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Revamping Aggregate Property Requirements for Portland Cement Concrete

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Revamping Aggregate Property Requirements for Portland Cement Concrete

by

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Abstract

Revamping Aggregate Property Requirements for Portland Cement Concrete

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Current Texas Department of Transportation (TxDOT) procedures for evaluating coarse aggregate for portland cement concrete (PCC) have been in place for over 39 years. Item 421 in the TxDOT "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges" describes the tests and test limits that must be met by aggregates before they can be approved for use in portland cement concrete applications. The intention of Item 421 is to ensure that only strong, durable aggregates are used in concrete so that the life of concrete is not cut short by common distress mechanisms which ultimately lead to costly repairs and replacements.

The two main tests currently used by TxDOT to evaluate aggregates are the magnesium sulfate soundness test and the Los Angeles abrasion and impact test. These tests are meant to characterize the overall soundness and resistance to abrasion and impact of an aggregate respectively. Unfortunately, past research has shown that the magnesium sulfate soundness and test and the Los Angeles abrasion and impact test are not able to successfully predict the field performance of an aggregate in concrete. The

requirements of item 421 have thus far done a reasonably good job of ensuring long-lasting concrete; however the current tests and test limits may be unnecessarily precluding the use of some local materials. As high quality aggregate sources are depleted and transportation costs increase, it will become more necessary to distinguish good performers from marginal and poor performers in the future.

If aggregate tests can be found that demonstrate better correlations with field performance, it may be possible to use more local aggregate sources and still provide the desired level of reliability for pavements, bridges, and other TxDOT concrete applications. Researchers are in the processing of collecting coarse and fine aggregates commonly used in Texas and testing these aggregates on a variety of alternative tests. Researchers will attempt to relate this test data to concrete behavior and ultimately recommend tests for improved TxDOT aggregate specifications.

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Chapter 1: Introduction

1.1 Need for Project

Current Texas Department of Transportation (TxDOT) procedures for evaluating coarse aggregate for portland cement concrete (PCC) have been in place for over 39 years. Item 421 in the TxDOT "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges" describes the tests and test limits that must be met by aggregates before they can be approved for use in portland cement concrete applications. The intention of Item 421 is to ensure that only strong, durable aggregates are used in concrete so that the life of concrete is not cut short by common distress mechanisms which ultimately lead to costly repairs and replacements.

The two main tests currently used by TxDOT to evaluate aggregates are the magnesium sulfate soundness test and the Los Angeles abrasion and impact test. These tests are meant to characterize the overall soundness and resistance to abrasion and impact of an aggregate respectively. However, these tests characterize intrinsic properties, and as such, the aggregates cannot be easily manipulated during production to ensure acceptance by these test criteria. Other tests are included in Item 421, such as gradation and decantation, but these tests can typically be accommodated for by aggregate producers through more stringent processing. It is imperative that the two most important tests in Item 421, the magnesium sulfate soundness and test and the Los Angeles abrasion and impact test, be able to successfully predict the field performance of an aggregate in concrete. Unfortunately, research conducted by many government transportation agencies and universities has shown that this is not necessarily the case. The requirements of item 421 have thus far done a reasonably good job of ensuring long-lasting concrete; however the current tests and test limits may be unnecessarily precluding the use of some local materials. A summary of the current concrete aggregate specifications in Item 421 can be found in Table 1. As high quality aggregate sources are depleted, and transportation costs increase, it will become more necessary to distinguish good performers from marginal and poor performers in the future.

Table 1: Summary of Current TxDOT Concrete Aggregate Specifications in Item 421

Coarse Aggregate Test	**Test Procedure**	**Test Limit**
LA Abrasion	Tex-410-A	40%
Soundness ($MgSO_4$)	Tex-411-A	18%
Deleterious substances	Tex-413-A	
clay lumps		0.25%
shale		1%
friable		5%
Decantation	Tex-406-A	1%
Gradation	Tex-401-A	

Fine Aggregate Tests	**Test Procedure**	**Test Limit**
Deleterious substances	Tex-413-A	
clay lumps		0.50%
Organic Impurities	Tex-408-A	Color
Acid Insoluble Residue	Tex-612-J	60%
Gradation	Tex-401-A	
Sand Equivalent Value	Tex-203-F	80
Fineness Modulus	Tex-402-A	2.6-2.8

A review of literature demonstrates that there is a variety of other potential aggregate tests available to evaluate aggregates for portland cement concrete applications. If aggregate tests can be found that demonstrate better correlations with field performance, it may be possible to use more local aggregate sources and still provide the desired level of reliability for pavements, bridges, and other TxDOT concrete applications. Currently, TxDOT uses aggregates from not only Texas, but Mexico, Canada, Arkansas, Louisiana, New Mexico, Kentucky, Missouri, and Illinois (Texas Department of Transportation, 2012). Hauling materials long distances from out of state is not always a sustainable or economical solution if local materials area available. An improved specification for the qualification of aggregates will ideally allow for the use of more local aggregates, while still providing high quality, durable concrete.

1.2 Project Objective

The objective of this research project is to examine alternative aggregate tests and correlate performance with the test results. By correlating aggregate performance with test results, researchers will be able to determine the most applicable aggregate tests for TxDOT's needs and ultimately make recommendations for tests and test limits.

1.3 Scope of Project

In order to correlate performance with test results, the research team is in the process of collecting coarse aggregates from more than 50 sources and fine aggregates from more than 35 sources used by TxDOT. Aggregates tested represent a variety of mineralogies and geographic locations in Texas. Fine aggregates tested also include both natural sands and manufactured sands.

Coarse aggregate tests that were performed include the Micro-Deval test (Tex-461-A), Los Angeles abrasion and impact test (Tex-410-A), magnesium sulfate soundness test (Tex-411-A), British aggregate crushing value test (BS 812 Part 110), British aggregate impact value test (BS 812 Part 112), specific gravity and absorption test

(Tex-403-A), flat and elongated particles test (Tex-280-F), Aggregate Imaging System (AIMS 2.0), thermal conductivity test (using Mathis TCi equipment), petrographic examination (ASTM C 295), and X-ray diffraction. Fine aggregate tests that were performed include the Micro-Deval test (ASTM D 7428), specific gravity and absorption test (Tex-403-A), Aggregate Imaging and System (AIMS 2.0), flakiness sieve (developed by Rogers and Gorman (2008)), X-Ray diffraction, acid insoluble residue test (Tex-612-J), petrographic examination (ASTM C 295), Grace methylene blue test (developed by W.R. Grace Co.), organic impurities test (Tex-408-A), and sand equivalent test (Tex 203-F). Because the field performance history of an aggregate was not always readily available, researchers also performed a variety of concrete tests in an attempt to correlate results with aggregate tests. Concrete tests that were performed include compressive strength, flexural strength, modulus of elasticity, and coefficient of thermal expansion. Detailed procedures for coarse aggregate tests, fine aggregate tests, and concrete tests are located in Chapter 5.

This thesis represents a summary of work performed and data collected as of April 2012, approximately halfway through the three year project.

Chapter 2: Review of Literature

Researchers conducted a comprehensive literature search to obtain pertinent information on coarse and fine aggregate tests to predict the performance of concrete. The following is a synopsis of relevant literature which includes aggregate performance, properties, and test methods.

2.1 Relating Aggregate Performance, Properties, and Test Methods

Perhaps the most relevant piece of literature on the state of the knowledge of aggregate tests is the final report from the National Cooperative Highway Research Program (NCHRP) Project 4-20C, published in 2002. NCHRP 4-20C was a research study led by Folliard and Smith which reviewed literature and current practice to identify and recommend a suite of aggregate tests that relate to portland cement concrete pavement performance.

Folliard and Smith first examined concrete pavement performance parameters affected by aggregate properties, determined which aggregate properties relate to concrete pavement performance, and then recommended the best test methods to evaluate those aggregate properties. According to the NCHRP 4-20C final report, the most important concrete pavement performance parameters are alkali-aggregate reactivity, blowups, d-cracking, roughness, spalling, surface friction, transverse cracking, corner breaks, faulting, punchouts, map cracking, scaling, and popouts (Folliard & Smith, 2002). Aggregate properties relating to these common distresses can be classified as physical properties, mechanical properties, chemical/petrographic properties, and durability properties. Physical properties include absorption, gradation, properties of microfines, shape, angularity, texture, and thermal expansion. Mechanical properties include abrasion resistance, elastic modulus, polish resistance, and strength. Chemical and petrographic properties are determined by mineralogy and durability properties are determined alkali-aggregate reactivity, and freeze-thaw resistance. Table 2, taken from the NCHRP 4-20C final report, provides a summary of the primary aggregate properties affecting key performance parameters.

Table 2: Summary of Primary Aggregate Properties Affecting Key Performance Parameters of Concrete Pavement (Folliard & Smith, 2002)

Aggregate Property	KEY PERFORMANCE PARAMETERS - from Folliard & Smith (2002)										
	AAR	Blowups	D-Cracking	Longitudinal Cracking	Roughness*	Spalling	Surface Friction**	Transverse Cracking	Corner Breaks	Joint Faulting	Punchouts
Absorption			X								
Abrasion Resistance				X			X	X	X	X	X
Angularity				X			X	X	X	X	X
Coefficient of Thermal Expansion		X		X		X		X	X	X	X
Elastic Modulus		X		X	X	X		X	X	X	X
Gradation				X	X	X		X	X	X	X
Hardness				X			X	X	X		
Mineralogy	X	X	X	X		X	X	X	X		
Porosity and Pore Structure	X		X								
Shape				X			X	X		X	X
Size	X		X	X		X		X		X	X
Strength				X		X		X	X		X
Texture				X		X	X	X	X	X	X

*Because roughness is affected by the presence of distresses, any aggregate property that influences the development of those distresses will also influence the development of roughness
** Surface friction is mainly affected by the polish resistance of fine aggregates because of the presence of the mortar-rich layer at the top surface of PCC pavements

To recommend aggregate tests based on the primary aggregate properties of interest, researchers identified important test criteria. These criteria included current or potential ability of test method to predict PCC pavement performance, repeatability and precision of test method, user-friendliness of test method, and availability and cost of test equipment. Tests that were recommended by researchers include both direct tests (performed directly on aggregate) and indicator tests (performed with mortar or concrete specimens containing the aggregate of interest).

Based on the aggregate properties of interest and the prioritized test criteria, the following test recommendations were made. Table 3, taken from the NCHRP 4-20C final report summarizes these recommendations. For each aggregate property, the recommended tests are listed, along with sample type required and the significance of the test. Sample type is listed as “A” for aggregate, “C” for concrete, or “M” for mortar.

Table 3: Test Methods Recommended by NCHRP 4-20C for Various Aggregate Properties

Aggregate Property	Recommended Test Methods (from Folliard & Smith (2002))*	Sample Type**	Significance
PHYSICAL PROPERTIES			
Absorption	AASHTO T 84 - Specific Gravity and Absorption of Fine Aggregates	A	Specific gravity needed for mixture proportioning; absorption values needed for moisture corrections at batch plant. Absorption may influence workability.
	AASHTO T 85 - Specific Gravity and Absorption of Coarse Aggregates	A	Specific gravity needed for mixture proportioning; absorption values needed for moisture corrections at batch plant. Absorption may influence workability and possibly freezing and thawing resistance.
Gradation	AASHTO T 27 - Sieve Analysis of Fine and Coarse Aggregates	A	Standard test to determine particle size distribution. Data essential for mixture proportioning. Gradation strongly affects workability of concrete.
Properties of Microfines	AASHTO T 11 - Materials Fine Than No. 200 Sieve in Mineral Aggregates by Washing	A	Measures amount of material passing #200 sieve, which influences water demand, workability, shrinkage, etc.
	AASHTO T 176 - Plastic Fines in Graded Aggregates and Soils by the Use of the Sand Equivalent Test	A	Determines relative amount of clay like material. Quick and simple to perform.
	AASHTO TP 57 - Standard Test Method for Methylene Blue Value of Clays, Mineral Fillers, and Fines	A	Quick, semi-quantitative test to assess clays (and organic materials) in aggregates. Does not identify type of clay present.
	X-ray Diffraction Analysis (XRD)	A	Useful in identifying type of clay minerals present in aggregates
Shape, Angularity, and Texture	AASHTO T 304 - Uncompacted Void Content of Fine Aggregates	A	Provides index of fine aggregate angularity, sphericity, and surface texture. Higher void contents result in higher water demands.
	AASHTO TP 56 - Uncompacted Void Content of Coarse Aggregates (As influenced by Particle Shape, Surface Texture, and Grading)	A	Provides index of coarse aggregate angularity, sphericity, and surface texture. Higher void contents result in higher water demands.
	ASTM D 7491 - Test Method for Flat or Elongated Particles in Coarse Aggregates	A	Assesses coarse aggregate shape by identifying ratio of length to width (for different size fractions). Flat and elongated particles affect water demand, mixture consistency, and finishability.
Thermal Expansion	AASHTO TP 60-00 - Standard Test Method for the Coefficient of Thermal Expansion of Concrete (*indicator test*)	C	Measures coefficient of thermal expansion of concrete, which is a major function of aggregate thermal properties. Affects thermal cracking of pavements, especially for CRCP.
MECHANICAL PROPERTIES			
Abrasion Resistance	CSA A23.2-23A - Resistance of Fine Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus	A	Wet attrition test to assess breakdown and slaking of concrete sand. Correlates well with Mg sulfate soundness test but has better precision. Max. loss of 20 percent recommended by CSA.
	AASHTO TP 58 - Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus	A	Wet attrition test to assess breakdown and slaking of concrete sand. Correlates well with Mg sulfate soundness test but has better precision. Max. loss of 14-17 percent recommended by CSA.
	AASHTO T 96 - Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine	A	Dry attrition test for coarse aggregates mainly used as a QC test. Can be used to assess breakdown during handling and may be related to abrasion from studded tires or chains.
Elastic Modulus	ASTM C 469 - Static Modulus of Elasticity and Poisson's Ration of Concrete in Compression (*indicator test*)	C	Indicator test to assess the impact of aggregate elastic modulus on concrete elastic modulus. Affects thermal and drying shrinkage cracking.
Polish Resistance	ASTM D 3042 - Insoluble Residue in Carbonates	A	Assesses the non-carbonate (usually siliceous) material intermixed with carbonate minerals in aggregates. Used as an index of polishing resistance of concrete sands.
Strength	British Standard 812 (Part 3) - Aggregate Crushing Value	A	Measures relative strength of graded coarse aggregate. Aggregate strength is most relevant for higher strength concrete.

CHEMICAL/PETROGRAPHIC PROPERTIES			
Mineralogy	ASTM C 295 - Guide for Petrographic Examination of Aggregates for Concrete	A	Useful in identifying reactive (ASR or ACR) minerals and other deleterious materials. Mineralogy of aggregates affects many concrete properties including thermal and drying shrinkage.
	X-ray Diffraction Analysis (XRD)	A	Reported to be helpful in identifying non-durable carbonate aggregates. Useful tool in identifying types of clay present in aggregates.
	Thermogravimetric Analysis (TGA)	A	Reported to be helpful in identifying non-durable carbonate aggregates.
	X-ray Fluorescence Analysis (XRF)	A	Reported to be helpful in identifying non-durable carbonate aggregates.
DURABILITY PROPERTIES			
Alkali-Aggregate Reactivity	ASTM C 295 - Guide for Petrographic Examination of Aggregates for Concrete	A	Identifies potentially reactive minerals and confirms findings from other tests (AASHTO T 303)
	AASHTO T 202 - Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction	M	Accelerated test suitable as screening test, but results should not be used independently to judge aggregate.
	ASTM C 1293 - Test Method for Concrete Aggregates by Determination of Length Change of Concrete due to Alkali-Silica Reaction	C	Long-term test (one year for aggregates, two years to assess pozzolans or slag) that relates well to field performance.
	CSA A23.2-26A - Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition	A	Useful, rapid screening test to identify aggregates with potential for ACR.
Freezing and Thawing Resistance	AASHTO T 161 (modified Procedure C) - Resistance of Concrete to Rapid Freezing and Thawing	C	Effective means of assessing D-cracking potential of aggregates but is time consuming. Various modifications of procedure can be used, including pre-soaking aggregates in salt solution.
	CSA A23.2-24A - Unconfined Freezing and Thawing of Aggregates in NaCl	A	Rapid test with good ability to predict D-cracking. Use of NaCl improves correlation with field performance. Max. loss of four to six percent recommended by CSA.
	AASHTO T 104 - Soundness by Use of Mg or Na Sulfate (recommend Mg)	A	Common test conducted by most DOTs. Magnesium sulfate recommended over sodium sulfate.
	Iowa Pore Index Test	A	Assesses volume of micropores in aggregates, which is critical for D-cracking. Good correlation with field performance. Secondary loading limit of 27 mL specified by Iowa DOT.
	Modified Washington Hydraulic Fracture Test (after Embacher and Snyder, 2001)	A	Measures fracture of aggregates in pressurized vessel. Recent modifications have improved precision of test and correlation with AASHTO T 161.

*Tests are recommended to assess aggregate property unless otherwise noted - some are "indicator" tests that indirectly assess property

**Sample Type: A=Aggregate, C=Concrete, M=Mortar

As part of the NCHRP 4-20C final report, authors presented a flowchart to assist agencies in selecting aggregate tests. Decision blocks rely on the user's judgment as to whether or not certain parameters (i.e. "Is elastic modulus a concern?") are important in the decision making process. Certain aggregate properties may have a greater impact on certain PCC pavement types (for example CoTE impact on continuously reinforced concrete pavements) but for the most part, the same aggregate properties influence the performance of all types of PCC pavement, therefore a single flowchart is logical for all pavement types. Figure 1 displays the flowchart for aggregate testing developed by Folliard and Smith (2002).

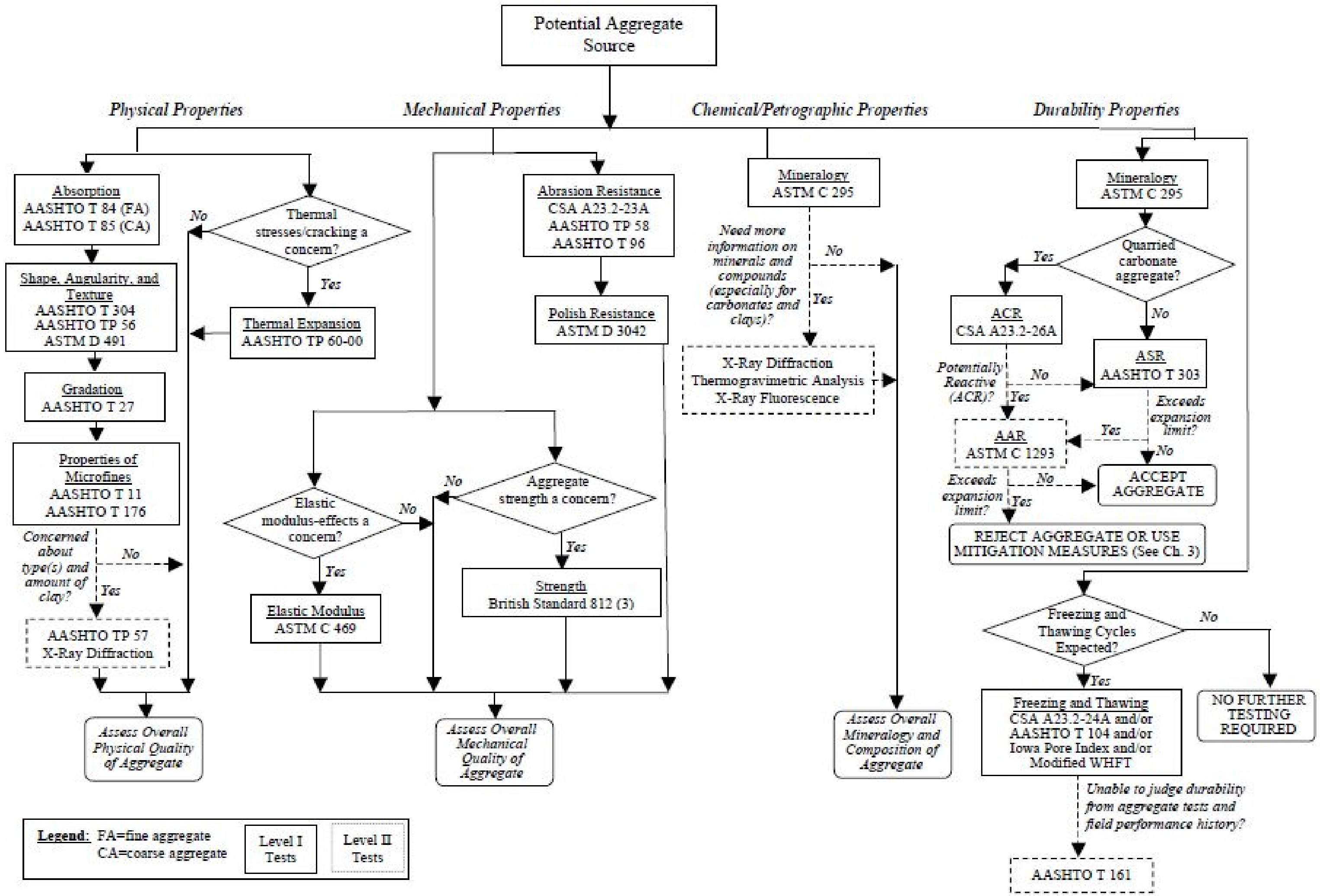

Figure 1: NCHRP 4-20C Recommendation for Aggregate Test Selection Process (Folliard & Smith, 2002)

When selecting test criteria for accepting or rejecting aggregates, engineers should first consider pavement type and design, climatic/environmental conditions, traffic loading, materials and mixture proportions, frequency of aggregate testing, and field performance histories of aggregates (Folliard & Smith, 2002). For example, freeze-thaw testing may not be as crucial in Texas as in Minnesota, or an aggregate may demonstrate good field performance history, despite not meeting a limit for a certain test.

2.2 Similar Research Projects

Research projects with similar scope and objective have been performed by various agencies in the past, although arguably none as comprehensive in size as the TxDOT research project ("Revamping Aggregate Property Requirements for Portland Cement Concrete") of which this thesis is a part. Other projects include: ICAR 507-1F by Fowler, Allen, Lange, and Range (2006), WHRP 06-07 by Weyers, Williamson, Mokarem, Lane, and Cady (2005), FHWA-SC-05-01 by Rangaraju, Edlinski, and Amirkhanian (2005), NCAT 98-4 by Wu, Frazier, Parker, and Kandhal (1998), and studies by the Minnesota DOT by Koubaa and Snyder (1999) and Ministry of Transportation of Ontario by Senior and Rogers (1991). These are all research projects that tested a variety of aggregates and attempted to relate test results with field performance. The findings of these research projects are presented in this section.

The International Center for Aggregates Research (ICAR) completed a study in 2006 (ICAR 507-1F) to determine the effectiveness of the Micro-Deval test in predicting performance of coarse aggregates for portland cement concrete, asphalt concrete, base course, and open-graded friction course (Fowler, Allen, Lange, & Range, 2006). Researchers collected coarse aggregates from 117 sources in Canada and the United

States representing a diverse mix of mineralogies, geographies, and performance histories. Aggregates were subjected to a variety of tests including: Micro-Deval, magnesium sulfate soundness, Los Angeles abrasion and impact, Canadian unconfined freeze-thaw, aggregate crushing value, aggregate crushing value at SSD, specific gravity and absorption, particle shape factor determination, and percent fractured test. State DOTs that submitted aggregates for this study were surveyed for relevant information about the performance histories of the aggregates such as: applications of use, years in service, average daily traffic, freeze-thaw exposure, failure characteristics, and time until failure. Researchers used this information to determine a rating for each aggregate. The rating criteria can be seen in Table 4.

Table 4: Performance Criteria for ICAR 507 (Fowler, Allen, Lange, & Range, 2006)

Rating	**Description - from Fowler et al. (2006)**
Good	used for 10 or more years with no reported non-chemical problems
Fair	used at least once where minor non-chemically related failures require minor repairs, but average service life extends beyond 10 years
Poor	used at least once where severe degradation or failure occurred within 2 years of service or during construction which severly inhibits and/or prevents the use of the application

After establishing performance rating for aggregates, the aggregates were subjected to the testing regimen and researchers examined the success rates for the Micro-Deval test and success rates for the Micro-Deval test combined with other tests. The best single test indicator of field performance for all applications was the Micro-Deval test, but the best overall indicator of field performance was the Micro-Deval test used in combination with the Canadian unconfined freeze-thaw test. A Micro-Deval loss of 21%, combined with a Canadian unconfined freeze-thaw loss of 3.6% was able to bound 77% of good performers for PCC applications. Figure 2 graphically displays the bounding of performers by these criteria.

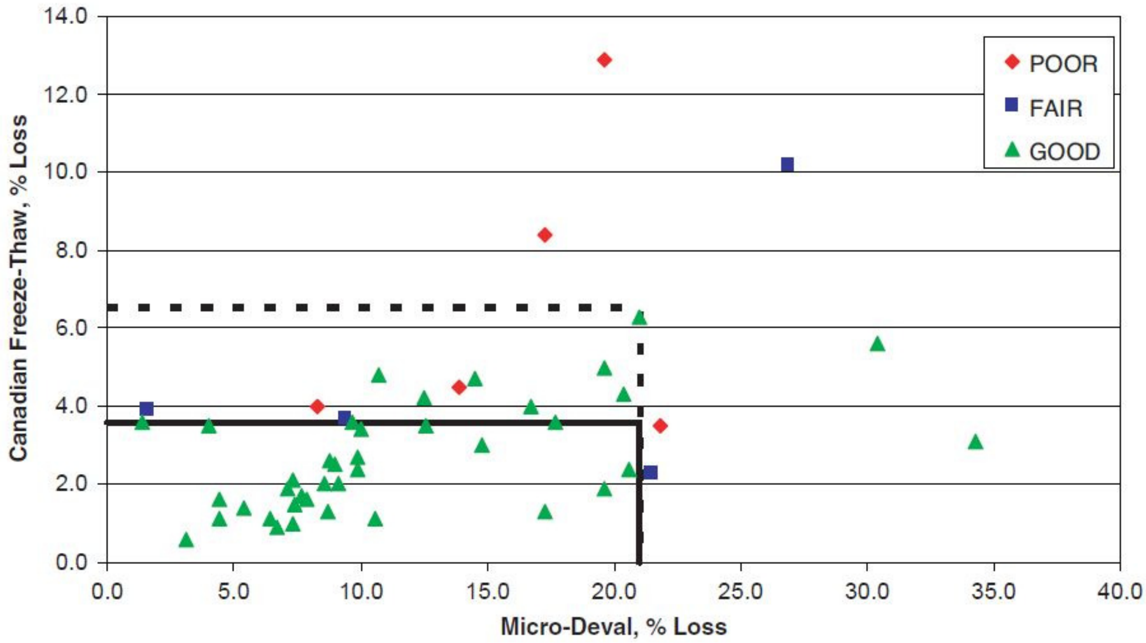

Figure 2: Canadian Freeze-Thaw vs Micro-Deval (Fowler, Allen, Lange, & Range, 2006)

Figure 2 demonstrates that the limits of 21% for Micro-Deval and 3.6% for Canadian Freeze-Thaw were also able to exclude 100% of the fair performers and 100% of the poor performers. If the Canadian Freeze-Thaw limit is raised to 6.5% (dashed line), 95% of good performers are included, 50% of the fair performers are included, and 60% of the poor performers are included. However, it is important to note that only four fair performers and five poor performers were subjected to testing. These researchers acknowledged that further research of more marginal performers may be needed to refine these limits. Success rates of other tests are shown below in Table 5.

Table 5: Success Rate Summary of Tests in ICAR 507 (Fowler, Allen, Lange, & Range, 2006)

Test	Test Alone (%)	Micro-Deval Combination (%)
Micro-Deval	83	N/A
Magnesium sulfate soundness	81	85
Canadian freeze-thaw	88	88
Absorption	83	83
Specific gravity (bulk)	85	87

*From Fowler et al. (2006)

ICAR 507 researchers also determined correlations between test methods to determine any trends that may exist. The most significant correlations were found to exist between Los Angeles abrasion and aggregate crushing value (R^2 = 0.65) and between absorption and specific gravity (R^2 = 0.65). The particle shape factor test (ASTM D 4791) was not a good indicator of field performance and did not have significant correlation with any other test. Surprisingly, the magnesium sulfate soundness test and Canadian unconfined freeze-thaw test had a low correlation (R^2 = 0.39) when all data points were considered (Fowler, Allen, Lange, & Range, 2006).

A smaller study within ICAR 507 included the use of the Aggregate Imaging System (AIMS). Twenty aggregates of varying performance histories were tested and the researchers concluded that higher average angularity and texture indices generally corresponded to better field performance but particle shape factor and sphericity factor were not good indicators of performance. By using texture and angularity indices on the same plot, researchers were able to pick limits (220 and 2850 respectively) that successfully identified most good and poor performers. However, the ability of these limits to identify fair performers was not good. Overall, the ICAR 507 researchers believe that the AIMS shows promise for providing relevant information to predict field performance, pending more comprehensive studies.

In 1991, Rogers and Senior published results from an Ontario Ministry of Transportation research study that examined testing and performance of 100 coarse aggregates from Ontario. The research was initiated because the specifications at the time (LA abrasion, magnesium sulfate soundness, and 24 hour water absorption) were prohibiting the use of a few aggregate sources that were known to have satisfactory characteristics. The tests were also stated to have poor precision, poor correlation with field performance, and poor ability to distinguish between marginal aggregates (aggregates with test results falling near test limits). As a result, this study examined alternative tests and the selected aggregates were tested for: Canadian unconfined freeze-thaw, Micro-Deval, aggregate impact value, polished stone value, and aggregate abrasion test. The aggregates were also rated as "good", "fair", or "poor" depending on pavement life and deterioration mechanisms.

Results of the Ontario study demonstrated that Canadian unconfined freeze-thaw and Micro-Deval were the best indicators of field performance and when combined, were fairly successfully at differentiating between "fair" and "poor" aggregates in portland cement concrete. Rogers and Senior also noted that water absorption can also be used as an indicator of poor performing aggregates. Other important conclusions included determining that the standard deviation for Micro-Deval was less than the magnesium sulfate soundness, even for high loss aggregates which is important for test precision. The two impact tests studied (LA abrasion and aggregate impact value) demonstrated relatively little relation to concrete field performance, yet had fairly high correlation to each other ($R^2 = 0.64$) (Rogers & Senior, 1991). The relationship between Micro-Deval abrasion loss and unconfined freeze-thaw loss can be seen in Figure 3.

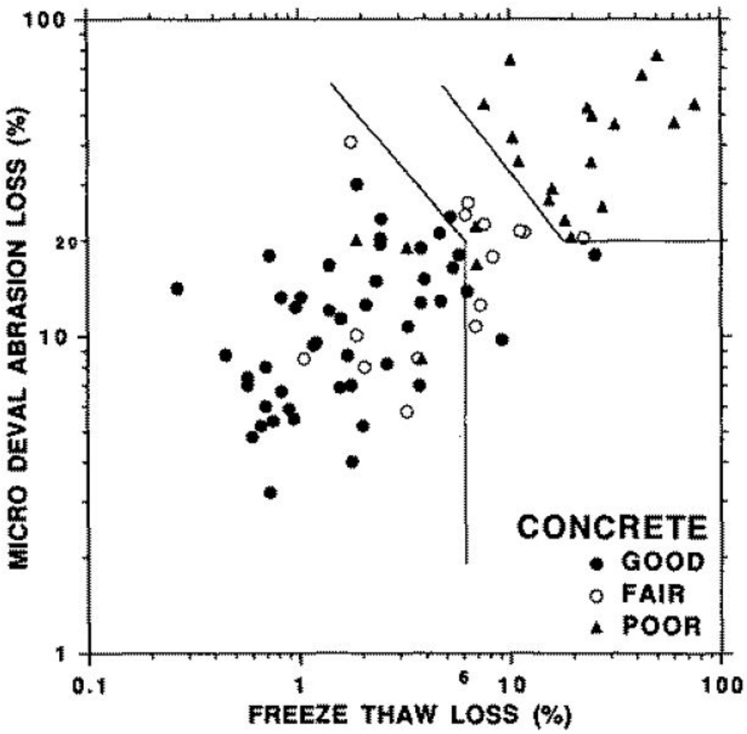

Figure 3: Relationship between Micro-Deval Loss and Canadian Unconfined Freeze-Thaw Loss with Performance Ratings for Ontario Aggregates (Rogers & Senior, 1991)

In 2005, researchers at the Virginia Transportation Research Council, in conjunction with the Wisconsin Department of Transportation (research project WHRP 06-07), evaluated 60 Wisconsin coarse aggregates on a variety of tests in an attempt to correlate results with field performance and recommend the most relevant tests. Tests in the study included sodium sulfate soundness, lightweight pieces in aggregate, frost resistance of aggregates in concrete, unconfined freeze-thaw, Micro-Deval, vacuum saturated specific gravity and absorption, and compressive strength of concrete. Wisconsin DOT officials classified each aggregate as “good” (20 total aggregates), “intermediate” (26 total aggregates), and “poor” (14 total aggregates).

Major conclusions of the WHRP 06-07 research project included finding a high correlation between Micro-Deval and vacuum saturated absorption (R^2 = 0.86) and the recommendation that absorption can be used as a primary indicator of durability (Weyers, Williamson, Mokarem, Lane, & Cady, 2005). The LA abrasion test was only able to identify the very worst aggregate sample as being poor but, although the LA abrasion test cannot directly predict the overall performance of an aggregate, it can be used to accurately estimate the strength. The sodium sulfate soundness test was determined to be highly variable. Although other literature has recommended an unconfined freeze-thaw limit of 10%, researchers determined that this limit would reject too many aggregates. As a result, they recommended a limit of 15% for Wisconsin aggregates. A similar finding was noted regarding the Micro-Deval. Weyers et al. (2005) recommended adding the Micro-Deval to Wisconsin DOT test procedures but again, the recommended limit (18%) would reject too many aggregates. Weyers et al. (2005) concluded that a limit of 25-30% for Micro-Deval loss would be more reasonable for Wisconsin aggregates. Because the aggregate crushing value and LA abrasion test were highly correlated and appear to measure the same property (aggregate strength), Weyers et al. (2005) saw no purpose in replacing the LA abrasion test. Based on these conclusions and additional recommendations made as part of this research project, a flow chart was created to assist DOT personnel in characterizing and qualifying coarse aggregates in the future. This flow chart can be found in Figure 4 (Weyers, Williamson, Mokarem, Lane, & Cady, 2005).

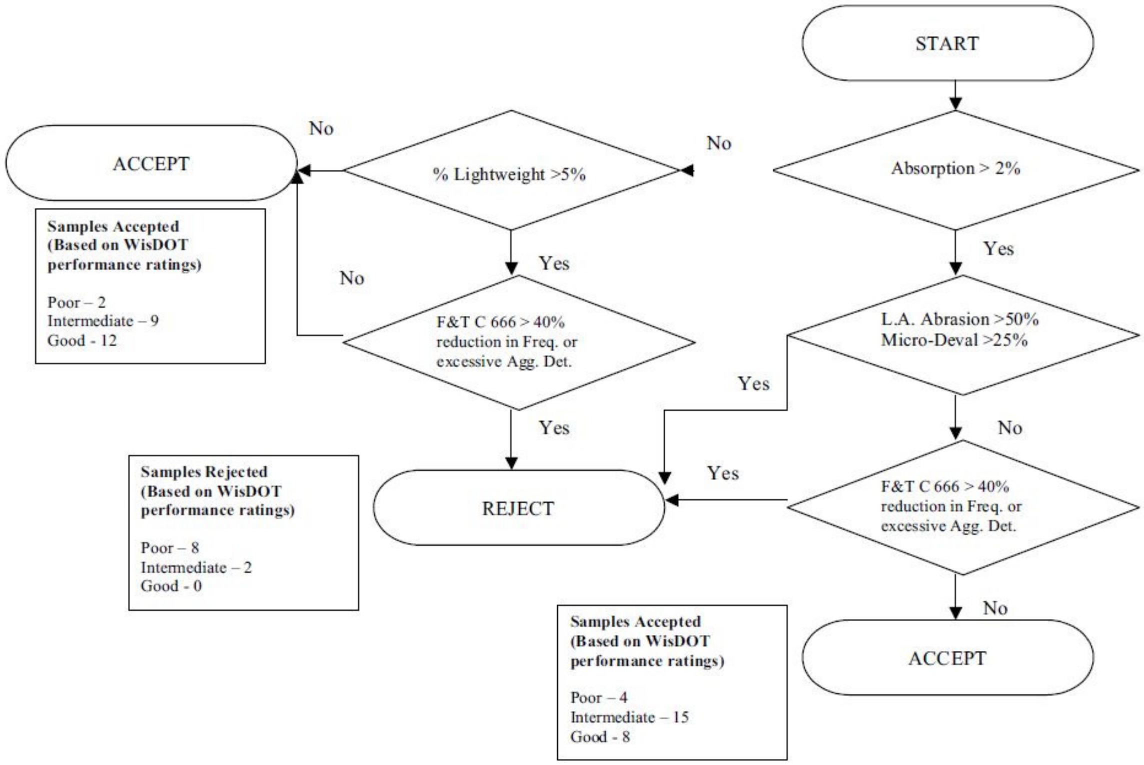

Figure 4: Aggregate Durability Testing Flowchart for Concrete Aggregates (Weyers, Williamson, Mokarem, Lane, & Cady, 2005)

In 2005, Rangaraju, Edlinski, and Amirkhanian from Clemson University published a report (FHWA-SC-05-01) in conjunction with FHWA and South Carolina DOT which described the results of a study evaluating South Carolina aggregate durability properties of 23 coarse aggregates. The Micro-Deval and magnesium sulfate soundness test were examined and compared to the two tests historically used by South Carolina DOT, the LA abrasion test and the sodium sulfate soundness test. Field performance ratings of each aggregate were provided by South Carolina DOT officials in order to attempt to identify the tests that most accurately distinguish performance. Results of this study demonstrated that there was no significant correlation between Micro-Deval and LA abrasion, or between Micro-Deval and either sulfate soundness test.

However, Rangaraju, Edlinski, and Amirkhanian did find a strong correlation between magnesium sulfate soundness and sodium sulfate soundness. The LA abrasion limit of 55% did not do a good job in identifying marginal ("fair" and "poor") aggregates in field performance but a Micro-Deval limit of 17% was able to identify all marginal aggregates. As a result, Rangaraju, Edlinski, and Amirkhanian concluded that the Micro-Deval test does a better job of predicting aggregate durability and as such, should be implemented in South Carolina DOT specifications (Rangaraju, Edlinski, & Amirkhanian, 2005).

The National Center for Asphalt Technology (NCAT) published a report in 1998 (NCAT 98-4) which detailed a research project about aggregate toughness, abrasion resistance, and durability in asphalt concrete performance in pavements. Although this study was focused on asphalt concrete applications, general trends from test results are still applicable for aggregates used in portland cement concrete, as many sources provide aggregates for both applications. In this study, Wu, Parker, and Kandhal from Auburn University and Georgia DOT examined 16 aggregate sources (from all regions of the US and varying in mineralogy) and subjected these aggregates to tests including: LA abrasion, Micro-Deval, aggregate impact value, aggregate crushing value, sulfate soundness, freeze-thaw soundness, Canadian unconfined freeze-thaw, and aggregate durability index. Aggregates included 5 carbonate sources, 4 gravels, 2 granites, 1 trap rock, 1 siltstone, 1 sandstone, and 1 steel slag. Surveys and discussions with DOT agencies allowed the researchers to provide a "worst case" characterization of aggregates as "good" (used for many years – no issues), "fair" (used for at least 8 years – some issues), and "poor" (problems occurred during first two years). Ratings were made independently for abrasion/toughness resistance and soundness/durability for each aggregate. The results of this study demonstrated that the Micro-Deval and magnesium sulfate soundness tests appear to be the best indicators of potential field performance of aggregates used in asphalt concrete. Limits of 18% for each test appear to separate good/fair performers from poor performers. Wu, Parker, and Kandhal concluded that the Micro-Deval test is the best choice for aggregate quality characterization (Wu, Parker, & Kandhal, 1998).

