NCHRP

REPORT 453

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers

TRANSPORTATION RESEARCH BOARD

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Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers

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SUBJECT AREAS
Pavement Design, Management, and Performance • Materials and Construction

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD — NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. — 2001

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NCHRP REPORT 453

Project D4-23 FY'96

ISSN 0077-5614

ISBN 0-309-06671-9

Library of Congress Control Number 2001-131335

Price $26.00

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Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.national-academies.org/trb/bookstore

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report contains recommendations for performance-based procedures to test and select aggregates for use in unbound layers of asphalt and portland cement concrete pavements. The report provides a comprehensive description of research intended to help materials engineers evaluate and select the aggregates that should contribute to good-performing pavements. Also, the report describes the test methods for the recommended aggregate tests. The contents of this report will be of immediate interest to materials engineers, researchers, and others concerned with the construction and performance of asphalt and portland cement concrete pavements.

The properties of coarse and fine aggregates used in unbound base and subbase layers are very important to the performance of the pavement system in which they are used. Because many of the currently used tests were developed to characterize aggregates empirically without, necessarily, demonstrating any relationship to the performance of the pavement structure incorporating the unbound aggregate layer, their use has contributed to inconsistent aggregate selection that has led to less than desired pavement performance.

Under NCHRP Project 4-23, “Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers,” Applied Research Associates, Inc., was assigned the task of recommending performance-based procedures for testing and selecting aggregates for use in unbound pavement layers. To accomplish this objective, the researchers reviewed relevant domestic and foreign literature; identified aggregate properties that influence the performance of pavements; identified and evaluated, in a laboratory investigation, the aggregate tests currently used in the United States and other countries as well as potential new aggregate tests to measure performance-related properties; and recommended a set of performance-based aggregate tests. The report documents the work performed under NCHRP Project 4-23 and discusses the link between aggregate tests and the performance of asphalt and concrete pavements.

The recommended set of aggregate tests can be used to evaluate and select aggregates for use in the unbound layers of asphalt and concrete pavements. The report includes descriptions of those recommended test methods that are not currently being used in the United States. These test methods will be particularly useful to highway agencies and, therefore, may be considered for adoption by AASHTO as standard test methods.

CONTENTS

PERFORMANCE-RELATED TESTS OF AGGREGATES FOR USE IN UNBOUND PAVEMENT LAYERS

SUMMARY

The performance of unbound, granular pavement layers depends greatly on the properties of the aggregates used. Failures in flexible pavement resulting from poor performance of granular layers are manifested as permanent deformation (rutting), fatigue and longitudinal cracking, depressions, corrugations, and frost heave. Failures in rigid pavements resulting from poor performance of granular layers include pumping, faulting, cracking, corner breaks, and fatigue cracking.

Many current aggregate tests were developed empirically to characterize aggregates and may not have any demonstrated relationship to the performance of the final product. The research conducted for this study was undertaken to evaluate existing tests, identify new tests that relate to performance, and develop better procedures for testing and selecting aggregates for use as unbound pavement base and subbase layers.

Factors contributing to distresses in both rigid and flexible pavements, due to the poor performance of unbound layers, include (1) shear strength, (2) density, (3) gradation, (4) fines content, (5) moisture level, (6) particle angularity and surface texture, (7) degradation during construction and under repeated loads, (8) freeze-thaw cycling, and (9) drainability. Aggregate properties that were determined to affect performance of unbound granular base and subbase layers are shear strength, frost susceptibility, durability, stiffness, and toughness.

For this study, aggregate samples were obtained from different climatic regions of the continental United States. Based on laboratory testing and data analysis, the following aggregate properties and tests related to the performance of aggregates in unbound pavement layers:

Aggregate Property	*Test(s)*
Classification/screening tests	Sieve Analysis Atterberg Limits Moisture-Density Relationship Specific Gravity and Absorption Flat and Elongated Particles Uncompacted Voids
Durability	Magnesium Sulfate Soundness
Shear Strength	Triaxial
Stiffness	Resilient Modulus
Toughness	Micro-Deval
Frost Susceptibility	Tube Suction

Requirements for test parameters for the aggregate properties were established to evaluate aggregates' suitability for use in a particular climate for a certain traffic level. The research team developed a decision chart incorporating aggregate shear strength, stiffness, toughness, durability, and frost susceptibility to provide a measure of the performance potential of a particular aggregate.

The researchers also developed a validation plan to evaluate the research results in the long term. This plan proposes accelerated pavement testing of specially constructed pavement sections and long-term performance monitoring of in-service test pavements.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

PROJECT BACKGROUND

In both flexible and rigid pavements, poor performance of unbound granular layers contributes to reduced life and costly maintenance. Failures in flexible pavements resulting from poor performance of granular bases are manifested as rutting, fatigue cracking, longitudinal cracking, depressions, corrugations, and frost heave. Poor performance of granular bases contributes to pumping, faulting, cracking, and corner breaks of rigid pavements. Clearly, proper aggregate selection is necessary for attaining desired performance.

The performance of unbound base and subbase layers in a pavement system depends on the properties of the aggregates used. Many current aggregate tests were developed empirically to characterize an aggregate, without demonstrating a clear effect on the performance of the final product. Although familiarity, widespread use, and availability of a database have perpetuated the popularity of some tests, the highway industry may be better served by other tests that would provide a clearer relationship to performance.

RESEARCH OBJECTIVE

The objective of the research was to recommend performance-based tests for testing and selecting aggregates for use in unbound pavement layers. The research included evaluating existing aggregate tests to assess their ability to predict pavement performance and, where this predictive ability or a clear relationship to performance was lacking, to develop new tests or modify existing tests. This research was concerned with aggregates used in unbound base and subbase layers of flexible and rigid highway pavements.

SCOPE OF STUDY

This study consisted of two phases. Phase I consisted of Tasks 1 through 7, and Phase II comprised Tasks 8 through 11.

Tasks 1 through 7 of Phase I are described below.

Task 1. Collect and review relevant domestic and foreign literature, research findings, performance data, current practices, and other information relative to the use, testing, and evaluation of aggregates used in unbound pavement layers.

Task 2. Identify the performance parameters of pavements that are affected by the aggregate properties.

Task 3. Identify and discuss the aggregate properties that influence the performance parameters identified in Task 2 and can be used to predict pavement performance.

Task 4. Identify and evaluate—considering performance predictability, precision, accuracy, practicality, cost, and other pertinent factors—those test procedures currently used in the United States and other countries for measuring properties of aggregates used in unbound pavement layers.

Task 5. Identify potential techniques or modifications of current test procedures for measuring those performance-related properties for which no suitable test method was identified in Task 4. Evaluate and rank these techniques considering the same factors used for evaluating available test procedures.

Task 6. Develop a plan, to be executed in Phase II, for an investigation to evaluate and validate the most promising techniques and test methods identified in previous tasks for measuring aggregate properties that relate to pavement performance.

Task 7. Prepare interim report to document the research performed in Tasks 1 through 6 and provide an updated work plan for Phase II, based on the work performed in Task 6.

Tasks 8 through 11 of Phase II are described below.

Task 8. Execute the plan developed in Phase I. Based on the results of this work, recommend tests for evaluating aggregates used in unbound base and subbase layers of flexible and rigid highway pavements.

Task 9. Develop protocols for the tests recommended in Task 8 for which standards are not currently available in a format suitable for consideration and adoption by AASHTO.

Task 10. For the test sets recommended in Task 8, develop a plan for validating their relationship to pavement performance in the long term. Also, recommend an implementation plan for moving the results of this research into practice.

Task 11. Submit a final report that documents the entire research effort.

RESEARCH APPROACH

The research approach for this project included a literature search that is summarized in Chapter 2 and included in Appendix A (not published herein). The approach also included selection of performance parameters that may be influenced by aggregate properties used in unbound pavement layers; the identification, description, and evaluation of aggregate properties that influence these parameters; and identification and evaluation of current aggregate test procedures and potential techniques that can be used to measure these properties.

Aggregate Performance Parameters

Granular base and subbase material properties that influence field performance include stiffness, shear strength, durability, permeability, toughness and abrasion resistance, and geometric characteristics of shape, angularity, and surface texture. These properties address the structural requirements needed to maintain satisfactory support for vehicle loadings and the durability properties associated with degradation and environmental effects.

Aggregate Test Procedures

Specifications for aggregates used in the unbound pavement layer construction in the United States and other countries were reviewed to identify and describe the aggregate test procedures currently used to determine selected aggregate properties. These test procedures were further evaluated on the basis of performance predictability, accuracy, practicality, complexity, precision, and cost.

Test Plan

The approach was to evaluate and select appropriate test methods and conduct tests on 12 different granular base/subbase samples of known performance history. The 12 samples represented both good and poor performers from 7 different states—some from seasonal frost areas and others from nonfrost areas. Test methods were selected and evaluated, and modifications were made to the test methods, for use in the assessment of the performance potential of unbound granular material. Selection of test methods and aggregate samples is discussed in Chapters 3 and 4, respectively.

Evaluation of Potential Test Methods

A plan for evaluating potential aggregate test methods was developed and executed. General guidelines were formulated for selection of aggregates to be tested, selection of test methods to be evaluated, and statistical analysis of data. A three-stage test scheme, described in Chapter 5, was used:

- Stage I: Evaluate and calibrate the test scheme using fabricated aggregate samples.
- Stage II: Identify the range of test parameters and finalize the test scheme using good- and poor-performing samples.
- Stage III: Validate test protocols using aggregate samples from test roads with controlled traffic and closely monitored distress and performance data.

Each aggregate sample was subjected to the selected test methods. Test data were analyzed to determine which test methods have the potential to predict the field performance of aggregates. Test results and analysis are presented in Chapters 5 and 6, respectively. Laboratory test results were correlated to subjective ratings of performance. Test protocols were then developed, in AASHTO format, for recommended test methods for which no standard is available. These protocols are provided in Appendix B.

A plan was developed for validation of the research results through a series of field performance studies. This plan is discussed in Chapter 7 of this report.

CHAPTER 2

FACTORS AFFECTING PERFORMANCE OF UNBOUND PAVEMENT LAYERS

BACKGROUND

Fatigue cracking, rutting/corrugations, depressions, and frost heave of flexible pavements can be attributed to poor performance of granular base and subbase layers. Cracking, pumping, faulting, and frost heave of rigid pavements can be attributed to poor performance of granular base and subbase layers. These distresses and granular base contributing factors are described in Tables 2.1 and 2.2 for flexible and rigid pavements, respectively.

FLEXIBLE PAVEMENTS

Thinly surfaced flexible pavements in which the aggregate performs a major structural role may not fail in the same manner as thicker surfaced pavements (*1*). Failure in the thin asphalt concrete (AC) surfaces will likely be manifested as surface rutting and some fatigue cracking, whereas thicker AC layers will more likely exhibit fatigue cracking as the predominant failure. Across the United States, rutting of flexible pavements is a predominant distress (much of rutting develops in the AC surface layer and is not attributable to the unbound granular layer); fatigue cracking may become a concern for those pavements that do not rut.

Fatigue Cracking

Repeated wheel loads moving along the pavement cause fatigue cracking in flexible pavements when the strain magnitude and/or number of strain repetitions exceed the strain tolerance for the AC mixture. The imposed strain is, in part, a function of the stiffness of the underlying layered structure and therefore a direct function of the base/subbase modulus and thickness. However, different AC mixes can have a range of strain tolerance depending on the mix properties, asphalt grade, temperature, and other factors. Cracking initiates from either the top or the bottom of the pavement, depending on the type of construction and conditions existing at the site. Fatigue cracking in flexible pavements generally develops as a longitudinal crack along the edge of the wheel path. With additional load repetitions, more longitudinal and transverse cracks develop. The interconnecting of these longitudinal and transverse cracks results in the well-known “alligator” cracking. Fatigue cracking is due to the ease with which a pavement system is deformed, caused by low modulus base, subbase, or subgrade and/or lack of adequate structural thickness. Fatigue cracking and rutting often occur in the same pavement, and fatigue-cracking failures can lead to an increase in rutting. Fatigue cracking caused by lack of base stiffness may contribute to failures in the AC surface, which are not necessarily failures within the base. This can be contrasted to shear failures in the base course that are manifested as rutting or displacements in the pavement surface that are truly failures within the base.

Rutting

Rutting is a progressive accumulation of plastic strain in each layer of a flexible pavement that occurs under repeated axle loading. Rutting can be the result of both densification (decrease in volume) and permanent shear deformation (change in shape without a reduction in volume). Test pits at both the AASHO and WASHO Road Tests indicated that rutting in the wheel paths was primarily due to the lateral movement of materials (*2*). Although density is directly related to rutting, the most important mechanism contributing to rutting in highway pavements is shear distortion, not densification. Shear distortion can be reduced or eliminated by increased shear strength and increased internal friction between aggregate particles.

Depressions

Depressions are localized pavement surface areas with elevations slightly lower than those of the surrounding pavement (*3*, *4*). Depressions are generally the result of localized areas in the base or subgrade caused by low initial density that was further compacted under traffic. They can also result from variability in the quality of the base material. Depressions can also result from subsidence of coarse, open-graded base or subbase materials into the weakened subgrade.

Frost Heave

Frost heave causes differential surface heave, which results in surface cracking and uneven surface conditions (*5*). If freezing occurs slowly, frost action results in thick ice lenses being

TABLE 2.1 Flexible pavement distresses and contributing factors

DISTRESS	DESCRIPTION OF DISTRESS	BASE FAILURE MANIFESTATION	CONTRIBUTING FACTORS
Fatigue Cracking	Fatigue cracking first appears as fine, longitudinal hairline cracks running parallel to one another in the wheel path and in the direction of traffic; as the distress progresses the cracks will interconnect, forming many-sided, sharp angled pieces (resulting in the commonly termed alligator cracking); eventually cracks become wider and in later stages some spalling occurs with loose pieces prevalent. Fatigue cracking occurs only in areas subjected to repeated traffic loading.	Lack of base stiffness causes high deflection/strain in the asphalt concrete surface under repeated wheel loads, resulting in fatigue cracking of the asphalt concrete surface. Alligator cracking only occurs in areas where repeated wheel loads are applied. High flexibility in the base allows excessive bending strains in the asphalt concrete surface. The same result can also be caused by inadequate thickness of the base. Changes in base properties with time can render the base inadequate to support loads.	Low modulus base Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Rutting	Rutting appears as a longitudinal surface depression in the wheel path and may not be noticeable except during and following rains. Pavement uplift may occur along the sides of the rut. Rutting results from a permanent deformation in one or more pavement layers or subgrade, usually caused by consolidation and/or lateral movement of the materials due to load.	Inadequate shear strength in the base allows lateral displacement of particles with applications of wheel loads and results in a decrease in the base layer thickness in the wheel path. Rutting may also result from consolidation of the base due to inadequate initial density. Changes in base properties with time due to poor durability or frost effects can result in rutting.	Low shear strength Low density of base material Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Depressions	Depressions are localized low areas in the pavement surface caused by settlement of the foundation soil or consolidation in the subgrade or base/subbase layers due to improper compaction. Depressions can contribute to roughness and can cause hydroplaning when filled with water.	Inadequate initial compaction or nonuniform material conditions result in additional reduction in volume with load applications. Changes in material conditions due to poor durability or frost effects may also result in localized densification with eventual fatigue failure.	Low density of base material
Frost Heave	Frost heave appears as an upward bulge in the pavement surface and may be accompanied by surface cracking, including alligator cracking with resulting potholes. Freezing of underlying layers resulting in an increased volume of material cause the upheaval. An advanced stage of the distortion mode of distress resulting from differential heave is surface cracking with random orientation and spacing.	Ice lenses are created within the base/subbase during freezing temperatures, particularly when freezing occurs slowly, as moisture is pulled from below by capillary action. During spring thaw, large quantities of water are released from the frozen zone, which can include all unbound materials.	Freezing temperatures Source of water Permeability of material high enough to allow free moisture movement to the freezing zone

TABLE 2.2 Rigid pavement distresses and contributing factors

DISTRESS	DESCRIPTION OF DISTRESS	BASE FAILURE MANIFESTATION	CONTRIBUTING FACTORS
Cracking	Longitudinal cracks parallel to the pavement centerline, generally along the wheel path (typically divide slab into two pieces). Cracks result from load applied stresses that exceed the flexural strength of the portland cement concrete (PCC). Fatigue cracking generally results from repeated load stresses but may also be caused by thermal gradients and moisture variations. Corner breaks appear as hairline cracks across slab corners where the crack intersects the joints less than 6 ft from the corner; cracking progresses to result in several broken pieces with spalling of crack and faulting at the crack or joint up to ½ in. or more. The corner break is a crack completely through the slab (as opposed to corner spalls, which intersect the joint at an angle).	Inadequate support from the base/subbase resulting from inadequate shear strength and/or stiffness can increase tensile stresses of slab under repeated wheel loads and result in longitudinal cracking; cracking initiates at the bottom of the slab and propagates to the surface and migrates along the slab; when a crack develops, increased load is placed on the base resulting in deformation within the base and surface roughness of the pavement; the crack introduces moisture to the base resulting in further loss of support and thereby further deformation and roughness. Corner breaks (and associated faulting) may be caused by lack of base support; loss of base support may result from erosion and pumping of the base material; freeze-thaw damage of the base may contribute to loss of support.	Low base stiffness and shear strength Pumping of base/subgrade fines Low density in base Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Pumping/Faulting	Begins as water seeping or bleeding to the surface at joints or cracks and progresses to fine material being pumped to the surface; ultimate condition is an elevation differential at the joint termed faulting. Pumping action is caused by repeated load applications that progressively eject particles of base and subgrade from beneath the slabs.	Pumping involves the formation of a slurry of fines from a saturated base or subgrade, which is ejected through joints or cracks in the pavement under the action of repetitive wheel loads.	Poor drainability (low permeability) Free water in base Low base stiffness and shear strength High fines content Degradation under repeated loads
Frost Heave	Differential heave during freezing and formation of ice lenses causes roughness due to uneven displacement of PCC slabs; thaw weakening results in loss of support from base and subgrade which may cause pumping and faulting and corner breaks; under heavy loads the loss of support can result in cracking of slabs.	Ice lenses are created within the base/subbase during freezing temperatures, particularly when freezing occurs slowly, as moisture is pulled from below by capillary action. Moisture migrates toward the freezing front. During spring thaw, large quantities of water are released from the frozen zone, which can include all unbound materials.	Freezing temperatures Capillary source of water Permeability of material high enough to allow free moisture movement to the freezing zone

formed in the frozen zone. During spring thaw, large quantities of water are then released from the frozen zone, which can include all nonstabilized pavement layers. The release of this water causes weakened pavement conditions leading to both cracking and surface roughness. After thawing, some residual differential settlement often remains. An advanced stage of the distortion mode of distress resulting from differential heave is surface cracking with random orientation and spacing.

RIGID PAVEMENTS

Cracking

Cracking in rigid pavements may be attributable to poor performance of base/subbase and may take the form of longitudinal cracking (fatigue cracking) and corner breaks. Longitudinal cracks will be parallel to the pavement centerline, generally along a wheel path (typically dividing a slab into two pieces). These cracks result from load-applied stresses that exceed the flexural strength of the portland cement concrete (PCC). Fatigue cracking generally results from the combined effects of repeated-load stresses, thermal gradients, and moisture variations. Pumping and faulting may also be associated with fatigue cracking. Corner breaks result when pumping and removal of base material, reduced strength due to increased moisture in the base, and uneven settlement reduce support to the slab corners. Loss of support during spring thaw, when the base material is susceptible to significant strength reduction with increased moisture content, can also result in corner breaks.

Pumping and Faulting

Pumping involves the formation of a slurry of fines from a saturated base or subgrade, movement of this material, and its ejection through joints or cracks in the PCC pavements under the action of repetitive wheel loads. As a load moves across the joint between the slabs, water is first forced under the leading slab and then forced back under the trailing slab. The action erodes and eventually removes particles of base and/or subgrade, resulting in a progressive loss of pavement support. Pumping can occur at transverse joints or along the slab edge as well as at cracks in the slab. Faulting is generally the result of severe pumping but can also result from settlement of underlying layers.

Frost Heave

Frost heave can produce pavement roughness and contribute to loss of shear strength. Differential surface heave results in faulting and uneven surface conditions (*5*). If freezing occurs slowly and water is available, frost action results in thick ice lenses being formed in the frozen zone. During spring thaw, large quantities of water are then released from the frozen zone. The release of this water causes greatly weakened base and subgrade support that leads to cracking, pumping, and faulting.

AGGREGATE PROPERTIES

In discussing aggregate material properties, the properties of the compacted mass and properties of the individual particles were separated. A detailed description of how mass and particle properties affect pavement performance is provided in Appendix A. Appendix A also describes and discusses various test methods available to measure these properties.

Mass Properties

Mass properties are those properties that describe the behavior of the aggregate layer as a continuum. Properties that are important in the performance of aggregate layers include shear strength, stiffness, density, resistance to permanent deformation, permeability, and frost susceptibility. In flexible pavements, the most important property is the shear strength; stiffness is of secondary importance and is closely related to shear strength. In practice, stiffness could probably be correlated to shear strength with sufficient accuracy to negate the need for measuring stiffness in rating an aggregate. For rigid pavements, the primary function of unbound granular bases is to prevent pumping and faulting; thus, the most important mass property is permeability. Although permeability is required to perform the primary task of drainage, shear strength cannot be ignored. A high degree of shear strength is required for construction purposes and to provide protection from base shear under the pavement joints. In areas where the aggregate mass is subject to freeze-thaw, frost susceptibility becomes one of the most important mass properties. Frost heave can be the direct result of a frost-susceptible base.

Particle Properties

Particle properties are important because they affect the mass properties. The researchers identified gradation (may be considered as mass property of particle size distribution), particle shape, particle texture, particle angularity, durability, specific gravity, toughness, and petrographical classification as having significant effects on mass properties.

CHAPTER 3

SELECTION OF CANDIDATE TEST METHODS

This chapter discusses test methods for unbound granular materials selected for evaluation in this research. The test methods were selected based on the criteria shown in Table 3.1 and on the apparent ability to assess the aggregate properties that influence performance of unbound pavement base/subbase layers in flexible and rigid pavements. Appendix A includes a discussion of aggregate properties and test methods for evaluating those properties.

RATING OF TEST METHODS

Table 3.1 gives a subjective, qualitative rating for each test method. The rating in each category for each test method was based on the research team's experience and judgment. The composite rating is based on the relative ratings for each of the followings six categories:

- **Performance Predictability:** The ability of the test method to measure aggregate properties through test parameters and the apparent relationship to performance potential.
- **Accuracy:** The ability of the test method to accurately measure the test parameters of interest.
- **Practicality:** Reasonableness and practicality associated with the test to make the measurement of interest; how easily the test can be applied/developed during the current research.
- **Complexity:** Difficulty of sample preparation, test equipment setup, test operation, and interpreting/understanding the test results.
- **Precision:** The ability to repeatedly provide correct results.
- **Cost:** Relative costs of equipment, sample preparation, and test operation including time for testing.

TEST SELECTION/DEVELOPMENT CONSIDERATIONS

The researchers set up guidelines to select or develop test methods to evaluate the properties of unbound granular materials based on pavement distress, material properties influencing aggregate performance, in situ factors influencing pavement behavior, available test procedures, and current practices. The following guidelines were used:

- Measured aggregate properties must relate to pavement performance.
- The test methods must include all factors that influence the aggregate's performance potential. The aggregate evaluation process must identify sources for possible problems (poor performance) and should be able to evaluate the aggregate with regard to the distresses identified in Chapter 2.
- Test results must be consistent with the current state of knowledge in regard to aggregate performance and must pass the test of reasonableness (i.e., for the most part, very good aggregate should rate as very good and very poor aggregate should rate as very poor).
- Test methods can be easily performed by most state DOTs (at a reasonable cost).
- In situ factors that influence aggregate performance must be considered.
- Test procedures should be as simple as possible.

