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EVALUATION AND CORRELATION OF LAB AND FIELD TENSILE STRENGTH RATIO (TSR) PROCEDURES AND VALUES IN ASSESSING THE STRIPPING POTENTIAL OF ASPHALT MIXES

FINAL REPORT

DECEMBER 1999

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CHAPTER ONE
INTRODUCTION

1.1 Background

The tensile Strength Ratio (TSR) test is used to predict the potential susceptibility of hot mix asphalt to moisture damage. According to the current Wisconsin Department of Transportation (WisDOT) specifications, when TSR values of asphalt mixes fall below 70%, the contractor is required to add anti-strip agents to the mix. The addition of anti-strip agents increases the cost of mix production and hence the overall project costs. The TSR test procedure and the threshold value used to indicate the failing mixes in Wisconsin have not been validated and it is not known if the threshold limit of 70% is the best for Wisconsin conditions. In addition, the TSR tests on the mix produced in the field have been observed to vary significantly from the laboratory results and could subsequently produce passing values. Since there are no provisions for accepting field TSR values, the contractor must continue to add the anti-strip agent, resulting in additional cost and unknown benefits. This problem is due to the fact that correlation between the lab and field values have not been established.

In an effort to understand the significance of the role of the anti-strip additives, the WisDOT constructed asphalt concrete pavements during the 1992 season without using any anti-strip additives, irrespective of the requirements as indicated by the AASHTO T 283 test on the mixes. It is anticipated that the performance evaluation of these pavements moisture damage would help in determining the effectiveness of the current AASHTO T 283 method and the threshold Tensile Strength Ratio (TSR) in predicting the moisture damage.

To accomplish these goals, the WisDOT sponsored a research project to the University of Wisconsin-Madison to investigate the extent to which the Wisconsin asphalt pavements are affected by moisture damage and the extent to which the laboratory moisture damage results correlated with the field performance. If the study indicates the presence of moisture damage problem in Wisconsin, then the research team is expected to come up with necessary modifications to the existing moisture damage evaluation test procedures and to the threshold values warranting the addition of anti-stripping agents.

To achieve the objectives of the project, it was essential to develop an understanding about the causes of moisture damage, the factors that influence its occurrence, and the pavement distresses that result from moisture damage. It was also necessary to study the TSR data collected for materials used in Wisconsin and to gather opinions from the neighboring states about their experience with the TSR test.

In a previous interim report, the results of an extensive literature search and the results of a survey conducted for the Midwestern states were presented. In addition, a preliminary analysis of the Wisconsin TSR database was presented. This report is prepared to summarize the final findings from the study and specifically includes the following results:

- Statistical analysis of the historical data on moisture damage collected for Wisconsin asphalt mixtures and the models derived from the data.
- Results of testing samples collected from 14 sections in Wisconsin and the findings from the laboratory testing of these samples.
- Relationship between TSR results and performance of pavement sections.
- Recommendations for revising the protocol for using the TSR testing and values to control moisture damage potential for Wisconsin.

1.2 Problem Statement

Reliable prediction of moisture damage is currently done in Wisconsin by using the Tensile Strength Ratio (TSR) of hot mix asphalt. It is not clear, however, if a correlation existed between values of TSR measured for field prepared mixtures and laboratory prepared mixtures. Recent observations indicated that field produced mixes show different TSR values than lab produced mixes. In addition, the threshold value of TSR of 0.70 is selected based on historical data and expert opinions. Currently other states are using values as high as 0.85 and as low as 0.60 depending on availability of aggregates and other local conditions. The suitability of 0.70 for Wisconsin conditions has not been verified with actual field performance.

1.3 Research Objectives

This study is proposed in two Phases. Phase I of the study will include three main objectives:

1. Review the literature on moisture damage evaluation. Evaluate the WISDOT test procedures, the threshold values, and the testing frequency to determine if the current procedure is capable of predicting the moisture damage of the mixes in the field. Examine the procedures and specifications adopted by the neighboring States and compare them to the WISDOT procedures.
2. Determine if Wisconsin pavements have moisture damage problems based on laboratory and field TSR values.
3. Determine the extent of moisture damage and the consequences on pavement performance based on cores extracted from selected pavement sections.

Should Phase I indicate that WisDOT has moisture damage problems, and that the existing tests and specifications are not adequate to identify the stripping problems, then Phase II of the research will be undertaken to develop revised procedures to identify the stripping potential of the mixes.

The special objective of Phase II study will be to:

4. Determine the promising procedures to measure the potential for moisture damage.
5. Establish revised or new procedures for testing and revise the threshold values used.

1.4 Research Methodology

The methodology used in this research is illustrated in Figure 1.1. It consisted of four main tasks.

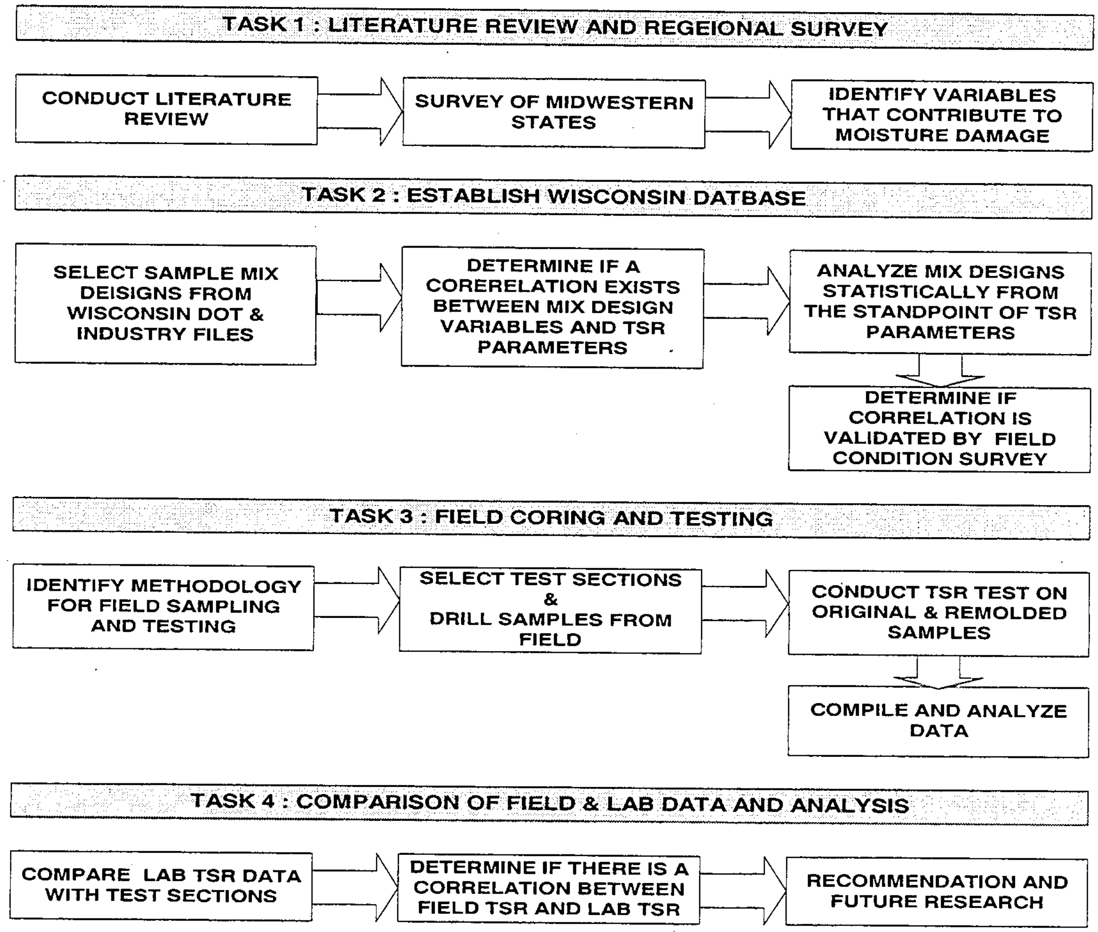


Figure 1.1 Research Methodology

Task 1: Literature Review and Midwestern survey

Literature review was conducted to understand the theories that may explain the nature and phenomenon of stripping, factors that are considered in prediction of moisture susceptibility, and variables that significantly affect the TSR values. Prevalent test methods to determine TSR were also reviewed to determine how accurately the tests measure the stripping potential of asphalt mixtures.

A survey of the Midwestern states was conducted to obtain information on what tests they use for measuring moisture susceptibility, whether or not moisture damage is a problem in their states, and threshold value of TSR used to pass a mixture. This survey was useful in chalking out a strategy for field sampling and subsequent lab testing of the drilled samples.

Task 2: Establishing Wisconsin Database

WisDOT, and main asphalt contractors of Wisconsin maintain a comprehensive database of mix designs that have been employed over the years. The data contains volumetric information, source and mineralogy of aggregates, source and grade of asphalt, anti-strip agents, and the TSR test parameters such as saturation, dry and wet strengths, etc.

A representative sample of this database was selected for statistical analysis with a view that it may yield a simple model to predict future TSR from the known variables that were reported affecting the TSR in the literature. A step-wise regression analysis was conducted using the SAS software to arrive at a simple yet reliable model to predict future strength of the mix from mix design data. This model was applied to the laboratory data of the fourteen field test sections of Wisconsin highways to determine the variability of the predicted and the actual indirect tensile strengths of the pavement mixtures.

Task 3: Field Coring and Testing

Fourteen sections of state and county highways of Wisconsin constructed in 1993 were selected for field coring of the samples. The sections were so selected as to include a diverse combination of mix designs, high and low TSR values, mineralogy etc. Top surface courses of the cores were sawed, and subsequently tested for in-situ, and remolded TSR value according to the same practice as is employed by the WisDOT and paving contractors. The results were compared with the TSR values of the corresponding sections as contained in the laboratory database of WisDOT, and contractors, to see if the lab and field TSR values are correlated.

Task 4: Analysis of Results and Recommendations

The results of tasks 1 and 2 were used to recommend changes in the current practices of the WisDOT procedure. The results of Task 3 were used to suggest the threshold value of TSR for Wisconsin conditions. Further the results from Task 3 were also used to determine whether TSR test itself possesses reliable potential for determining the moisture damage in the future or not.

1.5 Research Scope

Fourteen sections of state and county highways were selected obtaining actual pavement samples. The samples were removed using wet coring. Twelve cores from each section were drilled, and brought back to the laboratory for subsequent testing. The surface courses of the cores were sawed from the bottom portion to yield 168 samples of approximately 6-inch diameter and varying thickness. A subset of three cores out of twelve from each section were tested for in-situ dry strength (IDS), in-situ conditioned strength (ICS), remolded dry strength (RDS), and remolded conditioned strength (RCS). The results were analyzed in conjunction with the original mix design data to determine if any correlation existed between laboratory and field TSR values.

A population size of 317 mix designs out of all the designs maintained either by WisDOT or by the contractors that were used in construction of pavements from 1991 to 1996, were selected for statistical analysis. Statistical Analysis Software (SAS) was used for stepwise regression in choosing the significant among the numerous variables that contribute to moisture damage. The predictive model thus achieved was used to see if it matched the actual field data on TSR obtained from the samples of the fourteen pavement sections.

1.6 Summary

This report is organized into seven chapters. Chapter 1 has just described and consisted of an introduction, problem statement, objectives, research methodology, and scope of the research. Chapter 2 contains an overview of the theories, factors, and test methods relating to moisture damage. It also encompasses field studies that were aimed at providing correlation between lab tests and field performance of the asphalt mixtures. A comprehensive account of the Wisconsin mix design database is also included. Chapter 3 is devoted to the survey of the Midwestern states to document and understand their experience with the moisture damage problem. Practice of WisDOT in assessing the stripping potential of the mixtures is also included in this chapter. Chapter 4 describes the procedure followed for selecting the test sections, obtaining field samples, and testing the samples. Chapter 5 is a summary, and analysis of test results from the field samples. Statistical analysis of mix design database is carried out in chapter 6. Chapter 7 culminates this report with conclusions, and recommendations for future research.

CHAPTER TWO
LITERATURE REVIEW

2.1 Introduction

This chapter organizes information on past and on-going research on moisture damage in hot-mix asphalt (HMA) pavements within the scope of this research. Theories explaining the phenomenon of stripping are summarized. Critical factors that need to be considered in analyzing the TSR data are summarized. Test methods currently employed by various agencies in determining the moisture susceptibility are reviewed. Studies on correlation between TSR test results and field performance, and methodologies to identify moisture damage in the field are listed. The chapter concludes with a summary of findings from literature search.

2.2 Moisture Damage and Theories to Explain the Phenomenon of Stripping

Moisture damage in asphalt pavements is considered as a major problem encountered by the pavement engineers in the United States since the early 1900's (Nicholson, 1932). The moisture damage could manifest in asphalt pavements either due to stripping or from the softening of asphalt, both of which result in loss of stability to withstand traffic induced stresses (Taylor and Khosla, 1983).

Ever since the moisture damage was reported as a key distress affecting the asphalt pavements (Nicholson, 1932), researchers have conducted basic studies on adhesion-tension at the asphalt-aggregate surface and applied the principles of surface chemistry and physics to understand the stripping phenomenon. These studies have resulted in proposition of various stripping theories and development of several laboratory tests to quantify the degree of propensity of the asphalt mixes to moisture damage. Table 2.1 summarizes the theories by which researchers have explained the phenomenon of stripping in asphalt mixes. These theories generally indicate that moisture damage occurs in the presence of water and pore pressure, and is influenced by the properties of aggregates and asphalt. None of the theories listed in Table 2.1 could singly explain the phenomenon of field moisture damage due to the variability in highway materials, environment, construction practices, and evaluation methods. It is recognized that there are complex interactions between these different main factors.

Table 2.1 Summary of Theories used to Explain the Stripping Phenomenon

Theory **(1)**	**General Principle** **(2)**	**Supporting Research Source** **(3)**
Contact Angle Theory or Mechanical Adhesion Theory	Asphalt molecules are displaced from the aggregate surface, because the contact angle of water is less than that of the asphalt molecules.	Taylor and Khosla(1983) Stuart (1990) and Hicks et al. (1991)
Theory of Interfacial Energy or Molecular Orientation Theory	Asphalt molecules are displaced from the aggregate surface because the surface energy of water is less than that of asphalt.	Taylor and Khosla (1983) Stuart (1990) and Hicks et al. (1991)
Chemical Reaction Theory	Changes in the pH value of water around the aggregates affect the microscopic water at the mineral surface leading to the build-up of opposing, negatively-charged, electrical double layers on the aggregate and asphalt surfaces.	Taylor and Khosla (1983), and Hicks et al. (1991)
Pore Pressure or Hydraulic Scouring Theory	Pore pressure of the water entrapped due to mix densification under traffic results in the increased pore pressure on the asphalt films thus leading to rupture of the asphalt films.	Taylor and Khosla (1983), Hicks et al. (1991), and Kandhal (1994)
Theory of Spontaneous Emulsification	Adhesion between the asphalt and aggregates is lost due to the formation of an inverted emulsion.	Taylor and Khosla (1983) and Hicks et al. (1991)

2.3 Factors Affecting Moisture Damage in HMA Mixes

In addition to the theories for moisture damage, several researchers have classified the factors affecting the stripping in asphalt pavements into two groups: (1) material characteristics and (2) the in-place properties of HMA mix (Taylor and Khosla, 1983, Stuart, 1990, and Kandhal, 1994).

The material characteristics include properties of the binder, aggregates, asphalt mix, and those of the additives. The in-place properties include environmental conditions, traffic, construction practices, and drainage characteristics of the site. Table 2.2 summarizes the factors affecting the stripping potential of asphalt pavements, the desirable characteristics, the measurable indicators, and the supporting literature.

Certain factors listed in Table 2.2 can be measured/quantified using standardized procedures to correlate their effect on moisture damage while others are very difficult to quantify. In a survey conducted by Tunnicliff and Root (1984), the responding agencies related moisture damage to coarse and fine aggregates, geology, aggregate gradations, asphalt type and source, field compaction, and time of construction. Although scientists and practitioners agree on the list of factors that control moisture damage, defining the desirable or favorable characteristics is more ambiguous.

2.4 Test Methods Used to Evaluate Moisture Damage

Researchers have identified several test methods to evaluate the stripping potential of asphalt pavements (Lottman, 1978, Tunnicliff and Root, 1984, Hicks. et al., 1991, and Kandhal, 1994). The test methods to evaluate the moisture damage have been under development since the 1930's. The existing tests can be categorized as indicated in Table 2.3. Among the distinct categories of tests, the immersion -mechanical tests have been the most commonly used by several highway agencies. Table 2.4 summarizes the basic characteristics of moisture damage test methods used by the highway agencies across the United States.

Table 2.2 Summary of Factors Affecting the Moisture Damage

Factor (1)	Desirable Characteristics (2)	Measurable Indicator (3)	Supporting Literature (4)
Aggregate: Surface Texture	Roughness	Angularity & crushed faces	Taylor and Khosla (1983), Stuart (1990) and Hicks et al. (1991)
Porosity	Porous to absorb part of asphalt and increase the adhesion	Absorption	Yoon and Tarrer (1988)
Mineralogy	Basic aggregates	P^H value	Stuart (1990), Hicks et. al (1991)
Surface Chemistry	Ability to share electrons or form hydrogen bonds	Not defined	Taylor and Khosla (1983)
Dust Coatings	Clean aggregates	Wet sieve analysis	Kandhal (1994)
Surface Moisture	Dry	Water content	Taylor and Khosla (1983)
Adhesion	Good adhesion with asphalt	Percent coating retained after boiling; static/dynamic immersion	Tunnicliff and Root (1984) Hicks et al. (1991)
Asphalt: Viscosity	High	Viscosity tests	HMSO (1962), Schmidt and Graf (1972), Fromm (1974), and Stuart (1990)
Chemistry	Nitrogen and phenols	Elemental analysis	Stuart (1990)
Film Thickness	Thick and must coat the aggregates	Hveem guidelines	Hicks et. al (1991)
Asphalt Type and Source	Not defined	Not defined	Tunnicliff and Root (1984)
Asphalt Mix: Voids	Very low or very high. Not in Pessimum void region.	Volumetric analysis	Hicks et. al (1991), Al-Swalimi and Terrel (1993) and Kandhal (1994)
Gradation	Very dense or open	Sieve analysis	Hicks et. al (1991) and Kandhal (1994)
Asphalt Content	High	Mix design	Taylor and Khosla (1983) and Stuart (1990)
Additives	Compatible with aggregate (agg); must improve adhesion of asphalt with agg.	Improvements in Tensile Strength Ratio (TSR)	Tunnicliff & Root (1984)
Environmental Conditions: Temperature	Warm	Weather data	Stuart (1990), Hicks et al. (1991), and Kandhal (1994)
Rainfall during Construction	None	Weather data	Hallberg (1950) Stuart (1990) Kandhal (1994)
Rainfall after Construction	Minimal	Weather data	Hicks et al. (1991) and

Table 2.2 Summary of Factors Affecting the Moisture Damage (continued)

Factor (1)	Desirable Characteristics (2)	Measurable Indicator (3)	Supporting Literature (4)
Freeze-Thaw following Construction	Minimal	Weather data	Hicks et al. (1991)
Moisture Vapor (in arid areas)	Minimal	Weather data	Tunnicliff and Root (1984)
Pavement Drainage	Good drainage	Time to drain & inspection	Hicks et al. (1991) and Kandhal (1994)
Traffic	Low traffic	Average Daily Traffic (ADT)	Hicks et al. (1991) and Kandhal (1994
Construction Factors:			
Time of Construction	Summer & early Fall, Low Voids	Construction records	Tunnicliff and Root (1984)
Drum-Dryer Plant	Complete drying of aggregates, minimum bag-house fines and adequate viscosity of binder	Construction records	Kandhal (1989) and Tunnicliff and Root (1984)

2.5 Overall Experience with Moisture Damage Test Procedures

The general consensus among the users of moisture damage tests are that neither of the moisture damage test procedures proved to be superior nor can any one test correctly distinguish a moisture susceptible mix in all cases. The following are the views of various researchers with respect to the usefulness of the moisture damage tests.

1. Kandhal (1995) has identified three key issues that need to be addressed: (1) Proliferation of the test procedures and criteria due to a number of variations in the moisture conditioning, sample preparation (air voids) procedures, and the threshold limit to classify the moisture sensitive/insensitive mixes; (2) Need to refine the test procedures to increase reproducibility of the strength based tests by eliminating the influence of variables like air voids that have a significant effect on the strength ratios; (3) Need to establish a minimum wet strength value rather than relying on the strength ratios, a comment that strengthens the observations of Scherocman et al. (1986) and Busching et al. (1986).

Table 2.3 Typical Categories of Moisture Damage Test Methods

Test Method (1)	Principle (2)	Moisture Damage Indicators (3)	Supporting Literature (4)
Qualitative Coating Evaluation Tests	Immerse loose asphalt mix in water for specified time followed by optional agitation.	Percent of retained coating e.g. Texas Boiling Water Test ASTM D 3265	Taylor and Khosla (1983), Tunnicliff-Root (1984) Stuart (1990) and Hicks et al. (1991)
Quantitative Evaluation Tests	This is similar to the qualitative test but attempt is made to measure the percentage of aggregate surface exposed using a dye or a tracer.	Change in concentration of the dye or the percentage of light reflected. e.g.: Net Absorption Test	Taylor and Khosla (1983), Tunnicliff-Root (1984) and Hicks et al. (1991)
Immersion Mechanical Tests	Measures the changes in a specified mechanical property of the asphalt mix to be used in the actual pavement construction caused by the exposure to moisture.	Ratio of conditioned (wet) to unconditioned (dry) strength E.g.: Immersion-Compression Test, Modified Lottman (AASHTO T 283), Tunnicliff and Root Test (ASTM D 4867).	Taylor and Khosla (1983),Tunnicliff-Root (1984), Stuart (1990) and Hicks et al. (1991)
Non Destructive Tests	Measures the change in Resilient Modulus of the asphalt mix to be used in the actual pavement construction caused by the exposure to moisture.	Ratio of conditioned (wet) to unconditioned (dry) resilient modulus, M_r-ratio. e.g. ASTM D 4123 and SHRP –ECS M_r-Ratio	Taylor and Khosla (1983), Tunnicliff-Root (1984), Stuart (1990) and Hicks et al. (1991)
Wheel Tracking Test	Stresses applied on test samples (immersed in water bath at around 40 °C) using a Wheel Tracking Device to simulate the traffic effects.	Plot of wheel penetration (into the specimen) with time is compared for different mixes to obtain relative moisture damage potential of the mixes. e.g. Hamburg Wheel Tracking Device	HMSO (1962) and Aschenbrener (1995)
Other Tests (Texas Freeze-Thaw Pedestal Test)	Small briquettes prepared using an uniformly sized fraction of the proposed job aggregate and 2 % more asphalt than proposed for the field mix is immersed in water contained in a sealed jar and subjected to thermal cycling (10 °F, 140 °F, 10 °F) on a stress pedestal in water bottle.	Number of freeze-thaw cycles endured by the specimen is related to moisture susceptibility of the mix.	Taylor and Khosla (1983), Tunnicliff-Root (1984), Stuart (1990) and Hicks et al. (1991)

Table 2.4 Summary of Immersion-Mechanical Tests

Test Type (1)	Characteristics (2)	Comments (3)
Lottman Test (NCHRP 246)	9 samples compacted to 7 ± 1% air voids Subset I (S_1)– Water bath 5 hr – Tensile Test Subset II, III – Vacuum Saturation @ 26 in Hg for 30 min Subset II (S_2)– 3 hr @ 25 °C, conduct Tensile Test Subset III (S_3) – Freeze @ 0 °F for 15 hr, keep 24 hr @ 60 °C, & 3 hr @ 25 °C, conduct Tensile Test Ratios S_2/S_1 and S_3/S_1 are indicators of moisture damage under short and long term conditioning.	Minimum TSR 70% Provides quantitative measure of moisture damage at three phases of moisture conditioning. Moisture conditioning criticized as severe Good reliability (Lottman 1978, Maupin, 1982)
Tunnicliff-Root (ASTM D 4867)	6 samples compacted to 7 ± 1% air voids Subset I – No Conditioning- Tensile Test Subset II – 55-80% Vacuum Saturation. Soak 24 hr @ 60 °C. Soak 1 hr @ 25 °C. Conduct Tensile Test	Minimum TSR = 70% Effective predictor of moisture damage (Tunnicliff and Root, 1984)
Modified Lottman Test (AASHTO T 283) Proposed by Kandhal as a modification to Original Lottman (NCHRP Report 246) and the Root-Tunnicliff Method (ASTM D 4867)	8 samples compacted to 7 ± 1% air voids Subset I - Dry, No-Conditioning Subset II - Vacuum saturated to 55-80 % followed by optional freeze-thaw cycle TSR = Conditioned strength / Unconditioned strength in Indirect Tensile Strength Test	Minimum TSR = 70 % Reliable tool to identify mixes prone to severe moisture damage and strongly resistant to moisture damage Less reliable in predicting intermediate moisture resistance (Lottman 1978).
Immersion Compression Test (AASHTO T 165)	6 specimens compacted using Double Plunger to 6% void level. Dry subset: No Conditioning Wet subset: 4 days at 120 °F, 1 day at 140 °F Retained Compressive Strength Ratio = (wet/dry) Unconfined Compressive Strength	Min. ratio of 40-80 % used Not reliable and not recommended Retained ratios have been found to be >100 for stripped mixes (Stuart, 1990)
Environmental Conditioning System (SHRP - ECS)	Specimens: 100 mm height & 100 mm diameter Measure Preconditioned Resilient Modulus (M_1) Measure Air & water Permeability Samples conditioned according to desired sequence Resilient Modulus and water permeability determined after conditioning following each cycle (M_2). ECS - M_r ratio calculated (M_2/M_1)	Min ECS-M_r Ratio 80% Field validation and research on refining the device in progress. Measurement of permeability and conditioning process are different depending upon the environmental conditions. (Al-Swalimi 1993)

2. Busching and co-workers (1986) indicated that samples that have been saturated for long periods (60 days) have been found to exhibit very high saturation levels and exhibit very low relative strengths even in the absence of freeze-thaw cycles. They also indicated that the freeze thawing cycle generally accelerate the loss in strength of the mixes and one freeze thaw cycle could be used to make initial determination about the effectiveness of anti-stripping additives in the mixes. For long-term effectiveness of the anti-strip agents they however recommend the use of multiple freeze-thaw cycles.

3. Al-Swalimi (1993) indicates that the variability associated with the AASHTO T 283 test is relatively high and reports that the coefficient of variation in the TSR for a given mix was ranges between 11 and 39 percent. He indicates that there is a need for improving the mechanisms to simulate the asphalt-aggregate interactions during the conditioning process to enhance the repeatability of the test results.

4. Aschenbrener in his prepared discussions to Al-Swalimi's paper (Al-Swalimi, 1995), indicated that the Lottman test with many other modifications have allowed moisture susceptible mixes to be placed in the field. He has identified four specific areas in which the current tests are deficient and indicated that the ECS has the potential to overcome these deficiencies. He indicates that the AASHTO T 283 is not capable of modeling the extended presence of moisture and corresponding development of the pore pressures from the traffic. He also indicates that the AASHTO T 283 adopts one level of conditioning ignoring the fact that the heavy truck traffic on pavements conditions more severely than the low truck traffic.

5. Hicks et al. (1991) quotes Scherocman et al. (1986) and Busching et al. (1986) that it is possible for a mix to yield lower strength ratios even though the strength values for both conditioned and unconditioned samples have increased.

2.6 Correlation Studies between Laboratory Moisture Damage Test Results with Field Performance

From the above summary, it appears that several researchers have identified major gaps in the use of immersion-mechanical-testing methods. Several studies have included a field validation phase in which the researchers have studied the extent to which the lab results can be reproduced in the field in terms of performance. This section provides brief information on the studies that correlate the lab tests with the field performance.

1. Lottman - NCHRP Study 192 (1978)

The NCHRP study 192 conducted by Lottman (1978) resulted in the development of the original Lottman test procedure for moisture damage evaluation of HMA mix.

2. Lottman - NCHRP Study 246 (1982)

The field validation of the Lottman's moisture damage test procedure was extended through the NCHRP study 246 (Lottman 1982) in which seven participating highway agencies selected 8 test sections of new pavements constructed during 1975 and 1977 using aggregates having a history of moisture damage.

3. Taylor and Khosla (1983)

Taylor and Khosla (1978) indicate that the TSR values between 70% to 75% have been selected as a separation point between good performance and poor performance based on limited correlation studies conducted by the Virginia Department of Highways and Transportation. The field performance of 4 test sites constructed with and without anti-strip additives was included in the analysis.

4. Tunnicliff and Root - NCHRP Study 274 (1984)

This research is an extension of the Lottman's studies and involved the evaluation of asphalt concrete mixes modified using the anti-strip additives.

5. Hicks et al. – NCHRP Synthesis 175 (1991)

Hicks and his colleagues conducted a survey under the NCHRP Synthesis 175 (1991) to obtain feedback from the highway agencies pertaining to the test procedure used, criteria for classifying the moisture susceptibility of the mix, and the adequacy of the test method to determine the moisture susceptibility. The response from the highway agencies is summarized in Tables 2.5 and 2.6. It must be noted here that the agencies have used rating numbers between 0 and 9, 0 being least effective and 9 most effective. In addition, the highway agencies have described the effectiveness of the tests ranging as slight, moderate, and high. It is interesting to note that the boiling water test and the immersion-compression tests such as Modified Lottman test are widely used by state highway agencies.

6. Kiggundu and Roberts (1988)

Kiggundu and Roberts (1988) have compiled information pertaining to the success rates of the moisture damage tests (in terms of correctly identifying the moisture-susceptible mixes) from various research reports and papers. Their data indicated that the Modified Lottman Test with a minimum TSR criteria of 80% to be most effective. It can be seen from Table 2.6 that though the mechanical-based tests are common among the highway agencies, the qualitative coating evaluation method is still being pursued by some agencies with a level of success that can be compared with some of the strength based methods. A distinct observation from Tables 2.5 and 2.6 is that none of the test method has shown an overwhelming superiority.