A study performed by Koubaa and Snyder at The University of Minnesota in the late 1990s examined test methods for better characterizing freeze-thaw durability of Minnesota aggregates for concrete applications. After completing concrete pavement condition surveys to document a variety of freeze-thaw performance, the researchers collected and tested aggregates from 13 sources (11 carbonates – mostly dolomites, 2 gravels). Tests performed include absorption and bulk specific gravity, PCA absorption, Iowa pore index, acid insoluble residue, X-ray diffraction analysis, X-ray fluorescence analysis. Thermogravimetric analysis, Washington hydraulic fracture test, ASTM C 666, and the VPI single-cycle slow freeze test. Cores were also taken from the field for various tests and examinations. Results of this research project demonstrated that the tests with the best correlation to field performance were: modified ASTM C 666 (procedure B), VPI single-cycle slow freeze test, and the hydraulic fracture test. Other tests with correlation to field performance, though not as strong, included absorption, specific gravity, Iowa pore index and X-ray fluorescence. Field investigations demonstrated that fine-grained dolomites and aggregates with cracked shale particles caused poor freeze-thaw performance of the concrete. Due to discrepancies in otherwise strong patterns in test data, Koubaa and Snyder concluded that the best method for accepting or rejecting aggregates subject to freeze-thaw distress was to develop a flow chart, as no single test can accurately predict durability. The flow chart is not included in this literature review because in the scope of this project, freeze-thaw characterization is not a priority (only very northern parts of Texas experience more than a few yearly freeze-thaw cycles). However, the flow chart developed by Koubaa and Snyder can be found in conference proceedings from the 7th Annual ICAR Symposium (Koubaa & Snyder, 1999).

2.3 Coarse Aggregate Tests

Coarse aggregate tests are the focus of this study, and as such, it was necessary to conduct a comprehensive literature review to determine the most common and most effective coarse aggregate tests.

2.3.1 Abrasion Resistance

Abrasion resistance is an aggregate property that influences the breakdown of aggregate during production, handling, and mixing, the effect of studded tires on aggregates near exposed concrete surfaces, and the behavior of aggregates at concrete joints (Folliard & Smith, 2002). Breakdown of aggregate due to poor abrasion resistance can cause the production of microfines during mixing which is not always desirable and can lead to workability and placement problems for the concrete. The two most popular tests to assess a coarse aggregate's abrasion resistance are the Los Angeles abrasion and impact Test (also known simply as the LA abrasion test) and the Micro-Deval test. A review of coarse aggregate specifications used by United States DOTs showed that the LA abrasion test is used to evaluate aggregates by 49 of 50 DOTs (see Section 3.2.1 Coarse Aggregate Specifications). The Micro-Deval test is currently used more in Canada than in the U.S. but there has been a significant interest in the potential of the Micro-Deval test by American researchers and DOTs alike during the past two decades.

2.3.1.1 Micro-Deval Test for Coarse Aggregates

The Micro-Deval test originated from the Deval test which was developed in the 1900s to assess the quality of railroad ballast. French researchers desired to modify this test to abrade, rather than fracture, aggregates after determining that running this test in a wet condition increased loss by friction and abrasion (Dar Hao, 2010). As a result, French researchers developed the Micro-Deval test which subjects water, aggregate, and steel charge to 12,000 revolutions in a steel drum via a ball mill roller. Researchers in Canada further modified this test to allow for a larger aggregate sample size and slightly altered dimensions of the steel charge and drum (Rogers C. , 1998). Today, most Micro-Deval specifications are based on the AASHTO standards similar to those documented by the Canadian researchers.

There has been a significant amount of research in the past two decades examining how to best use the Micro-Deval test to characterize and qualify aggregates. Several studies have focused on the potential of the Micro-Deval to replace other durability tests (such as sulfate soundness tests) due to its higher repeatability and lower variability. Although research has consistently demonstrated the high repeatability of the Micro-Deval test, its correlation with other tests has shown mixed results. For example, a study from 2005 showed that for the 23 aggregate sources studied (all from South Carolina), there appeared to be no significant correlation between Micro-Deval and either sulfate soundness test (Rangaraju, Edlinski, & Amirkhanian, 2005). Another study in 2003 showed similar results after testing 72 aggregates from the Southeast United States, with Micro-Deval having a very low correlation (R^2 = 0.11) with sodium sulfate soundness (Cooley & James, 2003). However, a study in 2007 found that the Micro-Deval had a good correlation with sodium sulfate soundness (R^2 = 0.72) for 32 Montana aggregates, and thus recommended the Micro-Deval test as a good candidate to replace the sodium sulfate test due its high correlation, repeatability, and quick test time (Cuelho, Mokwa, & Obert, 2007). An examination of the Micro-Deval test for Virginia sources yielded similar conclusions about the enhanced repeatability of the Micro-Deval test compared to sulfate soundness. For twenty aggregates tested, researchers found the average coefficient of variance (COV) for Micro-Deval to be 4.8% compared to 20-30% for magnesium sulfate soundness (Hossain, Lane, & Schmidt, 2008). Rogers and Senior also found the COV for Micro-Deval to be much lower than magnesium sulfate soundness (Rogers & Senior, 1991). Their results are displayed graphically in Figure 5.

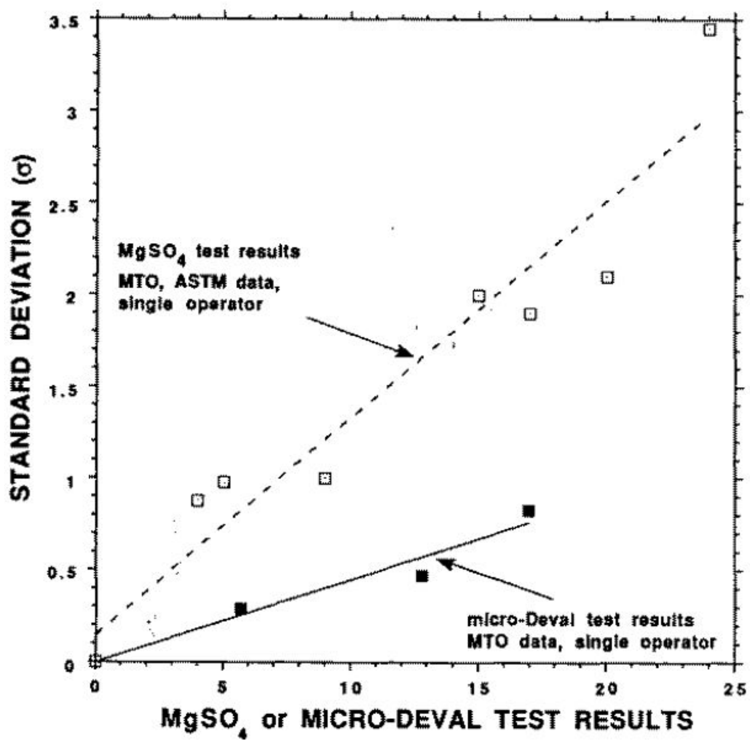

Figure 5: Standard Deviation Against Loss in Magnesium Sulfate Soundness or Micro-Deval Abrasion Test (Rogers & Senior, 1991)

A 2007 study in Texas also showed that the Micro-Deval test had much better repeatability (within-lab precision) and reproducibility (between lab precision) than the magnesium sulfate soundness test. This study also attempted to alter the Micro-Deval test to improve its correlation with the magnesium sulfate soundness test, but no alterations were successful in improving the correlation beyond R^2 value of 0.80. Because of this finding, researchers concluded that the correlation was not strong enough to justify replacing the magnesium sulfate soundness test with the Micro-Deval test, but the Micro-Deval test should be implemented as a quality control tool (Jayawickrama, Hossain, & Hoare, 2007).

For predicting field performance, the Micro-Deval test appears to be very promising. In 1991, Senior and Rogers collected over 100 aggregates from Ontario and tested them for: Canadian unconfined freeze-thaw, Micro-Deval, aggregate impact value, polished stone value, and aggregate abrasion test. Unconfined freeze-thaw and Micro-Deval were the best indicators of field performance for concrete aggregates. It was also determined that standard deviation for Micro-Deval was less than magnesium sulfate soundness, even for high loss aggregates (Rogers & Senior, 1991). A 2005 Wisconsin DOT study conducted by Weyers et al. recommended that the Micro-Deval test be added to Wisconsin DOT procedures after subjecting 60 aggregates of varying performance to a variety of tests. This study also found a high correlation (R^2 = 0.86) between Micro-Deval and vacuum saturated absorption (Weyers, Williamson, Mokarem, Lane, & Cady, 2005). A fairly comprehensive aggregate study in Texas (ICAR 507) examined 117 aggregate sources from North America and found the best correlation with field performance of concrete to be the Micro-Deval test when used in combination with the Canadian freeze-thaw test, as previously shown in Figure 2 (Fowler, Allen, Lange, & Range, 2006).

Recommended limits vary for each study which is not necessarily surprising considering different studies examined different aggregates and correlated performance to different environmental regions. The FHWA study examining South Carolina aggregates noted that a Micro-Deval limit of 17% was sufficient in identifying all marginal performers (Rangaraju, Edlinski, & Amirkhanian, 2005). An FHWA study examining Montana aggregates concluded that if Micro-Deval loss is more than the recommended cutoff of 18%-24%, then a second evaluation test should be performed before the aggregate is discredited for durability (Cuelho, Mokwa, & Obert, 2007). In a 1998 journal article summarizing the experience with the Micro-Deval test in Ontario, the author states that Micro-Deval limits in Ontario (as of 1998) were 13% for coarse aggregates used in concrete pavement and 17% for coarse aggregates used in structural concrete (Rogers C. , 1998).

2.3.1.2 Los Angeles Abrasion and Impact Test

The Los Angeles abrasion and impact test, also commonly referred to as simply the "LA abrasion test", is currently used by 49 of 50 state DOTs to evaluate aggregates (this is further discussed in Section 3.2 Survey of Other States and Organizations). The NCHRP 4-20C final report recommends this test, but some researchers studies have shown that that although the LA abrasion test correlates well with some other aggregate tests, it does not correlate well with field performance.

Several studies have been conducted to examine which rock properties influence the results of the LA abrasion test. In 2007, Turkish researchers tested 35 different rock types (9 igneous, 11 metamorphic, 15 sedimentary) for LA abrasion and uniaxial compressive strength (UCS), and also classified rocks by porosity. For the UCS testing, rocks were collected and inspected to ensure they had no fractures or defects before being cored to dimensions of 38-mm (1.5-in.) diameter and 76-mm (3.0-in) length, trimmed, and subjected to UCS testing. Regression analysis demonstrated logarithmic relationships, which varied by type of rock and porosity class, between LA abrasion and UCS. Correlation coefficients were highest when grouped by porosity ($R^2 = 0.68$ for porosity <1%, $R^2 = 0.79$ for porosity between 1% and 5%, and $R^2 = 0.75$ for porosity >5%). For relationships based on rock type alone, correlation coefficients were $R^2 = 0.50$ for igneous rock, $R^2 = 0.81$ for metamorphic rocks, and $R^2 = 0.50$ for sedimentary rocks. For all rocks included in the study, the correlation coefficient was $R^2 = 0.63$ relating LA abrasion loss to unconfined compressive strength. Overall, this study demonstrated that the LA abrasion test is dependent upon the strength of the aggregate, porosity, and rock type (Kahraman & Fener, 2007).

Another group of Turkish researchers published the results of a similar project in 2009 that evaluated four limestones, three crystalline marbles, and one andesite to determine correlations between LA abrasion and physical properties such as bulk density, Schmidt hardness, shore hardness, P-wave velocity, and mechanical properties such as uniaxial compressive strength, point load index, and indirect tensile strength of rocks. Results of the LA abrasion test were normalized by dividing by P-wave velocity to account for different porosities, densities, and presence of fractures. Researchers considered correlation coefficients of $R^2 > 0.50$ to be "statistically significant at a 99% confidence level with 10 degrees of freedom". The normalized LA abrasion loss showed highest correlations with compressive strength, tensile strength, Schmidt hardness, and point load index, and showed moderate correlations with bulk density and shore hardness. Thus, this research demonstrated that LA abrasion test results are not only influenced by aggregate strength and porosity, but by hardness and density as well (Ugur, Demirdag, & Yavuz, 2009).

A study with a more narrow scope in 1980 by researchers from Saudi Arabia examined igneous (particularly volcanic and plutonic) rocks and the influence of grain size and absorption capacity on LA abrasion loss. The study found that LA abrasion loss increased linearly with absorption capacity and that fine-grained (grain < 60-μm) igneous rocks were "tougher" (lower LA abrasion loss) than coarse-grained (grain > 2-mm) rocks of the same porosity (Kazi & Al-Mansour, 1980). These three studies suggest that the results of the LA abrasion test depend on, and thus indirectly characterize, absorption, porosity, strength (both tensile and compressive), density, and hardness.

When researchers have attempted to correlate LA abrasion with field performance, they have found little correlation between the two. In his paper explaining the Canadian experience with Micro-Deval testing, Rogers states anecdotally that in Ontario, researchers have noticed that LA abrasion results do not correlate well with field performance, although the test does have the capacity to identify aggregates prone to breakdown during handling (Rogers C. , 1998). The 2005 Wisconsin DOT study found that in their study of 60 aggregates, the LA abrasion test was only able to identify the very worst aggregate sample as being "poor". However, this study did confirm findings of the Turkish researchers that the LA abrasion test can accurately estimate aggregate strength (Weyers, Williamson, Mokarem, Lane, & Cady, 2005). The 2005 study conducted in South Carolina also found that the LA abrasion test was not a good predictor of field performance when they determined that the state specified 55% limit did not do a good of identifying marginal ("fair" and "poor") aggregates in field performance (Rangaraju, Edlinski, & Amirkhanian, 2005).

Despite research showing that the LA abrasion test does not correlate well with field performance, there are a few other common aggregate tests that it does correlate well with. The ICAR 507 study determined that LA abrasion had good correlation with aggregate crushing value ($R^2 = 0.65$) (Fowler, Allen, Lange, & Range, 2006). The Ontario study which explored a variety of aggregate tests found a similar correlation between LA abrasion and aggregate impact value (R^2 = 0.64) (Rogers & Senior, 1991). At least three studies (ICAR 507, FHWA-SC-05-01, and a Southeastern Superpave Center project) have demonstrated that although LA abrasion and Micro-Deval both attempt to characterize abrasion resistance of aggregates, the two tests do not correlate well at all (Rangaraju, Edlinski, & Amirkhanian, 2005), (Fowler, Allen, Lange, & Range, 2006), (Cooley & James, 2003). However, a study conducted on Montana aggregates found that Micro-Deval and LA abrasion actually had good correlation for low loss materials, but discontinuities existed for higher loss materials, causing a lower overall correlation coefficient (R^2 = 0.46) (Cuelho, Mokwa, & Obert, 2007).

Alternative methods to the LA abrasion test have been explored by at least one research study. In 2008, researchers published a report detailing the results of a study that examined the relationships between aggregate type and compressive strength, flexural strength, and abrasion resistance of high strength concrete. Researchers prepared 50-mm x 50-mm x 100-mm (1.97-in. x 1.97-in. x 3.94-in.) prismatic high strength concrete specimens which were saw-cut and placed in the LA abrasion machine without the traditional steel shot. After 28 days of curing, the specimens were subjected to 100 revolutions and 500 revolutions and a loss was measured at each stage. Other tests performed in this study include the traditional LA abrasion test, uniaxial compressive strength of rock, Bohme apparatus abrasion, and compressive and flexural strength of concrete. Results demonstrated that both aggregate strength and texture influenced the compressive strength, flexural strength, and abrasion resistance of high strength concrete. The traditional LA abrasion test had very high correlation (R^2 = 0.95) with uniaxial compressive strength of the aggregate, but interestingly the traditional LA abrasion test had a lower correlation (R^2 = 0.67) with the alternative LA abrasion method using prismatic concrete specimens (Kiliç, et al., 2008).

2.3.2 Soundness and Freeze-Thaw Resistance

Tests recommended by NCHRP 4-20C to characterize soundness and freeze-thaw resistance of aggregates include the magnesium sulfate soundness test, the Canadian unconfined freeze-thaw test, and the Iowa pore index test (Folliard & Smith, 2002).

2.3.2.1 Sulfate Soundness Test

The sulfate soundness of aggregates can be measured by using a magnesium sulfate solution or a sodium sulfate solution. This test involves cycles (typically five) soaking an aggregate in a sulfate solution and then drying the aggregate. This test was originally developed in 1828 to simulate freezing of water in stone before refrigeration was controllable and widely available (Rogers, Bailey, & Price, 1991). The idea was to simulate crystallization pressures of ice formation during freezing and thawing events by causing salt crystals to form during the heating stage of this test (Folliard & Smith, 2002). Of the state DOTs that do use the sulfate soundness test, twenty-eight states specify the use the use of sodium sulfate, nine states specify the use of magnesium sulfate, and two states allow the use of either magnesium sulfate or sodium sulfate. Eleven states do not use sulfate soundness testing. State specifications are further discussed in Section 3.2.1 Coarse Aggregate Specifications.

The most common complaints about the sulfate soundness test are its lack of correlation to field performance, its time of testing (7-10 days), and its high variability (Jayawickrama, Hossain, & Hoare, 2007). Several research studies have confirmed the assertion that the sulfate soundness tests have high variability compared to tests like the Micro-Deval and LA abrasion (Weyers, Williamson, Mokarem, Lane, & Cady, 2005), (Cuelho, Mokwa, & Obert, 2007), (Hossain, Lane, & Schmidt, 2008). Although Rangaraju et al. determined that the two sulfate soundness tests are highly correlated to each other, NCHRP 4-20 only recommends use of the magnesium sulfate soundness test due to the higher variability of the sodium sulfate soundness test (Folliard & Smith, 2002).

Several studies have examined the correlation between the Micro-Deval test and the sulfate soundness tests with mixed results. Rangaraju et al. found no correlation between Micro-Deval and either sulfate soundness test after testing 23 South Carolina aggregates (Rangaraju, Edlinski, & Amirkhanian, 2005). Cooley and James found a very low correlation (R^2 = 0.11) between Micro-Deval and sodium sulfate soundness after testing 72 aggregates in the southeast United States (Cooley & James, 2003). However, Cuelho et al. found that the sodium sulfate test had a good correlation (R^2 = 0.72) with Micro-Deval after testing 32 Montana aggregates (Cuelho, Mokwa, & Obert, 2007). Jayawickrama et al. found a high correlation (R^2 = 0.7) between Micro-Deval and magnesium sulfate soundness for Texas aggregates but said that the correlation wasn't high enough to justify replacing the magnesium sulfate soundness test (Jayawickrama, Hossain, & Hoare, 2007).

2.3.2.2 Canadian Unconfined Freeze-Thaw Test

Although less common than the sulfate soundness test, the Canadian unconfined freeze-thaw test has shown success in its correlation to freeze-thaw damage in PCC pavements and in its high precision (Folliard & Smith, 2002). This test was developed during the 1980s by researchers in Canada as a means to simulate realistic freezing and thawing cycles while the aggregate is exposed to moisture and salts (Rogers, Bailey, & Price, 1991). In the Canadian unconfined freeze-thaw test, three sizes of coarse aggregate are soaked in a 3% sodium chloride solution for 24 hours, then drained and subjected to five cycles of freezing (for 16 hours) and thawing (8 hours), and then re-sieved afterwards. A mass loss is calculated from the final material. Canadian specifications require a mass loss of 6% or less for severe exposure conditions and a mass loss of 10% or less for moderate conditions (Folliard & Smith, 2002).

Several research projects have examined the Canadian unconfined freeze-thaw test. In 1991, researchers from Ontario collected over 100 aggregates from Ontario and subjected them to a variety of tests (this research project is also discussed in Section 2.2 Similar Research Projects). They concluded that limits based on the Canadian unconfined freeze-thaw test and the 24-hour water absorption test was one way of identifying good, fair, and poor performers. Aggregates that had good field performance histories were mostly bounded by a Canadian unconfined freeze-thaw loss of 6% and an absorption capacity of 1.5%. Another boundary of 13% Canadian unconfined freeze-thaw loss and an absorption capacity of 2% was able to identify most aggregates with good or fair performance histories. It should be noted that although these limits captured most good and fair performers, several good and fair performers fell outside these limits as demonstrated by Figure 6, from the final report of this study.

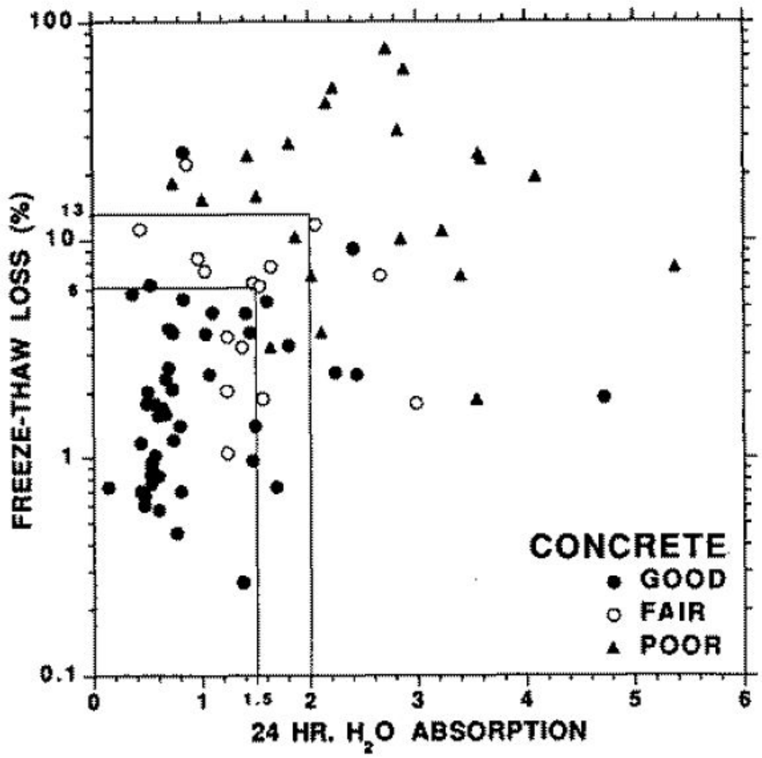

Figure 6: Canadian Unconfined Freeze-Thaw versus Absorption Capacity (Rogers & Senior, 1991)

ICAR 507 was another research study that had similar conclusions to Rogers & Senior's study in Ontario. ICAR 507 collected over 100 aggregate sources from the United States and Canada and subjected these samples to a variety of tests (as discussed in Section 2.2 Similar Research Projects). This study found that the best single test indicator of field performance for all applications was the Micro-Deval test, but the best overall indicator of field performance was the Micro-Deval test used in combination with the Canadian unconfined freeze-thaw test. A Micro-Deval loss of 21%, combined with a Canadian unconfined freeze-thaw loss of 3.6% was able to bound 77% of good performers for PCC applications. Figure 2, located in Section 2.2 Similar Research Projects, graphically displays the bounding of performers by these criteria. The limits of

21% for Micro-Deval and 3.6% for Canadian Freeze-Thaw were also able to exclude 100% of the fair performers and 100% of the poor performers. If the Canadian Freeze-Thaw limit is raised to 6.5% (dashed line), 95% of good performers are included, 50% of the fair performers are included, and 60% of the poor performers are included. However, it is important to note that only 4 fair performers and 5 poor performers were subjected to testing. The researchers acknowledged that further research of more marginal performers may be needed to refine these limits. This study found little correlation (R^2 = 0.39) between the Canadian unconfined freeze-thaw test and the magnesium sulfate soundness test (Fowler, Allen, Lange, & Range, 2006).

The Wisconsin DOT study, also discussed in Section 2.2 Similar Research Projects, examined 60 Wisconsin aggregates and determined that the recommended limit of 10% for Canadian unconfined freeze-thaw rejected too many aggregates. They instead proposed a 15% limit, which would ensure that only very non-durable aggregates were rejected by the limit. They also found that the Canadian unconfined freeze-thaw test had no correlation with the sodium sulfate soundness test (Weyers, Williamson, Mokarem, Lane, & Cady, 2005).

2.3.2.3 Iowa Pore Index

The Iowa pore index test was developed as means of identifying aggregates susceptible to freeze-thaw damage, particularly D-cracking behavior. This test uses a pressurized vessel to quantify the amount of macropores and micropores in an aggregate. Theoretically, a higher volume of micropores indicates that an aggregate will be more susceptible to D-cracking. When running the Iowa pore index test, a 9-kg (19.8-lb) sample of dried aggregate is placed in a pressure vessel along with water and then a 241-kPa (35.0-psi) pressure is applied. The amount of water absorbed by the aggregate under this pressure in the first minute is the primary load, an attempt at quantifying macropore volume. The amount of water absorbed by the aggregate at this pressure for the next 14 minutes is the secondary load, which is indicative of micropore volume. Iowa DOT places a 27-mL limit on the secondary load to avoid aggregates prone to D-cracking (Folliard & Smith, 2002). Figure 7, from a Michigan DOT report, demonstrates sample water uptake rates for various aggregates from a river gravel sample during the Iowa pore index test.

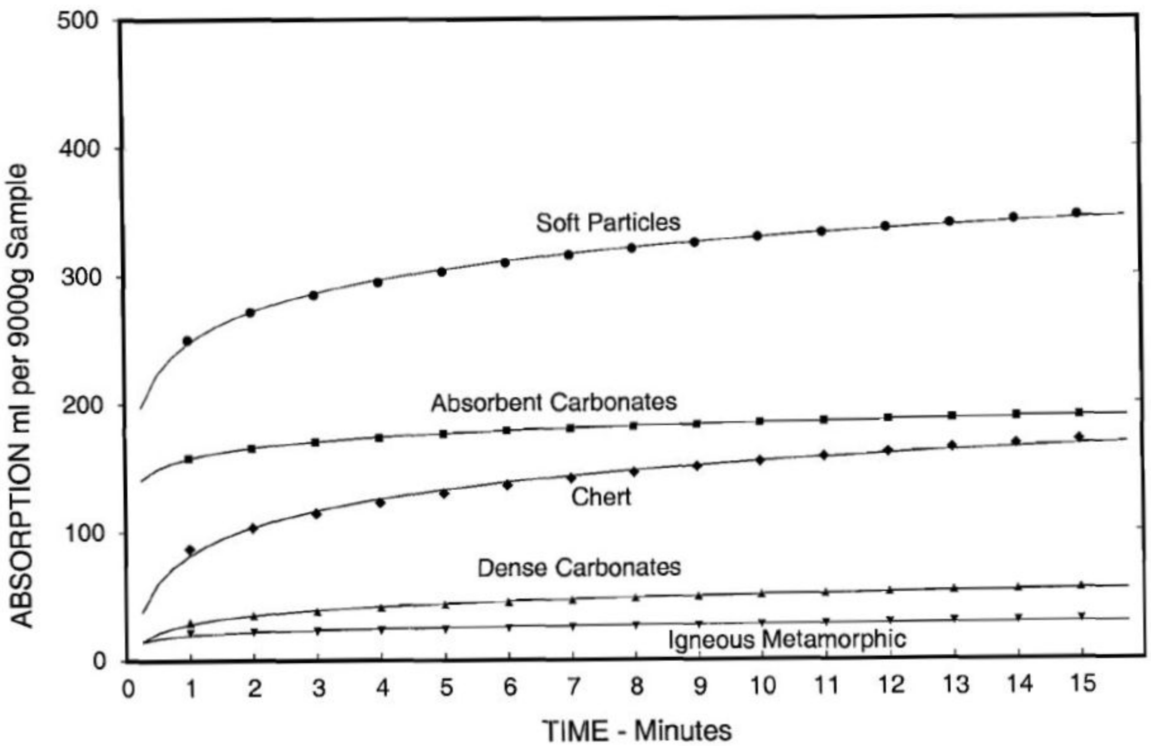

Figure 7: Water Uptake Rates for Various Aggregates in Iowa Pore Index Test (Muethel, 2007)

A study performed by researchers at The University of Minnesota in the late 1990s examined test methods for better characterizing freeze-thaw durability of Minnesota aggregates for concrete applications (discussed in Section 2.2 Similar

Research Projects). Results of this research project demonstrated that the Iowa pore index test was correlated with field performance, though not as well as other tests such as modified ASTM C 666 (procedure B), VPI single-cycle slow freeze test, and the hydraulic fracture test (Koubaa & Snyder, 1999).

2.3.3 Strength and Impact Resistance

Strength and impact resistance of aggregates is important for not only high strength concrete applications, when compression failure occurs due to aggregate fracture, but also for handling and transportation of aggregate as well. Perhaps the only realistic way to directly measure aggregate strength is through the unconfined compressive strength (UCS) testing of rock, however this method often yields highly variable results and is not always possible for river gravels which can have small maximum particle sizes unfriendly for coring devices (Folliard & Smith, 2002). The Los Angeles abrasion and impact test, British aggregate crushing value (ACV), and the British aggregate impact value (AIV) are all tests that indirectly measure aggregate strength and/or impact resistance. It is important to note that while some minimal level of integrity and strength is important for aggregates, concrete strength may not be highly important to all PCC applications. A TxDOT research project demonstrated that concrete strength had negligible correlation with performance of continuously reinforced concrete pavements (Won, 2001).

2.3.3.1 Los Angeles Abrasion and Impact Test

The Los Angeles Abrasion and Impact Test, also commonly referred to as simply the “LA abrasion test”, is currently used by 49 of 50 state DOTs to evaluate aggregates (this is further discussed in Section 3.2 Survey of Other States and Organizations). The NCHRP 4-20 final report recommends this test as an abrasion test, but some research studies have shown the LA abrasion test correlates well with aggregate tests which measure strength and impact resistance. ASTM International added “impact” to the test’s title, acknowledging the role of steel shot and aggregate impact in the test’s results. However, research has also shown that this does not correlate well with field performance.

Several studies have been conducted to examine which rock properties influence the results of the LA abrasion test. In 2007, Turkish researchers tested 35 different rock types (9 igneous, 11 metamorphic, 15 sedimentary) for LA abrasion and uniaxial compressive strength (UCS), and also classified rocks by porosity. For the UCS testing, rocks were collected and inspected to ensure they had no fractures or defects before being cored to dimensions of 38-mm (1.5-in.) diameter and 76-mm (3.0-in) length, trimmed, and subjected to UCS testing. Regression analysis demonstrated logarithmic relationships, which varied by type of rock and porosity class, between LA abrasion and UCS. Correlation coefficients were highest when grouped by porosity (R^2 = 0.68 for porosity <1%, R^2 = 0.79 for porosity between 1% and 5%, and R^2 = 0.75 for porosity >5%). For relationships based on rock type alone, correlation coefficients were R^2 = 0.50 for igneous rock, R^2 = 0.81 for metamorphic rocks, and R^2 = 0.50 for sedimentary rocks. For all rocks included in the study, the correlation coefficient was R^2 = 0.63 relating LA abrasion loss to unconfined compressive strength. Overall, this study demonstrates that the LA abrasion test is dependent upon the strength of the aggregate, porosity, and rock type (Kahraman & Fener, 2007).

Another group of Turkish researchers published the results of a similar project in 2009 that evaluated four limestones, three crystalline marbles, and one andesite to determine correlations between LA abrasion and physical properties such as bulk density, Schmidt hardness, shore hardness, P-wave velocity, and mechanical properties such as uniaxial compressive strength, point load index, and indirect tensile strength of rocks. Results of the LA abrasion test were normalized by dividing by P-wave velocity to account for different porosities, densities, and presence of fractures. Researchers considered correlation coefficients of $R^2 > 0.50$ to be "statistically significant at a 99% confidence level with 10 degrees of freedom". The normalized LA abrasion loss showed highest correlations with compressive strength, tensile strength, Schmidt hardness, and point load index, and showed moderate correlations with bulk density and shore hardness. Thus, this research demonstrated that LA abrasion test results are not only influenced by aggregate strength and porosity, but by hardness and density as well (Ugur, Demirdag, & Yavuz, 2009).

A study with a more narrow scope in 1980 by researchers from Saudi Arabia examined igneous (particularly volcanic and plutonic) rocks and the influence of grain size and absorption capacity on LA abrasion loss. The study found that LA abrasion loss increased linearly with absorption capacity and that fine-grained (grain < 60-μm) igneous rocks were "tougher" (lower LA abrasion loss) than coarse-grained (grain > 2-mm) rocks of the same porosity (Kazi & Al-Mansour, 1980). These three studies suggest that the results of the LA abrasion test depend on, and thus indirectly characterize: absorption, porosity, strength (both tensile and compressive), density, and hardness.

When researchers have attempted to correlate LA abrasion with field performance, they have found little correlation between the two. In his paper explaining the Canadian experience with Micro-Deval testing, Rogers states anecdotally that in Ontario, researchers have noticed that LA abrasion results do not correlate well with field performance, although the test does have the capacity to identify aggregates prone to breakdown during handling (Rogers C. , 1998). The 2005 Wisconsin DOT study found that in their study of 60 aggregates, the LA abrasion test was only able to identify the very worst aggregate sample as being “poor”. However, this study did confirm findings of the Turkish researchers that the LA abrasion test can accurately estimate aggregate strength (Weyers, Williamson, Mokarem, Lane, & Cady, 2005). The 2005 study conducted in South Carolina also found that the LA abrasion test was not a good predictor of field performance when they determined that the state specified 55% limit did not do a good of identifying marginal (“fair” and “poor”) aggregates in field performance (Rangaraju, Edlinski, & Amirkhanian, 2005).

Despite research showing that the LA abrasion test does not correlate well with field performance, there are a few other common aggregate tests that it does correlate well with. The ICAR 507 study determined that LA abrasion had good correlation with aggregate crushing value (R^2 = 0.65) (Fowler, Allen, Lange, & Range, 2006). The Ontario study, which explored a variety of aggregate tests, found a similar correlation between LA abrasion and aggregate impact value (R^2 = 0.64) (Rogers & Senior, 1991). At least three studies (ICAR 507, FHWA-SC-05-01, and a Southeastern Superpave Center project) have demonstrated that although LA abrasion and Micro-Deval both attempt to characterize abrasion resistance of aggregates, the two tests do not correlate well at all (Rangaraju, Edlinski, & Amirkhanian, 2005), (Fowler, Allen, Lange, & Range, 2006), (Cooley & James, 2003). However, a study conducted on Montana aggregates found that Micro-Deval and LA abrasion actually had good correlation for low loss materials, but discontinuities existed for higher loss materials, causing a lower overall correlation coefficient (R^2 = 0.46) (Cuelho, Mokwa, & Obert, 2007).

Alternative methods to the LA abrasion test have been explored by at least one research study. In 2008, researchers published a report detailing the results of a study that examined the relationships between aggregate type and compressive strength, flexural strength, and abrasion resistance of high strength concrete. Researchers prepared

50-mm x 50-mm x 100-mm (1.97-in. x 1.97-in. x 3.94-in.) prismatic high strength concrete specimens which were saw-cut and placed in the LA abrasion machine without the traditional steel shot. After 28 days of curing, the specimens were subjected to 100 revolutions and 500 revolutions and a loss was measured at each stage. Other tests performed in this study include the traditional LA abrasion test, uniaxial compressive strength of rock, Bohme apparatus abrasion, and compressive and flexural strength of concrete. Results demonstrated that both aggregate strength and texture influenced the compressive strength, flexural strength, and abrasion resistance of high strength concrete. The traditional LA abrasion test had very high correlation (R^2 = 0.95) with uniaxial compressive strength of the aggregate, but interestingly the traditional LA abrasion test had a lower correlation (R^2 = 0.67) with the alternative LA abrasion method using prismatic concrete specimens (Kiliç, et al., 2008).

2.3.3.2 Aggregate Crushing Value

The aggregate crushing value (ACV) test is a British standard that subjects an aggregate sample to confined compression using a machine typically used for compression testing of concrete cylinders. Approximately 2500-g (5.5-lb) of aggregate passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve is placed in a cylindrical containment apparatus consisting of steel plates and a steel ring. The aggregate is compressed for approximately 10-min. at a constant load rate, until the force has reached 400-kN (90,000-lb). This test procedure is described further in Section 5.1.6 Aggregate Crushing Value.

Of the literature reviewed for this project that has discussed the use of the aggregate crushing value, none have ultimately recommended the aggregate crushing value as a means of predicting or identifying field performance of aggregates. Perhaps the strength of aggregate is less crucial for typical DOT applications (pavements, bridges, etc.) than the integrity of the overall concrete mixture. Regardless, at least one study found that the ACV test correlated well (R^2 = 0.65) with the LA abrasion test (Fowler, Allen, Lange, & Range, 2006). However the LA abrasion test has itself been criticized for lack of correlation with field performance. The Wisconsin DOT study (discussed in Section 2.2 Similar Research Projects) concluded that there was no reason to change the LA abrasion requirement to an ACV requirement since they are highly correlated and appear to measure the same property (Weyers, Williamson, Mokarem, Lane, & Cady, 2005).

2.3.3.3 Aggregate Impact Value

The aggregate impact value (AIV) is a British standard that subjects a confined aggregate sample to a falling impact load. A sample of aggregate, passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve, of approximately 500-g (1.1-lb) in mass (depending on the density) is placed in a steel cup 38-cm (15-in.) below a steel hammer the same diameter as the inside of the cup. The user drops the steel hammer, guided by vertical rods, and raises it 15 times and the sample is sieved over a No. 8 (2.36-mm) sieve. The mass loss is recorded and the final result of the test is mass loss by percentage of original mass. The test procedure is further described in Section 5.1.5 Aggregate Impact Value.

Researchers in Ontario studied this test in the early 1990s but concluded that it had little correlation to field performance. They did determine that the AIV test had a good correlation (R^2 = 0.64) with the LA abrasion test (Rogers & Senior, 1991).

2.3.4 Absorption

Absorption capacity of aggregate is an important physical property as aggregates with more absorptive potential tend to be more porous and are thus typically weaker, less durable, and more prone to freeze-thaw damage. At least one state DOT, Minnesota, limits absorption capacity for aggregates used in portland cement concrete applications (Folliard & Smith, 2002).

2.3.4.1 Specific Gravity and Absorption of Coarse Aggregate

The most common procedure for testing coarse aggregate absorption is AASHTO T 85 (*Specific Gravity and Absorption of Coarse Aggregate*). In this test procedure, the user submerges a sample of aggregate in water for at least 15 hours and then removes it from water and dries all water from the surface of all the particles with a cloth or towel. When no free water can be observed on the particles' surface, it is considered to be at a "saturated surface dry" state, meaning internal pores are still occupied with water, but no water remains at the surface of the aggregate. The user records the saturated surface dry weight of the aggregate and places the sample in the oven to dry. When the sample has a consistent mass (completely dry) the user records the dry mass of the sample and the by percentage change in mass represents the aggregate's absorption capacity.

Many studies have determined that absorption capacity is an important parameter, but is often too variable to justify prescriptive limits. Rogers shows in Figure 6, in Section 2.3.2.2 Canadian Unconfined Freeze-Thaw Test, that lower absorption capacities and lower Canadian unconfined freeze-thaw values tend to signify aggregates with better performance histories. A Minnesota DOT study also found that absorption capacity of aggregates correlated to field performance, though the correlation was not as significant as other tests such as ASTM C 666 (Procedure B), VPI single-cycle slow freeze test, and the Washington hydraulic fracture test (Koubaa & Snyder, 1999). The Wisconsin DOT study discussed in Section 2.2 Similar Research Projects also includes absorption capacity as a decision parameter in its recommended aggregate selection flowchart shown in Figure 4.

2.3.5 Shape Characteristics – Shape, Angularity, and Texture

Shape characteristics of aggregates can influence workability, water demand, shrinkage, and strength of concrete mixtures. Literature has also shown that aggregate shape properties influence yield stress and modulus of elasticity of early age concrete, particularly when the aggregate elastic/viscous properties differ significantly from the cement (Mahmoud, Gates, Masad, Erdoğan, & Garboczi, 2010). Aggregates that are more cubical or angular in shape tend to require more water to achieve the same slump or workability. Aggregates with a rougher texture can improve bond strength at the aggregate-mortar interface leading to stronger concrete due to macroscopic mechanical adhesion (Folliard & Smith, 2002). There are traditional methods to determine shape characteristics, such as proportional calipers, which may warn of aggregates with flat and elongated particles, but newer methods, such as the Aggregate Imaging System (AIMS), are evolving to take advantage of current technologies to provide more data regarding these important shape characteristics.

2.3.5.1 Proportional Caliper for Determining Flat & Elongated Particles

Many state DOTs use a proportional caliper to manually measure particles in an aggregate sample and compare the maximum dimensions of a particle to its minimum dimension. An aggregate possessing an excess of particles above a 4:1 dimension ratio may lead to workability issues during concrete placement (Folliard & Smith, 2002). Some states incorporate this testing as a means of finding aggregates susceptible to breakage during compaction of HMA mixes. The most common standard of this test is

ASTM D 4791 and the Texas DOT version of this test, Tex-280-F is described in Section 5.1.8 Flat and Elongated Particles. A picture of the proportional caliper device can be seen in Figure 24, in Section 5.1.8 Flat and Elongated Particles.

