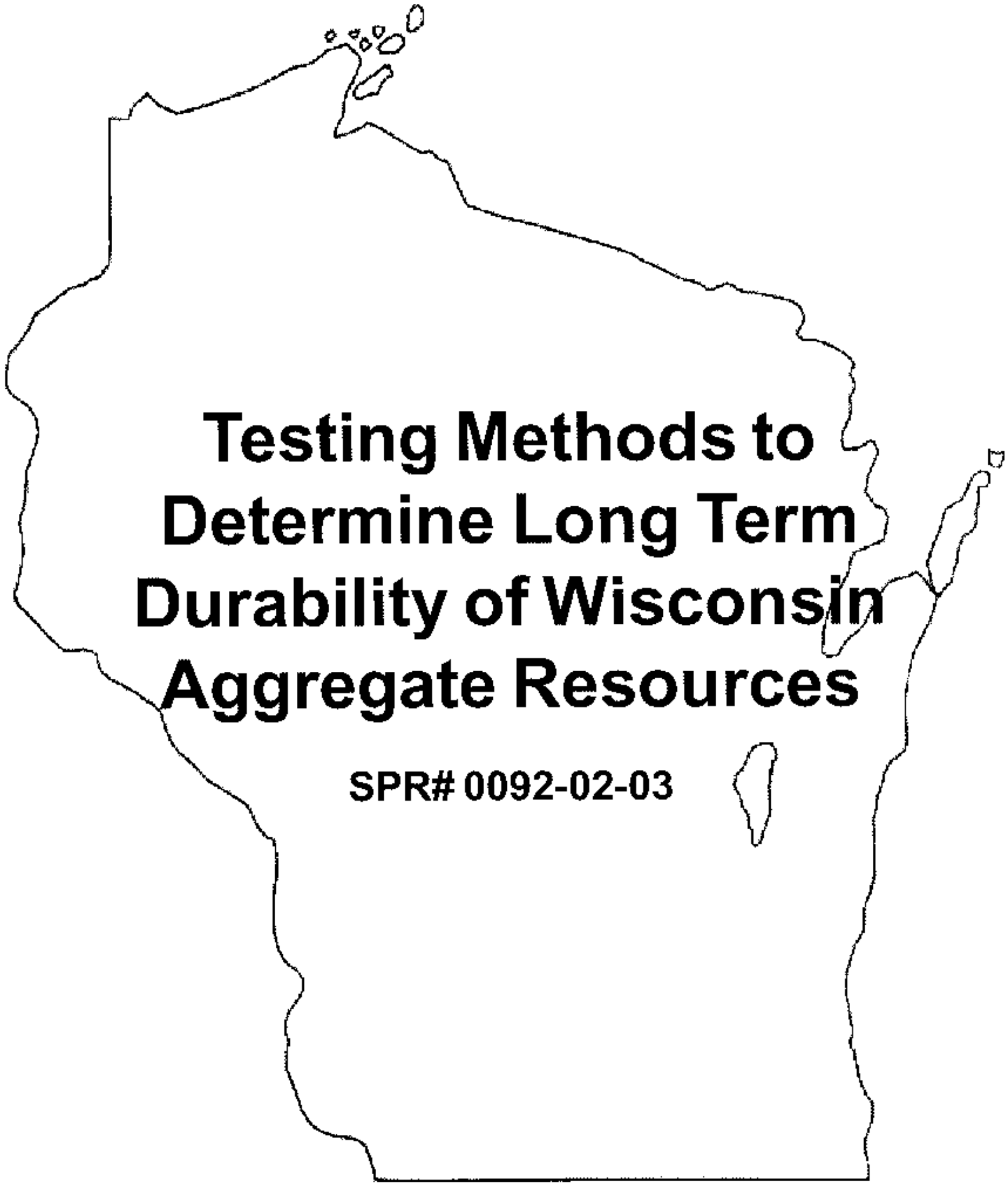
Wisconsin Highway Research Program



Testing Methods to Determine Long Term Durability of Wisconsin Aggregate Resources

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16. Abstract

The Wisconsin Department of Transportation (WisDOT) uses approximately 11,000,000 tons of aggregate per year for transportation projects. Being able to select durable aggregates for use in transportation projects is of considerable importance, if the aggregate deteriorates then the constructed facility requires premature repair, rehabilitation or replacement. Realizing the importance and also that deficiencies in the current WisDOT testing protocol may exist, it has been concluded that the durability-testing program for Wisconsin aggregates needs to be updated. It should also be noted that the use of recycled and reclaimed aggregates has increased in recent years and not all typical durability tests can be used for testing these aggregates.

This project has identified recent advances in the understanding and testing of aggregate durability. An in depth literature review has been conducted and from the compiled information a laboratory testing program was developed. Selection of the tests was based upon the tests' precision, efficiency, and predictive capabilities. In the laboratory-testing phase of this project the proposed durability tests along with current WisDOT durability tests were used to evaluate the full range of Wisconsin aggregates.

From the test results it was found that the WisDOT aggregate testing protocol could be reduced substantially by eliminating many of the testing requirements for aggregates that have a vacuum saturated absorption of less than 2%. Also, the addition of several tests was ruled out due to their lack of correlation with field performance records. The Micro-Deval abrasion test is recommended for inclusion in WisDOT testing protocol as a test to measure the abrasion resistance of aggregate while the L.A. Abrasion test is better suited as a measure of aggregate strength. Additional conclusions were made based on the durability testing conducted and an overall testing protocol has been developed and is recommended for implementation by WisDOT.

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EXECUTIVE SUMMARY

Project Summary

This project investigated the predictive capabilities of a wide range of aggregate durability tests for Wisconsin coarse aggregate resources with the goal of developing a coarse aggregate durability testing protocol. It is of great importance to be able to distinguish between durable and non-durable aggregate sources because if the aggregate deteriorates the life-cycle of the constructed facility will be adversely affected. Wisconsin Department of Transportation (WisDOT) officials have determined that the current aggregate durability testing protocol for Wisconsin may be deficient and that there is a need to update the testing program in light of recent developments in aggregate durability testing. Also, the use of recycled/reclaimed aggregate material has increased dramatically in recent years and typical durability tests may not be capable of accurately assessing the durability of these aggregates.

The investigators were charged with the responsibility of reviewing the state-of-the-practice of aggregate durability testing and selecting potential tests to be investigated in the laboratory phase of the project. Additionally, current WisDOT aggregate durability tests were carried out in the laboratory for comparative purposes. As a result of this research, revised aggregate durability testing protocols have been proposed for both natural and recycled/reclaimed aggregate material for use in the bound or unbound state.

Background

Natural and recycled/reclaimed aggregate resources are used in nearly all transportation projects with their uses ranging from unbound base course layers to structural concrete constituents. For each application the aggregate is exposed to a different set of physical and chemical degrading forces. Some of the forces that an aggregate may be exposed throughout its service life are: abrasive, tensile, shear, and compressive forces, sulfate exposure, wetting and drying cycles, and freezing and thawing cycles. It is important to investigate an aggregates susceptibility to deterioration by each of these modes. For an aggregate may be very resistant to abrasion but may not be freeze/thaw durable. As an example a concrete aggregate may have a high resistance to abrasion, high strength, and acceptable durability in an unconfined freezing and thawing test, but when it is tested in the confined state in concrete for freezing and thawing durability the pressure created by the expanding water inside the aggregate may be extruded into the cement paste at a rate high enough to fracture the cement paste.

It is also important to note that many of WisDOT's current aggregate durability tests have been in place for over 50 years and that recent developments in durability testing may be able to increase the predictive capabilities of WisDOT's aggregate durability testing protocol. Several new aggregate testing methods, such as the Micro-Deval, Canadian Unconfined Freezing and Thawing, and Aggregate Crushing Value tests, were investigated by this project. The method of application of current aggregate durability testing methods was also investigated and adjusted as necessary. Additionally, there are other promising aggregate durability tests that are currently in the developmental phase that may be considered for use by WisDOT in the future. However, it was deemed that there was not a thorough enough knowledge base to justify their incorporation into standard WisDOT testing protocol at this time.

This 2-year research project was completed through a collaborative effort between the Virginia Polytechnic Institute and State University, the Virginia Transportation Research Council (VTRC), and the Wisconsin Department of Transportation.

Process

When determining the durability of an aggregate resource it is important to not only test the durability of individual aggregate particles but also the durability of the inclusion material (i.e. Portland Cement Concrete or Bituminous Concrete). This is necessary due to the possible adverse effects that unsound aggregates may have on the inclusion material. Therefore, the proposed testing program has taken into account the differences in testing requirements for aggregates to be used in the unbound and bound conditions. The aggregate durability tests that were used in the laboratory investigation phase were selected based on the tests' precision, repeatability, efficiency, and predictive capabilities. A list of the tests that were performed is presented below in Table 1.

Aggregate Material	Crushed Stone and Gravel	Recycled Concrete Aggregate	Blast Furnace Slag	Recycled Asphalt Pavement
Lightweight Pieces in Aggregate (ASTM C 123-98)	X			
Vacuum Saturated Specific Gravity and Absorption (Modified ASTM C 127)	X	X	X	
Sodium Sulfate Soundness (ASTM C 88)	X	X	X	
Frost Resistance of Aggregates in Concrete (ASTM C 666)	X	X	X	
Unconfined Freezing and Thawing (CSA A23.2-24A)	X	X	X	
Micro-Deval Abrasion (AASHTO TP 58)	X	X	X	
L.A. Abrasion (ASTM C 131-01)	X	X	X	
Aggregate Crushing Value (British Standard 813 – Part 3)	X	X	X	
Compressive Strength of Cylindrical Concrete Specimens (ASTM C 39)	X	X	X	
Resistance of Compacted Asphalt to Moisture Induced Damage (AASHTO T 283)				X

TABLE 1

In total 70 natural aggregate resources (48 quarries, 22 pits) and 4 recycled/reclaimed aggregate resources were sampled. Of the recycled/reclaimed aggregate resources sampled two were Recycled Asphalt Pavement (RAP), one was a Recycled Concrete Pavement (RCP), and one was a Foundry Slag Aggregate. The sampled aggregates encompassed the full spectrum of available aggregate resources in Wisconsin. 1 ¼" base material was sampled where available. Otherwise, the next largest gradation was sampled. This was done to ensure that any durability issues that can be attributed to the critical size factor were evident in the testing.

Initially, all 70 natural aggregate resources were tested for Vacuum Saturated Absorption (VSA) and Vacuum Saturated Specific Gravity (VSSG). From this data a sample set of 60 aggregate sources was selected for additional testing based on VSA. Thirty samples were tested at Virginia Tech and 30 samples were tested at VTRC. Additionally, all recycled/reclaimed resources were subjected to all applicable tests at Virginia Tech.

Findings and Conclusions

This project resulted in the development of updated testing protocols for unconfined, bituminous concrete, and Portland cement concrete. These protocols are presented in flowchart format in Figures 1-3 below. The most significant finding was that the aggregate testing requirements can be greatly reduced by using an aggregates VSA as a preliminary durability indicator. It was found that aggregates with VSAs of less than 2% will meet the acceptance criteria for L.A. Abrasion, Micro-Deval, Unconfined and Confined Freezing and Thawing tests. This results in a large reduction in the required testing for WisDOT.

Recommendations for the testing of recycled/reclaimed aggregate resources were also made. It was concluded that RCP and Slag aggregates should be subjected to the same testing program as natural aggregates with the addition of iron and calcium disilicate unsoundness testing for slag. RAP aggregates should be tested in accordance with AASHTO T 283. Additional conclusions and recommendations for individual tests are presented in the report.

If the recommended testing protocols are implemented the result should be increased efficiency, increased ability to properly classify aggregate resources, and reduced costs. The proposed testing protocols also have the potential to be used as models for other state DOTs for developing their own state specific aggregate durability testing programs.

Recommendations for Action

It is recommended that WisDOT specifications be updated to reflect the conclusions and recommendations presented in this report. However, additional research relating to the use of the Canadian Unconfined Freezing and Thawing test may be justified prior to its institution. The requirement for additional equipment is limited with an estimated cost of less than $2000 per laboratory.

The responsibility for the implementation of this research lies on WisDOT management. It is felt that the information presented in this report can be effectively communicated through written correspondence between WisDOT offices and minimal training.