Table 3.2 shows the linkage between performance parameters and laboratory test measures.

PROPOSED AGGREGATE TESTS

For the most part, the proposed tests are well-documented standard tests. These tests are listed in Table 3.3.

Screening Tests

Screening tests can be used to classify and characterize aggregates by measurements of particle size, plasticity of fine materials, particle strength, and particle shape. The screening tests provide a quick and simple basis for comparing the test aggregates to state DOT specifications. It was also anticipated that some of the performance factors might be quantified by relationships with the results of these tests. The screening tests (at least those that give an indication of good performance potential) would likely become a part of the recommended test protocols. Many of the screening tests directly measure some of the contributing factors identified

TABLE 3.1 Rating of potential test methods

Property Measured	Test Name	Performance Predictability	Accuracy	Practi-cality	Com-plexity	Precision	Cost	Composite
Shear Strength	Static Triaxial Shear	F	G	H	FS	G	M	H
	Repeated Load Triaxial	G	G	H	C	G	M	H
	Texas Triaxial	F	G	M	FS	F	M	M
	Illinois Rapid Shear	F - G	G	M	FS	G	M	M – H
	Confined Compression	F	F	H	S	F	L	M
	Direct Shear	F	F	L	FS	F	M	L
	Gyratory Shear	F	F	M	C	F	M	M
	k-Mould	G	G	M	C	F	M	M - H
	CBR	F	G	M	S	F	L	M - H
	Hveem Stabilometer	F	F	M - H	S	F	L	L - M
	Hollow Cylinder	G	G	L	VC	L	H	L
	Dynamic Cone Penetrometer	F	F	M	S	F	L	M
	Lab Rut-Tester	G	F	L	C	F	H	L - M
Stiffness	Resilient Modulus	F	F	L	C	F	M	H
	Variable Confining Pressure Modulus	F	F	L	VC	F	H	M
	Resonant Column	P	P	L	C	P	M	L
Frost Susceptibility	Frost Susceptibility Test	F	F	L	C	P - F	H	L
	Index Tests	F	G	H	S	F	L	M
Permeability	Constant Head	F	F	M	FS - S	F	L	M
	Falling Head	F	F	H	FS	F	L	M
	Pressure Chamber	F	F	H	FS	F	M	L – M
	Horizontal Permeameter	F	G	H	FS	G	M	M
Toughness	LA Abrasion	F	F	H	S	F	L	M
	Aggregate Impact Value	F	G	H	S	G	L	M
	Aggregate Crushing Value	G	G	H	S	F	L	M – H
	Aggregate Abrasion Value	F	F	H	FS	F	L	M - H
	Micro-Deval	G	F	H	F	F	L	H
	Durability Mill	F	G	H	FS	F	L	M
	Gyratory Test	F	G	H	FS	F	M	M – H
Durability	Sulfate Soundness	F	F	H	F	F	L	M – H
	Freezing and Thawing	G	G	M	FS	F	M	M - G
	Canadian Freeze-Thaw	G	G	M	FS	F	M	M
	Aggregate Durability Index	F	F	H	FS	F	L	M - H
	Unconfined Freeze Thaw	F	F	H	FS	F	M	M
Mineralogical Composition	Petrographic Examination	F	F	L	C	F	H	L - M
Particle Geometric Properties	Particle Shape and Surface Texture Index	F	F	M	S	F	L	M
	Flat and Elongated Particles	P	F	L	S	F	L	M
	Percentage of Fractured Particles	G	G	H	FS	F	L	M – H
	Uncompacted Void Content	F	F	M	S	F	L	M
	Digital Image Analysis	P	F	M	C	F	H	L – M
	Atterberg Limits	F	G	M	S	F	L	M
	Sand Equivalent Test	F	F	M	S	F	L	M
	Methylene Blue Test	F	G	M	F	F	L	M

Rating Scale:

Performance Predictability - G = good, F = fair, P = poor
Accuracy - G = good, F = fair, P = poor
Practicality - H = high, M = medium, L = low
Complexity Levels - S = simple, FS = fairly simple, C = complex, VC = very complex
Precision - G = good, F = fair, P = poor
Cost - H = high, M = medium, L = low
Composite – H = high, M = medium, L = low (based on relative ratings of other factors)

Note: All ratings are "average" subjective evaluations of research team. The composite rating is based on the relative ratings for each category.

in Tables 2.1 and 2.2. The screening tests described below were selected as the best means of classifying and characterizing aggregates.

Gradation

Aggregate mass gradation is an important indicator of aggregate performance and is a factor for most agencies in the selection of an aggregate. The gradation is used to indicate permeability, frost susceptibility, and shear strength. The gradation tests for each aggregate material were determined in accordance with AASHTO T 27 (*6*).

Atterberg Limits

The liquid limit of aggregate fraction finer than the 0.425-mm (No. 40) sieve is determined using the test procedure given in AASHTO T 89 (*6*). The plastic limit for the fine fraction is determined in accordance with AASHTO T 90 (*6*). Atterberg Limits are index test results that are moisture

TABLE 3.2 Linkage between aggregate properties and performance

Pavement Type	Performance Parameter	Related Aggregate Property	Test Parameters That May Relate To Performance
Flexible	Fatigue Cracking	Stiffness	Resilient modulus, Poisson's ratio, gradation, fines content, particle angularity and surface texture, frost susceptibility degradation of particles, density
	Rutting, Corrugations	Shear Strength	Failure stress, angle of internal friction, cohesion, gradation, fines content, particle geometrics (texture, shape, angularity), density, moisture effects
	Fatigue Cracking, Rutting, Corrugations	Toughness	Particle strength, particle degradation, particle size, gradation, high fines
		Durability	Particle deterioration, strength loss
		Frost Susceptibility	Permeability, gradation, percent minus 0.02 mm size, density, fines type
		Permeability	Gradation, fines content, density
Rigid	Cracking, Pumping, Faulting	Shear Strength	Failure stress, angle of internal friction, cohesion, gradation, fines content, particle geometrics (texture, shape, angularity), density, moisture effects
		Stiffness	Resilient modulus, Poisson's ratio
		Toughness	Particle strength, particle degradation, particle size, gradation
		Durability	Particle deterioration, strength loss
	Cracking, Pumping, Faulting, Roughness	Permeability	Gradation, fines content, density
		Frost Susceptibility	Permeability, gradation, percent minus 0.02 mm size, density, fines type

TABLE 3.3 Tests selected for the laboratory test program

Aggregate Property	Test Method	Test Reference	Test Parameter
Screening Tests	Sieve Analysis	T 27,T 11[a]	Particle size distribution
	Atterberg Limits	T 89,T 90[a]	PL, LL, PI
	Specific Gravity and Absorption	T 84,T 85[a]	Specific gravity
	Moisture/Density Relationship	T 99, T 180[a]	Maximum dry density
	Flat and Elongated Particles	D 4971[b]	F or E, F and E
	Uncompacted Void Content	TP 33[a]	Percent uncompacted void
	Shape and Texture	D 3398[b]	Particle shape and texture index
Shear Strength	Static Triaxial Shear	T 296[a]	c, ϕ, shear strength
	Repeated Load Triaxial		Deviator stress
	California Bearing Ratio	T 193[a]	CBR
Stiffness	Repeated Load Triaxial	**	Resilient modulus
Frost Susceptibility	Tube Suction Test	*	Dielectric constant
	Index Method	*	F categories
Toughness and Abrasion	LA Abrasion	C 131[b]	% loss, passing #12 sieve
	Agg. Impact Value	BS 812[c]	% loss, passing BS 2.40 mm sieve
	Agg. Crushing Value	BS 812[c]	% loss, passing BS 2.40 mm sieve
	Micro-Deval Test	TP58-99[a]	% loss, passing #16 sieve
	Gyratory Degradation		Before and after gradation
Durability	Sulfate Soundness	T 104[a]	Weighted average loss
	Agg. Durability Index	T 210, T 176[a]	Durability index

a: AASHTO reference test method
b: ASTM reference test method
c: British reference test method
*: No test method is currently available
**: Test method developed in this research

contents at which the fine content (passing No. 200 sieve) change from one state into another (i.e., from solid to semisolid as moisture increases beyond Plastic Limit).

Specific Gravity and Absorption

Specific gravity is the ratio of the weight of a material of a given volume to the weight of a similar volume of water. Most natural aggregates have a specific gravity of 2.6 to 2.7, although values of 2.4 to 3.0 may be encountered. Specific gravity is not an indication of quality in itself; however, it can be an indication of potential problems and is needed for computations involving volume and mass. Absorption has been used as a crude indicator of aggregate durability as related to freezing and thawing. High absorption is often an indicator of unsound aggregates. Test methods used are AASHTO T 84 and T 85 (*6*).

Moisture/Density Relationship

Laboratory compaction is important to determine the anticipated density achievable in the field and for fabrication of laboratory specimens for other tests. Laboratory compaction is conducted in accordance with AASHTO T 99 and/or T 180 (*6*). Compaction of aggregate materials generally results in increasing density, shear strength, and stiffness and decreasing permeability with increasing moisture content to a point of maximum density beyond which the trends reverse. The point of maximum density is a function of compactive effort.

Flat and Elongated Particles

Flat and elongated particles can break under compaction and change aggregate gradation; they can also influence compactability. The shape of the aggregate particles in terms of the percentage of flat or elongated particles is determined in accordance with test procedure ASTM D 4791 (*7*). An excess of such particles can be detrimental to good performance.

Uncompacted Void Content

The uncompacted void content is an index that is a function of particle shape, angularity, and surface texture and could provide a good overall indicator of the potential for resisting permanent deformation. This test is conducted in two parts. The aggregate sample is split between a coarse fraction, which is the material retained on the 4.75-mm (No. 4) sieve, and the fine fraction, which is the material passing the 4.75-mm (No. 4) sieve. For the coarse fraction test, Method 1 as presented by Ahlrich (*8*) is used. For the fine fraction, the prescribed test is Method C of ASTM C 1252 (*7*) or AASHTO TP33 (*9*). This rather simple test appears to have considerable merit for assessing particle shape and texture.

Index of Aggregate Particle Shape and Texture

The Index of Aggregate Particle Shape and Texture, in accordance with ASTM D 3398 (*7*), is a measure of the voids in the aggregate determined at different compaction levels. The voids are determined for various size fractions ranging from ½- to 1-in. down to the No. 100 to No. 200 material. The index is an indirect, empirical measure of the particle shape and texture, but it does allow comparison of the various size fractions in the aggregate blend.

Shear Strength

Shear strength was identified as the single most important property that governs unbound pavement layer performance. Several test methods were identified for measuring shear strength; the triaxial shear test appeared to be the best candidate test for the following reasons:

- The test is the universally accepted one for measuring soil strength;
- Most state DOT laboratories have the capability to conduct the test;
- The test method can accommodate different stress states;
- The test method can consider repetitions of stress;
- The test method provides a measure of resilient and permanent deformation;
- The test method can accommodate changes in moisture content; and
- The degree of test complexities can be varied to suit the test objectives.

An additional advantage of the triaxial test is that, during the back-pressure saturation process, a measure of permeability of material can be obtained. Two versions of the triaxial tests were recommended: the static triaxial test and the repeated load test (described in Appendix A).

Although there is little doubt as to the merit of the triaxial shear test, there are some disadvantages to its use. The test, especially the repeated load tests, can be complex and require specialized equipment. Samples are difficult to prepare, and the test is time-consuming and difficult to set up. A major effort was devoted to simplifying the test procedure to the most usable method and still produce the desired aggregate evaluation. To simplify the test, it is probably desirable to limit the test procedure with respect to state of stress, number of stress repetitions, and state of moisture. This is possible because the objective of the test is not to provide design parameters, but to evaluate the aggregate's performance potential.

Static Triaxial Tests

The static triaxial test is simple to conduct and is well accepted in geotechnical applications. The recommended static triaxial test stipulated confining stresses of 5, 10, and 15 psi for both dry and wet conditions. Sample preparation for these tests is the same as for the repeated load triaxial shear tests. The time between sample preparation and testing may be shortened to permit testing in 1 day. AASHTO T 296 is the reference test method.

Repeated Load Triaxial Shear Tests

The repeated load triaxial shear test was selected to give a relative measure of an aggregate's ability to resist permanent deformation. The proposed test procedure considers repeated loads at a rate of loading similar to actual traffic loading but is also convenient for laboratory application.

Because changing the confining pressure from 5 to 20 psi did not affect the relative ranking of an aggregate's shear strength, one confining pressure was adopted for the test. The recommended test, which was carried to failure, was conducted at a confining pressure of 15 psi. Increasing levels of repeated load were applied until the aggregate sample developed sufficient permanent deformation (10 percent) to be considered failed. Each load level was applied for a set number of load applications. The time of testing depended on the number of load levels, the selected number of load cycles per load level, and the loading frequency. The number of load cycles per load level and the load cycle rate can be set, but the number of load levels depends on the strength of the aggregate sample. To keep the testing time reasonable, 1,000 load cycles were applied at each load level at a rate of 30 load cycles per minute. At this rate, each load level required approximately 35 min to complete.

The test was designed so that no more than eight load levels are required for failure of even the strongest material. Thus the time to conduct the test was no more than 5 hr. Both the dry and wet aggregate samples were prepared at 100 percent of the AASHTO T 180 (*6*) density and at optimum moisture content. For the dry test, the samples were allowed to consolidate overnight in the triaxial cell under a confining pressure of 15 psi for 12 to 16 hr. The test was conducted at the completion of the consolidation period. For the wet test, the sample was prepared in the same manner as for the dry sample but was saturated prior to consolidation. After saturation, the confining pressure was increased to the desired level and the sample was allowed to equalize, undrained, for the same length of time allowed for the dry sample to consolidate. After equalizing, the sample was allowed to drain for a period of 1 hr before testing. The testing started immediately after the 1-hr drainage, and drainage of the sample continued during testing. This procedure did not include sample conditioning or resilient modulus determination. However, the sequence of the loading is such that the first few load cycles can be considered conditioning for that load increment.

The primary concern in the measurement of permanent deformation is the proper seating of load-bearing plates at the top and bottom of the specimens. A correction for seating may be made using the load deformation curve for the first load cycle of each load increment. The test procedure is described in Appendix B.

The results of the repeated load testing can be presented as plots of permanent deformation versus time, permanent deformation versus principal stress ratio, and permanent deformation occurring within a load level versus the load cycles. Failure may be determined based on a limiting permanent strain, the shape of the load-deformation curve, the shape of the deformation-cycle curve, or a combination of criteria.

Cylindrical specimens with heights approximately twice their diameter are used for the triaxial shear test. This ratio is recommended so that small variations in length could not affect the strength considerably. Consideration was given to minimum specimen dimensions. Recent criteria for testing granular materials and subgrade soils (AASHTO T 294-94) require that the minimum specimen dimension be five times the "nominal" particle size (*6*). Nominal in this case is defined as the sieve opening for which 95 percent of the material passes. Based on these requirements, the specimen diameter was set at 6 in. with a height-to-diameter ratio between 2 and 2.5.

The orientation of particles in some soils is dependent on the method of compaction. This is particularly true for fine-grained soils. The method of compaction used in the laboratory should produce a material that is similar to that produced in the field. Test specimens for this research were fabricated using vibratory compaction as described in Appendix B.

While running the rapid shear test, Thompson and Smith (*10*) concluded that sample conditioning had a significant effect on test results and that conditioned samples are more representative of the strength of in-service granular base materials. For this research effort, conditioning of test specimens was provided by the initial repetitions for each load level.

Thadkamalla and George (*11*) studied the influence of the method and degree of saturation on the resilient modulus of soils. Coarse-grained soils were not significantly affected by the method or degree of saturation; however, both method and degree of saturation affected fine-grained soils. For this research, two methods of sample saturation, seepage saturation and back-pressure saturation, were studied; details are provided in Appendix B. All the samples were saturated using the seepage saturation method.

California Bearing Ratio (CBR)

The research team selected the CBR test (both soaked and unsoaked) because of its widespread use as a strength parameter in pavement structural design and because of its long-term historical acceptance as an indicator of performance. CBR tests on granular materials must be conducted using several samples to ensure accurate results. AASHTO T 193 is the reference test method (*6*).

Stiffness

Triaxial testing can be developed to obtain a stiffness value without adding significant complexities to the test method. Because the study was not aimed at determining design parameters, conducting a full resilient modulus test to determine the stiffness as a function of the state of stress was not necessary.

Because resilient modulus values can be determined from the repeated load triaxial test, a stiffness test was not needed. In the triaxial testing, the resilient modulus is determined at the end of each load increment and is then presented as a function of the principal stress ratio.

Frost Susceptibility

Rating of frost susceptibility, in terms of the F1, F2, F3, and F4 categories, from the grain size distribution and the Corps of Engineers method should give satisfactory assessment of frost susceptibility (*12*). Frost susceptibility was also determined using the dielectric constant value from the Tube Suction test (*13*).

Toughness and Abrasion

The aggregate toughness and abrasion resistance is determined using both the Micro-Deval test (*9*) and the Los Angeles abrasion test (*6*).

The Micro-Deval test is performed on an aggregate sample consisting of 250 grams of ¾- to ½-in. material and 250 gm of ½- to ⅜-in. material. The sample is soaked in water for 24 hr and placed in a jar mill with 2.5 l of water and an abrasive charge consisting of 11 lb of ⅜-in.-diam steel balls. The jar, aggregate, water, and abrasive charge are revolved at 100 rpm for 2 hr. The sample is then washed and dried. The amount of material passing the No. 16 sieve is determined, and the loss, expressed as a percent by weight of the original sample, is calculated.

The Los Angeles abrasion test has long been used by many agencies as an index for aggregate toughness. Because of its historical use and past acceptance, as well as documented relationships to aggregate abrasion, the test, as described in AASHTO T 96, was included in this research.

The Corps of Engineers gyratory test machine was selected to produce degradation of the aggregates. This test measured aggregate breakdown under selected revolutions at specified stress levels.

Additional tests recommended for study of aggregate toughness and abrasion resistance were the aggregate impact value (AIV), and the aggregate crushing value (ACV) tests. These British tests (*12*) are carried out on aggregate fractions between 14.0 and 10.0 mm. The test protocol followed is BS 812, part 3. For the AIV test, the aggregate particles are placed in a cylindrical rigid mold and subjected to 15 blows using a 30-lb drop hammer dropped from a height of 15 in. The percentage of material passing the No. 8 sieve is termed as the AIV. For the ACV test, 4.4 lb of aggregate are placed in a specially designed apparatus and subjected to a load of 90,000 lb (increased gradually over a period of 10 min). The percentage of fines created that pass the No. 8 sieve is designated as the ACV.

Durability

Aggregate durability was determined using the magnesium sulfate soundness test, and the Aggregate Durability Index procedure (*6*).

CHAPTER **4**

SELECTION OF AGGREGATE SAMPLES FOR LABORATORY TEST PROGRAM

In selecting or developing test procedures, it is important to understand how they will be used in aggregate evaluation. The proposed evaluation system was based on the assumption that aggregates leading to different levels of performance can be identified through tests. At the top of the performance scale is the very good aggregate, which is 100-percent crushed rock with nonplastic fines. This aggregate has a high permeability and high shear strength; moisture does not appreciably reduce shear strength, and the aggregate is not frost-susceptible. An aggregate that has all rounded (uncrushed) particles, poor gradation, and a large quantity of high-plasticity fines would be at the other end of the scale. This aggregate has low shear strength at optimum water content and loses shear strength when exposed to water.

The laboratory testing program included three stages:

- Stage I: Identification and calibration of test scheme by testing fabricated samples,
- Stage II: Tests on aggregate with known performance history, and
- Stage III: Tests on aggregate from field sites.

AGGREGATE MATERIAL SELECTION

All aggregates were selected from sources used by state DOTs in highway construction projects. Candidate aggregate sources represented different regions of the United States, covering a range of climatic conditions and aggregate types. For shear strength and stiffness tests, the aggregates ranged from materials with all crushed, hard aggregate particles to some combinations of rounded and crushed materials with lower hardness qualities and a range of fines content. For durability and toughness tests, a range of materials was included—those with high resistance to degradation and deterioration and those susceptible to particle breakdown.

Aggregates selected for the laboratory test program were based on the 1995 production (*14*) of processed aggregates by rock type, as shown in Table 4.1. The crushed aggregates were composed of limestone, granite, traprock, sandstone, and naturally occurring river gravels and glacial deposits. Table 4.2 shows the sources of aggregate materials used in this research.

Stage I Laboratory Samples

The primary objective of Stage I testing was to establish a shear strength range for aggregate materials. Aggregate samples were fabricated to obtain a range of potential performance. Stage I tests also established the methods for specimen preparation, moisture levels, and moisture conditioning procedure. The following three aggregates, designated Materials A, B, and C, were blended to fabricate samples for Stage I testing:

- Material A: Crushed dolomitic limestone from Vulcan materials in Vance, Alabama;
- Material B: Uncrushed natural gravel from American Sand and Gravel in Hattiesburg, Mississippi; and
- Material C: Clay obtained from AASHTO Materials Reference Laboratory (AMRL).

Five samples, designated Samples I-1 through I-5, were fabricated using these materials. Sample I-1 consisted of crushed limestone (Material A) with 8- to 10-percent fines (30 percent +4.75 mm, 70 percent −4.75 mm). This sample was fabricated to establish the upper range of shear strength. Sample I-1 was considered the strongest material, performing well when wet and dry, with durability, toughness, and frost resistance.

Sample I-2 consisted of crushed limestone (Material A) with no more than 3-percent fines (77 percent +4.75 mm, 23 percent −4.75 mm). This sample was fabricated to demonstrate the effect of gradation and moisture; it drained well with little strength reduction when wet.

Sample I-3 was fabricated with 50-percent Material A, 26 percent Material B (coarse fraction, +4.75 mm), and 24 percent Material C (fine fraction, −4.75 mm). This sample established the midrange of shear strength; it performed well when dry but poorly when wet, had poor frost resistance, and was tough and durable.

Sample I-4 was 100-percent Material B, established the lower end of the shear strength range, and was tough, durable, and frost-resistant.

Sample I-5 consisted of 61-percent Material B and 39-percent Material C. This sample established the lower end of the dry shear strength range. It was tough and durable, but had poor frost resistance due to the presence of clay fines.