2.3.5.2 Aggregate Imaging System (AIMS 2.0)

The aggregate imaging system (AIMS) is a machine consisting of a camera, lights, computer software, and movable trays, designed to capture and analyze the shape, angularity, and texture of coarse aggregates and the form and angularity of fine aggregates. The camera captures images of the aggregate particles , either lit directly or backlit, and the software analyzes these images and provides the user with data summarizing the shape characteristics. For coarse aggregates, the user places a set of aggregate particles on a transparent tray and places this tray into the machine. Figure 51, in Chapter 6: Discussion and Analysis, displays the aggregates placed on trays for testing and Figure 23, in Section 5.1.7 Aggregate Imaging System (AIMS 2.0), displays the AIMS machine. The AIMS machine rotates the tray three times to capture images of each particle. The first image captured is a backlit such that only the aggregates two-dimensional shape is captured and this image is analyzed by the fundamental gradient method. The second image captured requires the camera to focus on the particle, thus allowing a particle height (as a function of focal length) to be determined, and three-dimensional analysis to be realized. The third image captures a close-up surface texture of the aggregate particles which is analyzed by the wavelet method. The data are output in a spreadsheet file that includes a distribution of shape characteristics. Specific algorithms and analysis techniques used by the machine and software are described by the developer, Eyad Masad, and are available through the Texas Transportation Institute (Masad, 2005).

In order to determine the repeatability and accuracy of the AIMS machine, researchers from Texas A&M (including the developer himself) conducted a study which examined 500 particles and the effects of multiple operators and machines, as well as comparing AIMS results to results obtained from X-ray computed tomography and a digital caliper. In this study, AIMS was found to have a coefficient of variance of 11% for any give source (single operator) and a coefficient of variance of 5% for the same set of aggregates (single operator). Mahmoud et al. concluded that the effects of random placement of the same aggregate particles in different orientations had minimal effects on the angularity. In experiments with two AIMS machines (single operator) and multiple operators (single AIMS machine), the angularity measurements were found to be highly correlated (R^2 = 0.97 and R^2 = 0.98 respectively) as were texture measurements (R^2 = 0.97 and R^2 = 0.92), however slope and intercept for texture was not 1 and 0 (Mahmoud, Gates, Masad, Erdoğan, & Garboczi, 2010). Thus, it seems that the texture measurement is not as repeatable as the angularity measurement.

In the same study, Mahmoud et al. (2009) determined that length, thickness, and width dimensions as measured by AIMS correlated very well with both the X-ray computed tomography (R^2 = 0.96, 0.84, 0.91) and digital caliper (R^2 = 0.96, 0.81, N/A). However, AIMS underestimated those dimensions by about 10% compared to X-ray computed tomography. This effect was mostly cancelled out when overall ratios were computed (Mahmoud, Gates, Masad, Erdoğan, & Garboczi, 2010).

2.3.6 Thermal Properties

Thermal properties are important in understanding and predicting concrete behavior for applications such as continuously reinforced concrete pavements (CRCP) or mass pours. If thermal properties are not thoroughly understood or accounted for, thermal cracking may occur. Thermal properties are also input parameters for some computer programs such as *ConcreteWorks*.

2.3.6.1 Thermal Conductivity

The current method to measure thermal conductivity is ASTM C 177-04, however it requires a specimen in the shape of a large rectangular prism so the preparation of a field or lab sample is time-intensive. This method is also not recommended for highly heterogeneous materials and the varying mix designs of pavements means that ASTM C 177 may not be applicable or reliable (Carlson, Bhardwaj, Phelan, Kaloush, & Golden, 2010).

Acknowledging this problem, researchers from Arizona State University developed a new test method for thermal conductivity using cylindrical specimen geometry. Researchers sought to take advantage of the fact that concrete cylinders are either casted or cored on a regular basis as a means of quality control for mechanical properties. In their experimental setup, researchers drilled a 3/8-in. (9.5-mm) hole through a 4-in. (102-mm) by 6-in. (152-mm) concrete cylinder and placed a 3/8-in (9.5-mm). by 6-in. (152-mm) heating element in the hole. The top and bottom of the concrete cylinder was insulated as shown in Figure 8. Thermocouples placed on the inner and outer walls of the concrete cylinder allowed for measurement of temperature as heat diffused through the specimen (Carlson, Bhardwaj, Phelan, Kaloush, & Golden, 2010).

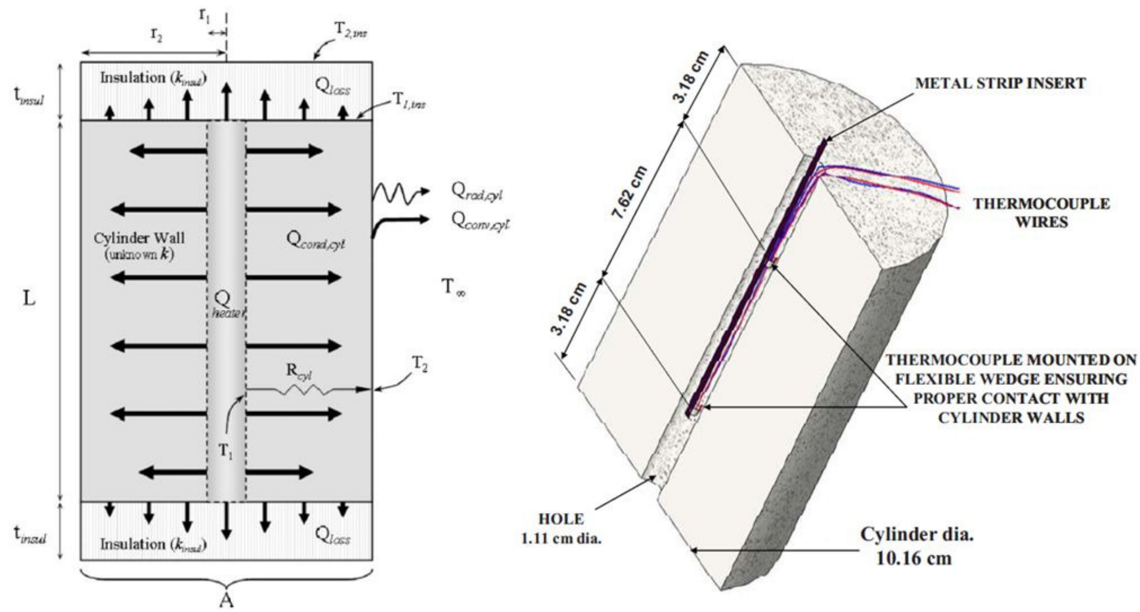

Figure 8: Test Setup for Thermal Conductivity Testing of a Cylindrical Specimen (Carlson, Bhardwaj, Phelan, Kaloush, & Golden, 2010)

After acquiring necessary data, fundamental heat transfer theory was used to determine the thermal conductivity of the specimen. The use of a sample with thickness larger than twice the maximum aggregate size prevents thermal bridging which would skew results. Thus, this method is able to more accurately characterize the composite nature of concrete.

Carlson et al. (2010) validated the accuracy of this test setup by using an ultrahigh molecular weight polyethylene (UHMWPE) with known thermal conductivity. Researchers also prepared an HMA cylindrical specimen and extracted a concrete core from an interstate in Arizona to test. Results compared favorably with values for HMA and concrete in the literature. The portland cement concrete cylinder test had higher variation than the HMA specimen, possibly due to the larger aggregate size. Overall, this test method shows potential for future use of determining thermal conductivity using cylindrical concrete specimens (Carlson, Bhardwaj, Phelan, Kaloush, & Golden, 2010).

2.3.6.2 Coefficient of Thermal Expansion (CoTE)

Coefficient of thermal expansion (CoTE) is a very difficult variable to measure for coarse aggregates. Linear displacement methods are not applicable due to the wide variety of shapes and sizes of aggregates, and volumetric measurements such as the dilatometer often have unacceptably high noise compared to the precision required to measure aggregate expansion under temperature change. At least one TxDOT research project attempted to back-calculate CoTE values of aggregate from concrete CoTE tests (Du & Lukefahr, 2007), and another research project attempted to use dilatometry to measure CoTE of aggregate (Mukhopadhyay & Zollinger, 2009). The current standard for measuring CoTE of a concrete specimen is AASHTO TP 60-00, which requires a core or cylinder 7-in. (178-mm) long and 4-in. (102-mm) in diameter, and it does not directly account for the aggregate CoTE.

Current AASHTO pavement design procedures ignore type of coarse aggregate selected. TxDOT researchers have found that coarse aggregates can significantly impact pavement performance and service life when other variables are held constant (Won, 2001). Spalling and wide/irregular cracks cause the most problems in Texas concrete pavements, particularly in continuously reinforced concrete pavements (CRCP). The AASHTO Road Test design procedures do not consider these modes of failure but rather emphasize fatigue cracking as the main failure mode of concern. According to at least one TxDOT researcher, thermal expansion of concrete (along with modulus of elasticity, drying shrinkage, and bond strength between aggregate and mortar) is a property that should be considered in pavement design and is significantly influenced by type of coarse aggregate used. Significantly different performance levels have been observed in Texas on the same roadways (thus same traffic loads and same environmental conditions) when different coarse aggregate is used in different sections. For example, the frontage road of

Beltway 8 in Houston used a crushed limestone aggregate in one section and siliceous river gravel in another section and the latter section has experienced major spalling. Another example is the frontage road of IH-610, also in Houston. This road used a lightweight aggregate in one section and siliceous river gravel in another section and again severe spalling has been observed in the latter case (Won, 2001).

Research continued at TxDOT to evaluate the coefficient of thermal expansion of 94 concrete mixtures where the coarse aggregate (41 siliceous gravels, 44 limestone, 3 dolomite, 1 mixed limestone and dolomite, 1 siliceous rhyolite, 1 blended limestone and siliceous gravel, 1 siliceous/limestone gravel, 1 siliceous sandstone, and 1 lightweight aggregate) was the only variable parameter. The mix design was a TxDOT class P pavement mixture with Type I cement, no supplementary cementitious materials, no admixtures, aggregate resieved to a TxDOT grade 5, and a control fine aggregate sieved to a TxDOT grade 1. CoTE values were back-calculated using a formula proposed by Emanuel and Hulsey. TxDOT used Tx-428-A as their testing standard (modified version of AASHTO TP 60) and compared results to a Federal Highway Administration (FHWA) Long Term Pavement Performance (LTPP) program which used AASHTO TP 60-00 on concrete cores from Texas highways. TxDOT researchers determined that concrete made with siliceous river gravel as coarse aggregate had, on average, 30% higher CoTE than did concrete made with limestone coarse aggregate. However, some overlap was observed as not all siliceous gravel had higher CoTE than limestone (Du & Lukefahr, 2007). Figure 9 displays the results of this testing.

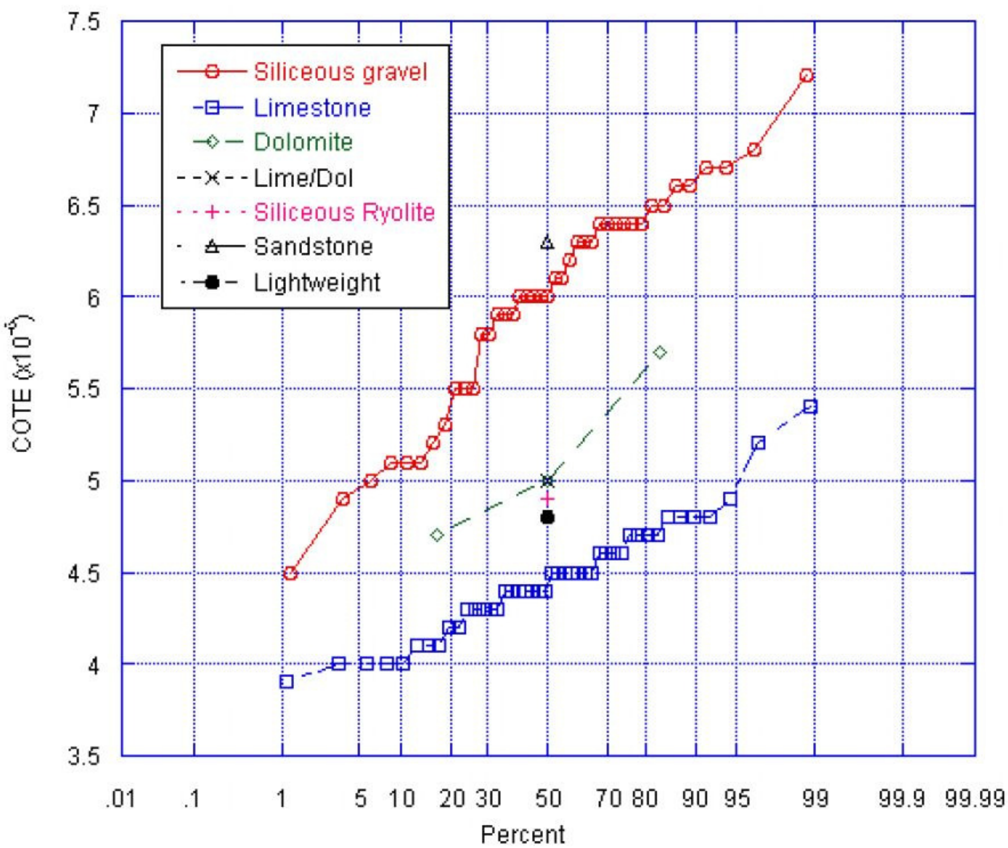

Figure 9: Probability Plot of Coefficient of Thermal Expansion Sorted by Aggregate Type (Du & Lukefahr, 2007)

As part of the same study, researchers found no significant correlation between CoTE and specific gravity, or between CoTE and dynamic modulus (3, 7, 14, 28 days), or between CoTE and compressive strength (3, 7, 14, 28 days). Comparisons between TxDOT CoTE values and FHWA CoTE values showed some discrepancies. However, it is important to note that TxDOT tested 94 specimens at known ages using a controlled mix design, whereas FHWA tested 182 specimens which were cores of unknown age and unknown mix proportions. For the limestone concrete specimens and siliceous river gravel specimens, TxDOT results showed smaller CoTE values and less variation than the FHWA tests results. Although both TxDOT and FHWA samples were saturated during CoTE testing so that age should not play a significant role (demonstrated by previous research conducted by Emanuel and Hulsey (1977)), it is possible that different aggregate sources and varying paste volumes were the primary causes between the discrepancies in TxDOT and FHWA data. The ultimate conclusion of this research study was that back-calculating a CoTE value for coarse aggregates may be helpful for state agencies to identify aggregates prone to early-age cracking (Du & Lukefahr, 2007).

Mukhopadhyay and Zollinger (2009) believe they have developed a test method to measure CoTE of aggregate using dilatometry. The dilatometry method is advantageous because it can measure CoTE of aggregates as-received and thus is significantly quicker (can be finished within 24 hours) than other methods and it may even be possible to use this as a quality control monitoring tool to monitor aggregate source variability (similar to Jayawickrama's proposal for use of the Micro-Deval as a project level quality control too). The new dilatometer test method developed by these researchers uses a stainless steel container, a brass lid, a glass float (to which an LVDT calibrated to 1/100-mm or 0.0004-in. is attached), a thermocouple, and a data acquisition system. Change in temperature of a water bath from 10˚C (50˚F) to 50˚C (122˚F) causes a change in water level due to thermal expansion of the tested material, water, and dilatometer container. Simple physics equations can be used to back-calculate the CoTE of the material tested, in this case aggregate.

Researchers validated results of this dilatometry testing by measuring CoTE for known materials such as steel alloys and comparing results to CoTE obtained from strain gauge based measurements and to literature. Researchers also compared tested aggregate (2 gravels, 2 limestones, 2 sandstones, and a granite) CoTE values to literature for validation. Researchers recommend that for homogenous aggregates, the average CoTE value from four heating and cooling cycles be taken to yield an acceptably low coefficient of variance (≤10%). For heterogeneous aggregates such as river gravel, researchers recommend running at least two runs (each with four heating and cooling cycles) with different samples to achieve the same low level of variance (Mukhopadhyay & Zollinger, 2009).

2.3.7 Mineralogical and Chemical Composition

Knowing the mineralogical and chemical composition of an aggregate can help engineers and scientists identify potential problems in concrete aggregates such as alkali-aggregate reaction, presence of detrimental clays and mica, shrinkage and thermal issues, and overly weathered material (Folliard & Smith, 2002). Keeping track of aggregate mineralogical composition can also help DOTs track and identify trends with good-performing and poor-performing aggregates. The two most common methods of identifying mineralogical and chemical composition are petrographic analysis and X-ray diffraction.

2.3.7.1 Petrographic Examination

Petrographic examination is typically performed by a trained and experienced petrographer, knowledgeable of local geology, following ASTM C 295 (*A Guide for Petrographic Examination of Aggregates for Concrete*). As part of the petrographic examination, petrographers identify key constituents and proportions of an aggregate sample using a variety of tools including microscopes, cameras, polishing/grinding wheels, and a variety of hand tools. Table 6, taken from the NCHRP 4-20C final report, displays the potential benefits of performing petrographic analysis on concrete aggregates. In addition to the benefits listed in Table 6, researchers from Ontario also determined that petrographic examination would be useful in identifying weak and weathered material (Rogers & Senior, 1991).

Table 6: Potential Benefits of Petrographic Analysis (Folliard & Smith, 2002)

Some Potential Benefits of Petrographic Analysis - from Folliard & Smith (2002)
Identification of minerals with potential for **ASR** or **ACR**
Estimation of **mica** content (from point count) in given size fraction of fine aggregate, especially relevant when analyzing material retained on the #200 Sieve (Rogers, 2002). Excessive mica contents (> 10% in specific size fraction) may lead to workability problems in fresh concrete, including increased water demand, segregation, and bleeding. Other materials that may adversely affect workability, such as muscovite, can also be identified.
Assessment of minerals and structure of **carbonate aggregates**, which may provide index of durability and soundness. A petrographic technique has been proposed to generate a petrographic number (PN), which has been reportedly linked to the **durability** of carbonate aggregates (Oyen et al., 1998)
Assessment of **thermal** and **shrinkage** potential, based on identifying the type and amount of minerals present in a given aggregate (Meininger, 1998)
Assessment of aggregates **surface texture** and **mineralogy**, which can be related to **bonding** with mortar
Development of petrographic database for aggregate sources by state DOTs to allow for correlation with aggregate type, source, and mineralogy to PCC pavement performance. This is essential in developing and maintaining field service records linking aggregate sources to PCC pavement performance (Meininger, 1998).

2.3.7.2 X-Ray Diffraction (XRD)

X-ray diffraction (XRD) is an advanced analysis technique that sometimes requires the material of interest to be ground to a fine powder (depending on the exact analysis method). A diffractometer, which fires an incident X-ray beam at the sample and receives the scattered beam, is used to gather data. Output from X-ray diffraction appears as a plot of scattering intensity versus scattering angle. From this plot, peaks can be identified which correspond to individual material components of the sample.

X-ray diffraction can be particularly useful for identifying deleterious clays as demonstrated by one study at the University of Wisconsin. Munoz et al. (2005) explored the impact of clay-coatings on concrete. Past research has shown that when an excess amount of clay is present in a concrete mixture (due to "dirty" coarse or fine aggregates), the water demand may be increased and the pozzolanic hydration products may be altered. These researchers were interested in exploring this topic further in this project.

Aggregates were cleaned and the then recoated with three known clay types. Concrete was mixed and researchers performed XRD and scanning electron microscopic (SEM) analysis to examine hydration products. Munoz et al. (2005) concluded that when aggregates that have clay-coatings are used in concrete mixtures, some of the clay will be dissolved by the water in the mixture but some of the clay will remain adhered to the coarse aggregate. The amount of clay that is dissolved in water or remains adhered to the aggregate depends on the type of clay. They also found that clays do have an impact on rate of hydration. Whether the rate of hydration is increased or decreased also depends on the type of clay: “The clay with macroscopic swelling (Na-montmorillonite) is the most difficult to detach and decreases the rate of hydration. Clays with crystalline swelling (Ca-montmorillonite) and no swelling (Kaolin) are easier to detach and increase the hydration reaction of cement pastes” (Munoz, Tejedor, Anderson, & Cramer, 2005). These researchers also used XRD analysis to examine the influence of microfines on concrete properties as discussed in Section 2.3.8 Presence of Microfines.

2.3.8 Presence of Microfines

Microfines are typically considered material finer than the No. 200 sieve (75-μm) and are usually classified as very fine particles of a parent rock, as opposed to clay. An excess of microfines can lead to decreased finishabiltiy of fresh concrete, air entrainment problems, and an increase in water demand, which can cause in increase in water to cement ratio, which can lead to reduced strength and increased drying shrinkage (Folliard & Smith, 2002). Microfines can either be found attached to the aggregate (due to handling, dust of fracture, etc.) or can be generated during mixing. Although excess microfines can be detrimental to concrete performance, calculated replacement of cement with microfines can be beneficial.

In the past few years, researchers at the University of Wisconsin explored the effects of microfines on fresh and hardened concrete properties. Munoz et al. identified ten aggregates from the state of Wisconsin that were suspected to have microfine coatings. After XRD analysis, three aggregates were selected (as representative) to be studied more in depth. Ten concrete mixes were created: “original coated aggregate series”, “washed aggregate series”, and “artificial coated aggregate series”. The P200 (ASTM C 117) test, California cleanness value, methylene blue value (MBV), and modified methylene blue value (product of P200 and MBV) were compared and correlated to concrete tests. Researchers concluded that even when microfines are present in amount under 1.5%, they influence fresh and hardened concrete properties, the extent of influence depending on nature and amount of microfines. For carbonate microfines, a small difference (< 0.2%) in cleaned and as-received aggregate did not influence slump or shrinkage, but did slightly improve tensile strength of the concrete. Coatings classified as “clay/carbonate” did not influence shrinkage or freeze-thaw durability, but did decrease slump and increase tensile strength. Coatings classified as “dust” or “clay/dust” both decreased slump and increased shrinkage, but did not influence concrete strength or freeze-thaw durability. Due to its absorptive nature, “clay” coatings had the most dramatic influence on concrete properties. These coatings decreased slump and increased shrinkage dramatically. The addition of water to maintain workability also caused a decrease in tensile strength and freeze-thaw durability. Perhaps the most important conclusion from this study is that quantifying the amount microfines alone (as currently quantified through ASTM C 117) is not a good enough measure for deleterious material. This study showed that the type of microfines is just as important as the quantity. Researchers also determined that because it identifies quantity of clay, the modified methylene blue value had the best correlation (out of the other microfine tests) with compressive strength and durability tests (Munoz, Gullerud, Cramer, Tejedor, & Anderson, 2010).

Rached et al. (2009) also examined the effects of microfines (specifically from limestone and granite sources) on concrete properties, with the objective of reducing cement content through replacement with microfines. Mortar mixes of three fine aggregates demonstrated that workability depends on paste volume, paste composition, and type of aggregate used. Shape and gradation of the fine aggregates affected workability of the mortar mixes. As expected, aggregates with higher angularity resulted in "increased paste volume and HRWRA demand. Aggregates with coarser grading generally required lower HRWRA demand but required higher paste volume to ensure adequate cohesiveness." (Rached, De Moya, & Fowler, 2009) Overall, Rached, De Moya, and Fowler determined that replacement of cement with microfines (up to 30%) was able to improve compressive strength, shrinkage, permeability, and abrasion resistance (Rached, De Moya, & Fowler, 2009).

2.4 Fine Aggregate Tests

2.4.1 Abrasion Resistance

Abrasion resistance of aggregates is an important property of aggregates, particularly for fine aggregates. Fine aggregates are more susceptible to mechanical breakdown during handling and mixing, which can lead to production of excess fines. Fine aggregates also provide surface texture for concrete, which is critical for applications where friction is necessary, such as area subject to direct traffic. Fine aggregates must be resistant to abrasion to provide the necessary friction.

2.4.1.1 Micro-Deval Test for Fine Aggregates

The Micro-Deval test for fine aggregates is very similar to the Micro-Deval test for coarse aggregates. A standard gradation of sand, ranging from No. 8 (2.36-mm) to No. 200 (75-µm), is placed in the Micro-Deval container along with steel charge and soaked for one hour prior to testing. The Micro-Deval container is then rotated for 15 min at approximately 100 revolutions per minute. The sample material is then removed and washed over a No. 200 (75-µm) sieve. The remaining material is oven-dried and weighed and the relative amount of material lost, as a percentage, signifies the final Micro-Deval loss.

The Ministry of Transportation in Ontario has studied Micro-Deval abrasion of fine aggregates in addition to Micro-Deval abrasion of coarse aggregates. In 1991, researchers in Ontario collected 86 natural sands and 21 quarry screenings from Ontario, all of which had satisfactory performance in portland cement concrete and/or asphaltic concrete. These researchers examined several tests for evaluating the quality of fine aggregate in concrete and asphalt. These tests included the Micro-Deval test, the ASTM attrition test, and the MTO attrition test, and the magnesium sulfate soundness test for fine aggregates. They found that the sulfate soundness test for fine aggregates suffers from poor multi-laboratory precision (COV of about 10.5%). The Micro-Deval was identified as a suitable replacement for the magnesium sulfate soundness test because it has good correlation with this test (R^2 = 0.88), is quicker (2 days vs. 10 days), and has a much better multi-laboratory precision (COV of 1.9%). Both ASTM attrition and MTO attrition tests had fairly high variance (COV of 11.0% of 14.1% respectively). The Micro-Deval loss also correlated well with absorption (R^2 = 0.81), which is logical considering more absorptive materials are typically more porous, and thus more susceptible to breakdown during wet abrasion (Rogers, Bailey, & Price, 1991).

In the years following, the Ontario researchers also determined that the Micro-Deval test for fine aggregates was useful in identifying weak and soft material such as shale. They also found a high correlation with Micro-Deval loss of fine aggregates and drying shrinkage of paste. A Micro-Deval loss of approximately 25% was the limit for negligible shrinkage. Sands with Micro-Deval loss higher than 25% loss showed much greater mortar shrinkage. Ontario specifications (as of 1998) dictate a maximum Micro-Deval loss of 20% for fine aggregate used in portland cement concrete (Rogers C. , 1998).

Researchers from the Virginia Transportation Research Council also explored the use of the Micro-Deval test for assessing fine aggregate durability. Ten fine aggregates were evaluated by Micro-Deval, petrographic examination, magnesium sulfate soundness test for fine aggregates, and freeze-thaw soundness. District materials engineers subjectively rated each source as "good", "borderline", or "poor", however ratings were not always based on performance but sometimes related to compliance with specifications. The Micro-Deval loss was measured by three methods: 1) traditional method of loss of No. 200 (75-µm) sieve, 2) weighted average based on test gradation, and 3) change in area under gradation curves before and after the test. Researchers concluded that the weighted average Micro-Deval loss differentiated between good and poor-performing aggregates 80% of the time (8 of 10 sources). The area between the curves loss calculation was also able to have the same success rate. As a result, modifying the loss calculation for Micro-Deval improved its ability to identify poor performers. A rated loss of less than 20% should provide good performance. For aggregates with less than 1.5% absorption, the MTO standard of 24 hour soaking can be reduced to one hour soaking without significantly affecting Micro-Deval loss results (note that the current ASTM D 7428 specification dictates one hour soaking time). This project also reached that same conclusion as Rogers et al. (1991): that the Micro-Deval test also had lower variability than the other soundness tests, and is thus more repeatable and reproducible. In this study, Micro-Deval COV was 2.3%, compared to 16.9% for magnesium sulfate soundness, and 28.7% for freeze-thaw soundness. Finally, researchers also concluded that the Micro-Deval test can be used as a quality control check to determine if material from a source has changed significantly (Hossain, Lane, & Schmidt, 2008).

One research project by Rached (2011) at The University of Texas examined the Micro-Deval test, among others, as an evaluative tool measuring potential friction loss. The scope of this project was to examine the use of manufactured sands in pavement concrete, since manufactured sands are becoming more necessary and are known to affect skid resistance, workability, and finishability. Rached examined the Micro-Deval test, acid insoluble residue (AIR) test, absorption, and the DFT60 test (coefficient of friction at 60 km/hr), among others, and determined that the Micro-Deval loss correlates well ($R^2 = 0.87$) with the DFT60 test in a polynomial relationship. Acid insoluble residue had a weaker correlation ($R^2 = 0.44$) with DFT60. Absorption had a reasonably high correlation ($R^2 = 0.62$) with DFT60, but the author would not recommend the ASTM C 128 absorption test due to its subjectivity and lack of repeatability. As a result, Rached concluded that the Micro-Deval test is more suitable than the AIR test for evaluating polish resistance of fine aggregates due to its higher correlation with DFT60 and the fact that it is a mechanical test and polishing is a mechanical behavior. As a final recommendation, Rached commented that the AIR and Micro-Deval test can be used in combination to indicate the presence of carbonates and determine the hardness of fine aggregates. The author also proposed blending guidelines for manufactured sands based on AIR and Micro-Deval results (Rached M. M., 2011).

2.4.2 Absorption

As with coarse aggregates, absorption is an important property of fine aggregates. Aggregates with higher absorption capacity are typically more porous, and therefore weaker and more susceptible to breakdown and abrasion.

2.4.2.1 Specific Gravity and Absorption of Fine Aggregates

A variety of methods for measuring bulk density and absorption were developed by researchers in the 1920's and 1930's but the ASTM standard for measuring these properties (ASTM C 128) has remain relatively unchanged since 1948. The ASTM standard has been widely accepted but a few agencies and DOTs use slightly altered versions of this test (the test procedure is described further in Section 5.2.2 Specific Gravity and Absorption Test for Fine Aggregates). Ontario MTO, Kentucky DOT, Kansas DOT, and Mississippi DOT have historically washed sands over a No. 100 (150-µm) sieve prior to testing them for the saturated surface dry (SSD) condition due to perceived inaccuracies in density calculations (Rogers & Dziedziejko, 2007).

Noting this discrepancy in test methods, researchers from Ontario conducted an investigation to determine the influence of microfines (<75 µm) on calculated absorption and density values of fine aggregates. The results of this investigation showed that stirring of the fine aggregate during testing can create artificial particles made up of conglomerated microfines. Because of this effect, sands with high microfines content (> 8% by mass) had higher absorption, lower density values, and higher variance compared to sands with microfines removed prior to testing. The presence of microfines in excess of 8% by mass caused relative density values to be off by as much as 0.13 and variance to be two times higher than a washed sand. When the microfines content is less than 4%, the error is negligible. Figure 10 demonstrates the severity of this problem. Researchers recommended that sands with high microfine content should be washed prior to ASTM C

128 testing to ensure more accurate specific gravity and absorption measurements (Rogers & Dziedziejko, 2007).

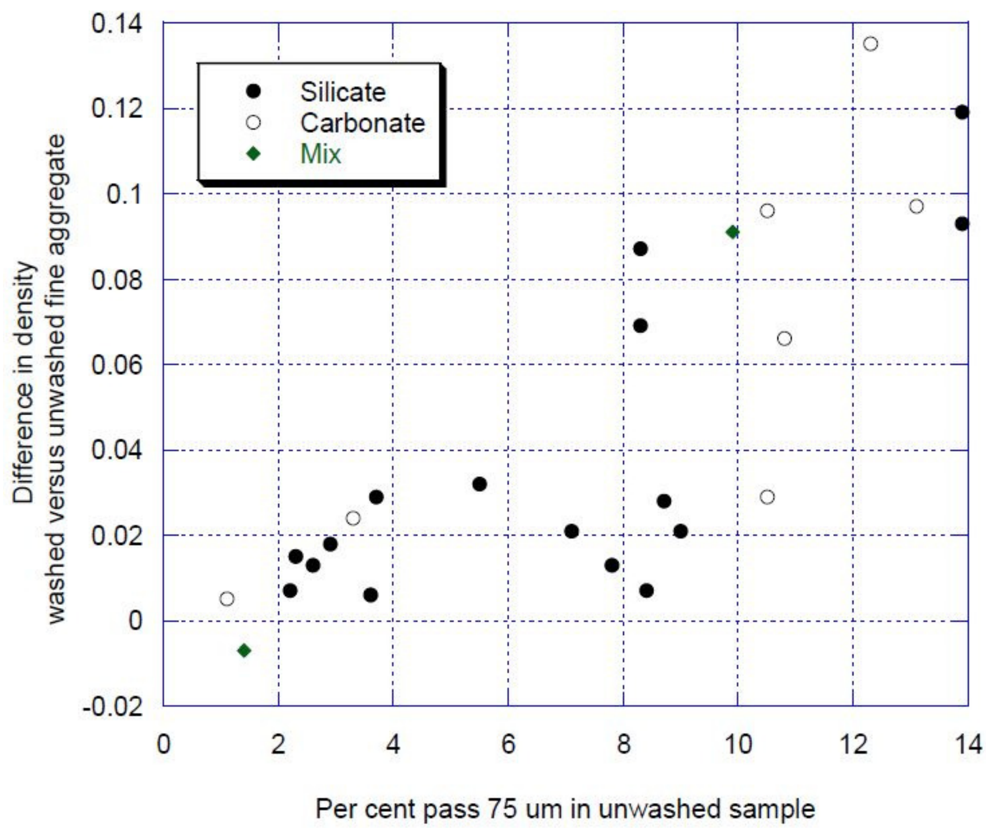

Figure 10: Difference in Density between Washed and Unwashed Samples of Fine Aggregate (Rogers & Dziedziejko, 2007)

2.4.3 Shape Characteristics

The shape characteristics of a fine aggregate are important particularly to fresh concrete properties such as workability and finishability. For example, manufactured sands tend to be more angular and therefore require a higher water demand to achieve the same slump as a natural sand. Two promising tests have evolved over the last few years which directly, in the case of AIMS, or indirectly, in the case of the flakiness sieve, quantify shape characteristics of fine aggregates.

2.4.3.1 Aggregate Imaging System (AIMS 2.0)

The Aggregate Imaging System (AIMS) is also capable of determining shape characteristics of fine aggregates. The aggregate imaging system (AIMS) is a machine consisting of a camera, lights, computer software, and movable trays, designed to capture and analyze the shape, angularity, and texture of coarse aggregates and the form and angularity of fine aggregates. The camera captures images of the aggregate particles , either lit directly or backlit, and the software analyzes these images and provides the user with data summarizing the shape characteristics. For fine aggregates, the user places a set of fine aggregate particles (separated by fraction size) on an opaque tray and places this tray into the machine. The camera in the AIMS apparatus is capable of capturing particles as small as 75-µm (retained on the No. 200 sieve). Output from the AIMS consists of quantified measurements of form and angularity for each particle, and a mean and standard deviation of each value as well. Aggregates typically follow a standard statistical distribution. Recent advances in this technology include a touching particle factor (TPF) to eliminate inaccurate angularity analysis of fine aggregates where particles touch or overlap (Mahmoud, Gates, Masad, Erdoğan, & Garboczi, 2010).

2.4.3.2 Flakiness Sieve

After observing problems in field compact of HMA, Rogers and Gorman (2008) sought to develop an inexpensive and quick test to determine a measurement of flakey particles in a sand. Past research has demonstrated that sands in excess of 30% flakey particles may have issues during compaction of hot mix asphalt. Rogers and Gorman (2008) briefly considered a flat and elongated test using a hand-held set of proportional calipers, but variations in accuracy of calipers and poor multi-laboratory variation caused researchers to abandon this test. ASTM C 1252 (*Standard Test Methods for Uncompacted Void Content of Fine Aggregate*) was also considered but no realistic limit could be found to reject poorly-compacting fine aggregate. Rogers and Gorman realized that slotted sieves, traditionally used for seeds and grains, could also be used to evaluate fine aggregate (Rogers & Gorman, 2008).

With this knowledge, Rogers and Gorman collected and tested 120 fine aggregates on Micro-Deval, specific gravity and absorption, uncompacted voids, compacted aggregate resistance (CAR) test developed by D. Jahn (2004), and the slotted sieve identified by this project. The flakiness sieve test, described by Rogers and Gorman (1998) in an appendix, uses two slotted sieves with slots of 1.8-mm and 1.0-mm respectively. The fine aggregate is sieved and broken down into separate size fractions. The sand retained on the No. 8 (2.36-mm) sieve is placed on the 1.8-mm slotted sieve and agitated. The same is done for the sand retained on the No. 16 (1.18-mm) sieve, except the 1.0-mm slotted sieve is used. The operator uses a set of tweezers to ensure that all flakey particles pass through the slots. All particles passing through the slotted sieves are considered flakey, and the final results are calculated by mass. The sieve is pictured in Figure 11 with sand particles retained.

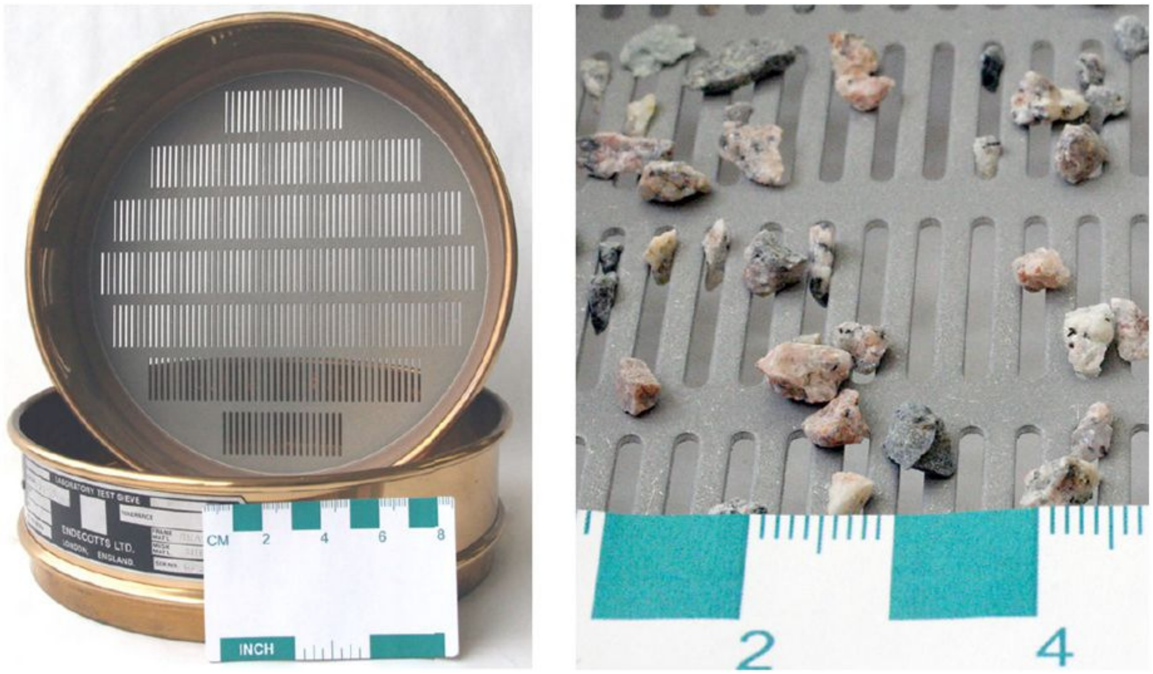

Figure 11: Slotted Sieve for Finding Flakey Particles in Fine Aggregate

After testing 120 fine aggregates, Rogers and Gorman concluded that natural sands tended to be less flakey than crusher screenings. No natural sands were found to have flakiness in excess of 25%. High flakiness on one sieve size tended to indicate high flakiness on the other sieve size, although regression coefficients were not strong. There was no obvious relationship between Micro-Deval loss and mean flakiness. Based on the two case studies in Ontario, contractors were unable to compact two sources which, when tested by the flakiness sieves, had flakiness in excess of 25% for No.8 (2.36mm) and 30% for No.16 (1.18mm). Rogers and Gorman recommend using the flakiness sieves as a means of detecting fine aggregate that is potentially difficult to compact (Rogers & Gorman, 2008). It is very possible that this test can also be applied towards workability and finishability of fresh portland cement concrete.

2.4.4 Mineralogical and Chemical Composition

Knowing the mineralogical and chemical composition of an aggregate can help engineers and scientists identify potential problems in concrete aggregates such as alkali-aggregate reaction, presence of detrimental clays and mica, shrinkage and thermal issues, and overly weathered material (Folliard & Smith, 2002). Keeping track of aggregate mineralogical composition can also help DOTs track and identify trends with good-performing and poor-performing aggregates. Some of the most common methods of identifying mineralogical and chemical composition of fine aggregate are petrographic analysis, X-ray diffraction, and acid insoluble residue.

2.4.4.1 Petrographic Examination

Petrographic examination is typically performed by a trained and experienced petrographer, knowledgeable of local geology, following ASTM C 295 (*A Guide for Petrographic Examination of Aggregates for Concrete*). As part of the petrographic examination, petrographers identify key constituents and proportions of an aggregate sample using a variety of tools including microscopes, cameras, polishing/grinding wheels, and a variety of hand tools. This technique and its benefits are further discussed in Section 2.3.7.1 Petrographic Examination.

2.4.4.2 X-ray Diffraction

X-ray diffraction (XRD) is an advanced analysis technique that sometimes requires the material of interest to be ground to a fine powder (depending on the exact analysis method). A diffractometer, which fires an incident X-ray beam at the sample and receives the scattered beam, is used to gather data. Output from X-ray diffraction appears as a plot of scattering intensity versus scattering angle. From this plot, peaks can be identified which correspond to individual material components of sample.

X-ray diffraction can be particularly useful for identifying deleterious clays as demonstrated by one study at the University of Wisconsin. A description and results of this study are discussed in Section 2.3.7.2 X-Ray Diffraction (XRD).