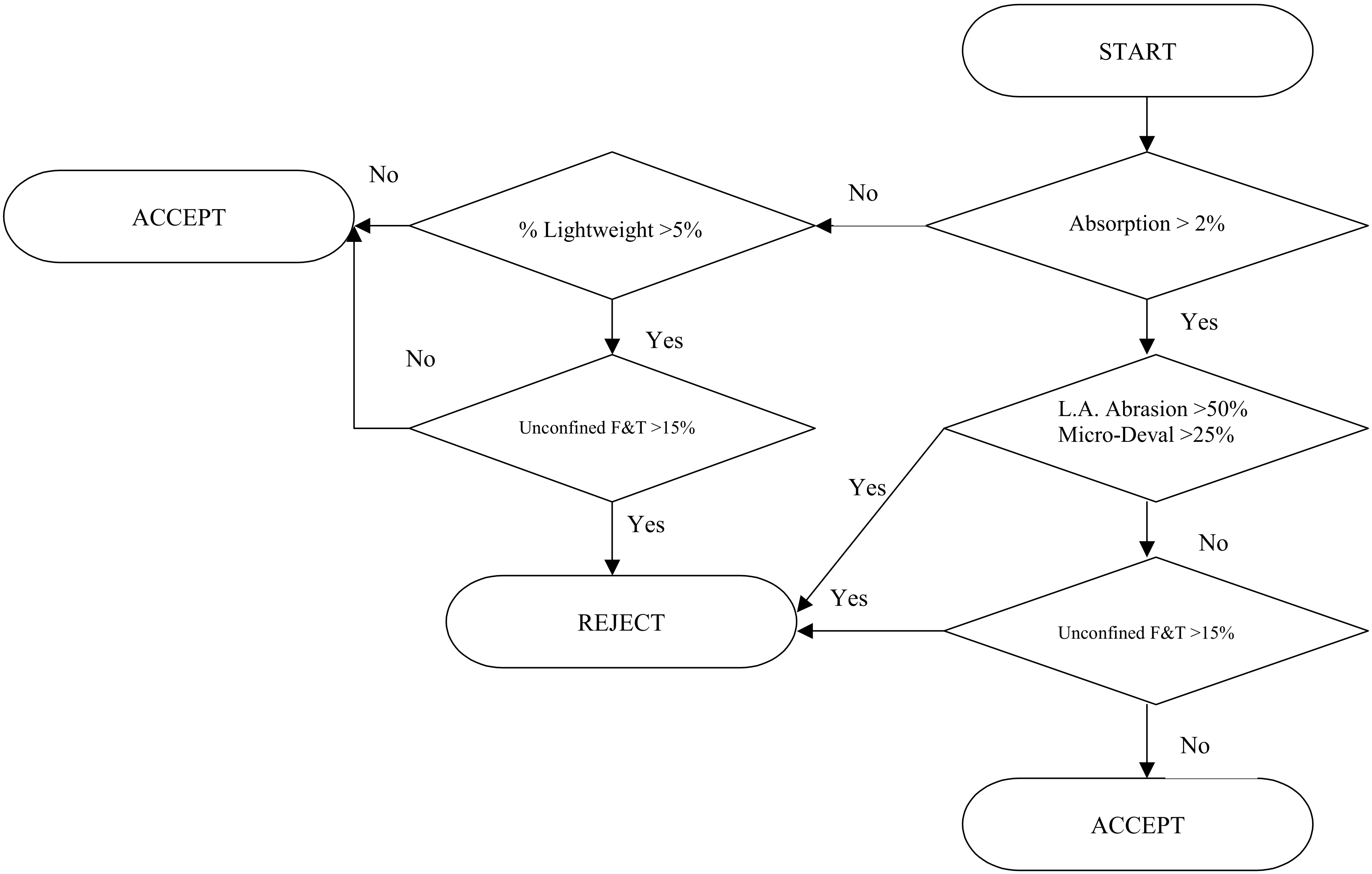

Figure 1 – Aggregate Durability Testing Flowchart for Unbound Aggregate

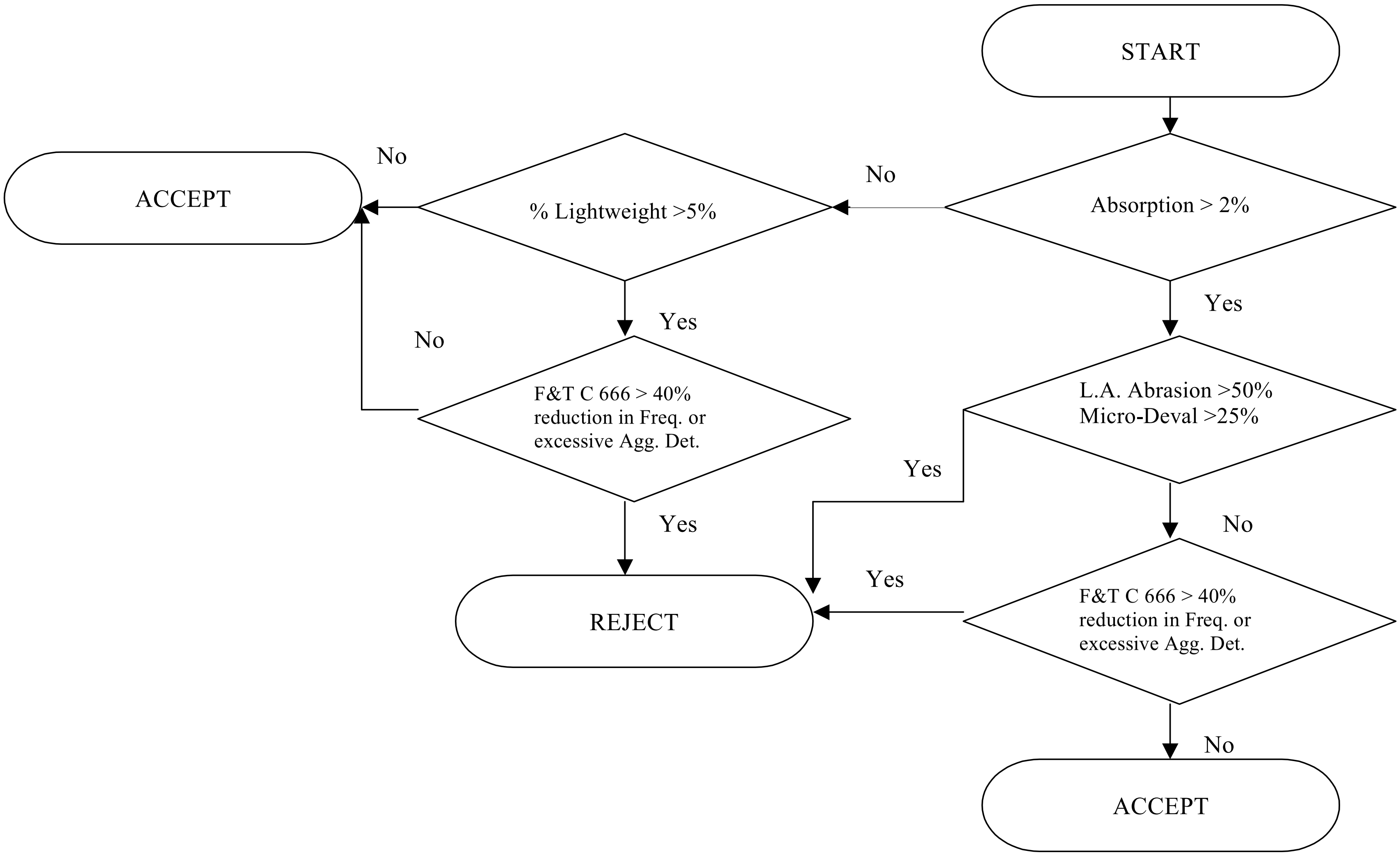

Figure 2 – Aggregate Durability Testing Flowchart for Concrete Aggregates

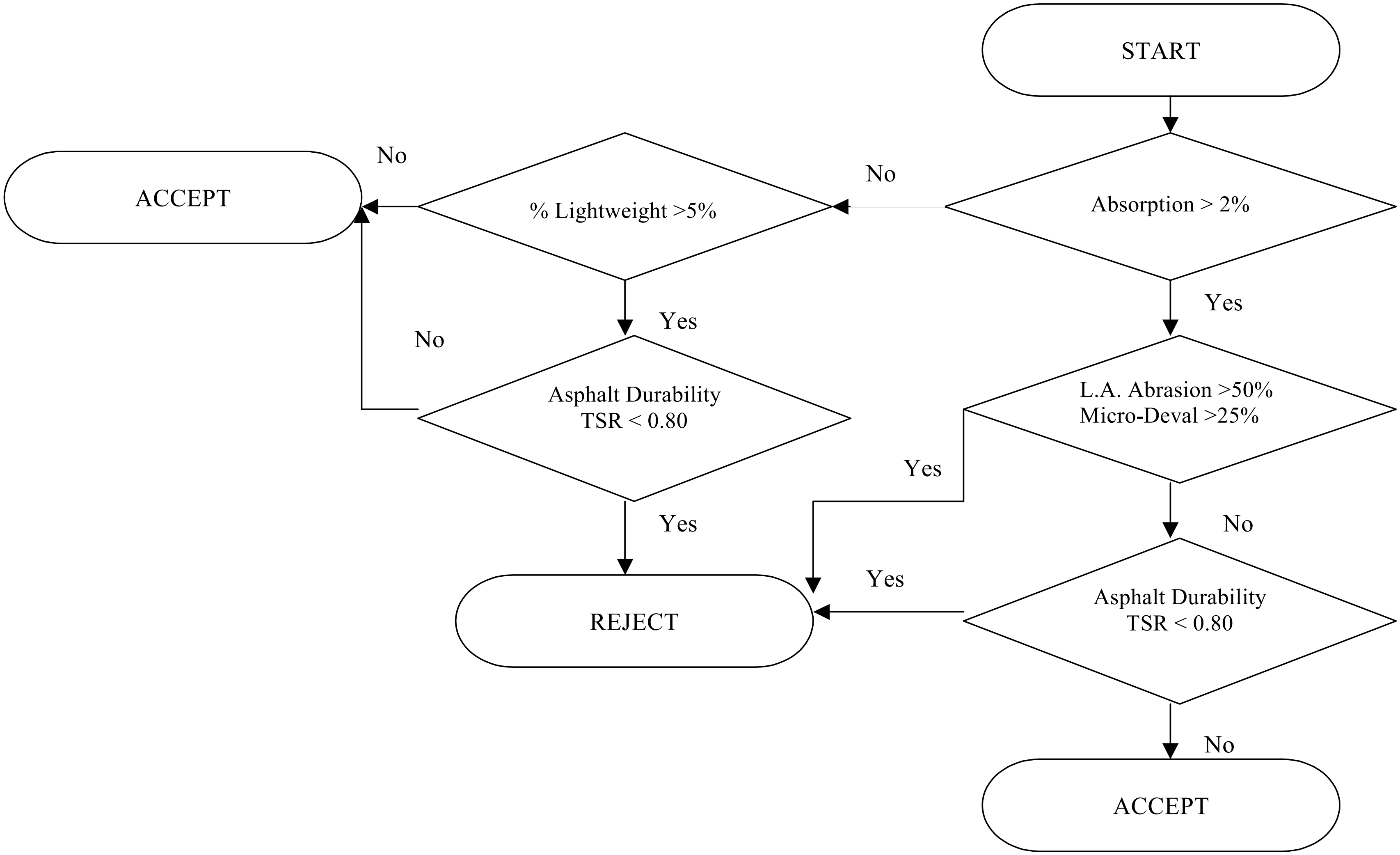

Figure 3 – Aggregate Durability Testing Flowchart for Bituminous Aggregates

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CHAPTER 1: INTRODUCTION

Aggregate is the most fundamental component of construction. It is used as an unbound material in base courses and as a bound material in bituminous and Portland cement concretes. Aggregate constitutes about 90% of the volume of base courses and bituminous concrete and 60 to 80% of the volume of Portland cement concrete. Aggregate is exposed to a number of physical and chemically degrading forces during processing, transporting, and construction as well as during the constructed facility's life. As the main load carrying component of unbound and bituminous and Portland cement concretes, if the aggregate fails, the facility fails to perform its designed intent.

Thus, State DOT's such as Wisconsin have employed a number of tests to ensure durable aggregate is used in their constructed facilities. However, the influence of State DOT specifications extends far beyond the limits of their constructed facilities. Their specifications and test methods serve as a guideline for private construction. Thus, it is imperative that WisDOT's aggregate testing protocol be applicable to the assessment of the long-term durability of constructed facilities. In this case applicability means that neither inferior products are accepted nor acceptable products rejected from their intended use. Durability is the chemical and environmental (physical) resistance to degradation of the material throughout the service life of the facility.

To ensure aggregate durability and to minimize their impact on the durability of pavements and structures, WisDOT has used a number of standard test methods. The standard test methods include gradation, plasticity, resistance to abrasion (impact), soundness, and freezing and thawing resistance. Some of the tests have been developed as aggregate quality assurance tests and others have been borrowed from other materials testing programs. These tests have been in use for well over 50 years and for the most part have served the highway industry well. However, these tests were developed when high quality natural sources of aggregate were abundant and social and political pressures on the use of industrial by-products and recycled/reclaimed materials were nonexistent. In addition, some of these tests have been kept in use in the name of "tradition" and "simplicity" rather than being replaced by other methods based on our ever-increasing understanding of the science of aggregate durability. Thus, given the social and political environment to use what once may have been termed "marginal or unacceptable materials" and the present state of the knowledge of aggregate degradation mechanisms, WisDOT has deemed it necessary to assess present aggregate testing protocol in light of "performance" criteria.

1.1 Background

The following discussion is presented to illustrate the underlying philosophy that will be used to develop the WisDOT aggregate durability testing protocol.

WisDOT projects use aggregates as unbound pavement base courses, bituminous and Portland cement concrete pavements and structural concretes for bridges, box culverts, and retaining walls. Each one of these facility components is exposed to a set of chemical and physical degrading forces during construction and throughout the facility's service life. For example, structural Portland cement concrete aggregates are exposed to:

- Abrasive forces while the aggregate is in a moist/wet state during stock piling, transporting, batching, mixing, and placing of the concrete.
- Tensile, shear, and compressive stresses during loading of the reinforced concrete structure.
- Chemical environments of a saturated solution of calcium hydroxide, sodium and potassium hydroxide, and sulfates in the concrete.
- Wetting and drying cycles of the concrete.
- Temperature changes including freezing and thawing of absorbed moisture of the concrete.

With respect to temperature changes, the aggregate volume changes must be compatible with the Portland cement paste, which binds the aggregate together. During the freezing of saturated aggregates bound together by Portland cement paste, the pressure being created by the freezing water must not be sufficient to fracture the aggregate nor be "extruded" into the surrounding cement paste at a rate which fractures the cement paste (Verbeck, 1960). Of these three potential aggregate temperature-related destructive mechanisms, two are related to the aggregate being bound within cement paste. An unbound aggregate test such as soundness or the freezing and thawing test would not assess these aggregate performance related aspects. This may be the reason why these tests have been poor predictors of the freezing and thawing durability of certain aggregate types. It is noteworthy that the "expulsion" distance mechanism may become more important in the future as more low permeable concretes are produced which have denser aggregate-paste transition zones.

The above could be interpreted that it is necessary to test all concrete aggregates exposed to freezing and thawing under saturated conditions in a concrete freezing and thawing test. This will not be necessary because an aggregate with porosity below 0.3% produces expansions within the elastic limits of the aggregate and surrounding cement paste (Verbeck, 1960). Thus, a concrete aggregate protocol may consist of a simple, precise test for water absorption and saturated specific gravity to identify the very good and very poor aggregates and a freezing and thawing or surrogate test for marginal aggregates.

However, since the freezing and thawing of base courses takes place in the unbound state, the soundness test may be sufficient if the lack of precision aspects of this test can be addressed. Also, Portland cement concrete pavement aggregates need to be assessed for the propensity of D-cracking (durability cracking). D-cracking is cracking that occurs along the edges or at the corners of concrete slabs due to the expansion of non-durable coarse aggregates during freezing and thawing. The resulting deterioration are crescent shaped cracks or spalls in the concrete.

The developed aggregate durability protocol is based on construction-service life performance criteria that can be addressed by performance tests that realistically simulate the field exposure and degradation process (Frondsdoff, 1995).

CHAPTER 2: LITERATURE REVIEW

A literature review was conducted using computer and manual search methods of databases including TRIS, the University of New Hampshire RMRC, NCHRP Digest and TxDOT online literature to assess the state-of-the-practice of the durability testing of "all aggregate" types (natural and recycled material aggregates). In addition, personal contact was made with Kevin Folliard, University of Texas at Austin, Jason Harrington, FHWA, and Brian Prowell, NCAT at Auburn University. The literature has been categorized by the specific test method due to the overlap between testing methods and structures and pavement types. The relevance that each testing method has to the facility/pavement types will be addressed and cross-referenced where applicable. An additional section will address the use and testing of recycled/reclaimed aggregates.

2.1 Tests

2.1.1 Specific Gravity and Absorption (ASTM C 127-01 AND ASTM C 128-01)

Tests for the specific gravity and absorption characteristics of aggregates have long been used to aid in determining batch quantities for concrete, but in recent years these characteristics have also been used to predict the freezing and thawing durability of aggregates. ASTM tests C127 (AASHTO T 84) and C 128 (AASHTO T 85) are the generally accepted test procedures and are currently used by WisDOT. There are several studies that identify the possibility of using specific gravity and absorption as predictors for aggregate soundness. A study conducted in 1995 by Pigeon and Pleau suggests that a limit of 2 percent be placed on the absorption of coarse aggregates to prevent freezing and thawing damage from occurring. Another report by Harman, Cady, and Bolling (1970) suggests that the vacuum saturated surface dry specific gravity can be used in conjunction with the vacuum saturated absorption as a preliminary acceptance test. Aggregates may be identified graphically as good, intermediate, or poor. Limits on aggregate absorption values have also been suggested to identify low shrinkage aggregates, 0.5 and 1.5 percent for fine and coarse aggregates, respectively. (Babaei and Purvis, 1994)

Several state DOT's have placed limits on the absorption of aggregates to help prevent freezing and thawing damage from occurring. For example, the Minnesota Department of Transportation limits the absorption of PCC aggregates to 1.7%. No changes to current WisDOT procedures would be necessary to incorporate this test as a durability indicator. This test is not suitable for evaluating Recycled Asphalt Pavements (RAP) or Recycled Concrete Aggregates (RCA) due to the adverse effects that adhered binder and mortar have on absorption data.

2.1.2 Soundness of Aggregates by Use of Sodium Sulfate (ASTM C 88-99a)

The Sulfate soundness test is one of the most widely used tests for the prediction of freezing and thawing durability of aggregates in the United States. The test is conducted in accordance with ASTM C88 or AASHTO T 104, and either sodium sulfate or magnesium sulfate may be used. Freezing and thawing cycles are simulated by immersing the aggregate in a sulfate solution, drying the aggregate, and then re-immersing the aggregate in the sulfate solution. Expansive forces are created when the sulfate crystals in the aggregate pores are re-hydrated. The expansion of the salt is assumed to simulate the forces that are created when water freezes in aggregate pores. The ranges of mass loss allowed in specifications vary from agency to agency with the type of sulfate used. Typical limits are 12 and 18 percent loss for sodium and magnesium sulfate, respectively. NCHRP studies on aggregates for use in unbound base and asphalt and Portland cement concrete pavements (Saeed et al, 2001; Kandhal and Parker, 1998; and Folliard and Smith, 2002) recommend that only the magnesium sulfate test be used due to the fact that it provides more precise values than the sodium sulfate, a statistic that is acknowledged in the ASTM and AASHTO test methods.

However, sulfate soundness tests do not address the condition where aggregate particles have a high porosity and permeability, which can cause fracturing of the surrounding Portland cement paste or cause stripping of asphalt cement.

The sulfate soundness test has come under scrutiny because of its lack of precision. A report by Harman et al (1970) suggests that specific gravity and absorption may serve as better indicators of aggregate soundness than the sulfate test. The deficiencies of the sulfate test are also noted in ASTM Standard C88, which states that it "may not be suitable for outright rejection of aggregates". However, a study conducted by Sheftick (1989) found that the single operator coefficient of variation ranged from 5.2 for a limestone to 13.5 for a dolomite. These values are significantly less than the reported value in ASTM C 88 of 24 percent for a single operator. Test procedure changes were suggested to reduce the variability of the sodium sulfate soundness test.

WisDOT is currently using the Sodium Sulfate Soundness test as an indicator for the freezing and thawing durability of aggregates for use in all types of pavement and base layers and also for aggregates used in structural concretes. At the very least the replacement of the sodium sulfate with magnesium sulfate will be considered by this project. Other possibilities such as the use of the Specific Gravity and Absorption Test as indicators will also be investigated. It is also noted that the Sulfate Soundness test is inappropriate for evaluating Recycled Concrete Aggregates because of the chemical reaction that occurs between the sulfate and mortar results in very high mass losses, which may not represent actual aggregate soundness characteristics. Specific gravity and absorption may not be applicable for all or some of the recycled materials or different limits may be needed.

A distribution analysis of Wisconsin Sodium Sulfate Soundness test results is presented in Appendix A (See Figure A-3)

2.1.3 Tube Suction Test

The tube suction test is used to predict the freezing and thawing durability of unbound aggregate base layers. The test monitors the capillary rise of moisture in a cylinder of compacted aggregate. The dielectric constant is measured at the surface of the sample through use of a probe. The dielectric constant is a measure of the unbound water in the aggregate sample. The amount of unbound water present is used to predict the freezing and thawing performance of the aggregate tested. (Syed 2000)

In 1995 Scullion et al conducted preliminary tube suction testing and concluded that the results were "promising". Syed, Scullion, and Randolph completed further research in 2000 that recommended that the tube suction test be "included in standard aggregate acceptance testing for design and construction of future pavement projects". However, testing was limited to only four marginal aggregates, all of which failed.

The scope of research on the tube suction test is too limited to recommend its inclusion in the WisDOT testing protocol. Also there are no set limits for the acceptance or rejection of aggregates tested using this procedure.

2.1.4 Los Angeles Abrasion Test (ASTM C 131-01)

The Los Angeles Abrasion Test was originally developed in Los Angeles in the 1920s. The test consists of placing an aggregate sample in a steel drum along with 6-12 steel spheres weighing approximately 420 gm each. The steel drum is then rotated for 500 revolutions. A steel shelf within the drum lifts and drops the aggregate and steel spheres with each revolution. Following the completion of the 500 revolutions the resulting sample is dry sieved over a No. 12 sieve to determine the amount of degradation that occurred during the test. The test can only be used with coarse, dry aggregate. WisDOT currently uses the L.A. Abrasion Test as an acceptance criterion for aggregates to be used in all pavement types. WisDOT conducts the test for 100 revolutions and 500 revolutions to estimate the degradation that will occur during handling and mixing and to estimate the overall durability of the aggregates, respectively. A linear regression analysis was conducted on L.A. Abrasion test data from the WisDOT approved aggregate resources database and it was found that it is generally not necessary to do two tests on the same sample because the results are highly correlated. (See Figure A-6 in Appendix A) Histograms of the test results are also presented in Appendix A. (See Figures A-4 and A-5)

The L.A. Abrasion test was identified as the fourth most important aggregate property (Cominsky, et. Al., 1944). A recent survey of 48 states and Provinces found that 96% of the agencies use the LA abrasion test (Prowell, 2004). A survey as part of another study showed that most agencies believe that L.A. Abrasion is more related to breakdown during compaction and they are satisfied with their current specifications (Amirkhanian, et. Al., 1991). Senior and Rogers 1991, summarized the concerns with the L.A. Abrasion test which led to investigations of other test methods.

- Severe impact of the steel balls overshadows particle abrasion, which dominates in pavements subject to traffic.

- Brittle materials such as granite and gneiss have high L.A. Abrasion loss but have adequate field performance.

- Soft aggregate such as limestone, dolomite and shale absorb the impact but tend to degrade when wet.

NCHRP Report 405 rates the field performance predictive capabilities of the test only as fair (Kandhal and Parker, 1998). However, early developmental studies indicated good correlations with performance (Woolf, 1937; Melville, 1948). Folliard and Smith(2002) recommend that the test only be used to estimate the amount of degradation that occurs during handling and mixing and that the test not be used in pavement specifications due to its "limited ability to predict PCC pavement performance". These sentiments are also reflected in NCHRP Reports #405 and #453, which relate to aggregate use in Asphalt Concrete and Unbound Pavement Layers (Kandhal and Parker, 1998; Seed and et. Al., 2001).

The L.A. Abrasion Test has also been used by state DOTs to predict the effect of studded tires on pavement surfaces, but its ability to predict pavement performance has not been demonstrated.

Other impact tests are the German Schlagversuch Impact Test and the British Aggregate Impact Test. The German impact test is poorly related to the L.A. Abrasion test while the relationship between the LA abrasion and British impact test is statistically significant, R = 0.403 and 0.731, respectively (Woodward, 1995). Prowell (2004) stated there is no evidence that suggests that the LA abrasion test should be replaced with another impact test. Whereas, Kandhal and Parker (1998) recommend that the Micro-Deval and magnesium sulfate tests be used in lieu of the LA abrasion test.

2.1.5 Micro-Deval Degradation Test (AASHTO TP 58-00)

The Micro-Deval test is a wet attrition test that is used to determine the potential for an aggregate to degrade during handling. The test was developed to include the effects of moisture on aggregate abrasion resistance, a property that cannot be determined by using the L.A. Abrasion test and does not include an impact component. NCHRP Report #453, #405, and NCHRP Research Results Digest #281 all recommend that the Micro-Deval test be used in place of the L.A. Abrasion test due to its ability to better reflect field conditions and predict field performance (Seed, et. Al., 2001; Kandhal and Parker, 1998; Folliard and Smith, 2003). Kandhal and Parker(1998) further recommended a coupling of the Micro-Deval with the magnesium sulfate test. Similarly, aggregate characteristics of absorption and specific gravity and the Micro-Deval test results may provide better assessment of aggregate durability performance.

The Micro-Deval test was originally developed in France in the 1960s and is now being used in the Province of Quebec and by the Ontario Ministry of Transportation as well as in the United States. AASHTO has currently only adopted a standard test method for coarse aggregates (AASHTO TP 58 – Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus). A suitable test method for evaluating fine aggregates is available in the Canadian Standards Association (CSA) specifications (CSA A23.2-23A – Resistance of Fine Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus). For Micro-Deval testing, a 1500 gm sample is soaked for a minimum of 1 hour in 2 liters of tap water and then the aggregate, water, and abrasive charge consisting of 5 kg of 9.5-mm-diameter steel balls are placed in a jar. The jar is then rotated at 100 rpm for 2 hours for coarse aggregates, 15 minutes for fine aggregates. After rotation, the sample is washed, oven dried, and then sieved over a 1.25 mm sieve for coarse aggregates and a 75 μm sieve for fine aggregates. The loss is expressed as a percent by mass of the original sample.

CSA specifications currently limit the percent loss to 20% and 14-17% for fine and coarse aggregates, respectively. A study conducted by Senior and Rogers (1991) reports that results from Micro-Deval testing are similar to those obtained using the Magnesium Sulfate test and have better within- and multi-laboratory precision and are less sensitive to aggregate grading. Kandhal and Parker, 1998, recommended a maximum loss of 18% for coarse aggregate. A reference material is available for periodic calibration of loss. WisDOT does not currently use the Micro-Deval test.

Woodward (1995) recommended specifications based on rock type. Ontario has implemented a specification for the Micro-Deval based on aggregate type (Rogers, et. Al., 2002). For high volume roads, maximum Micro-Deval percent loss values are as follows:

- Igneous gravel 5
- Dolomitic sandstone 15
- Traprock, diabase, and andesite 10
- Meta-arkose, meta-gabbro, and gneiss 15

Tarefder, et. Al. (2003), recommended a maximum loss of 25% for Oklahoma.

