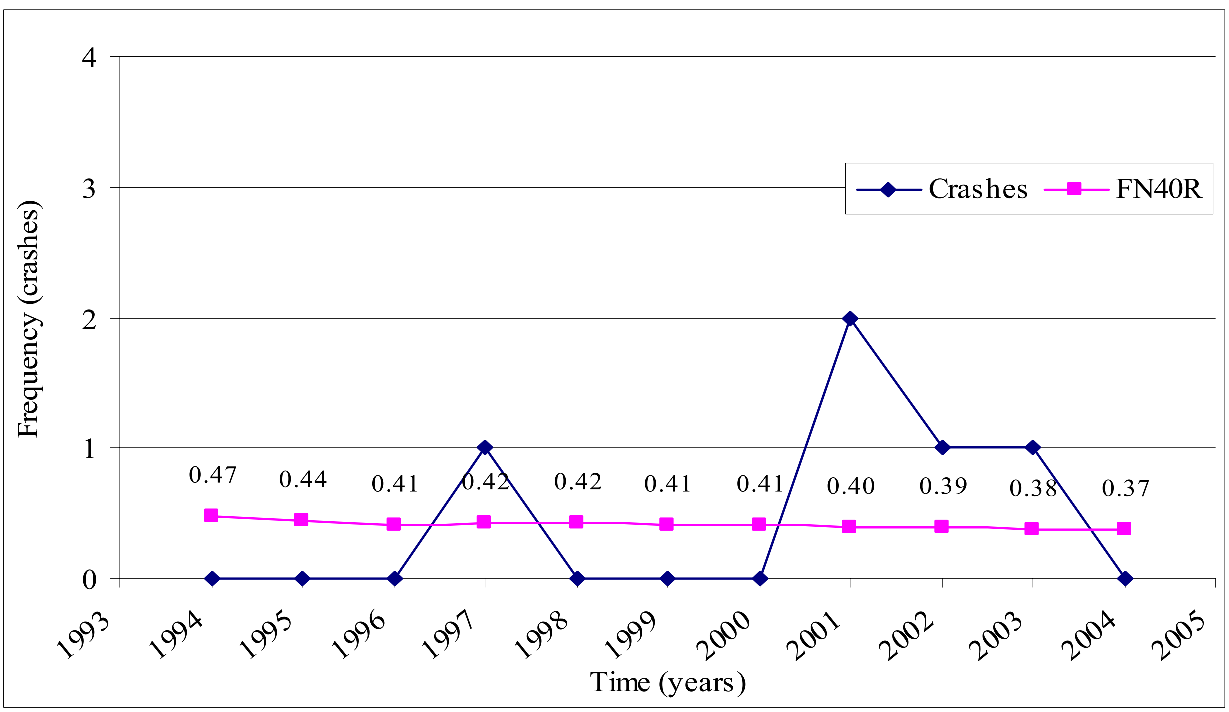


Figure 84 Wet Friction-Related Crashes for Site 6 NB (USH 151 Grant/Lafayette)

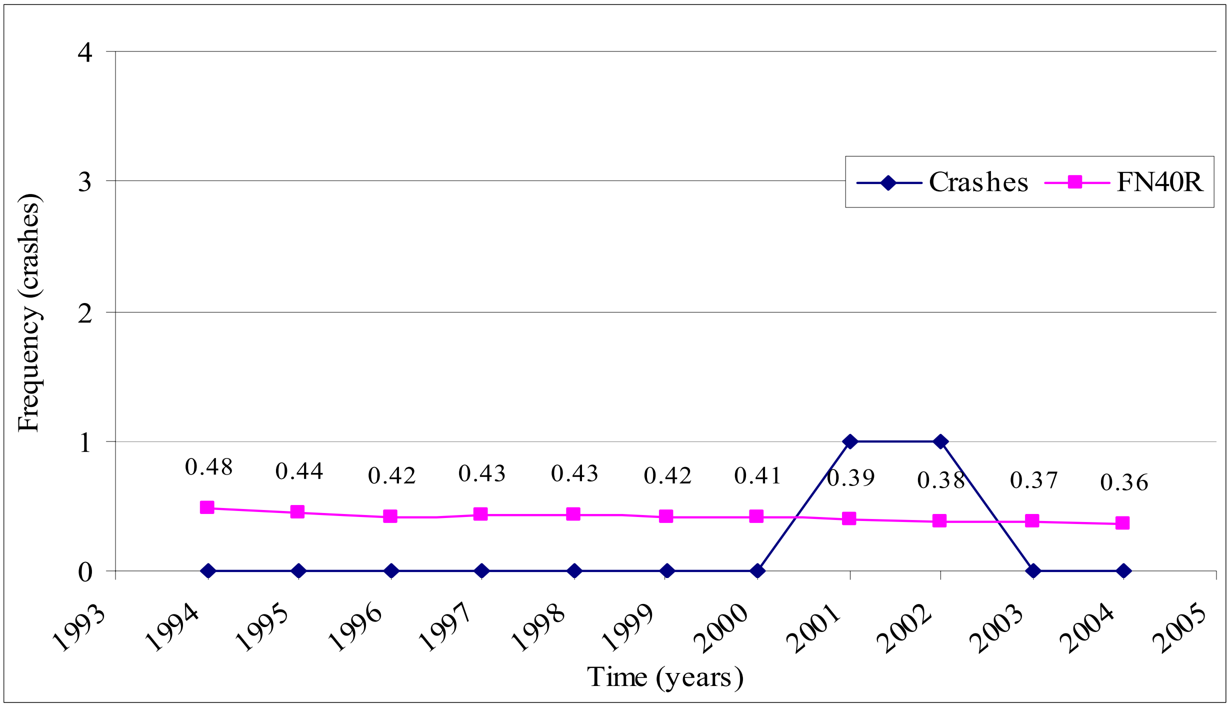


Figure 85 Wet Friction-Related Crashes for Site 6 SB (USH 151 Grant/Lafayette)

Although the frequency of crashes is quite low, there is some evidence to suggest that the crashes identified are happening in the latter years of the study time frame. Sites 2, 3, and 6 show an increase in wet pavement crashes after 1999. No trends of any significance were found at the other study sites. In fact, no wet pavement crashes were found at study site 5.

Statistical Analysis

Wet Weather Crashes

For analysis purposes, wet weather skid crashes were summed with respect to FN recorded during the year of the crash. This was done to account for the low frequency of crashes at each study site and the associated statistical insignificance. FNs for the period 1999 through 2003 represent interpolated values since no actual friction tests were performed in those years. Figure 86 shows wet weather skid crash frequency for FNs recorded during FN40R tests at each site. Note that friction numbers were rounded to the nearest integer. Statistical analysis results were obtained using a statistical software package.

As shown in Figure 86, no pattern is evident from the data points plotted. Consequently, no statistical correlation can be measured. Figure 86 also shows a possible outlier value that could be misleading during the analysis of the data. Removing the outlier value does not statistically improve the data set. The coefficient of determination (R-squared) for both conditions (with and without the outlier) is not high enough to prove a statistically significant correlation. R-squared values tend to approach zero. Figure 87 shows wet weather skid crashes against FN50R for all sites combined. Again, friction numbers were rounded to the nearest integer.

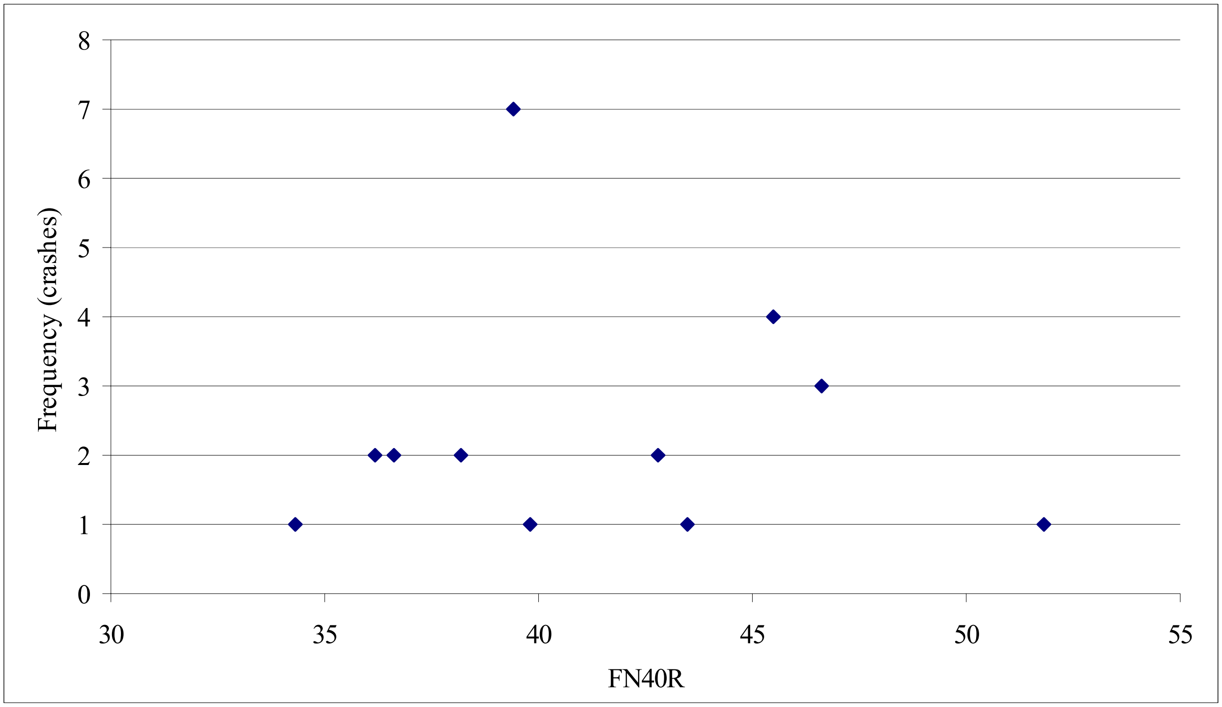


Figure 86 Wet Friction-Related Crashes Correlated to FN40R

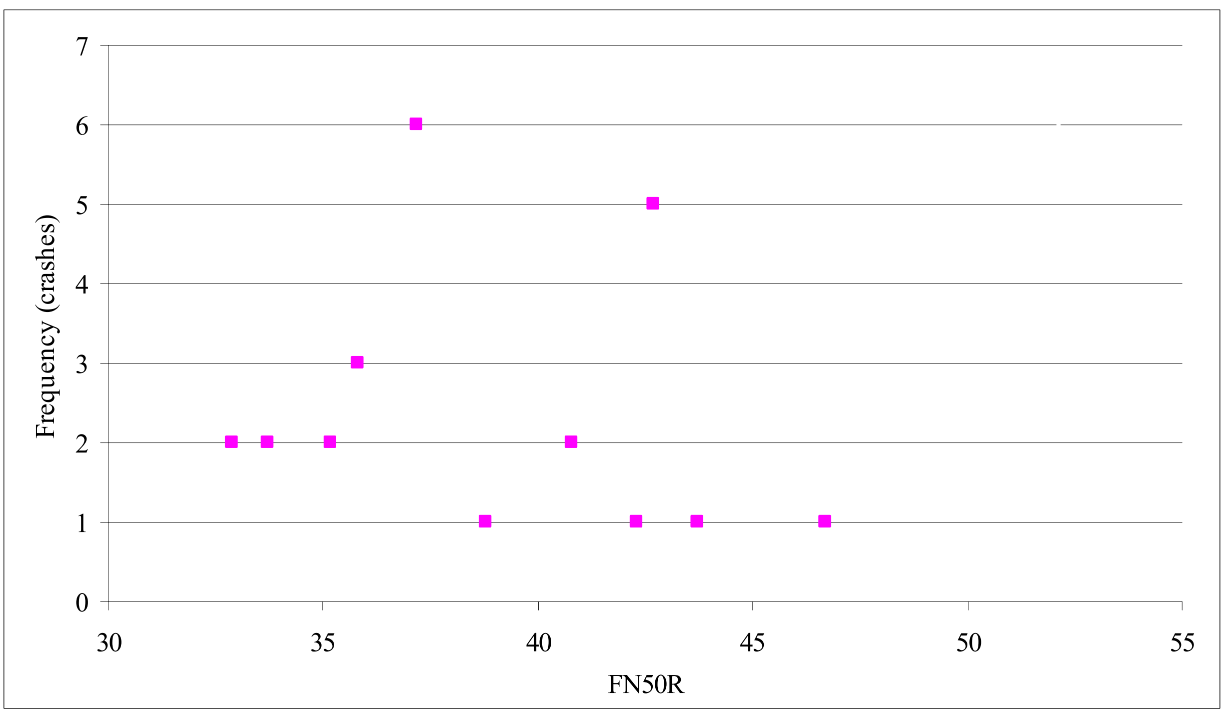


Figure 87 Wet Friction-Related Crashes Correlated to FN50R

Scattered data points shown in Figure 87 do not identify a trend in the data. Therefore, no statistical correlation can be measured. The R-squared for the data was such that no statistically significant correlation could be found. Figure 87 also shows two possible outlier values that could be misleading during the analysis of the data. If in fact they are outliers, removing them would improve the correlation between wet friction-related crashes and FN50R data. A regression equation for data without the outliers results in an inversely proportional relation between frequency and FN. For $\alpha = 0.05$, the p-values for each coefficient and the model as a whole would be statistically significant. The R-squared was 0.406. Therefore, less than half of the variation crashes can be explained by changes in skid friction values. Nevertheless, the current data set is very limited to prove this analysis to be valid.

CHAPTER VI

MIX DESIGN EVALUATION

This chapter covers results of the research conducted to study the effects of the surface texture of an asphalt pavement (at both the micro and macro level) on improving skid resistance. It also covers the subject of considering mix design variables such as aggregate type, gradation, and asphalt content in improving the skid resistance of asphalt mixtures.

TEXTURE AND FRICTION MEASUREMENT TESTING

Background

A review of literature related to skid resistance to date shows that there are no detailed guidelines for improving skid resistance in terms of asphalt mixture design. This is particularly true for the States in the Midwest region. Therefore, such guidelines are needed in asphalt mixture design. In addition, the existing models used by the various States to predict FN values need to be further developed or revised to include the micro- and macrotexture of asphalt mixtures used. Such models can then be used in the asphalt mixture design. For example, the current practice of WisDOT of using a prediction model for the FN values including merely the dolomite content and the aggregate LA wear in terms of materials properties is not sufficient because while it could relate to microtexture, it could not relate to the accurate contribution of surface texture (resulting form aggregate gradation) to skid resistance.

The hypothesis of this part of the study is that surface texture of asphalt pavement, micro- and macro-surfacing has significant effects on improving skids resistance in terms of traffic safety. These effects can be changed by adjusting the mixture design variables such as aggregate type, gradation, and asphalt content. It is expected that by considering effects of mixture design variables in estimation of skid resistance and introducing guidelines for including these effects, a better estimation of skid resistance can be realized.

Research Objectives

The main objectives of this part of the study were:

(1) To develop model of skid resistance values based on the macro-texture of asphalt mixtures after compaction; and
(2) To develop guidelines for improving skid resistance of asphalt pavements based on mixture design variables.

Laboratory Testing - Asphalt Mixture Design

In this study experimentation included varying asphalt mixture components, such as binder and aggregate types and gradations, percentage of air voids, and asphalt content and measuring effect of these variables on macrotexture. The sand patch method for measuring macrotexture (mean thickness depth: MTD) measured with the Volumetric Technique (ASTM E 965-96) was used in this study for compare the effects of various variables on pavement skid resistance. A significant effort was spent in developing a method for measuring the FN on gyratory compacted samples that is similar to the British Pendulum Tester (ASTM E 303-93). The efforts however were not successful and did not result in a repeatable method due to difficulty in generating sufficient surface area required for measurements.

In addition to the laboratory study, a new FN model which incorporates both micro- and macro-surfacing was evaluated through field experimentation. The field testing utilized a Locked Wheel Tester (ASTM E 274-77) in addition to the sand Volumetric Technique. Results obtained from field testing before and after application of an overlay on existing pavement was used to determine what, if any, improvements were realized in the skid resistance of the asphalt pavement.

Variations in microtexture and macrotexture were studied through the manipulation of selected variables in the asphalt mix design. Asphalt mixes are designed to meet specific volumetric and stability requirements through the manipulation of a number of variables. The controlled variables which could affect microtexture and macrotexture were the following:

- Maximum nominal aggregate size;
- Aggregate angularity;
- Percentage air voids; and
- Aggregate gradation type.

As presented in Chapter II, previous research has indicated that macrotexture is affected by the maximum nominal aggregate size, which is defined as the smallest sieve through which 100 percent of the aggregate sample will pass. The maximum nominal aggregate sizes used in this research were:

- Fine (12.5 mm);
- Coarse (19 mm);
- Stone Matrix Asphalts (SMA) (9.5 mm); and
- Porous (12.5 mm).

The aggregate types used in this study were limestone (a sedimentary rock) and gravel. These types were selected because they are commonly used and are both abundant in Wisconsin.

The percentage of air voids in an asphalt mix can significantly affect the performance of an asphalt pavement. Voids in dense-graded hot mix asphalt (HMA) (at optimum asphalt content) can drop from around eight percent at the early service life of a pavement to around three percent after significant traffic loading. The voids used in this study were between four and eight percent for dense-graded asphalt mixtures and for porous asphalt mixtures the voids were between 15 and 20 percent.

Aggregates are usually described as dense-graded, uniformly-graded (open), or gap-graded. For a dense-graded aggregate, the fine gradation (above the maximum density gradation) and coarse gradation (below the maximum density gradation) are commonly used. As previously stated, this study used four types of gradation (fine, coarse, porous, and stone matrix). This variety in gradations was used because previous studies have indicated that aggregate gradation can have significant effects on the pavement macrotexture.

After a consideration of the variables of interest, approximately 19 specimens were tested. To obtain optimum asphalt content and repeatability, a total of 114 (19 specimens x 6 samples) samples should were prepared. Table 57 presents the list of HMA mixes which were prepared and compacted in the laboratory.

Table 57 HMA Mixtures Design Plan

Gradation	Aggregate Type	Max. Nominal Size(mm)	Air voids (%)	Mix Type
Coarse	Gravel	19	4	CG-19-4
			8	CG-19-8
		12.5	4	CG-12.5-4
			8	CG-12.5-8
	Limestone	19	4	CL-19-4
			8	CL-19-8
		12.5	4	CL-12.5-4
			8	CL-12.5-8
Fine	Gravel	19	4	FG-19-4
			8	FG-19-8
		12.5	4	FG-12.5-4
			8	FG-12.5-8
	Limestone	19	4	FL-19-4
			8	FL-19-8
		12.5	4	FL-12.5-4
			8	FL-12.5-8
Stone matrix	Limestone	9.5	4	SL-9.5-4
			8	SL-9.5-8
Porous	Limestone	12.5	18	PL-12.5-18

Coarse Gradation

Binder Evaluation

A chemically unmodified asphalt binder type PG 58-28 provided by the Payne & Dolan Co. was selected as the best performing binder for this research. Generally, in order to evaluate binder performance, Superpave physical tests are conducted. These tests are described in Table 58.

Aggregate Evaluation

The type "K & N" aggregate provided from Payne and Dolan was selected as the best aggregate for this research because of historically good performance and availability. The "K & N" aggregate used consisted of 7/8 inch chip, 5/8 inch chip, 3/8 inch chip, ¼ inch MFGD sand, and washed natural sand. Four mix designs (Gravel-19 mm, Gravel-12.5 mm, Limestone-19 mm, & Limestone-12.5 mm) were selected. Aggregate properties and results of test conducted are listed in Table 59. Table 60 presents the applicable WisDOT Superpave mix requirements for the tests performed. As shown in Table 60, most tests performed satisfied the WisDOT Superpave mix requirements. For this research, a value of ten million equivalent single axle loads (ESALs) was assigned. For ten million ESALs, the WisDOT Superpave mix requirements indicate an E-30 or E-10 mix type can be selected. The E-30 specification requirement was applied for Gravel-19 mm and Gravel-12.5 mm, and the E-10 specification requirement was applied for Limestone-19 mm and Limestone-12.5 mm. An S-shape gradation shown in Figure 88 was used for the coarse blend, Gravel-19 mm. Restricted zones that might induce tender mixtures were avoided for both gradations.