2.4.4.3 Acid Insoluble Residue

The acid insoluble residue test is one way of determining carbonate content of fine aggregate. In this test, a fine aggregate sample is subjected to hydrochloric acid and carbonate aggregates are dissolved by the aggregate while siliceous aggregates remain. Carbonate aggregates polish more easily than siliceous aggregates which reduce skid resistance of friction-critical concrete applications. Texas is one state that specifies use of this test (limit of 60% insoluble) due to the increased interest in using manufactured sands in concrete pavement applications. However, at least one research study by Rached has shown that the acid insoluble residue test may not correlate well with the property that it is trying to measure.

The scope of Rached's project was examining the use of manufactured sands in pavement concrete, since manufactured sands are becoming more necessary and are known to affect skid resistance, workability, and finishability. Rached examined the Micro-Deval test, acid insoluble residue (AIR) test, absorption, and the DFT60 test (coefficient of friction at 60 km/hr), among others, and determined that the acid insoluble residue test had a weaker correlation ($R^2 = 0.44$) with DFT60 than did the Micro-Deval test ($R^2 = 0.87$). As a result, Rached concluded that the Micro-Deval test is more suitable than the AIR test for evaluating polish resistance of fine aggregates due to its higher correlation with DFT60 and the fact that it is a mechanical test and polishing is a mechanical behavior. As a final recommendation, Rached commented that the AIR and Micro-Deval test can be used in combination to indicate the presence of carbonates and determine the hardness of fine aggregates. The author also proposed blending guidelines for manufactured sands based on AIR and Micro-Deval results (Rached M. M., 2011).

2.4.5 Deleterious Substances

Anecdotal evidence has long suggested that concrete mixtures containing clay content (from coarse or fine aggregate) will have a detrimental effect on the fresh and hardened properties of the concrete and reduce rheology. Norvell et al. (2007) explored this issue further by “doping” sand with clay minerals and (non-clay) microfines and examining the effects on concrete. Microfines replaced sand by 1-4% and mortar mixes were created to determine the impacts of water demand, compressive strength, and shrinkage. Norvell et al. (2007) found that all clays increased water demand and HRWR demand to achieve a constant flow. Montmorillonite had the highest absorption of all clays studied. Figure 12, taken from a separate study by W.R. Grace & Co. displays the implications of this trend.

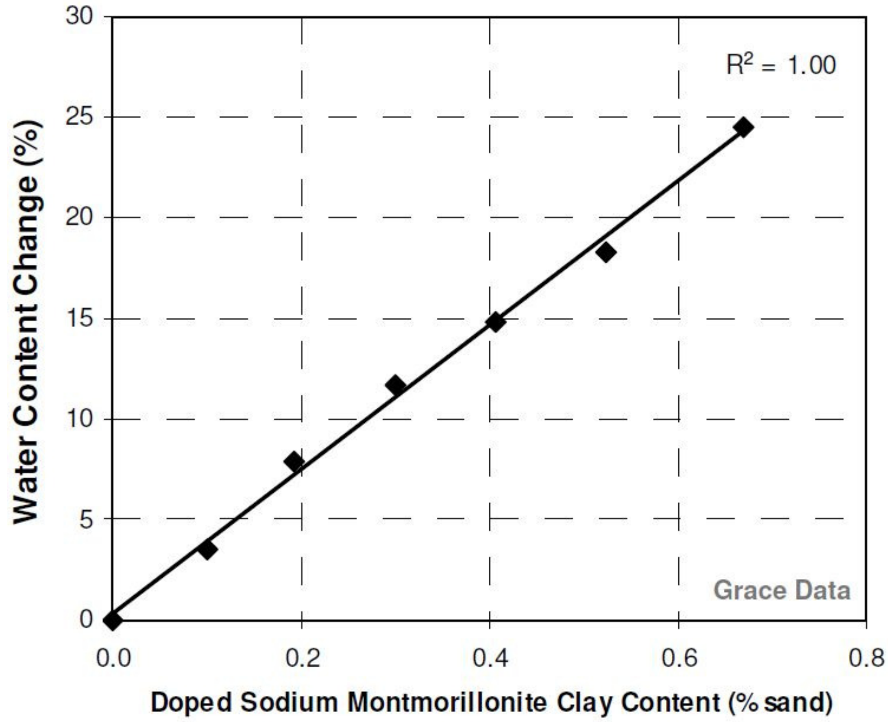

Figure 12: Effect of Sodium Montmorillonite on Concrete Water Demand for a 3 in. Slump (Koehler, Jeknavorian, Chun, & Zhou, 2009)

Surprisingly the microfines decreased the HRWRA demand and only increased the water demand slightly to achieve constant flow at a constant water to cement ratio. Interestingly, Norvell et al. (2007) also determined that the only reason clays caused lower compressive strengths of mortar was due to the necessary increase of water to achieve the same level of workability. When the water to cement content was held constant (and flow ignored), the impact of clay on compressive strength was negligible, except in the case of montmorillonite. Regarding drying shrinkage, microfines showed no effect, nor did kaolinite and illite clays. However, montomrillonite did have an adverse effect on drying shrinkage of the mortar mixes (Norvell, Stewart, Juenger, & Fowler, 2007).

Improved aggregate performance can be realized when polycarboxylate-based HRWR admixtures are dosed appropriately according to the clay content. However, this relies on accurate knowledge of the sand's clay content. Current methods to measure clay in aggregates include: methylene blue value (MBV), sand equivalent value (SE) and durability index, plasticity index, X-ray diffraction (XRD), and thermo-gravimetric analysis.

2.4.5.1 Methylene Blue Test

Methylene blue is a dye that has a strong affinity for clay particles. As such, this test (note that there are several versions, the most common in the US being AASHTO TP 57) subjects a fine aggregate sample to diluted methylene blue and the color of the mixed, filtered solution will depend on the clay content. Higher methylene blue values suggest higher clay content which can indicate problematic aggregates.

In the past few years, researchers at the University of Wisconsin explored the effects of microfines on fresh and hardened concrete properties. Munoz et al. identified ten aggregates from the state of Wisconsin that were suspected to have microfine coatings. After XRD analysis, three aggregates were selected (as representative) to be studied more in depth. Ten concrete mixes were created: "original coated aggregate series", "washed aggregate series", and "artificial coated aggregate series". The P200 (ASTM C 117) test, California cleanness value, methylene blue value (MBV), and modified methylene blue value (product of P200 and MBV) were compared and correlated to concrete tests. Researchers concluded that even when microfines are present in amount under 1.5%, they influence fresh and hardened concrete properties, the extent of influence depending on nature and amount of microfines. For carbonate microfines, a small difference (< 0.2%) in cleaned and as-received aggregate did not influence slump or shrinkage, but did slightly improve tensile strength of the concrete. Coatings classified as “clay/carbonate” did not influence shrinkage or freeze-thaw durability, but did decrease slump and increase tensile strength. Coatings classified as “dust” or “clay/dust” both decreased slump and increased shrinkage, but did not influence concrete strength or freeze-thaw durability. Due to its absorptive nature, “clay” coatings had the most dramatic influence on concrete properties. These coatings decreased slump and increased shrinkage dramatically. The addition of water to maintain workability also caused a decrease in tensile strength and freeze-thaw durability. Perhaps the most important conclusions from this study is that quantifying the amount microfines alone (as currently quantified through ASTM C 117) is not a good enough measure for deleterious material. This study showed that the type of microfines is just as important as the quantity. Munoz et al. (2010) also determined that because it identifies quantity of clay, the modified methylene blue value had the best correlation (out of the other microfine tests) with compressive strength and durability tests (Munoz, Gullerud, Cramer, Tejedor, & Anderson, 2010).

Because there is an inherent level of subjectivity to the methylene blue test, researchers at W.R. Grace & Co. sought to improve this test method by removing subjectivity and enhancing repeatability and reproducibility. The test developed by Grace is similar to the traditional AASHTO test, but the test is performed on an entire sample of sand (not just the microfines) and uses a UV colorimeter to analyze color of the final filtered solution sample. The new methylene blue test allows the entire sample to be measured, which is important because all clay in the sand in measured which ensures more representative results. The reproducibility and repeatability were comparable to the traditional AASHTO method. Researchers at Grace demonstrated that inadequate sieving can cause the traditional methylene blue test to produce inaccurate results, as shown in Figure 13 (Koehler, Jeknavorian, Chun, & Zhou, 2009).

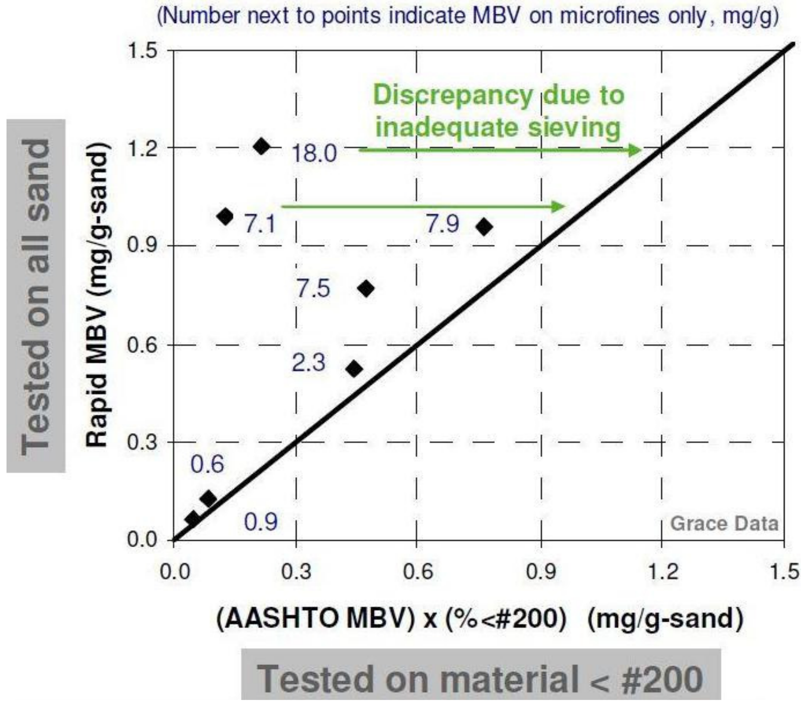

Figure 13: Methylene Blue Testing of Full Sand versus Microfines (Koehler, Jeknavorian, Chun, & Zhou, 2009)

2.4.5.2 Organic Impurities

Investigations by early concrete researchers at the Lewis Institute showed that even small amount of tannic acid or surface loam (from decomposing organic materials) can significantly reduce concrete strength due to interference with the hydration process (Lewis Institute, 1921). As a result, most state DOTs specify use of the AASHTO test for organic impurities. In this test, a fine aggregate sample is subjected to a sodium hydroxide solution and allowed to remain undisturbed to react for 24 hours. Any organic material in the sample will react with the sodium hydroxide solution to produce a dark liquid. The operator examines the color of the supernatant liquid and if it is darker than a standardized color, the fine aggregate is subjected to a 7-day mortar cube strength test.

The fine aggregate is typically deemed to have an unacceptable amount of organic content if the compressive strength of the mortar cube is less than 90-95% of a control sample.

2.4.5.3 Sand Equivalent Test

The sand equivalent test is a test method that is used to determine the proportion of "detrimental fine dust of clay-like particles in soils or fine aggregates" (Texas Department of Transportation, 2009). This test subjects a fine aggregate sample to a flocculating solution (calcium chloride) in order to separate fine particles from the coarser sand. The higher the sand equivalent value, the cleaner the sand is perceived to be. From the survey of other DOT specifications (discussed further in Section 3.2 Survey of Other States and Organizations), the research team determined that only 11 state DOTs specify this test for aggregate quality control. Of these 11 states, Texas has the highest (most restrictive) limit at 80.

Alhozaimy (1998) examined 100 natural sands and 100 crushed manufactured sands in Saudi Arabia to compare the sand equivalent test (ASTM D 2419) with another test that measures fine particles in a sand: the ASTM C 117 test (*Materials Finer than No. 200 Sieve by Washing*). This study determined that, for the natural sand, the sand equivalent test was strongly correlated to the materials finer than No. 200 test. However, no correlation existed between the two tests for crushed manufactured sands, leading the author to call the sand equivalent test "misleading" for these types of fine aggregates. Furthermore, investigations between the two tests and water demand of mortar, showed a correlation between sand equivalent and water demand for only the natural sands (Alhozaimy, 1998). Although this was only one research study, it strongly suggests that the sand equivalent test for manufactured sands may not sufficiently serve its purpose.

Chapter 3: Development of Testing Program

3.1 Survey of TxDOT

The research team surveyed TxDOT District Members and Construction Division personnel to obtain more information about aggregates used in Texas, as well as concrete conditions and recurring problems.

3.1.1 Development of District Surveys

The research team developed a standardized survey for TxDOT district personnel for the purpose of gleaning information from the tremendous amount of knowledge and experience of area engineers and laboratory personnel. The focus of the district surveys was to identify fine and coarse aggregate types that have been used in concrete over the years, particularly those that performed poorly so that they could be obtained for testing in order to determine limits for test procedures. The survey also focused on identifying specific field problems related to aggregates.

It was important that the survey be standardized and performed by only one or two researchers to ensure consistency and to eliminate bias. The standardized survey included a statement of purpose to brief the interviewee on the goals and methods of the project and continued with specific questions regarding aggregate performance in that district. Topics of questions included aggregate performance, aggregate testing procedures, and aggregate sources that are commonly used in the district as well as sources that are no longer used.

Prior to implementation, the standardized survey was sent to the project committee members for feedback and approval. After including feedback in the final revision of the surveys, the surveys were sent via email to district engineers and lab personnel to allow district personnel time to evaluate the questions and provide the most appropriate responses. Researchers followed up the emails with phone interviews.

3.1.2 Results of District Surveys

Researchers were able to communicate with TxDOT personnel in all 25 districts across the state and get information from 22 of those districts. The interviewees range from lab supervisors to materials engineers to traffic engineers. In most cases, the interviewee was the district lab supervisor as these personnel seemed to have the most information about aggregate sources and testing in their district. In some cases, district personnel polled area engineers to obtain other relevant information. Observations and knowledge of area engineers are an important piece of the district surveys, particularly for the larger districts.

One of the questions addressed by the surveys involved district laboratory testing procedures and equipment. Not surprisingly, all districts perform quality monitoring tests on aggregates when necessary. However, most district labs do not have equipment for LA abrasion, magnesium sulfate soundness, or acid insoluble residue testing so aggregates are typically sent to the TxDOT Construction Materials and Testing Laboratory in Cedar Park, Texas for these tests. A few districts (Waco and Odessa) did possess equipment for LA abrasion and/or magnesium sulfate soundness but typically sent aggregates to the TxDOT materials lab anyway. Despite the fact that there is no Micro-Deval requirement for Item 421, all districts reported possessing the equipment to run this test. Results varied by district as to how frequently the Micro-Deval test is performed. Many districts run and record Micro-Deval loss values for bituminous aggregates on a regular basis (as recommended by Jayawickrama (2007)) but do not perform the Micro-Deval test for concrete aggregates as frequently.

When asked about their opinions regarding the usefulness of the current Item 421 test methods, interviewee responses varied. Some interviewees felt that they did not possess the expertise to comment on these tests, while others freely gave their opinion. One popular response was that the current sand equivalent limit of 80 may be slightly higher than necessary. Several district personnel felt that this limit may be precluding the use of sands that could still make strong, durable concrete. Interestingly, one district raised the limit on the sand equivalent test to 90 after reporting problems with a sand possessing sand equivalent values in the 80s. One laboratory supervisor felt that the decantation limit may be too high in addition to the sand equivalent limit being too high. Another laboratory supervisor felt that the fineness modulus limit may cause some aggregates to be rejected that would perform well otherwise. Other personnel felt that the Micro-Deval test, sieve analysis, and the sand equivalent test are useful, and the organic contents test and decantation test are relevant. The research team also contacted aggregate producers for collection of aggregates (see Chapter 4) and several producers have suggested if gradation limits were slight adjusted, more aggregates could be used locally to make good-performing concrete. However, gradation requirements of concrete aggregates are not in the scope of this project.

Most districts have no trouble with aggregates meeting specification limits so they do not allow deviations to these limits, or they do allow deviations but only on an as-needed basis. There are some districts that do allow deviations due to recurring issues in their district (i.e. sand equivalent limit of 90 as previously discussed). The Beaumont district relies on aggregates being shipped in due to poor local geology and as a result, the decantation limit for these aggregates is raised. The Laredo district has trouble with coarse and fine aggregates not meeting gradation so they have altered the fine aggregate gradation to allow the No. 8 (2.36-mm) sieve to pass with 75-100%. Although the Fort

Worth district does not allow deviations to specification limits, they noticed severe problems with aggregate quality in the fall of 2010 when 9 of 15 producers failed quality monitoring (QM) tests for gradation and/or decantation in September and 5 of 15 producers failed QM tests for gradation and/or decantation the following month. The cause of this widespread drop in quality was unknown.

As far as evaluation of the aggregate sources, it seems that sources in the Aggregate Quality Monitoring Program (AQMP) have been used very successfully in districts throughout the state (the AQMP is discussed further in 4.1.1 TxDOT Aggregate Quality Monitoring Program (AQMP)). A survey of TxDOT district personnel revealed that most districts have access to at least a few aggregate sources within their district, with the exception of a few districts in eastern Texas. Most districts are able to use somewhat local resources, though some districts are forced to ship aggregates in from other districts or states. Survey results showed that districts use anywhere from one to more than a dozen aggregate sources on a regular basis for TxDOT projects. Many districts have also used local non-AQMP sources with success and there are several of these producers in the process of being approved and added to the current AQMP list. Survey results also showed that TxDOT projects used local aggregates where possible but local geology conditions, particularly in the eastern districts, prevented local sources from being utilized. In the Atlanta, Beaumont, Bryan, and Lufkin districts, coarse aggregates are shipped in from elsewhere (from other districts in Texas or Arkansas, Louisiana, Oklahoma, Canada, and Mexico). There are also non-AQMP sources in several districts that have been used successfully for non-TxDOT concrete applications but due to specification limits (particularly magnesium sulfate soundness and sand equivalent) are not used in TxDOT projects.

It was somewhat difficult to discern the common concrete aggregate problems in Texas due to the fact that not all interviewees were aware of specific problems in their district or did not know the nature or causes of distress. District surveys showed reports of alkali silica reaction (ASR) in at least six districts (Beaumont, Bryan, Dallas, Houston, San Angelo, and Waco) though specific locations had already been investigated in all of these districts. Minor to moderate spalling and non-ASR cracking were also reported in at least five districts (Atlanta, Beaumont, Fort Worth, Paris, Tyler) though interviewees did not necessarily know the cause of distress due to the widespread nature of the problem, age of the concrete, or lack of information. Atlanta TxDOT personnel believe that the cause of most cracking in that district is due to siliceous aggregates with high CoTE values. The problem is prevalent in pavement applications (but not structural applications) due to the use continuously reinforced concrete pavements. There is currently no CoTE limit in the district and brining in aggregates with low CoTE values would be cost prohibitive. The Lufkin district also experienced a thermal expansion problem that was investigated within the last two years. The same expansion problem has been observed in Houston in CRCP pavements, so aggregates with high CoTE values (particularly siliceous river gravel) are now avoided when possible. This trend was documented by Won (2001) and Du & Lukefahr (2007) and was discussed in Section 2.3.6.2 Coefficient of Thermal Expansion (CoTE). Another common response to the question of aggregate issues was the occasional problem of “dirty” or rounded aggregates causing low compressive strengths. Other concrete issues were typically attributed to age.

3.1.3 Survey of TxDOT Construction Pavement and Materials Division

The goal of the survey of TxDOT Construction (CST) Pavement and Materials Division was to obtain the results of aggregate tests for frequently run sources, aggregate test methods, an aggregate data base, and performance data.

The results of the aggregate tests for frequently run sources were provided by CST for all aggregates meeting the AQMP requirements for concrete and bituminous applications. Results include values for rated source LA abrasion, magnesium sulfate soundness, Micro-Deval, coefficient of thermal expansion (where applicable), alkali silica reaction (where applicable), and acid insoluble residue.

Aggregate test methods, as described in Item 421, were also provided by TxDOT personnel. These test methods include LA abrasion (Tex-410-A), magnesium sulfate soundness (Tex-411-A), organic impurities (Tex-408-A), and sand equivalent (Tex-203-F). A more detailed description of these test methods can be found in Chapter 5: Laboratory Testing.

A recommendation of aggregates to test for the project was made by TxDOT personnel to reflect a diversity of geographic locations (including river basins for gravel), mineralogies, and applications. Determining the final list of aggregates was an iterative process which involved several discussions on how to cost-efficiently transport and acquire those aggregates and is discussed further in Section 4.1 Selection of Aggregates and Section 4.2 Aggregate Collection.

3.1.4 Summary

The most important results of the survey of TxDOT are the findings that districts do use local aggregate sources when possible, although this is very difficult to do in eastern Texas where local coarse aggregates are non-existent or poor quality. The most common problems reported that were known to be relate to aggregates were cracking due to ASR and thermal expansion. Neither of these issues is currently addressed directly with limits in Item 421. It is also important to note that in some cases, limits (particularly magnesium sulfate soundness and sand equivalent) in Item 421 have caused local aggregates to be rejected even though they have been used successfully in other non-TxDOT concrete projects around the state.

3.2 SURVEY OF OTHER STATES AND ORGANIZATIONS

A review of testing and standards for aggregates in other states was conducted to understand the differences in the approach of how states qualify aggregate use in portland cement concrete. The most recent versions of DOT state construction specifications were reviewed for all 50 states to determine general trends in testing procedures and limits and to identify any significant deviations from how Texas approaches aggregate use.

3.2.1 Coarse Aggregate Specifications

The most common coarse aggregate qualification test is the LA abrasion test. Texas allows aggregates with up to 40% LA abrasion loss which is close to the national average. Forty-nine states use the LA abrasion test to evaluate aggregates. The highest allowable limit is 60%, specified by both Georgia and Kentucky, while the lowest allowable limit is 30%, specified by Oregon, Illinois, Indiana, and Massachusetts. Maine is the only state to not specify use of the LA abrasion test. Kansas has different limits dependent on aggregate type and New Mexico uses an aggregate index value which combines LA abrasion, sulfate soundness, and absorption. Figure 14 displays LA abrasion limits for each state.

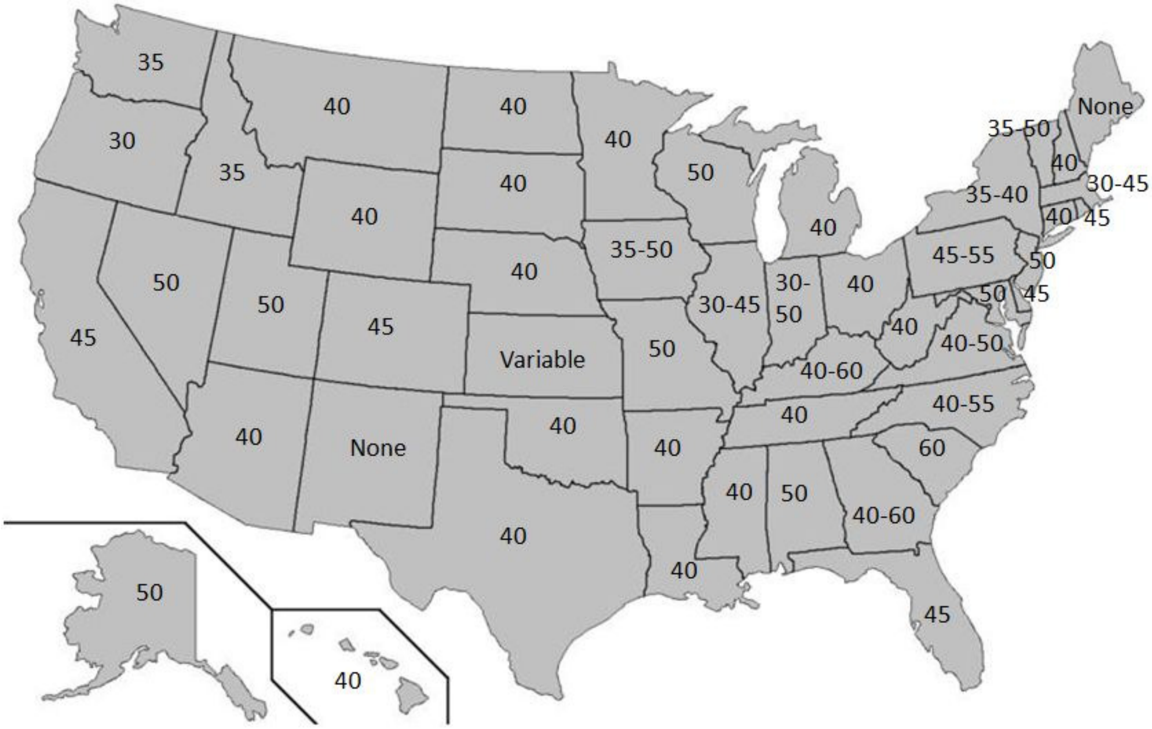

Figure 14: Limits for the LA Abrasion Test as Specified by State DOTs

The sulfate soundness test is also a very common test procedure used to evaluate aggregates. However, there appears to be no clear consensus with how to interpret and apply the results of this test. Twenty-eight states specify the use of sodium sulfate, nine states specify the use of magnesium sulfate, two states allow the use of either magnesium sulfate or sodium sulfate, and eleven states do not use sulfate soundness testing. Ten states that do use the sulfate soundness test have an "opt out" clause for the test, allowing aggregate use with 5 years of field performance data and engineer approval. Arizona only requires this test for use above 4500-ft. elevation, while another state (New York) even specifies a ten-cycle sulfate soundness test, instead of the typical five-cycle version. The Texas limit of 18% for five-cycle magnesium sulfate soundness is somewhat high compared to other states, but is not the highest. Virginia has the highest limit and allows up to 30% for the five-cycle magnesium sulfate soundness test. Connecticut has the lowest limit for the five-cycle magnesium sulfate soundness test at 8%. The highest allowable limit for the five-cycle sodium sulfate soundness test was 25% (Illinois and Indiana), and the lowest allowable limit for the five-cycle sodium soundness test was 8% (Vermont). Figure 15 displays sulfate soundness limits for each state. Note that "Na" signifies the use of sodium sulfate soundness, while "Mg" signifies the use of magnesium sulfate soundness.

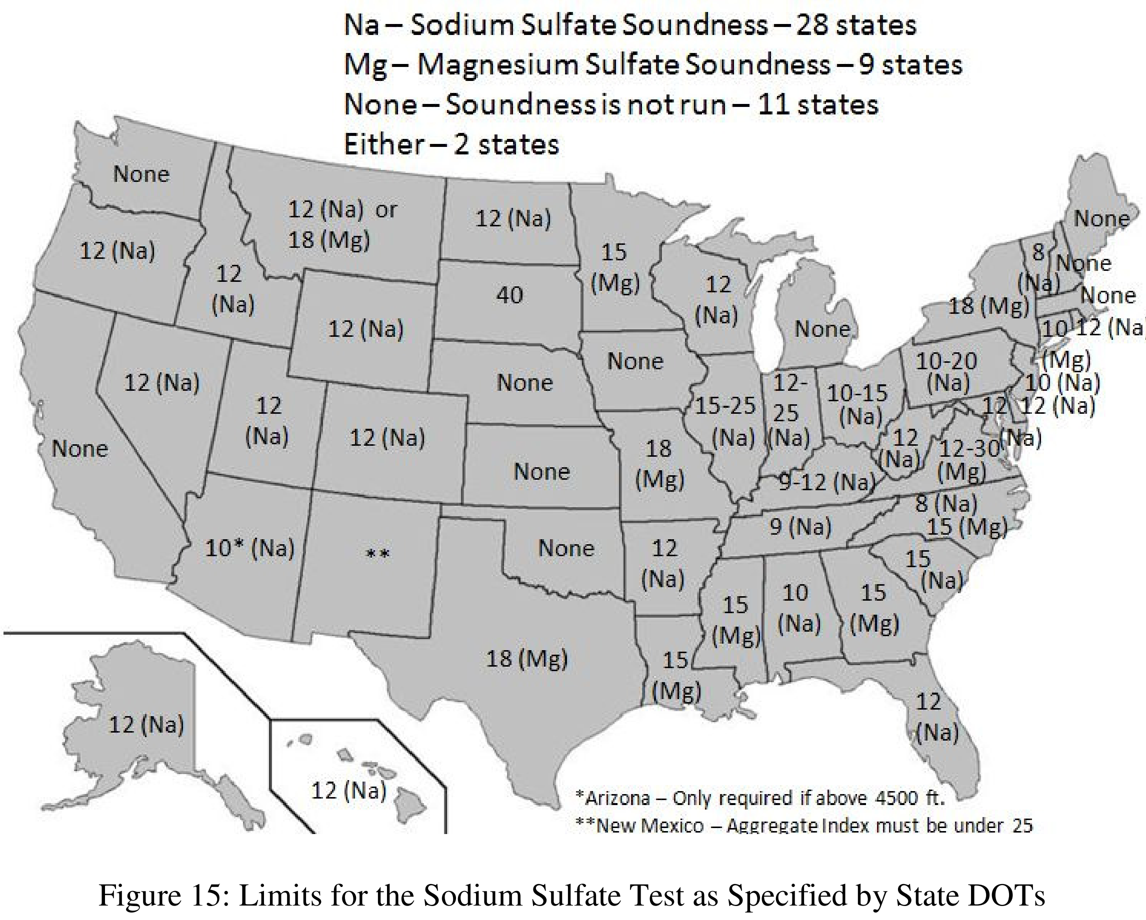

Figure 15: Limits for the Sodium Sulfate Test as Specified by State DOTs

Other important coarse aggregate tests include those used to measure the type and amount of deleterious material and microfines. Some states specify total limits for cumulative amount of deleterious materials based on clay lumps, shale, and friable particles, but states that do list limits for each type of deleterious materials are discussed as follows. Also, it is important to note that five states simply specify that the aggregate must be “free from deleterious material” and the state engineer of record is responsible for determining aggregate usage. For clay lump content, Texas has the most conservative limits in the country at 0.25% allowed. However, eight other states match this limit for clay lump content. The highest allowable clay lump content is 2.5%, specified by New Mexico. For shale content, the Texas requirement of 1.0% is similar to most other states. The lowest allowable shale content is 0.4% (Minnesota) and the highest allowable is 10% (Pennsylvania). However, this limit only applies to curb and gutter applications; otherwise a 2.0% limit is specified. For friable particles, the limit in Texas of 5% is one of the highest in the country. The lowest allowable limit is 0.25% (West Virginia) and the highest allowable limit is 8% (Illinois). For decantation, the Texas limits of 1.0% to 3.0% (these limits depend on the aggregate type and composition of microfine material) are comparable to limits by other states (Texas Department of Transportation, 2004).

Other coarse aggregate specification limits included by state DOTs include an expansion limit for ASR related expansion, a maximum content of flat and elongated particles (on either a 5:1 or 3:1 basis), a limit for coal and lignite content, and freeze-thaw limits which depend on the test. Several states use AASHTO T 103 for freeze-thaw testing while other states have state specific unconfined freeze-thaw limits. As previously mentioned, New Mexico has a unique “Aggregate Index” to rate aggregates which is based on a composite score from LA abrasion, sulfate soundness, and absorption. Alaska uses a machine that is smaller than the LA abrasion machine, but larger than the Micro-Deval apparatus, to simulate studded tire abrasion. This test uses small diameter steel charge and is conducted in the presence of water. A review of several Canadian specifications showed that Canadian provinces typically specify an absorption limit as well as Micro-Deval limits and unconfined freeze-thaw limits.

3.2.2 Fine Aggregate Specifications

Fine aggregate specifications in the state of Texas include tests for deleterious material, organic impurities, acid insoluble residue, sand equivalent, and fineness modulus. For deleterious materials, Texas limits clay lump content to 0.5% which is one of the most conservative limits in the country. Approximately 75% of states run this test and the highest allowable limit is 3% (eleven states) while the lowest allowable limit is 0.25% (Missouri and Virginia). Canadian provinces typically specify a total deleterious content.

For organic impurities, Texas requires that if the color of the sample supernatant liquid is darker than a standard color (Gardner No. 11), then the aggregate should be run for AASHTO T 71 mortar cube test and meet 95% compressive strength when compared to a control mixture. This test is run by 48 states with the same procedures. For samples failing the Gardner color test, states require from 90% (three states) to 100% (three states) mortar cube strength, however most states specify a 95% mortar cube strength requirement.

The acid insoluble residue test is run by only four states, including Texas. Texas, along with Oklahoma, has the strictest requirements at 60% insoluble by weight. The least restrictive requirements are specified by North Carolina and Ohio at 25%. In general, states require engineer approval for a fine aggregate depending on the application (friction surface or structural). Three states require natural sand on bridge decks and one state (Minnesota) requires all fine aggregates to be of natural origin.

The sand equivalent test is run by 11 states and the limit specified by Texas (80%) is the most conservative in the country but is also matched by two other states. The lowest allowable sand equivalent limit is Oregon at 68%. Most of the states that do not run sand equivalent specify a maximum loss by decantation.

Fineness modulus values are allowed to vary from 2.3 to 3.1 in the state of Texas which is very similar to requirements by most other states. The value of 2.3 is the lowest allowable fineness modulus by any state and the highest allowable fineness modulus is 3.5 (allowed by Connecticut and Michigan).

Other somewhat common fine aggregate specification limits included by state DOTs include an expansion limit for ASR related expansion, sodium sulfate soundness limit (typically 10% over 5-cycle test), and absorption (typically 2.0-2.5%). Canadian provinces specify maximum total deleterious content as well as Micro-Deval limits. Indiana requires fine aggregates to be tested in a 3% brine freeze-thaw test. Hawaii specifies a limit of 40% for LA abrasion testing of fine aggregates.

3.2.3 Summary

For the majority of tests, Texas aggregate qualifications and limits are very similar to other states. For coarse aggregate tests, TxDOT standards represent an approximately average value for LA abrasion, loss by decantation, sulfate soundness, and shale content. TxDOT coarse aggregate standards are less conservative for friable particles but more conservative for clay lumps when compared to other state specifications. The most common tests performed by other states are content of flat and elongated particles, ASR testing, and aggregate freeze-thaw testing.

For fine aggregate tests, TxDOT standards represent an approximately average value for organic impurities and fineness modulus. TxDOT fine aggregate standards are more conservative for clay lump content, acid insoluble residue, and sand equivalent value. The most common fine aggregate tests performed by other states are loss by decantation, ASR testing, and sulfate soundness testing.

3.3 Aggregate Workshop

An aggregate workshop was held on June 22, 2011 at the Pickle Research Campus in Austin, Texas to provide the researchers the opportunity to gather information from the experience of TxDOT personnel, a retired representative from the Ontario Ministry of Transportation and representatives from major aggregate producers in Texas. A complete list of workshop attendees can be found in the Appendix. This meeting provided for a free and open discussion of (1) current problems in portland cement concrete related to aggregates; (2) most important aggregate properties and tests to measure those properties; (3) number and types of aggregates to be used in the study; (4) the appropriate methods for establishing performance test limits and criteria; and (5) determining the most appropriate test methods to establish the required performance of aggregates in concrete. During this meeting, attendees were able to discuss past failures observed in concrete pavements and structure as well as to discuss the properties and tests that would be most useful in screening for quality materials. The research team was present during the discussion, but did not make comments in order to provide an unbiased discussion between the producers and TxDOT.

From a TxDOT perspective, a failure should be defined as a distress that causes money to be spent on repair or replacement earlier in the concrete life than anticipated. For example, even though minor pop-outs may be only a cosmetic issue, they must eventually be dealt with and would therefore be considered a failure. There have not been very many concrete failures in TxDOT projects that can be directly attributed to aggregates. There are perhaps two main explanations for this. The first explanation is that it is very difficult to pinpoint the cause of a concrete failure because of the composite nature of the material. The second explanation may be that the Aggregate Quality Monitoring Program (AQMP) has been successful in ensuring that good quality aggregates are used in TxDOT projects, and therefore failures are rare. Despite the general success of concrete aggregate usage by TxDOT, it is possible that current specifications are too conservative and preclude the use of good aggregates around the state. The desire for differing specifications based on application was suggested. The main categories would be for structural needs and paving needs. Limits and testing would need to be established that would best predict and screen for materials to be used in these applications.

One aggregate issue in Texas includes excessive cracking in continuously reinforced concrete pavements (CRCP) using siliceous river gravels, likely due to the high coefficient of thermal expansion (COTE) of this aggregate type. This issue has primarily occurred in the Houston District. Because of this problem, many districts have banned the use of river gravels in CRCP. However, the Fort Worth district has successfully used river gravels blended with 50% limestone in CRCP projects with no issues. Current research is investigating mitigation options for CRCP projects which use river gravels. TxDOT is currently in the process of introducing a statewide COTE requirement for CRCP projects.

Other concrete issues around the state include freezing and thawing in the Panhandle and D-cracking which has been identified at the Abilene Airport. However, these aggregate sources were later abandoned because of these problems. There have also been isolated incidents of polishing when carbonate fine aggregate was used, e.g. on

I-35 near San Antonio and in the Dallas and Fort Worth area, which was a 100% carbonate fine aggregate pavement. In areas where high volume paving was done and mass concrete was placed, issues with heat generation and management have been seen; this problem typically results in thermal cracking. Issues with aggregate thermal conductivity seem to have been a likely cause.

One specific example of a concrete failure due to an aggregate was in the Dallas District in Collin County where an aggregate from southern Oklahoma (Lattimore Stringtown) was used in a bridge deck. This aggregate had pyrite, shale, and asphaltic material which made it perform very poorly in service. Aggregates with high contents of pyrites and other sulfides should be avoided. Aggregates with high shale content should be avoided as well. Producers can usually deal with shale during processing but this process can sometimes be tricky. If the shale is not handled correctly, an aggregate with a 0.4% decant at the quarry can result in a 1.0% or higher decant when the material reaches the ready mix plant.

The use of optimized gradation was highly supported by both producers and many of the TXDOT district personnel. It was commented that reductions of 1 sack of portland cement per cubic yard could be achieved by using optimized gradation. One comment made, however, suggested that the extra testing required for optimized gradations are often complicated and either not run or run incorrectly. One major problem concerning optimized gradation is the lack of storage bins at ready mix plants and hesitation of plants to have multiple aggregate piles.

Once the issue of common concrete distresses had been discussed, a list of the material properties and corresponding test methods was developed to provide a basis for selecting tests to be performed to screen aggregates. Properties that were given high priority by workshop attendees included combined gradation (more important for producer than buyer), resistance to degradation (can be measured by aggregate impact value, LA abrasion, and Micro-Deval), shape characteristics (can be measured by AIMS), texture (can be measured by AIMS), strength (important for structures – can be measured by compression point load index and concrete cylinder compression), modulus of elasticity (of concrete), coefficient of thermal expansion (CoTE), modulus of elasticity (of aggregate), freezing and thawing behavior (can be measured by modified ASTM C 666, Iowa pore index, Canadian freeze-thaw, and sodium or magnesium sulfate soundness), resistance to dimensional change (can be measured by wetting/drying cycles, Canadian unconfined freeze-thaw, CoTE, creep testing), resistance to abrasion (Micro-Deval with AIMS), lack of objectionable substances (ex - chloride ions, sulfides, and clays), skid resistance (acid insoluble test), thermal conductivity, and petrography. Other properties and methods that were mentioned by attendees were considered less important and thus given lower priority. These properties and methods included: discrete measurements of decantation, the difference between TxDOT gradation and ASTM C33, strength (less important for pavements), absorption, and chemical resistance.

After the discussion of aggregate properties and relevant test methods, a discussion was held to determine the number and types of aggregates to collect for the study. The original aggregate list provided by TxDOT for the project was created to encompass a good representation of the Texas geology (Edwards formation, etc.) however, it was stated that more materials from Edwards formation may be required due to the complex geologic formations found within. Additionally, there should be special interest taken in materials that are relatively new to use in Texas such as granites and dolomites. Attendees suggested that it may be possible to get bad sources from other states such as D-cracking susceptible aggregates from Michigan or Kansas.

Next, a discussion concerning the procedure for establishing the limits for use with the new tests was conducted. Chris Rogers, formerly with the Ontario Ministry of Transportation, provided crucial insight on how Ontario established limits and also addressed some recurring problems encountered during the process. Several studies were conducted involving numerous tests and also determined known performance of existing concrete through surveys and field visits. One common problem was that if a source yields poorly performing aggregates, it tends not to be reused. Therefore, it is sometimes difficult to gather enough information about these sources. Rogers also emphasized that Ontario had different limits for different classes of concrete; however it is important to remember that Ontario has very different environmental conditions than Texas. Other ideas and advice provided by attendees during this discussion included correlating AIMS data (shape and texture) to concrete strength of volumetrically constant mixture designs, the need for cubical aggregates during testing to eliminate erroneous results due to flakey aggregates, and the necessity for petrographic examination.