Table 2.5 Rating of the Moisture Susceptibility Test Procedures by Hicks Survey (1991)

Test method (1)	**Agencies Using** (2)	**Rating (Max 9)** (3)	**Effectiveness** (4)
Boiling Water	9	5	Slight to Moderate
Static-Immersion Test	3	4	Slight
Lottman (NCHRP 246)	3	7.5	High
Tunnicliff and Root (ASTM D 4867)	9	5	Slight to Moderate
Modified Lottman (AASHTO T 283)	9	7.5	High
Immersion-Compression Test	11	5	Slight to Moderate
Total	44	-	-

Table 2.6 Success Rate of Test Methods (Kiggundu and Roberts, 1988)

Test Method (1)	**Minimum Test Criteria** (2)	**% Success** (3)
Modified Lottman (AASHTO T 283)	TSR = 70%	67
	TSR = 80%	76
Static-Immersion Test (ASTM D 4867)	TSR = 70%	60
	TSR = 80%	67
	TSR = 70-80%	67
10-Minute Boil Test	Retained Coating 85-90%	58
Immersion-Compression Test	Retained Strength 75%	47

7. Tunnicliff and Root - NCHRP Study 373 (1995)

This study was an extension of the NCHRP study 274 to evaluate the field performance of the anti-stripping agents. In this study, 19 full-scale pavement sections (each with control and anti-strip sections) were built in eight states. The field evaluation included the periodical testing of cores for moisture damage and pavement condition surveys over a period of 6 to 8 years.

8. Aschenbrener et al. (1995)

Aschenbrener and co-workers have also compared the pavement performance (from moisture damage considerations) of about 20 sites with the laboratory moisture susceptibility results determined using the Modified Lottman (AASHTO T 283), Texas Boiling Water Test, the ECS, and the Hamburg wheel track device. They indicate that under the current protocols, none of the testing procedures could correlate well with the field performance. The laboratory procedures with respect to moisture conditioning and/or the threshold values of the test methods were adjusted to obtain a better correlation (in some cases) between the lab results and the field performance. For the TSR test, the level of saturation was raised from existing 80% maximum to 90% to obtain better correlation. The recommendation from this study relative to the AASHTO T 283 test was the need to increase the degree of saturation for the AASHTO T 283 from 80 to 90 percent as obtained by a 30 minute vacuum saturation.

9. Maupin (1995, 1997)

Maupin (1995,1997) studied the performance of test sections constructed using hydrated lime (3 projects) and chemical anti-strip additives (9 projects) to determine the effectiveness of chemical anti-strip additives over the hydrated lime. His study indicated that neither the TSR test nor the boiling test could identify the moisture susceptible mixes since none of the 12 mixes failed the boiling water test and only one mix that stripped in the field showed unacceptable TSR value.

10. Dukatz and Phillips (1987)

Dukatz and Phillips employed Lottman and Modified Lottman procedures to determine the effect of air voids on the TSR. They used 10 combinations of additives, 4 types of aggregates, and 4 types of asphalt mixes (base, binder, coarse surface, and fine surface) to prepare specimen for subsequent testing. They observed that although all the procedures required that the asphalt mixtures should be compacted to a given level of air voids, plus or minus 1 percent, differences as high as 40 percent were found between TSR values for mixes tested at low and high side of the air void specifications.

11. Gilbert Y. Baladi et al. (1988)

Though not related directly to the scope of this research, this study related structural properties such as resilient modulus and Poisson's ratio with asphalt mix parameters such as voids, and test temperatures. The main objective of the study was to select a simple test and test procedure to determine the fundamental engineering properties for the design of asphalt pavements. It found substantial variability in the material properties using different tests such as triaxial tests, cyclic flexure tests, Marshall tests, indirect tensile tests, and creep tests. However, results from the indirect tensile test were the most promising though they were not consistent. The study found that the percentage of air voids and test temperature significantly influence the resilient characteristics of the mix. The study may be used to investigate graphically any relationship if it exists, between the indirect tensile strength as determined for TSR test and the modulus of the mix.

2.7 Identification of Moisture Damage in the Field

Literature review and survey of Midwestern states (Chapter Three) identifies potholes, rutting, raveling, flushing, cracking and bleeding/blisters as the key pavement distresses that could be associated with moisture damage. Researchers are aware of the need to develop an investigative methodology to identify the moisture damage problem in specific pavement sections. The most commonly recommended procedure is taking cores and splitting them apart to visually evaluate the stripping of asphalt from aggregates. This procedure is arbitrary and could be uneconomical. Also, conclusions based on pavement distresses such as raveling, flushing, and rutting could also be erroneous since these distresses could be caused by factors other than stripping (Kandhal, 1994). The literature review conducted for this research task identified three procedures that show promise.

1. Maupin's Procedure (1989)

This procedure was developed by Maupin and published in 1989. The procedure includes the development of a deterioration curve of the pavement layer based on the in-situ, remolded, and conditioned tensile strength of the mix. It is based on the concept that as the pavement ages, the stiffness of the mix (hence the indirect tensile strength) increases primarily due to age hardening as shown in curve 1 of Figure 2.1. If the pavement is affected by moisture damage, the strength of the mix drops after the initial peak as shown in curve 2 on Figure 2.1. To evaluate the present and future strengths of the mix, cores need to be tested for the following conditions:

- In-situ strength: Cores taken from the pavement are to be tested for tensile strength directly after removal and as soon as practical. Un-stripped strength: Cores are either reheated and remolded to the field void content or, in cases where it is not possible to remold the mixes due to large aggregate size, cores are dried to constant weight and tested for un-stripped tensile strength.
- Future strength: Cores are conditioned as per NCHRP 274 specifications and tested for tensile strength.

Using the in-situ, un-stripped, and the future strength, the deterioration curve is plotted as shown in the figure 2.1. A given pavement layer is said to be a candidate for rehabilitation if the present and future strengths of the pavements is less than 275.8 kN/m^2. This specification is based on Georgia DOT's experience. Also, the ratios of present (in-situ), and future (conditioned) strength, to the dry remolded strength must be greater than 0.3.

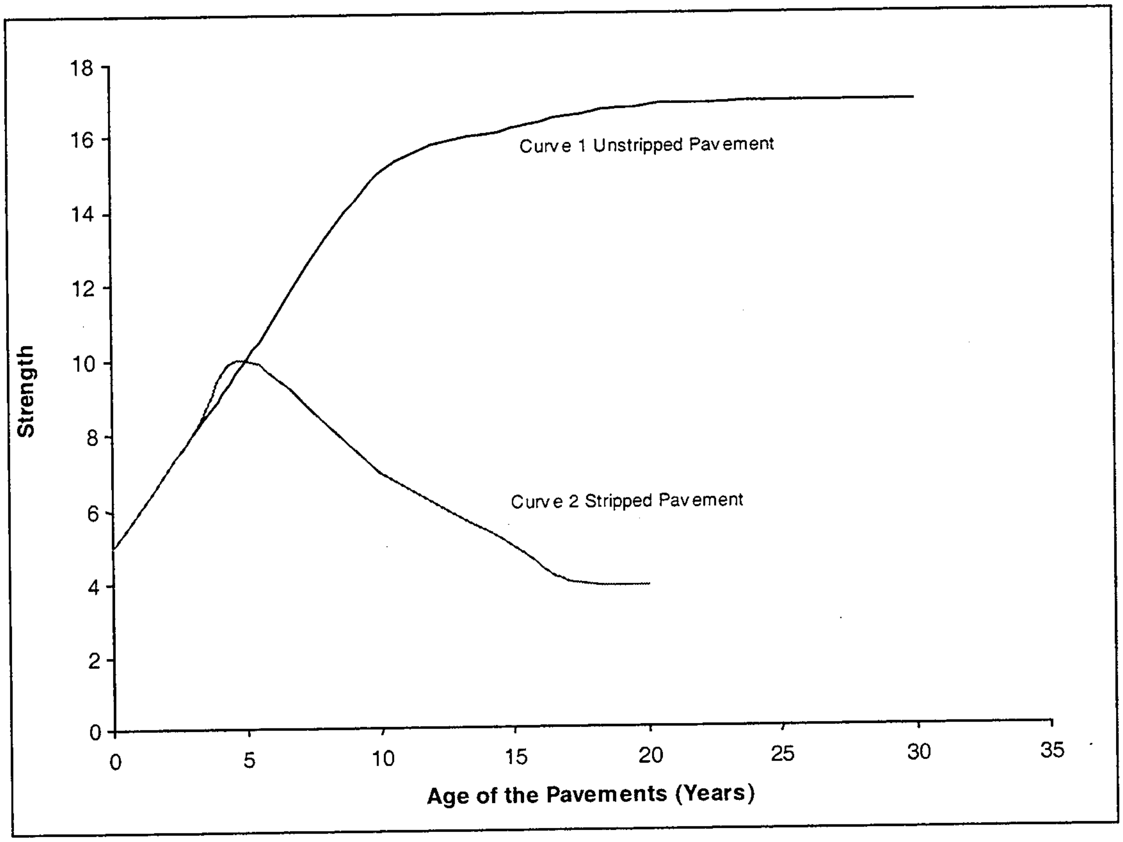


Figure 2.1 Deterioration Curve for an Asphalt Pavement (Maupin 1989)

2. Kandhal's Procedure (1994)

Kandhal (1994) suggests an investigative methodology based on forensic experience to identify the stripping problems in a given HMA pavement project. The procedure is accomplished in three phases, (1) sampling phase, (2) testing phase, and (3) analysis phases.

Kandhal's procedure has been used in the states of Georgia and South Carolina to identify if stripping problems existed in their HMA pavements. In South Carolina, about 800 kilometers of HMA pavements were surveyed, 1324 cores were taken, and 4503 pavement layers were evaluated for stripping rates. Two cores were taken at random from every 3.218 kilometers segment of each highway section sampled. The Georgia DOT has a similar program in which the crews revisit the sites each year to sample and test the cores. The results from this program have been reported to be successful in identifying moisture damage in the field.

3. Tunnicliff and Root's Procedure (1995)

The Tunnicliff and Root procedure was developed as a part of NCHRP study 373 on the field evaluation of anti-stripping additives in asphalt concrete mixtures. To evaluate the performance of the anti-stripping additives over a period of time, the researchers suggested a procedure that involves taking field cores form the pavement periodically and testing them for tensile strength under varying conditions.

2.8 Summary of Findings from Literature Search

The review of literature on moisture damage resulted in the identification of theories associated with moisture damage, the factors affecting moisture damage, tests used to evaluate the moisture damage potential of asphalt mixes, and their effectiveness. The following are the summary of findings from the literature review.

1. Although several theories have been proposed to describe the phenomenon of stripping (moisture damage), none of the theories of stripping can singly explain the phenomenon of field moisture damage due to the variability in highway materials, environment, construction practices, and evaluation methods.
2. The aggregate characteristics and asphalt properties are said to influence the asphalt-aggregate bond in presence of traffic and environment. There is no clear information in the literature about the degree of interaction and the contribution of the individual factors to the moisture damage.
3. Subsequent to the development of the Lottman moisture damage test, the original test procedure has undergone modifications in the conditioning process and the

criteria adopted by the agencies to identify mixes susceptible to moisture damage have changed. Several agencies have used the threshold value of 70% to identify the moisture susceptible mixes but others varied this criterion to adjust to the pavement performance they observed in the field.

4. Even though the AASHTO T 283 test results do not correlate extremely well with field performance, this test still is rated as an effective method to predict the moisture damage potential of asphalt mixes and it is still being used by a majority of highway agencies. It is concluded that the moisture damage problem must be considered as regional problem and that the selection of test methods for moisture damage evaluation must be based on the regional conditions. The criteria and conditioning procedure need to be calibrated based on the field performance results and comparison with lab results.
5. Among the moisture damage identification procedures used by the researchers, the Maupin's procedure appears to be the best suited for this study because of its quantitative approach to the problem and potential success in comparing the field performance and laboratory moisture damage. This procedure has been modified and recommended for use in Task 3 of this study.

CHAPTER THREE
MIDWESTERN SURVEY

3.1 Introduction

A survey of current practices of State Highway Agencies to deal with the problem of moisture damage was conducted by Hicks et al. (1991). The response to survey was reported in NCHRP synthesis No. 175. The survey focussed on questions regarding extent of moisture damage, pavement problems related to moisture damage, effect of aggregate, effect of asphalt, criteria for determining the moisture damage, tests used, field procedures, environmental factors, and traffic levels. To update the information, the research team of this project conducted a new survey to exclusively incorporate information on current practices of Midwestern States' DOTs. The results of this survey along with the relevant portions of NCHRP Synthesis No. 175 are presented in this chapter.

3.2 Practices to Control Moisture Damage in Asphalt Mixes in the Mid-Western Region as Reported in NCHRP Synthesis 175

Hicks et al. (1991) conducted a nation-wide survey of state highway agencies' perspectives on various aspects of moisture damage detection and prevention as a part of the NCHRP Synthesis No. 175. The respondents in this study were asked to rate their views on a scale of zero to nine, with zero being least effective and 9 being most effective. Although the Hick's study included about 29 questions on various aspects of moisture damage, in this report, the responses to only those questions significant to the scope of this study have been summarized. Also, the responses of only the mid-western states are presented. The details of the responses from the Hicks (1991) study are summarized below.

- **Percentage of Pavements Affected by Moisture Damage**

It can be seen from Figure 3.1 that among the responding states, moisture damage is prevalent in an average 15% pavements in Minnesota and South Dakota and is not considered as a major problem (only 5%) for Wisconsin, Illinois, Nebraska, and Missouri.

Since the author is not aware of the procedure adopted to attribute the extent of moisture damage in the pavements, the use of Figure 3.1 is limited to obtaining a general information about the extent of moisture damage problem.

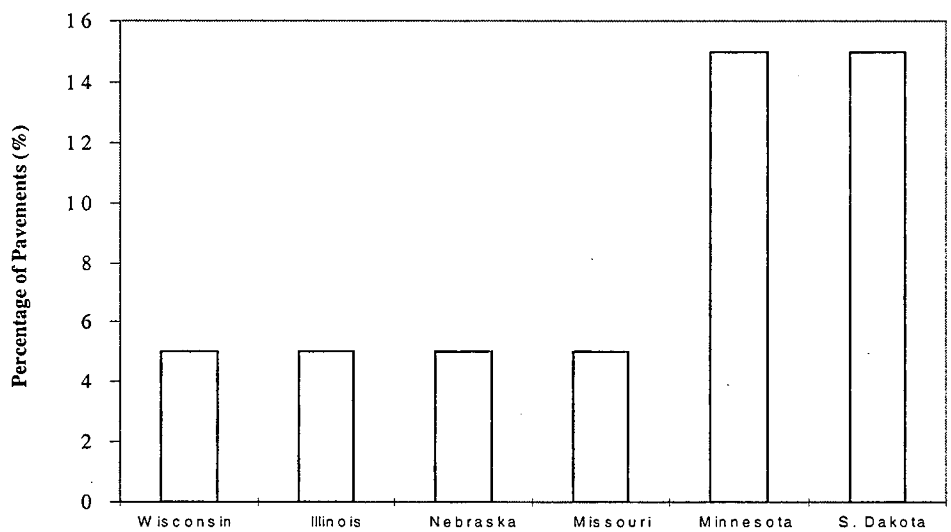


Figure 3.1 Average Percentage of pavements Affected by Moisture Damage (Hicks, 1991)

- **Distresses Associated with Moisture Damage**

In response to this question, 4 of the 6 responding states identified potholes as the key indicator of moisture damage (mean effective indicator rating 5.6, Figure 3.2). The effectiveness of potholes as an indicator of moisture damage is expected because the potholes are usually formed during the periods of high rainfall during which water penetrates the HMA surface through the cracks, and softens the base course. Here, the presence of water may aid in the stripping of asphalt film from the aggregates. This is said to accelerate the formation of potholes. Rutting and flushing were 3.25 and 3.0, respectively for their effectiveness as indicators of moisture damage (Figure 3.2), while raveling and reflection cracking received a least effective indicator scoring of 2.6 and 2.0, respectively. It can be seen from Figure 3.2 that the WisDOT does not consider potholes as a primary distress since it is preceded with cracking.

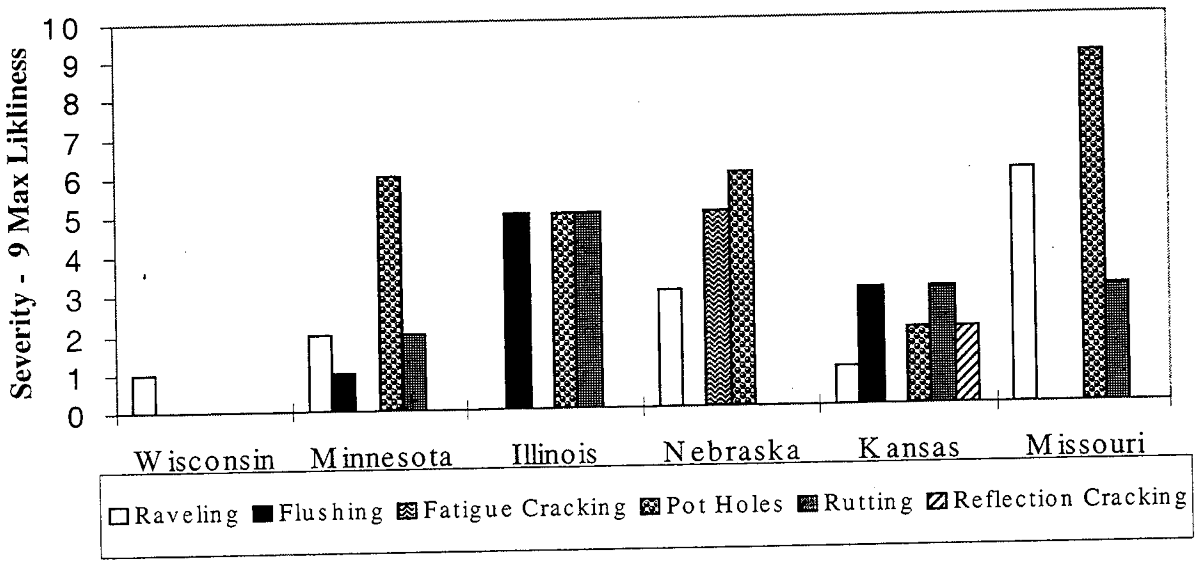


Figure 3.2 Pavement distress related to moisture damage (Hicks, 1991)

- **Relation between the Age of the Pavements and Moisture Damage**

In response to identifying the typical age of pavements at which moisture damage distresses are observed, Figures 3.3a and 3.3b show that moisture damage related distresses (as identified above) like, raveling, flushing and reflection cracking are observed at an early age (within 3 years) in case of mixes without additives. Distresses such as, potholes, and rutting that received high effective indicator rating cracking were observed at around 6 years. In case of modified mixes, although the age of pavements was higher in case of raveling, the highway agencies did not indicate any delays in the occurrence of potholes for mixes with additives. It is interesting to note that rutting was reported to accelerate in case of modified mixes.

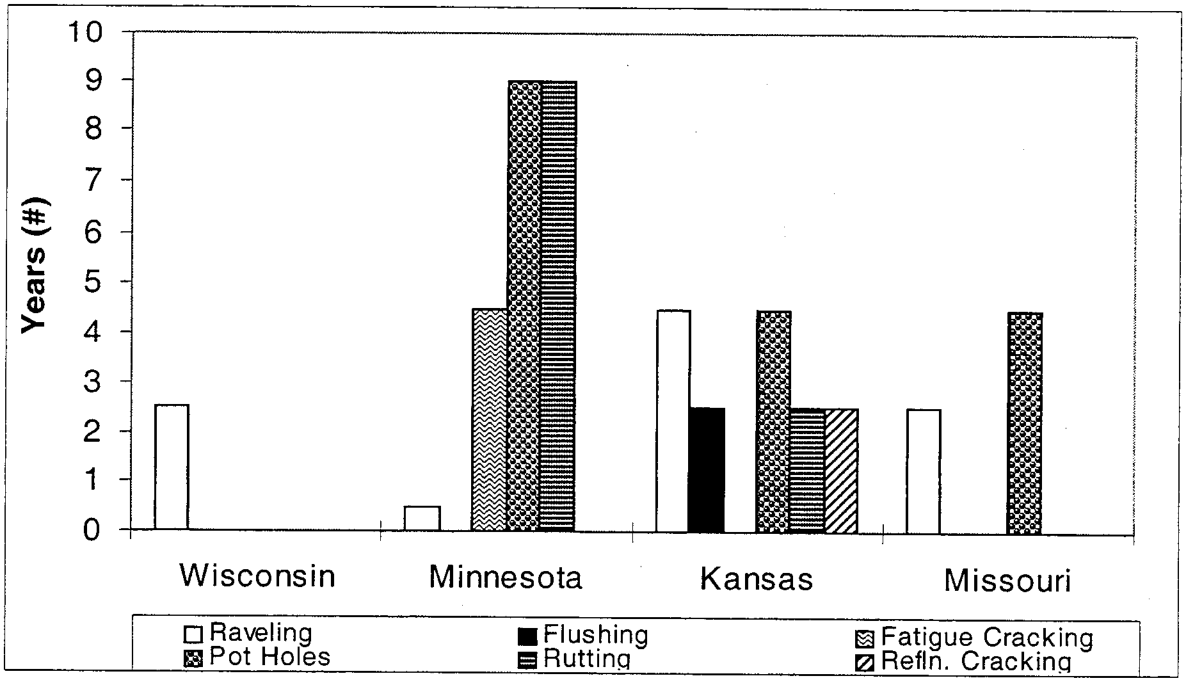


Figure 3.3a Age Of Pavements when Moisture Related Distresses are First Experienced – Mixes Without Additives (Hicks, 1991)

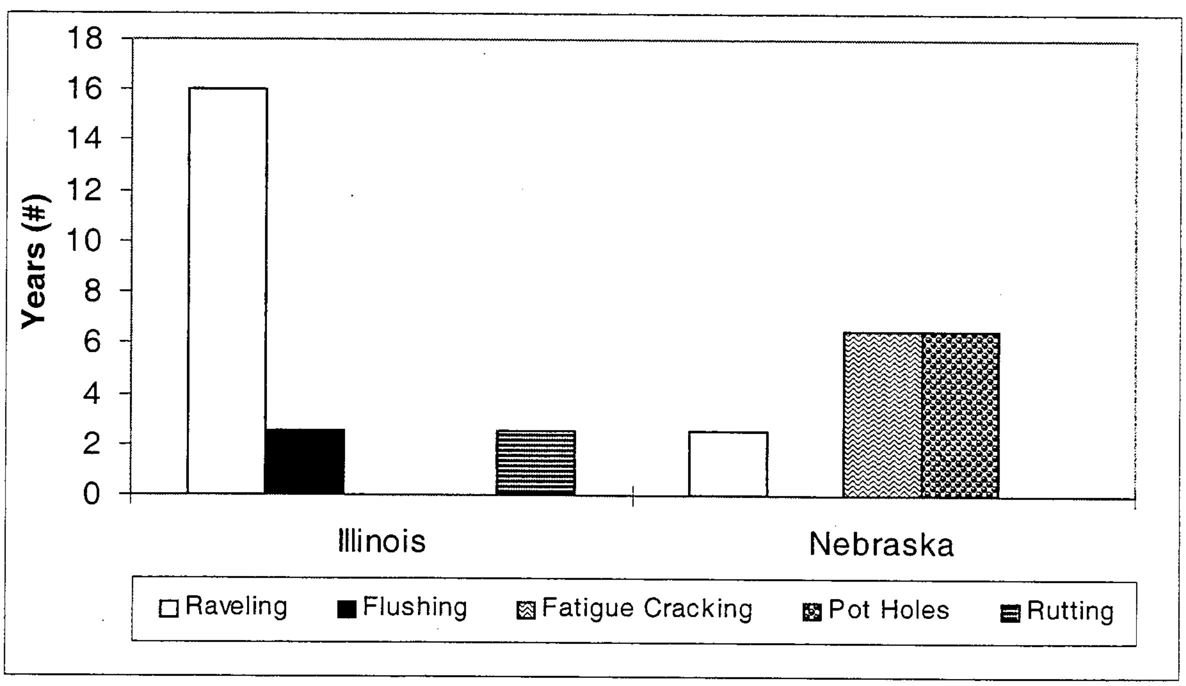


Figure 3.3b Age of Pavements when Moisture Related Distresses are First Experienced – Mixes With Additives (Hicks, 1991)

- **Relation between the Aggregate Type and Moisture Damage**

The responding mid-western states indicated that limestone and dolomite were the predominantly used aggregates among the quarried materials, while basalt, rhyolite, quartzite, and granite were the commonly used aggregates used from gravel pits (Figures 3.4a and 3.4b). Among the aggregates obtained from the quarries, traprock and dolomite were highly related to moisture damage, while chert (from gravel pits in Missouri) was associated with moisture damage (Figures 3.5a and 3.5b). It must be noted that dolomite formations are predominant in Wisconsin and it would be interesting to see if significant relation exist between dolomite and the moisture damage based on the statistical analysis of the Wisconsin TSR database.

- **Field Practices to Control Moisture Damage**

In response to the field practices adopted by the mid-western states to control moisture damage problems, Wisconsin rated changing the aggregate source (Figures 6a and 6b) and controlling compaction and placement temperature of the mat as the most effective methods.

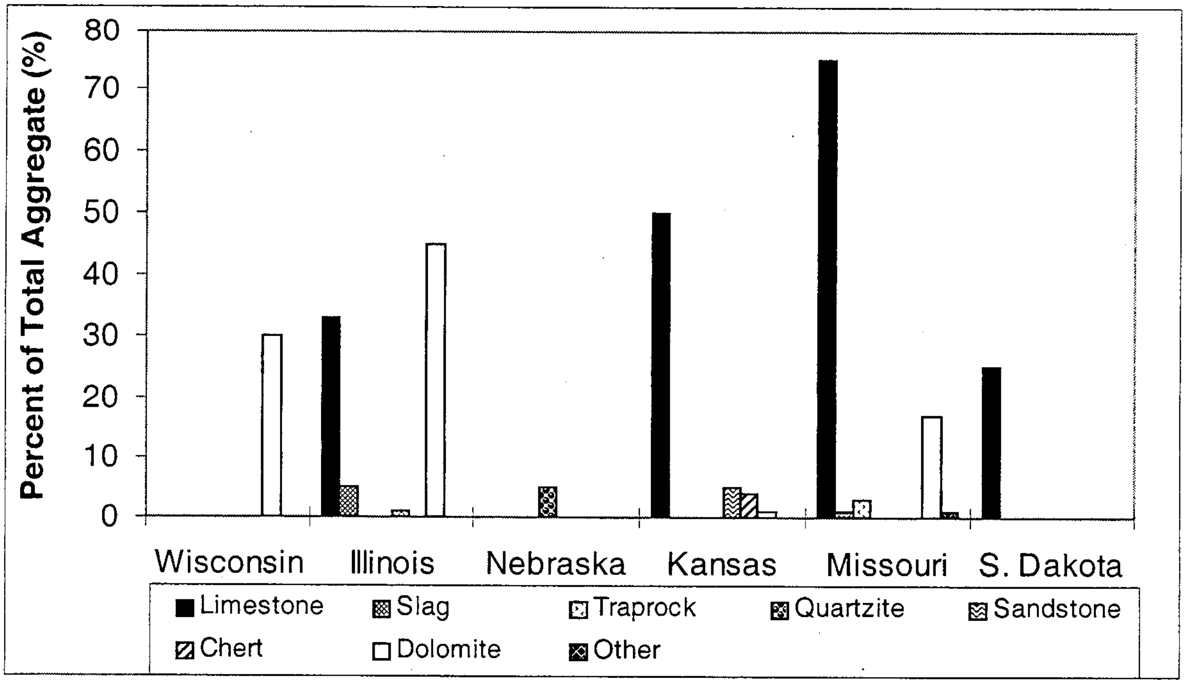


Figure 3.4a Type and Percentage of Quarried Aggregates used in Asphalt Pavements (Hicks, 1991)

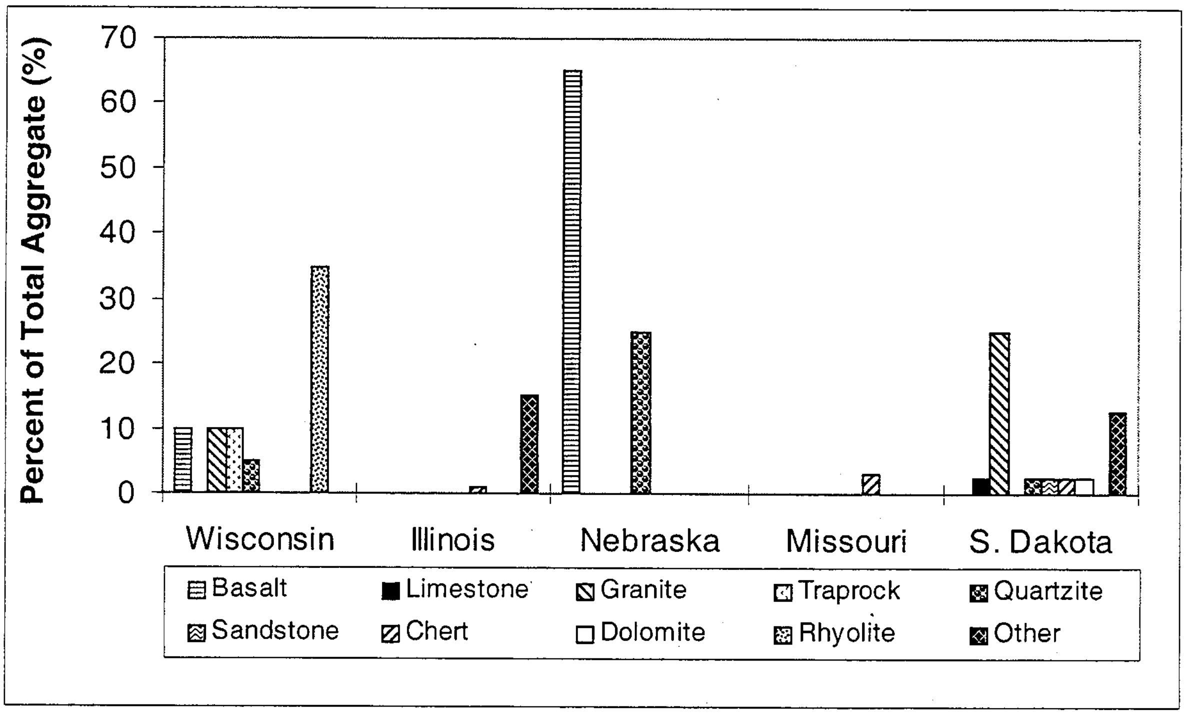


Figure 3.4b Type and percentage of Pit - Aggregates Used in Asphalt Pavements (Hicks, 1991)

