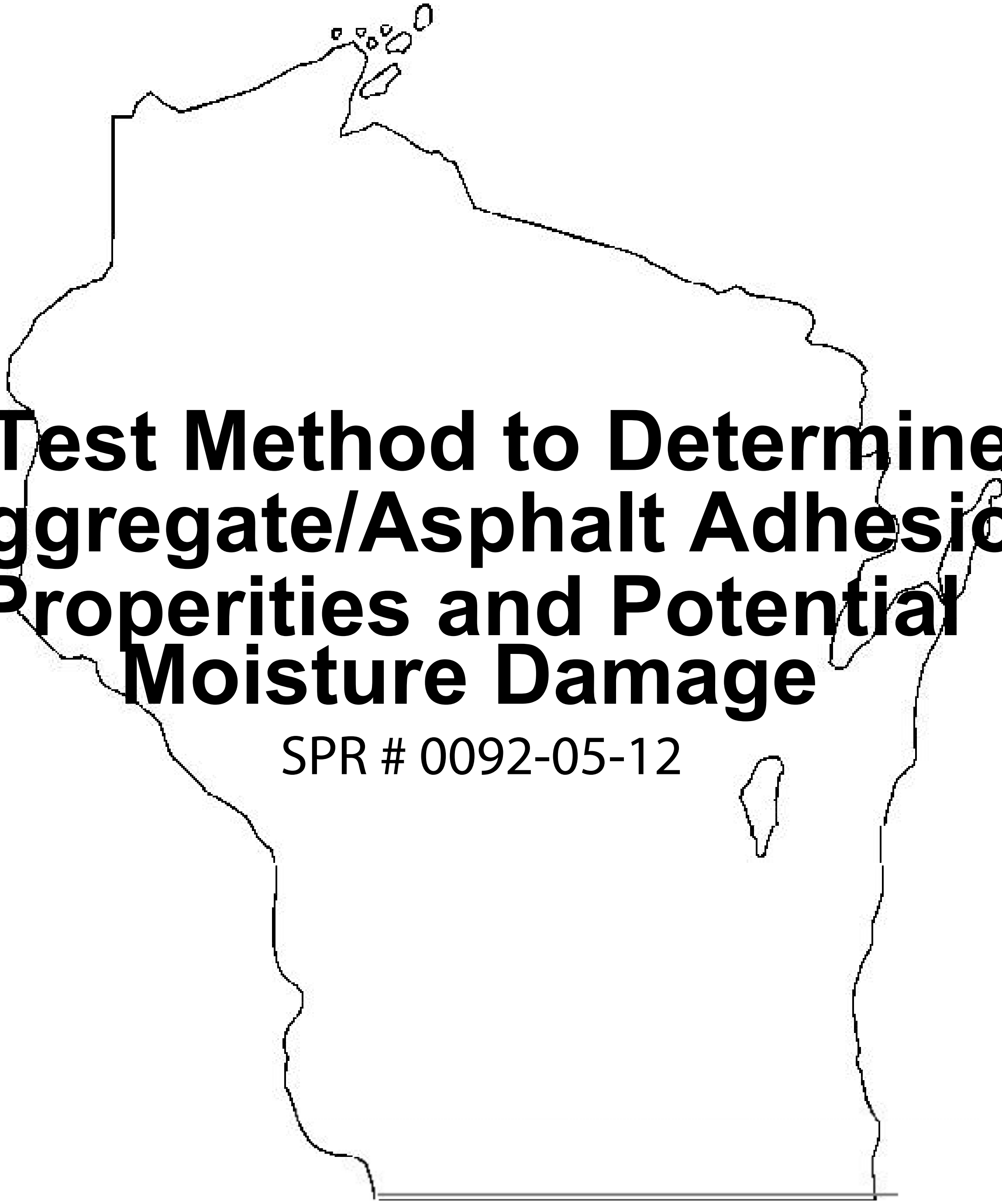
Wisconsin Highway Research Program



Test Method to Determine Aggregate/Asphalt Adhesion Properities and Potential Moisture Damage

SPR # 0092-05-12

Hussain Bahia, Andrew Hanz
University of Wisconsin-Madison
Dr. Kunnawee Kanitpong
Asian Institute of Technology
Dr. Haifang Wen
Bloom Consultants

May 2007

WHRP 07-02

DISCLAIMER

This research was funded through the Wisconsin Highway Research Program by the Wisconsin Department of Transportation and the Federal Highway Administration under Project # 0092-05-12. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Wisconsin Department of Transportation or the Federal Highway Administration at the time of publication.

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof. This report does not constitute a standard, specification or regulation.

The United States Government does not endorse products or manufacturers. Trade and manufacturers' names appear in this report only because they are considered essential to the object of the document.

Technical Report Documentation Page

1. Report No. WHRP 07-02	2. Government Accession No	3. Recipient's Catalog No
4. Title and Subtitle Test Method to Determine Aggregate/Asphalt Adhesion Properties and Potential Moisture Damage		5. Report Date May 2007 6. Performing Organization Code Univ. of Wisconsin - Madison
7. Authors A. Hanz, H.U. Bahia, K. Kanitpong, and H. Wen		8. Performing Organization Report No. WHRP 07-02
9. Performing Organization Name and Address University of Wisconsin-Madison Department of Civil and Environmental Engineering, Rm 2205 1415 Engineering Dr., Madison, WI 53706		10. Work Unit No. (TRAIS) 11. Contract or Grant No. WisDOT SPR# 0092-05-12
12. Sponsoring Agency Name and Address Wisconsin Department of Transportation Division of Business Services Research Coordination Section 4802 Sheboygan Ave. Rm 801 Madison, WI 53707		13. Type of Report and Period Covered Final Report, 2004-2007 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The aim of this research project was to improve current practice by investigating different test methods to quantify moisture damage in an effort to serve the short and long term needs of the agency and industry. In the short term, more efficient test methods were investigated for use as a screening test to supplement current testing procedures. The Stripping Test was chosen as a candidate to serve this need. This test measures the mass loss due to moisture conditioning of a HMA loose mix. The test was evaluated in terms of serving as a screening test to waive mechanical testing requirements for mixes that display resistance to moisture damage. To serve the long term needs of the agency the use of fracture energy, a fundamental property of the asphalt mixture, was investigated as a first step in identifying a parameter that relates to pavement performance. The research involved selection and testing of common HMA mixes used in Wisconsin using the stripping test to measure percent mass loss and the indirect tension test to measure tensile strength and fracture energy. Test results were evaluated on an individual basis in their ability to reliably identify moisture damage due to sample conditioning and the effects of the addition of an anti-stripping agent or change in aggregate type or gradation. The results of the mechanical testing were compared to the non-mechanical test results to evaluate the possibility of incorporating a non mechanical screening test into current WisDOT asphalt mixture testing specifications. Furthermore, the statistical method of analysis of variance was performed on the mechanical testing results to identify significant effects. The quality of the mechanical tests was then determined by evaluating if the significant effects were consistent between the mechanical testing parameters and with common engineering knowledge. The results of this research concluded that the Stripping Test is not adequate for use by WisDOT due to high variability within testing results for a certain mix. The fracture energy test gave promising results however, the high variability and complex test protocol prevent recommendation for the test to be implemented at this time. Recommendations from this research include improvement of the fracture energy test procedure to reduce variability, and further investigation into the physical causes of moisture damage. Specifically asphalt/aggregate adhesion, the effect of the mastic (fine aggregate and binder), and physiochemical interaction between the asphalt binder and aggregate must be understood.

17. Key Words Moisture Damage, TSR, adhesion, cohesion, fracture energy		18. Distribution Statement No restriction. This document is available to the public through the National Technical Information Service 5285 Port Royal Road Springfield VA 22161	
19. Security Classif.(of this report) Unclassified	19. Security Classif. (of this page) Unclassified	20. No. of Pages 107	21. Price

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

Acknowledgements

The authors would like to take this opportunity to thank Ms. Judie Ryan from the Wisconsin Department of Transportation and the rest of the Wisconsin Highway Research Program Flexible Technical Oversight Committee for their help in providing data and input to the project. The authors would also like to thank Mr. John Montgomery, Mr. Erv Dukatz, Mr. Randall Brockington, and Ms. Signe Reichelt for providing mix designs and raw materials for this project.

Executive Summary

Overview

Current Wisconsin Department of Transportation specifications require that the potential for moisture damage of all asphalt mixtures be quantified using the tensile strength ratio as specified by ASTM D4867. Past research has concluded that the use of this procedure is uneconomical due to the variability of the test results and lack of correlation to pavement performance. Furthermore, requirement of tensile strength testing for mixtures from aggregate sources that consistently far exceed the requirements in the specification is an inefficient use of time and resources. The aim of this research project was to improve current practice by investigating different test methods to quantify moisture damage in an effort to serve the short and long term needs of the agency and industry. In the short term, more efficient test methods were investigated for use as a screening test to supplement current testing procedures. To serve the long term needs of the agency the use of fracture energy, a fundamental property of the asphalt mixture, was investigated as a first step in identifying a parameter that relates to pavement performance.

Background

The Wisconsin Department of Transportation adopted the use of the Tensile Strength Ratio to quantify the effects of moisture damage on asphalt mix design in the early 1990's. In the initial stages of the implementation of the TSR specification, 80%-90% of mixes were unable to achieve the required TSR without the use of anti-stripping agent. The sudden increase in the use of anti-stripping agent in mixes that had been previously used throughout Wisconsin raised flags within both industry and the agency. Concerns were voiced questioning the validity and repeatability of the TSR test and the value of the use of anti-stripping additive to correct a problem that may not exist at all or could be remedied through mix design.

WisDOT recognized that these concerns could not be addressed by the agency alone. Therefore, partnerships between the agency, industry, and academia were formed to define, scope, and oversee research projects evaluating the TSR testing procedure and how it relates to field performance. This collaborative effort resulted in the funding of two research projects. The results of these two projects concluded that current practices for quantifying moisture susceptibility in the laboratory have no relationship to what is happening in the field. Furthermore, the TSR test currently being used poses a significant commitment in terms of time and investment for contractors in the mix design process. These findings served as the motivation for the current research project, an investigation of potential test methods to better relate results to pavement performance and/or reduce the time commitment required for meeting moisture damage testing specifications.

Methodology

A literature review was conducted to investigate potential surrogate or replacement mixture test methods to assess moisture susceptibility of asphalt mixtures. The review involved both the investigation of non-mechanical test methods to be used as an efficient surrogate to current testing procedures and different mechanical testing parameters that reflect more fundamental properties of the asphalt mixture. Based on the results of the literature review and consultation with WisDOT, it was decided that the mechanical testing parameter of fracture energy and the non-mechanical parameter of Percent Mass Loss would both be compared to tensile strength testing results. The measurement of the mechanical parameters of tensile strength and fracture energy were achieved through indirect tensile strength testing of compacted asphalt mixtures using a modified ASTM D4867 procedure. Gyratory samples were cut into two inch slices and instrumented to record both stress and strain during testing, allowing for the measurement of both tensile strength and fracture energy using one test.

The non-mechanical testing parameter was measured by the use of the Stripping Test, a modified version of a test developed by the Quebec Department of Transportation to measure stripping in loose mixtures induced by moisture. The test quantifies stripping using a mass loss calculation by comparing unconditioned sample weight to sample weight after mechanical agitation in a water bath of elevated temperature. The mass loss represents the stripping of the asphalt from coarse aggregate and a loss of fine aggregate/asphalt adhesion.

The research had two overall objectives. The first objective was evaluation of the ability of the Stripping Test to accurately identify moisture susceptible mixtures. This investigation was meant to serve the short term goal of the agency of reduced time and resource expenditure in moisture damage testing by potentially providing an efficient screening test to waive mechanical testing results for mixes found to be consistently resistant to moisture damage. The second objective involved initial evaluation of the fracture energy parameter to replace tensile strength as the mechanical test parameter used to quantify the effect of moisture on asphalt mixtures. Fracture energy is a fundamental property of asphalt mixtures, potentially providing a link between pavement performance and mechanical testing results in the lab. This study evaluated the ability of the fracture energy parameter to reliably assess moisture damage in asphalt mixtures and the potential for future work in the investigating the relationship between fracture energy and pavement performance.

Mixes ranging from moisture resistant to moisture susceptible were selected to allow for evaluation and analysis of all three test methods at a wide range of materials and sensitivities. Mixes were selected using historical TSR values in the Wisconsin Mix Design database. The results of this analysis provided five mix designs, one mix design from a source that uses anti-strip agent in virtually all its mixes and four mix designs with TSR values ranging from 0.70-0.85. The mix designs included granite, gravel, and limestone aggregate types. To further aide the evaluation of the test methods mixture design components were varied to evaluate their effects on mix moisture sensitivity and the relative response of the results of each test method to these changes. The research team worked closely with WisDOT to identify aggregate gradation and the use of anti-stripping additive as the two mix components to vary. To study the effects of differing aggregate gradation fine and coarse aggregate blends were used. Furthermore, evaluation of the ability of the testing methods to identify the use of anti-stripping additive was accomplished by preparing all mixes using asphalt binders with and without liquid anti-stripping additive. The worst performing mix was also prepared using hydrated lime and polymer (SBS) modification.

Mechanical and non-mechanical testing results were then evaluated individually in terms of their ability to reliably identify moisture damage due to sample conditioning and the effects of the addition of an anti-stripping agent or change in aggregate type or gradation. The results of the mechanical testing were compared to the non-mechanical test results to evaluate the possibility of incorporating a non mechanical screening test into current WisDOT asphalt mixture testing specifications. Furthermore, the statistical method of analysis of variance was performed on the mechanical testing results to identify significant effects. The quality of the mechanical tests was then determined by evaluating if the significant effects were consistent between the mechanical testing parameters and with common engineering knowledge.

The effects of specific binder and mixture properties on mechanical testing results was also investigated. Specifically, the binder properties of cohesion, and the SuperPave rutting and fatigue parameters and the mixture properties of air voids, percent fine aggregate, and percent natural sand in the fine aggregate blend were investigated. Regression analysis was used to identify the relative contributions of the binder and mixture properties and the significance of their effects on mixture testing.

The results of materials testing and data analysis were incorporated into an economic analysis to assess the implications and potential benefits of incorporating a non-mechanical screening test or fracture energy testing into current practices.

Findings

Based on the analysis of results collected in this study, the following is a summary of the findings that can be stated:

1. The stripping test on loose mixtures was able to detect the potential for weak adhesion between asphalt binders and aggregates and thus differentiate between aggregate types. It was also able to identify the presence of anti-stripping additive in moisture susceptible mixes. However, high variability between test results prevents the definition of a threshold value to be used as a screening test for mixtures.

2. Both the ASTM D4867 tensile strength test and the fracture energy tests were able to identify moisture susceptible mixes and the contribution of liquid anti-stripping additives. However, there was high variability within fracture energy. This problem was reflected in the results of the analysis of variance, which was unable to identify effects consistent expectations, based on common engineering knowledge. The complexity of the test protocol and the variability prevent from recommending this test as a replacement to ASTM D4867.

3. For the testing temperature used in this study, aggregate properties and/or asphalt/aggregate adhesion control mixture failure in indirect tension. Regression analysis on the effects of the binder properties of cohesion and the Superpave rutting and fatigue parameters found that their effect on mixture tensile strength is insignificant. The extent to which adhesion and aggregate properties effect testing results needs to be investigate further.

4. Investigation of the individual and combined effects of binder properties, fine aggregate proportion and composition in the mixture, and percent air voids found that these factors did not have significant effects on the tensile strength of the mixtures tested.

5. The time and cost analysis of the three tests indicates that both the agency and industry could realize a significant economic benefit with the implementation of a screening test into current moisture susceptibility testing requirements. The use of a screening test to waive the mechanical testing requirement for exceptional mixes would significantly reduce time and resources needed for mix moisture susceptibility testing.

Recommendations

1. The stripping test as defined in this report is not suitable for use as a screening test for the agency. The variability of the test is too high to be used as a surrogate test to predict moisture susceptibility at an acceptable level of risk for the agency or industry. There is significant economic benefit to the implementation of a simple screening test, however no alternative can be recommended at this time.

2. Since the Fracture Energy Test is a more fundamental test, further research is needed to reduce the variability of this test. The fracture energy parameter shows promise in its ability to quantify moisture damage, as seen in the stress-strain plots of behavior of various mixtures. However, the variability of the test must be reduced before further investigation into development of any specification involving this parameter can begin.

3. The role of adhesion in the performance of the mixtures used in this study must be fully understood. The adhesion between the various asphalts and aggregate sources used in this study must be incorporated into the results of mixture testing to define the contributions of the asphalt binder to mixture performance. Previous work by Kanitpong has used the PATTI test to measure asphalt aggregate adhesion and the how moisture effects the bond. A similar procedure should be used to test the asphalts and aggregates used in this study.

4. The effect of the mastic (fine aggregate and asphalt) should be quantified using unconditioned and conditioned torsion cylinders. This testing would provide an opportunity to evaluate the effect of the amount of natural sand in the aggregate blend. Investigation of the mastic would also allow for better understanding of the significance of moisture damage in the fine aggregate. Testing of torsion cylinders has been performed previously by Massad and Little, and also at UW Madison. This work should be reviewed and the procedures adjusted to incorporate the effects of moisture.

5. To fully understand the moisture damage phenomena the physio-chemical interaction between the asphalt binder and aggregate must be investigated further. Previous research by Ken Thomas of Western Research Institute provides a starting point for this investigation. Another aspect of asphalt-aggregate interaction that should be investigated is the concept of surface tension and its effects on adhesive strength. Work has been done by Little at Texas A&M in this area.

Table of Contents

LIST OF TABLES

List of Figures

Chapter 1: Introduction

1.1: Background

Moisture damage contributes significantly to many pavement failures throughout the United States. It weakens the resistance of the pavement to two of the three primary distresses used by the majority of pavement technologists to quantify pavement performance: rutting and fatigue cracking. Furthermore, moisture damage increases the susceptibility of the pavement to raveling, a distress that causes the loss of skid resistance on the surface of the road. The sensitivity of pavement performance to moisture has been known for over six decades. Over this time period academia, industry, and state agencies have been researching and developing laboratory test methods to predict this phenomena [25]. The two main mechanisms of moisture damage have been common knowledge in these circles for years. It is well established that these mechanisms are the loss of the bond between the asphalt and aggregate (adhesion) and the loss of bond between asphalt molecules (cohesion) in the binder. However, significant challenges have been realized in developing a cost effective laboratory test that can accurately simulate how moisture damage occurs in the field. This problem raises both policy issues within the agency and economic issues within industry. Numerous studies have questioned the accuracy of the Tensile Strength Ratio (TSR), the parameter currently used by most states to quantify moisture damage (ASTM D8467 or AASHTO T-283). Furthermore, a disconnect exists between the introduction of moisture damage in the field and moisture damage induced in test specimens by conditioning in the water bath [7]. Given these circumstances, it is very difficult for any agency to establish and enforce policies requiring asphalt mixtures to attain a certain TSR value with confidence that they are preventing pavement failures due to moisture induced damage. Conversely, industry is committing resources in terms of technician time and wages to perform the TSR as part of their mix design procedure, even though the repeatability and application to the field have been questioned.

The Wisconsin Department of Transportation's (WisDOT) experience with moisture damage testing of asphalt mixes has been very similar to experiences on the national level. The department began to consider the effects of moisture on asphalt mix design in the early 1990's with the implementation of the Tensile Strength Ratio (TSR) Test as defined by ASTM D-4867 [1]. The TSR test established a criterion of 70%, mixes below this level were considered to be moisture susceptible. These mixtures were required to use an anti-stripping additive to enhance their resistance to the effects of moisture. Subsequently, the mixtures containing the anti-stripping additives were required to achieve a TSR of 75% [27]. In the early stages of implementation of the TSR specification, 80-90% of mixes were unable to achieve the required TSR without the addition of anti-stripping agent to the mix. The sudden increase in the use of anti-stripping agent for mixes that had been used throughout the state of Wisconsin raised flags within both industry and the agency. Concerns were voiced questioning the validity and repeatability of the TSR test and the value of the use of anti-stripping additive to correct a problem that may not exist at all or could be remedied through mix design. In summary, the shortcomings of TSR testing discovered through Wisconsin's experience were congruent with those experienced nationally.

WisDOT recognized that in order to adequately address these concerns partnerships between the agency, industry, and academia must be formed to define, scope, and oversee research projects related to evaluating the TSR testing procedure and how it relates to field performance. This collaborative effort resulted in WisDOT funding of two research projects:

- 0092-45-94: Evaluation and Correlation of Lab and Field Tensile Strength (TSR) Procedures and Values in Assessing the Stripping Potential of Asphalt Mixes.
- 0092-03-07: Evaluation of the Extent of HMA Moisture Damage in Wisconsin as it Relates to Pavement Performance.

To properly communicate the motivation for this current research project it is imperative to use the proceeding sections to describe Wisconsin's past research efforts. The following sections will provide a summary of WisDOT practices in measuring pavement performance and the research objectives and findings of the past research projects funded by WisDOT.

1.1.2: Wisconsin Measures for Pavement Distress

The Wisconsin Department of Transportation defines procedures for measuring pavement distress in the PDI Survey Manual [26]. The manual specifies that pavement distress on a given interstate or state trunk highway is measured by performing a bi-annual pavement distress survey. The distress survey involves the selection of a 0.1 - mile test section of the highway that is a representative sample of the distress realized over an approximately 1 mile section of roadway. Distress data is collected using an instrumented vehicle that uses laser measurements to record permanent deformation in the wheel path of the roadway and collects video of the test section. The video is then reviewed to evaluate the cracking distresses realized in the pavement. Results of pavement distress surveys are kept in the Pavement Information File (PIF). Through this methodology WisDOT measures the following pavement distresses as shown in Table 1.1:

Table 1.1: Summary of Flexible Pavement Distresses Measured by WisDOT

Alligator/Block Cracking (ALCR/BLCR)	Edge Raveling (ER)
Transverse Cracking (TRC)	Surface Raveling (SR)
Longitudinal Cracking (LCR)	Rutting (RT)
Patching (PA)	Longitudinal Distortion (LDT)
Flushing (FL)	Transverse Distortion (TDT)

These distresses are analyzed, defined, and categorized into ranges of extent and severity. Where extent is the frequency of occurrence of a given distress over the test section and severity is the level of distress in the pavement. These categorical measures are combined algebraically to form the Pavement Distress Index (PDI). PDI is an aggregate index used to summarize the level of pavement distress using a single value. The formula for PDI is provided in Equation 1.1.

$$\text{(Eq. 1.1) PDI (Asphalt Pavement)} = 100 * (1-(ALCR/BLCR * LCR * TCR * PT * FL * ER * SR * RT * LDT * TDT))$$

The measure of PDI serves as an initial triggering mechanism in WisDOT pavement management to indicate possible needs for maintenance or rehabilitation. As previously stated, PDI is an aggregate measure of pavement distress; therefore PDI increases over time as distress increases in the pavement. WisDOT policy is then used to set PDI thresholds to identify pavements in need of maintenance or rehabilitation. The individual distresses of pavements exceeding the aforementioned thresholds are then investigated to prioritize and plan for future maintenance and rehabilitation activities.

Previous work by Kanitpong [4] has identified two specific distresses in pavements experiencing moisture damage: surface raveling and rutting. The following is a detailed definition and description of how each are measured as provided in the Wisconsin Department of Transportation PDI Survey Manual [26].

Surface Raveling: Progressive disintegration of the pavement from the surface downward. This distress is caused by the loss of bond between the asphalt binder and aggregate particles, resulting in aggregates being dislodged from the surface of the pavement. The loss of aggregates on the pavement surface presents a safety hazard due to loss of skid resistance on affected surfaces of the road. Surface raveling is measured in severity only. Severity is defined in three levels:

- Slight: The aggregate and/or asphalt binder has worn away from the surface and the surface texture is slightly rough or pitted.
- Moderate: Same as definition for slight except the surface texture is moderately rough or pitted.
- Severe: Same as definition for slight except the surface texture is severely rough or pitted.

The qualitative severity levels discussed above are supplemented with pictures showing examples of each severity level in the PDI Manual [26] to provide more guidance for WisDOT employees performing the pavement distress survey.

Rutting: Rutting is characterized by permanent deformation in the wheel path of the pavement after repeated traffic loading. This distress is caused by either insufficient support provided by the base and sub-grade layers of the pavement structure or material deficiencies in the hot mix asphalt. In terms of moisture damage, rutting would be realized in the HMA layer due to a moisture susceptible mix losing its ability to resist repeated load due to the moisture effects. Rutting is measured using the following severity levels:

- Slight: ¼” – ½” rut depth
- Moderate: ½” – 1” rut depth
- Severe: Rut depths greater than 1”

Understanding how WisDOT measures PDI and the distresses directly related to moisture damage is a necessary first step in analyzing the methodologies and results used in the previous WisDOT sponsored projects reviewed in the subsequent sections.

1.1.3: Project 1: Evaluation and Correlation of Lab and Field Tensile Strength (TSR) Procedures and Values In Assessing the Stripping Potential of Asphalt Mixes [2].

The main focus of this study was to assess the impact of moisture damage on the performance of Wisconsin pavements and to evaluate the ability of the existing laboratory test methods to predict real world construction and in-place moisture susceptibility. The impact of moisture damage on pavement performance was studied using two different approaches, the first of which was the evaluation of the performance of pavement sections with a wide range of mix properties using the Pavement Distress Index (PDI) measured by the WisDOT Pavement Management Database. Initial investigation using this approach found no correlation between the TSR measured during the mix design process and PDI. Based on the lack of correlation, further investigation into relationship between TSR and PDI was stopped in favor of using another approach to investigate correlation between laboratory and field performance.

The second approach used to investigate the relationship between moisture damage measured in the lab and moisture damage experienced in the field was performed by testing the TSR of reproduced cores taken from 17 pre-determined in-place pavements. It was hoped that the TSR from actual in-place pavements would be more closely related to pavement performance, thus allowing for more refined policies related to moisture damage thresholds. Projects were selected to provide a wide range of aggregate types, aggregate structures, binders, and laboratory measured TSR values. Cored samples were remolded and again tested for TSR, test results were compared to pavement performance of the specific sections as measured by PDI and to the TSR values originally measured during the mix design process. Analysis of the results produced no correlation between TSR measured from the pavement cores and pavement distress.

The testing of the TSR of in-place pavement cores also allowed for evaluation of the ability of the TSR test from laboratory mixes to relate to the moisture susceptibility of in-place pavements. This objective was attained through comparison of the TSRs measured in the laboratory relative to those measured using field cores. It was predetermined that TSR values for the two different samples would not be the same due to increased stiffness of the binder in the cores caused by aging in the field. However, it was expected that the two samples (laboratory and field cores) would provide similar rankings of the different mixtures in terms of moisture susceptibility. This hypothesis was rejected through analysis of the test results, the TSR values measured with the laboratory prepared mixes did not rank the mixes in the same order as the TSR values measured with the field cores. In summary, this study found that no relationship could be drawn between the TSR values measured in laboratory mix design and pavement performance as measured by PDI, nor did a relationship exist between TSR values from laboratory mixes and field cores. Furthermore, the recommendations of the report questioned the repeatability of the test due to variation in individual test results for wet and dry tensile strength. Based on these findings it was recommended that, the usefulness of the TSR test in predicting moisture damage in the field and the validity of the threshold TSR value of 70% be further examined.

1.1.4: Project 2: Evaluation of the Extent of HMA Moisture Damage in Wisconsin as it Relates to Pavement Performance [4].

Based on results of the previously discussed study, another research project was funded to further investigate the effect of moisture damage on Wisconsin's HMA pavements and if it can be accurately quantified by the existing TSR testing procedures. The project had two main tasks.

1. Further investigation of the relationship between pavement performance and TSR laboratory values.

The first task was an expansion on the investigation in the previous project regarding the relationship between TSR and pavement performance by considering more pavement sections from a wider variety of locations and used specific measures of pavement distress. A total of 21 pavement sections were selected for analysis for this project from nine geological areas in Wisconsin. This allowed for examination of the effects of aggregate mineralogy on pavement performance as well. Theoretically, different aggregate mineralogies exhibit different levels of moisture susceptibility in asphalt mixtures, which should relate to distress.

The specific pavement distresses related to moisture damage, surface raveling and rutting were included with PDI as the distress measures used to examine the relationship between moisture damage and pavement performance. There are two benefits to this expanded approach. The main benefit is that as seen in Formula 1.1, PDI is an aggregate measure of pavement performance that includes all pavement distresses. Therefore, the effects of the distresses related to moisture damage could be lost when combined with other distress measures in the PDI calculation. Thus, the use of the specific distresses of rutting and surface raveling allow for analysis of the relationship between pavement performance and the TSR test without the confounding effects inherent to the calculation of PDI. Furthermore, the increased number of sections allows for verification of the findings of the previous report, namely, that PDI and TSR are unrelated.

The findings of the Task 1 of the study were as follows:

- As was concluded in the previous study, no relationship exists between pavement performance as measured by PDI and TSR values of laboratory mixes.
- No relationship exists between TSR and the specific pavement distresses of surface raveling and rutting.
- There is no discernable relationship between aggregate mineralogy and pavement performance.

2. Evaluation of the effects of the use of anti-stripping additives on HMA and asphalt binders.

The evaluation of anti-stripping additives on HMA were quantified by comparing the pavement performance parameters specified in Task 1, namely PDI, surface raveling, and rutting for mixes prepared with and without anti-stripping additive for a given aggregate mineralogy. Results showed that anti-stripping additives affect pavement performance; however, no discernable trends could be identified from these efforts due to lack of data for some aggregate mineralogies and wide variation between performance results.

To evaluate the effect of anti-stripping additive on the binder properties, binders without anti-strip agent and with anti-strip agent added were tested for damage resistance and rutting resistance using the measure of ratio of dissipated energy and accumulated strain, respectively, as measured by the Dynamic Shear Rheometer (DSR). Results of these test showed no differences between binders with and without anti-stripping agent that could be differentiated from the variability inherent to the testing equipment. The adhesive properties of the asphalt binder with and without anti-stripping additive were also tested using a pneumatic device to measure the force required to break the bond between an asphalt and aggregate surface. In order to quantify the effect of the anti-strip agent samples were conditioned in water for 24 hours and tested. Results showed that for certain aggregate mineralogies the addition of anti-strip agent greatly increases the adhesive properties of the asphalt binder in the presence of water.

The overall findings of this study concluded that the TSR test has the ability to identify the effects of anti-strip agent in a mix; however it is unable to correlate to any measure of pavement performance relevant to moisture induced damage. The main recommendation was the investigation of test methods that could provide information regarding the presence of anti-stripping additive in a more efficient manner should be pursued.

1.1.5: Motivation for Current Project

The results of the two projects previously funded by WisDOT concluded that current practices for quantifying moisture susceptibility in the laboratory have no relationship to what is happening in the field. Furthermore, the TSR test currently being used poses a significant commitment in terms of time and investment for contractors in the mix design process. Current practices also present a certain level of risk for the agency in that they do not specify a test that can accurately predict field performance in terms of moisture susceptibility. This shortcoming prevents them from assessing any moisture related performance issues related to conventional mix designs or new mix design technologies.

The current research project hopes to serve both the short and long term needs of the agency and industry by investigating possible modifications or replacement of current procedures to create the link between laboratory test results and field performance. In the short term, this includes the investigation of more efficient test methods that could be used to screen mixes and determine when TSR testing is appropriate, as opposed to the current practice of requiring TSR for all mixes. To address long term needs, this project investigates more sophisticated methods of indirect tension testing aimed at decreasing the variability of the TSR test or using other mechanical properties of the asphalt mixture to quantify moisture damage. If these testing methods prove to be viable options, further research would be required to compare the laboratory test results to field performance.

1.2: Problem Statement

This project investigates three approaches that quantify the moisture susceptibility of asphalt mixtures. These approaches are: the indirect tensile strength test as defined by ASTM D-8467 [1], the fracture mechanics framework developed by Birgsson, et al in Florida [6,21,28], and a version of the Evaluation of Stripping in Loose Mixtures test developed by the Quebec Department of Transportation (QDOT) [15]. All three tests use a relative comparison of performance of conditioned and unconditioned mixtures to quantify moisture damage. The aim of this research is to present the consequences of moisture damage on physical characteristics of asphalt mixes and to compare these test methods in terms of both their ability to predict the changes in these characteristics, and the economical and logistical requirements necessary to implement them for use by both state agencies and contractors.

1.3: Hypotheses

The results of this research will provide WisDOT a variety of testing options and approaches to modify their current moisture damage specifications. Specifically, a relationship exists between the results obtained from the mix provided by the non-mechanical stripping test and the mechanical properties of indirect tensile strength and fracture energy measured by the indirect tensile test. This relationship can be used to define a threshold value of the stripping test to identify mixes for which TSR testing is appropriate.

A second hypothesis for this research is that the IDT testing results will provide a more accurate measure of TSR through reduced variability between individual tests. The IDT testing will also provide a more refined approach to quantifying moisture damage through analysis of the measurements of the fracture energy of mixes or the associated stresses and strains measured using the fracture energy approach. Furthermore the results of the IDT test will display a relationship between asphalt binder and mixture properties and test results.

1.4: Research Objectives

The following objectives have been established to define the path of this research:

1. Evaluate the feasibility of using the modified version of the evaluation of stripping in loose mixtures developed by the Quebec Department of Transportation to accurately predict moisture damage in asphalt mixtures through quantifying the loss of adhesion between certain asphalt/aggregate combinations due to moisture effects. Determine if this test will serve as a viable supplement to current TSR testing requirements, by providing a screening test to identify mixtures for which the TSR testing requirement can be waived.
2. Compare the variability of the indirect tensile strength test results found in this study to ranges of TSR values published in WisDOT project 0092-95-04.
3. Evaluate the ability of the application of the principles of the fracture energy framework developed by Birgsson et al. to clearly differentiate the effects of moisture on the asphalt mixture through calculated fracture energies or comparison of stress/strain plots generated during testing.

4. Identify any binder properties significant to mixture performance.
5. Examine the significance of percent air voids and aggregate gradation on mixture performance.
6. Address the logistical and economical requirements associated with each test studied to assess possibilities for agency implementation of the test methods.

1.5. Research Methodology

The research methodology used in this study is illustrated in Figure 1.1 and consists of the following main tasks.

1. <u>Identify and collect mix designs with a wide range of resistance to moisture damage.</u>

To provide a meaningful comparison of the three test methods it was imperative to obtain asphalt mix designs with a wide range of moisture susceptibility. Furthermore, it was desirable to evaluate different types of aggregate. This task was achieved through use of the Wisconsin Department of Transportation Mix Design Database [23]. Mix design data from 1997 to 2004 was used to identify mix designs of granite, gravel, and limestone aggregate types. The mix designs selected using this procedure all had adequate to exceptional resistance to moisture damage. All of them were able to come very close to or exceed the current TSR threshold of 70% without the use of anti-stripping additive. To obtain a mix design with a high level of moisture susceptibility the database was mined further to identify aggregate sources that showed a history of using anti-strip additive in their mix designs. This procedure allowed for identification of an aggregate source that requires the addition of anti-strip additive in virtually all of its mix designs (information provided by contractor). This task produced materials for five mix designs using granite, limestone, and gravel aggregate types. TSR values for these mix designs ranged from 0.43-0.87, providing an acceptable range of moisture damage characteristics to serve as a means of comparison between the proposed testing methods.

2. <u>Test HMA mixtures using the three test methods proposed.</u>

This task involves the physical testing of the mixes, collection of the data, analysis of the data, and comparison of results to examine any relationships that exist and how they correlate. Specifically, the following relationships will be examined further:

- Stripping test (non-mechanical) correlation with the TSR and Fracture Energy (mechanical) test results.
- Variability of the TSR values measured for this project and comparison of variability of results published in project 0092-95-04.
- Variability of Fracture Energy results.
- Plot of stress vs. strain curves for all mixes tested. Evaluation of these plots to assess their ability to identify moisture damage.
- Statistical analysis using Analysis of Variance (ANOVA) to evaluate the ability of the mechanical test methods to identify the presence of moisture damage and changes in mixture design properties pertinent to moisture damage.

- Statistical Analysis of percent air voids, and gradation to identify properties that have significant effect on mixture performance.

3. Test asphalt binders used in the mixtures and examine any relationships between mixture test results and binder properties.

All asphalt binders used in the HMA mixes were tested to determine the parameters of complex modulus (G*) and phase angle (sin δ). The cohesion of the binders was also tested by measuring the pull off strength of the asphalt binder. All tests were performed using the dynamic shear rheometer (DSR). Test results will be analyzed and used to examine the following relationships:

- Examination of correlation between cohesion (Tack Factor) and mechanical testing results to evaluate the relationship between cohesion and dry strength, wet strength, and TSR.
- Regression analysis to identify any binder properties that have significant effects on the mixture testing results.
- Identification through statistical analysis of binder and mixture properties that have significant effects on moisture susceptibility.

4. Economic Analysis

In evaluating the possibility of the implementation of a test method on a state and industry level consideration of the economic and logistical issues associated with the new test is necessary. In order for a test to be cost effective it must be able to clearly and reliably identify the effects of moisture while minimizing agency/contractor investment in terms of training, time, and labor. In this study equipment and time requirements were used to estimate the unit cost associated with using each test in the mix design process. The reliability and technical benefit of each test method will be addressed in Task 2. This task will focus on incorporating the economic and logistical requirements into the technical information provided in Task 2 to identify the cost effectiveness of each test method. Recommendations will be made in identifying if and how these test methods should be incorporated into current WisDOT procedures. These recommendations will include the potential impact of each test method on current procedures and the associated benefits in terms of cost savings that will be realized with their implementation.

5. Final Report and Recommendations

This task involves consideration of the results from all of the data collected in tasks 2 and 3 and using the criteria discussed in task 4 to make overall conclusions and recommendations for the project. The main recommendations will include discussion of each test method in terms of the benefit of the technical information they provide relative to the cost associated with them. Based on this information, a decision will be made to determine the usefulness and feasibility of incorporating these tests into current WisDOT practices.

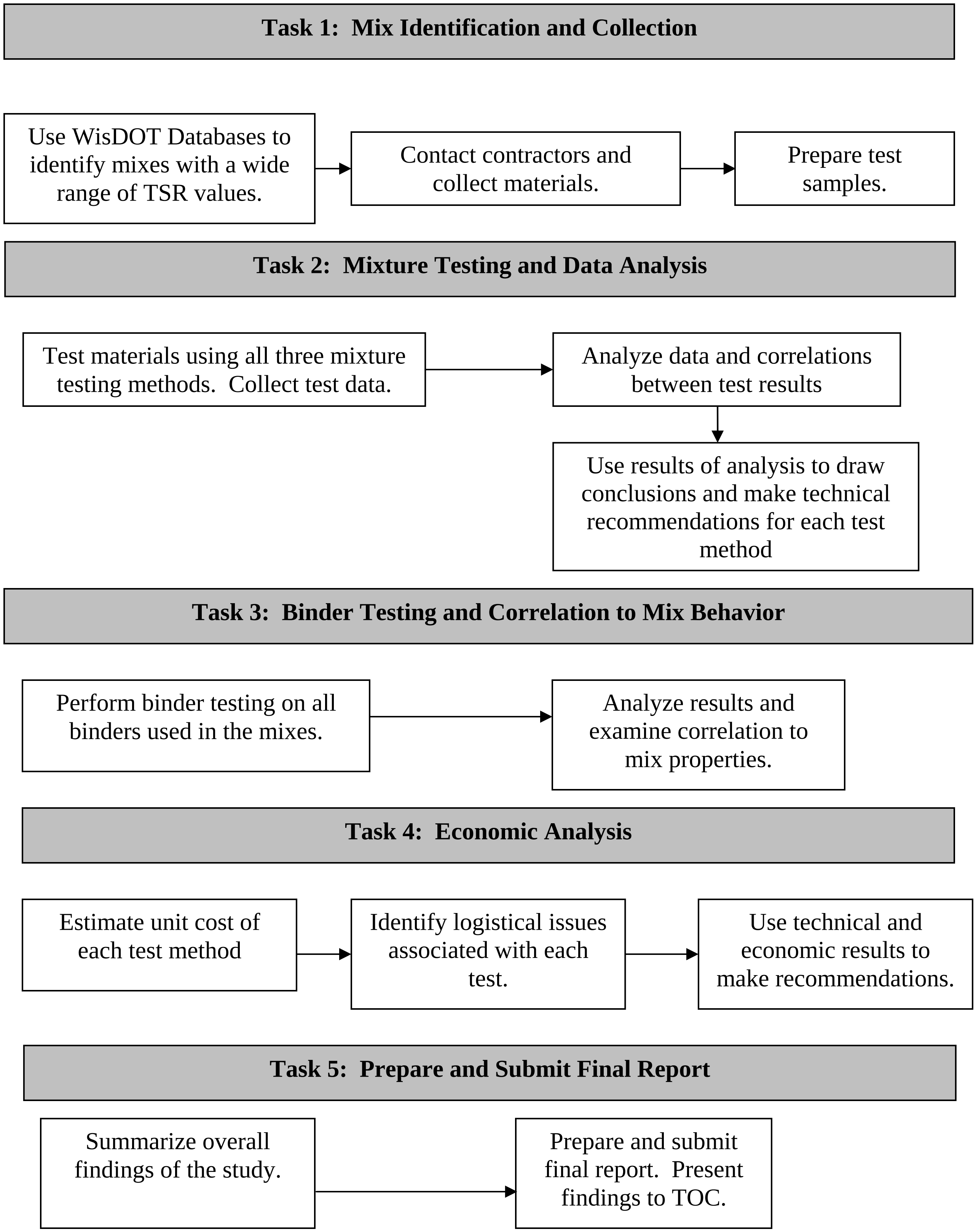


Figure 1. 1: Diagram of Research Methodology

1.6: Summary

This report is organized into six chapters. Chapter 1 contains the introduction, problem statement, hypothesis, objectives, and research methodology. Chapter 2 summarizes the literature review in terms of material properties and physical mechanisms that influence adhesion, mechanisms of stripping, summary of moisture damage tests on loose mixtures, and background information regarding the effects of anti-stripping agents. Chapter 3 provides the experimental plan and summarizes the concepts behind and the procedures in the tensile strength, fracture energy, and stripping tests, and the motivation for the binder testing. Chapter 4 contains analysis of the test results including how they are related and identification of significant factors. Chapter 5 provides analysis of the test methods from an economic point of view. Finally Chapter 6 presents the overall findings and recommendations of the study.

Chapter 2: Literature Review

2.1: Introduction

Moisture damage is a complex phenomenon that cannot be attributed to a singular material property or physical mechanism. The aim of this chapter is to present the various material properties related to moisture damage, the physical mechanisms of stripping and different test methods to quantify them, and additives used to reduce moisture susceptibility of mixes. There are a variety of material characteristics in both the asphalt binder and aggregate that contribute to moisture damage. These characteristics and how their effects can be minimized through production and construction quality control will be addressed. Furthermore, different interactions between the three phase asphalt-aggregate-water system present different mechanisms that cause stripping to occur in asphalt mixtures. These mechanisms will be defined and related to the material properties that facilitate the stripping of asphalt mixtures.

In certain instances consideration of the material properties and mechanisms of stripping are insufficient in preventing a mix from being moisture susceptible. In these cases there are a variety of additives that can be used to enhance the resistance to moisture damage of the mix. These additives include liquid anti-stripping agent, hydrated lime, and polymer modification of the asphalt binder. The physical and chemical reasons these additives increase resistance to moisture damage will be addressed.

To accurately assess the moisture damage phenomena the theoretical hypotheses formed through consideration of the contribution of material properties and additives must be verified through actual asphalt mixture testing. This study considers both non-mechanical and mechanical test methods to assess the moisture susceptibility of asphalt mixtures. The non-mechanical testing method used in this study is a modified version of the stripping test to determine asphalt-aggregate adhesion developed by the Quebec Department of Transportation [15]. The study also used two mechanical testing methods to quantify moisture damage, namely, the tensile strength ratio as measured by ASTMD-8467 [1] and the fracture mechanics framework developed by Birgsson et al [6,21,28]. In order to compare these test methods and determine their applicability to WisDOT it is imperative to first establish the motivation behind these tests and how they apply to the material characteristics and physical mechanisms of stripping in asphalt mixtures. These test methods and the concepts behind them will be described in detail in Chapter 3. Furthermore, it is necessary to understand the contributions of the various anti-stripping additives used in asphalt mixtures to ensure that these contributions are realized in both the non-mechanical and mechanical mixture testing results.

2.2: Material Properties that Affect Susceptibility to Moisture Damage

The strength and stability of asphalt concrete is derived from the cohesive strength of the binder and the frictional resistance produced by aggregate interlock. Assuming the cohesion of the asphalt is strong, and the aggregates and asphalt are fully bonded, failure in the mix must occur either within the binder or through the aggregates [13]. Both of these materials are very strong. However, when moisture is introduced into the mixture aggregates with high affinity for water relative to affinity for asphalt will try to create bonds with the water [13]. This competition between asphalt and water for bonding sites on the aggregate surface causes a weakening of the bond. This provides a weak spot in the mix, allowing it to fail at the asphalt aggregate interface. This phenomenon is commonly referred to as stripping. The mechanical and physical properties of both the aggregate and asphalt binder affect the adhesion and stripping phenomena significantly.

There are two main characteristics of asphalt binders that are important to stripping: viscosity and asphalt chemistry [13]. It has been observed in many studies that binders of high viscosity are able to resist displacement by water much better than low viscosity binders. However, solving the moisture damage problem by simply specifying highly viscous binders would be detrimental to overall performance of the pavement in terms of constructability, low temperature cracking, and fatigue cracking. The other governing characteristic of asphalt behavior in terms of stripping potential is asphalt chemistry. Asphalt chemistry is dependent on two factors: crude oil source and refining methods [8]. The chemical composition of asphalts is crude source dependent. This allows for the potential to define asphalts with a propensity for stripping by determining the main compounds of its chemical structure. Studies by Peterson, cited in Kanitpong identified asphalts containing compounds such as certain forms of carboxylic acids and sulfoxides have displayed more moisture susceptibility [13]. To fully describe the chemical composition of asphalt, crude oil refining methods must be considered as well. Different chemical reactions used in the refining of the crude have the potential to leave undesirable reactants in the crude oil which will be present in the asphalt binder. An example of the refining process controlling moisture susceptibility can be found in the following section in the description of stripping by spontaneous emulsion.

Equally important in determining the effects of adhesion and stripping potential of the asphalt-aggregate interface are the mineralogical and physical properties of the aggregates. The mineralogical property of aggregates most important for stripping is the aggregates affinity for water. In terms of the three phase system of aggregate, asphalt, and water, the aggregate affinity for water can be defined by the Table 2.1.

Table 2. 1: Relationship Between Aggregate Mineralogy and Resistance to Moisture Damage

Aggregate Affinity	Definition	Composition	Silica Content	Resistance to Moisture Damage
Hydrophillic	Have a greater attraction for water than for asphalt binder.	Acidic	High	Poor
Hydrophobic	Have a greater attraction to binder than water.	Basic	Low	Good

In general, hydrophobic aggregates provide a better resistance to moisture damage than hydrophilic aggregates [13]. In terms of basic chemistry this idea is conceptually sound. Hydrophilic aggregates are acidic and prefer water, which is basic in order to reach a level closer to chemical equilibrium. Conversely, the basic, hydrophobic aggregates would rather stay bonded with the acidic asphalt binder in a state closer to equilibrium than become more basic by creating bonds with water [5]. These concepts can be used as a rule of thumb in identifying aggregate susceptibility to stripping; however there are always exceptions to the rule, necessitating experimental evaluation of the mix for susceptibility to moisture damage.

The physical properties of the aggregate also contribute significantly to the asphalt aggregate bond and its resistance to stripping [8]. Aggregate properties such as roughness, porosity, and dust coatings all greatly affect adhesive strength. Surface roughness increases bond strength by providing more surface area to accommodate the asphalt-aggregate bond. Furthermore, an optimum level of porosity is desirable to allow more interlocking in the bond between the asphalt and the aggregate. Finally, the aggregate should be as clean from dust as practically possible. Dust coating the aggregate does not allow for the asphalt binder to bond directly to the aggregate, creating a space between the asphalt film and the surface of the aggregate. This allows water to be absorbed into the aggregate and initiate the stripping mechanism of displacement.

The properties discussed above are either attributed to the nature of the aggregate and asphalt materials or are controlled by construction materials selection and quality control. The effects of both of these on HMA moisture susceptibility must be realized in considering solutions for this problem. Research can work to investigate and continue improvements on current agency practices. However, the agency must realize a responsibility in how they decide to use the research and how they write specifications to control the properties of the materials during construction. Based on pavement performance data presented in the previous two WHRP projects it is clear that WisDOT has been successful in developing specifications and allowing them to adapt as more knowledge is gained [27]. These practices have allowed WisDOT to control the inherent variability of construction materials to the extent that moisture damage failure due deficient construction materials is unlikely.

2.3: Aggregate Blend Components and Their Contribution to Moisture Damage

Conventional asphalt mixes include the blending of five or six different aggregate types proportioned such that they meet the job mix formula gradation requirements specified by Superpave. A typical aggregate blend consists of two or three different sizes of coarse aggregate, manufactured sand, natural sand, and RAP. In evaluating the potential for moisture damage in any asphalt mixture the proportions of these components in the mix and their individual gradation must be considered. Simple analysis of the job mix formula is insufficient in determining reasons for mixture moisture susceptibility. Specific consideration must be given to the relative proportions of coarse and fine aggregate in the mix and the natural sand fraction in the fine aggregate. Taking these factors into account allows for better characterization of the contact points between aggregate particles that provide strength to the mixture and further insight into the differing effects of moisture on the integrity of these contact points.

The aggregate specific component to moisture damage is asphalt aggregate adhesion in both the coarse and fine aggregate. Furthermore, asphalt/aggregate adhesion maintains the contact points between coarse aggregates, coarse and fine aggregates, and the fine aggregates in the asphalt mixtures. A loss of adhesion at any of these interfaces reduces the strength between contact points, facilitating the movement of particles past each other under loading, thus compromising the strength of the mix. As previously stated, this adhesion is dependent on aggregate mineralogy, surface texture, and particle shape. All of these factors are directly related to the proportions of the aggregates used in the mixture. In general, it is preferable for mixtures to be composed of mostly angular aggregates produced through mechanical crushing because these particles allow for greater interlock and internal friction than the rounded particles found in a natural sand source [20]. Work by Kholsa, confirms these concepts by showing that the presence of natural sand in a mixture increased moisture susceptibility as shown by an approximately 8% greater reduction in tensile strength due conditioning than a mix using only manufactured sand. However, the use of strictly manufactured aggregates in an asphalt mix is not practical. Natural sand is added to a mix to improve mixture workability and reduces cost [14].

The contradiction between practice and ideal mixture properties for moisture resistance necessitates consideration of the natural sand contribution to the fine aggregates in the blends used in this study. To address this issue the proportion of natural sand in the fine aggregate (passing the number 4 sieve) in the job mix formula will be examined. Further examination of the individual contributions of the mix constituents to the fine aggregate proportion could reveal that the fine aggregates in a mix thought to be composed of mainly manufactured materials could actually be primarily natural sand. This would result in more rounded fine aggregate particles in the mix, potentially reducing moisture susceptibility. The shape of the natural sand particles is determined by measuring fine aggregate angularity (FAA). Natural sands in Wisconsin are considered to be relatively angular, a survey taken by Stakston and Bahia reported most natural sands used in Wisconsin had FAA values of approximately 40, whereas the manufactured sands had FAA values greater than 43 [5]. However, it is expected that even a small change in FAA would have adverse effects on the moisture resistance of a mixture. A mix with a fine aggregate composed of mainly natural sand with a lower FAA relative to that of the manufactured sand would be expected to be more moisture susceptible.