Rismantojo (2002) and Senior and Rogers (1991) showed good correlations between the Micro-Deval and Magnesium Sulfate Soundness. The two tests are used to assess performance of two completely different deterioration modes, abrasion and freezing and thawing, respectively. The good correlation indicates an interrelationship of factors affecting the tests; presumably absorption and low strength (in particular, moisture-related strength reduction). Similarly, Rismantojo(2000) also indicated a correlation between Micro-Deval and absorption. In contrast, Cooley (2002) reports that Micro-Deval testing has no relationship with data collected using sodium sulfate testing and LA abrasion testing, and suggests an indication that the test is measuring something else. Cooley did find differences in results based on rock type and suggests that Micro-Deval specifications may need to be “based upon the parent aggregate type” as Ontario has done.

If two tests correlate perfectly, it may be advisable that only one test be used. In the case of the Micro-Deval and sulfate soundness, the choice would be the Micro-Deval test because it has a greater precision. On the other hand, if the good correlation was obtained on a limited suite of aggregate materials, it may be advisable to perform both tests to avoid passing non-durable aggregate. An example would be an absorptive rock with a pore structure susceptible to frost damage that does not lose strength when saturated and thus performs well in the wet abrasion test. For this reason some continue to recommend performing soundness tests along with Micro-Deval tests.

2.1.6 Aggregate Crushing Value (British Standard 812-110 1990)

Aggregate strength needs to be assessed for aggregates that are to be used in high strength concrete, but not for normal strength due to the fact that the strength of concrete is generally controlled by the strength of the hydrated cement matrix rather than the aggregate in normal strength concretes. There are currently no AASHTO or ASTM testing methods to determine aggregate strength directly. However, the British Aggregate Crushing Value Test has been used extensively to determine the relative strength of graded concrete aggregates. This test consists of applying a load to an aggregate sample in a steel cylinder for 10 minutes. After the 10 minute loading period the sample is analyzed for changes in gradation and a value is determined.

Although the Aggregate Crushing Value Test may provide insight into the strength of the aggregate it may not always reflect the strength of the concrete in which the aggregate is placed. Therefore, it is recommended that concrete cylinders be cast and tested for strength to provide a more accurate estimate of the concrete strength.

Aggregate strength is equally important for unbound aggregate and asphalt concrete where the aggregate is subjected to high stresses at contact points. Should the Micro-Deval test replace the LA Abrasion test then the Aggregate Crushing Value may need to be used in concert with the Micro-Deval test since the LA Abrasion test includes impact as a surrogate for strength. No clear trends were noted between the results of the LA abrasion, Aggregate impact or Aggregate crushing and subjective field performance ratings (Prowell, 2004).

2.1.7 Resistance of Concrete to Rapid Freezing and Thawing (ASTM C 666, AASHTO T 161)

ASTM C 666 (or AASHTO T 161) is the most commonly used test for determining the freezing and thawing resistance of aggregates in concrete. One problem in implementing this test into a testing protocol is the amount of time required to complete the test. It may take over two months to complete one test depending on the length of the freeze-thaw cycle. It has also been suggested by Stark (1976) that the number of freezing and thawing cycles be reduced to two per day, which would extend the amount of testing time required to almost six months.

ASTM C 666 presents two procedures that may be used, as follows:

- Procedure A – Freezing and Thawing in Water
- Procedure B – Freezing and Thawing in Air

A third procedure was developed by Janssen and Snyder (1994) to better simulate field conditions and is referred to as Procedure C.

- Procedure C – Freezing and Thawing while wrapped in terry cloth

The procedures begin with the concrete beams being moist-cured for 14 days at 23^{o}C before they are submitted to freezing and thawing cycles. This aspect of the test was criticized in NCHRP Report No. 129 (1986) because the high level of saturation of the beams does not simulate actual field conditions. After the moist curing is complete the beams are cycled between –17.8 and 4.4^{o}C in a two to five hour time period. The rate of temperature change has also been questioned for being too rapid (harsh). ASTM C 666-97 states, "Neither procedure (A or B) is intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete."

It is argued that Procedure A is too severe because of the freezing and thawing in water and that it is done in rigid containers that may cause sample damage unrelated to the freezing and thawing process. Procedure B is criticized for allowing the sample to dry during the freezing cycle making the test not severe enough. Procedure C was developed in 1994 by Janssen and Snyder in order to create a more appropriate testing environment. By wrapping the beams in terry cloth the use of the rigid containers is eliminated while allowing for the beam to retain its moisture. However, Procedure C has not been adopted by either AASHTO or ASTM standards.

There are several ways to interpret test results. The following recommendations were presented by Folliard and Smith (2003):

- AASHTO T 161 (Procedure C) appears to be the most viable concrete test to assess D-cracking. The use of a cloth wrap avoids potential premature damage sometimes observed in rigid containers (Procedure A) and prevents specimen drying during freezing cycles (Procedure B).
- Length change or dilation measurements are recommended to assess D-cracking. Stark (1976) proposed a failure criterion of 0.035 percent expansion after 350 cycles, an approach currently adopted by CSA. This proposed method specifies only two freezing and thawing cycles per day, which may be more indicative of field performance.
- The calculation of a durability factor (DF) using dynamic modulus measurements may be used to assess AASHTO T 161 results, but the data may not be indicative of field performance as length change measurements.
- Pre-soaking carbonate aggregates in chloride solution before casting and testing concrete in AASHTO T 161 is a viable method of assessing salt susceptibility of aggregates in a freezing and thawing environment.

- Field performance records are essential in relating laboratory freezing and thawing tests to D-cracking in PCC pavements. State DOTs and other relevant agencies are urged to correlate laboratory data to the performance of their local aggregates in PCC pavements; this comparison can be used to select the version of AASHTO T 161 and method of data interpretation that relates best to field performance.

Agencies have modified these standard procedures in various ways to suit their specific purposes. For instance the Virginia Department of Transportation (VDOT) which does not have a significant soundness problem with aggregates, uses Procedure A with a 2% NaCl solution (Newlon,1978). In Kansas where D-Cracking aggregates present a major problem, Procedure B is used with the curing time extended to 90 days(Clowers,1999)

2.1.8 Frost Resistance of Coarse Aggregate in Concrete (ASTM 682/ASTM 671)

Powers in 1955 published a critical review of existing freezing and thawing tests. He cited unrealistic rates as compared to natural cooling rates of 3^{o}C/hr (5^{o}F/hr) and the saturated moisture state of aggregates and concretes during testing. The California Division of Highways (Tremper and Spellman, 1961) reported a practical application of Power's proposed method. The method was used to evaluate several aggregates for major highway projects. The freezing and thawing performance of the concrete has been satisfactory after 30 years of exposure. Larson and Cady (1969) refined and standardized the procedures ASTM C 671 and C 682 in 1971.

The method uses 75mm (3 in) diameter and 150mm (6 in) long specimens. Specimens are cooled from 1.7^{O}C (35^{o}F) to –17.8^{o}C (0^{o}F) at 3^{o}C/hr (5^{o}F/hr) in water-saturated kerosene every two weeks. In between cycles, the specimens are stored in water at 1.7^{o}C (35^{o}F). Frost resistance criterion is a sharp increase in dilation of a factor of two or more between cycles. The Frost resistance period is considered to be 12 cycles (24 weeks or one winter period). However, aggregate moisture states other than saturated or dry are difficult to achieve and maintain. Variability of results will be affected by the moisture state and the moisture state would have to be representative of field conditions for reliable performance predictions. Thus, best and conservative conditions would be to vacuum saturate the aggregate prior to specimen casting.

Equipment is relatively extensive, freezing chamber, strain frames, length and temperature measuring apparatus, and conditioning cabinet. However, a large number of specimens can be tested in the 24-week test period as 20 sets of specimens can be tested within the two-week cycle period. Although, the test method has been reported as being theoretically sound, precise, and a better performance predictor, a 1987 survey reported that only one highway agency was using the test method (Vogler and Grove, 1989). Additionally, these testing methods were withdrawn by ACI Committee C09 in 2003.

2.1.9 Soundness of Aggregates by Freezing and Thawing (AASHTO T 103)

AASHTO T 103 presents three procedures for testing the resistance of unconfined fine and coarse aggregates to disintegration by freezing and thawing. The procedures are as follows:

- Procedure A – Total Immersion – Aggregate samples are soaked in water for 24 hours before the test begins. The sample is then cycled through freezing and thawing conditions, while remaining in the completely immersed condition.
- Procedure B – Partial Immersion – Aggregate samples are saturated in an alcohol-water solution (0.5 percent by mass ethyl-alcohol) in a vacuum with an air pressure not to exceed 3.4 kPa. The saturation process will be for a period of 15 minutes. After saturation the sample is frozen in the surface-wet condition and thawed for 30 minutes in the alcohol-water solution. The freeze time will generally be 2 hours, but will depend upon the sample size.
- Procedure C – Partial Immersion – This procedure is the same as Procedure B except water is used in place of the alcohol-water mixture.

The generally accepted number of freezing and thawing cycles are 50, 16, and 25 for Procedures A, B, and C, respectively. Results are reported as the weight loss of each size fraction and the weighted average loss.

The performance predictability and reproducibility for the test are unknown due to the fact that it is not widely used. The New York Department of Transportation (NY DOT) uses a modified version (NY 703-08), in which 3% NaCl solution is used rather than water. A Canadian test (CSA A23.2-24A) is similar to AASHTO T 103 and was recommended by Folliard and Smith (2003) over AASHTO T 103. A study conducted by Brown (1999) concerning aggregates from the Sinnipee Group in Wisconsin suggests that AASHTO T 103 (Procedure B) “may be a more sensitive and therefore a more reliable test” than the Sodium Sulfate Soundness test.

This test will be described later. WisDOT is currently using AASHTO T 103 as an indicator for the freezing and thawing durability of aggregates. A distribution analysis of Wisconsin test results is presented in Appendix A, see Figure A-2.

2.1.10 Washington Hydraulic Fracture Test

The intent of this test is to simulate freezing and thawing action on aggregates by creating internal forces in the aggregate using hydraulic pressures. The aggregates are placed in a chamber and submerged; the pressure is then increased causing water to penetrate the aggregate pore structure. The pressure is released and the compressed air within the pores forces the water back out of the aggregate. This action simulates the pressures that occur during a freezing and thawing cycle. The release of these pressures causes the fracturing of the aggregate, which is then quantified through sieving and used to predict the potential soundness of the aggregate.

The test was found to be highly sensitive to the pressure release rate. (Alford and Janssen, 1995) A study completed by Issa et al. in 2000 concludes that there was a "lack of a direct correlation" between WHFT tests and ASTM C 666 tests. The study also reports that the two testing devices used, WHFT 94 and WHFT 97; only correctly identify aggregate sources 67% and 76% of the time, respectively. The coefficients of variation (CV) for the two tests were found to be 21-24% for WHFT 94 and 16% for WHFT 97. There have been several modifications made to the procedure by Embacher and Snyder (2003) in a recent research project. Among these modifications were placing a neoprene lining in the crushing chamber, and using a larger testing chamber. These modifications show better correlations to ASTM C 666 test results, which suggest that the test may possibly be used successfully in the future; however, they note the failure of the test to fracture susceptible chert aggregate. The same report also recommends that further testing be done to verify the results obtained using the modified testing procedures. Due to the lack of precision and relevant results of this test it does not appear that it is a viable option for replacing current testing procedures at this time.

2.1.11 Test Method for the Resistance of Unconfined Coarse Aggregate to Freezing and Thawing (CSA A23.2-24A)

In this test, aggregate samples are soaked in a 3 percent sodium chloride solution for 24 hours prior to testing. The sample is then drained and put through 5 freezing and thawing cycles, while saturated with the sodium chloride solution as compared to 25 cycles for AASHTO T 103 or NY 703-08. The use of the sodium chloride solution is specified in order to provide a better simulation of the field environment by representing the effects that de-icing salts have on the aggregates. The Canadian test is recommended over AASHTO T 103 and the Sulfate Soundness Test by Senior and Rogers (1991-2). The test has better precision and better correlation with field performance, which are among the reasons cited for its recommendation. However, calcium and magnesium chloride de-icing salts may be significantly more aggressive than sodium chloride on specific rock types, for example magnesium chloride on limestone.

2.1.12 Petrographic Examination of Aggregates for Concrete (ASTM C 295)

Petrographic Examination of aggregates may be useful in determining vital aggregate characteristics relating to durability, especially those associated with Alkali-Aggregate Reactivity (AAR). When using this procedure in the context of its importance to aggregate durability it can provide information concerning the physical and chemical characteristics of the aggregate as well as the relative amounts of the aggregate's constituents.

Petrographic examination is only discussed with respect to its use in analyzing aggregates to be used in Portland cement concrete. Aggregates in HMA or unbound pavement layers will not be addressed because Kandhal and Parker (1998) could not identify any "strong relationships between mineralogy or petrology and the general performance of

HMA pavements", and Saeed et al (2001) only rated the performance predictability as fair for aggregates used in unbound pavement layers.

SHRP Report # C-342 (1993) suggests that when used in conjunction with ASTM C 227 (Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method) and ASTM C 289 (Test Method for Potential Alkali-Sillica Reactivity of Aggregate-Chemical Method), petrographic examination may be able to "determine the potential for expansive ASR in highway structures". SHRP Report # C-343 concluded later that ASTM Tests C 227 and C 289 "fail particularly to identify slowly reacting aggregates that cause abnormal expansion in highway structures". Another recent study completed in 2001 by Touma et al reports that the "petrographic examination results available failed to detect the reactive materials present in the slowly reactive aggregates investigated". This same report proposes that ASTM C 1293 (Test Method for Concrete Aggregate by Determination of Length Change of Concrete Due to Alkali-Silica Reaction) may be used in conjunction with ASTM C 1260 (Test Method for Potential Alkali Reactivity of Aggregate-Mortar-Bar Method) to properly identify potentially reactive aggregates. These tests (ASTM C 1260 and ASTM C 1293) are discussed below. It should also be noted that petrography results are dependent solely upon the interpretations of individual petrographers and thus can produce varying results.

2.1.13 Potential Alkali Reactivity of Aggregates, Mortar Bar Method (ASTM C 1260)

ASTM C 1260 can be used as a rapid indicator of alkali aggregate reactivity, taking only 16 days in comparison with the 12 months required for ASTM C 1293. The test consists of casting specimens and then placing them in a moist room for 24 hours. Following the initial curing, the specimens are removed from the molds and comparatory readings are taken. The specimens are then placed in tap water and stored at 80^{o}C for 24 hours. After the 24-hour period zero readings are taken, the specimens are placed in a 1N NaOH solution at 80^{o}C. Readings are taken at 14 days and for a minimum of 3 additional intermediate readings. Expansions of less than 0.10% indicate innocuous aggregates and expansions greater than 0.20% indicate potentially deleterious aggregates. Mortar bars that have expansions between 0.10% and 0.20% are known to have varying levels of reactivity and should be subjected to additional testing.

A report by Touma et al (2001) recommends that the test be used as a preliminary indicator of aggregate reactivity. The report suggests that aggregates found to be innocuous need not be tested further and can be accepted, but aggregates with expansions greater than 0.10% should be further tested using ASTM C 1293 before rejection. The AASHTO counterpart to ASTM C 1260, AASHTO T 303, was also recommended by NCHRP Project 4-30 (2002) for use as an indicator for aggregate reactivity. However, it has been noted that some deleteriously reactive coarse aggregate do not expand excessively in this test (SHRP C-343 and Lane, 1993). Conversely, with fine aggregate the method seems to over predict their potential for causing damage to field concrete (Lane, 2000), thus limiting its usefulness as a screening tool.

2.1.14 Determination of Length Change of Concrete Due to Alkali-Silica Reaction (ASTM 1293)

The purpose of ASTM C 1293 is to "evaluate the potential of an aggregate to expand deleteriously due to any form of alkali-silica reactivity". The procedure requires that concrete prisms be cast with a total alkali content of 1.25% by mass of cement. The alkali content is obtained by adding NaOH to the concrete mixing water. The concrete prisms are then stored over water in a 38°C environment for one year. An expansion limit of 0.04% is used as the acceptance criterion. The Canadian Standard Association has recently modified the interpretation limits of ASTM 1293 as follows (CSA, 2000a):

Expansions
- < 0.04%, non-reactive
- 0.04 to 0.12%, marginally-reactive
- > 0.12%, highly reactive

Touma et al (2001) report that if the storage temperature is increased to 60°C the required testing time can be reduced to 3 months. The same expansion limit would apply to the modified test. Folliard et al (2004) are currently conducting the modified and standard versions of ASTM 1293, storage above water at 60 and 38°C, respectively. The findings to date show that the long-term (up to 6 months) expansions at 60°C are significantly less than the 38°C expansions at one year. The reduction in expansion was attributed to specimen drying and greater leaching of alkali at 60°C, in addition to the associated changes in the concrete pore water solution composition.

ASTM C 1293 is generally accepted as the most reliable test method for identifying reactive aggregates. Presently, if time permits the standard form of ASTM C 1293 should be used instead of other available test methods. As previously stated, Touma et al (2001) recommended that ASTM C 1293 be used in conjunction with ASTM C 1260 to evaluate aggregates.

The test method may also be used to evaluate the potential for Alkali-Carbonate Reactivity (ACR). It is recommended that the aggregates be tested first using CSA A23.2-26A (Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition) and if they are found to be potentially reactive they should be tested using ASTM C 1293 (Folliard and Parker 2002).

The best approach appears to be the use of ASTM C 1293 in conjunction with field performance since both ACR and ASR rock will expand in the test. Subsequent examination of the rock would determine if the problem is ACR or ASR.

2.1.15 Thermogravimetric Analysis

Thermogravimetric analysis is a recent test that has been developed in order to create a rapid testing method for tracking carbonate aggregate durability. Its use is based on the assumption that the mineralogic characteristics of durable and non-durable lithologies will serve as markers within the range of deposition. Research completed in Kansas and Iowa have shown promising results. Thermogravimetric analysis consists of heating carbonate aggregates to a temperature of 1200^{o}C and evaluating the weight loss of the sample. Weight loss is achieved through calcite and dolomite transition. Carbon dioxide is used as a purge gas in the oven because it showed good results with carbonates. Calcite transition generally occurs between 905^{o}C and 940^{o}C, and dolomite transition takes place between 705^{o}C and 745^{o}C.

A research project completed by Cross and Abou-Zeid (1996) was able to use thermogravimetric data to obtain the percent acid insoluble minerals (AI) of aggregates. The AI value was used in conjunction with the absorption of the aggregate to calculate a Pavement Vulnerability Factor (PVF). The PVF is currently used by the Kansas DOT as an indicator for aggregate durability. Prior to the use of the PVF, 48% of Kansas Concrete Highways experienced D-cracking and out of the roadways built since, less than 1% show signs of D-cracking (Clowers, 1999). Two research projects, one conducted by Dubberke (1994) and the other by Cross and Abou-Zeid (1996) suggest that the slopes of the weight loss plots of the aggregates prior to dolomite and calcite transition correlate well with field performance. If the slope is relatively flat it suggests durable aggregates and if the slope is steeper the opposite is expected. Dubberke (1994) states that the "sample size, rate of heating, and test method" have an influence on the weight loss of the sample during testing, and concludes that a "standardized test method is necessary to obtain repeatable results". It was also concluded that the percent of acid insoluble minerals may not be a good indicator of performance for dolomites and that analysis of coarse-grained dolomites may produce erroneous results due to micro explosions during the heating process that cause sample loss. Use of thermogravimetric analysis as an indicator for the durability of aggregates is not recommended at this time. If a significant problem is found to exist with durability of carbonate rocks, TGA may warrant examination to determine if it can be used as a more efficient monitoring tool to reduce the frequency of soundness testing.

2.1.16 **Lightweight Pieces in Aggregate (ASTM C 123-98)**

ASTM C 123 (AASHTO T 113) is used to determine the amount of deleterious materials present in a given aggregate sample. The test may be used for both fine and coarse aggregates. Test results are based on the principle that deleterious materials such as clay, coal, and low density chert have lower specific gravities than durable aggregate. The percentage of these lightweight materials is determined by placing an aggregate sample in a heavy liquid and skimming off the materials that float to the surface. These materials are then washed, dried, and weighed to determine the overall percentage of lightweight materials in the aggregate sample.