TABLE 4.1 Production of aggregate in the United States in 1995 (*14*)

Rock Type	1995 Tons Produced	Producer States	Leading Producers
Limestone	804×10^6	47	Texas, Florida, Missouri, Kentucky, Illinois, Alabama
Dolomite	93×10^6	26	Pennsylvania, Ohio, Illinois, Michigan, Indiana
Granite	194×10^6	34	Georgia, North Carolina, Virginia, South Carolina, Arkansas
Traprock	101×10^6	27	Oregon, Virginia, Washington, New Jersey, Idaho
Sandstone	33×10^6	25	Arkansas, Pennsylvania, South Dakota

TABLE 4.2 Sources of test aggregates

Rock Type	Source Location	Selected Source
Basalt	Oregon	Oregon
Chert River Gravel	Mississippi, New Jersey	Mississippi
Dolomite	Pennsylvania, Ohio, Indiana	Indiana
Gabbro	California	California
Glacial Till (Gravels)	Minnesota, Wisconsin, Iowa	Minnesota
Granite	Georgia, Virginia, North Carolina	Virginia
Limestone	Alabama, Kentucky, Texas	Alabama
Sandstone	Arkansas, Pennsylvania	Pennsylvania

TABLE 4.3 Stage II aggregates

Sample No.	Supplier State	Rock Type	State DOT-Provided Information	
			Performance	Comments
II-6	Pennsylvania	Sandstone	Poor	Used mainly for subbase
II-7	Pennsylvania	Sandstone	Good	
II-8	Virginia	Granite	Good	
II-9	Virginia	Granite	Poor	Similar to sample II-8, except has mica and often is slightly off the gradation specifications
II-10	Minnesota	Glacial Till	Good	Sand and gravel
II-11	Minnesota	Limestone	Poor	Limestone
II-12	California	Gabbro	Good	Good quality aggregate; similar to aggregate used in hot-mix asphalt concrete
II-13	Texas	Caliche limestone	Good	Material is used as base layer after treating with 1% (by weight) lime
II-14	Indiana	Dolomite	Good	
II-15	Indiana	Dolomite	Poor	Absorption is higher than sample II-14
II-16	Oregon	Basalt	Good	No incidence of base failure during the last 5-10 years
II-17	Texas	Gravel	Marginal	Base layer incorporating this material has a propensity to crack due to environment

Stage II Laboratory Samples

Stage II laboratory tests were conducted to develop the full range of the limiting test parameters and to finalize the test procedure. Laboratory tests were conducted on aggregates that covered a range of geologic compositions from various climatic regions within the United States. The test samples consisted of traprock, dolomite, limestone, sandstone, granite, crushed river gravel, pit-run gravel, and glacial gravel with known performance histories ranging from good to poor as provided by supplier state DOTs.

Twelve samples, designated Samples II-6 through II-17, were used in Stage II laboratory testing. Table 4.3 describes the Stage II samples. The aggregates were selected with consideration of known performance and ranged from materials with all crushed, hard aggregate particles to some combinations of rounded and crushed materials with borderline hardness qualities. The samples also ranged from highly resistant to degradation to materials with poor durability properties. The materials selected represented materials used on highway projects meeting the high and low end of state base/subbase specifications.

Stage III Laboratory Samples

Stage III test samples were designated Samples III-18 through III-20 and included one sample from the Ohio Test Road and two samples from the Minnesota Road Research Facility (MnRoad). Test materials, similar to those used in the construction of these test roads, are identified in Table 4.4.

TABLE 4.4 Stage III aggregates

Sample No.	Description	Performance Rating
III-18	MnRoad Class 6 (Granite)	Good
III-19	MnRoad Class 5 (Gravel)	Marginal
III-20	Ohio Test Road Gravel	Good

CHAPTER 5

LABORATORY TEST PROGRAM AND TEST RESULTS

TEST PROGRAM BACKGROUND

The laboratory tests included in this research are shown in Table 3.3. All Stage I, II, and III samples were subjected to these tests with the exception that flat and elongation, particle shape and texture, and uncompacted voids tests were not conducted on Stage III samples.

Test conditions were designed to simulate in-service conditions of stress levels, moisture, density, and material drainage. For example, triaxial tests were conducted on partially saturated compacted aggregate samples with free drainage using applied stress levels to simulate imposed repeated vehicle loads.

Sample preparation followed, as closely as possible, standard methods and procedures of sample splitting, handling, compaction, and moisture control. The samples received from supplier states consisted of a range of particle sizes. Some tests required sample separation into various sieve sizes to conduct tests on certain fractions, and a few tests were conducted on aggregate particles (e.g., crushing strength of base rock).

STAGE I LABORATORY TEST RESULTS

The primary objective of Stage I testing was to establish a shear strength range for aggregate materials. Tests results are presented in the following sections.

Grain Size Including Hydrometer Analysis

Aggregate gradation is an indicator of aggregate performance and is currently used by most agencies in aggregate selection. The gradation can be used to estimate permeability, frost susceptibility, and shear strength. Test methods AASHTO T 27 and T 11 were used. Figure 5.1 shows the gradation of Stage I samples.

Moisture/Density Relationship

Laboratory compaction is required for determining the density achievable in the field and for manufacture of laboratory specimens for other tests. Compaction of aggregate materials generally increases density, shear strength, and stiffness, and decreases permeability. Test Method D of AASHTO T 180 was used. The results are shown in Table 5.1.

California Bearing Ratio

The CBR tests, both soaked and unsoaked, were conducted in accordance with AASHTO T 193. Figure 5.2 shows the CBR curves for Sample I-1, and CBR test results for Stage I samples are shown in Figure 5.3. The soaked CBR was always less than the unsoaked CBR. Sample I-5 had higher CBR values than Sample I-4, demonstrating that a certain amount of fines can contribute to an increased strength. Under both unsoaked and soaked conditions, Sample I-4 produced the lowest CBR values.

Static Triaxial Test

The static triaxial test was conducted in accordance with AASHTO T 234 (ASTM D 2850) on each sample to measure shear strength and to serve as a reference for the repeated load triaxial tests. The tests were conducted at confining stresses of 5, 10, and 15 psi, to determine both the dry and wet strengths. Sample preparation for these tests is described in Chapter 3.

Both the dry and wet aggregate samples were prepared at 100 percent of the AASHTO T 180 density and at optimum moisture content. The dry condition was defined by optimum moisture, and the wet condition was represented by approximately 90-percent saturation (but determined by the drainability of the material). For the dry test, the sample was allowed to consolidate in the triaxial cell under the test confining pressure for 12 to 16 hr before the test was conducted.

The sample, for the wet test, was prepared in the same manner as for the dry sample, but was saturated prior to consolidation. After saturation, the confining pressure was increased to the desired level and the undrained sample was allowed to equalize for the same length of time allowed for the dry sample to consolidate and then allowed to drain for a period of 1 hr before testing was performed. Sample drainage was allowed during testing.

Table 5.2 shows failure stresses obtained for the five Stage I samples at the three levels of confining stresses for the moisture conditions. Table 5.3 shows the cohesion (c) and angle of internal friction (*10*) values.

Figure 5.4 shows the effect of the confining stresses on the deviator stresses for Stage I samples. For each sample, the deviator stress increased as the confining stress increased.

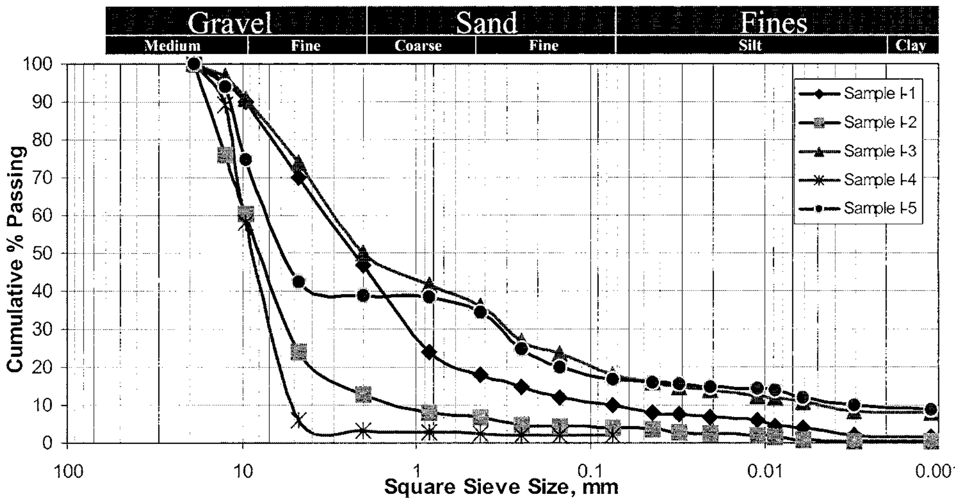

Figure 5.1. Grain size–hydrometer analysis for Stage I samples.

Results of the static triaxial test confirmed the expected order of the five samples with respect to shear strength.

Repeated Load Triaxial Test

Repeated load triaxial shear tests were conducted to obtain a relative measure of an aggregate to resist permanent deformation. Because no standard test procedure was available, one was developed to consider repeated loads at a rate of loading to simulate field loading but also convenient for laboratory application.

The repeated load tests were conducted at a confining pressure of 15 psi. An array of load increments was applied, with 1,000 repetitions at each load level. The load level was increased until the aggregate sample failed. The sample was considered failed when the permanent deformation reached 10 percent. The test time depended on the selected number of load cycles per load level, the load cycle rate, and the number of load levels. The number of load cycles per load level and the load cycle rate can be set, but the number of load levels depends on the strength of the aggregate sample. To keep the testing time reasonable, the 1,000 cycles at each load level were applied at a rate of 30 cycles per min. At this rate, each load level required approximately 35 min to complete. The test was designed so that seven load levels were adequate for even the strongest material, and thus, no more than 5 hr was required to complete the test.

TABLE 5.1 Optimum moisture/density for Stage I samples (AASHTO T 180)

Sample No.	Optimum Moisture Content (%)	Maximum Dry Density (pcf)
I-1	6.3	142.0
I-2	4.5	129.5
I-3	5.0	140.0
I-4	--[a]	--[a]
I-5	6.5	135.0

a: 100% uncrushed river gravel (free draining), moisture density curve could not be developed

The load levels were based on the failure strength exhibited by the samples during static triaxial testing. Six load increments representing 15, 25, 50, 75, 100, and 110 percent of the static triaxial failure strength were used in the test. If the specimen did not fail, testing was continued until 120 percent of the static triaxial test failure strength was reached. A haversine load pulse of 0.1-sec load duration, shown in Figure 5.5, was used to apply load to the test specimen. Each load pulse was followed by a 0.9-sec relation period. The procedure does not allow for sample conditioning, but the first few load cycles at each load increment were considered conditioning for that load increment.

Figure 5.6 shows the test results for Sample I-1; similar curves were obtained for each sample. However, Samples I-1, I-2, and I-3 did not fail even when the load was increased to 120 percent of the failure stress of the static triaxial test. Figure 5.7 compares the curves for the Stage I samples. The “start” and “end” curves represent the permanent strain at the beginning (average of repetitions from 96 to 100) and at the end (average of repetitions from 996 to 1,000) of a repeated stress loading, respectively.

An examination of the repeated load triaxial test data suggests that the deviator stress at 2-percent strain provides a better criterion for ranking the samples than failure stress. Sample rankings based on the deviator stress at 2-percent strain are shown in Figure 5.8. The 2-percent strain criteria corresponds to a 0.2-in. deformation in a 10-in. unbound aggregate base. A second option was to use the deviator stress at which a rapid increase, “break-over” point, in permanent strain takes place, as shown in Figure 5.9. This approach could not be fully evaluated because Samples I-1, I-2, and I-3 did not fail during the tests.

Revised Repeated Load Triaxial Testing Protocol

Based on the observations made during Stage I testing, the repeated load triaxial test protocol was modified for Stage II

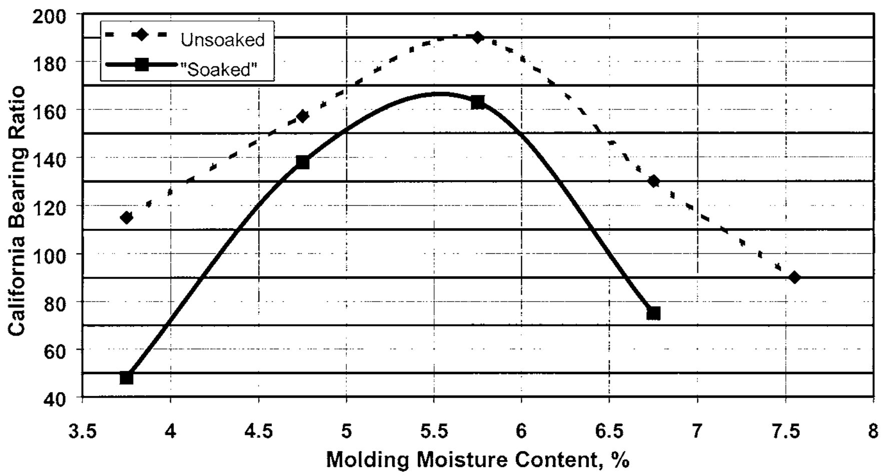

Figure 5.2. CBR for Sample I-1.

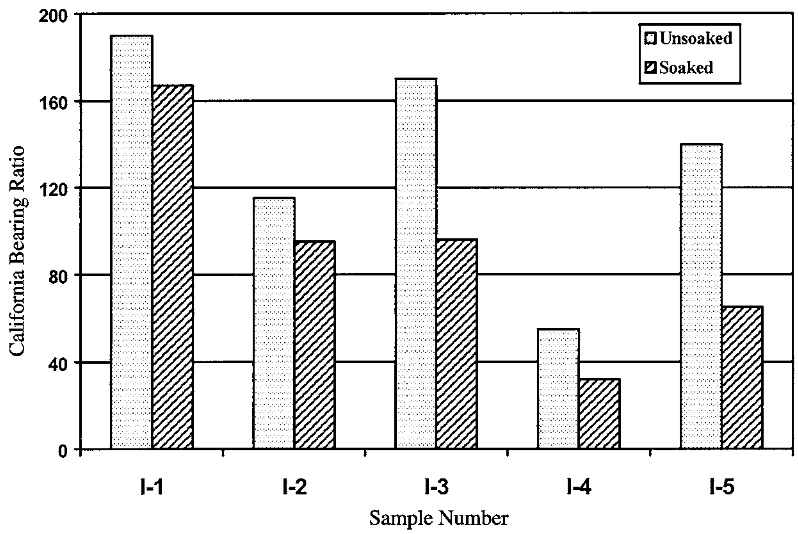

Figure 5.3. CBR results for Stage I samples.

TABLE 5.2 Failure deviator stresses for Stage I samples during static triaxial testing

Sample No.	Confining Pressure, σ_c (psi)								
	5			10			15		
	σ_d	Saturation[1], %		σ_d	Saturation[1], %		σ_d	Saturation[1], %	
		Before	After		Before	After		Before	After
I-1 (Dry)	76.7	79.4	59.7	111.3	91.2	73.1	155.5	89.7	71.5
I-2 (Wet)	38.4	37.4	36.1	82.1	30.2	27.1	109.4	36.5	22.7
I-3 (Dry)	46.4	64.0	60.0	89.1	59.8	54.7	106.8	67.7	63.1
I-3 (Wet)	48.1	75.3	76.1	57.1	50.7	96.6	71.8	44.3	81.0
I-4 (Dry)	24.0	37.2	8.8	46.8	36.3	10.1	69.8	33.5	19.5
I-5 (Wet)	33.4	63.0	94.4	56.6	64.6	80.1	85.3	58.5	72.5

1. Calculated saturation

TABLE 5.3 Values for c and ϕ for Stage I samples

Sample No.	c (psi)	ϕ°
I-1 - Dry	5.9	53.0
I-2 - Wet	0.8	51.5
I-3 - Dry	3.3	49.4
I-3 - Wet	9.5	33.0

to continue testing until failure of the specimen occurs. For the first two stress levels, the deviator stress was 10 and 20 psi and increased by 20 psi thereafter until specimen failure, defined by 10-percent strain, occurred or limit of the load frame was reached.

Flat and Elongated Particles

To evaluate the influence of flat or elongated particles on aggregate performance, the percentage of flat and/or elongated particles was determined. An excess of these particles may interfere with compaction and consolidation. Also, weak particles may break during transportation, placing, handling, and compaction, causing the gradation to change.

Aggregate materials A (crushed limestone) and B (uncrushed river gravel) used to fabricate the Stage I samples were tested in accordance with ASTM D 4791 to determine the percentage of flat or elongated, and flat and elongated particles, in the aggregate sample. The test results are presented in Table 5.4.

Uncompacted Voids

The uncompacted void content was determined in accordance with the procedure described in Chapter 3. The uncompacted voids for Material A were 52 percent and 49 percent for the fine and the coarse fractions, respectively. The uncompacted voids for Material B coarse fraction were 43 percent; there was no fine fraction.

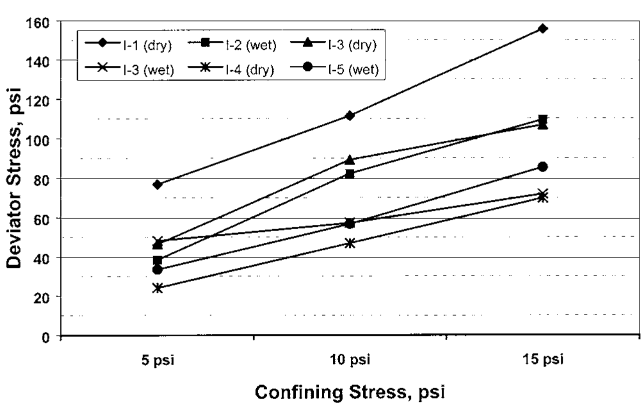

Figure 5.4. Static triaxial test results for Stage I samples.

Aggregate Durability Index

The aggregate durability index is a measure of the resistance of aggregates to deterioration when subjected to agitation in the presence of water. The test protocol followed was AASHTO T 210 (ASTM D 3744). Material A coarse fraction had an index of 86 and Material B had an index value of 98.

Magnesium Sulfate Soundness

The magnesium sulfate soundness test, conducted according to AASHTO T 104, gives an estimate of an aggregate's resistance to weathering. This test simulates the weathering action using crystallization of soluble salts in aggregate pores. The test results are represented as percent loss. For the 19 to 9.5 mm fraction, both Materials A and B had a 0.6 percent loss. For the 9.5 to 4.75 mm fraction, Material A had a 1.6 percent loss and Material B had a 1.1 percent loss.

Micro-Deval Test

The Micro-Deval test provides an indication of an aggregate's degradation potential. The test results are represented as percent loss. The test was conducted in accordance with AASHTO Provisional Standard TP58-99 (*9*). Material A had a 10.1-percent loss for the 19 to 9.5 mm fraction and 9.6-percent loss for the 9.5 to 4.75 mm fraction. Material B had a 1.0-percent loss for the 19 to 9.5 mm fraction and 1.3-percent loss for the 9.5 to 4.75 mm fraction.

Los Angeles Abrasion Test

The Los Angeles abrasion test, conducted according to AASHTO T 96, is an estimate of the degradation (the percent loss) of an aggregate. Material A had a 20.7-percent loss for the 19 to 9.5 mm fraction and a 24.4-percent loss for the 9.5 to 4.75 mm fraction. Material B had a 17.0-percent loss for the 19 to 9.5 mm fraction and a 22.0-percent loss for the 9.5 to 4.75 mm fraction.

Aggregate Impact and Crushing Value Tests

The AIV and ACV are British tests that provide a measure of an aggregate's degradation potential. A lower AIV or ACV represents a better-performing aggregate. The AIV and ACV for Material A were 22.0 and 23.1, respectively; the AIV and ACV for Material B were 17.6 and 12.1, respectively.

Gyratory Degradation

The Strategic Highway Research Program (SHRP) gyratory compactor can be used to produce degradation of aggregate under shearing force. This method was used to measure aggregate breakdown under selected revolutions of the

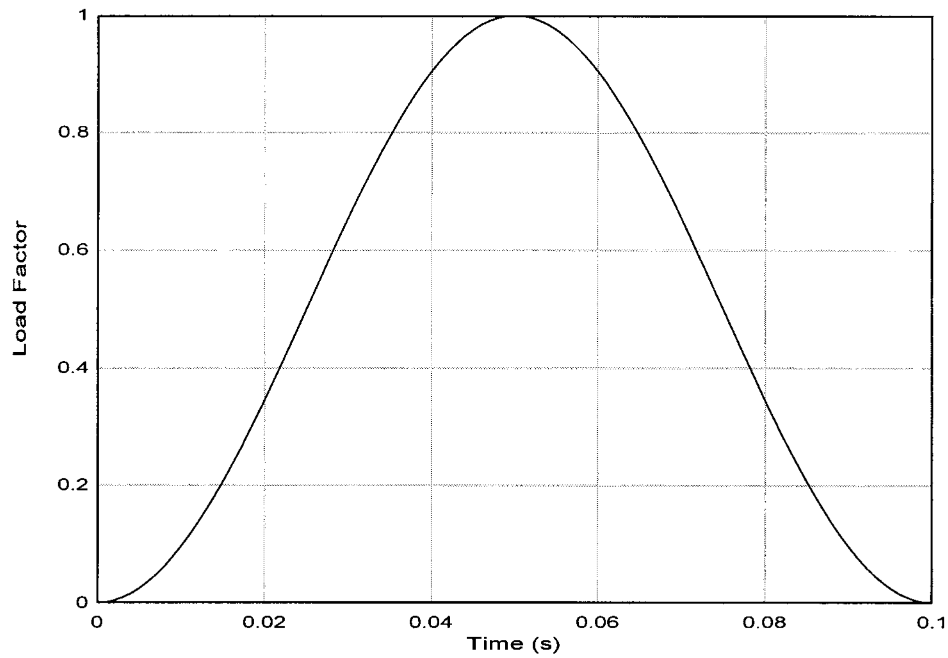

Figure 5.5. Load pulse for repeated load triaxial tests.

gyratory machine. Significant degradation was not observed for the five samples tested during the course of Stage I testing; Figure 5.10 shows test results for Sample I-1.

Frost Susceptibility

The frost susceptibility of Stage I aggregate samples was determined in terms of the Corps of Engineers "F" categories (*12*) and from the results of the Tube Suction Test (TST) (*13*). The Corps of Engineers method categorizes soils into several categories based on their degree of susceptibility, from F1 (least susceptible) to F4 (most susceptible). The "F" categories are based on general soil type and the amount of material finer than 0.02 mm. The TST measures the amount of free water that exists within an aggregate sample. The asymptotic dielectric constant value (DCV) at the end of the test can be used to characterize an aggregate as a poor (DCV > 16), marginal (10 < DCV < 16), or good (DCV < 10) performer in terms of its moisture susceptibility and frost resistance. Table 5.5 presents a summary of Corps of Engineers "F" categories and the TST test results for Stage I samples. TST test details are provided in Appendix D (not published herein).

Permeability

Permeability of Stage I samples was calculated using the equation given in Chapter 3. The permeabilities of Samples I-1 and I-2 were 0.02 and 29.12 ft/day, respectively. Samples I-3 and I-5 both had a permeability of 0.00 ft/day. Sample I-4 was free draining, and permeability could not be established.

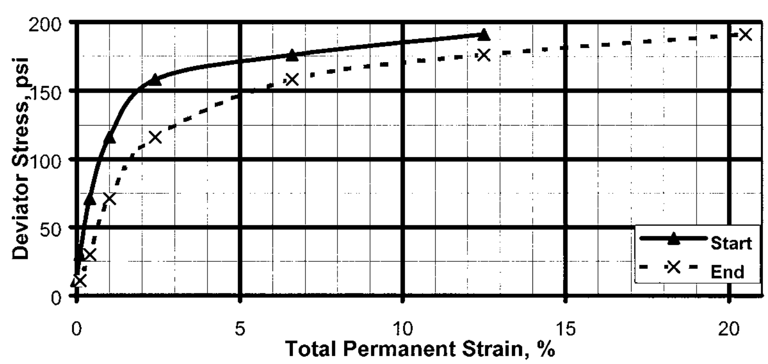

Figure 5.6. Repeated load triaxial test results for Sample I-1.