2.4: Mechanisms of Aggregate/Asphalt Adhesion and Stripping

The research of Kanitpong (2004) and others have suggested that the moisture damage phenomenon is much too complex to be investigated through solely testing of asphalt mixtures as placed in the field. An understanding of how moisture affects the bond between asphalt/aggregate and asphalt/asphalt is necessary in characterizing total mix behavior. Therefore, the properties of asphalt/aggregate adhesion and asphalt cohesion must initially be studied separately to develop systems to determine these properties and incorporate these measurements into consideration of the moisture susceptibility of the entire asphalt mixture [13]. This research project was solicited and proposed with the intent to investigate test methods to quickly and efficiently quantify the adhesion of asphalt to aggregates. However, investigation of the mechanisms of aggregate/asphalt adhesion and stripping must be understood before a test to evaluate the adhesion of asphalt to aggregates in the presence of moisture can be selected and developed.

2.4.1: Mechanisms of Aggregate Asphalt Adhesion

There are four main theories that have been developed in research dating 1938 to attempt to explain adhesion between asphalt-aggregate systems. The following is a list of these theories and the researchers who developed them:

- Mechanical Theory (Knight 1938, Lee and Nicholas 1954, Rice 1958)
- Chemical Reaction Theory (Rice 1958, Maupin 1982)
- Molecular Orientation Theory (McBain and Lee 1932, Mack 1957)
- Interfacial Energy Theory (Thelen 1958, Ishai and Craus 1977)

Various research efforts have concluded that the phenomenon of aggregate-asphalt adhesion is too complex to be explained solely by any of the theories listed above [13]. Rather, the adhesion of asphalt to aggregate is better explained by a combination of the mechanical, chemical, and thermodynamic principles developed in these theories. The following is a summary of the basic principles that govern aggregate-asphalt adhesion:

- **Mechanical:** Mixing forces asphalt into the pores and irregularities of the aggregate surface, providing sites for interlock.
- **Chemistry**: Chemical reactions between the absorbed asphalt and the aggregate form the bond. The bonding is facilitated by reorganization of the molecular structure of the asphalt to reach an energy equilibrium and bond with the aggregate surface.
- **Thermodynamic**: Adhesion is related to the surface energy of the asphalt-aggregate system. Wetting of the aggregate surface or stripping of the asphalt from the aggregate surface is caused by changes in the free energy of the system. Changes in bonding are due to the introduction of other materials (i.e. water or air) to the system.

Fundamental research to better explain asphalt-aggregate adhesion and the effects of moisture is currently being performed at Texas Transportation Institute [16]. This research involves incorporating the relatively new science of Fracture Mechanics into previously established surface energy concepts to model the damage and healing at the micro-level to explain the behavior of asphalt mixtures. The results of this work are promising and may result in new opportunities to develop different test methods to evaluate adhesion.

In summary, asphalt/aggregate adhesion is a complex phenomenon that is dependent on the mechanical, chemical, and thermodynamic interaction of the materials in the system. The complexity of this issue continues to prevent researchers from agreeing on the use of a single test to characterize this phenomenon. The current state of knowledge has led to this research project. Given the information available regarding the mechanism of adhesion and causes of stripping of asphalt from aggregate, different test methods must be investigated to aide state agencies and industry in evaluating mix designs to ensure pavement performance is not hindered by severe damage caused by moisture.

2.4.2. Mechanisms of Stripping and Practical Applications

To accurately assess moisture damage understanding of the mechanism of aggregate-asphalt adhesion as well as the effects of moisture on the established bond is necessary. Understanding of the effects of moisture is best achieved by understanding the causes of stripping and why they affect aggregate-asphalt adhesion. Stripping is characterized as an adhesion failure caused by the loss of a bond between the asphalt and aggregate surface produced by water action. Prior research performed by numerous parties has identified the following mechanisms of stripping [7, 13]:

- Detachment
- Displacement
- Spontaneous Emulsification
- Pore Pressure
- Hydraulic Scour

The following discussion will provide a brief overview of each of these mechanisms in greater detail. The practical aspects of reducing stripping potential in the field will also be discussed.

2.4.2.1. Detachment

Detachment is defined as the separation of asphalt film from an aggregate surface by a thin film of water with no obvious break in the asphalt film [6, 11]. When stripping in a mix is due to detachment, the asphalt film can be totally peeled off the aggregate, indicating a complete loss of adhesion. The detachment mechanism is explained by the theory of interfacial surface energy [7]. The basic premise of this theory is that thermodynamically stable systems tend to prefer equilibrium states of lowest surface energy. Furthermore, Kanitpong cites work done at Texas A&M University that found that the introduction of water to an asphalt-aggregate system causes a release in energy. This energy release allows the system to tend to a state of equilibrium with lower surface energy, implying that the surface of the aggregate has a preference to water over asphalt [13].

The effects of detachment on an asphalt mix can be mitigated by using dry aggregate and by using a less permeable asphalt binder. Both of these measures minimize the chances of water entering the asphalt aggregate system, thus reducing the probability of detachment occurring in the mix.

2.4.2.2. Displacement

Displacement is characterized by the water pushing back the asphalt from the aggregate surface and replacing it. It is a result of penetration of water to the aggregate surface from a break in the asphalt film. The mechanism of displacement can be described by surface energy principles similar to those found in detachment. The system will allow the water to displace the asphalt on the aggregate surface in order to reach an equilibrium state of lower surface energy. The displacement mechanism may also be explained by chemical reaction theories [16].

To minimize the effects of displacement on an asphalt mix, the potential sources of film rupture or spaces in the asphalt aggregate interface must be minimized. Spaces in the asphalt film coating the aggregate are caused by incomplete coating of the aggregate in the mixing process, pinholes formed by dust in the binder caused by a dusty aggregate, and film rupture on sharp edges of aggregate. Two of these three sources can be easily controlled in the production of the mix. Quality control measures can be implemented to ensure that the percent binder used in production remains close to optimum asphalt content to ensure that enough asphalt binder is available to fully coat the aggregates. Pinholes forming from the use of dusty aggregate in the mix can be eliminated by washing aggregates with high dust contents as part of the aggregate production process to ensure that there is a clean surface for the asphalt to bond to [27]. Eliminating these two sources of displacement greatly reduces the probability of this mechanism causing stripping in a mix.

2.4.2.3. Spontaneous Emulsification

In general asphalt is not considered to be significantly affected by the presence of water in the mix. However, certain compositions of asphalts allow water and asphalt to combine and form an inverted emulsion of water droplets in asphalt [13]. The emulsification will trigger the detachment mechanism for stripping when the emulsion reaches the aggregate surface. The moisture damage caused by this mechanism is reversible, if the water evaporates from the asphalt pavement, the emulsified asphalt returns to its regular form [8].

Both of the defining factors of asphalt chemistry play a significant role in determining the potential for spontaneous emulsification in the mix to occur. Basic chemical properties vary by the crude oil source from which the asphalt is produced. This knowledge allows for the understanding of the ideal chemical composition of each asphalt. To correctly evaluate the potential for emulsification, the refining processes used to produce the asphalt must also be considered. Certain crude oil refining methods result in the presence of emulsifying agents in the asphalt. Two of these methods are the use of caustics to neutralize acidic crude oils and the desalting of crude oil. The failure to desalt at all or to desalt after the addition of caustics leads to the same result: salts contained in the asphalt binder. These salts can act as emulsifying agents, promoting emulsification when the pavement is exposed to moisture [8].

2.4.2.4. Pore Pressure

The mechanism of pore pressure is a result of the reduction of air voids in the HMA pavement due to traffic loading [13]. Under loading the water present in the air voids is compressed, increasing pressure against the asphalt film. Repeated loading causes the pore pressure to reach a sufficient level to cause rupture of the asphalt film, allowing water to infiltrate the aggregate surface and begin to strip the aggregate via displacement. This mechanism is usually realized in newly placed HMA pavements. New pavements are compacted to approximately 7-8% air voids under the assumption that densification to 4% air voids will occur due to traffic loading. Therefore early in pavement life the voids are sufficiently large to be interconnected, allowing for water to freely move through the pavement. As the pavement begins to further densify the air voids close and water is trapped inside of them. This entrapped water creates pore pressures in the pavement from both traffic loading and thermal expansion/contraction.

The effects of the mechanism of pore pressure can be mitigated through the correct prediction of traffic levels in the mixture design and consistent compaction during construction. The predicted level of traffic defines the compactive effort to reach 4% air voids in gyratory specimens. Overestimation of the traffic loading will result in an underestimation of design asphalt binder content due to the higher compactive effort used to reach the design air voids percentage [8]. This will make less binder available to coat the aggregates. The thinner coatings will be more susceptible to the effects of pore pressure. Consistent compaction during construction also will aid in helping the pavement resist the effects of pore pressure. Proper compaction will help ensure the mix densifies at the rate planned under traffic loading allowing the air voids distribution in the pavement to be consistent.

2.4.2.5. Hydraulic Scour

Hydraulic scour is a mechanism of stripping that only occurs in surface courses. This mechanism is due to the movement of a tire over a wet pavement. As a tire moves on a pavement the water in front of the tire is pressed down into the void spaces of the road. As the back of the tire passes the water is immediately sucked out behind the tire. This action subjects pavements to thousands of compression-tension loading cycles daily [8]. The cumulative effect of these cycles results in stripping of the aggregate.

2.5: Test Methods on Loose Mixtures to Quantify the Loss of Adhesion Investigated in this Study

Over the years there have been a number of tests developed to qualitatively and quantitatively asses stripping potential using loose mixtures. One aim of this study was to find a test on loose mixtures to be used as a screening test to reduce mechanical testing requirements on the agency and industry. In order to fit these parameters, the test must be both practical and produce quantifiable results. Based on these two requirements two tests were selected as potential candidates for this study: The modified version of the Stripping Test on Loose Mixtures as developed by QDOT (described in Chapter 3) and the Pneumatic Adhesion Test. (PATTI).

The PATTI test was developed by the National Institute of Standards and Technology (NIST) and has been adopted by ASTM as method D 4541 “Pull Off Strength of Coatings Using Portable Adhesion Testers.” The PATTI test involves measurement of the air pressure required to remove a pull stub attached to a surface by a certain adhesive [11]. The air pressure at failure is then converted into a pull-off tensile strength to provide a measure of the adhesive ability of a given material. The method is directly applicable to evaluation of the role of moisture in the adhesive and cohesive properties of different asphalt/aggregate combinations [12]. Samples are prepared such that a variety of aggregates with different mineralogies can be used as the adhesive surface, allowing for examination of the role of aggregate mineralogy in adhesion, the samples can also be conditioned in water. Furthermore, the test method is able to differentiate between adhesive and cohesive failures by examination of the failure surface. However, there are some disadvantages to using this test method in terms of reliability and sample preparation that influenced the research team in choosing not to pursue this study. The variability of the test has been previously studied and was found to be approximately 10%. Furthermore, a study by Kanitpong cites difficulty in controlling the variables inherent to sample preparation, a significant source of variability. Given that the variability of current test methods used by WisDOT equals or exceeds 10%, the variability of the PATTI test was not the deciding factor in electing not to further pursue it. Instead, the PATTI test was not selected due to the rigorous requirements for sample preparation. Sample preparation involves preparing smooth aggregate surfaces suitable for testing, application of an asphalt film of uniform thickness, sometimes conditioning, and testing. All of these steps require significant investment in terms of time and money, reducing the economic benefit of using the PATTI test as a screening test for mechanical testing of mixtures [11,12]. Based on review of the literature, the PATTI test is an excellent candidate for measuring the adhesion of asphalts to aggregate, however due to the logistical requirements associated with the test method; it was found not to fit the scope of this project.

The QDOT Stripping Test was selected as the non-mechanical test to compare in this study due to its potential to serve as a true screening test. A true screening test is one which has the ability to provide technically sound information regarding the mixture at reduced testing time requirements [25]. Preliminary tests using this method displayed the ability to differentiate between moisture resistant and moisture susceptible mixtures and also to identify the presence of anti-stripping agent. Furthermore, the methods used in this test procedure required minimal effort in terms of sample preparation and testing relative to the PATTI test. The QDOT Stripping Test is discussed in greater detail in Chapter 3.

2.6: Anti-stripping Additives

Many state agencies add anti-stripping additive to the mixes that do not fulfill the requirements of their specified test for moisture damage (usually the TSR). The anti-stripping agent is added to prevent stripping in the mix, improving pavement performance by negating the effects of moisture damage on pavement rutting or fatigue. Anti-stripping additives are defined as substances that convert the aggregate surface to one that is more easily wetted with asphalt than water [8]. Testing performed by Kanitpong has shown increased adhesion in binders with anti-stripping agent added [4, 11, 13]. There are two main types of anti-stripping additives currently available: liquid and hydrated lime. Both additives have been proven effective in field trials and various studies in resisting stripping [8, 11].

Liquid anti-strip agents are surface active agents, meaning they reduce the surface tension of asphalt cement, which promotes asphalt adhesion to aggregate. Most liquid anti-stripping additives contain amines as their active ingredient. In addition to improving the mix in regards to stripping, the liquid anti-strip agents must also be heat stable so they are able to maintain their effectiveness at high temperatures [8]. The most common method of application in practice is to combine small volumes (0.5% by weight) of anti-stripping agent to the binder. This method is inefficient because not all of the agent reaches the surface of the aggregate. However, it is much more economical than the alternative, which involves full coating of the aggregates [8]. The overall performance of the mixture is very dependent on the amount of agent added to the binder.

The use of too much additive may be detrimental to the mix, weakening it's resistance to permanent deformation.

The anti-stripping mechanism of lime additives is not well understood, however many studies have proven its effectiveness as an anti-stripping agent [24]. The effectiveness of lime may be due to the fact that it is directly applied to the aggregate and therefore more of it has a chance to contribute to the stripping resistance of the mixture. Hydrated lime can be applied in one of two ways: either by wet application as a slurry or a completely dry application directly to the aggregates. Various projects have been conducted to evaluate the relative effectiveness of the dry addition process to a variety of wet processes [8]. The results of the studies have been inconclusive; therefore the dry method of addition is preferred based on economic considerations.

Conceptually both of these additives achieve the same objective in reducing the amount of stripping realized in an asphalt mix. However, to be determined successful in a practical sense the reduction of stripping caused by each additive must correlate to improved pavement performance and decreased life cycle cost. In this regard the additives are drastically different. In general, the long term effectiveness of liquid anti-stripping additives has not been fully established [8]. This general trend applies to Wisconsin pavements as stated by Bahia in [4] when it was found that the lifecycle cost between pavements with and without anti-stripping agent are basically the same. This trend is further enforced by personal research using Wisconsin's Mix Design Database [23] showing a significant decline in the use of anti-stripping agent across the state of Wisconsin over the last 3 years. Conversely, a study conducted by Seebaly on Nevada highways found that the use of hydrated lime anti-stripping agent resulted in an average of a 3 year (38%) extension of pavement surface life [24].

The use of polymer modified asphalts in pavements also reduces mixture moisture susceptibility; however, polymers are rarely added to the asphalt binder with the sole intent of improving mixture resistance to moisture damage. Polymer modification improves the performance of asphalt pavement to permanent deformation and fatigue resistance due to increased elasticity of the asphalt binder, allowing for faster recovery between loading cycles. Another effect of polymer modification is increased asphalt binder viscosity. The higher viscosity promotes better adhesion of asphalt to aggregate [22], thus improving resistance to failure in a pavement due to loss of asphalt aggregate adhesion caused by moisture damage.

Chapter 3: Experimental Plan and Introduction of Test Methods

3.1: Experimental Plan

The goal in designing this experiment was to select mixes that would provide meaningful results while representing the aggregates commonly used in Wisconsin mix designs. A representative sample was obtained by using the following types of aggregate sources from varying regions in Wisconsin:

- Granite: North Central Wisconsin
- Limestone: Northeast and Southern Wisconsin
- Gravel: Southeast Wisconsin

Mixes ranging from moisture resistant to moisture susceptible were selected to allow for evaluation and analysis of the TSR, Fracture Energy, and Stripping Test methods at a wide range of sensitivities. The use of mixes with a wide range of behaviors provides a valid means of comparison by ensuring that the results of the test would not be confounded with the variability inherent to the test method. The mix selection process was facilitated by the use of the WisDOT Mix Design Database [23] to identify mix designs from aggregate sources that have a history of producing moisture sensitive or moisture insensitive mixes from the 1997-2004 construction seasons. Moisture sensitivity in this analysis was evaluated based on TSR values and historical use of anti-stripping agent. The results of this analysis provided five mix designs, one mix design from a source that uses anti-strip agent in virtually all its mixes and four mix designs with TSR values ranging from 0.70- 0.85.

To further aide the evaluation of the test methods mixture design components were varied to evaluate their effects on mix moisture sensitivity and the relative response of the results of each test method to these changes. The research team worked closely with WisDOT to identify aggregate gradation and the use of anti-stripping additive as the two mix components to vary. To study the effects of differing aggregate gradation fine and coarse aggregate blends were used. In this project a coarse graded aggregate blend was defined as a gradation with less than 40% passing the 4.75mm (No.4) sieve. Two fine-graded mix designs from the same aggregate source were also used for testing. The only differences in these mixes were the use of 10% RAP in the aggregate blend and the use of different asphalt binder sources. The use of two mix designs from one aggregate source does not provide a sufficient data set to draw any conclusions or make recommendations regarding the effects of RAP and asphalt binder source on moisture sensitivity of the mix. The second mix from the same source was incorporated into the testing matrix purely for exploratory/investigative purposes.

Previous discussion in Chapter 2 revealed the importance of further examination of the aggregate blends used in this study to properly assess the impact of different aggregate proportions in the mix on moisture damage. Pertinent aggregate properties identified were the geology of the manufactured aggregates and natural sands, the proportion of coarse aggregate to fine aggregate, and the proportion of natural sand in the fine aggregate of the job mix formula. These values are provided in Tables 3.1 and 3.2. For detailed Job-Mix-Formula gradations of all the aggregate blends used, please refer to the Mix Design Data section of the Appendix.

Table 3. 1: Summary of Aggregate Type for Coarse and Fine Aggregates

		Manufactured Sand		Natural Sand	
Aggregate Type	Gradation/Mix	Type	Blend % in Mix	Type	Blend % in Mix
Granite	Fine Mix 1	Granite	30%	Igneous	15%
	Coarse Mix 1	Granite	7%	Igneous	8%
	Fine Mix 2	Granite	15%	Igneous	20%
Gravel	Fine	Crushed Gravel Fines	28%	Calcercious	31%
	Coarse	Crushed Gravel Fines	10%	Calcercious	20%
Limestone NE	Fine	Limestone	27%	Calcercious	29%
Limestone South	Fine	Limestone	40%	Calcercious	27%

Table 3. 2: Summary of Aggregate Proportions in Job Mix Formulas

Aggregate Type	Gradation/Mix	Coarse Aggregate Contribution	Fine Aggregate Contribution	Natural Sand Proportion
Granite	Fine Mix 1	35.35%	64.65%	21.81%
	Coarse Mix 1	50.77%	49%	15.28%
	Fine Mix 2	33.95%	66.05%	28.46%
Gravel	Fine	31.81%	68.19%	40.10%
	Coarse	56.84%	43.16%	40.87%
Limestone NE	Fine	28.43%	71.57%	36.59%
Limestone South	Fine	34.77%	65.23%	42.95%

The values reported above will be used to evaluate the significance of the effects of the properties of the aggregate blend on the results of the mechanical testing. This comparison will be presented with the mechanical testing results in Chapter 4.

The use of a variety of anti-stripping additives was also incorporated into the mix designs. The incorporation of anti-strip agents into the test matrix was expected to provide an excellent opportunity to evaluate the ability of the various test methods to diagnose moisture susceptibility. A test method must be able to identify the use of anti-stripping additive in a previously moisture susceptible mix to be considered valid. The additives used were liquid anti-stripping agent, hydrated lime, and polymer modification. In the interest of time and materials all aggregate blends were not used to produce mixes with each of the possible additives. Instead, it was decided to use the liquid anti-strip agent Mor-life™, at 0.5% by binder weight for all aggregate blends and perform all of the tests, the amount of anti-strip additive in the mix is consistent with WisDOT specification [27]. The mix with the worst performance was then selected to be prepared using hydrated lime at a level consistent with those found in the literature [24] (1% by weight of aggregate) and SBS modified asphalt binder. The base binder used in the study was graded PG 58-28 this was used for all mixes, the SBS polymer modification raised the binder grade to PG 64-28.

The following table provides a summary of all the independent variables considered in this study and how they were varied for each mix design.

Table 3. 3: Summary of Hot Mix Asphalt Mix Designs

Type	Source	Fine Gradation	Coarse Gradation	Liquid Anti-strip	RAP	Hydrated Lime	SBS Modified
Granite	Central WI Mix #1	X	X	X	-	-	-
	Central WI Mix #2	X	-	X	X	-	-
Gravel	SE WI	X	X	X	X	-	-
Limestone	NE WI	X	-	X	X	X	X
	Southern WI	X	-	X	X	-	-

All mixes presented in the table above were compacted and tested using the Indirect Tensile Test to determine the tensile strength ratio and fracture energy ratio. The loose mix was also tested using the stripping test on loose mixtures developed in this study.

The properties relevant to moisture damage of the asphalt binders used in the mixtures were also tested. The properties measured in this study included: complex modulus (G*), phase angle (δ), and cohesion as measured by the tack test. All binder properties were measured using the Dynamic Shear Rheometer (DSR). The binders used in the mix designs and corresponding tests performed are provided in the table below.

Table 3. 4: Summary of Binder Testing

Grade	Source	Modifier	Mixes	Cohesion	G* and phase angle
PG 58-28	Koch	None	Granite 1 Fine and Coarse Limestone NE Fine	X	X
PG 58-28	Koch	.5% Morlife		X	X
PG 58-28	MIF	None	Granite 2 Fine	X	X
PG 58-28	MIF	.5% Morlife		X	X
PG 58-28	CRM	None	Gravel Fine and Coarse Limestone S. Fine	X	X
PG 58-28	CRM	.5% Morlife		X	X
PG 64-28	Koch	SBS	Limestone NE Fine	X	X

The experimental plan involves both non-mechanical and mechanical asphalt mixture testing. It was imperative that both types of testing be used to achieve the goal of this project. The overall objective of this research was to investigate different test methods to improve the current practice in testing for moisture susceptibility of asphalt mixtures. This objective could be achieved through two avenues:

1. Development of a practical, efficient test method for evaluating asphalt aggregate adhesion to reduce the amount of time and resources invested in moisture damage testing.
2. Investigation of more advanced test methods that provide a better link between moisture susceptibility measured in the lab and field performance.

The Stripping Test was selected as the test method to evaluate to satisfy the first criteria established in objective one. The mechanical testing aspect of this project satisfied objective two by investigating improvements to current TSR testing procedures and the use of fracture mechanics to better link moisture susceptibility measured in the field to pavement performance. Furthermore, mechanical testing results provided means to verify that the results of the stripping test were reasonable. The subsequent sections of this chapter are dedicated to presenting the general concepts and procedures of the tests developed in the experimental plan. Presentation of test results and detailed analysis of how they are related will be addressed in Chapter 4.

3.2: Mechanical Testing of Asphalt Mixtures: The Indirect Tension Test

The mechanical testing of the asphalt mixtures was performed using the Indirect Tensile Test. Testing was conducted in a James Cox and Sons Testing Machine with an environmentally controlled chamber. The current TSR test procedure (ASTM D4867) was modified to allow for one test to measure both the tensile strength and fracture energy. The remainder of this section will introduce the key concepts behind the current parameter used to quantify moisture damage (tensile strength) and the more advanced fracture energy framework investigated in this study. General testing methodology, testing procedures, and sample preparation procedures will also be provided.

3.2.1: Current Practices: The Tensile Strength Ratio (ASTM D4867)

ASTM D4867 is the test method currently used by the Wisconsin Department of Transportation to quantify the moisture susceptibility of asphalt mixtures [1]. This method evaluates resistance to moisture damage caused by a particular combination of variables such as different aggregate sources or gradations or the use of mixes with and without anti-stripping agent. All of these measures focus on the relative change of one parameter before and after conditioning to quantify moisture damage [4]. In this study the relative change of the indirect tensile strength was used to evaluate moisture damage by the tensile strength ratio (TSR), which is the ratio of conditioned to unconditioned tensile strength of an asphalt mixture. This test method uses a certain value of TSR to determine mixture susceptibility to moisture, for Wisconsin this threshold is set at 70% [27].

The ability of the TSR to predict moisture susceptibility that correlates to field performance and the repeatability of the test have been questioned on both state and national levels. National recognition of these shortcomings is evident in the development of project NCHRP 9-34: "Improved Conditioning Procedure for Predicting Moisture Susceptibility of Asphalt Mixtures" in 2002. This project began in March of 2002 and currently has a draft final report under review. The project involves a $400,000 investment to develop conditioning and testing procedures that are more reliable and better able to correlate to field performance. The abstract for this study cites frequent false positives and/or negatives have been recognized by many states [18], meaning that laboratory testing is not correctly predicting field performance.

On a state level, Wisconsin has proven they have been cognizant of the shortcomings of the TSR as specified by ASTM D4867 through their funding of the two previous research studies summarized in Chapter 2 of this report. The findings of these studies questioned both the repeatability of this test method and it's usefulness as a parameter due to lack of correlation with field performance [2, 4]. The study by Bahia and Kanitpong concluded, "The TSR protocol adopted by WisDOT cannot be used to quantify the effects of moisture damage on pavement performance."[4]. Two main reasons exist for these shortcomings in both the test method and its relation to pavement performance: current conditioning procedures do not simulate field conditions and the inability of a single parameter measured in the laboratory to correlate to field performance.

The sample conditioning specified by ASTM D4867 is insufficient in simulating the mechanisms of moisture damage discussed previously in this report. The conditioning may be sufficient in evaluating moisture susceptibility due to differing material properties of aggregates or asphalt binder, however it is too simplistic to account for the various mechanisms of stripping that effect pavement performance. Specifically, the current method is unable to simulate the mechanisms of stripping by hydraulic scour or pore water pressure due to the absence of consideration of the dynamic traffic loading that causes these stripping mechanisms. Stripping due to pore water pressure is the result of the increase of pressure in voids due to traffic loading; this cannot be simulated by simply placing a sample in a water bath. The compression/tension loading cycles caused by tire action in hydraulic scour are also neglected. A conditioning method that accounts for these mechanisms of stripping would possibly improve the correlation of laboratory results to field performance.

Due to the complexity of the effects of moisture damage on asphalt pavements, the use of a single parameter is insufficient to accurately characterize behavior. The use of a single testing parameter does not allow for information regarding the different mechanisms of moisture damage present in the conditioned mix [4]. Separation and analysis of the mechanisms of adhesive, cohesive, and other moisture related damage in the conditioned sample are an integral part of accurately quantifying the effects of moisture damage on laboratory samples. In order to sufficiently separate the different mechanisms of moisture damage more advanced methods of testing that are able to measure a variety of parameters are necessary.

The investigation of different conditioning procedures is beyond the scope of this project and will hopefully be addressed in the findings of NCHRP 9-34. However, this project did include investigation of the use of the fracture energy parameter as a possible replacement for tensile strength as an indicator of moisture susceptibility.

3.2.1.1: The Effects of Air Voids on Tensile Strength Test Results

The literature review and experimental design have already identified that mix type in terms of aggregate type, gradation, and binder grade/modification have a significant effects on the both the measures of tensile strength and the tensile strength ratio. It is expected that these relationships will hold for the fracture energy parameter. Another important factor that must be considered is the percent air voids of the individual test samples. Previous work by Dukatz and Phillips [9] cites a direct relationship between air voids and unconditioned and conditioned tensile strength. In both cases, tensile strength decreases with increasing air voids, however the rate of decrease for unconditioned and conditioned samples is not the same. This difference in behavior is present even within the 7% +/- 1% range of air voids specified in ASTM D4867 [1]. An example of this phenomenon is presented in the study by Dukatz. A limestone mix was prepared at 6%, 7%, and 8% air void contents, tensile strength testing results showed a 28% decrease in unconditioned tensile strength and a 48% decrease in conditioned tensile strength with increasing air voids [9]. This difference in behavior reflects the importance of controlling air voids between conditioned and unconditioned samples when evaluating moisture susceptibility. It is obvious that solely using the specification to control air voids could result in a wide range of measured TSR values, thus rejecting a mix that is moisture resistant or vice versa.

To minimize the potential for incorrect characterization of the mix due to air void variation Dukatz suggests compaction of mixtures to a wide range of air voids and measuring the corresponding tensile strength, then using statistical or graphical methods to estimate the tensile strength at 7% air voids. This method provides a baseline for evaluation of moisture susceptibility by considering the differing variation in the relationship between tensile strength and air voids for conditioned and unconditioned mixtures [9].

Evaluation of the effect of varying levels of tensile strength was not directly in the scope of this study, however the effects of air voids on tensile strength and the differing behavior for conditioned and unconditioned mixes was incorporated into specimen preparation by controlling the air voids of all specimens more stringently than the specification. Efforts were made to keep air void variation between mixes to 0.5%; this requirement was met for all but two mixes, with no variation exceeding 1.1%. Specific evaluation of the air voids in test samples will be presented in Chapter 4. It is hypothesized that this higher level of control will minimize the effects of air voids on the tensile testing results. Refinement of measured results similar to the methods described by Dukatz does not seem applicable due to the small range of air void variation within samples. There is simply insufficient data in terms of sample size and air void range to accurately estimate tensile strength at 7% air voids for all mixtures.

3.2.2: Innovative Testing: Evaluation of Moisture Damage Using Fracture Mechanics Principles

An improvement to the current use of tensile strength or another single parameter to quantify the effects of moisture damage has been proposed by Roque and Birgisson of the University of Florida. The proposed fracture mechanics framework uses concepts developed [21] and verified [28] through the comparison of theoretical models to data obtained from fracture tests. The development of the fracture mechanics framework involved the definition of a suitable crack growth law for asphalt mixtures. Initial difficulties in this approach stem from the inability of linear elastic fracture mechanics (LEFM) to correctly model the physical damage realized by HMA pavements. LEFM assumes that crack growth is continuous and that the rate of crack growth is governed by the Paris Law. Given that an asphalt mixture is a composite material consisting of asphalt, aggregates, and air that have different resistances to fracture, the use of this concept is invalid. Crack propagation will occur through areas of least resistance to fracture; the orientation of the constituent materials in asphalt mixtures prohibits crack growth from propagating continuously [21]. Furthermore, the visco-elastic nature of the asphalt binder in the mix allows for healing of micro-damage realized during loading cycles, again rendering the assumption of continuous crack growth inapplicable. Based on the behavior of the materials in the HMA mixture it was decided that crack growth will be modeled as growth in a discrete step-wise manner throughout the mixture using the threshold concept.

The main assumption of the threshold concept is that a damage threshold exists that defines the development of discontinuous macro-cracks at any point in the mixture. This concept involves the use of a threshold, such as the fracture energy density, to define a failure criterion for initiation and propagation of cracks [21]. If the threshold is not reached, micro-damage occurs that will heal given adequate time between loading cycles. This concept and a comparison to how it differs from using an approach based purely on LEFM is best illustrated through the following plot taken from [6].

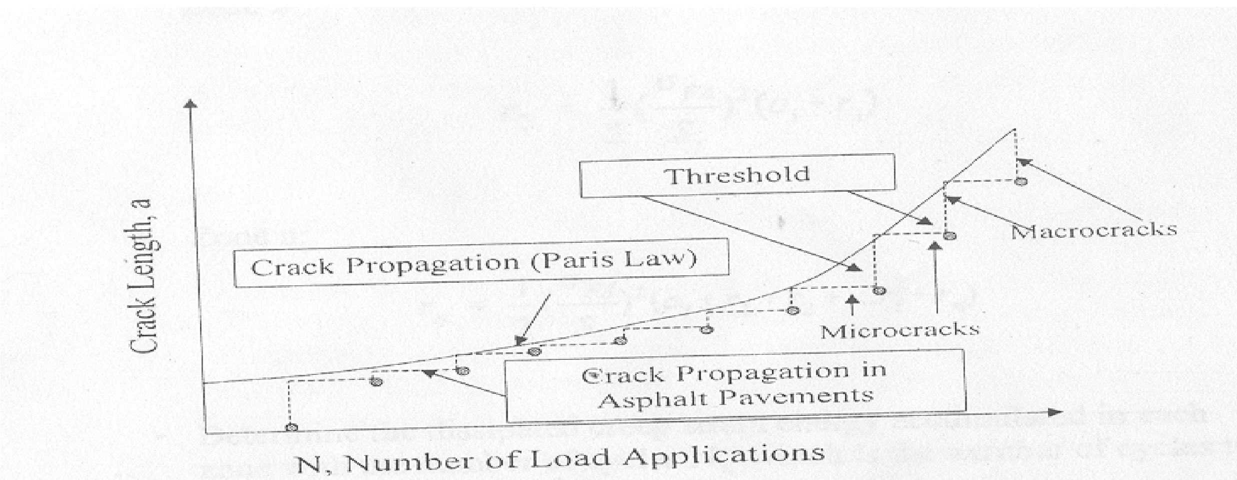


Figure 3. 1: Conceptual Illustration of Crack Growth Law

The final step in establishing a fracture mechanics model for HMA was determining appropriate mixture parameters to use as thresholds. The established thresholds must be able to accurately indicate the cracking performance of asphalt pavements. Two parameters were identified as suitable for use in defining thresholds: Fracture Energy (FE) and Dissipated Creep Strain Energy (DSCE). The appropriate threshold to use is dependent on the loading mode of the mixture test. For a strength test, in which a sample is loaded continuously until failure, fracture energy was deemed appropriate. Quantitatively, fracture energy is defined as the area under the stress-strain curve of a failed specimen. This represents the work required to fracture a specimen for a given volume. For a test in which the sample is subjected to a cyclic load, the DSCE is the appropriate threshold parameter to use. DSCE is represented by the area under the stress strain curve from one loading and unloading cycle. The area calculation is bound by the difference between initial strain and permanent strain realized from the load cycle and the associated stresses. Qualitatively, this is defined as the absorbed energy that damages the specimen. The relationship between Fracture Energy and DSCE is given in the figure below [21].

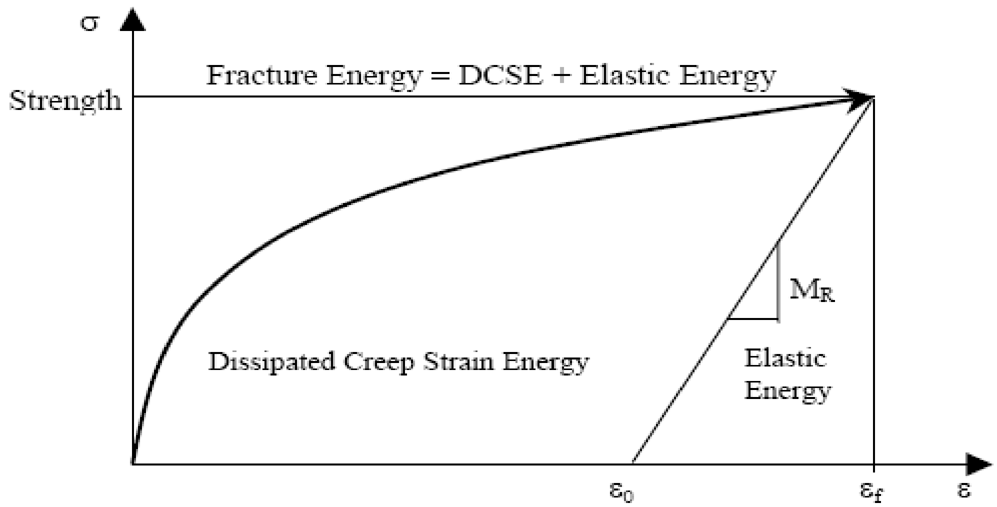


Figure 3. 2: Conceptual Diagram to Explain FE and DSCE

The development of a crack growth law in studies by Roque and Birgisson [21, 28] was predicated on the use of DSCE as measured by a cyclic loading test. The use of the cyclic loading test was chosen because it better represents loading conditions experienced by pavements in the field. Furthermore, the use of DSCE allows for consideration of discontinuous crack growth and the effects of healing associated with asphalt mixtures. The crack growth law developed using DSCE was used to verify the HMA Fracture Mechanics Model developed as part of their study. Fracture energy was cited as a viable substitute for DSCE because comparison of DSCE test results from cyclic loading and theoretical DSCE results for a given level of permanent strain showed a 1:1 relationship that correlated strongly. Therefore, fracture energy is also a viable parameter to characterize the crack performance of pavements.

3.2.2.1. Use of the HMA Fracture Mechanics Model to Evaluate Moisture Susceptibility in HMA

The fracture mechanics concepts presented above were used to define a performance based specification criterion, the energy ratio (ER) as a parameter to measure the fracture resistance of mixtures. The ER was developed based on investigations of 36 field pavement sections of known crack performance in Florida. The ER can be calculated using the Equation 3.1 [6]:

(Eq. 3.1) $ER = DCSE_f / DCSEmin = a * DCSE_f / m^{2.98} * D1$

where: $DSCE_f$: measured dissipated strain energy
DSCEmin = Min. dissipated strain energy for adequate cracking performance
D1 and m are creep parameters
$a = 0.0299\sigma^{-3.1} * (6.63\text{-}St)+2.46E\text{-}8$

in which: σ = tensile stress in asphalt layer
St = tensile strength of asphalt sample

Research cited by Birgisson, states that the minimum DSCE for adequate cracking performance could be determined for all mixtures used [6]. With this definition a performance based law could be developed in which a minimum required ER was defined for various traffic levels. The following minimum ER's were determined for different ESALS of loading:

Table 3. 5: Definition of ER_{min} for Different Levels of Traffic

ESALS (Million)	ER_{min}
3	1.1
10	1.3
30	1.7

It has been well established that moisture damage severely weakens the fracture resistance of asphalt mixtures. The performance based criterion described above can be used to evaluate the moisture susceptibility for a mixture given a certain level of traffic. If the conditioned sample demonstrates an energy ratio that exceeds ER_{min}, the mix can be deemed acceptable without anti-stripping agent modification. This concept was tested to verify the hypothesis established by Birgisson in [6]. Experimental results using this method were consistent with conceptual ideas in terms of both the effects of moisture conditioning and the effects of anti-stripping agent. In terms of moisture conditioning a drastic reduction in ER was shown in samples made with aggregate known to strip. Furthermore, there was no change in ER for aggregates known to not be susceptible to stripping. The results for the anti-stripping mixtures were also conceptually sound. In both the cases of conditioned and unconditioned samples, the use of anti-stripping agent resulted in an increase in ER. This behavior is expected because the anti-stripping agent allows the asphalt to bond well to the aggregate. For specific data and plots please refer to the paper by Birgisson [6].

The performance based criterion established in the paper by Birggison [6] is a significant upgrade from simply using the tensile strength of the mixture. Although the relation was empirically developed, using pavement performance data to determine the constants in equation 3.1, the concepts driving the performance based criterion are very useful. This criterion allows the evaluation of moisture damage to go from being dependent on one parameter, to being dependent on many more parameters pertinent to pavement performance. The performance based criterion considers the following parameters:

- Traffic: ER_{min} specific values can be adjusted to region.

- Pavement structure: σ – tensile stress in asphalt pavement layer, can be modified to account for different pavement structures or different seasonal effects
- Material Parameters: Dynamic modulus, creep, tensile strength.

The consideration of these parameters and the consistency found with conceptual data make the ER performance fracture criterion a promising starting point for evaluating both the effects of moisture damage and the fracture properties of asphalt mixtures. Furthermore, the ER is able to evaluate the effects of moisture damage, independent of conditioning procedures [6]. This allows the ER performance criterion to be used to evaluate the effects of various conditioning procedures on mixes.

3.2.3: Mechanical Testing Approach Used in this Study

The fracture mechanics framework is promising; however it represents a possible long term solution to a problem that is very real today. Currently the TSR and all of its inherent flaws are used to quantify moisture damage in asphalt mixtures. The objectives of this study were to investigate a non-mechanical test to serve as a screening test for the TSR and to investigate more advanced testing parameters or methods to improve or replace the TSR. This serves as a first step towards the development of more advanced testing methods that are better able to correlate to field performance. Furthermore, fracture energy is a much simpler parameter to measure than those specified in the energy ratio approach, this study allows for investigation into the ability to obtain relevant information regarding moisture damage by this simpler means. This section focuses on the mechanical tests performed in this study and how they were used to obtain the tensile strength and fracture energy of the asphalt mixtures tested.

All mechanical mixture testing in this study was conducted using strength testing in indirect tension. Samples prepared were conditioned using ASTM D4867 procedures, and cut into 2 inch (50.8 mm) thick discs. A detailed outline of the procedures used will be given later in the chapter. Samples were cut to allow for smooth surfaces to mount LVDT's on the sample in the horizontal and vertical directions to obtain strain measurements throughout the testing. All testing was performed at 10º C and the crosshead placed load on the sample at a rate of 50 mm/min. Figures 3.3 and 3.4 show an instrumented sample that is ready for testing.

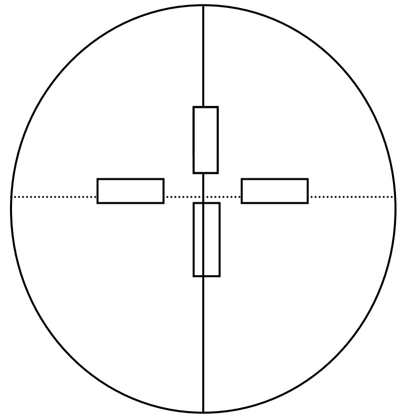


Figure 3. 3: IDT Sample Prior to Testing	**Figure 3. 4: Cross-Section of IDT Sample**

Lab View Testing Software was used to record the load and horizontal and vertical deformations over the duration of the test. These readings were converted to stress and strain measurements that allowed for the plotting of the stress-strain curves for each mixture.

Load was used to calculate horizontal tensile stress (σ_{xy}) by the Equation 3.2 [19]:

(Eq. 3.2) $$\sigma_{xy} = \frac{2P}{(\pi dt)}$$

Where:

P = Load
d = Diameter of HMA sample
t = Thickness of HMA sample

Horizontal and vertical deflections on each side of the sample were calculated by subtracting the initial reading of the LVDT before loading from the LVDT reading at a given point. The horizontal and vertical deflection readings were then averaged. These deflections were used to calculate the horizontal tensile center strain (ε_{xx}) using the Equation 3.3 [15]:

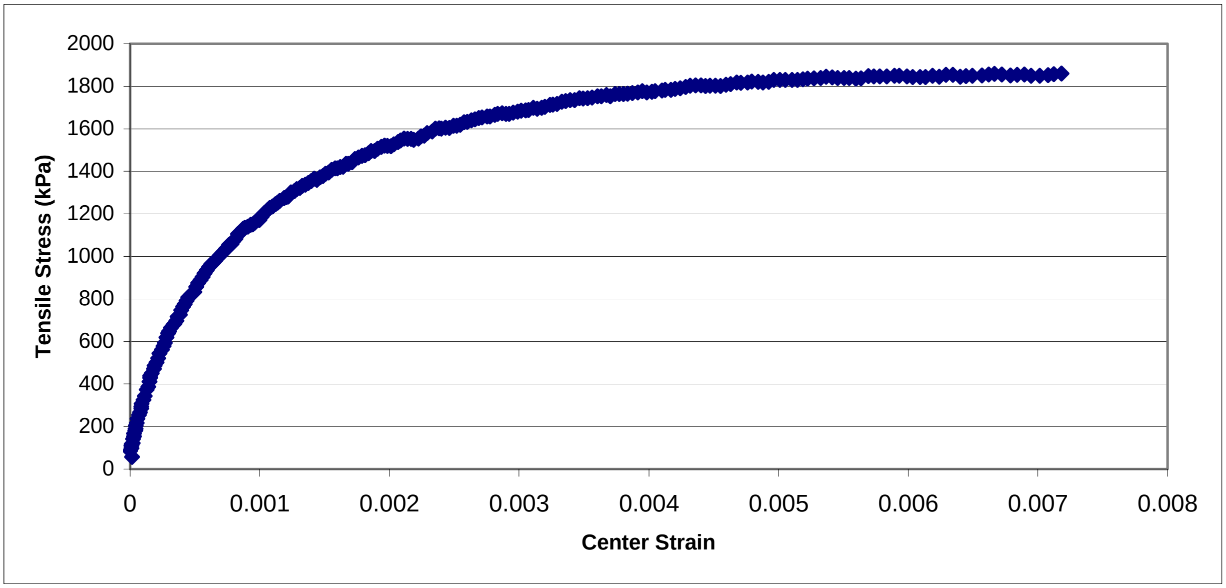
(Eq. 3.3) $$\varepsilon_{xx} = \delta_{xx} * \frac{(2*(1+3\upsilon))}{(d*(a+b\upsilon)*\pi)}$$

Where:

δ_{xx} = average horizontal deformation
υ = Poisson's Ratio (assumed to be 0.32 [10])
a,b,d = Integration constants that are dependent on specimen geometry.

The center strain and stress were plotted to give the stress-strain curve for each mix tested. An example of a plot is given in Figure 3.5:

Figure 3. 5: Stress vs. Strain for HMA Mixture Tested in Indirect Tension



The stress-strain plots were used to obtain the tensile stress at failure and fracture energy for each specimen. Birgisson et al defined the fracture energy as the area under the stress strain curve [21]. For calculation purposes this area was calculated as a sum of the area between individual data points given by the testing software. This statement is summarized Equation 3.4:

(Eq. 3.4) Fracture Energy (FE) (in Pa) = $\Sigma((\varepsilon_{xx(n+1)} - \varepsilon_{xx(n)})^{*} \sigma_{xy(n)})/1000$ For n = 1 to final data point

The final step in the analysis was visual examination of individual test specimens to ensure that the specimen failed in tension. A tensile failure is characterized by a diametetral vertical crack along the surface of the specimen. Some specimens failed in the compression failure zone, by shoving of the material directly under the applied load. Samples exhibiting compressive failure were identified and disregarded in the data analysis portion of this project. Of the 96 samples tested, 14 exhibited compressive failures, thus 15% of the samples tested were deemed inappropriate for further analysis. Figures 3.6 and 3.7 show the difference between a compressive and tensile failure.

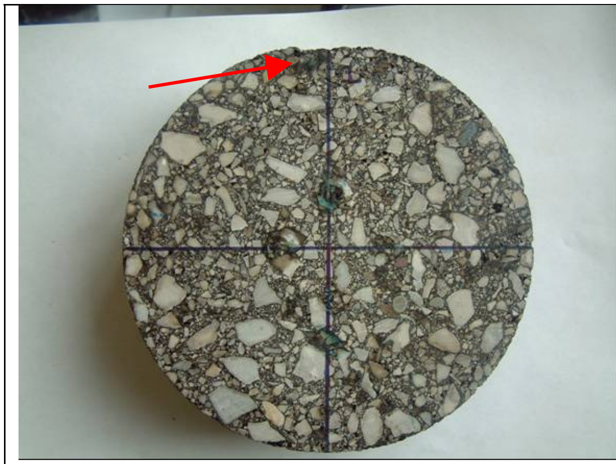
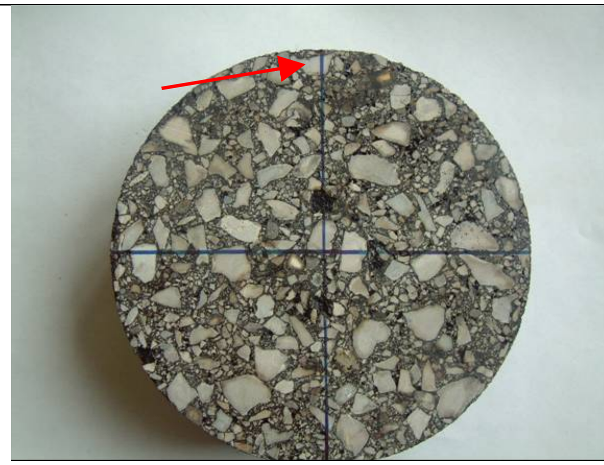
	

Figure 3. 6: HMA Sample Exhibiting Compressive Failure	**Figure 3. 7**: HMA Sample Exhibiting Tensile Failure

3.2.3.1: Sample Preparation Procedures

All samples were prepared and conditioned as specified by ASTM D4867: Effect of Moisture on Asphalt Concrete Paving Mixtures [1]. The following is a step by step procedure of the sample preparation and conditioning:

1. Combine aggregates at proportions specified in mix design. Prepare materials for 6 compacted samples (4700g) and three rice samples (1500g)
2. Heat aggregates and asphalt binder to mixing temperature (135ºC). Any mixes using anti-strip additive contained asphalt binders with 0.5% of anti-strip added by weight.
3. Combine aggregates and asphalt binder at optimum binder content. Mix by hand until aggregates are coated.
4. Cover asphalt mixtures with aluminum foil and allow to cool for 2 hours.
5. Place all covered mixes back in the oven at 60ºC (140ºF) for 16 hours for long term aging.
6. Uncover and reheat samples to mixing/compaction temperature for two hours. After one hour stir each mix.
7. Prepare rice samples. Compact all other samples to 7% +/- 1% air voids.
8. Determine bulk density of compacted samples and maximum density of rice samples using the Corelok Machine.
9. Perform volumetric calculations to ensure that samples are within specification.
10. Saturate all samples to 55%-85% saturation.
11. Select three samples for moisture conditioning. Place these samples in a water bath at 60ºC (140ºF) for 24 hours.
12. Cut all samples into discs of approximately 50.8 mm (2 inch) thickness.
13. Allow samples to dry and prepare them for mounting of the Linear Variable Transducers (LVDTs).
14. Place samples in the environmental chamber of testing machine at 10ºC and allow two hours for samples to reach 10ºC. Verify that mixes have reached appropriate

temperature by measuring temperature in the center of a dummy sample place in the chamber.

15. Conduct indirect tensile test and examine individual samples for mode of failure.

Measures were taken to reduce variability of test results by controlling air voids and level of saturation to lower tolerances than required in the specification.

As stated previously, all IDT testing performed used strength testing, allowing for the one test to provide both the tensile strength and fracture energy of each mixture. The effects of moisture were quantified by defining the two ratios in Equations 3.5 and 3.6:

- (Eq. 3.5) Tensile Strength Ratio: $\frac{\text{Average Conditioned Tensile Strength}}{\text{Average Unconditioned Tensile Strength}}$
- (Eq. 3.6) Fracture Energy Ratio: $\frac{\text{Average Conditioned Fracture Energy}}{\text{Average Unconditioned Fracture Energy}}$

All test results and corresponding analysis will be presented in Chapter 4.

3.3: Non-Mechanical Testing Approach – The Stripping Test

The stripping test procedure used in this study is a modified version of a procedure developed by the Quebec Department of Transportation to evaluate the adhesion characteristics of different asphalts to aggregates using loose mixtures [15]. Loose mixture tests are suited best for comparison between mixtures in terms of affinity of asphalt to aggregates, strength of adhesion, and stripping. These tests are useful to identify extremely poor mixes that exhibit excessive moisture damage [25]. The original test involved aggregates of the same size coated with asphalt binder. The mix is then conditioned in water at constant temperature and agitated for periods of time in a gyratory shaker bath. After filtration, it is washed with water and dried. The residual percentage of asphalt coating aggregates and the percent of aggregate stripped are evaluated qualitatively [15]. The agitation of the mix will allow water to better compete with asphalt to adhere to aggregates, simulating water circulation in pavement mixtures, and potentially causing stripping by both the displacement and detachment mechanisms, as previously discussed. An example of stripping test results for mixes using neat binder and binder modified with anti-stripping agent is shown in Figure 3.8.