AASHTO T 113 is currently being used by WisDOT, and should continue to be used to ensure that excessive amounts of deleterious materials are not present in aggregate sources that are to be used in WisDOT projects. A distribution analysis of Wisconsin test results is presented in Appendix A (See Figure A-1)

2.2 Recycled/Reclaimed Aggregate Material

2.2.1 Foundry Slag

Foundry Slag for use as aggregate is said to be "at least as good as natural aggregates." (OECD, 1997) Testing of the physical properties of slag in the unbound form can be carried out just as testing would be for virgin aggregates, however the chemical properties of the slag must be taken into consideration. With respect to the chemical properties of slag, two types of unsoundness may occur, these are Iron Unsoundness and Calcium Disilicate Unsoundness.

"Iron Unsoundness is very rare, it arises when partially reduced iron oxides in the slag oxidize. The expansive reaction causes the slag to disintegrate. It is detected by immersing 12 pieces of slag in water for a period of 14 days and observing whether any of the particles crack or disintegrate." (OECD, 1997)

Calcium Disilicate Unsoundness is caused by an increase in volume due to a phase change. The following two conditions must be met:

$CaO + 0.8MgO < 1.2SiO_2 + 0.4Al_2O_3 + 1.75S$ (% by mass)
$CaO < 0.9SiO_2 + 0.6Al_2O_3 + 1.75S$ (% by mass)

If the slag fails to meet the previous requirements it may not necessarily be considered unsound, but is subject to a microscopic examination. (OECD, 1997)

The sulphur and sulphate content of the slag is also an important factor. In order for slag to be used as a concrete aggregate, it must have a sulphur content of less than 2 percent and a sulphate content of less than 0.7 percent. If used in the unbound state the slag may not have a soluble sulphate content of more than 2g/litre. (OECD, 1997)

Foundry Slag is currently being used by WisDOT as a concrete aggregate and as a pavement base material. It is not recommended that WisDOT test slag specimens for every project, but rather test slag sources on a periodic basis (every 5-years) for iron unsoundness and chemical composition. This would allow for slag to be accepted on a past performance basis and also set up a list of approved slag sources. WisDOT should also require specimen testing if there are any significant changes in the slag production process or source. By doing so, WisDOT would alleviate testing requirements while also ensuring that high quality slag is provided for use in WisDOT projects.

Steel slag is also another source for slag aggregate. ASTM D 4798 may be used to determine the expansion potential of steel slag aggregates. (ASTM, 1991)

2.2.2 Recycled Asphalt Pavement

Recycled Asphalt Pavement (RAP) is used as a base material or in new asphalt mixtures. When using RAP as a base material proper gradation must be checked as it can be altered during the recycling process. WisDOT is currently using RAP as a base material and the testing procedures appear to be in general agreement with accepted procedures, although the original testing protocol for the asphalt aggregates will need to be updated.

WisDOT is not incorporating the use of RAP in the Superpave mix design method at this time. If in the future WisDOT decides to use RAP in Superpave, testing should be done in accordance with NCHRP Report #452 (Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual). This report was completed in 2001 and provides the most up to date information concerning the use of RAP in Superpave mixtures. After reviewing NCHRP Report #452, there are no additional durability tests that need to be conducted in addition to the standard testing of aggregate used in the original asphalt pavement layers. This is due to the fact that the virgin aggregates contained in the original asphalt mixture have already met aggregate performance durability requirements. Of course any pavements that exhibit poor performance should not be reused as RAP. The additional testing required for the implementation of RAP into Superpave mixtures relates to the moisture content, gradation, and binder content of the RAP.

2.2.3 Recycled Concrete Pavement

Recycled Concrete Pavement (RCP) is generally used as a base material in pavements, but there are also recent research projects that address the use of RCP in concrete. This research project will only address the use of Recycled Concrete Pavements. If concrete from demolished buildings is to be used there are other considerations that must be accounted for such as the presence of gypsum and brick in the aggregate. WisDOT currently uses RCP as a base material and specifies that no durability testing is necessary if the RCP is taken from within the project limits. A limit of 50% loss from the L.A. Abrasion test is otherwise imposed. Kuo et al (2002) suggest that a 48% limit of abrasion loss be used for RCP. It has been determined by many investigations that the sulfate soundness test is "inappropriate to apply to RCA because of the nature of the chemical attacks that take place on concrete materials" (Kuo et al, 2002). It is recommended that the sulfate test be waived for Recycled Concrete Aggregates (RCA).

When RCA is considered for use in new concrete other characteristics must be addressed. Water absorption is much higher for RCA because of the adhered mortar. When designing the concrete mix this additional water absorption must be accounted for. Also, "the dry and saturated-surface-dry bulk specific gravities in conventional concrete aggregate are about 10% higher than those of recycled aggregate concrete". (Barra and Vazquez, 1998) Because of these differences between recycled aggregates and virgin aggregates it does not appear that specific gravity and absorption will be able to be used as durability indicators for RCA.

Desai (1998) found that RCA concretes that contain "less than 30% coarse RCA or 20% fine RCA" had no "significant reduction in compressive strength compared with the equivalent natural aggregate concrete". The report also states that the "strength and durability of RCA can be enhanced with adjustments to the water-cement ratio and with use of fillers." Another report by the OECD (1997) suggests that up to 100% of coarse aggregate can be substituted with RCA if a satisfactory mixture can be obtained, but substitution of fine aggregate should be limited to 20% due to its high absorption characteristics. Other reports recommend that RCA should be limited to 25% to prevent loss of strength and durability. After an extensive literature review, an accepted set of durability testing specifications was unable to be determined. Very little information concerning freezing and thawing durability testing was found. L.A. Abrasion testing was found to be adequate in testing for wear, and the use of the Micro-Deval test should also be acceptable. It can be concluded that sulfate soundness, specific gravity, and absorption tests should not be used as RCA durability indicators, but the other recommended durability testing procedures can be used without adjustment.

Won (2004) reported on the use of 100% RCA in continuous reinforced concrete pavement (CRCP). The study consisted of a laboratory evaluation of CRCP sections in the Texas Houston District and development of guidelines for the use of RCA in CRCP. The RCA test findings were consistent with those reported elsewhere, lower specific gravity, higher water absorption, higher sulfate soundness loss and higher LA abrasion loss. The validity of the sulfate soundness and LA abrasion tests for evaluating the performance was questioned.

Comparing virgin aggregate with 100% RCA concrete properties, it was found that there was no effect on compressive strength, reduction in flexural strength and modulus of elasticity and significant increase in the thermal coefficient. Moisture control of the RCA was critical in producing consistent and workable concrete. Guidelines for the effective use of RCA for CRCP are being developed.

The University of New Hampshire Recycled Materials Resource Center (RMRC) assessed the use of RCA for unbound soil aggregate base course (RMRC and Chesner Engineering 2001-2). The final project development was AASHTO 319-02, Standard Specification for Reclaimed Concrete Aggregate for Unbound Soil-Aggregate Base Course. Durability performance parameters are limited to LA abrasion test with a maximum loss of 50% and the aggregate soundness testing is left to the discretion of the engineer. If the soundness loss is too high, alternative freezing and thawing tests may be used. AASHTO T 103 sets a maximum of 20% loss and the New York State DOT (Test Method NY 7003-08) sets a maximum loss limit of 20% when tested in a sodium chloride brine solution.

The RMRC also assessed the use of RCA as a coarse aggregate in Portland cement concrete (RMRC and Chesner Engineering, 2001-2). The standard specification durability limits are placed on chert (specific gravity less than 2.40), LA abrasion or Micro-Deval (50 and 13% loss, respectively), and sodium sulfate soundness (12% loss). Freezing and thawing tests listed are AASHTO T103, NY 7003-08, and the Ontario Ministry of Transportation Test Method LS 614 (Freezing and Thawing of Coarse Aggregate).

Although the soundness test is generally considered not applicable to RCA, it may be of some value when RCA is used as a base course in areas of high soil sulfate content.

As with RAP, RCP should not be used if the pavement has exhibited freezing and thawing damage, regardless of whether it is related to aggregate or cement paste performance.

The specification also addresses AAR as the RCA source may have been constructed using a reactivity abatement method, low alkali cement, flyash, or ground granulated blast furnace slag. Also, the RCA source aggregate may be a slow reactivity aggregate, which has not demonstrated significant deterioration to date. The processing of RCA may open new reactive aggregate sites. The specification lists a number of AAR tests, which have been discussed in the AAR test section of this report.

FHWA Pavement Recycling Team recently completed a review of RCA state-of-the-practice. The goal was to transfer present knowledge to other State highway agencies. The final report is in preparation. The material specifications are generally to treat RCA as virgin course aggregate (Harrington, J., 2004).

2.3 Current WisDOT Durability Testing Procedures and Specifications

2.3.1 Aggregate Testing Procedures

The following tests are currently used by WisDOT as aggregate durability indicators:

- Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (AASHTO T 96)
- Soundness of Aggregate by Use of Sodium Sulfate (AASHTO T 104)
- Soundness of Aggregates by Freezing and Thawing (AASHTO T 103)
- Lightweight Pieces in Aggregate (AASHTO T 113 and CMM 13.22)

Freeze-Thaw Soundness Testing (WisDOT Standard Specifications)

The department will conduct freezing and thawing soundness testing (AASHTO T 104), on the fraction retained on the No. 4 (4.75 mm) sieve, for virgin crushed stone aggregates produced from limestone/dolomite sources in one or more of the following counties or from out of state:

Brown	Columbia	Crawford	Dane	Dodge	Fond du Lac	Grant
Green	Green	Lake	Iowa	Jefferson	Lafayette	Marinette
Oconto	Outagamie	Rock	Shawano	Walworth	Winnebago	

The department may waive freezing and thawing testing for existing quarries determined to be in either the Silurian system or the Prairie du Chien group of the Ordovician system of rocks.

2.3.2 Aggregate Specifications

The following specifications are currently used by WisDOT as aggregate quality control measures:

301.2.3.1 General (WisDOT Standard Specifications)

(1) Furnish aggregates that are substantially free of deleterious materials.

(2) The department may prohibit the use of crushed stone from limestone/dolomite deposits that have thinly bedded strata or strata of a shale nature. The department may also prohibit the use of aggregate from deposits or sources known to produce unacceptable material.

301.2.3.4 By-Product Materials (WisDOT Standard Specifications)

(1) The contractor may provide an aggregate with one of the following by-product materials mixed with crushed gravel, crushed concrete, or crushed stone up to the listed maximum percentage, by weight.

BY-PRODUCT MATERIAL	MAXIMUM PERCENTAGE (by weight)
Glass	12
Foundry slag	7
Steel mill slag	15
Bottom ash	8
Pottery cull	7

(2) Furnish by-product materials substantially free of deleterious substances.

(3) Crush, screen, and combine materials to create a uniform mixture conforming to the predominant material specifications.

(4) If the aggregate contains a by-product material, the department will test the final product for gradation, wear, soundness, liquid limit, plasticity, and fracture as required for the predominant material.

(5) Do not use aggregate containing a by-product material in the top 3 inches of a temporary or permanent aggregate wearing surface.

Table 2.3.1 below presents the current WisDOT base aggregate requirements, although not all of the tests shown are considered to measure aggregate durability.

Property	Crushed Stone and Crushed Gravel	Crushed Concrete	Reclaimed Asphaltic Pavement	Reprocessed Material	Blended Material	
Wear **AASHTO T 96** **loss by weight**	<= 50%	note [1]	-----	note [1]	note [2]	
Sodium Sulfate Soundness **AASHTO T 104** **loss by weight**						
dense	<= 18%	------	------	------	note [2]	
open	<= 12%	------	------	------	note [2]	
Freeze/thaw soundness **AASHTO T 103** **loss by weight**						
dense	<= 18%	------	------	------	note [2]	
open	<= 18%	------	------	------	note [2]	
[1] No requirement for material taken from within the project limits. Maximum of 50 percent loss,						
by weight for materials supplied from a source outside the project limits.						
[2] Required as specified for the individual componenet materials defined in columns 2-5 of the table						
before blending.						

Table 2.3.1 Current WisDOT Base Aggregate Requirements

SIEVE SIZE	PERCENT PASSING (by weight)
1 inch (25.0 mm)	90 - 100
3/8 inch (9.5 mm)	45 - 65
No. 4 (4.75 mm)	15 - 45
No. 10 (2.00 mm)	0 - 20
No. 40 (425 µm)	0 - 10
No. 200 (75 µm)	0 - 5.0

Table 2.3.2 Open Gradation Requirements

SIEVE SIZE	PERCENT PASSING BY WEIGHT 3-INCH (75 mm)	1 1/4-INCH (31.5 mm)	3/4-INCH (19.0 mm)
3 inch (75 mm)	90 - 100	____	____
1 1/2 inch (37.5 mm)	60 - 85	____	____
1 1/4 inch (31.5 mm)	____	95 - 100	____
1 inch (25.0 mm)	____	____	100
3/4 inch (19.0 mm)	40 - 65	70 - 93	95 - 100
3/8 inch (9.5 mm)	____	42 - 80	50 - 90
No. 4 (4.75 mm)	15 - 40	25 - 63	35 - 70
No. 10 (2.00 mm)	10 - 30	16 - 48	15 - 55
No. 40 (425 µm)	5 - 20	8 - 28	10 - 35
No. 200 (75 µm)	2.0 - 12.0	2.0 - 12.0[1][3]	5.0 - 15.0[2]

[1] Limited to a maximum of 8.0 percent for base placed between old and new pavement.

[2] 8.0 - 15.0 percent if base is ≥ 50 percent crushed gravel.

[3] 4.0 - 10.0 percent if base is ≥ 50 percent crushed gravel.

Table 2.3.3 Dense Gradation Requirements

2.3.3 Additional Specifications for Structural Concrete

501.2.5.3 Fine Aggregates (WisDOT Standard Spec.)

(1) Fine aggregate consists of a combination of sand with fine gravel, crushed gravel, or crushed stone consisting of hard, strong, durable particles.

501.2.5.3.1 Deleterious Substances

(1) Do not exceed the following percentages:

SUBSTANCE	PERCENT BY WEIGHT
Material passing the No. 200 (75 µm) sieve	3.5*[1]*
Coal	1.0
Clay lumps	1.0
Shale	1.0
Other deleterious substances like alkali, mica, coated grains, soft and flaky particles	1.0

[1] Reduce to 2.3 percent if used in grade E concrete.
Grade E concrete is used for overlays and repairs on decks. Mixture proportions are located in Section 501.3.2.2 of the WisDOT standard specifications.

501.2.5.4 Coarse Aggregates (WisDOT Standard Spec.)

501.2.5.4.1 General

(1) Use clean, hard, durable gravel, crushed gravel, crushed stone or crushed concrete free of an excess of thin or elongated pieces, frozen lumps, vegetation, deleterious

substances or adherent coatings considered injurious. Do not use coarse aggregates obtained from crushing concrete in concrete for bridges, culverts, or retaining walls.

501.2.5.4.2 Deleterious Substances

(1) The amount of deleterious substances shall not exceed the following percentages:

DELETERIOUS SUBSTANCE	PERCENT BY WEIGHT
Shale	1.0
Coal	1.0
Clay lumps	0.3
Soft fragments	5.0
Any combination of above	5.0
Thin or elongated pieces based on a 3:1 ratio	15.0
Materials passing the No. 200 (75 µm) sieve	1.5
Chert*[1]* for all grades of concrete other than for prestressed members	5.0*[2]*
Chert*[1]* for concrete for prestressed concrete members	2.0

[1] Material classified lithologically as chert and having a bulk specific gravity (saturated surface-dry basis) of less than 2.45. Determine the percentage of chert by dividing the weight of chert in the sample retained on a 3/8-inch (9.5 mm) sieve by the weight of the total sample.
[2] The engineer may accept aggregates exceeding this value if aggregates from the same deposit or from one of similar geological origin demonstrated a satisfactory service record, or tests the engineer select indicate no inferior behavior.

(2) If using 2 sizes of coarse aggregates, the engineer will determine the percentages of harmful substances based on one of the following: a sample consisting of 50 percent of size No. 1, and 50 percent of size No. 2; or a sample consisting of the actual percent of size No. 1 and No. 2 used in the work.

(3) The engineer will not require the contractor to wash coarse aggregate produced within specified gradations, free of coatings considered injurious, and conforming to the above limits for harmful substances.

501.2.5.4.3 Physical Properties

The department will conduct 5 cycles of the sodium sulfate soundness test, according to AASHTO T 104, on aggregate retained on the No. 4 (4.75 mm) sieve. The weighted loss shall not exceed 12 percent.

CHAPTER 3: AGGREGATE PERFORMANCE ASSESSMENT PROGRAM

Aggregate performance durability issues may be categorized as physical or chemical. Physical degradation mechanisms include:

- Attrition during handling and construction
- Degradation under in-service loads
- Environmental degradation from freezing and thawing, wetting and drying, and/or thermal expansion and contraction.

Chemical degradation mechanisms include but are not assessed in this study:

- Hydration of anhydrous oxides of CaO and MgO and oxidation of ferrous sulfides
- Alkali-silica reaction
- Alkali-carbonate reaction

Aggregate performance properties have a direct influence on the stability of aggregate particles in the unbound and bound state. For example, an aggregate with a high porosity and low permeability defines the aggregate freezing and thawing critical size whether in the unbound state or the bound state as Portland cement concrete or bituminous concrete. Whereas, an aggregate with a high porosity and high permeability may not fracture as unbound material but degrades the binding forces in bituminous and Portland cement concrete.

Thus, aggregate performance properties not only influence aggregate durability, but also the durability of their inclusion material. It is the authors opinion that "durability of aggregate" addresses the durability of the aggregate particles and its influence on the durability of the material of which they are a component, in this case bituminous or Portland cement concrete. Thus, the proposed aggregate testing program is presented for unbound, bituminous concrete and Portland cement concrete pavement and structural elements and their associated durability performance parameters.

3.1 Aggregate Physical Properties

Aggregate physical durability properties are typically interrelated. For example, both the freezing and thawing degradation mechanisms, aggregate fracture and degradation of binder-aggregate forces occur when the aggregate has a high porosity. Porosity, absorption, and specific gravity are interrelated. Aggregate that has a high specific gravity generally has a low absorption. These aggregates would generally have a high strength, high abrasion resistance, and a high resistance to dimensional changes. Relative to physical degrading forces, some aggregates may be accepted based on specific combinations of specific gravity and absorption. Where as, other aggregate physical performance characteristics will have to be determined by abrasion, strength, and freezing and thawing testing. Thus, the objective of the proposed testing program is to develop a tiered aggregate assessment protocol.

3.2 Aggregate Chemical Properties

Whereas, the aggregate physical durability properties are generally related, chemical degradation mechanisms are generally mineralogic composition related. Thus, the tiered aggregate assessment protocol shall include assessing the chemical degradation mechanisms based on the proposed testing program.

CHAPTER 4: AGGREGATE TEST SELECTION

Table 4.1 presents the laboratory test matrix. Selection of the tests was based upon the tests' precision, efficiency, and predictive capabilities. Wisconsin's current aggregate tests were also conducted for comparative purposes. Some of the tests that were presented in the literature review were not selected due to their lack of proven successful use and/or standard specifications. Tests that may be considered for future adoption by WisDOT once the present shortcomings of testing and their predictive performance capabilities are more mature are the Tube Suction Test, Modified Washington Hydraulic Fracture Test, and Thermogravimetric Analysis.