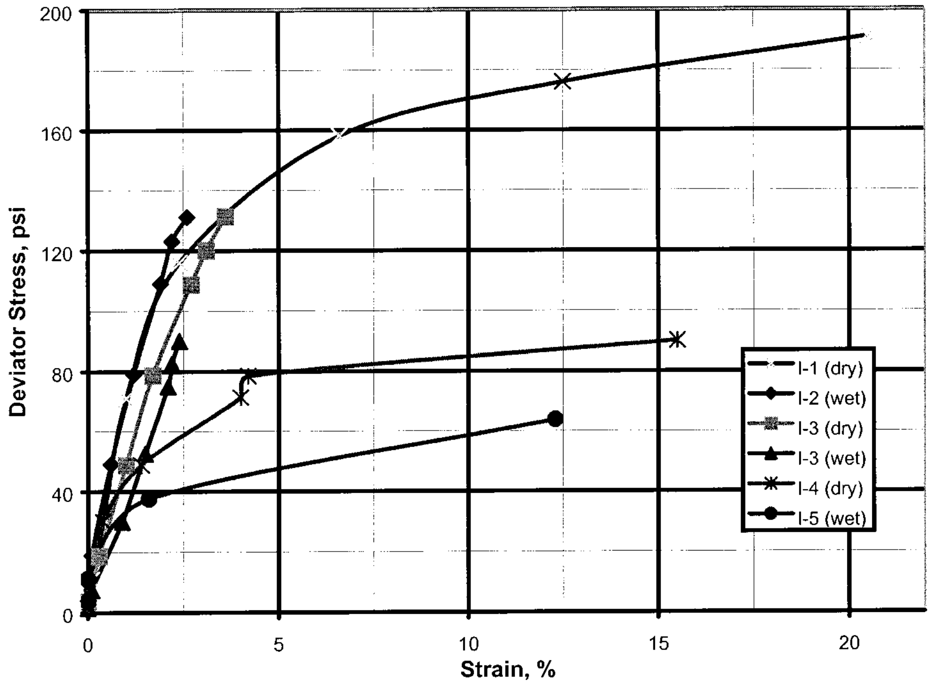

Figure 5.7. Repeated load triaxial test results for Stage I samples.

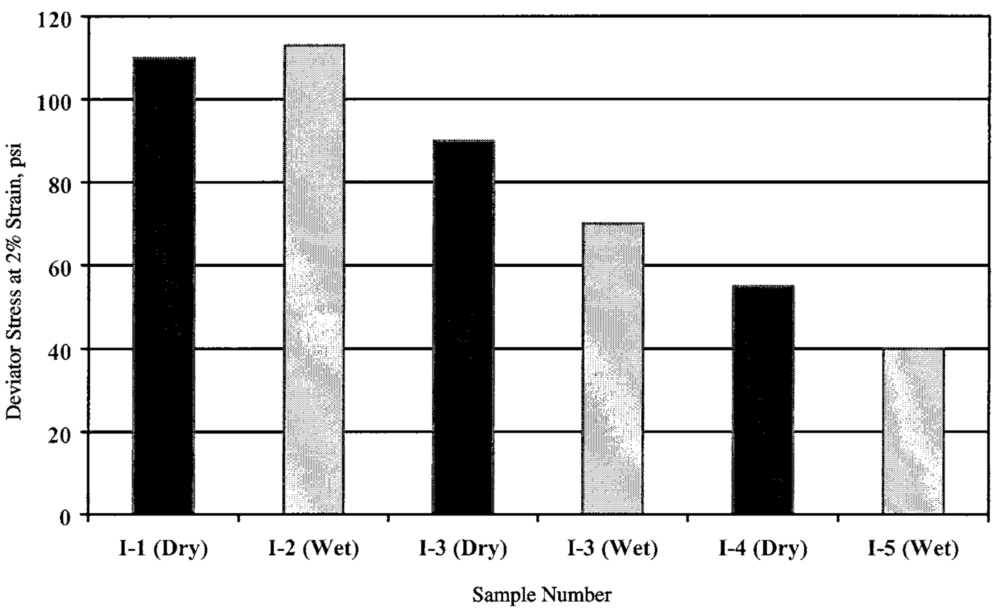

Figure 5.8. Deviator stress at 2-percent strain for Stage I samples.

STAGE II LABORATORY TEST RESULTS

Each of the Stage II samples (described in Chapter 4) was subjected to a full set of tests that included classification, shear strength, toughness, durability, frost susceptibility, and permeability tests. Replicate tests were conducted in Stage II to evaluate the repeatability of some test results. Replicates were not conducted on those tests with established precision and accuracy. Laboratory test results for Stage II samples are provided in Appendix C (not published herein).

Static Triaxial Test Results

The static triaxial test was conducted in accordance with ASTM D 2850 on all samples in the same manner as in

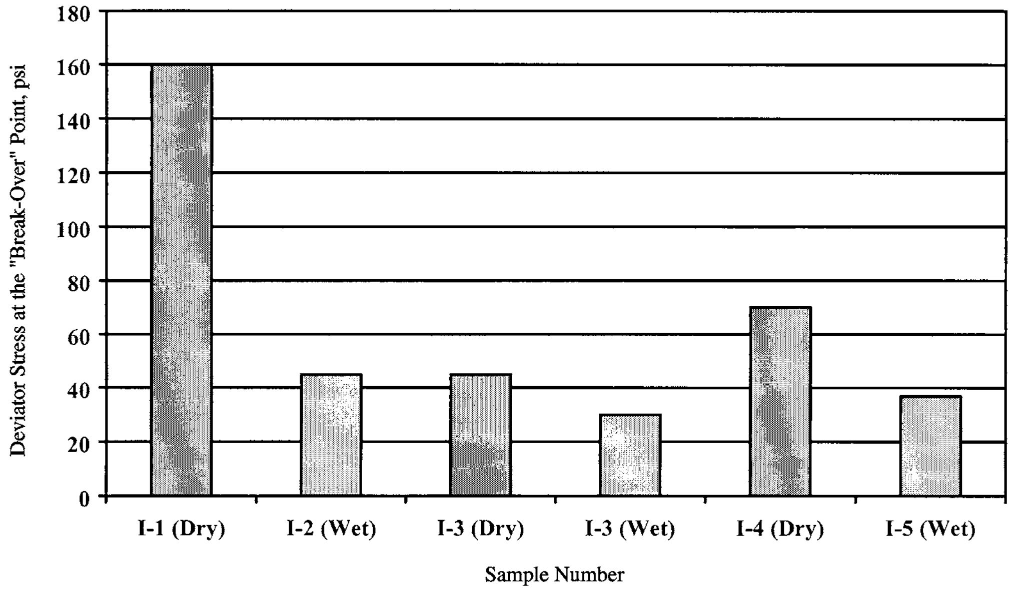

Figure 5.9. Deviator stress at the "break-over" point for Stage I samples.

Stage I tests. Table 5.6 shows the failure stresses for the three confining stresses, along with the cohesion (c) and angle of internal friction (ϕ).

Figure 5.11 presents the samples in the order of decreasing deviator stress at 15 psi confining stress and shows the deviator stress at failure for each confining stress. Note that the same order does not hold true for the 5 and 10 psi confining stresses. The failure stress increased with increased confining stress for all samples, but the percent increase was not the same for all the samples. For example, Sample II-6 showed an increase of almost 41 percent in deviator stress when tested at a confining stress of 10 psi instead of 5 psi, and an increase of 68 percent when tested at a confining stress of 15 psi instead of 10 psi. For Sample II-7, the increase was almost 75 percent when confining stress was increased from 5 to 10 psi. However, only about 4 percent increase was measured when the confining stress was increased from 10 to 15 psi.

Repeated Load Triaxial Test Results

The purpose of conducting repeated load triaxial shear tests on Stage II samples was to define the range of test parameters that could predict the potential for permanent deformation. The test procedure was revised based on the results obtained from the Stage I test program. The test procedure was modified to continue testing until the specimen fails. Specimen failure was defined as reaching a strain of 10 percent. The deviator stress for the first two load increments was 10 and 20 psi and increased by 20 psi thereafter until specimen failure occurred or until limit of the load frame was reached.

Tests were conducted at the optimum moisture content and a wet condition defined by a period of saturation and drainage, referred to as "dry" and "wet" tests, respectively. Figure 5.12 shows typical results for a repeated load triaxial test. The strain values at the first and the last load increment for the dry

TABLE 5.4 Flat and elongated particles measured on Stage I component aggregates

Size/Gradation			Limestone		Gravel	
Specified Ratio	*Size*		*F or E (%)*	*F & E (%)*	*F or E (%)*	*F & E (%)*
	Passing	*Retained*				
3:1 (mass)	-3/4"	+1/2"	0.0	5.0	2.2	10.7
	-1/2"	+3/8"	2.4	7.5	2.1	7.3
3:1 (count)	-3/4"	+1/2"	0.0	4.6	3.0	11.7
	-1/2"	+3/8"	3.2	9.2	3.2	8.7
5:1 (mass)	-3/4"	+1/2"	0.0	0.0	0.0	0.0
	-1/2"	+3/8"	0.0	0.0	0.0	0.5
5:1 (count)	-3/4"	+1/2"	0.0	0.0	0.0	0.0
	-1/2"	+3/8"	0.0	0.0	0.0	0.6

F or E: Aggregate particles having a ratio of width to thickness or length to width greater than the specified ratio
F & E: Aggregate particles having a ratio of length to thickness and length to width greater than the specified ratio

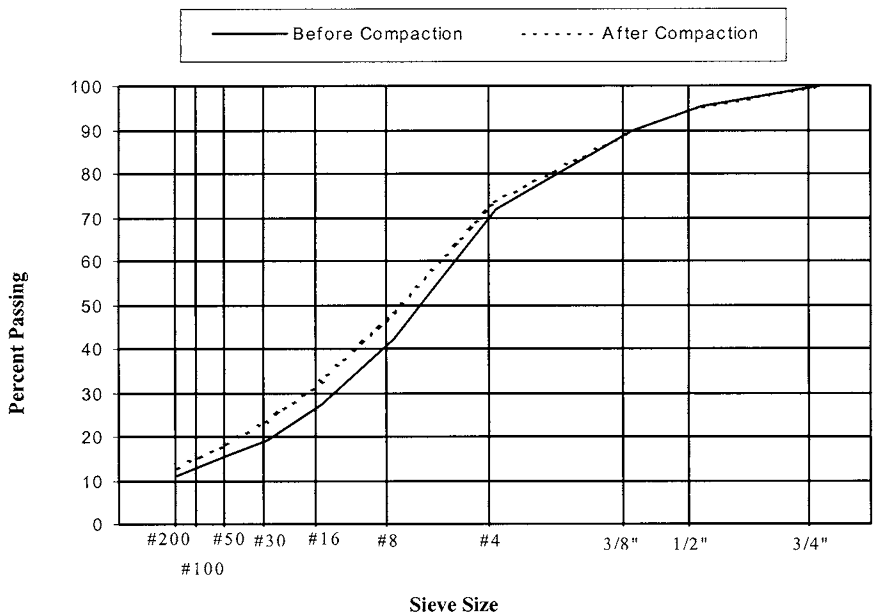

Figure 5.10. Results of gyratory degradation test for Sample I-1.

test are labeled "First-D" and "Last-D," respectively. The strain values at the first and the last load increment for the wet test are labeled "First-W" and "Last-W," respectively.

Reference Deviator Stress

Three strength levels were proposed as indicators of sample strength:

- Deviator stress at initial shear failure of the sample—defined by the deviator stress at which an increased rate of permanent deformation occurs; determining the point for initial shear failure is a subjective criterion.
- Deviator stress at 2-percent strain that was found to agree sufficiently well with the initial shear failure but eliminates the subjectivity associated with the initial shear criterion.
- Deviator stress at termination of the test—defined by reaching 10-percent strain or reaching the capacity of the test equipment.

TABLE 5.5 F categories and DCV from Tube Suction Tests for Stage I samples

Sample No.	F Category	Tube Suction Test	
		DCV	Performance Classification
I-1	F-1	9	Good
I-2	F-1	3	Good
I-3	F-2	6	Good
I-4	F-1	2	Good
I-5	F-3	4	Good

Figure 5.12 shows data required for determining the three reference strengths. For the stress–strain curve labeled "Last-D," the rapid increase in the rate of permanent deformation beyond 80 psi suggests it to be point of initial shear. The deviator stress at 2-percent strain is estimated at about 100 psi, and deviator stress at termination of the test was 160 psi.

Test Results

Table 5.7 presents reference deviator stress values for all Stage II samples at each moisture condition for the beginning cycles and the end cycles of load. These values ranged from 10 to 192 psi (the capacity of the test equipment did not permit testing higher than 192 psi). Figures 5.13 and 5.14 show plots of these values for the dry and wet tests, respectively.

Test results showed little difference between the shear strengths of saturated and dry samples for many of the samples. However, test results indicated a significant difference between the dry and soaked CBR. Table 5.8 shows the difference in the optimum moisture content and moisture content measured after triaxial testing. The slight difference in moisture contents of wet and dry samples is attributed to the 1-hr drainage period of wet samples before testing.

Because of the limited difference in moisture contents, the effects of moisture on the strength of the materials could not be evaluated. For some samples, the moisture added prior to testing could have aided in compaction during the test. The test materials were, for the most part, good draining materials and the hour allowed for drainage after saturation permitted the moisture content to approach optimum.

TABLE 5.6 Static triaxial test results for Stage II samples

Sample	Dry Tests: σ_c (psi) 5	10	15	c psi	ϕ^o	Wet Tests: σ_c (psi) 5	10	15	c psi	ϕ^o
	Deviator Stress at Failure					Deviator Stress at Failure				
II-6	45.20	63.80	107.3	1.40	49.80	73.20	88.10	108.80	12.70	39.90
II-7	63.90	111.60	116.10	6.80	48.80	51.90	63.50	85.30	7.80	39.00
II-8	96.50	112.00	154.40	11.20	48.90	94.10	130.00	176.80	8.30	53.70
II-9	70.20	78.20	105.70	12.20	40.90	67.00	86.00	131.00	5.00	50.40
II-10	26.40	76.60	117.60	0.00	55.10	36.70	76.70	93.00	1.90	48.20
II-11	47.90	98.10	159.00	0.00	58.00	108.00	128.30	171.00	12.90	49.90
II-12	98.50	156.80	176.30	10.00	53.70	88.40	151.90	177.50	7.10	55.50
II-13	29.90	66.00	78.00	1.40	46.40	30.20	70.40	77.20	1.40	46.40
II-14	84.50	98.20	152.60	6.40	52.00	92.30	143.90	152.20	11.10	50.50
II-15	111.80	133.10	185.10	181.4	56.40	100.60	177.20	185.10	12.10	51.60
II-16	102.60	138.40	169.00	12.70	50.20	102.60	138.40	169.00	0.00	59.30
II-17	47.10	87.20	103.60	3.90	48.30	41.00	55.30	58.90	9.70	29.10

Resilient Modulus Test Results

The repeated load triaxial test was selected to determine shear strength. Test data were also used to calculate the resilient modulus (M_R). The $M_R$values were reported for each 100 repetitions as the average of the last 5 repetitions. Thus, 10 values of M_R were reported for each load increment. The final values of M_R were used to develop a plot of M_R versus deviator stress; a typical plot is shown in Figure 5.15.

The M_R versus deviator stress plot can be divided into four phases. In the first phase, M_R decreases with increased load due to the breakdown of the initial sample structure, the seating of the sample in the test setup, or measurement errors at low load levels. In the second phase, a gradual increase in the M_R occurs probably due to densification of the sample. For good quality materials, this phase continues until the onset of failure (as defined by a rapid increase in plastic deformation). At the end of phase 2, the sample reaches the maximum density. The end of the second phase and start of the third phase is marked by a peak of M_R. During phase 3, the M_R continues to decline, probably due to dilation of the sample caused by plastic shear. The start of phase 3 coincides with the onset of failure and a decrease in sample density. As the plastic failure continues, the sample begins to bulge, leading to errors calculating the M_R value, which produces

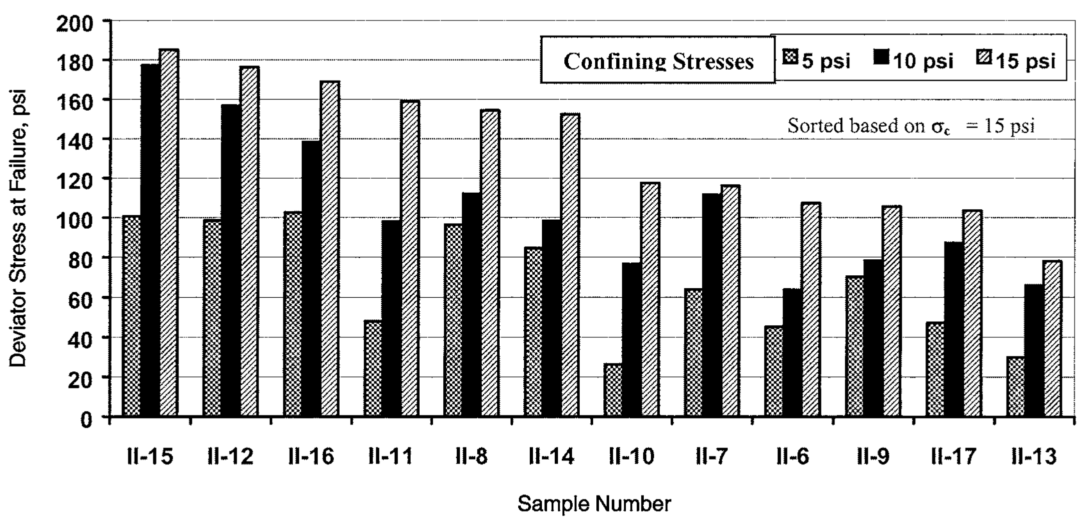

Figure 5.11. Deviator stress for Stage II samples from static triaxial test results (tested dry).

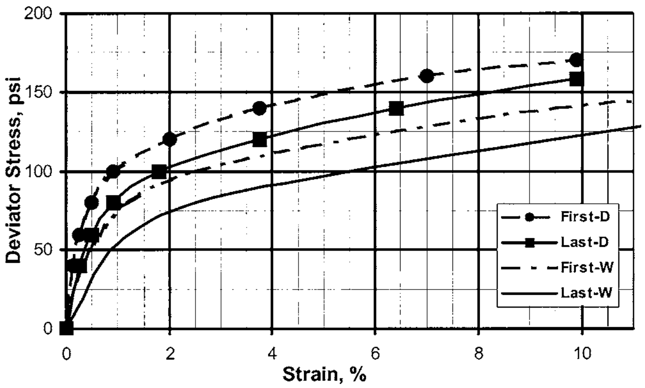

Figure 5.12. Repeated load triaxial test results for Sample II-6.

an apparent increase in M_R value in phase 4 of the test. There is a linear relationship between M_R and deviator stress during phase 2, and the end of phase 2 corresponds to the onset of failure defined by a rapid increase in permanent deformation of the sample.

Most samples exhibited a behavior similar to that shown in Figure 5.15. However, in some cases, phase 1 was not apparent and, in others, phase 3 could not be defined. In the latter cases, a peak value for the M_R was not defined, and an M_R value was estimated from the stress–strain curve.

The values for the peak M_R ranged from 28,000 to 105,000 psi with a mean value of 51,100 psi and standard deviation of 16,060 psi. There was no noticeable difference between the M_R for the dry and wet samples.

TABLE 5.7 Repeated load triaxial test results for Stage II samples

Sample No.	Quality*	Moisture	Deviator Stress, psi							
			2% strain		Termination			Rapid strain increase (1)		
			First Load Increment	Last Load Increment	First Load Increment	Last Load Increment	Termination Strain, %	First Load Increment	Last Load Increment	Strain Value, %
II-6	P	Dry	120.0	102.0	170.0	160.0	9.95	100.0	80.0	1.00
II-6	P	Wet	95.0	75.0	192.0	175.0	10.00	75.0	55.0	1.00
II-7	G	Dry	140.0	120.0	157.5	137.5	3.90	130.0	117.5	1.50
II-7	G	Wet	120.0	100.0	175.0	158.0	7.45	105.0	85.0	1.50
II-8 (2)	G	Dry	155.0	140.0	187.0	187.0	8.00	135.0	115.0	1.00
II-8 (2)	G	Wet	165.0	147.5	187.0	187.0	4.75	160.0	130.0	1.75
II-9	P	Dry	105.0	85.0	152.0	132.5	10.00	90.0	70.0	1.50
II-9	P	Wet	130.0	112.0	157.5	90.0	6.25	100.0	80.0	1.00
II-10	G	Dry	130.0	111.0	140.0	130.0	2.75	112.5	95.0	0.75
II-10	G	Wet	140.0	120.0	140.0	120.0	2.00	70.0	65.0	0.50
II-11	P	Dry	145.0	124.0	180.0	160.0	9.25	120.0	100.0	0.75
II-11	P	Wet	155.0	135.0	185.0	185.0	9.00	140.0	120.0	1.50
II-12	G	Dry	190.0	180.0	190.0	190.0	2.50	190.0	180.0	2.00
II-12	G	Wet	160.0	140.0	190.0	190.0	6.25	160.0	140.0	2.00
II-13	P	Dry	127.5	97.5	157.5	137.5	7.65	100.0	80.0	1.25
II-13	P	Wet	130.0	120.0	157.5	137.5	5.00	117.5	100.0	0.75
II-14	G	Dry	180.0	160.0	185.0	185.0	3.00	185.0	180.0	2.25
II-14 (2)	G	Wet	182.0	178.0	182.0	182.0	2.40	180.0	178.0	2.00
II-15	P	Dry	162.0	145.0	187.0	187.0	3.75	180.0	178.0	2.00
II-15 (2)	P	Wet	180.0	162.0	187.0	187.0	2.90	185.0	185.0	0.75
II-16	G	Dry	187.0	187.0	190.0	180.0	1.75	175.0	180.0	1.75
II-16	G	Wet	185.0	170.0	190.0	190.0	3.75	175.0	160.0	1.75
II-17	P	Dry	74.0	54.0	90.0	80.0	8.50	57.0	40.0	0.75
II-17	P	Wet	60.0	10.0	100.0	80.0	8.50	30.0	10.5	0.25

(1) Point at the stress-strain curve at which the curve adopts a more horizontal asymptote
(2) Load frame limit was reached before a strain of 10%
* P and G designate poor and good performance rating, respectively

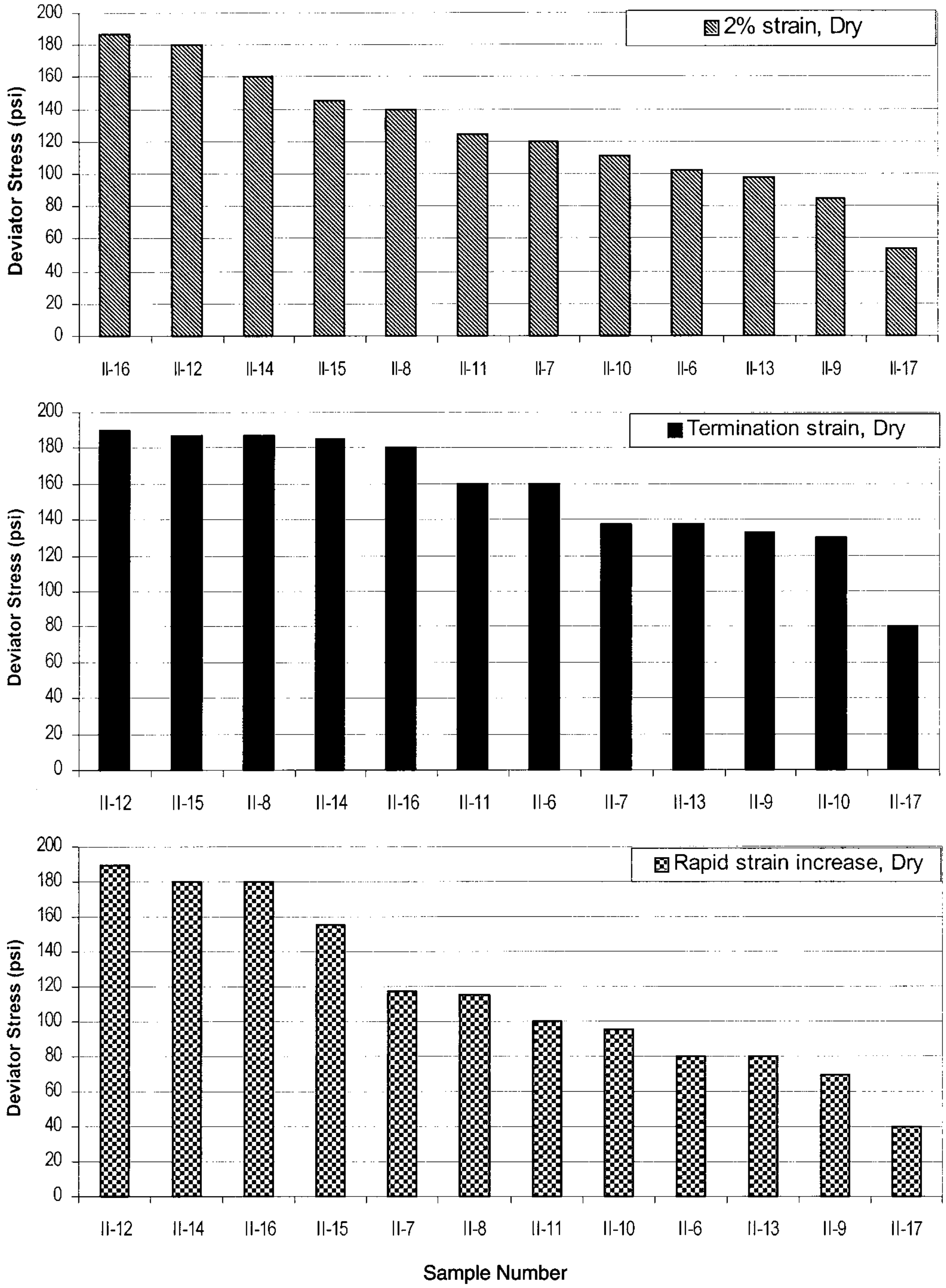

Figure 5.13. Order of Stage II samples based on repeated load triaxial test results (tested dry).