At the end of the day, the workshop focused on selecting the best tests to be performed during this project. This discussion focused on identifying tests that would be valuable to run from an academic standpoint as well as tests that would be important to have for incorporating into a new test standard. During the discussion, an agreement was reached between the researchers and the project management committee that Los Angeles abrasion testing, and magnesium sulfate soundness testing would be conducted by TxDOT, since these two tests are not very good predictors of performance and will likely be excluded from future specifications. Additional testing by the research team will be selected to offset the work that would no longer be required (LA abrasion, magnesium sulfate soundness, and petrographic examination).

3.4 Finalizing Testing Plan

The research team met with TxDOT project committee members to discuss the testing plan in the context of the literature review and the suggestions provided by attendees of the aggregate workshop. For the most part, suggestions made at the aggregate workshop were supported by project committee members but a few changes and revisions were made with the goal of creating the most relevant and complete testing plan possible.

Although the Canadian unconfined freeze-thaw test has shown promise as a good indicator of field performance for coarse aggregates, project committee members felt that its only slight improvement in prediction of performance compared to magnesium sulfate soundness (according to results from ICAR 507) and its limited relevance to Texas, which has very little freeze-thaw exposure, was not enough to justify its inclusion in the final testing plan. Similarly, the Iowa Pore Index test, which has been shown to potentially identify aggregates susceptible to D-cracking, was deemed too costly and not relevant enough to Texas environmental conditions to pursue further. Although LA abrasion and magnesium sulfate soundness tests have shown limited correlation to field performance, they were desired to be included in the final testing plan since they are currently part of Item 421 specifications. Decantation was eliminated from the testing plan because TxDOT has already gathered a significant amount of relevant data in an internal study. Project committee members were somewhat skeptical of the value and importance of the aggregate impact value test and the aggregate crushing value test, but suggested that these tests be run on approximately fifteen aggregates to determine potential of these tests before investing the time to test all coarse aggregates sources. The Micro-Deval, thermal conductivity, chemical composition (X-ray diffraction), AIMS 2.0, specific gravity and absorption tests were all approved unanimously.

Fine aggregate tests supported for inclusion in the final testing plan include Micro-Deval, AIMS 2.0, chemical composition (X-ray diffraction), acid insoluble residue, specific gravity, and absorption. The organic impurities and sand equivalent tests were added to the final testing plan because since they are currently part of Item 421 specifications. Test methods recently developed were also added to the final testing plan to determine their potential for future use. These tests are the flakiness test developed by Chris Rogers and the Grace Methylene Blue test which uses a colorimeter.

Concrete tests were included in the final testing plan due to the limited data gathered from district surveys about specific aggregate performance. These tests include compressive strength testing, flexural strength testing, coefficient of thermal expansion, and modulus of elasticity.

The final testing plan is listed in Table 7. Descriptions of these test methods are provided in Chapter 5: Laboratory Testing.

Table 7: Finalized Testing Plan

Coarse Aggregate Tests	
LA Abrasion	*Tex-410-A*
Magnesium Sulfate Soundness	*Tex-411-A*
Micro-Deval	*Tex-461-A*
Thermal Conductivity	
X-ray Diffraction	
AIMS 2.0	
Specific Gravity and Absorption	*Tex-403-A*
Aggregate Crushing Value	*BS 812.110*
Aggregate Impact Value	*BS 812.112*

Fine Aggregate Tests	
Micro-Deval	*ASTM D 7428*
AIMS 2.0	
X-ray Diffraction	
Grace Methylene Blue	
Organic Impurities	*Tex-408-A*
Acid Insoluble Residue	*Tex-612-J*
Specific Gravity and Absorption	*Tex-403-A*
Flakiness	*Rogers (2008)*
Sand Equivalent	*Tex-203-F*

Concrete Tests	
Compressive Strength	*Tex-418-A*
Flexural Strength	*Tex-448-A*
CoTE	*Tex-428-A*
Modulus of Elasticity	*ASTM C 469*

Chapter 4: Materials Acquisition

4.1 Selection of Aggregates

A survey of TxDOT district personnel revealed that most districts have access to at least a few aggregate sources within their district, with the exception of a few districts in eastern Texas. Most districts are able to use somewhat local resources, though some districts are forced to ship aggregates in from other districts or other states.

As a large, geologically diverse state, Texas has a tremendous amount of aggregate resources, particularly of the carbonate variety. Limestones, dolomitic-limestones, and dolomites are common throughout much of Texas, as are river gravels on a regional level. Quality of limestones and dolomites varies by quarry and region, primarily due to geologic features of the Edwards formation (Fisher & Rodda, 1969). The composition of river gravels varies widely by region and river basin but may contain sedimentary, metamorphic, or igneous aggregates. Many river gravels are composed of predominantly carbonate and siliceous rocks, weathered by water. Although less common, there are also a few sandstone, granite, and rhyolite sources in Texas and nearby in Oklahoma.

A list of aggregates to test was suggested by TxDOT geologists and engineers to capture a representative sample of lithologies and geographies. The list was created with the understanding that the research team would be responsible for collecting aggregates close to Austin (the location of the laboratory testing) and TxDOT personnel would assist in the collection and delivery of aggregate sources far from Austin. The list went through several iterations to ease logistical transportation constraints and ensure that aggregates could be acquired or delivered in a timely manner. The final list of aggregates also took advantage of the fact that several aggregate samples were already present at the

Construction Materials Research Lab at The University of Texas due to testing by other research projects.

Some of the selected aggregates will be used to make concrete for mechanical testing. The aggregate sources on the concrete list were rated as “good”, “moderate”, or “poor”, depending on how consistently they meet specification requirements. As a result, some aggregates on the testing list are members of the TxDOT Aggregate Quality Monitoring Program (AQMP), as discussed in the following section, and some are not.

4.1.1 TxDOT Aggregate Quality Monitoring Program (AQMP)

The Aggregate Quality Monitoring Program (AQMP) was created by TxDOT in 1977 to accelerate the acceptance of aggregate sources for use in TxDOT projects and to improve the overall efficiency of TxDOT operations. The AQMP involves quality monitoring (as the name implies), testing, and statistical analysis of aggregates to ensure consistency and compliance with specifications. TxDOT certifies personnel in both sampling and testing to ensure fair and representative data. To be accepted in the AQMP, producers must provide a test history of at least five TxDOT project samples produced one month apart within the last two years and allow periodic quarry/pit inspection and testing of materials by TxDOT personnel for each quarry or pit. Acceptance in the AQMP means that an aggregate source can bypass strenuous and time consuming testing typically required at the beginning of TxDOT projects. Once accepted in the AQMP, the aggregate source is monitored frequently for material consistency, material quality, production trends, production rate, frequency of use in TxDOT projects, and test results (Texas Department of Transportation, 2007).

The AQMP lists (one list for portland cement concrete approved sources and one list for bituminous approved sources) are published semi-annually, and periodically include new data about the aggregate sources. Initially, only polish value ratings were available on the AQMP document, but in 1994, TxDOT began publishing data for LA abrasion and magnesium sulfate soundness (Jayawickrama, Hossain, & Hoare, 2007). Recent iterations of the AQMP document have added data for sources for Micro-Deval, coefficient of thermal expansion, alkali-silica reactivity for coarse aggregates, and acid insoluble residue, and alkali-silica reactivity for fine aggregates. However, not all data are available for every source and a disclaimer states that this information is only for reference. All sources are now also categorized by material type (i.e. partly crushed siliceous and limestone gravel).

4.2 Aggregate Collection

The list of aggregates to be tested as part of this research project was created with the understanding that the research team would be responsible for collecting aggregates close to Austin (the location of the laboratory testing) and TxDOT personnel would assist in the collection and delivery of aggregate sources far from Austin. Districts close to Austin included the Austin, Waco, Bryan, and San Antonio districts. A TxDOT district map is displayed in Figure 16.

For each aggregate source members of the research team asked employees a basic set of questions to obtain more information about the material. When possible, the research team spoke with quality control managers who have a good idea of material availability and quality. Questions directed to these personnel fostered discussions about what types of materials are available on a regular basis, problems meeting TxDOT specifications, and history and use of the products. Specific collection strategies are discussed in the following sections.

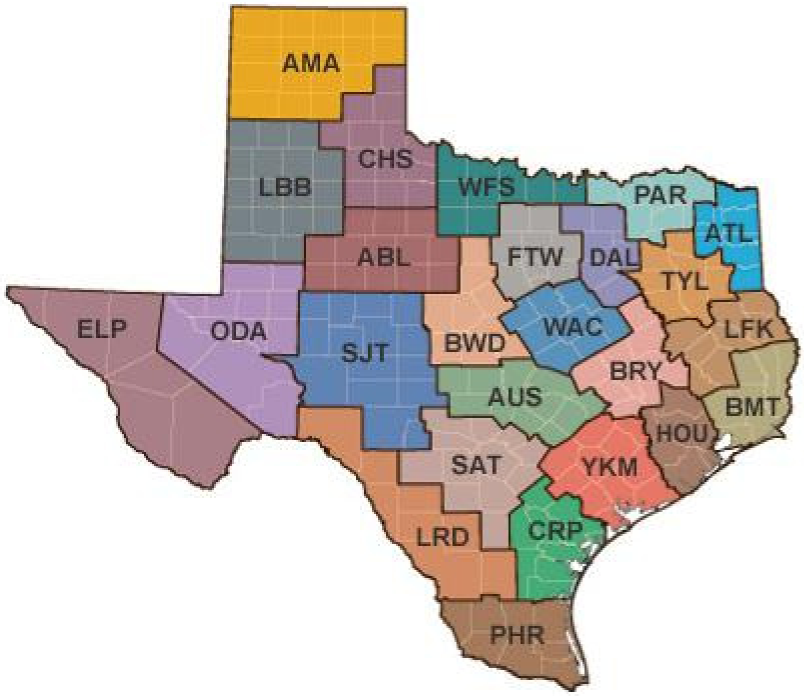

Figure 16: State Map of TxDOT Districts

4.2.1 Collection of Aggregate Sources Close to Austin

Aggregates located within a three hour drive of Austin were considered "close", and therefore the research team's responsibility to acquire. At least 10 sources on the final list of aggregates were located in the Ausitn district, 8 in the San Antonio district, and 5 more in the Waco district. For many of these sources, two or three members of the research team traveled to the quarry or pit to personally sample the material.

After arriving on location, the research team viewed and acknowledged safety procedures and then discussed which materials would be best to sample for the project with producer personnel. A producer employee, often the quality manager, then guided the research team to the material of interest. This material was often located in large stockpiles on site. Research members proceeded to obtain sample material, following sampling procedures outlined by TxDOT in the Tex-400-A (*Sampling Flexible Base,* *Stone, Gravel, Sand, and Mineral Aggregates*) specification. Understanding of and compliance with this sampling standard ensured that a representative sample of material was obtained for testing. If a front-end loader was available, the research team instructed the operator to approach the stockpile and cut into the stockpile from bottom to top in one continuous cut, exposing a clean, interior vertical face. The material from the first cut was discarded and the front-end loader operator then cut into the newly exposed face of the stockpile and lowered material from this cut to the ground. This process was repeated several times, until a large sample of material was available at ground level. The research team then took flat-nosed shovels and loaded the material of interest into 0.3-ft^3 (0.009-m^3) TxDOT approved canvas bags or 8-ft^3 (0.23-m^3) *Super Sacks®*, pictured in Figure 17. If a front-end loader was not available, research team members used shovels and 5-gallon buckets to sample material at quarter points of the stockpile, with sample points at bottom, middle and top of the stockpile for a total of 12 points. This material was then transferred to the TxDOT approved canvas bags or 8-ft^3 (0.23-m^3) *Super Sacks®*. The canvas bags or *Super Sacks®* were then loaded onto a trailer towed by the laboratory truck. Researchers proceeded to sample nearby stockpiles in the same manner if interested in additional material such as fine aggregate or crusher screenings. After sampling material at the first few sources, the research team decided that it would be more efficient to unload and handle material at the laboratory if it was in 8-ft^3 (0.23-m^3) *Super Sacks®*. These *Super Sacks®* were obtained from *B.A.G. Corp* and are made of woven polypropylene fibers. Handles on the tops of the *Super Sacks®* allow a forklift to easily lift and transport a full bag.

Figure 17: “Super Sack” Used to Collect and Transport Aggregates

To save time and reduce transportation costs, the research team contacted the Texas Aggregate and Concrete Association (TACA) in order to obtain assistance in gathering materials. The research team, with generous assistance from TACA and TxDOT employees, devised a plan in which the research team would ship “super sacks” to aggregate producers, who would then sample and transport material to a local TxDOT yard. A TxDOT truck would then travel to the local TxDOT yards and obtain the aggregates and transport them to Austin for testing. The research team is extremely grateful for everyone involved in this process (TACA, TxDOT, and producer personnel) which allowed the acquisition of material to be expedited and more cost-effective.

4.2.2 Collection of Aggregate Sources Far From Austin

Aggregates located farther than three hours were acquired in a similar manner to aggregates located close to Austin that were collected with assistance from TACA and TxDOT. Again, the research team shipped “super sacks” to aggregate producers, who then sampled and transport material to a local TxDOT yard. A TxDOT truck would then travel to the local TxDOT yards and obtain the aggregates and transport them to Austin for testing. However, prior to shipping the “super sacks”, the research team contacted producers to ask employees the same basic set of questions to obtain more information about the material. When possible, the research team spoke with quality control managers who have a good idea of material availability and quality. Questions directed to these personnel fostered discussions about what types of materials are available on a regular basis, problems meeting TxDOT specifications, and history of use of the products.

4.2.3 Collection Status-to-Date

As of April 2012, the research team has acquired 24 coarse aggregates and 38 fine aggregates (including material available at the laboratory through previous projects), representing 48 unique sources, 11 TxDOT districts, and a variety of lithologies. Several aggregate samples from additional sources are expected to arrive at the laboratory for testing in the next few weeks.

Chapter 5: Laboratory Testing

5.1 Coarse Aggregate Tests

Coarse aggregate tests were selected based on a review of literature, a review of other state DOT specifications, and a discussion with personnel in industry and academia. Chapter 2: Review of Literature and Chapter 3: Development of Testing Program contain more information about the selection of aggregate tests. Because this research project is funded by TxDOT, TxDOT standards were used when possible. Otherwise, ASTM standards or other widely accepted test methods were used to ensure repeatable and consistent results. Members of the research team were certified on several tests by TxDOT to ensure that standard procedures were followed correctly. Refer to Table 7: Finalized Testing Plan to view the complete list of coarse aggregate tests performed.

In the following sections, the test methods are listed and followed by a general background, description of procedures, documentation of precision, and comments regarding the approach of the research team for each test.

5.1.1 Micro-Deval Test for Coarse Aggregates

French researchers developed the Micro-Deval test which subjects water, coarse aggregate, and steel charge to approximately 12,000 revolutions in a steel drum via a ball mill roller. Researchers in Canada further modified this test to allow for a larger aggregate sample size and slightly altered dimensions of the steel charge and drum (Rogers C. , 1998). Today, most Micro-Deval specifications are based on the standards documented by the Canadian researchers. The standard used by this research project was Tex-461-A (*Degradation of Coarse Aggregate by Micro-Deval Abrasion*).

The Micro-Deval test requires that the aggregate sample be washed and dried prior to testing. A standard gradation is specified for concrete aggregate for this test and provided in Table 8. The total mass of the sample should be 1500 ± 5-g.

Table 8: Coarse Aggregate Gradation Required for Micro-Deval Test (Tex-461-A)

Sieve Size	Target Mass
3/4" - 1/2"	660 ± 5 g
1/2" - 3/8"	330 ± 5 g
3/8" - 1/4"	330 ± 5 g
1/4" – No. 4	180 ± 5 g

Once the sample has been weighed out, 5000 ± 5-g of stainless steel balls, of diameter 9.5 ± 0.5-mm, should be placed in the Micro-Deval container. The Micro-Deval container is a small stainless steel drum and can be seen in Figure 19. The aggregate sample can then be placed in the Micro-Deval container and soaked in 2000 ± 500-mL of tap water at 20 ± 5˚C (68 ± 9°F) for a minimum for one hour. After one hour, the Micro-Deval container is sealed and placed in the Micro-Deval machine. The Micro-Deval machine is a simple ball mill roller and can be seen in Figure 18.

Figure 18: Micro-Deval Test Machine

The operator sets the appropriate time on the machine such that it will run for 120 ± 1 minute. The machine should be calibrated to revolve at 100 ± 5 revolutions per minute and the final revolution count should be 12,000 ± 600 revolutions. After two hours of revolutions, the Micro-Deval container is removed and the contents are washed over a No. 16 (1.18-mm) sieve (see Figure 19). Material passing the sieve is discarded. The retained material is oven-dried to a constant weight and weighed after drying. The oven-dried weight is recorded and compared to the original weight to get a percent loss calculation for the final Micro-Deval loss.

Figure 19: Sample Ready to be washed after Micro-Deval Test

The research team performed two Micro-Deval tests per source and recorded the mean loss unless the test results were different by more than 1.0%. In this case, a third Micro-Deval test was performed and the mean of the three tests was considered acceptable unless an outlier existed. Conditions of the inside surface of the steel drum should not affect results for testing of coarse aggregates, but the research team used a local limestone aggregate (Alamo Weir) as a control to monitor test conditions over time (Rogers C. , 1998). Research has shown the Micro-Deval to be a very consistent test, but standard deviation increases with higher loss materials (Jayawickrama, Hossain, & Hoare, 2007). Figure 5, in Section 2.3.1.1 Micro-Deval Test for Coarse Aggregates, displays typical standard deviation results for this test based on average loss. The

AASHTO standard of this test also provides information about the multi-laboratory precision of this test method, and is found in Table 9, taken from AASHTO T 327-09.

Table 9: Multi-Laboratory Precision Values for Micro-Deval Testing of Coarse Aggregate (AASHTO T 327-09)

Aggregate Abrasion Loss, Percent	Coefficient of Variation, Percent of Mean[a]	Acceptable Range of Two Results, Percent of Mean[a]
5	10.0	28
12	6.4	18
17	5.6	16
21	5.3	15

[a] These numbers represent, respectively, the (1s percent) and (d2s percent) limits as described in ASTM C 670.

5.1.2 Specific Gravity and Absorption Test for Coarse Aggregates

The specific gravity and absorption test requires that a representative sample of an aggregate be obtained and soaked for 15 to 24 hours, depending on the exact standard used. The aggregate is then dried to saturated surface dry state, weighed in a calibrated pycnometer, and dried to an oven-dry state as described by the following procedures. The standard used by the research team was Tex-403-A (*Saturated Surface-Dry Specific Gravity and Absorption of the Aggregates*).

After a representative sample of aggregate is obtained, the aggregate is soaked (in a non-metal tub to avoid reaction) to ensure that all permeable pores of the aggregate become filled with water. The water should be at a temperature of 23 ± 2˚C (73 ± 3˚F). After the soaking period, the operator removes the aggregate and places it on a towel or cloth to absorb free moisture. The operator should dry the aggregate with the towel or cloth until no moisture can be seen on the surface of the aggregate particles. At this point, the aggregate has reached the saturated surface dry (SSD) condition where the outside surface of the aggregate has no free water but all internal pores remain filled with water. The aggregate is then placed in a container with a lid so that free evaporation does not remove additional moisture from the aggregate. This step should be repeated until approximately 1500-g (3.31-lb) of aggregate in the SSD state has been obtained and the weight of this sample recorded.

While the SSD aggregate is covered, the mass of a controlled volume of water must be measured. The controlled volume of water is obtained by filling a calibrated pycnometer with water at 23 ± 2˚C (73 ± 3˚F). The pycnometer is a 2000-mL mason jar with a metal pycnometer cap. A rubber bulb or syringe is used to fill the pycnometer with water until a rounded bead of water can be seen on top of the pycnometer cap. The outside of the pycnometer is dried and the pycnometer is weighed and this weight recorded. The pycnometer cap can be removed and the jar can be emptied until about ¼ of the water remains. At this point, the SSD aggregate is placed in the pycnometer. The operator should take extra care in placing aggregate in the pycnometer so that all material is accounted for. The jar is filled with water at 23 ± 2˚C (73 ± 3˚F) until it reaches the brim. The pycnometer should be agitated so that any entrapped air is freed. The pycnometer cap is again placed on the jar and a rubber bulb or syringe is used to fill the pycnometer until a rounded bead of water can be seen on top of the pycnometer cap. The outside of the pycnometer is dried and the pycnometer with aggregate is weighed and this weight recorded.

Finally, the aggregate is removed from the pycnometer, placed in a pan, and the pan placed in an oven where the aggregate is dried to constant mass. The weight of the aggregate at SSD, the weight of the pycnometer with water, and the weight of the pycnometer with aggregate are used to calculate the specific gravity and absorption capacity of the aggregate. Specific gravity is typically reported to the nearest 0.01 and absorption to the nearest 0.1% (Texas Department of Transportation, 1999).

According to the ASTM standard of this test (ASTM C 127), the standard deviation is 0.007 for SSD specific gravity for single-operator, and 0.011 for SSD specific gravity for multi-laboratory testing. The acceptable range of two results is 0.020 and 0.032 respectively (ASTM International, 2007).

Because this test is somewhat variable, the research team ran at least two tests per aggregate source. If the results of these two tests deviated more than the amount dictated by the ASTM C 127-07 standard, a third test was run and an average value taken of all three tests unless one test was determined to be an outlier.

5.1.3 Los Angeles Abrasion and Impact Test

The Los Angeles abrasion and impact test is performed on a sample of approximately 5000 g of aggregate. The aggregate sample, along with several large stainless steel balls, are loaded in the Los Angeles machine (pictured in Figure 20) and rotated approximately 500 times at about 33 rpm. The aggregate sample is then removed and sieved over a No. 12 (1.70-mm) sieve. The mass of material loss is determined as a percentage of the original sample mass to get the final LA abrasion results. The standard used by the research team was Tex-410-A (*Abrasion of Coarse Aggregate Using the Los Angeles Machine*), which simple refers to procedures described in ASTM C 131 (*Resistance to Degradation of Small-Sized Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*) and is described as follows.

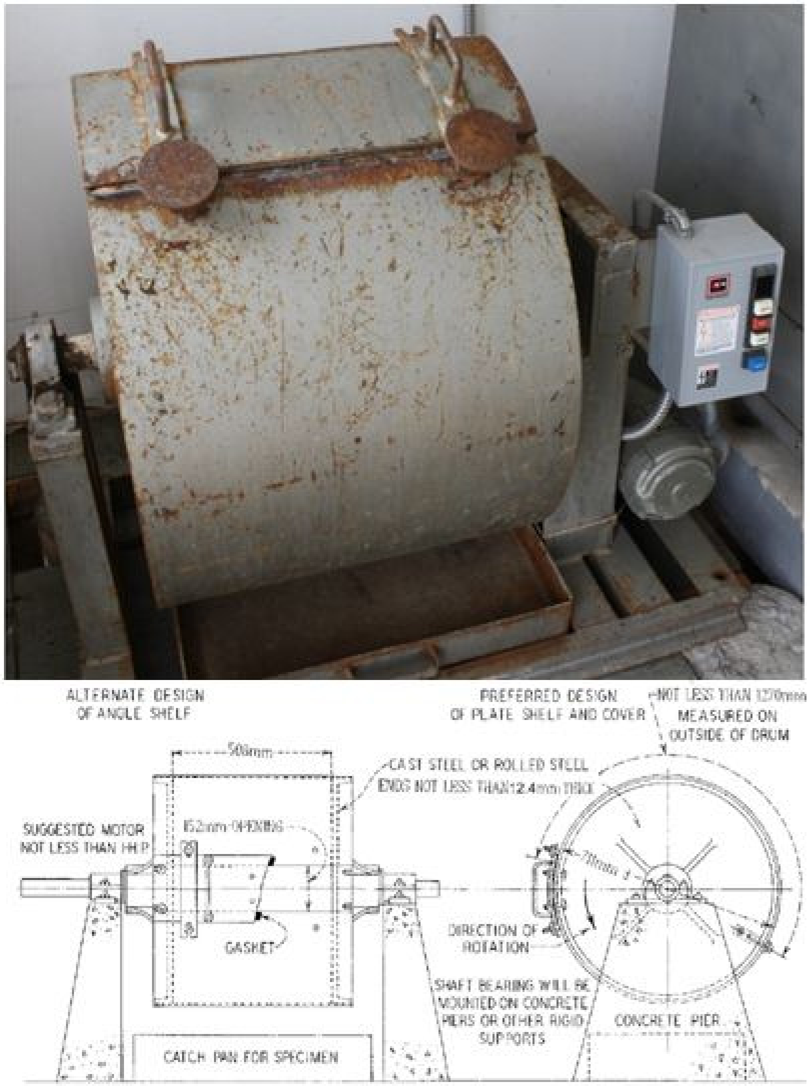

Figure 20: LA Abrasion Test Setup (Diagram from ASTM C 131)

The Los Angeles machine is a large, rotating steel drum with an inside diameter of 711 ± 5 mm (28 ± 0.2 in.) and inside width of 508 ± 5 mm (20 ± 0.2 in.). The steel walls of the machine are no less than 12.4-mm (0.49-in.) thick. A steel shelf is also located inside the drum and protrudes 89 mm (3.5 in.) into the drum. The function of the steel shelf is to cause the aggregate and steel shot to be lifted and dropped repeatedly as the drum rotates. A steel plate, with handles and a gasket seal, functions as the access point for the drum through which the aggregate and steel shot is placed. The Los Angeles machine should be calibrated to run at 30-33 revolutions per minute. The Los Angeles machine setup is displayed in Figure 20.

Prior to performing the LA abrasion test, a representative sample of aggregate must be obtained, washed, and oven-dried. The LA abrasion test can be performed on several sample gradations, so the amount of each size fraction necessary will depend on the gradation. This project used ASTM C 131 Grade B, which consists of 2500 ± 10 g of aggregate passing the 19.0-mm (¾-in.) sieve and retained on the 12.5-mm (1/2-in.) sieve, and 2500 ± 10 g of aggregate passing the 12.5-mm (1/2-in.) sieve and retained on the 9.5-mm (3/8-in.) sieve. This grading requires that 11 steel spheres be placed in the Los Angeles machine along with the aggregate. The steel spheres should be approximately 47 mm (1.9 in.) in diameter and weigh between 390 g and 445 g. The total mass of the steel shot should thus be 4584 ± 25-g.

Once the correct mass of aggregate and steel shot is obtained, the operator places both materials in the Los Angeles machine. A counter is set at 500 revolutions and the machine is started. After the prescribed number of revolutions, the operator places a catch pan beneath the machine and removes the contents. The steel spheres are cleaned, removed, and placed aside for the next test. The aggregate sample is then sieved over a No. 12 (1.70-mm) sieve and the finer material is discarded. The remaining aggregate is weighed and this weight is recorded as the final mass. The operator may wet-sieve the material, but this is only necessary if dust remains adhered (and accounts for more than 0.2% of original mass) to the aggregate particles after initial sieving. The final LA abrasion loss is calculated as a percentage of mass lost compared to the original sample.

According to ASTM C 131-06, the single-operator coefficient of variance is approximately 2.0%, and thus two tests run by the same operator should not differ by more than 5.7%. The multi-laboratory coefficient of variance is 4.5% and thus two tests should not differ by more than 12.7% (ASTM International, 2006).

To expedite testing, TxDOT construction materials laboratory volunteered to test aggregates for the LA abrasion test and provide results to the research team. Members of the research team prepared samples and delivered samples in batches to the TxDOT laboratory.

5.1.4 Magnesium Sulfate Soundness Test

The sulfate soundness of aggregates can be measured by using a magnesium sulfate solution or a sodium sulfate solution. This test involves cycles (typically five) of soaking an aggregate in a sulfate solution and then drying the aggregate. This test was originally developed in 1828 to simulate freezing of water in stone before refrigeration was controllable and widely available (Rogers, Bailey, & Price, 1991). The idea was to simulate crystallization pressures of ice formation during freezing and thawing events by causing salt crystals to form during the heating stage of this test (Folliard & Smith, 2002). Of the state DOTs that do use the sulfate soundness test, twenty-eight states specify the use the use of sodium sulfate, nine states specify the use of magnesium sulfate, and two states allow the use of either magnesium sulfate or sodium sulfate. Eleven states do not use sulfate soundness testing. The standard method used for this project was Tex-411-A (*Soundness of Aggregate Using Sodium Sulfate or Magnesium Sulfate*).

Prior to performing the magnesium sulfate soundness test, a representative sample of aggregate must be obtained, washed, and oven-dried. The gradation of the sample to be tested for concrete aggregates is listed in Table 10. The magnesium sulfate solution must also be prepared in advance and allowed to sit for 48 hours prior to testing. The solution is prepared by adding magnesium sulfate in anhydrous or crystalline hydrate form to water at a temperature of at least 25˚C (75˚F) until the solution is beyond saturated (evident by presence of excess crystals). After allowing the solution to sit and cool to room temperature, the specific gravity of the magnesium sulfate solution should be between 1.295 and 1.308.

Table 10: Concrete Aggregate Size Fractions for Magnesium Sulfate Soundness Testing (Tex-411-A)

Sieve Size	Target Mass
¾-in. - ½-in.	670 ± 10 g
½-in. - 3/8-in.	330 ± 5 g
3/8-in. - ¼-in.	180 ± 5 g
¼-in. - No. 4	120 ± 5 g

After the solution and aggregate sample have been prepared, the operator places each fraction of aggregates in individual containers with holes (to allow for fluid circulation and draining) and submerges the container in the solution for 16 to 18 hours. The solution should be maintained at 20˚C to 24˚C (68˚F to 75˚F). After the submersion period, the sample is removed, drained for 15 min, and dried to constant mass in an oven. After drying, the sample is removed from the oven, allowed to cool to room temperature, and placed back in the magnesium sulfate solution. This process is repeated for a total of five cycles. After the fifth cycle, the sample is removed from the oven, allowed to cool to room temperature, and then washed by circulating hot water through the sample containers. After all salt is removed by washing, the aggregates are again dried in the oven. The aggregates are then sieved over their respective retained sieve sizes and a mass loss is calculated for each fraction size. This information is then used in combination with a normalized aggregate gradation (provided in Tex-411-A) to obtain the final magnesium sulfate loss.

As noted in the discussion of the literature review in Section 2.3.2.1 Sulfate Soundness Test, both sulfate soundness tests have fairly low precision and repeatability. According to the ASTM standard for the sulfate soundness tests, the coefficient of variation for a magnesium sulfate test (with total loss between 9% and 20%) is 25% (ASTM International, 2005).

To expedite testing, TxDOT construction materials laboratory volunteered to test aggregates for the magnesium sulfate soundness test and provide results to the research team. Members of the research team prepared samples and delivered samples in batches to the TxDOT laboratory.

5.1.5 Aggregate Impact Value

The aggregate impact value (AIV) is a British standard test method that subjects a confined aggregate sample to a falling impact load. A sample of aggregate, passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve, of approximately 500-g in mass (depending on the density) is placed in a steel cup 38-cm (15-in.) below a steel hammer the same diameter as the inside of the cup. The user drops the steel hammer, guided by vertical rods, and raises it 15 times and the sample is sieved over a No. 8 (2.36-mm) sieve. The mass loss is recorded and the final result of the test is mass loss by percentage of original mass. The specific test method used by this research project was British Standard 812 Part 112 (*Methods for Determination of Aggregate Impact Value*) and the procedures are described in this section.

The required apparatus to operate this test consists of a cylindrical metal base, a cylindrical steel cup, a metal hammer, and a means for raising and dropping the hammer in a controlled manner. The cylindrical metal base should be between 22 and 30-kg (48 and 66-lb) and should be affixed to a concrete block no less than 450-mm (17.7-in.) thick.

The cylindrical steel cup should have an internal diameter of 102 ± 0.5-mm (4.0 ± 0.02-in.), a depth of 50 ± 0.25-mm (1.97-in ± 0.01-in.), and should have a thickness of at least 6-mm (0.24-in.) The metal hammer should have a mass between 13.5 and 14.0-kg (29.8 and 30.9-lb) and should fit snugly inside the metal cup. The hammer should be supported such that it may freely slide vertically and drop from a height of 380 ± 5-mm (15.0 ± 0.2-in.). The test setup used by this research project is shown in Figure 21.

To begin the test, an oven-dry sample of aggregate, passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve, of at least 500-g in mass should be obtained and placed in the metal cup. The operator then uses a tamping rod to compact the aggregate by dropping the rod in a controlled manner 25 times at a height of 50-mm (1.97-in.) above the sample. Any aggregate that protrudes above the edge of the cup is removed. The mass should be recorded so that additional tests on the same aggregate source use the same mass. The user then places the cup in the testing apparatus below the hammer. The cup is secured to the testing apparatus and the operator raises and drops the hammer 15 times, waiting at least 1 second between drops. The cup is removed and then the aggregate sample is removed from the cup and sieved over the 2.36-mm (No. 8) sieve. The amount of material passing the 2.36-mm (No. 8) sieve divided by the original mass is the aggregate impact value (BSi, 1995).

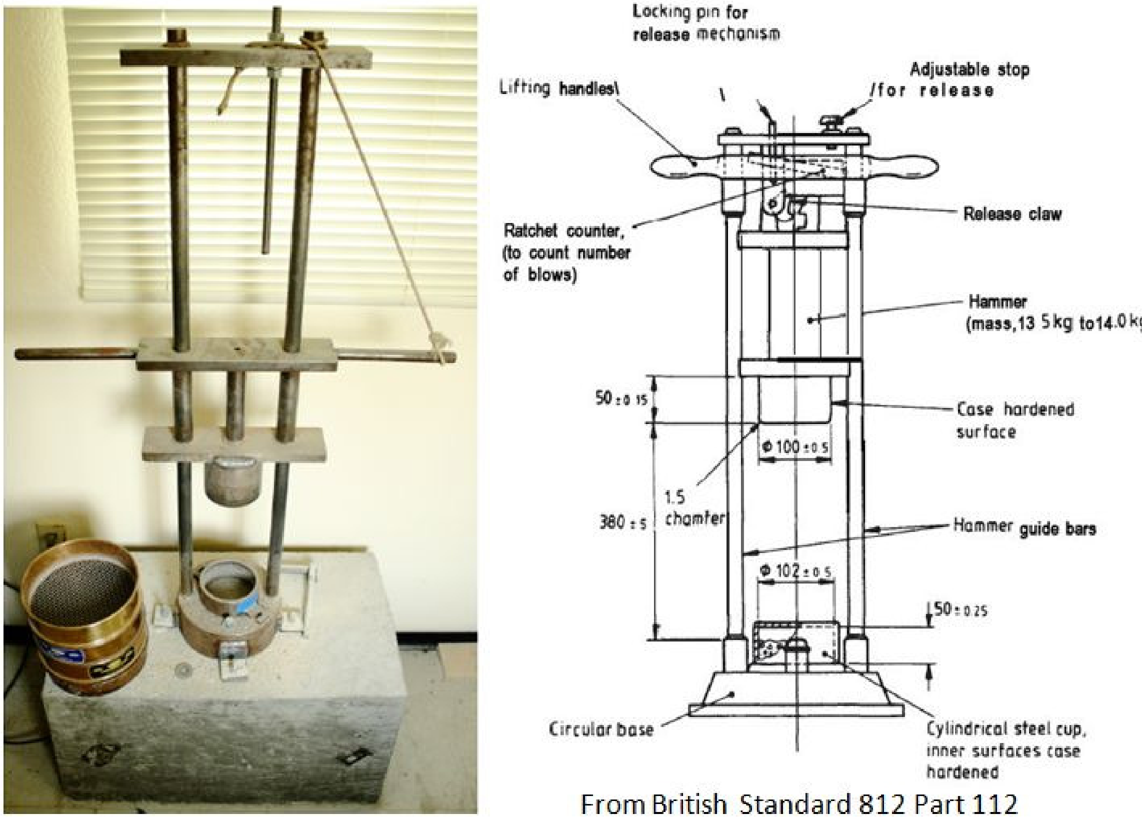

Figure 21: Aggregate Impact Value Test Setup

The aggregate sample may be tested in a dry or SSD condition. The research team tested aggregates in the dry condition and ran the test four times per sample. The mean value of the four tests was recorded as the AIV. Repeatability and reproducibility of this test vary by lithology and can be found in British Standard 812 part 112.

5.1.6 Aggregate Crushing Value

The aggregate crushing value (ACV) test is a British standard that subjects an aggregate sample to confined compression using a machine typically used for compression testing of concrete cylinders. Approximately 2500-g of aggregate passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve is placed in a cylindrical containment apparatus consisting of steel plates and a steel ring. The aggregate is compressed for approximately 10-min. at a constant load rate, until the force has reached 400-kN (90,000-lb). The standard test method used by this research project was British Standard 812 Part 110 (*Methods for Determination of Aggregate Crushing Value*) and the procedures are described in the following section.

The testing components for the aggregate crushing value include an open-ended steel cylinder, a plunger, and a base plate. The cylinder should have a diameter of 154 ± 0.5-mm (6.06 ± 0.02-in.), a depth of 125 to 140-mm (4.92 to 5.51-in.), and a wall thickness of at least 16.0-mm (0.63-in.). The plunger should have a diameter of 152 ± 0.5-mm (5.98 ± 0.02-in.) and an overall length of 100 to 115-mm (3.94-in. to 4.53-in.). The baseplate should be a square with side lengths of 200 to 230-mm (7.87 to 9.06-in.) and a minimum thickness of 10-mm (0.39-in.). These components are pictured in Figure 22.

Figure 22: Equipment Used for Aggregate Crushing Value Test

To begin the test, an oven-dry sample of aggregate, passing the ½-in. (12.5-mm) sieve and retained on the 3/8-in. (9.5-mm) sieve, of at least 1500-g in mass should be obtained. The cylinder should be placed on the base plate and aggregate is placed in the cylinder in three layers, each layer subjected to 25 tamps from a tamping rod. The total depth of the aggregate sample in the cylinder should be about 100-mm (3.9-in) after the three layers are added. The aggregate surface should be leveled and then the plunger is then placed on top of the aggregate. The entire apparatus is loaded into a compression testing machine (as shown in Figure 22) and the aggregate is compressed for 10 ± 0.5 min. at a constant load rate, until the force has reached 400-kN (90,000-lb). The operator then releases the load and removes the testing apparatus. The testing apparatus is disassembled and the aggregate is removed by tapping a rubber mallet on the side of the cylinder. The aggregate sample should be sieved over the 2.36-mm (No. 8) sieve. The amount of material passing the 2.36-mm (No. 8) sieve divided by the original mass is the aggregate crushing value. At least two tests should be performed and the mean recorded (BSi, 1990).

The research team performed this test three times per aggregate source and recorded the mean of the three tests as the aggregate crushing value. Repeatability and reproducibility of this test vary by lithology and can be found in British Standard 812 part 110.

5.1.7 Aggregate Imaging System (AIMS 2.0)

The aggregate imaging system (AIMS) is a machine consisting of a camera, lights, computer software, and movable trays, designed to capture and analyze the shape, angularity, and texture of coarse aggregates and the form and angularity of fine aggregates. The camera captures images of the aggregate particles , either lit directly or backlit, and the software analyzes these images and provides the user with data summarizing the shape characteristics. For coarse aggregates, the user places a set of aggregate particles on a transparent tray and places this tray into the machine. There are four different trays for each size fraction of aggregate. Figure 23 displays the AIMS machine and Figure 51, in Chapter 6: Discussion and Analysis, displays the aggregates placed on trays for testing. The AIMS machine rotates the tray three times to capture images of each particle. The first image captured is backlit such that only the aggregates particle's two-dimensional shape is captured, and this image is analyzed by the fundamental gradient method. The second image captured requires the camera to focus on the particle, thus allowing a particle height (as a function of focal length) to be determined, and three-dimensional analysis to be realized. The third image captures a close-up surface texture of the aggregate particles which is analyzed by the wavelet method. The data are output in a spreadsheet file and includes a distribution of shape characteristics. Specific algorithms and analysis techniques used by the machine and software are described by the developer, Eyad Masad, and are available through the Texas Transportation Institute (Masad, 2005).