Table 58 Superpave Asphalt Binder Testing Equipment and Purpose

Equipment	Purpose	Performance Parameter
Rolling Thin Film Oven (RTFO)	Simulate binder aging during HMA production and construction	Resistance to aging during construction
Pressure Aging Vessel (PAV)	Simulate binder aging during HMA service life	Resistance to aging during service life
Rotational Viscosity (RV)	Measure binder properties at high construction temperatures	Handling and pumping
Dynamic Shear Rheometer (DSR)	Measure binder properties at high and intermediate service temperatures	Resistance to permanent deformation and fatigue cracking
Bending Beam Rheometer (BBR)	Measure binder properties at low service temperatures	Resistance to thermal cracking
Direct Tension Tester (DTT)	Measure binder properties at low service temperatures	Resistance to thermal cracking

Table 59 Aggregate Properties and Test Result Measured

Aggregate properties and tests	**G-19mm (E-30)**
Bulk aggregate specific gravity (Gsb)	2.722
Max. theoretical specific gravity (Gmm)	2.582
Bulk specific gravity (Gmb)	2.479
Apparent aggregate specific gravity (Gsa)	2.804
Effective aggregate specific gravity(Gse)	2.768
Density (kg /m^3)	2479
Optimum asphalt content	5.25%
Fine aggregate angularity	45>**45 OK**
Elongated particles	1.9<**5 OK**
Dust to binder ratio	**0.6**<0.8<**1.2**
Sand equivalent	92.6>**45 Ok**
Crush	94.9/94.1 **98/90 Ok**
L.A wear	5.5(100)**<13 Ok,** 25.6(500)**<45 OK**
Soundness	2.4**<12 OK**
Voids in mineral aggregate (VMA)	**13%**<13.39% **OK**
Voids filled with binder (VFA)	**67<69.5<75 OK**

Table 60 WisDOT Superpave Mixture Requirement

Mixture Type	E-0.3	E-1	E-3	E-10	E-30	E-30x	SMA
ESALs x 10^6 (20 yr design life)	< 0.3	0.3 -< 1	1 -< 3	3 -< 10	10 -< 30	≥ 30	----
LA Wear (AASHTO T 96) 100 revolutions (max % loss) 500 revolutions (max % loss)	 13 50	 13 50	 13 45	 13 45	 13 45	 13 45	 13 45
Soundness (AASHTO T 104) (sodium sulfate, max % loss)	12	12	12	12	12	12	12
Freeze/Thaw (AASHTO T 103) (specified counties, max % loss)	18	18	18	18	18	18	18
Fractured Faces (ASTM 5821) (one face/2 face, % by count)	60/__	65/__	75/60	85/80	98/90	100/100	100/90
Thin or Elongated (ASTM D4791) (max %, by weight)	5 (5:1)	5 (5:1)	5 (5:1)	5 (5:1)	5 (5:1)	5 (5:1)	20 (3:1)
Fine Aggregate Angularity (AASHTO T304, method A, min)	40	40	43	45	45	45	45
Sand Equivalency (AASHTO T 176, min)	40	40	40	45	45	50	50
Gyratory Compaction Gyrations for N_{ini} Gyrations for N_{des} Gyrations for N_{max}	 6 40 60	 7 60 75	 7 75 115	 8 100 160	 8 100 160	 9 125 205	 8 100 160
Air Voids, $\%V_a$ ($\%G_{mm}$ @ N_{des})	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)
$\%G_{mm}$ @ N_{ini}	<91.5	<90.5	<89.0	<89.0	<89.0	<89.0	---
$\%G_{mm}$ @ N_{max}	≤98.0	≤98.0	≤98.0	≤98.0	≤98.0	≤98.0	---
Dust to Binder Ratio (% passing $0.075/P_{be}$)	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2	0.6-1.2	1.2-2.0
Voids filled with Binder (VFB or VFA, %)	70-80	65-78	65-75	65-75	65-75	65-75	70-80
Tensile Strength Ratio – TSR (ASTM 4867) No antistripping additive With antistripping additive	 0.70 0.75	 0.70 0.75	 0.70 0.75	 0.70 0.75	 0.70 0.75	 0.70 0.75	 0.70 0.75
Drainddown @ Production Temperature (%)	---	---	---	---	---	---	0.30

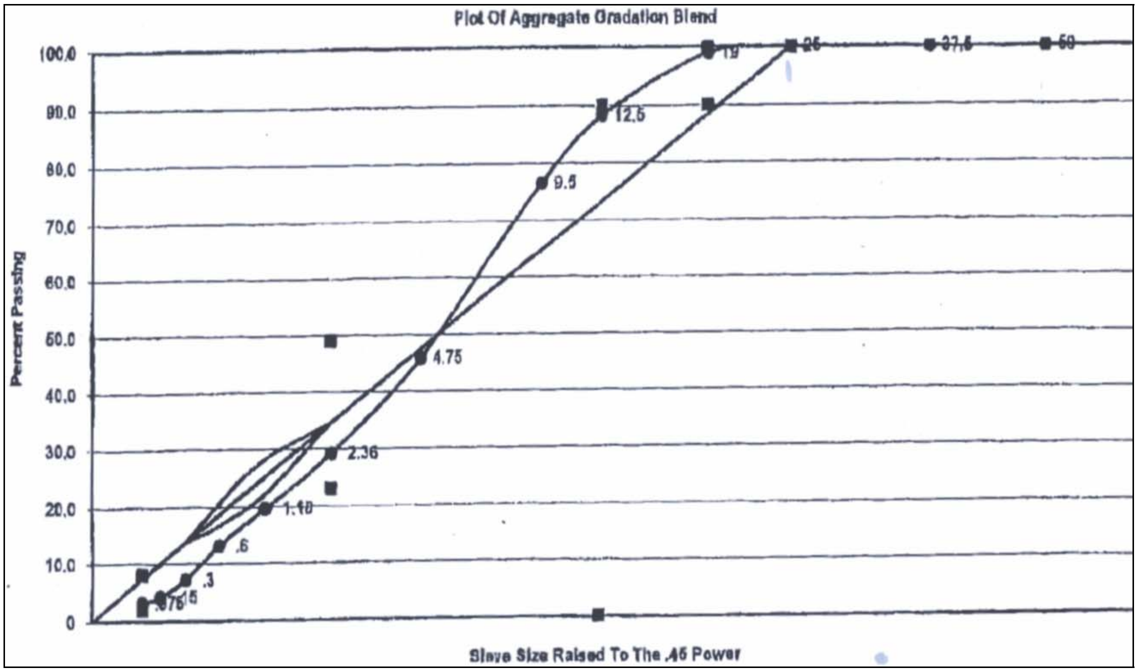


Figure 88 Aggregate Gradation for Gravel -19mm nominal size (Coarse)

Compaction Results

Estimated Asphalt Content

An estimated asphalt content of five percent was selected for the coarse blend mixture. Samples with asphalt contents ± 0.5 percent of the estimated optimum asphalt content selected were also chosen for volumetric analysis.

Mixing and Compaction Temperatures

A zero shear viscosity test was required to estimate the acceptable mixing and compaction temperature for an HMA pavement. Tests were conducted with a rotational viscometer and acceptable mixing and compaction temperatures can be estimated in terms of the Excel spreadsheet for Zero shear viscosity analysis. From these tests, the acceptable mixing and compaction temperatures estimated were 149ºC and 135ºC, respectively. These temperatures were then used as the mixing and compaction temperatures.

Densification Plots for Each Sample

Densification plots of 19 mm Gravel coarse mixture were used for optimum asphalt content and repeatability at asphalt content of 4.5, 5.0, and 5.5 percent.

Volumetric Analysis

Basic parameters of the mix geometry are listed in Table 61. Additionally, the diameter for the HMA sample puck was 150 mm and the density of water at 25 ºC was 0.997.

Table 61 Summary of Volumetric Parameters Measured

Mix type	Compact Effort	AC Content (%)	Mass of Puck (g)	Height (mm)	Volume (cm^3)
G-19mm (E-30) for Optimum AC	Nmax = 160 Gyr.	4.5	4871.4	113.5	2005.7
		5.0	4905	113.5	2005.7
		5.5	4913.4	112.9	1995.1
G-19mm (E-30) for Repeatability	Nmax = 160 Gyr.	4.0	4887.4	113.1	1998.6
		5.0	4871.9	112.6	1989.8
		5.5	4967.8	113.5	2005.7

Bulk specific gravity of aggregates (Gsb) is given in Table 62. Bulk specific gravity for HMA (Gmb) from mixtures and theoretical maximum specific gravity for HMA (Gmm) from asphalt rice was measured with the Core Lock machine. The correction factor (CF) was obtained in terms of theoretical Gmb at 160 gyrations divided by the actual Gmb (as measured with Core Lock).

With the CF computed, the actual Gmb (as varies by gyration number) was calculated. The graph of percent Gmm (which is a ratio of Gmm at Nmax and actual Gmb) is shown in Table 62. The criteria to satisfy the required percent air voids in mixes were:

- N_{ini} = 8 percent Gmm < 89 percent;
- N_{des} = 100 percent Gmm = 96 percent, and
- N_{max} = 160 percent Gmm = 98 percent.

Air void is the number that subtracts percent Gmm at N_{des} from 100 percent. The equations for percent VMA, percent VTM, and percent VFA are as follows:

$$VMA = 100 \times (1 - Gmb\,(1 - Pb) / Gsb); \tag{15}$$

where,

Pb = Asphalt content

$$VTM = 100 \times (1 - Gmb / Gmm); \text{ and} \tag{16}$$

$$VFA = (VMA - VTM) / VMA \times 100 \tag{17}$$

Table 62 Summary of Volumetric Properties Calculated

Mix Type	AC Content (%)	Gsb	Gmm	Gmb at Ndes	%Gmm at Ni	%Gmm at Nd	%Gmm at Nm	Air Void (%)	VMA	VTM	VFA
19mm CGO (E-30)	4.5	2.722	2.546	2.409	85.2	94.6	96.1	5.40	13.26	5.38	57.61
	5.0		2.538	2.423	85.5	95.5	97.0	4.50	13.37	4.53	65.62
	5.5		2.524	2.433	86.0	96.4	98.0	3.60	13.41	3.61	74.84
19mm CGR (E-30)	4.5	2.722	2.555	2.412	85.0	94.4	95.9	5.60	13.27	5.60	56.69
	5.0		2.534	2.410	85.3	95.1	96.7	4.90	13.36	4.89	66.71
	5.5		2.524	2.435	86.5	96.5	98.0	3.50	13.42	3.53	75.11

Determination of Optimum AC & Mix Properties (WisDOT Specification)
An asphalt content which allows for four percent air voids was optimal per WisDOT specifications. Using this value, the optimal asphalt content can be determined by plotting the relationship between percent air voids and asphalt content. For this research, optimal asphalt content was 5.25 percent as shown in Figure 89.

As shown in Figure 89, percent Gmm at N_{ini} and percent Gmm at N_{max} satisfy WisDOT Superpave requirements from Table 63. Also a VMA of 13.39 percent and a VFA of 69.5 percent satisfy, respectively, the VMA requirement from Table 63, and the VFA requirement from Table 60. Since the measured values satisfy WisDOT requirements, repeatability of the experiment was not an issue.

Mix Design Final Results

Mix design results from laboratory compacted specimens are summarized in Table 64.

Table 63 VMA Requirements

Nominal Maximum Aggregate Size (mm)	Percent minimum VMA
9.5	15.0
12.5	14.0
19.0	**13.0**
25.0	12.0
37.5	11.0

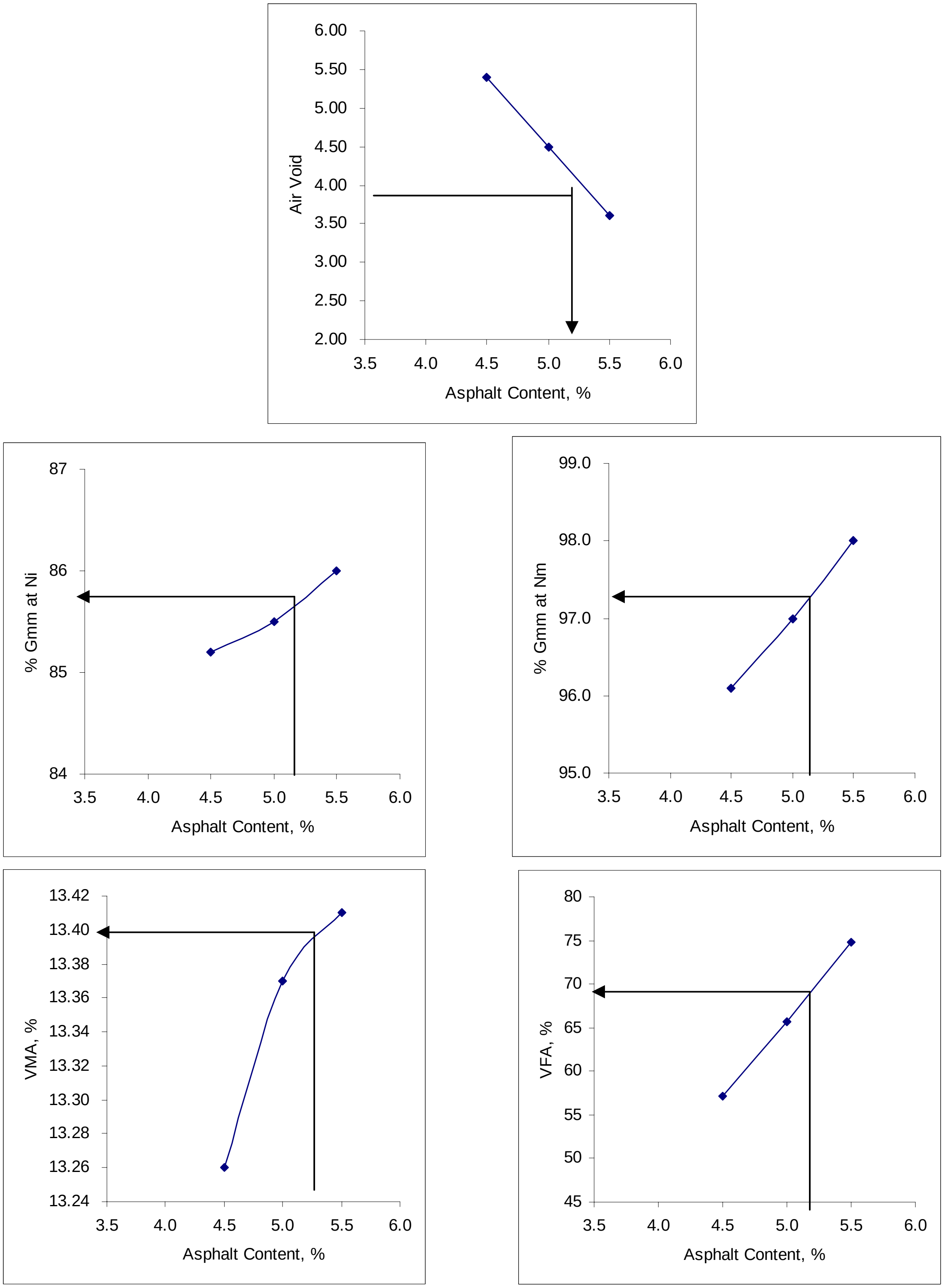


Figure 89 G-19 mm Mix Properties versus Asphalt Content (for optimum)

Table 64 Mix Design Results

Mix Type	Optimum AC Content (%)	Gsb	Gmm	Gmb	%Air Void	%VMA	%VFA
CG-19-4	5.25	2.722	2.582	2.479	4	13.39	69.5
CG-19-8	5.25	2.722	2.586	2.471	8	13.21	68.3
CG-12.5-4	5.4	2.725	2.557	2.455	4	14.51	72.4
CG-12.5-8	5.4	2.725	2.563	2.449	8	14.15	71.5
CL-19-4	4.7	2.751	2.579	2.513	4	13.72	70.8
CL-19-8	4.7	2.751	2.581	2.508	8	13.25	69.3
CL-12.5-4	4.8	2.746	2.575	2.473	4	14.62	72.6
CL-12.5-8	4.8	2.746	2.578	2.468	8	14.34	71.7
FG-19-4	4.6	2.692	2.541	2.439	4	13.91	71.2
FG-19-8	4.6	2.692	2.545	2.428	8	13.75	70.6
FG-12.5-4	4.8	2.693	2.532	2.424	4	14.81	73.1
FG-12.5-8	4.8	2.693	2.536	2.419	8	14.75	72.5
FL-19-4	4.4	2.697	2.531	2.438	4	14.21	71.8
FL-19-8	4.4	2.697	2.535	2.434	8	14.05	71.1
FL-12.5-4	4.8	2.708	2.531	2.435	4	15.17	73.3
FL-12.5-8	4.8	2.708	2.532	2.431	8	15.01	72.8
SL-9.5-4	6.3	2.748	2.536	2.429	4	16.83	76.9
SL-9.5-8	6.3	2.748	2.539	2.423	8	16.51	75.5
PL-12.5-18	4.0	2.579	2.51	2.078	18	22.6	23.5

Gradation	Agg. Type	Max. Nominal Size	Air voids
Coarse: C	Gravel: G	19mm	4%
Fine F	Limestone: L	12.5mm	8%
Stone Matrix: S		9.5mm	18%
Porous: P			

Macrotexture Measurement

Sand Patch Method

Traditionally, macrotexture measurements have been made using a volumetric test known as the Sand Patch Test. As presented in Chapter II, the Sand Patch Test is used by many transportation departments in the U.S. and is a relatively simply test to perform. However, the results from this test are dependent upon the individual performing the test. The Sand Patch Test is performed using a known volume of material and a spreading tool. The material traditionally used for this test is Ottawa sand, passing the number 50 sieve and retained on the number 100 sieve. Currently, the specifications in ASTM E965 recommend the use of glass spheres because of the consistency of the particle shapes and the commercial availability of the spheres. However, the test is still traditionally performed using the Ottawa sand. Results from tests performed using Ottawa sand are adequate as long as the sand meets the required specifications.

Procedure for this test consists of placing a known volume of material on the pavement surface and spreading it using a circular motion until the sand is dispersed around the voids in the pavement surface. The diameter of the area covered with the sand is measured and then used to calculate the mean texture depth (MTD) of the pavement macrotexture. The MTD is calculated from the test results using the following equation (ASTM E965):

$$MTD = 4V/\pi D^2 \tag{18}$$

where,

MTD = Mean texture depth of the pavement macrotexture
V = Volume of the sample material used
D = Average diameter of the area covered by the material

ASTM defines the mean texture depth as the average depth of the pavement macrotexture when determined using the Sand Patch Method. The term "mean texture depth" should only be applied when the test is performed to ASTM E965 specifications.

MTD Measurement

The macrotexture depths for the 19 specimens compacted in the lab were measured by the sand patch test. The known volume of Ottawa sand was placed on the surface of a compacted specimen and spread via circular motion until the voids in the surface of specimen were filled with sand. The diameter of the area covered with the sand was measured and four equally-spaced diameter measurements were taken by per the specifications of ASTM E 965. The final MTD value was calculated by the above MTD equation using the computed average diameter. Figure 90 shows four different specimens on which the sand patch test was conducted.

Results of the sand patch test for all 19 specimens, including MTD values, are listed in Table 65.

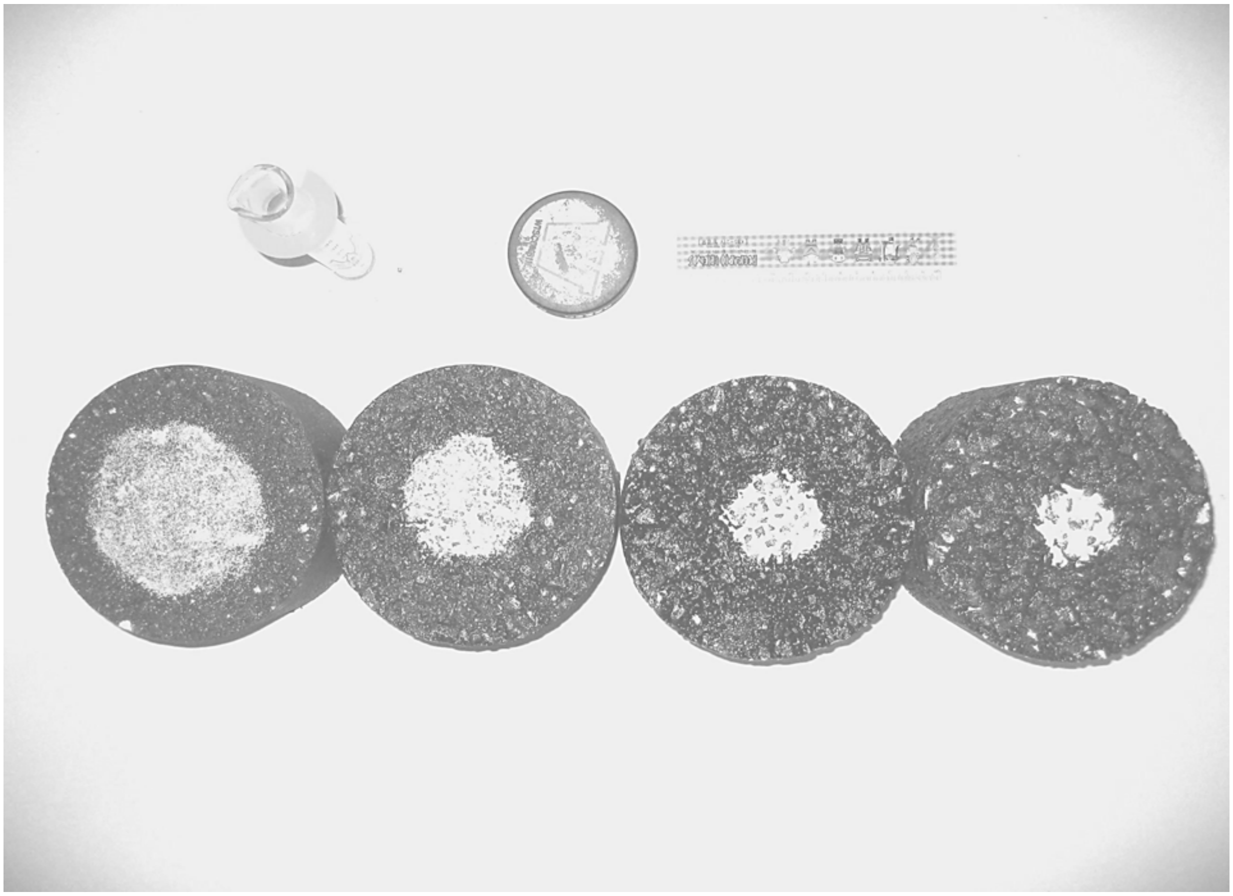


Figure 90 Sand Patch Test (Fine, Coarse, SMA, and Porous Mixture by Order)

Table 65 MTD Measurement Results

Mix Type	Sand Patch Diameter(cm)				Average Diameter (cm)	Volume (cm^3)	MTD (mm)
CG-19-4	8.0	8.3	8.5	8.1	8.2	4	0.757
CG-19-8	7.5	7.4	7.1	7.2	7.3	4	0.956
CG-12.5-4	9.0	9.1	8.9	9.1	9.03	3	0.468
CG-12.5-8	8.0	7.9	7.8	8.1	7.95	3	0.604
CL-19-4	9.5	9.2	9.6	9.4	9.43	3	0.615
CL-19-8	8.5	8.2	8.5	8.4	8.4	3	0.751
CL-12.5-4	9.1	9.3	9.2	9.3	9.23	3	0.448
CL-12.5-8	8.1	8.0	8.2	8.3	8.15	3	0.575
FG-19-4	10.0	10.1	10.3	10.0	10.1	2	0.249
FG-19-8	8.5	8.3	8.1	8.3	8.3	2	0.369
FG-12.5-4	10.3	10.5	10.3	10.6	10.43	2	0.234
FG-12.5-8	8.7	8.5	8.4	8.7	8.58	2	0.346
FL-19-4	10.4	10.4	10.3	10.7	10.45	2	0.233
FL-19-8	9.7	9.8	9.6	9.9	9.75	2	0.268
FL-12.5-4	11.5	11.3	11.2	11.0	11.25	2	0.221
FL-12.5-8	10.0	10.8	10.3	10.6	10.43	2	0.234
SL-9.5-4	9.3	9.5	9.2	9.4	9.35	10	1.456
SL-9.5-8	9.2	9.2	9.1	9.1	9.15	10	1.521
PL-12.5-18	9.5	9.8	9.4	9.5	9.55	15	2.094

Data Analysis and Results

Mix Property Influences on MTD
Mix properties used in this analysis were:

- Gradation type: Coarse, Fine, SMA, Porous;
- Aggregate type: Gravel, Limestone;
- Maximum nominal aggregate size: 19 mm, 12.5 mm, 9.5 mm; and
- Percent air voids: 4, 8, and 18 percent.