Figure 3. 8: Examples of Stripped Aggregates after Water Conditioning Before and After Using Anti-Stripping Additive

The results clearly show the difference in performance of the two mixes as the mix using the original binder experienced considerably more stripping than the mix with the additive. Based on the above results, it was decided that this procedure was a viable candidate for further investigation and development as a non-mechanical test to identify combinations of asphalt and aggregate that are susceptible to stripping.

The objective of the non-mechanical test as defined by the scope of this project is to provide a quick, efficient test that could identify mixtures prone to stripping and serve as a screening test for TSR testing. The original Quebec procedure had to be modified to accommodate the practicality requirement and to provide a quantifiable measure that could be written into specifications. In terms of practicality, the current Quebec method, which uses a single sized aggregate as a test sample is not acceptable because it requires materials preparation above and beyond current practices. Instead, it was decided to perform the test on loose mixture samples of material that was used in the actual mix design. This modification also allows for consideration of materials actually being used in the field, providing an opportunity to quantify the stripping of the entire asphalt mixture, not just a single sized aggregate.

The other main modification to the Quebec Stripping Test procedure was an effort to make it quantitative test. The original test method involves visual inspection of the test sample to qualitatively evaluate the percent of total aggregate stripped. This qualitative measure is similar to other mixture tests in that it gives an output of pass/fail that is awarded subjectively by visual inspection [25]. Efforts were made to modify the stripping test to differentiate it from other tests on loose mixtures by providing an objective, quantifiable result. This result was accomplished by using the objective measure of the loss of materials after washing over a sieve to quantify stripping and loss of fine aggregate adhesion. Loss of materials was calculated using the Equation 3.7:

(Eq. 3.7) $$\%\text{Weight Loss} = \frac{(\text{Total Mass} - \text{Conditioned Mass}) * 100}{\text{Total Mass}}$$

The advantage of using a quantifiable measure to define the loss of materials in a mixture due to stripping and loss of fine aggregate adhesion is that it provides an opportunity for the agency to establish a threshold for TSR testing requirements. Assuming that the stripping test is found to be reliable, a threshold value could be established to identify moisture susceptible mixes. Specifications could be written to waive the TSR testing requirements for mixes exhibiting a mass loss below the threshold, leading to reduced time and monetary investment in moisture sensitivity testing.

3.3.1: Procedures for the Stripping Test

The following is the general procedure for the modified stripping test performed in this study:

1. Place 150 grams of loose mixture into a glass jar. Three 150 gram samples can be tested at the same time.
2. Add approximately 400 g of distilled water to each jar and cover.
3. Fill gyratory shaker bath with water and establish a temperature of 60ºC.
4. Secure samples in gyratory shaker bath.
5. Agitate samples in bath for 24 hours at 60ºC.
6. Remove samples from bath and allow them to cool.
7. Manually separate conditioned loose mixture over a sieve (#8 for coarse mixes, #16 for fine mixes).
8. Wash over the appropriate sieve size with distilled water to ensure all particles passing the sieve size are removed.
9. Place retained loose mixture in a pan of known weight and oven dry for 24 hours at 80ºC.
10. Weigh retained materials after drying to obtain conditioned mass.
11. Calculate %Mass Loss using Formula (4) provided above.

Extra measures were taken in performing the stripping test by ensuring that all samples were approximately 150 grams in weight and that the temperature and conditioning time remained consistent for all tests performed. Variability inherent to the manual separation and washing over the appropriate sieve size was also minimized by using a single operator to run the test and ensuring that all samples were separated and washed using similar methods.

Further consideration was given to the potential of the %Mass Loss results obtained using the procedure above being confounded with materials being lost because of their size, not because of moisture damage. To address this issue 150 grams of all loose mixtures were hand sieved over the appropriate sieve size (#8 for coarse mixes, #16 for fine mixes) and the amount retained was measured. This measurement was used to correct original calculations. The original stripping test results and the effects of this modification will be presented in Chapter 4.

3.4: Asphalt Binder Testing to Relate to Mechanical Testing Results

Moisture damage in asphalt mixtures is characterized as failure of the asphalt's adhesive bonds to the aggregate surface or loss of cohesion between other asphalt molecules. Regardless of failure mode, asphalt binder properties must be considered in evaluating the moisture susceptibility of asphalt mixtures. The properties selected for further investigation in this study were the Superpave rutting parameter, $G^*/\sin\delta$, the Superpave fatigue parameter $G^* \sin\delta$, and the binder cohesion as measured by the Tack Test. Consideration of these properties and their effects on tensile strength and fracture energy of both conditioned and unconditioned samples allow for determination of the asphalt binder's overall effect on mixture performance. Previous work performed by Kanitpong and Bahia found that binder properties have an effect on both the unconditioned and conditioned tensile strength of asphalt mixtures. Specifically, the research found that for constant aggregate structure there was a strong correlation between cohesion and tensile strength for unconditioned mixtures and a statistically derived function of cohesion and adhesion for conditioned mixtures. Further work by Kanitpong in [12] also shows strong correlation between binder adhesion and cohesion and mixture performance in the Hamburg Wheel Tracking Test, another test commonly used to predict moisture damage in asphalt mixtures. In summary, the results of past research suggest that a relationship exists between asphalt binder adhesion and cohesion and asphalt mixture performance. This study will examine if the trend in binder properties remains applicable to the mechanical testing results over a wide range of aggregate types and gradations. Furthermore, the correlation of indirect tensile test results in terms of both strength and fracture energy to the Superpave rutting and fatigue parameters will be investigated. The remainder of this section is dedicated to description of the procedures used to measure the Superpave binder rutting and fatigue parameters and the tack test.

3.4.1: Measurement of the Superpave Binder Rutting and Fatigue Parameters

The Superpave rutting and fatigue parameters both serve as a measure of the ability of the binder to minimize energy dissipated due to damage caused by loading. These parameters are derived from the properties of complex modulus (G^*) and phase angle (δ). Where the complex modulus indicates the ability of the asphalt to resist permanent deformation and the phase angle defines the non-elastic response of the binder, as shown by the time lag in the loading and response curves of the test samples. All binders used in this study were tested using a Dynamic Shear Rheometer (DSR) at a frequency of 10 radians/second, in accordance with AASHTO TP5 binder specifications. The only deviation from Superpave binder specifications was in the testing temperature. All asphalt binders were tested at a temperature of 10ºC to determine the binder properties present during asphalt mixture testing.

The rutting of asphalt pavements is a result of accumulated permanent deformations under repeated, cyclic traffic loading. Under the assumption that rutting is due inadequate materials in the pavement surface layer and not a weakness in base and sub-grade materials, rutting can be modeled as a stress controlled phenomenon. With each loading cycle, work is done by traffic to deform the pavement layer. This energy is divided into two parts, some is recovered through the elastic response of the hot mix asphalt, and the remaining portion is dissipated through permanent deformation and heat in the surface layer. For a viscoelastic material the work dissipated with each load cycle can be characterized by the following derivation leading to Equation 3.10 [3]:

(Eq. 3.8) $Wc = \pi * \sigma * \varepsilon * \sin \delta$

Furthermore, the following substitution can be made for a stress controlled (σ_0) phenomenon:
(Eq. 3.9) $\varepsilon = \sigma_0/G^*$

Thus, (Eq. 3.10) $Wc = \pi * \sigma_0^2 * (1/(G^*/ \sin \delta))$

Based on this relationship, derived by Bahia and Anderson [3], it is clear that in order to minimize permanent deformation (minimize dissipated energy), an asphalt requires a higher complex modulus or more elasticity (lower δ).

The contribution of the asphalt binder to resistance to fatigue cracking can be conceptually explained in a similar fashion to that of permanent deformation. It has been established that fatigue cracking is the distress that governs pavement failures for thin pavements [3]. Under this assumption, fatigue in the pavement layer can be characterized as a strain controlled phenomena caused by large deformations under traffic loading. These deformations are caused by inadequate pavement response due to the inability of the thin pavement layer and underlying base materials to withstand the traffic loadings. The ability of the pavement layer to dissipate energy is characterized by the same response equation for a viscoelastic material as presented in Equation 3.10, however in a strain controlled situation (ε_0), strain is related to applied stress and complex modulus by the following:

(Eq. 3.11) $\sigma = \varepsilon_0 * G^*$

Thus, (Eq. 3.12) $Wc = \pi * \varepsilon_0^2 * (G^* \sin \delta))$

This relationship shows that in order to minimize fatigue cracking, asphalts with lower complex modulus and/or lower values of δ should be selected. Conceptually, the use of an asphalt with a lower complex modulus has the ability to deform more without developing large stresses and more elastic asphalts (lower δ) have the ability for quicker recovery after loading. Preference of a lower complex modulus to resist fatigue cracking is in contradiction to desired material properties for resistance to permanent deformation. This contradiction shows that selection and specification of asphalt binders for pavements requires consideration of a variety of pavement responses to traffic loading and cannot be accomplished by simply specifying the most elastic binder with the highest modulus.

The conceptual definitions of the Superpave asphalt binder parameters for rutting and fatigue are by no means comprehensive. However, in order to use these parameters to compare to the results of the mixture testing in this study and hypothesize in regards to any possible relationships, it is necessary to demonstrate a basic understanding of how these parameters were developed. All mixture testing in this study was performed using strength tests, so consideration of the binder's ability to dissipate energy over loading cycles is not necessary. However, if the binder has a significant effect on mixture performance in indirect tension, mixture results should show correlation with both of these parameters due to its contribution to the visco-elasticity of the asphalt mixture.

3.4.2: Measurement of Binder Cohesion using the Tack Test

The integrity of an asphalt mixture is dependent on the strength of the aggregate, the adhesion of the asphalt to the aggregate surface, and the cohesion within the asphalt. Failure in any mode or frequency occurs when the applied load exceeds the strength of the weakest constituent of the asphalt mixture. Previous work by Kanitpong [11] and others suggests that the probability of a mixture failure due to loss of cohesion is much greater than failure due to loss of adhesion provided the adhesive bond between the asphalt and aggregate surface has not been compromised. Therefore, asphalt binder cohesion should correlate with both tensile strength and fracture energy test results on the mixtures used in this study, especially for unconditioned specimens. To investigate this correlation the cohesion of the asphalts used in this study was measured using the Tack Test as developed by Paar Physica and the University of Wisconsin – Madison [12]. This test involves the use of the DSR to measure the force applied over time to pull apart two surfaces adhered by asphalt. The head of the DSR is displaced at a constant rate (0.01 mm/s) and the elapsed time and force acting on the asphalt is measured. To maintain consistency with mechanical testing procedures, all tests were conducted at a temperature of 10º C. A schematic of the test is provided in Figure 3.9 [12]:

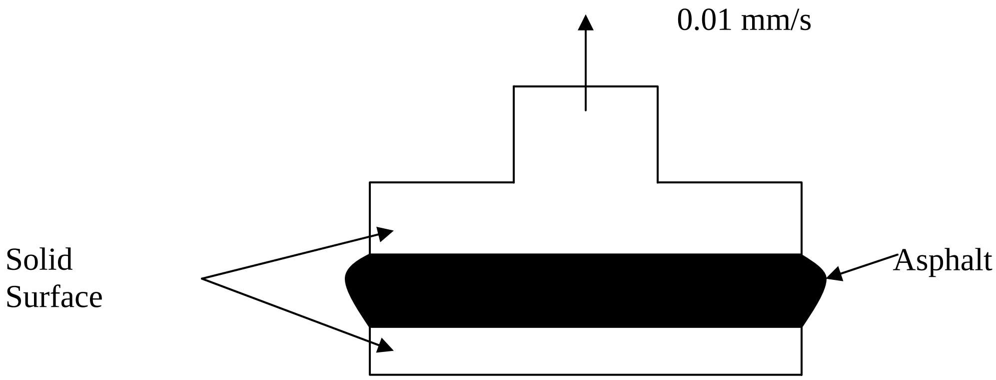


Figure 3. 9: Schematic of the Tack Test

The cohesion of the asphalt is quantified by the tack factor (C_T), which is defined as the area under the force vs. time curve. Conceptually the tack factor relates to the energy required to overcome the cohesion of the asphalt and separate the two solid surfaces. In this study, this parameter was estimated using the Equation 3.13:

(Eq. 3.13) $C_T = \Sigma((F_{(n+1)} + F_{(n)})/2)*(t_2-t_1))$ For n = 1 to final data point

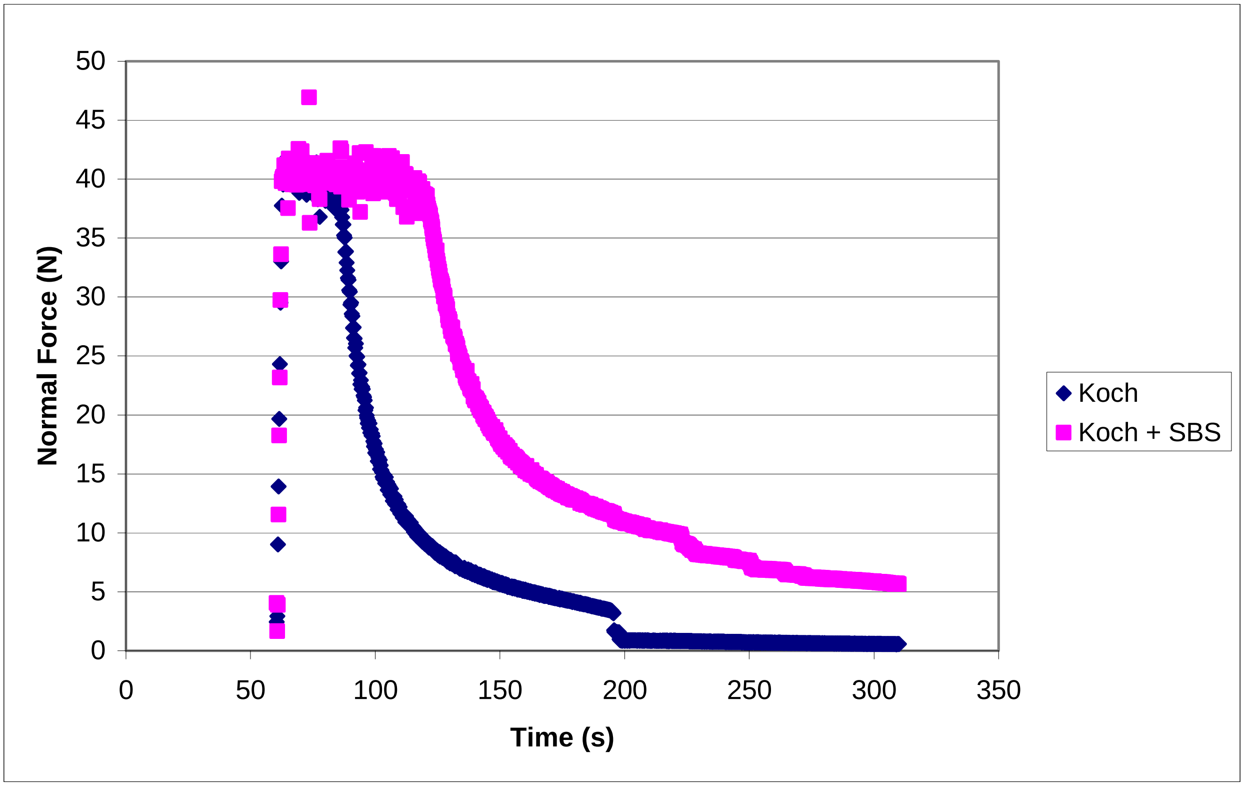
Where:

C_T = Tack Factor
F = Normal Force (N)
T = Time (s)

In order for this test procedure to be valid, it must be sensitive to the effects of cohesion on the addition of an additive to the asphalt. To evaluate the sensitivity of the test at the temperature used in this study, plots of a neat binder were compared to an SBS modified binder.

It is predicted that the SBS polymer should significantly increase the tack factor of the asphalt binder. This plot is shown in Figure 3.10.

Figure 3. 10: Comparison of Cohesion of Neat and SBS Modified Binder



As expected, the SBS modified binder displays significantly higher tack factor, as characterized by the larger area under the curve. Based on these results, it can be concluded that the Tack Test is valid at the testing temperature of 10°C and therefore can be used to quantify the cohesion of the asphalt binder in the mixtures tested. This plot is meant to be conceptual, specific values of C_T and plots for all the other binders used in this study will be provided in Chapter 4.

Chapter Four: Test Results and Data Analysis

The results of the previously described test methods and a statistical analysis will be presented in this chapter. The test results will be presented to show each test's ability to identify moisture damage due to sample conditioning and the effects of the addition of an anti-stripping agent or change in aggregate type or gradation. Furthermore, multiple replicates for each mix were tested, allowing for the examination of the reliability of each test method. The results of the mechanical testing will then be compared to the non-mechanical test results to evaluate the possibility of incorporating a non mechanical screening test into current WisDOT asphalt mixture testing specifications. The statistical method of analysis of variance will be used to further examine and verify that factors found pertinent to moisture susceptibility of asphalt mixtures are consistent with common engineering knowledge. Key asphalt binder and mixture properties will also be measured and statistically analyzed to identify binder properties that are significant to asphalt mixture behavior.

4.1: Mechanical Testing Results

The fracture energy and tensile strength tests conducted in this study served two main purposes: to evaluate their individual ability to accurately predict moisture damage and to examine their correlation with the stripping to test to determine if the stripping test was a viable alternative for evaluating moisture damage in asphalt mixes. This section will present the air voids present in individual mixes and the results of the Tensile Strength and Fracture Energy tests individually. Analysis of the results will focus on the ability of the test methods to identify various factors that affect moisture susceptibility of asphalt mixtures and their repeatability. Results of all tests will be provided in Tables 4.1A - 4.1C of the Appendix. Subsequent sections will address the individual results of the stripping test and how they correlate with the tensile strength and fracture energy ratios.

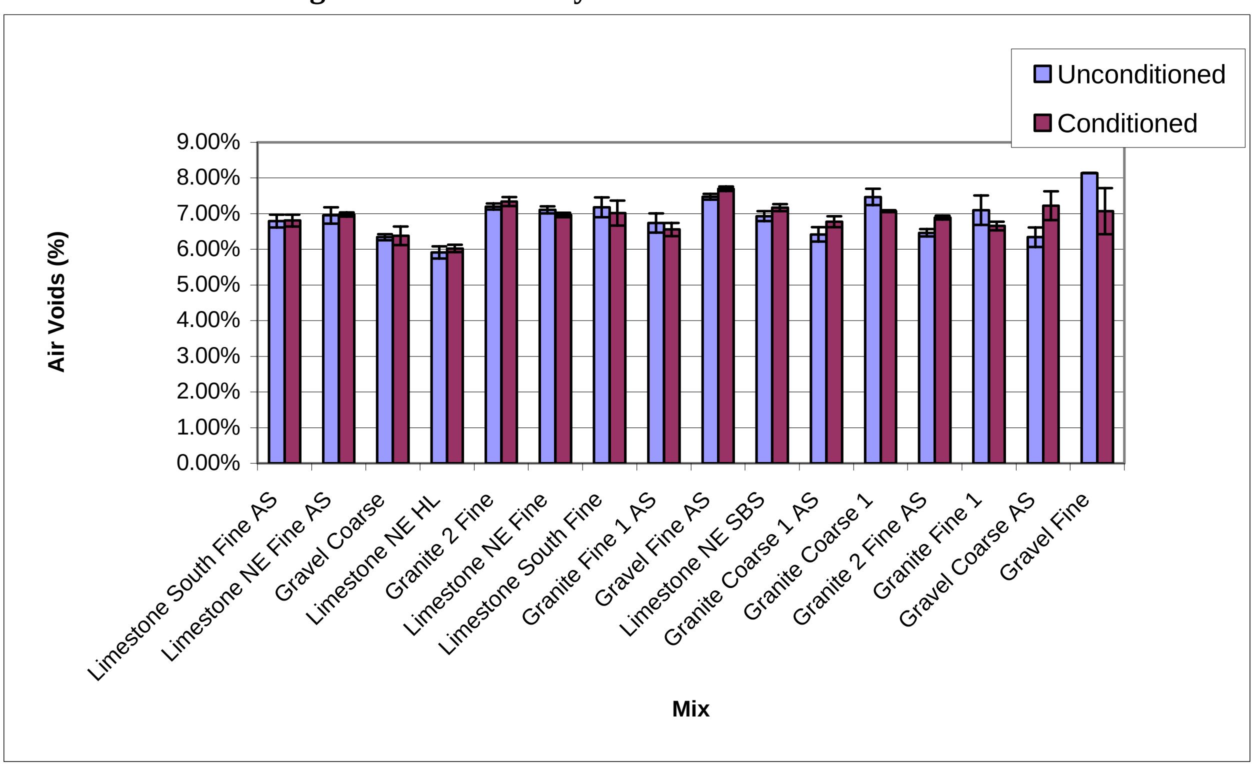
4.1.1: Summary of Air Void Levels and Variation Within Each Mix

Conceptual knowledge verified by the work of Dukatz detailed in Chapter 3 identified the importance of controlling the variation of air voids to accurately assess the potential for moisture damage in asphalt mixes. As previously stated, control of air voids in this study was attempted through reducing the variation within individual mixes. The average air void levels for each mix and their corresponding standard deviations are provided in Table 4.1, the data is also presented as a bar chart with standard deviations reported in the y-error bars for each mix and mixes ordered from smallest to largest difference in air voids between unconditioned and conditioned specimens in Figure 4.1.

Table 4. 1: Summary of Air Void Variation within Mixture Samples

Mix	Average Air Voids - Unconditioned	Standard Deviation	Average Air Voids - Conditioned	Standard Deviation	Difference (UC - C)
Granite Fine 1	7.10%	0.41%	6.66%	0.12%	0.44%
Granite Coarse 1	7.47%	0.23%	7.07%	0.03%	0.40%
Granite 2 Fine	7.20%	0.08%	7.34%	0.13%	-0.14%
Granite Fine 1 AS	6.74%	0.27%	6.56%	0.19%	0.18%
Granite Coarse 1 AS	6.42%	0.20%	6.77%	0.15%	-0.35%
Granite 2 Fine AS	6.46%	0.10%	6.89%	0.06%	-0.43%
Gravel Fine	8.14%	0%	7.07%	0.65%	1.07%
Gravel Fine AS	7.47%	0.08%	7.70%	0.06%	-0.22%
Gravel Coarse	6.34%	0.08%	6.38%	0.26%	-0.04%
Gravel Coarse AS	6.34%	0.27%	7.22%	0.41%	-0.88%
Limestone NE Fine	7.11%	0.10%	6.96%	0.06%	0.14%
Limestone NE Fine AS	6.95%	0.23%	6.98%	0.06%	-0.03%
Limestone NE HL	5.91%	0.17%	6.03%	0.11%	-0.12%
Limestone NE SBS	6.93%	0.14%	7.17%	0.10%	-0.24%
Limestone South Fine	7.18%	0.28%	7.02%	0.35%	0.16%
Limestone South Fine AS	6.79%	0.18%	6.81%	0.17%	-0.01%

Figure 4. 1: Summary of Air Voids for Each Mix



Review of Table 4.1 and Figure 4.1 shows that both the variability within prepared samples and between unconditioned and conditioned samples is much less than the 2% range prescribed by ASTM D4867 [1]. The variability within prepared samples is measured using the standard deviation of the air voids for a given mix. This yields a maximum standard deviation of 0.41% for the unconditioned mixes (Granite Fine 1) and 0.65% for conditioned mixes (Gravel Fine). Control of the air voids between unconditioned and conditioned samples for a given mix is also a key factor in evaluating the moisture sensitivity of a mix. Large differences between conditioned and unconditioned samples could result in falsely classifying a mix as moisture resistant or moisture susceptible. The difference between unconditioned and conditioned samples is shown in both tabular and visual form. Examination of these figures shows that for most mixes the difference between air void levels for unconditioned and conditioned samples is negligible, thus providing a solid means of evaluation of moisture susceptibility. There were only two mixes reported that had a difference larger than 0.5%, Gravel Fine (1.07%) and Gravel Coarse (0.88%). Ideally these large differences could be negated using the method described by Dukatz, however, the lack of data points to adequately characterize the effects of a range of air voids on tensile strength prevents the use of such an analysis. Furthermore, analysis of the tensile strength and fracture energy results for these mixes presented in subsequent sections of this chapter do not show deviations from expected behavior based on test results from similar mixes.

4.1.2: Tensile Strength Testing Results

The tensile strength tests conducted in this study displayed quality results in terms of expected outcomes and reliability. All mixes tested exhibited losses in tensile strength due to moisture conditioning. Furthermore, all tests were able to identify changes in other mix properties pertinent to moisture damage. Specifically, test results were able to differentiate between mixes using aggregates that are known to cause stripping and those that are known to be resistant to moisture damage. The Granite 1 and Limestone NE mixes were expected to be moisture susceptible, conversely the Gravel and Limestone South mixes were not. This expected trend was verified by the TSR testing results showing reduced conditioned tensile strength in the mixes expected to exhibit moisture damage. Furthermore, the effect of anti-stripping additive was correctly identified by a gain in conditioned tensile strength of previously moisture susceptible mixes. Results are shown in Figure 4.2 by the comparison of conditioned to unconditioned samples for all mixes. The reliability of the tensile strength test is realized by the small standard deviations between test results for individual sets of mixes. The mixes are organized from highest to lowest Tensile Strength Ratio. The results are also provided in a table along with some general mix properties and basic statistics in order to reinforce what is presented in the bar chart and to serve as a means of comparison to the precision statements in ASTM D 4867.

Test results also show that prediction of moisture damage is specific to the aggregate source or mix design, not necessarily the aggregate type. Two limestone mixes from different sources (Limestone NE and Limestone South) were tested; each showed very different results in terms of conditioned tensile strength and the contribution of anti-stripping agent to conditioned tensile strength. Furthermore, two granite mixes from the same source, but different mix designs were used (Granite 1 and Granite 2). The gradation of the mixes was very similar, with the only differences being in binder source and the use of RAP. The mix design using RAP showed significantly higher unconditioned and conditioned tensile strengths and less moisture susceptibility.

Figure 4. 2: Comparison of Tensile Strength for Conditioned and Unconditioned Samples

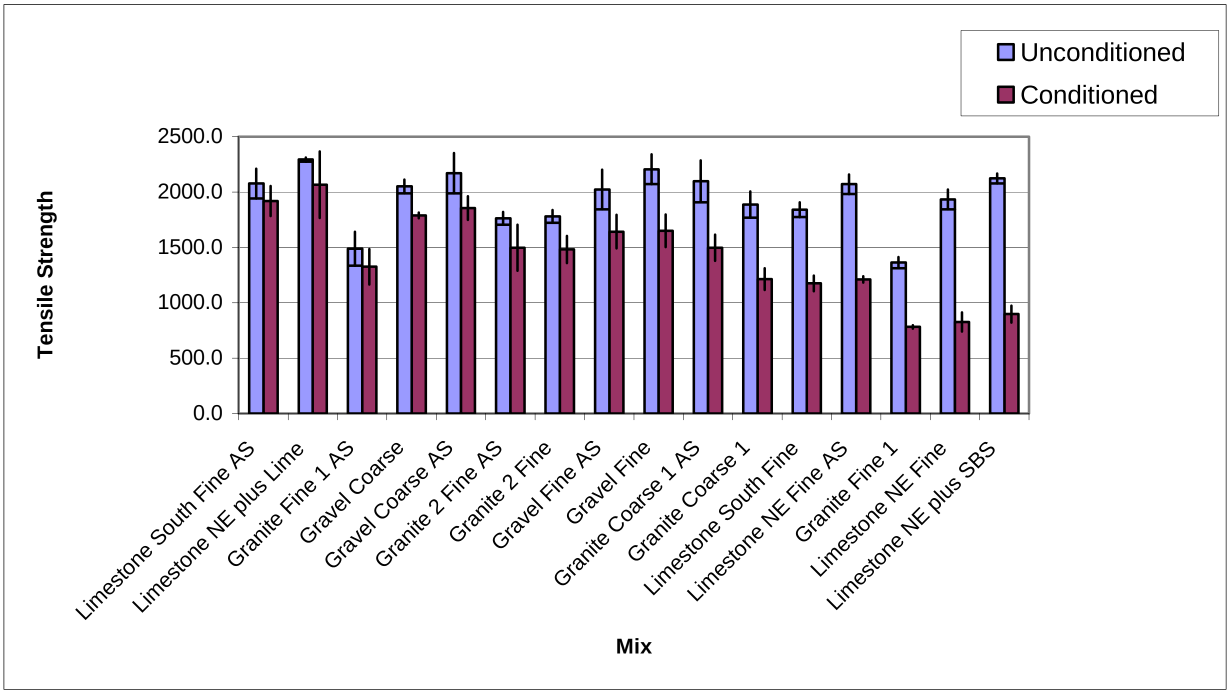


Table 4. 2: Summary of Tensile Strength Testing Results

				Air Voids		Tensile Strength						
Aggregate Type	Gradation/Mix	Fine Aggregate Contribution	Natural Sand Proportion	Average	Standard Deviaton	Average Strength Reduction (kPa)	Average TSR	TSR Min	TSR Max	TSR Range	Standard Deviation	ASTM D4867 Precision
Granite	Fine Mix 1	64.65%	21.81%	6.92%	0.38%	580.0	0.57	0.54	0.60	0.06	2%	PASS
	Fine Mix 1 AS			7.31%	0.27%	161.9	0.89	0.74	1.11	0.37	12%	FAIL
	Coarse Mix 1	49%	15.28%	7.29%	0.13%	672.8	0.64	0.58	0.73	0.15	6%	PASS
	Coarse Mix 1 AS			6.65%	0.23%	600.1	0.71	0.63	0.83	0.20	7%	PASS
	Fine Mix 2	66.05%	28.46%	6.63%	0.24%	298.6	0.83	0.74	0.91	0.17	6%	PASS
	Fine Mix 2 AS			6.63%	0.25%	266.5	0.85	0.74	0.96	0.22	9%	FAIL
Gravel	Fine	68.19%	40.10%	7.50%	0.74%	555.7	0.75	0.67	0.86	0.19	7%	PASS
	Fine AS			7.59%	0.14%	380.2	0.81	0.72	0.96	0.24	9%	FAIL
	Coarse	43.16%	40.87%	6.37%	0.19%	262.1	0.87	0.84	0.90	0.06	3%	PASS
	Coarse AS			6.78%	0.57%	314.6	0.86	0.76	0.96	0.20	7%	PASS
Limestone NE	Fine	71.57%	36.59%	7.04%	0.11%	1106.9	0.43	0.36	0.48	0.12	4%	PASS
	Fine AS			6.97%	0.12%	859.4	0.59	0.55	0.62	0.06	2%	PASS
	Fine Hydrated Lime			5.97%	0.13%	226.6	0.90	0.80	1.00	0.20	11%	FAIL
	Fine SBS			7.05%	0.17%	1224.9	0.42	0.38	0.47	0.09	3%	PASS
Limestone South	Fine	65.23%	42.95%	7.10%	0.30%	665.5	0.64	0.58	0.69	0.11	4%	PASS
	Fine AS			6.80%	0.16%	158.2	0.92	0.80	1.04	0.24	8%	FAIL
									Average Range	0.17	Percent Passing	69%

Data presented in Table 4.2 serves as a supplement to the bar chart in Figure 4.1, by providing a detailed summary of both the mix and testing results to further investigate possible sources of variability. The mix properties in the table include the fine portion of the job mix formula (passing No. 4), the proportion of natural sand in the job mixed fine aggregate, the %air voids and their variation for the test samples. Consideration of all these mix properties and their effects on the values and variation within tensile strength results is difficult using solely the table above. This issue will be addressed in more detail through regression analysis later in this chapter.

The table also allows for a more detailed analysis of the variability within the tensile strength ratio and comparison with the precision statements in ASTM D4867. Maximum and minimum TSR values were derived using the maximum and minimum tensile strength values of the conditioned and unconditioned samples. This analysis provides interesting results in terms of the large variation in range of TSR and standard deviation between mixes. The table above shows that TSR values of specific mixes had ranges of anywhere from 6% to 37%, with an average range of 17%. This large range reinforces the conclusions of previous work cited in this study [2,9] that under the current procedure, there is a certain level of uncertainty that a mix with an average TSR greater than 0.7 is moisture resistant. Examination of the standard deviation within test results and comparison to the precision statement in ASTM D4867 further verifies this claim. ASTM D4867 requires that the standard deviation of TSR measurements between labs be less than 8% [1]. This requirement was extended to examination of the standard deviation of the TSR values used in this study. The standard deviation was calculated from all possible combinations of TSR values from the test results for an individual mix and compared to the 8% threshold established in ASTM D4867. Based on this requirement, 70% of the mixes were deemed acceptable in terms of precision, of the passing mixes three, Granite Coarse 1, Granite Coarse 1 AS, and Gravel Fine had ranges that crossed the .70 (.75 for mixes using Anti-strip additive) criterion currently used by WisDOT to identify moisture resistant mixes. A similar summary of the mechanical testing results is available in Table 4.1B of Appendix A, these results could not be compared to the precision statement in ASTM D4867 which limits variation between results to 55 kPa because this variation is based on a testing temperature of 25ºC, whereas tests in this study were all conducted at 10ºC.

4.1.3: Fracture Energy Test Results

The fracture energy test results were similar to tensile strength test results in that both were able to detect moisture damage by displaying a general trend of a decrease in their respective parameters for moisture susceptible mixes. However, the reliability of the fracture energy test results must be questioned. The standard deviations for both unconditioned and conditioned mixes are provided visually in Figure 4.3 and in numerical form in Table 4.3. Figure 4.3 is ordered from largest to smallest Fracture Energy Ratio. The large standard deviations demonstrated by some of the mixtures prohibit the definition of a general trend to define the behavior of all mixtures. Figure 4.3 provides a summary of all fracture energy testing results; it can be observed that mixes identified as moisture susceptible by the TSR tests also realized significant drops in fracture energy due to moisture conditioning. Furthermore, all moisture susceptible mixes showed an increase in the fracture energy of conditioned mixes due to the addition of anti-stripping agent. However, the wide range of measurements taken for individual replicates prevent the definition of the relationship provided as anything more than a general observation.

Fracture Energy testing results had general trends similar to those identified in TSR testing in regards to the dependence of aggregate source and mix design on moisture susceptibility. Mixes from the same aggregate source using a different mix design (Granite 1 and 2) showed different fracture energy results, possibly due to the presence of RAP in the mixture. Furthermore, test results of mixes of the same aggregate type from different sources were consistent with the trends found through TSR testing.

Figure 4. 3: Comparison of Fracture Energy of Conditioned and Unconditioned Samples

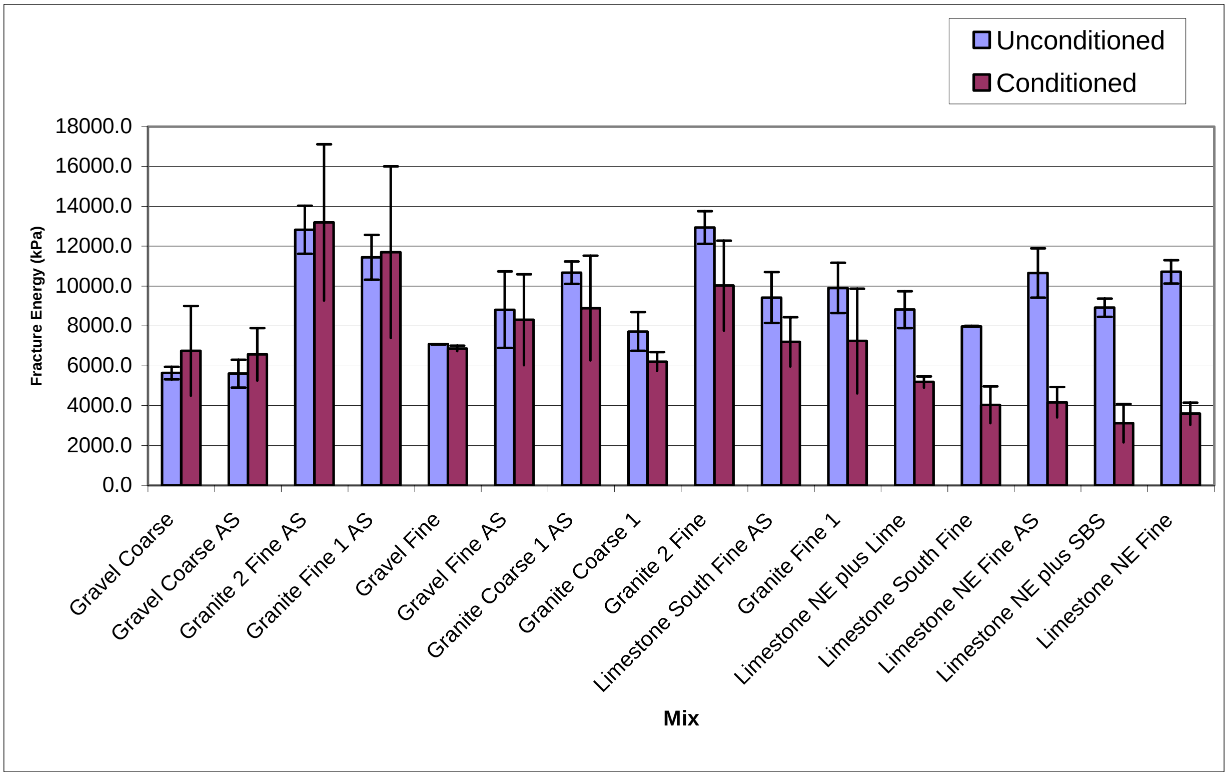


Table 4. 3: Summary of Fracture Energy Testing Results

				Air Voids		Fracture Energy					
Aggregate Type	Gradation/Mix	Fine Aggregate Contribution	Natural Sand Proportion	Average	Standard Deviaton	Average FE Reduction (kPa)	Average FER	FER Min	FER Max	FER Range	Standard Deviation
Granite	Fine Mix 1	64.65%	21.81%	7.10%	0.41%	2657.9	0.73	0.48	1.04	0.56	22%
	Fine Mix 1 AS			6.74%	0.27%	-254.5	1.02	0.62	1.55	0.93	34%
	Coarse Mix 1	49%	15.28%	7.47%	0.23%	1515.5	0.80	0.67	0.95	0.28	10%
	Coarse Mix 1 AS			6.42%	0.20%	1774.9	0.83	0.61	1.15	0.54	22%
	Fine Mix 2	66.05%	28.46%	7.20%	0.08%	2913.1	0.77	0.63	1.02	0.39	16%
	Fine Mix 2 AS			6.46%	0.10%	-376.4	1.03	0.74	1.35	0.62	25%
Gravel	Fine	68.19%	40.10%	8.14%	0.00%	212.8	0.97	0.95	0.99	0.04	2%
	Fine AS			7.47%	0.08%	496.9	0.94	0.61	1.38	0.76	27%
	Coarse	43.16%	40.87%	6.34%	0.08%	-1108.2	1.20	0.88	1.54	0.66	33%
	Coarse AS			6.34%	0.27%	-969.0	1.17	0.95	1.58	0.64	24%
Limestone NE	Fine	71.57%	36.59%	7.11%	0.10%	7115.7	0.34	0.28	0.40	0.12	5%
	Fine AS			6.95%	0.23%	6485.5	0.39	0.31	0.51	0.20	7%
	Fine Hydrated Lime			5.91%	0.17%	3634.9	0.59	0.53	0.66	0.13	6%
	Fine SBS			6.93%	0.14%	5791.8	0.35	0.25	0.50	0.25	10%
Limestone South	Fine	65.23%	42.95%	7.18%	0.28%	3939.7	0.51	0.42	0.64	0.22	10%
	Fine AS			6.79%	0.18%	2223.9	0.76	0.54	0.98	0.43	14%

The data presented in Table 4.3 provides similar results to those observed in the bar chart in Figure 4.3, namely large variations in fracture energy testing results. These large variations are evident in the large range in possible ranges of fracture energy ratio (FER) and the standard deviation of FER. Both of these parameters were calculated in the same fashion as presented above for the tensile strength testing results. Based on the data in Figure 4.3 and Table 4.3, it is clear that only general observations based on the effects of mixture properties on test results can be made. Therefore, regression analysis to determine significant mix properties will not be performed on the fracture energy results because any of results would be inaccurate due to the variability of the test results.

Although the high variability of certain test results prevents the establishment of any definite moisture damage criterion based on fracture energy, further analysis of the stress-strain behavior of individual mixes shows promise in identifying moisture susceptible mixes. The stress-strain plots of a moisture susceptible mix and a moisture resistant mix are shown in Figures 4.3 and 4.4 respectively. Plots of all mixes used in the study can be found in Figures 4.1.3A – 4.1.3P in the Appendix. Visual inspection of the figures given below shows a clear difference in results for moisture susceptible mixes relative to those that resist moisture damage. In Figure 4.4 the moisture damage realized in the mix due to conditioning is apparent by the large distance between stress strain curves for conditioned and unconditioned mixes. Figure 4.5 exhibits more similar stress-strain behavior of conditioned and unconditioned mixes, signifying a mix that is not prone to moisture damage. These two plots verify that conceptually the Fracture Energy Method is sound in that it is able to differentiate between moisture susceptible mixes and those that are not. However, before any fracture energy criteria is established the reliability of the test must be reduced through further refinement of the test procedures.

Figure 4. 4: Limestone NE Stress-Strain Relationship for Unconditioned and Conditioned Mixes

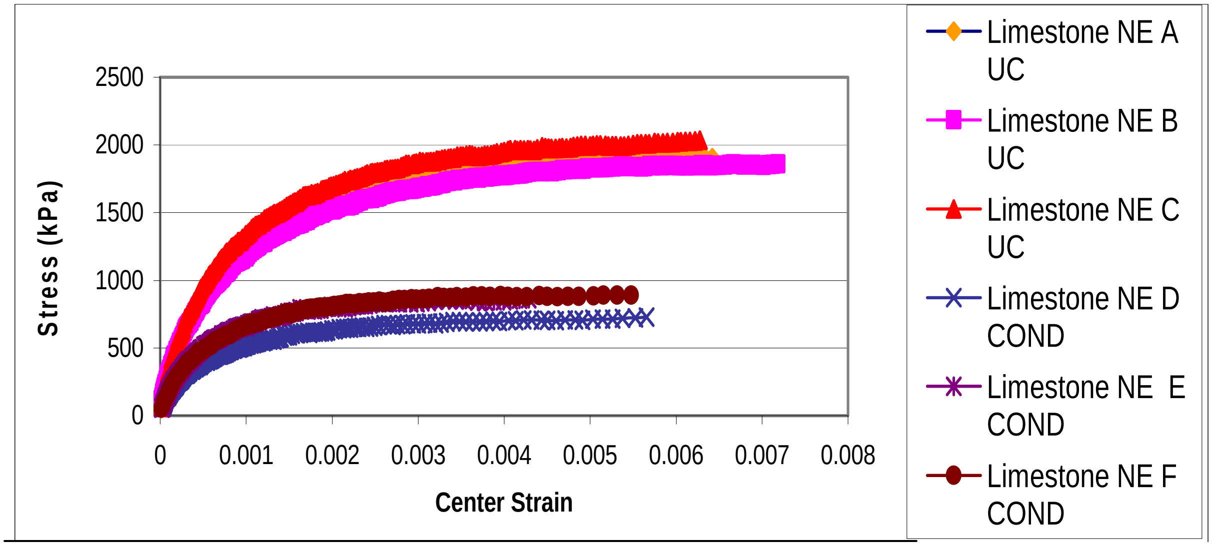
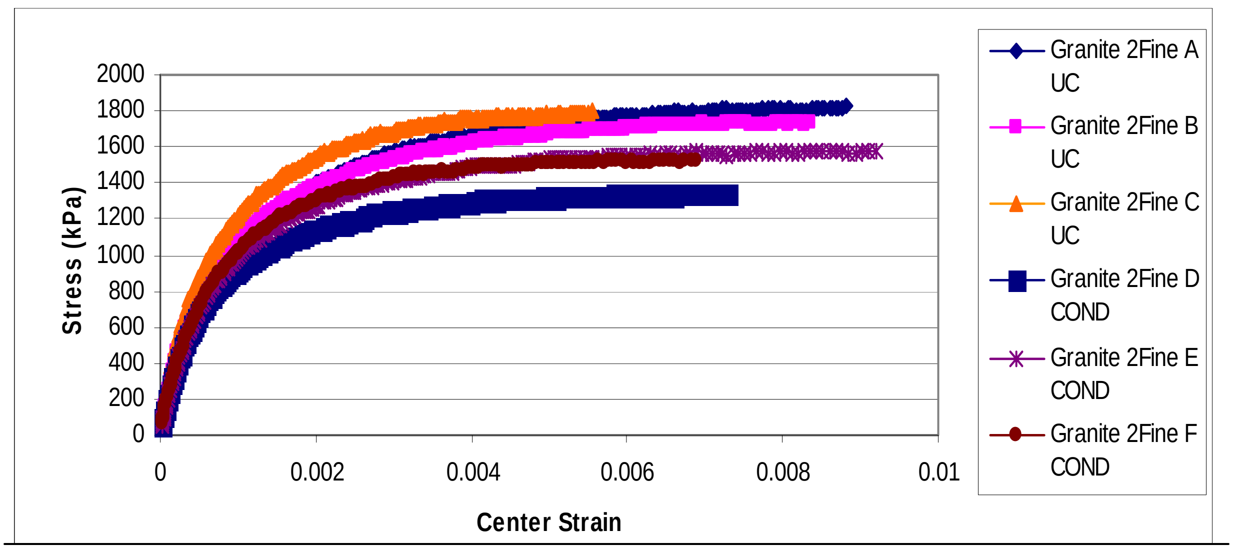


Figure 4. 5: Granite 2 Fine Stress-Strain Relationship for Unconditioned and Conditioned Mixes



Comparison of Figures 4.2 and 4.3 show that mixes are not ranked the same in terms of decreasing Tensile Strength Ratio (TSR) relative to Fracture Energy Ratio (FER). This indicates that the presence of moisture, aggregate type, and gradation all have different effects on mixture performance for each of the measures. The relationship between TSR and FER was further examined by plotting TSR vs. FER. Two different plots were constructed in order to evaluate this relationship. The first plot, Figure 4.6, was constructed by calculating all possible values of the TSR and FER using all combinations of measured wet and dry strengths. The second plot was constructed to address the issue of variability. The variability of a test method is of utmost importance when considering it for implementation, or when using it to identify any trends or correlations. To capture the variability inherent to each mix, the standard deviations of all possible TSR and FER measurements were calculated and shown as x and y error bars to represent the possible ranges of the test results. This plot is shown in Figure 4.6.

Figure 4. 6: Tensile Strength Ratio vs. Fracture Energy Ratio for all Possible Values

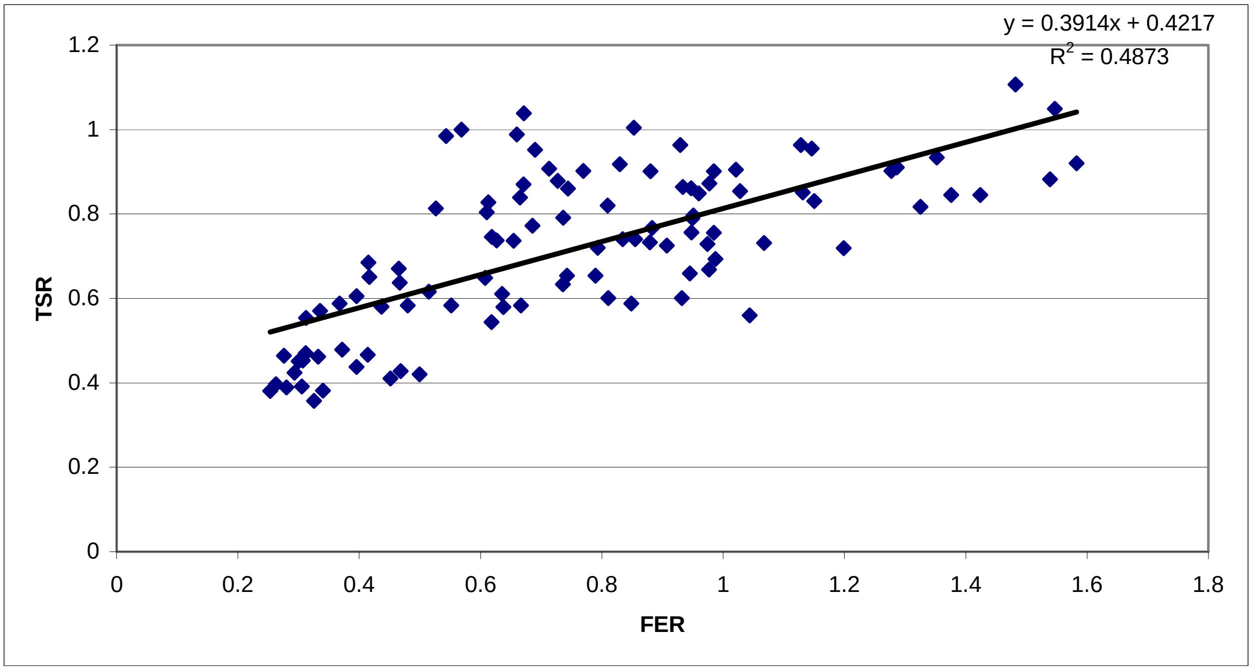
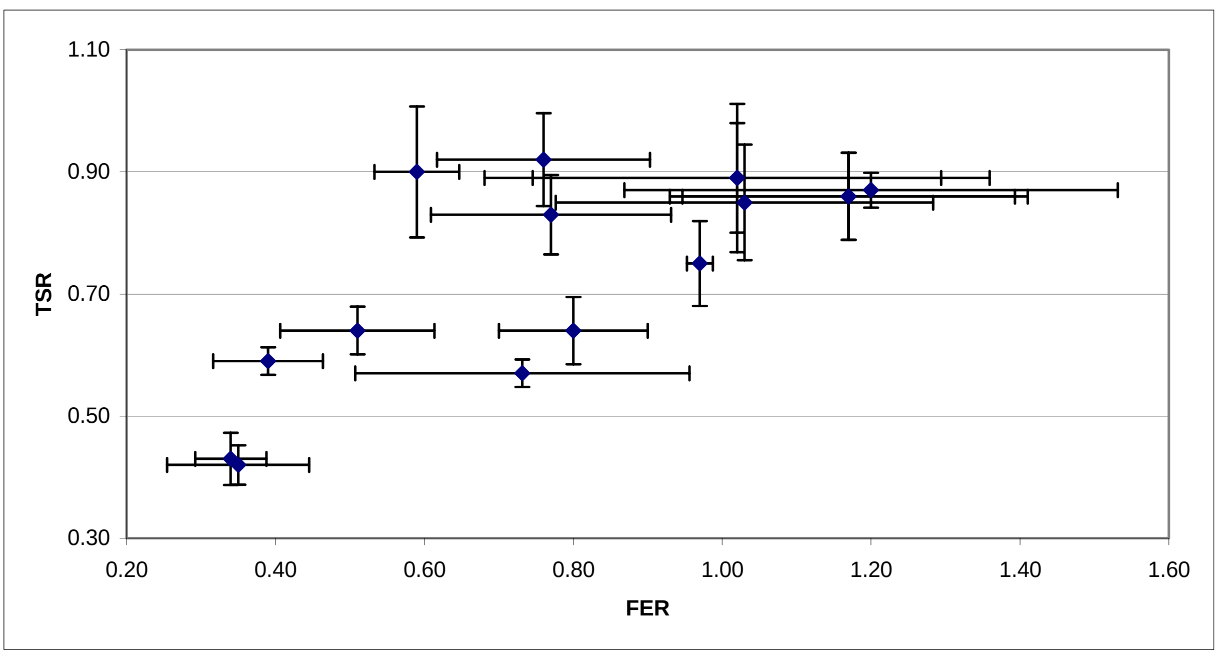


Figure 4. 7: Tensile Strength Ratio vs. Fracture Energy Ratio Showing Variability of Each Mix



Both figures show the general trend of increasing TSR with increasing FER, however Figure 4.6 shows no strong correlation exists. Furthermore, the wide ranges shown, especially for the fracture energy ratio, in Figure 4.7, indicate that the variability associated with both the TSR and FER is too large to develop any meaningful correlation between the two parameters. Also, Figure 4.6 shows that using the TSR and other mix characteristics to predict FER is probably not feasible.