Aggregate Material	Crushed Stone and Gravel	Recycled Concrete Aggregate	Blast Furnace Slag
Lightweight Pieces in Aggregate (ASTM C 123-98)	X		
Vacuum Saturated Specific Gravity and Absorption (Coarse) (ASTM C 127)	X	X	X
Sodium Sulfate Soundness (ASTM C 88)	X	X	X
Frost Resistance of Aggregates in Concrete (ASTM C 666)	X	X	X
Unconfined Freezing and Thawing (CSA A23.2-24A)	X	X	X
Micro-Deval Abrasion (Coarse) (AASHTO TP 58)	X	X	X
L.A. Abrasion (ASTM C 131-01)	X	X	X
Aggregate Crushing Value (British Standard 813 – Part 3)	X	X	X
Compressive Strength of Cylindrical Concrete Specimens (ASTM C 39)	X	X	X

Table 4.1 – Laboratory Test Matrix

4.1 Aggregate Testing for Aggregates to be used in Unbound Pavement Layers, Hot Mix Asphalt and Portland Cement Concrete

Crushed stone and gravel aggregates were all subjected to the same testing protocol. Although, the requirement for testing aggregates for Alkali Aggregate Reactivity may be waived if the aggregate source will not be used for Portland Cement Concrete production. The use of the same testing procedures for all sources will increase efficiency and will also encompass the full spectrum of durability testing needs. Testing for Lightweight Pieces in Aggregate is an important screening test used to determine the percentage of non-durable aggregates for crushed stone and gravel, as excessive amounts of low density chert, a lightweight aggregate, will result in a reduction in durability. The test for specific gravity and absorption, although being used for mixture proportioning for HMA and PCC pavements, can also be used as an indicator for aggregate soundness. Other soundness tests that were investigated were the Sodium Sulfate Soundness (ASTM C88), Unconfined Freezing and Thawing of Aggregates (CSA A23.2-24A), and Frost Resistance of Aggregates in Concrete (ASTM C 666) tests.

Testing of the resistance of aggregates to abrasion was conducted using both the L.A. Abrasion test and the Micro-Deval test. The Micro-Deval test was selected because it can be applied to both fine and coarse aggregates, and recent reports have shown that test results correlate better with field performance records. Also, the L.A. Abrasion and Impact Test included the effects of impact on the aggregate sample, thus both the effects of abrasion and impact are present in the test results. As a result, brittle aggregates may have a higher L.A. Abrasion loss due to the impact forces in the test. There is no clear understanding on how to interpret the results so that mass losses of the aggregate sample can be attributed to the abrasion resistance or the impact resistance of the aggregate. For the Micro-Deval test, the aggregate strength must also be considered. The British Aggregate Crushing test has been chosen for testing unbound and asphalt concrete and ASTM C 39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) was selected for hydraulic cement concretes.

ASTM C 1260 and ASTM C 1293 may be used in conjunction with one another to determine the potential for aggregates to react with alkalis found in cements. ASTM C 1260 was selected because of its' ability to provide relatively rapid information that can be used to accept an aggregate. ASTM C 1293 may be used if an aggregate fails to meet ASTM C 1260 specifications or if time permits. Results obtained from ASTM C 1293 will take precedence over any other AAR (alkali silica and carbonate reactions) test results.

4.2 Testing for Recycled Asphalt Pavement

Durability testing for Recycled Asphalt Pavements (RAP) is recommended because the recycled aggregates may not meet durability requirements for inclusion in asphalt. RAP samples will be tested using AASHTO T 283 (Resistance of Compacted Asphalt to Moisture Induced Damage). By conducting AASHTO T 283, the Tensile Strength Ratio (TSR) can be determined. The TSR is a measure of the loss of tensile strength that occurs in an asphalt sample as a result of conditioning the sample. The asphalt sample is conditioned by vacuum saturating the sample, freezing the sample, and then heating the sample at elevated temperatures prior to testing. This conditioning process is used to model the effects of temperature changes on the asphalt under field conditions. The TSR is then calculated by dividing the tensile strength of the conditioned sample by the tensile strength of the dry sample. For asphalt mixes it is generally accepted that the TSR should be greater than 0.8 to ensure durable pavements. The RAP samples will also be tested for gradation, angularity, and % flat and elongated particles after burning off the bitumen at high temperatures. These tests are more appropriate for estimating the rutting susceptibility of the asphalt pavement rather that the freeze thaw durability, but the results have been included for completeness.

For RAP aggregate particles, the remaining adhered binder may make the aggregate more resistant to freezing and thawing because the binder inhibits water intrusion. Also the binder may have no effect on the abrasion resistance of the RAP aggregate particles. Other testing such as gradation and water absorption does need to be addressed when using RAP in unbound pavement layers or HMA, but these properties are not important to the durability of the aggregate.

4.3 Testing for Recycled Concrete Aggregate

Durability testing was conducted for Recycled Concrete Aggregates (RCA) because the adhered mortar will affect the abrasion resistance and possibly the freeze-thaw resistance of the aggregates. The recycled aggregates have a significantly higher absorption, therefore the absorption cannot be used as a durability indicator, but it will be necessary if the aggregates are to be used in concrete. It may also be necessary to conduct AAR testing on the recycled aggregates depending on whether or not the alkali content of the cement to be used is significantly different from the original cement.

4.4 Testing for Foundry Slag

Foundry slag should be tested using the same procedures and specifications that were used for aggregates in unbound pavement layers, HMA, and PCC. If Iron Unsoundness or Calcium Disilicate Unsoundness have been a problem, additional testing may be required. Due to the rarity of these problems it is not recommended that this additional testing be used on a regular basis.

4.5 Laboratory Testing Program

For this project 74 aggregate samples, representing the full range of aggregate available in Wisconsin, were collected for testing. Initially all 70 crushed stone and crushed gravel samples were tested for vacuum saturated specific gravity (VSSG) and absorption (VSA) in a test procedure similar to ASTM C 127. The modification was to place the aggregate under a vacuum of 635 mm of mercury for 5 minutes prior to saturating the aggregates. Aggregate saturation consisted of the introduction of tap water while the aggregate was under vacuum and subsequent submersion in water for 24 +/- 1 hour. VSA testing was selected over standard absorption testing because the VSA more closely mimics the long-term field absorption of an aggregate. From these results 30 aggregates were selected for further analysis at Virginia Tech and 30 for analysis at VTRC throughout the range of the VSA values. The selected aggregate were then subjected to the following tests: Lightweight Particles in Aggregate (ASTM C 123-98), Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (ASTM C 131-01), Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus (AASHTO TP 58), Methods for Determination of Aggregate Crushing Value (BS 812-110 1990), Soundness of Aggregates by Use of Sodium Sulfate (ASTM C 88-99a), and Test Method for the Resistance of Unconfined Coarse Aggregate to Freezing and Thawing (CSA A23.3-24A). Additionally, 9 natural and 2 recycled aggregates were tested in concrete for freezing and thawing durability using ASTM C 666-97 (Resistance of Concrete to Rapid Freezing and Thawing) and for compressive strength using ASTM C 39 (Compressive Strength of Cylindrical Concrete Specimens). ASTM procedures C 666-97 Method A and C 39 were modified to 28 days of curing in

lime saturated water rather than the 14 days specified and the aggregate was saturated by soaking it in water for a period of 24 hours prior to inclusion in concrete.
One recycled concrete aggregate and one slag aggregate were subjected to the same testing listed above with the exception of Lightweight Particles in Aggregate. Table 4.5.1 presents the testing that was conducted for inclusion in this report.

	All 70 Natural Aggregate Samples	60 Selected Natural Aggregate Samples	Recycled Concrete Aggregate	Blast Furnace Slag
Vacuum Saturated Specific Gravity and Absorption	X	X	X	X
L.A. Abrasion		X	X	X
Micro-Deval		X	X	X
Sodium Sulfate Soundness		X	X	X
Aggregate Crushing Value		X	X	X
Unconfined Freezing and Thawing		X	X	X
Freezing and Thawing in Concrete		X	X	X
Lightweight Pieces in Aggregate		X		

Table 4.5.1 –Laboratory Test Matrix

In addition to the testing listed above VTRC conducted durability testing on Recycled Asphalt Pavement (RAP) using AASHTO T 283 (Resistance of Compacted Asphalt to Moisture Induced Damage).

CHAPTER 5: SELECTION OF AGGREGATES

With the assistance of WisDOT officials, 74 aggregate samples were selected for testing from across the state. The selected aggregates encompassed the full spectrum of aggregates that can be found in Wisconsin. Among the aggregates tested were glacial deposits of gravel, ledge rocks from different geologic groups, recycled aggregate stockpiles, and foundry slag. A variety of glacial deposits were tested because the material often varies widely depending upon from which direction the depositing glacier originated. It was also important to test ledge rocks from groups with good and poor field performance records. Table 5.1 lists the geologic formations and glacial lobes that were sampled from throughout Wisconsin. Figures 5.1 and 5.2 show the sample site locations in conjunction with the local geology for the quarries and pits, respectively. Also, a petrographic analysis was conducted on the aggregate samples and the results are presented in Appendix F.

Crushed Stone (System/Group/Formation)	Gravel (Glacial Lobe)
Keweenawan	Superior
Galena	Chippewa
Platteville	Pre-Wisconsin
Prarie du Chien	Green Bay
Decorah	Ontonogan
Mayville/Maquoketa	Lake Michigan
Silurian	

Table 5.1 – Sample Geology

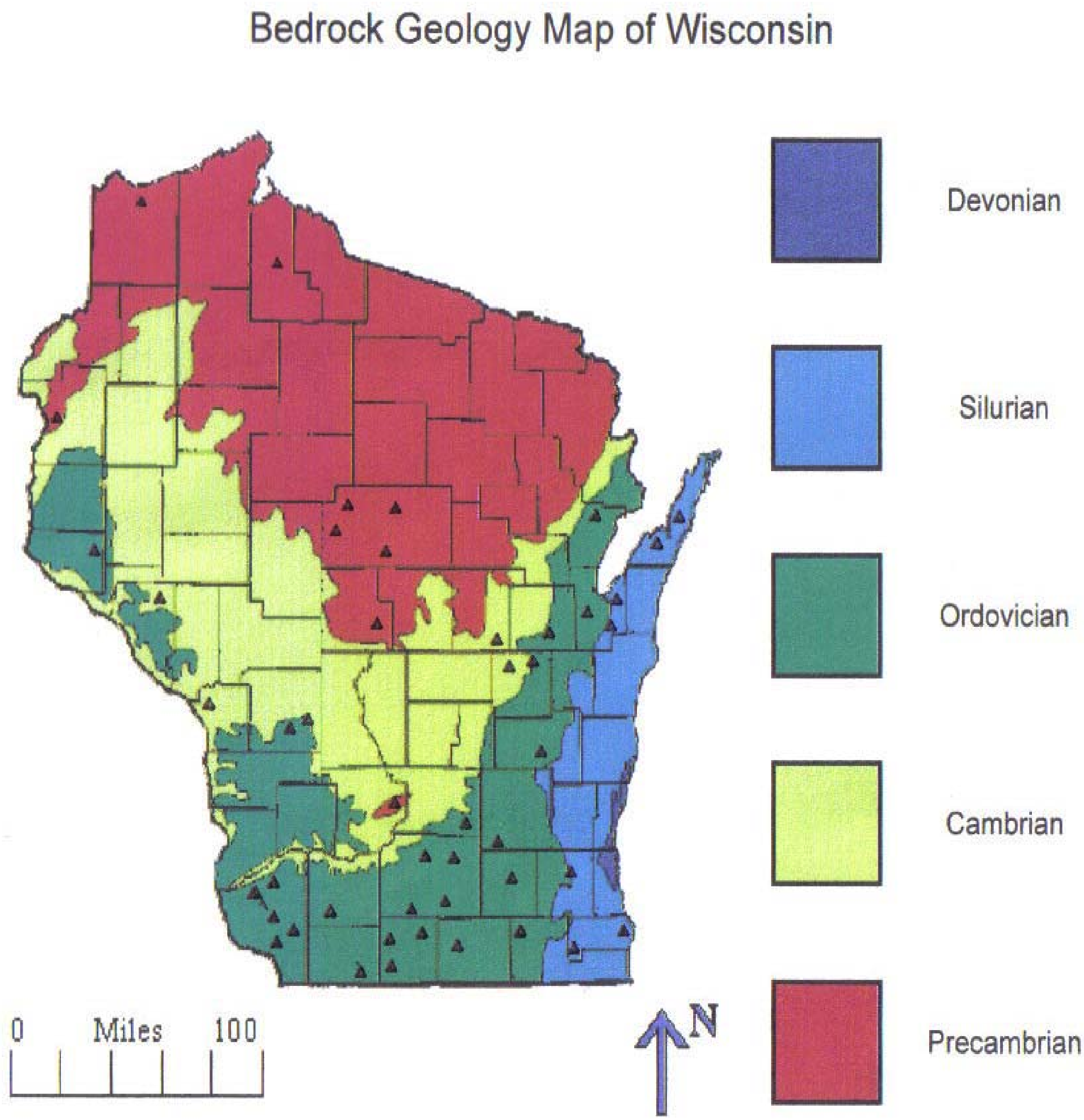


Figure 5.1 – Quarry Sample Sites (Milwaukee Public Museum)

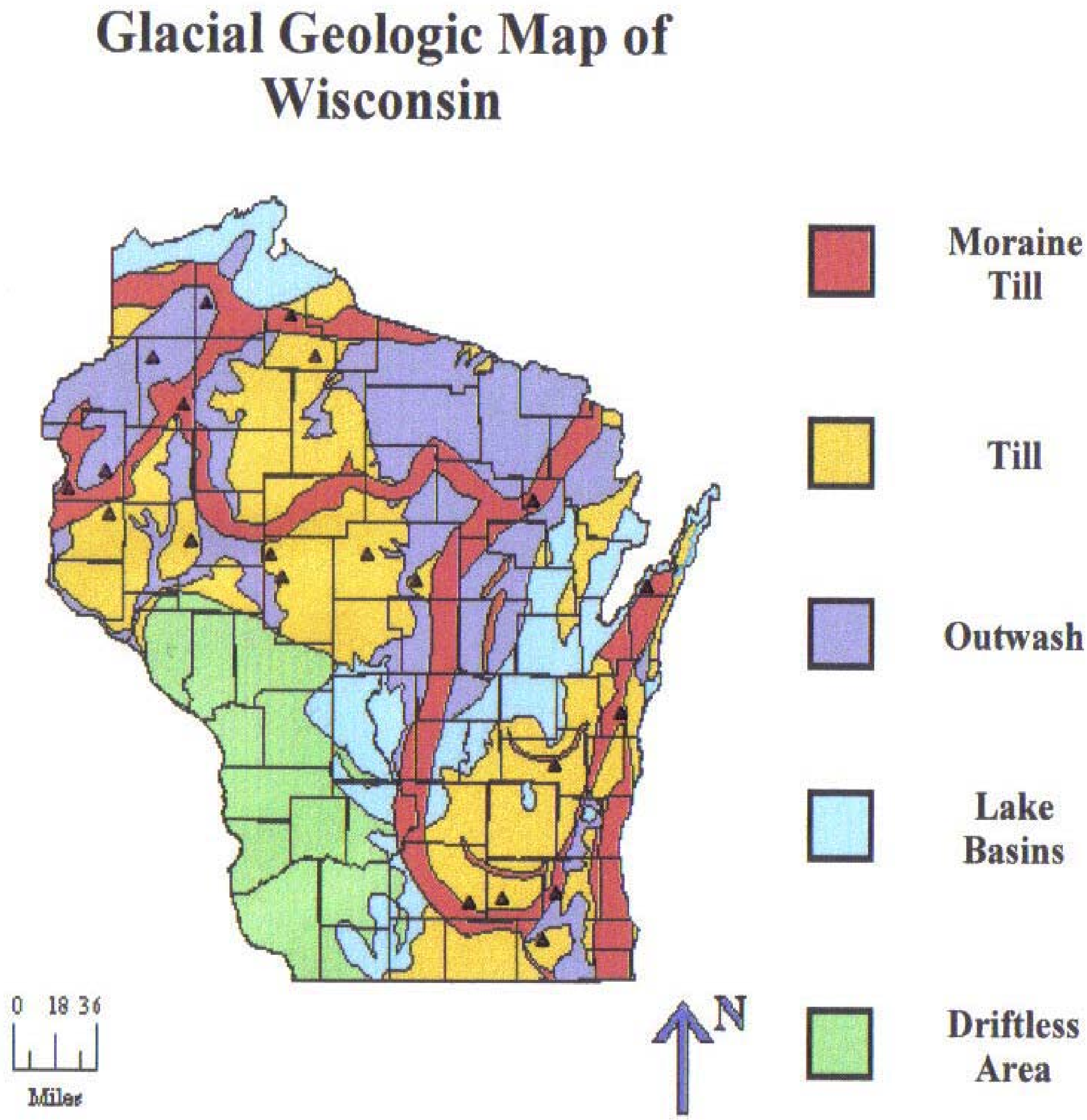


Figure 5.2 – Pit Sample Sites (Milwaukee Public Museum)

The importance of testing the range of aggregate durability performance based on field performance records cannot be over emphasized. It is often very easy to identify good and poor performing aggregates based on laboratory test results. However, it is much more difficult to identify aggregates that have adequate field performance histories but would be classified as intermediate aggregate based on laboratory testing. It is undesirable from both economic and social issues to reject a good aggregate or accept a poor aggregate.

For the 60 aggregate samples that were selected for further testing it was important to ensure that the full range of aggregate qualities was reflected in the sample set with an emphasis on intermediate quality aggregates. The poor, intermediate, and good ratings are based on field performance and/or test records provided by WisDOT officials. Due to the subjective nature of these classifications it is recommended that WisDOT verify these ratings to further validate the conclusions of this report. Table 5.2 shows the distribution of aggregate samples tested.

	Poor	**Intermediate**	**Good**
Quarries	10	20	15
Pits	4	6	5

Table 5.2 – Aggregate Sample Performance Distribution

Once the sample distribution had been determined, the individual samples within the performance categories were selected based on VSA data as there is a strong relationship between VSSG and VSA. Aggregates with low, moderate, and high absorption values were selected from each group in order to be certain that aggregate qualities can be identified without any dependence on absorption i.e. a poor aggregate with a low absorption can be identified as poor just the same as a poor aggregate with a high absorption.

CHAPTER 6: RESULTS AND DISCUSSION

6.1 Introduction

This chapter presents the results of all aggregate durability testing conducted at Virginia Tech and the Virginia Transportation Research Council (VTRC).

6.2 Vacuum Saturated Specific Gravity and Absorption

The VSSG and VSA were determined for all 70 natural aggregate samples with a modified version of ASTM C 127. Aggregate was held under a vacuum of 635 mm of mercury for 5 minutes prior to saturation, and then allowed to soak for 24 hours. VSA was then plotted against VSSG to investigate the relationship (see Figure 6.2.1.) As shown, there is a general linear relationship between VSA and VSSG. The relationship is much stronger for carbonate rocks than what is shown in Figure 6.2.1. This is due to the very low absorptions that are characteristic of igneous and metamorphic materials of all specific gravities.

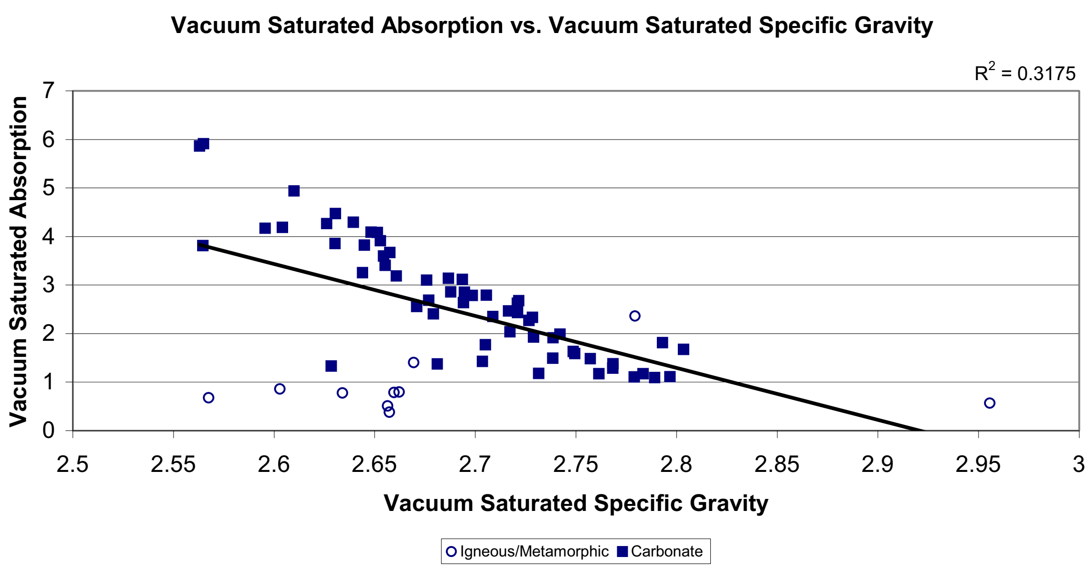


Figure 6.2.1 – Vacuum Saturated Absorption vs. Vacuum Saturated Specific Gravity

The distribution histograms for VSA and VSSG are shown in Figures 6.2.2 and 6.2.3. The VSA data was used to select the aggregates to be subjected to further testing because of the dependence of VSSG on VSA. The use of the VSA as a preliminary durability indicator will also be investigated in comparisons with other test data.