These properties were used to aid in determining the contribution of HMA mix properties to the frictional properties of the wearing surface. While Table 66 shows MTD values for each of mix types (in terms of mix properties included in the analysis), Figure 91 graphically illustrates the MTD difference according to mix types.

Figure 92 shows that MTD values vary by gradation type. It appears, generally, that the MTD value for the porous mixture was the largest. For the fine, coarse, and SMA mixtures, the MTD values for them increase respectively.

Figure 92 and Table 66 both indicate that there is a measurable difference between MTD values due to different aggregate types for a given HMA mixture type, when the gradation type, maximum nominal aggregate size, and percent air voids are held constant. MTD values for mixes with gravel aggregate were found to be greater than the MTD values for mixes with limestone aggregate.

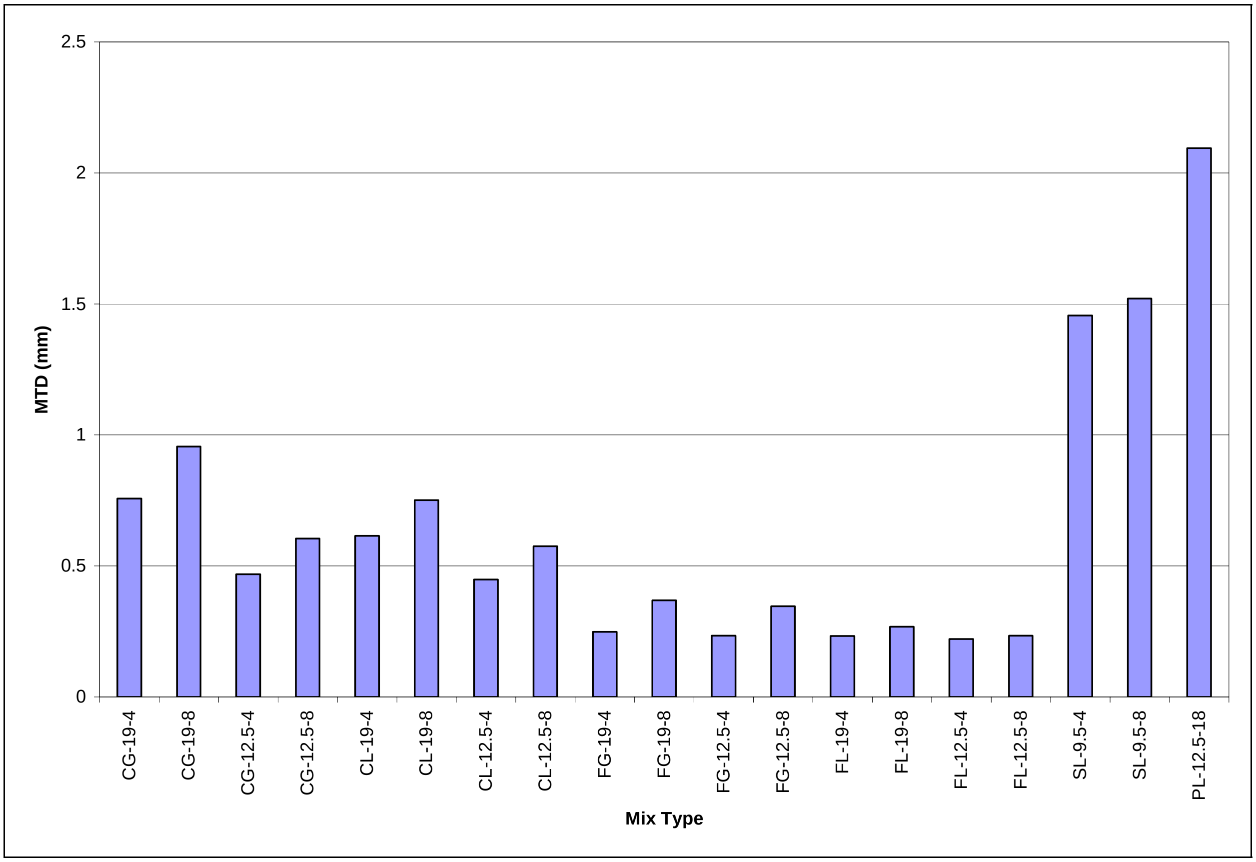


Figure 91 Mix Type- MTD

Table 66 Mix Type-MTD According to Aggregate Type

No	Mix Type	MTD	Mix Type	MTD
1	CG-19-4	0.757	CL-19-4	0.615
2	CG-19-8	0.956	CL-19-8	0.751
3	CG-12.5-4	0.468	CL-12.5-4	0.448
4	CG-12.5-8	0.604	CL-12.5-8	0.575
5	FG-19-4	0.249	FL-19-4	0.233
6	FG-19-8	0.369	FL-19-8	0.268
7	FG-12.5-4	0.234	FL-12.5-4	0.221
8	FG-12.5-8	0.346	FL-12.5-8	0.234

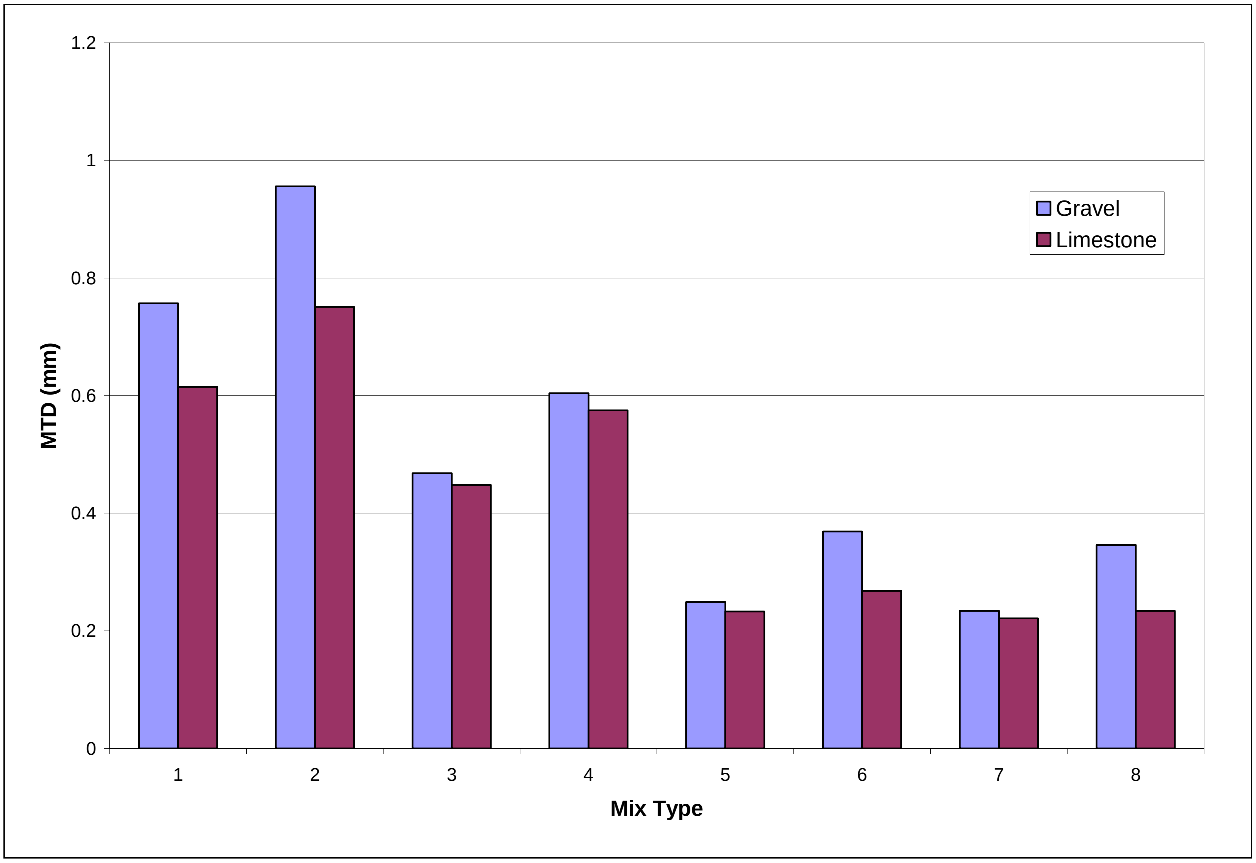


Figure 92 Mix Type–MTD according to Aggregate Type

Figure 93 and Table 67 both indicate that there is a measurable difference between MTD values due to different maximum nominal aggregate types for a given HMA mixture type, when the gradation type, maximum nominal aggregate size, and percent air voids are held constant. MTD values for mixes with a larger maximum nominal aggregate size are greater than the MTD values for mixes with a smaller maximum nominal aggregate size. MTD values are also greater for coarse mixes than for fine mixes. Figure 94 and Table 68 both indicate that MTD values for mixes with a higher percent air voids are greater than the MTD values for mixes with a lower percent air voids.

Table 67 Mix Type-MTD According to Maximum Nominal Aggregate Size

No	Mix Type	MTD	Mix Type	MTD
1	CG-19-4	0.757	CG-12.5-4	0.468
2	CG-19-8	0.956	CG-12.5-8	0.604
3	CL-19-4	0.615	CL-12.5-4	0.448
4	CL-19-8	0.751	CL-12.5-8	0.575
5	FG-19-4	0.249	FG-12.5-4	0.234
6	FG-19-8	0.369	FG-12.5-8	0.346
7	FL-19-4	0.233	FL-12.5-4	0.221
8	FL-19-8	0.268	FL-12.5-8	0.234

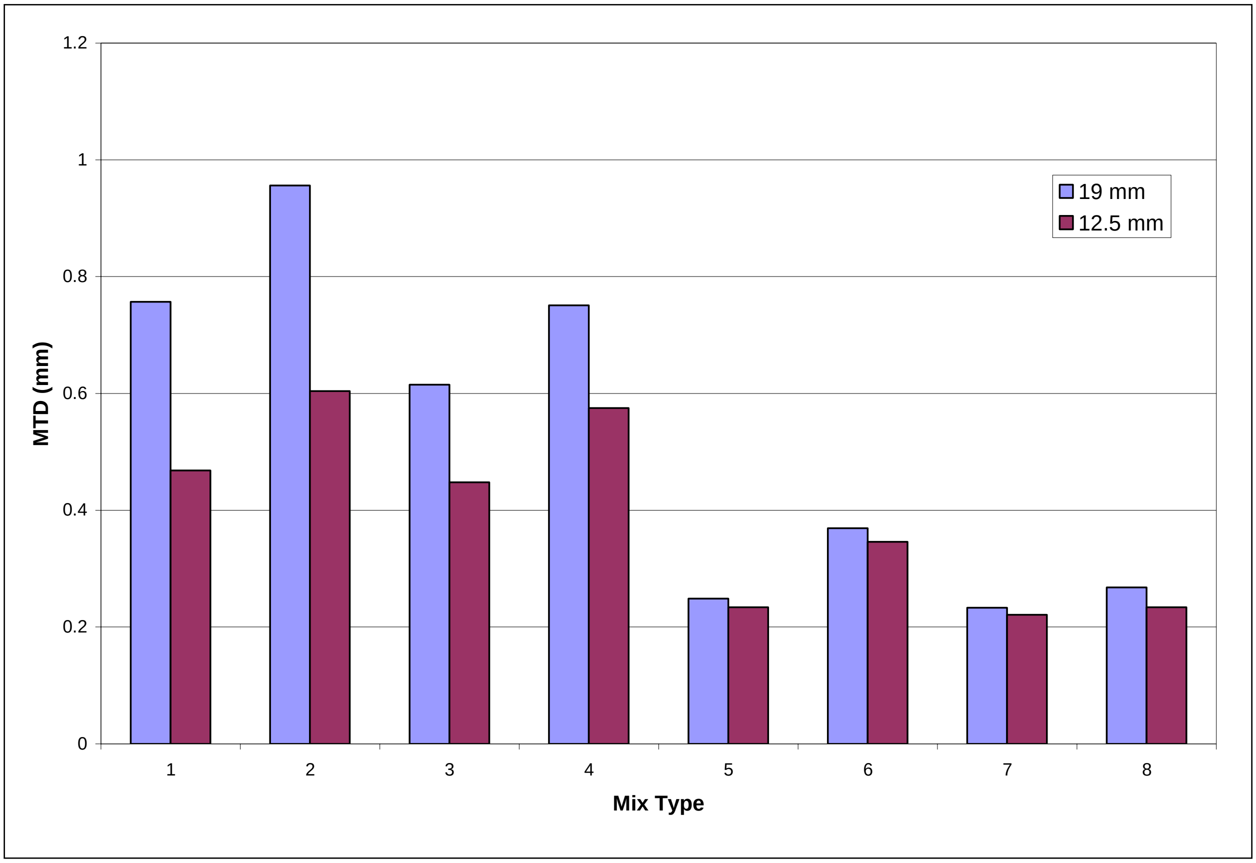


Figure 93 Mix Type–MTD according to Maximum Nominal Aggregate Size

Table 68 Mix Type-MTD According to Air Voids

No	Mix Type	MTD	Mix Type	MTD
1	CG-19-4	0.757	CG-19-8	0.956
2	CG-12.5-4	0.468	CG-12.5-8	0.604
3	FG-19-4	0.249	FG-19-8	0.369
4	FG-12.5-4	0.234	FG-12.5-8	0.346
5	CL-19-4	0.615	CL-19-8	0.751
6	CL-12.5-4	0.448	CL-12.5-8	0.575
7	FL-19-4	0.233	FL-19-8	0.268
8	FL-12.5-4	0.221	FL-12.5-8	0.234

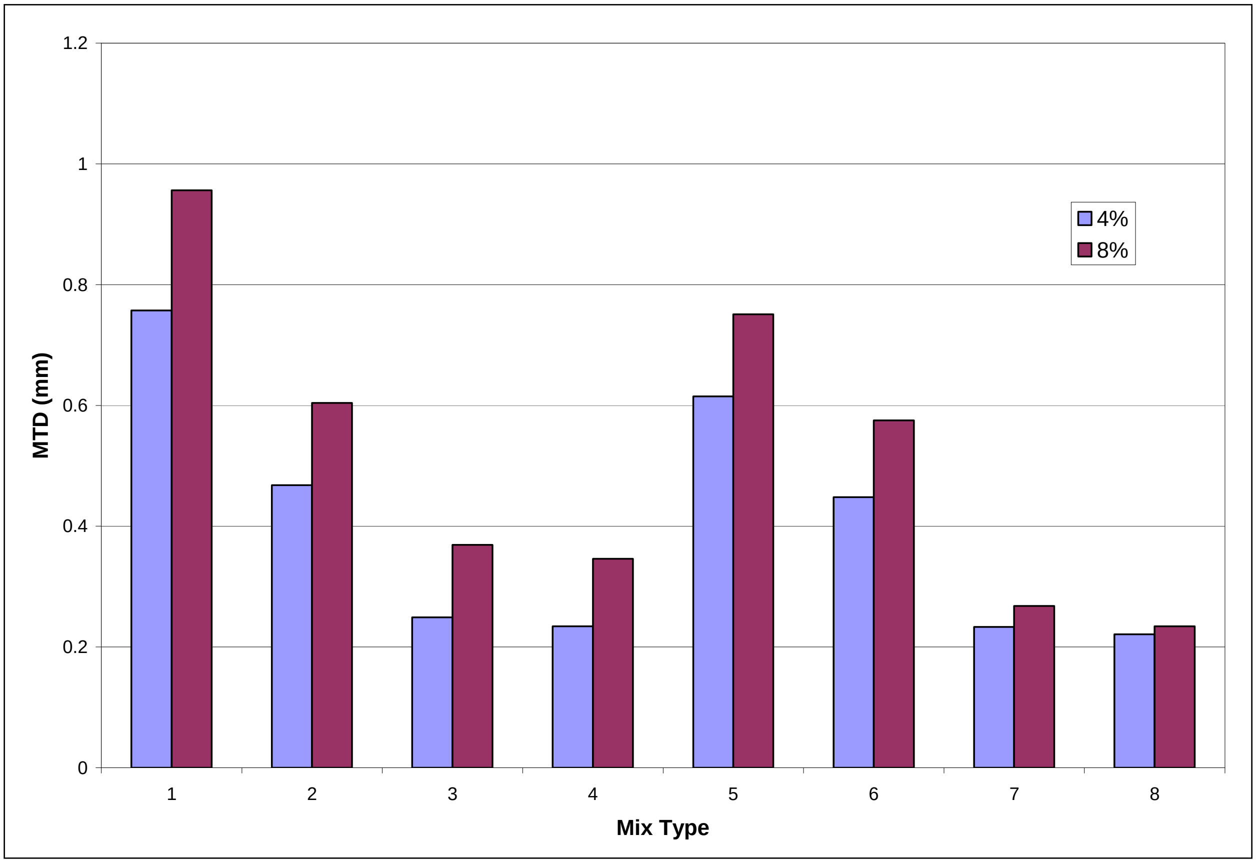


Figure 94 Mix Type–MTD according to Air Voids

Relationship between MTD and Mix Properties

A statistical software program was used to study the influence of specific mix properties on the MTD of the pavement surface. The stepwise regression function was used to establish relevant mix properties for each variable. The variables used in the analysis were:

- Percent passing rate No. 4 sieve, No.8 sieve, (No.4 – No.50) sieve, and (9.5-No.8) sieve;
- Percent passing rate obtained from gradation types (Coarse, Fine, SMA, and Porous mixture);
- Aggregate type: Gravel, Limestone;
- Maximum nominal aggregate size: 19, 12.5, 9.5 mm;
- Percent air voids: 4, 8, 18 percent;
- Fine aggregate angularity; and
- Elongation.

Table 69 shows a list of variable values and MTD according to mixture types.

The MTD values obtained by the sand patch test were analyzed according to mix properties of the pavement to determine which properties had the largest effect on MTD. This analysis produced an MTD regression model:

$$\begin{aligned} MTD = & -85.2 + 1.91*PP\#4 - 0.187*PP\#8 - 1.57*PP(\#4\text{-}\#50) \\ & + 0.821*PP(9.5\text{-}\#8) + 5.44*AT + 1.36*MNAS \\ & + 0.0262*AV - 2.61*AT*PP\#4 + 0.747*AT*PP\#8 \\ & + 2.47*AT*PP(\#4\text{-}\#50) \end{aligned} \tag{19}$$

where,

PP#4: % passing rate No. 4 sieve
PP#8: % passing rate No. 8 sieve
PP9.5: % passing rate 9.5mm sieve
PP#50: % passing rate No. 50 sieve
AT: Aggregate Type
AV: Air Voids
MNAS: Max. Nominal Aggregate size

The regression coefficient for this equation was 0.997, indicating an excellent fit. Table 70 shows the ANOVA table for analysis performed for the MTD. Since the aggregate types (Gravel: 0, Limestone: 1) are the categorized variables, the interactions between aggregate types and other variables were considered. It is expected that the maximum nominal aggregate size would have the largest influence on MTD because of the larger aggregate protruding from the pavement surface.

Table 69 Variables Values and MTD According to Mix Type

Mix Type	%P No.4	%P No.8	%P No.4 - %P No.50	%P 9.5 -%PNo.8	Agg. Type	Max. Nominal Size	Air voids (%)	Fine Aggregate Angularity	Elong.	MTD
CG-19-4	46.8	29.1	39.8	47.4	0	19.0	4	45.0	98.1	0.757
CG-19-8	46.8	29.1	39.8	47.4	0	19.0	8	45.0	98.1	0.956
CG-12.5-4	61.9	38.9	53.2	50.6	0	12.5	4	45.5	98.9	0.468
CG-12.5-8	61.9	38.9	53.2	50.6	0	12.5	8	45.5	98.9	0.604
CL-19-4	52.0	34.5	45.3	37.7	1	19.0	4	46.3	96.3	0.615
CL-19-8	52.0	34.5	45.3	37.7	1	19.0	8	46.3	96.3	0.751
CL-12.5-4	58.9	39.5	50.9	44.6	1	12.5	4	45.8	97.7	0.448
CL-12.5-8	58.9	39.5	50.9	44.6	1	12.5	8	45.8	97.7	0.575
FG-19-4	62.8	46.4	50.6	34.2	0	19.0	4	43.7	98.8	0.249
FG-19-8	62.8	46.4	50.6	34.2	0	19.0	8	43.7	98.8	0.369
FG-12.5-4	69.0	52.1	53.3	37.0	0	12.5	4	43.0	98.8	0.234
FG-12.5-8	69.0	52.1	53.3	37.0	0	12.5	8	43.0	98.8	0.346
FL-19-4	58.7	48.2	46.7	32.0	1	19.0	4	41.5	96.8	0.233
FL-19-8	58.7	48.2	46.7	32.0	1	19.0	8	41.5	96.8	0.268
FL-12.5-4	68.7	52.5	56.7	37.4	1	12.5	4	43.0	97.8	0.221
FL-12.5-8	68.7	52.5	56.7	37.4	1	12.5	8	43.0	97.8	0.234
SL-9.5-4	40.2	18.0	26.6	76.0	1	9.5	4	47.7	93.1	1.456
SL-9.5-8	40.2	18.0	26.6	76.0	1	9.5	8	47.7	93.1	1.521
PL-12.5-18	27.1	16.3	21.1	67.4	1	12.5	18	39.9	98.2	2.094

* Gravel: 0, Limestone: 1

Table 70 ANOVA Table for MTD

Source	DF	ANOVA SS	Mean Square	F Value	P value
Model	10	4.8604	0.48604	293.68	0.000
Constant					0.004
PP#4					0.004
PP#8					0.005
PP(#4-#50)					0.004
PP(9.5-#8)					0.004
AT					0.004
MNAS					0.004
AV					0.001
AT*PP#4					0.004
AT*PP#8					0.004
AT*PP(#4-#50)					0.004
Residual Error	8	0.01324	0.00165		
Corrected Total	18	4.87361			
Root Mean Square Error	0.04068				
R^2	0.997		**R^2 (Adjusted)**	0.994	

The regression analysis indicated that many variables influenced each of the different study variables. Although not all mix property variables fit the MTD regression equation with a high regression coefficient, many of them are still relevant to the characteristics of the mixes and the overall impact on MTD. The established regression equation may be improved by incorporating a wider variety of mix properties into the analysis. This study does, however, show a strong relationship between the MTD and the study variables, which can therefore be considered directly influential on the pavement surface texture.