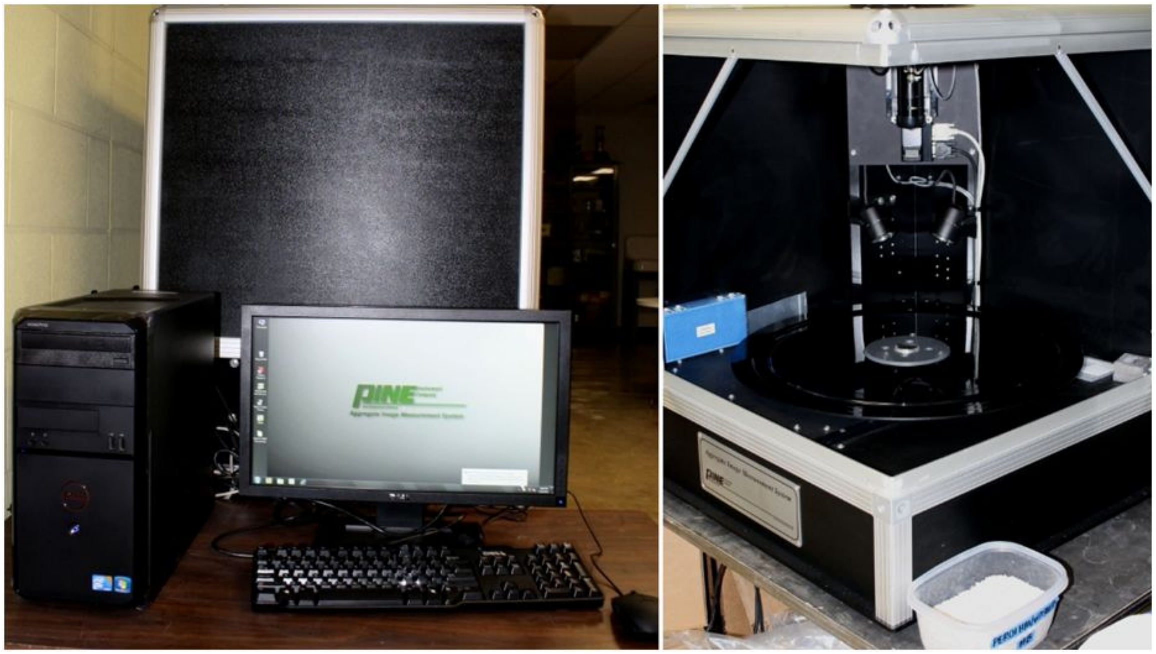

Figure 23: Setup of Aggregate Imaging System (AIMS 2.0)

For AIMS testing, the research team analyzed each aggregate source for all four size fractions before and after running the Micro-Deval test. The analysis before Micro-Deval testing provides general information about the aggregate and also provides a baseline to compare after Micro-Deval results to. The analysis after Micro-Deval testing provides data for change in shape, angularity, and texture. Some researchers have hypothesized that the change in shape, angularity, and texture will provide useful data for characterizing the abrasion resistance of an aggregate.

Mahmoud et al. (2010) found the AIMS to have a coefficient of variance of 11% for any give source (single operator) and a coefficient of variance of 5% for the same set of aggregates (single operator). Mahmoud et al. (2010) concluded that the effects of random placement of the same aggregate particles in different orientations had minimal effects on the angularity. In experiments with two AIMS machines (single operator) and multiple operators (single AIMS machine), the angularity measurements were found to be highly correlated (R^2 = 0.97 and R^2 = 0.98 respectively) as were texture measurements (R^2 = 0.97 and R^2 = 0.92), however slope and intercept for texture was not 1 and 0. Thus, it seems that the texture measurement is not as repeatable as the angularity measurement.

5.1.8 Flat and Elongated Particles

Many state DOTs use a proportional caliper to manually measure particles in an aggregate sample and compare the maximum dimensions of a particle to its minimum dimension. An aggregate source possessing an excess of particles above a 4:1 dimension ratio may lead to workability issues during concrete placement (Folliard & Smith, 2002). Some states incorporate this testing as a means of finding aggregates susceptible to breakage during compaction of HMA mixes. The most common standard of this test is ASTM D 4791. Researchers used the Texas DOT version of this test, Tex-280-F (*Determining Flat and Elongated Particles*) is and it is described in the following section.

A proportional caliper is required to run this test and is a device consisting of a metal base plate, a fixed vertical post, and a swinging arm. Additional vertical posts are located such that their radial distance from the pivot point of the swinging arm is twice, three times, four times, and five times the distance from the pivot point to the farthest vertical post. The swinging arm allows the operator to adjust the proportional caliper for as necessary for each particle measured. A picture of the proportional caliper used by the research team is pictured in Figure 24.

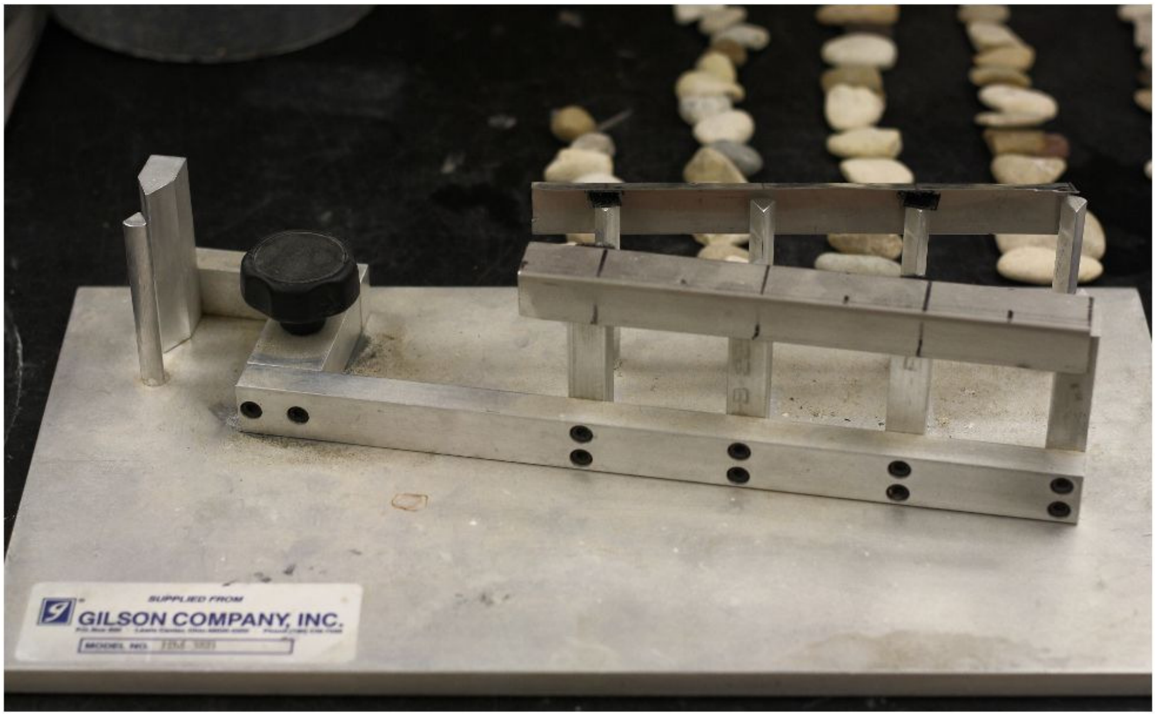

Figure 24: Proportional Caliper for Testing Flat & Elongated Particles

To begin the test, oven-dry samples of aggregate of each size fraction should be obtained. In this case, the research team measured aggregate particles retained on the ½-in. (12.5-mm), 3/8-in. (9.5-mm), ¼-in. (6.35-mm), and No. 4 (4.75-mm) sieves. Approximately 100 particles were tested for each size fraction, thus a total of approximately 400 particles was tested for each aggregate source. For each aggregate particle, the operator sets the left most opening to the minimum dimension, and then determines if the maximum dimension of the particle can pass through a critical opening on the right side of the proportional caliper. Typically, a critical ratio (e.g. 5:1) is selected and aggregate particles are considered "flat and elongated" if the maximum dimension perpendicular to the length and width cannot be placed through the opening matching the critical ratio. Once all particles are measured and classified as "flat and elongated" or "not flat and elongated", the flat and elongated percentage of each size fraction is multiplied by the percentage of each size fraction occurring in a sieve analysis of the original aggregate sample. This calculation provides a weighted average to obtain the final flat and elongated percentage for the aggregate source (Texas Department of Transportation, 2005).

For this research project, the operator placed each particle in a pile according to the dimension ratio it matched. For example, the most spherical particles would be classified as “2:1” and the most flat and elongated particles would be classified as “5:1” or “over”. This method of recording ratios for every particle provides more data for analysis. Instead of getting a final flat and elongated percentage for one critical ratio, the researchers can manipulate the data to obtain a flat and elongated percentage for any ratio. The precision of this test is described in the ASTM standard of this test (ASTM D 4791) and is shown in Table 11 (ASTM International, 2010).

Table 11: Precision Values for Flat and Elongated Particles Test, from ASTM D 4791-10

19.0-mm to 12.5-mm Flat and Elongated (Percent)

Precision	Test Result (%)	(1S) %	(D2S) %
Single Operator	2.7	51.2	144.8
Multilaboratory		88.5	250.3

12.5-mm to 9.5-mm Flat and Elongated (Percent)

Precision	Test Result (%)	(1S) %	(D2S) %
Single Operator	34.9	22.9	64.7
Multilaboratory		43.0	121.8

9.5-mm to 4.75-mm Flat and Elongated (Percent)

Precision	Test Result (%)	(1S) %	(D2S) %
Single Operator	24.1	19.0	53.6
Multilaboratory		46.1	130.3†

5.1.9 Thermal Conductivity

Thermal properties, such as thermal conductivity, are important in understanding and predicting concrete behavior for applications such as continuously reinforced concrete pavements (CRCP) or mass pours. If thermal properties are not thoroughly understood or accounted for, thermal cracking may occur. Thermal properties are also input parameters for some computer programs such as *ConcreteWorks*.

There is currently no standard test method for measuring the thermal conductivity of aggregates. However, researchers have determined a method for evaluating the thermal conductivity of aggregate using a *Mathis TCi Thermal Conductivity Analyzer*. The *Mathis TCi Thermal Conductivity Analyzer* is a tool for rapid, non-destructive thermal conductivity and effusivity testing which can be used for solids, liquids, powders, and pastes. This machine operates based on the "modified transient place source technique. It uses a one-sided, interfacial, heat reflectance sensor that applies a momentary, constant heat source to the sample" (Mathis Instruments Ltd., 2012). A known current is applied to a heating element and the temperature rise at the sensor-material interface causes a change in voltage. The thermal conductivity of the material being tested is inversely proportional to the change in voltage. This test is completed in a matter of seconds and the thermal conductivity range that can be tested is 0 to 100 W/mK. Software provided with the machine creates output data in the form of a spreadsheet (Mathis Instruments Ltd., 2012).

The minimum sample size required for testing is 0.67-in. (17-mm). The research team plans to obtain large aggregate particles with a 3/4-in. (19-mm) minimum dimension and use a mechanical polisher to get a clean, polished, and even surface. The minimum sample thickness required for testing is 0.02-in. (0.5-mm) but because the calibration material is 3/8-in. (9.5-mm) thick, the research team will attempt to polish aggregates to a thickness of 3/8-in. (9.5-mm). Because aggregates in concrete are effectively saturated, it will be more relevant to test aggregate samples in a saturated state rather than a dry state. Therefore after polishing, aggregates will be soaked for 24 hours prior to testing.

According to the manufacturer, the accuracy of the Mathis TCi Thermal Conductivity Analyzer is better than 5% and the precision is better than 1% (Mathis Instruments Ltd., 2012). Because some aggregate sources, particularly river gravels, are composed of a wide range of minerals, the research team will prepare and test approximately 15 samples per source to ensure that the source is accurately represented.

5.1.10 Petrographic Analysis

Petrographic examination is typically performed by a trained and experienced petrographer, knowledgeable of local geology, following ASTM C 295 (*A Guide for Petrographic Examination of Aggregates for Concrete*). As part of the petrographic examination, petrographers identify key constituents and proportions of an aggregate sample using a variety of tools including microscopes, cameras, polishing/grinding wheels, and a variety of hand tools.

The petrographic examination of aggregates collected in this research project was performed by a trained TxDOT petrographer. Members of the research team provided a 100-g raw sample of each aggregate to the TxDOT petrographer and analysis results were relayed to the research team.

5.1.11 X-Ray Diffraction

X-ray diffraction (XRD) is an advanced analysis technique that sometimes requires the material of interest to be ground to a fine powder (depending on the exact analysis method). A diffractometer, which fires an incident X-ray beam at the sample and receives the scattered beam, is used to gather data. Output from X-ray diffraction appears as a plot of scattering intensity versus scattering angle. From this plot, peaks can be identified which correspond to individual material components of sample.

The research team will prepare samples for XRD analysis by crushing and pulverizing raw coarse aggregate samples. A representative 2500-g sample will be obtained and crushed to pass the 1/4-in. (6.25-mm) sieve. This material will be reduced to a 100-g sample by splitting the original sample. This 100-g sample will be pulverized to pass the No. 100 (150-µm). The new pulverized sample will be taken to the TxDOT laboratory where it will undergo a final grind to pass the No. 325 (45-µm) sieve. This sample will then be subjected to XRD analysis. Researchers will be trained to use the X-ray diffractometer at the TxDOT laboratory.

5.2 Fine Aggregate Tests

Fine aggregate tests were selected based on a review of literature, a review of other state DOT specifications, and a discussion with personnel in industry and academia. Chapter 2: Review of Literature and Chapter 3: Development of Testing Program contain more information about selection of aggregate tests. Because this research project is funded by TxDOT, TxDOT standards were used when possible. Otherwise, ASTM standards or other widely accepted test methods were used to ensure repeatable and consistent results. Refer to Table 7: Finalized Testing Plan to view the complete list of coarse aggregate tests performed.

5.2.1 Micro-Deval Test for Fine Aggregates

The Micro-Deval test for fine aggregates is very similar to the Micro-Deval test for coarse aggregates. A standard gradation of sand, ranging from No. 8 (2.36-mm) to

No. 200 (75-µm), is placed in the Micro-Deval container along with steel charge and soaked for one hour prior to testing. The Micro-Deval container is then rotated for 15 min at approximately 100 revolutions per minute. The sample material is then removed and washed over a No .200 (75-µm) sieve. The remaining material is oven-dried and weighed and the relative amount of material lost, as a percentage, signifies the final Micro-Deval loss. The standard test method used by the research team was ASTM D 7428 (*Resistance of Fine Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus*) and is described in the following section.

The Micro-Deval test requires that the sample aggregate be washed over the No. 200 (75-µm) sieve and dried prior to testing. A standard gradation is specified for concrete aggregate for this test and provided in Table 12. The total mass of the sample should be 500 ± 5-g.

Table 12: Fine Aggregate Gradation Required for Micro-Deval Test (ASTM D 7428)

Sieve Size	Target Mass	
No. 4 – No. 8	50	g
No. 8 – No. 16	125	g
No. 16 – No. 30	125	g
No. 30 – No. 50	100	g
No. 50 – No. 100	75	g
No. 100 – No. 200	25	g
Total	500 ± 5	g

Once the sample has been weighed out, 1250 ± 5-g of stainless steel balls, of diameter 9.5 ± 0.5-mm, should be placed in the Micro-Deval container. The Micro-Deval container is a small stainless steel drum and can be seen in Figure 19. The aggregate sample can then be placed in the Micro-Deval container and soaked in 750 ± 50-mL of tap water a temperature of 20 ± 5˚C (68 ± 9°F) for a minimum for one hour. After one hour, the Micro-Deval container is sealed and placed in the Micro-Deval machine. The Micro-Deval machine is a simple ball mill roller and can be seen in Figure 18.

The operator sets the appropriate time on the machine such that it will run for 15 minutes. The machine should be calibrated to revolve at 100 ± 5 revolutions per minute and the final revolution count should be 1500 ± 10 revolutions. After fifteen minutes of revolutions, the Micro-Deval container is removed and the contents are washed over a No. 200 (75-μm) sieve. Material passing the sieve is discarded. The retained material is oven-dried to a constant weight and weighed after drying. The oven-dried weight is recorded and compared to the original weight to get a percent loss calculation for the final Micro-Deval loss.

The research team performed two Micro-Deval tests per source and recorded the mean loss, unless the test results were different by more than 1.0%. In this case, a third Micro-Deval test was performed and the mean of the three tests was considered acceptable unless an outlier existed. The inside surface of the stainless steel drum can affect the results of the Micro-Deval test for fine aggregates, so it is recommended that a reference sand be run periodically. Testing of carbonate sands can lead to "polishing" of the steel drum surface which has been shown to affect test results (Rogers C. , 1998).

According to the ASTM standard, the single-operator coefficient of variation is approximately 3.4% and the multi-laboratory coefficient of variation in approximately 8.7% (ASTM International, 2008).

5.2.2 Specific Gravity and Absorption Test for Fine Aggregates

The specific gravity and absorption test for fine aggregate requires that a representative sample of an aggregate approximately 3000-g in mass and passing the No. 4 (4.75-mm) sieve be obtained and soaked for at least 24 hours. The aggregate is then dried to saturated surface dry state, weighed in a calibrated pycnometer, and dried to an oven-dry state as described by the following procedures. The standard used by the research team was Tex-403-A (*Saturated Surface-Dry Specific Gravity and Absorption of the Aggregates*).

After a representative sample of aggregate is obtained, the aggregate is soaked (in a non-metal tub to avoid reaction) to ensure that all permeable pores of the aggregate become filled with water. The water should be at a temperature of 23 ± 2˚C (73 ± 3˚F). After the soaking period, the operator removes a sample of the fine aggregate and places it a clean, smooth surface such as a metal pan. The sample is spread out and allowed to air dry for several hours. While drying, the operator should stir the sand periodically to ensure that none of the sample gets drier than the saturated surface dry state. Some standards allow the use of external heat, but the TxDOT test method does not. Drying may take several hours or several days depending on the aggregate sample. A fan may be used to accelerate convection and thus speed the drying time. To determine if the sample is at SSD, the operator places a metal cone (wide end down) on a metal base plate and fills the cone with sand until slightly overflowing. The operator then lightly tamps the sand in the cone 25 times by dropping the tamper about 5-mm (0.2-in.) above the surface of the fine aggregate. The moisture condition of the fine aggregate sample will be evident upon lifting the cone. If the material slumps completely, then it is too dry. If the material maintains its shape completely, then it is too wet. A sample in the SSD condition should slump slightly. Test materials are pictured in Figure 25.

Figure 25: Testing a Sand for Specific Gravity and Absorption

At this point, the aggregate has reached the saturated surface dry (SSD) condition. The operator should be obtain and immediately weight approximately 1200-g of aggregate in the SSD. Next, the SSD aggregate is placed in the pycnometer which should be calibrated by the same methods described in Section 5.1.2 Specific Gravity and Absorption Test for Coarse Aggregates. The operator should take extra care in placing aggregate in the pycnometer so that all material is accounted for. The jar is filled with water at 23 ± 2˚C (73 ± 3˚F) until it reaches the brim. The pycnometer should be agitated so that any entrapped air is freed. The pycnometer cap is again placed on the jar and a rubber bulb or syringe is used to fill the pycnometer until a rounded bead of water can be seen on top of the pycnometer cap. The outside of the pycnometer is dried and the pycnometer with aggregate is weighed and this weight recorded.

Finally, the aggregate is removed from the pycnometer, placed in a pan, and the pan placed in an oven where the aggregate is dried to constant mass. The weight of the aggregate at SSD, the weight of the pycnometer with water, and the weight of the pycnometer with aggregate are used to calculate the specific gravity and absorption capacity of the aggregate. Specific gravity is typically reported to the nearest 0.01 and absorption to the nearest 0.1% (Texas Department of Transportation, 1999).

The research team noticed that allowing the sand to cool to room temperature also allowed an uptake of moisture from the air. A pan of oven-dry sand could easily gain a few tenths of a percent of mass from moisture in the air alone in a matter of minutes, thereby skewing absorption values. Therefore, it is recommended that the sand be weighed immediately after removal from the drying oven.

5.2.3 Aggregate Imaging System (AIMS 2.0)

The aggregate imaging system (AIMS) is a machine consisting of a camera, lights, computer software, and movable trays, designed to capture and analyze the shape, angularity, and texture of coarse aggregates and the form and angularity of fine aggregates. The camera captures images of the aggregate particles , either lit directly or backlit, and the software analyzes these images and provides the user with data summarizing the shape characteristics. The camera in the AIMS apparatus is capable of capturing particles as small as 75-µm (retained on the No. 200 sieve).

For fine aggregates, the user places a set of fine aggregate particles (separated by fraction size) on an opaque tray and places this tray into the machine. Figure 51, in Chapter 6: Discussion and Analysis, displays coarse aggregates placed on trays for testing and Figure 23, in Section 5.1.7 Aggregate Imaging System (AIMS 2.0), displays the AIMS machine. The AIMS machine analyzes at least 150 particles for analysis for each size fraction. After the AIMS machine has analyzed one size fraction, the user removes the tray, cleans it, and places the next size fraction of fine aggregate on the tray. This process is repeated until all size fractions have been analyzed. Because the fine aggregate particles are so small, many of them touch as they are distributed across the tray by the operator. Fortunately recent advances in the AIMS technology include a touching particle factor (TPF) to eliminate inaccurate angularity analysis of fine aggregates where particles touch or overlap (Mahmoud, Gates, Masad, Erdoğan, & Garboczi, 2010). Output from the AIMS consists of quantified measurements of form and angularity for each particle, and a mean and standard deviation of each value as well. Aggregates typically follow a standard statistical distribution. The research team performed AIMS analysis of fine aggregate before Micro-Deval and after Micro-Deval to determine if a change in form and angularity would provide meaningful data.

5.2.4 Flakiness Sieve

After observing problems in field compact of HMA, Rogers and Gorman in Ontario sought to develop an inexpensive and quick test to determine a measurement of flakey particles in a sand. Past research has demonstrated that sands in excess of 30% flakey particles may have issues during compaction of hot mix asphalt. Rogers and Gorman realized that slotted sieves, traditionally used for seeds and grains, could also be used to evaluate fine aggregate (Rogers & Gorman, 2008). There is currently no accepted standard for this flakiness test for fine aggregate, but Rogers and Gorman have provided a description of this test and its procedures in the appendix of their 2008 paper entitled “A Flakiness Test for Fine Aggregates”.

The flakiness sieve test, described by Rogers and Gorman, uses two slotted sieves with slots of 1.8-mm and 1.0-mm respectively. The fine aggregate is sieved and broken down into separate size fractions. The sand retained on the No. 8 (2.36-mm) sieve is placed on the 1.8-mm slotted sieve and agitated. The same is done for the sand retained on the No. 16 (1.18-mm) sieve, except the 1.0-mm slotted sieve is used. The operator uses a set of tweezers to ensure that all flakey particles pass through the slots. All particles passing through the slotted sieves are considered flakey, and the final results are calculated by mass. Rogers and Gorman recommend testing at least 60-g of fine aggregate per fraction size to obtain a representative result. To calculate the final flakiness value, the results from the No. 8 and No. 16 can be averaged, or a weighted average can be calculated based on the gradation of the fine aggregate of interest. A slotted sieve is pictured in Figure 11, located in Section 2.4.3.2 Flakiness Sieve, with sand particles retained.

Researchers tested each fine aggregate for flakiness using the test method described by Rogers and Gorman. Slotted grain sieves 200-mm (8-in.) in diameter meeting ISO 5223 were obtained from Endecott. Fine aggregate samples were obtained by first oven-drying and sieving a sample, and then using a mechanical splitter. At least three flakiness tests of 20.0-g each were run per fraction size, thus a total of at least 120.0-g was tested for each fine aggregate source.

5.2.5 Grace Methylene Blue Test

Because there is an inherent degree of subjectivity to the AASHTO methylene blue test (AASHTO TP 57), researchers at W.R. Grace & Co. sought to improve this test method by removing subjectivity and enhancing repeatability and reproducibility. The test developed by Grace is similar to the traditional AASHTO test, but the test is performed on an entire sample of sand (not just the microfines) and uses a UV colorimeter to analyze the color of the final filtered solution sample. The new methylene blue test allows the entire sample to be measured, which is important because all clay in the sand is measured which ensures more representative results.

W.R. Grace & Co. generously supplied the testing equipment and test procedures for their Grace methylene blue test. Testing materials needed for this test include a Hach DR 850 Colorimeter, a micropipette, a portable balance, several plastic and glass test tubes, a plastic weigh boat, a 3-mL syringe with luer-lok adapter, and a 0.2-μm syringe filter. Prior to testing a fine aggregate sample, the concentration of methylene blue solution should be calculated and a correction factor used if the solution concentration differs from 0.50% by weight.

Once the methylene blue solution concentration is confirmed, a slightly moist sample of fine aggregate weighing at least 20-g should be obtained. A sample of 20-g of sand should be weighed and added to a 45-mL plastic test tube with 30-g of methylene blue solution. The tube should be capped and agitated for one minute. A three minute rest period should follow and then the sample should be agitated for an additional minute. Approximately 2-mL of this test solution should be transferred to a 3-mL syringe with a luer-lok filter fitted. The syringe should be depressed such that 0.5 – 1.0-mL of solution is filtered and transferred to a new 1-mL plastic tube. Using the micro-pipette, 130-μL of solution is transferred from the 1-mL plastic tube to a new, empty 45-mL plastic test tube. The 130-μL sample is diluted by adding water to make the total diluted solution 45-g. The 45-mL plastic tube is then capped and mixed. Next, the diluted solution is transferred to a clean 16-mm glass tube and is ready to be measured by the colorimeter. Before measuring the diluted solution, the colorimeter should be zeroed by measuring a sample of water. Finally, the diluted solution can be measured by the colorimeter. The output of the colorimeter is a reading of miligrams of methylene blue per gram of sand.

After using the correction factor, the methylene blue value can be divided by 1.60 to obtain an equivalent amount of sodium montmorillonite clay (W.R. Grace & Co., 2010).

The research team performed three Grace methylene blue tests per sample. Researchers at Grace determined that reproducibility and repeatability were comparable to the traditional AASHTO method (Koehler, Jeknavorian, Chun, & Zhou, 2009).

When performing this test, researchers noticed that when placing the sand in the initial weigh dish, the entire sample was not successfully transferred to the 45-mL test tube. A small amount of microfines remained adhered to the initial weigh dish or was fine enough to disperse into the air as the sample was being transferred. This error led to changes of up to 0.3% final clay content. As a result, researchers recommend placing the sand sample directly into the 45-mL test tube after tarring the scale appropriately.

5.2.6 Organic Impurities

In the organic impurities test, a fine aggregate sample is subjected to a sodium hydroxide solution and allowed to remain undisturbed to react for 24 hours. Any organic material in the sample will react with the sodium hydroxide solution to produce a dark liquid. The operator examines the color of the supernatant liquid and if it is darker than a standardized color, the fine aggregate is subjected to a 7-day mortar cube strength test. The fine aggregate is typically deemed to have an unacceptable amount of organic content if the compressive strength of the mortar cube is less than 90-95% of a control sample. The standard test method used by the research team was Tex-408-A (*Organic Impurities in Fine Aggregates for Concrete*) and is described in the following section.

Materials needed for this test include small glass bottles with volume of 12 to 16 oz. (355 to 473-mL), a Gardner glass color standard, and 3% sodium hydroxide solution. An air-dried sample of approximately 300-g should be obtained prior to testing. The glass bottle should be filled with the air-dried sand up to the 4.5-oz (133-mL) mark. A small amount of sodium hydroxide (enough to cover the sample) should be added to the glass bottle. The bottle should be capped and shaken. More sodium hydroxide is then added to the 7-oz (207-mL) mark and the bottle is capped and shaken again. The bottle is then placed aside and allowed to sit undisturbed for 24 hours. After the 24-hour resting period, the operator should observe the color of the supernatant liquid and compare it to the Gardner color standard. If the color of the liquid is darker than the Gardner No. 11, the fine aggregate is subjected to a 7-day mortar cube strength test. The fine aggregate is typically deemed to have an unacceptable amount of organic content if the compressive strength of the mortar cube is less than 95% of a control sample (Texas Department of Transportation, 1999).

5.2.7 Sand Equivalent

The sand equivalent test is a test method that is used to determine the proportion of "detrimental fine dust of clay-like particles in soils or fine aggregates" (Texas Department of Transportation, 2009). This test subjects a fine aggregate sample to a flocculating solution (calcium chloride) in order to separate fine particles from the coarser sand. The higher the sand equivalent value, the cleaner the sand is perceived to be. The standard test method used by researchers for this test was Tex-203-F (*Sand Equivalent Test*) and it is described in the following section.

Materials needed for this test include calcium chloride solution, a transparent graduated plastic cylinder, rubber stopper, agitator tube, an 85-mL (3-oz.) measuring can, weighted foot assembly, 3.8-L glass bottle, and plastic or rubber tubing. The graduated plastic cylinder should have a 1.25-in. (31.8-mm) inside diameter and a height of approximately 17-in. (432-mm). To prepare the calcium chloride solution, 577-g of ACS calcium chloride dehydrate should be dissolved in a 1.9-L of distilled water. This solution should be further diluted by adding 88-cc of the solution to 3.8-L of distilled water. This final solution is the working calcium chloride solution. All mixing and dilution should be performed at 22 ± 3˚C (72 ± 5˚F). The workstation should be setup such that the calcium chloride solution is located 914 ± 25-mm (3-ft. ± 1-in.) above the work surface and can be siphoned into the graduated plastic cylinder.

Prior to testing, a representative, oven-dried fine aggregate sample of at least 500-g should be obtained. This material should be sieved over the No. 4 (4.75-mm) sieve to remove any coarse material. After obtaining the sample material, the operator siphons calcium chloride solution into the graduated plastic cylinder such that the measurement reading is 101.6 ± 2.5-mm (4 ± 0.1-in.). The operator fills the 85-mL (3-oz.) measuring can with sample material and then slowly scrapes a scapula at a 45˚ angle over the measuring can to remove excess sand and ensure that 85-mL of bulk sand is obtained. The sample is then transferred to the graduated plastic cylinder. The cylinder is then agitated to remove air bubbles and allowed to sit for 10 ± 1 minutes (see Figure 26-A). The operator then stops the cylinder and places it in the mechanical sand equivalent shaker for 45 ± 1 seconds. The stopper is removed and the agitator is forced through the material with a gentle twisting and jabbing motion as the cylinder is rotated about its vertical axis (see Figure 26-B). This process should be continued so that all microfines and clay material are flushed from the coarse material until the level of the liquid reaches 381-mm (15-in.). The cylinder is allowed to sit for 20 ± 0.25-min and then the operator reads the level at the top of the clay layer. This value is recorded as the clay reading. Figure 26-C displays the sand equivalent test at this point for a sand with an unusually high clay reading. The operator then obtains the sand reading by gently dropping the weighted foot assembly into the cylinder until it comes to rest on the sand layer. The sand reading is calculated by subtracting 254-mm (10-in.) from the top indicator level. The final sand equivalent (SE) value is calculated by dividing the sand reading by the clay reading and multiplying by 100.

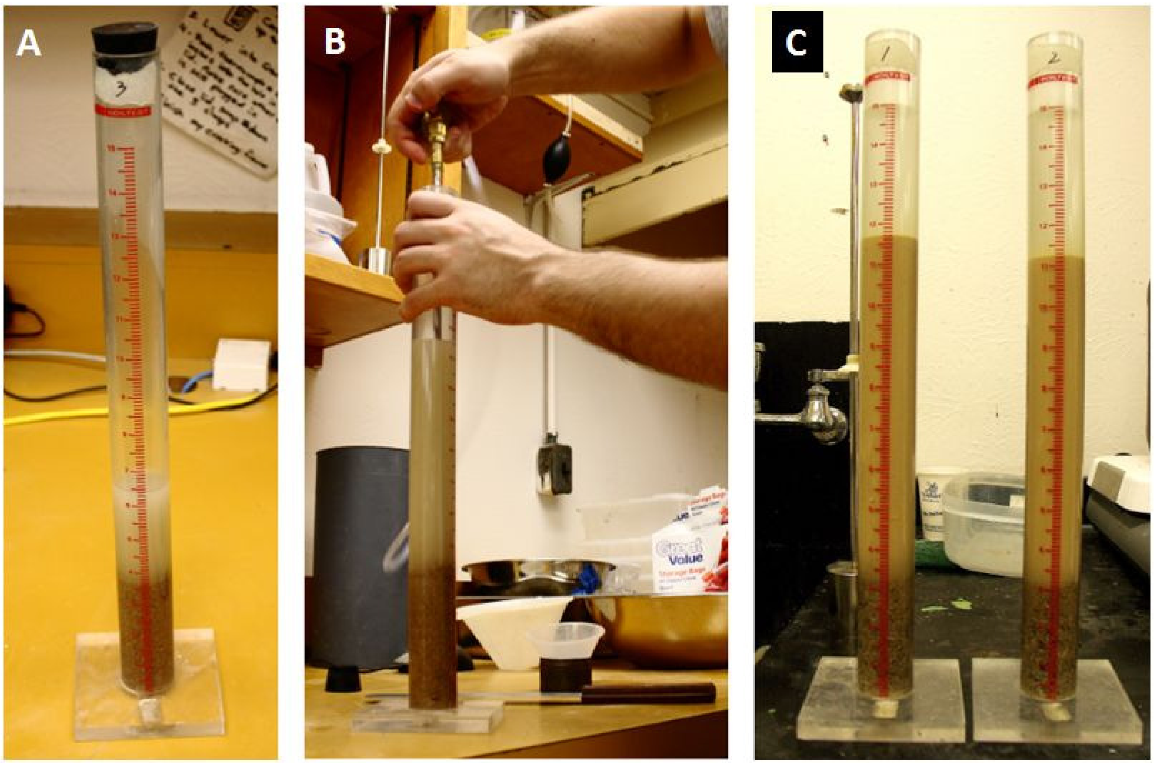

Figure 26: Sand Equivalent Testing

As per the Tex-203-F specification, the researchers performed this test twice per source and calculated the mean value. If the two tests differed by more than four points, a third test is performed and the mean of all three recorded. If the three tests differ by more than four points, the test results are discarded and the test must be performed again. The sand equivalent reading is typically reported to the nearest whole number.

According to the ASTM standard of this test (ASTM D 2419), the single-operator standard deviation is 1.5 for SE values greater than 80 and 2.9 for SE values less than 80.

Therefore, two tests should not differ by more than 4.2 for SE values greater than 80 and 8.2 for SE values less than 80 (ASTM International, 2009). This precision is reflected by the Tex-203-F test standard which requires discarding results differing by more than 4 points.

5.2.8 Acid Insoluble Residue

The acid insoluble residue test is one way of determining carbonate content of fine aggregate. In this test, a fine aggregate sample is subjected to hydrochloric acid and carbonate aggregates are dissolved by the acid while siliceous aggregates remain. The standard test method used by researchers for this project was Tex-612-J (*Acid Insoluble Residue for Fine Aggregate*) and the procedures for this test are described in the following section.

Materials needed for this test include a 2000-mLbeaker, a stirring rod, a porcelain filtration apparatus, No. 2 filter paper, plastic tubing, a drying dish, and a hydrochloric acid solution. Prior to testing, a representative, oven-dried fine aggregate sample of at least 500-g should be obtained. From this sample, 25-g of fine aggregate is placed in the 2000-mL beaker. The operator places the beaker in a fume hood and slowly adds hydrochloric acid until reaction ceases. An additional 25-mL of hydrochloric acid is added and stirred to ensure complete reaction. The remaining solution is decanted without loss of any sample material and then the remaining sample and solution is filtered over a No. 2-H filter paper. The filter paper is placed over the porcelain filtration apparatus such that distilled water can be washed over the sample to remove all acid without removing remaining aggregate. The filter paper, with remaining aggregate sample, is then oven-dried for two hours, and the final mass is recorded. The final mass of the sample is compared to the original mass to calculate the acid insoluble residue percentage by weight (Texas Department of Transportation, 2000).

The research team performed three tests per source and calculated a mean value to report as the acid insoluble residue for that source.

5.2.9 Petrographic Analysis

Petrographic examination is typically performed by a trained and experienced petrographer, knowledgeable of local geology, following ASTM C 295 (*A Guide for Petrographic Examination of Aggregates for Concrete*). As part of the petrographic examination, petrographers identify key constituents and proportions of an aggregate sample using a variety of tools including microscopes, cameras, polishing/grinding wheels, and a variety of hand tools.

The petrographic examination of aggregates collected in this research project was performed by a trained TxDOT petrographer. Members of the research team provided a 100-g raw sample of each aggregate to the TxDOT petrographer and analysis results were relayed to the research team.

5.2.10 X-ray Diffraction

X-ray diffraction (XRD) is an advanced analysis technique that sometimes requires the material of interest to be ground to a fine powder (depending on the exact analysis method). A diffractometer, which fires an incident X-ray beam at the sample and receives the scattered beam, is used to gather data. Output from X-ray diffraction appears as a plot of scattering intensity versus scattering angle. From this plot, peaks can be identified which correspond to individual material components of sample.

Samples for XRD analysis will be prepared by crushing and pulverizing raw fine aggregate samples. A representative 100-g sample of fine aggregate will be pulverized to pass the No. 100 (150-µm). The new pulverized sample will be taken to the TxDOT laboratory where it will undergo a final grind to pass the No. 325 (45-µm) sieve. This sample will then be subjected to XRD analysis. Researchers will be trained to use the X-ray diffractometer at the TxDOT laboratory.

5.3 CONCRETE TESTS

In order to relate aggregate tests to performance criteria, the research team decided to conduct mechanical concrete tests with the coarse aggregate as the only variable. Ideally, performance histories of aggregates would be available and could be used to rate an aggregate as "good", "fair", or "poor", but unfortunately this information is not always readily available. As a result, researchers will attempt to glean data and interpret results of mechanical concrete tests and their relation to the aggregate tests.

The concrete mixture design was selected such that it is a volume controlled concrete mixture meeting requirements for TxDOT CoTE testing. As such, the concrete mixture will be a Class P pavement mixture with Type I cement from Alamo Cement. No supplementary cementitious materials or admixtures will be used. The fine aggregate will function as a control and is a clean, natural river sand that has been obtained from TXI Webberville.

Researchers hope to begin mixing concrete samples soon and will cast ten 4-in. x 8-in. (100-mm x 200-mm) cylinders and three 4-in. x 4-in. x 14-in. (100-mm x 100-mm x 255-mm) beams per aggregate source. A total of 10 to 15 aggregate sources will be used for mixing and casting concrete.

5.3.1 Compressive Strength

Compressive strength testing of concrete is a common quality control procedure to ensure that the concrete is hydrating properly and that strength is gained at the necessary rate. The research team will cast 4-in. x 8-in. (100-mm x 200-mm) concrete cylinders, and use Tex-418-A (*Compressive Strength of Cylindrical Concrete Specimens*) to determine the compressive strength of these concrete mixtures. The Tex-418-A standard simply refers users to the ASTM version of this test, which is ASTM C 39 (*Compressive Strength of Cylindrical Concrete Specimens*). The procedures of this test are discussed in the following section.

The compression testing machine available to the research team is a Forney FX-700, more than capable of performing this test within the prescribed constraints. Other necessary equipment such as steel caps and neoprene bearing pads were also available to the research team. After curing the cylindrical concrete specimens for 28 days (± 20 hours), the specimens are removed from the curing room and placed in a 5-gallon bucket full of water so that the specimen remains moist until immediately prior to testing. The operator then takes the steel caps and places the neoprene bearing pads inside the steel caps. After 100 tests, or the first visible sign of cracking, the neoprene pads should be discarded, and replaced with new pads. The operator then removes the concrete specimen from the water and places the caps on both ends of the specimen. The concrete specimen is then placed in the compression machine. The operator should ensure that the surface of the bearing blocks in the machine is free from debris, and that the specimen is centrally placed on the blocks. At this point, the cage or screen on the testing machine is closed and locked for the safety of the test operator. The operator can then begin applying load to the specimen. The load should be applied at a rate of 0.25 ± 0.05 MPa/s (35 ± 7 psi/s) which means that the test may take several minutes, depending on the strength of the specimen. This load rate should be held as constant as possible for the duration of the test. The test is complete when the specimen displays a "well-defined fracture pattern" and supports no additional load. The operator then removes the load from the specimen, notes the fracture pattern, and records the final load. The final load divided by the average cross-sectional area of the specimen is the strength of the specimen (ASTM International, 2012).

For 4-in. x 8-in. (100-mm x 200-mm) specimens with strengths between 17 and 32 MPa (2500 and 4700 psi), the coefficient of variation for single-operator use has been found to be 3.2%. Therefore, the acceptable range of individual cylinder strengths is 9.0% for two cylinders or 10.6% for three cylinders (ASTM International, 2012). The research team will test the compressive strength of 3 specimens per aggregate source, with testing to be performed at 28 days.

5.3.2 Modulus of Elasticity

The modulus of elasticity is an important property of concrete because it will dictate the stress imposed by a given strain. Strains, caused by thermal expansion for example, will cause higher stresses for concrete with higher elastic modulus of elasticity. Modulus of elasticity in the linear range can be calculated by obtaining two data points of stress and strain in the linear elastic range. The most common test method for measuring modulus of elasticity of concrete in compression is ASTM C 469 (*Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*). The research team will use ASTM C 469 to determine the modulus of elasticity for several concrete mixtures.

The two main pieces of equipment necessary to perform this test are a compression testing machine and a compressometer. The compressometer is a device that fits around the concrete cylinder and is capable of measuring very precise changes in length between two gauge points. The gauge points should be separated by a distance at least three times the length of the maximum size aggregate. The deformation may be measured by a dial gauge, strain gauge, or by a linear variable transformer. Prior to the test, a matching cylinder should be tested in compression so that the ultimate load can be determined.