4.2: Non - Mechanical (Stripping) Test Results

On an individual level, the stripping test was evaluated based on two criteria: reliability, and its ability to show a change in results (% mass loss) due to the addition of anti-stripping additive. The numerical results are provided in Table 4.2 in the Appendix. The results are presented in the form of bar charts in Figures 4.7 and 4.8. Figure 4.7 provides the test results and corresponding standard deviations of each mix. The experimental design called for identifying the worst performing mix based on TSR and FER measurements and preparing more mix designs using hydrated lime and SBS modification. The Limestone NE mix was chosen to undergo further modification. The results of the stripping test of the mix with these modifications are provided in Figure 4.8.

Figure 4. 8: Summary of Stripping Test Results for Unmodified and Anti-strip Modified Mixtures

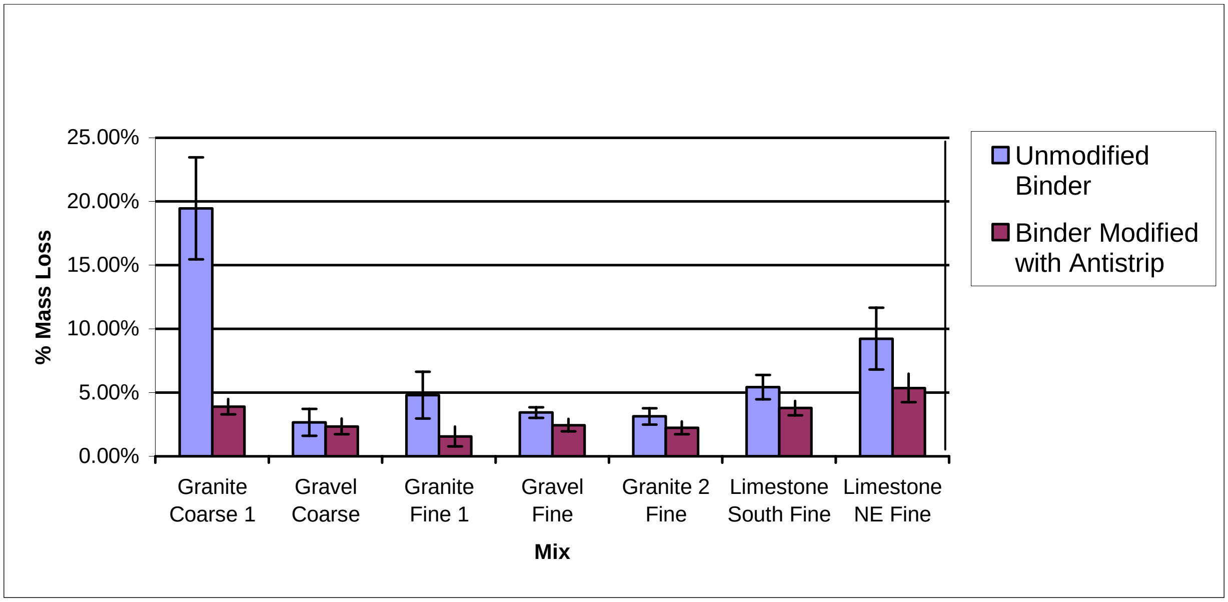
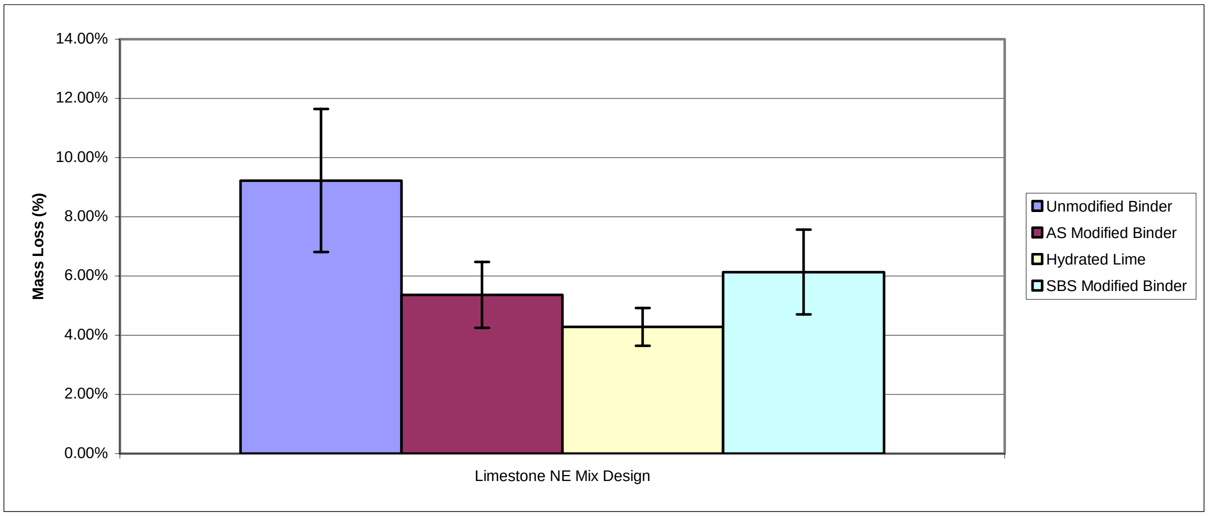


Figure 4. 9: Comparison of Different Modification Methods for the Limestone NE Mix Design



The figures above show that the stripping test is able to differentiate between moisture susceptible mixes using anti-stripping additives and those that do not. This is shown by the reduction in % Mass Loss in the mixes using anti-stripping additive. Furthermore, the wide range in results (1.55%-9.23% for fine and 2.66% to 19.55% for coarse) show that the stripping test is able to identify mixes with a wide range of stripping potentials in both coarse and fine graded mixes. However, the stripping test was unable to satisfactorily meet the reliability criterion previously established. In some cases, multiple replicates were necessary to minimize the variability between tests. Even with the implementation of these measures standard deviations for individual mixes ranged between 0.5% and 4.3% Mass Loss. This corresponds to a coefficient of variation of 15%-40%. Specific values, including the amount of replicates used for each test are provided in Table 4.1.4 of the Appendix. This large experimental error was also noticed qualitatively. Three replicates were tested in the gyratory shaker bath at one time for each mix. In moisture susceptible mixes, samples of the same batch exhibited significant loss of materials as noted by the separation of fine aggregate from the loose mix as shown by significant clouding of the water Whereas, some mixes tested in the same testing batch would not exhibit this behavior. This difference is show in Figure 4.10.

Figure 4. 10: Sample Showing Separation of Fine Aggregate From Loose Mix

4.2.1: Adjusted Stripping Test Results

As previously noted in Chapter 3, concerns were raised after testing was complete that stripping test results were being confounded with gradation because of the inability of the test method to differentiate between mass loss due to moisture damage and mass loss due to particle size. To account for this difference all loose mixes were hand sieved and the amount of material passing the appropriate sieve size (#8 for coarse #16 for fine mixes) was weighed and used to adjust the mass loss percentages for each mix. The results of these adjustments are shown Figures 4-11 and 4-12. An average correction factor was applied to all mixes therefore, the standard deviation remained unchanged.

Figure 4. 11: Summary of Adjusted Stripping Test Results for Unmodified and Anti-strip Modified Mixtures

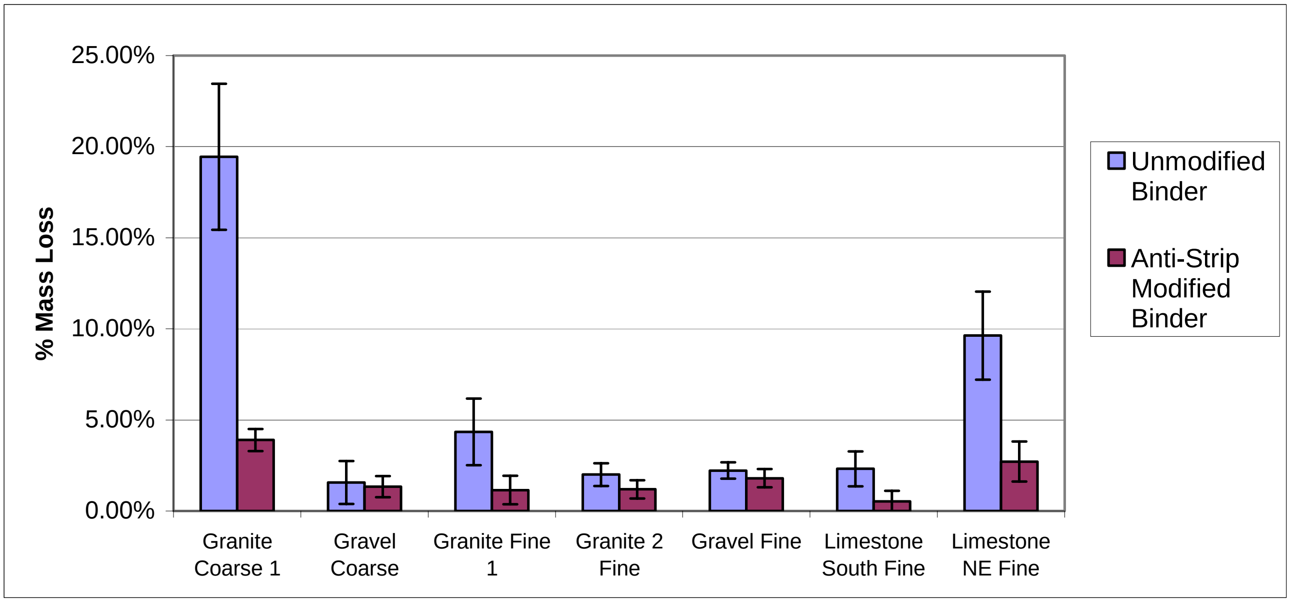
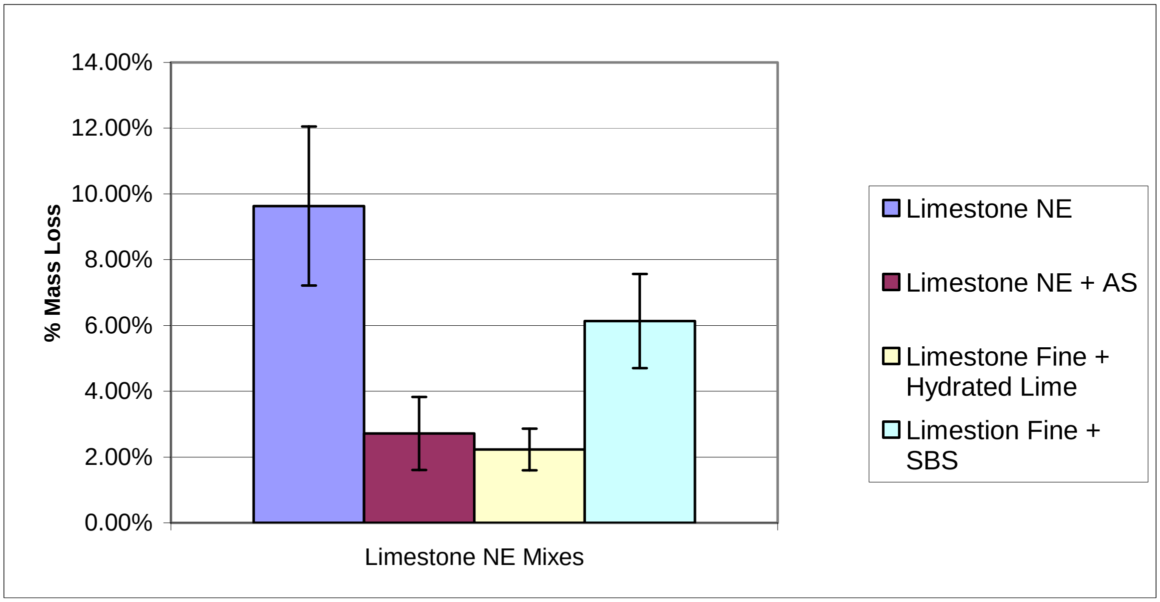


Figure 4. 12: Comparison of Adjusted Mass Loss of Limestone NE using Different Additives



The adjusted stripping test procedure resulted in a downward shift of all of the previous test results. This adjustment had no effects on the previous findings that the stripping test was able to identify the presence of anti-stripping additive and show a wide range of mass loss results. It also has the same reliability issues associated with the previous test method.

4.3: Relationships Between Non-Mechanical and Mechanical Testing Results

The overall objective of the mechanical and non-mechanical testing conducted in this study was to determine if an efficient non-mechanical test could be developed that could accurately identify moisture susceptible mixes. The efficacy of the non-mechanical testing results would be evaluated through comparison to mechanical testing results. Assuming that all test methods produced reliable results that made engineering sense, the relationship between the non-mechanical and mechanical testing results could be used to define a % Mass Loss threshold to be used as a screening test in WisDOT moisture sensitivity evaluation. However, given the results presented above, it is clear that this assumption does not hold. All test methods were able to produce results that clearly showed mixes with a range of moisture susceptibility due to differing aggregate type and were able to identify the presence of anti-stripping additives. However, the reliability of the test results, especially the FER and % Mass Loss results were not sufficient to warrant the definition of such a threshold. Examination of plots of %Mass Loss vs. TSR and FER respectively and their corresponding variabilities, reinforce that results from this testing are inadequate to define such a threshold. Figures 4.13 and 4.14 show these plots for fine mixes. A similar analysis for coarse mixes was conducted, however only four data points were available, it was felt this data set was insufficient to evaluate this relationship. Plots for the coarse mixes are available in Figures 4.3A and 4.3B in the Appendix. Both figures include the standard deviations of each test to show the possible ranges of results.

Figure 4. 13: Mass Loss vs. Fracture Energy Ratio

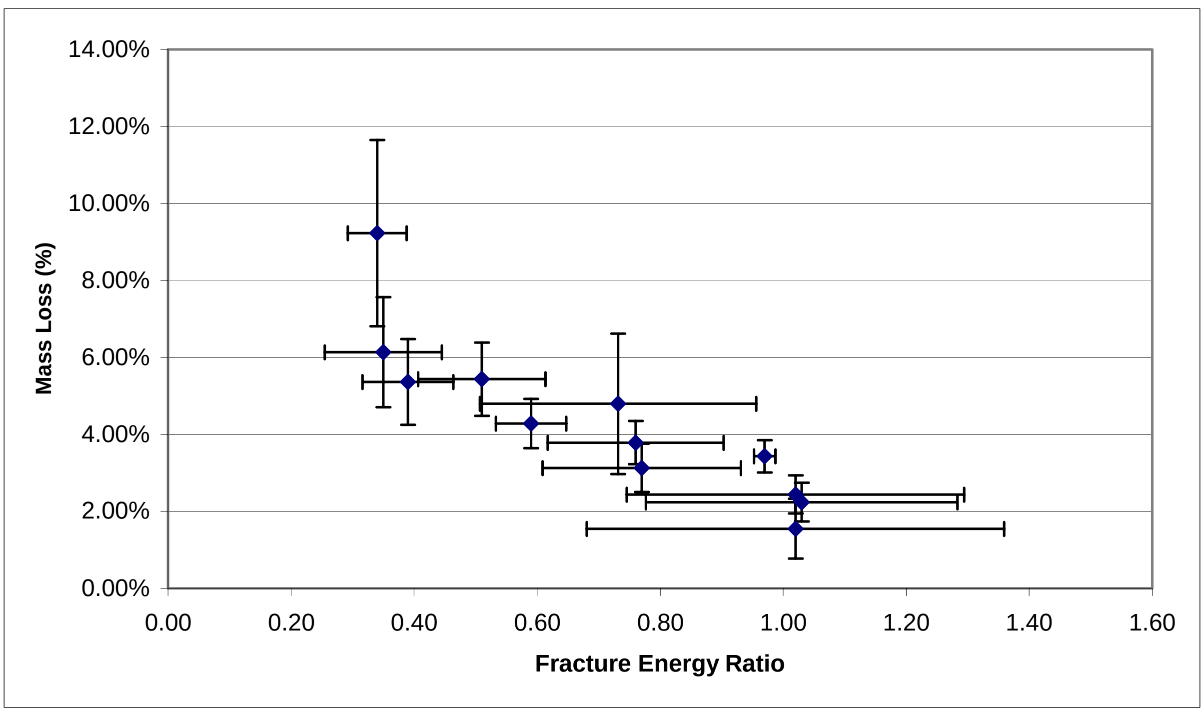
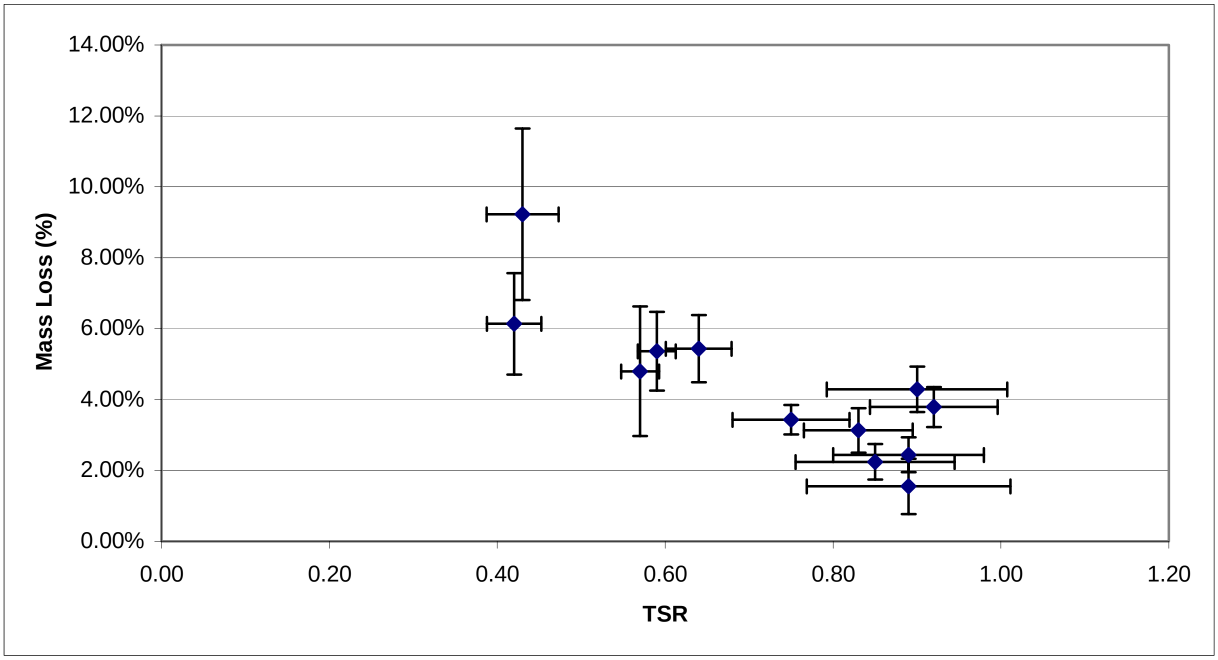


Figure 4. 14: Mass Loss vs. Tensile Strength Ratio

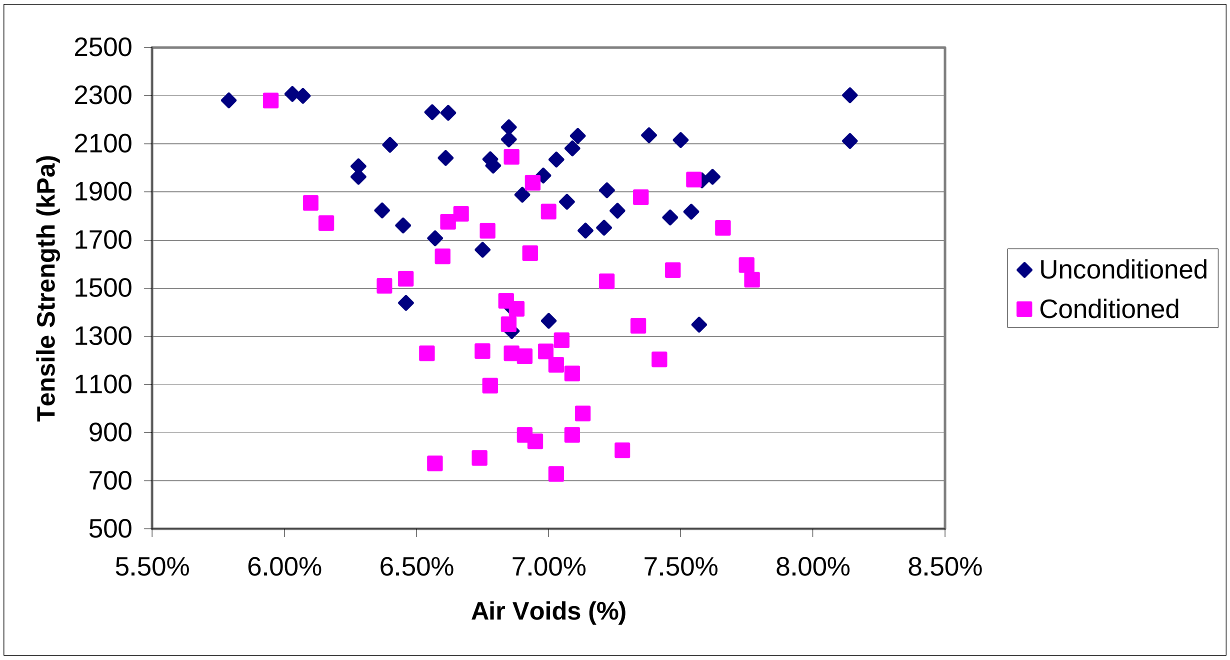


Based on these comparisons it is clear the inherent variability of all three measures is too great to clearly establish a Percent Mass Loss threshold that could accurately identify moisture susceptible mixes. Based on this variability, establishment of such a threshold could potentially result in false positives, leading to mechanical testing of a mixture being waived for a mix that is moisture susceptible. This would result in acceptance and placement of a mixture that has increased potential for premature failure in the field due to distress caused by moisture damage. The same analysis was performed on the adjusted percent mass loss results, as expected these results lead to the same conclusion. Plots for the Adjusted % Mass Loss vs. FER and TSR are available in Figures 4.3C – 4.3F in Appendix A.

4.4: Examination of the Effects of Air Voids on Tensile Strength

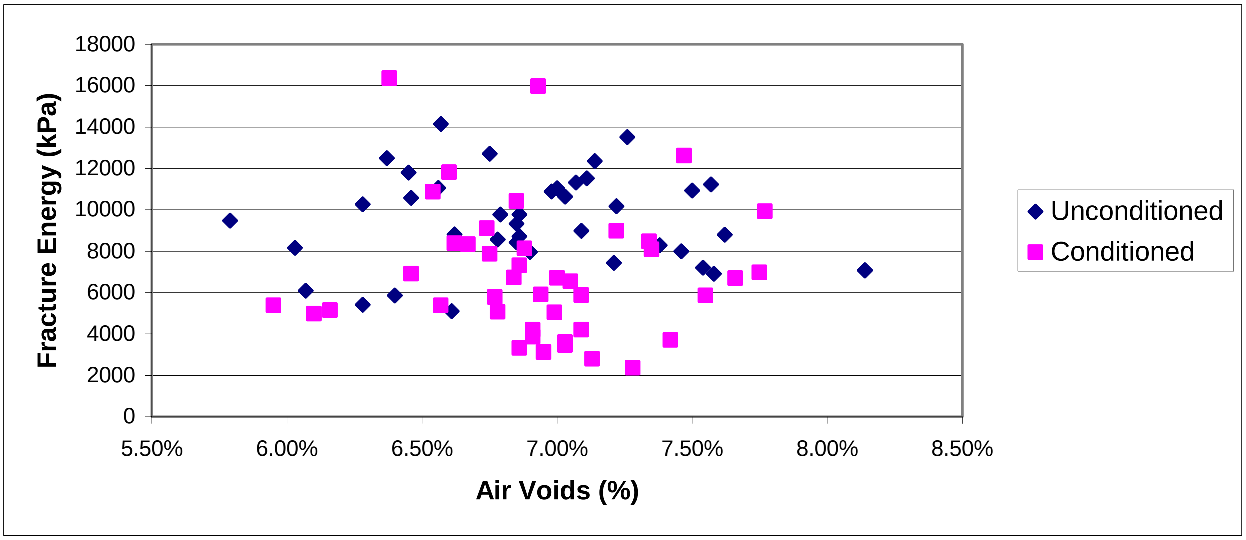
Discussion in Chapter 3 cited that a relationship exists between % air voids and conditioned and unconditioned tensile strength. Specifically that tensile strength decreases with increasing air voids and that the rate of decreasing tensile strength is greater for conditioned mixes relative to unconditioned mixes. This relationship was examined by plotting the tensile strength vs. air voids for all test results from this study. It is expected that the results of this plot will only be able to provide general trends due to the variety of other mix properties used in this study and lack of tensile strength testing data for a wide range of air voids. This plot is provided in Figure 4.15, a similar plot for the fracture energy parameter is provided in Figure 4.16.

Figure 4. 15: Plot of Tensile Strength vs. Air Voids



Examination of the plot of Tensile Strength vs. Air Voids shows general trends consistent with expectations. Discounting the outliers in the upper right corner of the plot, tensile strength values seem to be decreasing with increasing air voids for both conditioned and unconditioned mixtures. The quality of the definition of this relationship is constrained by the small range in air voids and wide varieties of mixture properties. Mixtures in this study were selected with the expectation that they would exhibit a wide range in performance, which is the reason for the wide range in scatter of conditioned and unconditioned tensile strength values. Furthermore, the study attempted to minimize the effects of air void variation on tensile strength by trying to maintain consistency in the air voids of all samples for a given test set. This practice prevents more detailed analysis of the tensile strength of a certain mix for a wide range of air voids. A more complete analysis of this topic would require compaction of the same mix at the entire range of air voids levels ranging from 6% - 8% permitted by ASTM D4867.

Figure 4. 16: Plot of Fracture Energy vs. Air Voids

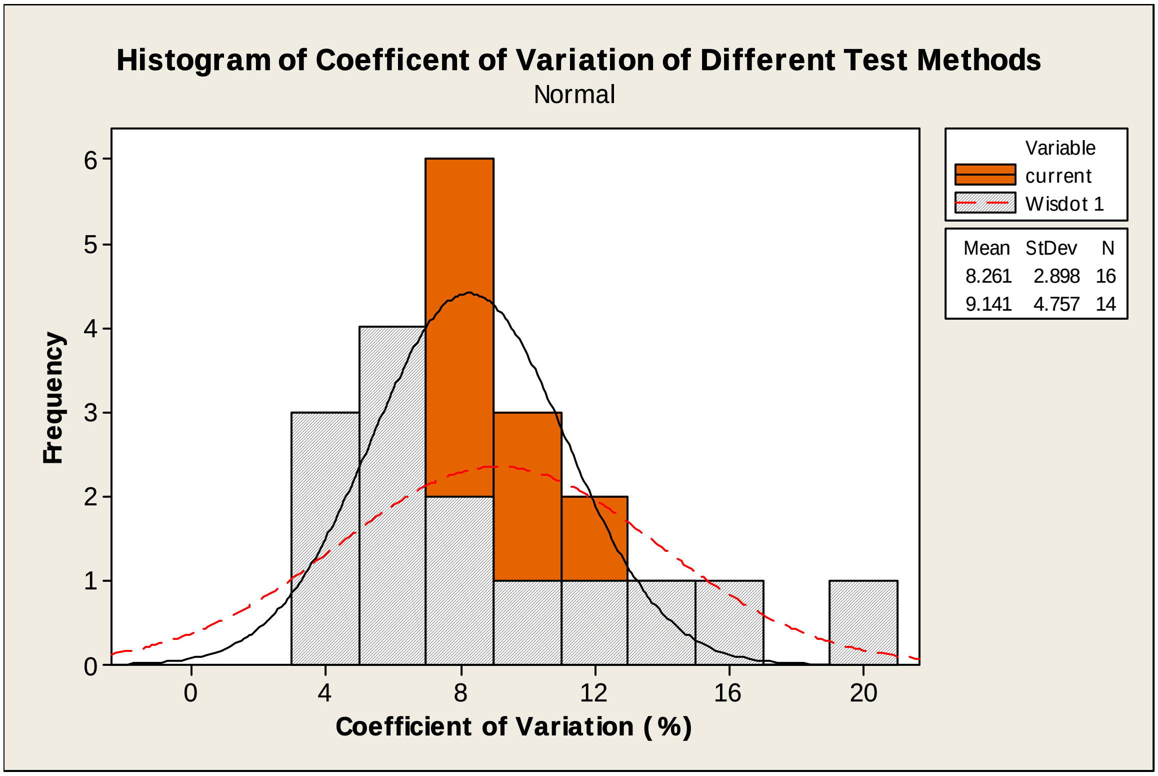


The plot of the effects of air voids on fracture energy shows no discernable trends. Based on conceptual knowledge of fracture mechanics and the previously established trend that showed FER increasing with TSR, it was hypothesized that a general trend of decreasing fracture energy with increasing air voids would be present. Conceptually, a higher amount of air voids would relate to lower resistance of the mix to fracture, however this concept was not shown in the test data. Deviations from test results could be due to the high variability within the results, the fracture energy used in the plot was average fracture energy. Table 4.3 shows that there are large ranges of fracture energy that exist between these averages, the variability of the test results must be improved before future analysis of the relationship between fracture energy and air voids.

4.5: Does the Advanced TSR Testing Procedure Reduce Reliability?

One objective of this study was to evaluate if any of the advanced testing procedures used were an improvement over current WisDOT procedures. Data published in the first moisture damage study commissioned by WisDOT (0092-45-94) [2] provided an opportunity to investigate if the variability of the TSR test was reduced by using the advanced testing methods in this study. The variability of each of the test methods was calculated by using the coefficient of variation associated with TSR testing in each study. For the current study, all of the possible TSR values were calculated using all combinations of conditioned and unconditioned tensile strengths. The coefficient of variation of the TSR values in the previous study were calculated by using the high, low, and average TSR values reported by WisDOT for the mixes used in the research [2]. A histogram was used to compare the coefficients of variation for tests in each study. The histogram is provided in Figure 4.17. A table of the raw data used to create the histogram and individual histograms for each data set are available in Tables 4.4A – 4.4B and Figures 4.5A -4.5B in the Appendix.

Figure 4. 17: Comparison of Coefficient of Variation for WisDOT Research Projects 0092-05-12 and 0092-95-04



The histogram provided above shows that the advanced procedure used in this study could potentially reduce the variability of the TSR measurement. The coefficients of variation associated with the test methods used in this study resulted in a mean of approximately 8% with a maximum of 13%, whereas the current methods resulted in a mean coefficient of variation of 9% with a maximum of 21%. The difference in mean and more importantly maximum coefficients of variation associated with each test method indicate that the method used in this study results in lower variability. However, the nature of this comparison did not allow for consideration of a number of variables including mix type and variability associated with operator error. Ideally, the two test methods would be compared using the same mix designs and operators running the tests to eliminate these effects. Based on the inability of this comparison to consider these factors, it cannot be taken as fact that the test method used in this study has the ability to reduce the variability of TSR measurements realized in current procedures. However, the data presented here shows there is potential for reduced variability using this method. The absence of a coefficient of variation greater than 15% is promising.

4.6: Statistical Analysis: Using Analysis of Variance to Investigate the Results of Mechanical Testing

To further investigate the ability of the Tensile Strength and Fracture Energy tests to accurately predict moisture damage test results were analyzed by the statistical method of Analysis of Variance (ANOVA) [17]. In order to be considered accurate a mechanical test must be able to identify the effects of conditioning on an asphalt mixture. Furthermore, the test must also have the ability to identify changes in mixture design that are pertinent to moisture damage. Specifically, a mechanical test must be able identify the presence of an anti-stripping additive and differentiate between aggregate source and type. To evaluate the mechanical tests based on these criteria Three-Way ANOVA was used to investigate the effects of the independent variables of conditioning, anti-stripping agent, and aggregate type on the response variables of fracture energy and tensile strength. The experimental design used in this analysis is provided in Table 4.4.

Table 4. 4: Experimental Design – 3-Way ANOVA

Coded Factors	Factor	Level
A	Aggregate Type	Granite
		Gravel
		Limestone NE
		Limestone S
B	Antistripping Agent	No
		Yes
C	Conditioned	No
		Yes

The ANOVA was developed by selecting data from the overall testing results that fit the parameters defined in the experimental design. This resulted in the use of 48 data points to complete the analysis. The use of Three-Way ANOVA allows for the investigation of the significance of the effects of the independent variables and the effects of their interaction on the response parameters used in this study to characterize moisture damage. For this analysis only two-factor interactions were considered. The analysis was conducted by considering the F-Distribution at a level of significance $\alpha = 0.05$. The results of the ANOVA are presented below for Fracture Energy in Table 4.5 and for Tensile Strength in Table 4.6.

Table 4. 5: Results of 3-Way ANOVA – Fracture Energy

Effect	DOF	F Distribution	p-value
Aggregate Type	3	12.23	0.000
Anti-strip	1	17.22	0.000
Conditioning	1	62.51	0.000
Aggregate Type*Anti-strip	3	1.62	0.205
Aggregate Type*Conditioning	3	16.24	0.000
Anti-strip*Conditioning	1	5.03	0.032

Table 4. 6: Results of 3-Way ANOVA – Tensile Strength

Effect	DOF	F Distribution	p-value
Aggregate Type	3	59.03	0.000
Anti-strip	1	45.16	0.000
Conditioning	1	226.61	0.000
Aggregate Type*Anti-strip	3	11.07	0.000
Aggregate Type*Conditioning	3	15.4	0.000
Anti-strip*Conditioning	1	21.38	0.000

* = interaction
(yellow box) = significant effect

Significant factors are defined as those having a p-value<α = 0.05. In the analysis Type III (adjusted) sum of squares was used. The appropriate diagnostic plots were examined to evaluate the validity of the results above. Tensile strength data was found to meet the assumptions of independence, normally distributed, and constant variance required by ANOVA. However, the fracture energy data required the inverse square root data transformation (1/square root) to satisfy the ANOVA assumptions. These diagnostic plots and Minitab Outputs are provided in Figures 4.6A and 4.6B of the Appendix. The figures above show differing results in terms of what effects were found to be significant. The results of the ANOVA analysis on Tensile Strength found the effects of all main factors and two factor interactions to be significant, whereas, the Fracture Energy results found the interaction between anti-stripping agent and conditioning to be insignificant. This difference can be attributed to the high variability of the individual fracture energy results relative to the tensile strength results. The higher variability will have an adverse effect on the quality of the model used to predict results in the ANOVA analysis, leading to errors in defining significant factors. However, in regards to the main effects, the results of the ANOVA analysis for both test methods were consistent with expected results. Based on the design of the overall research project it was expected that aggregate type, the presence of anti-stripping additive, and moisture conditioning would all have a significant impact on both the tensile strength and fracture energy of the asphalt mixtures.

The work-plan for this study also included the investigation of the effects of gradation on moisture damage. The testing matrix included coarse and fine mix designs for two aggregate types, with all four mixes being modified by liquid anti-stripping additive. The tensile strength and fracture energy results from these mixes were used to perform a 4-Way ANOVA to investigate the effects of gradation on moisture damage. The experimental design used in the analysis is provided in Table 4.7.

Table 4. 7: Experimental Design – 4-Way ANOVA

Coded Factors	Factor	Level
A	Aggregate Type	Granite
		Gravel
B	Gradation	Fine
		Coarse
C	Anti-stripping Agent	No
		Yes
D	Conditioned	No
		Yes

The factors of conditioning and anti-stripping agent were left in the 4-Way ANOVA to examine the consistency of the results with the 3-Way ANOVA presented above. The ANOVA analysis to investigate the effects of gradation was developed in the same manner as the previous analysis. Test results that met the conditions developed in the experimental design were selected from the overall test matrix to fill the data set. Again, the data set consisted of 48 data points. Furthermore, only main factors and two factor interactions and using the F-Distribution at a level of significance $\alpha = 0.05$. The results of the 4-Way ANOVA are presented in Tables 4.8 and 4.9.

Table 4. 8: Results of 4-Way ANOVA – Fracture Energy (Log Transformation)

Effect	DOF	F Distribution	p-value
Aggregate Type	1	20.51	0.000
Gradation	1	1.68	0.204
Anti-Strip	1	15.51	0.000
Conditioning	1	47.82	0.000
Aggregate Type*Gradation	1	5.69	0.023
Aggregate Type*Anti-stripping	1	0.19	0.662
Aggregate Type*Conditioning	1	30.4	0.000
Gradation*Anti-stripping	1	6.89	0.353
Gradation*Conditioning	1	8.14	0.008
Anti-Strip*Conditioning	1	3.66	0.065

Table 4. 9: Results of 4-Way ANOVA – Tensile Strength

Effect	DOF	F Distribution	p-value
Aggregate Type	1	95.61	0.000
Gradation	1	28.79	0.000
Anti-Strip	1	8.95	0.006
Conditioning	1	81.46	0.000
Aggregate Type*Gradation	1	12.16	0.002
Aggregate Type*Anti-stripping	1	8.03	0.008
Aggregate Type*Conditioning	1	1.47	0.235
Gradation*Anti-stripping	1	0.06	0.814
Gradation*Conditioning	1	0.69	0.414
Anti-Strip*Conditioning	1	3.34	0.078

* = interaction

= significant factor

Significant factors were defined as those with a p-value<α = 0.05. Again, in the analysis Type III (adjusted) sum of squares was used. The appropriate diagnostic plots were constructed and are provided with the Minitab outputs in Figures 4.6C -4. 6D of the Appendix. Examination of the diagnostic plots confirmed the validity of the Tensile Strength Analysis, but gave indications that data transformation was necessary for the fracture energy results. Trial and error was used to select the log transformation as most appropriate for the data set. This transformation allowed the data set to meet the ANOVA assumptions; however the model used to predict fracture energy still did not correlate well with actual results. The inaccuracy of the ANOVA for fracture energy was further verified by comparison of the significant effects identified using this parameter relative to those identified using tensile strength. Most importantly, the ANOVA results for fracture energy testing show the effects of gradation to be insignificant. This result is not consistent with other data analyses presented in this study. It is believed this error further reinforces that the reliability of the fracture energy tests must be questioned and that the procedures must be refined before the method is considered for widespread use. Based on this error, the 4-Way ANOVA for Fracture Energy will no longer be considered in comparison of the results of this analysis to the 3-Way ANOVA results. The tensile strength analysis was successful in that it was consistent with common engineering knowledge and the results of the 3-Way ANOVA conducted previously. Both analyses found the effects of conditioning and the addition of liquid anti-strip additive to have significant effects on tensile strength. Furthermore, the effect of gradation was found to significantly affect tensile strength, this outcome was also expected.

In conclusion, the results of these statistical analyses show that the tensile strength parameter has the ability to accurately predict moisture damage in asphalt mixtures. This parameter demonstrated sensitivity to all changes of the conditioning and mix design factors pertinent to moisture damage investigated in this study. The fracture energy parameter has potential to be an accurate measure as well; however the repeatability of the test must be improved. High standard deviations within certain test samples were reported previously in this chapter. As expected this high variability had an adverse effect on the ability to perform quality statistical analysis to evaluate the sensitivity of fracture energy to the previously defined factors.

4.7: Binder Testing Results

The performance of asphalt mixtures is governed by both the properties of the aggregates and asphalt in the mixture. The contributions of the constituent materials of the asphalt mixture and the significance of their individual material properties can be examined using the measured mixture and binder properties in this study. The binder properties determined as pertinent to mixture performance in indirect tension in terms of both tensile strength and fracture energy were the Superpave rutting parameter of $G^*/\sin\delta$, the Superpave fatigue parameter of $G^* \sin\delta$, and the cohesion of the asphalt as quantified by the tack factor (C_T). The following is a presentation of the binder test results and an examination of their correlation with mixture behavior.

4.7.1: Binder Testing Results – Superpave Rutting and Fatigue Parameters

The experimental design included the use of PG58-28 graded binders from three different sources, CRM, MIF, and Koch. The Koch binder was also SBS modified to attain a grade of PG 64-28. Furthermore, all binders were modified with anti-stripping agent at a concentration of 0.5% by weight. The binders were tested at a temperature of 10ºC at a frequency of 10 radians/sec to measure the complex modulus (G^*) and the phase angle (δ). These values were used to calculate the Superpave rutting and fatigue parameters. The results are presented below in the form of bar charts in Figures 4.18 and 4.19 respectively. Individual test results for these parameters and the tack factor are available in Table 4.7.1 of the Appendix.

Figure 4. 18: Comparison of Binder Rutting Performance

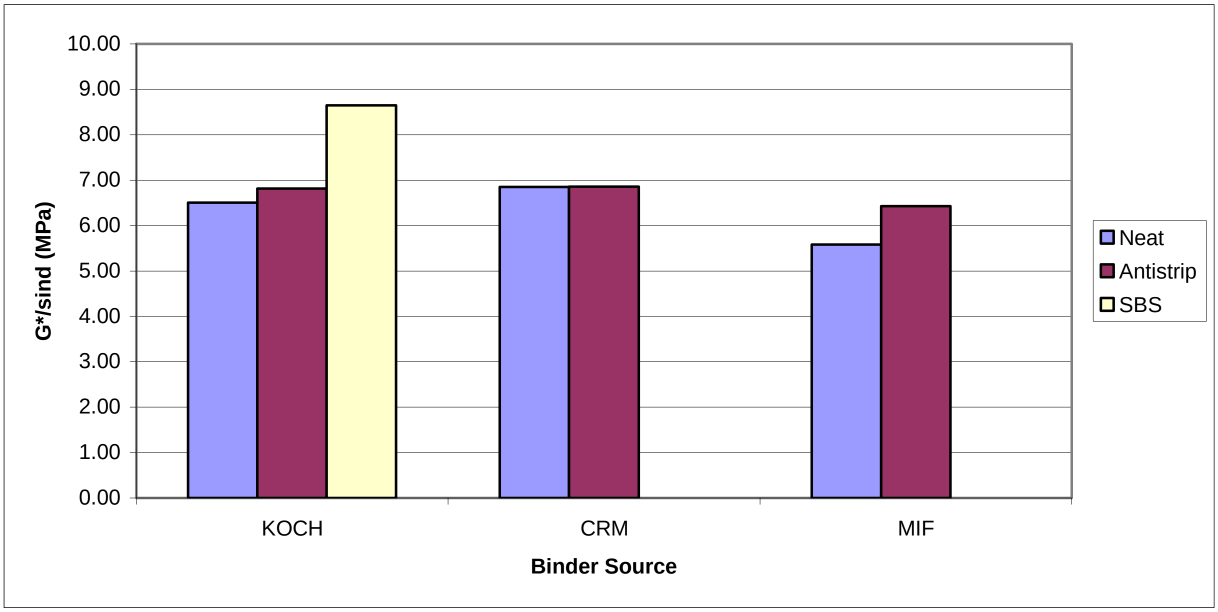
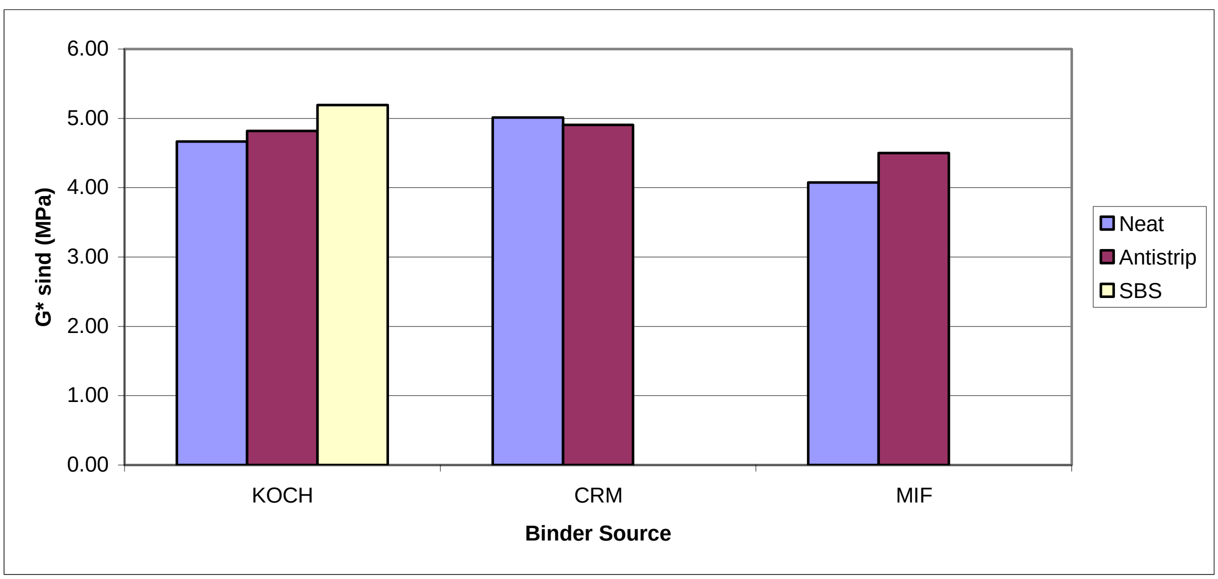


Figure 4. 19: Comparison of Binder Fatigue Performance



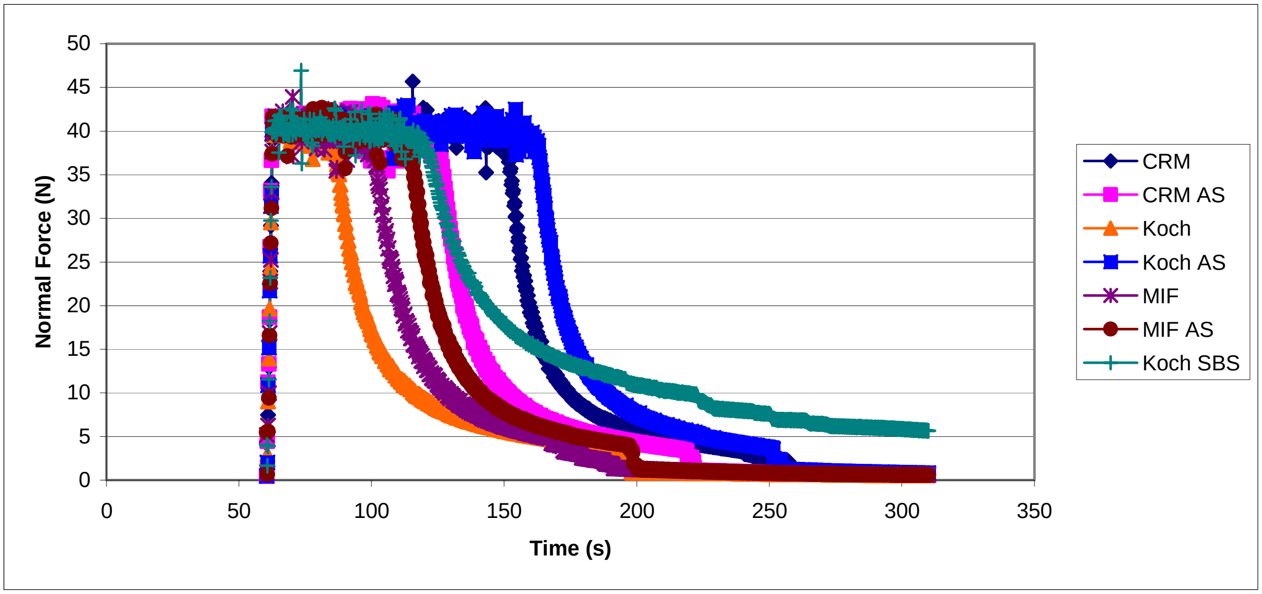
The results presented above are consistent with expectations. In a previous WHRP research project, Kanitpong concluded that the addition of anti-stripping agent had no significant effect on the rutting or fatigue resistance of asphalt binders [4]. The above results are consistent with these conclusions, showing differences in performance indiscernible from the 10% variability inherent to the DSR testing machine [4]. Furthermore, the SBS modified Koch asphalt showed improved performance relative to the neat and anti-strip modified Koch asphalts. Conceptually, this behavior is expected due added elasticity from the SBS polymer modification.

In conclusion, the results of the binder tests for fatigue and rutting resistance are consistent with what was expected and are suitable for use in comparison to mixture test results.

4.7.2: Binder Testing Results – Cohesion Test

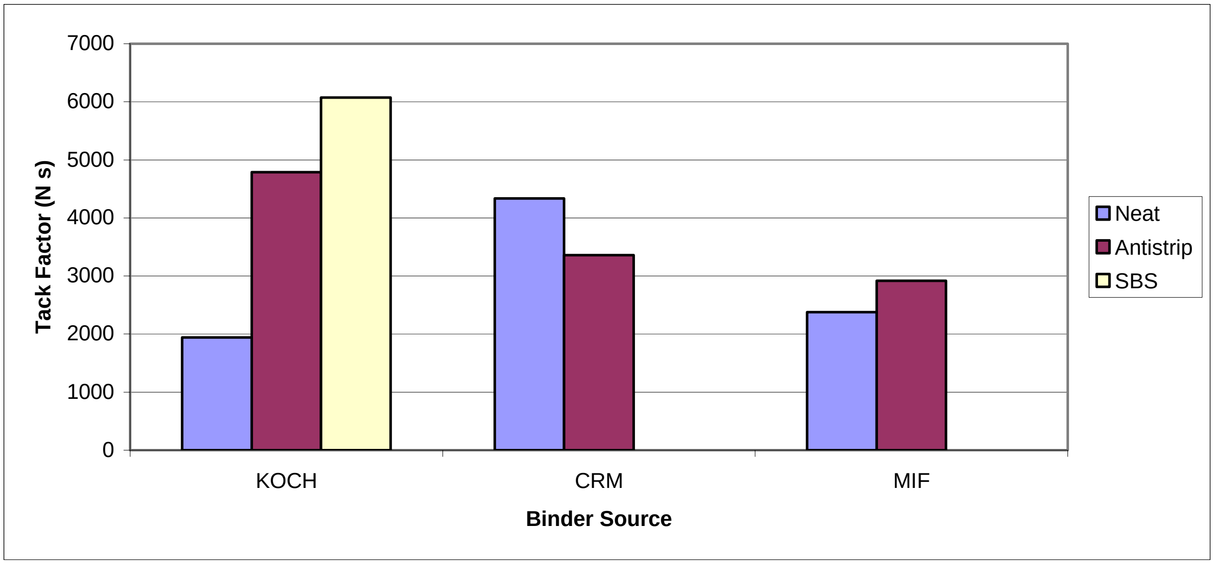
The cohesion of all binders used in this study was measured using the Tack Test as described in Chapter 3. Individual binder behavior during testing is shown by the plots of Normal Force vs. Time for each of the binders used in this study. The plots are provided in Figure 4.20.