FIELD TEST

As presented in Chapter III, field testing on selected roadways was performed to also include the analysis of surface characteristics of different HMA mixes.

Tested Wisconsin Roadways

A total of six test sites were selected on roadways in Wisconsin. A review of these roadways is presented in Tables 71 and 72. Each roadway selected had various subsections where different asphalt mix designs were present. The pavement type used in each subsection is repeated in Table 73. Mix design information for each of these subsections was acquired from WisDOT design records.

Table 71 Selected Roads for Study

Site	Hwy	County	Location	Pave. Year	Hwy. Type	Pave. Type	Sec. Lgth. (mi)	Pave. Subsec.	Subsec. Lgth. (mi)
1	IH 43	Walworth	USH 12 to STH 120	1993	Four-Lane Divided	3 - AC/RB	7.3	1 F1	1.06
								1 F2	1.03
								1 P1	1.05
								1 P2	1.08
								1 CT	0.79
								1 HV	0.36
								1 E2	1.02
2	IH 43	Waukesha	STH 164 to CTH Y	1992	Four-Lane Divided	3 - AC/RB	3.2	2 CT	0.55
								2 SH	0.48
								2 F1	0.60
								2 F2	0.57
								2 P1	0.57
								2 SX	0.38
								2 P2	0.57
								2 E2	0.57
								2 E1	0.55
3	IH 94	Monroe	CTH E to STH 21	1992	Four-Lane Divided	3 - AC/RB	5.2	3 SH	0.55
								3 SX	0.33
								3 WI	1.15
								3 SS	1.00
								3 SF	0.99
								3 SP	0.91
4	IH 94	Waukesha	STH 67 to CTH BB	1991	Four-Lane Divided	3 - AC/RB	1.8	4 SM	0.79
								4 SD	0.52
								4 SA	0.46
5	STH 21	Juneau	7th Ave to 2nd Ave	1994	Two-Lane Undivided	1 - AC/FB	5.3	5 CT	0.75
								5 E1	0.77
								5 E2	0.74
								5 P2	0.76
								5 P1	0.73
								5 F2	0.77
								5 F1	0.77

Table 72 Selected Roads for Study (Con't)

Site	Hwy	County	Location	Pave. Year	Hwy. Type	Pave. Type	Sec. Lgth. (mi)	Pave. Subsec.	Subsec. Lgth. (mi)
6	USH 151	Grant/ Lafayette	STH 126 to East Side Rd	1993	Two-Lane Undivided	3 - AC/RB	5.7	6 E2	0.80
								6 E1	0.64
								6 P2	0.87
								6 P1	0.79
								6 F2	0.88
								6 F1	1.17
								6 CT	0.51

Notes:
1. IH: Interstate Highway, STH: State Trunk Highway, USH: United States Highway
2. TYPE 1 - AC/FB (Asphaltic Concrete pavement over Flexible Base)
   TYPE 2 - ARM (Asphaltic Road Mix)
   TYPE 3 - AC/RB (Asphaltic Concrete Pavement over Portland Cement Concrete)
3. 1F1 (F1 type of site1)

Table 73 Pavement Subsection Descriptions

Pavement Subsection	Description
CT (Control)	Dense Graded Asphalt Mix
WI (Control)	Wisconsin's A1 Mix
F1	SMA w/ Cellulose Fiber Stabilizer
F2	SMA w/Mineral Fiber Stabilizer
P1	SMA w/Polymer (Thermoplastic) Stabilizer (Lo %)
P2	SMA w/Polymer (Thermoplastic) Stabilizer (Hi %)
E1	SMA w/Polymer (Elastomeric) Stabilizer (Lo %)
E2	SMA w/Polymer (Elastomeric) Stabilizer (Hi %)
SH	Strategic Highway Research Project (SHRP) Mix
SX	SHRP Mix Gradation w/Wisconsin 85-100
SP	SMA w/Polymer (EVA)
§	SMA Surface w/Vestoplast over Wisconsin A1 Binder
SF	SMA Full Depth w/Vestoplast
SM	SMA over SMA
SA (Control)	SMA over A Mix
SD	SMA w/Polymer (Vestoplast) over A Mix
HV	HV Asphalt w/ MAC 10

Friction Measurement by WisDOT Locked Wheel Skid Trailer
A locked wheel skid trailer provided by WisDOT was used to measure the surface characteristics of the different HMA mixes. A complete description of the skid trailer was provided in Chapter III.

Macrotexture Measurement by Sand Patch Method
The macrotexture depth of field compacted HMA pavement was measured by the sand patch test at a total of 39 subsections over six sites. A known volume of sand was placed on the surface of each subsection and spread by a circular motion until the voids in the surface of the subsection were filled with sand as shown in Figure 95. The diameter of the area covered with the sand was measured in accordance with ASTM E 965. The macrotexture texture depth (MTD) was then calculated by the MTD equation.

Data Analysis and Results

Relationship between Friction and MTD
Measurements from the selected roadways were used to determine the impact of material properties on parameters of interest. Data obtained during summer 2004 testing by WisDOT was used in studying pavement surface characteristics. Table 74 presents the friction numbers and MTD measured at in-service pavement, using a ribbed tire at target speeds of 40 mph and 50 mph.

Figure 95 Sand Patch Test in the Field

Table 74 FN and MTD for Subsection Pavements

Site	Highway	Pave. Subsec.	FN 40R	FN 50R	MTD (mm)
1	IH 43	1 F1	32.3	32	1.05
		1 F2	35.4	33.9	0.65
		1 P1	33.7	34.4	0.88
		1 P2	34.7	34.9	0.53
		1 CT	43	41.4	0.28
		1 HV	43	42.4	0.75
		1 E2	37	35.3	0.96
2	IH 43	2 CT	40.9	42	0.37
		2 SH	33.8	34.3	0.5
		2 F1	34.3	31.4	0.88
		2 F2	33.3	32.5	0.96
		2 P1	34.1	32.3	0.96
		2 SX	35.3	33.8	0.65
		2 P2	35	34	1.05
		2 E2	35.1	33.6	0.65
		2 E1	35.4	33.6	0.65
3	IH 94	3 SH	41.5	39.1	0.32
		3 SX	40.2	38.8	0.32
		3 WI	45.4	41.7	0.26
		3 SS	37.4	35	0.53
		3 SF	37.3	34.5	0.53
		3 SP	38.3	35.8	1.41
4	IH 94	4 SM	29.9	27.8	1.05
		4 SD	37.7	35	0.53
		4 SA	39.1	36	0.32
5	STH 21	5 CT	52.7	47.9	0.5
		5 E1	46.7	44.6	0.7
		5 E2	45.8	42.5	0.65
		5 P2	45.9	42.9	0.65
		5 P1	46	43.4	0.57
		5 F2	47.4	43.8	0.57
		5 F1	46.3	42.9	0.35
6	USH 151	6 E2	34.2	32	1.27
		6 E1	35.1	33.4	1.27
		6 P2	37.7	35.5	1.33
		6 P1	37.3	34.9	1.27
		6 F2	35	33.4	0.61
		6 F1	37.5	30.4	0.86
		6 CT	36.7	33.9	0.42

Note:
FN 40R and FN 50R: Friction number at 40 mph and 50 mph

A statistical analysis was completed to evaluate the relationship between Friction number (FN) and MTD values. The following equations were found to show the relationship between FN and MTD at 40 mph and 50 mph, respectively. Tables 75 and 76 show the ANOVA table for this analysis.

$$FN\ 40R = 44.1 - 7.53\ MTD\ (mm)\text{: at 40 mph} \quad (20)$$

$$FN\ 50R = 41.8 - 7.21\ MTD\ (mm)\text{: at 50 mph} \quad (21)$$

Surface friction is known to be closely related to surface texture. The above equations show that as MTD increases, FN decreases. Generally a smooth (or blank) tire is more sensitive to the macrotexture skid properties of the pavement surface and a treaded (ribbed) tire is more sensitive to the microtexture skid properties of the pavement. In this study, a WisDOT-provided ribbed tire was used due to time and equipment limitations. If a smooth tire had also been used for measuring friction numbers, it is theorized that another set of equations could be derived, showing that as FN increases, macrotexture depth increases.

Table 75 ANOVA Table for Relationship between FN and MTD (for 40 mph)

Source	**DF**	**ANOVA §**	**Mean Square**	**F Value**	**P value**
Model	1	220.37	220.37	10.17	0.003
Constant					0.000
MTD(mm)					0.003
Residual Error	37	801.57	21.66		
Corrected Total	38	1021.94			
Root Mean Square Error	4.65446				
R^2	0.216		**R^2 (Adjusted)**	0.194	

Table 76 ANOVA Table for Relationship between FN and MTD (for 50 mph)

Source	DF	ANOVA §	Mean Square	F Value	P value
Model	1	202.29	202.29	11.66	0.002
Constant					0.000
MTD(mm)					0.002
Residual Error	37	642.15	17.36		
Corrected Total	38	844.44			
Root Mean Square Error	4.16598				
R^2	0.240		**R^2 (Adjusted)**	0.219	

Relationship between FN and Mix Properties

Many HMA design parameters influence pavement skid resistance. This research was performed to determine the effect of aggregates within a paved surface on skid resistance, microtexture, and macrotexture. Other properties, such as asphalt content and percent air voids, also have been found to affect skid resistance. Recall that mix design data was obtained from WisDOT's HMA mix design database. Table 77 shows mix design data for each pavement subsection and for vehicle speeds (at 40 and 50 mph). This research sought to develop functional models showing the influence that mix design factors have on the skid resistant behavior of a pavement surface. A statistical analysis was completed to develop design models at 40 and 50 mph. The equations for the relationship between FN and mix design factors are:

At 40 mph:

$$\text{FN 40R} = 41.3 + 1.19\ \text{PP\#4} - 0.214\ \text{PP\#8} - 0.916\ \text{PP (\#4-\#50)} + 0.179\ \text{PP (9.5-\#8)} + 0.21\ \text{MNS} - 8.98\ \text{AC} + 1.97\ \text{AV} + 0.98\ \text{VMA} \tag{22}$$

At 50 mph:

$$\text{FN 50R} = 44.5 + 0.495\ \text{PP\#4} + 0.016\ \text{PP\#8} - 0.359\ \text{PP (\#4-\#50)} + 0.172\ \text{PP (9.5-\#8)} + 0.19\ \text{MNS} - 8.73\ \text{AC} + 1.01\ \text{AV} + 1.22\ \text{VMA} \tag{23}$$

where,

PP#4: % passing rate No. 4 sieve
PP#8: % passing rate No. 8 sieve
PP9.5: % passing rate 9.5mm sieve
PP#50: % passing rate No. 50 sieve
AT: Aggregate Type
AV: Air Voids
MNAS: Maximum nominal aggregate size
VMA: voids in the mineral aggregate

Table 77 FN and Mixture Properties in Subsection Pavements

Pave. Subsec.	FN 40R	FN 50R	%P No.4	%P No.8	%P No.4 -%P No.50	%P 9.5 - No.8	MNS (mm)	AC (%)	AV (%)	VMA (%)
1 F1	32.3	32.0	29.0	19.0	18.0	59.0	19.0	6.2	2.6	15.3
1 F2	35.4	33.9	29.0	19.0	18.0	59.0	19.0	6.1	2.2	14.5
1 P1	33.7	34.4	34.0	22.0	20.0	50.0	19.0	6.0	2.4	15.1
1 P2	34.7	34.9	34.0	22.0	20.0	50.0	19.0	6.0	3.2	14.8
1 CT	43.0	41.4	76.0	51.0	64.0	39.0	19.0	5.9	3.5	15.5
1 HV	43.0	42.4	76.0	51.0	64.0	39.0	19.0	5.9	3.5	15.5
1 E2	37.0	35.3	34.0	22.0	20.0	50.0	19.0	6.1	2.9	14.9
2 CT	40.9	42.0	75.0	55.0	65.0	37.0	19.0	6.2	5.2	17.2
2 SH	33.8	34.3	54.0	28.0	42.0	54.0	19.0	6.1	4.1	17.1
2 F1	34.3	31.4	37.0	21.0	25.0	78.0	12.5	6.2	3.6	16.9
2 F2	33.3	32.5	37.0	21.0	25.0	78.0	12.5	6.3	3.1	16.6
2 P1	34.1	32.3	37.0	21.0	25.0	78.0	12.5	7.2	4.4	18.7
2 SX	35.3	33.8	53.0	28.0	42.0	54.0	19.0	5.0	4.2	16.8
2 P2	35.0	34.0	37.0	21.0	25.0	78.0	12.5	7.0	4.0	18.0
2 E2	35.1	33.6	37.0	21.0	25.0	78.0	12.5	7.2	3.3	17.5
2 E1	35.4	33.6	37.0	21.0	25.0	78.0	12.5	7.0	2.7	17.9
3 SH	41.5	39.1	54.0	28.0	42.0	54.0	19.0	6.1	4.1	17.1
3 SX	40.2	38.8	53.0	28.0	42.0	54.0	19.0	5.0	4.2	16.8
3 WI	45.4	41.7	73.0	52.0	57.0	37.0	19.0	6.0	3.0	14.9
3 SS	37.4	35.0	32.0	17.0	19.0	50.0	19.0	6.5	3.0	18.2
3 SF	37.3	34.5	32.0	17.0	19.0	50.0	19.0	6.5	3.0	18.2
3 SP	38.3	35.8	29.0	16.0	18.0	50.0	19.0	5.9	3.0	15.8
4 SM	29.9	27.8	27.9	20.1	14.7	49.5	19.0	5.5	2.4	15.9
4 SD	37.7	35.0	27.9	20.1	14.7	49.5	19.0	6.5	3.1	16.4
4 SA	39.1	36.0	27.9	20.1	14.7	49.5	19.0	5.4	3.5	16.1
5 CT	52.7	47.9	74.6	53.6	57.6	39.2	19.0	5.9	4.2	16.2
5 E1	46.7	44.6	35.0	21.0	22.1	75.2	12.5	6.0	2.9	16.0
5 E2	45.8	42.5	35.0	21.0	22.1	75.2	12.5	5.8	3.0	15.2
5 P2	45.9	42.9	35.0	21.0	22.1	75.2	12.5	5.7	3.0	15.2
5 P1	46.0	43.4	35.0	21.0	22.1	75.2	12.5	5.6	3.1	15.0
5 F2	47.4	43.8	35.0	21.0	22.1	75.2	12.5	5.9	3.0	15.1
5 F1	46.3	42.9	35.0	21.0	22.1	75.2	12.5	5.8	3.1	15.3
6 E2	34.2	32.0	29.0	17.0	18.0	55.0	19.0	6.5	4.5	16.6
6 E1	35.1	33.4	29.0	17.0	18.0	55.0	19.0	6.7	4.4	16.8
6 P2	37.7	35.5	29.0	17.0	18.0	55.0	19.0	6.6	4.5	16.9
6 P1	37.3	34.9	29.0	17.0	18.0	55.0	19.0	6.5	4.4	16.7
6 F2	35.0	33.4	29.0	17.0	18.0	55.0	19.0	6.3	4.4	15.9
6 F1	37.5	30.4	29.0	17.0	18.0	55.0	19.0	6.7	4.4	16.4
6 CT	36.7	33.9	63.0	48.0	46.0	33.0	19.0	5.3	2.6	13.3

Tables 78 and 79 show the ANOVA table for this statistical analysis. The regression equation indicates that as the maximum nominal aggregate size increased, so did the measured skid number. Larger aggregates incorporated into wearing surface mixes may increase the contact of small asperities in the aggregate surface at the pavement-to-tire interface. There was also a measurable increase in FN as voids in the mineral aggregate (VMA) increased. Although the ribbed tire allows for the dispersal of water from the roadway surface through the treads, additional removal of water would increase the skid resistance. An increase of VMA will increase the surface voids, thereby increasing the rate at which water is repelled from the pavement-to-tire interface. Also found was that the asphalt content affects skid resistance. If an excessive amount of asphalt is incorporated into the pavement mix, there is a tendency for bleeding of the pavement surface. Bleeding of asphalt prevents the aggregates from properly contacting the vehicle tire and can cause large decreases in the skid resistance properties of the surface.

Table 78 ANOVA Table for Relationship between FN and Mix Factors (for 40 mph)

Source	**DF**	**ANOVA SS**	**Mean Square**	**F Value**	**P value**
Model	8	514.4	64.3	4.05	0.004
Constant					0.008
PP#4					0.005
PP#8					0.032
PP(#4-#50)					0.072
PP(9.5-#8)					0.063
AC					0.052
MNAS					0.002
AV					0.004
VMA					0.009
Residual Error	24	514.4	64.30		
Corrected Total	32	895.43			
Root Mean Square Error	3.98450				
R^2	0.574		**R^2 (Adjusted)**	0.433	

Table 79 ANOVA Table for Relationship between FN and Mix Factors (for 50 mph)

Source	DF	ANOVA SS	Mean Square	F Value	P value
Model	8	456.39	57.05	4.83	0.001
Constant					0.005
PP#4					0.003
PP#8					0.012
PP(#4-#50)					0.003
PP(9.5-#8)					0.087
AC					0.010
MNAS					0.002
AV					0.003
VMA					0.007
Residual Error	24	456.39	57.05		
Corrected Total	32	739.74			
Root Mean Square Error	3.43604				
R^2	0.61.7		**R^2 (Adjusted)**	0.489	

Macrotexture and Skid Resistance

To explore the macrotexture relationship with skid resistance, FNs for 2004 were combined with Mean Texture Depth (MTD) data. The intention was to see if the relationship between macrotexture and skid resistance can be understood and quantified. MTD was defined in Chapter II as a volumetric method for measuring surface texture. MTD measurements were taken at the same study sites during the end of the summer of 2004 following the Sand Patch Method procedure. MTD data for the study sites were provided by fellow researchers in the University of Wisconsin Department of Civil and Environmental Engineering. Table 80 shows the MTD values for the different mix types along the study sections. Figure 96 shows a plot of the combined MTD and friction number data for FN40R.

Table 80 MTD Measurements

Mix Type/MTD (mm)					
Site 1	**Site 2**	**Site 3**	**Site 4**	**Site 5**	**Site 6**
IH 43 Walworth	**IH 43 Waukesha**	**IH 94 Monroe**	**IH 94 Waukesha**	**STH 21 Juneau**	**USH 151 Grant/Lafayette**
CT/0.28	CT/0.37	WI/0.26	SA/0.32	CT/0.5	CT/0.42
E1/-	E1/0.65	SS/0.53	SD/0.53	E1/0.7	E1/1.27
E2/0.96	E2/0.65	SF/0.53	SM/1.05	E2/0.65	E2/1.27
F1/1.05	F1/0.88	SP/1.41		F1/0.35	F1/0.86
F2/0.65	F2/0.96	SH/0.32		F2/0.57	F2/0.61
P1/0.88	P1/0.96	SX/0.32		P1/0.57	P1/1.27
P2/0.53	P2/1.05			P2/0.65	P2/1.33
HV/0.75	SX/0.65				
	SH/0.5				

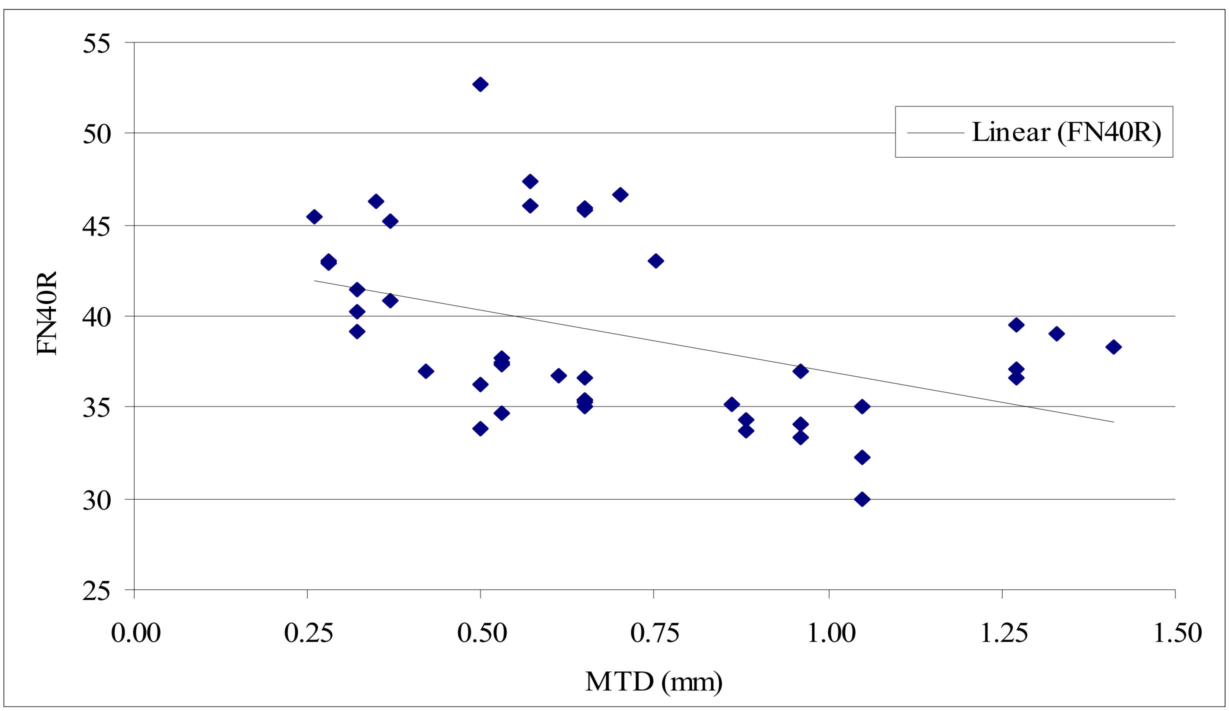


Figure 96 Relationship between FN40R and MTD (Macrotexture)

The regression equation for data in Figure 96 is as follows: FN40R = 43.7 – 6.76 MTD. This equation shows an inversely proportional relation between MTD and skid resistance. Table 81 shows statistical results for the linear regression.