To begin the test, the operator removes a concrete cylinder from a curing room or bath, attaches the compressometer, and places the assembly in the compression testing machine. The specimen should be loaded twice, such that the first loading is used to correct any errors in the placement of the compressometer. During the second loading, the load rate should be 250 ± 50 kPa/s (35 ± 7 psi/s). The applied load and the longitudinal strain should be recorded when the longitudinal strain reaches 50 millionths and when the applied load reaches 40% of the ultimate load (as determined from the previous test). The modulus of elasticity can then be calculated by determining the slope of the stress-strain curve from a simple equation provided in ASTM C 469 (ASTM International, 2010).

Researchers plan on performing two modulus of elasticity tests per coarse aggregate for a total of 10 to 15 sources. Researchers will test one concrete cylinder in compression to obtain the ultimate strength, then test two cylinders for modulus of elasticity, and then test the same two cylinders for compressive strength.

According to ASTM C 469, the results of tests of two cylinders from the same batch should be no more than 5% different (ASTM International, 2010).

5.3.3 Flexural Strength

Flexural strength of concrete is important for resistance to tension and cracking. Flexural strength is typically determined by subjecting an unreinforced concrete beam to flexure by imposing one or two point loads in the middle of the span. The TxDOT method used by the research team is Tex-448-A (*Flexural Strength of Concrete Using Simple Beam Third-Point Loading*) and simply refers the user to ASTM C 78 (*Flexural Strength of Concrete*).

The flexural strength of concrete will be tested at third-point loadings as prescribed by the ASTM standard. The test setup should be similar to that shown in Figure 27. The bearing surfaces for the test specimen should be case-hardened steel and consist of partial cylinders with at least 0.80 radians of curved surface so that rotation is not restricted during loading.

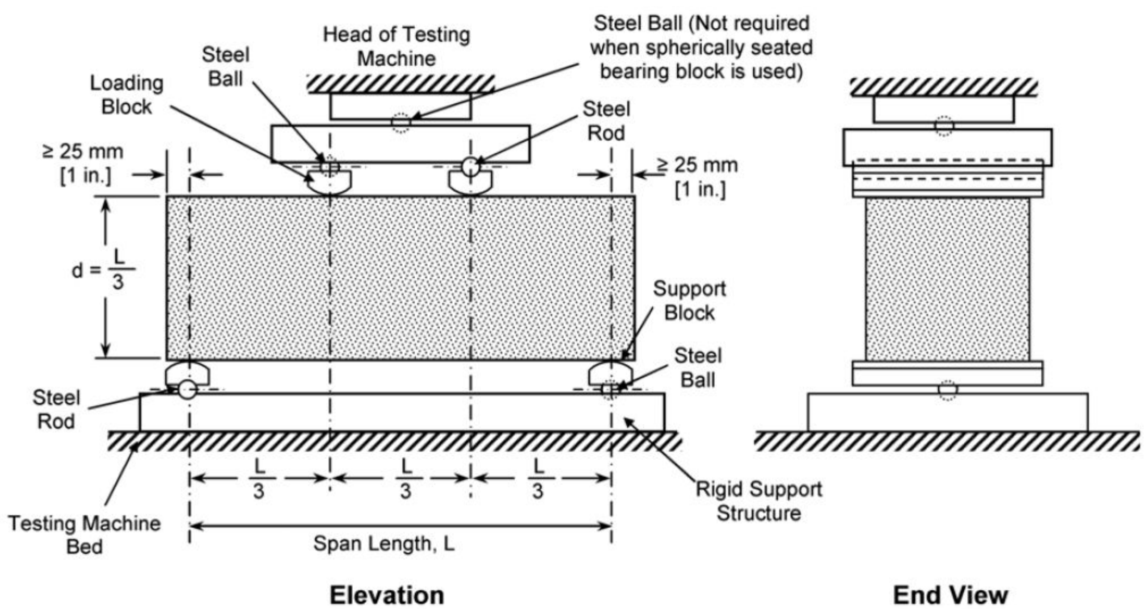

Figure 27: Setup of Concrete Flexure Test from ASTM C 78 (ASTM International, 2010)

Before placing the prismatic concrete specimen in the test apparatus, it should be placed on its side relative to its casting position. The operator should then center the concrete specimen in the test apparatus while conforming to the ASTM tolerances for position. The operator can then load the specimen up to 6% of ultimate load before again checking the position of the specimen and determining if there is any gap between the specimen and test blocks. Leather shims can be used to eliminate any gaps. The operator can then begin loading the specimen such that the tension face experiences a stress rate of 0.9 to 1.2 MPa/min (125 to 175 psi/min). The ASTM standard provides an equation for calculating the loading rate based on the geometry of the concrete specimen. The concrete specimen is loaded to failure and the operator should note the location of the fracture that initiated failure. If the fracture is located more than 5% outside the middle third of the beam, then the test results must be discarded. Otherwise, the modulus of rupture can be calculated based on simple equations located in ASTM C 78.

Strength of concrete will influence the precision of the test but the ASTM standard provides general information about precision. The single-operator coefficient of variation has been found to be 5.7% and the multilaboratory coefficient of variation has been found to be 7.0% for specimens from the same batch. Therefore, two tests using specimens from the same batch should not differ by more than 16% or 19% respectively (ASTM International, 2010).

The research team will create three 4-in. x 4-in. x 14-in. (100-mm x 100-mm x 255-mm) beams per aggregate source. A total of 10 to 15 aggregate sources will be used for mixing and casting concrete. The Tex-448-A standard provides an adjustment factor when beams are tested that are not 6-in. x 6-in. (150-mm x 150-mm) in cross section. However, the standard does not provide the adjustment factor for beams 4-in. x 4-in. in cross-section so researchers will consult with TxDOT laboratory personnel to determine an appropriate adjustment factor.

5.3.4 Coefficient of Thermal Expansion

Coefficient of thermal expansion of concrete is a particularly relevant test considering the issues that TxDOT has seen from high CoTE concrete mixtures in CRCP pavements. Section 2.3.6.2 Coefficient of Thermal Expansion (CoTE) documents the observed and perceived problems further. Researchers will use Tex-428-A (*Determining the Coefficient of Thermal Expansion of Concrete*) to test for CoTE.

This test requires a water bath, support frame for the cylinder, a measuring device such as an LVDT, and a saw to cut cylinders down to size. The support frame should be made of stainless steel with the exception of the vertical members which should be made of Invar, a material which has low coefficient of thermal expansion. The base plate should be 10-in. (254-mm) in diameter with three equally spaced support buttons to form circles of 2-in. (50-mm) and 3-in. (75-mm) diameter. A measuring device such as a spring loaded submersible LVDT should be attached to the top of the frame to provide expansion measurements.

Prior to testing, the concrete cylinder should be saw-cut such that the length is 6-in. (150-mm) and both ends are plane to within 0.002-in. (0.050-mm). The specimens should be conditioned in a saturated limewater solution at 73 ± 4˚F (23 ± 2˚C) for at least 48 hours. After conditioning, the specimens are removed from the limewater, measured, and the original length recorded with a caliper within 0.004-in. (0.1-mm). The specimens are then placed in the support frame in the water bath with LVDT attached (note that a typical standard water bath can hold two or three support frames with specimens attached). The water bath is set to 50 ± 2˚F (10 ± 1˚C) for an hour. At this point the test has begun and the water temperature and displacement (as measured by the LVDT) should be recorded at one minute intervals for the remainder of the test. The water bath should be heated and cooled again in accordance with Figure 28. After the test cycle is complete, the operator performs a regression analysis on the temperature versus displacement plot for a heating or cooling phase. The slope of the linear regression line divided by the length of the specimen will provide the coefficient of thermal expansion. A correction factor may be added based on the type of support frame used (Texas Department of Transportation, 2011). No precision information for this test is available in either the TxDOT or AASHTO standards.

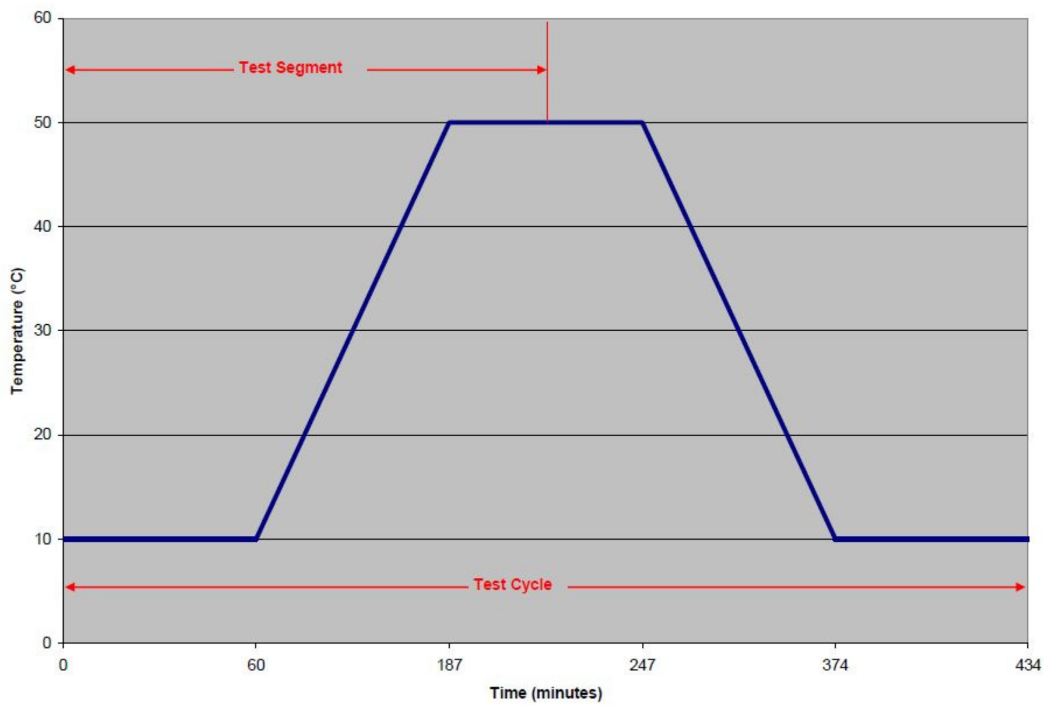

Figure 28: Temperature Cycle of Water Bath Required by Tex-428-A for CoTE Testing (Texas Department of Transportation, 2011)

The research team will perform CoTE tests on two concrete specimens per concrete mixture for a total of 10 to 15 aggregate sources. The cylinders will be tested at an age of no earlier than 28 days and no later than 90 days. When possible, the research team will also subject cores from aggregate sources to the same test and compare results.

Chapter 6: Discussion and Analysis

The majority of the first half of this project was spent collecting information, reviewing literature, and acquiring aggregates. Although the research team has been busy performing aggregate tests over the last several months, there is not enough data to draw significant conclusions at this point. However, there is enough data to examine trends in the aggregate tests and compare results to literature.

6.1 Analysis of AQMP Data

Before examining the data obtained by researchers for this project, it may be useful to take a look at data already obtained and documented by TxDOT. As discussed in Section 4.1.1 TxDOT Aggregate Quality Monitoring Program (AQMP), the AQMP is a TxDOT program that involves quality monitoring (as the name implies), testing, and statistical analysis of aggregates to ensure consistency and compliance with specifications. Due to the numerous tests run on various aggregates used in Texas, there is a significant amount of data available, particularly for the two main qualification tests: the magnesium sulfate soundness test and the LA abrasion test. As part of a project level quality monitoring tool, the Micro-Deval is also used extensively by TxDOT so these data are available as well. The AQMP list provided by TxDOT has information for 170 aggregate sources in the state ranging in location, mineralogy, and application.

The literature review showed mixed results for how well the Micro-Deval test correlated to other aggregate tests. Linear regression analysis performed on the 122 aggregate sources that had data for magnesium sulfate soundness, LA abrasion, and Micro-Deval demonstrated a good correlation (R^2 = 0.64) between magnesium sulfate soundness and Micro-Deval and a weaker correlation (R^2 = 0.54) between LA abrasion and Micro-Deval as shown in Figure 30 and Figure 31. Both correlations had positive slope, indicating increasing loss for both tests, as would be expected.

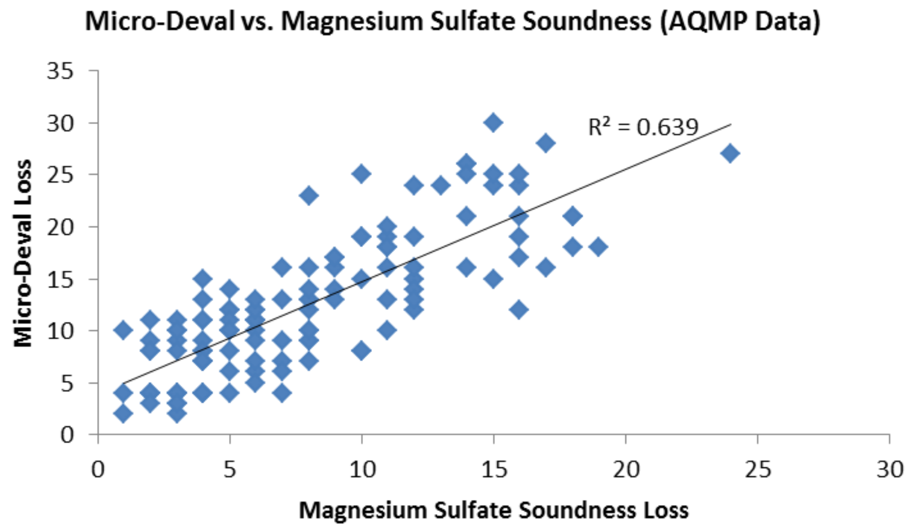

Figure 29: Micro-Deval Loss vs. Magnesium Sulfate Soundness (AQMP Data)

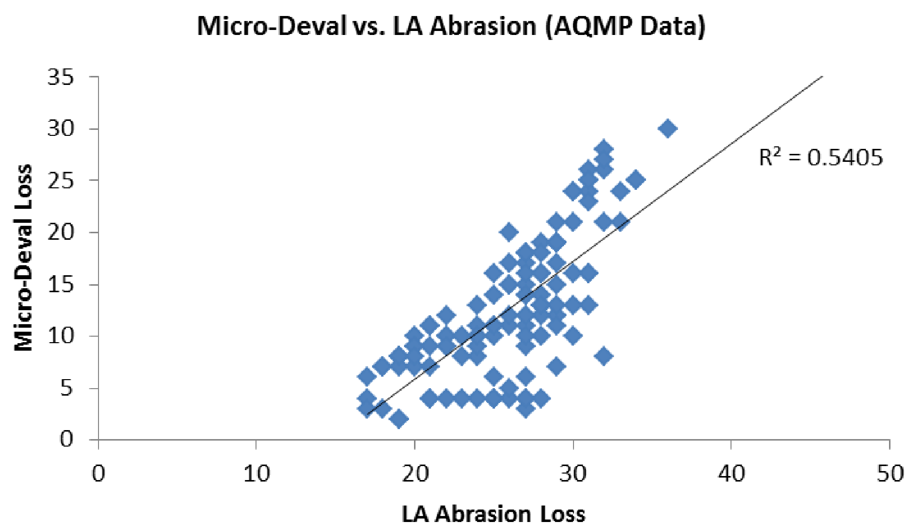

Figure 30: Micro-Deval vs. LA Abrasion (AQMP Data)

The correlation coefficient from the Micro-Deval and magnesium sulfate soundness comparison (R^2 = 0.64) is close to that obtained by the ICAR 507 study (R^2 = 0.54) which examined 117 American and Canadian sources and by the Jayawickrama et al. (2007) study (R^2 = 0.69) which examined Texas aggregates. However, at least one study, by Rangaraju, Edlinski, and Amirkhanian (2005), found no significant correlation between Micro-Deval and magnesium sulfate soundess. Although the LA abrasion has better correlation with tests like ACV and AIV, the correlation coefficient obtained by analyzing AQMP sources for LA abrasion and Micro-Deval is reasonably high. The correlation coefficient from the Micro-Deval and LA abrasion comparison (R^2 = 0.54) is close to that obtained by the Cuehlo, Mokwa, and Obert (2007) study (R^2 = 0.46) which examined Montana aggregates. However, the ICAR 507 study found little correlation between Micro-Deval and LA abrasion (R^2 = 0.12). It is possible that the correlation using AQMP data is higher for both test comparisons because AQMP data uses mean results which reduces the variability of the test.

It is also useful to examine the distribution of Micro-Deval values for AQMP sources. Despite the requirement of good historical performance to participate in the AQMP, many aggregates in Texas would fail to qualify for aggregate use in concrete if limits recommended by other studies were used in Texas. A variety of research projects have recommended Micro-Deval limits of anywhere from 13% to 20%, but analyzing AQMP data demonstrated that 15% of Texas aggregates on the AQMP list have Micro-Deval losses of 20% or higher as shown in Figure 31. Thus, recommended Micro-Deval limits would reject far too many aggregates. Weyers et al. (2005) recognized this same problem in the Wisconsin DOT study and recommended that a limit of 25-30% Micro-Deval loss would be more reasonable for Wisconsin aggregates. If the Micro-Deval test is ever used as a qualification test in Texas, the DOT should be aware that limits typically used by other DOTs would simply be too low and result in rejection of too many good aggregates.

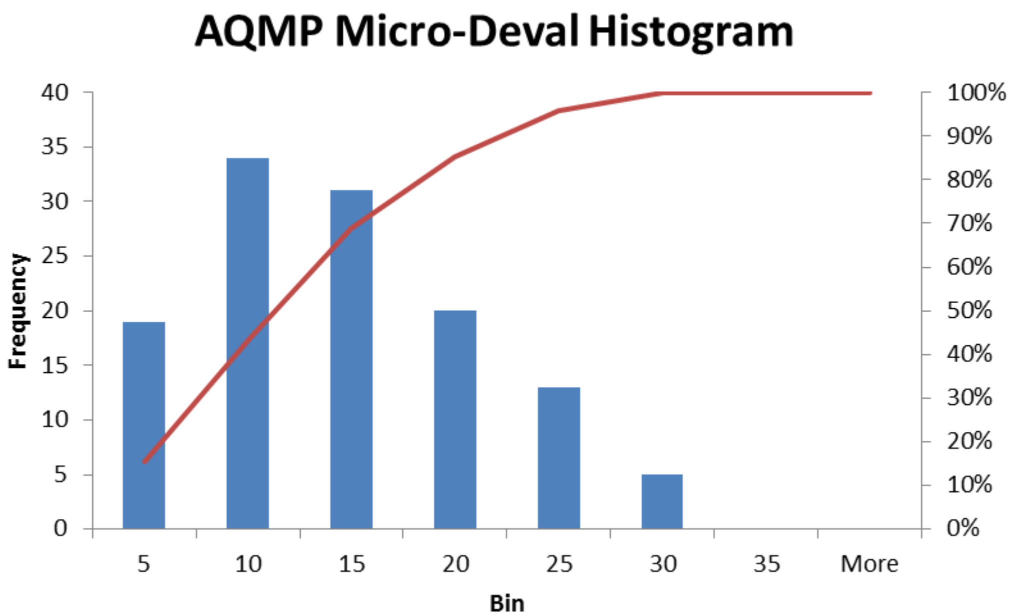

Figure 31: Histogram of Micro-Deval Values of Texas Aggregate Sources

6.2 CORRELATION OF AGGREGATE TESTS

Not all aggregates have been received or tested but there are enough data to examine testing trends and determine which aggregate tests show the strongest correlation so far. Ideally, results would have obtained from LA abrasion testing, but the TxDOT laboratory is still in the process of running the test samples that have already been prepared. Fortunately AQMP data from LA abrasion tests are available and these data can be used to compare to AIV and ACV, all of which measure strength of aggregate. Figure 32 shows the correlation between LA abrasion and AIV and between LA abrasion and ACV. With the limited amount of data points currently available, both

ACV and AIV have strong correlations to LA abrasion with ACV having a slightly stronger correlation (R^2 = 0.901 and 0.786 respectively).

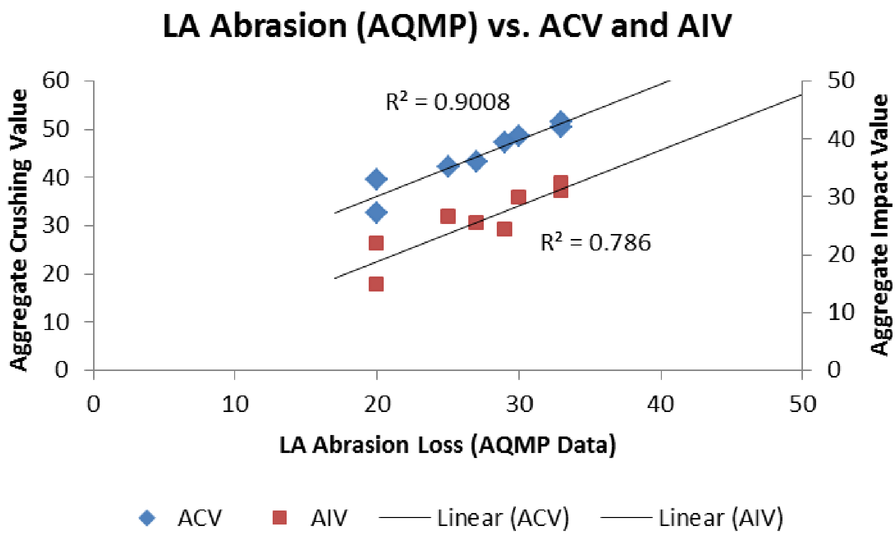

Figure 32: LA Abrasion (AQMP) vs. ACV and AIV

Another strong correlation has been observed between ACV and AIV. This trend is not exactly surprising as both tests measure some component of the aggregate strength. Specifically, AIV likely represents resistance to impact loads and ACV represents resistance to static loads. Linear regression analysis provides a correlation coefficient of 0.79 for these tests based on 10 aggregates that have been tested so far, as shown in Figure 33.

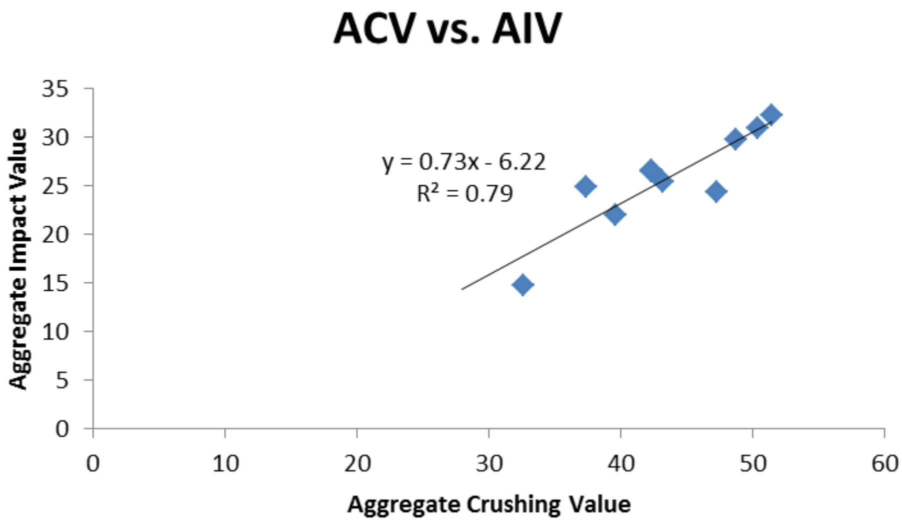

Figure 33: ACV vs. AIV

A closer look at this relationship, based on material type, is displayed in Figure 34. Based on just a few sources tested so far, limestone seems to be the weakest material as defined by higher losses for both ACV and AIV. More material will be tested before any conclusions can be drawn from the correlation of these two tests.

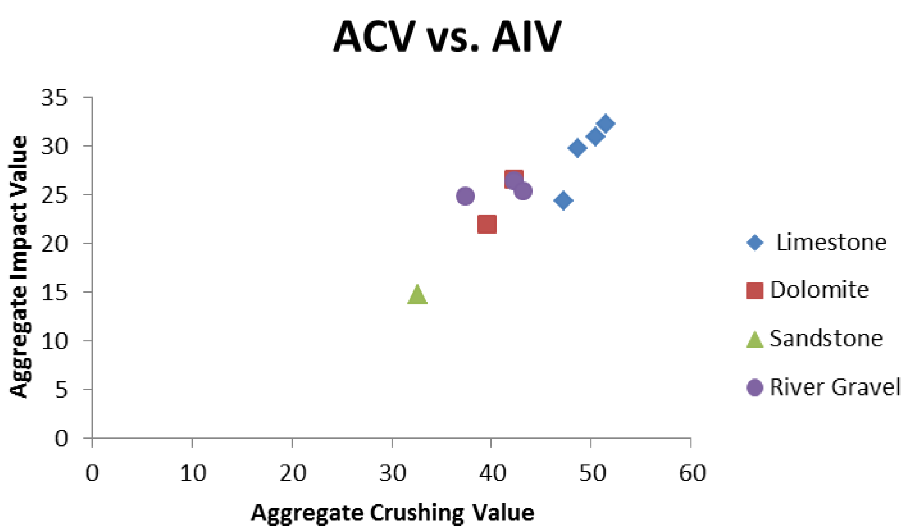

Figure 34: ACV vs. AIV – by Material Type

Comparing Micro-Deval results to ACV and AIV yields a lower correlation. For the 18 aggregate sources that have been tested for Micro-Deval and ACV, there appears to be a good correlation (R^2 = 0.61) between the two tests. The correlation is much weaker (R^2 = 0.34) for Micro-Deval and AIV based on the 11 aggregate sources tested thus far. Although Micro-Deval simulates abrasion in a wet environment, stronger aggregates (as defined by ACV and AIV) tend to have greater loss. Figure 35 graphically depicts these relationships and linear regression trend lines.

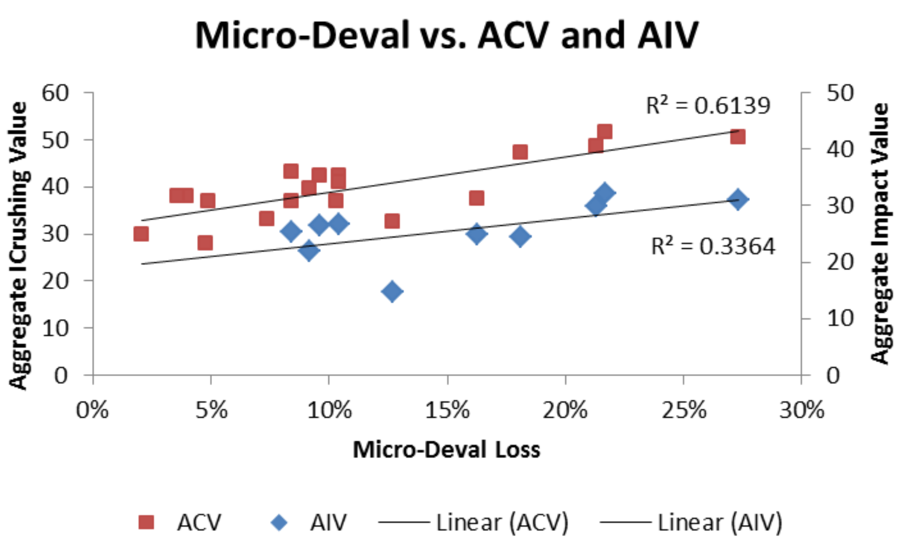

Figure 35: Micro-Deval vs. ACV and AIV

Another interesting trend is the correlation between Micro-Deval loss and absorption capacity of coarse aggregate. Linear regression analysis provides a correlation coefficient of 0.63 for these tests based on 22 aggregates that have been tested so far, as shown in Figure 36. This correlation is much stronger than the correlation found by ICAR 507 (R^2 = 0.40) between these two tests. It is possible that the addition of other sources to this plot will cause the correlation coefficient to decrease, but for now the correlation appears fairly strong. Figure 37 displays the same data based on material type. Denser, less absorptive material, such as igneous rocks (granite and rhyolite) and dolomite, tend to be more resistant to abrasion as represented by Micro-Deval loss.

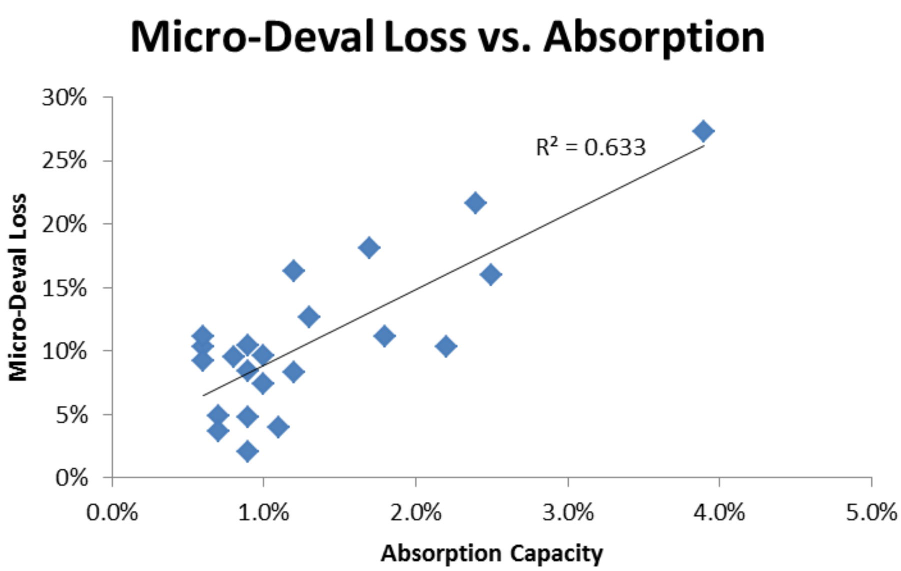

Figure 36: Micro-Deval vs. Absorption

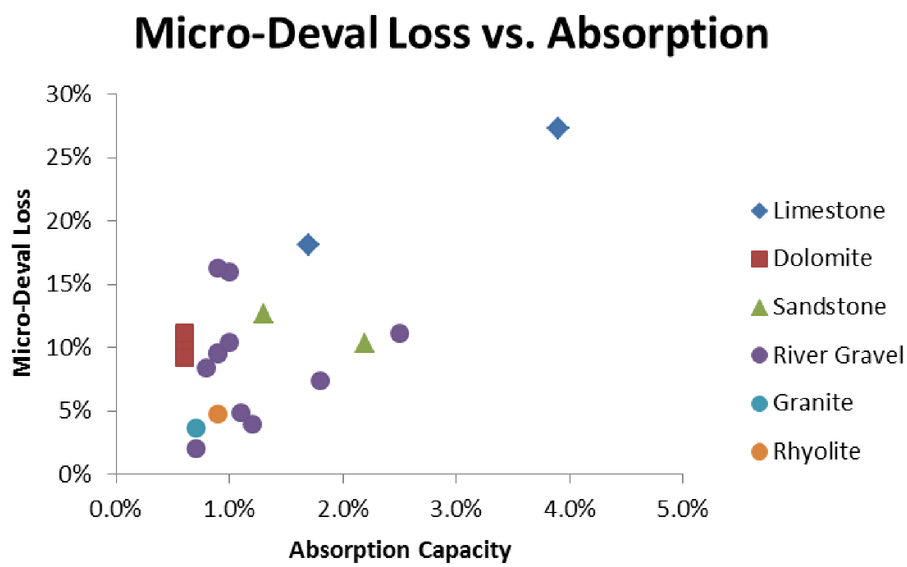

Figure 37: Micro-Deval vs. Absorption – by Material Type

Researchers also examined the correlation between fine aggregate tests. Two tests, which attempt to measure detrimental material in fine aggregates, are the sand equivalent tests (measures quantity of microfines) and the Grace methylene blue test (measures quantity of only clay). An examination of the relationship between these tests should will indicate whether any trends exist which might suggest that fine aggregates with higher microfines also have higher clay content. Figure 38 depicts the relationship between these two tests. Despite testing a total of 35 fine aggregates, consisting of both natural and manufactured sands, no clear correlation or trend emerged from the analysis. Even the removal of three outliers (manufactured sands not meeting ASTM C 33 gradation requirements) still only demonstrated a correlation that was weak at best. Plotting the test results by material type (natural or manufactured sand), as seen in Figure 39, only demonstrated that the natural sands that were tested have higher sand equivalent values than the manufactured sands. Again, this is not unexpected due to the fact that many of the manufactured sands that were tested are not used in concrete and are therefore not meeting the ASTM C 33 gradation requirements.

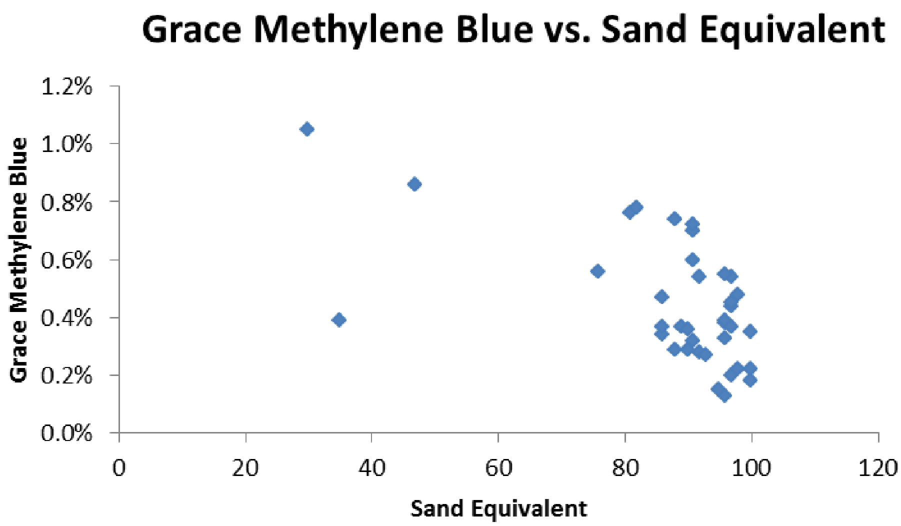

Figure 38: Grace Methylene Blue vs. Sand Equivalent

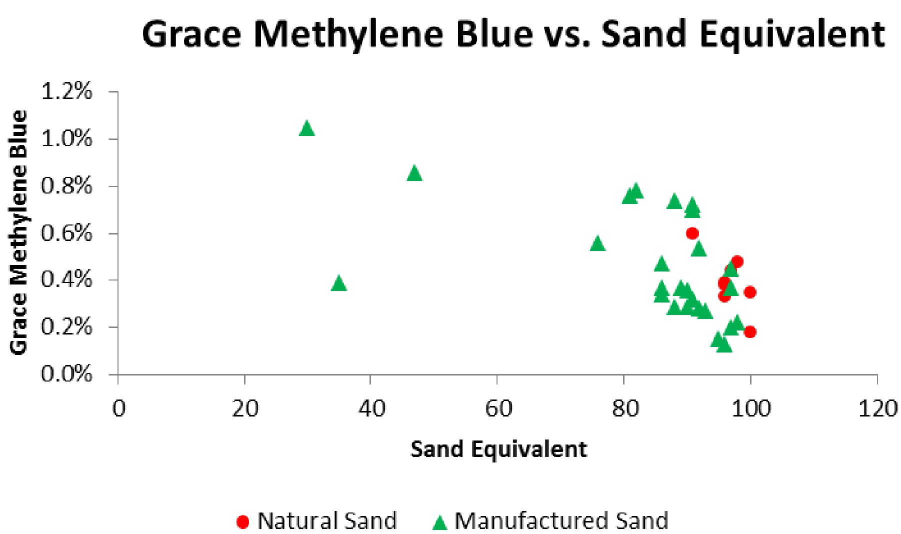

Figure 39: Grace Methylene Blue vs. Sand Equivalent – by Material Type

At least one recent research project demonstrated that, for fine aggregates, the Micro-Deval loss had a high correlation (R^2 = 0.81) with acid insoluble residue. However, this project only examined manufactured sands which are predominantly carbonate in nature, so this trend is not necessarily surprising. Sands with higher carbonate content will be less resistant to abrasion and also less resistant to acid. Of the sands tested thus far by the research team, no such trend has been established. Figure 40 depicts results of Micro-Deval loss and acid insoluble residue.

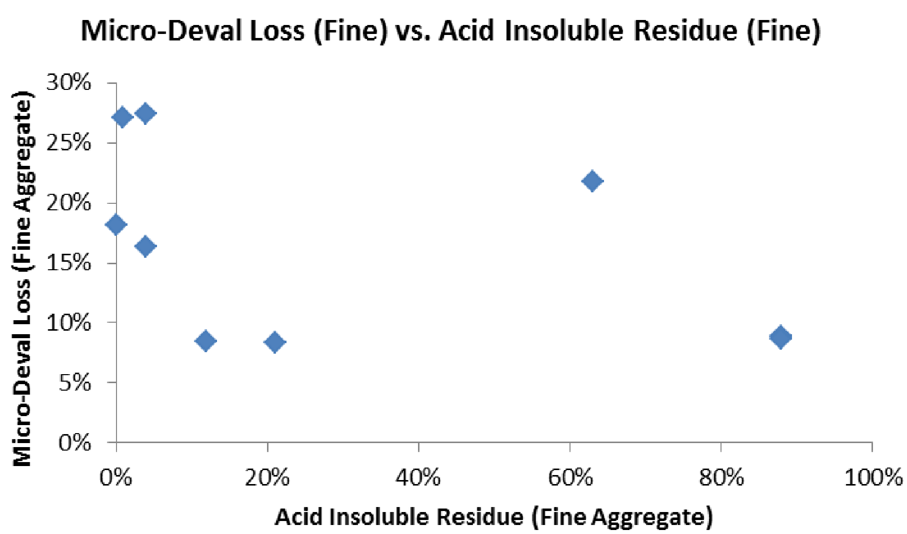

Figure 40: Micro-Deval Loss vs. Acid Insoluble Residue for Fine Aggregates

6.3 Comparison of Tests for Material from the same Source

Coarse aggregate and fine aggregate coming from the same source should intuitively have very similar physical and mechanical properties. Differences in size, processing, and shape of the aggregate may cause small differences in properties but generally, one would expect very similar results if subjected to the same tests.

Examining the specific gravity and absorption of materials from the same source provides the expected results. The correlation coefficients are quite high and the linear regression slope is close to 1 with an intercept close to 0 as indicated in Figure 41 and Figure 42. Variance in test results and material, along with the inherent subjectivity of Tex-403-A (see Section 5.2.2 Specific Gravity and Absorption Test for Fine Aggregates), may explain why the correlation coefficients are not exactly 1.0. However, they are very high, as one would expect. Rogers and Dziedziejko (2007) showed that sands with high microfines content (> 8% by mass) had higher absorption, lower density values, and higher variance compared to sands with microfines removed prior to testing. So presence of microfines may also cause the correlation to be lower than 1.0.

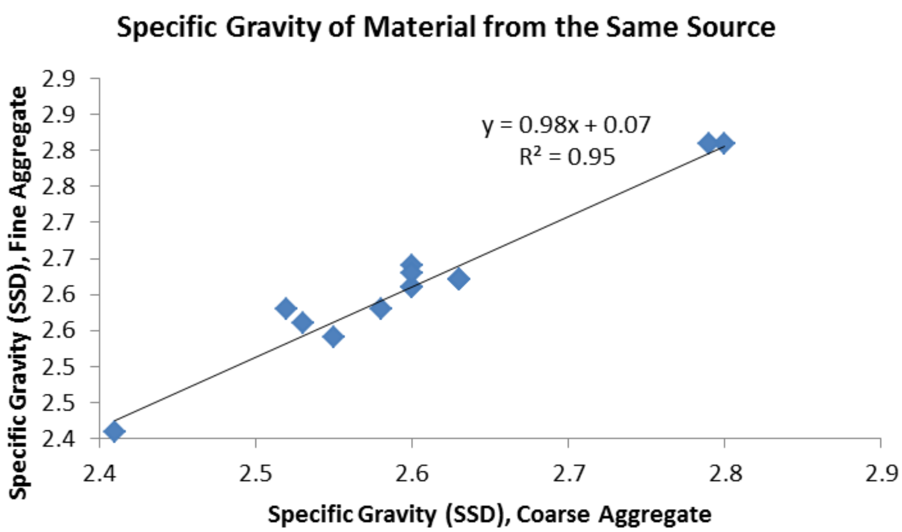

Figure 41: Specific Gravity of Material from the Same Source

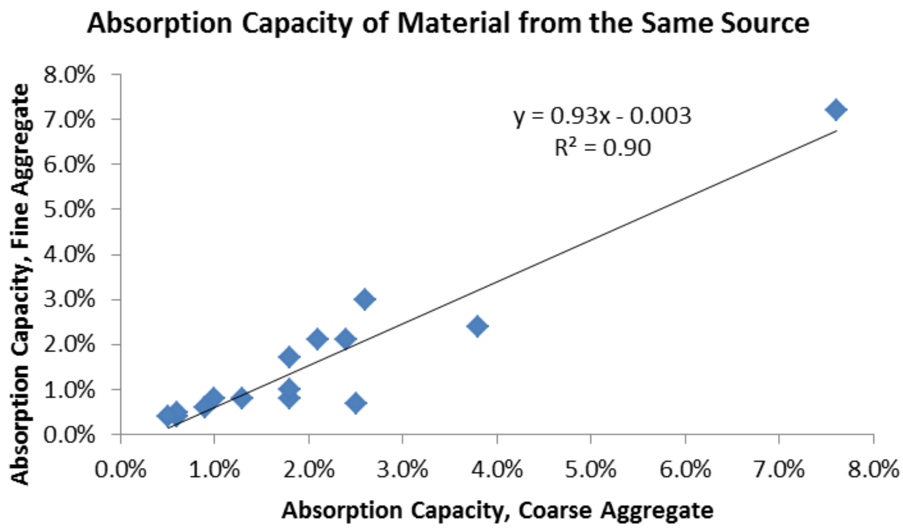

Figure 42: Absorption Capacity of Material from the Same Source

Interestingly, comparison of Micro-Deval loss of coarse aggregate and fine aggregate from the same source does not demonstrate the same level of very high correlation. Linear regression analysis actually shows that the correlation coefficient between Micro-Deval loss for fine aggregates and Micro-Deval loss for coarse aggregates is only 0.61. The linear regression is still positive, as one would expect, but the amount of deviation from the best-fit line, as shown in Figure 43, is somewhat surprising. Granted, only 9 aggregate sources have been tested for Micro-Deval for both coarse and fine aggregates, so it is possible that the addition of more data points will cause the correlation coefficient to increase.