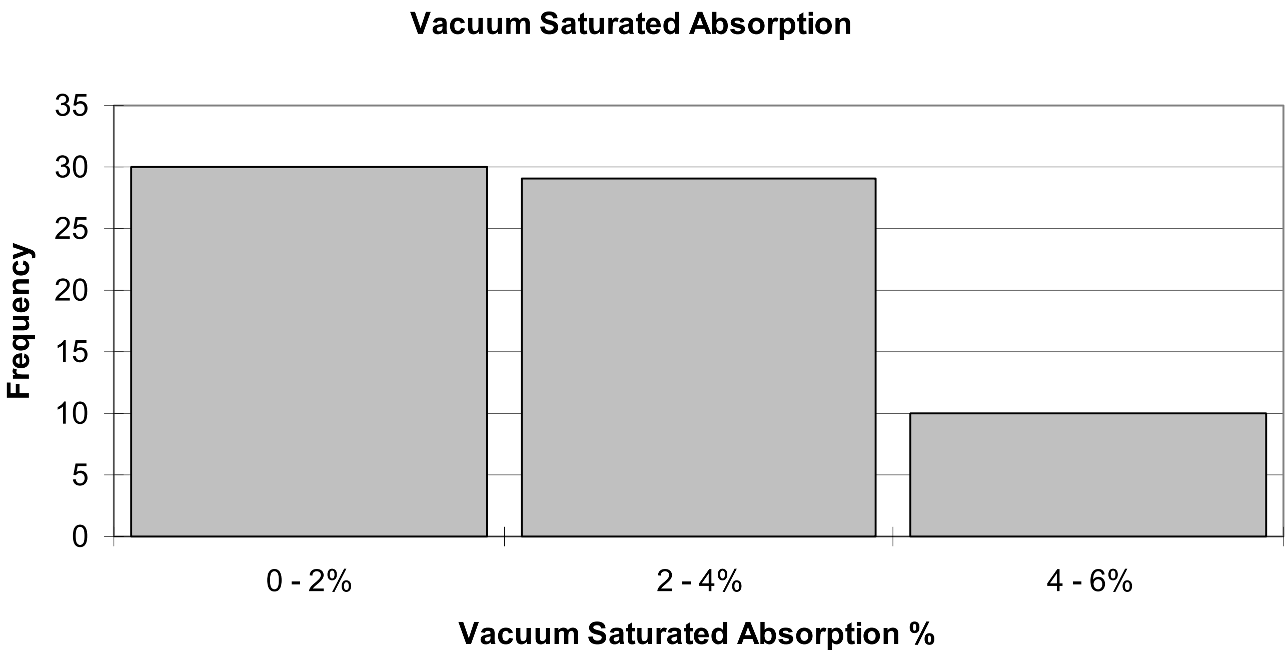


Figure 6.2.2 – Vacuum Saturated Absorption Histogram

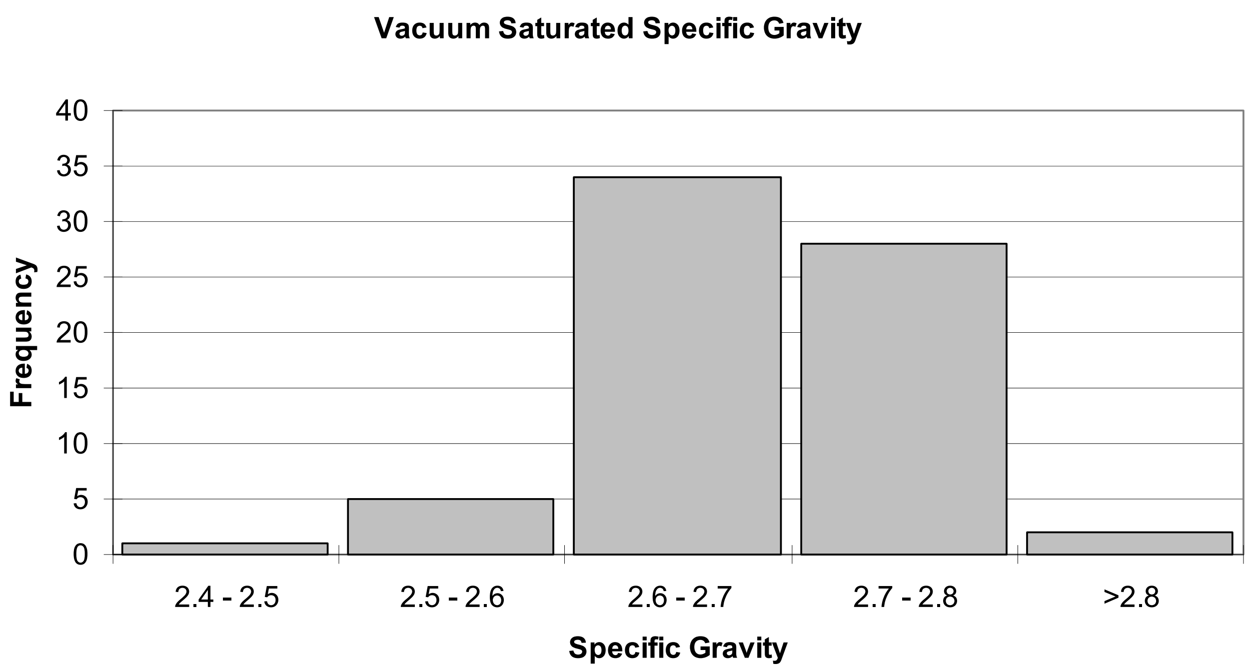


Figure 6.2.3 – Vacuum Saturated Specific Gravity Histogram

6.3 Lightweight Particles in Aggregate

The percentage of lightweight particles in aggregate was determined using WisDOT modified ASTM procedure 123-98. The ASTM procedure was modified to test only coarse aggregate material retained on the 3/8 in. sieve to mimic WisDOT specifications. WisDOT sets a limit of 5% and 2% lightweight chert by mass for standard concrete and pre-stressed concrete, respectively. Lightweight aggregate is defined as material with a SSD specific gravity of less than 2.45. The lightweight material was then classified as chert by petrographic analysis. The relationship between these values and absorption was investigated, see Figures 6.3.1, 6.3.2, 6.3.3, and 6.3.4. Distribution histograms were also developed and are presented in Figures 6.3.5 and 6.3.6. The chert percentages were determined for all of the aggregates tested, but it was decided that it was more desirable to set a limitation on the percentage of lightweight material rather than on the chert material. This was done because it was found that lightweight aggregate that would not be classified as chert may also result in durability problems for aggregate in the bound state.

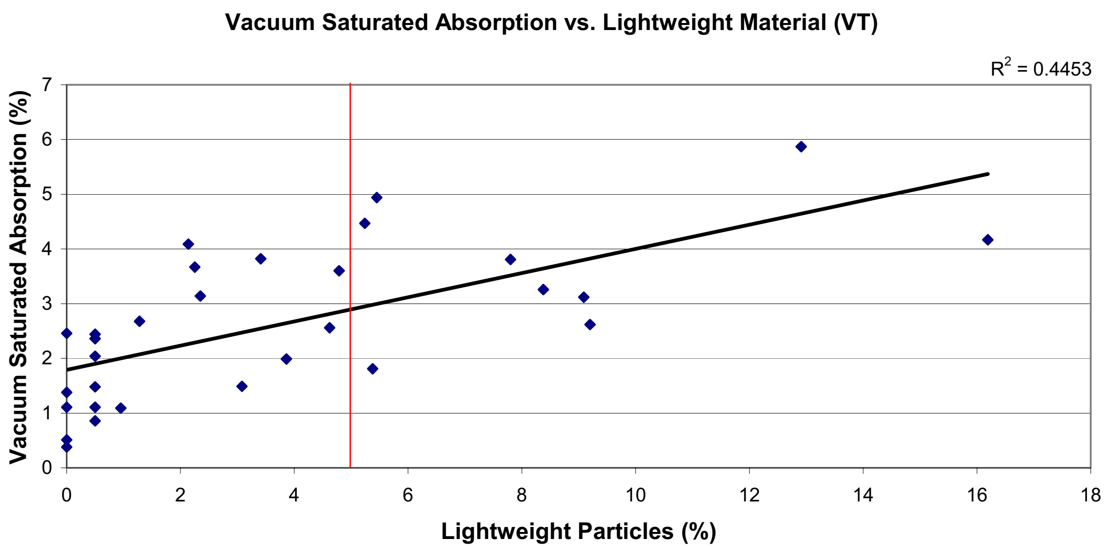


Figure 6.3.1 – Vacuum Saturated Absorption vs. % Lightweight Particles (VT)

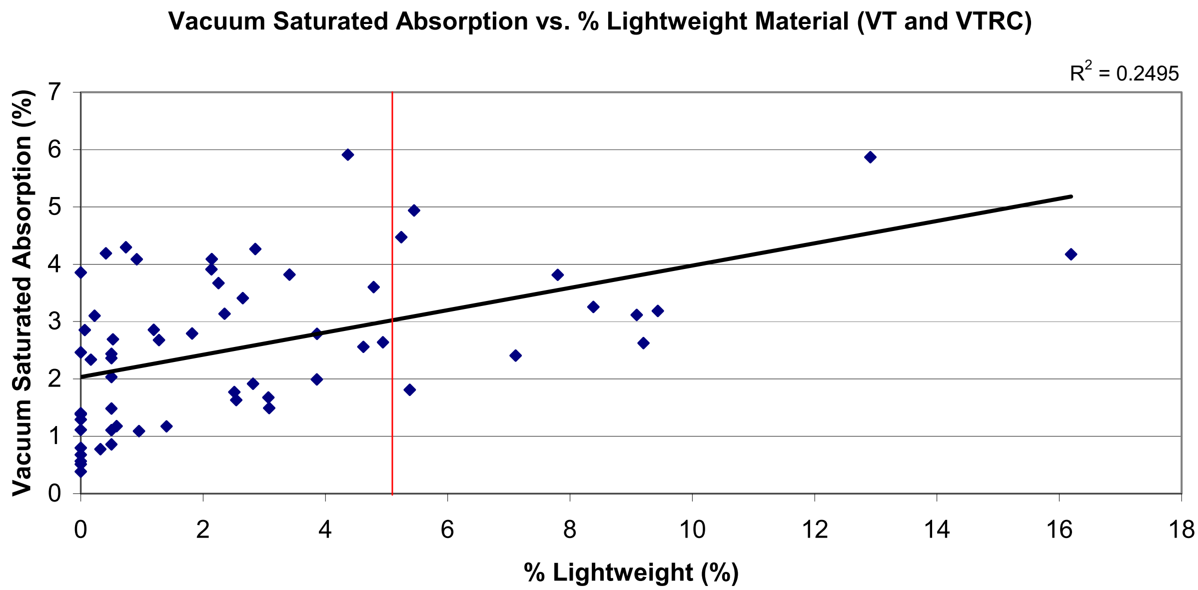


Figure 6.3.2 – Vacuum Saturated Absorption vs. % Lightweight Particles (VT and VTRC)

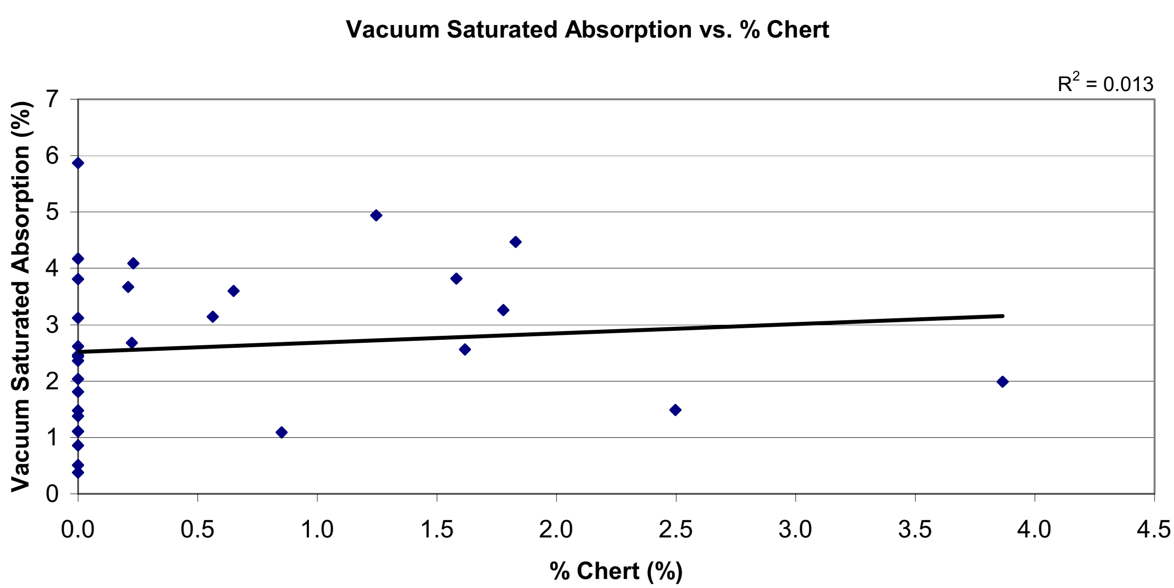


Figure 6.3.3 – Vacuum Saturated Absorption vs. % Chert (VT)

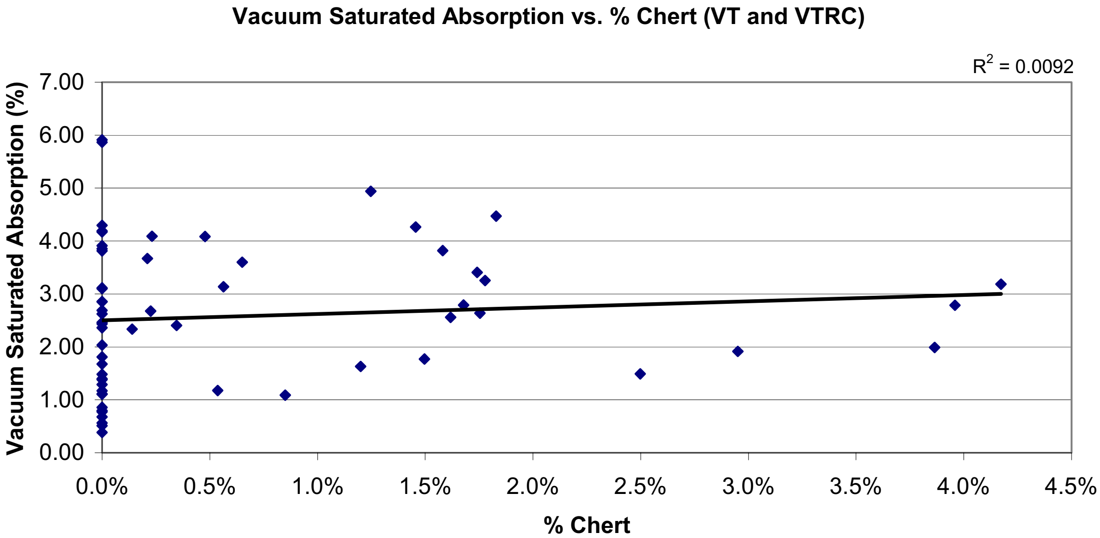


Figure 6.3.4 – Vacuum Saturated Absorption vs. % Chert (VT and VTRC)

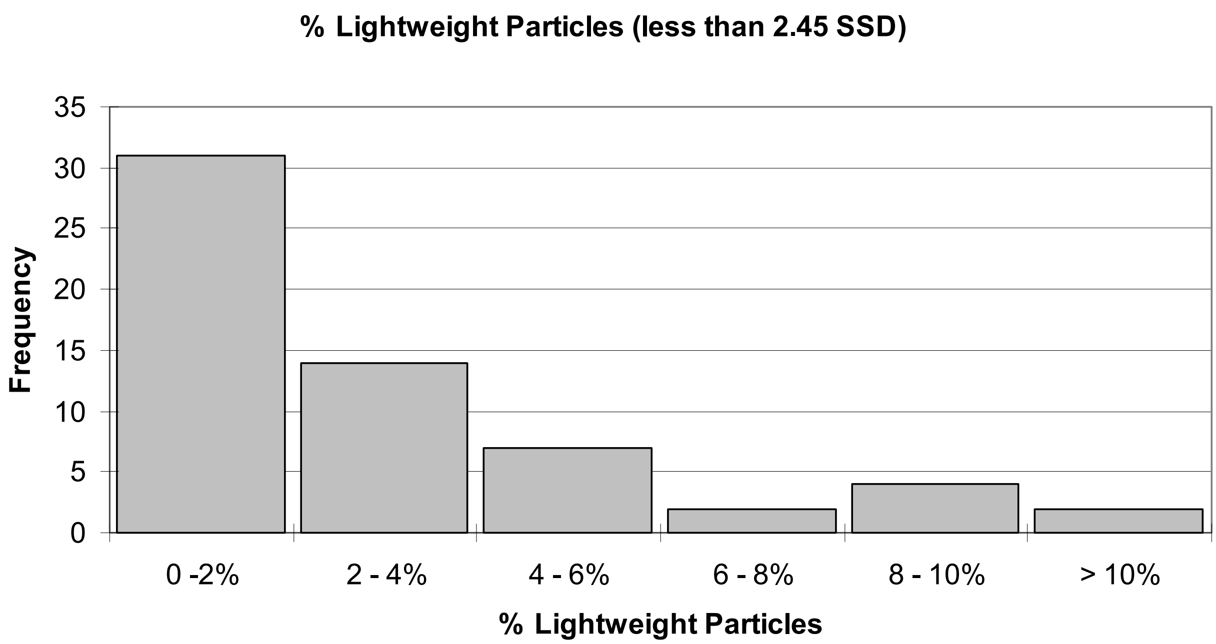


Figure 6.3.5 - % Lightweight Particles Distribution Histogram (VT and VTRC)

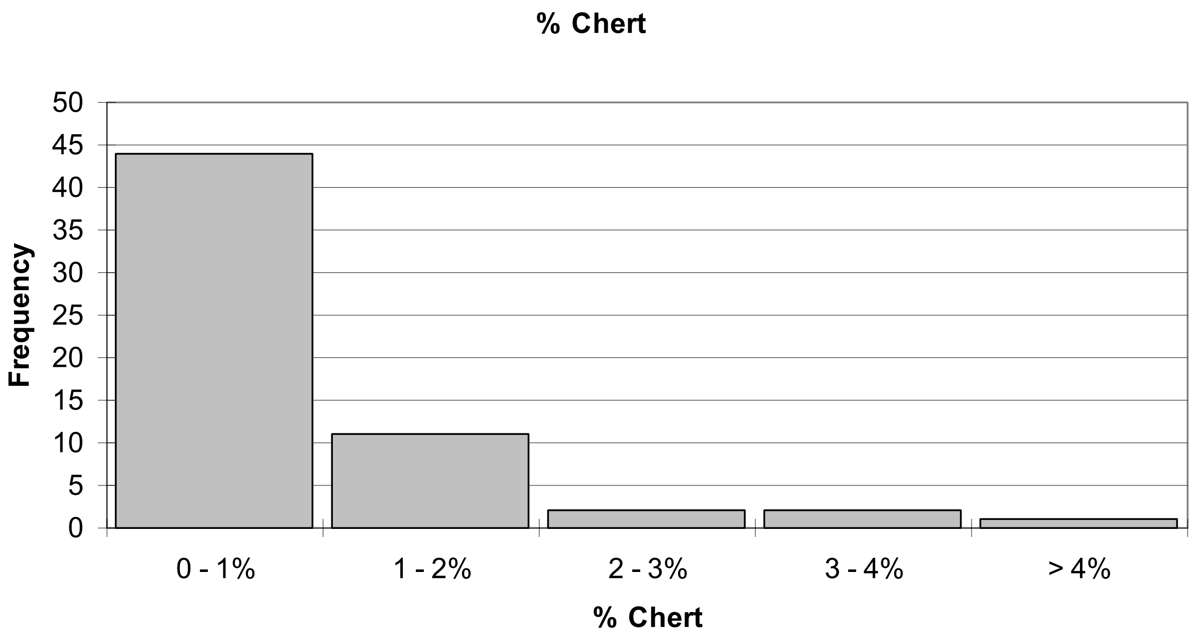


Figure 6.3.6 - % Chert Distribution Histogram (VT and VTRC)

As shown there appears to be a general correlation between lightweight material and VSA and no relationship between VSA and chert. However, the linear relationship is weaker between the lightweight material and VSA when the VTRC test data is included, $R^2 = 0.44$ compared to 0.25.