For $\alpha = 0.05$, the p-value for the model is statistically significant. Nonetheless, the coefficient of determination (R^2) was 0.162 indicating a weak relationship. A Pearson correlation among both variables yielded a value of -0.427 indicating that the linear relationship is not a negative perfect relationship. The p-value for Pearson correlation is statistically significant (0.004). Table 82 shows the analysis of variance for the regression.

For FN50R and MTD, the regression equation resulted as FN50R = 40.8 – 5.83 MTD. Data are plotted in Figure 97. This equation shows an inversely proportional relation between MTD and friction number. Table 83 shows statistical results for the linear regression.

For $\alpha = 0.05$, the p-values for both coefficients are statistically significant. Nonetheless, the coefficient of determination (R^2) was 0.147 indicating a weak relationship. For $\alpha = 0.05$, the p-value for the model was also statistically significant. A Pearson correlation among both variables yielded a value of -0.409 indicating that the linear relationship is not negative perfect. For $\alpha = 0.05$ the p-value for Pearson correlation is statistically significant (0.006). Table 84 shows the analysis of variance for the regression.

Table 81 T-Statistic for Coefficients (MTD and FN40R)

Predictor	Coefficient	Standard Error Coefficient	t statistic	p-value
Constant	43.670	1.704	25.63	0.000
MTD	-6.759	2.236	-3.02	0.004
S = 4.59869, R-Sq = 18.2%, R-Sq(adj) = 16.2%				

Table 82 ANOVA of FN40R and MTD

Source	DF	SS	MS	F	p-value
Regression	1	193.27	193.27	9.14	0.004
Residual Error	41	867.06	21.15		
Total	42	1060.34			

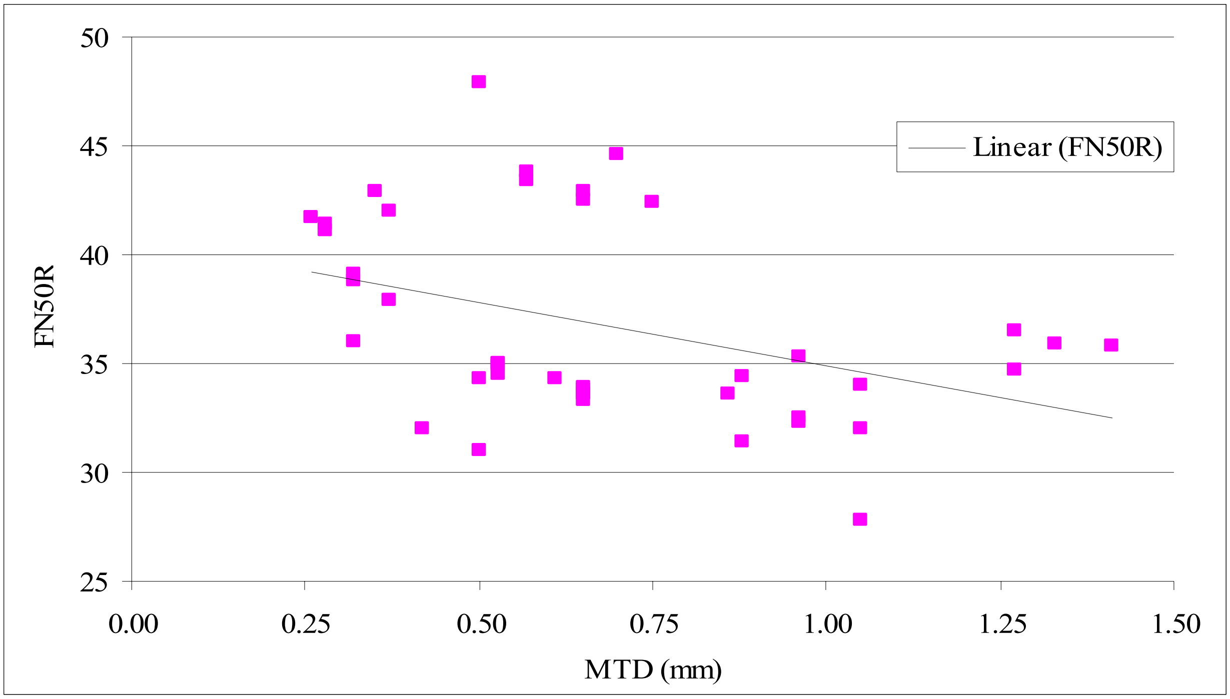


Figure 97 Relationship between FN50R and MTD (Macrotexture)

Table 83 T-Statistics for Coefficients (MTD and FN50R)

Predictor	Coefficient	Standard Error Coefficient	t statistic	p-value
Constant	40.757	1.548	26.33	0.000
MTD	-5.828	2.032	-2.87	0.006
S = 4.17921, R-Sq = 16.7%, R-Sq(adj) = 14.7%				

Table 84 ANOVA of FN50R and MTD

Source	DF	SS	MS	F	p-value
Regression	1	143.72	143.72	8.23	0.006
Residual Error	41	716.10	17.47		
Total	42	859.82			

CHAPTER VII

ASSET MANAGEMENT PLAN

The final step of this research was to provide consideration of how skid friction can become part of the asset management plan of roadway agencies. Specifically, this research considered the development of guidelines for using skid numbers or other safety measures in project level management. One of the major challenges is that WisDOT, like many departments of transportation, have discontinued the annual skid testing programs. Without this data, the development of a regular management program becomes significantly more challenging.

This research has generated two key findings that can directly impact an asset management approach to skid friction safety. First, although there was not a strong statistical correlation found between skid friction values and safety, the results of this research, supported by the literature, show that FN values less than 35 negatively contribute to safety under wet pavement conditions. Therefore, pavement sections identified with FN values at or near 35 should be programmed for rehabilitation or reconstruction activities. Second, it is possible to develop asphalt mix designs with properties that maintain the integrity of the design while developing a higher initial FN value.

The construction of asphalt roadways is a challenge when FN is of focus. Construction methods typically include a significant amount of compaction rolling to achieve specified densities. Additionally, final pavement smoothness to reduce tire noise is desired and rewarded. The combination of these effects tends to reduce the potential FN values of the mix design. Proposed enhancements to asphalt mix designs as proposed in Chapter VI may lead to an asset management plan to accommodate the safety benefits of higher friction values with the same ride-ability and densities benefits.

The 2005 Austroads report on pavement friction includes a categorization scheme for management of pavements as deterioration occurs (*54*). There are two levels which require action to be taken. The first is the “Investigatory Level”, which is triggered whenever the predetermined skid friction values are reached. The second is the “Threshold Level”, which is triggered whenever the skid friction values drop to a predetermined level below the “Investigatory Level” (Austroads suggests a value of 0.10 for this differential).

When the Investigatory Level is reached, Austroads recommends that remedial alternatives be reviewed and the roadway should undergo an extensive site investigation to determine what, if any, minor modifications might be made to improve the pavement section. Reaching the Investigatory Level should also place the pavement on a priority list for rehabilitation, and it should also receive heightened scrutiny regarding the cause(s) of any crashes that have already or may yet occur. When the Threshold Level is reached, Austroads recommends that the pavement section be placed on an immediate fast-track for remedial action, without first undergoing a site investigation.

An additional area of focus for pavement management is also included in the Austroads report. It recommends setting a differential value (0.10 is suggested) between travel lanes for a multi-lane roadway, such that if the skid friction values of multiple lanes of travel are more than the differential apart, then the Investigatory Level is reached. The same differential scheme is recommended for the two wheel paths in a given lane. While the skid friction differential between wheel paths is of little issue for many, two-wheeled vehicles are particularly susceptible to a significant difference in skid friction from one wheel path to the other.

One of the primary methods of pavement maintenance that leads to reduced skid friction values is the use of bituminous sealants. Crack sealants are used extensively to prevent water infiltration into (and below) asphalt pavements, and can be quite effective in accomplishing this desirable task. When too much sealant is used, as is a quite common occurrence, it tends to be spread out by traffic across the road surface during hot weather conditions, resulting in an expanding area of extremely low wet skid friction.

Therefore, an effective asphalt pavement asset management approach will include an annual testing program to monitor skid friction (i.e., FN) values. As presented in Table 3, FN values less than 35 should trigger a safety monitoring program and those pavements scheduled for future rehabilitation or reconstruction. As mentioned, the type of rehabilitation is critical to safety such that friction values are enhanced, especially during wet road conditions. The final aspect of the asset management program should include a detailed review of asphalt mix design and construction practices to assure that the initial FN value of newly constructed or rehabilitated pavement is maximized.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

This research explored the relationship between asphalt mix design, skid friction, and roadway safety. Initial tasks attempted to find a relationship between pavement skid resistance (friction) and crash frequency, particularly wet weather crashes. Friction and crash data collected over 10 years at six study sites in Wisconsin were analyzed.

The results of the analysis did not indicate a relationship between crash frequency and pavement skid friction. Although some evidence suggests that the number of wet pavement crashes increased as the pavement life increased (and skid friction values decreased), the frequency of crashes was not sufficient to support this claim statistically. Coefficients of determination and p-values were low for friction measurements taken at 40 mph and 50 mph. Nevertheless, the fact that the relationship seems to behave inversely proportional, that is to say more crashes occurred at low friction numbers (FNs), is an important indication that skid resistance may indeed be a factor affecting wet weather crashes.

It was not possible to determine a skid friction threshold value that indicates the critical point where pavement maintenance would be needed. No data trend or statistical significance was found between the standard FN40 values and number of crashes that occurred. Therefore, no threshold value can be assumed from this data. Nonetheless, it should be noted that the lowest FN registered in this study was 30 on subsection SM at Site 4. Because of the low frequency of crashes occurring at the site, it could be inferred that an FN of 30 is still a safe level for motorists. This inference cannot be proven due to the lack of significance of the FN40 data.

There were no study sections with a history of crashes occurring during low skid resistance conditions, making it harder to detect a critical point between low and high skid resistance. The fact that friction levels on study roadways were relatively high could help explain why wet weather skid-related crashes were infrequent along the study sections. Consequently, this may validate the belief that high-friction values prevent wet weather-related crashes.

Finding a threshold value is important to help as an indicator of required maintenance and safety of the roads. Adequate pavement skid resistance is critical to accomplish a safe vehicle braking maneuver, as is reflected in AASHTO's braking distance requirements. As described in Chapter II, the FN assumed in AASHTO's stopping sight distance equation is 35. This friction value is derived from the default value for deceleration. Hence, roadways throughout Wisconsin and the U.S. are designed with the assumption that a minimum FN of 35 is available for emergency braking. Although the data obtained in the research could not support a specific value, it is clear that friction values less than 35 are problematic from a safety standpoint.

A possible indicator of friction on high-speed roadways is macrotexture. Therefore, macrotexture (measured as MTD) combined with friction data was of great interest in this research. It was expected that a directly proportional relationship existed between both variables. In other words, it was expected that the larger the MTD value for a given section of pavement, the larger the FN. However, according to the data analysis the relationship resulted in the opposite for the study sections. Plots of MTD and FN values did not show a clear relationship between the two values, although it was evident that the larger FNs were concentrated in low MTD values. A correlation between MTD and pavement skid friction could allow the prediction of pavement skid friction from volumetric measurements. Volumetric measurements are simpler and less expensive to perform than friction measurements taken with a skid trailer. Ultimately using volumetric measurements to predict pavement skid friction could help in the design of pavement mixes that yield high FNs and increase the safety of the road by maximizing grip between vehicle tires and pavement.

It is difficult to explain why the relationship between MTD and friction was found to be more inversely proportional than directly proportional, as was expected. Data suggest that dense graded asphalt mixes, although less porous than SMA mixes, yield better skid resistance. Dense graded asphalt mixes tested had the lowest MTD values, but the highest friction numbers. The lack of a consistent relationship between FN and MTD could be related to the use of a ribbed tire for friction measurements. The literature review in Chapter II showed that a ribbed tire is not a good indicator of macrotexture, instead it mainly detects microtexture. A possible explanation for the unexpected directly proportional shapes of the graphs could be that sections with high MTD had low microtexture. Note that historical FN information used in this research was taken with a ribbed tire and this is why no smooth tire measurements were taken.

Although no statistically significant results were obtained, it has become clear that skid resistance is an important feature which should be considered while evaluating roadway safety. Research around the globe has demonstrated that there is a skid resistance involvement in the incidence and frequency of many crashes. An effective asphalt pavement asset management approach will include an annual testing program to monitor skid friction values. FN values less than 35 should trigger a safety monitoring program and those pavements scheduled for future rehabilitation or reconstruction. The final aspect of the asset management program should include a detailed review of asphalt mix design and construction practices to assure that the initial FN value of newly constructed or rehabilitated pavement is maximized.

Recommendations

After reviewing the outcome of this research study, the following recommendations are made:

- New research should be performed using a smooth tire skid tester and include all sites, especially those with known low skid resistance.
- Wisconsin MV4000 Accident Reports should add a section to identify crashes in which skidding or pavement friction may have been a contributing factor to the crash.
- New research should evaluate correlation between skid resistance and macrotexture in order to incorporate macrotexture values and maximum friction in mix design.
- Incorporation of Portland cement concrete pavements (PCC) in skid resistance research is needed to determine the friction-crash correlation on rigid and flexible pavements.
- An FN threshold value of 35, as inferred by AASHTO, should be maintained until new evidence is gathered.

Knowing these interactions in greater detail will help to maintain safer roadways and improve methods to determine when maintenance is needed.

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APPENDIX A

REQUEST FOR INFORMATION – FORM

The University of Wisconsin - Madison
Department of Civil and Environmental Engineering
In Cooperation with
The Midwest Regional University Transportation Center

ASPHALT PAVEMENT SKID RESISTANCE

REQUEST FOR INFORMATION

Version 5 (02.03.04)

The Department of Civil and Environmental Engineering at the University of Wisconsin - Madison is conducting an analysis on the current use of skid resistance in asphalt pavement design, construction, maintenance, and safety, focusing on U.S. DOT Region 5 states and selected others. Please assist us by completing the following questions. You may submit via mail, e-mail, or fax to the address/number listed at the end of the survey. **THANK YOU FOR YOUR PARTICIPATION!!!**

RESPONDENT

Name: ____

Title: ____

Agency: ____

Address 1: ____

Address 2: ____

City: ____

State: _________________ **Zip Code:** __________________

Telephone: (_____)_______________ **Ext:** ______________

FAX: __________________________

Email: _________________________

REQUEST FOR INFORMATION

1. Does your agency consider skid resistance requirements in asphalt mix design specifications?
 □ Yes □ No
 (If you answered No, please skip to question 4)

2. Do the specifications consider micro texture and macro texture for the skid resistance design?
   □ Only micro texture
   □ Only macro texture
   □ Both, micro texture and macro texture
   □ None (skip to question 4)

3. What are the skid resistance specifications? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?
   ____
   ____
   ____
   ____

4. Does your agency use laboratory testing methods for measuring asphalt mixture or aggregate properties related to micro and macro texture?
   □ Only micro texture
   □ Only macro texture
   □ Both, micro texture and macro texture
   □ None (skip to question 6)

5. What laboratory methods are used? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?
   ____
   ____
   ____
   ____

6. Does your agency use field testing methods to measure asphalt pavement micro and macro texture?
   □ Only micro texture
   □ Only macro texture
   □ Both, micro texture and macro texture
   □ None (skip to question 8)

7. What field methods are used? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?
   ____
   ____
   ____
   ____

8. Does your agency use a skid trailer or other equipment to measure skid resistance of asphalt pavements?
   □ Skid trailer (skip to question 10)
   □ Other equipment
   □ Both, skid trailer and other equipment
   □ None (skip to question 12)

9. Please describe the other equipment.
   ____
   ____
   ____
   ____

10. What are the criteria for use?
    ____
    ____
    ____
    ____

11. What is the frequency of use?

____
____
____
____

12. Has your agency developed a model or equation to estimate a friction number (FN)?
 Yes No
(If you answered No, please skip to question 14)

13. Please describe the model used and define the variables included. Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

14. Does your agency have any skid resistance requirements for the design of pavements?
 Yes No
(If you answered No, please skip to question 16)

15. What are the requirements? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

16. Does your agency have any skid resistance requirements for the maintenance of pavements?
 Yes No
(If you answered No, please skip to question 20)

17. Is the skid number used as a measure for decision making regarding maintenance activities?
 Yes No
(If you answered No, please skip to question 19)

18. How is the skid number used in decision making? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

19. Are there any other requirements for the maintenance of pavements? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

20. Does your agency maintain a database of skid values?
 Yes No
(If you answered No, please skip to question 22)

21. Is this database available for research purposes? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

22. Does your agency correlate skid numbers to crash/safety data?
 Yes No
(If you answered No, please skip to question 24)

23. How is it correlated? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

24. Has your agency conducted any studies related to skid resistance?
 Yes No
(If you answered No, please skip to question 26)

25. What were the study objectives? Would you please attach a copy, provide the URL if contained on a web site, or fill in a brief statement in the space below?

____
____
____
____

26. If you have any additional comments, please write them in the space below:

____
____
____
____

END
Thank you for completing this request for information. Please direct all correspondence to:

Dr. David A. Noyce, P.E.
University of Wisconsin – Madison
Department of Civil and Environmental Engineering
1210 Engineering Hall
1415 Engineering Drive
Madison, WI 53706-1691
E-mail: noyce@engr.wisc.edu
Tel: 608/265-1882
Fax: 608/262-5199

APPENDIX B

REQUEST FOR INFORMATION – AGENCY RESULTS

Table 1. Participant State DOT's Summary	
States where a RFI was sent	**States that responded the RFI so far**
Illinois	Illinois
Indiana	Indiana
Iowa	Iowa
Michigan	Michigan
Minnesota	Missouri
Missouri	Nebraska
Nebraska	Ohio
Ohio	Pennsylvania
Pennsylvania	Washington
Washington	Wisconsin
Wisconsin	

Table 1A. General Responses to Question 1	
Question 1: Does your agency consider skid resistance requirements in asphalt mix design specifications?	
Responses	**Number of Responses**
Yes	5
No	5

Table 1B. Detailed Responses to Question 1	
Agencies that consider skid resistance requirements in asphalt mix design specifications:	**Agencies that do not consider skid resistance requirements in asphalt mix design specifications:**
Illinois	Michigan
Indiana	Nebraska
Iowa	Ohio*
Pennsylvania	Washington
Missouri	Wisconsin
**Individual cases only*	

Table 2A. General Responses to Question 2	
Question 2: Do the specifications consider micro texture and macro texture for the skid resistance design?	
Responses	**Number of Responses**
Only micro texture	2
Only macro texture	0
Both, micro texture and macro texture	2
None	1

Table 2B. Detailed Responses to Question 2			
Only micro texture	**Only macro texture**	**Both, micro texture and macro texture**	**None**
Illinois		Iowa	Indiana
Pennsylvania		Missouri	

Table 3. Responses to Question 3	
Question 3: What are the skid resistance specifications?	
Illinois	Illinois DOT requires specific aggregates for hot mix asphalt based upon traffic volumes (ADT). A copy of the aggregate specifications will be attached to the e-mail with this survey.
Indiana	-
Iowa	Levels of friction criteria for HMA mix design are based on speed and traffic/truck volume. (see attachment #1) The criteria specify the amount and quality of friction aggregate in the coarse fraction of the gradation blend. (spec 2303.02, B.1)(attachment #2)
Missouri	http://www.modot.mo.gov/design/specbook/perffiles.htm, Section 403.3.5
Pennsylvania	<u>Initial or Current Two-Way ADT*</u> — <u>SRL Designation</u> Above 20,000 — E 5,001 – 20,000 — H; Blend of E and M; Blend of E and G 3,001 – 5,000 — G; Blend of H and M; Blend of E and L 1,001 – 3,000 — M; Blend of H and L; Blend of G and L; Blend of E and L; 0 – 1,000 — L *When all traffic for an SR travels in one direction, divide the ADT values shown above by 2 to determine the required SRL.

Table 4A. General Responses to Question 4	
Question 4: Does your agency use laboratory testing methods for measuring asphalt mixture or aggregate properties related to micro and macro texture?	
Responses	**Number of Responses**
Only micro texture	2
Only macro texture	0
Both, micro texture and macro texture	2
None	6

Table 4B. Detailed Responses to Question 4

Only micro texture	Only macro texture	Both, micro texture and macro texture	None
Michigan		Indiana*	Illinois
Pennsylvania		Iowa	Missouri
			Nebraska
			Ohio
			Washington
			Wisconsin

**Specialty projects only*

Table 5. Responses to Question 5	
Question 5: What laboratory methods are used?	
Indiana	British Pendulum Tester
Iowa	Geology and other aggregate source criteria are check for all friction aggregate sources. (IM T-203) HMA mix designs are reviewed for specification compliance. No specific HMA mixture testing is done to measure friction micro/macro texture.
Michigan	http://www.michigan.gov/documents/mdot_MTM_CombinedManual_83501_7.pdf. See MTM 111 & MTM 112 aggregate wear index (for bit pavements).
Pennsylvania	Petrographic examination, Polished Stone Value using the British pendulum and the British wheel.