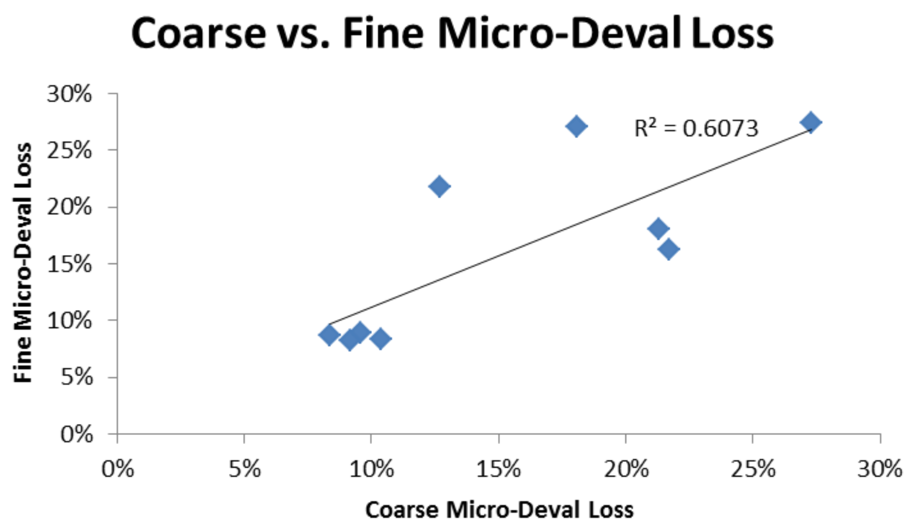

Figure 43: Coarse vs. Fine Micro-Deval Loss

Of the nine aggregates tested for Micro-Deval for both coarse aggregate and fine aggregate, two are from natural sources (river sand and gravel) and seven are from crushed sources (crushed stone and manufactured sand). Plotting the material type shows that the crushed material deviated more from the best-fit line than did the natural material. This plot is shown in Figure 44. It is important to note that the natural material tested so far is from two gravel pits very near each other. It will be interesting to watch this trend as more sources are tested and additional data points added to the plot.

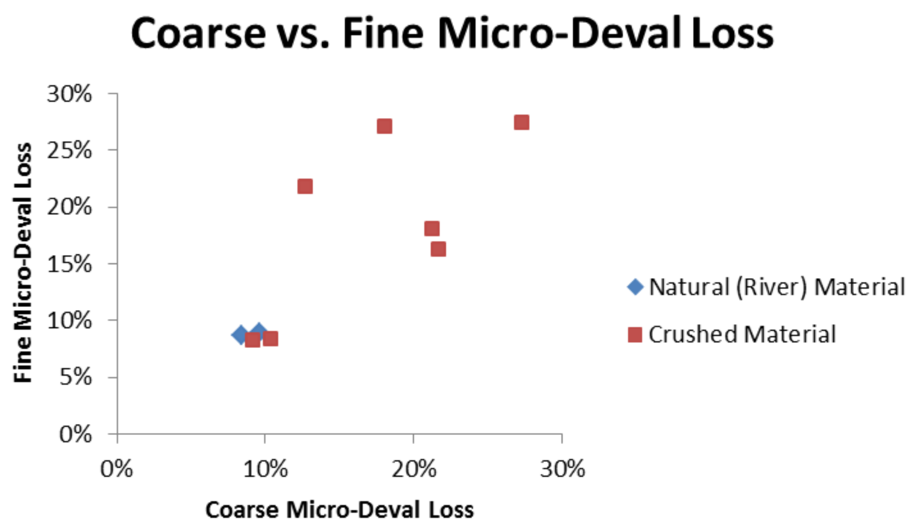

Figure 44: Coarse vs. Fine Micro-Deval Loss – by Material Type

6.4 AIMS Analysis

Analyzing an aggregate sample on the AIMS machine generates a significant amount of data. The AIMS software provides data for coarse aggregates that describes angularity, texture, sphericity, and flat and elongated particles. However, these data are in the form of index values that are arbitrarily defined and, thus, not necessarily intuitive.

6.4.1 AIMS Data for Coarse Aggregates

One facet of the AIMS test that the research team has been evaluating is the change in angularity, texture, and sphericity after performing a Micro-Deval test. A set of aggregates is prepared for Micro-Deval testing and analyzed by the AIMS prior to Micro-Deval testing. The sample is then subjected to Micro-Deval testing and afterwards is oven-dried and regraded to be analyzed again by the AIMS machine. After the second AIMS analysis, a change in angularity, texture, and sphericity can be calculated. A decrease in angularity, texture, or sphericity is represented by a negative percentage and an increase in angularity, texture, or sphericity is represented by a positive percentage. Intuitively, one would expect that the abrasion experienced by aggregates during the Micro-Deval test would cause a decrease in angularity, texture, and sphericity. However, preliminary results from 19 aggregates (total of 76 size fractions analyzed) show that this is not always the case. Figure 45 shows that at least 18 of the 76 size fractions tested before and after Micro-Deval experienced an increase in texture. Of these 18 size fractions, 6 also experienced a change in angularity.

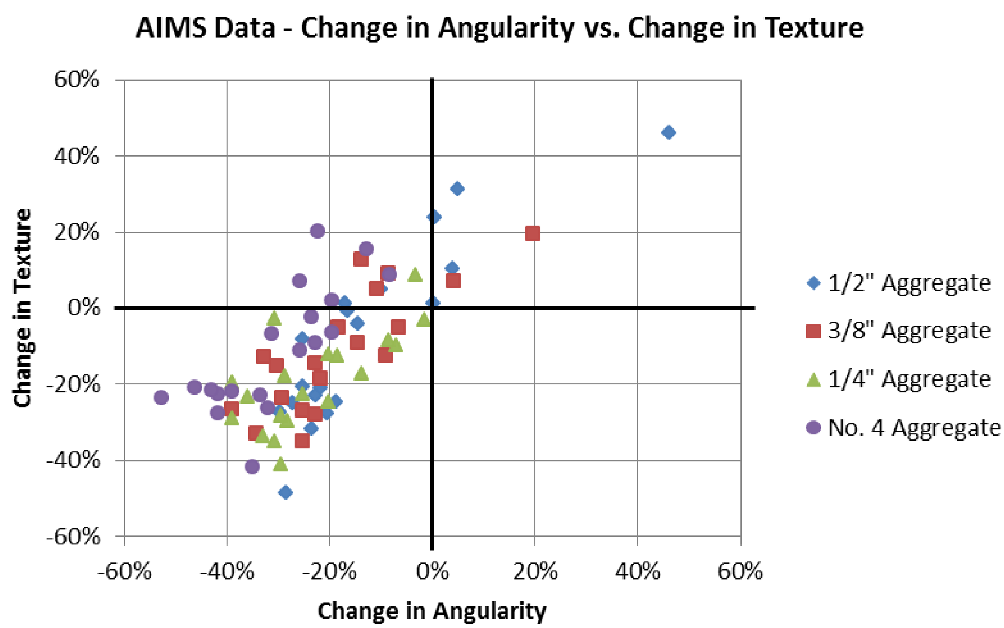

Figure 45: Change in Angularity vs. Change in Texture – by Size Fraction

Changes in angularity or texture close to 0% may be dismissed due to variance of the test and particles analyzed by the AIMS, but there are several examples of aggregates having an increase of 20% or more for texture and/or angularity. Since this information is so counterintuitive, the issue was explored further. The AIMS software provides an average index for each size fraction of aggregate but does not provide an overall average index for the aggregate. To calculate an overall average index, a weighted average was used, where weights were assigned to each size fraction based on the Micro-Deval grading (44% for ½-in., 22% for 3/8-in., 22% for ¼-in., and 12% for No. 4). As a result, an average index was attained for each source such that one number would represent the angularity, texture, or sphericity of that source. Figure 46 displays the results of plotting change in angularity and change in texture based on this weighted average.

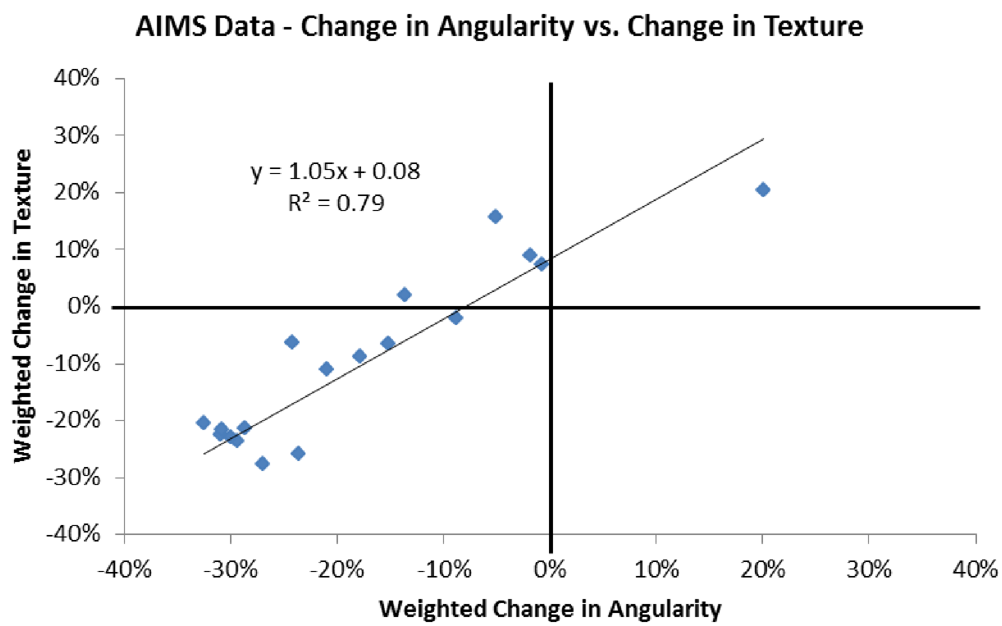

Figure 46: Weighted Change in Angularity vs. Weighted Change in Texture

A linear regression analysis, also displayed in Figure 46, shows that change in angularity and change in texture are strongly correlated ($R^2 = 0.79$). The negative slope indicates that a decrease in texture is accompanied by a decrease in angularity which is behavior that should be expected due to the abrasion of the Micro-Deval test. Identifying the material type of each data point provides more insight. Figure 47 demonstrates that four of the five aggregates displaying an increase in texture are river gravels.

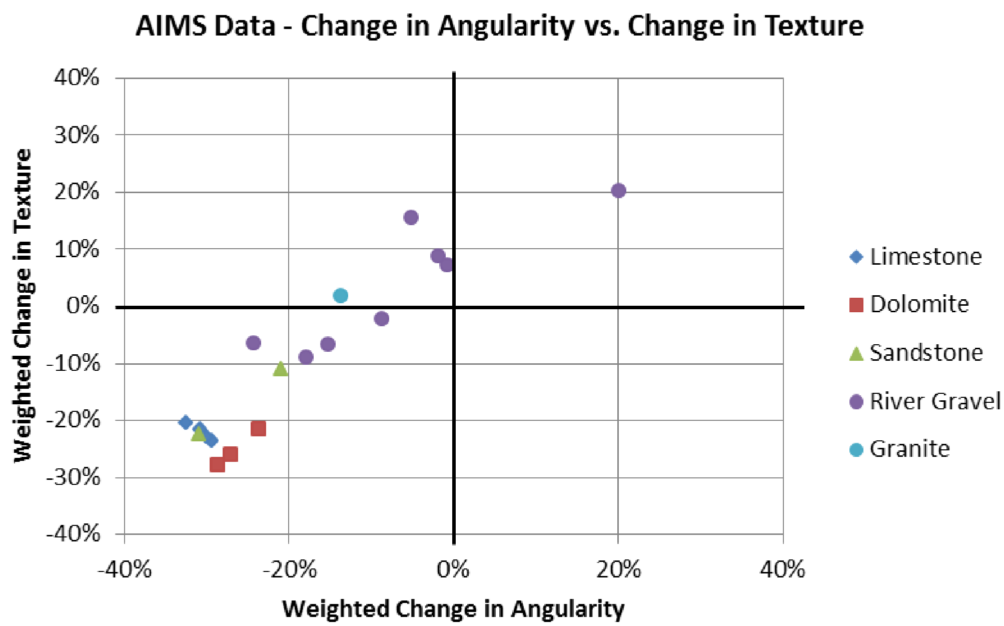

Figure 47: Weighted Change in Angularity vs. Weighted Change in Texture – by Material Type

Including Micro-Deval loss in the data analysis shows that all five of the aggregates showing an increase in texture are materials that are highly resistant to abrasion (Micro-Deval losses of less than 10%) as shown in Figure 48. The one aggregate that showed an increase in angularity is also highly resistant to abrasion as shown in Figure 49. Change in sphericity did not seem to be correlated to Micro-Deval loss (Figure 50).

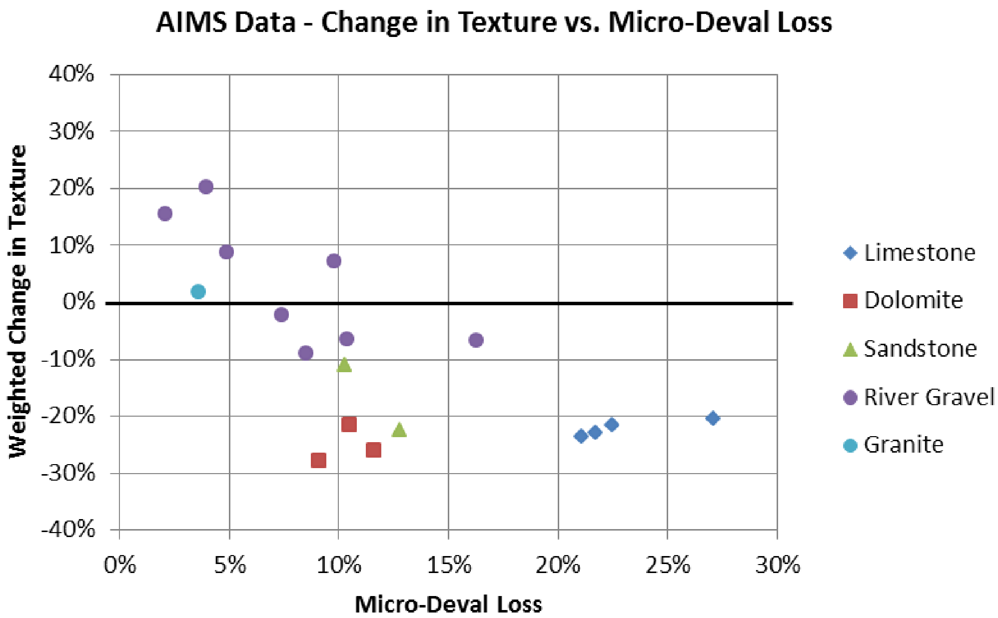

Figure 48: Change in Texture vs. Micro-Deval Loss – by Material Type

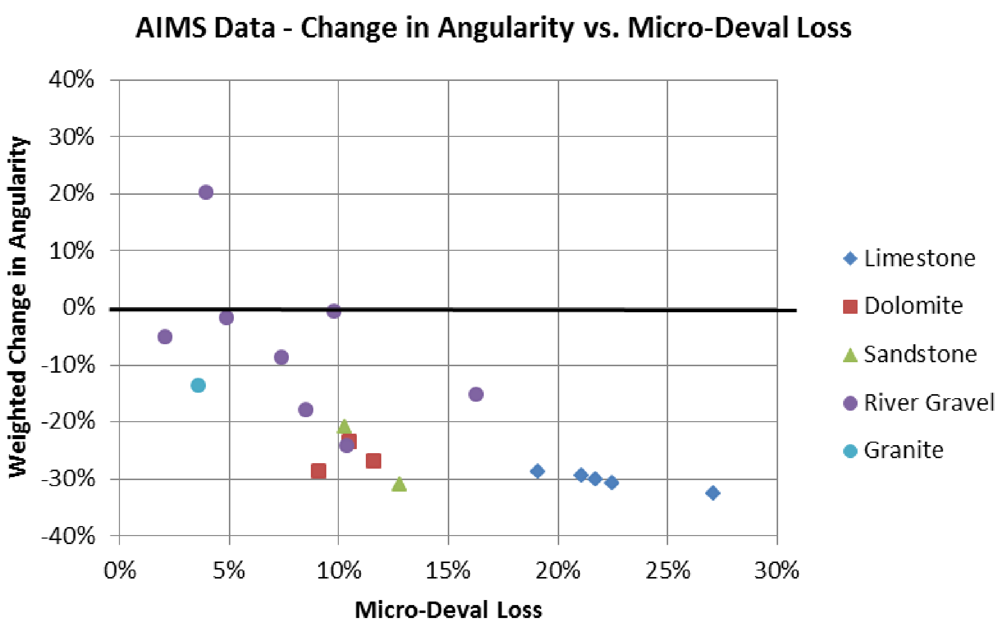

Figure 49: Change in Angularity vs. Micro-Deval Loss – by Material Type

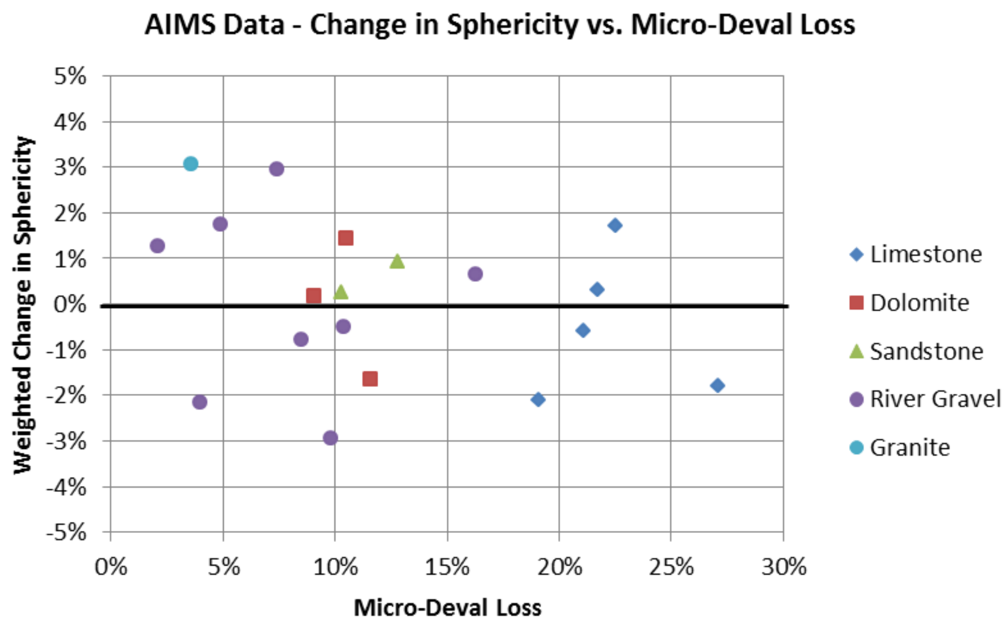

Figure 50: Change in Sphericity vs. Micro-Deval Loss – by Material Type

After data analysis demonstrated that river gravels with low Micro-Deval loss were the sources demonstrating the counterintuitive increase in texture, the research team discussed the issue with a TxDOT geologist who postulated that aggregate color was influencing the measurement of texture. To explore this idea further, set up a simple test was set up to determine if color of aggregate influences AIMS texture index.

6.4.1.1 AIMS Color Experiment

The AIMS texture index is based on an algorithm that uses a grey-scale image of the aggregate which is captured by a camera during AIMS analysis. In the AIMS color experiment, the research team selected a white limestone aggregate (1/2-in.) to use as a control. The control aggregate was analyzed by the AIMS machine and then the tray was removed from the machine. Researchers used a black permanent marker to color the top surface of the aggregate black, being careful to not disturb the placement of the aggregate particles. Figure 51 shows the aggregates before and after coloring, and Figure 52 shows texture images captured by AIMS before and after coloring.

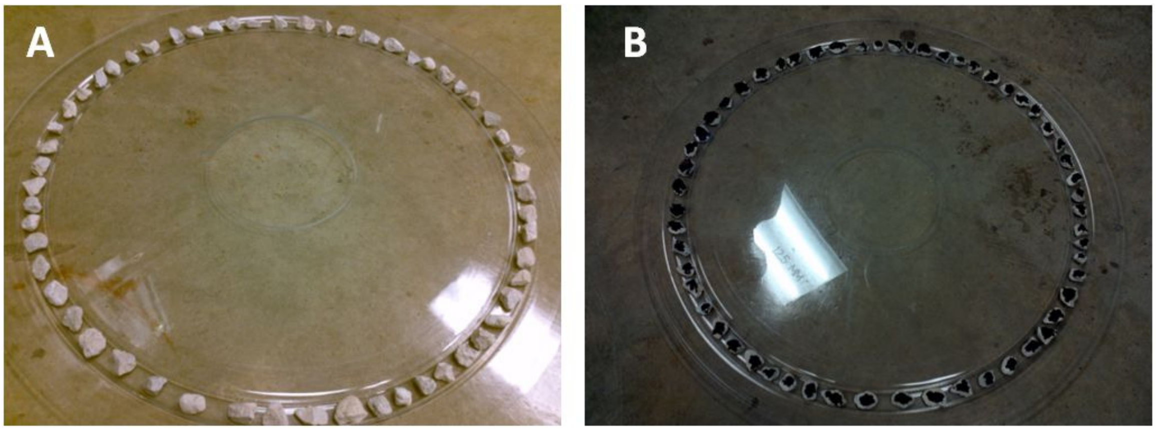

Figure 51: AIMS Color Experiment – Control Limestone (A), Colored Limestone (B)

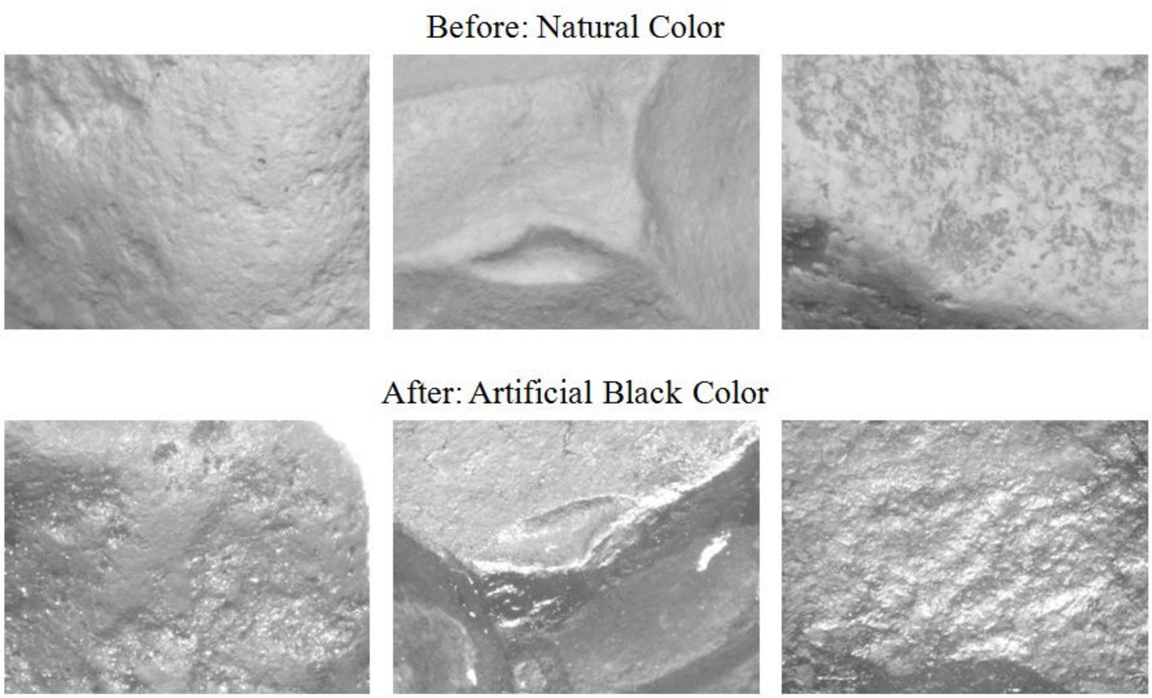

Figure 52: AIMS Photographs of Aggregates from Color Experiment

The results of the AIMS color experiment are shown in Figure 53. Simply coloring the aggregate black was enough to cause a 165% increase in texture, which is statistically significant. Angularity decreased by 1% (well within the variance of this test). Clearly, color is affecting the texture index of AIMS. Time permitting, the issue of color influence on AIMS texture will be explored further by others on the research team in the next several months.

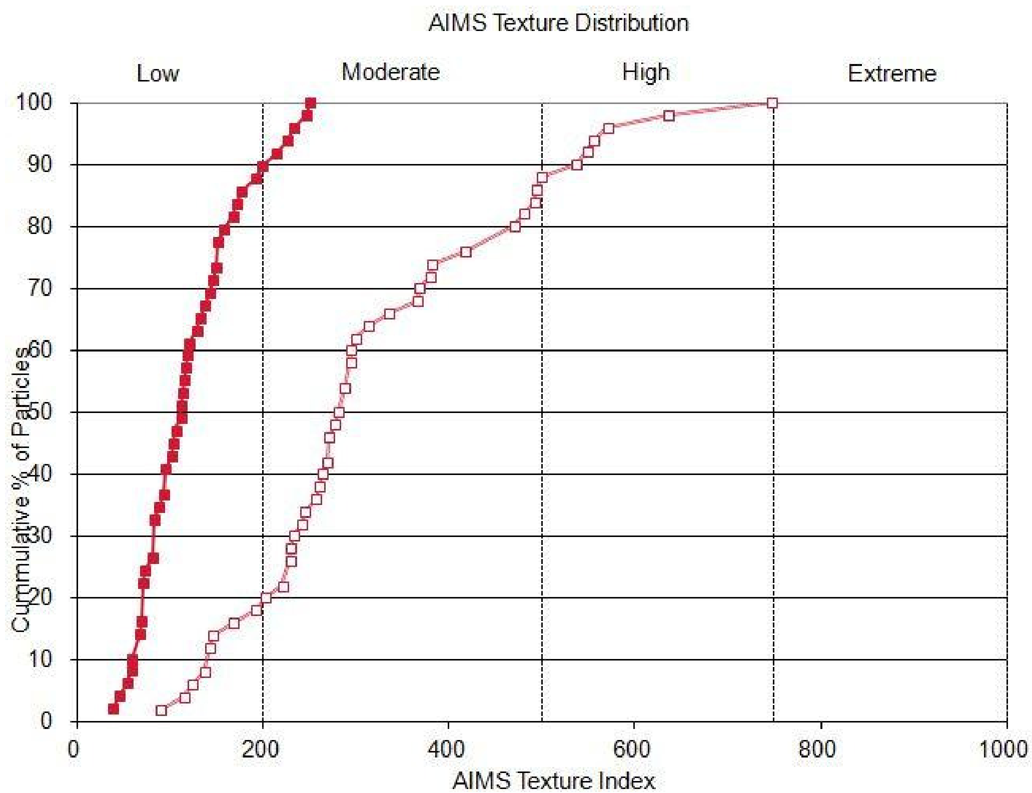

Figure 53: Texture Distribution of Control Aggregate (Red Points) and Colored Aggregate (White Points)

6.4.2 AIMS Data for Fine Aggregates

Fine aggregates were also analyzed by the AIMS machine. To determine if shape characteristics affected the flakiness of an aggregate (as defined by the 2008 Rogers and Gorman Flakiness test), researchers plotted flakiness results versus AIMS Form 2D and AIMS Angularity. These results are displayed in Figure 54 and Figure 55. Surprisingly, in both cases, fine aggregate retained on the No. 16 (1.18-mm) sieve shows much higher correlation (R^2 = 0.73 and 0.52) for flakiness vs. AIMS shape characteristics than does fine aggregate retained on the No. 8 (2.36-mm) sieve (R^2 = 0.19 and 0.26). Due to the limited number of tests performed so far, it is not reasonable to draw any conclusions from these relationships.

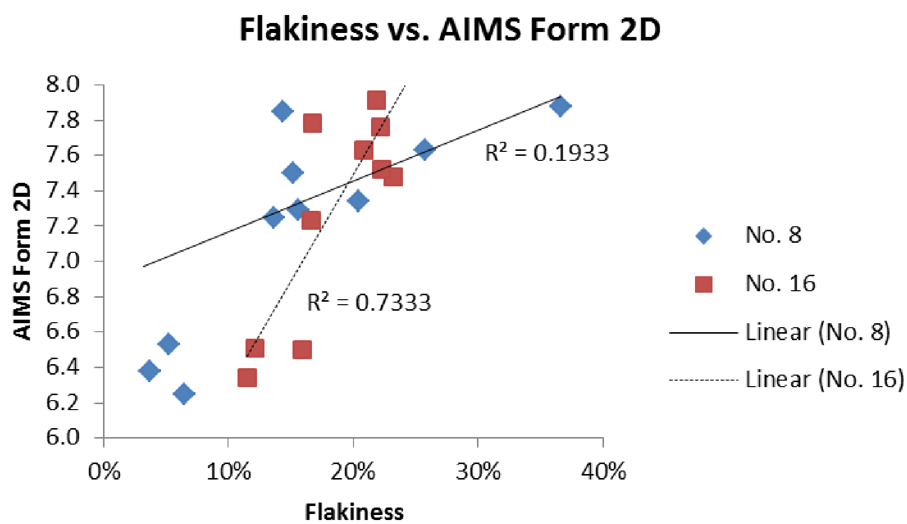

Figure 54: Flakiness vs. AIMS Form 2D

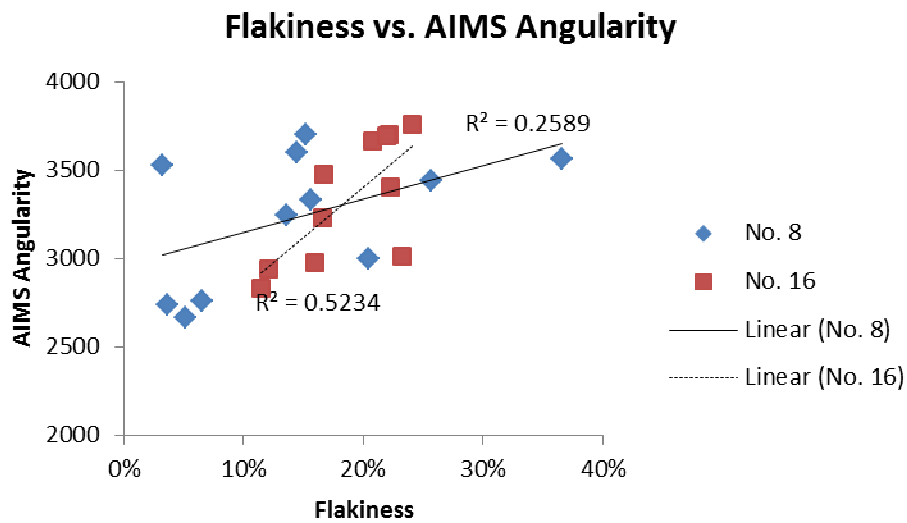

Figure 55: Flakiness vs. AIMS Angularity

Classifying the sands by material type (natural and manufactured) also provides an interesting look at the same test comparison. Natural sands tend to be grouped together and in general demonstrate lower flakiness and lower angularity than manufactured sands (as shown in Figure 56 and Figure 57). These figures also demonstrate that although manufactured sands tend to have high angularity (as defined by AIMS), this does not necessarily mean that they will have high flakiness. Crushing action during production will produce highly angular particles, but these particles can sometimes be cubical (low flakiness) instead of flakey (high flakiness).

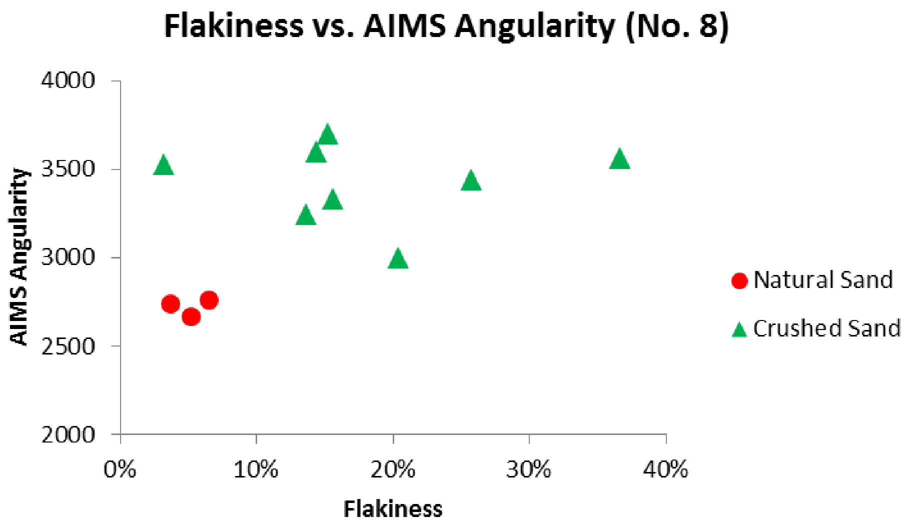

Figure 56: Flakiness vs. AIMS Angularity – by Material Type

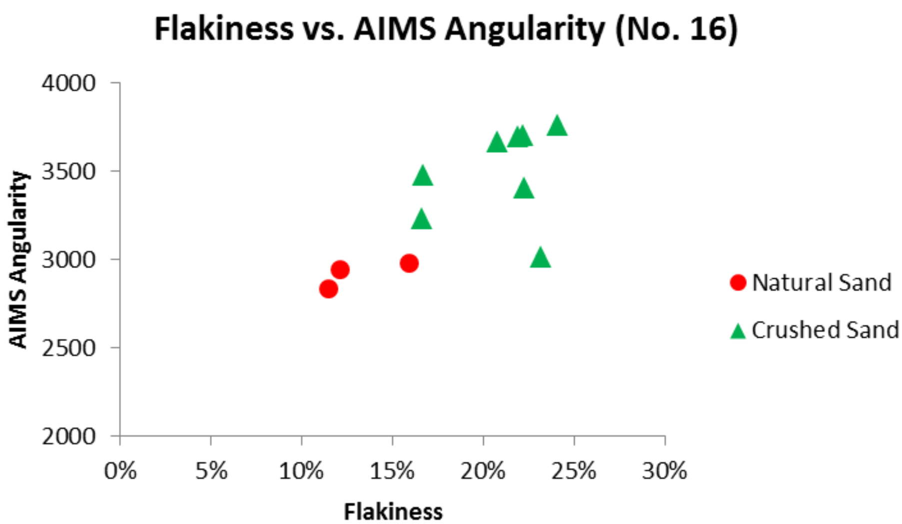

Figure 57: Flakiness vs. AIMS Angularity – by Material Type

Chapter 7: Summary and Conclusions

7.1 Project Summary to Date

Significant progress has been made during the first 18 months of this project. The first half of this project was spent reviewing literature, collecting information and data from TxDOT and industry personnel, developing the test plan, collecting aggregates, and testing aggregates. As of April 2012, 24 coarse aggregates and 38 fine aggregates have been acquired (including material available at the laboratory through previous projects), representing 48 unique sources, 11 TxDOT districts, and a variety of lithologies. Several aggregate samples from additional sources are expected to arrive at the laboratory for testing in the next few weeks.

Testing has been performed on a variety of coarse aggregates for Micro-Deval, AIMS 2.0, flat and elongated particles, specific gravity and absorption, aggregate crushing value, and aggregate impact value. Coarse aggregate samples have been prepared to be tested at TxDOT for LA abrasion, magnesium sulfate soundness, petrography, and X-ray diffraction. Over the coming months, researchers will be performing additional coarse aggregate tests which include thermal conductivity and X-ray diffraction. Testing has also been performed on a variety of fine aggregates for Micro-Deval, specific gravity and absorption, AIMS 2.0, flakiness sieve, Grace methylene blue, organic impurities, sand equivalent, and acid insoluble residue. Fine aggregate samples have been prepared to be tested at TxDOT for petrography and X-ray diffraction. Over the coming months, researchers will also be performing additional fine aggregate testing for chemical composition through X-ray diffraction. Concrete mixtures will be prepared soon so that tests can be performed for compressive strength, modulus of elasticity, flexural strength, and coefficient of thermal expansion.

7.2 CONCLUSIONS

Although manyaggregate tests have been completed over the last several months, there are not enough data to draw significant conclusions at this point. However, there are enough data to examine trends in the aggregate tests as discussed in Chapter 6: Discussion and Analysis. These trends will be monitored as additional data become available through testing.

Some important observations from the first half of this project include:

- In several instances, limits in Item 421 (particularly magnesium sulfate soundness and sand equivalent) have caused local aggregates to be rejected even though they have been used successfully in other non-TxDOT concrete projects around the state.
- Although the Grace methylene blue test shows great promise for detecting quantity of clay minerals in a fine aggregate, it is recommended that a slight change in procedure be made to achieve accurate results (see Section 5.2.5 Grace Methylene Blue Test).
- 15% of Texas aggregates on the concrete AQMP list have Micro-Deval losses of 20% or higher (as shown in Figure 31). Typical Micro-Deval limits would reject too many Texas aggregates that have demonstrated good performance.
- Thus far, both ACV and AIV have strong correlations to LA abrasion, with ACV having a slightly stronger correlation (R^2 = 0.901 and 0.786 respectively).

- Linear regression analysis actually showed that the correlation coefficient between Micro-Deval loss for fine aggregates and coarse aggregates from the same source was only 0.61, somewhat lower than expected.
- Linear regression analysis showed that Micro-Deval abrasion of coarse aggregates caused change in angularity and changes in texture that are strongly correlated (R^2 = 0.79). The negative slope indicates that a decrease in texture is accompanied by a decrease in angularity, which is behavior that should be expected.
- Several river gravels with low Micro-Deval loss have demonstrated a counterintuitive increase in texture after abrasion by Micro-Deval.
- A simple test involving the color of a white aggregate demonstrated that color significantly affects the AIMS texture index. This issue will be explored further, time permitting.

Appendix

Table 13: List of Attendees of June 2011 Aggregate Workshop

Organization	Name	Email
	The University of Texas	
CTR	David Whitney	dpwhitney@mail.utexas.edu
CTR	Chris Clement	chris.clement@utexas.edu
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CTR	David Fowler	dwf@mail.utexas.edu
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CST Division	Lisa Lukefahr	elizabeth.lukefahr@txdot.gov
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Bridge Division	Graham Bettis	graham.bettis@dot.gov
Bridge Division	Kevin Pruski	kevin.pruski@txdot.gov
RTI	German Claros	german.claros@txdot.gov
District Personnel	Steve Swindell	steven.swindell@txdot.gov
District Personnel	Darlene Goehl	darlene.goehl@txdot.gov
District Personnel	Richard Willammee	richard.williammee@txdot.gov
District Personnel	Charles Chance	charles.chance@txdot.gov
District Personnel	Ron Johnston	ron.johnston@txdot.gov
	Researchers	
Ontario MTO	Chris Rogers (Retired)	rogers.chris@rogers.com
	Industry / Producers	
Jobe Materials	Martin Alerette	martin@jobeco.com
Vulcan	Harry Bush	bushh@vmcmail.com
Martin Marietta	Mike Carney	mike.carney@martinmarietta.com
Martin Marietta	Jason Ford	jason.ford@martinmarietta.com
Fordyce Materials	Matt Champion	matt@fordyceco.com
TACA	Richard Szecsy	rich.szecsy@tx-taca.org

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