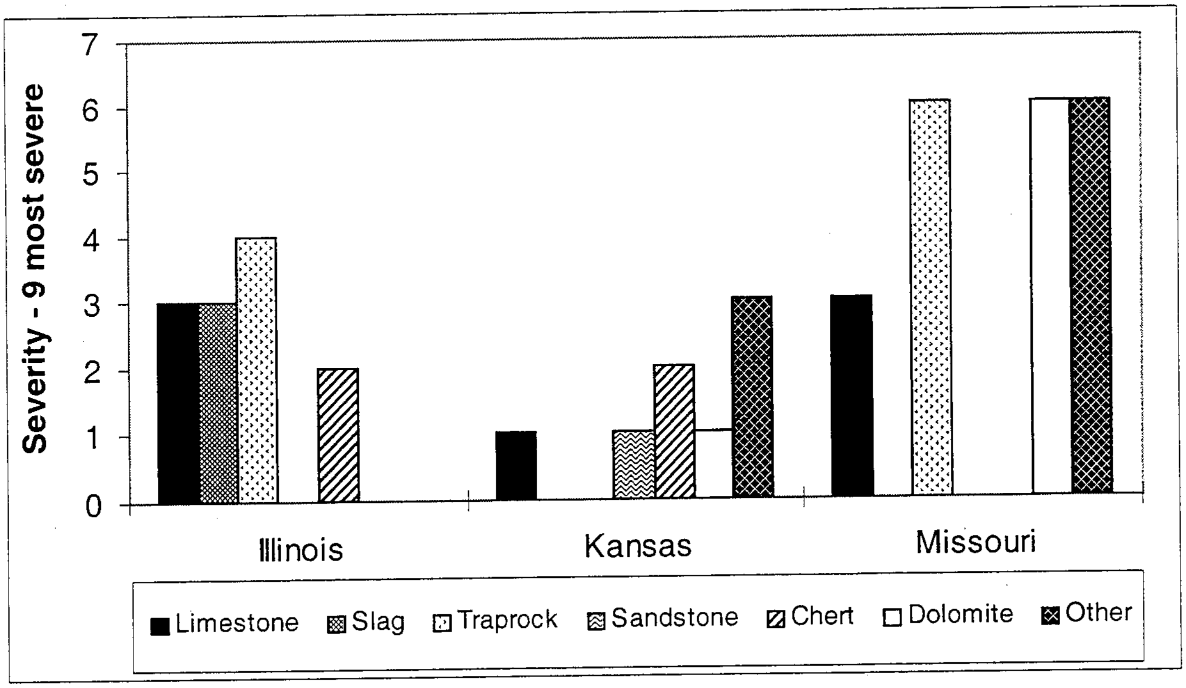


Figure 3.5a Severity of Moisture Damage Problems for Each Aggregate Type – Quarries (Hicks, 1991)

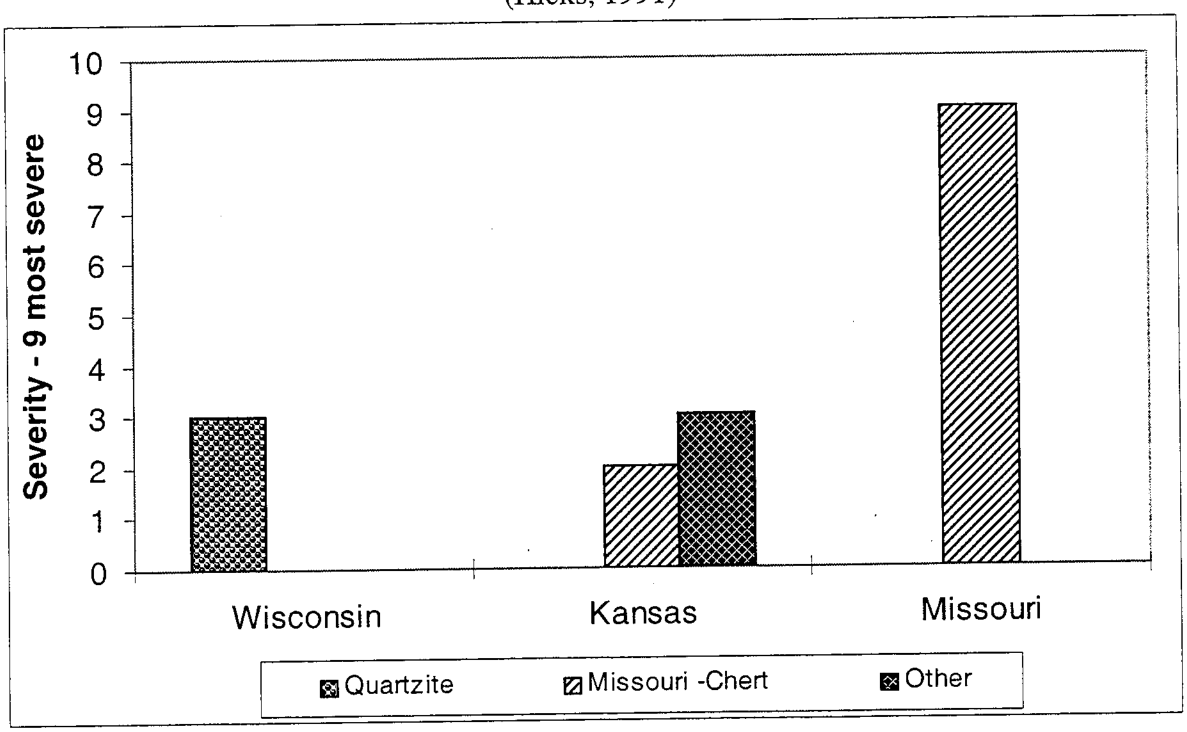


Figure 3.5b Severity of Moisture Damage Problems for Each Aggregate Type - Pits (Hicks, 1991)

3.3 WisDOT-UW-Madison Survey of Mid-western States on Moisture Damage

The survey conducted by the Hicks and his colleagues did not solely analyze and report results relevant to the Midwest states. This is partly due to the fact that not all the mid-western states responded. To obtain relevant and the most recent information on moisture damage from the mid-western states, a new questionnaire (henceforth referred to as WisDOT-UW survey) was sent out to the mid-western states in August 1997. The states included Illinois, Indiana, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Ohio and South Dakota. A copy of the WisDOT-UW questionnaire is shown in Appendix A. Table 3.1 summarizes the responses from the responding states. It should be noted that the discussions are limited only to the states that responded to the questionnaire.

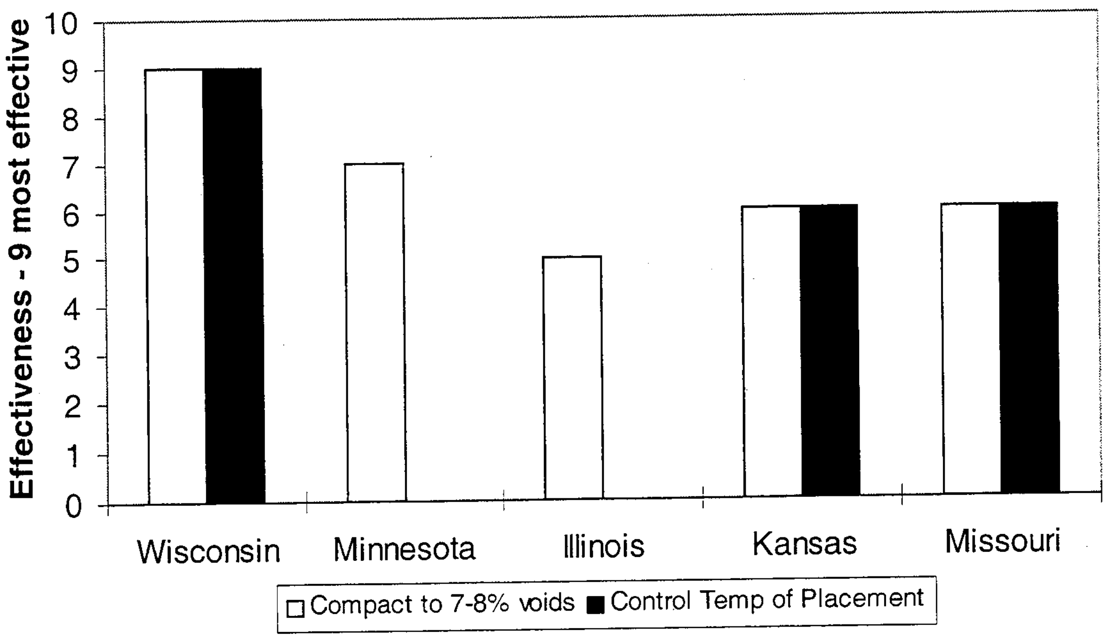


Figure 3.6a Effectiveness of Field Procedure to reduce Extent of Moisture Damage Problems (Hicks, 1991)

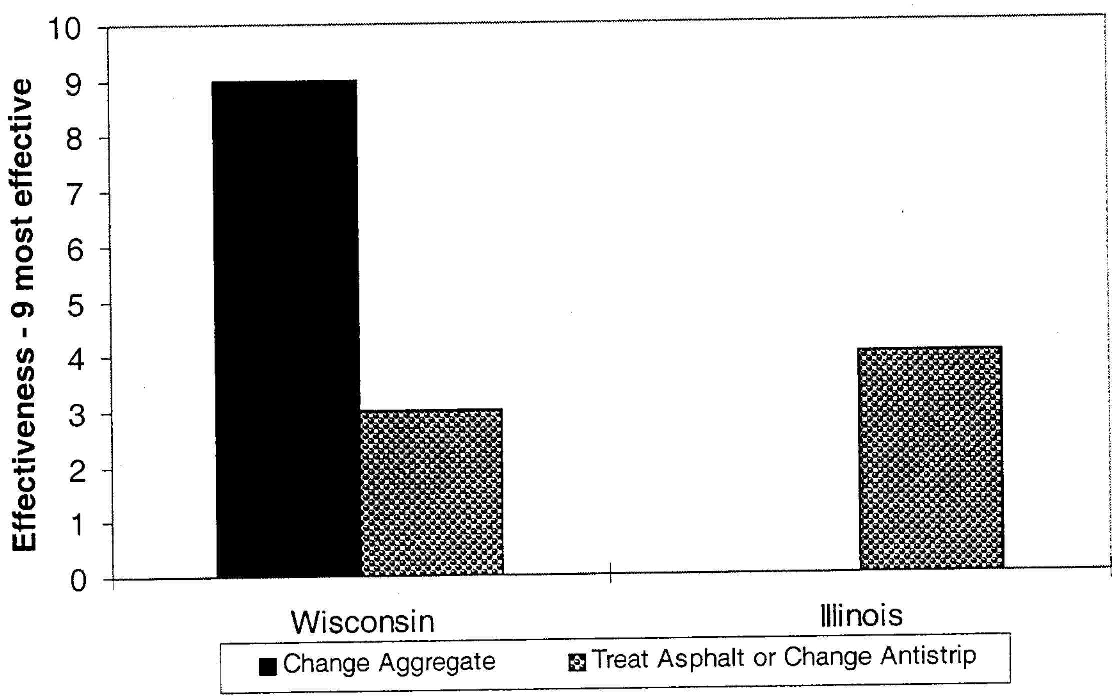


Figure 3.6b Effectiveness of Guidelines Implemented upon Detection of Moisture Damage (Hicks, 1991)

Table 3.1 Summary of the WisDOT-UW Survey on Mid-western States Perspectives on Moisture Damage

Question	Iowa	Illinois	Kansas	Minnesota	Michigan	Missouri	Nebraska	N. Dakota	Ohio	S. Dakota
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Is Moisture Damage a Concern	Yes	Yes	Yes	Yes	Yes	Yes	No	No	Yes	Yes
Do you use Anti-strip additives	Yes	Yes	Yes	Yes	No	Yes	No	No	Yes	Yes
Identifying Moisture Damage in field	Visual observation of Split Tensile Cores	Visual Observa-tion of cores	Monitor during construction	AASHTO T 283	None	Visual Observation of cores	NA	NA	Random coring and evaluation by DOT lab.	Core Inspection
Distresses perceived to Moisture Damage	Stripping, Raveling, Rutting	Rutting, Raveling, Shoving, Fatigue Cracking	Disintegration Longitudinal cracking from bottom upwards	Thermal Cracking, Deterioration of fatigue cracks, and raveling	Raveling, Segrega-tion	Raveling, Bleeding, Rutting, Potholes	NA	NA	Raveling, Potholes	Stripping
How early Moisture Damage occurs	8 years	< 3 years	Depends on Previous year Climate, Compaction, Aggregates	3-5 years	2-5 years	Related to Environment	NA	NA	4-6 years	No response
Aggregate types related to Moisture Damage	Quartzite, Granite, Siliceous, aggregates,	Chert, Gravel, Carbo-nate Rocks	Sand Gravel	Quartzite, Limestone, Argillite	Lime stone, Dolomite, Natural sand	Chert sands, Gravel, Siliceous Aggregates	NA	NA	Glacial Till, Gravel, Natural sand with fines	Quartzite
Tests for Moisture Damage	AASHTO T 283	IL-Modified T 283	Kansas Modified T 283	AASHTO T 283	AASHTO T 283	AASHTO T 283	AASHTO T 283	NA	AASHTO T 283	ASTM D 4867

Table 3.1 Summary of the WisDOT-UW Survey on Mid-western States Perspectives on Moisture Damage (continued)

Question (1)	Iowa (2)	Illinois (3)	Kansas (4)	Minnesota (5)	Michigan (6)	Missouri (7)	Nebraska (8)	N. Dakota (9)	Ohio (10)	S. Dakota (11)
Criteria for passing TSR value	80%	75%	80%	80% for 152.4 mm Cores 70% for 101.6 mm Cores	80%	80%	NA	NA	70% without anti-strip 80% with anti-strip	Low traffic 60% Medium traffic 70% High traffic 80%
How Effective	Good but not 100% reliable	No response	No idea	No Idea	Very effective, No failure	Effective	NA	NA	Effective for materials + #152.4 mm sieve	No response
Version of T 283 Used	T 283 No freeze thaw	Illinois T 283	55% Saturation level	T 283 No Freeze thaw	Fixed saturation level	No modifica-tion	NA	NA	No modifica-tion	No freeze thaw
Correla-tion of test with field	Yes	Yes	Not known	Yes	No	Yes	NA	NA	Sometimes	No response
Ideal test for Moisture Damage	ALF	Hamburg	No such test exists	No response	Simulate actual mix produc-tion in lab	Multiple freeze thaw with T 283	NA	NA	No response	No response
Corrective measure for Moisture Damage	Lime	Anti-strip and re-test	Anti-strip	Anti-strip	None	More lime, New aggregate source	NA	NA	Change aggregate source or anti-strip	Hydrated Lime

- ***Is Moisture Damage a Factor for Premature Failure of Pavements?***

In response to this question, 8 out of the 10 states replied yes while Nebraska and North Dakota replied no. Nebraska indicated that their mixes were designed and constructed as dense mixes (impermeable) and only recently they have started to open up the gradation, and are observing the behavior of the mixes from moisture damage considerations. North Dakota indicated that the presence of adequate aggregates in the state and low traffic levels have contributed to very minimal moisture damage problems, but their main concern was the low temperature cracking and fatigue cracking.

- ***What Procedure is Adopted for Identifying the Moisture Damage in the Field?***

This is considered as one of the most important question of the WisDOT-UW survey in view of the absence of a definite quantitative methodology to determine the presence of moisture damage problem in a given asphalt pavement. Four states indicated the use of visual inspection of cores. The Kansas and Minnesota DOT indicated that they monitor the TSR value of the mixes during project construction at a frequency of one on the first lot of production followed by one per week or 10,000 tons, whichever is lesser. Ohio DOT indicated that they randomly sample the cores and send it to the DOT lab for further evaluation, while Michigan DOT indicated that they had no procedure to identify the moisture damage problem in the field.

The above responses emphasize the need for the development of a quantitative method to identify moisture damage in the field. Development of such a method would help in refining the existing laboratory moisture damage test methods.

- ***Which Distresses are Associated with Moisture Damage?***

Table 3.2 prepared from the WisDOT–UW survey responses indicate that majority of the mid-western states consider raveling, rutting, and fatigue cracking as the key indicators of moisture damage. It is also important to note that in the literature some states have related bleeding (or the appearance of blisters/blobs of asphalt on the pavements surface) as a potential indicator of moisture damage (Campbell Crawford, prepared discussions on

Kandhal et al., 1989). In this study, it is proposed select the pavement sections for field coring by giving adequate considerations to raveling, rutting, fatigue cracking, potholes, and bleeding/blisters.

Table 3.2 States Attributing Different Distresses to Moisture Damage

Type of Distress (1)	Number of States (Total 10) (2)
Raveling	6
Rutting	4
Fatigue Cracking	3
Shoveling	1
Bleeding	1
Potholes	1
Disintegration (bottom up)	1
Stripping	2

- ***What is Typical Age of the Pavements at which Moisture Damage Related Distresses are Observed?***

Based on WisDOT –UW survey (Table 3.3), it is evident that there is no definite age that moisture damage manifests in a pavement and is dependent on factors such as quality of aggregates (Illinois versus Iowa), construction practices and environmental factors (Kansas and Ohio).

- ***What is Role of the Aggregates in Moisture Damage?***

Table 3.4 summarizes the moisture susceptible aggregates in mid-western states. It indicates that asphalt pavements constructed using gravel, quartzite, limestone chert and natural sand aggregates are most susceptible to moisture damage for a given set of traffic, environmental and construction conditions.

Table 3.3 Average age of Visible Moisture Damage

State (1)	Average age of Visible Moisture Damage (2)
Michigan	2-5 years
Illinois	Less than 3 years
Minnesota	3-5 years
Ohio	4-6 years
Iowa	Seldom occurs due to hydrated lime in the mix. But was recently observed in an 8 year old pavement
Kansas	Depends upon the previous year's climate, compaction control, and type of aggregate.
Missouri	Environment related

Table 3.4 Moisture Susceptible Aggregates Identified by Mid-Western States

Aggregate (1)	Number of States (2)
Gravel	5
Quartzite	4
Limestone	3
Chert	2
Natural sand with fines	2
Carbonate Rocks	1
Dolomite	1
Glacial Till	1

- ***What is Moisture Damage Test Adopted in the Mid-Western States?***

In response to question on the moisture damage tests adopted by the states, 9 out of 10 states indicated the use of a version of AASHTO T 283 test. South Dakota reported the use of ASTM D 4867 and Ohio DOT reported the use of Methylene Blue Adsorption test (for material finer than 75 microns) in addition to the ASHTO T 283 test. From Table 3.1, it can be seen that 4 states require a minimum TSR of 80%, 1 state (Illinois) requires TSR of 75%. Three states have distinct criteria based on:

- Size of the test sample: Minnesota - 70% and 80% for 101.6 mm and 152.4 mm specimens respectively;
- Anti-strip additives: Ohio - 70% and 80% for plain and anti-strip modified mixes respectively; and
- Traffic levels: S. Dakota - 60%, 70%, and 80% for low, medium and heavy traffic levels.

It must be noted that WisDOT and MinDOT are the only two states that have the least TSR criteria among the Mid-western states.

- ***What Version of the AASHTO T 283 Test is Used ?***

In response to the WisDOT-UW question on the versions of AASHTO T 283 used, Iowa and Minnesota DOT, and S. Dakota (ASTM D 4867) indicate need to eliminate the freeze-thaw testing, while Kansas and Michigan required a fixed level of saturation of the specimens, and S. Dakota. Only 4 states indicated good correlation between the lab results with the field experience of moisture damage. Michigan indicated that no correlation existed between the lab results and the field experience of moisture damage. Kansas and Ohio indicated that they were not sure about the usefulness of the test in correlating with the field performance.

- ***What is Ideal Test for Moisture Damage?***

When asked to identify an ideal test for moisture damage, Kansas indicated that there has been no ideal test, 2 states recommended Hamburg/ALF type testing, Michigan indicated the need to simulate the field mix production in the lab, and Missouri recommended the use of multiple freeze thaw cycles. Detailed responses are given in Table 3.1.

- ***What are Field Procedures Adopted to Control Moisture Damage?***

The WisDOT-UW survey reported that six states used hydrated lime (anti-strip agents), and/or changed aggregate sources to ensure that the mixes failing the moisture damage tests passed the criteria.

3.4 Summary of the Mid-Western States Experience on Moisture Damage

The results obtained from the UW-Survey and NCHRP 175 survey regarding the experiences of the mid-western states with moisture damage indicate the following points:

- Moisture damage is a concern among most of the mid-western states.
- Visual inspection of the field cores is the most commonly adopted method to determine the presence of moisture damage.
- Rutting, raveling, fatigue cracking, potholes and bleeding/blisters are considered as key indicators of moisture damage.
- For untreated mixes the general consensus is that the moisture damage should appear within the first 5 - 6 years.
- Gravel, quartzite and limestone were ranked as the aggregate types most susceptible to moisture damage. In addition, chert, carbonate rocks, dolomite and glacial till were also identified by at least one state as problematic aggregates.
- Most of the states (9 out of 10) used a version of the AASHTO T 283 test as a moisture damage test. Wisconsin and Minnesota are the two states with the lowest Tensile Strength Ratio criteria (70%).
- There is no overwhelming consensus about the correlation between the lab results and field performance and some states recommend using a constant saturation of samples, eliminating (or increasing) the freeze-thaw cycles.

CHAPTER FOUR
TESTING METHODOLOGY

4.1 Introduction

This chapter provides a comprehensive account of fieldwork and laboratory testing of the samples drilled from fourteen highway sections located in the state of Wisconsin. It covers selection of test sections, selection of procedure for identification of moisture damage in the field, drilling of cores for laboratory testing, and the investigative procedure adopted to determine the moisture susceptibility in the laboratory. All these activities constitute Task 3 as described in section 1.4 entitled Research Methodology.

4.2 Selection of Test Sections

The objective of Task 3 of this study is to evaluate the effect of moisture damage, if any, for mixes with a wide range of TSR values. This evaluation should result in explaining the effectiveness of the existing TSR test results and should provide an assessment of the moisture damage in the field. The TSR database and the WisDOT pavement management database were judiciously used to select the test sections (surface mixes only) having both unmodified (no anti-strip additives) and modified mixes. The selection criteria included a wide range of aggregate geology, low and high TSR values, and low and high Pavement Distress Index (PDI) values.

Based on the above criteria, 50 sections were preliminarily selected from 1991-1995 construction seasons. The information pertaining to the actual location of the test sections was identified with the help of the WisDOT and asphalt industry personnel. Subsequently, the WisDOT files were searched to locate the Reference Points (RP) of the test projects. These RP's were cross-referenced in the WisDOT Pavement Management Database to obtain the PDI for the test section.

It was possible to identify the PDI values of about 30 test sections. It must be noted that an average PDI of each test section was obtained by taking into account the length of each test section. Table 4.1 shows the list of test sections with their TSR values and corresponding PDI values. To correlate the TSR values with the PDI values, a scatter plot was prepared as shown in Figure 4.1.

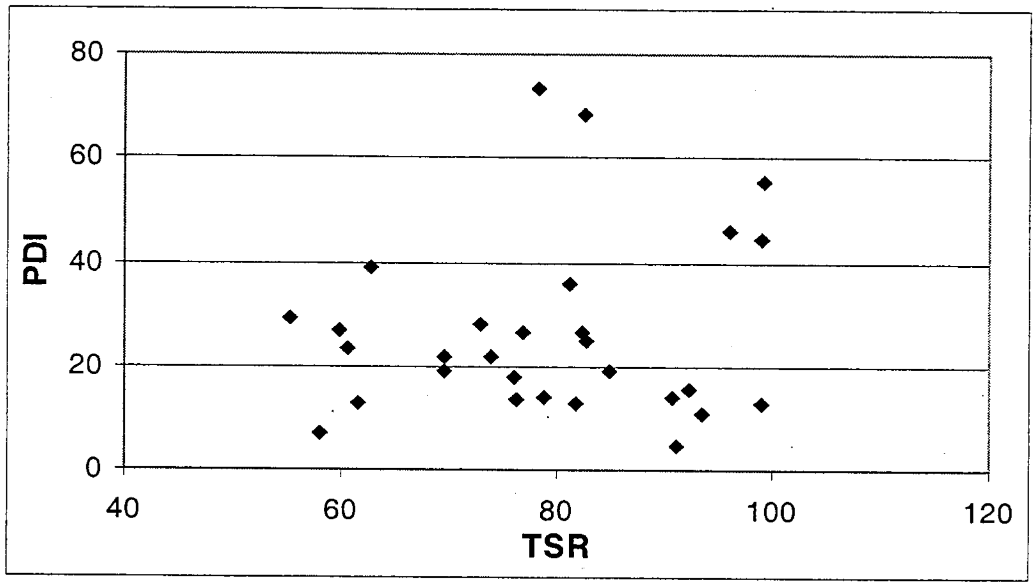


Figure 4.1 Relationship between TSR values and Pavement Distress Index (PDI)

It can be seen that there is no definite relation between the TSR and PDI for these sections. As a result, it was concluded that the use of PDI as a factor to select the test sections for field coring, and for the evaluation of the moisture damage problem may not be effective.

Due to the absence of relationship between the TSR and the PDI as seen above, further analysis of the TSR and PDI data was stopped and it was decided to select the test sections from considerations of geological type of aggregates and TSR values. Since 1992 WisDOT TSR data were suspect (personal communications, of Ms. Judy Ryan of WisDOT), it was decided to focus on the pavements/overlays constructed during the 1993 season only. This decision was made to consider pavements with longest pavement life and the most possible moisture damage. The details of the pavements/overlays constructed during the 1993 season are shown in Table 4.2.

Table 4.1 Comparison of Lab TSR with Actual Field Performance Indicator (PDI)

Year (1)	Design Number (2)	Length (km) (3)	Additive Used (4)	Average TSR (5)	Average PDI (6)
1991	250-2002	22.526	Yes	78	73.4
	250-2108	3.218	No	81	36.0
1992	250-2248	6.414	No	55	29.3
	250-2193	30.571	No	60	26.9
	250-2182	9.654	No	61	23.3
	250-2082	14.481	No	83	25.0
	250-2245	8.045	No	76	13.6
	250-2047	4.827	No	62	13.0
	250-2020	3.218	Yes	73	28.0
1993	250-2328	1.609	No	58	7.0
	250-2113	9.654	No	70	19.0
	250-2239	8.045	No	77	26.6
	250-2111	3.218	No	82	13.0
	256-2081	4.827	No	70	21.7
	250-2094	17.699	No	74	21.9
	250-2046	1.609	No	62	39.0
	256-2286	1.609	Yes	99	13.0
	250-2174	1.609	Yes	92	15.5
	250-2173	1.609	Yes	96	46.0
	250-2279	4.827	Yes	85	19.0
	250-2071	4.827	Yes	91	14.0
	250-2205	4.827	Yes	94	11.0
	250-2083	4.827	Yes	99	55.3
1994	250-2032	12.872	Yes	76	17.9
	250-2220	8.045	No	79	14.2
	250-2036	8.045	No	82	26.6
1995	250-2087	8.045	Yes	99	44.5
	250-2087	3.218	Yes	99	13.0
	250-2104	12.872	Yes	83	68.3
	256-2178	4.827	No	91	4.7

The selection of test sections was made to include a high and low TSR values from each aggregate mineralogical type. In all, a total of 14 test sections representing 7 geological types of aggregates were selected for field coring in Task 3. A specific designation number (GEO code) based on mineralogy was assigned to every aggregate type. The test sections represent 7 of the 9 mineralogies that were used in mixes from 1991 to 1996. Tables 4.3 lists the properties of the mixes used for paving the test sections that were cored. The table indicates that in addition to covering a wide range of aggregate geological properties, wide range of TSR values is also included. The table also shows that the wet and dry strength values vary within a significantly wide range.

The locations of 14 sections selected for drilling are given in Figure 4.2. The coring locations were selected with help from contractors that built the pavements. Coring was done with significant help from the contractors. All samples were sealed and transferred to the UW – Madison laboratories on the same day.

4.3 Moisture Damage Identification Procedure

The test procedure recommended for this project is based on Maupin's quantitative procedure (Maupin, 1989), as described in chapter 2 of this thesis. Few minor modifications were made in the procedure to meet the objectives of this study. The procedure involved following phases to determine the presence of moisture damage problems in the field.

Phase 1: Field Coring

In this phase, random cores were taken over a 0.805km stretch of the test section using Kandhal's recommendation (1994). For four-lane highways, the cores were obtained from the inside wheel track of the slow-traffic (outside) lane and for two lane highways, cores were taken from the outside wheel track of the lane. As described in Appendix B, 12 cores from each test section were found sufficient for subsequent testing. The 12 samples were drilled and transported to the laboratory from each of the fourteen sections in sealed plastic bags. The field cores were blotted to Saturated Surface Dry (SSD) condition right after the coring and sealed.