Table 6A. General Responses to Question 6	
Question 6: Does your agency use field testing methods to measure asphalt pavement micro and macro texture?	
Responses	**Number of Responses***
Only micro texture	0
Only macro texture	0
Both, micro texture and macro texture	4
None	5
**Ohio did not answer*	

Table 6B. Detailed Responses to Question 6			
Only micro texture	**Only macro texture**	**Both, micro texture and macro texture**	**None**
		Illinois	Indiana
		Iowa	Michigan
		Missouri	Nebraska
		Pennsylvania	Washington
			Wisconsin

Table 7. Responses to Question 7	
Question 7: What field methods are used?	
Illinois	The Illinois DOT uses ASTM E 274 test method to monitor skid resistance of pavements. We use both a treaded tire and a smooth tire.
Iowa	HMA mixture compliance, including friction aggregate criteria (sources and quantity), is continuously monitored during HMA production and paving operations by routine HMA sampling and testing. No specific testing is performed to measure micro/macro texture.
Missouri	MoDOT uses AASTO T242 for determining frictional properties of paved surface after construction is done.
Pennsylvania	Skid friction testing using a skid trailer.

Table 8A. General Responses to Question 8	
Question 8: Does your agency use a skid trailer or other equipment to measure skid resistance of asphalt pavements?	
Responses	**Number of Responses**
Skid trailer	9
Other equipment	1
Both, skid trailer and other equipment	0
None	0

Table 8B. Detailed Responses to Question 8			
Skid trailer	**Other equipment**	**Both, skid trailer and other equipment**	**None**
Illinois		Ohio	
Indiana			
Iowa			
Michigan			
Missouri			
Nebraska			
Pennsylvania			
Washington			
Wisconsin			

Table 9. Responses to Question 9	
Question 9: Please describe the other equipment.	
Ohio	The department is experimenting with a side force measuring 5th wheel mounted at a slight skew designed by Halliday Technologies.

Table 10. Responses to Question 10	
Question 10: What are the criteria for use?	
Illinois	ASTM E 274
Indiana	For inventory friction testing, the friction flag value is 20 (smooth tire, 40 mph). For warranty projects, the friction flag value is 35 (smooth tire, 40 mph).
Iowa	Skid trailer used to monitor pavement for the PMS.
Michigan	Entire state maintained network. Tested on a 3 year cycle.
Missouri	Interstates, NHS, and Special request.
Nebraska	Test all new surfacing projects and the State highway system.
Ohio	Our office responds to special requests from our districts or counties which may have been generated by an accident history, user complaints, or problems encountered during construction. Also we have a list of research sections that change from year to year that come both from in house projects and programs in various districts.
Pennsylvania	-
Washington	See attachment.
Wisconsin	Measurements are made at the request of WI D.O.T. approved agencies or projects. Primarily the measurements are made to provide performance data on experimental pavements. Additionally, measurements may be made when liability issues arise. All measurements are made in accordance of ASTMs E-274, E-501, and E-524.

Table 11. Responses to Question 11	
Question 11: What is the frequency of use?	
Illinois	High accident locations are tested upon request. Aggregates used in the various hot mix asphalt mixtures are evaluated biennially.
Indiana	Interstate: every year, State and U.S. highways: every three years Anytime if necessary
Iowa	The schedule of skid trailer use is based on the type of route (Interstate or State), the traffic level and existing friction number. Others are measured at the request of field offices. The Interstate routes are covered in a two-year cycle.
Michigan	3 year cycle. 30-40,000 test per year.
Missouri	Weather permitting, as needed.
Nebraska	All new surfacing is tested within a few months of placement, and the system is tested on a three year cycle.
Ohio	Generally testing is performed once or twice per season on most research projects. On occasion it has been more frequent. We generally test almost exclusively at 40 MPH at either half mile or quarter mile increments depending on section length. If the location is a site where corrective action is going to take place, we sometimes test before any action takes place and then afterward to document the friction improvement.
Pennsylvania	Approximately 2100 miles of roadway were tested in 2003. All testing is done as a result of a request from an Engineering District office, or the Materials and Testing Lab.
Washington	Entire state highway system every two years. We test half one year and the rest the following year.
Wisconsin	10 days a year.

Table 12A. General Responses to Question 12	
Question 12: Has your agency developed a model or equation to estimate a friction number (FN)?	
Responses	**Number of Responses**
Yes	2
No	8

Table 12B. Detailed Responses to Question 12	
Agencies that have developed a model or equation to estimate a friction number (FN):	**Agencies that have not developed a model or equation to estimate a friction number (FN):**
Iowa	Illinois
Wisconsin	Indiana
	Michigan
	Missouri
	Nebraska
	Pennsylvania*
	Washington
**No estimation, skid testing results in a Skid Number determined as follows: SN = Horizontal Tractive Force / Vertical Load X 100 (The coefficient of friction, is Horizontal Tractive Force / Vertical Load.)*	

Table 13. Responses to Question 13	
Question 13: Please describe the model used and define the variables included.	
Iowa	This can be done through the regression of the PMS database, but is not done on a regular basis except for system-wide analysis.
Wisconsin	FN= Friction Number @40mph, LAVP= Lane Accumulated Vehicle Passes, LAWEAR= Los Angles Wear, HV= % Heavy Vehicles in the design lane, DOLOMITE= % Dolomite in the mix AC FN= 41.4 - 0.00075 DOLOMITE^2 - 1.45 ln(LAVP) + 0.245 LAWEAR PC Ln(FN)= 3.99 - 0.0419 ln(LAVP) - 0.00129 DOLOMITE + 0.00474 HV

Table 14A. General Responses to Question 14	
Question 14: Does your agency have any skid resistance requirements for the design of pavements?	
Responses	Number of Responses
Yes	3
No	7

Table 14A. Detailed Responses to Question 14	
Agencies with skid resistance requirements for the design of pavements:	**Agencies without skid resistance requirements for the design of pavements:**
Illinois	Indiana
Iowa	Michigan
Pennsylvania	Missouri
	Nebraska
	Ohio*
	Washington
	Wisconsin
**For mix design, only in individual cases. For pavement design, No.*	

Table 15. Responses to Question 15	
Question 15: What are the requirements?	
Illinois	The only requirements are to use the appropriate aggregate based upon level of ADT as mentioned earlier.
Iowa	See survey question 3 for details.
Pennsylvania	Defined in #3.

Table 16A. General Responses to Question 16	
Question 16: Does your agency have any skid resistance requirements for the maintenance of pavements?	
Responses	**Number of Responses**
Yes	5
No	5

Table 16B. Detailed Responses to Question 16	
Agencies with skid resistance requirements for the maintenance of pavements:	**Agencies without skid resistance requirements for the maintenance of pavements:**
Indiana	Illinois
Iowa	Missouri
Michigan	Nebraska
Pennsylvania	Ohio*
Washington	Wisconsin
**Only in isolated cases.*	

Table 17A. General Responses to Question 17	
Question 17: Is the skid number used as a measure for decision making regarding maintenance activities?	
Responses	**Number of Responses**
Yes	6
No	0

Table 17B. Detailed Responses to Question 17	
Agencies with skid number used as a measure for decision making regarding maintenance activities:	**Agencies without skid number used as a measure for decision making regarding maintenance activities:**
Indiana	
Iowa	
Michigan	
Ohio	
Pennsylvania	
Washington	

Table 18. Responses to Question 18	
Question 18: How is the skid number used in decision making?	
Indiana	All low FN together with their locations are reported to the individual districts for field investigation and further action. The annual network pavement friction testing report is sent to each district and PMS engineer for planning maintenance and resurfacing.
Iowa	Minimum threshold levels are used to notify field offices about pavement sections with values below the threshold.
Michigan	If a pavement falls below a FN of 30 and has a crash history, corrective action is taken.
Ohio	Districts are given safety reports with accident rates and SN for sections. Activities are conducted to reduce rates through improving skid resistance or other improvements.
Pennsylvania	Action should be taken when all of the following occurs: -site is on the wet pavement accident cluster list or a known skid friction problem exists (8 or more wet pavement accidents within 3000 feet, wet accidents / total accident equals or exceeds 0.30) -one or more high friction needs exists within the cluster area (such as curves with low design speed, high speed areas with a high frequency of access points, short stopping sight distance for vertical curves, short accel./decel. lanes at interchanges, flushed and/or polished surfaces) -either the ribbed tire SN is less than 35, or the smooth tire SN is less than 20
Washington	See attachment.

Table 19. Responses to Question 19	
Question 19: Are there any other requirements for the maintenance of pavements?	
Indiana	IRI and PCR
Iowa	Question too broad – no response.
Michigan	-
Ohio	-
Pennsylvania	Depending on the criteria defined in #18, "Slippery When Wet" signs may need to be installed, and/or corrective action to the pavement may be warranted (overlay, grinding, etc.)
Washington	-

Table 20A. General Responses to Question 20	
Question 20: Does your agency maintain a database of skid values?	
Responses	**Number of Responses**
Yes	9
No	1

Table 20B. Detailed Responses to Question 20	
Agencies with database of skid values:	**Agencies without database of skid values:**
Illinois	Wisconsin
Indiana	
Iowa	
Michigan	
Missouri	
Nebraska	
Ohio	
Pennsylvania	
Washington	

Table 21. Responses to Question 21	
Question 21: Is this database available for research purposes?	
Illinois	We keep the database for internal use only.
Indiana	This database is for agency use only.
Iowa	No, the PMS database is not accessible outside the DOT.
Michigan	We would need a letter explaining the intent and objectives of the research before we would release any data.
Missouri	No, this database is not available.
Nebraska	The database has only limited accessibility. If you have specific requests, we can query the database for you.
Ohio	Currently our skid data is not very accessible. We are working on creating an online database that is accessible and usable for research purposes. Since we don't test the entire network, said database will be a hodge podge of short projects and sections scattered throughout the entire state.
Pennsylvania	No.
Washington	-

Table 22A. General Responses to Question 22	
Question 22: Does your agency correlate skid numbers to crash/safety data?	
Responses	**Number of Responses***
Yes	5
No	4
**Washington did not answer.*	

Table 22B. Detailed Responses to Question 22	
Agencies that correlate skid numbers to crash/safety data:	**Agencies that do not correlate skid numbers to crash/safety data:**
Illinois	Indiana
Iowa	Nebraska
Michigan	Pennsylvania*
Missouri	Wisconsin
Ohio	
**No, not correlated. But crash data and skid data are both used to determine potential wet pavement accident cluster sites, as defined in #18.*	

Table 23. Responses to Question 23	
Question 23: How is it correlated?	
Illinois	The Division of Traffic Safety monitors the accident locations.
Iowa	On a statewide system basis, roadway segments with low friction are reviewed for accident history.
Michigan	Individual projects are reviewed on a case by case basis. This is a decentralized process so no database is maintained.
Missouri	A correlation between wet and dry accidents to pavement friction.
Ohio	ODOT has a newly created Office of Safety. They generate a statewide list of high accident locations. Skid testing these locations along with other data gathering is done on an annual basis. See question 18.

Table 24A. General Responses to Question 24	
Question 24: Has your agency conducted any studies related to skid resistance?	
Responses	**Number of Responses***
Yes	7
No	2
**Washington did not answer.*	

Table 24B. Detailed Responses to Question 24	
Agencies that conducted studies related to skid resistance:	**Agencies that have not conducted studies related to skid resistance:**
Indiana	Illinois
Iowa	Michigan*
Missouri	
Nebraska	
Ohio	
Pennsylvania	
Wisconsin	
**No recent studies. Lots of work done in 1960's.*	

Table 25. Responses to Question 25	
Question 25: What were the study objectives?	
Indiana	The study is to upgrade the agency's friction testing program with the state-of-the-art technologies, evaluate network pavement friction performance, and examine the agency's state-of-the-practice in friction testing. To request a report, please contact Karen Hatke at (765) 494-9310 (JTRP, Purdue University). The name of the report is "Upgrading the INDOT Pavement Friction Testing Program." FHWA/IN/JTRP-2003/23.
Iowa	A list of research projects is attached (#3). We are currently working through Iowa State University and the North Central Superpave Center to develop a lab procedure to measure HMA mixture surface friction characteristics (micro/macro texture).
Missouri	The objective of the study was to sample the skid resistance of roadway surfaces in Missouri
Nebraska	Studies have been done at specific accident and high accident sites. Studies have also been done for various surface textures and specific mixes.
Ohio	ODOT has finished a study on polish and lab testing. A new study is starting to look at a quicker approach. http://www.dot.state.oh.us/research/Materials.htm (Blending Proportions of High Skid and Low Skid Aggregate) We have also conducted a limited study of the effects of a certain pavement treatment to on friction over an extended period of time. We are in the midst of studies correlating data between the standard ribbed and smooth tires and studies on the long term performance of Novachip and Smoothseal.
Pennsylvania	We established the SRL requirements, defined in #3. We also established the SRL Evaluation program based on research.
Wisconsin	-Analytical study of surface friction of Wisconsin state trunk highway system pavements. Technical report No. 114. -Statewide pavement friction inventory program -The wet weather accident reduction program - the first seven years Copies of the above reports are available thru WI DOT Library, Rm 803, HFSTB, 4002 Sheboygan Ave, Madison, WI, 53707. All of the above reports are 10 to 20 years old. Other pavement studies have been performed where friction measurement were part of the study.

Table 26. Responses to Question 26	
Question 26: If you have any additional comments, please write them in the space below:	
Illinois	-
Indiana	-
Iowa	Detailed skid resistance research sharply declined as agency liability issues rose in the 1980's. Our current research efforts are focused on maintaining a good pavement friction without specifying "minimum" levels. Your regional state-of-the-practice study should refrain from state-to-state comparison without a FULL understanding of the criteria used in each state.
Michigan	Lots of work done in 1960's tracking different surface types.
Missouri	-
Nebraska	The friction testing program provides information for Pavement Management, and to determine the need for driver advisory signs. The procedures generally follow NCHRP 37.
Ohio	The department is considering reinstating full network level friction testing on an annual basis. But this would require first a large increase in personnel and also require a rigorous policy regarding skid number values and any treatments or pavement decisions they would trigger.
Pennsylvania	-
Washington	-
Wisconsin	-

APPENDIX C
WISDOT FRICTION DATA SHEET

PAVEMENT EVALUATION
(Friction Data Sheet)
E-L 10(R) 282

State of Wisconsin / Department of Transportation
DIVISION OF TRANSPORTATION FACILITIES
Materials Research Unit

Project I.D.		Route:	District:	County: / Code:
Date	Location:			Pavement Width: ☐ 2 Lane ☐ 6 Lane ☐ 4 Lane ☐ Divided
Page	Type of Pavement:			
Test Section I.D.				Length of Test Section:
Field Assignments: Driver: Recorder:	Texture: OT ☐ FA ☐ FC ☐ CA ☐ BL ☐ OP ☐ CL ☐	☐ Clear ☐ Overcast ☐ Hazy ☐ Partly Sunny		Surface Temperature: °F. Posted Speed/s: MPH

Reference: | Reference:

Test No.	Lane: 3 2 1 P D	Direction: N S E W	Test Speed: MPH		Lane: 3 2 1 P D	Direction: N S E W	Test Speed: MPH	
	Zero:		Friction		Zero:		Friction	
	Reference Points	Distance	Force	FN	Reference Points	Distance	Force	FN
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								

NO. TESTS	____		NO. TESTS ____
TEST SPEED	____	$\frac{FN_{40} - FN_{50}}{\text{Change of Speed}}$ = S.G.	TEST SPEED ____
RANGE	____		RANGE ____
STD. DEV.	____		STD. DEV. ____
MEAN	☐	Speed Gradient ____	MEAN ☐

COMMENTS:

APPENDIX D

SUPPLEMENTAL SPECIFICATIONS FOR SECTION 1004

State of Illinois
Department of Transportation

SUPPLEMENTAL SPECIFICATION
FOR
SECTION 1004. COARSE AGGREGATE

This Supplemental Specification amends the provisions of the Standard Specifications for Road and Bridge Construction, adopted January 1, 2002 and shall be construed to be a part thereof, superceding any conflicting provisions thereof applicable to the work under the contract.

1004.01 Materials. Add the following paragraph after the third paragraph of Article 1004.01(c):

" Any aggregate produced under the Department's current Policy Memorandum, 'Aggregate Gradation Control System (AGCS)', shall meet the gradation requirements set under the AGCS program."

1004.03 Coarse Aggregate for Bituminous Courses. Revise subparagraph (a) of this Article to read:

" (a) Description. The coarse aggregate for bituminous courses shall be according to the following table.

Class	Mixture	Aggregates Allowed
A	Seal or Cover	Gravel Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag Crushed Concrete
B		Gravel Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF) Crushed Concrete
I and Superpave	A or B and IL-25.0 or IL-19.0 Binder	Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF)
I and Superpave	C Surface	Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag (except when used as leveling binder) Gravel (only when used in Class I Type 3CL or Superpave IL-9.5L)
I and Superpave	D Surface	Crushed Gravel Crushed Stone (other than Limestone) Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag Limestone may be used in Mixture D if blended by volume in the following coarse aggregate percentages: Up to 25% Limestone with at least 75% Dolomite. Up to 50% Limestone with at least 50% any aggregate listed for Mixture D except Dolomite. Up to 75% Limestone with at least 25% Crushed Slag (ACBF) or Crushed Sandstone.

Class	Mixture	Aggregates Allowed
I and Superpave	E Surface	Crushed Gravel Crushed Stone (other than Limestone and Dolomite) Crushed Sandstone No Limestone. Dolomite may be used in Mixture E if blended by volume in the following coarse aggregate percentages: Up to 75% Dolomite with at least 25% Crushed Sandstone, Crushed Slag (ACBF) or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 25% to a maximum of 75% of either Slag by volume. Up to 50% Dolomite with at least 50% of any aggregate listed for Mixture E. If required to meet design criteria, Crushed Gravel or Crushed Stone (other than Limestone or Dolomite) may be blended by volume in the following coarse aggregate percentages: Up to 75% Crushed Gravel or Crushed Stone (other than Limestone or Dolomite) with at least 25% Crushed Sandstone, Crushed Slag (ACBF), or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 25% to a maximum of 50% of either Slag by volume.
I and Superpave	F Surface	Crushed Sandstone No Limestone. Crushed Gravel or Crushed Stone (except Limestone) may be used in Mixture F if blended by volume in the following coarse aggregate percentages: Up to 50% Crushed Gravel or Crushed Stone with at least 50% Crushed Sandstone, Crushed Slag (ACBF), or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 50% to a maximum of 75% of either Slag by volume”

APPENDIX E
IOWA SPECIFICATIONS 2303.02

Section 2303. Hot Mix Asphalt Mixtures.

2303.01 DESCRIPTION.

This work shall consist of mixture design, production, placement, and compaction of HMA using proper quality control practices for the construction of surface, intermediate, or base course on a prepared sub base, base, or pavement, to the proper dimensions specified in the contract documents.

The Contractor shall be responsible for all aspects of the project, provide quality control management and testing, and maintain the quality characteristics specified.

Quality Management - Asphalt (QM-A) shall apply to contracts with HMA quantities of 5000 tons (5000 Mg) or greater and all Interstate contracts. The Contractor shall follow the procedures and meet the criteria established in Article 2303.02, Section 2521, and Materials I.M. 510 and 511.

For contracts with less than 5000 tons (5000 Mg) quality control will be the responsibility of the Engineer. The Contractor shall be responsible for the mix design. This does not change the mix requirements from gyratory to Marshall, unless specified in the contract documents.

2303.02 MATERIALS AND EQUIPMENT.

Materials used in these mixtures shall meet the following requirements:

A. Asphalt Binder.

The Performance Graded asphalt binder, PG XX -XX, will be specified in the contract documents to meet the climate, traffic, and pavement conditions. The asphalt binder shall meet the requirements of Section 4137.

B. Aggregates.

1. Individual Aggregates.

Virgin mineral aggregate shall meet the following requirements:

VIRGIN MINERAL AGGREGATES		
Mixture	**Aggregate Type**	**Aggregate Requirements**
Base	Type B	Section 4126[1] & 4127
Intermediate and Surface	Type B	Section 4126
Intermediate and Surface	Type A	Section 4127
[1]When the size of the mixture is not specified, 1/2 inch (12.5 mm) shall be used.		

When frictional classification of the coarse aggregate is required, the contract documents will specify the friction level and location. The friction aggregate shall be furnished from sources identified in Materials I.M. T203.

For friction classification L-2, at least 80% of the combined aggregate retained on the No. 4 (4.75 mm) sieve shall be Type 4 or better friction aggregate; and at least 25% of the combined aggregate retained on the No. 4 (4.75 mm) sieve shall be Type 2 or better friction aggregate.

For friction classification L-3, at least 80% of the combined aggregate retained on the No. 4 (4.75 mm) sieve shall be Type 4 or better friction aggregate; and at least 45% of the combined aggregate retained on the No. 4 (4.75 mm) sieve shall be Type 3 or better friction aggregate. If Type 2 is used in place of Type 3, the minimum shall be 30% of the combined aggregate retained on the No. 4 (4.75 mm) sieve.

For friction classification L-4, at least 50% of the combined aggregate retained on the No. 4 (4.75 mm) sieve shall be Type 4 or better friction aggregate.

2. Blended Aggregates.

The blended aggregates shall meet the combined aggregate requirements in Materials I.M. 510.

When mixtures include RAP, the blended mineral aggregate gradation shall be a mixture of extracted RAP aggregate combined with virgin aggregate.

C. Recycled Asphalt Pavement.

1. Designated RAP.

When RAP is taken from a project, or is furnished by the Contracting Authority, the contract documents will indicate quantity of RAP expected to be available. The Contractor is responsible for salvaging this material unless otherwise specified in the contract documents. The RAP not used shall be incorporated into other parts of the project or placed in active stockpiles as directed in the contract documents.

The Contracting Authority will test samples of this material. For mix design purposes, the amount of asphalt binder in the RAP will be based on extraction tests. The Contractor shall designate the exact proportions of RAP material in the hot mix within the allowable range.

When the work is completed, the Contractor shall return unused material to the stockpile or other designated location, rebuild the stockpile, and restore the area, in accordance with Article 1104.08.

Test information, if known, will be included in the contract documents.