Figure 4. 20: Cohesion Test Results for All Binders



The cohesion of each of the binders used in this study can be characterized by calculating the area under the curve. This area is used to define the tack factor (C_T), a quantifiable measure of the binder cohesion. The tack factors calculated using the plots in Figure 4.23 are presented in the form of a bar chart in Figure 4.21. The bar chart is organized such that it provides a comparison of the effects of anti-stripping or SBS modification on each binder source.

Figure 4. 21: Comparison of Binder Cohesion



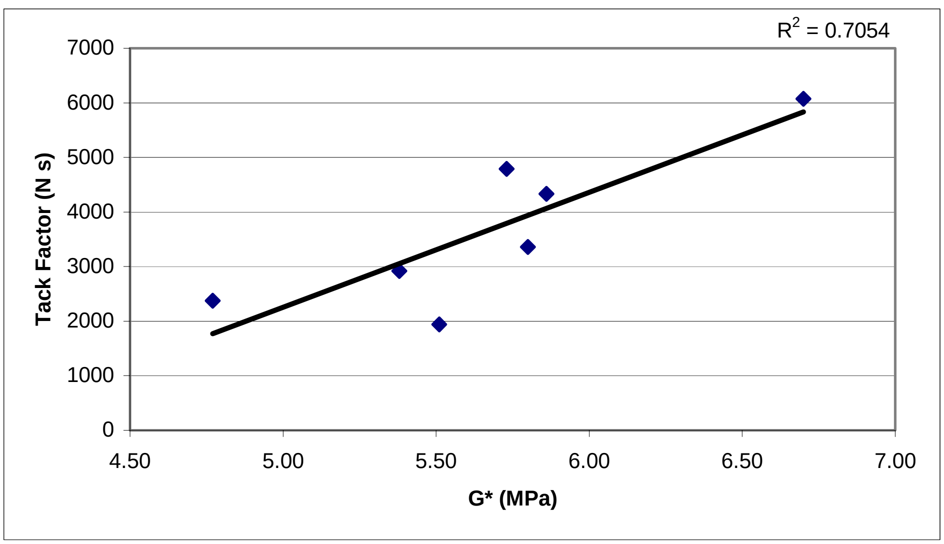
The results presented in Figures 4.20 and 4.21 are consistent with past measures of binder cohesion published in work by Kanitpong and Bahia [12,13] in terms of the behavior of the material during the tack test, however tack values measured in this study are considerably higher than those previously published. Both of these results were expected. The fact that the behavior of the asphalt binder over time is consistent with similar plots published in previous work in terms of shape, implies that the results of cohesion measurement at 10ºC are accurate and can be applied to further analysis. Higher values of tack factor were also expected due to the difference in testing temperature in this study and previous work. In previous work, the tack test was performed at testing temperatures of 25ºC, a 15ºC difference from the testing temperatures used in this study. The temperature dependency of asphalt materials dictates that there is an inverse relationship between temperature and cohesion. As temperature of the asphalt binder decreases, cohesion of the asphalt should increase and vice versa, regardless of binder source or modification. A general comparison of testing results from this study and previously published data are consistent with these concepts. Testing at 10ºC resulted in measured tack factors approximately one order of magnitude higher than those measured at higher testing temperatures. Furthermore, both theory and previous experimental data dictate that the addition of SBS modification should significantly increase the cohesive strength of the asphalt binder. This assertion is consistent with data gathered for the Koch binder source, which shows an increase of approximately 300% increase in binder cohesion due to SBS modification.

Testing results also presented deviations from expected behavior, namely the difference in cohesion within binder sources and the effects of anti-stripping agent on the cohesion of the binders. This study was by no means an extensive review of binder cohesion and its variation amongst binder sources; however the preliminary data shows that in terms of cohesion all binders of the same performance grade are not equal. Neat Koch and MIF binders demonstrated a tack factor of approximately 2000 N s, whereas the CRM binder had a tack factor of 4000 N s. This stark contrast for binders of the same grade (PG 58-28) suggests that further testing is needed to quantify asphalt binder cohesion. Another interesting observation was the differences in the effect of anti-stripping agent on asphalt binder cohesion for different sources. The effects of anti-strip additive on cohesion ranged from a 600 N s – 3000 N s improvement in tack factor for the MIF and Koch binders respectively, to a 1000 N s reduction in tack factor for the CRM. This variety of behavior suggests that the effect of anti-stripping agent varies with binder source. The variation suggests that there is a possibility that in instances where binder cohesion is a significant factor, the addition of anti-stripping agent could have an effect on all types of mixture performance, not just resistance to moisture damage.

4.7.3: Examination of Correlation Between Binder Properties

The examination of the relationship between the tack factor and the binder properties of complex modulus (G*) and the Superpave rutting and fatigue parameters was performed as a last measure of the quality of the binder data gathered in this study. Conceptually, these are all fundamental properties of the asphalt; therefore they should be related to some extent. Previous work by Kanitpong also suggests that a moderate relationship exists [11,12]. Based on consideration of these concepts and previous work it can be asserted that if the results of this analysis were a scatter plot with virtually no correlation, the quality of the data and its use as a means of comparison with mixture testing results would need to be questioned. The plots of the tack factor vs. the previously mentioned binder properties are provided below in Figures 4.22-4.24.

Figure 4. 22: Plot of Tack Factor vs. Complex Modulus



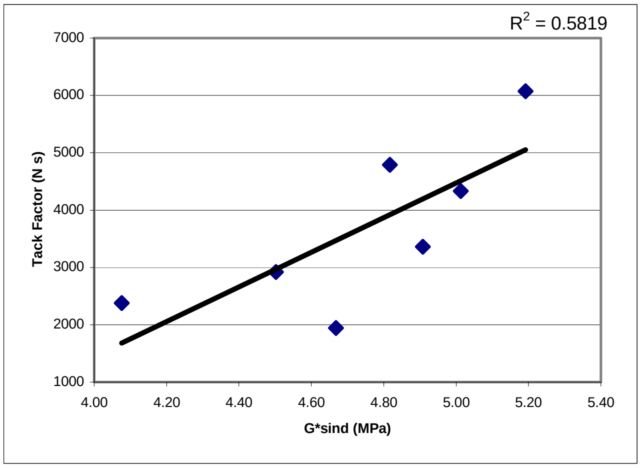


Figure 4. 24: Plot of Tack Factor vs. Fatigue Parameter

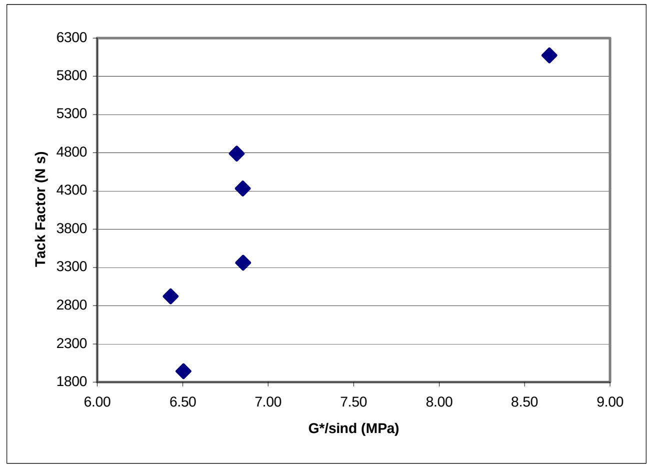


Figure 4. 23: Plot of Tack Factor vs. Rutting Parameter

The tack factor shows a moderate relationship with the other binder properties measured in this study, which is consistent with expected behavior and with previous work [11,12]. The existence of some sort of relationship indicates that the binder results are reasonable and able to be compared to mixture testing results.

4.8: Investigation of the Relationship Between Binder and Mixture Testing Results

Previous work by Kanitpong and others has established that the adhesive strength of the asphalt-aggregate bond is greater than the cohesive strength of the asphalt. Therefore, in an unconditioned mix failure will be controlled by the cohesive strength of the asphalt, rendering the tensile strength of the mixture a function of asphalt cohesion. Conversely, a mix subjected to water conditioning is a function of both the adhesive strength of the asphalt aggregate bond and the cohesive strength of the asphalt. In the case of moisture conditioning, adhesion must be considered due to the moisture effects on the adhesive bond. Moisture weakens the adhesive bond to the extent that the cohesive strength of the asphalt could exceed the adhesive strength of the asphalt-aggregate bond, necessitating the consideration of both phenomena in predicting mixture performance [11].

The previously discussed effects of the change in binder properties caused by moisture damage were verified experimentally through comparison of the cohesion to dry tensile strength and a statistically derived function of cohesion and adhesion to wet tensile strength. Both plots showed a linear relationship between their respective mixture and binder properties with a correlation greater than 95%. The tests that led to these results were conducted on a limited data set, in which only the effect of binder grade and modification were examined by holding aggregate source and gradation constant [11] at 25ºC. The experimental plan in this study prohibits the use of a similar analysis to verify the accuracy of the mechanical methods previously discussed due to a lack of data points to fit the constraints defined in Kanitpong and Bahia’s study and the previously established variability inherent to the measurement of adhesion in the stripping test used in this study. Theoretically, the measures of tensile strength and to a certain extent fracture energy would produce similar results, given an adequate data set.

However, the various combinations of asphalt binders, aggregate sources, and aggregate gradations used in this study allow for identification of the relative effects of the constituent materials in the asphalt mixture on performance as measured by tensile strength and fracture energy. This investigation will be conducted using an iterative process consisting of three steps of analysis:

1. The assumption will be made that the cohesive strength of the asphalt binder as measured by the Tack Test governs mixture performance in terms of both Fracture Energy and Tensile Strength.
2. If no relationship is established in step one, the assumption will be made that mixture performance is only dependent on the binder properties measured in this study. A regression analysis will be performed to identify significant binder factors.
3. Binder properties found significant will be combined with aggregate properties to determine the relative contribution of aggregate and binder properties to mixture performance.

4.8.1: Investigation of the Relationship between Cohesion and Mixture Testing Results

The first hypothesis was formed under the assumption that binder cohesion is the controlling factor in asphalt mixture performance in terms of indirect tensile strength and fracture energy for unconditioned and conditioned mixtures. This hypothesis will be investigated by plotting Tack Factor of all mixes vs. dry and wet tensile strength and fracture energy measures and the TSR and FER. If binder cohesion is the dominating factor governing asphalt mixture performance, these plots should show strong correlation between mixture and binder performance. The plots of the various tensile strength vs. tack factor are given below in Figures 4.25 – 4.27.

Figure 4. 25: Dry Average Tensile Strength vs. Tack Factor

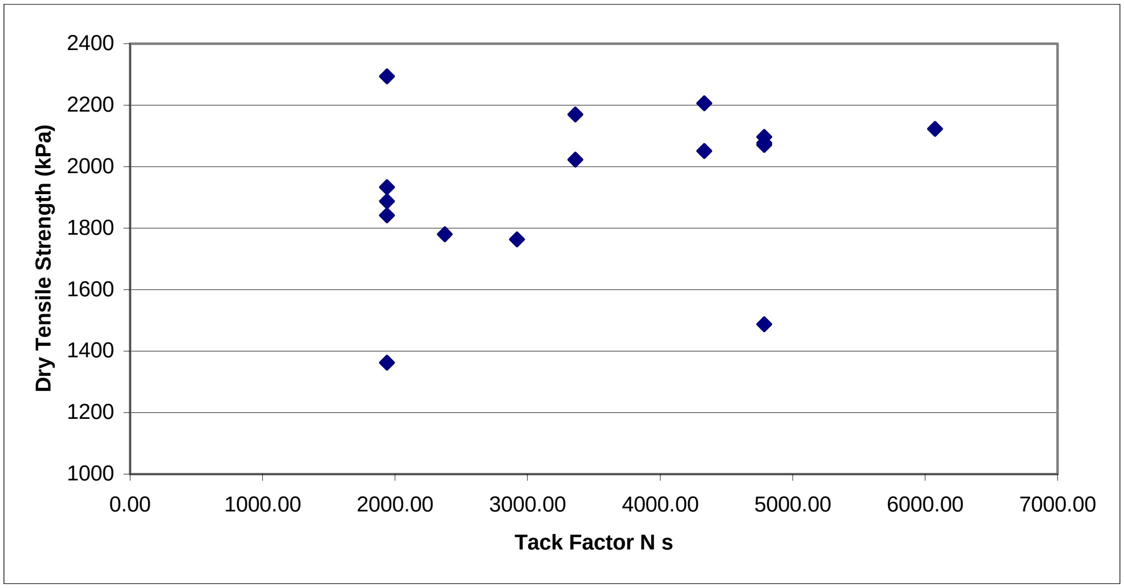


Figure 4. 26: Wet Tensile Strength vs. Tack Factor

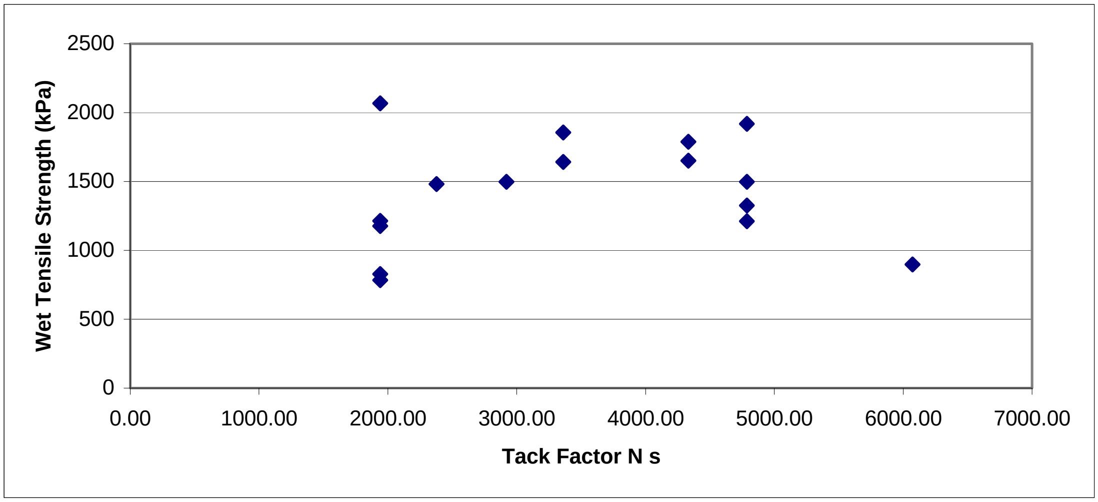
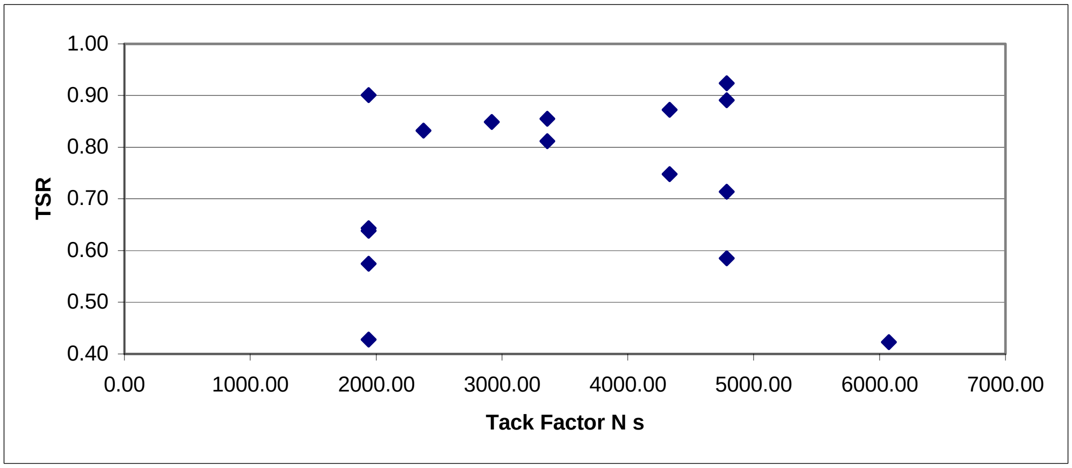


Figure 4. 27: TSR vs. Tack Factor



Based on the above plots it is apparent that when aggregate source, aggregate gradation, and binder type are varied asphalt cohesion alone cannot predict mixture behavior. The data for all three graphs is scattered and provides no correlation. The same plots prepared using the fracture energy parameter showed similar results, with a wide variety of scatter between data points. These plots are provided in Figures 4.8.1A – 4.8.1C of the Appendix. Based on these results it is clear that consideration of other factors is necessary in order to adequately relate binder and mixture performance.

4.8.2: Investigation of the Effects of Measured Binder Properties on Mixture Performance

The analysis presented in the previous section clearly shows that binder cohesion alone is insufficient in predicting asphalt mixture behavior in terms of indirect tensile strength. Therefore, the consideration of the binder parameters, G* sinδ and G*/sinδ must be considered in examining the contribution of the asphalt binder to overall mixture performance. In order to consider the combined effects of all these binder properties on mixture testing results regression analysis was performed. In this analysis the binder properties of cohesion, G* sinδ, and G*/sinδ were defined as dependent variables and the mixture test results of dry tensile strength, wet tensile strength, and TSR were defined as dependent variables. Theoretically, the regression procedure will produce an equation that identifies the binder properties that have significant effects on mixture testing results. The results of the regression analysis are presented as plots of the measured mixture test results vs. results predicted by the regression equation. The plots of Measured vs. Predicted values are provided in Figures 4.28-4.30. The numerical results of the regression analysis are presented in Figures 4.8.2A-4.8.2C of the Appendix.

Figure 4. 28: Measured vs. Predicted Dry Tensile Strength

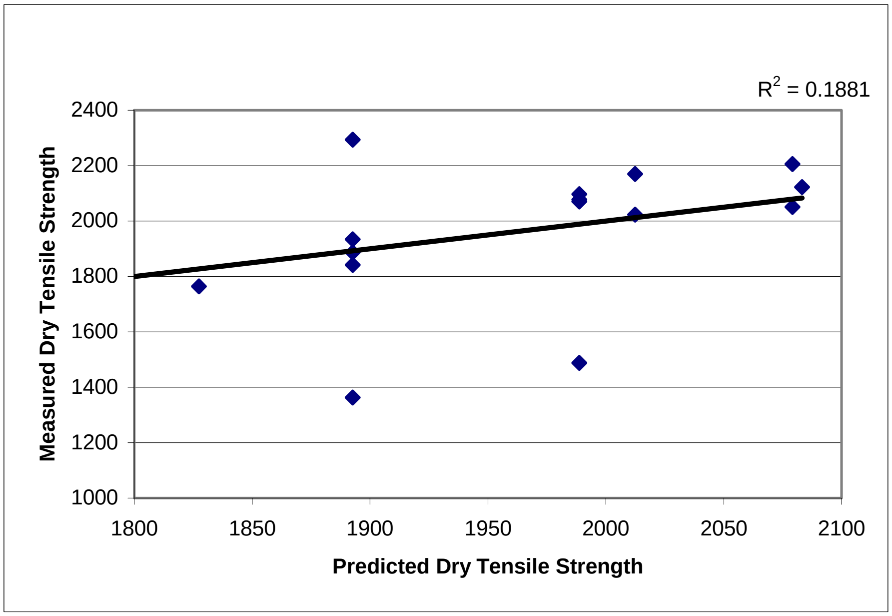


Figure 4. 29: Measured vs. Predicted Wet Tensile Strength

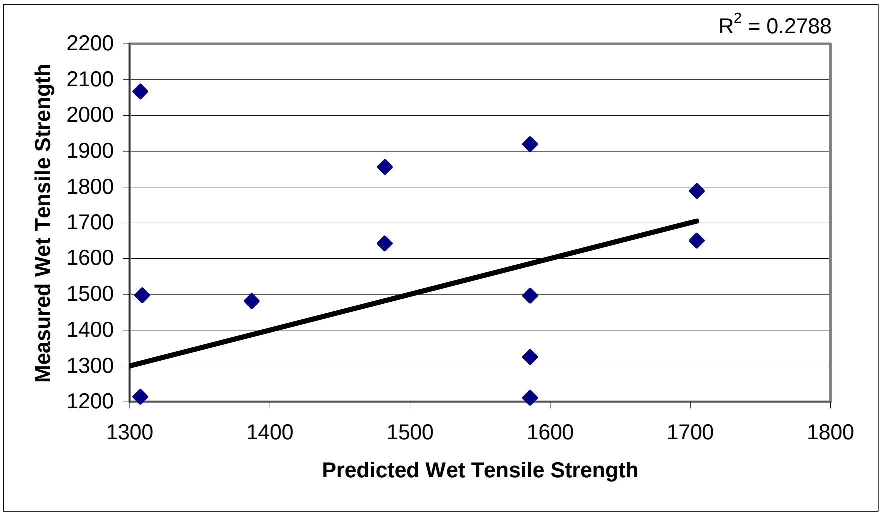
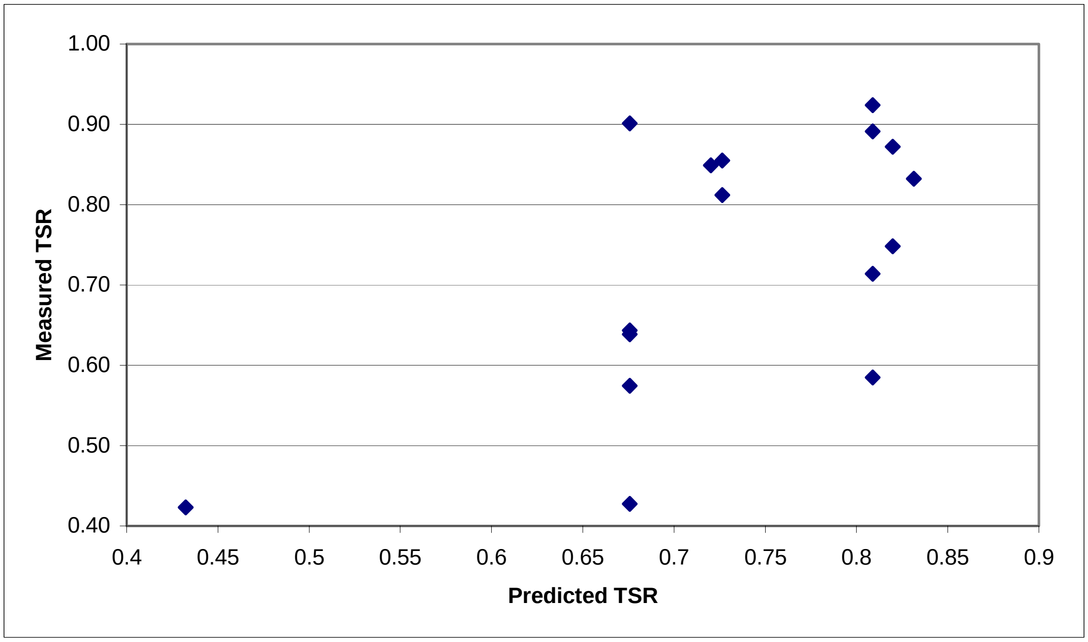


Figure 4. 30: Measured vs. Predicted TSR Values



The results of the regression analysis provided above show that no significant relationships can be established between mixture performance and binder properties. The regression equations basically yield scatter plots of measured vs. predicted values. Based on these results it is clear that using the asphalt binder properties of cohesion and the Superpave parameters for rutting and fatigue alone cannot predict asphalt mixture performance in terms of the tensile strength measured in this study. Due to the failure of the regression models to identify any significant factors (p-value<0.05), the incorporation of the binder properties into the previously presented detailed ANOVA analysis is not necessary. A similar analysis was performed for the fracture energy testing results with plots of measured vs. predicted values Figures 4.8.2D-4.8.2F and numerical results presented in Figures 4.8.2G-4.8.2I of the Appendix. The results were less scattered, with the prediction model for Dry Fracture Energy producing an R^2 value of 0.67, however, considering the level of variability in the mixture testing results established previously in this report, the usefulness of any regression model must be questioned. Therefore, the same conclusion can be drawn for the fracture energy results, in that the results are independent of asphalt binder properties.

The conclusions formed using this data set are a direct contradiction to the findings of previous work performed by Kanitpong and Bahia [11,12]. Those findings established that binder cohesion was a significant factor in asphalt mixture performance. The source of this deviation is believed to be the change in testing temperature from 10ºC to 25ºC. As previously stated, this change in temperature resulted in tack factor values an order of magnitude larger than those measured in Kanitpong's study. It is hypothesized that the large increase in cohesion changed the mixture component that governs failure. The failure of an asphalt mixture occurs through failure of the weakest component of the mix, either the aggregate, cohesive failure of the asphalt binder, or adhesive failure at the asphalt aggregate interface. The large increase in binder cohesion due to the temperature change reached such a level that the asphalt binder was no longer the weakest component of the mix, forcing failure to occur through the aggregate or asphalt/aggregate adhesion. This phenomenon was visually observed to some extent through visual inspection of test samples. Certain test samples exhibited cracked coarse aggregate through the zone of failure. Further investigation into the affects of asphalt/aggregate adhesion are needed to confirm this hypothesis, however that testing is beyond the scope of this project. In conclusion, deviations from past results were caused by the increase in cohesive strength of the asphalt which rendered cohesion an insignificant factor in asphalt mixture performance in the indirect tension test.

4.9: Effects Mixture and Binder Properties on Tensile Strength Testing Results

Regression analysis was used to examine the contribution to the mixture properties of proportion of fine materials in the aggregate blend, proportion of natural sand in the fine aggregate, and air voids to the parameters of dry and wet tensile strength, TSR, and reduction in average tensile strength. Furthermore, a separate regression analysis incorporating the binder properties reported in Section 4.7 was performed. Both analyses were conducted in hopes of better quantifying the effects of these materials and their properties on mixture results.

4.9.1: Effects of Mixture Properties on Tensile Strength Testing Results

The use of the air voids and properties of the aggregate blend discussed in Section 3.1 allows for definition of quantifiable measures for some of the mixture properties identified as having significant effects on moisture damage. Specifically, incorporation of the proportion of fine aggregates and natural sands in the fine aggregate addresses the previously qualitative measure of aggregate gradation. Aggregate gradation was identified as a significant factor in the ANOVA analysis presented previously. It is hoped that a more quantifiable definition of the aggregate gradation of each mix will provide a means to evaluate the contribution of the fine aggregates and natural sands to the effect of gradation. Air voids were identified in the literature review as having a significant effect on tensile strength. Therefore, the air voids of each sample were included in the regression analysis to evaluate if the results of the testing were consistent with the findings in the work performed by Dukatz [9]. The results of the regression analysis are presented in Figures 4.31-4.33 as plots of Measured vs. Predicted Values for the parameters of dry tensile strength, wet tensile strength, TSR, and reduction in TSR due to conditioning. Numerical results are provided in the Appendix (Figures 4.9.1A – 4.9.1D). Analysis was not performed on the Fracture Energy parameter due to the variability of the results.

Figure 4. 31: Measured Vs. Predicted Values for Tensile Strength

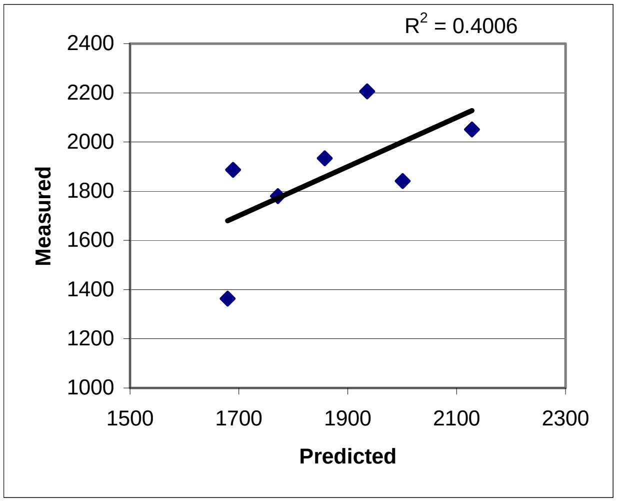


Figure 4. 32: Measured vs. Predicted Values for TSR

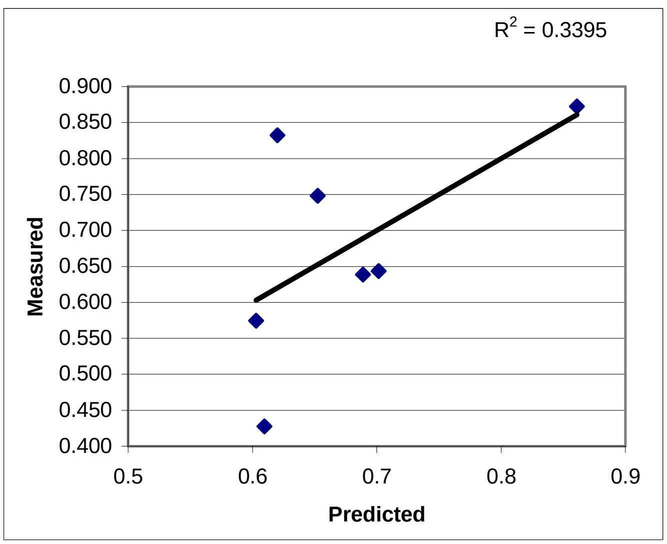


Figure 4. 33: Measured Vs. Predicted Values for Wet Tensile Strength

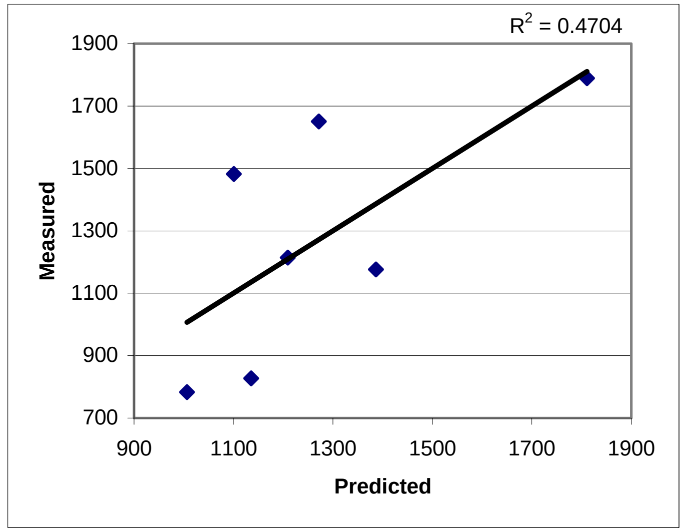
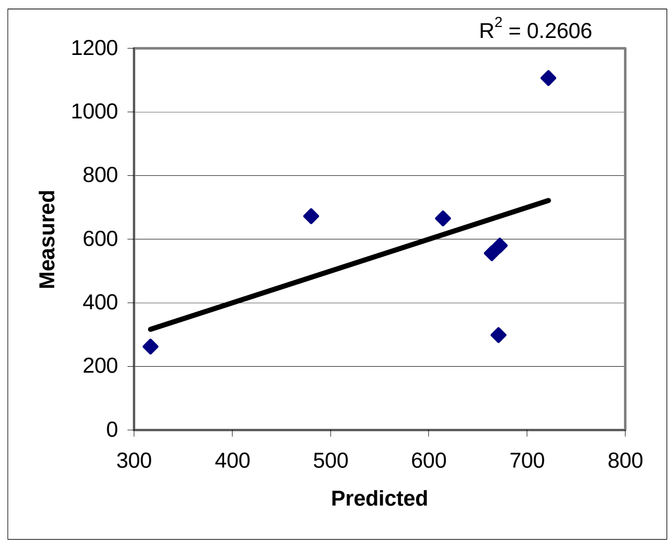


Figure 4. 34: Measured vs. Predicted Values for Tensile Strength Reduction



The results of the regression analysis show that no significant relationship can be established between the factors of air voids and defined components of the aggregate blend, and the tensile strength testing results. To some extent, these results were expected. The air voids were expected to be insignificant because of the efforts spent limiting the variation between unconditioned and conditioned specimens for the same mixture. If test samples were prepared using only the ASTM D4867 specified range of 6%-8% [1], it is expected that the effects of air voids would be much more profound. Furthermore, the definition of gradation using the fine aggregate proportion and percent natural sand in the fines serves as an intermediate step in using the mixture components to characterize overall mixture behavior. It is expected that in order to establish a strong relationship between the effects of fine aggregate properties on tensile strength, the resistance of the fine aggregate asphalt mastic and how it changes in the presence of moisture must be quantified. Proposed work to take this next step will be discussed in the recommendations and conclusions section in Chapter 6.

4.9.2: Combined Effects of Mixture and Binder Properties on Tensile Strength Parameters

A regression analysis was performed for the combined effects of binder properties described in Section 4.7 and the properties of the mixture and aggregate blend discussed in Sections 3.1 and 4.8.1. This analysis was conducted to examine the combined effects on all of the measured properties including the dry tensile strength, wet tensile strength, TSR, and tensile strength reduction due to conditioning. Based on the results of previous regression analyses, it was expected that no strong correlations would be found, however it was anticipated that the relationships would be marginally better due to inclusion of the constituents of the mix: aggregate, asphalt, and air voids in the analysis. The presentation of results is similar to previous regression analyses in that plots of the Measured vs. Predicted values for the tensile strength parameters are provided in Figures 4.35 – 4.38. Numerical results of the analysis are provided in the Appendix (Figures 4.9.2A – 4.9.2D). Again, the fracture energy results were not included in the analysis because of their variability.

Figure 4. 35: Measured vs. Predicted Values for Dry Tensile Strength

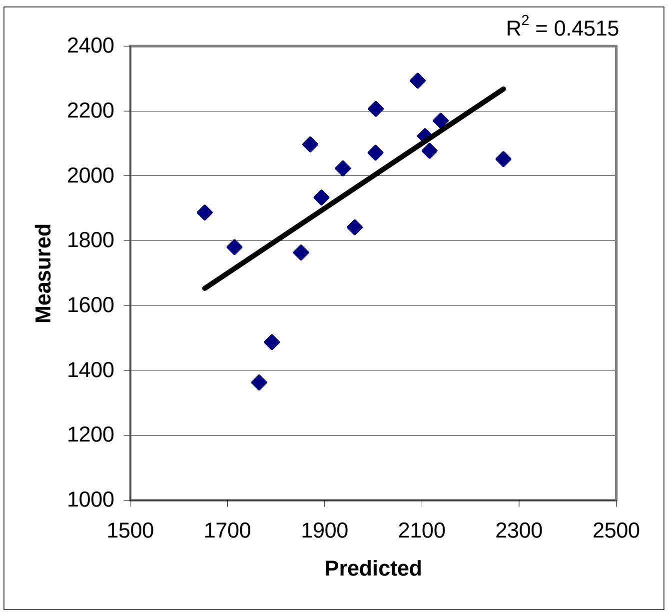
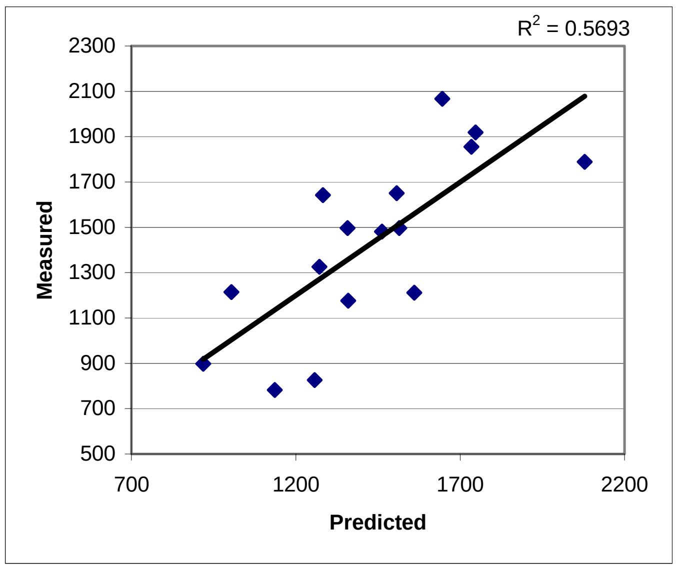


Figure 4. 36: Measured Vs. Predicted Values for Wet Tensile Strength



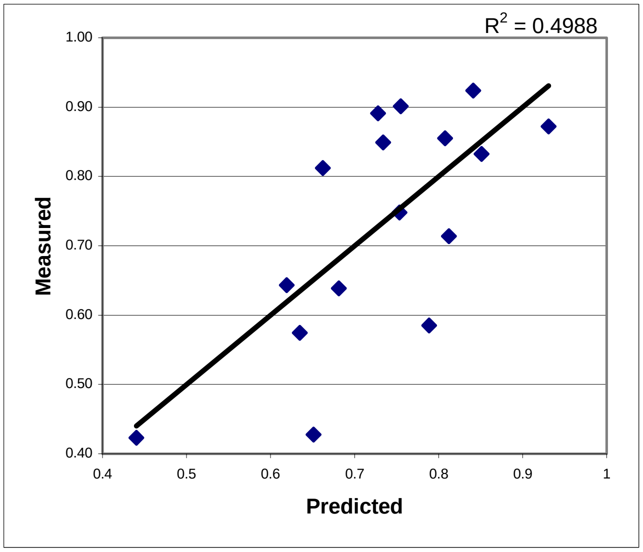


Figure 4. 37: Measured vs. Predicted Values for TSR

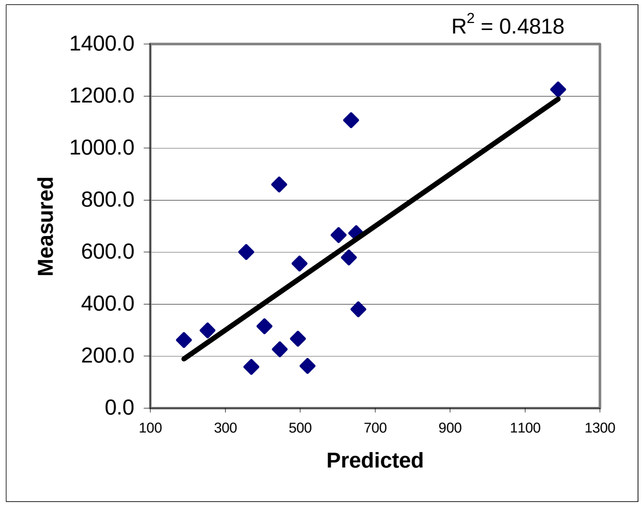


Figure 4. 38: Measured vs. Predicted Values for Tensile Strength Reduction

Examination of the plots of measured vs. predicted values show marginal improvement in the correlation between measured and predicted values. However, these improvements were not sufficient to identify significant relationships. These findings were reinforced by the numerical results of the regression analysis which show the materials' variables to be insignificant at a level of 95% confidence. Significance was defined by the p value for the model or any factor being less than 0.5.

The results of this and previous regression analyses reinforce the hypothesis that the properties of the asphalt binder and aggregates and the effects of moisture on them must be measured and quantified in more detail to accurately relate their effects to moisture damage as measured by tensile strength testing. For example the fine aggregate in the blend, or the proportion of natural sand in the fine aggregate, need to be better defined by quantifiable measures and these could be related to moisture damage. Also, the role of aggregate type in aggregate-asphalt adhesion must be investigated. It is also important to consider the physio-chemical interaction between the asphalt and aggregate into the evaluation of the mechanisms of moisture damage. Evaluation of these factors is beyond the scope of this study, however, possible methods and corresponding literature will be sited in the recommendations section of this report.

Chapter 5: Economic Analysis of Test Methods

5.1: Introduction

The quality of a test method for evaluating the moisture susceptibility of asphalt mixtures for use by state agencies and contractors is dependent on how cost effective it is. A cost effective test method should allow to clearly and reliably, identify the effects of moisture while minimizing agency/contractor investment in terms of training, time, and labor. The experimental results presented previously in this report indicate that the current stripping test and fracture energy testing procedures produce results with questionable reliability. Furthermore, informal comparison of the coefficient of variation for TSR testing using WisDOT procedures with those used in this study was inconclusive in determining if the more advanced testing method resulted in lower test variability. From a technical aspect, this study is the baseline for Wisconsin's moisture damage testing beyond ASTM D-4867. The research provided many promising results, but few firm conclusions. In making the decision to further pursue certain aspects of these results, consideration must be given to both the potential technical benefit and to the economic and logistical issues associated with each test method. Table 5.1 was constructed to compare the initial cost of equipment and unit cost per test related to each test method as compared to current practices as specified by ASTM D4867.

Table 5. 1: Comparison of Three Test Methods to Predict Moisture Damage

	IDT Testing	TSR (ASTM D4867)	Stripping Test
Equipment	SuperPave IDT Test Apparatus	Testing Machine	Gyratory Shaker Bath
Cost	$70,000.00	None	$5,000
Additional Technician Training	Short Course and on-site training for SuperPave IDT	None	Minimal (Short demonstration and copy of procedure in lab manual)
Time	32 hours (classroom) 8 hours laboratory	0 hours	2 hours
Initial Investment	**$ 70,800.00**	**$ -**	**$ 5,080.00**
Time Invested (Hrs)			
Sample Preparation/Monitoring	3	3	5
Compaction	3	3	NA
Volumetrics	2	2	NA
Conditioning	1	1	NA
Cutting	2	NA	NA
Testing	5	3	1
Total Time /Mix Design	16	12	6
Cost of Testing/Mix Design	$ 320.00	$ 240.00	$ 120.00
Technician Rate	$20.00/hr		

In order to make use of this analysis, the assumption was made that any modifications to the test methods to improve reliability would not result in significant changes in time requirements. The analysis shows for moisture damage testing on the agency/contractor level a screening test (such as the stripping test) is the most cost effective. From an economic perspective, the agency can realize a maximum of 60% reduction in time investment relative to fracture energy testing and a 40% reduction relative to TSR testing by implementing the stripping test. Over time, this reduction in testing requirements would outweigh the initial costs associated with the mechanical testing of every mix. The remainder of this chapter will be dedicated to further describing the logistical issues associated with a screening test, the advanced TSR testing, and FER testing.

5.2: The Use of a Screening Test for Evaluation of Asphalt Mixtures

Common practices in Wisconsin define a mix with a tensile strength ratio greater than 0.70 as suitable in resistance to moisture damage. TSR testing is required for all mixes placed on WisDOT projects [27]. It seems however impractical to require extensive moisture damage testing for mixes produced using aggregates from sources that consistently show no moisture damage problems. Results of this study and others have found adhesion of asphalt to an aggregate surface to be significantly affected by aggregate source. The current WisDOT practice is therefore uneconomical in terms of commitment of time and resources, especially considering the questions regarding the accuracy of the TSR test results.

A more sensible approach would be to use a screening test to identify asphalt mixes that are not moisture susceptible and waive their tensile strength testing requirement. A practical example of the benefit of a screening test can be realized by use of the WisDOT Mix Design Database [23]. In 2004, 210 mix designs were submitted to the WisDOT central office for mix design verification. Of those mixes, 150 exceeded the TSR requirement of 70% by more than 5% without using any modification to mitigate the effects of moisture damage. Using the information listed in Table 5.1 above, these mixes required 1800 hours of additional testing by agency staff or contractors to produce TSR results. This time investment could have been reduced by 30-50% if the screening test would have been used in conjunction with the TSR to evaluate moisture damage. Based on this analysis, the development of an efficient screening test would be beneficial to WisDOT and industry from an economic standpoint. The evaluation of moisture damage using a combination of a screening test, a historical knowledge of aggregate sources and limited TSR testing seems to be an ideal interim solution until more advanced test methods that better relate moisture susceptibility in the laboratory to pavement performance are fully developed.

5.3: The Use of the Advanced IDT Procedures to Test HMA Tensile Strength

The only possible benefit realized by using the advanced IDT testing conducted in this study to obtain the TSR, as compared to using the conventional TSR testing, is the increase in the reliability of the test results. However, the limited data collected in this study for the comparison of the two methods could not conclusively identify a significant increase in reliability. This study has also shown that the use of advanced IDT testing methods would require at least an additional 2 hours of labor. If a future study found that the advanced IDT methods produced consistently more reliable results than ASTM D4867, this extra time investment would be worth the cost from an agency standpoint. More reliable test results would result in decreased possibility of results providing false/positives, thus minimizing the opportunity for pavements to fail due to distresses caused by moisture damage.

5.4: The Use of Fracture Energy to Evaluate Moisture Damage

The use of fracture energy to quantify moisture damage in asphalt mixtures is very different than the tensile strength approach in that it provides information regarding changes in both stresses and strains in the mixture due to moisture conditioning. This information is clearly an improvement on the use of only tensile strength. However, to realize this advantage further development of the test methods would be necessary to reduce the variability of the fracture energy measurements presented in this study. Also, for the benefits of this new technology to be fully realized, an entire testing framework and performance-based specification must be available for use. Furthermore, there are other logistical requirements in terms of testing time and expertise. Personnel and equipment to perform testing at this level of complexity may not be available for wide spread industry and state agency use, limiting the practicality of this method. Furthermore, the Energy Ratio (ER) only provides information regarding changes in mixture properties due to moisture. To realize the significance of these changes, the change in ER must be linked to field performance of pavements at different traffic levels.

The calibration of ER can be accomplished through the use of pavement performance databases to establish minimum acceptable values of ER for different traffic (ESAL) levels. Such performance-based specification would allow agencies to require the use of anti-stripping additives for any mix with an ER below the minimum limit, for a given traffic level. Previous research in this area shows that this is a promising concept, however given the required equipment, technical expertise, and quality pavement performance databases, the implementation of this testing is not feasible in the near future.

In conclusion, consideration of the economic and logistical of further development and possible implementation of each test method plays an integral role in the selection of the appropriate test. Tests more suited for agency use in design or more appropriate for use as a quality control/quality assurance test cannot be properly identified without incorporating both technical and practical issues in the evaluation process

Chapter 6: Findings and Recommendations

6.1: Findings

Based on the analysis of results collected in this study, the following is a summary of the findings that can be stated:

1. The stripping test performed in this study on loose mixtures is able to detect the possibility of weak adhesion between asphalt binders and aggregates and thus differentiate between aggregate types. It can also be used to identify the presence of anti-stripping additive in moisture susceptible mixes. The high variability within the test results, however, prevented the definition of a threshold value to be used as a screening test for mixtures. Establishment of such a threshold using the current test procedures could result in incorrect screening of mixtures due to false positives caused by the variability of the test.

2. The ASTM D4867 tensile strength test was able to distinguish between mixes using aggregates with different affinity for asphalt. This test was also able to identify the added moisture resistance caused by the use of liquid anti-stripping additives. Factors identified as significant through analysis of variance performed on the test results were consistent with expectations.

3. The fracture energy test was also able to identify moisture susceptible mixes and identify the contributions of liquid anti-stripping additives. However, similar to the stripping test, suffered from high variability between test results for individual mixes. Problems with excessive variability were reflected in the results of the analysis of variance, which was unable to identify effects consistent with expectations, based on engineering knowledge. The complexity of the test protocol and the variability of the results make it difficult to recommend this test as a better alternative than the ASTM D4867.

4. For the testing temperature used in this study, aggregate properties and/or asphalt/aggregate adhesion control mixture failure in indirect tension. Regression analysis on the effects of the binder properties of cohesion and the Superpave rutting and fatigue parameters found that their effect on mixture tensile strength is insignificant. The extent to which adhesion and aggregate properties effect testing results needs to be investigate further.

5. Informal comparison of test results measured in this study to previous work suggests that the advanced testing methods used in this study may reduce the variability of the TSR test. Results using these methods resulted in a lower average and peak coefficient of variation. A more formal comparative study is needed to confirm these initial findings.

6. Investigation of the individual and combined effects of binder properties, fine aggregate proportion and composition in the mixture, and percent air voids found that these factors did not have significant effects on the tensile strength of the mixtures tested.

7. The time and cost analysis of the three tests indicates that both the agency and industry could realize a significant economic benefit with the implementation of a screening test into current

moisture susceptibility testing requirements. The use of a screening test to waive the mechanical testing requirement for exceptional mixes would significantly reduce time and resources needed for mix moisture susceptibility testing.

6.2: Recommendations

1. The stripping test as defined in this report is not suitable for use as a screening test for the agency. The variability of the test is too high to be used as a surrogate test to predict moisture susceptibility at an acceptable level of risk for the agency or industry. There is significant economic benefit to the implementation of a simple screening test, however no alternative can be recommended at this time.

2. A comparative study between the TSR testing procedures used in this study and current WisDOT procedures is needed to investigate if the new procedures reduce the variability of the TSR test. The same mixes must be tested using the same operator to fully understand if the procedure used in this study does in fact reduce variability of test results. The reduction in variability would need to be significant to justify the extra expense and time involved in using the new procedure.

3. Since the Fracture Energy Test is a more fundamental test, further research is needed to reduce the variability of this test. The fracture energy parameter shows promise in its ability to quantify moisture damage, as seen in the plots of behavior of various mixtures. However, the variability of the test must be reduced before further investigation into development of any specification involving this parameter can begin.

4. The role of adhesion in the performance of the mixtures used in this study must be fully understood. The adhesion between the various asphalts and aggregate sources used in this study must be incorporated into the results of mixture testing to define the contributions of the asphalt binder to mixture performance. Previous work by Kanitpong has used the PATTI test to measure asphalt aggregate adhesion and the how moisture effects the bond. A similar procedure should be used to test the asphalts and aggregates used in this study.

5. The effect of the mastic (fine aggregate and asphalt) should be quantified using unconditioned and conditioned torsion cylinders. This testing would provide an opportunity to evaluate the effect of the amount of natural sand in the aggregate blend. Investigation of the mastic would also allow for better understanding of the significance of moisture damage in the fine aggregate. Testing of torsion cylinders has been performed previously by Massad and Little, and also at UW Madison. This work should be reviewed and the procedures adjusted to incorporate the effects of moisture.

6. To fully understand the moisture damage phenomena the physio-chemical interaction between the asphalt binder and aggregate must be investigated further. Previous research by Ken Thomas of Western Research Institute provides a starting point for this investigation. Another aspect of asphalt-aggregate interaction that should be investigated

is the concept of surface tension and its effects on adhesive strength. Work has been done by Little at Texas A&M in this area.

References

1. ASTM D4867. "Standard Test Method for Effect of Moisture on Asphalt Concrete Paving Mixtures." Annual Book of ASTM Standards, Vol. 04.03, American Society for Testing and Materials, Philadelphia, PA.