Table 4.2 Selection of Test Sections from 1993 Construction Season Based on Geology and TSR Values

Geology (1)	Projects (2)	No Additive (3)		With Additive (4)	
		Low TSR (3)a	High TSR (3)b	Low TSR (4)a	High TSR (4)b
1	5	72.2, 72.3, 72.9	82 -	77.3 -	- -
2	8	75.3, 74.1, 76.4, 79.3, 70.7	83.6 - -	75.7 - -	84.4 - -
3	4	73.6, 76.6	86.6, 99.4	-	-
4	-	-	-	-	-
5	15	77.3, 77.6, 71.1, 78.2, 78	94.5, 89.1, 88.6, 90.9, 85.0, 85.1, 84.1, 87.5	- - - -	83.1, 89.1 - - -
6	4	76.3	81.6, 83.3, 86.6	-	-
7	41	72.9, 79.3, 77.0, 79.4, 71.6, 79.0, 79.9, 75.0, 70.9, 77.6, 73.5, 70.0, 70.2, 78.7, 73.6, 70.3, 75.5, 72.4, 74.4, 77.7, 73.3, 70.2, 72.9, 79.3	80.9, 80.2, 93.2, 99.5, 86.5, 96.3, 92.7, 80.7, 97.6, 95.2, 93.1, 95.8, 95.2, 90.3, 86.5, 86.1 - - - -	- - - - - - - - - - - -	- - - - - - - - - - - -
8	5	77.8, 73.6, 75.4	- -	- -	86.6, 84.7
Combined	17	-	-	-	-
Total	95	-	-	-	-

Figure 4.2 Location of Test Sections

Table 4.3 Properties of the Mixes Selected for Field Coring in Task 3

#	HWY	LIMITS	DOT ID OF PROJECT	CONTRACTOR'S ID OF MIX	MIX TYPE	CONTRACTOR	DOT DIST.	COUNTY
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	78	USH 14 - USH 12 Rd	250-2339-93	230F93	MV3	PAYNE & DOLAN	1	Dane
2	64	USH 63 - E. County Line	250-2083-93	8035MVS	MV3	MATHY	6	St. Croix
3	14	Spring Green –Wisconsin River Road	250-2046-93	134F93	HV3	PAYNE & DOLAN	1	Sauk
4	35	Desoto - Genoa Road	250-2075-93	8059MVS	MV3	MATHY	5	Vernon
5	10 - Mondovi	Mondovi - 6th St in Eleva	250-2324-93	8093S	MV3	MATHY	5	Buffalo
6	51-Mathy	E. Albert St. – CTH CX	250-2124-93	8857HVS	HV2	MATHY	1	Columbia
7	51-P&D	STH 16 – Ontario Street	250-2205-93	180F93	HV3,HV2	PAYNE & DOLAN	1	Columbia
8	100	13th to 27th Street, City of Oak Creek	250-2238-93	101F93	HV3	PAYNE & DOLAN	2	Milwaukee
9	116	South Corporate Limits - STH 21/Bear Creek Bridge	250-2135-93	146F93	MV2	PAYNE & DOLAN	3	Winnebago
10	10 - Clark	CTH B - Collier Road	250-2330-93	8769MVS	MV3	MATHY	6	Clark
11	12 - Harding Ave	Harding Ave uphill	250-2067-93	8325HVS	HV3	MATHY	6	Eu Claire
12	62	City Limits, City of St. Francis	256-2071-93	108F93	MV2R	PAYNE & DOLAN	2	Milwaukee
13	29	STH 128 - CTH N	250-2039-93	8023MVS	MV3	MATHY	6	St. Croix
14	30	CTH T to TT STH 30 - CTH N	250-2179-93	167F93	MV3	PAYNE & DOLAN	1	Dane

Table 4.3 Properties of the Mixes Selected for Field Coring in Task 3 (continued)

#	HWY	GEO CODE	AGGREGATE SOURCE	MINEROLOGY	AC CONTENT	AC SOURCE	AC GRADE	BLOWS/END	ADDITIVE	DRY ITS (kN/m^2)	WET ITS (kN/m^2)	TSR (DOT)
(1)	(2)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
1	78	7	Ballweg Pit	Igneous + Carbonate Dolomite	5.3	KOCH	120-150	8	No	453	337	0.74
2	64	9	Simonson Quarry	Chippewa/St. Croix Older Gravel	5.7	KOCH	120-150	20	No	417	414	0.99
3	14	2	Yanggen quarry	Platteville Dolomite	5.8	KOCH	85-100	30	No	737	460	0.63
4	35	6	Pedretti Quarry	Prairie Du Chein Dolomite	6.2	KOCH	85-100	20	No	762	461	0.61
5	10-Mondovi	9	Windsand Quarry	Chippewa/St. Croix Older Gravel	5.3	KOCH	85-100	12	No	683	443	0.65
6	51-Mathy	7	Kettleson Quarry	Igneous + Carbonate Dolomite	5.5	KOCH	85-100	20	No	601	441	0.73
7	51-P&D	6	Kohn Quarry	Prairie Du Chein Dolomite	5.7	KOCH	85-100	26	No	742	694	0.94
8	100	5	Franklin quarry	Niagra Dolomite	6.2	SENECA	MAC 10	19	No	676	574	0.85
9	116	1	Larsen quarry	Platteville & Prairie Du Chein Dolomite	5.3	KOCH	120-150	15	No	368	265	0.72
10	10-Clark	6A	Boone Quarry	Precambrian Crystalline Rock	6.2	KOCH	85-100	14	Wetfix	565	539	0.95
11	12-Harding Ave	6	Parker Quarry	Prairie Du Chein Dolomite	5.5	KOCH	85-100	22	No	638	447	0.70
12	62	5	Franklin quarry	Niagra Dolomite	6.2	AMOCO	120-150	21	No	721	654	0.91
13	29	6	Cotts Quarry	Prairie Du Chein Dolomite	6.4	KOCH	85-100	18	No	678	574	0.85
14	30	7	Anderson Pit	Igneous + Carbonate Dolomite	5.9	KOCH	120-150	18	No	445	279	0.63

Phase 2: Lab Testing

Figure 4.3 describes overall procedure followed in testing. On average, twelve surface cores were separated from the samples drilled from the field for each of the fourteen test sections. The samples were visually inspected to determine the boundaries of the surface layer and were cut using a diamond saw with water-cooling. The cores were numbered numerically from 1 to 12. Three cores (marked RICE) were used for determining the theoretical maximum specific gravity, G_{mm}. Following is a detailed description of the testing of these cores. For the purpose of clear understanding, examples of a test section from Highway 14 are used frequently.

Step 1. Determination of Theoretical Maximum Specific Gravity, G_{mm}

The purpose of determining the G_{mm} is to facilitate the determination of the in-situ air voids of the field cores. Three cores, 1 - 3 (RICE), were used for determination of theoretical maximum specific gravity, G_{mm}, according to the flask method as given in AASHTO T 209-94. The cores were melted by heating for 120 minutes in an oven maintained at 135 °C. The loose mixture from the cores was distributed into three piles by a quartering technique as described in AASHTO T 209-94. Average of the three G_{mm} was then taken as the basis for determining the air voids of the field cores in subsequent stages of the testing. An example of the measured G_{mm} values is given in Table 4.4.

Table 4.4 Determination of G_{mm} by Flask Method According to AASHTO T 209-94

SAMPLE ID (1)	A (2)	D (3)	E (4)	G_{mm} (5)	Average G_{mm} (6)
RICE 1	1287.3	7479.1	8254.5	2.5147	-
RICE 2	1294.6	7479.1	8258.4	2.5123	2.517
RICE 3	1254.7	7479.1	8236.5	2.5230	-

In Table 4.4, A = Weight of the material in grams;

D = Weight of the flask filled with water in grams; and

E = Weight of the flask with water and material

Step 2: Determination of Bulk specific gravity, G_{mb}

G_{mb} of the remaining nine cores 4 - 9 were determined in accordance with the test method AASHTO T 166. Table 4.5 shows an example of the results of this step.

Table 4.5 Determination of Bulk Specific Gravity, G_{mb}

HW (1)	**Core ID** (2)	**Dry Mass** (3)	**Mass in Water** (4)	**SSD Mass** (5)	$\mathbf{G_{mb}}$ (6)
14	4	1422.2	844.6	1422.9	2.4593
14	5	1569.4	936.4	1570.0	2.4770
14	6	1570.5	930.8	1571.3	2.4520
14	7	1456.6	866.1	1457.4	2.4634
14	8	1421.6	850.1	1422.1	2.4853
14	9	1522.7	902.5	1523.8	2.4508
14	10	1532.8	912.9	1534.2	2.4671
14	11	1514.8	903.9	1515.7	2.4760
14	12	1561.8	933.8	1562.4	2.4846

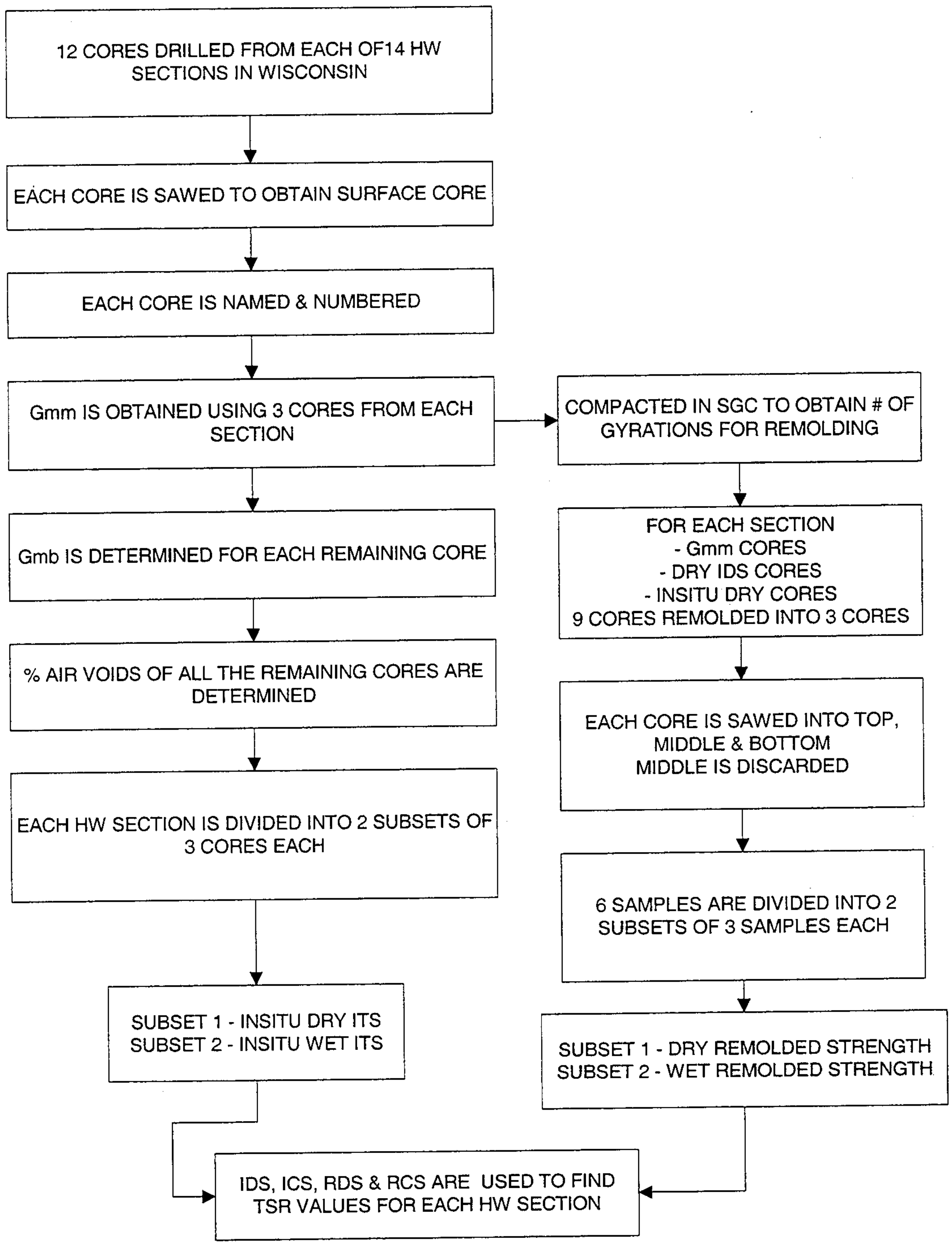


Figure 4.3 Flow Chart Depicting Testing Procedure

Step 3: Determination of % Air Voids

G_{mm} and G_{mb} from the above steps was used to determine the % air voids of the individual cores as well as the average of all the cores in each section as per AASHTO T 269-94. Table 4.6 summarizes an example of the results from this step.

Table 4.6 Determination of Percent Air Void Content of Field Cores

HWY (1)	Core No. (2)	G_{mb} (3)	G_{mm} (4)	% Voids (5)	% Voids in Order (6)
14	4	2.4593	2.5177	2.3	1.3
14	5	2.4770	2.5177	1.6	1.3
14	6	2.4520	2.5177	2.6	1.6
14	7	2.4634	2.5177	2.2	1.7
14	8	2.4853	2.5177	1.3	2.0
14	9	2.4508	2.5177	2.7	2.2
14	10	2.4671	2.5177	2.0	2.3
14	11	2.4760	2.5177	1.7	2.6
14	12	2.4846	2.5177	1.3	2.7
14	1(RICE)	-	-	-	-
14	2(RICE)	-	-	-	-
14	3(RICE)	-	-	-	-

Step 4: Distribution of the Cores in Subsets for In-Situ TSR

Two subsets of three cores each were selected for determining the In-Situ Dry Strength (IDS), and In-Situ Conditioned (wet) Strength (ICS) for calculation of the in-situ TSR value. The selection was based on matching the average air voids of the two subsets with the average air voids of the field cores. The method is typically illustrated in Figure 4.4 and Table 4.7. Figure 4.4 shows the arrangement of per cent air voids in ascending order of nine field cores below and above the average air voids. Table 4.7 shows the selected cores for two subsets, in-situ ITS testing, and in-situ conditioned ITS testing. Cores for both the subsets were selected in such a manner that would yield the average air voids of each subset close to the average of all the nine cores. Also the selection was based on practical aspects of the subsequent testing.

For example, will the saturation be practically achievable in real time if the cores selected had very low air voids such as 1.3% in case of cores # 8 and #12. As such, the selection was made from the remaining cores, #s 4,5, 6,7,9,10, and 11. Dividing the remaining cores in pairs such that the air voids in both the cores of a pair are closely similar, facilitated the distribution of cores for each subset. For example, cores 5 and 11 having air voids as 1.6% and 1.7% respectively were distributed in two subsets. Similarly pair of cores 7 and 10, and 4 and 6, contributed one core each in each subset. The final selection therefore resulted in cores as shown in Table 4.7. This procedure was followed to match the average air voids for the dry in-situ subset (IDS) and the conditioned in-situ subset (ICS) to the average field voids for all highway sections.

Table 4.7 Average Air Voids of Subsets for In-situ ITS Tests

Cores and Their Distribution (1)	**Average Air Voids** (2)
IDS Cores - # 4, 5, 7	2.04 %
ICS Cores - # 6,10, 11	2.09 %

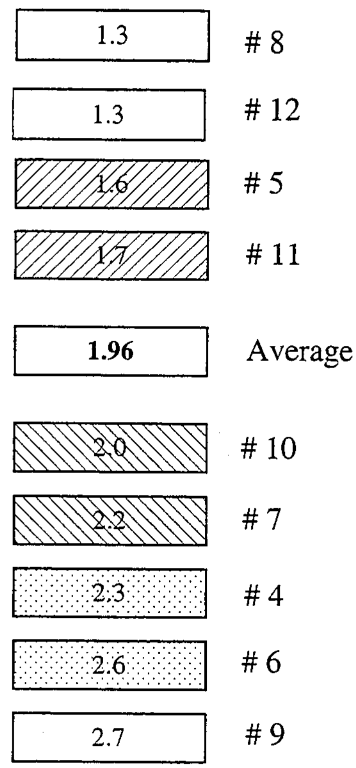


Figure 4.4 Distribution of Percent Air Voids in Field Cores (# 4 - # 12)

Step 5: Determination of Densification Characteristics for Remolding

Material from three cores used to measure G_{mm} cores was heated and compacted in the Superpave Gyratory Compactor (SGC) to a preset number of gyrations selected to achieve air voids close to the average percent of voids of the field cores. Generally the compaction was first done to 100 gyrations to establish the relationship between number of gyrations and % air voids. This relationship was then used to determine required number of gyrations to achieve in-situ air voids. The SGC yielded a sample of 152.4 mm diameter and varying height. G_{mb} of the compacted sample, G_{mm} of the mix, and the height of the specimen were used to determine the number of gyrations required for remolding the cores in a subsequent stage of testing to achieve in-situ air voids. Figure 4.5 and Table 4.8 depict the densification curve and sample data from the spreadsheet used in this analysis.

SGC typically is sensitive to the density required because the relationship between number of gyrations and % G_{mm} is exponential. Therefore, sensitivity analysis was done to determine the minimum number of gyrations that was required to achieve the required density, in this case, 98% of the G_{mm} or 2% air voids in the compacted sample. Table 4.9 and Figure 4.5 were used to determine the number of gyrations for subsequent remolding of the cores. It may be seen from table and the figure that 49 gyrations caused 98% G_{mm}. However, the total change in % G_{mm} from 40 gyrations to 49 gyrations was only 0.44%. Depending upon this sensitivity analysis and using the engineering judgement the number of gyrations were chosen as 45 to remold the cores for step 5 of the testing program.

Step 6: Determination of In-situ Dry ITS (IDS)

The following procedure describes the treatment of the three cores selected for the in-situ dry tensile strength tests from step 3.

Average diameter and thickness of the cores were taken according to AASHTO standard procedures. The cores were placed at 25° C in a water-bath for 20 minutes to equilibrate the temperature. Thereafter ITS test was performed according to AASHTO T 283.

The peak load of each core was used to determine the ITS according to the following formula:

$$\sigma_c = 2\,P / \pi\,D\,t \tag{4.1}$$

where σ_c = Indirect tensile strength, ITS;

P = Load in kN;

D = Average diameter of the core in m; and

t = Average thickness of the core in m.

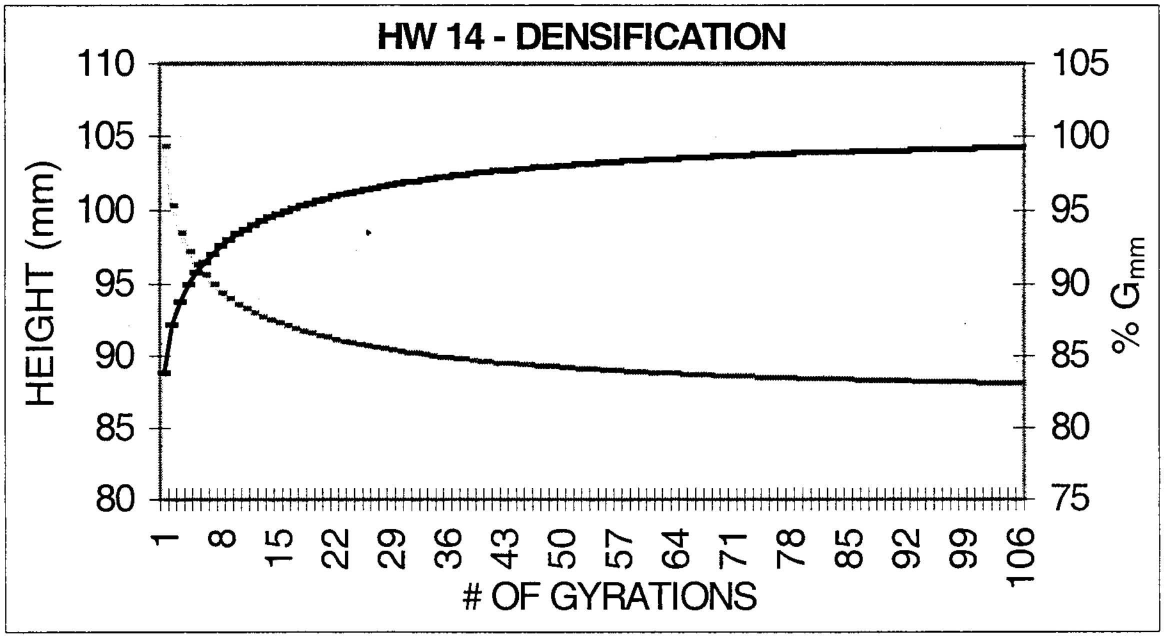


Figure 4.5 Densification Curve to Determine Number of Gyrations for Remolding

Amount of stripping in percentage was noted visually according to the procedure given in SHRP Protocol P05 for SHRP Test Designation AC05. The cores from seven HWY sections were also either video-imaged or shot by digital camera to determine if the stripping is better visualized by this technique and to document the stripping conditions. It was found that the technique does not capture stripping. Therefore, the procedure was not further pursued. Typical images of some cores are shown in Appendix C to this report.

Step 7: Determination of In-situ Conditioned ITS (ICS)

Three cores from step 3 selected for ICS test were tested according to AASHTO T 283. The cores were subjected to vacuum saturation to achieve a level of more than 55%. They were conditioned in water at 60 °C for 24 hours, and then were placed in water at 25 °C for two hours prior to ITS testing. The test yielded in-situ conditioned ITS (ICS) according to equation given in step 6. Stripping was noted as described in step 6.

Table 4.8 Sample of data Used to Determine Densification Required for Remolding

No. of Gyrations	Height (mm)	Gmb Estimated	Gmb Corrected	Gmm %	Change in Height (mm)	Change in % G_{mm}
0	104.3	2.05	2.10	83.81	-	-
1	100.3	2.14	2.19	87.15	4.0	3.34
2	98.5	2.18	2.23	88.75	1.8	1.59
3	97.2	2.20	2.26	89.93	1.3	1.19
4	96.3	2.22	2.28	90.77	0.9	0.84
5	95.6	2.24	2.30	91.44	0.7	0.66
6	95.0	2.26	2.31	92.02	0.6	0.58
7	94.4	2.27	2.32	92.60	0.6	0.58
8	94.0	2.28	2.33	93.00	0.4	0.39
9	93.6	2.29	2.34	93.39	0.4	0.40
10	93.3	2.30	2.35	93.69	0.3	0.30
11	93.0	2.30	2.36	94.00	0.3	0.30
12	92.7	2.31	2.37	94.30	0.3	0.30
13	92.5	2.32	2.37	94.50	0.2	0.20
14	92.3	2.32	2.38	94.71	0.2	0.20
15	92.1	2.33	2.38	94.91	0.2	0.21
16	91.9	2.33	2.39	95.12	0.2	0.21
17	91.7	2.34	2.39	95.33	0.2	0.21
18	91.6	2.34	2.40	95.43	0.1	0.10
19	91.4	2.34	2.40	95.64	0.2	0.21
20	91.3	2.35	2.40	95.75	0.1	0.10
21	91.1	2.35	2.41	95.96	0.2	0.21
22	91.0	2.35	2.41	96.06	0.1	0.11
23	90.9	2.36	2.41	96.17	0.1	0.11
24	90.8	2.36	2.42	96.27	0.1	0.11
25	90.7	2.36	2.42	96.38	0.1	0.11
26	90.6	2.36	2.42	96.49	0.1	0.11
27	90.5	2.37	2.42	96.59	0.1	0.11
28	90.4	2.37	2.43	96.70	0.1	0.11
29	90.3	2.37	2.43	96.81	0.1	0.11
30	90.2	2.38	2.43	96.91	0.1	0.11
31	90.2	2.38	2.43	96.91	0.0	0.00
32	90.1	2.38	2.44	97.02	0.1	0.11
33	90.0	2.38	2.44	97.13	0.1	0.11
34	89.9	2.38	2.44	97.24	0.1	0.11
35	89.9	2.38	2.44	97.24	0.0	0.00
36	89.8	2.39	2.44	97.34	0.1	0.11
37	89.8	2.39	2.44	97.34	0.0	0.00
38	89.7	2.39	2.45	97.45	0.1	0.11
39	89.6	2.39	2.45	97.56	0.1	0.11
40	89.6	2.39	2.45	97.56	0.0	0.00

Specimen Mass	3786.4g
G_{mm} (measured)	**2.5102g**

Dry Mass	3786.4g
Mass in Water	2269.9g
SSD Mass	3790.1g
G_{mb}	**2.4907g**

Step 8: Determination of In-situ TSR (TSR 1)

The ratio of ICS (Step 7) to the IDS (Step 6) yielded in-situ TSR (TSR 1).

Step 9: Remolding of Samples

Mass of nine samples, 3 each from G_{mm} test, IDS test, and in-situ samples was heated and remolded in SGC at a pre-determined number of gyrations (Step 5), into three cores of approximately equal size. G_{mb} of the samples was determined to ascertain that the air voids of the remolded samples lied within 1% of the average field voids.

Step 10: Distribution of the Remolded Samples into Dry and Wet Sub-sets

The remolded samples were designated R1, R 2, and R 3. Each one of the three remolded samples (Step 9) was cut to yield three cores such that the thickness of the top and bottom cores was the same as the average thickness of the field samples. Middle cores of the three remolded samples were discarded because they always were thin due to loss of material in remolding, and sawing. As such three remolded samples yielded six cores (3 top, 3 bottom) for subsequent testing. G_{mb} of the six cores were determined to calculate the average voids. The six cores were then divided into two sub-sets for determining dry and wet ITS in the same way as described in Step 4. Table 4.9 shows the end result for Highway 14.

Step 11: Determination of Remolded Dry ITS (RDS)

It was carried out in a similar way as described in Step 6.

Step 12: Determination of Remolded Conditioned ITS (RCS)

Same procedure as described in Step 7 was followed.

Step 13: Determination of Remolded TSR (TSR 2)

It was determined as a ratio of RCS (step 13) to RDS (step 11).

Table 4.9 Distribution of Remolded Cores into Dry and Conditioned Sub-Sets

% Voids of Cores (1)	% Voids in Order (2)	Core Identity (3)	Dry Sub-set (RDS) (4)	Conditioned Sub-set (RCS) (5)
2.01 for R1-B	1.65	R1-T	R1-T;	R1-B;
1.65 for R1-T	1.65	R2-B	R2-B; and	R2-T; and
1.65 for R2-B	1.70	R2-T	R3-T	R3 -B
1.70 for R2-T	2.01	R1-B	-	-
2.02 for R3-B	2.02	R3-B	-	-
2.22 for R3-T	2.22	R3-T	-	-
Sum = 11.26	Average = 1.88	-	Average % Voids = 1.84	Average % Voids = 1.91

Phase 3: Lab Analysis and Interpretation of Results

The laboratory testing yielded the following information:

- The in-situ dry indirect tensile strength of field cores (IDS),
- In-situ conditioned indirect tensile strength (ICS),
- Remolded dry indirect tensile strength (RDS),
- Remolded conditioned indirect tensile strength (RCS), and
- Visual rating of amount of stripping.

The ratio of "in-situ conditioned strength (ICS) to the in-situ dry strength" is calculated as the in-situ TSR (TSR 1) and the ratio of "remolded conditioned strength to the dry-remolded strength" is calculated as the remolded TSR (TSR 2) value of the mix. These TSR values are compared with the lab TSR value for the lab prepared samples (mix design) and the Pavement Performance Data of the test. It is anticipated that the TSR ratios will be used as follows:

1. Determine if any correlation exists between the lab prepared samples and the in-situ or the future in-situ TSR values

2. Determine the degree of correlation between each of the TSR values and the pavement performance indicators as determined using the WisDOT Pavement Distress Index database.

Should a correlation exist between the lab prepared samples and the in-situ TSR values then this finding will answer the WisDOT's first concern about the usefulness of the laboratory TSR values in predicting the moisture susceptibility of the field mixes. Also, should the pavements with high pavement distress consistently relate to low TSR values (<70%), then it can be concluded that there is a moisture damage problem in the given mix and the threshold value of 70% adequately distinguishes between moisture susceptible and non-susceptible mixes.

The analysis of the data generated by testing the samples from the field has been carried out in Chapter 6.

4.4 Pilot Project (Highway 23)

The preceding sections described the overall framework of testing. It also concluded in selection of a procedure, referred to as modified Maupin's procedure, for evaluating and correlating the lab and field TSR procedures. The procedure was discussed with WisDOT Technical Oversight Committee (TOC) in October 1997, and the need for a pilot study was felt due to the following considerations:

- Will the 12 cores be sufficient to conduct all the tests as planned in the testing phase? Because the tests were planned on the surface course only, the densification of the mixture under service conditions over the years may yield very thin slices when the surface course was sawed from the field sample. Subsequently it may not provide enough material for determining the G_{mm} from three cores, and their remolding for densification characteristics.
- What is the back up if some test samples give erroneous results, or are damaged in any phase of testing process?

There has been a concern that the specified testing required at least 3 cores in each subset. If during conditioning or remolding, one of the cores becomes unusable from the standpoint of test provisions, the average results may not be comparable.

- How will the required saturation level be achieved for conditioning?

The void contents of the original mix are usually about 8%. Over the years in service, the voids reduce due to densification of the pavement. If the voids are less than 3%, the desired saturation is hard to achieve in lab. Sometimes it is not attainable at all. Therefore it was required to estimate in what time and how the desired level of saturation be achieved in the lab.

WisDOT identified a section of Highway 23 near Dodgeville that exhibited distresses. It was therefore selected as a first project to test the modified Maupin procedure, and to address the above-mentioned concerns. During the third week of October 97, the UW-Madison, WisDOT, and the Payne and Dolan Inc. collected 12 field cores from HW 23 north-bound section on the outskirts of Dodgeville, Wisconsin. The main objective of this study was to determine if the reported pavement failures were related to stripping, and also ascertain the number of field cores required to evaluate the moisture damage potential of a given mix by the modified Maupin's procedure before carrying out the coring of the actual 14 sections selected for the study.

The testing procedure and the results are shown in Appendix B. The findings from the pilot program were as follows:

- It appears that 12 cores are sufficient to conduct the TSR study in the conceptual framework. The cores yielded enough material to conduct all the tests according to modified Maupin's procedure.
- The testing of the cores identified the loopholes in the practicality of test procedures required that need to be addressed before testing of the 14 sections is undertaken. The loopholes include: (1) how to attain desired saturation when air voids are significantly low and (2) how to avoid over-compaction in SGC during remolding of the cores.
- The TSR values of present as well as future condition represent a passing value (more than 70%) as per WisDOT specification.

- It seems that conditioning of cores with and without one freeze–thaw cycle, and remolding of the samples did not cause a significant moisture damage to the samples.

It may be noted here that WisDOT uses AASHTO T 283 with some modifications to determine TSR. For example, AASHTO T 283 requires that the samples to be used for determining dry ITS should be sealed in leak-proof plastic bags and placed in a water-bath for two hours at 25 °C. WisDOT testing procedure involves placing the samples in water without sealing them from water. For the results to be comparable, the cores were tested in the same manner as WisDOT does.

CHAPTER FIVE
COMPILATION OF RESULTS AND ANALYSIS

5.1 Introduction

This chapter includes a summary of the data generated from the lab testing of samples taken from the 14 highway sections that were selected for this project. The data include volumetric properties and indirect tension test results for the samples as recovered from the field and after conditioning them in the laboratory. The data set also includes the results of the remolded samples produced using the Superpave Gyratory Compactor (SGC) before and after conditions according to the AASHTO protocols. The chapter includes the analysis conducted to compare the tensile strength test results for the field and lab samples and also the comparison to the original mixture design results. A study of the factors affecting the TSR values is presented to address the objectives of the project.

The data have been organized in a database that consists of two distinct sets of data. One data set, henceforth referred to as WisDOT data set, includes the information as provided by the WisDOT or contractors on Job Mix Formulae (JMF), and TSR tests. This data is essentially the same as tabulated in Table 4.3. The other data set includes the results of laboratory tests performed on samples taken from the 14 highway sections of the project, henceforth referred to as UW-Madison data set. A table 5.1 summarizes this data.

Data from the two tables were used to compare the indirect tensile strengths and the TSR results of the fresh mix samples and the densified pavement samples. A detailed analysis to verify the potential of TSR test in predicting the future moisture damage is presented in this chapter. Also the threshold value for Wisconsin as specified by the WisDOT was evaluated in comparison to the Pavement Deterioration Indices (PDIs) of the selected sections. The findings from the analysis are used to recommend revisions of the current criteria to predict moisture damage.

5.2 Comparison of In-Situ and Remolded ITS

The indirect tension strength for mixtures used in each of the test sections studied were measured in four different conditions, as described in chapter 4:

- In-situ Dry Strength (IDS)
- In-situ Conditioned Strength (ICS)
- Remolded Dry Strength (RDS)
- Remolded Conditioned Strength (RCS)

Table 5.3 lists the results of in-situ and remolded ITS of the samples from all the 14 test sections.

5.2.1 Comparison of the Dry and Conditioned Strength Values

In theory the conditioned in-situ strength (ICS) should be lower than the in-situ dry strength (IDS) because conditioning in water according to the AASHTO T 283 is expected to induce more moisture damage. Comparison of the sections also reveals that ICS in 9 Sections HWY 78 through HWY 100 and HWY 30 is less than or nearly equal to the respective IDS, which support this theory. However, the remaining 5 Sections, HWY 116 through HWY 29, show an opposite trend.