6.4 L.A. Abrasion

L.A. Abrasion tests were performed using 500 revolutions in accordance with ASTM C 131-01. Figures 6.4.1 and 6.4.2 present the relationship between L.A. Abrasion and VSA for aggregate tested at VT and VT and VTRC combined, respectively. The failure limit is shown as a red line and the dashed blue line represents the range for which all L.A. Abrasion values will lie for VSA of less than 2%.

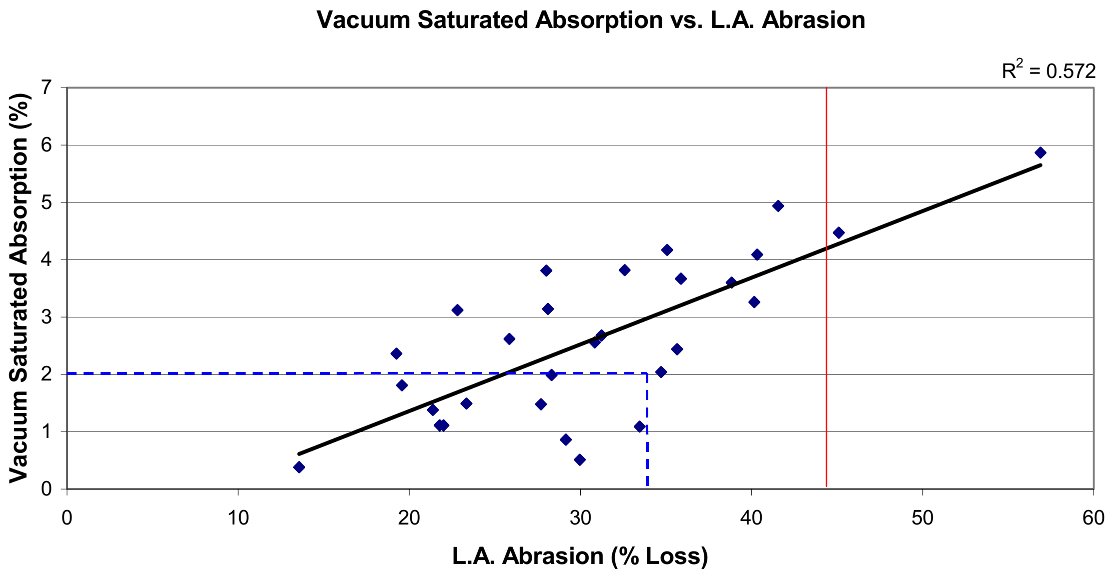


Figure 6.4.1 – Vacuum Saturated Absorption vs. L.A. Abrasion (VT)

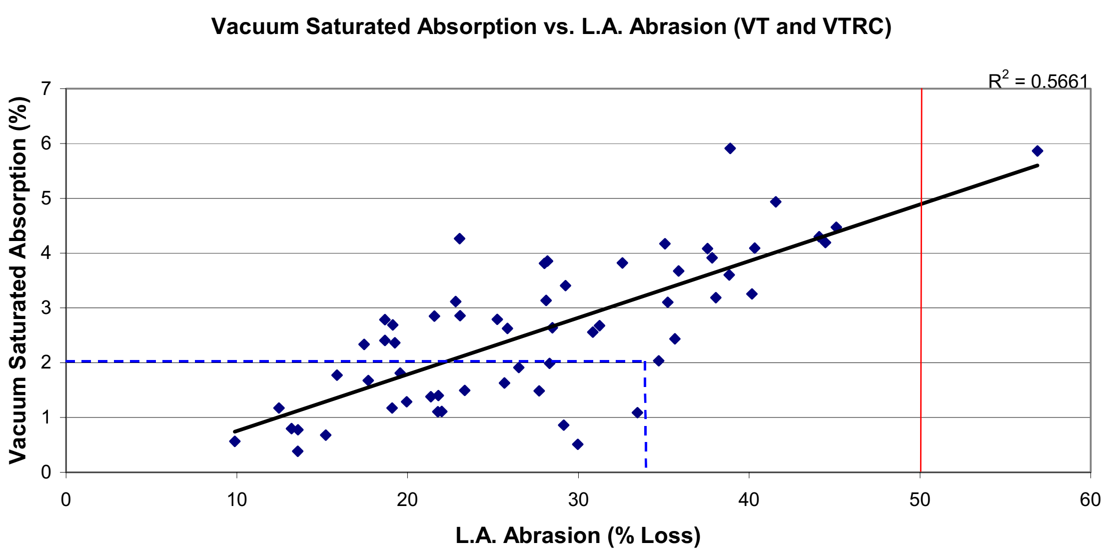
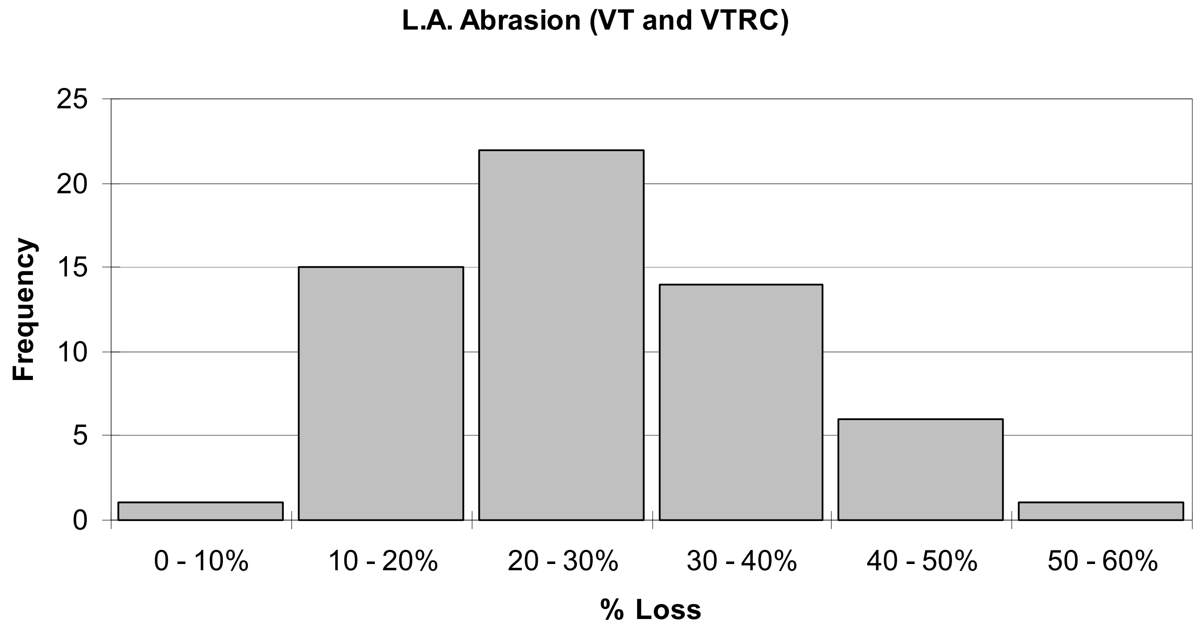


Figure 6.4.2 – Vacuum Saturated Absorption vs. L.A. Abrasion (VT and VTRC)

As demonstrated, aggregate with VSA of less than 2% will have a L.A. Abrasion value of less than 35%, which is substantially lower than the 50% loss failure criterion used by WisDOT. It should be noted that only one aggregate had a L.A. Abrasion value of greater than 50%. The strength of the general linear relationship between VSA and L.A. Abrasion is the same for both the VT and VT and VTRC combined test results, R^2 = 0.57. Figure 6.4.3 presents the distribution histogram for the L.A. Abrasion results.



6.4.3 – L.A. Abrasion Distribution Histogram (VT and VTRC)

6.5 Micro-Deval

Figures 6.5.1 and 6.5.2 present the relationship between VSA and Micro-Deval % loss for aggregate tested at VT and VT and VTRC, respectively. A loss of 18% was used as the failure criterion as recommended by Kandhal and Parker for HMA (1998). Senior and Rogers recommend that the maximum allowable losses be 15% for HMA, 20% for concrete, and 40% for base material (1991). The Ontario standard specification currently allows a maximum loss of 13% for concrete pavement, 17% for structural concrete, 5-17% for HMA, and 30% for granular sub-base.

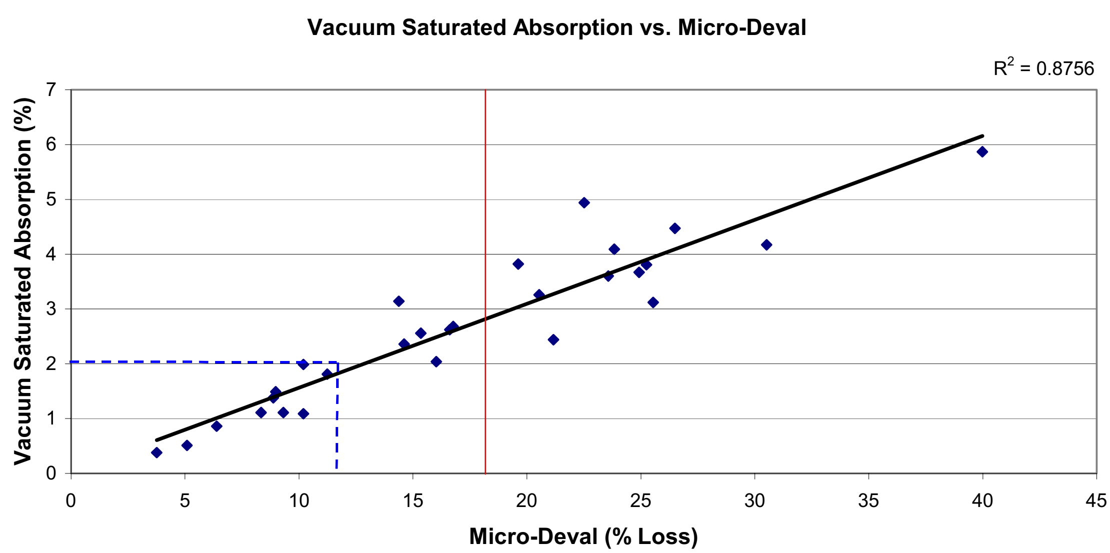


Figure 6.5.1- Vacuum Saturated Absorption vs. Micro-Deval (VT)

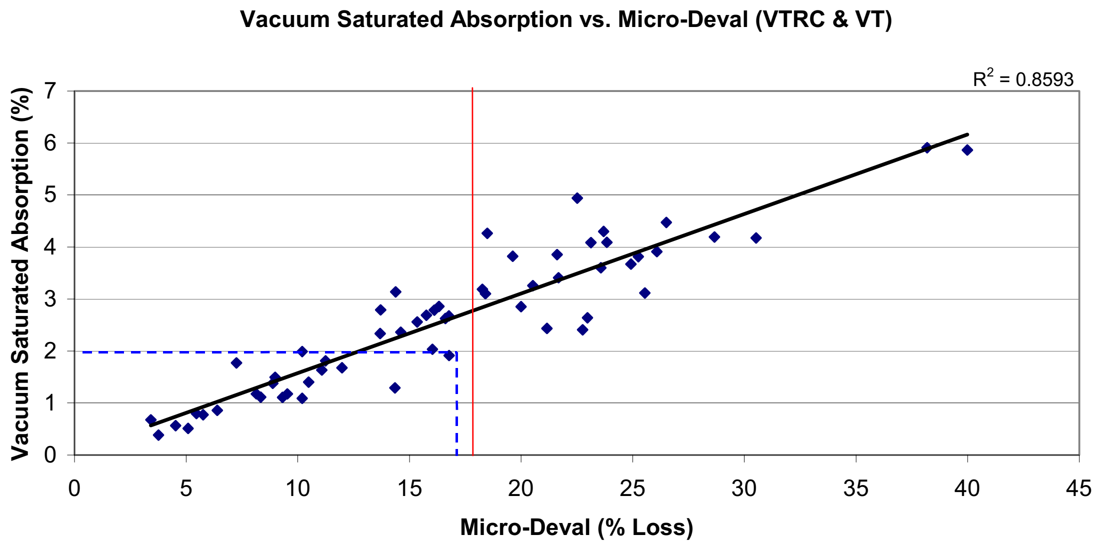


Figure 6.5.2 – Vacuum Saturated Absorption vs. Micro-Deval (VT and VTRC)

As demonstrated, a failure limit of 18% may be too stringent in general and a limit of 25-30% may be more appropriate for Wisconsin aggregate. Also, for aggregate with VSA less than 2% the Micro-Deval loss will be less than 17%. Therefore, it is reasonable to conclude that aggregate with VSA less than 2% will have a low the Micro-Deval value. The strength of the linear relationship between the VSA and Micro-Deval results is approximately equal for the VT and VT and VTRC combined results with R^2 values equal to 0.87 and 0.86, respectively. The Micro-Deval distribution histogram is shown in Figure 6.5.3.

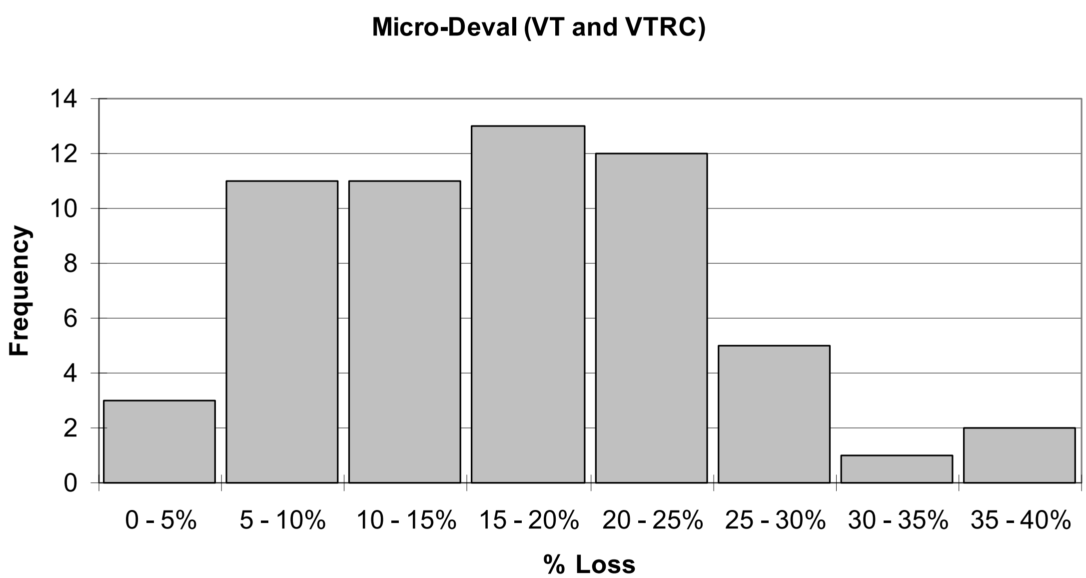


Figure 6.5.3 – Micro-Deval Distribution Histogram (VT and VTRC)

6.6 Aggregate Crushing Value

Figure 6.6.1 presents the Aggregate Crushing Values of the aggregates tested at VT with respect to the VSA. Figure 6.6.2 presents the results of the aggregates tested at VT and VTRC. The British Standard suggests that the allowable aggregate crushing loss should be based on the parent material. Average losses range from 16% for igneous material to 27% for argillaceous limestone.

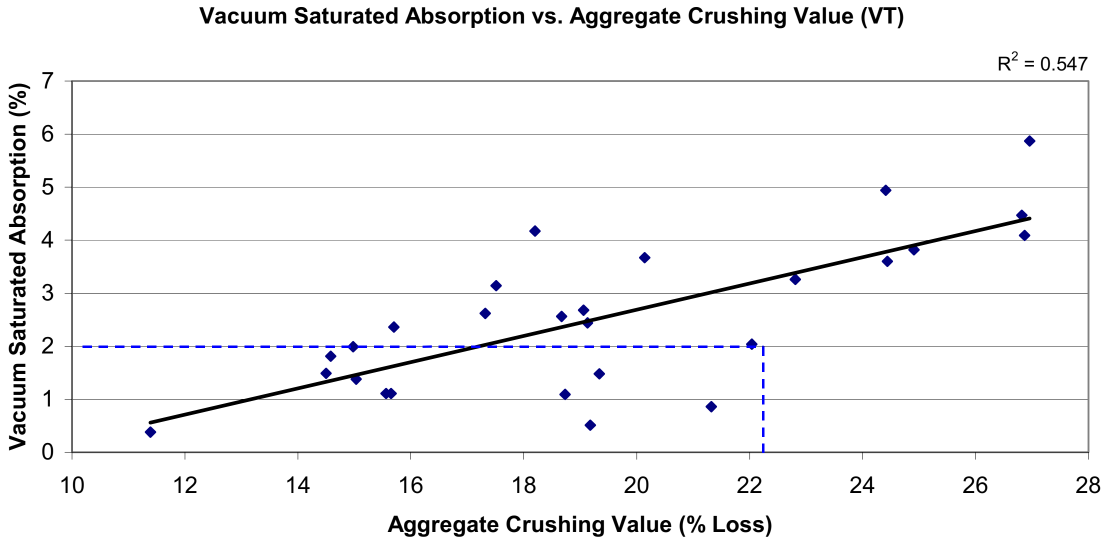


Figure 6.6.1 – Vacuum Saturated Absorption vs. Aggregate Crushing Value (VT)

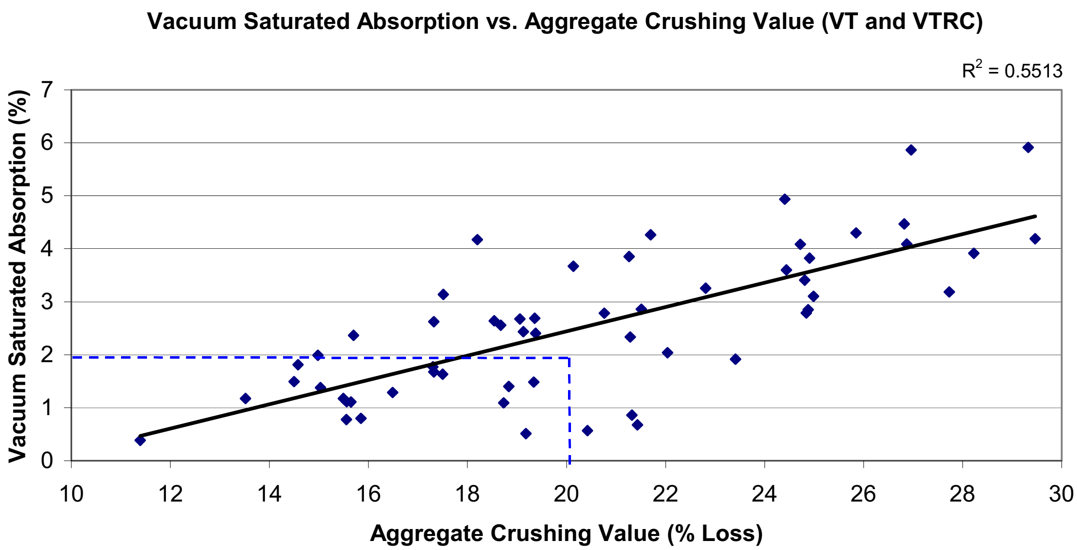


Figure 6.6.2 – Vacuum Saturated Absorption vs. Aggregate Crushing Value (VT and VTRC)

With no discernable relationship or recommended failure criteria it is not possible to draw any conclusions from these data. The distribution data presented in Figure 6.6.3 offers no additional useful information. However, VSA's of less than 2% generally have aggregate crushing values of less than 22%. Also, the strength of the linear relationship for the VT and the VT and VTRC combined test results are the same, $R^2 = 0.55$.