2. Bahia, H., Ahmad, S. " Evaluation and Correlation of Lab and Field Tensile Strength Ratio (TSR) Procedures and Values in Assessing the Stripping Potential of Asphalt Mixes. WisDOT Study 0092-95-04. Wisconsin Department of Transportation, Madison, WI 1999. http://ntl.bts.gov/lib/16000/16300/16308/PB2000103391.pdf

3. Bahia, H., Anderson D. "The New Rheological Properties of Asphalt Binders: Why Are They Required and How Do They Compare To Conventional Properties.: Physical Properties of Asphalt Cement Binders: ASTM STP 1241, John C. Harden, Ed., American Society of Testing and Materials. Philadelphia, PA, 1994. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=aw%3Abahia+AND+aw%3Aanderson+AND+yr%3A1995&r=1&d=tr&p=1&z=1&s=yr&o=1&new=n

4. Bahia, H., Kanitpong, K. "Evaluation of the Extent of HMA Moisture Damage in Wisconsin as it Relates to Pavement Performance." WHRP Report to WisDOT Project 0092-01-03. Wisconsin Highway Research Program. Madison, WI, 2004. http://www.whrp.org/Research/publications/Final%20Reports/WHRP%2003-07%20Evaluation%20of%20the%20Extent%20of%20HMA%20Moisture%20Damage%20in%20Wisconsin%20as%20it%20Relates%20to%20Pavement%20Performance.pdf

5. Bahia, H., Stakston, A. "The Effect of Fine Aggregate Angularity, Asphalt Content, and Performance Graded Asphalts on Hot Mix Asphalt Performance." WHRP Report to WisDOT, Project 0092-45-98. Wisconsin Highway Research Program. Madison, WI, 2003. http://www.whrp.org/Research/publications/Final%20Reports/WHRP%2003%2004%20The%20Effect%20of%20Fine%20Aggregate%20Angularity,%20Asphalt%20Content%20and%20Perfomance%20Graded%20Asphalts%20on%20Hot%20Mix%20Asphalt%20Performance.pdf

6. Birgisson, Roque, Page. "The Use of a Performance-Based Fracture Criterion for the Evaluation of Moisture Susceptibility of Hot Mix Asphalt." Transportation Research Record. Journal of the Transportation Research Board. Transportation Research Board 2004. Taken from the Annual Meeting CD ROM.

7. Birgisson, B., Roque, R., Tia, M., Masad, E. "Development and Evaluation of Test Methods to Evaluate Water Damage and Effectiveness of Anti-Stripping Agents." Chapters 2,3,6.University of Florida Project No. 4910-4504-722-12. Florida Department of Transportation, Tallhassee, FL, 2005. http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_SMO/FDOT_BC354_11_rpt.pdf

8. D'Angelo, J., Anderson, R.M.. "Material Production, Mix Design, and Pavement Design Effects on Moisture Damage." Proceedings of Moisture Sensitivity of Asphalt Pavements-A National Seminar, February 4-6, 2003. Transportation Research Board 2003. http://trb.org/publications/conf/reports/moisture_seminar.pdf

9. Dukatz, E., Phillips, R.. "The Effect of Air Voids on the Tensile Strength Ratio." Journal of the Association of Asphalt Paving Technologists. Vol. 56. Pages 517-554. Association of Asphalt Paving Technologists, White Bear Lake, MN, 2003.

10. Huang, Y. "Pavement Analysis and Design, Second Edition." Prentice Hall 2004.

11. Kanitpong, K., Bahia, H. "Role of Adhesion and Thin Film Tackiness of Asphalt Binders in Moisture Damage of HMA." Journal of the Association of Asphalt Paving Technologists. Vol 72. Pages 502-528. Association of Asphalt Paving Technologists, White Bear Lake, MN 2003. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Abahia&r=1&d=tr&p=22&z=1&s=yr&o=1&new=n

12. Kanitpong, K., Bahia, H. "Relating Adhesion and Cohesion of Asphalts to Effect of Moisture on Asphalt Mixtures' Laboratory Performance." Transportation Research Record. Journal of the Transportation Research Board. No. 1901. Pages 33-43. Transportation Research Board, Washington DC, 2005. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Akanitpong&r=1&d=tr&p=2&z=1&s=yr&o=1

13. Kanitpong, K., "Evaluation of the Roles of Adhesion and Cohesion Properties of Asphalt Binders in Moisture Damage of HMA." Dissertation, University of Wisconsin – Madison Department of Civil and Environmental Engineering, 2004. http://madcat.library.wisc.edu/cgi-bin/Pwebrecon.cgi?v1=3&ti=1,3&BOOL1=all%20of%20these&CNT=50&SAB1=kanitpong&FLD1=Author%20Name%20%28NKEY%29&PID=13753&SEQ=20061229183002&SID=1

14. Kholsa, N., Sadasivam, S., and Malpass, G. "Performance Evaluation of Fine Graded Superpave Mixtures for Surface Courses." Report Submitted to North Carolina Department of Transportation and Federal Highway Administration. North Carolina State University, Department of Civil Engineering, Raleigh, NC 27695. www.ncdot.org/doh/preconstruct/tpb/research/download/2000-06FinalReport.pdf -

15. LC 25-009. "The Evaluation of Binder Resistance to Stripping for a Given Aggregate Surface." Quebec Department of Transportation, 2002.

16. Lytton, R. Masad, E., Zollinger, C., Bulut, R., Little, Dallas., "Measurements of Surface Energy and its Relationship to Moisture Damage." Report 0-4524-2 Submitted to Texas Department of Transportation and Federal Highway Administration. Texas

Transportation Institute, Texas A&M University, College Station, TX. 2005. http://tti.tamu.edu/documents/0-4524-2.pdf

17. Montgomery, D. "Design and Analysis of Experiments 6th Edition." John Wiley & Sons, Inc. 2005.

18. NCHRP Project 9-34, Abstract. "Improved Conditioning Procedure for Predicting the Moisture Susceptibility of HMA Pavements." Transportation Research Board, Washington DC, 2002. http://www.trb.org/trbnet/projectdisplay.asp?projectid=968

19. NCHRP Report 444, "Compatibility of a Test for Moisture-Induced Damage with Superpave Volumetric Mix Design.", Transportation Research Board, National Research Council, Washington DC. 2000. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Anchrp+kw%3A444&r=1&d=tr&p=0&z=1&s=yr&o=1

20. Roberts, F., Kandhal, P., Brown, E., Lee, D., Kennedy, T. "Hot Mix Asphalt Materials, Mixture Design, and Construction. Second Edition" National Center for Asphalt Technology. NAPA Education Foundation, Lanham Maryland, 1997.

21. Roque, Zhang, Sankar. "Determination of Crack Growth Rate Parameters of Asphalt Mixtures Using the Superpave IDT." Journal of the Association of Asphalt Paving Technologists Vol. 68 pages 404-433. Asphalt Paving Technologists, White Bear Lake, MN, 1999. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Aroque&r=1&d=tr&p=67&z=1&s=yr&o=1

22. Roque, Birgisson. "Guidelines for used of Modified Binders." University of Florida Project 4910-4504-964-12. Florida Department of Transportation, Tallahassee, FL, 2005. http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_SMO/FDOT_BC354_77_rpt.pdf

23. Ryan, Judith. "Wisconsin Asphalt Mixture Design Database." Wisconsin Department of Transportation. Madison, WI. 2004.

24. Sebaaly P., Hitti, E., Weitzel, D. "Effectiveness of Lime in Hot-Mix Asphalt Pavements." Transportation Research Record No. 1832. Transportation Research Board, Washington DC, 2003. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Asebaaly&r=1&d=tr&p=23&z=1&s=yr&o=1

25. Solaimanian, M., Harvey, J., Tahmoressi, T., Tandon, V. "Test Methods to Predict Moisture Sensitivity of Hot-Mix Asphalt Pavements." Proceedings of Moisture Sensitivity of Asphalt Pavements-A National Seminar, February 4-6, 2003. Transportation Research Board 2003. http://trb.org/publications/conf/reports/moisture_seminar.pdf

26. Wisconsin Department of Transportation: PDI Survey Manual. Wisconsin Department of Transportation, Madison, WI 2001

27. Wisconsin Department of Transportation Standard Specifications. Section 460.2. Wisconsin Department of Transportation, Madison, WI. 2005. http://roadwaystandards.dot.wi.gov/standards/stndspec/index.htm

28. Zhang, Z., Roque, R., Birgisson, B., Sangpetngam, B. "Identification and Verification of a Suitable Crack Growth Law." Journal of the Association of Asphalt Paving Technologists. Vol. 70, Pages 206-241. Asphalt Paving Technologists, White Bear Lake, MN, 2001. http://ntlsearch.bts.gov/tris/search.do?b1=1&f1=0&t1=kw%3Aroque&r=1&d=tr&p=60&z=1&s=yr&o=1

Appendix: Summary of Figures and Graphs

Mix Design Data

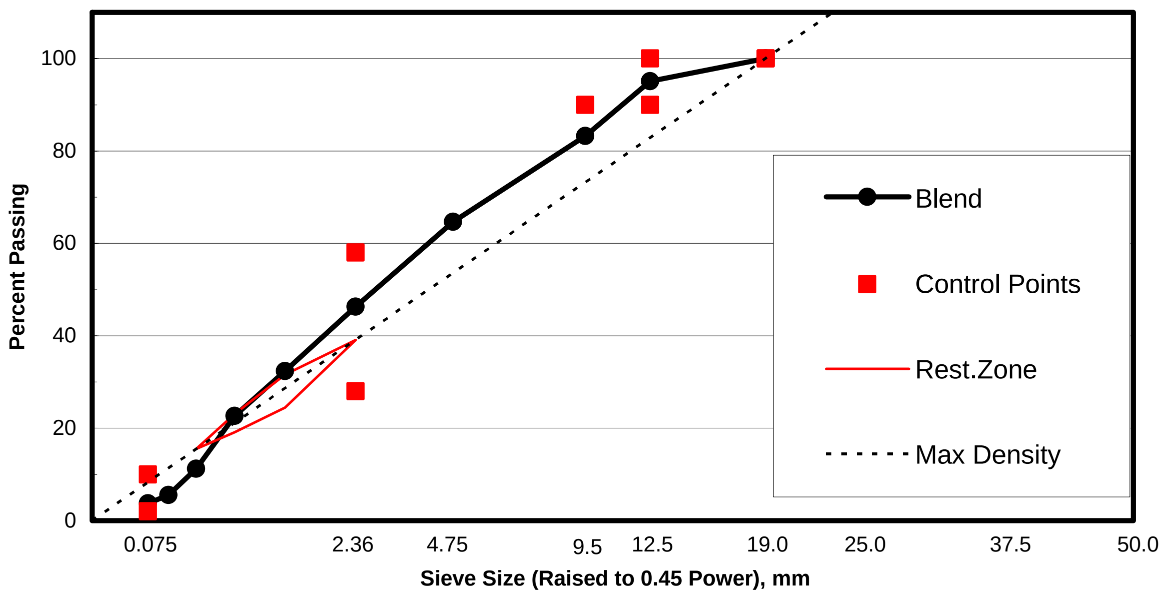
Table 3.1: Summary of Mix Designs							
	Percent Passing for Each Mix						
Sieve Size (mm)	**Granite 1 Fine**	**Granite 1 Coarse**	**Granite 2 Fine**	**Gravel Fine**	**Gravel Coarse**	**Limestone South Fine**	**Limestone NE Fine**
19	100.00	100.00	100.00	100.00	100.00	99.73	100.00
12.5	95.10	93.27	95.70	96.48	91.21	92.33	96.79
9.5	83.25	76.69	85.80	88.52	73.14	83.82	88.48
4.75	64.65	49.23	66.05	68.19	43.16	65.23	71.57
2.36	46.30	32.61	48.75	49.15	30.44	50.14	54.54
1.18	32.37	22.50	36.23	33.67	21.78	40.96	42.76
0.6	22.68	15.69	25.44	20.96	14.30	33.67	33.60
0.3	11.23	8.84	9.59	10.87	8.08	18.41	18.05
0.15	5.55	5.41	4.82	6.17	5.17	8.18	7.33
0.075	3.72	4.04	3.79	4.68	4.13	5.30	4.86
Asphalt Source	Koch	Koch	MIF	CRM	CRM	Koch	Koch
Asphalt Content	6.24%	5.15%	5.90%	5.30%	4.35%	4.80%	5.90%

Aggregate Properties of Individual Mixes

Granite Fine Mix 1

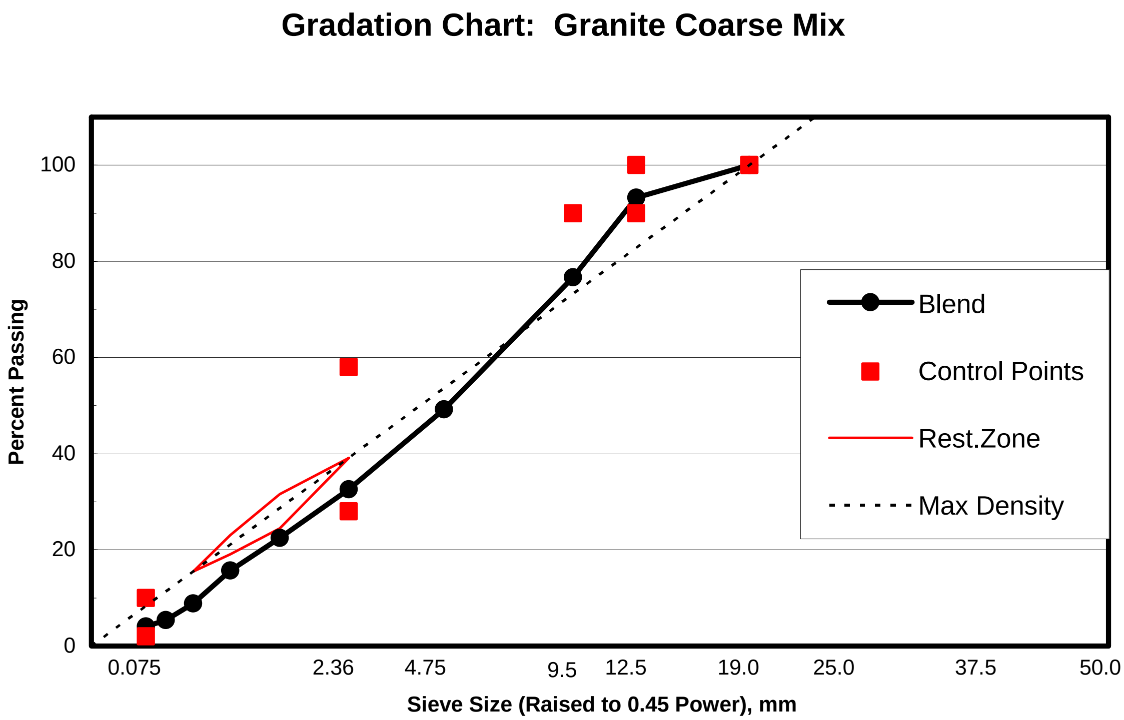
Proportion of Natural Sand in the Fine Aggregate							
Granite Fine Mix 1							**Blend**
Material	1/2" Crush Rock	3/8" Crush Rock	1/4" Screenings	Man Sand	Blend Sand		
% in Blend	25%	15%	15%	30%	15%		100%
R4	0.94	0.67	0.02	0.02	0.06	**CA Portion**	35.35%
P4	0.06	0.33	0.98	0.98	0.94	**FA Portion**	64.65%
Sand Fraction	1.50%	4.95%	14.70%	29.40%	14.10%	**FA Portion**	64.65%
Sand Proportion	**21.81%**						

Gradation Chart: Granite Fine Mix 1



Granite Coarse Mix 1

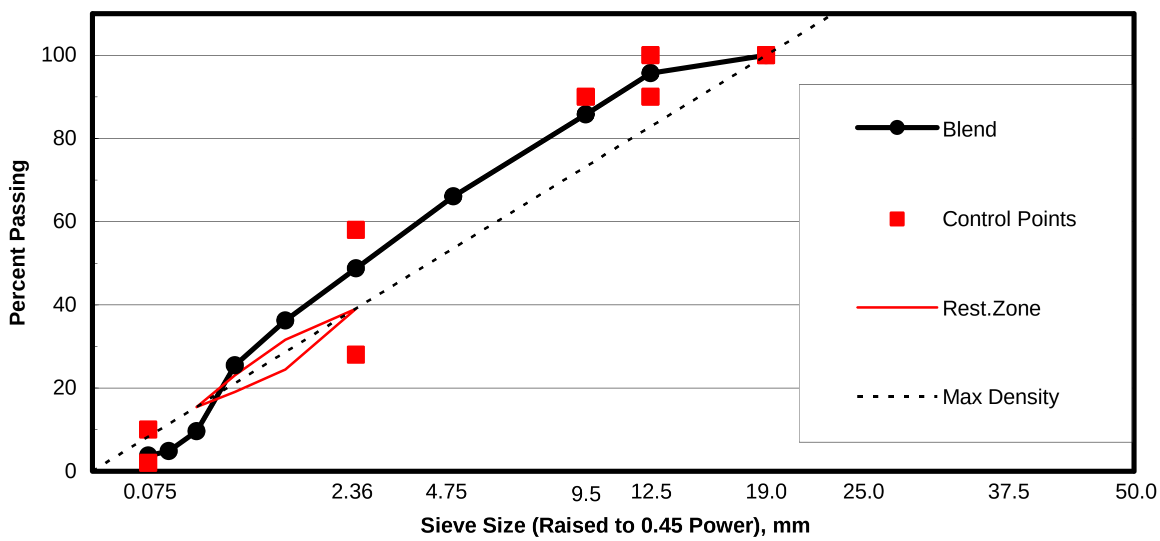
Granite Coarse Mix 1							Blend
Material	1/2" Crush Rock	3/8" Crush Rock	1/4" Screenings	Man Sand	Blend Sand		
% in Blend	35%	25%	25%	7%	8%		100%
R4	0.94	0.67	0.02	0.02	0.06	**CA Portion**	50.77%
P4	0.06	0.33	0.98	0.98	0.94	**FA Portion**	49.23%
Sand Fraction	2.10%	8.25%	24.50%	6.86%	7.52%	**FA Portion**	49.23%
Sand Proportion	**15.28%**						



Granite Fine Mix 2

Granite Fine Mix 2								Blend
Material	1/2" Crush Rock	3/8" Crush Rock	3/16" Screenings	Man Sand	Blend Sand	RAP		
% in Blend	20%	15%	20%	15%	20%	10%		100%
R4	94.00%	71.00%	3.00%	0.00%	6.00%	0.27	**CA Portion**	33.90%
P4	6.00%	29.00%	97.00%	100.00%	94.00%	0.73	**FA Portion**	66.10%
Sand Fraction	1.20%	4.35%	19.40%	15.00%	18.80%	7.30%	**FA Portion**	66.05%
Sand Proportion	**28.46%**							

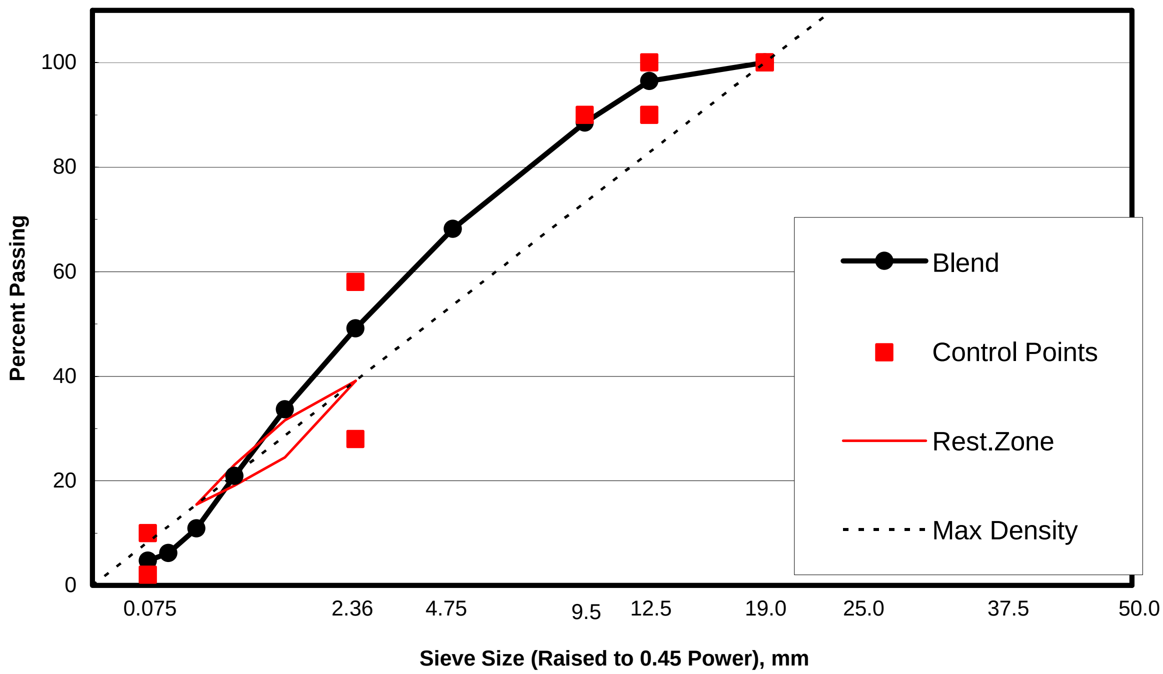
Gradation Chart: Granite Fine Mix 2



Gravel Fine Mix

Gravel Fine Mix							Blend
Material	5/8" x 3/8" HF Chip	3/8" x 1'/4" Chip	Man Sand	Natural Sand	RAP		
% in Blend	12%	15%	28%	31%	14%		100%
R4	97.50%	84.50%	2.90%	11.80%	21.20%	**CA Portion**	31.80%
P4	2.50%	15.50%	97.10%	88.20%	78.80%	**FA Portion**	68.20%
Sand Fraction	0.30%	2.33%	27.19%	27.34%	11.03%	**FA Portion**	68.19%
Sand Proportion	**40.10%**						

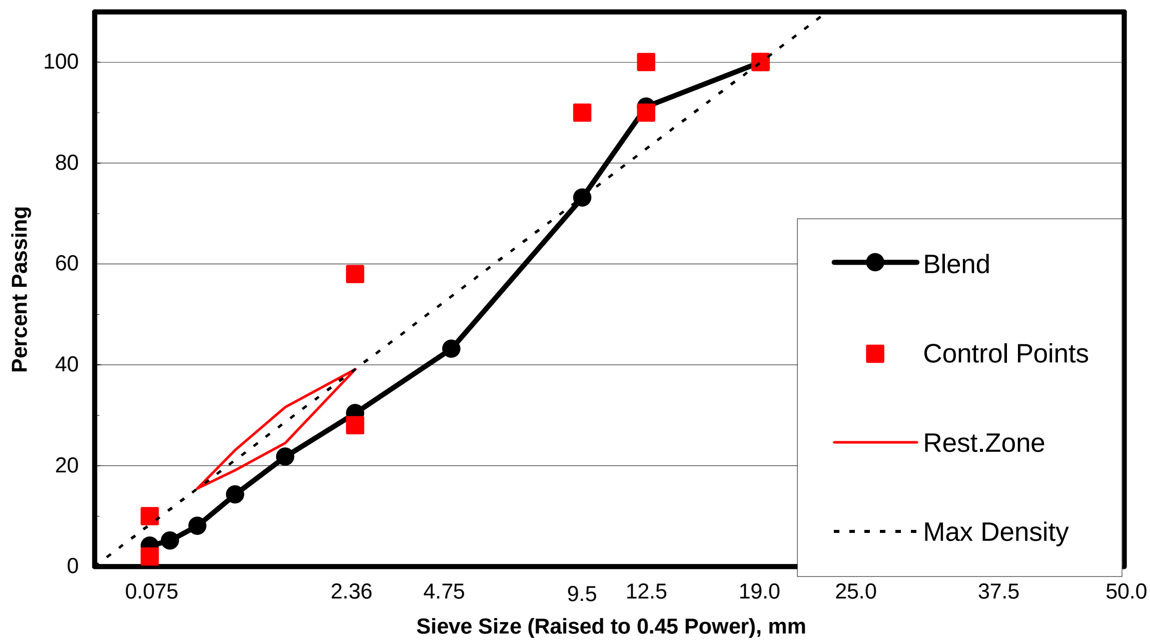
Gradation Chart: Gravel Fine Mix



Gravel Coarse Mix

Gravel Coarse Mix							Blend
Material	5/8" x 3/8" HF Chip	3/8" x 1'/4" Chip	Man Sand	Natural Sand	RAP		
% in Blend	30%	26%	10%	20%	14%		100%
R4	97.50%	84.50%	2.90%	11.80%	21.20%	**CA Portion**	56.84%
P4	2.50%	15.50%	97.10%	88.20%	78.80%	**FA Portion**	43.16%
Sand Fraction	0.75%	4.03%	9.71%	17.64%	11.03%	**FA Portion**	43.16%
Sand Proportion	**40.87%**						

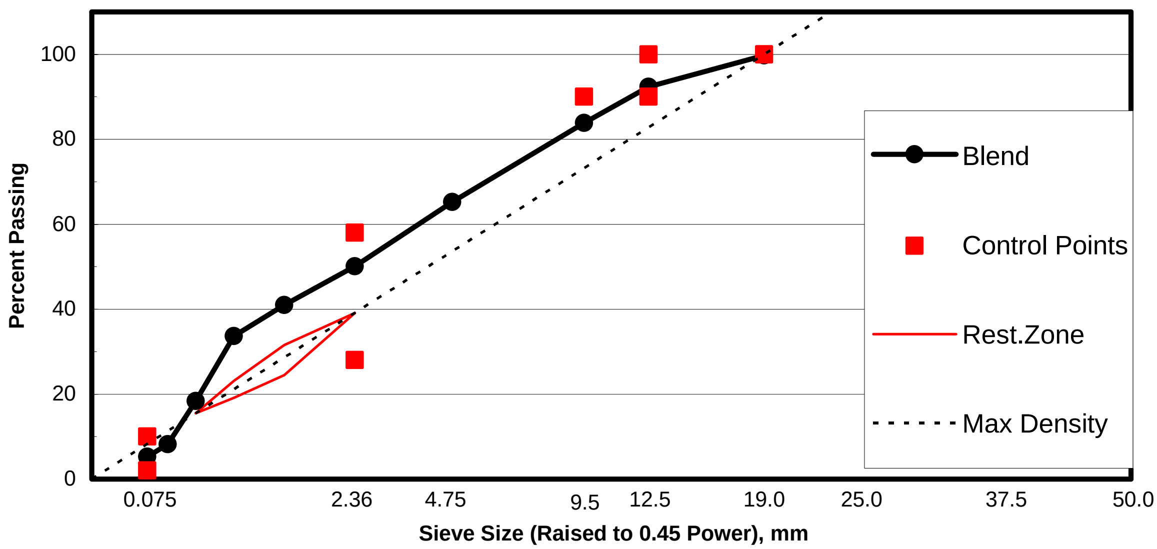
Gradation Chart: Gravel Coarse Mix



Limestone South Fine

Limestone South Fine Mix								Blend
Material	5/8" Chip	3/8" Chip	1/4" Screenings	Man Sand	Natural Sand	RAP		
% in Blend	8%	18%	5%	27%	29%	13%		100%
R4	89.70%	86.50%	17.00%	27.30%	3.40%	21.70%	**CA Portion**	34.77%
P4	10.30%	13.50%	83.00%	72.70%	96.60%	78.30%	**FA Portion**	65.23%
Sand Fraction	0.82%	2.43%	4.15%	19.63%	28.01%	10.18%	**FA Portion**	65.23%
Sand Proportion	**42.95%**							

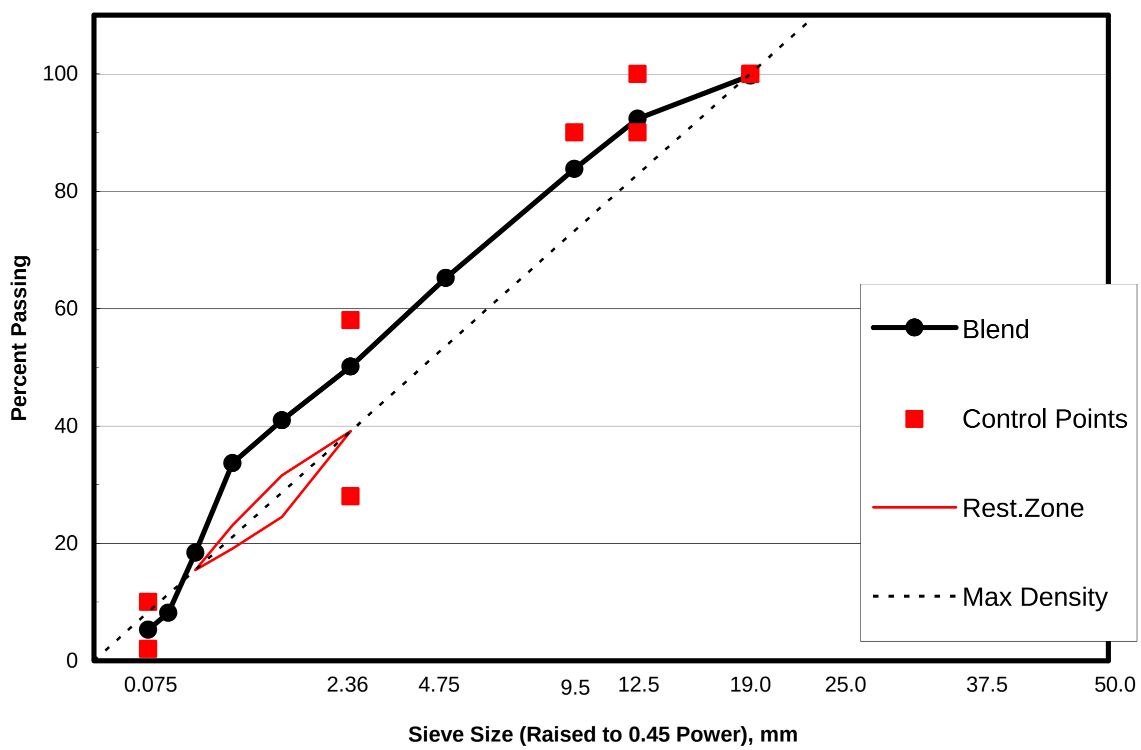
Gradation Chart: Limestone South Mix



Limestone NE Fine

Limestone NE Fine Mix							Blend
Material	5/8" x 1/2" HF Chip	1/2" x 1'/4" Chip	Man Sand	Natural Sand	RAP		
% in Blend	10%	18%	40%	27%	5%		100%
R4	95.20%	86.40%	3.30%	3.00%	24.60%	**CA Portion**	28.40%
P4	4.80%	13.60%	96.70%	97.00%	75.40%	**FA Portion**	71.60%
Sand Fraction	0.48%	2.45%	38.68%	26.19%	3.77%	**FA Portion**	71.57%
Sand Proportion	**36.59%**						

Gradation Chart: Limestone NE Fine Mix



4.1 – 4.1.3: Mechanical Testing Data

Note: Labels in Titles Correspond to the Section of the Report the Table is Referenced in.

Table 4.1A: Summary of Mechanical Testing Results

Mix	Condition	Avg Fracture Energy	Std Dev	Avg Tensile Strength	Std Dev
Granite Fine 1	Unconditioned	9903.2	1258.7	1362.8	49.8
Granite Fine 1 AS		11443.8	1126.8	1487.3	153.4
Granite Coarse 1		7720.6	973.3	1887.0	117.8
Granite Coarse 1 AS		10664.5	563.5	2097.1	189.2
Gravel Fine		7081.1	11.2	2206.2	134.4
Gravel Fine AS		8811.3	1919.3	2023.0	178.3
Gravel Coarse		5635.4	309.6	2051.4	62.5
Gravel Coarse AS		5603.8	700.9	2170.1	181.7
Granite 2 Fine		12939.7	821.2	1780.4	58.2
Granite 2 Fine AS		12819.5	1206.8	1763.9	58.2
Limestone NE Fine		10713.3	580.5	1933.7	90.6
Limestone NE Fine AS		10651.4	1235.1	2071.0	87.5
Limestone NE plus Lime		8819.3	925.5	2293.4	19.0
Limestone NE plus SBS		8916.1	455.7	2123.0	44.0
Limestone South Fine		7977.3	21.7	1841.3	67.1
Limestone South Fine AS		9425.1	1273.2	2077.6	134.8
Granite Fine 1	Conditioned	7245.3	2625.0	782.8	16.0
Granite Fine 1 AS		11698.3	4305.0	1325.4	159.7
Granite Coarse 1		6205.1	472.7	1214.2	98.2
Granite Coarse 1 AS		8889.6	2625.8	1496.9	117.4
Gravel Fine		6868.3	137.0	1650.5	147.3
Gravel Fine AS		8314.4	2279.6	1642.8	152.2
Gravel Coarse		6743.6	2251.5	1789.3	26.6
Gravel Coarse AS		6572.8	1310.3	1855.5	107.9
Granite 2 Fine		10026.6	2259.4	1481.8	122.2
Granite 2 Fine AS		13195.9	3919.2	1497.4	208.1
Limestone NE Fine		3597.6	556.8	826.8	87.4
Limestone NE Fine AS		4165.9	764.6	1211.6	28.2
Limestone NE plus Lime		5184.4	283.4	2066.8	301.1
Limestone NE plus SBS		3124.3	965.7	898.0	77.1
Limestone South Fine		4037.7	923.1	1175.8	71.3
Limestone South Fine AS		7201.2	1237.4	1919.5	135.7

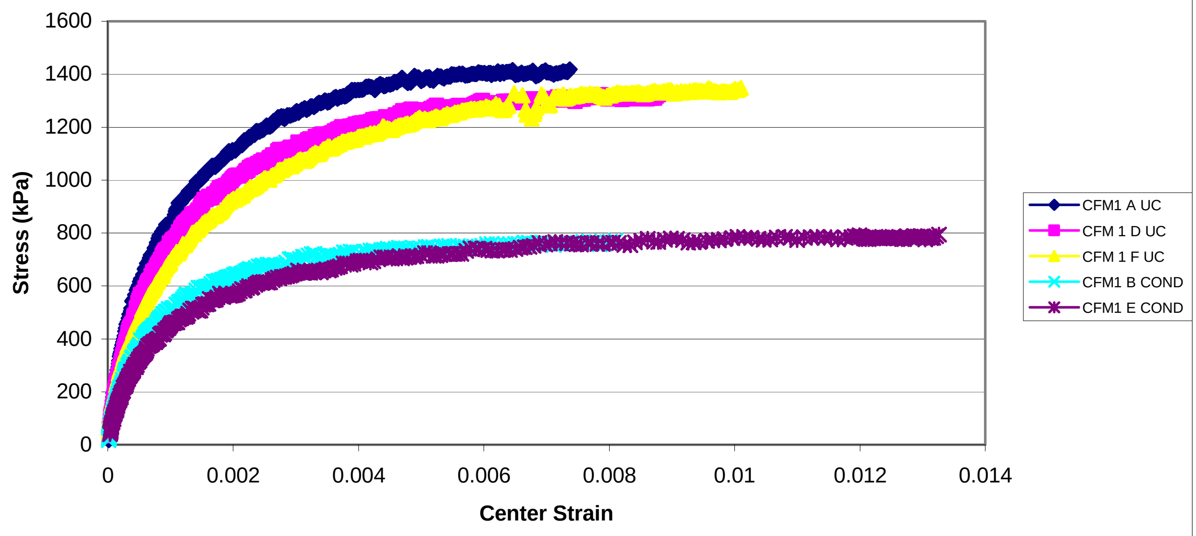
Table 4.1B: Summary of Aggregate Proportions in Job Mix Formula

Aggregate Type	Gradation/Mix	Fine Aggregate Contribution	Natural Sand Proportion	Air Voids Average	Air Voids Standard Deviaton	Dry Average (kPa)	Dry Maximum (kPa)	Dry Minimum (kPa)	Dry Range	Dry Standard Deviation	Wet Average (kPa)	Wet Maximum (kPa)	Wet Minimum (kPa)	Wet Range	Dry Average (kPa)	Wet Average (kPa)
				Air Voids		Tensile Strength										
Granite	Fine Mix 1	64.65%	21.81%	7.10%	0.41%	1362.8	1418.38	1322.32	96.06	49.77	782.8	794.13	771.46	22.67	1362.8	782.8
	Fine Mix 1 AS	64.65%	21.81%	6.74%	0.27%	1487.33	1659.11	1363.94	295.17	153.42	1325.42	1509.72	1228.85	280.87	1487.33	1325.42
	Coarse Mix 1	49%	15.28%	7.47%	0.23%	1887	1962.18	1751.28	210.9	117.75	1214.2	1283.62	1144.75	138.87	1887	1214.2
	Coarse Mix 1 AS	49%	15.28%	6.42%	0.20%	2097.07	2230.88	1963.26	267.62	189.23	1496.93	1631.07	1413	218.07	2097.07	1496.93
	Fine Mix 2	66.05%	28.46%	7.20%	0.08%	1780.4	1821.55	1739.19	82.36	58.24	1481.8	1574.38	1343.25	231.13	1780.4	1481.8
	Fine Mix 2 AS	66.05%	28.46%	6.46%	0.10%	1763.87	1823.57	1707.2	116.37	58.24	1497.35	1644.48	1350.22	294.26	1763.87	1497.35
Gravel	Fine	68.19%	40.10%	8.14%	0.00%	2206.2	2301.28	2111.22	190.06	134.4	1650.5	1817.32	1538.37	278.95	2206.2	1650.5
	Fine AS	68.19%	40.10%	7.47%	0.08%	2023.02	2135.65	1817.42	318.23	178.33	1642.78	1750.42	1535.14	215.28	2023.02	1642.78
	Coarse	43.16%	40.87%	6.34%	0.08%	2051.4	2095.6	2007.26	88.34	62.47	1789.3	1808.13	1770.56	37.57	2051.4	1789.3
	Coarse AS	43.16%	40.87%	6.34%	0.27%	2170.1	2298.58	2041.6	256.98	181.71	1855.54	1950.46	1738.18	212.28	2170.1	1855.54
Limestone NE	Fine	71.57%	36.59%	7.11%	0.10%	1933.7	2034.64	1859.33	175.31	90.62	826.8	890.09	727.13	162.96	1933.7	826.8
	Fine AS	71.57%	36.59%	6.95%	0.23%	2071.04	2132.88	2009.2	123.68	87.46	1211.64	1237.19	1181.46	55.73	2071.04	1211.64
	Fine Hydrated Lime	71.57%	36.59%	5.91%	0.17%	2293.38	2306.78	2279.98	26.8	18.95	2066.83	2279.73	1853.94	425.79	2293.38	2066.83
	Fine SBS	71.57%	36.59%	6.93%	0.14%	2122.97	2168.9	2081.24	87.66	43.98	898.03	978.76	825.25	153.51	2122.97	898.03
Limestone South	Fine	65.23%	42.95%	7.18%	0.28%	1841.3	1888.81	1793.85	94.96	67.15	1175.8	1229.13	1094.81	134.32	1841.3	1175.8
	Fine AS	65.23%	42.59%	6.79%	0.18%	2077.64	2228.34	1968.65	259.69	134.78	1919.45	2044.97	1775.46	269.51	2077.64	1919.45

Table 4.1C: Summary of Mechanical Test Ratios and Stripping Test Results

Mix	Fracture Energy Ratio	TSR	Stripping Result	Standard Deviation
Granite Coarse 1	0.80	0.64	19.45%	4.01%
Granite Coarse 1 AS	0.83	0.71	3.90%	0.61%
Gravel Coarse	1.20	0.87	2.66%	1.05%
Gravel Coarse AS	1.17	0.86	2.34%	0.62%
Granite Fine 1	0.73	0.57	4.80%	1.83%
Granite Fine 1 AS	1.02	0.89	1.55%	0.78%
Gravel Fine	0.97	0.75	3.43%	0.42%
Gravel Fine AS	0.94	0.81	2.44%	0.49%
Granite 2 Fine	0.77	0.83	3.13%	0.63%
Granite 2 Fine AS	1.03	0.85	2.24%	0.50%
Limestone NE Fine	0.34	0.43	9.23%	2.42%
Limestone NE Fine AS	0.39	0.59	5.36%	1.11%
Limestone NE plus Lime	0.59	0.90	4.28%	0.64%
Limestone NE plus SBS	0.35	0.42	6.14%	1.43%
Limestone South Fine	0.51	0.64	5.43%	0.95%
Limestone South Fine AS	0.76	0.92	3.79%	0.56%

Figure 4.1.3A: Granite Fine Mix 1 Stress Strain Relationship for Unconditioned and Conditioned Mixes



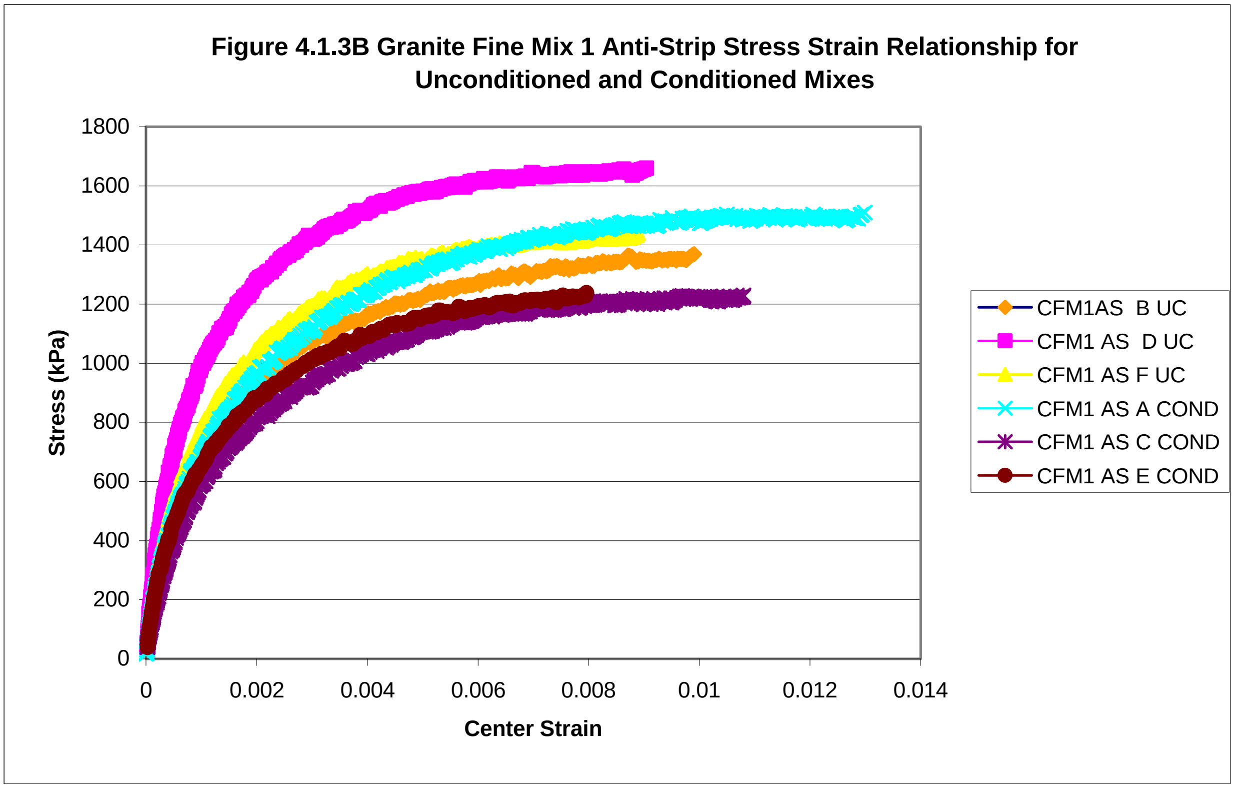


Figure 4.1.3B Granite Fine Mix 1 Anti-Strip Stress Strain Relationship for Unconditioned and Conditioned Mixes
Stress (kPa)
1800
1600
1400
1200
1000
800
600
400
200
0
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
Center Strain
CFM1AS B UC
CFM1 AS D UC
CFM1 AS F UC
CFM1 AS A COND
CFM1 AS C COND
CFM1 AS E COND

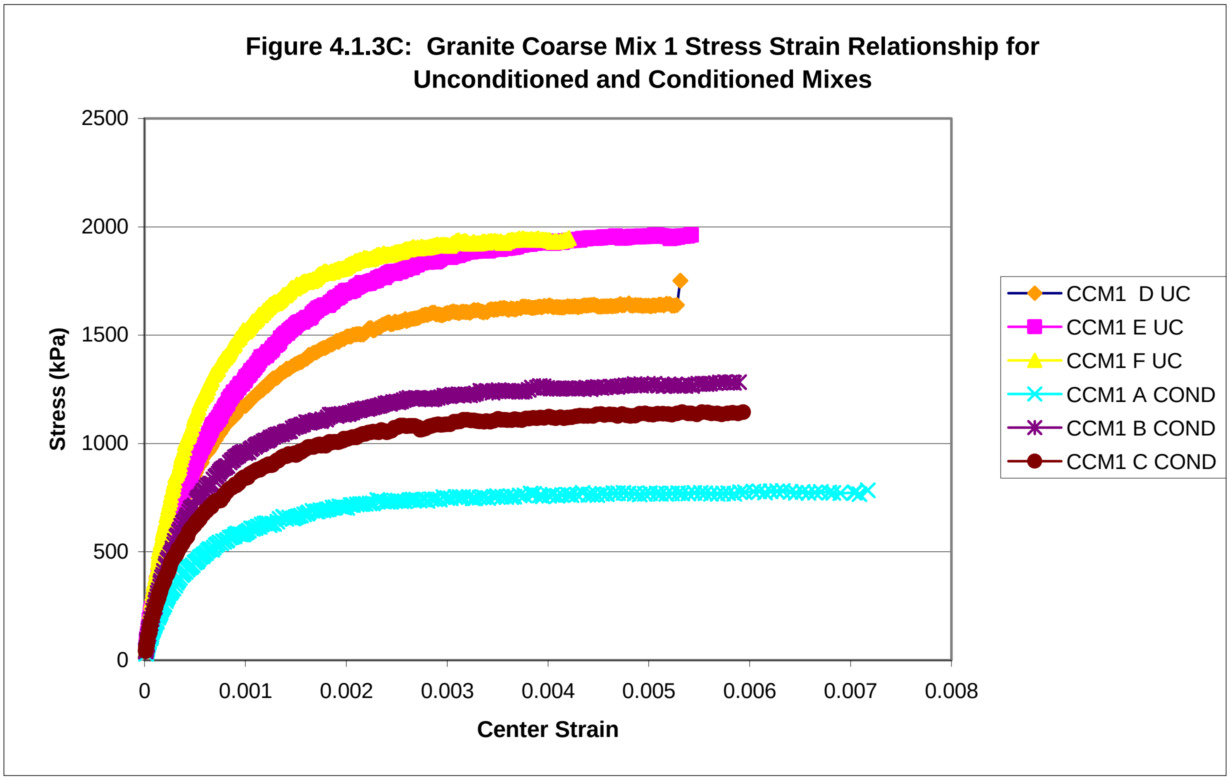


Figure 4.1.3C: Granite Coarse Mix 1 Stress Strain Relationship for Unconditioned and Conditioned Mixes
Stress (kPa)
2500
2000
1500
1000
500
0
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
Center Strain
CCM1 D UC
CCM1 E UC
CCM1 F UC
CCM1 A COND
CCM1 B COND
CCM1 C COND

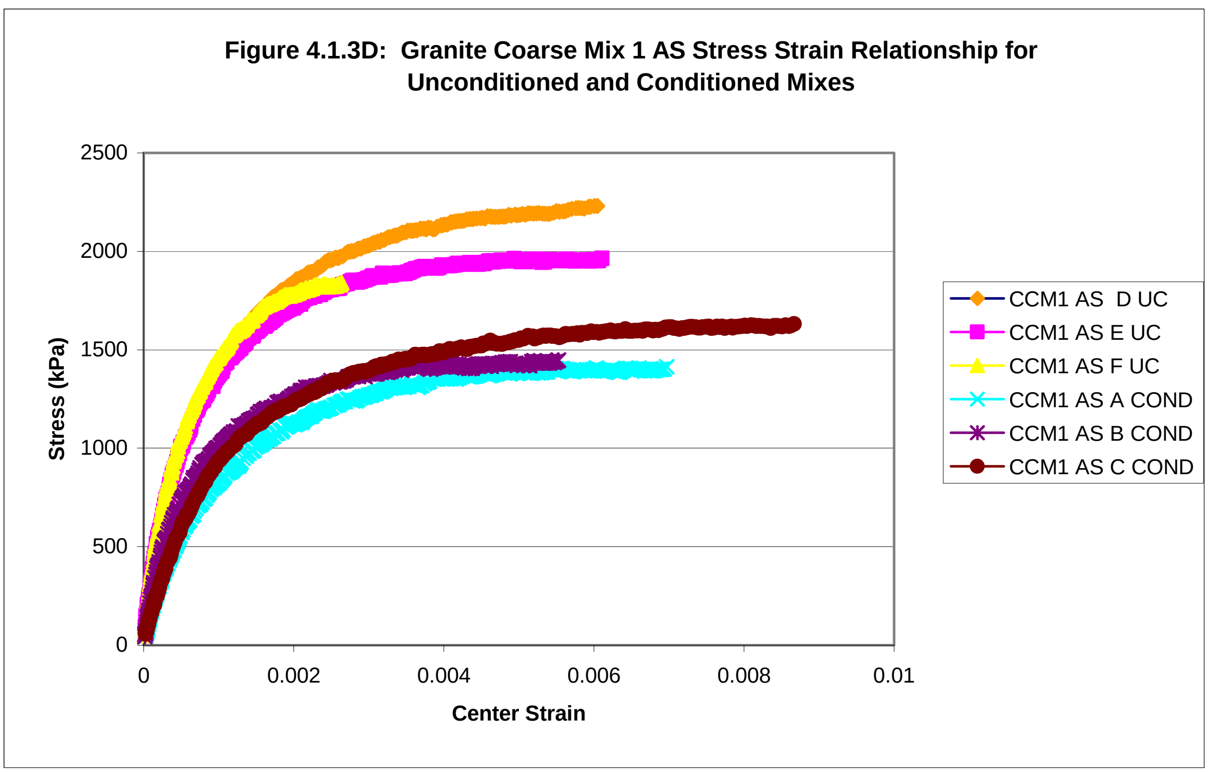


Figure 4.1.3D: Granite Coarse Mix 1 AS Stress Strain Relationship for Unconditioned and Conditioned Mixes
Stress (kPa)
2500
2000
1500
1000
500
0
0
0.002
0.004
0.006
0.008
0.01
Center Strain
CCM1 AS D UC
CCM1 AS E UC
CCM1 AS F UC
CCM1 AS A COND
CCM1 AS B COND
CCM1 AS C COND

Figure 4.1.3E: Granite Fine Mix 2 Stress Strain Relationship for Unconditioned and Conditioned Mixes