Although some of this unexpected trend can be explained by the variability in the strength value, the level of variability observed could not account for the relatively high increase in strength after conditioning. The only speculation that could be offered to explain the increase in strength for the field samples is healing of the asphalt cracks resulting from the coring process as a result of conditioning at relatively high temperatures. In addition, the development of pore pressure within the capillary voids during the saturation and the change in air voids can have an effect on the strength.

Table 5.1 UW-Madison Data-Set

Col	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
#	**HWY SECTION**	**ACTUAL AV. THICKNESS** (mm)	**IN-SITU AV. DIA.** (mm)	**ACTUAL VOIDS** (%)	**IDS VOIDS** (%)	**IDS THICKNESS** (mm)	**IDS DIA.** (mm)	**IDS** (kN/m^2)	**ICS VOIDS** (%)	**ICS THICK-NESS** (mm)	**ICS DIA.** (mm)	**ICS AV. INITIAL SAT.**(%)	**ICS AV. FINAL SAT.** (%)	**ICS DELTA SAT** (%)	**ICS** (kN/m^2)
1	78	32.4	144.6	3.98	4.3	33.02	144.8	1143	4.2	30.5	144.8	75.4	79.5	4.1	938
2	64	32.7	146.1	5.89	6.3	30.48	147.3	1222	5.6	30.5	147.3	75.9	114.0	38.1	540
3	14	35.5	144.7	1.96	2.0	35.56	144.8	1638	2.1	38.1	144.8	69.9	105.5	35.6	1218
4	35	37.1	144.2	4.10	4.2	35.56	144.8	1043	4.2	38.1	144.8	61.3	75.3	14.0	896
5	10-Mondovi	57.9	144.1	4.30	4.3	58.42	144.8	1597	4.4	58.4	144.8	58.1	62.7	4.6	1085
6	51-Mathy	45.4	144.1	3.23	3.3	43.18	144.8	1196	3.3	45.7	144.8	60.2	70.7	10.4	900
7	51-P&D	43.1	144.6	3.20	3.2	43.18	144.8	1281	3.2	43.2	144.8	67.5	113.9	46.4	832
8	100	35.5	144.3	4.40	4.4	38.10	144.8	946	4.5	35.6	144.8	63.6	77.9	14.2	785
9	116	38.5	143.9	2.98	3.2	40.64	144.8	821	3.4	35.6	144.8	61.4	68.2	6.8	1071
10	10-Clark	42.7	144.3	4.17	4.4	40.64	144.8	725	4.3	43.2	144.8	57.7	74.9	17.2	1016
11	12-Hrading Ave	40.9	144.3	7.41	7.3	40.64	144.8	578	7.4	38.1	144.8	77.4	91.5	14.2	750
12	62	41.0	144.4	6.91	7.0	40.64	144.8	536	6.9	40.6	144.8	63.9	73.2	9.4	887
13	29	40.5	144.1	5.86	6.4	45.72	144.8	436	6.0	35.6	144.8	61.0	68.1	7.1	737
14	30	31.0	144.1	1.90	2.0	33.02	144.8	1403	2.2	30.5	142.2	58.0	66.9	8.9	1443

Table 5.2 UW-Madison Data Set (continued)

Col.		17	18	19	20	21	22	23	24	25	26	27	28	29
#	HW SECTION	RDS	RDS	RDS	RDS	RCS	RCS	RCS	RCS	RCS	RCS		TSR 1	TSR 2
		VOIDS	THICKNESS	DIA.	RDS	VOIDS	THICKNESS	DIA.	AV. INITIAL	AV. FINAL	DELTA	RCS	IN-SITU	REMOL-DED
		(%)	(mm)	(mm)	(kN/ m^2)	(%)	(mm)	(mm)	SAT. (%)	SAT. (%)	SAT. (%)	(kN/ m^2)		
1	78	4.0	33.0	149.9	1038	4.1	33.0	149.9	56.9	72.5	10.3	1012	0.8	1.0
2	64	5.8	35.6	149.9	1237	5.9	35.6	149.9	66.9	110.1	5.2	506	0.4	0.4
3	14	1.8	35.6	149.9	1261	1.9	35.6	149.9	58.4	83.8	12.6	1233	0.7	1.0
4	35	5.0	38.1	149.9	945	4.8	38.1	149.9	66.2	75.5	8.4	825	0.9	0.9
5	10-Mondovi	5.1	58.4	149.9	1165	5.1	58.4	149.9	69.6	100.0	8.9	874	0.7	0.7
6	51-Mathy	4.2	45.7	149.9	1067	4.3	48.3	149.9	62.5	82.1	10.5	1034	0.8	1.0
7	51-P&D	3.4	45.7	149.9	1043	3.5	45.7	149.9	67.6	98.3	9.8	965	0.6	0.9
8	100	3.9	35.6	149.9	854	3.9	35.6	149.9	63.7	82.5	10.1	989	0.8	1.2
9	116	1.7	40.6	149.9	1707	1.9	40.6	149.9	58.9	71.8	15.5	1524	1.3	0.9
10	10-Clark	3.3	43.2	149.9	1035	3.3	43.2	149.9	72.6	87.9	10.6	1044	1.4	1.0
11	12-Harding Ave	7.1	43.2	149.9	1111	7.1	43.2	149.9	74.4	128.9	6.7	662	1.3	0.6
12	62	5.8	40.6	149.9	1481	6.0	40.6	149.9	66.7	80.5	12.6	1240	1.7	0.8
13	29	6.3	45.7	149.9	954	6.3	45.7	149.9	62.0	104.9	6.7	654	1.7	0.7
14	30	3.0	33.0	149.9	1250	3.1	33.0	149.9	66.5	88.4	12.6	1240	1.0	1.0

The results of the remolded samples are also shown in Table 5.3. With only one exception, the conditioned samples (RCS) show lower values than the remolded dry samples (RDS). Unlike the results for the in-situ samples, these test results confirm the theory that conditioning in water should result in reduction of the tensile strength values. The remolded sample results can also be used to support the speculation of the healing effect of the in-situ samples.

5.2.2 Comparison of In-situ and Remolded Strength Values

According to Modified Maupin Procedure remolded strength should be higher than the in-situ strength values because it is expected that by heating the field samples the water damage induced during the service life of the mixture is removed and the asphalt regains its bond to the aggregates. The results shown in Table 5.3 do not always support this theory. In fact in 8 cases (out of 14) the remolded dry strength (RDS) is lower than the in-situ dry strength (IDS). In a few of these cases, the inherent variability of the strength values can explain this unexpected result while in other cases the variability could not explain the results.

This finding is very significant because it makes the comparison between the laboratory and the field samples difficult. It appears that remolding is changing the structure of the mixture to such a level that it is not possible to assume the mixtures to be similar in properties. There are several factors that may contribute to this discrepancy:

- The gyratory compactor is not simulative of the rolling pattern in the field.
- Remolding the samples is resulting in some crushing of the aggregates.
- Asphalt content is changing due to losses and due to the exposed aggregates resulting from the coring.
- Cutting the remolded samples could have resulted in damage.

Regardless of the reason, it is clear that the in-situ strengths could not be directly compared to the remolded strength values. This observation, however, does not lead to the conclusion that the TSR values of the in-situ samples and the remolded samples could not be compared. Since the TSR values are ratios related to the same mixture, produced by the same procedure, they should be useful to understand the reaction of the mixtures to the moisture conditioning.

Table 5.3 Comparison of In-situ and Remolded Strength

HWY SECTIONS (1)	IDS (kN/ m²) (2)	High IDS (kN/ m²) (3)	Low IDS (kN/ m²) (4)	ICS (kN/ m²) (5)	High ICS (kN/ m²) (6)	Low ICS (kN/ m²) (7)	RDS ((kN/ m²) (8)	High RDS (kN/ m²) (9)	Low RDS (kN/ m²) (10)	RCS (kN/ m²) (11)	High RCS (kN/ m²) (12)	Low RCS (kN/ m²) (13)
78	1143	1335	110	938	964	916	1038	1055	1014	1012	1055	989
64	1222	1285	1109	540	597	425	1237	1328	1145	506	556	439
14	1638	1798	1514	1218	1337	1123	1261	1373	1225	1233	1336	1058
35	1043	1071	1003	896	948	864	945	1002	843	825	880	743
10-Mondovi	1597	1654	1560	1085	1209	952	1165	1198	1143	874	1004	781
51-Mathy	1196	1309	1040	900	1025	691	1067	1104	1002	1034	1098	973
51-P&D	1281	1313	1250	832	1020	721	1043	1055	1031	965	994	937
100	946	965	915	785	829	705	854	916	788	989	1063	942
116	821	883	743	1071	1089	1052	1707	1747	1651	1524	1586	1478
10-Clark	725	776	642	1016	1093	906	1035	1066	977	1044	1071	1009
12-Harding Ave	578	655	510	750	772	709	1111	1191	991	662	732	606
62	536	563	512	887	956	856	1481	1594	1343	1240	1385	1112
29	436	519	376	737	881	563	954	965	943	654	776	576
30	1403	1844	1129	1443	1515	1327	1250	1362	1183	1240	1253	1219

5.2.3 Comparison of % Air Voids and ITS

To study the effect of air voids on the tensile strength ration, plots of IDS, ICS, RDS, and RCS as a function of % air voids were prepared and are shown in Figure 5.1. The correlation coefficients are listed in Table 5.4. It appears that the IDS, ICS, and RCS have a moderate co-relation with the % Air Voids. However, RDS has a very low correlation coefficient. The effect of voids on tensile strength is always negative (low strength at higher air voids) and ranges between – 41.4 kN/m^2 and – 137.9 kN/m^2 per 1% change in air voids. It also appears that the effect is higher for the conditioned samples compared to the dry samples. There is significant scatter in the data. Therefore one cannot draw any generalizable conclusions.

Table 5.4 R^2 for % Air Voids and ITS

Strength	**R^2 with % Air Voids**
(1)	(2)
IDS	50.33%
ICS	48.03%
RDS	9.78%
RCS	58.27%

5.3 Comparison of the TSR Values

To understand the relationship between the reaction of different mixtures to moisture conditioning, three TSR values of each mixture were calculated:

- TSR 1: the TSR value of the in-situ samples before and after conditioning.
- TSR 2: the TSR values of the remolded samples before and after conditioning.
- TSR 3: the TSR values reported by WisDOT Laboratory prior to the construction.

Table 5.5 provides a summary of the values calculated and also lists the two extremes of the 95% confidence interval calculated using the standard deviation determined from the indirect tension test results of the dry and conditioned samples. The following sections give a detailed analysis of the listed results.

5.3.1 Variability of TSR Values

The inherent variability of the TSR 1 (in-situ), TSR 2 (remolded), and TSR 3 (WisDOT Lab.) was analyzed to evaluate the scatter or range of the TSR values. High and Low TSR values were calculated from the formulae contained in Section 6.5 of Chapter Six. Figure 5.2 depicts the variability of the TSR values.

It is observed that the TSR 1 (in-situ) has the maximum variability while TSR 3 (pre-construction) has the least variability. The range in variation of TSR1 based on the 95 % confidence interval is between 0.18 and 1.08. The range in TSR3 confidence interval is 0.07 and 0.26. The effect of using thinner samples cut from the field cores and the effect of variation in the samples are some of the reasons for this significant difference. The variability of the TSR values of the remolded samples TSR2 are between 0.10 and 0.31, which is much closer to the TSR3 variability.

5.3.2 Relationships between the TSR values

The degree of relationship between TSR 1, TSR 2, and TSR 3 are shown in Figures 5.5 and 5.6 in the format of scatter plots. It is observed that the relationships are almost non-existent.

This lack of relationship can be explained by a number of factors. The inherent variability of TSR test itself and the lack of reproducibility of the test in different conditions is a major factor. In addition, although the parameters of the TSR test such as dry, and conditioned tensile strengths, conditioning and testing practices of WisDOT and UW-Madison are the same, there are some differences that need to be highlighted for comparison of the results from the two data sets. A description of such differences is as follows:

i. ***Void content***: WisDOT performed TSR test for an average air voids of 7 ± 1%. UW performed the in-situ TSR test on the field void content that varied for each and every section. Generally it ranged from 1.9% to 7.4% for the field samples. The samples for remolded TSR matched their field voids ± 1%.

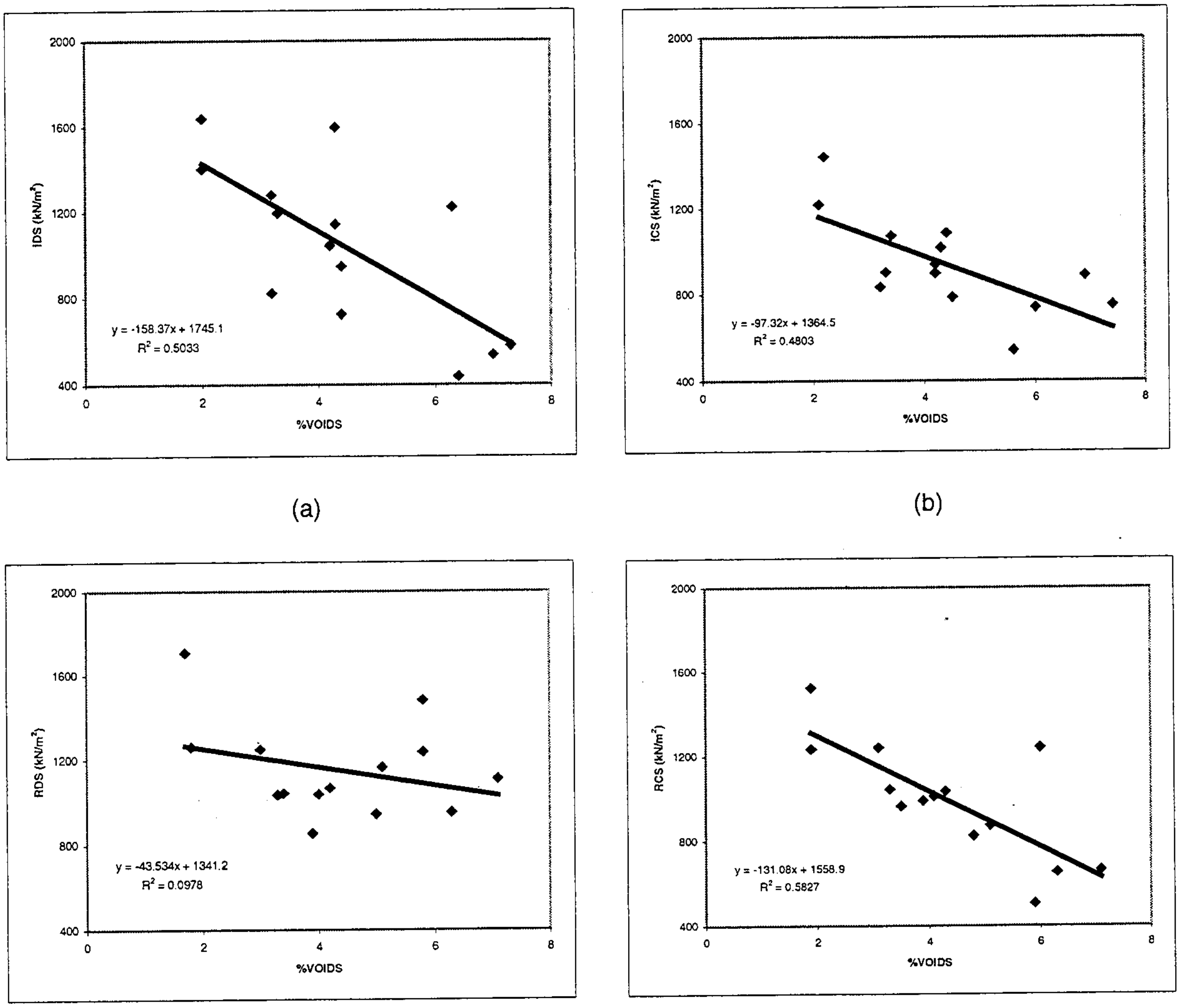


Figure 5.1 Plot of Indirect Tensile Strength versus % air Voids

(a) In-situ Dry Strength (IDS), (b) In-situ conditioned Strength (ICS)

(b) Remolded Dry Strength, (d) Remolded Conditioned Strength (RCS)

Table 5.5 Range of TSR Values Due to Variability in TSR Test Parameters

HW Sections (1)	TSR 1 (in-situ) (2)			TSR 2 (remolded) (3)			TSR 3 (DOT) (4)		
	TSR1-low (2)a	TSR1 (2)b	TSR1-high (2)c	TSR2-low (3)a	TSR2 (3)b	TSR2-high (3)c	TSR3-low (4)a	TSR3 (4)b	TSR3-high (4)c
78	0.62	0.83	1.02	0.93	0.98	1.02	0.69	0.74	0.80
64	0.35	0.44	0.53	0.34	0.43	0.48	0.93	0.99	1.06
14	0.64	0.74	0.85	0.81	0.93	1.12	0.59	0.63	0.67
35	0.80	0.87	0.92	0.74	0.88	1.00	0.58	0.61	0.64
10-Mondovi	0.58	0.67	0.78	0.63	0.76	0.87	0.52	0.65	0.78
51- Mathy	0.54	0.74	0.95	0.88	0.98	1.06	0.62	0.73	0.85
51-P&D	0.50	0.67	0.80	0.89	0.93	0.96	0.82	0.94	1.05
100	0.74	0.82	0.92	1.02	1.17	1.29	0.80	0.85	0.90
116	1.17	1.31	1.44	0.84	0.90	0.94	0.67	0.72	0.77
10-Clark	1.18	1.41	1.63	0.94	1.01	1.08	0.88	0.95	1.03
12-Harding Ave	1.10	1.28	1.50	0.50	0.58	0.69	0.63	0.70	0.78
62	1.50	1.68	1.81	0.70	0.85	0.97	0.86	0.91	0.95
29	1.15	1.64	2.23	0.55	0.70	0.81	0.81	0.85	0.88
30	0.69	0.98	1.37	0.90	0.98	1.08	0.54	0.63	0.72

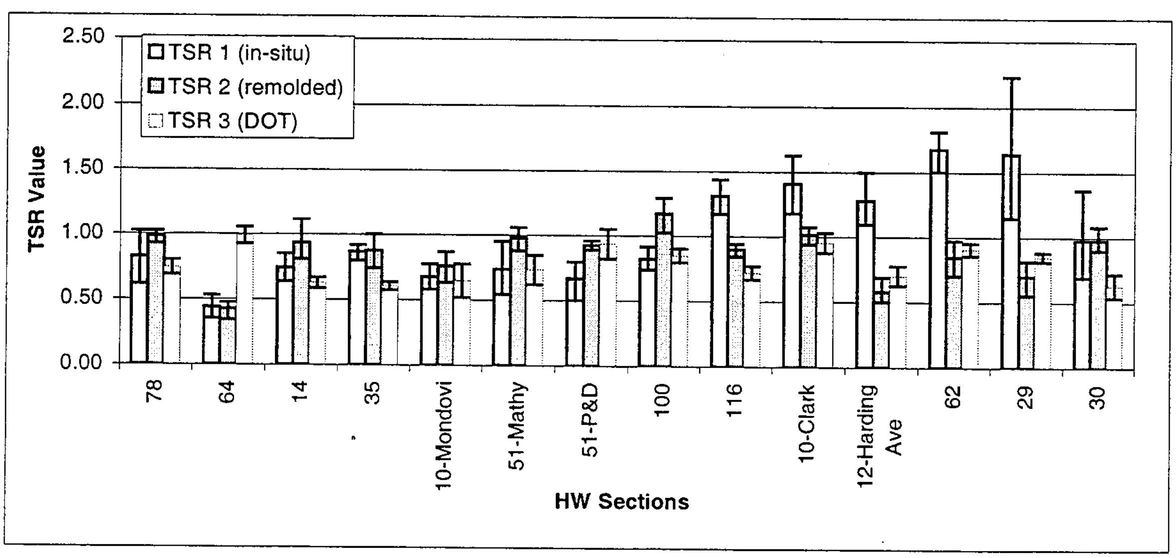


Figure 5.2 Range of TSR Values

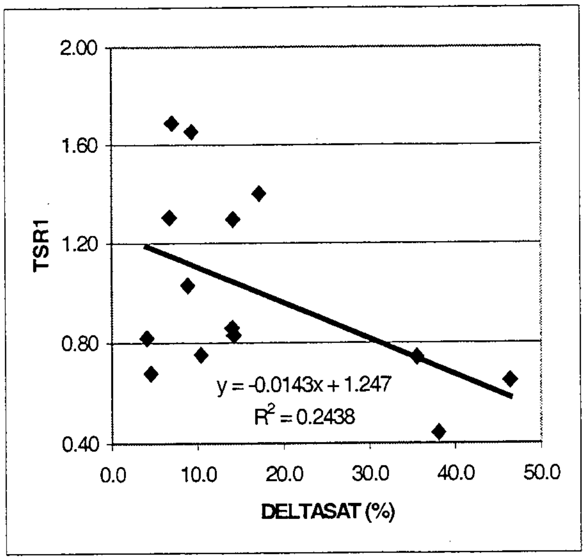


Figure 5.3 TSR1 Vrs. DELTASAT.

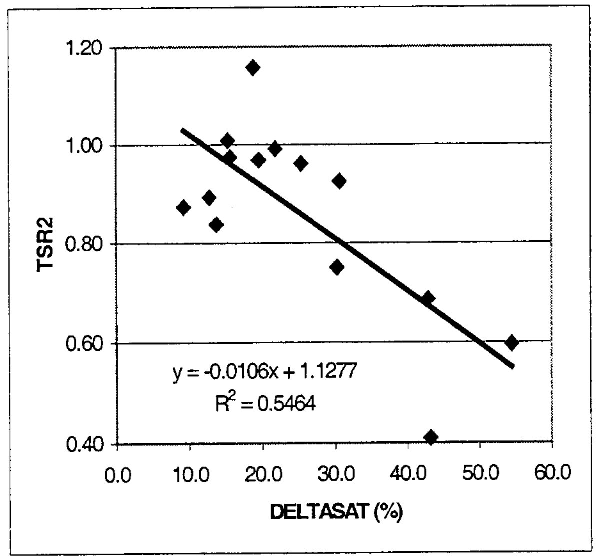
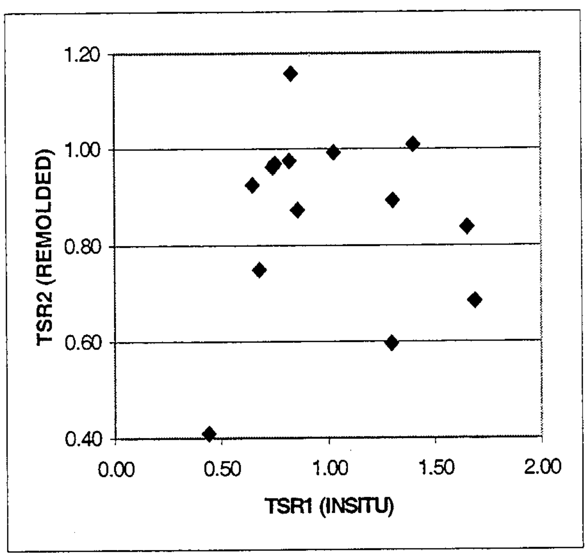


Figure 5.4 TSR2 Vrs. DELTASAT.



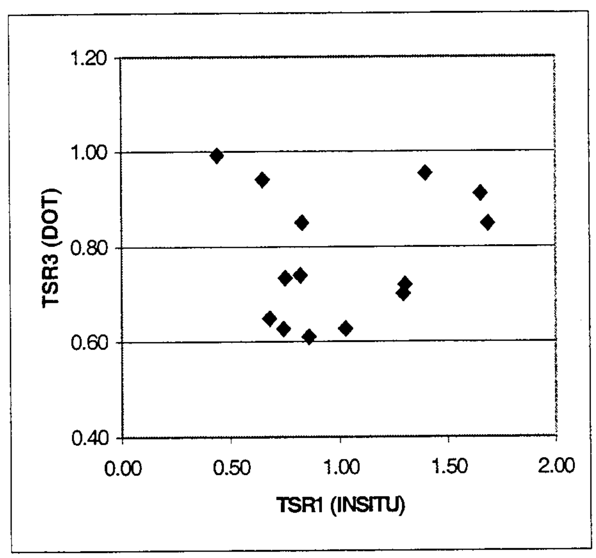


Figure 5.5 TSR1 (In-situ) Vrs. TSR2 and TSR3

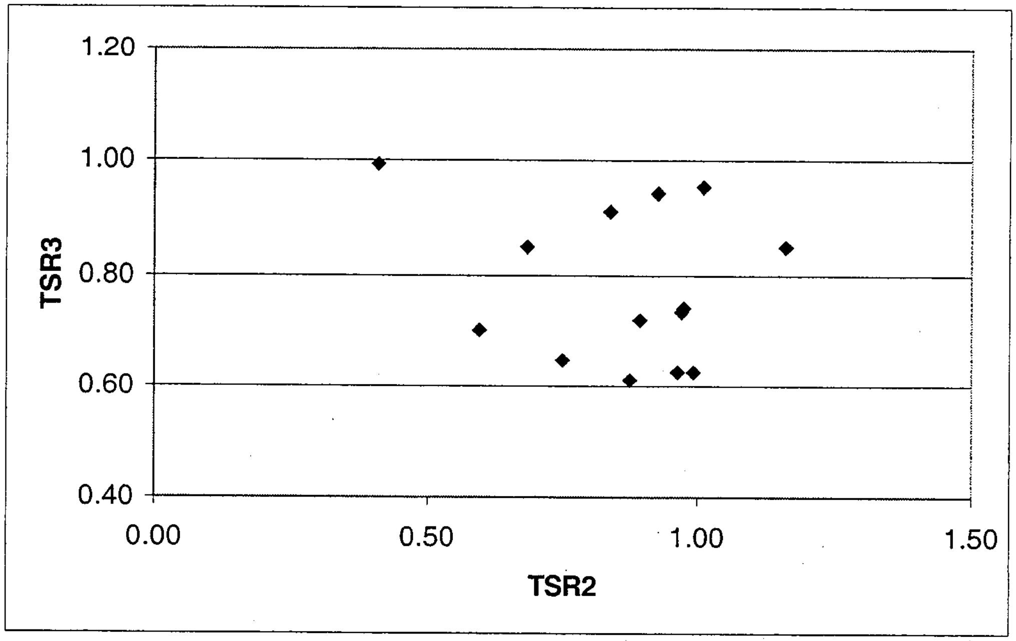


Figure 5.6 TSR2 (Remolded) Vrs. TSR3 (DOT)

ii. ***Thickness***: The thickness of the WisDOT samples was according to the specification of AASHTO T 283 test method i.e. 63.5 mm. The UW samples were as thick as the thickness of the surface course was for a particular highway section. The average thickness ranged from 31mm to 57.9 mm.

iii. ***Diameter***: Little variation was noticed in the diameters of the field samples, and remolded samples. Field samples were wet drilled by a core cutter. The average diameters of the field and remolded samples were about 144.78 mm to 149.86 mm respectively. However when compared to the average diameter of the WisDOT samples, the difference is significant. WisDOT samples have typically a 101.6 mm diameter as specified by AASHTO T 283 test method.

iv. ***Compaction Effort***: The WisDOT samples were prepared by Marshall Design Method in the lab using materials provided by the contractors. Compaction effort was in terms of blows per end of the specimen depending upon the density required at Optimum Asphalt Content (OAC). The UW samples were divided in two distinct categories. The samples used for determining the In-situ TSR were the same samples that were drilled from the field. No treatment was done to the exposed surfaces of the aggregates of the cores. The lack of asphalt coating that was cut away from the aggregate surfaces during sampling was inherent to these samples. The remolded samples prepared for determining the Remolded TSR, consisted of samples from G_{mm} test, IDS test, and three virgin samples from the field. All these nine samples were heated in the oven to break the integrity of the HMA, homogenized in a bucket mixer, and then compacted in SGC using predetermined number of gyrations to match the field density of a particular highway section. Therefore, the mode of compaction, and compaction effort for the UW samples is significantly different from that of WisDOT samples. The remolding has already been described in section 4.3 of Chapter 4.

5.4 Effect of Aggregate Mineralogy

The aggregates used in production of the HMA mixtures for the test sections contained a variety of mineralogical compositions. Table 5.7 summarizes the geological code assigned to different aggregate compositions, their mineralogy, and TSR values pertinent to the test sections. Geological code was used in statistical analysis in chapter 6 of this report. The mineralogy was identified by using WisDOT database, geological map of Wisconsin, and by consultation with the Department of Geological Sciences of UW - Madison.

The aggregates appear to be mostly Dolomites in nature, which have historically and experimentally shown resistance to stripping caused by moisture damage. The highway sections having such aggregates showed almost always satisfactory values. Only one source of aggregate, Chippewa / St.Croix Older Gravel, (HWY 64), has shown questionable TSR values. However the same aggregate performed reasonably well in case of HWY 10-Mondovi. Mineralogy alone therefore does not seem to explain the disagreement and variability of different TSR values. It is well known that production and construction methods can have significant effect on the moisture damage behavior. This issue is related to not having a defined relationship between lab produced and field produced samples.

Table 5.6 Aggregate Mineralogy and TSR Values

HW Section (1)	**GEO Code** (2)	**Aggregate Source** (3)	**Aggregate Mineralogy** (4)	**TSR1** (5)	**TSR2** (6)	**TSR3** (7)
78	7	Ballweg Pit	Igneous + Carbonate Dolomite	0.82	0.97	0.74
64	9	Simonson Quarry	Chippewa/St. Croix Older Gravel	0.44	0.41	0.99
14	2	Yanggen Quarry	Platteville Dolomite	0.74	0.96	0.63
35	6	Pedretti Quarry	Prairie Du Chein Dolomite	0.86	0.87	0.61
10-Mondovi	9	Windsand Quarry	Chippewa/St. Croix Older Gravel	0.68	0.75	0.65
51-Mathy	7	Kettleson Quarry	Igneous + Carbonate Dolomite	0.75	0.97	0.73
51-P&D	6	Kohn Quarry	Prairie Du Chein Dolomite	0.65	0.92	0.94
100	5	Franklin Quarry	Niagra Dolomite	0.83	1.16	0.85
116	1	Larsen quarry	Platteville & Prairie Du Chein Dolomite	1.30	0.89	0.72
10-Clark	6A	Boone Quarry	Precambrian Crystalline Rock	1.40	1.01	0.95
12-Harding Ave	6	Parker Quarry	Prairie Du Chein Dolomite	1.30	0.60	0.70
62	5	Franklin Quarry	Niagra Dolomite	1.65	0.84	0.91
29	6	Cotts Quarry	Prairie Du Chein Dolomite	1.69	0.68	0.85
30	7	Anderson Pit	Igneous + Carbonate Dolomite	1.03	0.99	0.63

5.5 Pavement Performance and TSR values

WisDOT has maintained a detailed database for indicators of Pavement Performance. Periodically each mile of a pavement is sampled – $1/10^{th}$ mile in length - for distress survey according to the current WisDOT Distress Survey Manual. An index number, termed as Pavement Distress Index PDI, calculated from measuring or estimating different components of distresses pertinent to a pavement type, represents the condition of a pavement. The higher the PDI number is, the more a pavement is assumed to have accumulated surface damage. Higher PDI number may or may not result from moisture damage. However, moisture damage can be a significant contributor to observed distresses. Because of the complexity of the effect of moisture damage, and because of the difficulty of associating one type of distress with the effect of moisture damage, the use of the overall PDI number is considered the best available choice. PDI data on the test sections, as provided by Ms. Judy Ryan of WisDOT, were examined to see if TSR values, as indicator of potential for moisture damage, are related to PDI numbers.