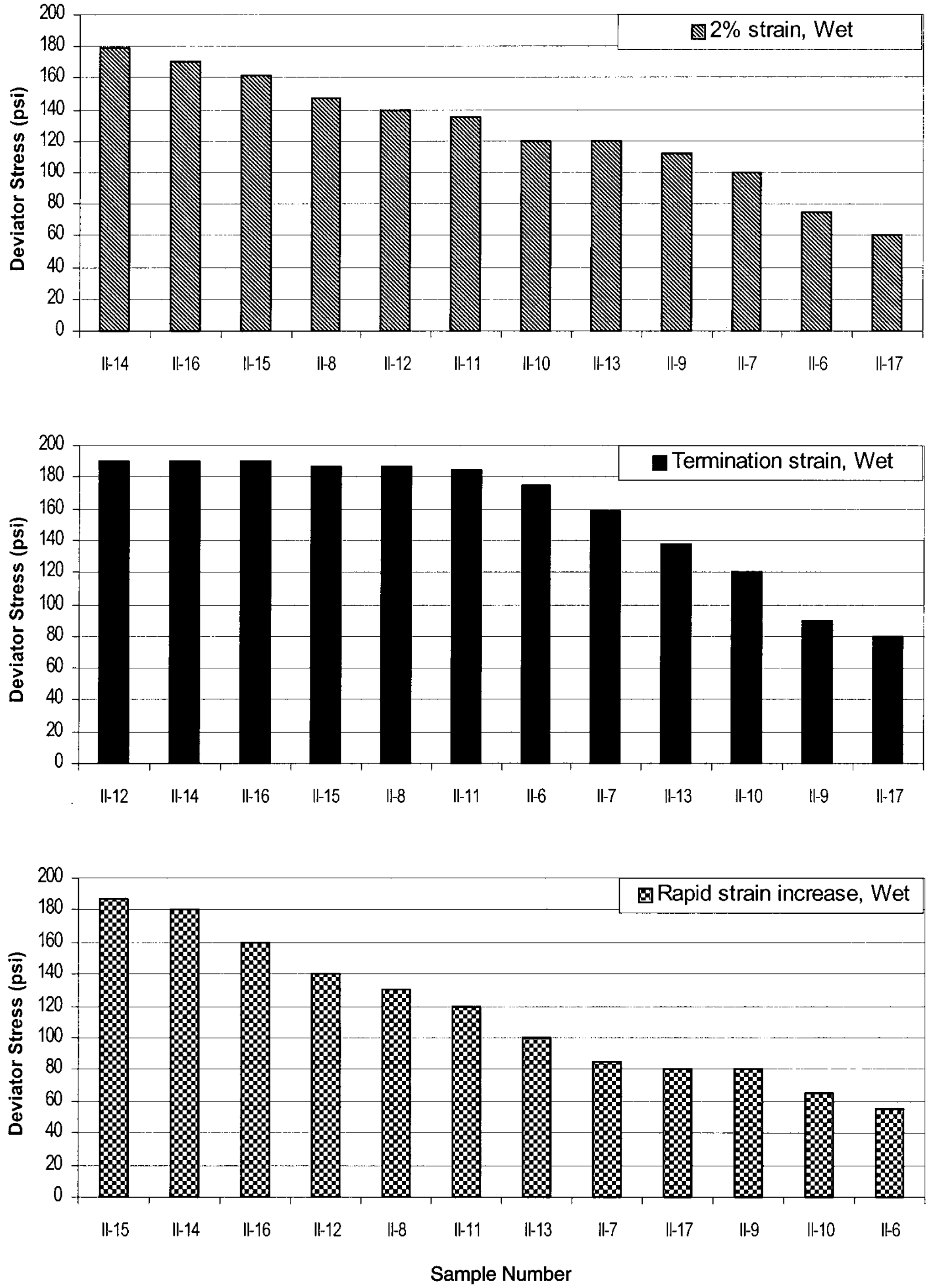

Figure 5.14. Order of Stage II samples based on repeated load triaxial test results (tested wet).

TABLE 5.8 Molded and after triaxial testing moisture contents for Stage II samples

Sample No.	Moisture	OMC [a]	Test Replicate 1		Test Replicate 2		Difference[b]
			As Molded	After Test	As Molded	After Test	
II-6	Dry	6.5	6.4	6.0	6.7	5.9	-0.60
II-6	Wet	6.5	6.1	5.6	6.3	6.7	-0.05
II-7	Dry	8.5	9.4	7.2	8.4	3.8	-3.40
II-7	Wet	8.5	8.6	5.1	8.3	3.2	-4.30
II-8	Dry	5.5	6.1	5.4	5.5	4.4	-0.90
II-8	Wet	5.5	6.3	5.8	6.0	5.2	-0.65
II-9	Dry	5.5	6.0	5.7	6.0	5.5	-0.40
II-9	Wet	5.5	5.5	6.1	6.3	6.3	0.30
II-10	Dry	6.5	5.5	5.3	5.7	5.5	-0.20
II-10	Wet	6.5	5.0	5.8	5.4	5.2	0.30
II-11	Dry	6.0	4.9	4.5	4.9	4.6	-0.35
II-11	Wet	6.0	5.2	5.1	5.1	4.6	-0.30
II-12	Dry	7.5	7.3	5.0	7.0	5.1	-2.10
II-12	Wet	7.5	6.7	5.1	7.2	5.5	-1.65
II-13	Dry	8.5	9.5	8.9	9.5	9.2	-0.45
II-13	Wet	8.5	8.6	8.9	8.9	9.6	0.50
II-14	Dry	9.5	9.9	4.6	9.1	5.1	-4.65
II-14	Wet	9.5	8.6	4.0	8.6	5.3	-3.95
II-15	Dry	11.0	12.0	8.1	12.1	8.0	-4.00
II-15	Wet	11.0	11.2	7.6	12.1	7.7	-4.00
II-16	Dry	7.5	6.6	4.5	6.6	5.4	-1.65
II-16	Wet	7.5	6.9	6.2	6.7	5.3	-1.05
II-17	Dry	6.0	7.3	7.0	7.2	6.9	-0.30
II-17	Wet	6.0	6.6	5.8	5.6	6.6	0.10

a: optimum moisture content
b: difference between average molded and tested moisture contents of replicates, − and + designate lower and higher moisture contents after the tests than before the test, respectively

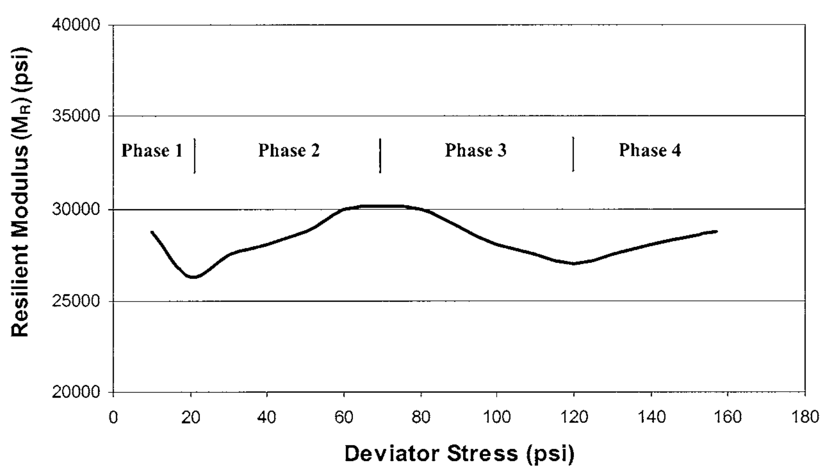

Figure 5.15. Typical plot of M_R versus deviator stress.

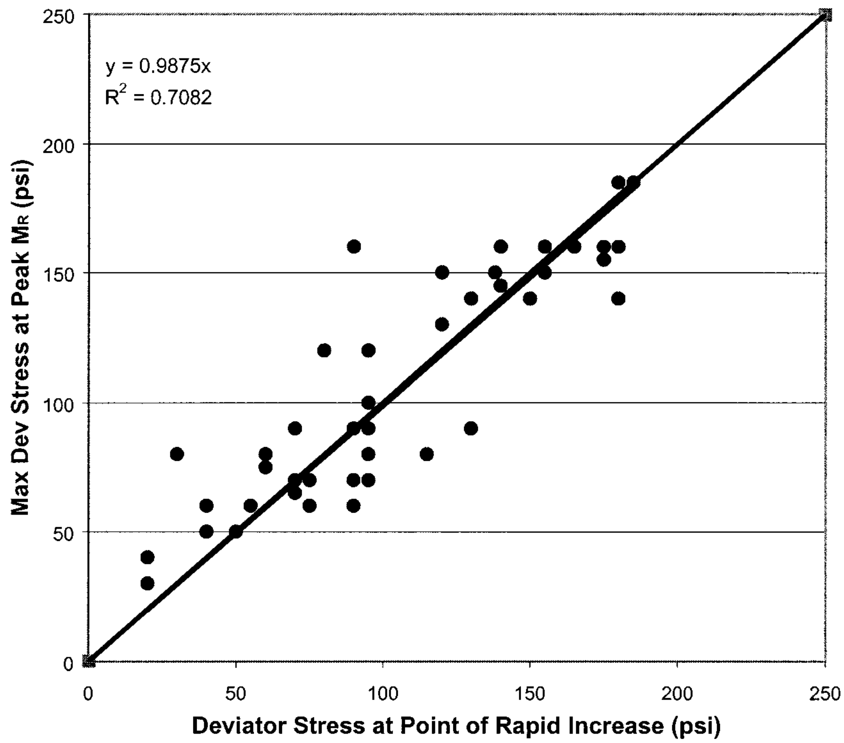

Figure 5.16. Deviator stress at peak M_R versus initial failure deviator stress.

Relationship Between Peak Resilient Modulus and Shear Strength

As discussed previously, sample strength can be defined by the stress at 2-percent strain or the stress at the onset of rapid permanent deformation (rapid strain increase). The stress at peak $M_R$appears to be related to the point of rapid strain increase. Figure 5.16 is a plot of the deviator stress at the peak $M_R$versus the deviator stress at rapid strain increase. Evaluation of Stage II results indicates that these stresses represent the beginning of failure of the sample. Figure 5.17 relates the peak $M_R$to the deviator stress at failure, and Figure 5.18 relates the peak M_R to the deviator stress at which the peak M_R was selected. The linear regression results of both plots indicate similar correlations of the stress values to the peak M_R.

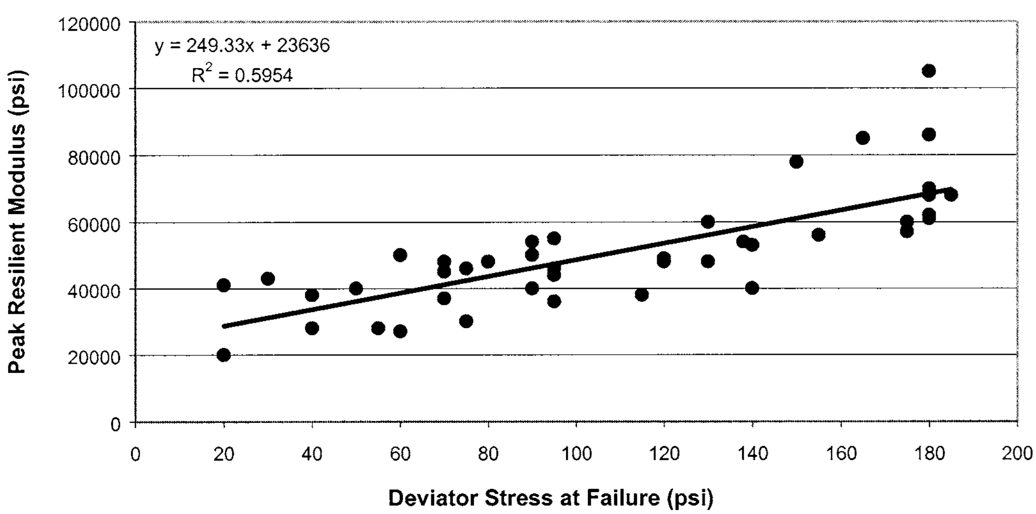

Figure 5.17. Relationship between peak resilient modulus and deviator stress at initial failure.

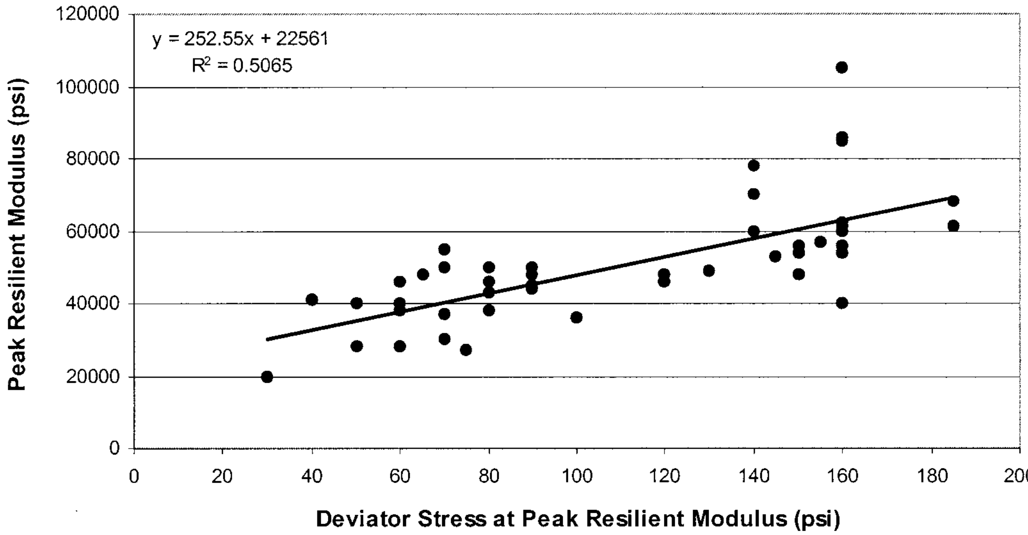

Figure 5.18. Peak resilient modulus versus deviator stress at peak M_R.

Order of Stage II Samples

The peak M_R was used to place Stage II samples in the order of decreasing value, as shown in Figures 5.19 and 5.20, for dry and wet tests, respectively.

Resilient Modulus at a Strain of 2 Percent

Stress at 2-percent strain was used as the reference for judging shear strength. Figure 5.21 shows an excellent agreement between the peak M_R and the M_R at 2-percent strain.

For Stage II samples, the M_R values at 2-percent permanent strain were nearly identical to the peak M_R values (Figure 5.21). For two samples, the data indicate that the M_R at 2-percent permanent strain was greater than the M_R at peak strain, because failure had started earlier than 2-percent strain.

STAGE III LABORATORY TEST RESULTS

Stage III laboratory tests were conducted on aggregate samples from MnRoad and Ohio Test Road as described in Chapter 4. Because the primary concern of Stage III tests was shear strength assessment, Stage III samples were subjected to all tests except particle flatness and elongation, index of particle shape and texture, and uncompacted voids tests.

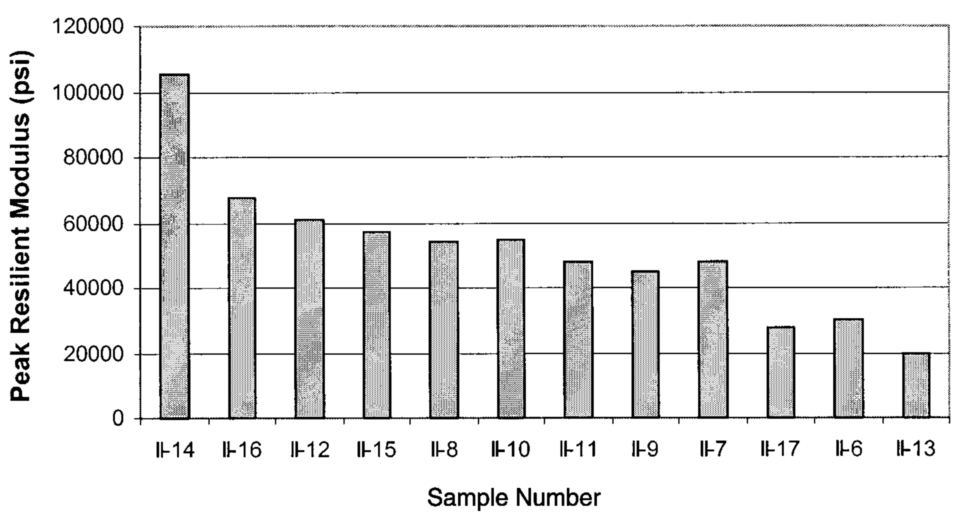

Figure 5.19. Peak resilient modulus for Stage II samples when tested dry.

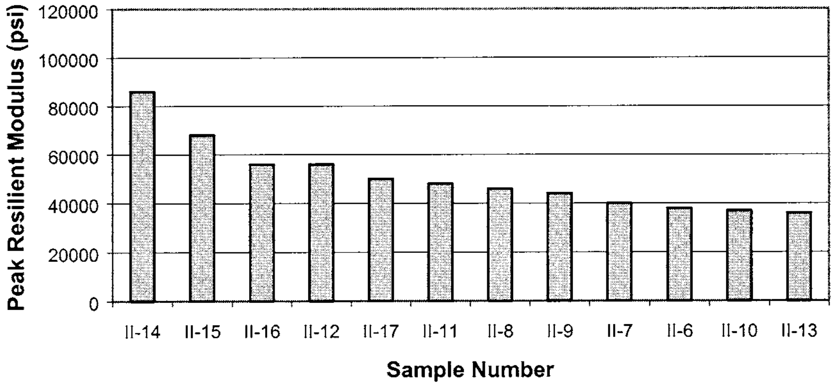

Figure 5.20. Peak M_R for Stage II samples when tested wet.

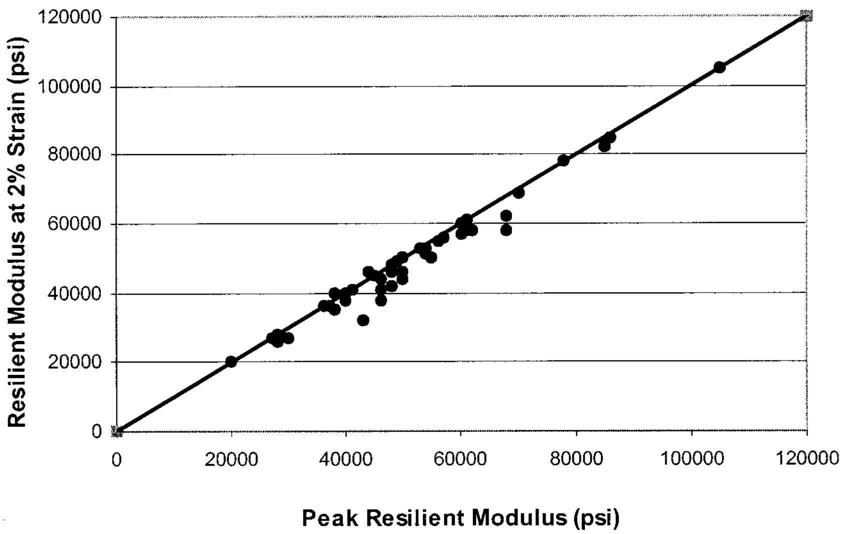

Figure 5.21. Relationship between peak resilient modulus and modulus at 2-percent strain.

Results of grain size distribution tests are shown in Figure 5.22. Laboratory compaction was conducted in accordance with AASHTO T 180; results are shown in Table 5.9. Permeabilities of Stage III samples, calculated using the equation discussed in Chapter 3, were 0.085, 0.007, and 0.002 ft/day for Samples III-18, III-19, and III-20, respectively.

Triaxial Test Results

Both the static and repeated load triaxial tests (as used in Stage II) were conducted on Stage III samples under a confining stress of 15 psi. The static triaxial deviator stresses at failure were 189, 190, and 116 psi for samples III-18, III-19, and III-20, respectively.

Figures 5.23, 5.24, and 5.25 show the repeated load triaxial test results for Samples III-18, III-19, and III-20, respectively. The deviator stress values at 2-percent strain, at failure, and at the point of rapid strain increase are shown in Figures 5.26 and 5.27 for dry and wet tests, respectively. For each sample, the moisture content was measured after sample molding and after triaxial testing; results are shown in Table 5.10.

Sample III-18, which consisted of all crushed particles and nonplastic fines, exhibited high shear strength in both the static and the repeated load triaxial tests. When tested dry, a strain of 2 percent was not reached; the test was discontinued because the load frame limit was reached. As shown in Figure 5.23, there was no point of initial failure on the stress–strain curve. When tested under wet conditions, Sample III-18 exhibited a relatively rapid increase in strain; the test was discontinued because the load frame limit was reached. For Sample III-19, the water drained out freely, but the sample had a relatively lower deviator stress at failure when tested wet as compared to the dry test. Sample III-19 did not reach a strain of 2 percent under dry conditions before the load frame limit was reached. In contrast to Samples III-18 and III-19, Sample III-20 performed slightly better when tested wet compared to when tested dry. This sample exhibited deformations during the test, as shown in Figure 5.25.