2. Certified RAP

The RAP shall be from a known source and of the proper quality for the intended use, with no material added from other sources during the time in stockpile. The Contractor shall certify to this before use. RAP from not more than two known sources at a time will be allowed.

Certified RAP may be used in the base and intermediate course of mixes for which the RAP aggregate qualifies. RAP may also be used in surface courses when authorized by the Engineer. Not more than 30% of the asphalt binder in a final surface course mixture shall come from the RAP.

A certified RAP stockpile shall be sealed or protected in accordance with Materials I.M. 505.

3. Unclassified RAP.

Up to 10% of unclassified RAP may be incorporated into intermediate mixes for under 3,000,000 ESALs and all base mixes with the following safeguards:

a. Unclassified RAP shall not be used in surface courses.

b. Unclassified RAP shall not be used in intermediate or base mixtures containing designated or certified RAP.

c. The Engineer will inspect the unclassified RAP stockpile visually for uniformity. Unclassified RAP stockpiles containing concrete chunks, grass, dirt, wood, metal, coal tar, or other foreign or environmentally restricted materials shall not be used, unless approved by the Engineer. If foreign material is discovered in any unclassified stockpile, the Engineer may stop the continued use of the pile.

d. Representative samples will be taken by the Engineer. These samples are to be tested for gradation and asphalt content.

e. No credit will be given for crushed particles.

f. Stockpiles, when used, shall be worked in such a manner that the materials removed are representative of a cross section of the pile as approved by the Engineer.

D. Hot Mix Asphalt Mixture.

The surface course is the upper lift for a wearing surface of a designated thickness. The intermediate course is the next lower lift or lifts of a designated thickness. Leveling, strengthening, and wedge courses shall be of the intermediate course mixture. The base course is the lift or lifts placed on a prepared sub grade or sub base.

The job mix formula (JMF) is the percentage of each material, including the asphalt binder, to be used in the HMA mixture. The JMF gradation shall be within the control points specified for the particular mixture designated and shall establish a single percentage of aggregate passing each required sieve size.

If the asphalt binder demand for the combination of aggregates submitted for an acceptable mix design exceeds the basic asphalt binder content by more than 0.75%, the mix design will include an economic evaluation prepared by the Contractor. This evaluation will be based on past job mix history, possible aggregate proportion changes, and aggregate availability and haul costs for any changes or substitutions considered.

The basic asphalt binder content is the historical, nominal mixture asphalt binder content, expressed as percent by weight (mass) of the asphalt binder in the total mixture. The following values, based on mixture size and type, shall apply.

BASIC ASPHALT BINDER CONTENT (%)					
Size	**Aggr. Type**	**1 inch (25 mm)**	**3/4 inch (19 mm)**	**1/2 inch (12.5 mm)**	**3/8 inch (9.5 mm)**
Intermediate and Surface	Type A	4.75	5.50	6.00	6.00
Intermediate and Surface	Type B	5.25	5.75	6.00	6.25
Base	Type B	5.25	6.00	6.00	6.25

The HMA mixture designed shall meet gyratory design and mixture criteria corresponding to the design level specified in the contract documents. The Engineer may approve the substitution of any mixture which meets requirements for a higher mixture than specified in the contract documents at no additional cost to the Contracting Authority. Shoulders placed as a separate operation shall be HMA 1,000,000 ESAL base mixture. For outside shoulders on Interstate projects, the Contractor has the option to substitute the mainline intermediate or surface mixture for a specified base mixture at the Contractor's expense.

The Contractor shall prepare gyratory HMA mixture designs for all base, intermediate, and surface mixtures. The gyratory design procedure used shall follow the procedure outlined in Materials I.M. 510. The gyratory mixture designs submitted shall comply with Materials I.M. 510. The gyratory compactor used for design and field control shall meet the AASHTO protocol for Superpave gyratory compactors. Compactors for which compliance with this protocol is pending may be used at the discretion of the District Materials Engineer.

E. Other Materials.

1. Tack Coat.

Tack coat may be SS-1, SS-1H, CSS-1, or CSS-1H. Mixing of CSS and SS grades will not be permitted. RC-70 and MC-70 may also be used after October 1, at the Contractor's option.

2. Hydrated Lime.

Hydrated lime shall meet the requirements of AASHTO M 303, Type I. Section 4193 shall not apply. Hydrated lime will not be considered part of the aggregate when determining the job mix formula and the filler/bitumen ratio.

On Interstate highways, if 25% or more of the plus No. 4 (4.75 mm) (virgin and RAP) aggregate is gravel, quartzite, granite, trap rock, steel slag, or other siliceous aggregate (not a limestone or dolomite), hydrated lime will be required in the affected intermediate and surface course mixture.

On Primary highways other than Interstate highways, if 25% or more of the plus No. 4 (4.75 mm) (virgin and RAP) aggregates or more than 40% of the total (virgin and RAP) aggregates is quartzite, granite, or other siliceous aggregates (not limestone or dolomite) which is obtained by crushing from ledge rock, hydrated lime will be required in the affected mixtures requiring Type A aggregate.

Hydrated lime will not be required for base repair, patching, or temporary pavement.

When hydrated lime is required based on aggregate source, the Contractor may arrange for Superpave moisture sensitivity evaluation of the proposed HMA mixture design according to AASHTO T 283, "Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage." When results of this evaluation indicate more than 80% tensile strength retained (TSR), hydrated lime will not be required. Confirmation of AASHTO T 283 test results will be completed by the Central Materials Laboratory during placement of the test strip.

3. Sand for Tack Coats.

Sand shall meet requirements of Section 4109, Gradation No. 1.

4. Fabric Reinforcement.

Fabric reinforcement shall meet requirements of Article 4196.01, D.

F. Equipment

The Contractor shall provide sufficient equipment of the various types required to produce, place, and compact each layer of HMA mixture as specified.

Equipment shall meet requirements of Section 2001 with the following modifications:

1. Plant Calibration.

Each plant scale and metering system shall be calibrated before work on a contract begins. Calibration equipment shall meet the manufacturer's guidelines and Materials I.M. 508. The Engineer may waive calibration of permanent plant scales when a satisfactory operational history is available. The Engineer may require any scale or metering system to be recalibrated if operations indicate it is necessary. Calibration data shall be available at the plant.

Each aggregate feed shall be calibrated throughout an operating range wide enough to cover the proportion of that material required in the JMF. A new calibration shall be made each time there is a change in size or source of any aggregate being used.

For continuous and drum mixing plants, the asphalt binder metering pump shall be calibrated at the operating temperature and with the outlet under pressure equal to that occurring in normal operations.

2. Paver.

Article 2001.19 shall apply. Spreaders, as described in Article 2001.13, D, may be used to place paved shoulders. Spreaders used to place the final lift of paved shoulders shall meet additional requirements of Article 2001.19.

3. Rollers.

For initial and intermediate rolling, self-propelled, steel tired, pneumatic tired, or vibratory rollers meeting requirements of Article 2001.05, B, C, or F, shall be used. Their weight (mass) or tire pressure may be adjusted when justified by conditions.

For finish rolling, self propelled, steel tired rollers or vibratory rollers in the static mode meeting requirements of Article 2001.05, B or F, shall be used.

4. Scales.

Article 2001.07, B, shall apply to all paving operations regardless of the method of measurement.

APPENDIX F

IOWA ASPHALT SELECTION GUIDE

IOWA DOT
HOT MIX ASPHALT SELECTION GUIDE
(July 2001)

DAILY ESALs	20yr DESIGN ESALs (Millions)	LAYER DESIGNATION	LIFT THICKNESS MIN	LIFT THICKNESS MAX	MIX SIZE (1)	BID ITEM DESIGNATION	BASIC BINDER CONTENT
< 15	< 0.1	SURFACE	1.5 (40mm)	2 (50mm)	½" (12.5mm)	HMA 100K	6.00
		INTERMEDIATE	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
		BASE	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
15 – 40	0.1 - 0.3	SURFACE	1.5 (40mm)	2 (50mm)	½" (12.5mm)	HMA 300K	6.00
		INTERMEDIATE	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
		BASE	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
40 – 140	0.3 – 1.0	SURFACE	1.5 (40mm)	2 (50mm)	½" (12.5mm)	HMA 1M	6.00
		INTERMEDIATE	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
		BASE (2)	1.5 (40mm)	3 (80mm)	½" (12.5mm)		
140 – 400	1.0 – 3.0	SURFACE	1.5 (40mm)	2 (50mm)	½" (12.5mm)	HMA 3M	6.00
		INTERMEDIATE	2 (50mm)	3.5 (90mm)	¾" (19.0mm)		5.50
		BASE	2 (50mm)	3.5 (90mm)	¾" (19.0mm)		6.00
400 – 1350	3.0 – 10.0	SURFACE	1.5 (40mm)	2 (50mm)	½" (12.5mm)	HMA 10M	6.00
		INTERMEDIATE	2 (50mm)	3.5 (90mm)	¾" (19.0mm)		5.50
		BASE	3 (75mm)	4 (100mm)	1" (25.0mm)		5.25
	10.0 – 30.0	SURFACE	2 (50mm)	2 (50mm)	¾" (19.0mm)	HMA 30M	5.50
		INTERMEDIATE	2 (50mm)	3.5 (90mm)	¾" (19.0mm)		5.50
		BASE	3 (75mm)	4 (100mm)	1" (25.0mm)		5.25
	30.0 – 100.0	SURFACE	2 (50mm)	2 (50mm)	¾" (19.0mm)	HMA 100M	5.50
		INTERMEDIATE	2 (50mm)	3.5 (90mm)	¾" (19.0mm)		5.50
		BASE	3 (75mm)	4 (100mm)	1" (25.0mm)		5.25

(1) On urban routes with adjacent pedestrian traffic, a 3/8" surface mix size may be considered (1" minimum lift thickness).
(2) This design should be used for paved shoulders that are paved separately.

TOTAL ADT	TRUCK ADT	FRICTION LEVEL
0–2000	0–300	no special friction required
2000-5000	300–500	L - 4
5000-10000	500–2000	L - 3
>10000 all Interstate	>2000 all Interstate	L - 2

APPENDIX G

MISSOURI SPECIFICATIONS 403.3.5

SECTION 403

ASPHALTIC CONCRETE PAVEMENT

403.3.5 Surface Mixtures. Design level B surface mixtures, except as described in Sec 403.15.3, containing limestone coarse aggregate shall contain a minimum amount of non-carbonate aggregate. The LA abrasion values, AASHTO T 96, of the limestone will determine the type and amount of non-carbonate aggregate required as shown in the table below. The LA abrasion value will be determined from the most recent source approval sample. In lieu of the above requirements, the aggregate blend shall have an acid insoluble residue (AIR), MoDOT Test Method TM-76, meeting the plus No. 4 (4.75 mm) criteria of crushed non-carbonate material. Non-carbonate aggregate shall have an AIR of at least 85 percent insoluble residue.

Coarse Aggregate (+ No. 4)	*Minimum Non-Carbonate by Volume*
Limestone, LA ≤ 30	30% Plus No. 4
Limestone, LA > 30	20% Minus No. 4
Dolomite	No Requirement

403.15.3 Non-traffic Areas. Sec 403 mixtures used for surfacing medians and similar areas, shoulders adjacent to rigid or flexible pavement and shoulders adjacent to resurfaced pavement shall be compacted to the specified densities for the mixture. Once an established rolling pattern has been demonstrated to provide the required density for shoulders, at the engineer's discretion, the pattern may be used in lieu of density tests provided no changes in the material, typical location or temperatures are made. Regardless of the method, density will still be required and subject to testing as deemed necessary by the engineer. In lieu of roller and density requirements, temporary bypasses to be maintained at the expense of the contractor shall be thoroughly compacted. The rolling shall be performed at proper time intervals and shall be continued until there is no visible evidence of further consolidation.

APPENDIX H

MICHIGAN SPECIFICATIONS MTM 111

MICHIGAN TEST METHOD
FOR
DETERMINING AN AGGREGATE WEAR INDEX (AWI)
BY WEAR TRACK POLISHING TESTS

1. Scope

   1.1. This method covers the determination of an Aggregate Wear Index (AWI) for aggregates or blends of aggregates proposed to be used in HMA wearing course mixtures.

   1.2. The AWI determined by this method is the result of wear track polishing tests conducted on the exposed aggregate or blend of aggregates cast into test slabs. The AWI represents the average initial peak force measurement determined on duplicate test slabs after four million wheel passes of wear track polishing. Only material finer than the 3/8 inch (9.5 mm) sieve and coarser than the No. 4 (4.75 mm) sieve is to be used in the wear track tests for aggregates to be used in HMA wearing course mixtures. Sizes of aggregates may be adjusted for special investigations or research projects.

2. Referenced Documents

   2.1. *ASTM Standards*:

   C 136 Test for Sieve Analysis of Fine and Coarse Aggregates
   E 29 Standard Recommended Practice for Indicating Which Places of Figures are to be Considered Significant in Specific Limiting Values

   2.2. *MDOT Research Reports:*
   Research Report R-1098 MDOT Circular Wear Track-Results of Preliminary Aggregate Polishing Tests

3. Significance and Use

   3.1. Aggregates that are readily traffic-polished have been shown to contribute to the wet-road slipperiness of HMA pavements. Aggregate Wear Index ratings are used in conjunction with traffic count data to develop HMA top course mixtures that will resist the anticipated amount of traffic polishing for the design life of pavements.

   3.2. AWI determinations by wear track testing are conducted on quarried carbonates, slags and other aggregates that cannot be analyzed by MTM 112, Test Method for Determining an Aggregate Wear Index (AWI) From Sample Petrographic Composition and Wear Track AWI Factors.

4. Apparatus and Supplies

4.1. Double mold and vibratory table for casting test slabs (Figure 1).

4.2. Concrete mixer and assorted tools for casting of test slabs.

4.3. Medium-etch concrete retarding agent for coating the molds to produce an exposed aggregate surface.

4.4. Sand, oven-dry and graded to Michigan Aggregate No. 2NS with material coarser than the No. 4 (4.75 mm) sieve removed.

4.5. Portland cement, Type I.

4.6. Molds and vibratory table for casting wear track test slabs, (Figure 1).

4.7. Circular Wear Track Assembly, (Figure 2).

4.8. Static Friction Tester with a load sensor calibrated to detect force impulses up a maximum of 1000 lbf, and a recording oscillograph or other device with sensitivity and range to record the friction test measurements, (Figure 3).

5. Samples

5.1. A minimum quantity of 60 pounds of aggregate from produced stock from each source to be evaluated shall be submitted for tests.

5.2. Wash each sample, then sieve according to ASTM C 136, using the 3/8 inch (9.5 mm) and No. 4 (4.75 mm) sieves.

5.3. Cast duplicate test slabs for each sample, using the sample material finer than the 3/8 inch (9.5 mm) sieve and coarser than the No. 4 (4.75 mm) sieve, following the detailed procedure outlined in MDOT Research Report R-1098.

6. Procedure

6.1. Test the prepared test slabs, after curing, on the static friction tester to obtain initial friction values.

6.2. Place a complete set of 16 test slabs on the wear track, and polish for increments of 500,000 wheel passes, with friction testing on the static tester at the end of each increment, through four million wheel passes, as described in MDOT Research Report R-1098.

7. Calculations

7.1. Determine the Aggregate Wear Index (AWI) for each aggregate sample by plotting the average friction values versus wheel passes at the 1.5 million through 4.0 million wheel pass increments. Report the least-square best fit friction value for each sample at 4.0 million wheel passes as the sample AWI.

7.1.1. Computations shall be made according to the rounding off procedure described in ASTM E 29.

8. Report

8.1. Include the AWI values determined for the submitted aggregates in standard test reports. Wear track AWI values are also to be entered into the MDOT Aggregate Source AWI Summary.

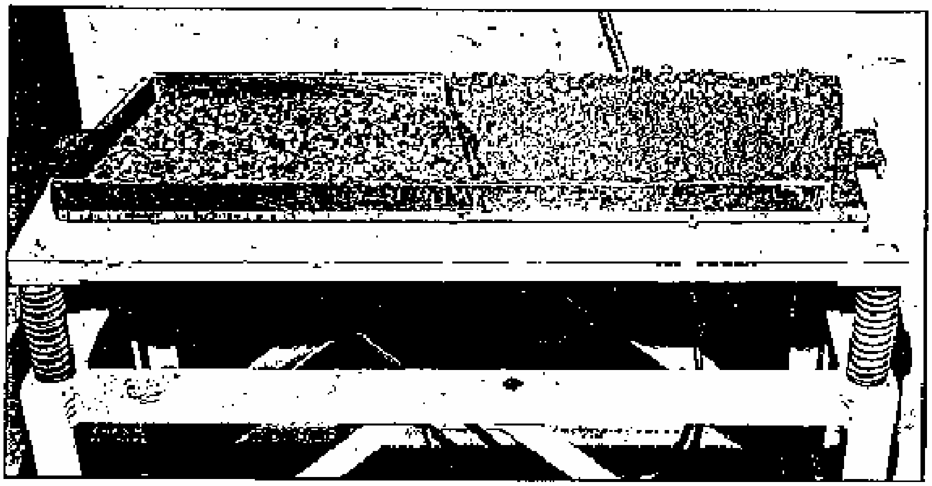

Figure 1. Vibratory table and test slab molds.

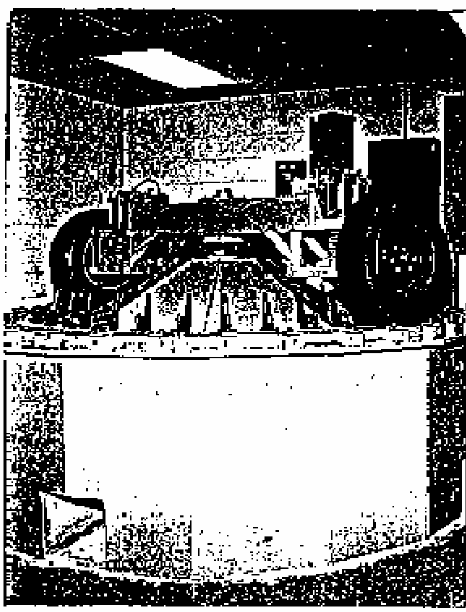

Figure 2. MDOT wear track.

Figure 3. MDOT static friction tester.

APPENDIX I

MICHIGAN SPECIFICATIONS MTM 112

MICHIGAN TEST METHOD FOR DETERMINING AN AGGREGATE WEAR INDEX (AWI) FROM SAMPLE PETROGRAPHIC COMPOSITION AND WEAR TRACK AWI FACTORS

1. Scope

1.1. This method covers the determination of an Aggregate Wear Index (AWI) for gravel aggregates or blends of aggregates proposed to be used in HMA wearing course mixtures.

1.2. The AWI determined by this method is computed from the petrographic composition of the aggregate sample and reference AWI factors established for rock types contained in Michigan glacial gravels. The reference AWI factors were determined on 100 percent crushed particles.

1.3. The final computed AWI is based upon a grading-weighted summation of the calculated AWI values for the rock type categories present in the sample. In the case of blended aggregates, the AWI of the composite material is determined from the grading-weighted AWI of each aggregate and the given blend ratio.

1.4. The final computed AWI of the sample is reduced by a factor of 0.26 percent for each percent of uncrushed material in the sample, based upon wear track tests comparing the polishing resistance of crushed versus uncrushed aggregates with similar composition.

1.5. The AWI is determined for only the sample fraction coarser than a No. 4 (4.75 mm) sieve. The sample fraction coarser than the No. 4 (4.75 mm) sieve may be changed for special investigations or research projects.

2. Referenced Documents

2.1. *ASTM Standards*:

C 136 Test for Sieve Analysis of Fine and Coarse Aggregates

C 294 Descriptive Nomenclature of Constituents of Natural Mineral Aggregates

C 95 Practice for Petrographic Examination of Aggregate for Concrete

C 702 Methods for Reducing Field Samples of Aggregate to Testing Size

E 11 Wire Cloth Sieves for Testing Purposes

E 29 Standard Recommended Practice for Indication Which Places of Figures Are to be Considered Significant in Specified Limiting Values

2.2. *MDOT Standards*:

MTM 111 Test Method for Determining an Aggregate Wear Index (AWI) by Wear Track Polishing Tests

2.3. *MDOT References*:

Research Report R-1098	MDOT Circular Wear Track-Results of Preliminary Aggregate Polishing Tests
Research Report R-1232	An Aggregate Wear Index Reduction Factor for Uncrushed Material in Gravel

3. Significance and Use

3.1. This procedure is used to evaluate the polish resistance of gravel coarse aggregates for HMA top course mixtures. The evaluation is based on wear track tests of aggregates tested according to MTM 111 and as reported in Research Report R-1098.

3.2. This procedure was developed as a rapid means for determining the Aggregate Wear Index of typical gravel aggregates as an alternative to the actual wear track procedure that requires three months for completion. The procedure may also be used to determine the Aggregate Wear Index of recycled asphalt and Portland cement concrete pavement material.

3.3. Wear track testing as reported in Research Report R-1232 indicates that naturally smooth aggregate particle surfaces polish to a greater extent than the fractured surfaces of crushed particles. A reduction factor developed from wear track comparison tests is included to adjust the AWI results for samples with rounded (uncrushed) particles.

4. Apparatus and Supplies

4.1. *The following items are recommended for the proper analysis of the aggregate samples*.

- Sample splitter
- Mechanical sieve shaker
- Sieves conforming to ASTM E 11, with openings of 1 inch (25.0 mm), 3/4 inch (19.0 mm), _ inch (12.5 mm), 3/8 inch (9.5 mm) and No. 4 (4.75 mm)

5. Computations

5.1. Computations shall be made according to the rounding-off procedure in ASTM E-29.

6. Samples and Sample Preparation

6.1. Samples submitted for AWI determination shall be accompanied by proper identification per MDOT procedures.

6.2. minimum of 1500 grams of aggregate coarser than a No. 4 (4.75 mm) sieve is required for an AWI determination. A 60-pound sample of dense-graded aggregate provides adequate material. Approximately 1500 grams of aggregate coarser than a No. 4 (4.75 mm) sieve are also required for an AWI determination on material extracted from HMA pavements.