Figures 5.7 (a to c) are the plots of average PDI numbers versus TSR1, TSR2, and TSR3. Table 5.8 gives R^2 of the relationships. The figure illustrates that there is hardly any significant correlation between the PDI numbers and different sets of TSR values. Therefore, using PDI to estimate moisture damage may be misleading.

5.6 Distress Analysis of 14 test Sections

Fourteen sections selected for field coring from 1993 paving season were identified on the basis of mix design database of WisDOT and the contractors. Ms. Judy Ryan of WisDOT identified the same sections on PDI database of DOT. Eleven of the 14 sections were further analyzed to determine whether a particular distress or combination of the distresses is related to low TSR values. Three sections that could not be analyzed were highway 14, 30, and 12 - Harding Ave. Identification of two sections, highways 14 and 30, in PDI database was not possible. Highway 12 - Harding Ave was identified as JPCP in PDI database. PDIs of test sections were determined in 1997 and 1998, after 4 - 5 years of pavement performance.

Table D - 1 of Appendix D entitled "Distresses and TSR" provides a breakdown of PDI number in measured or observed distresses pertinent to test sections. Ten out of 11 sections have passing values of in-situ TSR (TSR1) and remolded TSR (TSR2). Only exception is HWY 64 that has TSR 1 and TSR 2 values as low as 0.4. TSR value of DOT (TSR 3) is also passing for all 11 sections except for HWY 35 and HWY 10-M (Mondovi) which have values of 0.6. Given the inherent variability of TSR test values, the mixes with value of TSR as 0.6 can not be considered failing mixes. The most common distresses associated with these sections are transverse and longitudinal cracks, which are basically shrinkage and reflective cracks and normally not associated with moisture damage. Although the extent of these distresses vary, the severity of almost all the cracks is towards the low end (1) on a scale of 0 - 3 (none to most severe). Rare cases of rut (HWY 116) and surface raveling (HWY 64) are not enough to establish that the distresses were caused by moisture damage. In both the cases the severity (1) is again low. Seal coated and crack fills are not considered distress indicators.

Highway 64 is identified as the most damaged section with block, transverse and longitudinal cracks, and surface raveling. Its low TSR1 and TSR2 values may be attributed to a combination of these distresses. However, to base the opinion on only one instance is not tangible.

In summary, no conclusive evidence could be found as to which distresses are associated with low TSR values on the basis of limited information on 14 test sections. Only 14 sections are not enough to draw any conclusions on the basis of available PDI database.

5.7 Effect of Air Voids on TSR Values

In the literature, it is indicated that air voids in the field can have a significant effect on the TSR values. The values of TSR1 and TSR2 are plotted as a function of air voids in Figures 5.8 and 5.9. As indicated there is a poor relationship between TSR1 and the average % air voids (R^2 = 21%). Moreover the trend of the curve shows an increase in TSR value with increase in the air voids, which is not supported by the available research (Dukatz et al., 1987). Air voids are expected to be inversely related to TSR value according to the available research. It has been indicated before that the TSR1 values are not reliable and are believed to be affected by the coring process.

Figure 5.9 depicts the relationship between the TSR2 values and the average % air voids. As shown the data depict the expected trend of TSR values decreasing with increasing air voids. The R^2 value of about 47% is a fairly good number to suggest that the remolded samples with fewer voids would give higher TSR values. The average change in the values of the TSR is 0.08 for every one-percentage change in air voids.

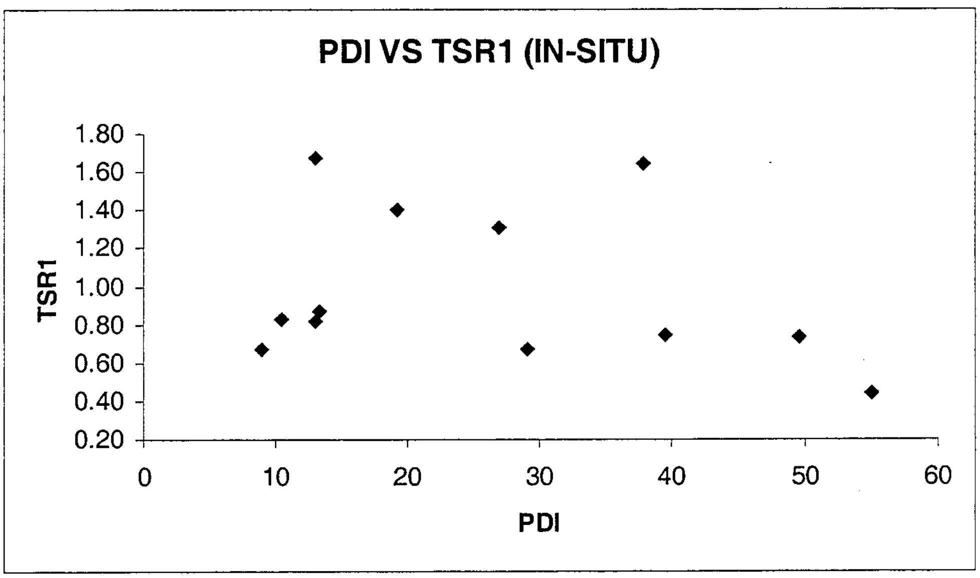


Figure 5.7(a) PDI versus TSR1

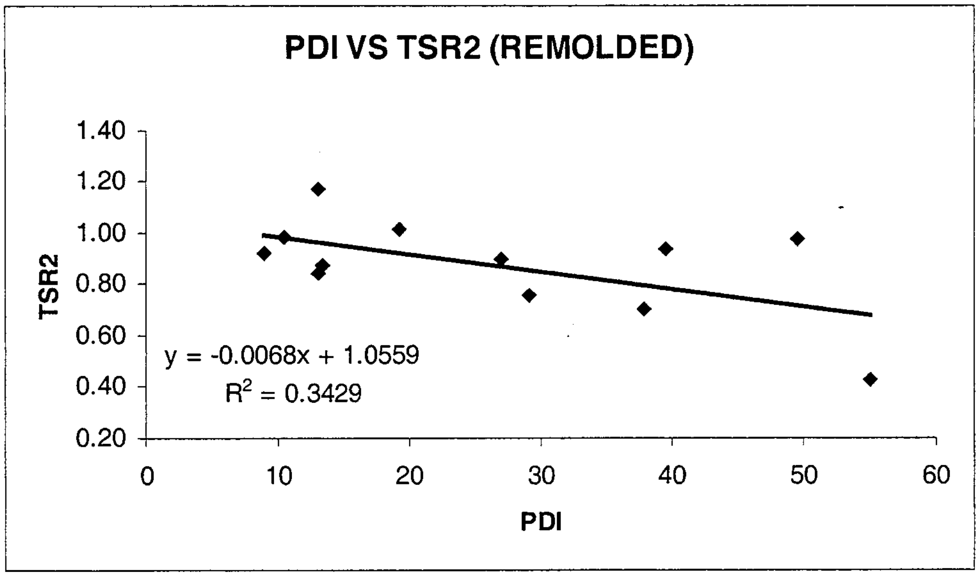


Figure 5.7(b) PDI versus TSR2

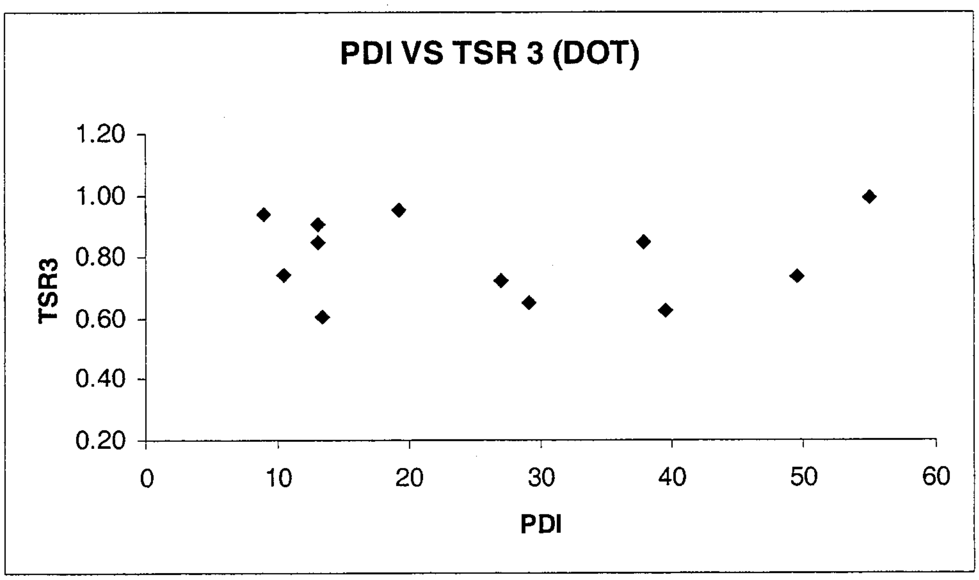


Figure 5.7(c) PDI versus TSR3

Figure 5.7 Correlation between Average PDI Numbers and TSR Values

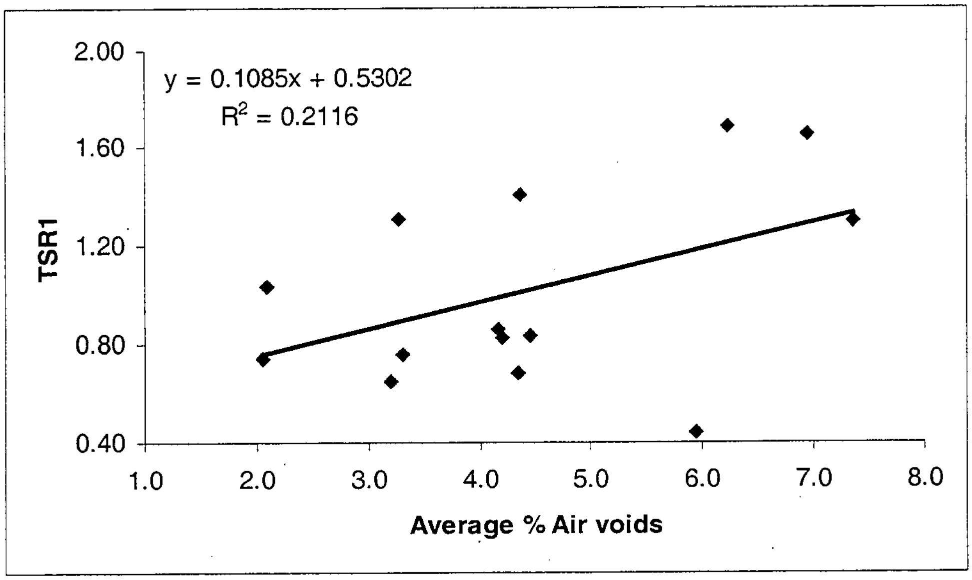


Figure 5.8 Average %Air Voids Vrs. TSR1

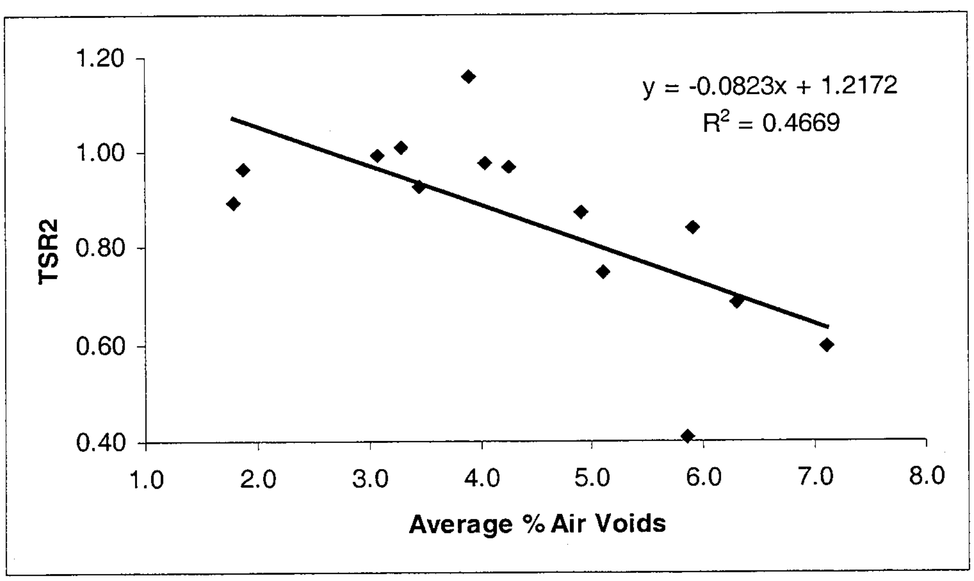


Figure 5.9 Average % Air Voids Vrs. TSR2

5.8 Summary of Field Study

In this chapter, the results of testing samples recovered from 14 sections were presented. The analysis of the results lead to the following findings:

- High variation in the TSR1 values was observed. In several of the sections, TSR1 values were higher than the value of 1.0. It appears that the coring process damages the field samples. After conditioning the samples with water at 60 °C, it appears that a partial healing of the damage is occurring, that results in increasing the TSR value.
- In case of 9 sections TSR1 is less than or equal to the TSR2 (TSR value of remolded sample), which is conceptually logical because remolding restores the HMA sample. On the other hand, the results from the other 5 sections are not logical (TSR1 is higher than TSR2). All the five sections have TSR1 values greater than 1, which identifies the problem with the field samples.
- No relationship was found between TSR1 and TSR2. It is speculated that degradation of aggregate might have occurred during the remolding process, resulting in lower values of TSR2 than the values of TSR1. Different compaction methods, in field and in lab, might have caused a reduction in strength for remolded samples. Healing of asphalt during the conditioning could have increased the conditioned strength, giving a higher TSR1 value.
- No strong agreement was found between the remolded TSR2 and the original TSR3 value. This was expected because the sample geometry, air voids, and materials are not identical. It was expected, however, that the TSR values would rank the materials similarly. Because of the wide confidence intervals calculated for the TSR values, it is very difficult to decide whether the field samples give the same ranking. For five of the sections the TSR values were similar.
- Average % Air Voids and TSR Values: Due to the damage of field samples the TSR1 (in-situ) values show very poor and illogical relationship with average % air voids. In TSR2 and TSR3, there is a clear trend that TSR values decrease with increasing air voids.

- Geology: Majority of the sections contained a mineralogical composition with Dolomite as the main component. Dolomites impart anti-stripping properties to the HMA. All such sections have generally good values in all of the TSR sets. Only one aggregate mineralogy, Chippewa / St. Croix Old Gravel used in HWY 64 section, did not perform well in TSR1 and TSR2 tests, with values of 0.44 and 0.41 respectively. However, TSR3 (DOT) value for the same source is 0.99. It is logical to believe that this source of aggregate is different and is expected to show lower TSR values. It is, however, difficult to explain the results from the mixture design. The statistical analysis indicated that geology of the commonly used aggregates in Wisconsin conditions does not significantly affect the TSR value results.
- Saturation in case of TSR1 and TSR2 does not seem to play an important role. In statistical analysis, it marginally improves the model to predict WET response.
- Average PDI values are not related to either of the TSR tests. It may be because the measurement of PDIs is done by sampling $1/10^{th}$ of a mile of a pavement, and the samples collected for TSR test may not fall in that surveyed section of a road.

CHAPTER SIX
ANALYSIS OF WISCONSIN TSR DATABASE

6.1 Need for Analysis of Historical TSR Database

Although there are different methods to measure the potential for moisture damage, a review of recent literature, including the advancements made during the Strategic Highway Research program, indicates that the retained-strength based tests are among the most widely used tests. The Modified Lottman test (AASHTO T 283) was proposed by Kandhal as a combination of the Root-Tunnicliff test and the original Lottman test (Kandhal, 1995). Since this test was adopted by AASHTO in 1985, State Highway Agencies (SHAs) have introduced several changes to the test procedure and the threshold values to adjust to the observed field performance. An NCHRP study by Hicks and his co-workers in 1991 (which included a survey of the SHAs) concluded that the T 283 is rated as the most effective test method to determine the potential for moisture damage of asphalt mixes. Recent studies, however, have identified several drawbacks of this method. Al Swalimi and Terrel (1993,1994) indicated that the variability associated with the AASHTO T 283 is relatively high and can be as high as 39%. They also indicated that the test procedure could show values higher than 1.0, which is unrealistic. Other researchers have indicated that the test may have been misused and that it is necessary to consider that absolute value of the wet strength in the criteria rather than looking at the TSR value only (Kandhal, 1995). Kandhal (1995) associated the problems with the use of this standard with the wide variability in the sample preparation procedures and the moisture conditioning methods that different states have used. Kandhal (1995) also indicated that a laboratory versus field correlation have to be the basis for developing a criteria.

WisDOT has been using the T 283 for approximately 5 years. There is a growing concern that this method is resulting in unnecessary use of anti-stripping additives. Since the moisture damage in asphalt mixes has been related to the aggregate, asphalt, and mix properties, there is a need to examine the historical TSR database maintained by the Wisconsin DOT and Asphalt Contractors. This can help in identifying the key factors that can contribute to moisture damage of asphalt pavements in Wisconsin. To achieve this, a database was established for mixture properties and TSR test results. The database, which was obtained from the WisDot and from the asphalt industry, was used in a statistical analysis to identify the key factors and to develop a statistical model for prediction of TSR values.

The database includes approximately 320 mixtures designed using aggregates from more than 100 quarries or gravel pits with a wide range of mixture volumetric properties. The statistical analysis was designed to explain the wide range in TSR values that are observed and possible sources of variability in the test results.

6.2 Description of the Wisconsin TSR Database

The materials testing laboratory of WisDot, Payne and Dolan Inc. (P&D) and Mathy Construction provided mix design data from 1991 through the 1996 paving seasons. The data was organized in a structured database according to a number of key variables. A brief description of the structure of the database is given below.

Of special importance was the data for the 1992 construction season WisDOT mix designs. This set of data was important because no additives were used in the mixes irrespective of whether the mixes passed or failed the Modified-Lottman test. The TSR data was grouped by district and by county, and each mix design had information pertaining to materials (aggregates, asphalt and anti-strip additives) and mix design and the TSR test results. The aggregate properties utilized in the development of the database were geological formation, unit weight, and the gradation properties of the Job Mix Formula (JMF). The asphalt cement information included unit weight, type, and the asphalt supplier. Information pertaining to the mix design and TSR test included the optimum asphalt content, Marshall stability, degree of compaction, wet and dry strength, and the degree of saturation. The database was structured to include 5 indicator variables, 10 measured variables, and 6 calculated variables. The response variables were identified as TSR and the wet strength of the mix. The details of the variables accounted for in this study are explained next.

6.2.1 Indicator variables

- **Aggregate Mineralogy (GEO):** More than 100 sources of aggregates were identified from the collected data. These sources were located on the geological map of Wisconsin, which shows the distribution of the bedrock in the state. A total of 9 predominant geological formations were identified in the State of Wisconsin with the assistance of the geologists at the Wisconsin Geological and Natural History Survey (WNHS) department. These aggregate types were predominantly dolomite formations. This guideline was used to group the aggregates used in each of the 316 mix designs.
- **Mixture Type (MT):** The individual mix designs were broadly are classified as A1, B1, AERO, Heavy traffic (HV), LV (Low Traffic), MV (Medium Traffic), Superpave, and SMA mixes. The mix types A1, B1, and AERO mixes correspond to the gradations used prior to 1993. The rest are mixes used after 1993. A detailed information about the WisDOT mixes can be obtained from Wisconsin DOT materials specifications (1990,1996). It must be noted that the mix type was accounted for in the analysis by accounting for its distinct properties such as Surface Area, Coarse/Fines Ratio, and Percent Fines.
- **Asphalt Type and Source (ACTYPE & ACSOURCE):** The asphalt cement used in the database was classified from source and penetration grade considerations. The asphalt types were 85/100, 120/185, 200/300, and AC-10 grades and were coded as indicator variables ranging from 1 - 4. The asphalt suppliers typically included Amoco, Koch, Murphy, and Gladstone. They were coded as 1 - 4.
- **Anti-Strip Type (ADDTYPE):** The database indicated the use of 8 distinct types of anti-strip additives, namely, hydrated lime, ACRA-2000, and Kling Beta. These were coded as indicator variables between 0 and 8 with 0 being the no-additive (unmodified mix). In addition, indicator variable ADD (0 and 1) was used to differentiate the unmodified (0) and the anti-strip additive modified mixes.

6.2.2 Measured Variables

- **Optimum Asphalt Content (ACCON):** This corresponds to the optimum asphalt content obtained from the Marshall mix design. In only two cases the mix designs were obtained using the Superpave volumetric mix design method.
- **Percentage Fines (FINES):** Since researchers have identified that excess fines in the mix could inhibit the coating of the asphalt over the aggregate surface, the percentage fines were included as a predictor variable in the database.
- **Air Voids (VOID):** Some of the mixes indicated air-voids that varied between a low of 4% and a high of 9%. Although the AASHTO T283 method requires that the air voids be at 7.0 ± 1.0 percent, the data included testing done using a constant compaction effort rather than a constant air void content. This data was intentionally included in this database to evaluate the effect of air voids outside the range of standard test method.
- **Compaction Effort (BLOW)**: Although most of the database indicated that the Marshall compaction effort was adjusted to ensure that the test mixes had an air-voids in the range of 7± 1 percent air voids. About 36 mix designs were evaluated for their TSR by compacting the mixes to 20 blows without exercising a control over the air voids.
- **TSR Values**: The TSR values of the mixes in the database corresponds to the ratio of the average conditioned (wet strength) to the unconditioned (dry strength) tensile strength of the mixes as per the AASHTO T 283 method, without performing the freeze-thaw conditioning. The TSR was determined as a simple average without accounting for the standard deviations in the dry and the wet strengths of the mix. The dry and wet strengths of the mixes were identified as **DRY** and **WET** respectively
- **Saturation:** The AASHTO T283 procedure involves vacuum saturation followed by conditioning in water at 140 F for 24 hours. During conditioning, the samples undergo saturation and swelling. To assess the effect of saturation on the TSR value, the percent initial and final saturation of the samples were used in the database and the predictor variables were coded as **SATI** and **SATF**

6.2.3 Calculated Variables

In addition to the measured variables, the following calculated variables were added to explore their relationship to TSR. These variables were selected based on the understanding of the moisture damage phenomenon. They were calculated using the aggregate gradation and the mix properties at OAC.

- **Surface Area (SAREA), Film Thickness (FILM):** Several researchers are of the opinion that mixes with gradations having low surface area of the aggregates and thicker asphalt film offer higher resistance to the moisture damage. To test this hypothesis, two variables (Surface Area (SAREA) and Film Thickness (FILM)) were added to the database. The calculation of the Surface Area of the aggregate gradations and Asphalt Film Thickness were calculated in accordance with the procedure outlined in the National Stone Association handbook (Barksdale, 1991) and Roberts et al. (1996).
- **Mastic Volume (MVOL) Partial Surface Area (PSA)**: Since it is recognized that mineral filler (P200) can be considered to be embedded in the asphalt film, and since in many cases moisture damage is observed at the surface of large aggregates, it is hypothesized by the authors that the true asphalt film is the mastic component of the mix. This is based on the assumption that the stripping in the asphalt mixes is more predominant with the coarse aggregates rather than with the fine aggregates. Based on this understanding, two more variables, Partial Surface Area-PSA (Surface area calculated without accounting the Fines) and the Mastic Volume-MVOL calculated as the summation of the volume of asphalt and the fines were included in the database.
- **Coarse to Fines Ratio (CF):** This variable was included to account for the differences in the aggregate gradation.
- **Net Saturation (DELTASAT):** Examining the database indicated that there is a wide range in the difference between the initial and the final saturation of the samples. To evaluate the effect of net saturation, this variable was added to the database.

6.3 Statistical Analysis of the TSR Database

The Statistical analysis of the TSR database was performed using the latest version of the Statistical Analysis System (SAS) Software. Two types of analyses were performed: Simple correlation and regression analysis.

6.3.1 Simple Correlation Analysis

Table 6.1 shows the correlation matrix for the variables included in the database. Of particular interest is the correlation of the TSR with other variables that are listed in the second column of the table. It can be observed that none of the values is higher than 0.500. The best correlation is with the final saturation of the specimen and the second best is with the dry strength.

Also of interest is the correlation of the wet strength with the other variables, which is shown in the 19th row. It is observed that the best correlation is with the dry strength at a value of 0.824. This correlation is rather impressive and indicates that a strong mixture can maintain its strength and resist the moisture damage efficiently.

There are other variables that are also highly correlated with the wet strength. The number of blows (correlation of 0.720), Marshall stability values, and the existence of the additive. These variables are however also correlated to the dry strength, which indicates that the improvement in correlation can be minimal if these variables are all combined in a linear model.

The results of the correlation analysis were somewhat surprising. They indicate that several of the well-known parameters that are often claimed to be related to moisture damage are not showing significant relationship to moisture damage. Of special interest is the geological type of the aggregates. The correlation of this parameter with TSR is 0.037 and with wet strength, it is 0.220, which indicates lack of relationship. To show the lack of relationship Figure 6.1 is prepared. In addition Table 6.2 lists the range of the TSR values for each of the aggregate sources. The data shown in the table and in the figure clearly show the lack of relationship and indicate that none of the sources can be classified as inferior or superior as measured by the TSR test.

Table 6.1 Correlation Matrix of Variables Included in Wisconsin Database of HMA Mixes (No. of Mixes = 317)

	GEO (1)	TSR (2)	STAB (3)	ADD (4)	ADTP E (5)	FINE (6)	CF (7)	SARE A (8)	PSA (9)	MVO L (10)	FILM (11)	ACTP E (12)	ACSO RCE 13)	ACCO N (14)	BLO W (15)	VOID (16)	SATI (17)	SATF (18)	DRY (19)	WET (20)
TSR	0.037																			
STAB	-0.227	-0.209																		
ADD	-0.202	0.291	0.406																	
ADTPE	-0.205	0.071	0.266	0.665																
FINES	-0.043	0.061	0.013	0.118	0.025															
CF	0.239	-0.025	-0.047	-0.039	0.009	-0.023														
SAREA	0.117	-0.030	-0.315	-0.166	-0.130	0.550	-0.254													
PSA	0.145	-0.044	-0.433	-0.309	-0.194	0.287	-0.285	0.820												
MVOL	0.107	0.054	-0.408	-0.153	-0.144	0.554	-0.283	0.580	0.463											
FILM	0.034	0.112	-0.043	0.001	-0.052	-0.460	0.033	-0.668	-0.507	-0.020										
ACTPE	0.060	0.167	-0.463	-0.216	-0.151	0.043	-0.004	0.229	0.312	0.219	-0.050									
ACSORCE	0.183	-0.106	-0.255	-0.107	-0.117	-0.080	0.126	0.199	0.271	0.114	-0.068	0.243								
ACCON	0.009	0.154	-0.223	-0.101	-0.162	0.216	-0.447	0.175	0.169	0.614	0.311	0.156	0.038							
BLOW	-0.178	-0.142	0.717	0.408	0.299	-0.056	-0.096	-0.339	-0.450	-0.366	0.005	-0.417	-0.294	-0.193						
VOID	-0.196	0.163	0.053	0.064	-0.025	0.019	-0.161	-0.055	-0.044	-0.070	0.037	0.065	-0.191	0.119	-0.059					
SATI	0.078	-0.116	-0.108	-0.158	-0.051	0.004	0.144	0.069	0.111	0.051	-0.065	0.081	0.098	-0.185	-0.089	-0.103				
SATF	0.025	-0.450	0.018	-0.152	-0.049	-0.033	0.217	0.137	0.186	-0.012	-0.176	0.122	0.199	-0.243	-0.057	-0.166	0.410			
DRY	-0.232	-0.410	0.735	0.372	0.341	-0.037	0.008	-0.264	-0.371	-0.395	-0.108	-0.563	-0.220	-0.352	0.741	-0.153	-0.013	0.070		
WET	-0.220	0.155	0.674	0.587	0.404	-0.009	-0.006	-0.303	-0.423	-0.395	-0.049	-0.504	-0.295	-0.310	0.720	-0.075	-0.088	-0.199	0.824	
DSAT	-0.036	-0.369	0.103	-0.031	-0.010	-0.037	0.109	0.085	0.104	-0.052	-0.129	0.062	0.127	-0.103	0.012	-0.089	-0.364	0.700	0.082	-0.135

The limitation of first order correlation procedure, similar to the one shown in Table 6.1, is that interactions and higher order relationships cannot be identified. The analysis of variance and regression analysis was used to overcome this limitation as described next.