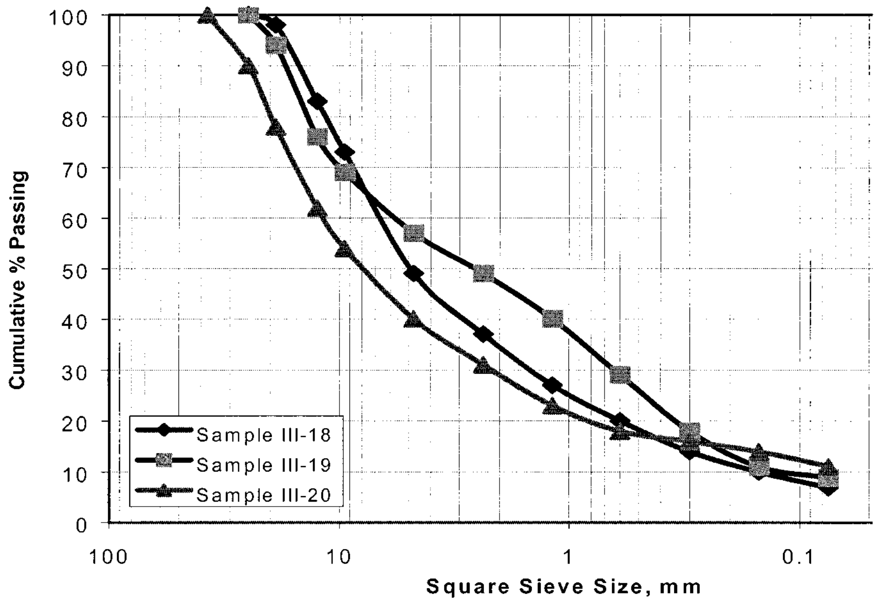

Figure 5.22. Grain size distribution for Stage III samples.

TABLE 5.9 Optimum moisture/density for Stage III samples

Sample	Optimum Moisture Content (%)	Maximum Dry Density (pcf)
III-18	7.50	142.0
III-19	6.50	134.5
III-20	9.00	135.5

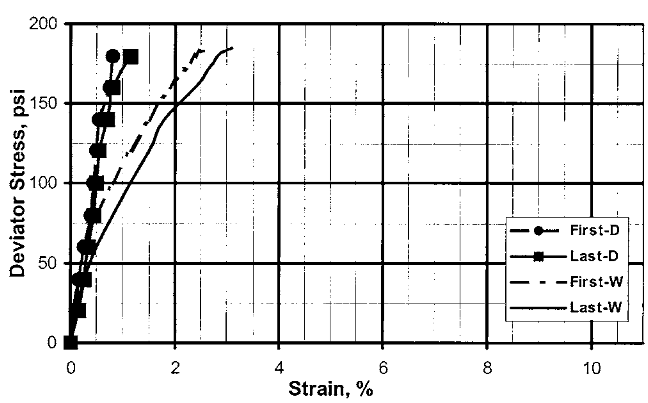

Figure 5.23. Repeated load triaxial test results for Sample III-18.

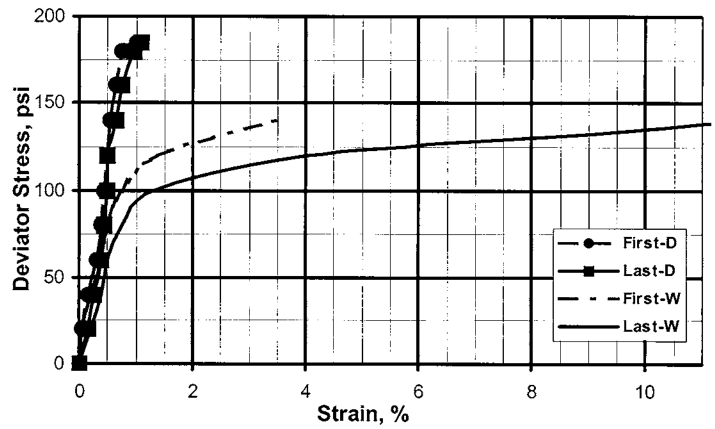

Figure 5.24. Repeated load triaxial test results for Sample III-19.

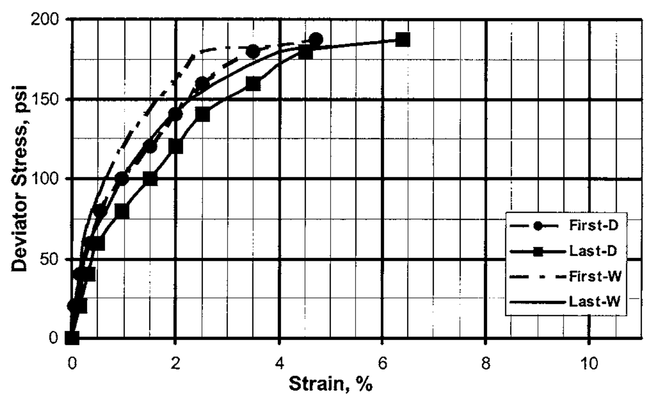

Figure 5.25. Repeated load triaxial test results for Sample III-20.

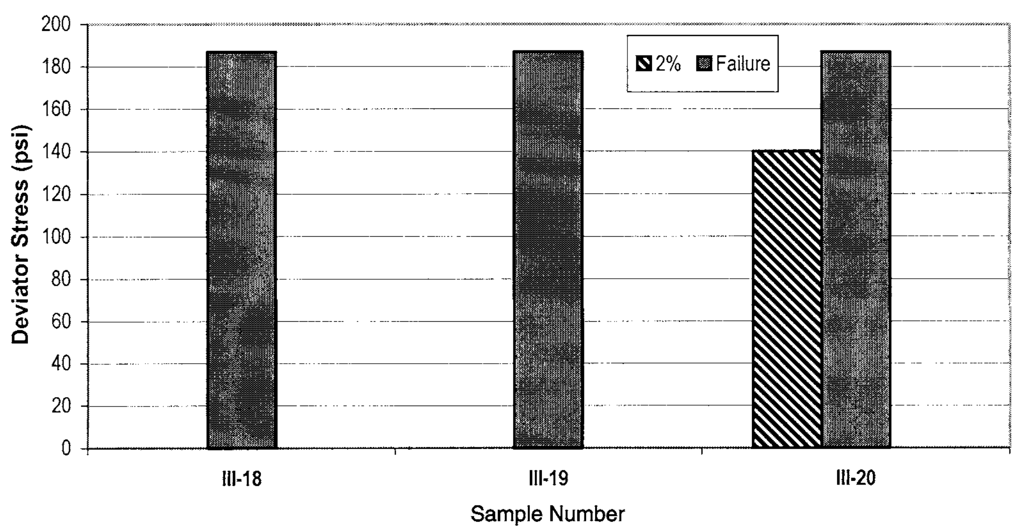

Figure 5.26. Repeated load triaxial test results for Stage III samples (tested dry).

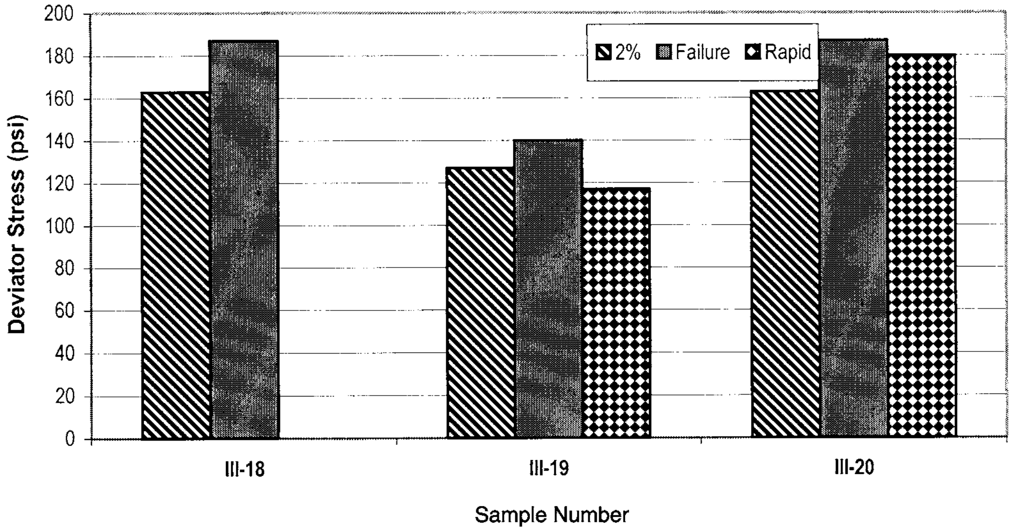

Figure 5.27. Repeated load triaxial test results for Stage III samples (tested wet).

TABLE 5.10 As molded and after triaxial testing moisture contents for Stage III samples

Sample No.	Test Condition	Moisture Content		
		As Molded	After Test	Difference
III-18	Dry	7.1	5.1	-2.0
	Wet	6.6	4.5	-2.1
III-19	Dry	5.9	5.6	-0.3
	Wet	6.0	7.3	1.3
III-20	Dry	8.4	6.9	-1.5
	Wet	7.9	6.5	-1.4

CHAPTER **6**

ANALYSIS OF TEST RESULTS

STAGE I DATA

The main purpose of Stage I tests was to establish an initial range of shear strength and to refine the test protocols. Three different aggregate materials were combined to fabricate five aggregate samples. These samples and their characteristics are listed in Table 6.1 in order of descending shear strength as perceived by the research team.

Performance Prediction

Because the performance potential of an unbound pavement layer depends on its dry and wet shear strengths, durability and toughness, and resistance to freeze/thaw damage, these properties were considered in the laboratory test program. Stage I data, shown in Table 6.2, were analyzed to determine if regression models could be developed to predict the performance potential of an unbound pavement layer.

Although the primary intent of Stage I tests was to establish the shear strength parameter ranges, data were also collected for a number of other tests. Some of the tests, such as those for durability and toughness, were made only on the limestone and gravel used to fabricate Stage I samples (i.e., not on Samples I-1 through I-5). These tests did not provide adequate data for statistical analysis.

Correlation Analysis

Correlation analysis of Stage I data was conducted; the results are shown in Table 6.3. The definitions of variables in the correlation matrix are:

- CBR: Result of the soaked CBR test,
- RO: Rating assigned to each fabricated aggregate sample,
- D5: Deviator stress (σ_d) for static triaxial test at $\sigma_c = 5$ psi,
- D10: Deviator stress (σ_d) for static triaxial test at $\sigma_c =$ 10 psi,
- D15: Deviator stress (σ_d) for static triaxial test at $\sigma_c =$ 15 psi,
- RLTT: Repeated load triaxial test deviator stress (σ_d) at $\sigma_c = 15$ psi at the end of test, and
- DCV: Dielectric constant value from the Tube Suction Test.

The top values in each cell of Table 6.3 are the correlation coefficients, R, and the bottom values are the significance levels, P. High R-values represent a stronger correlation between the two variables; a lower P value indicates a greater significance. The deviator stress levels from the static triaxial tests are highly correlated with each other. D10 was selected to be included in the model because it showed the best correlation with the dependent variable. CBR correlates well with D10 and DCV; however, its correlation with DCV has a relatively low significance. The results of the correlation analysis indicate that most of the variability in material rating can be explained by D10 and then by the remaining variables.

Regression Models to Predict Performance

Four different multiple regression equations were developed to relate the test data shown in Table 6.2 to the rating of Stage I samples; the equations are shown in Table 6.4.

All models have a high coefficient of regression values, R^2. Model 1, which uses RLTT and D10 as the independent variables, has an R^2 value of 0.965 and the lowest significance level of 0.018. The other three models are not statistically significant at the 5-percent level ($P \leq 0.05$).

The results of multivariable regression analysis indicate that shear strength is the single variable most strongly related to material rating; as the shear strength increases, the rating increases.

STAGE II DATA

Aggregate Performance Rating by State DOTs

The state DOTs that supplied the Stage II samples provided performance ratings of the material when used in base/subbase layers on a scale of 1 to 5, with 1 being poor and 5 being excellent. The rating criteria provided to the state DOTs are shown in Table 6.5. Table 6.6 shows the results of the performance survey of Stage II aggregate samples.

Laboratory Data

Stage II tests were conducted on aggregate samples, poor and good performing materials, from seven states across the

TABLE 6.1 Expected performance of Stage I aggregate samples

Sample No.	Material Type	Test	Characteristics	Rating [a]
I-1	Crushed stone 30% +4.75mm, 70% -4.75mm	dry [b]	Strongest of all samples; exceptional performance under dry conditions; tough, durable and frost resistant	5.0
I-2	Crushed stone 77% +4.75mm, 23% -4.75mm	wet [c]	Fabricated to demonstrate the effect of gradation and moisture on Sample I-1, drains well and little strength reduction when wet.	4.0
I-3	Combinations Gravel 26% +4.75mm limestone 50%, -4.75mm clay, 24%	dry	Middle of the shear strength range; performs well when dry but poorly if wet; durable and tough but poor frost resistance	3.0
		wet		2.5
I-4	Unaltered gravel	dry	Tough, durable, frost resistant, and low shear strength	2.0
I-5	Gravel plus Clay gravel, 57% +4.75mm, limestone 4% -4.75mm; clay, 39%	wet	Lower end of the strength range; a combination of 3 and 4; tough, durable, poor frost resistance	1.5

a: rating assigned based on the expected relative shear strength performance potential
b: dry tests were conducted at the optimum moisture content
c: wet tests were conducted after saturation

TABLE 6.2 Stage I test results used in correlation analysis

Sample No.	Rating [(1)]	σ_d, Static Triaxial Test			σ_d, RLTT[(2)]	DCV [(3)]	CBR-soaked
		σ_c = 5 psi	σ_c = 10 psi	σ_c = 15 psi	σ_c = 15 psi		
I-1	5.0	76.7	111.3	155.5	110	9	165
I-2	4.0	38.4	82.1	109.4	113	3	95
I-3	3.0	46.4	89.1	106.8	90	6	170
I-3	2.5	48.1	57.1	71.8	70	6	96
I-4	2.0	24.0	46.8	69.8	55	2	32
I-5	1.5	33.4	56.6	85.3	40	4	65

1. Rating order assigned by research team based on expected shear strength of fabricated samples
2. RLTT: Repeated load triaxial test
3. Dielectric Constant Values: poor (DCV > 16), marginal (10 < DCV < 16), or good (DCV < 10)

TABLE 6.3 Correlation matrix for Stage I samples

Variable	CBR	D10	D15	D5	RO	RLTT	DCV
CBR	1.0 —	0.8904 0.017	0.7677 0.075	0.8149 0.048	0.6948 0.125	0.694 0.126	0.832 0.040
D10		1.0 —	0.9665 0.002	0.8429 0.035	0.9026 0.014	0.8452 0.034	0.7414 0.092
D15			1.0 —	0.8364 0.038	0.8936 0.016	0.7702 0.073	0.7081 0.115
D5				1.0 —	0.7946 0.059	0.6346 0.176	0.9581 0.003
RO					1.0 —	0.9448 0.004	0.6063 0.202
RLTT						1.0 —	0.4497 0.371
DCV							1.0 —

Top and bottom numbers in each cell are the correlation coefficient and significance, respectively.

TABLE 6.4 Multiple regression models to predict the rating of Stage I samples

Model No.	Equation	R^2	P
1	RO = -0.66 + 0.02 RLTT + 0.02 D10	0.965	0.018
2	RO = -1.20 - 0.02 CBR + 0.03 D10 + .03 RLTT + 0.21 DCV	0.999	0.082
3	RO = -0.89 + 0.04 RLTT + 0.02 D5 - 0.05 D10 + 0.03 D15	0.998	0.089
4	RO = -1.10 + 0.18 DCV + 0.04 RLTT + 0.01 D15 - 0.01 CBR	0.998	0.084

TABLE 6.5 Performance rating criteria of Stage II samples

Performance Rating	Quality	Aggregate Characteristics
5	***Excellent***	Highest quality aggregate material with history of excellent performance on heavy traffic roads with minimum cover.
4	***Good***	Aggregate material that has a history of good performance under medium to high traffic.
3	***Fair***	Material has a history of good performance when used in highways with medium traffic; usually does not work well under high traffic.
2	***Marginal***	A marginal material; usually used as subbase on high type roads and as a base on roads with light traffic.
1	***Poor***	This aggregate material is recognized as a poor performer under all traffic conditions, and is normally used with maximum cover.

TABLE 6.6 Ratings provided by state DOTs for Stage II aggregate samples

Sample No.	Rating	Pavement Type	Environment		Traffic	
			Moist.	Temp.	Category	Definition
II-6	2	F	M	FT	M	H: >20K vpd L: <5K vpd
II-7	4	B	M`	FT	H	H: >20K vpd
II-8	4.25	B	M	FT	A	–
II-9	3	B	M	FT	A	–
II-10	4	B	M	F	H, M	H: >1000K ESALs L: <100K ESALs
II-11	2	B	M	F	M, L	H: >1000K ESALs L: <100K ESALs
II-12	4.5	F	–	–	–	–
II-13	3	F	L	NF	A	H: > 20K ADT L: < 1K ADT
II-14	4.25	B	M	FT	H	H: > 1000K ESALs
II-15	1.8	B	M	FT	M	H: > 1000K ESALs L: <100K ESALs
II-16	4.3	F	H	NF	H	H: >150K ESALs
II-17	3	F	L	FT	M	H: > 8K ADT L: < 1K ADT

Material categories: P: Poor, M: Marginal, G: Good
Moisture categories: H: High, M: Medium, L: Low
Temperature categories: F: Freeze, NF: No Freeze, FT: Freeze-Thaw
Traffic categories: L: Low, M: Medium, H: High, A: All
Pavement type: F: Flexible, R: Rigid, B: Both
–: No information provided

United States that encompass different climatic conditions and aggregate sources. This section deals with analysis and interpretation of these test data. The following aggregate properties and test methods were included in this evaluation:

Aggregate Property	*Test Methods*
Screening	Sieve Analysis Atterberg Limits Moisture-Density Relationship California Bearing Ratio Specific Gravity & Absorption Flat & Elongated Particles Particle Index Uncompacted Voids
Shear Strength	Static Triaxial–dry Static Triaxial–wet Rep. Load Triaxial–dry Rep. Load Triaxial–wet
Stiffness	Resilient Modulus–dry Resilient Modulus–wet
Durability	Sulfate Soundness Durability Index
Toughness	LA Abrasion Micro-Deval Aggregate Impact Value Aggregate Crushing Value Gyratory Degradation
Frost Susceptibility	Tube Suction Test Corps of Engineers Frost Classification

Correlation Analysis

To select the test methods that could best relate to the performance of aggregates in unbound pavement layers, correlation analysis of Stage II data was undertaken. The analysis was performed on the data obtained from laboratory tests.

Screening Tests

Tests included in this category are (1) sieve analysis, (2) Atterberg limits, (3) moisture-density relationship, (4) specific gravity and absorption, (5) flat and elongated particles, (6) particle shape and texture index, and (7) uncompacted voids. These tests are currently included in most state DOTs' specifications and will determine if candidate aggregates warrant further consideration as a base or subbase layer. Tests 1 through 4 provided test results in the form of a single test parameter. Selection of test parameters for flat and elongated particles, particle shape index, and uncompacted voids is discussed in the following sections.

Flat and Elongated Particles. Results for the Flat and Elongated Particles Test, performed according to ASTM D 4791, are given in Appendix C; a summary is provided in Table 6.7. Both Flat and Elongated (FAE) and Flat or Elongated (FOE) particles were tested as defined in the test protocol. The data are reported for two ratios of 5 : 1 and 3 : 1, using both the mass method and the count method. Because no coarse fraction was available for three of the samples, 12 samples were used for fine aggregate and 9 samples were available for coarse fraction. A composite value for mass- and count-based FOE and FAE statistics using the particle size distribution of the samples was also calculated using the following equation:

$$FOE_{Composite} = \frac{G_A \times FOE_A + G_B \times FOE_B + G_C \times FOE_C}{G_A + G_B + G_C}$$

where:

G_A is the aggregate percent passing the 25 mm sieve and retained on the 19 mm sieve,

G_B is the aggregate percent passing the 19 mm sieve and retained on the 12.5 mm sieve, and

G_C is the aggregate percent passing the 12.5 mm sieve and retained on the 9.5 mm sieve.

The correlation matrix for combined count- (c) and mass- (m) based composite FAE and FOE parameters with ratios of 5 : 1 and 3 : 1 is shown in Table 6.8. These data indicate that FAE 3 : 1 c, FAE 3 : 1 m, FAE 5 : 1 c, and FAE 5 : 1 m correlate with all parameters except FOE 5 : 1 c and FOE 5 : 1 m; FOE 3 : 1 c and FOE 5 : 1 c correlate with all parameters, except FOE 5 : 1 m. FOE 5 : 1 c and FOE 5 : 1 m correlate well with each other.

A paired samples *t*-test was used to test if two related samples came from populations with the same mean. To determine whether the counting procedure (i.e., 3 : 1 and 5 : 1) changes the test results, the paired samples *t*-test was used to determine if the average results are the same. Based on the results of the *t*-test, one test parameter can then be selected for the analysis. Table 6.9 provides results of the paired samples *t*-test.

Table 6.8 indicates that FAE 5 : 1m can be used to represent all the test parameters except FOE 5 : 1c. Table 6.9 indicates that if the paired differences are calculated, then these two parameters come from populations with statistically similar means. Based on *t*-test results, either of these can be used to represent the test results; FOE 5 : 1m was selected because it had the highest correlation with performance.

Particle Shape and Index. The Particle Shape and Index test, ASTM D 3398, was conducted for different constituent fractions of each aggregate sample. A combined index, based on the gradation of the entire aggregate sample, and those of individual fractions were highly correlated with each other. The correlation coefficient was greater than 0.95 in all cases with a test significance of 0.00. The results are presented in terms of the composite particle shape and index value in Appendix C.