6.3. A separate sample must be submitted for each aggregate to be combined in an blend. Sample information must include the blend ratio, the total amounts retained above the No. 4 (4.75 mm) sieve, and the percent crushed for each aggregate in the blend.

6.4. Prepare a sieve analysis sample of approximately 1500 grams by reducing the sample to the required quantity according to ASTM C 702. Samples containing clay lumps must be washed and dried before sieving.

6.5. Prepare a petrographic analysis sample of 300 representative particles retained in each of the following sieve fractions:

1 inch (25 mm) to 3/4 inch (19 mm)

3/4 inch (19 mm) to ½ inch (12.5 mm)

½ inch (12.5 mm) to 3/8 inch (9.5 mm)

3/8 inch (9.5 mm) to No. 4 (4.75 mm)

6.5.1. Material from the sieve analysis sample fraction may be incorporated in the petrographic analysis sample fraction after completion of the sieve analysis. Use all particles in a size fraction if less than 300 particles are present. If the sample contains less than 30 particles in a size fraction, that material shall be combined with the next smaller size fraction before selection of particles for examination.

7. <u>Sieve Analysis</u>

7.1. Conduct a sieve analysis of the sample portion prepared in 6.4, following ASTM C 136. Determine the grading on the basis of 100 percent retained on the No. 4 (4.75 mm) sieve, as shown in Table 1. Since the AWI determination is conducted on the sample fraction coarser than the No. 4 (4.75 mm) sieve, material passing the No. 4 (4.75 mm) sieve may be discarded after the sieve analysis if not needed for other tests.

TABLE 1:
EXAMPLE DATA SHEET FOR AWI SIEVE ANALYSIS

Sieve Size Opening	Grading of Sample, Amount Retained on Individual Sieve		Grading of AWI Fraction Amount Retained on Individual Sieve	
	Wt., g	Percent	Wt., g.	Percent
1 inch (25 mm)	0	0.0	0	0.0
3/4 inch (19 mm)	0	0.0	0	0.0
½ inch (12.5 mm)	88	3.5	88	5.0
3/8 inch (9.5 mm)	620	24.8	620	35.2
No. 4 (4.75 mm)	1052	42.1	1052	59.8
Pan	740	29.6		
TOTALS	2500	100.0	1760	100.0

8. <u>Petrographic Analysis</u>

8.1. Determine the petrographic composition of the sample portion prepared in 6.5, following ASTM C 294 and ASTM C 295, as shown in Table 2, using the rock type categories indicated. Additional categories may be used, providing wear track factors are available from actual wear track tests.

TABLE 2:
EXAMPLE DATA SHEET FOR GRADING-WEIGHTED PETROGRAPHIC COMPOSITION

Rock Type	Sieve Fraction Analyzed								Grading-Weighted Petrographic Sample Composition, Percent
	1 inch (25 mm) to 3/4 inch (19 mm)		3/4 inch (19.0 mm) to ½ inch (12.5 mm)		½ inch (12.5 mm) to 3/8 inch (9.5 mm)		3/8 inch (9.5 mm) to No. 4 (4.75 mm)		
	Count	Weighted Percent	Count	Weighted Percent	Count	Weighted Percent	Count	Weighted Percent	
Igneous/ Metamorphic	0	0.0	60	1.0	57	6.7	67	13.4	21.1
Sedimentary									
Carbonates	0	0.0	230	3.8	225	26.4	217	43.2	73.4
Sandstone	0	0.0	0	0.0	5	0.6	14	2.8	3.4
Siltstone	0	0.0	0	0.0	0	0	0	0.0	0.0
Shale	0	0.0	0	0.0	0	0	0	0.0	0.0
Clay Ironstone	0	0.0	0	0.0	0	0	0	0.0	0.0
Chert	0	0.0	10	0.2	13	1.5	2	0.4	2.1
AWI Sample Grading, %		0.0		5.0		35.2		59.8	100.0
Particle Counts	0		300		300		300		

8.1.2 Calculate a grading-weighted sample petrographic composition, shown in Table 2, as follows:

Grading-weighted composition, percent = (P/TP) x F

Where: P = Particle count of rock type, from 6.5

TP = Total particles in sieve fraction, from 6.5

F = AWI sample grading, percent, from 7.1

8.2. Either use the percent crushed stated on the mix design communication sheet or calculate the grading-weighted uncrushed particle content in each size fraction of the sieve analysis sample prepared in 6.4, shown in Table 3, as follows:

Grading-weighted uncrushed content, percent = U x F

Where: U = Uncrushed content in sieve fraction, percent, from 8.1

F = AWI sample grading, percent, from 7.1

8.2.1. Uncrushed particles are those which have no fractured faces, except that all sandstone particles are to be considered as crushed particles. The grading weighted uncrushed composition is the summation of the weighted percent values.

8.2.2. Crag, a lime-cemented conglomerate of sand and gravel particles common in some gravel deposits, presents a special case when determining crushed content. By convention, when the largest rock fragment in a crag particle comprises less than 50 percent of the particle, the crag particle is classified as a crushed particle; when the largest rock fragment comprises greater than 50 percent of the particle, and that fragment, by itself, would be retained on a No. 4 (4.75 mm) the crag particle is considered as a crushed particle if the rock fragment has a fractured appearance. When the largest rock fragment would pass a No. 4 (4.75 mm) the crag particle is included in the sandstone rock type category.

TABLE 3:
EXAMPLE DATA SHEET FOR GRADING-WEIGHTED UNCRUSHED CONTENT

Material in Sample	Sieve Fraction Analyzed								Grading-Weighted Uncrushed Content, Percent
	1 inch (25 mm) to 3/4 inch (19 mm)		3/4 inch (19 mm) to ½ inch (12.5 mm)		½ inch (12.5 mm) to 3/8 inch (9.5 mm)		3/8 inch (9.5 mm) to No. 4 (4.75 mm)		
	Percent by Weight	Weighted Percent	Percent by Weight	Weighted Percent	Percent by Weight	Weighted Percent	Percent by Weight	Weighted Percent	
Crushed	0	0.0	80	4.0	70	24.6	62	37.1	65.7
Uncrushed	0	0.0	20	1.0	30	10.6	38	22.7	34.3
AWI Sample Grading, percent		0.0		5.0		35.2		59.8	100.0
Material Analyzed, percent	0		100		100		100		

9. AWI Determination

9.1. Calculate a grading-weighted sample AWI from the results of 7.1 and the wear track AWI factors for each rock type as shown in Table 4. Note that the calculated AWI for each rock type category is computed from the sample percent expressed as a decimal, as follows:

Grading-weighted rock type AWI = RT x AWIF

Where: RT = Grading-weighted rock type sample content, percent, from Table 2.

AWIF = Wear track AWI factor

The grading-weighted sample AWI is the summation of the calculated AWI values determined for the rock type categories contained in the sample.

9.1.1. The rock types listed in Table 4 are typical of Michigan gravel deposits. If particles of indeterminate type are encountered in samples, it is appropriate to classify them with known types with similar texture and hardness for the purpose of the AWI determination.

9.1.2. Quarried carbonate material of indeterminate origin contained in samples of recycled asphalt or Portland cement concrete shall be assigned the wear track control limestone AWI factor of 170.

9.1.3. Sand-cement fragments in samples of recycled Portland cement concrete shall be assigned the wear track AWI factor of 360. Crag (lime-cemented sand and gravel) shall be assigned the wear track AWI factor of 435.

TABLE 4:
DATA SHEET FOR DETERMINATION OF GRADING-WEIGHTED AWI

COMPOSITION OF SAMPLE			
Rock Type	Percent	Wear Track AWI Factor	Calculated AWI
IGNEOUS/METAMORPHIC	21.1	370	78.1
SEDIMENTARY			
Carbonates	73.4	250	183.5
Sandstone	3.4	490	16.7
Siltstone	0.0	475	0.0
Shale	0.0	335	0.0
Clay Ironstone	0.0	275	0.0
Chert	2.1	345	7.2

Grading-Weighted Sample AWI (E) 285.5
Grading-Weighted Uncrushed Particles, Percent (U) 34
AWI Reduction Based on Uncrushed Reduction in Percent...................................8.8
Adjusted Sample AWI..260.3

9.2. *Calculate an adjusted sample AWI, shown in Table 4, as follows:*

Adjusted sample AWI = E (1 - 0.0026U)

Where: E = Grading-weighted sample AWI, from 9.1
U = Grading-weighted uncrushed content, percent, from Table 3

9.3. *Calculate the AWI for a blend, as follows:* **Refer to Note at the end of this section.**

$$\text{Blend AWI} = \frac{[(AWI_A \times P_{A4} \times P_A \times F_A) + (AWI_B \times P_{B4} \times P_B \times F_B)]}{[(P_{A4} \times P_A \times F_A) + (P_{B4} \times P_B \times F_B)]}$$

Where: AWI_A = AWI of Aggregate A

AWI_B = AWI of Aggregate B

P_{A4} = percent of Aggregate A > No. 4 (4.75 mm) in blend

P_{B4} = percent of Aggregate B > No. 4 (4.75 mm) in blend

P_A = percent of Aggregate A in blend

P_B = percent of Aggregate B in blend

F_A = specific gravity factor of Aggregate A

F_B = specific gravity factor of Aggregate B

9.3.1. The percentages of aggregates in the blend are obtained from the blend ratio and gradation information submitted with the sample.

9.3.2. The use of specific gravity factors is required in AWI computations for blends since the AWI is related to the surface area of the aggregates exposed on a pavement surface, which is related to the volumes of the various aggregate components in the blend. A specific gravity adjustment for blend aggregates with markedly different specific gravity values is particularly important. In general, the use of the given specific gravity factors in lieu of actual values determined for the aggregate components in a blend will result in a blend AWI that is sufficiently accurate for acceptance/rejection purposes. However, if the specific gravities of the blend aggregates differ considerably from those indicated, actual specific gravity factors should be determined.

The following specific gravity factors to be used are based upon averages of typical samples analyzed in the laboratory:

	Bulk Specific Gravity Gravity	Specific Gravity Factor
Natural Aggregates	2.68	1.0
Blast Furnace Slag	2.24	1.2
Steel Furnace Slag	3.24	0.8

Note: Calculate the AWI for the blend using the cumulative percent retained on the No. 16 sieve for each of the individual aggregates as follows:

Quarried Stone, Mine Rock, and Slag sources will use the AWI number established by MDOT's circular wear track testing.

Natural Aggregate Sand and Gravel sources with established nomographs will run the Michigan Test Method for Measuring Fine Aggregate Angularity (MTM 118) on all aggregates with more than 80 percent passing the No. 4 sieve. The percent crushed in these fine aggregates for use in determining the AWI value from the nomograph is obtained from Table 5.

TABLE 5:
AWI VALUES

ANGULARITY INDEX	PERCENT CRUSHED
< = 3.0	30
> 3.0 to < = 4.0	70
> 4.0	95

Natural Aggregate Sand and Gravel sources without a nomograph must follow the Procedures Manual for Mix Design Processing for submitting aggregate samples, but must include approximately 200 grams of each of the No. 8 and No. 16 fractions. MDOT will conduct the necessary tests and report the results to the aggregate supplier. Natural Aggregate Sand and Gravel sources with an established nomograph and less than 80 percent passing the No. 4 sieve, will use the percent crushed of the retained No. 4 aggregate to determine the appropriate AWI from the nomograph.

10. Report

10.1. A report summarizing the results of the AWI determination shall include the grading weighted petrographic composition, computation of the grading-weighted AWI, computed adjustment for uncrushed material, and the grading-weighted adjusted AWI.

APPENDIX J
IOWA RESEARCH PROJECTS

Skid / Friction Projects

PROJECT #		TITLE	END
HR/TR	121	Skid Resistance of Highway Pavements	6/1/73
HR/TR	170	Maintenance of Pavement Skid Resistance	1/1/80
HR/TR	199	Upgrading Asphalt Surface Friction by Aggregate Sprinkle Treatments (Demo #50)	12/1/83
HR/TR	205	Effects of Special Aggregate on Bridge Deck Overlay Frictional Properties	12/1/85
HR/TR	224	Restoration of Frictional Characteristics on Older PCC Pavement	2/1/84
HR/TR	281	Effects of Pavement Surface Texture on Noise & Friction Characteristics	1/1/87
HR/TR	400	The Potential of Friction as a Tool for Winter Maintenance	7/31/97
HR/TR	450	Identification of Laboratory Techniques to Optimize Superpave HMA Surface Friction Characteristics	12/31/04
HR	1035	Longitudinal Grinding & Transverse Grooving to Improve Friction & Profile	10/1/85
MLR	7602	A Lab Investigation of the Accelerated Polishing Method for Determining Skid Resistance Potential of Aggregate	1/1/76
MLR	7701	Skid Resistance of the Secondary Road System in Iowa	6/1/77
MLR	7803	Frictional Properties of New ACC Pavements in Iowa	11/1/78
MLR	8104	A Review of the Frictional Classification of Aggregates	3/1/81
MLR	8705	A Review of the Frictional Properties of Asphaltic Concrete Pavements in Iowa	7/1/87
MLR	8603	Comparison Study of Frictional Properties of Recycled vs Virgin Aggregates in Asphalt Pavements	3/1/86
MLR	9304	Noise & Friction Evaluation of Alternative PCC Pavement Tining Schemes	1/1/96

APPENDIX K
SUBSECTIONS FN DISPERSION PARAMETERS

Site 1 - IH 43 Walworth (09/03/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
NB (40)	CT	42.9	0.2	9	41.8	44.4	2.6	0.8	0.6
	F1	32.3	0.0	10	29.1	35.7	6.6	2.1	4.6
	F2	35.4	0.2	11	31.9	41.4	9.5	2.9	8.2
	P1	33.7	-0.1	10	31.5	35.4	3.9	1.6	2.5
	P2	34.7	0.0	10	32.1	37.7	5.6	1.5	2.4
NB (50)	CT	41.4	-	9	39.2	42.9	3.7	1.2	1.3
	F1	32.0	-	10	29.9	34.8	4.9	1.6	2.6
	F2	33.9	-	9	32.8	35.6	2.8	0.9	0.9
	P1	34.4	-	11	32.2	37.1	4.9	1.4	1.8
	P2	34.9	-	9	33.4	37.2	3.8	1.1	1.3
SB (40)	CT	43.0	0.2	10	41.2	45.5	4.3	1.3	1.8
	E1	36.1	0.1	10	30.9	43.3	12.4	3.5	12.5
	E2	37.0	0.2	13	35.2	39.2	4.0	1.1	1.2
	HV	43.0	0.1	5	42.1	44.1	2.0	0.8	0.7
SB (50)	CT	41.1	-	10	40.1	42.6	2.5	0.8	0.6
	E1	34.7	-	11	32.2	42.3	10.1	2.9	8.5
	E2	35.3	-	13	32.9	40.8	7.9	1.9	3.6
	HV	42.4	-	3	41.7	43.5	1.8	1.0	0.9

Site 2 - IH 43 Waukesha (09/16/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
NB (40)	CT	40.9	0.3	10	35.8	44.4	8.6	2.8	7.7
	F1	34.3	0.3	8	32.6	36.1	3.5	1.2	1.5
	F2	33.3	0.1	8	32.4	33.9	1.5	0.5	0.3
	P1	34.1	0.2	9	32.3	37.0	4.7	1.5	2.2
	SH	33.8	0.3	8	32.0	36.6	4.6	1.6	2.6
	SX	36.6	0.3	6	35.0	38.1	3.1	1.0	1.0
NB (50)	CT	37.9	-	8	30.3	43.8	13.5	5.0	24.9
	F1	31.4	-	7	30.6	32.5	1.9	0.8	0.6
	F2	32.5	-	9	30.4	33.9	3.5	1.2	1.5
	P1	32.3	-	8	29.4	34.7	5.3	1.5	2.3
	SH	31.0	-	7	28.9	34.7	5.8	2.0	4.1
	SX	33.3	-	6	29.9	35.6	5.7	2.1	4.2
SB (40)	CT	45.2	0.3	10	40.4	49.4	9.0	3.1	9.4
	E1	35.4	0.2	9	32.6	36.7	4.1	1.3	1.6
	E2	35.1	0.2	10	32.1	37.5	5.4	1.9	3.6
	P2	35.0	0.1	8	33.4	37.0	3.6	1.2	1.4
	SH	36.3	0.2	8	34.0	37.6	3.6	1.1	1.2
	SX	35.3	0.2	7	33.4	36.9	3.5	1.3	1.6
SB (50)	CT	42.0	-	9	35.1	46.3	11.2	3.5	12.2
	E1	33.6	-	8	31.1	34.9	3.8	1.2	1.4
	E2	33.6	-	9	31.8	35.4	3.6	1.1	1.2
	P2	34.0	-	8	32.5	35.0	2.5	0.8	0.6
	SH	34.3	-	6	33.4	35.4	2.0	0.8	0.6
	SX	33.8	-	6	31.7	36.2	4.5	1.5	2.1

Site 3 - IH 94 Monroe (09/30/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
EB (40)	WI	45.4	0.4	22	42.2	47.7	5.5	1.2	1.4
	SS	37.4	0.2	14	36.1	39.2	3.1	0.9	0.9
	SF	37.3	0.3	13	35.5	38.7	3.2	1.0	1.0
	SP	38.3	0.3	14	36.7	40.6	3.9	1.2	1.4
	SH	41.5	0.2	9	39.7	44.3	4.6	1.5	2.2
	SX	40.2	0.1	4	39.6	40.8	1.2	0.6	0.3
EB (50)	WI	41.7	-	22	39.0	43.7	4.7	1.4	1.9
	SS	35.0	-	14	33.1	37.8	4.7	1.4	2.0
	SF	34.5	-	13	33.0	36.2	3.2	0.9	0.7
	SP	35.8	-	14	33.9	38.0	4.1	1.4	2.1
	SH	39.1	-	8	38.1	40.0	1.9	0.6	0.4
	SX	38.8	-	5	37.9	39.8	1.9	0.8	0.6
Site 4 - IH 94 Waukesha (08/20/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
WB (40)	SA	39.1	0.3	8	36.9	41.9	5.0	1.7	2.7
	SM	29.9	0.2	14	27.9	33.3	5.4	1.3	1.8
	SD	37.7	0.3	9	34.7	40.8	6.1	2.1	4.5
WB (50)	SA	36.0	-	4	32.9	38.4	5.5	2.3	5.2
	SM	27.8	-	8	26.3	30.1	3.8	1.6	2.6
	SD	35.0	-	6	32.9	36.9	4.0	1.5	2.1
Site 5 - STH 21 Juneau (09/30/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
WB (40)	CT	52.7	0.5	8	51.9	54.3	2.4	0.8	0.6
	E1	46.7	0.2	7	45.9	48.1	2.2	0.8	0.7
	E2	45.8	0.3	8	43.3	48.7	5.4	2.0	3.9
	F1	46.3	0.3	6	43.8	49.0	5.2	2.3	5.1
	F2	47.4	0.4	8	42.7	50.3	7.6	2.3	5.3
	P1	46.0	0.3	7	42.8	49.3	6.5	2.2	4.7
	P2	45.9	0.3	8	42.3	49.1	6.8	2.2	4.7
WB (50)	CT	47.9	-	8	47.1	49.3	2.2	0.6	0.4
	E1	44.6	-	7	44.1	46.0	1.9	0.7	0.4
	E2	42.5	-	8	40.7	44.8	4.1	1.4	2.1
	F1	42.9	-	7	40.9	45.6	4.7	1.8	3.1
	F2	43.8	-	8	40.7	46.7	6.0	1.8	3.4
	P1	43.4	-	7	40.2	46.0	5.8	2.1	4.5
	P2	42.9	-	8	41.4	45.1	3.7	4.5	2.1

Site 6 - USH 151 Grant/Lafayette (08/17/2004)									
Hwy Dir (Spd)	**Mix Type**	**FN**	**Speed Gradient**	**Lock-ups**	**Min**	**Max**	**Range**	**S.D.**	**Var**
NB (40)	CT	37.0	0.5	8	34.1	41.1	7.0	2.7	7.7
	E1	39.5	0.3	9	37.4	41.4	4.0	1.5	2.4
	E2	37.1	0.2	9	34.9	38.7	3.8	1.1	1.3
	F1	35.2	0.2	6	32.0	37.3	5.3	2.0	3.9
	F2	36.7	0.2	11	32.0	43.5	11.5	3.0	9.0
	P1	36.6	0.2	8	33.5	38.8	5.3	1.8	3.3
	P2	39.0	0.3	10	37.4	41.2	3.8	1.2	1.3
NB (50)	CT	32.0	-	7	25.3	35.3	10.0	3.3	11.0
	E1	36.5	-	9	34.2	37.5	3.3	1.1	1.2
	E2	34.7	-	6	33.6	35.6	2.0	0.9	0.7
	F1	33.6	-	4	31.7	34.7	3.0	1.3	1.7
	F2	34.3	-	10	30.0	42.0	12.0	3.2	10.3
	P1	34.7	-	7	32.0	36.0	4.0	1.4	2.0
	P2	35.9	-	8	34.5	37.5	3.0	1.3	1.6
SB (40)	CT	36.7	0.3	6	32.7	42.9	10.2	4.5	20.2
	E1	35.1	0.2	12	33.6	37.3	3.7	1.2	1.4
	E2	34.2	0.2	15	30.7	38.6	7.9	1.9	3.7
	F1	37.5	0.7	7	33.1	42.7	9.6	3.1	9.6
	F2	35.0	0.2	11	32.4	36.9	4.5	1.5	2.2
	P1	37.3	0.2	14	33.1	39.8	6.7	1.9	3.5
	P2	37.7	0.2	16	36.3	39.1	2.8	0.8	0.7
SB (50)	CT	33.9	-	3	33.1	34.8	1.7	0.9	0.7
	E1	33.4	-	6	31.8	35.4	3.6	1.4	2.0
	E2	32.0	-	8	29.0	34.1	5.1	1.6	2.4
	F1	30.4	-	4	29.0	31.5	2.5	1.2	1.3
	F2	33.4	-	8	30.4	36.6	6.2	1.9	3.6
	P1	34.9	-	7	31.7	36.6	4.9	1.6	2.7
	P2	35.5	-	9	34.0	36.9	2.9	1.2	1.4