Table 6.2 Range in TSR values for the different aggregate sources in Wisconsin

Source ID (1)	**Source Description (2)**	**Mean TSR (3)**	**Low TSR (4)**	**Hi TSR (5)**	**N* (6)**	**Sd (7)**	**95% CI (8)**
1	Platteville Prairie Du Clion	73.7	70.3	77.1	36.0	10.4	3.4
2	Platteville Dolomite	73.3	68.6	78.0	24.0	11.8	4.7
3	Galena Dolomite	81.1	73.3	88.9	9.0	11.9	7.8
4	Niagara Dolomite	78.8	76.1	81.4	68.0	11.1	2.6
5	Prairie Du Chein dolomite	74.5	69.2	79.7	27.0	14.0	5.3
6	Igneous and Dolomite	77.4	75.3	79.5	106.0	11.1	2.1
7	Igneous	77.1	72.8	81.3	20.0	9.7	4.3
8	Precambrian Rock	68.1	58.2	78.1	2.0	7.2	9.9
9	Chippewa Glacial	70.6	62.2	79.0	9.0	12.8	8.4

N* = No. of observations

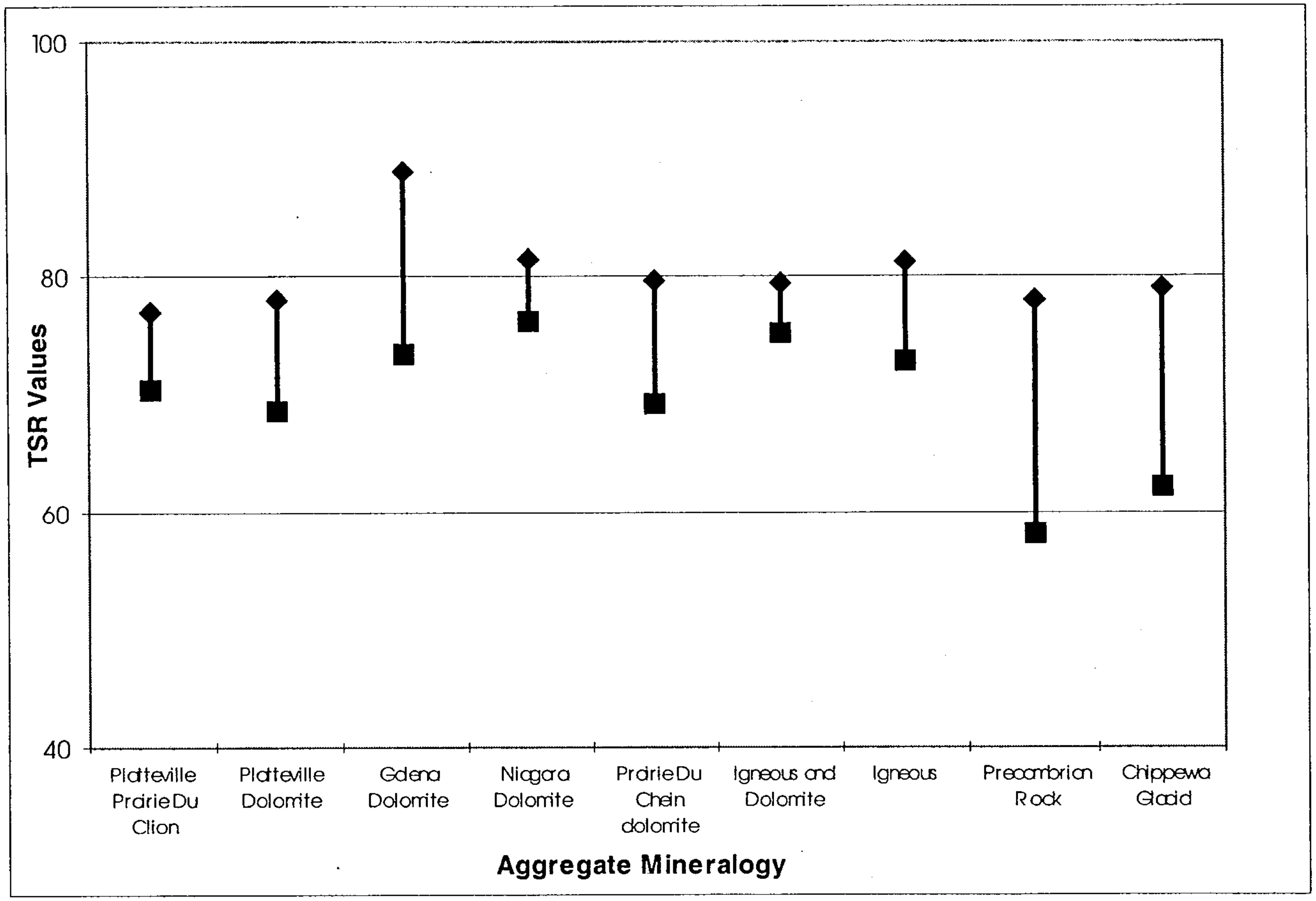


Figure 6.1 Range of TSR values for aggregates sources in Wisconsin

6.3.2 Multivariate Linear Regression Analysis

The first modeling effort focused on using the TSR as the response variable. However, since TSR is a derivative of the dry and wet strength, it was decided that a more realistic approach was to use the wet strength as the response variable and to use one of the step wise regression procedures in SAS. The method selected is called the backward elimination procedure. This procedure starts with a second order model, evaluating the p-values of the terms in each model, removing the insignificant second-order terms, and lastly removing the insignificant first-order terms. What are called parsimonious models are achieved by selecting the level of significance to evaluate the contribution of the variable to the regression model. The level in this case was selected at 0.0001. In other words the terms with p-values larger than 0.0001 are considered insignificant. Using WET as the response, the model selection method follows the following steps:

- Establish a second-order polynomial. For this analysis all possible interactions, except the interactions between discrete and continuous variables, were included.
- The non-estimable/insignificant two-factor interactions and quadratic terms in the second-order polynomial were removed which resulted in a model with fewer second-order.
- The insignificant main (first order) effects, which are not included in the two factor interactions, were removed. The insignificant main effects that are shown to be significant through a two-factor interaction were not removed.
- The back elimination process was then used to find the best model. Important factors that affect the validity of the model, such as equal variances and normality were monitored during the reduction of the model.

The resulting models are given in Table 6.3. The detailed analysis of variance tables for each of the models is given in Appendix E. Table 6.3 lists the models derived from the complete data set of approximately 320 mixtures. As shown it appears that there is no important interaction terms and the number of main effect variables are rather small. The dry strength, the existence of the additives and the saturation are the only important factors needed to result in a R-squared value of 0.819. Such high value is indicative of highly reliable model. The other models listed show that adding the air voids in a first and second order term does not improve the model significantly. The model with geology of aggregates is also shown in the table to emphasize the point that geology of aggregates is not found to be very important. Adding the geology as an indictor variable improved the R-squared of the model by only 0.008.

From Table 6.3, it may be seen that Model 3 with a fairly high R^2 value of 81.9 %, and a few variables may be used reasonably to predict the WET strength of a mix and therefore calculate TSR value. It is also concluded that the inclusion of 9 levels of variable GEO marginally improves the model as shown in Model 7. Therefore Model 3 is preferred for its simplicity and high level of reliability.

6.4 Variability Associated in the Wet and Dry Strength Values

Although the Modified-Lottman test has been widely adopted by most of the mid-western highway agencies, not much attention has been directed towards understanding the effect of variance associated with the dry and wet strength values of the samples tested. The importance of this issue could be realized from the significant variation in the range of the values of the wet set and dry set of the testing results assembled in this database.

Statistical averaging procedures for calculating the uncertainty of a given measured quantity can be best represented by the use of the propagation of error formulas (Ku, 1996, Wardrop, 1997). To determine the variability associated with the Tensile Strength Ratio (TSR) value using these formulas, standard deviations associated with the dry and wet strength of the mixes are used. Given the average strengths and standard deviations of the dry and wet mixes as X and Y, and σ_x and σ_y, the variance associated with the TSR is given by:

$$\textbf{Variance of TSR} = [\ \overline{X} / \overline{Y}]^2\ [\{(\sigma_x^2/n)/\ \overline{X}^2\} + \{(\sigma_y^2/n)/\ Y^2\}] \ ....(6.1)$$

where n is the number of tests performed to evaluate dry and wet strength.

The range of the TSR values at 95% probability is determined using the relation

$$\textbf{Confidence Interval} = \textbf{TSR} \pm 2\ (\textbf{Variance of TSR})^{1/2} \ .... (6.2)$$

Table 6.3 Models for Estimating Wet Strength

# (1)	Model (2)	R^2 (3)	CV (4)	Std. Error of Coefficient (5)	Root MSE (6)
1	WET = 99.17 + C1 Where C1 = -28.57 when ADD = 0 & C1 = -0 when ADD = 1	0.345	22.297	1.90170000 2.21820000 0.00000000	17.43
2	WET = 38.17 +C2 + 0.496 DRY Where C2 = -15.86 when ADD = 0 & C2 = 0 when ADD = 1	0.771	13.2122	2.76587000 1.41587000 0.00000000 0.02053800	10.33
3	WET = 10.26 + C3 + 1.059 DRY - 0.003 DRY^2 Where C3 = -16.09 when ADD = 0 & C3 = 0 when ADD = 1	0.787	12.7419	6.22634000 1.36624000 0.00000000 0.11517000 0.00052290	9.96
4	WET = 21.06 +C4 +1.034 DRY + 0.47 DELTASAT + 0.002 DRY^2 Where C4 = -15.45 when ADD = 0 & C4 = 0 when ADD = 1	0.819	11.7838	5.94282000 1.26644000 0.00000000 0.10656690 0.06399386 0.00048426	9.21
5	WET = 17.92 + C5 + 1.039 DRY - 0.468 DELTASAT + 0.386 VOID - 0.002 DRY^2 Where C5 = -15.39 when ADD = 0 & C5 = 0 when ADD = 1	0.819	11.7994	9.59876580 1.27808180 0.00000000 0.10746172 0.06425589 0.92727935 0.00048677	9.22
6	WET = -131.797 + C6 +1.05 DRY - 0.459 DELTASAT + 44.73 VOID - 0.002 DRY^2 - 3.276 $VOID^2$ Where C6 = -15.276 when ADD = 0 & C6 = 0 when ADD = 1	0.826	11.5770	42.4853570 1.25437340 0.00000000 0.10548358 0.06309444 12.3042378 0.00004778 0.90656211	9.05
7	WET = -133.74 + C7 + C8 +1.12 DRY - 0.422 DELTASAT + 44.84 VOID - 0.003 DRY^2 - 3.34 $VOID^2$ Where C7 = -2.09 when GEO = 1 C7 = 2.97 when GEO = 2 C7 = 3.18 when GEO = 3 C7 = 3.00 when GEO = 4 C7 = 1.14 when GEO = 5 C7 = 1.84 when GEO = 6 C7 = -1.33 when GEO = 7 C7 = -5.48 when GEO = 8 C7 = 0 when GEO = 9 C8 = -15.60 when ADD = 0 C8 = 0 when ADD = 1	0.834	11.4657	42.8693160 3.71852258 3.96042686 4.58324367 3.60952137 3.68069249 3.47666694 3.69376071 6.08788590 0.00000000 1.27542862 0.00000000 0.10821298 0.06798486 12.5545950 0.00049424 0.92229088	8.96

The above concept is illustrated in Table 6.4. The average TSR value is 0.711. Considering the variance in the wet strength and the dry strength data sets, it can be shown that the 95 % confidence range for the TSR value is between 1.6 and - 0.179. The criteria for the TSR values used currently do not consider this variance in the initial values. Using the simple statistical analysis, the concept of reliability can be used to establish a statistically based criterion that should be a more reliable measure of potential for moisture damage.

The calculations in Table 6.4 demonstrate the effect of variability in dry and wet strengths. This is not accounted for in the current methods of TSR calculation and could be a major factor in the uncertainty seen in the field with using the average TSR values.

Table 6.4 Method for determining the variability of the TSR values

Test Parameter	**Dry subset**	**Wet subset**
Average height (mm)	67.4	66.7
Sample diameter (mm)	101.6	101.6
Air voids (%)	7.13, 7.76, 7.88, 7.64, 7.64	7.28, 7.24, 7.64, 7.92
Average Voids (%) / Std Dev (%)	7.60 / 0.33	7.52 / 0.32
Tensile Strengths (kN/m²)	670.9, 748.1, 830.2, 788.1	468.2, 548.2, 581.2, 566.8
Avg. Strength (kN/m²)	**759.3**	**541.1**
Std. deviation of strength (kN/m²)	**67.81**	**50.44**
TSR	**541.1 / 759.3 = 0.713**	
Variance of TSR	$= [\bar{X} / \bar{Y}]^2 [\{(\sigma_x^2/n)/\bar{X}^2\} + \{(\sigma_y^2/n)/Y^2\}]$ $= (0.713)^2 [\{(50.44)^2/4)/(541.1)^2\}$ $+ ((67.81)^2/4)/(759.3)^2\}] = 0.00214$	
Confidence Interval for TSR at 95% probability	**= TSR ± 2 (Variance of TSR)$^{1/2}$** **= 0.713 ± 0.0926**	
95% Confidence Range of the TSR	**0.805 - 0.620**	

6.4.1 Variability Associated With Wisconsin Database

WisDOT database was examined to determine the deviation of TSR test results from their mean value due to the variability of strengths of dry and wet samples. A scatter plot of TSR value versus standard deviation of dry and wet strengths is shown in Figure 6.2. No relationship could be determined between the two variables.

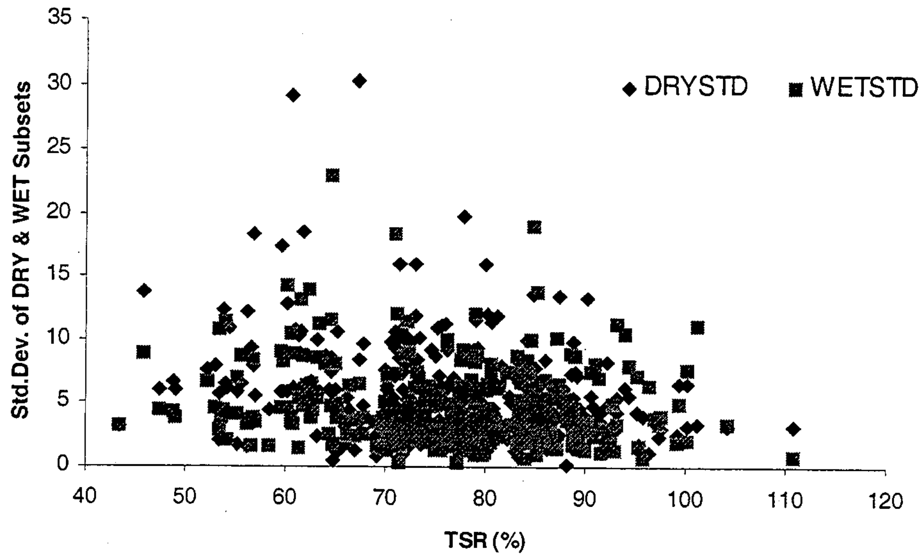


Figure 6.2 Variability of Dry and Wet Strengths of Wisconsin Mix data

6.4.2 Variability of Estimating TSR Value by Statistical Model

Statistical model No. 3 was applied to estimate wet strength of 14 test sections from variables measured by WisDOT. Estimated values of wet strength were then compared with the measured wet strength values of respective highway sections. Figure 6.3 shows that fairly reliable correlation exists between measured and predicted values of wet strength with an R^2 of 61.47%. Considering the inherent variability of TSR test results, use of statistical model provides a reliable tool in estimating stripping potential of a mix by plugging a few variables such as dry strength, and presence or absence of anti-strip additives in the model to determine wet strength and therefore, calculating TSR.

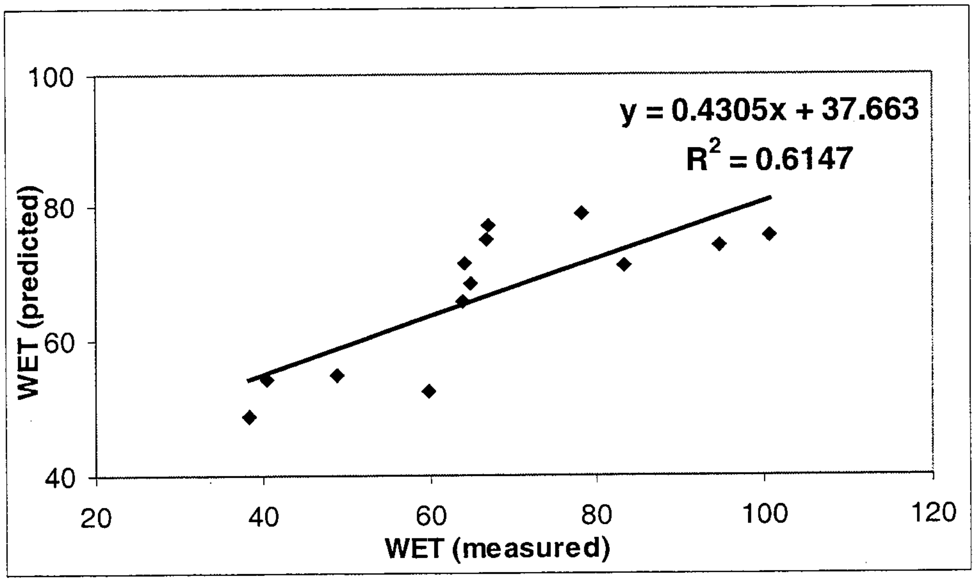


Figure 6.3 Measured Wet Strength versus. Predicted Wet Strength

6.5 Summary

TSR database of Wisconsin was used to arrive at a model that can predict WET strength from DRY strength and a few other variables for estimating TSR value within reasonable limits. A procedure for estimating the variability of the TSR value from the variability associated with its constituent parameters, DRY and WET strength has been developed.

CHAPTER SEVEN
FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

This study was focused on evaluating the practices of controlling moisture damage of asphalt paving mixtures in Wisconsin. The study included several tasks that were designed to offer a comprehensive study of importance of control of moisture damage in Wisconsin. The tasks included an extensive literature review, a survey of current practices of the Mid-Western States to control moisture damage, a statistical evaluation of moisture damage data collected from a large number of Wisconsin projects, a field study and a laboratory study of samples recovered from 14 field sections. The findings were used to recommend specific changes in protocols currently used to identify potential for moisture damage and require the addition of anti-stripping additives.

7.1 Findings

This study included several tasks that covered different activities. The following sections give the summary of findings from each of these tasks.

7.1.1 Literature Review

- There has been a number of theories introduced to explain moisture damage mechanism in asphalt mixtures. In addition, a number of tests have been used to accelerate moisture damage in the laboratory and measure the potential for moisture damage. Each theory has its merits but is difficult to prove that it alone can explain this complex phenomenon. With regard to test methods, saturation and storage at high temperature and freezing and thawing are the most widely used techniques. Both are used in the AASHTO T283 standard procedure.
- The literature review indicates that the AASHTO T283 is considered the most widely used test method despite the fact that in many studies no good correlation could be found between the results of this test and the actual moisture damage in the field.

- Although few modifications have been suggested and a new test (the Environmental Conditioning System, ECS) has been introduced, the AASHTO T 283 remained to be the test of choice. The recently recommended Superpave volumetric design procedure requires conducting this test without modification.
- It is recognized that there are several factors that affect the repeatability of the TSR values. Currently averaging of the dry and wet subsets is used to calculate the average TSR value. It is recognized that this is not the best statistical method and it raises some concerns about the value of the test results. Variation in the measurement of the indirect tensile strength values is the main problem in this test. Variations as high as 40% is cited in some studies.
- It appears that the indirect tensile strength measure is too sensitive to minor changes in voids, moisture conditioning, saturation level, aggregate orientation, and temperature. It is, however, the most practical test available.
- The lack of relationship between the TSR results and the moisture damage in the field has been mentioned in many studies. In few studies a modification of the test procedure to improve its relationship to field performance based on field calibration has been proposed.

7.1.2 Survey of the Mid-Western States

A total of 10 states in the Mid-Western region responded to the survey. Several questions were asked to gather information about the experience of the Mid-Western states with moisture damage. The following points summarizes the findings:

- The majority of the 12 State Highways Agencies indicated that moisture damage is a concern.
- Most of the states (9 out of 10) used a version of the AASHTO T 283 procedure to identify moisture damage. The experience in relating the TSR values to actual moisture damage is not consistent among the states.
- Acceptable TSR values vary among the states. Wisconsin and Minnesota are the two states with the lowest ratio (0.70).

- The effect of moisture damage on pavement performance is not well understood. Raveling, rutting, fatigue cracking, potholes, bleeding/blisters are among the important distress indicators that are related to moisture damage.
- Visual inspection of the field cores is the most commonly adopted method to determine the presence of moisture damage. For mixtures without additives, the general consensus is that effect of moisture damage should appear within the first 5-6 years.
- There is no consensus regarding the mineralogy of the aggregates that are most susceptible to moisture damage. Based on experience, gravel, quartzite, and certain types of limestone are ranked as the most susceptible. In addition, chert, carbonate, dolomite, and glacial till have been mentioned by at least one state as problematic aggregates.

7.1.3 Statistical Analysis of Wisconsin TSR Database

- The database which includes 5 indicator variables, 10 measured variables, and 6 calculated variables, was used to derive a statistical model to determine the factors controlling the TSR. The analysis indicated that the dry strength and the use of additives are the best predictors of the TSR values.
- Using the average wet strength as the response variable, none of the indicator variables which include geological type of the aggregates, the asphalt binder source, the mixture type (HV, MV, LV), and type of additive could improve the predictability of the wet strength.
- None of the measured variables, which included asphalt content, percentage fines, air voids, compaction effort, and final saturation could improve the predictability of the model.

- Among the calculated variables, which include film thickness, mastic volume, coarse to fine ratio, and net saturation, the latter was the only factor that is found to be marginally important.
- The average wet strength is found to be a nonlinear function of the dry strength and it appears that by including a second order function of the dry strength a significant improvement in the model is achieved.
- The TSR average value can be misleading if the variability associated with it is not indicated. The variability within the wet set and the dry set of samples can be used to estimating the TSR variability. A procedure has been proposed to calculate the standard deviation of the TSR values. The standard deviation of the TSR values in the database is relatively high and it raises some questions about the utility of this test.
- The procedure for calculating the TSR standard deviation is best used if a large number of replications are available. There is no optimum number supported by the statisticians but as a general rule 10 replicates are recommended for a true representation of standard deviation. It is difficult to tell how much accuracy is lost because of the use of only 3 replicates.
- There is no indication that any of the 9 different geological types of aggregates has a bad record of moisture damage. Similarly there does not appear to be a source of asphalt that shows poor performance with regard to moisture damage.
- For a large number of sections for which the PDI numbers were identified in 1997 (4-5 years after construction) there is no relationship between the TSR values and the PDI numbers. This raises some questions about the value of this test and the actual performance of the pavement sections.

7.1.4 Findings from the Laboratory Evaluation of Field Samples

The field samples were used to measure the in-situ dry strength and the in-situ strength after conditioning according to the AASHTO T 283. Some of the field samples were also heated and remolded to measure the tensile strength before and after conditioning according the AASHTO T 283 procedure. The in-situ values were used to calculate TSR 1 while the values for the remolded were used to calculate TSR 2. The original mixture design values, reported by the WisDot are represented by TSR3 in the following discussion. The following points summarized at the end of chapter 5 are reproduced for the findings:

- High variation in the TSR1 values was observed. In several of the sections, TSR1 values were higher than the value of 1.0. It appears that the coring process damages the field samples. After conditioning the samples with water at 60 °C, it appears that a partial healing of the damage occurred, which resulted in increasing the TSR value.
- In 9 sections TSR 1 was less than or equal to the TSR 2 (TSR value of remolded sample), which is conceptually logical because remolding restores the HMA sample. On the other hand the results from the other 5 sections are not logical (TSR1 is higher than TSR2). All the five sections have TSR 1 values greater than 1, which identifies the problem with the field samples.
- No relationship was found between TSR 1 and TSR 2. It is speculated that degradation of aggregate might have occurred during the remolding process, causing less value of TSR 2 than the value of TSR 1. Different compaction methods, in field and in lab, might have caused a reduction in strength for remolded samples. Healing of asphalt during the conditioning could have increased the conditioned strength, giving a higher TSR 1 value.
- No strong agreement was found between the remolded TSR 2 and the original TSR3 value. This was expected because the sample geometry, air voids, and materials are not identical. It was expected, however, that the TSR values would rank the materials similarly. Because of the wide confidence intervals calculated for the TSR values, it is very difficult to decide whether the field and samples give the same ranking. For five of the sections the TSR values were similar.
- Average % Air Voids and TSR Values: Due to the damage of field samples the TSR1 (in-situ) values show very poor and illogical relationship with average %

air voids. In case of TSR2 and TSR3, there is a clear trend that TSR value decrease with increasing air voids.

- Geology: Majority of the sections contained a mineralogical composition with Dolomite as the main component. Dolomites impart anti-stripping properties to the HMA. All such sections have generally good values in all of the TSR sets. Only one aggregate mineralogy, Chippewa / St. Croix Old Gravel used in HWY section 64, did not perform well in TSR 1 and TSR 2 tests, with values of 0.44 and 0.41 respectively. However, TSR 3 (DOT) value for the same source is 0.99. It is logical to believe that this source of aggregate is different and is expected to show lower TSR values. It is, however, difficult to explain the results from the mixture design. The statistical analysis indicated that geology of the commonly used aggregates in Wisconsin conditions does not significantly affect the TSR value results.
- Saturation in case of TSR1 and TSR2 does not seem to play an important role. In statistical analysis, it marginally improves the model to predict WET response.
- Average PDI values are not related to either of the TSR sets. It may be because the measurement of he PDIs is done by sampling 1/10th of a mile of a pavement, and the samples collected for he TSR test may not fall in that surveyed section of the road.

7.2 Conclusions

The following points summarize the conclusions that could be drawn from the findings as related to the objectives of the project.

- Based on the analysis of the results from the database and from the 14 sections sampled in this study there appears to be no relation between the between performance of pavements as measured by the PDI numbers and any of the lab TSR values measured during the mixture designs or TSR values for recovered samples.

- The relationship between lab prepared mixes and the field recovered mixes was difficult to establish in this study. Coring and remolding using the gyratory compactor could have confounded the results significantly. Based on the experience shown in this project with the sensitivity of the TSR values to test parameters and mixture preparation, it is believed that such a relationship is very difficult to be found in any project.
- TSR test results are very sensitive to procedure and details of testing. Its value in predicting moisture damage is questionable because of significant variability inherent of the test procedure.
- From statistical analysis, highly reliable model was derived to predict wet strength from dry strength of Wisconsin mixes. The model indicates that the source of aggregate is not a significant factor. It also indicates that existence of additives and net saturation can enhance predictability of the wet strength.
- The currently used TSR procedure is not capable of predicting the moisture damage of the mixes in the field.
- The presence of a moisture damage problem in Wisconsin pavements cannot be determined at this time due to difficulties inherent in the test procedures.
- The extent of moisture damage in Wisconsin pavements cannot be determined at this time due to difficulties inherent in the test procedures.

7.3 Recommendations

1. It is recommended that testing for moisture damage using the AASHTO T-283 procedure be made optional.
2. It is recommended that mixture acceptance should be based on the results of the measured or predicted values of the wet strength.
3. It is recommended that the model developed in this study be used to estimate the wet strength.

4. It is recommended that the values from the field prepared samples should be used for quality acceptance.
5. It is recommended that the TSR values database be continuously updated and the prediction model should be revised annually.

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APPENDIX A TO CHAPTER THREE

QUESTIONAIRE TO STATE HIGHWAY AGENCIES OF MID-WEST

Wisconsin Department of Transportation
University of Wisconsin-Madison
Asphalt Pavement Research Group

Questionnaire to the Mid-Western States on Moisture Damage Test Practices

1. Is moisture damage one of your concerns regarding premature failure of pavements?

   Y ----- N -----

2. Do you use anti-strip agents?

   Y ---- N ----

   If YES, please provide us with the specifications
   If the answer is NO to both Q1 and Q2, please fax this sheet to the number indicated below

3. What is the procedure adopted by your agency to identify the moisture damage in the field?

4. What are the common distresses that you attribute to Moisture Damage ?

5. How early do the moisture damage problems occur in your pavements?

6. List the common types of aggregate mineralogy (eg. Dolomite) used in your asphalt pavement mixes that you think are more prone to moisture damage?

7. What specific tests and the threshold values do you use to detect the tendency of the aggregate for moisture damage?

   AASHTO T 283 ---------- , Criteria ----------
   Others ---------- Criteria -----------

8. How effective are these aggregate tests?

9. What version or modification of AASHTO T 283 and criteria do you specify to identify the moisture damage problems?

10. Do the lab moisture damage results and the field performance correlate well? YES ----- NO -----

11. What would be an ideal test to determine the tendency of a mix for moisture damage?

12. What corrective measures do you recommend if a mix design fails the moisture damage test?

Please Fax the completed survey to
Attn: Prof. Hussain U. Bahia / Dr. Gary Gowda
CEE Dept., UW-Madison
Fax: (608) 262 5199, Phone: 608 265 4481

APPENDIX B TO CHAPTER FOUR

LAB TESTING PROCEDURE FOR THE PILOT PROGRAM

(HWY 23)

APPENDIX B TO CHAPTER FOUR
LAB TESTING PROCEDURE FOR THE PILOT PROGRAM

The lab testing of twelve cores drilled from HWY 23 was accomplished in the following stages:

Step 1: Separation of Surface Course: The twelve field cores were sawed to separate the surface and binder courses. The separated cores were numbered from 1-12. Average thickness from four quarters and average diameter from two halves was determined for each core as shown in Table B-1 below.

Table B-1: Volumetric Data of Highway 23

Core #	Avg. Thickness (mm)	Avg. Diameter (mm)	Dry Mass (g)	Mass in Water (g)	SSD Mass (g)	G_{mb}	G_{mm}	% Voids
1	35.7	144.6	1360.2	788.9	1361.5	2.3755	2.4945	4.7
2	38.5	144.7	1500.5	881.4	1501.6	2.4194	2.4945	3.0
3	37.0	144.7	1428.4	836.8	1429.9	2.4084	2.4945	3.4
4	39.6	144.6	1544.3	906.4	1545.5	2.4164	2.4945	3.1
5	32.3	144.9	1252.2	735.4	1253.1	2.4188	2.4945	3.0
6	33.0	144.7	1279.6	751.7	1280.6	2.4194	2.4945	3.0
7	38.1	144.5	1474.7	864.1	1475.8	2.4108	2.4945	3.3
8	27.3	144.7	1061.7	626.5	1062.6	2.4345	2.4945	2.4
9	32.4	145.0	1257.6	739.3	1258.8	2.4208	2.4945	2.9
10	33.8	144.6	1310.0	773.0	1311.6	2.4322	2.4945	2.4
11	26.5	144.8	1013.4	596.3	1015.2	2.4192	2.4945	3.0
12	31.1	144.3	1205.8	713.5	1208.4	2.4365	2.4945	2.3

Step 2: Bulk specific gravity: G_{mb} of all 12 cores was determined at 25 °C according to AASHTO T 166.

Step 3: Theoretical Maximum Specific Gravity, G_{mm} (core # 10,11,12 –Rice cores): G_{mm} was determined from 3 cores (# 10-12). As per AASHTO T 209-94. The cores were heated to disintegrate, and the material was combined together. The combined material was split into three parts by quartering. G_{mm} of each part was determined, and an average was worked out as **2.4945**.

Step 4: Air Voids: From the above two steps air voids of each core were determined as per AASHTO T 269-94. Average field voids of all the cores was also determined as 3.08 %. As shown in Table 2, arranging air voids in descending order, selection of three subsets of remaining 9 cores was made in such a way that the average air voids of a subset are close to the average of all the cores. The abbreviation of tests is given in the subsequent steps.