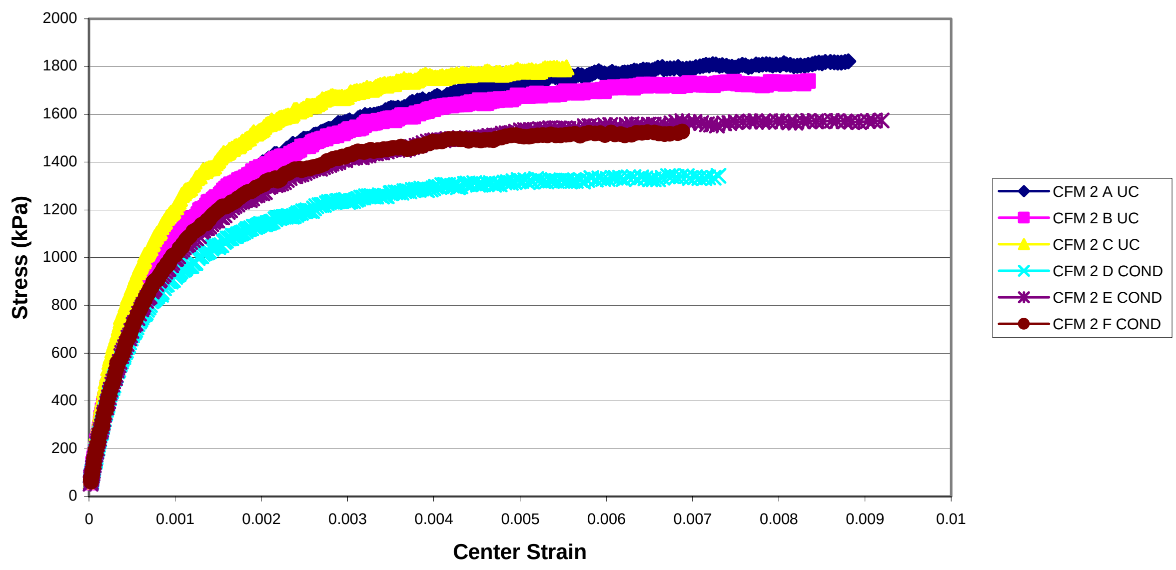


Figure 4.1.3F: Granite Fine Mix 2 Anti-Strip Stress Strain Relationship for Unconditioned and Conditioned Mixes

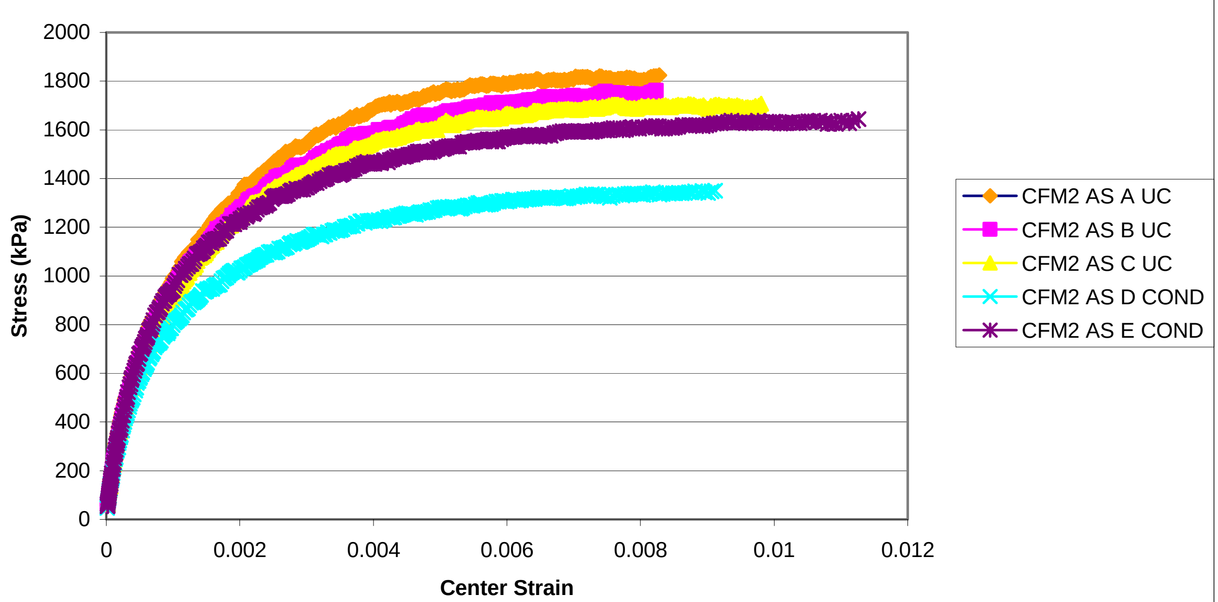


Figure 4.1.3G: Gravel Fine Mix Stress Strain Relationship for Unconditioned and Conditioned Mixes

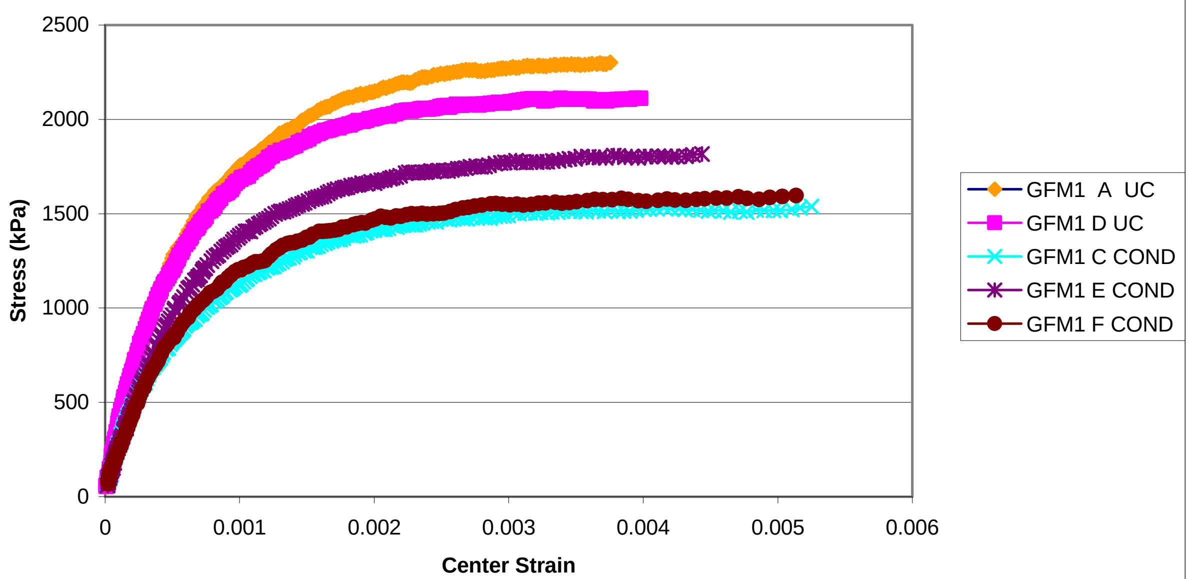


Figure 4.1.3H: Gravel Fine Mix Anti Strip Stress Strain Relationship for Unconditioned and Conditioned Mixes

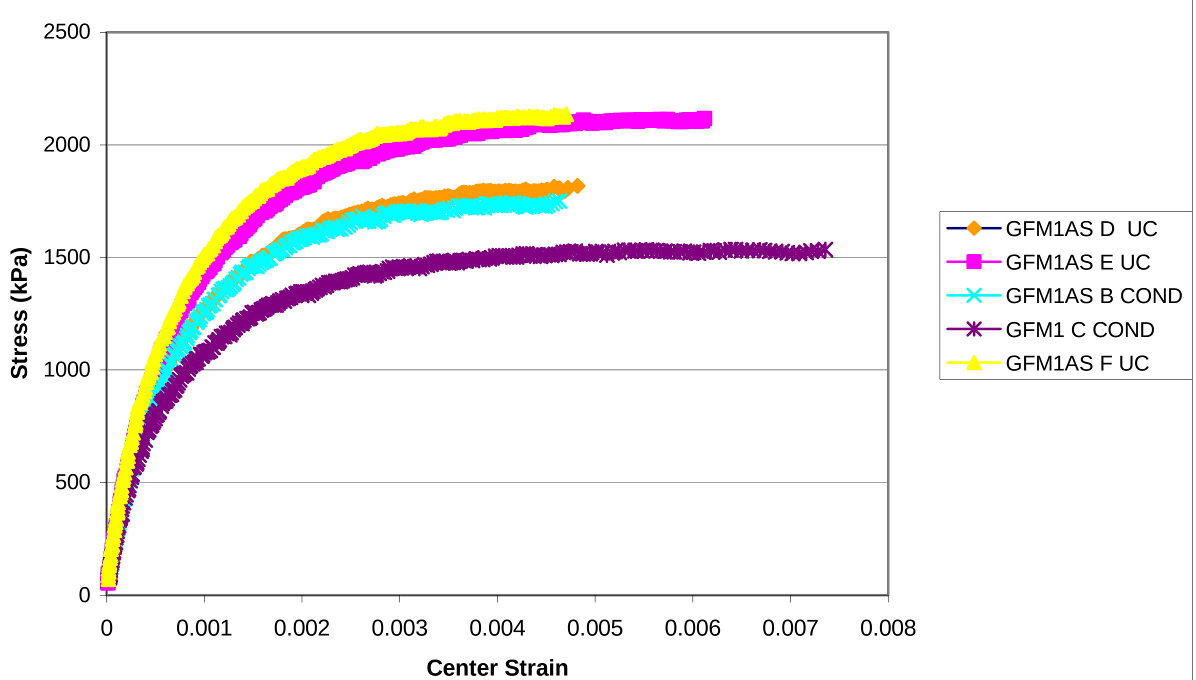


Figure 4.1.3I: Gravel Coarse Mix Anti-Strip Stress Strain Relationship for Unconditioned and Conditioned Mixes

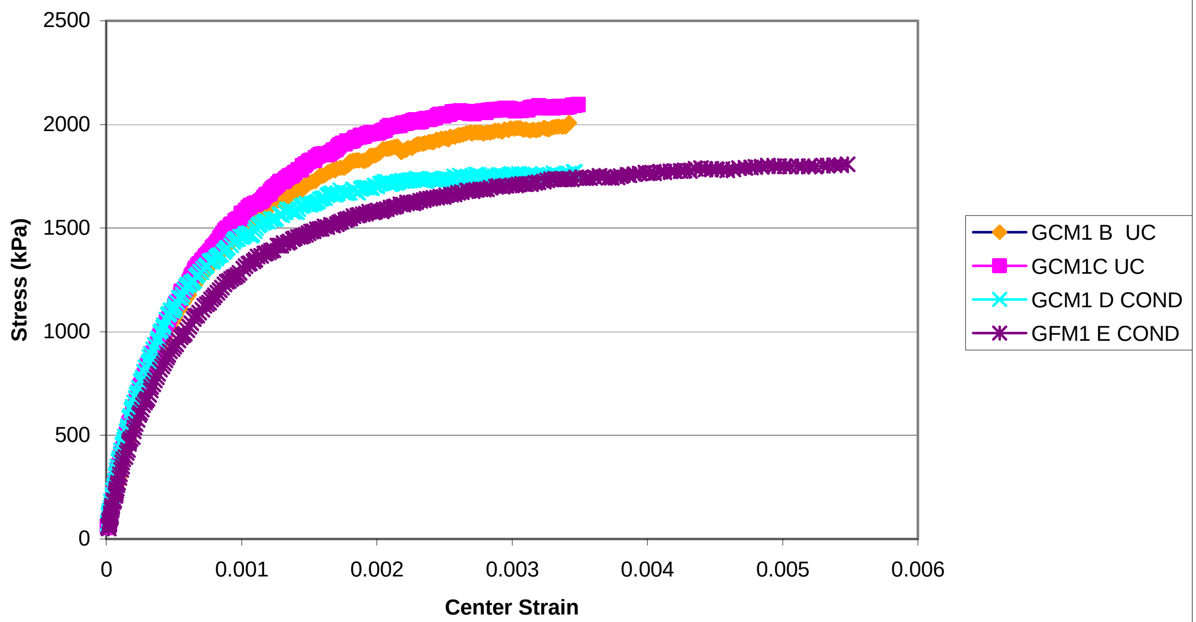
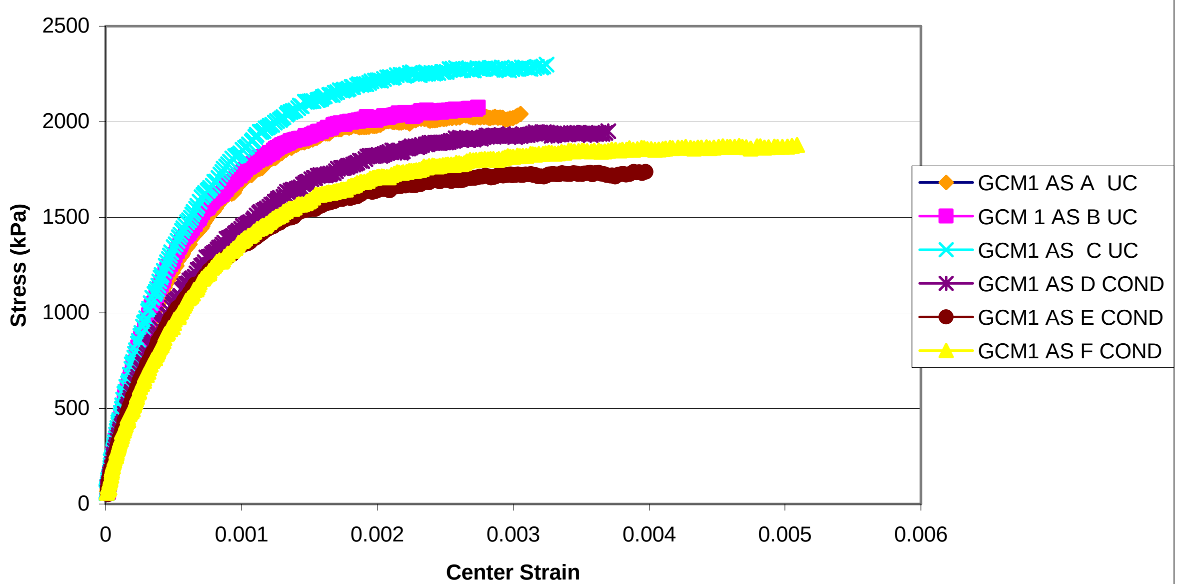


Figure 4.1.3J: Gravel Coarse Mix Anti-Strip Stress Strain Relatoinship for Unconditioned and Conditioned Mixes



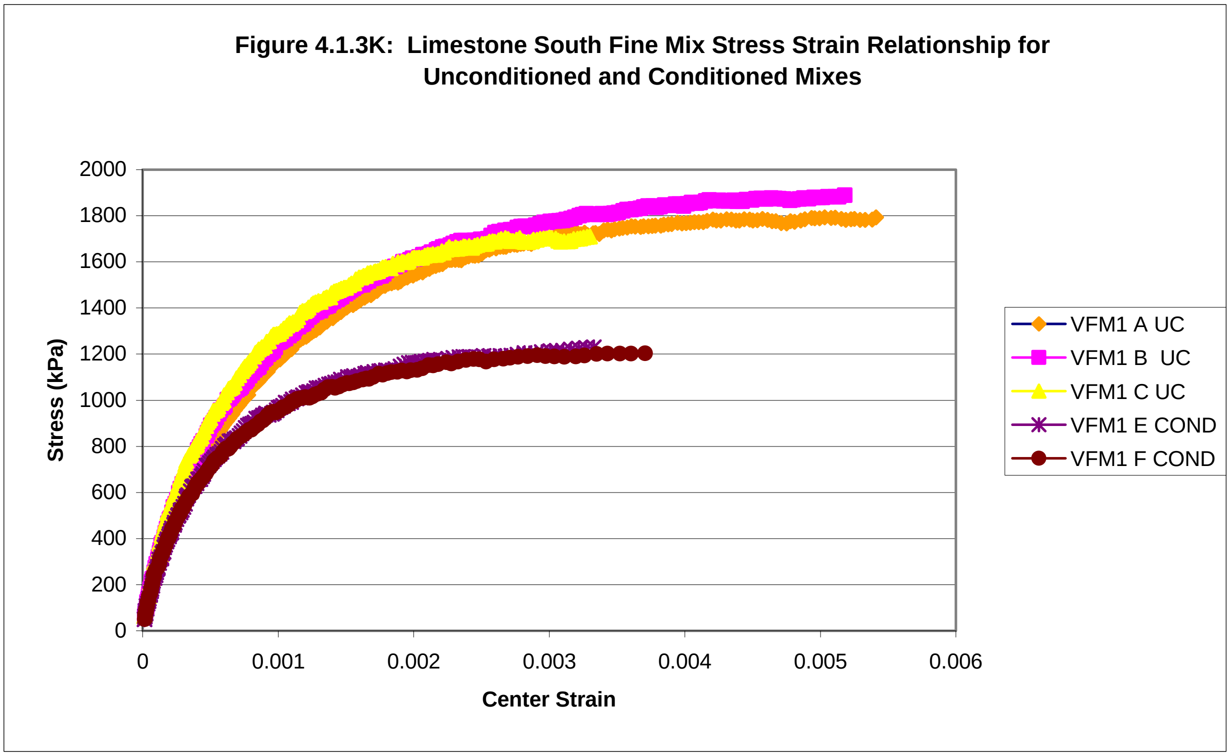


Figure 4.1.3K: Limestone South Fine Mix Stress Strain Relationship for
Unconditioned and Conditioned Mixes
Stress (kPa)
2000
1800
1600
1400
1200
1000
800
600
400
200
0
0
0.001
0.002
0.003
0.004
0.005
0.006
Center Strain
VFM1 A UC
VFM1 B UC
VFM1 C UC
VFM1 E COND
VFM1 F COND

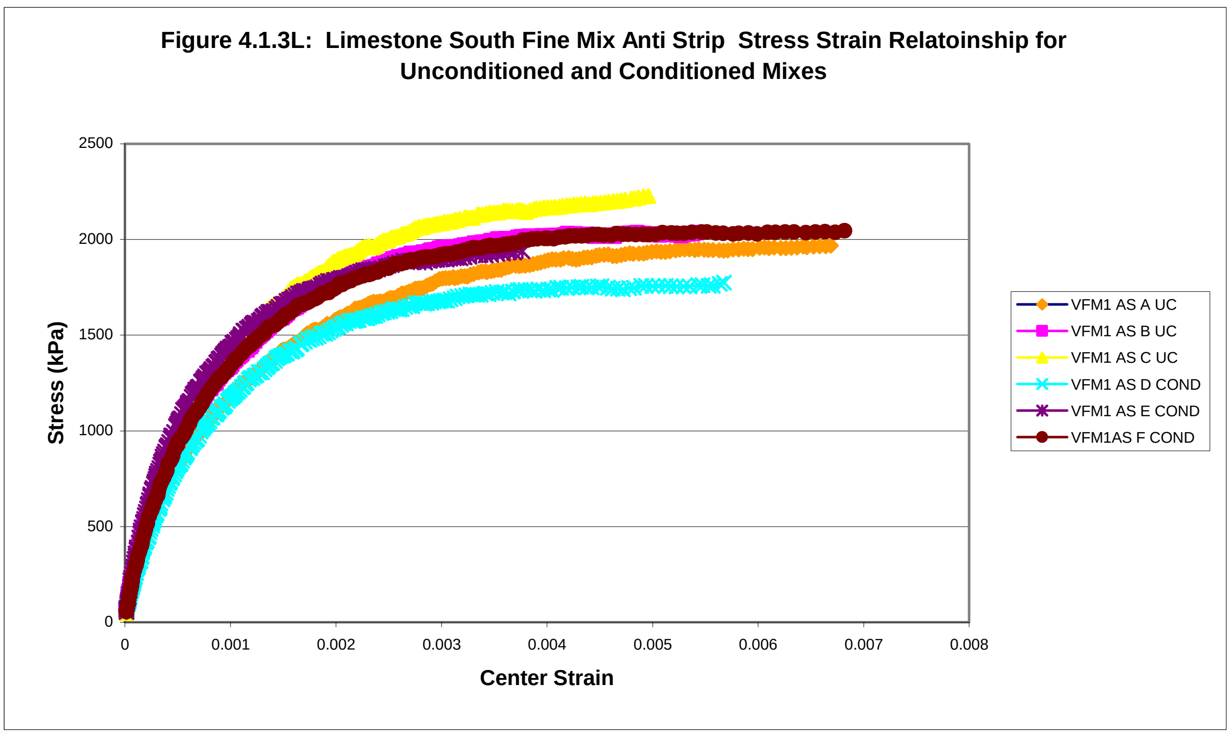


Figure 4.1.3L: Limestone South Fine Mix Anti Strip Stress Strain Relatoinship for
Unconditioned and Conditioned Mixes
Stress (kPa)
2500
2000
1500
1000
500
0
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
Center Strain
VFM1 AS A UC
VFM1 AS B UC
VFM1 AS C UC
VFM1 AS D COND
VFM1 AS E COND
VFM1AS F COND

Figure 4.1.3M: Limestone NE Fine Mix Stress Strain Relationship for Unconditioned and Conditioned Mixes

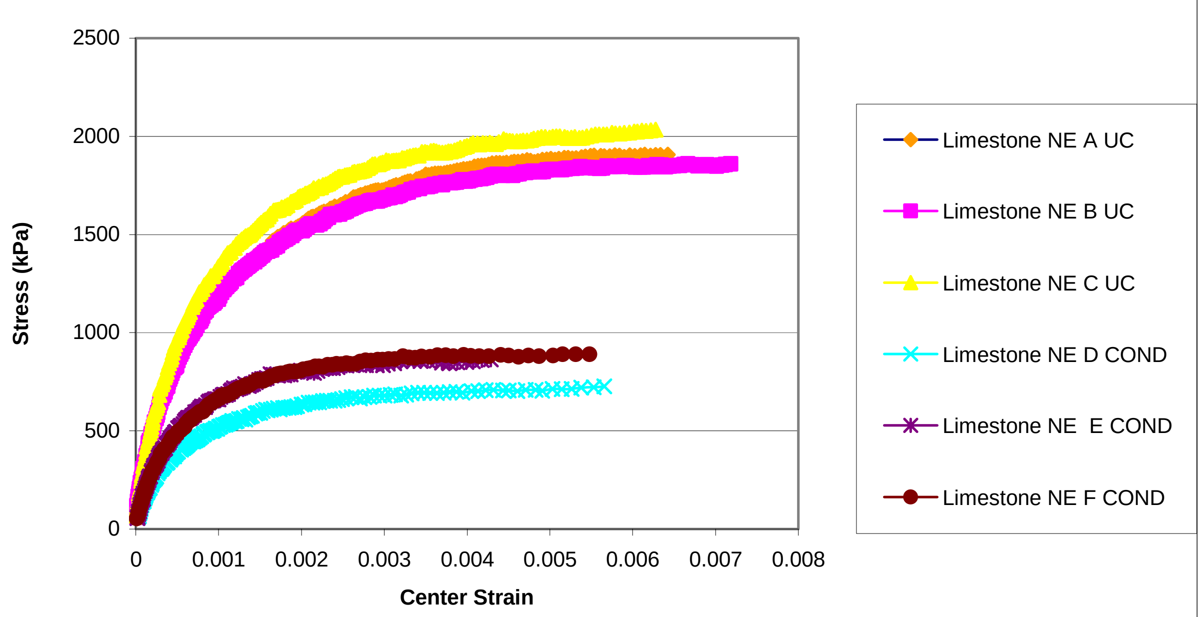


Figure 4.1.3N: Limestone NE Fine Mix Anti-Strip Stress Strain Relationship for Unconditioned and Conditioned Mixes

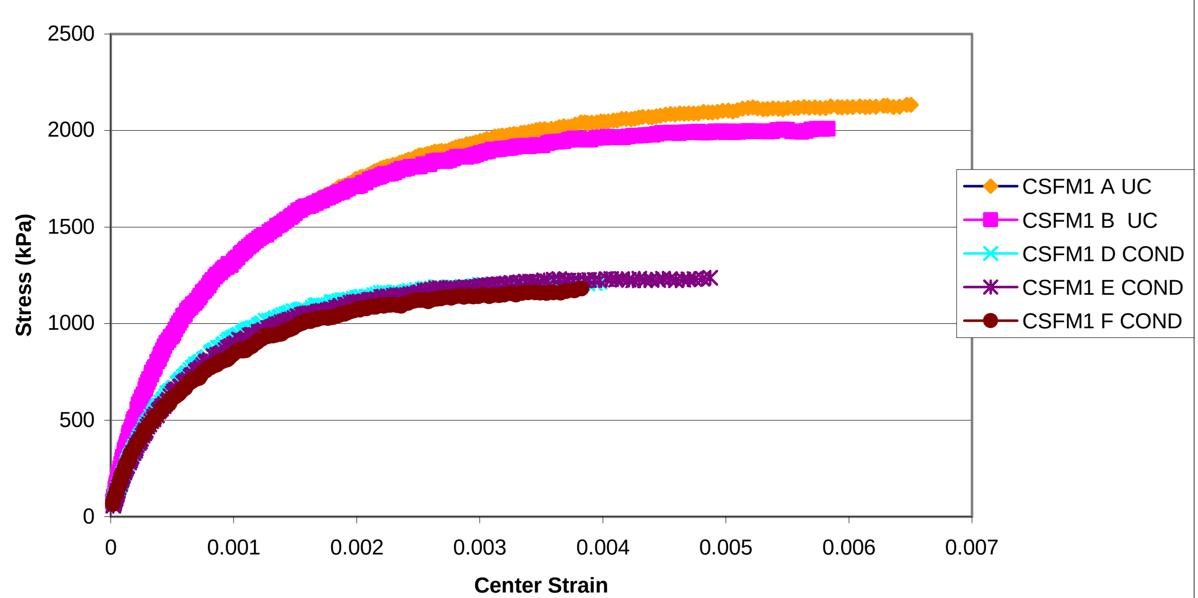


Figure 4.1.3O: Limestone NE Fine Mix Hydrated Lime Stress Strain Relationship for Unconditioned and Conditioned Mixes

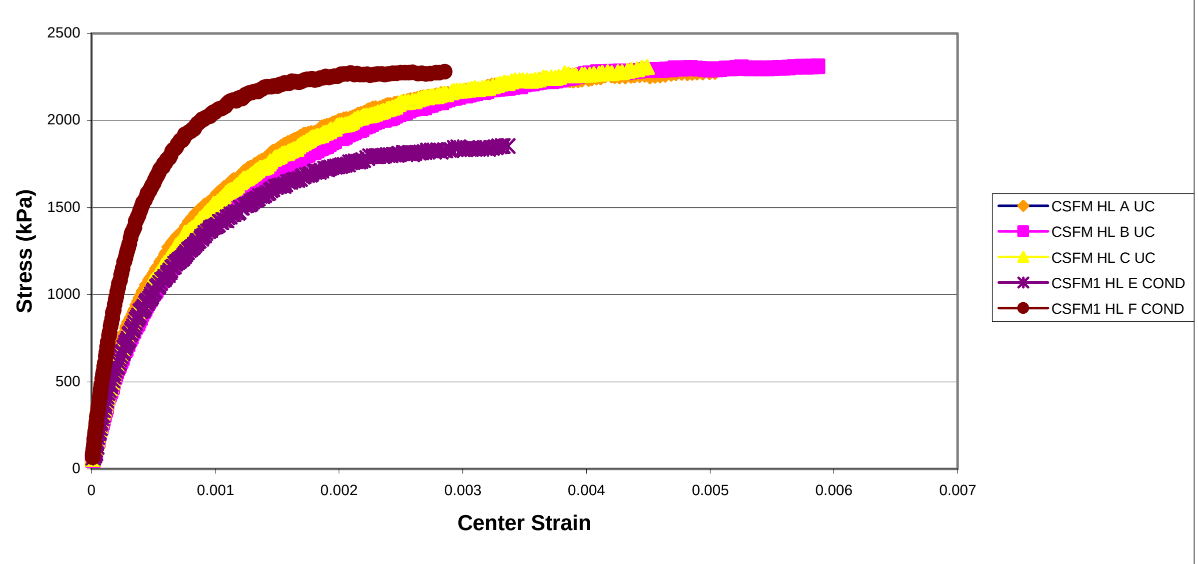
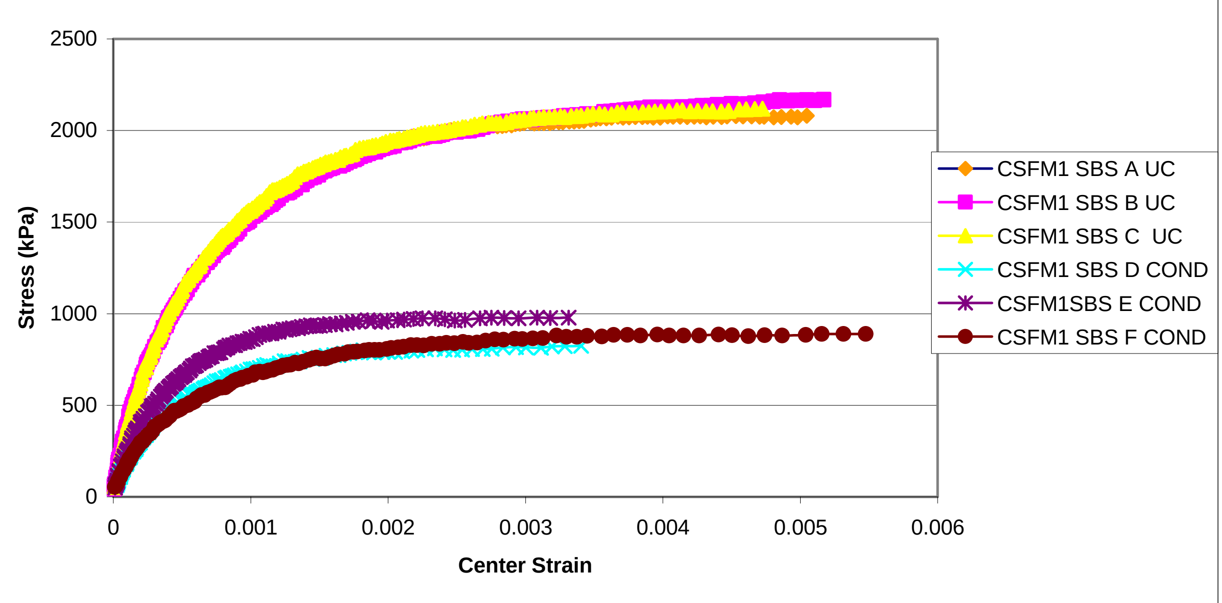


Figure 4.1.3P: Limestone NE Fine Mix SBS Stress Strain Relationship for Unconditioned and Conditioned Mixes

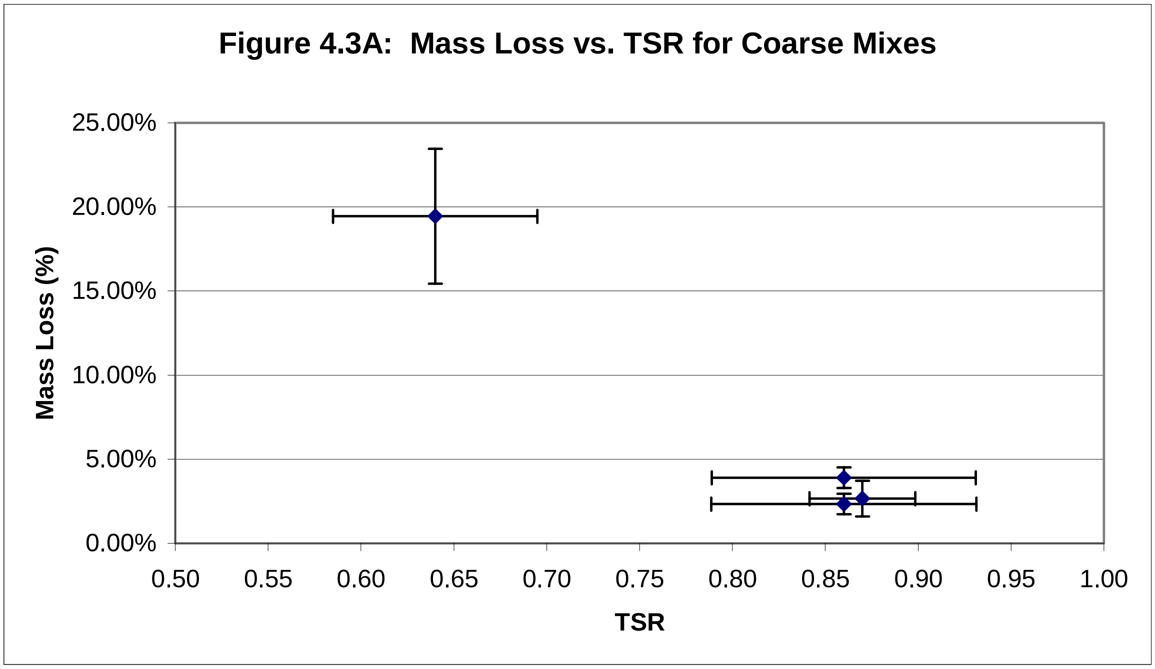


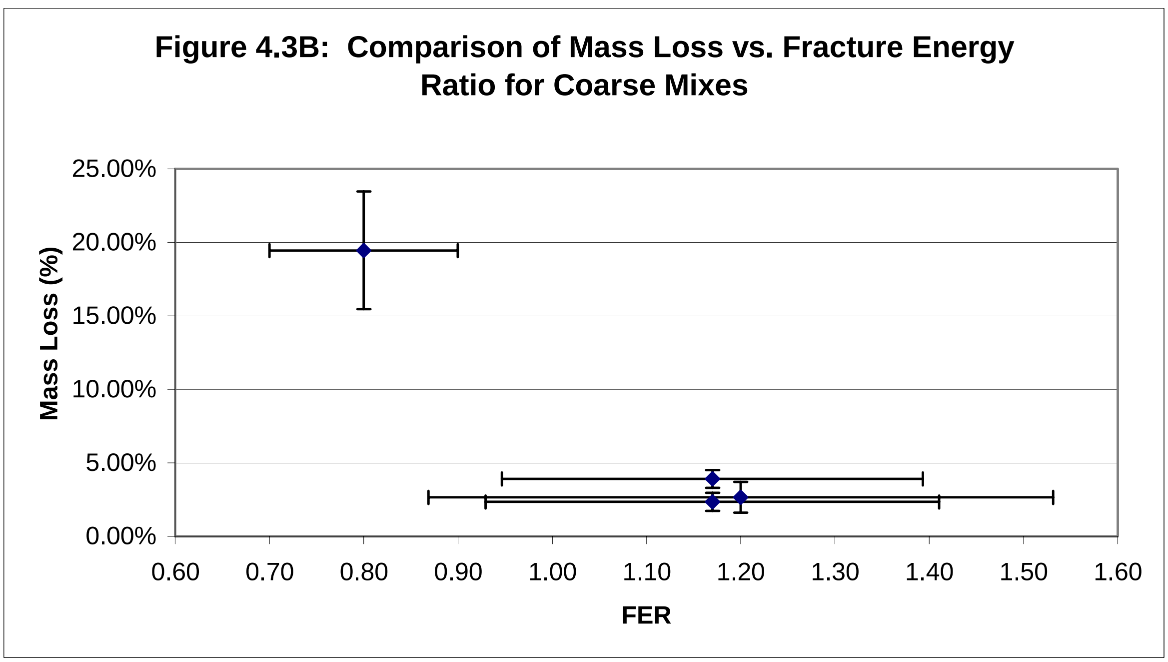
Section 4.2: Detailed Non-Mechanical Testing Results

Table 4.2: Summary of Stripping Test Results

Mix	Stripping Result	Standard Deviation	Coefficient of Variation	Trials
Granite Coarse 1	19.45%	4.01%	20.60%	4
Granite Coarse 1 AS	3.90%	0.61%	15.60%	5
Gravel Coarse	2.66%	1.05%	39.65%	6
Gravel Coarse AS	2.34%	0.62%	26.33%	6
Granite Fine 1	4.80%	1.83%	38.07%	5
Granite Fine 1 AS	1.55%	0.78%	50.38%	3
Gravel Fine	3.43%	0.42%	12.19%	3
Gravel Fine AS	2.44%	0.49%	20.19%	6
Granite 2 Fine	3.13%	0.63%	20.11%	3
Granite 2 Fine AS	2.24%	0.50%	22.37%	3
Limestone NE Fine	9.23%	2.42%	26.23%	3
Limestone NE Fine AS	5.36%	1.11%	20.73%	6
Limestone NE plus Lime	4.28%	0.64%	14.96%	3
Limestone NE plus SBS	6.14%	1.43%	23.31%	4
Limestone South Fine	5.43%	0.95%	17.49%	3
Limestone South Fine AS	3.79%	0.56%	14.81%	3

4.3: Relationship Between Mechanical and Non-Mechanical Test Results





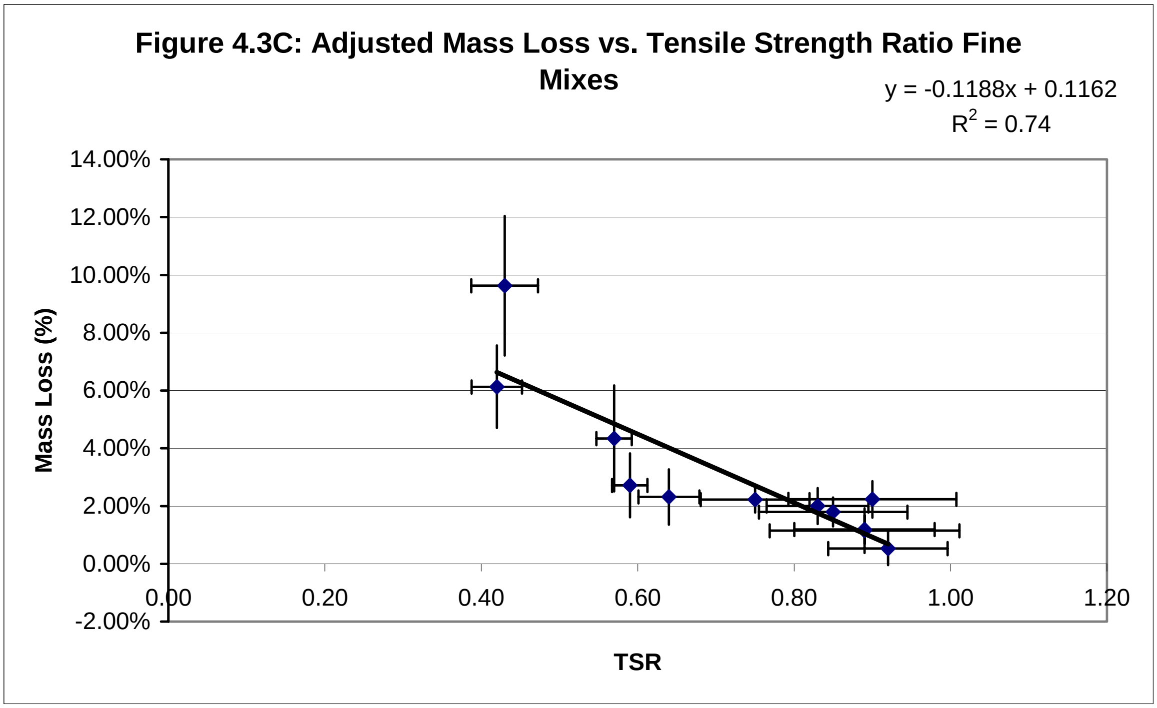


Figure 4.3C: Adjusted Mass Loss vs. Tensile Strength Ratio Fine Mixes
y = -0.1188x + 0.1162
R2 = 0.74
Mass Loss (%)
14.00%
12.00%
10.00%
8.00%
6.00%
4.00%
2.00%
0.00%
-2.00%
0.00
0.20
0.40
0.60
0.80
1.00
1.20
TSR

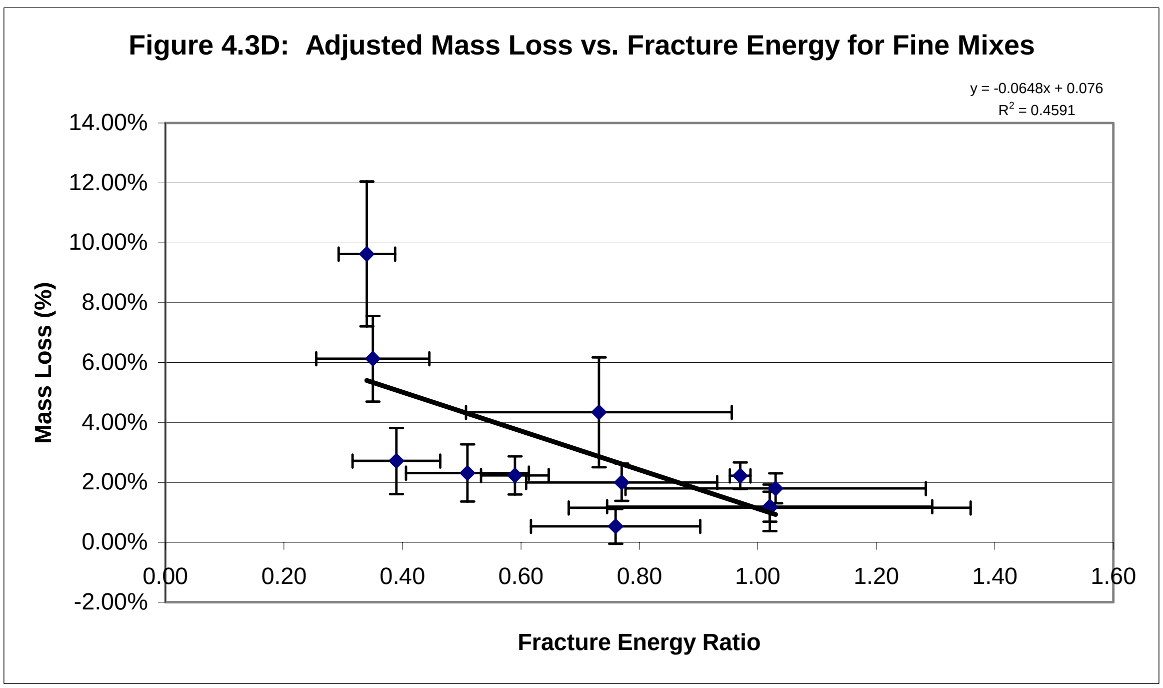


Figure 4.3D: Adjusted Mass Loss vs. Fracture Energy for Fine Mixes
y = -0.0648x + 0.076
R2 = 0.4591
Mass Loss (%)
14.00%
12.00%
10.00%
8.00%
6.00%
4.00%
2.00%
0.00%
-2.00%
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
Fracture Energy Ratio

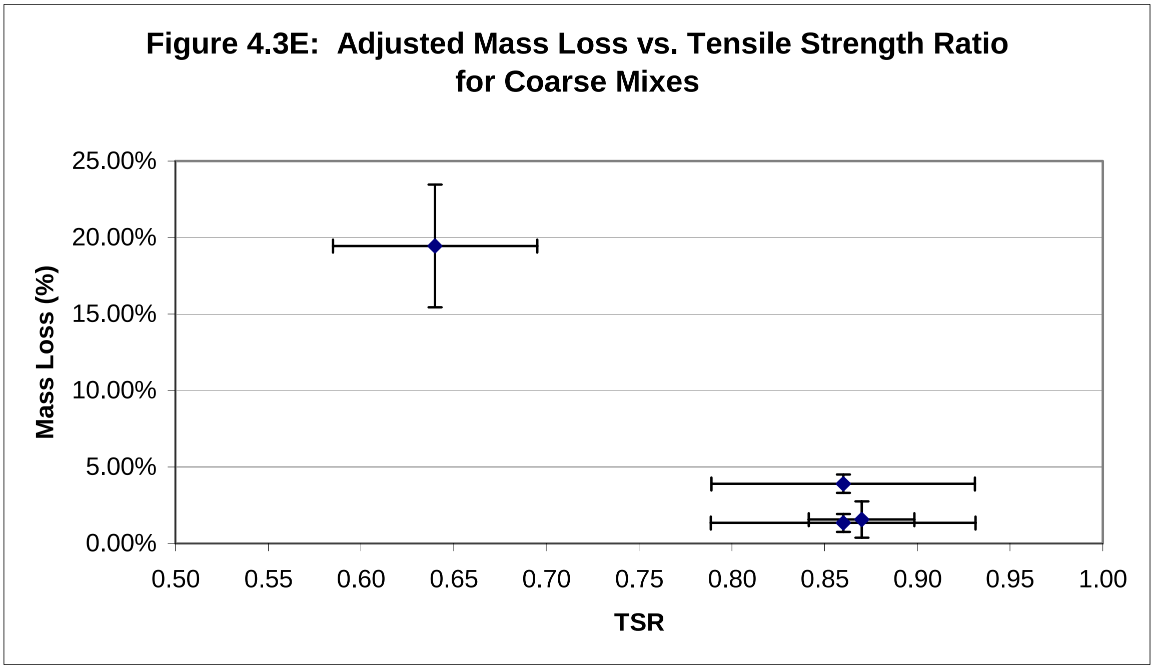


Figure 4.3E: Adjusted Mass Loss vs. Tensile Strength Ratio for Coarse Mixes
Mass Loss (%)
25.00%
20.00%
15.00%
10.00%
5.00%
0.00%
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
1.00
TSR

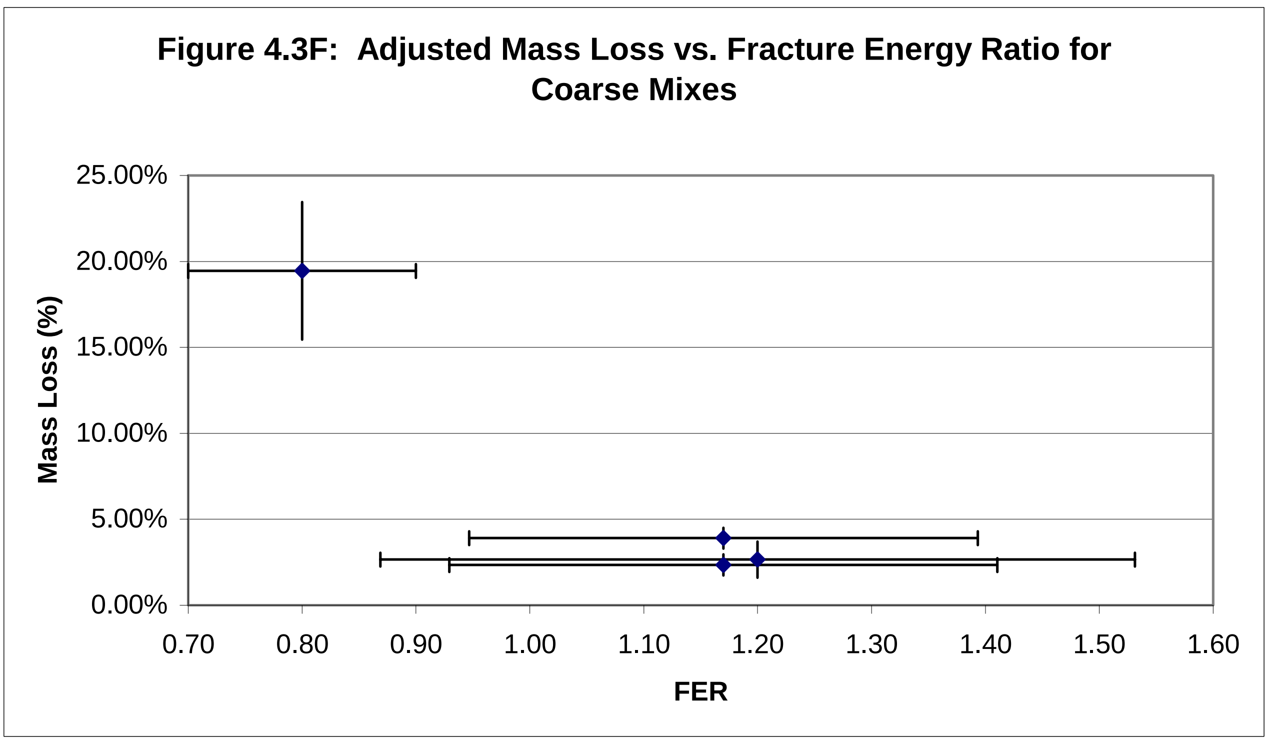


Figure 4.3F: Adjusted Mass Loss vs. Fracture Energy Ratio for Coarse Mixes
Mass Loss (%)
25.00%
20.00%
15.00%
10.00%
5.00%
0.00%
0.70
0.80
0.90
1.00
1.10
1.20
1.30
1.40
1.50
1.60
FER

4.5: Comparison of New to Current TSR Testing Method

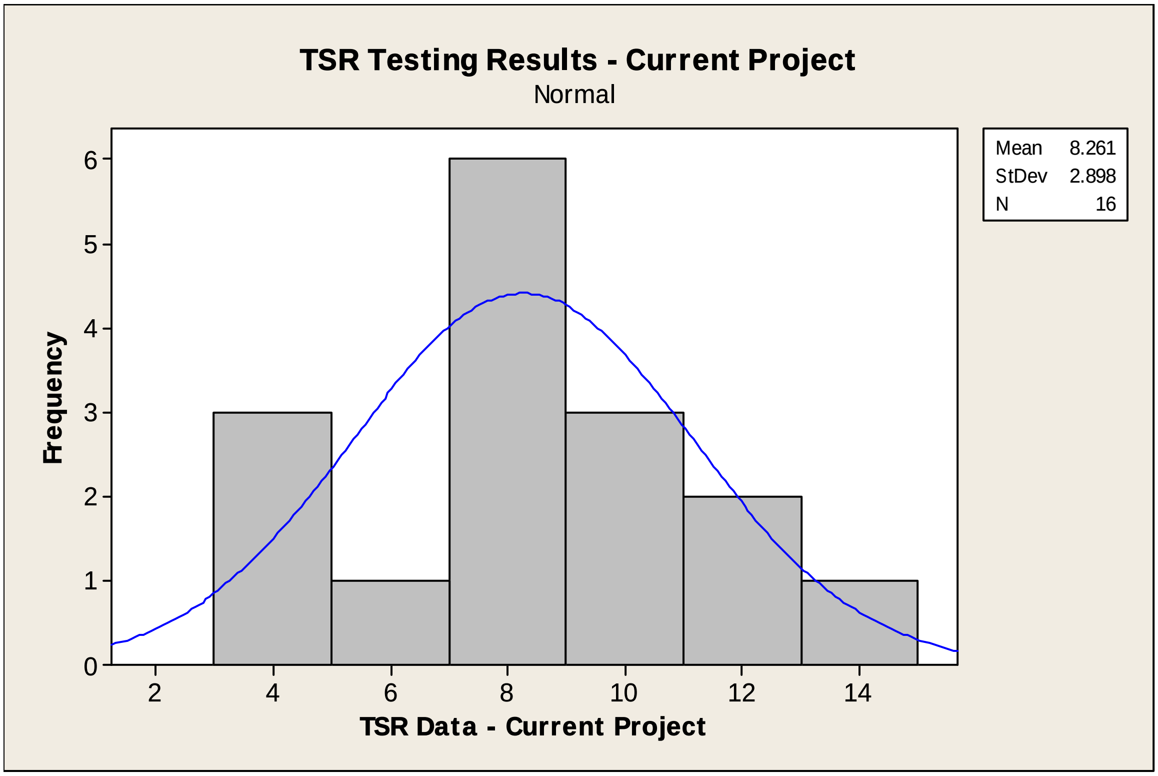
Table 4.5A: Summary of Mix Data for Histogram