TABLE 6.7 Summary statistics of the particle shape test

Test Results	No. of Samples	Minimum	Maximum	Mean	Std Dev
FAE3:1c-comp.	12	8.27	36.10	20.68	8.15
FAE3:1cC	9	6.40	25.20	15.46	7.15
FAE3:1cF	12	6.50	40.80	24.00	9.09
FAE3:1cM	12	8.50	34.60	20.37	8.04
FAE3:1m-comp.	12	8.61	32.70	19.57	7.60
FAE3:1mC	9	6.40	24.20	14.38	7.15
FAE3:1mF	12	6.80	37.90	23.47	8.63
FAE3:1mM	12	9.90	30.50	19.01	7.36
FAE5:1c-comp.	12	0.09	5.40	2.77	1.88
FAE5:1cC	9	0.00	3.40	1.44	1.18
FAE5:1cF	12	0.00	7.50	3.43	2.43
FAE5:1cM	12	0.00	6.20	2.95	2.12
FAE5:1m-comp.	12	0.03	4.10	2.13	1.34
FAE5:1mC	10	0.00	3.20	1.29	1.14
FAE5:1mF	12	0.00	6.00	2.71	1.75
FAE5:1mM	12	0.00	5.70	2.47	1.94
FOE3:1c-comp.	12	0.60	10.70	5.89	3.12
FOE3:1cC	9	0.60	11.80	4.90	3.85
FOE3:1cF	12	0.00	12.90	6.59	3.44
FOE3:1cM	12	1.00	10.40	5.96	3.05
FOE3:1m-comp.	12	0.36	7.96	4.28	2.39
FOE3:1mC	10	0.50	9.30	4.04	3.04
FOE3:1mF	12	0.00	8.40	4.68	2.37
FOE3:1mM	12	0.50	7.70	4.28	2.43
FOE5:1c-comp.	12	0.00	1.30	0.20	0.37
FOE5:1cC	9	0.00	0.30	0.07	0.13
FOE5:1cF	12	0.00	1.90	0.29	0.54
FOE5:1cM	12	0.00	1.60	0.22	0.46
FOE5:1m-comp.	12	0.00	0.70	0.13	0.19
FOE5:1mC	9	0.00	0.20	0.03	0.07
FOE5:1mF	12	0.00	1.00	0.17	0.29
FOE5:1mM	12	0.00	0.80	0.17	0.30

FOE: Flat or Elongated c: Count Based 5:1 = 5:1 ratio
FAE: Flat and Elongated m: Mass Based 3:1 = 3:1 ratio
C: Coarse Fraction (-1.0" +3/4"), M: Middle Fraction (-3/4" +1/2"), F: Fine Fraction (-1/2" +3/8")
Comp.: Composite value representing the weighted mean of C, M, and F fractions

TABLE 6.8 Correlation matrix of combined FOE and FAE parameters

Test Results	Rating	FAE3:1c	FAE3:1m	FAE5:1c	FAE5:1m	FOE3:1c	FOE3:1m	FOE5:1c	FOE5:1m
Rating	1.0 —	0.1655 0.607	0.1976 0.538	0.0051 0.987	-0.0542 0.867	0.2152 0.502	0.2102 0.512	-0.4431 0.149	-0.4531 0.139
FAE3:1c		1.0 —	0.9871 0.000	0.8548 0.000	0.7595 0.004	0.8674 0.000	0.8458 0.001	0.1979 0.537	0.3926 0.207
FAE3:1m			1.0 —	0.8012 0.002	0.707 0.010	0.8537 0.000	0.8395 0.001	0.1326 0.681	0.3497 0.265
FAE5:1c				1.0 —	0.9618 0.000	0.8576 0.000	0.8518 0.000	0.4349 0.158	0.5096 0.091
FAE5:1m					1.0 —	0.8241 0.001	0.8184 0.001	0.5192 0.084	0.5871 0.045
FOE3:1c						1.0 —	0.9894 0.000	0.2993 0.345	0.4593 0.133
FOE3:1m							1.0 —	0.3544 0.258	0.4996 0.098
FOE5:1c								1.0 —	0.9255 0.000
FOE5:1m									1.0 —

FAE: Flat and Elongated, FOE: Flat or Elongated
c: Count-based composite value, m: Mass-based composite value

TABLE 6.9 Results of the paired samples *t*-test for particle shape test

Test Results	95% Confidence Interval		Means Test	
	Minimum	Maximum	T-statistic	Significance
FAE 5:1 m vs FAE 3:1 c	13.990	23.121	8.95	0.000
FAE 5:1 m vs FAE 3:1 m	13.176	21.710	9.00	0.000
FAE 5:1 m vs FAE 5:1 c	0.198	1.082	3.18	0.009
FAE 5:1 m vs FOE 3:1 c	-5.134	-2.397	-6.06	0.000
FAE 5:1 m vs FOE 3:1 m	-3.110	-1.202	-4.97	0.000
FAE 5:1 m vs FOE 5:1 c	1.174	2.688	5.61	0.000
FAE 5:1 m vs FOE 5:1 m	1.212	2.783	5.60	0.000

c: Count-based composite value, m: Mass-based composite value

Uncompacted Voids. The modified uncompacted voids content test was performed on different constituent fractions of all aggregate samples. Results of sieve analysis tests were combined to develop the composite index for all samples, as shown in Appendix C. The correlation coefficient between the composite uncompacted voids content and that of individual fractions was at least 0.9679 in all cases with test significance of 0.00.

Shear Strength Tests/Stiffness

Shear strength tests included CBR, static triaxial, and repeated load triaxial tests. Static and repeated load triaxial tests were conducted both wet and dry.

CBR test results did not correlate well with the rating and were excluded from further analysis. Dry CBR at maximum dry density had a correlation coefficient of −0.1572 with the rating with a test significance of 0.626, whereas soaked CBR at maximum dry density had a correlation coefficient of −0.0287 with a test significance of 0.929. Test results are shown in Appendix C.

Static Triaxial Test Results. Both wet and dry static triaxial tests were conducted according to ASTM D 2850. Table 6.10 provides summary statistics for test parameters; test results are provided in Appendix C. The correlation matrix for the static triaxial test results is provided in Table 6.11. Cohesion (C) did not correlate well with any of the test parameters, except σ_d at $\sigma_c = 5$ psi when tested dry. On the other hand, the deviator stresses at each of the three confining stresses correlated with each other, with varying test significance of generally less than 5 percent. Deviator stress at 5 psi for the test at dry conditions correlated fairly well to all the deviator stresses at the three confining pressures, whether tested wet or dry.

TABLE 6.10 Summary statistics of the static triaxial test

Test Results	Minimum	Maximum	Mean	Std Dev
C-dry	-2.8	12.7	5.84	5.32
C-wet	-0.5	12.9	7.46	4.62
ϕ-dry	40.9	58.0	50.71	4.71
ϕ-wet	29.1	59.3	47.79	8.26
σ_d, σ_c=5 psi dry	26.4	102.6	67.78	28.35
σ_d, σ_c =5 psi wet	30.2	111.8	73.14	27.87
σ_d, σ_c =10 psi dry	63.80	177.2	107.01	35.08
σ_d, σ_c =10 psi wet	55.30	151.90	101.34	33.85
σ_d, σ_c =15 psi dry	58.9	183.60	133.06	46.40
σ_d, σ_c =15 psi wet	78.0	185.1	135.39	34.58

Based on the results of the correlation analysis, deviator stress at 5 psi confining stress (tested dry) and deviator stresses at 15 psi confining stress (tested wet) were selected for final analysis.

Repeated Load Triaxial Test. Table 6.12 provides summary statistics for the repeated load triaxial at a confining pressure of 15 psi. Test results are provided in Appendix C. Table 6.13 shows the correlation matrix developed for the repeated load triaxial test. The deviator stress at the dry moisture condition correlated well with the number of load repetitions to failure, the wet M_R, and the wet deviator stress. Deviator stresses from wet and dry tests were selected for the final analysis.

Aggregate Durability Determination

Aggregate durability was measured in terms of the Aggregate Durability Index, ASTM D 3744, and Magnesium Sulfate Soundness test, AASHTO T 104; test results are provided in Appendix C. Table 6.14 provides summary statistics for aggregate durability, and Table 6.15 provides the correlation matrix.

The Magnesium Sulfate Soundness test results for the coarse fraction correlated well with the rating, and other test parameters were selected to represent the category in further analysis.

Aggregate Toughness and Abrasion Resistance

The LA Abrasion, Micro-Deval, Aggregate Impact Value, Aggregate Crushing Value, and Gyratory Degradation tests were conducted to get a measure of the aggregate toughness; test results are provided in Appendix C. Table 6.16 provides the descriptive statistics for the aggregate toughness category. The correlation matrix for aggregate toughness is provided in

TABLE 6.11 Correlation matrix of static triaxial test parameters

Test Results	Rating	C-dry	C-wet	ϕ-dry	ϕ-wet	σ_d, σ_c=10 psi, dry	σ_d, σ_c=10 psi, wet	σ_d, σ_c=15 psi, dry	σ_d, σ_c=15 psi, wet	σ_d, σ_c=5 psi, dry	σ_d, σ_c=5 psi, wet
Rating	1.0 —	0.4557 P=.137	-0.0930 P=.774	-0.0297 P=927	0.3781 P=.234	0.2601 P=.414	0.3703 P=.236	0.3770 P=.227	0.4119 P=.183	0.4834 P=.111	0.2973 P=.348
C-dry		1.0 —	-0.1684 0.601	-0.4106 0.185	0.456 0.136	0.6359 0.026	0.2777 0.382	0.4208 0.173	0.5186 0.058	0.8638 0.000	0.3716 0.234
C-wet			1.0 —	0.3644 0.244	-0.326 0.301	0.0850 0.793	0.4247 0.382	0.2562 0.422	0.1593 0.621	0.0893 0.783	0.5441 0..067
ϕ-dry				1.0 —	0.2295 0.473	0.3603 0.250	0.5181 0.084	0.6405 0.025	0.4082 0.188	0.0825 0.799	0.4881 0.107
ϕ-wet					1.0 —	0.5306 0.076	0.6499 0.022	0.6621 0.019	0.8423 0.001	0.612 0.034	0.5704 0.053
σ_d, σ_c=10 psi, dry						1.0 —	0.5625 0.057	0.7973 0.002	0.8412 0..001	0.8463 0.001	0.6390 0.025
σ_d, σ_c=10 psi, wet							1.0 —	0.9292 0.000	0.9421 0.000	0.6360 0.026	0.8514 0.000
σ_d, σ_c=15 psi, dry								1.0 —	0.9017 0.000	0.8002 0.002	0.8639 0.000
σ_d, σ_c=15 psi, wet									1.0 —	0.7996 0.002	0.9087 0.000
σ_d, σ_c=5 psi, dry										1.0 —	0.746 0.005
σ_d, σ_c=5 psi wet											1.0 —

Table 6.17. The correlation analysis indicates that only Micro-Deval parameter MD1 (percent loss for coarse fraction) has a significant correlation with the rating. MD1 also correlates well with other test parameters, except LA abrasion test results for coarse fraction. MD1 was selected to represent the category in further analysis.

Selection/Elimination of Between-Tests Parameters

The main objective of Stage II data analysis was to establish a set of tests that could be used to predict aggregate performance in unbound pavement layers. Using the analyses

TABLE 6.12 Summary statistics for the repeated load triaxial test

Test Results	Minimum	Maximum	Mean	Std Dev
Mean strain at failure (wet), %	1.84	14.94	8.82	4.24
Failure strain test 1 (wet), %	0.83	15.67	8.96	5.20
Failure strain test 2 (wet), %	2.45	15.3	9.38	4.50
Mean strain at failure (dry), %	3.02	15.97	10.23	5.14
Failure strain test 1 (dry), %	2.09	15.97	9.71	5.54
Failure strain test 2 (dry), %	3.59	16.17	10.07	4.78
Mean σ_d (dry), psi	98.45	188.90	161.62	31.39
Mean σ_d (wet), psi	58.95	189.55	162.73	37.63
σ_d for test 1 (dry), psi	97.9	189.40	163.13	31.09
σ_d for test 2 (dry), psi	99.00	189.30	163.62	30.16
σ_d for test 1 (wet), psi	95.00	190.50	164.97	31.65
σ_d for test 2 (dry), psi	100.00	189.20	168.82	28.94
Mean load rep. at failure (dry)	5300.00	11000.00	9084.79	2013.43
Mean load rep. at failure (wet)	3137.50	11000.00	9147.92	2838.11
Test 1 load rep. at failure (dry)	5100.00	11000.00	9125.83	2075.75
Test 2 load rep. at failure (dry)	5500.00	11000.00	9239.58	1862.29
Test 1 load rep. at failure (wet)	6275.00	11000.00	9710.42	1712.83
Test 2 load rep. at failure (dry)	5750.00	11000.00	9689.58	1827.49
Mean M_R (dry), psi	26750.00	81250.00	45395.84	13572.14
Test 1 M_R (dry), psi	27500.00	90000.00	45500.00	15948.78
Test 2 M_R (dry), psi	26000.00	72500.00	45291.67	11195.49
Mean M_R (wet), psi	22500.00	72500.00	44125.00	13260.29
Test 1 M_R (wet), psi	35000.00	70000.00	45791.67	10349.48
Test 2 M_R (wet), psi	27500.00	65000.00	45375.00	11405.99

TABLE 6.13 Correlation matrix for repeated load triaxial test

Variables	Rating	σ_d dry	M_R dry	N dry	Strain dry	σ_d wet	M_R wet	N wet	Strain wet
Rating	1.0 —	0.4445 0.148	0.1820 0.571	0.4524 0.140	-0.2503 0.433	0.3487 0.267	0.2427 0.447	0.4623 0.130	-0.2071 0.518
σ_d Opt		1.0 —	0.1014 0.754	0.9871 0.000	-0.5649 0.056	0.903 0.000	0.5805 0.048	0.8391 0.001	-0.4609 0.132
M_R dry			1.0 —	0.1029 0.750	-0.3269 0.300	0.1003 0.756	0.4662 0.127	-0.0853 0.792	-0.1943 0.545
Mean N dry				1.0 —	-0.6599 0.020	0.8606 0.000	0.5518 0.063	0.7941 0.002	-0.4967 0.100
Strain dry					1.0 —	-0.463 0.130	-0.3566 0.255	-0.255 0.424	0.5268 0.078
σ_d wet						1.0 —	0.6739 0.016	0.8845 0.000	-0.2475 0.438
Mean M_R wet							1.0 —	0.5586 0.059	-0.4549 0.137
Mean N wet								1.0 —	-0.3031 0.338
Strain wet									1.0 —

TABLE 6.14 Summary statistics for aggregate durability tests

Variables	Minimum	Maximum	Mean	Std Dev
Soundness Coarse (% loss)	1.10	43.10	13.95	16.60
Soundness Fine (% loss)	1.50	49.20	17.77	18.40
Durability Index Coarse	29.20	100.00	64.33	25.99
Durability Index Fine	27.00	90.10	65.45	23.10

TABLE 6.15 Correlation matrix for test parameters used to test aggregate durability

Variables	Rating	Durability Index Coarse	Durability Index Fine	Soundness Coarse	Soundness Fine
Rating	1.0 —	0.1704 0.597	0.2912 0.385	-0.5458 0.066	-0.5747 0.051
Durability Coarse		1.0 —	0.7416 0.009	-0.6946 0.012	-0.6713 0.017
Durability Fine			1.0 —	-0.5171 0.103	-0.4904 0.126
Soundness Coarse				1.0 —	0.9892 0.000
Soundness Fine					1.0 —

TABLE 6.16 Summary statistics for the aggregate toughness tests

Variables	Minimum	Maximum	Mean	Std Dev
MD1	5.0	42.7	16.54	12.75
MD2	6.5	46.4	19.32	13.77
CRUSH	12.4	28.6	20.32	6.00
IMPACT	14.0	33.4	21.63	6.48
LA2	14.4	40.4	26.16	9.91
LA3	15.4	39.5	26.35	9.02
LA1	17.7	40.6	31.28	9.21

Crush: Aggregate crushing value
Impact: Aggregate impact value
LA1: LA Abrasion, 37.5-9.5 mm, LA2: LA Abrasion, 18.0-9.5 mm, LA3: LA Abrasion, 9.5-4.75 mm
MD1: Micro Deval, 19-9.5 mm, MD2: Micro Deval, 13.2-4.75 mm

TABLE 6.17 Correlation matrix for the aggregate toughness tests

Variables	Rating	Crush	Impact	LA1	LA2	LA3	MD1	MD2
Rating	1.0 —	-0.3381 0.281	-0.4787 0.115	-0.4179 0.115	-0.4792 0.115	-0.5413 0.085	-0.4984 0.009	-.4642 0.130
Crush		1.0 —	0.8891 0.000	0.9495 0.004	0.9469 0.000	0.9515 0.000	0.6251 0.030	0.6372 0.026
Impact			1.0 —	0.7304 0.099	0.8466 0.001	0.9319 0.000	0.5681 0.054	0.5673 0.054
LA1				1.0 —	0.9805 0.001	0.9881 0.002	0.7441 0.090	0.7451 0.089
LA2					1.0 —	0.9836 0.000	0.6535 0.021	0.6675 0.018
LA3						1.0 —	0.692 0.018	0.7023 0.016
MD1							1.0 —	0.996 0.000
MD2								1.0 —

conducted in this project, the test parameters shown in Table 6.18 were selected. Also the following tests were selected:

Screening:
- Sieve Analysis
- Atterberg Limits
- Moisture-Density Relationship
- Specific Gravity & Absorption
- Flat & Elongated Particles
- Uncompacted Voids Test

Toughness:
- Micro-Deval

Stiffness:
- Resilient Modulus

Shear Strength:
- Static Triaxial (Dry & Wet)
- Rep. Load Triaxial (Dry & Wet)

Durability:
- Sulfate Soundness

Frost Susceptibility:
- Tube Suction

The screening tests are well established and are used by most state DOTs. These tests can quickly and inexpensively determine if an aggregate is a candidate for base or subbase aggregate.

Aggregate Performance Prediction

A process was developed for evaluating the aggregates using selected test parameters, performance ratings, traffic categories provided by state DOTs that supplied test samples, and engineering judgment. In this approach, tests are conducted in sequence while comparing results with the suggested criteria selected based on traffic and climatic or regional factors. At each step in the testing program, the aggregate is either rejected because of failure to meet suggested criteria, or it is advanced to the next phase of tests and is eventually accepted if it meets all the suggested test criteria.

The process requires the selection of test parameters based on traffic and climatic conditions. Three traffic levels are proposed:

- Low traffic (<100,000 ESALs/year),
- Medium traffic (100,000–1,000,000 ESALs/year), and
- High traffic (>1,000,000 ESALs/year).

TABLE 6.18 Description of test parameters selection for performance prediction

Test Category	Property	Description	Range
Screening Tests	Cu	Coefficient of uniformity to represent aggregate gradation	2 - >10
	D	Maximum dry density (psi)	126 - 143
	FAE 5:1 m	Flat and elongated particles in a ratio of 5 to 1, based on the mass of the sample (%)	0.0 - 4.1
	Uc	Composite uncompacted voids value (%)	13.9 - 52.2
Toughness Tests	MD1	Micro Deval test result for 19.0 to 9.5 mm fraction (% loss)	5.0 - 42.7
Shear Strength Tests	σ_d dry	Repeated load triaxial test result, tested at the OMC at 15 psi confining stress (psi)	98 - 190
	σ_d wet	Repeated load triaxial test result, tested at when saturated at 15 psi confining stress (psi)	95 - 190
	σ_{d5} dry	Standard triaxial test deviator stress at 5 psi confinement when tested dry (psi)	26 - 103
	σ_{d15} wet	Static triaxial test deviator stress at 15 psi confinement when tested wet (psi)	59 - 184
Stiffness	M_R	Resilient modulus when tested dry (ksi)	26 - 90
Durability	S	Aggregate soundness value for coarse fraction	1.0 - 43.0
Frost Susceptibility	DCV	Dielectric constant value from the Tube Suction Test	3.0 - 30.0

TABLE 6.19 Significance levels of traffic moisture and temperature combinations on aggregate performance potential

Temperature Condition	Moisture Condition	Traffic		
		High	Medium	Low
Freezing	High	4	4	3
	Low	4	3	2
Non Freezing	High	3	2	2
	Low	3	2	1

4 = Most significant; 1 = least significant

The climatic conditions of moisture and freezing, based on the AASHTO definitions (*15*), are also incorporated in this approach. Table 6.19 shows the significance levels of traffic, moisture, and climate combinations on a scale of 1 to 4, where 4 is the most significant and 1 is the least significant on aggregate performance potential.

The following variability ranges are proposed for the four significance levels:

- Level 4: X ≥ mean + 1 SD,
- Level 3: mean ≤ X < mean + 1 SD,
- Level 2: mean − 1 SD ≤ X < mean, and
- Level 1: min ≤ X < mean − 1 SD,

where X is the test parameter and SD is the standard deviation.

Proposed ranges for selected test parameters that relate to aggregate performance are shown in Table 6.20. These ranges are based on the Stage II test results and other considerations. The table suggests that pavement type has no effect on aggregate selection. The fact that unbound aggregate pavement layers under flexible and rigid pavements fail due to different mechanisms was recognized and addressed while developing Table 6.20. The user agency will adjust these values to suit project-specific conditions.

ANALYSIS OF STAGE III DATA FOR DECISION CHART VALIDATION

Stage III laboratory data was used to validate the aggregate performance prediction procedure illustrated in Table 6.20. For this purpose, Stage III data were used to determine the traffic, moisture, and temperature combination where Stage III samples could successfully be used according to this research. These combinations were then compared to the conditions where these materials have successfully been used (as reported by state DOTs).

Table 6.21 shows the laboratory data for Stage III samples and the combination of traffic, moisture, and temperature levels for which these samples are suited according to Table 6.20.

Table 6.22 compares the predicted conditions for good performance to the field conditions of the sites where the samples were obtained. This comparison confirms the validity of the process for determining performance potential, illustrated in Table 6.20.

TABLE 6.20 Recommended tests and test parameters for assessment of aggregate performance potential

<table>
<tr><th rowspan="3">TESTS</th><th>Traffic</th><th colspan="2">High</th><th colspan="2">Medium</th><th colspan="2">High</th><th>Low</th><th colspan="2">Medium</th><th colspan="3">Low</th></tr>
<tr><th>Moisture</th><th>High</th><th>Low</th><th>High</th><th>Low</th><th>High</th><th>Low</th><th>High</th><th>High</th><th>Low</th><th>Low</th><th>High</th><th>Low</th></tr>
<tr><th>Temperature</th><th>F</th><th>F</th><th>F</th><th>F</th><th>NF</th><th>NF</th><th>F</th><th>NF</th><th>NF</th><th>F</th><th>NF</th><th>NF</th></tr>
<tr><td colspan="2">Screening Tests [a]</td><td colspan="3"></td><td colspan="4"></td><td colspan="4"></td><td></td></tr>
<tr><td colspan="2">Gradation, Cu</td><td colspan="3">≥ 6</td><td colspan="4">≥ 6</td><td colspan="4">≥ 2</td><td>≥ 2</td></tr>
<tr><td colspan="2">Max. Aggregate Size</td><td colspan="3">≥ 3/4”</td><td colspan="4">≥ 3/4”</td><td colspan="4">≥ 3/4”</td><td>≥ 3/4”</td></tr>
<tr><td colspan="2">Minus #200, #</td><td colspan="3">≤ 5</td><td colspan="4">≤ 8</td><td colspan="4">≤ 10</td><td>0 ≤ # ≤ 12</td></tr>
<tr><td colspan="2">Atterberg Limits [b]</td><td colspan="3">Nonplastic</td><td colspan="4">Nonplastic</td><td colspan="4">Nonplastic</td><td>Nonplastic</td></tr>
<tr><td colspan="2">Uncompacted Voids, Uc</td><td colspan="3">< 35</td><td colspan="4">< 45</td><td colspan="4">< 55</td><td>Uc < 65</td></tr>
<tr><td colspan="2">Flat and elongated 5:1m</td><td colspan="3">< 0.10</td><td colspan="4">< 0.10</td><td colspan="4">< 0.32</td><td>< 0.32</td></tr>
<tr><td colspan="2">Toughness/Abrasion</td><td colspan="3"></td><td colspan="4"></td><td colspan="4"></td><td></td></tr>
<tr><td colspan="2">Micro-Deval, MD</td><td colspan="3">≤ 5</td><td colspan="4">≤ 15</td><td colspan="4">≤ 30</td><td>≤ 45</td></tr>
<tr><td colspan="2">Durability</td><td colspan="3"></td><td colspan="4"></td><td colspan="4"></td><td></td></tr>
<tr><td colspan="2">Sulfate Soundness, S</td><td colspan="3">≤ 13</td><td colspan="4">≤ 30</td><td colspan="4">≤ 30</td><td>≤ 45</td></tr>
<tr><td colspan="2">Frost Susceptibility [c]</td><td colspan="3"></td><td colspan="4"></td><td colspan="4"></td><td></td></tr>
<tr><td colspan="2">Tube Suction Test, DCV</td><td colspan="3">≤ 7</td><td colspan="4">≤ 10</td><td colspan="4">≤ 15</td><td>≤ 20</td></tr>
<tr><td colspan="2">F - Category</td><td colspan="3">F-1</td><td colspan="4">≥ F-2</td><td colspan="4">≥ F-2</td><td>≥ F-3</td></tr>
<tr><td colspan="2">Shear Strength [d]</td><td colspan="3"></td><td colspan="4"></td><td colspan="2"></td><td colspan="2"></td><td rowspan="7">Not Required</td></tr>
<tr><td colspan="2">Std. dry, σc=5psi, σd</td><td colspan="3">≥ 100</td><td colspan="4">≥ 60</td><td colspan="2">≥ 40</td><td colspan="2">≥ 25</td></tr>
<tr><td colspan="2">Std. wet, σc=15psi, σd</td><td colspan="3">≥ 180</td><td colspan="4">≥ 135</td><td colspan="2">≥ 90</td><td colspan="2">≥ 60</td></tr>
<tr><td colspan="2">Rep. dry, σc=15psi, σd</td><td colspan="3">≥ 180</td><td colspan="4">≥ 160</td><td colspan="2">≥ 130</td><td colspan="2">≥ 90</td></tr>
<tr><td colspan="2">Rep. wet, σc=15psi, σd</td><td colspan="3">≥ 180</td><td colspan="4">≥ 160</td><td colspan="2">≥ 125</td><td colspan="2">≥ 60</td></tr>
<tr><td colspan="2">Stiffness</td><td colspan="3"></td><td colspan="4"></td><td colspan="2"></td><td colspan="2"></td></tr>
<tr><td colspan="2">Res. Modulus, M_R</td><td colspan="3">≥ 60 ksi</td><td colspan="4">≥ 40 ksi</td><td colspan="2">≥ 32 ksi</td><td colspan="2">≥ 25 ksi</td></tr>
</table>

a. Screening tests should also include specific gravity and moisture-density relationship tests
b. Most state DOTs allow some plastic fines in their base/subbase layers; all test samples were nonplastic
c. Frost susceptibility tests are not required in non-frost areas
d. Triaxial tests are optional for the low traffic category

TABLE 6.21 Test results and potential application for Stage III samples

Tests	Sample III-18	Sample III-19	Sample III-20
Screening Tests			
Gradation, Cu	>6	>6	>6
Max. Aggregate Size	>3/4"	3/4"	>3/4"
Minus #200, #	7%	10%	10%
Atterberg Limits	- [a]	- [a]	- [a]
Uncompacted Voids, Uc	- [a]	- [a]	- [a]
Flat/elongated 5:1 m	- [a]	- [a]	- [a]
Toughness/Abrasion			
Micro-Deval, MD	- [a]	- [a]	- [a]
Durability			
Sulfate Soundness, S	- [a]	- [a]	- [a]
Frost Susceptibility			
Tube Suction Test, DCV	- [a]	- [a]	- [a]
F - Category	F-1	F-1	F-1
Shear Strength			
Std. dry, σc=5psi, σd	- a	- a	- a
Std. wet, σc=15psi, σd	189 psi	116 psi	190 psi
Rep. dry, σc=15psi, σd	187 psi	187 psi	187 psi
Rep. wet, σc=15psi, σd	187 psi	140 psi	187 psi
Stiffness			
Res. Modulus, M_R	60 ksi	50 ksi	60 ksi
Performance Potential			
Traffic	High	Medium	High
Moisture	High	Low	High
Temperature	Freezing	Freezing	Freezing

a. Test not conducted on Stage III samples

TABLE 6.22 Comparison of predicted and actual field performance potential

Sample No.	Field Condition		Agreement
	Actual [1]	Predicted [2]	
III-18	**Traffic:** High **Moisture:** High **Temperature:** Freezing	**Traffic:** High **Moisture:** High **Temperature:** Freezing	Yes
III-19	**Traffic:** Medium **Moisture:** Low **Temperature:** Freezing	**Traffic:** Medium **Moisture:** Medium **Temperature:** Freezing	Yes
III-20	**Traffic:** High **Moisture:** High **Temperature:** Freezing	**Traffic:** High **Moisture:** High **Temperature:** Freezing	Yes

1. Field condition under which the aggregate is expected to provide good performance as reported by state DOT and/or based on state's geographic location
2. Predicted field conditions under which the aggregate is expected to provide good performance based on Table 6.20

CHAPTER 7

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

Based on the results of the research in this project, the following conclusions are made.