Table B-2: Distribution of Air Voids

Core #	**% Voids**	**Avg % Voids**	**Deviation From Av. % Voids**	**Selected For**
1	4.771	3.081	1.690	**IDS**
3	3.453	3.081	0.372	**F**
7	3.355	3.081	0.274	**ICS**
4	3.132	3.081	0.051	**IDS**
5	3.036	3.081	-0.045	**F**
11	3.019	3.081	-0.062	**Rice**
6	3.012	3.081	-0.069	**ICS**
2	3.011	3.081	-0.070	**ICS**
9	2.955	3.081	-0.126	**F**
10	2.496	3.081	-0.585	**Rice**
8	2.404	3.081	-0.677	**IDS**
12	2.327	3.081	-0.754	**Rice**

Step 5: Densification characteristics (# of gyrations): Mixture material from step 3 was heated at 140 °C for 90 minutes, re-mixed, and remolded in Superpave Gyratory Compactor (SGC) to yield a 150 mm diameter sample. G_{mb} of the compacted sample was determined, and fed to a spread sheet along with data from the printout of Superpave Gyratory Compactor (SGC) to determine the number of gyrations needed to remold the cores at field void content i.e., about 3%. The number of gyrations came out to be 70 as shown in Table B-3 and Figure B-1.

Step 6: Subset I for In-situ Dry ITS - IDS (core # 1,4,8): The three cores were placed in water bath at 25 °C for 20 minutes, and then tested for indirect tensile strength. The average dry strength came out to be about 1103.2 kN/m^2.

Step 7: Remolded Dry ITS - RDS: The cores tested from step 6 were visually examined for stripping, and the amount of stripping was estimated as about 5%. The cores were remolded at 70 gyrations in the Superpave Gyratory Compactor to yield one sample. The sample was cooled overnight, sawed into three cores (#1*, 4*, 8*) which were left to dry overnight. G_{mb} of the cores were determined the following day, thereby yielding the average air void content of about 2% (compare it to the target air voids of 3%). Average thickness and diameter of the cores were measured. The cores were tested in the same way as in step 6 to determine remolded dry ITS as about 1034.3 kN/m^2. Visual inspection showed minimal stripping.

Step 8: Subset II for In-situ Conditioned ITS - ICS (core # 2,6,7): The three cores were subjected to vacuum saturation to achieve a level of more than 55%. They were conditioned in water at 60 °C for 24 hours, and then were placed in water at 25 °C for two hours prior to ITS testing. The test yielded ICS of about 1031.3 kN/m^2. 5% stripping was observed.

Step 9: Subset III for In-situ Conditioned ITS with one Freeze-thaw Cycle - F (core # 3,5,9): A departure from the modified Maupin procedure was made in this step with the consent of Wis DOT. The objective was to see the effect of severity of conditioning on ITS by adding one freeze-thaw cycle. The three cores were vacuum saturated to a level of more than 55%, wrapped separately with a plastic film, placed in a plastic bag containing 10 ml. of water, and frozen at about 3 °C for 16 hours. Soon after removal from the freezer, the cores were unwrapped and placed in water bath at 60 °C for 24 hours. Afterward the cores were put in water at 25 °C for two hours before testing for ITS. The conditioned ITS was determined as about 1036.9 kN/m^2. Observed stripping was about 5%.

Step 10: Remolded Conditioned ITS - RCS: Remolded sample from Rice cores from step 5 was remolded at 70 gyrations. The remolded sample was then sawed into three cores Rice-top, Rice-mid, and Rice-bot. The cores were conditioned, and tested in the same way as those from step 8. The average remolded conditioned ITS was determined as about 1205.2 kN/m^2. Very little stripping was observed.

Step 11: Tensile strength ratio TSR: Various TSR values were determined as listed in Table B-4.

Discussion on the Test Procedure

- **Saturation level:** Achieving the desired saturation level in the cores (>55%) has been cumbersome. In some cases it required more than six trials consuming more than two hours for one sample. In the author's opinion, this can be attributed to very low air voids of the in-situ, and the remolded samples.

- **Remolding:** It has been observed that remolding SGC at the number of gyrations (70) as determined from the densification curve (step 5) over-compacted the sample yielding less than 2 % air voids as compared to average in-situ void content of 3 %. Further the middle sawed core from the remolded sample yielded the lowest air voids (less than 1.5%).

Table B-3: Densification Characteristics

Gyrations	Ht., mm	G_{mb}(est.)	G_{mb}(corr)	$\%G_{mm}$
1	98.4	2.106	2.143	85.9
5	94.5	2.192	2.232	89.5
8	93.1	2.225	2.265	90.8
10	92.4	2.242	2.282	91.5
20	90.4	2.292	2.333	93.5
30	89.3	2.320	2.362	94.7
40	88.6	2.338	2.380	95.4
50	88.1	2.352	2.394	96.0
60	87.7	2.362	2.405	96.4
70	87.4	2.371	2.413	96.7
80	87.1	2.379	2.421	97.1
90	86.9	2.384	2.427	97.3
95	86.9	2.384	2.427	97.3
100	86.8	2.387	2.430	97.4
110	86.6	2.392	2.435	97.6
120	86.5	2.395	2.438	97.7
130	86.4	2.398	2.441	97.8
140	86.2	2.404	2.446	98.1
150	86.1	2.406	2.449	98.2

Specimen mass = 3659.5 g

Gmm (measured) = 2.4945

Gmb (measured) = 2.4493

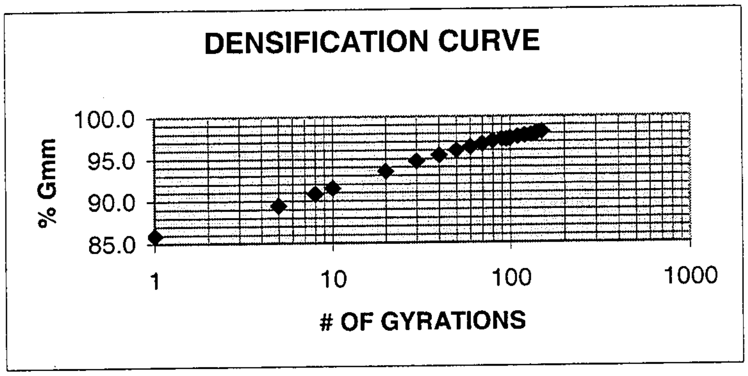


Figure B-1: Densification Curve

Discussion on the Test Results

- Perusing Table B-4, it may be seen that the average IDS is greater than the RDS. It might happen because of age hardening of the mix. The increased stiffness imparts strength to the mixture.
- In case of RDS, and ICS with or without freeze-thaw cycle, the average ITS is almost the same and very close to average IDS. It seems that conditioning or remolding of the cores did not cause a significant moisture damage to the samples.
- Remolded conditioned ITS yielded the highest average ITS of 1205.2 KN/m^2. It implies that the conditioning in this case actually increased the strength. Coplantz et al. (1988) and Lottman (1982) have reported similar results when vacuum saturation resulted in increase of strength.
- As per the criteria of WisDOT the mixture is not susceptible of stripping because each TSR is greater than 70%.

Table B-4: Consolidated Test Results of Pilot Program - HW 23

Sample	Avg. Dia,D	Avg. Ht, T	Air Voids	Peak Load, P	ITS	Avg. ITS	STDEV
#	mm	mm	%	kN	KN/m^2	KN/m^2	
IN-SITU DRY ITS (IDS)							
1	144.6	35.7	4.771	8.896	1097.4	-	-
4	144.6	39.6	3.132	10.23	1136.6	1103.1	31.1
8	144.7	27.3	2.404	6.670	1075.2	-	-
REMOLDED DRY ITS (RDS): Samples 1,4,8 from IDS were remolded, cut into 3 cores, & tested.							
1* top	149.9	27.8	2.06	6.230	950.0	-	-
4* middle	150.0	24.6	1.48	6.340	1094.1	1036.6	371.2
8* bottom	150.0	28.3	2.43	7.120	1065.8	-	-
IN-SITU CONDITIONED ITS (ICS): Samples conditioned in water without freezing cycle.							
2	144.7	38.5	3.011	9.560	1092.2	-	-
6	144.7	33.0	3.012	4.450	1127.8	1031.3	76.3
7	144.5	38.1	3.355	7.560	874.0	-	-
IN-SITU CONDITIONED ITS WITH FREEZING CYCLE (F)							
3	144.7	37.0	3.453	8.896	1056.9	-	-
5	144.9	32.3	3.036	8.674	1179.7	1036.9	153.8
9	145.0	32.4	2.955	6.450	874.0	-	-
REMOLDED CONDITIONED ITS (RCS): Rice samples 10,11,12 remolded, cut into 3 cores, conditioned in water and tested.							
Rice-top	5.902	1.102	2.8	7.117	1079.8	-	-
Rice-mid	5.902	1.022	1	could not be saturated		1205.2	177.3
Rice-bot	5.902	1.034	1.94	8.451	1330.6	-	-

TSR	% TSR	Variance*	C.I.**
IN-SITU CONDITIONED ITS/IN-SITU DRY ITS	93.50%	0.00182	0.9350 ± 0.085
IN-SITU CONDITIONED ITS WITH FREEZE CYCLE/IN-SITU DRY ITS	93.99%	0.00672	0.9399 ± 0.164
REMOLDED CONDITIONED ITS/REMOLDED DRY ITS	116.27%	0.05924	1.1627 ± 0.487

Variance of TSR = {(Average Conditioned ITS/ Average Dry ITS)^2}[{(STDEV of DryITS^2/No. of Tests)/(Dry ITS^2)
+ {(STDEV of Conditioned ITS^2/No. of Tests)/(Conditioned ITS^2)}]

**Confidence Interval = TSR ± 2*(Variance of TSR)^0.5

APPENDIX C TO CHAPTER FOUR

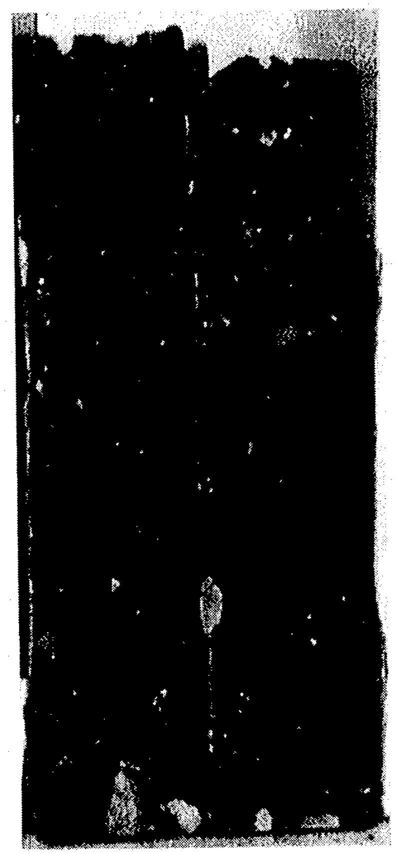
ASSESSSMENT OF AMOUNT OF STRIPPING

APPENDIX C TO CHAPTER FOUR
ASSESSSMENT OF AMOUNT OF STRIPPING

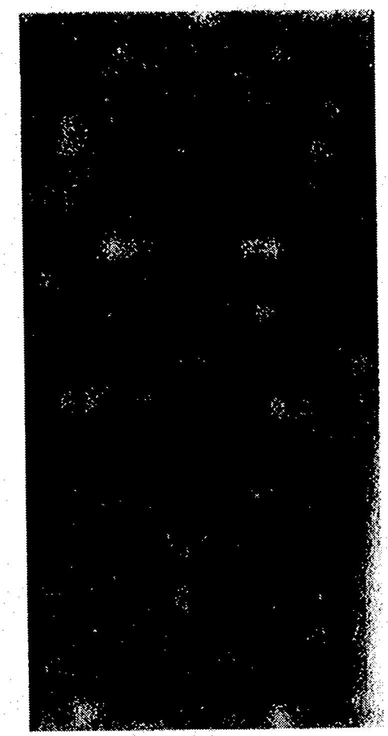
Amount of stripping in percentage was noted visually according to the procedure given in SHRP Protocol P05 for SHRP Designation AC05. The cores from a few HWY sections were also either video-imaged or shot by digital camera to determine if the stripping is better visualized by this technique and to document the stripping conditions. It was found that the technique does not capture stripping. Therefore, the procedure was not further pursued. Table C-1 shows the amount of stripping as determined by visual examination of the samples. Typical images of some cores are also shown.

HWY 78

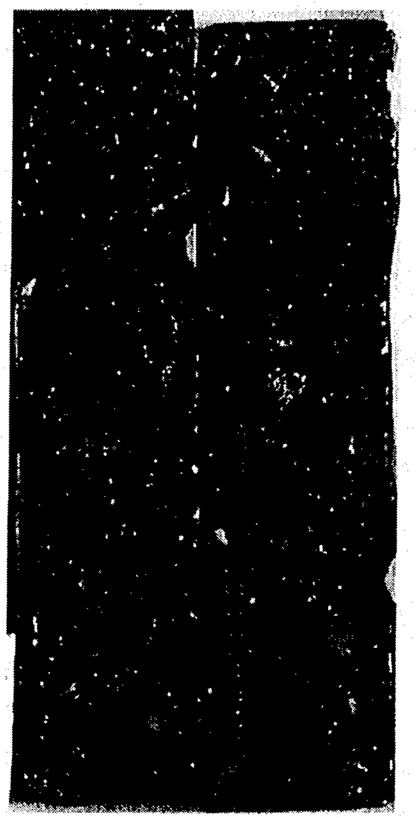
ICS 4



ICS 10



ICS 11



RCS 3-T

RCS 2-B

RCS 2-T

RDS 2-T

RDS 2-B

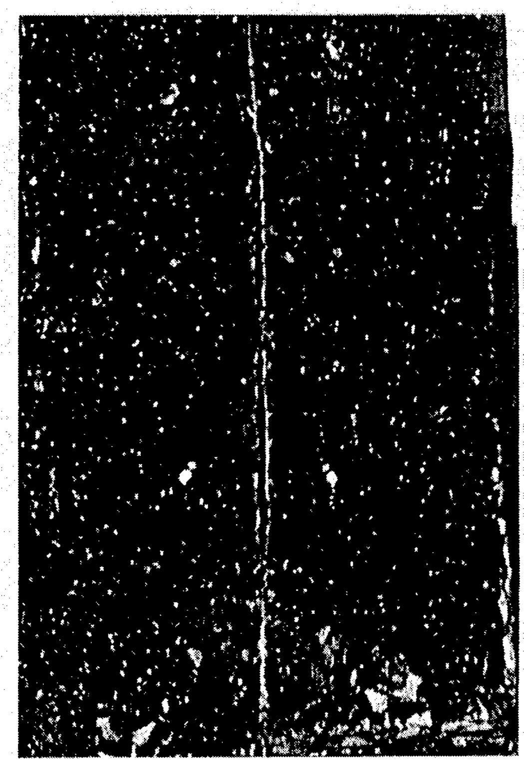
RDS 3-B

HWY78

HWY 62

ICS # 1

ICS # 5

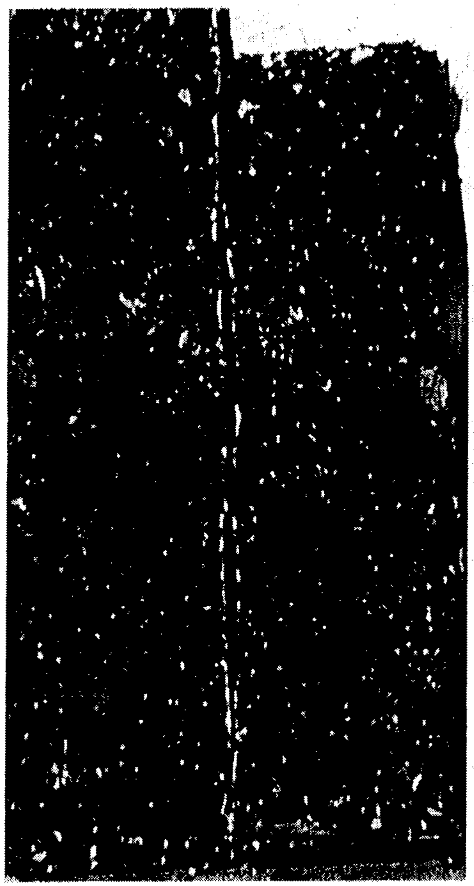


ICS # 7

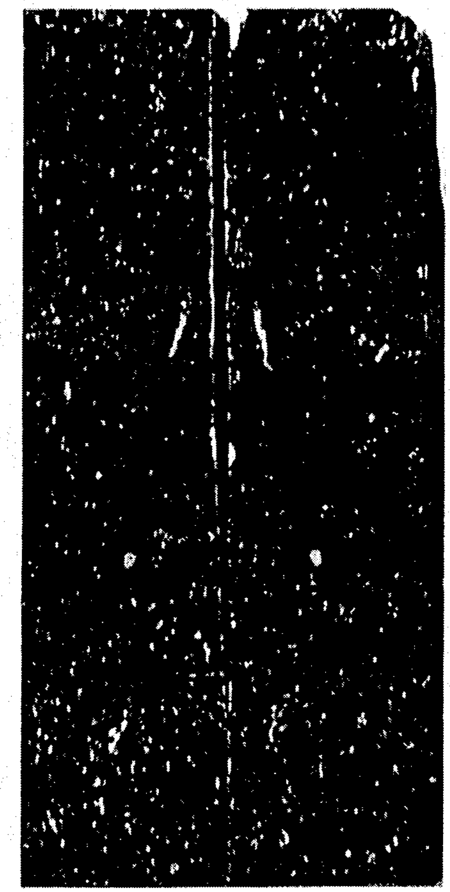
IDS # 1

IDS # 5

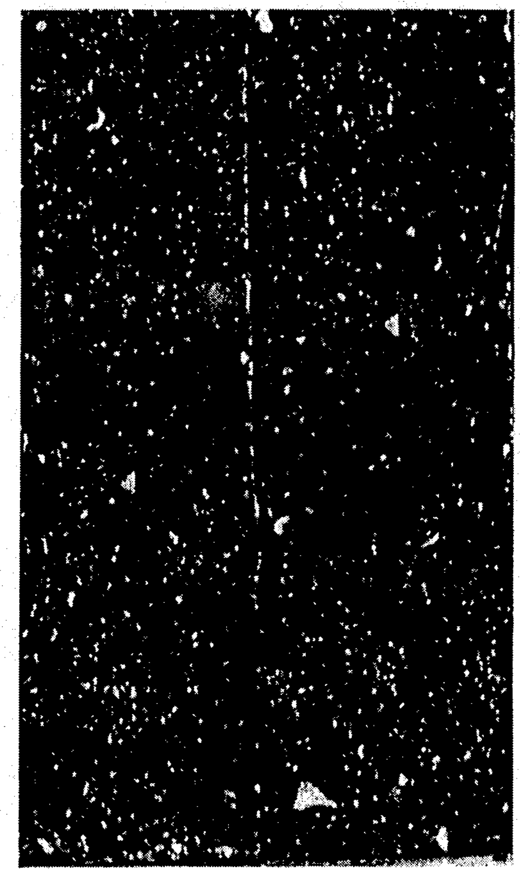
IDS # 7



RCS R1-B



RCS R2-B



RCS R3-B

HWY 62

Table C-1: Stripping of Cores as Determined by Visual Examination

No. (1)	HW Section (2)	IDS Cores (3)				ICS Cores (4)				RDS Samples (5)				RCS Samples (6)			
		#	Stripping (%)		Broken Stones (%)	#	Stripping (%)		Broken Stones (%)	#	Stripping (%)		Broken Stones (%)	#	Stripping (%)		Broken Stones (%)
			C. A.*	F. A.*			C. A.*	F. A.*			C. A.*	F. A.*			C. A.*	F. A.*	
1	78	5	10	-	2	4	10	5	2	R1-B	20	5	-	R2-B	5	-	1
		9	10	5	1	10	10	5	2	R1-T	30	5	1	R2-T	10	5	2
		12	15	5	1	11	15	5	3	R3-T	20	5	-	R3-B	10	5	5
2	64	4	10	-	2	6	40	10	2	R2-B	5	-	1	R1-B	5	40	1
		5	5	-	2	7	40	10	2	R2-T	10	-	2	R1-T	5	40	1
		8	10	-	5	9	40	10	-	R3-T	15	-	3	R3-B	5	40	1
3	14	4	60	-	20	6	20	-	5	R1-T	30	-	2	R1-B	15	-	5
		5	50	-	10	10	15	-	3	R2-B	30	-	2	R2-T	35	-	5
		7	50	-	10	11	15	-	3	R3-T	35	-	3	R3-B	10	-	5
4	35	4	10	5	1	6	5	-	1	R1-B	5	5	2	R1-T	10	5	1
		9	10	-	-	10	10	-	1	R2-T	5	5	3	R2-B	5	-	2
		13	10	-	1	11	10	-	3	R4-B	5	5	2	R4-T	5	5	2
5	10 - Mondovi	1	20	5	3	3	10	5	1	R1-T	10	5	1	R1-B	5	50	1
		6	15	5	2	4	5	10	3	R3-B	5	5	2	R2-B	10	30	2
		8	10	-	3	5	5	20	1	R3-T	5	5	1	R2-T	5	50	2
6	51- Mathy	4	5	5	-	5	10	5	-	R1-T	5	-	-	R1-B	5	5	1
		8	15	10	-	6	5	10	-	R2-B	5	-	-	R3-B	5	5	-
		9	10	10	2	7	5	15	2	R2-T	5	-	2	R3-T	5	10	-
7	51- P&D	1	10	1	2	2	5	5	1	R1-T	10	5	1	R1-B	5	20	-
		3	5	5	1	4	5	5	-	R2-B	5	5	2	R2-T	5	20	1
		6	5	5	1	5	5	15	2	R3-T	5	5	1	R3-B	5	15	1
8	100	1	50	-	10	2	25	-	2	R1-T	30	-	3	R1-B	60	-	10
		5	506	-	5	4	20	-	5	R2-B	25	-	3	R3-T	50	-	5
		7	0	-	5	6	15	-	2	R2-T	20	-	5	R3-B	50	-	5
9	116	4	10	-	-	6	10	-	-	R1-T	10	-	-	R1-B	20	-	-
		5	10	-	-	7	10	-	-	R3-B	5	-	-	R2-B	15	-	-
		8	10	-	-	9	10	-	-	R3-T	10	-	-	R2-T	5	-	1

10	10 - Clark	8	5	-	-	7	15	5	1	R1-B	201	5	1	RI-T	302	10	-
		10	5	-	-	9	25	5	2	R3-B	0	5	-	R2-B	030	10	-
		11	5	-	-	12	15	5	-	R3-T	10	5	2	R2-T		10	1
11	12 - Harding Ave	7	5	5	-	6	5	5	1	R1-T	5	5	2	R1-B			
		10	5	5	-	8	5	5	-	R2-B	5	10	2	R2-T			
		12	5	5	-	11	5	5	1	R3-B	5	5	2	R3-T			
12	62	6	30	10	-	1	30	10	1	R1-B				R1-T	303	-	2
		8	30	10	-	5	30	10	1	R2-B				R2-T	0	2	-
		9	30	10	-	7	30	10	2	R3-T				R3-B	35	2	-
13	29	1	5	-	-	2	10	5	-	R1-T	5	2	1	R1-B	2	10	1
		3	5	-	-	6	-	-	-	R2-T	5	2	-	R2-B	2	10	2
		5	5	-	-	7	10	5	2	R3-T	10	2	2	R3-B	2	10	1
14	30	1	50	-	2	4	20	5	5	R1-B	30	-	2	R2-T	30	-	3
		2	40	-	3	5	20	10	2	R1-T	40	-	1	R3-B	20	-	2
		3	30	-	1	6	30	5	1	R2-B	25	5	1	R3-T	40	-	1

C.A. – Coarse Aggregate
F.A. – Fine Aggregate

APPENDIX D TO CHAPTER FIVE

DISTRESS ANALYSIS OF TEST SECTIONS

Table D-1: Distresses and TSR of 14 Test Sections

HW No.	Section	Block/ Allegator	Block Crack 25 - 49 %	Block Crack 50-74 %	Transverse Crack 1-5 STA	Transverse Crack 6-10 STA	Transverse Crack 11⁺ STA	Longitudinal Crack 1-100 STA	Longitudinal Crack 101-200 STA	Longitudinal Crack 201-300 STA	Surface ravelling	Rut	Seal Coated	Crack Fill	PDI	PDI_{av}	TSR 1 in-situ	TSR 2 remolded	TSR 3 DOT
78	19 E Int.	0													0				
	Raceck Rd. Int.	0			1			1							13				
	Sec. 35 & 26	0			1			1							13				
	Sec. 26 & 25	0			1			1							13	9.8	0.83	0.98	0.74
64	USH 63	1		1			1		1					1	58				
	CTH D Int.	1		1			1		1					1	58				
	280th St. Int.	1	1				1		1					1	53				
	290th St. Int.	1	1				1		1					1	53				
	STH 128 S Int.	1	1				1		1					1	53				
	310th St. Int.	1	1				1		1		1			1	55	55	0.44	0.43	0.99
35	Craw/Vern County Line	0			1			1						2	13				
	Ghelf Rd.	0			1			1						2	13				
	Battle Hollow Rd.	0													0				
	Mcmichael Rd. Int.	0			1			1						2	13				
	Twn of Genova	0			1			1						2	13				
	Heck's Pt. Int.	0				1		1						2	23				
	Mundsack Rd. Int.	0			1			1						2	13				
	Edgewater Lane Int.	0			1				1					2	19	13	0.87	0.88	0.61
10-M	Sec. Line	0				1		1							23				
	STH 37 Int.	0				1		1							23				
	Allemann Rd.	0				1		1							23				
	Page Rd. Int.	0				1		1							23				
	CTH WW Int.	0				1		1							23				
	Buff. Trem. Co Line	0					1		1						38				
	CTH FF Int.	0					1		1						38				
	Hutchins Rd. Int.	0					1	1							33	28	0.67	0.76	0.65
51(P&D)	Ontario St.	0			1				1					2	13	13	0.74	0.98	0.73
51(Mathy)	Sec Line	0			1								1	2	7				
	STH 16	0			1									2	7				
	CTH P	0			1			1						2	13	9	0.67	0.93	0.94
100	I - 94 STR	0			1			1						2	13	13	0.82	1.17	0.85
116	CTH E	0			1			1				1		2	27	27	1.31	0.90	0.72
10- Clark	Bachelors Ave	0			1				1					2	13				
	CTH B Int	0			1				1					2	13				
	Columbia Rd. Int.	0			1				1					2	13	13	1.41	1.01	0.95
62	Norwich Ave	0			1				1					2	13	13	1.68	0.85	0.91
29	STH 128 N	0					1	1						2	33				
	CTH T	0					1	1						2	33				
	310th St.	0					1	1						2	33				
	320th St.	0					1	1						2	33				
	STCR/ DUNN Co. Line	0			1				1					2	38				
	Sec. 30 & 29	0				3			1					2	52				
	CTH Q Int.	0			1				1					2	38	37	1.64	0.70	0.85
14	Data Not Trace-able																		
30	Data Not Trace-able																		
12 - Harding Ave.	JPCP (DOT Record)																		

APPENDIX E TO CHAPTER SIX

ANALYSIS OF VARIANCE OF PREDICTIVE MODELS FOR WET STRENGTH

APPENDIX E TO CHAPTER SIX

ANALYSIS OF VARIANCE OF PREDICTIVE MODELS FOR WET STRENGTH

Model 1 : Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	50409.4352	1	50409.4352	165.93	0.0001
Error	95695.9086	315	303.7965		
TOTAL (Corrected)	146105.3438	316	$R^2 = 0.345021$		

REDUCED MODEL 1 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	50409.4352	1	50409.4352	165.93	0.0001
			$R^2 = 0.345021$		

Model 2: Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	112613.844	2	13382.9342	527.91	0.0001
Error	33491.5	314	106.661		0.0001
TOTAL (Corrected)	146105.344	316	$R^2 = 0.770772$		

REDUCED MODEL 2 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	133382.9342	1	13382.9342	125.47	0.0001
DRY	62204.4088	1	62204.4088	583.20	0.0001
			$R^2 = 0.770772$		

Model 3: Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	115055.035	3	38351.678	386.6	0.0001
Error	31050.309	313	99.202		
TOTAL (Corrected)	146105.344	316	$R^2 = 0.78748$		

REDUCED MODEL 3 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	13753.1390	1	13753.1390	138.64	0.0001
DRY	8384.1625	1	8384.1625	84.52	0.0001
DRY*DRY	2441.1912	1	2441.1912	24.61	0.0001
			$R^2 = 0.78748$		

Model 4: Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	119634.057	4	29908.514	352.51	0.0001
Error	26471.287	312	84.844		
TOTAL (Corrected)	146105.344	316	$R^2 = 0.818821$		

REDUCED MODEL 4 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	12632.6129	1	12632.6129	148.89	0.0001
DRY	7985.0786	1	7985.0786	94.11	0.0001
DELTASAT	4579.0216	1	4579.0216	53.97	0.0001
DRY*DRY	2112.3195	1	2112.3195	24.90	0.0001
			$R^2 = 0.818821$		

Model 5 : Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	119648.812	5	23929.762	281.30	0.0001
Error	26456.531	311	85.069		
TOTAL (Corrected)	146105.344	316	$R^2 = 0.818922$		

REDUCED MODEL 5 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	12330.0435	1	12330.0435	144.94	0.0001
DRY	7954.2952	1	7954.2952	93.5	0.0001
DELTASAT	4515.4316	1	4515.4316	53.08	0.0001
VOID	14.7556	1	14.7556	0.17	0.6773
DRY*DRY	2127.0350	1	2127.0350	25	0.0001
			$R^2 = 0.818922$		

Model 6 : Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	120718.405	6	20119.734	245.68	0.0001
Error	25386.939	310	81.893		
TOTAL (Corrected)	146105.344	316	$R^2 = 0.826242$		

REDUCED MODEL 6 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
ADD	12145.5967	1	12145.5967	148.31	0.0001
DRY	8121.9940	1	8121.9940	99.18	0.0001
DELTASAT	4336.3131	1	4336.3131	52.95	0.0001
VOID	1082.3526	1	1082.3526	13.22	0.0003
DRY*DRY	2209.5400	1	2209.5400	26.98	0.0001
VOID*VOID	1069.5922	1	1069.5922	13.06	0.0004

Model 7 : Analysis of Variance					
Source of variation	Sum of Squares	d.f.	Mean square	F Value	Sig. level
MAIN EFFECTS					
Model	121847.010	14	8703.358	108.35	0.0001
Error	24258.333	302	80.326		
TOTAL (Corrected)	146105.344	316	$R^2 = 0.833967$		

REDUCED MODEL 7 : FOR PREDICTING WET STRENGTH					
MAIN EFFECTS					
GEO	1128.6058	8	141.0757	1.76	0.0852
ADD	12021.4871	1	12021.4871	149.66	0.0001
DRY	8529.8114	1	8529.8114	106.19	0.0001
DELTASAT	3095.6306	1	3095.6306	38.54	0.0001
VOID	1024.5762	1	1024.5762	12.76	0.0004
DRY*DRY	2633.9017	1	2633.9017	32.79	0.0001
VOID*VOID	1052.2986	1	1052.2986	13.10	0.0003