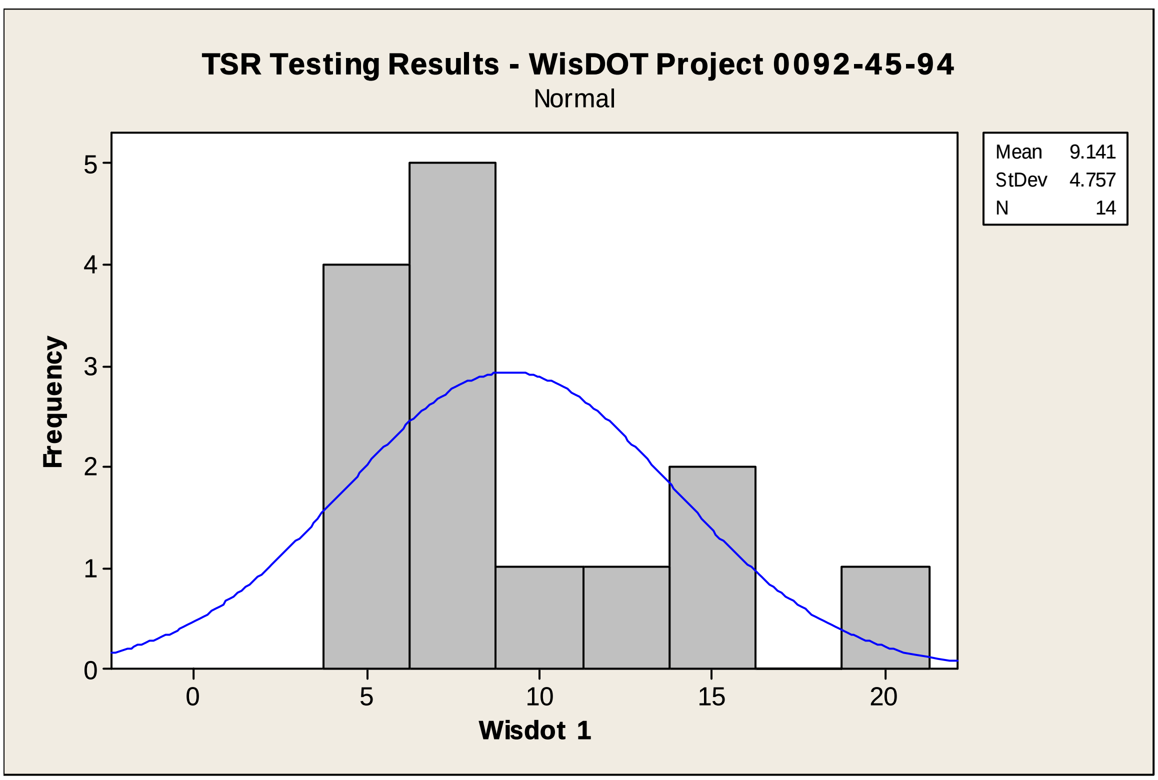
Mix	TSR	Standard Deviation	Coefficient of Variation
Granite Coarse 1	0.64	0.06	8.60%
Granite Coarse 1 AS	0.86	0.07	8.26%
Gravel Coarse	0.87	0.03	3.27%
Gravel Coarse AS	0.86	0.07	8.30%
Granite Fine 1	0.57	0.02	3.97%
Granite Fine 1 AS	0.89	0.12	13.64%
Gravel Fine	0.75	0.07	9.27%
Gravel Fine AS	0.89	0.09	10.08%
Granite 2 Fine	0.83	0.06	7.83%
Granite 2 Fine AS	0.85	0.09	11.16%
Limestone CS Fine	0.43	0.04	9.94%
Limestone CS Fine AS	0.59	0.02	3.85%
Limestone CS HL	0.9	0.11	11.93%
Limestone Poly	0.42	0.03	7.70%
Limestone Vienna Fine	0.64	0.04	6.10%
Limestone Vienna Fine AS	0.92	0.08	8.28%

Table 4.5B: Summary of WisDOT 1 Data for Histogram

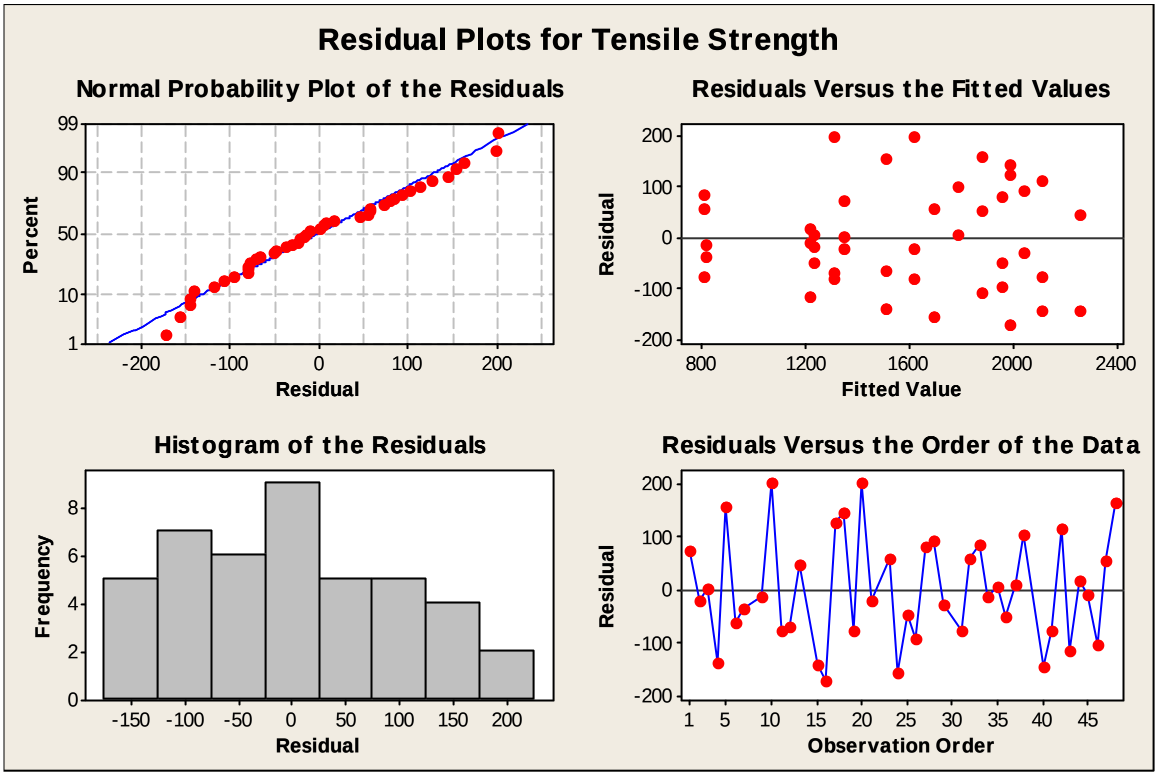
HWY Sections	TSR WisDOT 1	Standard Deviation WisDOT 1	Coefficient of Variation WisDOT 1
78	0.74	0.06	7.41%
64	0.99	0.07	6.55%
14	0.63	0.04	6.35%
35	0.61	0.03	4.92%
10-Mondovi	0.65	0.13	20.00%
51-Mathy	0.73	0.12	15.69%
51-P&D	0.94	0.12	12.28%
100	0.85	0.05	5.88%
116	0.72	0.05	6.94%
10-Clark	0.95	0.08	7.87%
12-Harding	0.70	0.08	10.67%
62	0.91	0.05	4.97%
29	0.85	0.04	4.15%
30	0.63	0.09	14.29%

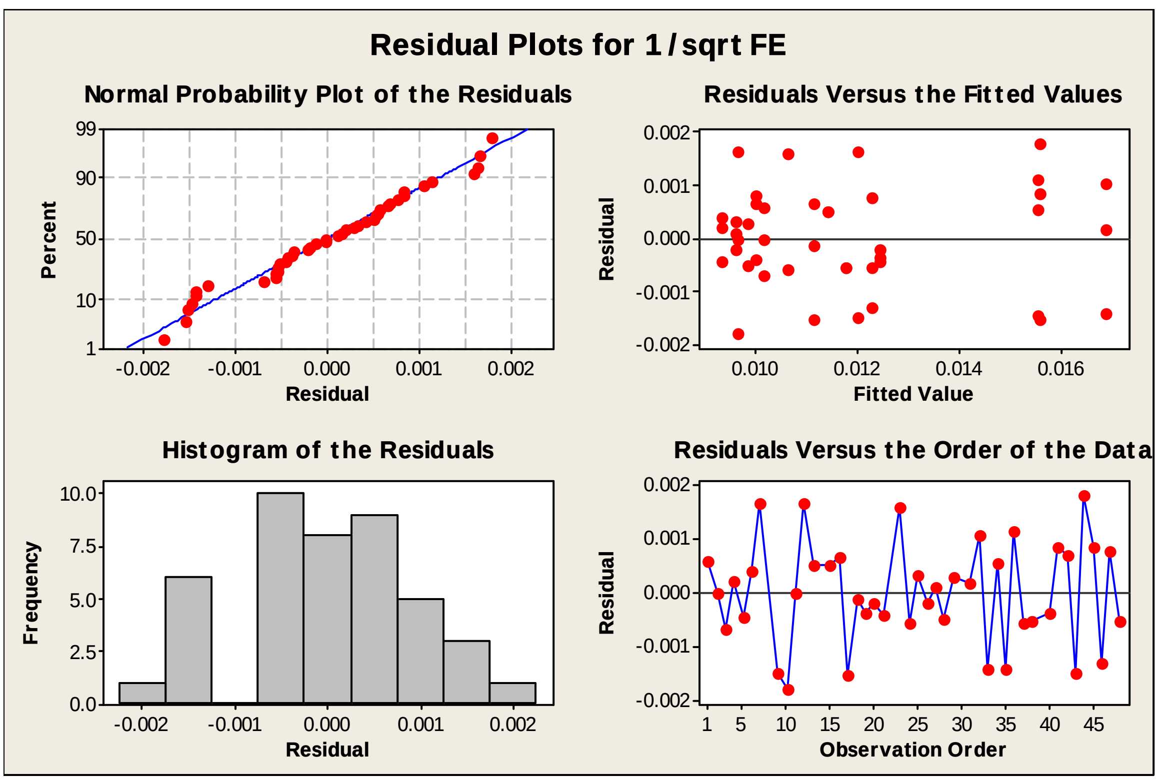
Figures 4.5A and 4.5B: Individual Histograms for TSR Testing Results



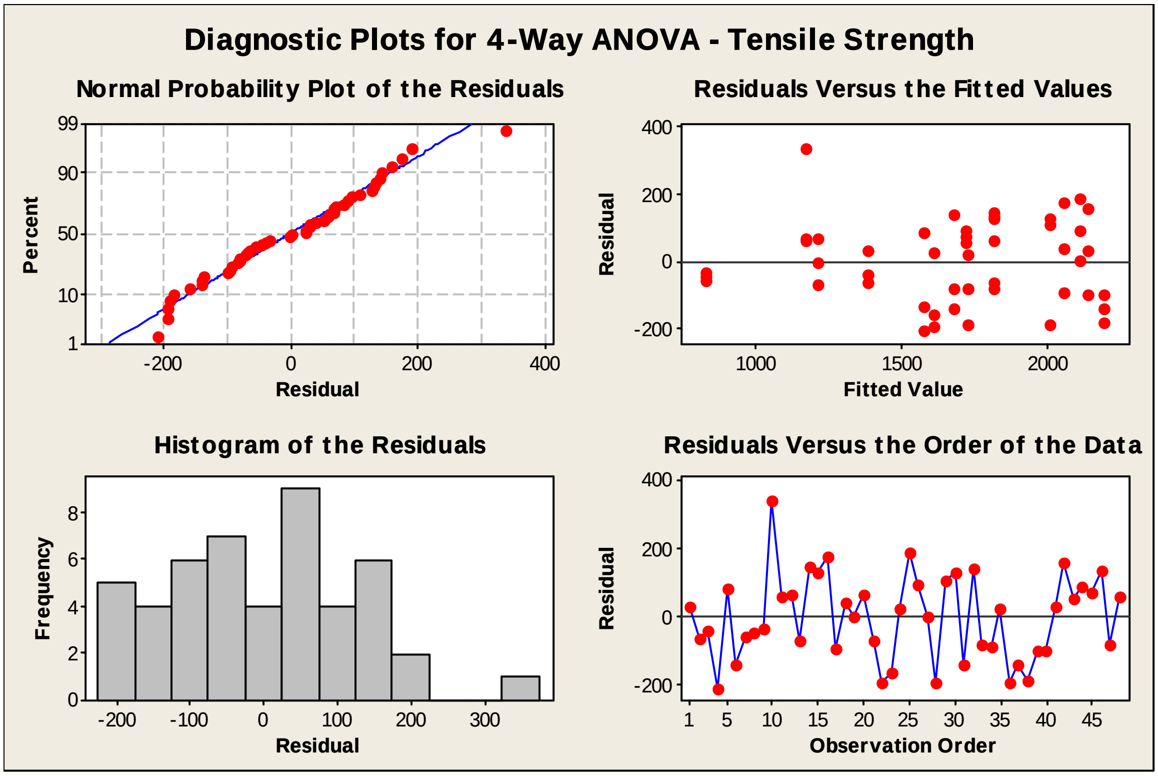


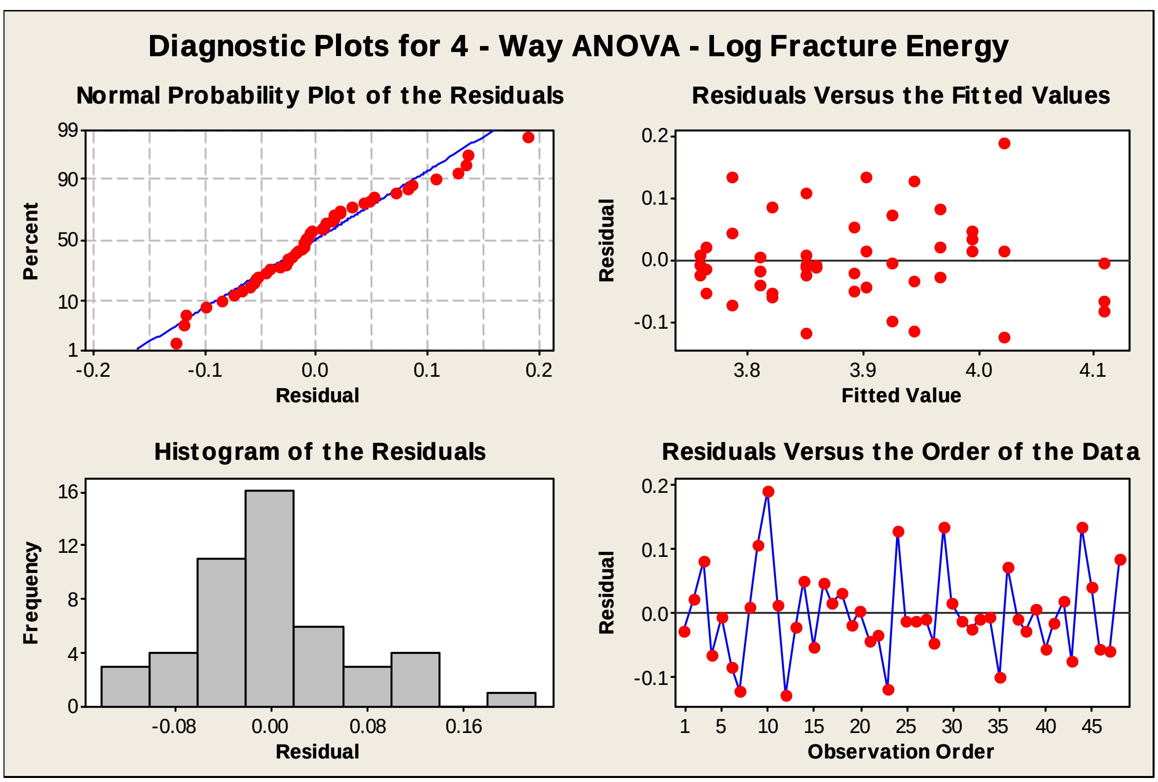
Figures 4.6A-4.6B: Diagnostic Plots for 3 Way ANOVA





Figures 4.6C-4.6D: Diagnostic Plots for 4 Way ANOVA





4.7: Binder Testing Results

Table 4.6.1: Summary of Binder Testing Results

Source	Grade	Mod	Temp	G*	d	G* sind	G*/sind	Tack
			(in C)	(in MPa)	(in deg)			(N s)
CRM	58-28		10	5.86	58.80	5.01	6.85	4333
CRM	58-28	5%	10	5.80	57.80	4.91	6.85	3360
Koch	58-28		10	5.51	57.90	4.67	6.50	1940
Koch	58-28	5%	10	5.73	57.20	4.82	6.82	4787
MIF	58-28		10	4.77	58.70	4.08	5.58	2376
MIF	58-28	5%	10	5.38	56.80	4.50	6.43	2920
Koch	64-28	SBS	10	6.70	50.80	5.19	8.65	6074

4.8.1: Investigation of the Relationship between Cohesion and Mixture Testing Results

Figure 4.8.1A: Dry Fracture Energy vs. Tack Factor

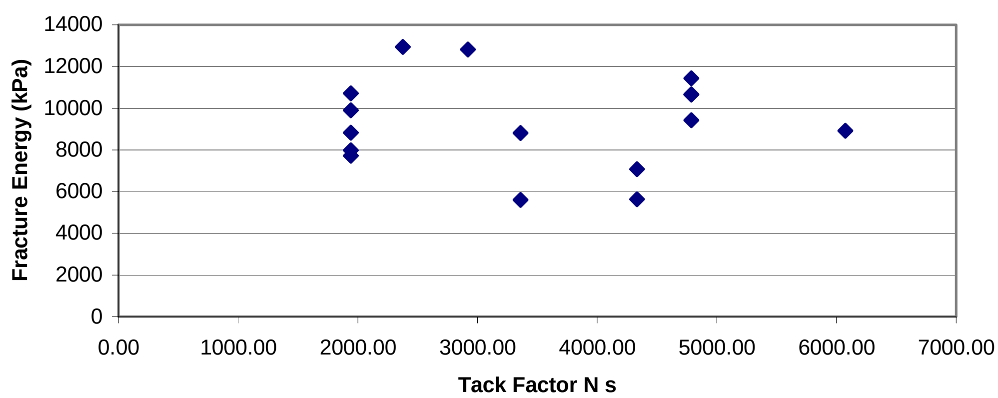
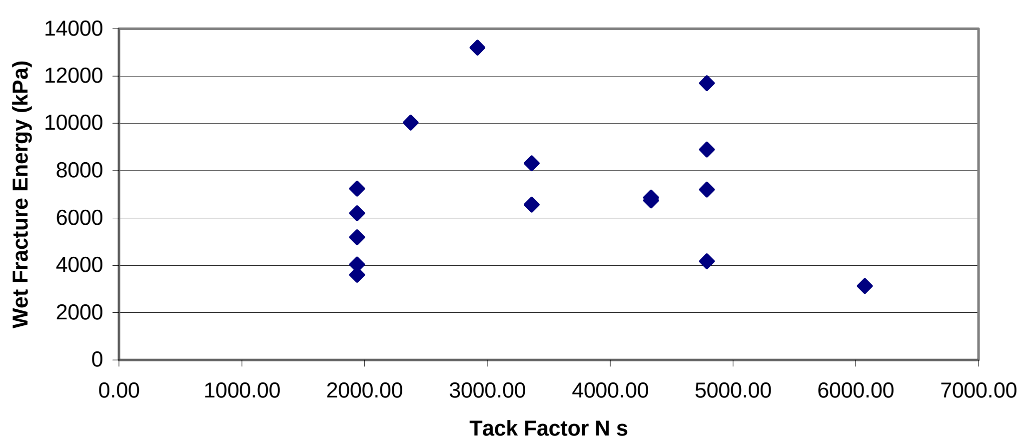


Figure 4.8.1B Wet Fracture Energy vs. Tack Factor



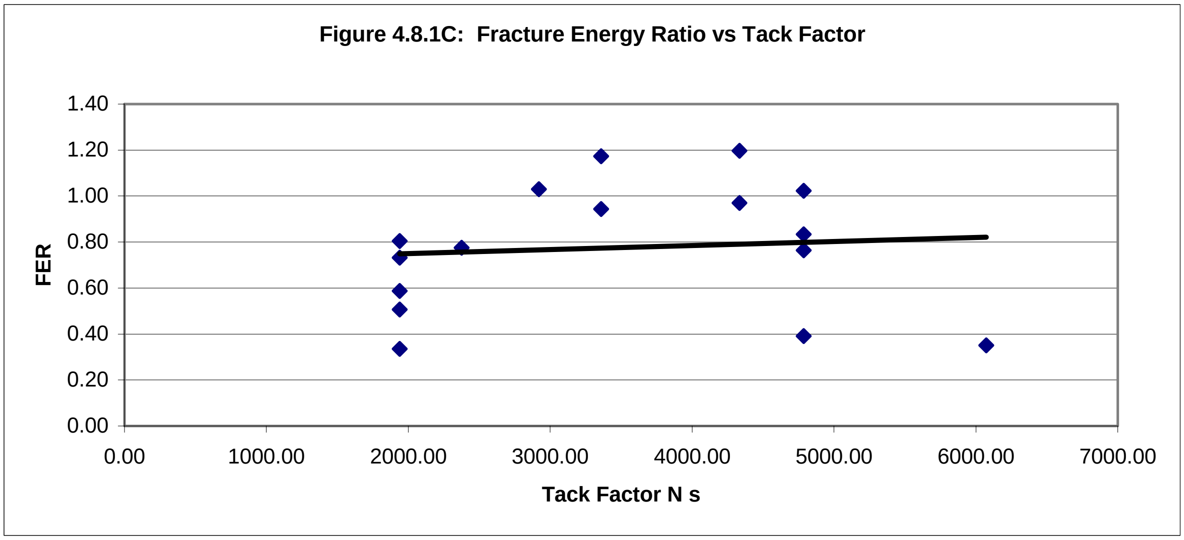


Figure 4.8.1C: Fracture Energy Ratio vs Tack Factor
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
FER
0.00
1000.00
2000.00
3000.00
4000.00
5000.00
6000.00
7000.00
Tack Factor N s

4.8.2: Investigation of the Effects of Measured Binder Properties on Mixture Performance

Figure 4.8.2A: Regression Analysis: Effect of Binder Properties on Dry Tensile Strength

Regression Statistics	
Multiple R	0.4336889
R Square	0.1880861
Adjusted R Square	-0.014892
Standard Error	255.9704
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	182140.8783	60713.62609	0.926631	0.4576228
Residual	12	786250.1585	65520.84655		
Total	15	968391.0368			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-88.51885	1554.87911	-0.05692973	0.955538	-3476.3094	3299.2717	-3476.3094	3299.271702
G*sind	504.99159	496.0909392	1.017941569	0.328791	-575.897713	1585.8809	-575.897713	1585.880891
G*/sind	-62.03017	220.4990126	-0.281317242	0.783259	-542.456251	418.3959	-542.456251	418.395903
Tack	0.0141768	0.068801748	0.206053389	0.840204	-0.1357293	0.164083	-0.1357293	0.164082963

RESIDUAL OUTPUT

Observation	*Predicted Dry Avg*	*Residuals*	*Standard Residuals*
1	2079.1852	-27.78523537	-0.121361053
2	2079.1852	127.0147646	0.554778299
3	2012.4049	157.6950753	0.688784535
4	2012.4049	10.59507528	0.046277438
5	1892.6364	-5.636439269	-0.024618982
6	1892.6364	-529.8364393	-2.314232988
7	1892.6364	41.06356073	0.179358458
8	1892.6364	400.7635607	1.750465208
9	1892.6364	-51.33643927	-0.224228597
10	1988.7615	108.3384838	0.473203567
11	1988.7615	-501.4615162	-2.190296282
12	1988.7615	82.28032985	0.359386104
13	1988.7615	88.83848384	0.388030975
14	2083.273	39.69849253	0.173396079
15	1657.1119	123.2880749	0.538500769
16	1827.4198	-63.51983224	-0.277443528

Figure 4.8.2B: Regression Analysis: Effect of Binder Properties on Wet Tensile Strength

Regression Statistics	
Multiple R	0.527974
R Square	0.278757
Adjusted R Square	0.098446
Standard Error	371.5021
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	640100.1	213366.7	1.545981	0.253492223
Residual	12	1656166	138013.8		
Total	15	2296266			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	779.6125	2256.67	0.34547	0.735721	-4137.250078	5696.475	-4137.25	5696.475
G*sind	999.3776	720.0005	1.388023	0.190361	-569.3687394	2568.124	-569.3687	2568.124
G*/sind	-671.5373	320.0208	-2.098418	0.057707	-1368.802626	25.7281	-1368.803	25.7281
Tack	0.119138	0.099855	1.193107	0.255884	-0.0984279	0.336704	-0.098428	0.336704

RESIDUAL OUTPUT

Observation	*Predicted Wet Avg*	*Residuals*	*Standard Residuals*
1	1704.53	84.77009	0.255115
2	1704.53	-54.02991	-0.162603
3	1481.913	373.5865	1.124307
4	1481.913	160.8865	0.484187
5	1307.546	-93.34597	-0.280924
6	1307.546	-524.746	-1.579221
7	1307.546	-480.746	-1.446803
8	1307.546	759.254	2.284972
9	1307.546	-131.746	-0.396489
10	1585.62	-88.71983	-0.267002
11	1585.62	-260.2198	-0.78313
12	1585.62	-373.9769	-1.125482
13	1585.62	333.8802	1.004811
14	886.19	11.84381	0.035644
15	1387.073	94.72727	0.285081
16	1308.818	188.582	0.567537

Figure 4.8.2C: Regression Analysis: Effect of Binder Properties on TSR

Regression Statistics	
Multiple R	0.6163497
R Square	0.379887
Adjusted R Square	0.2248587
Standard Error	0.1451022
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	0.154779	0.051593	2.450437	0.113782426
Residual	12	0.252656	0.021055		
Total	15	0.407435			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1.310319	0.881416	1.486607	0.162914	-0.61012059	3.230759	-0.61012059	3.230759
G*sind	0.2642087	0.281219	0.939511	0.365996	-0.34851586	0.876933	-0.348515863	0.876933
G*/sind	-0.307034	0.124994	-2.456379	0.030238	-0.57937331	-0.034694	-0.579373308	-0.034694
Tack	6.663E-05	3.9E-05	1.708403	0.113275	-1.8347E-05	0.000152	-1.83468E-05	0.000152

RESIDUAL OUTPUT

Observation	*Predicted TSR*	*Residuals*	*Standard Residuals*
1	0.8199062	0.052327	0.403191
2	0.8199062	-0.071787	-0.553131
3	0.7264341	0.128596	0.990848
4	0.7264341	0.085627	0.65977
5	0.6757508	-0.032296	-0.248842
6	0.6757508	-0.101345	-0.78088
7	0.6757508	-0.248177	-1.912239
8	0.6757508	0.225444	1.737079
9	0.6757508	-0.03718	-0.286479
10	0.8088299	-0.095035	-0.732256
11	0.8088299	0.082315	0.634251
12	0.8088299	-0.22379	-1.724332
13	0.8088299	0.115073	0.886653
14	0.4322923	-0.009284	-0.071538
15	0.8314769	0.000808	0.006226
16	0.7202108	0.128704	0.99168

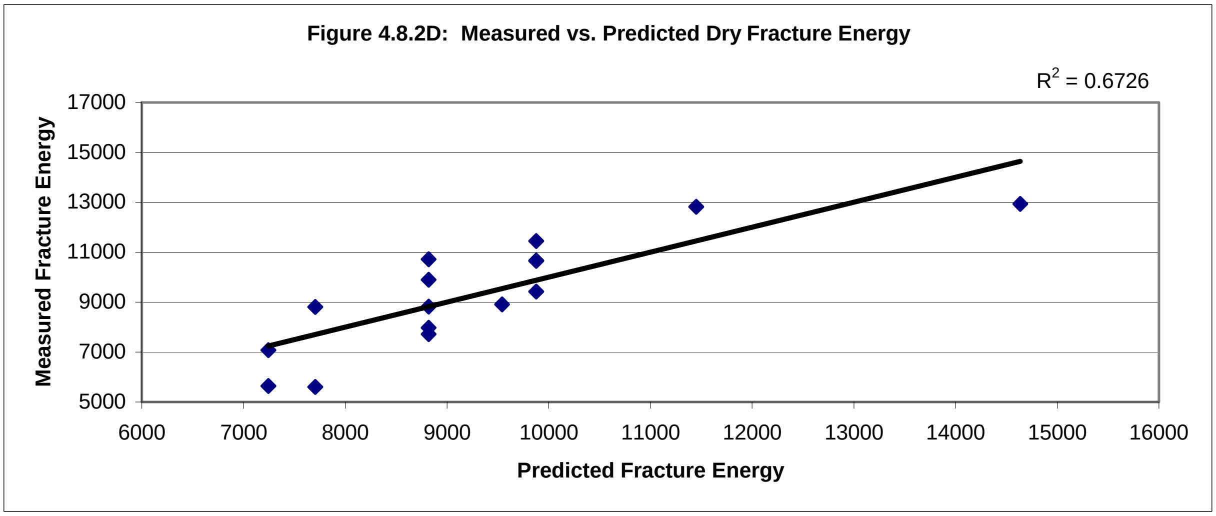


Figure 4.8.2D: Measured vs. Predicted Dry Fracture Energy
$R^2 = 0.6726$
Measured Fracture Energy
17000
15000
13000
11000
9000
7000
5000
6000
7000
8000
9000
10000
11000
12000
13000
14000
15000
16000
Predicted Fracture Energy

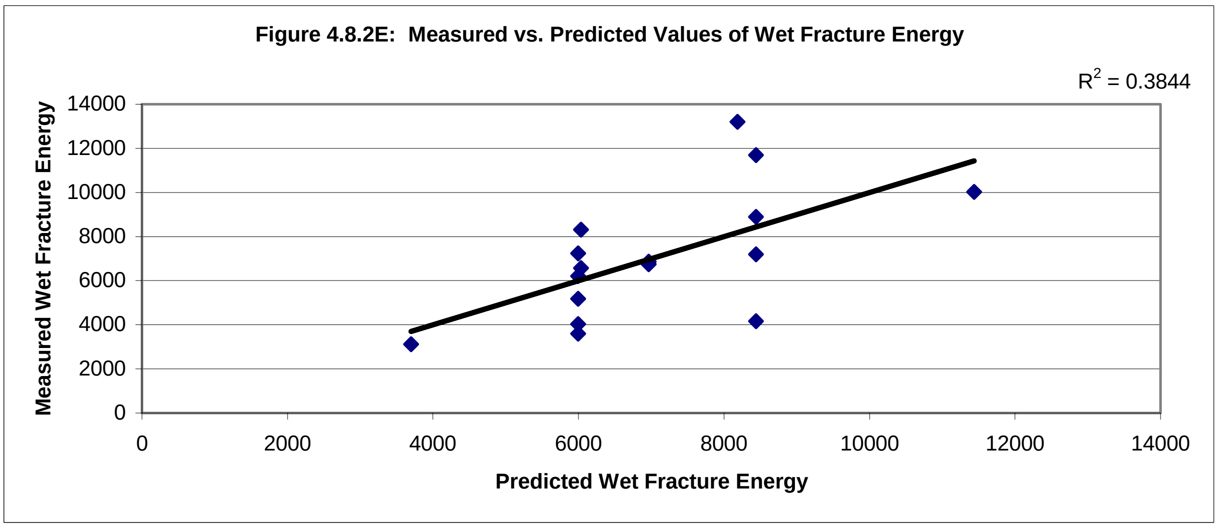


Figure 4.8.2E: Measured vs. Predicted Values of Wet Fracture Energy
$R^2 = 0.3844$
Measured Wet Fracture Energy
14000
12000
10000
8000
6000
4000
2000
0
0
2000
4000
6000
8000
10000
12000
14000
Predicted Wet Fracture Energy

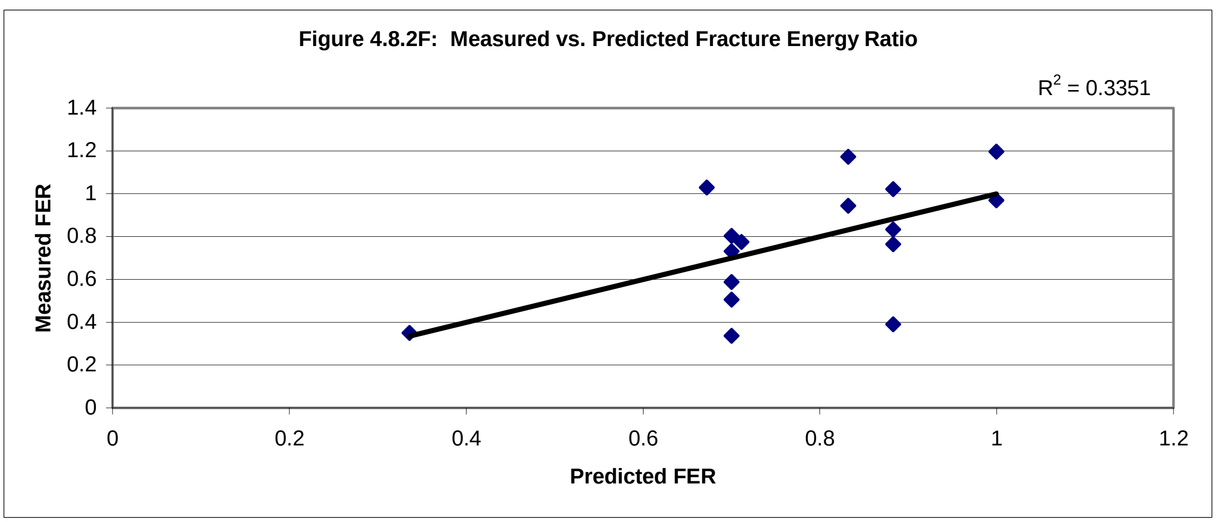


Figure 4.8.2F: Measured vs. Predicted Fracture Energy Ratio
$R^2 = 0.3351$
Measured FER
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
1.2
Predicted FER

Figure 4.8.2G: Regression Analysis: Effect of Binder Properties on Dry Fracture Energy

Regression Statistics	
Multiple R	0.820118562
R Square	0.672594455
Adjusted R Square	0.590743069
Standard Error	1412.876846
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	49210425	16403475	8.217264	0.003062528
Residual	12	23954652	1996221		
Total	15	73165077			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	51658.07386	8582.4481	6.0190371	6.04E-05	32958.5258	70357.62	32958.53	70357.62
G*sind	-0.011862905	0.0027383	-4.332267	0.000975	-0.017829077	-0.005897	-0.017829	-0.005897
G*/sind	0.0016864	0.0012171	1.3856052	0.19108	-0.000965402	0.004338	-0.000965	0.004338
Tack	0.806236074	0.3797642	2.1229912	0.05524	-0.021199046	1.633671	-0.021199	1.633671

RESIDUAL OUTPUT

Observation	*Predicted FE Dry*	*Measured*
1	14637.5319	12939.7
2	11450.71091	12819.52
3	8819.363182	7720.58
4	9876.381721	10664.48
5	8819.363182	9903.21
6	9876.381721	11443.84
7	7242.789678	5635.37
8	7703.804202	5603.83
9	7242.789678	7081.1
10	7703.804202	8811.3
11	8819.363182	10713.35
12	9876.381721	10651.41
13	8819.363182	8819.28
14	9541.676646	8916.06
15	8819.363182	7977.34
16	9876.381721	9425.08

Figure 4.8.2H: Regression Analysis: Effect of Binder Properties on Wet Fracture Energy

Regression Statistics	
Multiple R	0.620019
R Square	0.384424
Adjusted R Square	0.23053
Standard Error	2507.298
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	47110907.3	15703636	2.497976	0.109339394
Residual	12	75438513.2	6286542.8		
Total	15	122549420			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	39450.79	15230.4528	2.590257	0.023647	6266.480823	72635.09	6266.481	72635.09
G*sind	-0.003902	0.00485934	-0.802908	0.437642	-0.014489202	0.006686	-0.014489	0.006686
G*/sind	-0.00275	0.00215985	-1.273435	0.226979	-0.007456324	0.001955	-0.007456	0.001955
Tack	1.364465	0.67393134	2.0246354	0.065746	-0.103904971	2.832836	-0.103905	2.832836

RESIDUAL OUTPUT

Observation	*Predicted FE Wet*	*Measured*
1	11436.53	10026.58
2	8186.888	13195.94
3	5996.768	6205.07
4	8441.442	8889.55
5	5996.768	7245.3
6	8441.442	11698.29
7	6963.65	6743.61
8	6034.6	6572.83
9	6963.65	6868.27
10	6034.6	8314.36
11	5996.768	3597.63
12	8441.442	4165.9
13	5996.768	5184.36
14	3701.362	3124.31
15	5996.768	4037.66
16	8441.442	7201.23

Figure 4.8.2l: Regression Analysis: Effect of Binder Properties on Fracture Energy Ratio

Regression Statistics	
Multiple R	0.578867
R Square	0.335087
Adjusted R Square	0.168859
Standard Error	0.253796
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	3	0.389531	0.129844	2.015825	0.165497842
Residual	12	0.772947	0.064412		
Total	15	1.162479			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-0.033442	1.541669	-0.021692	0.98305	-3.392449809	3.325566669	-3.392449809	3.325566669
G*sind	8.66E-07	4.92E-07	1.759718	0.103899	-2.06143E-07	1.93727E-06	-2.06143E-07	1.93727E-06
G*/sind	-5.31E-07	2.19E-07	-2.430827	0.031685	-1.00779E-06	-5.50967E-08	-1.00779E-06	-5.50967E-08
Tack	7.73E-05	6.82E-05	1.1337	0.279062	-7.12947E-05	0.00022597	-7.12947E-05	0.00022597

RESIDUAL OUTPUT

Observation	*Predicted FER*	*Measured*
1	0.711388	0.77487
2	0.672054	1.029363
3	0.700039	0.803705
4	0.882969	0.833566
5	0.700039	0.731611
6	0.882969	1.022235
7	0.999401	1.196658
8	0.831911	1.172917
9	0.999401	0.969944
10	0.831911	0.943602
11	0.700039	0.335808
12	0.882969	0.391113
13	0.700039	0.587844
14	0.3357	0.350414
15	0.700039	0.506141
16	0.882969	0.76405

4.9.1: Effects of Mixture Properties on Mechanical Testing Results

Figure 4.9.1A: Regression Analysis: Effect of Mixture Properties on Dry Tensile Strength

Regression Statistics	
Multiple R	0.632915795
R Square	0.400582403
Adjusted R Squai	0.100873605
Standard Error	249.7629925
Observations	7

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	166754.8789	83377.43946	1.336572	0.359301455
Residual	4	249526.2097	62381.55242		
Total	6	416281.0886			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1808.123061	625.9545053	2.888585426	0.044626	70.19473889	3546.051383	70.19473889	3546.051383
Fine Agg Contribı	-719.7673905	980.1862349	-0.734316975	0.503479	-3441.200664	2001.665883	-3441.200664	2001.665883
Natural Sand Pro	1542.542457	977.2131888	1.578511705	0.18959	-1170.636318	4255.721231	-1170.636318	4255.721231

RESIDUAL OUTPUT

Observation	*Predicted Dry Strength*	*Residuals*	*Standard Residuals*
1	1679.218016	-316.4180161	-1.55159633
2	1689.408617	197.5913832	0.968914693
3	1771.774961	8.625038537	0.042293983
4	1935.872468	270.3275324	1.325585731
5	2127.883105	-76.48310469	-0.375044715
6	1857.486674	76.21332579	0.373721819
7	2001.156159	-159.8561591	-0.783875181

Figure 4.9.1B: Regression Analysis: Effect of Mixture Properties on Wet Tensile Strength

Regression Statistics	
Multiple R	0.68584711
R Square	0.470386258
Adjusted R Square	0.205579388
Standard Error	346.1481266
Observations	7

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	425676.2	212838.1	1.776337	0.280490715
Residual	4	479274.1	119818.5		
Total	6	904950.3			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*
Intercept	1958.715877	867.5143	2.257848	0.086887	-449.8900851	4367.322	-449.8901
Fine Agg Contributio	-2097.87344	1358.446	-1.544318	0.197394	-5869.525198	1673.778	-5869.525
Natural Sand Propor	1853.765502	1354.326	1.368773	0.242903	-1906.446292	5613.977	-1906.446

RESIDUAL OUTPUT

Observation	*Predicted Wet Strength*	*Residuals*	*Standard Residuals*
1	1006.742222	-223.9422	-0.792355
2	1209.099887	5.100113	0.018045
3	1100.713035	381.087	1.348366
4	1271.572047	378.928	1.340727
5	1810.852415	-21.55241	-0.076257
6	1135.687279	-308.8873	-1.092908
7	1386.533115	-210.7331	-0.745618

Figure 4.9.1C: Regression Analysis: Effect of Mixture Properties on TSR

Regression Statistics	
Multiple R	0.582688256
R Square	0.339525604
Adjusted R Square	0.009288406
Standard Error	0.153461636
Observations	7

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.048426	0.024213	1.028126	0.436226428
Residual	4	0.094202	0.02355		
Total	6	0.142628			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1.039831701	0.384605	2.703638	0.053891	-0.028001935	2.107665	-0.028002	2.107665
Fine Agg Contributi	-0.82013384	0.602255	-1.361772	0.244921	-2.492261487	0.851994	-2.492261	0.851994
Natural Sand Propo	0.428467404	0.600428	0.713603	0.514898	-1.238588434	2.095523	-1.238588	2.095523

RESIDUAL OUTPUT

Observation	*Predicted TSR*	*Residuals*	*Standard Residuals*
1	0.603062818	-0.028657	-0.228707
2	0.701529228	-0.058074	-0.463476
3	0.620089197	0.212196	1.693489
4	0.652416264	0.095703	0.763783
5	0.860957088	0.011277	0.089995
6	0.609674104	-0.1821	-1.453302
7	0.688914225	-0.050344	-0.401782

Figure 4.9.1D: Regression Analysis: Effects of Mixture Properties on Tensile Strength Reduction

Regression Statistics	
Multiple R	0.510531769
R Square	0.260642688
Adjusted R Square	-0.10903597
Standard Error	295.8187456
Observations	7

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	123396.4	61698.21	0.705052	0.546649235
Residual	4	350034.9	87508.73		
Total	6	473431.3			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-150.592816	741.3792	-0.203125	0.848952	-2208.99134	1907.80571	-2208.991345	1907.805712
Fine Agg Contributic	1378.106051	1160.93	1.18707	0.300879	-1845.15361	4601.36571	-1845.153605	4601.365707
Natural Sand Propoı	-311.223045	1157.409	-0.268896	0.801309	-3524.70609	2902.26	-3524.70609	2902.26

RESIDUAL OUTPUT

Observation	*Predicted Reduction*	*Residuals*	*Standard Residuals*
1	672.4757938	-92.47579	-0.382867
2	480.3087296	192.4913	0.79695
3	671.0619268	-372.4619	-1.542062
4	664.3004205	-108.6004	-0.449626
5	317.0306899	-54.93069	-0.227423
6	721.7993953	385.1006	1.594388
7	614.6230441	50.87696	0.21064

Section 4.9.2: Combined Effects of Binder and Mixture Properties on Dry Tensile Strength

Figure 4.9.2A: Regression Analysis: Combined Effects of Binder and Mixture Properties on Dry Tensile Strength

Regression Statistics	
Multiple R	0.671904075
R Square	0.451455086
Adjusted R Square	0.085758477
Standard Error	242.9428815
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	437172.9	72862.15	1.234507	0.372424857
Residual	9	531191.2	59021.24		
Total	15	968364.1			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1423.799292	1853.033	0.768362	0.461957	-2768.051555	5615.65	-2768.052	5615.65
Fine Aggregate Contrib	-174.3310239	793.5543	-0.219684	0.831019	-1969.475648	1620.814	-1969.476	1620.814
Natural Sand Proportior	1091.929345	792.8891	1.377153	0.201746	-701.7104652	2885.569	-701.7105	2885.569
Air Voids Average	-18570.04729	17117.98	-1.084827	0.306201	-57293.60136	20153.51	-57293.6	20153.51
G*sind	337.7804693	615.3137	0.548957	0.59639	-1054.15585	1729.717	-1054.156	1729.717
G*/sind	-17.06954329	236.5389	-0.072164	0.94405	-552.1577216	518.0186	-552.1577	518.0186
Tack	0.018367184	0.065797	0.27915	0.786438	-0.130475562	0.16721	-0.130476	0.16721

RESIDUAL OUTPUT

Observation	*Predicted Dry Average (kPa)*	*Residuals*	*Standard Residuals*
1	1765.437849	-402.6378	-2.139611
2	1791.38212	-304.0521	-1.615728
3	1653.00113	233.9989	1.243466
4	1870.093088	226.9769	1.206152
5	1714.423773	65.97623	0.350597
6	1851.402806	-87.53281	-0.465148
7	2005.332702	200.8673	1.067405
8	1937.527444	85.49256	0.454306
9	2267.587811	-216.1878	-1.148818
10	2138.748997	31.351	0.166599
11	1893.461891	40.23811	0.213825
12	2004.642679	66.39732	0.352834
13	2091.697146	201.6829	1.071739
14	2106.733416	16.23658	0.086281
15	1961.834775	-120.5348	-0.64052
16	2115.912373	-38.27237	-0.203379

Figure 4.9.2B: Regression Analysis: Combined Effects of Binder and Mixture Properties on Wet Tensile Strength

Regression Statistics	
Multiple R	0.754538447
R Square	0.569328268
Adjusted R Square	0.28221378
Standard Error	331.4852787
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	1307336.487	217889.4	1.982931	0.170975883
Residual	9	988942.4102	109882.5		
Total	15	2296278.898			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	3697.078216	2528.38448	1.462229	0.177701	-2022.524834	9416.681	-2022.525	9416.681267
Fine Aggregate Cont	-593.3656397	1082.771304	-0.548006	0.597015	-3042.764495	1856.033	-3042.764	1856.033216
Natural Sand Proport	1383.110475	1081.86365	1.278452	0.233073	-1064.235126	3830.456	-1064.235	3830.456076
Air Voids Average	-36397.9461	23356.75498	-1.558348	0.153579	-89234.59657	16438.7	-89234.6	16438.70436
G*sind	755.7579398	839.5695163	0.900173	0.391478	-1143.480251	2654.996	-1143.48	2654.996131
G*/sind	-572.256113	322.7473245	-1.773078	0.109971	-1302.361283	157.8491	-1302.361	157.8490575
Tack	0.120633689	0.089776976	1.343704	0.211937	-0.082455939	0.323723	-0.082456	0.323723317

RESIDUAL OUTPUT

Observation	*Predicted Wet Average (kPa)*	*Residuals*	*Standard Residuals*
1	1135.849754	-353.0497539	-1.374979
2	1271.870385	53.54961514	0.208553
3	1003.750814	210.4491856	0.81961
4	1514.427637	-17.49763724	-0.068146
5	1461.856037	19.94396259	0.077673
6	1357.753444	139.5965558	0.543669
7	1507.067949	143.4320508	0.558607
8	1282.449362	360.3306375	1.403335
9	2078.242507	-288.9425069	-1.125308
10	1733.996004	121.543996	0.473362
11	1257.433511	-430.6335114	-1.677135
12	1560.520715	-348.8807149	-1.358742
13	1645.981586	420.8484139	1.639026
14	918.2103817	-20.18038174	-0.078594
15	1359.296726	-183.4967261	-0.714642
16	1746.463185	172.9868149	0.67371

Figure 4.9.2C: Regression Analysis: Combined Effects of Binder and Mixture Properties on TSR

Regression Statistics	
Multiple R	0.706262287
R Square	0.498806418
Adjusted R Square	0.164677364
Standard Error	0.150627504
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	0.20322522	0.033871	1.492856	0.282482095
Residual	9	0.204197805	0.022689		
Total	15	0.407423026			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	2.18464656	1.148902436	1.901507	0.089672	-0.41435131	4.783644	-0.41435131	4.783644
Fine Aggregate Con	-0.260923895	0.49201322	-0.530319	0.608724	-1.373935122	0.852087	-1.373935122	0.852087
Natural Sand Propor	0.308338392	0.49160078	0.627213	0.5461	-0.803739832	1.420417	-0.803739832	1.420417
Air Voids Average	-9.70143044	10.61335129	-0.914078	0.38451	-33.71049903	14.30764	-33.71049903	14.30764
G*sind	0.181425945	0.381501892	0.475557	0.64572	-0.68159129	1.044443	-0.68159129	1.044443
G*/sind	-0.269394134	0.146656962	-1.8369	0.0994	-0.601155231	0.062367	-0.601155231	0.062367
Tack	6.61654E-05	4.07948E-05	1.621908	0.139273	-2.61189E-05	0.000158	-2.61189E-05	0.000158

RESIDUAL OUTPUT

Observation	*Predicted Average TSR*	*Residuals*	*Standard Residuals*
1	0.634819462	-0.060413827	-0.517793
2	0.72797153	0.16316897	1.398485
3	0.6193983	0.02405692	0.206186
4	0.812410425	-0.098590667	-0.844998
5	0.850878366	-0.018593486	-0.159361
6	0.733892219	0.11500821	0.985709
7	0.753245374	-0.005126436	-0.043938
8	0.662283562	0.149759819	1.283558
9	0.93073823	-0.058504633	-0.50143
10	0.80789105	0.047157105	0.404173
11	0.651199386	-0.223625306	-1.916642
12	0.78888102	-0.203841619	-1.747081
13	0.754762156	0.146453517	1.25522
14	0.440184763	-0.017178315	-0.147231
15	0.681034808	-0.042464233	-0.363951
16	0.841126746	0.082733981	0.709094

Figure 4.9.2D: Regression Analysis: Combined Effects of Binder and Mixture Properties on Tensile Strength Reduction

Regression Statistics	
Multiple R	0.694083155
R Square	0.481751426
Adjusted R Square	0.136252377
Standard Error	303.4542267
Observations	16

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	770395.5848	128399.3	1.394364	0.313655683
Residual	9	828760.2091	92084.47		
Total	15	1599155.794			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-2273.278925	2314.579278	-0.982156	0.351686	-7509.221006	2962.663	-7509.221	2962.663
Fine Aggregate Contri	419.0346158	991.2100166	0.422751	0.682399	-1823.238218	2661.307	-1823.238	2661.307
Natural Sand Proportic	-291.1811299	990.3791162	-0.29401	0.775418	-2531.574337	1949.212	-2531.574	1949.212
Air Voids Average	17827.89881	21381.66149	0.833794	0.425972	-30540.77978	66196.58	-30540.78	66196.58
G*sind	-417.9774705	768.5738542	-0.543835	0.599766	-2156.612316	1320.657	-2156.612	1320.657
G*/sind	555.1865697	295.4551712	1.879089	0.092941	-113.1794607	1223.553	-113.1795	1223.553
Tack	-0.102266505	0.082185257	-1.244341	0.244803	-0.288182471	0.083649	-0.288182	0.083649

RESIDUAL OUTPUT

Observation	*Predicted Average Strength Reduction (kPa)*	*Residuals*	*Standard Residuals*
1	629.5880949	-49.58809491	-0.210964
2	519.5117351	-357.6017351	-1.521356
3	649.2503155	23.54968449	0.100188
4	355.6654506	244.4745494	1.040075
5	252.5677359	46.03226406	0.195836
6	493.6493619	-227.1293619	-0.966283
7	498.2647531	57.43524691	0.244348
8	655.0780813	-274.8380813	-1.169252
9	189.3453037	72.7546963	0.309522
10	404.7529927	-90.19299267	-0.383711
11	636.0283797	470.8716203	2.003243
12	444.1219643	415.2780357	1.76673
13	445.7155599	-219.1655599	-0.932403
14	1188.523034	36.41696561	0.15493
15	602.5380493	62.96195066	0.267861
16	369.4491877	-211.2591877	-0.898766

Wisconsin Highway Research Program
University of Wisconsin-Madison
1415 Engineering Drive
Madison, WI 53706
608/262-2013
www.whrp.org