1. Fatigue cracking, rutting, corrugations, depressions, and frost heave are distresses associated with poor performance of unbound granular layers in flexible pavements. Distresses attributed to poor performance of unbound granular layers in rigid pavements include cracking, pumping/faulting, and frost heave.
2. Properties of unbound granular aggregates used as base and subbase pavement layers that affect pavement performance are shear strength, toughness and abrasion, stiffness, durability, frost susceptibility, and permeability.
3. Shear strength of aggregate has much greater influence on the performance of an unbound aggregate pavement layer than any other aggregate property. Because stiffness is directly related to shear strength, it has a similarly large effect on performance.
4. The following tests relate to the performance of aggregates in unbound pavement layers:
   - **Screening Tests**
     - Sieve Analysis
     - Atterberg Limits
     - Moisture–Density Relationship
     - Flat and Elongated Particles
     - Uncompacted Voids
   - **Durability**
     - Magnesium Sulfate Soundness
   - **Shear Strength Tests**
     - Triaxial Tests: Wet/Dry
     - California Bearing Ratio
   - **Stiffness**
     - Resilient Modulus: Wet/Dry
   - **Toughness and Abrasion Resistance**
     - Micro-Deval Test
   - **Frost Susceptibility**
     - Tube Suction Test
5. A process was developed that can be used to evaluate the potential performance of an aggregate for use in ranges of traffic and climatic conditions. The process has been validated by limited laboratory and field data.

SUGGESTED RESEARCH

The recommended aggregate tests and the ranges of selected test parameters for different traffic and climatic conditions were based on laboratory test results. These ranges were subjected to the test of reasonableness using current state DOT specifications.

Modifications to the current test procedure for triaxial testing are proposed. The sample saturation process for triaxial testing should be revised so that water does not drain out completely before test completion by maintaining a small head of water on the sample during testing. Also, testing needs to be conducted using a load frame with at least 300 psi loading capacity or at a lower confining stress of 10 psi to establish the shear strength range.

Field Validation Plan

The following field validation plan is suggested to validate performance-related tests of aggregates identified in this research in unbound pavement layers. The plan makes use of accelerated pavement testing of specially constructed pavement sections and implementation of the procedure to actual highway construction.

Accelerated Pavement Testing

Accelerated loading devices can be used to apply full-scale loadings to specially constructed test pavements with unbound pavement layers. It is recommended to construct AC and PCC pavement sections with unbound aggregate layers to evaluate the merits of the reported research findings. By varying the characteristics of aggregates used for unbound pavement layers, the effects of various aggregate properties on the performance of unbound pavement layers can be assessed. This field performance can then be compared to the performance predicted using the methodologies developed in this research.

The primary advantage of this approach would be that the factors that affect pavement performance could be more closely controlled. This is particularly important if the test involves studying the effect of a single factor or a group of factors on pavement performance. The disadvantage of this approach is that long-term strength loss due to poor durability, poor drainability, and frost effects cannot be fully assessed.

In-Service Test Pavements

Testing in-service pavements is proposed to validate the procedure in actual practice. The study would test the adaptability of the laboratory to highway department methods of aggregate evaluation and compare the test results with current evaluation procedures. The study would provide for monitoring the pavement construction and performance to evaluate the accuracy of the performance prediction.

This study would involve the help and resources of several state DOTs. Pavement structures currently under design would be identified. Projects representing a range in traffic and climatic conditions would be chosen from various parts of the country for study. In each project, the aggregate being considered would be tested by the procedures recommended in this study. The test procedures to be used and aggregates to be tested would be in agreement with procedures used by the host state. Comparisons could be made using the recommended evaluation procedures in this report and standard evaluations of the host state. Construction of the pavement project would be followed to document the construction practices. Aggregate samples would be collected from the site for laboratory evaluation. Samples would also be saved for later testing, should such a need arise. The performance of the pavement would then be monitored by the host state.

There are benefits in testing in-service pavements, but there are also major disadvantages. Testing the in-service pavement can provide the comfort of knowing that the pavement is “real” in all respects. In most cases, however, in-service pavements do not allow for good control of the factors that may affect pavement performance. Testing in-service pavement also limits, or at least makes more difficult, the use of instrumentation installed in the pavement structure to measure response and performance.

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APPENDIXES A, C, D and E

UNPUBLISHED MATERIAL

Appendixes A, C, D, and E contained in the research agency's final report are not published herein. For a limited time, copies of this report, "Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers," containing these appendixes will be available on a loan basis or for purchase ($22.00) on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C. 20055.

Appendix A: Literature Review
Appendix C: Stage II Test Results, Database Quarry Output
Appendix D: Technical Memorandum on Tube Suction Testing and Results
Appendix E: Aggregate Performance Index

APPENDIX B

RECOMMENDED NEW AGGREGATE TESTS

These proposed testing methods are the recommendations of NCHRP Project 4-23 staff at Applied Research Associates, Inc. These methods have not been approved by NCHRP or any AASHTO committee or formally accepted for the AASHTO specifications.

PROPOSED STANDARD METHOD OF TEST FOR SHEAR STRENGTH OF AGGREGATE BY THE REPEATED LOAD TRIAXIAL TEST

1. Scope

1.1 This method covers a procedure for repeated load triaxial test for shear strength of aggregates.

2. Reference Documents

2.1 AASHTO Standards:

T 27 Standard Method for Sieve Analysis of Fine and Coarse Aggregates
M 92 Standard Specification for Wire-Cloth Sieves for Testing Purposes

3. Apparatus

3.1 Axial Loading Device.

The axial loading device shall be capable of applying axial force to the specimen using either controlled strain or controlled stress. In the controlled stress mode, the device shall be capable of applying a uniform cyclic sinusoidal force above an initial static force on the piston. The device shall be capable of applying a maximum force of at least 5,000 lbf and of cycling the force at a frequency range of 0.1 to 2.0 Hz. In the controlled stress mode, the device shall have the capability of controlling the static axial force to within ± 0.25 percent of the desired axial force and the dynamic force to within ±3 lbf of the desired peak to peak (double amplitude) cyclic axial force. In the controlled strain mode, the rate of advance of the loading device shall not vary by more than ± 1 percent from the selected value.

3.2 Axial Force Measurement Device.

The axial force measuring device shall have the capability of measuring axial force to within ± 1 percent of the applied axial force.

3.3 Pressure and Vacuum Control Devices.

The chamber, back pressure, and vacuum control devices shall be capable of applying and controlling pressures or partial vacuums to within ± 0.25 psi.

3.4 Pressure and Vacuum Measurement Devices.

The chamber, back pressure, and vacuum measurement devices shall be capable of measuring pressures and partial vacuums to within ± 0.25 psi.

3.5 Volume Change Measurement Devices.

The volume of water entering or leaving the specimen during the permeability phase of the test shall be measured with an accuracy of within ± 0.05 percent of the total volume of the specimen. The volume measurement devices shall be two burettes. One burette shall be attached to a line from the specimen cap and the other to the line from the specimen base.

3.6 Axial Deformation Indicator.

Movement of the piston relative to the top of the cell shall be measured by an LVDT (linear variable differential transformer). Axial deformation will be assumed to be the measured piston movement. The piston travel shall be measured with an accuracy of at least ± 0.02 percent of the initial height of the specimen. The LVDT shall have a range of travel of at least 15 percent of the initial height of the specimen.

3.7 Recorders.

Applied axial forces and axial deformation during static shear or repeated loading shall be recorded by electronic analog recorders. It shall be necessary to calibrate the measuring devices through the recorder using known input standards. Resolution of each variable should be within the accuracy requirements for the deformation and force measurement devices.

3.8 Specimen Size Measurement Devices.

Devices used to determine the height and diameter of the specimen shall measure the respective dimensions to within ± 0.001 inch. A circumferential measuring tape has been found to be the best device for measuring specimen diameters.

3.9 Triaxial Cell.

The triaxial chamber should be made of clear lucite or acrylic in order to aid in attachment of the piston to the specimen cap and to observe the specimen during testing. The chamber must be sufficiently thick to safely perform tests at cell pressures up to 100 psi. The top and bottom of the cell shall be constructed to seal the ends of the chamber and to ensure proper alignment of the loading piston with the specimen axis. The loading piston should have a diameter of ¾ in. and have provision for a threaded boss at one end (for connecting the piston to the specimen cap) and a spherical surface at the other end for transferring the axial load applied by the load actuator. The connection of the piston to the top cap shall be by straight threads. A teflon washer may be used to avoid over-tightening. The top of the cell shall have a piston guide containing two linear ball bushings to maintain alignment of the piston. The piston seal may be a rubber O-ring having an unstretched inside diameter of approximately 90 percent of that of the piston. The top of the cell shall also have a vent valve to provide for quick reduction of the confining pressure. The bottom of the cell shall have an inlet through which the confining liquid is supplied to the cell and inlets leading to the specimen base and to provide for connection to the cap to allow saturation and drainage of the specimen.

3.10 Specimen Cap and Base.

Aluminum caps and bases may be used. Provision should be made for drainage of the specimen through both the cap and the base. The diameter of the cap and base should be equal to that of the specimen. Perforated aluminum plates shall be attached to the cap and base so that bearing surfaces can be replaced when necessary and to aid specimen drainage. Radial grooves shall be made in the cap and base to guide water passing through the plates to a central hole in the cap and base. Holes in the bearing plates should be aligned with the grooves.

3.11 Compaction Mold.

A split compaction mold equipped with a collar and provided with a means to attach the mold and collar to the cell base shall be used to prepare specimens. The mold shall have a 0.1-in. thick porous plastic liner that will allow a membrane to be pulled against the liner surface when a vacuum is applied to the liner. The mold shall be constructed so that the liner will extend slightly below the surface of the specimen base with sufficient space between the base and liner for a membrane. The mold shall also be constructed so that the O-ring placed around the membrane to seal it against the specimen base will bear against an inner surface of the mold. In addition, the mold shall be constructed so the collar will be supported by a flange on the outside of the mold so a space can be made to keep the collar from bearing on the membrane after it is pulled over the top of the mold. An aluminum piston with a flange shall be used to complete compaction of the last layer of the specimen.

3.12 Membranes.

The specimen will be encased in two latex membranes. The inner membrane shall be 0.050 in. thick and have a diameter of 90 to 95 percent of that of the specimen. This membrane will form the inner surface of the mold and the aggregate will bear against it during compaction. The outer membrane shall be 0.012 in. thick and have a diameter of 90 to 95 percent of that of the specimen. It will be placed around the specimen after the mold has been removed and the specimen is being supported by a partial vacuum of 5 psi. The purpose of the outer membrane is to seal holes formed in the inner membrane during compaction. Prior to placing the outer membrane around the specimen, an ice pick may be used to punch additional holes in the inner membrane to ensure removal of any air trapped between the membranes. Membranes shall be sealed to the specimen cap and base using rubber O-rings having unstressed inside diameters between 75 and 85 percent of the diameter of the cap and base.

3.13 Vibratory Compactor.

Vibratory compaction shall be provided using electric rotary or demolition hammers with a rated input of 750 to 1,250 watts and capable of 1,800 to 3,000 blows per minute. The compactor head shall be at least 0.5 in (13 mm) thick and have a diameter of not less than 5.75 in. (146 mm).

4. Specimen Preparation

Specimens shall have a diameter of 6 in. and have a height-to-diameter ratio of between 2 and 2.5. The maximum particle size shall be 1 in.

Specimens shall be compacted in six equal weight and height layers using a vibratory compaction device. The top of each layer shall be scarified prior to the addition of material for the next layer.

Material for each layer shall be prepared just prior to compaction by combining air-dry gravel with previously prepared minus No. 4 sieve material batched at the water content of the minus No. 4 material in a total sample at optimum water content. Air-dry gravel for each layer shall be prepared according to the desired gradation of the gravel fraction in the total sample. Slightly more than the required wet weight of minus No. 4 material for the total specimen shall be prepared in a single batch and allowed to cure overnight prior to compaction. The required wet weight of minus No. 4 material per layer shall be based on the required dry weight and the batch water content. The combined layer weights shall be the initial wet weight of the specimen.

To prepare the specimen, place the required amount of prepared material for one layer in the mold; avoid spillage. Using a spatula, draw the material away from the inside edge of the mold to form a small mound at the center. Insert the vibrator head and vibrate the material until the required layer thickness is achieved. This may require the removal and insertion of the vibrator several times until experience is gained in gauging the vibration time that is required.

A moisture content determination will be made using left-over material after specimen preparation.

After completion of the triaxial test, the specimen will be laid flat on its side and divided into three equal portions. Moisture content will be determined for the top, middle, and bottom one-third using the divided specimen. Care must be exercised to properly label each portion of the specimen. The final moisture content reading reported should include initial portion weight of the specimen to facilitate calculation of a weighted average.

5. Procedure

5.1 Specimen Measurement.

Base the initial specimen conditions on measurements taken after the mold has been removed (with a partial vacuum of 5.0 psi applied to the specimen). Take three uniformly spaced diameter measurements along the axis of the specimen and measure the specimen height at four locations.

5.2 Prior to Shear.

Assemble the cell with a completely dry specimen drainage system (cap, drainage lines, and burettes). Set the axial deformation indicator so it will have sufficient travel to perform the test and record an initial reading. Simultaneously, decrease the 5.0-psi partial vacuum acting on the specimen to atmospheric pressure while increasing the confining pressure to 5.0 psi. Air shall be used as the confining fluid for all of the testing. If the test is to be a "dry test," proceed to the consolidation phase of the test.

5.3 Seepage Saturation.

If the test is to be a "wet test," fill the burette to the bottom of the specimen with de-aired water and allow the top of the burette access to atmospheric pressure. Open valves so the top of the specimen has access to atmospheric pressure through the burette to the top of the specimen. Next, open valves so that the water in the burette will slowly enter the bottom of the specimen. Allow water to seep through the specimen until it appears in the burette to the top of the specimen. It may be necessary to refill the burette to the bottom of the specimen several times before water appears in the other burette. If the seepage process is proceeding too slowly, apply a partial vacuum of 2.0 psi to the top of the specimen. When water appears in the burette to the top of the specimen or after approximately 8 hours of seepage, close the drainage valves to the specimen and fill the burette to the top of the specimen with de-aired water.

5.4 Back-Pressure Saturation.

After filling the burette to the top of the specimen with de-aired water, open valves in the drainage system so water in the burette under atmospheric pressure has access to both the top and bottom of the specimen. The cell pressure should still be 5.0 psi. When the water level in the burette stabilizes, simultaneously increase the confining pressure and the back pressure acting on the burette to the top of the specimen in increments of 10.0 psi until the total back pressure is 40 psi and the cell pressure is 45 psi. Remember to increase the force acting on the loading piston by an amount equal to the cell pressure times the piston cross-sectional area each time the cell pressure is increased. Allow each increment of back pressure (total of four) to remain on the specimen for approximately 15 minutes. When the water level in the burette stabilizes under the total back pressure of 40 psi, proceed to the consolidation phase of the test. A falling-head permeability test may be performed prior to consolidation. Procedures for performing falling-head permeability tests using triaxial test equipment are given in ASTM Test Method D 5084. There is currently no AASHTO test method for this test.

5.5 Consolidation.

A single confining pressure of 15.0 psi will be used for both wet and dry tests. Dry test specimens shall be consolidated in the same manner as the standard test specimens. Wet test specimens shall be allowed to stabilize under the 15.0-psi confining pressure overnight with the specimen drainage valves closed. Drainage valves will be opened for a period of 1 hour prior to testing the wet test specimens.

5.6 Axial Loading.

Repeated loads shall be applied to both wet and dry specimens in stages. Each stage, characterized by a certain deviator stress, shall consist of 1,000 repetitions of axial load applied to the specimen using a haversine waveform. Deviator stress for the first two stages will be 10 and 20 psi and will increase by 20 psi thereafter until specimen failure. Specimen failure is defined as reaching a strain of 10 percent or the load-frame limit, whichever comes first. The rate of cyclic loading will be 0.05 Hz. Sufficient time shall be allowed for the specimens to stabilize between stages. Drainage valves shall remain open during repeated loading. Divide the sample in three portions to determine the final water content.

PROPOSED STANDARD METHOD OF MEASURING THE DIELECTRIC VALUE OF AGGREGATE BY THE TUBE SUCTION TEST DEVICE

1. Scope

1.1 This method covers a procedure for assessing the moisture susceptibility of aggregates using the tube suction test apparatus.

2. Reference Documents

2.1 AASHTO Standards:

T 27 Standard Method for Sieve Analysis of Fine and Coarse Aggregates
M 92 Standard Specification for Wire-Cloth Sieves for Testing Purposes
T 180 Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop

3. Apparatus

3.1 Molds: Standard plastic molds of 6-in. diameter and 12-in. height.

3.2 Soaking pans.

3.3 Distilled water.

3.4 Drill: Electric drill with 1⁄16-in. drill bit.

3.5 Sieves: 1-in. sieve.

3.6 Dielectric probe: A capacitance-based 60-mm-diameter surface probe generating a 50 MHz electrical field between a central node and an outer ring arranged coaxially.

3.7 Multimeter: A handheld or bench-top multimeter capable of measuring capacitance.

3.8 Drying oven: A thermostatically controlled drying oven capable of maintaining a temperature of 230°F ± 9°F for drying moisture samples.

3.9 Balances: A balance or scale conforming to the requirements of AASHTO M 231.

4. Procedure

4.1 Mold Preparation

At 1⁄4 in. above the outside bottom of the mold, drill 1⁄16-in.-diameter holes around the circumference of the mold at a horizontal spacing of 1⁄2 in. This equates to 38 or 39 holes around the cylinder base. In addition, drill one 1⁄16-in.-diameter hole in each quadrant of the circular bottom of the mold, with each hole about 2 in. from the center. Weigh the mold.

4.2 Sample Preparation

4.2.1 Thoroughly dry in an oven approximately 30 lb of the aggregate to be tested. Maintain the oven at a temperature of 230°F.

4.2.2 After removing the aggregate from the oven, allow it to cool to a temperature at which it can be comfortably handled.

4.2.3 Sieve the aggregate through the 1-in. sieve screen and discard all material retained on the screen.

4.2.4 Mix the aggregate at optimum moisture as determined by Modified Proctor.

4.2.5 In the prepared plastic mold, compact the aggregate in four lifts of 2 in. each at Modified Proctor. Compact each lift with 50 blows. Use PVC wraps or a metal sleeve around the mold as necessary to prevent failure of the plastic walls during compaction.

4.2.6 After compaction, carefully smooth the sample surface. Remove or reposition any coarse aggregate protruding from the sample surface. Fill all large voids at the surface with fines, but avoid a full cover of fines to help preserve the uniformity of the particle size gradation throughout the sample.

4.2.7 Measure the final height of the sample and record its weight.

4.2.8 Place the sample in an oven maintained at a temperature around 100°F for drying. Dry each sample for a minimum of 72 hr.

4.3 Sample Testing

4.3.1 Record the weight of each aggregate sample, including the mold.

4.3.2 Use the probe and multimeter set up to take six capacitance readings on the surface of each sample. Take five readings around the perimeter of the sample and the sixth in the center. Press down on the probe with a force of about 20 lb to ensure adequate contact of the probe on the sample surface. Use minimal twisting as needed to seat the probe. Follow this pattern for each sample each time dielectric readings are made. The change in capacitance due to probe contact with the material under investigation is used to determine the dielectric value according to the following relationship (*1*):

$$\Delta C = C_a(\epsilon_r - 1)$$

where ΔC is the measured change in capacitance, C_a is the active probe capacitance, and ϵ_r is the dielectric value.

4.3.3 Place each sample in the empty soaking basin.

4.3.4 Use distilled water at 77°F to fill the soaking basin to a depth of ½-in. and record the time. Maintain the water bath at this temperature throughout the testing period.

4.3.5 Take additional capacitance or dielectric readings at the recommended time intervals of ½ hr, 1 hr, 2 hr, 4 hr, 8 hr, 12 hr, 24 hr, and 24 hr thereafter until testing is completed. These equate to total test times of ½ hr, 1½ hr, 3½ hr, 7½ hr, 15½ hr, 27½ hr, 51½ hr, 75½ hr, and so forth.

4.3.6 If the water content is also to be monitored through time, record the weight of each sample at the same or similar time intervals. Wipe the bottom of the mold dry before weighing.

4.3.7 The test is completed when the elapsed time exceeds 240 hr. Take final capacitance or dielectric readings and record the final weight.

4.3.8 If the actual dry density of each sample is to be calculated, place the samples in an oven for complete drying. The oven should be maintained at a temperature of 230 degrees Fahrenheit. Record the dried weight.

5. Reference

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation

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