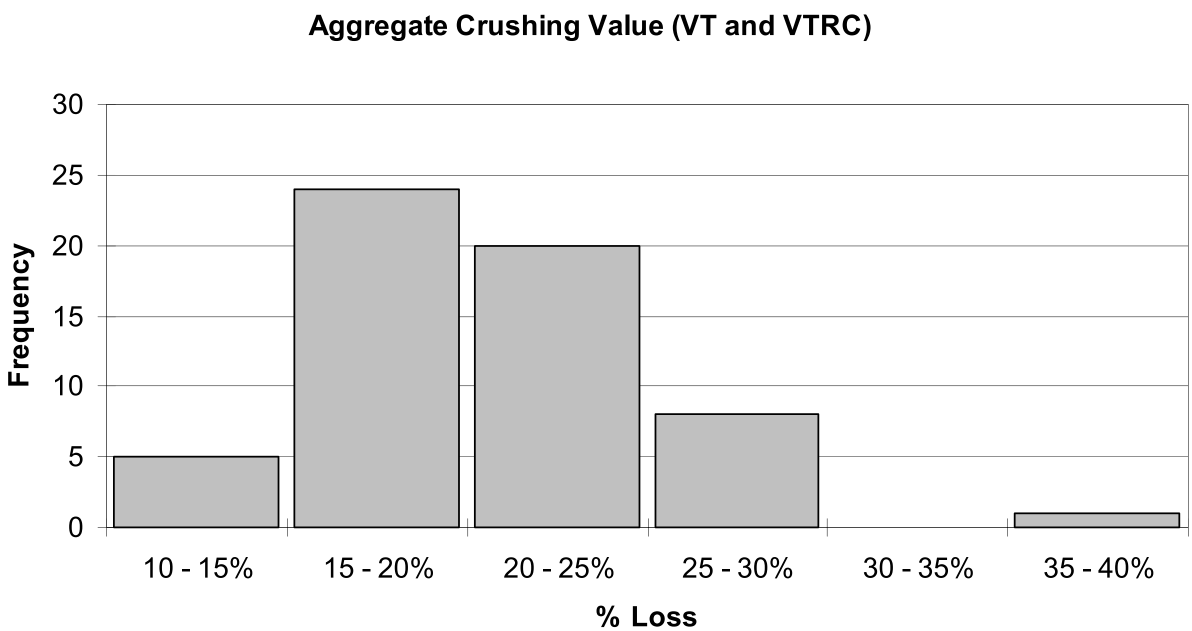


Figure 6.6.3 – Aggregate Crushing Value Distribution Histogram (VT and VTRC)

6.7 Compressive Strength of Concrete Cylinders

Concrete cylinders were made for 9 natural and 2 recycled aggregates. These cylinders were cast from the same concrete batches that were used for the Freezing and Thawing in Concrete tests (ASTM C 666). Compressive strength testing of the concrete cylinders was conducted in accordance with ASTM C 39. The results are shown in Table 6.7.1. The initial results were then adjusted for differences in w/c ratio and air content using Abrams' Rule (Abrams, 1918) and an expression recommended by Rykewitsch and Nurse (Ryskewitsch, 1953; Nurse, 1968), respectively. The compressive strengths shown in Table 6.7.2 represent the relative compressive strengths of the concrete specimens adjusted to reflect a w/c ratio of 0.41 and an air content of 4.2%.

Sample #	Specimen A (kips)	Specimen B (kips)	Specimen C (kips)	Specimen D (kips)	Specimen E (kips)	Average (kips)	28-day Moist-Cured Compressive Strength (psi)	w/c	Air Void Content (%)
36	70.5	71.5	73.5	69.0	75.0	71.9	5720	0.48	6.0
42	77.5	71.5	70.5	67.0		71.6	5700	0.41	6.5
12	75.0	77.5	76.0	75.0		75.9	6040	0.43	6.2
31	81.0	80.0	82.0			81.0	6450	0.43	6.2
22	74.0	76.0	75.5	76.0		75.4	6000	0.43	5.9
60	69.5	72.5	72.0			71.3	5680	0.48	6
55	73.0	75.5	72.5	73.0		73.5	5850	0.46	5.8
50	55.0	56.5	55.0	53.0		54.9	4370	0.47	5.7
39	91.5	97.5	95.0			94.7	7530	0.43	4.2
71	60.0	56.0	56.0	57.5		57.4	4570	0.47	7.7
73	78.0	78.0	78.0	79.0		78.3	6230	0.44	5.8

Table 6.7.1 – Compressive Strength of 4"x8"Concrete Cylinders

Sample #	28-day Moist-Cured Compressive Strength (psi)	w/c Adjustment Factor	Air Content Adjustment Factor	w/c Adjustment	Air Content Adjustment	Relative Compressive Strengths	Performance Rating
50	4370	0.885	0.936	502	278	5150	Poor
12	6040	0.960	0.916	241	506	6780	Intermediate
22	6000	0.960	0.928	239	430	6670	Intermediate
55	5850	0.903	0.932	566	395	6811	Intermediate
60	5680	0.867	0.924	754	430	6864	Intermediate
31	6450	0.960	0.916	257	540	7240	Good
36	5720	0.867	0.924	759	433	6910	Good
39	7530	0.960	1.000	300	0	7830	Good
42	5700	1.000	0.904	0	546	6250	Good
71	4570	0.885	0.858	525	649	5740	RCP
73	6230	0.941	0.932	369	421	7020	Slag

Table 6.7.2 – Relative Compressive Strength of 4”x8” Concrete Cylinders

As shown in Table 6.7.2, the relative compressive strengths of the cylinders generally increases with the WisDOT provided performance ratings. The results indicate that compressive strength is related to aggregate quality. Thus, verifying the previous conclusion that aggregates need to be tested in concrete particularly when high strength concrete is to be used. It should be noted that the compressive strength of the RCP concrete is lower than most of the other natural aggregate concrete, which is to be expected. Also, the slag concrete is one of the strongest, which corresponds with the results of the other tests that were conducted.

6.8 Strength and Abrasion Test Comparison

A comparison of L.A. Abrasion and Micro-Deval test results presented in Figure 6.8.1 for aggregate tested at VT and Figure 6.8.2 for aggregate tested at VT and VTRC demonstrated that there is a linear relationship although not a strong one with $R^2 = 0.56$. This indicates that the tests are measuring different properties i.e. L.A. Abrasion measures dry aggregate impact strength and abrasion and the Micro-Deval measures wet abrasion resistance.

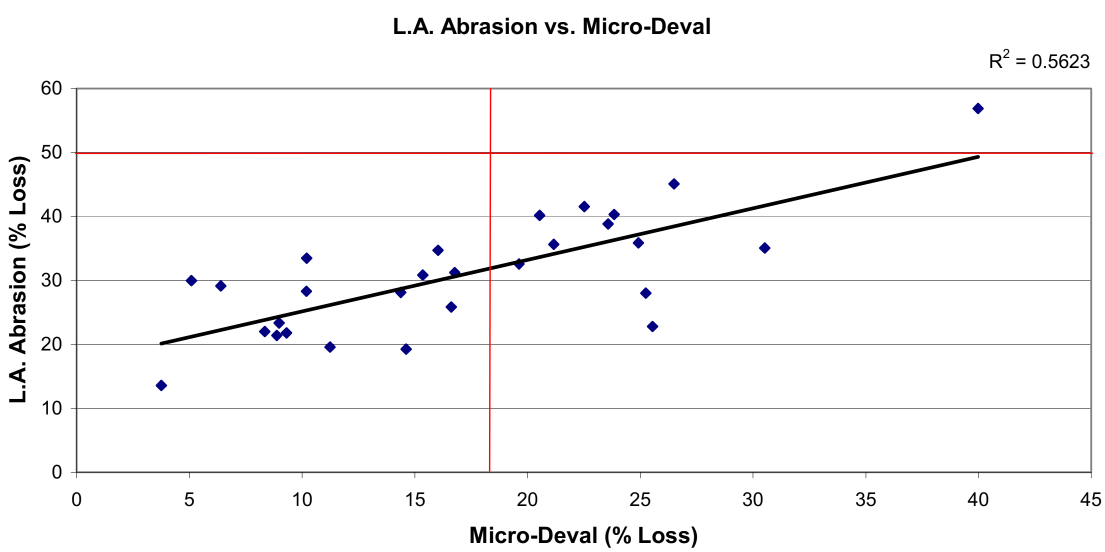


Figure 6.8.1 – L.A. Abrasion vs. Micro-Deval (VT)

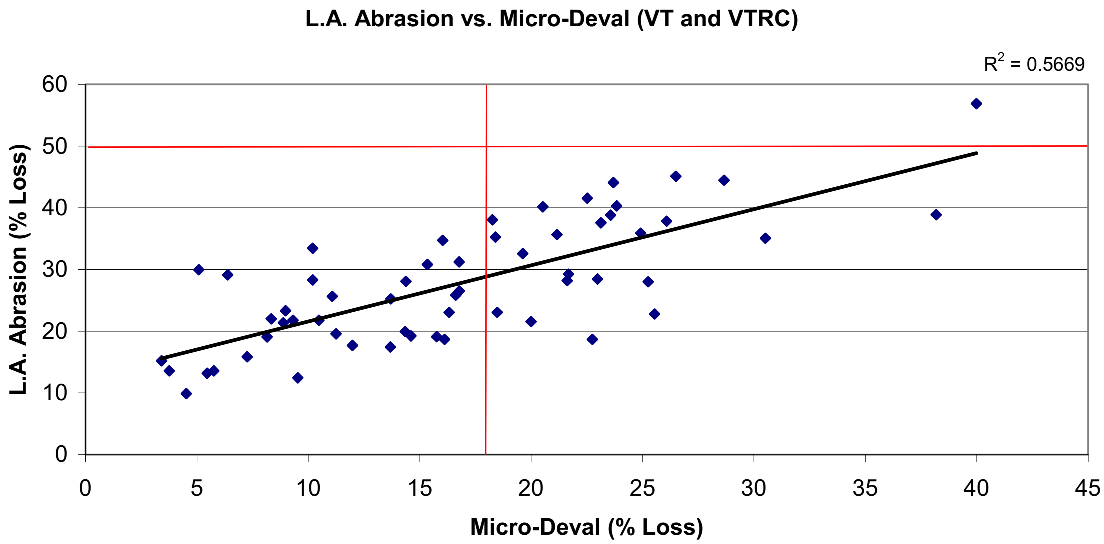


Figure 6.8.2 – L.A. Abrasion vs. Micro-Deval (VT and VTRC)

Figures 6.8.3 and 6.8.4 present the relationship between L.A. Abrasion and Aggregate Crushing Value. As shown in Figure 6.8.3, the two tests are highly correlated (R^2 = 0.79) implying that the L.A. Abrasion test is more a test of aggregate strength than of abrasion resistance. The combined data set shown in Figure 6.8.4 does not have as strong a relationship as the VT results shown in Figure 6.8.3, which may be the result of a larger test pool. This may also indicate that the two test pools are not equivalent with regards to the aggregate tested. The results indicate that it may be necessary to perform both the L.A. Abrasion Test and the Micro-Deval Test or possibly the Micro-Deval and Aggregate Crushing test.

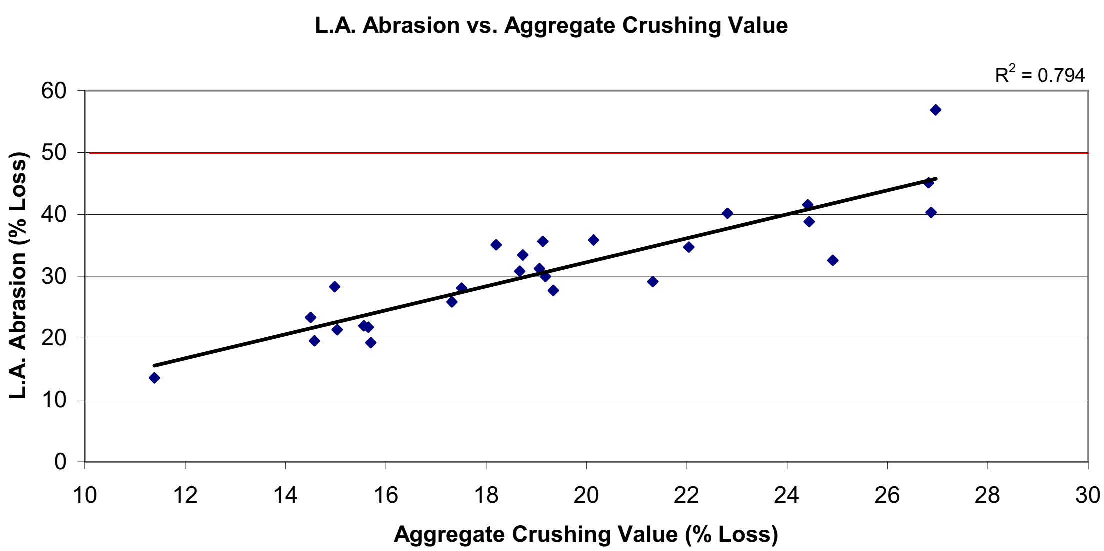


Figure 6.8.3 – L.A. Abrasion vs. Aggregate Crushing Value (VT)

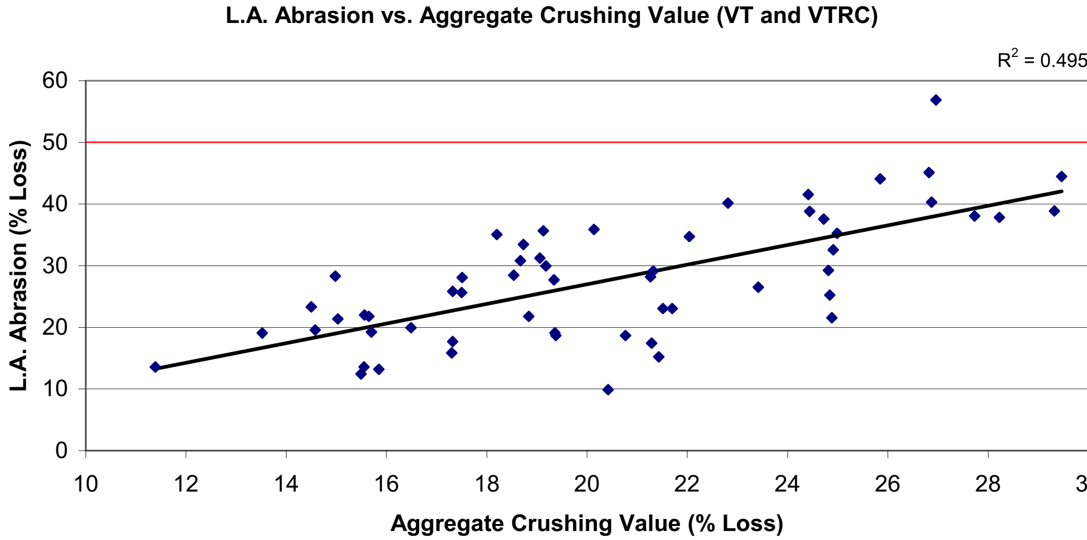


Figure 6.8.4 – L.A. Abrasion vs. Aggregate Crushing Value (VT and VTRC)

The relationship between Micro-Deval Loss and Aggregate Crushing Value is shown in Figures 6.8.5 and 6.8.6. There is only a general correlation between the two tests, which demonstrates that the tests are measuring different properties of the aggregate. The strength of the VT and VT and VTRC combined test results are about equal with R^2's of about 0.50. The Micro-Deval Test measures the abrasion resistance of the aggregate while the Aggregate Crushing Value measures strength.

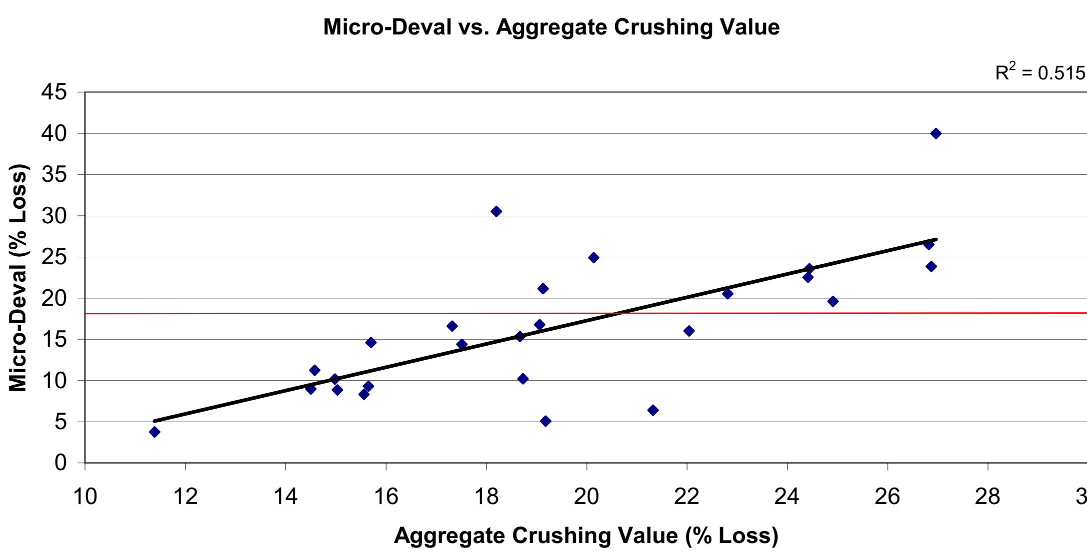


Figure 6.8.5 – Micro-Deval vs. Aggregate Crushing Value (VT)

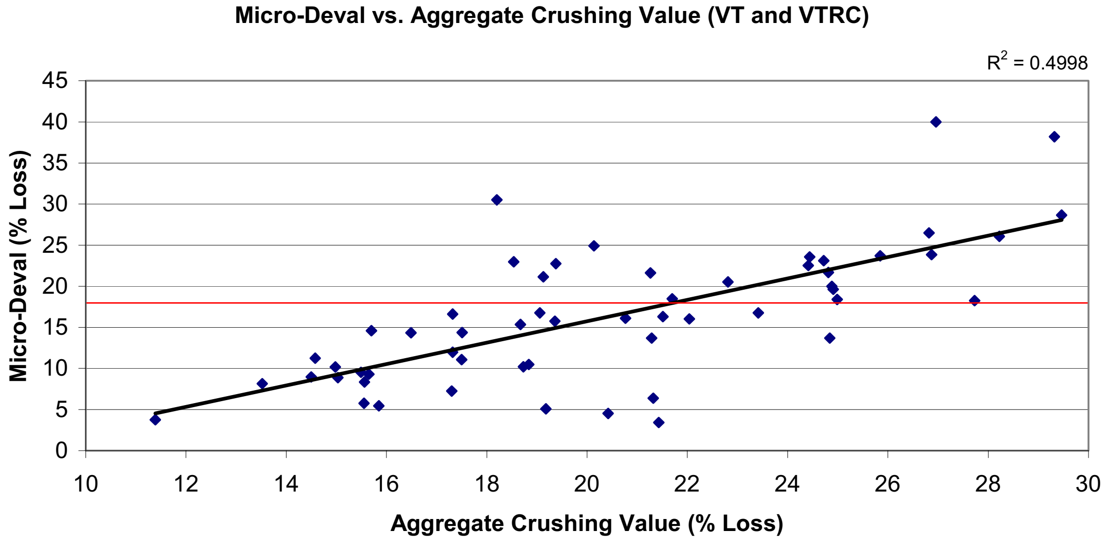


Figure 6.8.6 – Micro-Deval vs. Aggregate Crushing Value (VT and VTRC)

6.9 Sodium Sulfate Soundness

Figures 6.9.1 and 6.9.2 present the relationship between VSA and sodium sulfate soundness loss with a failure criterion of 12% loss, which is currently being used by WisDOT. There is no apparent correlation between the two data sets. However, aggregates with VSA's of less than 2% will have sulfate soundness losses of less than approximately 8%. A distribution histogram is shown in Figure 6.9.3.

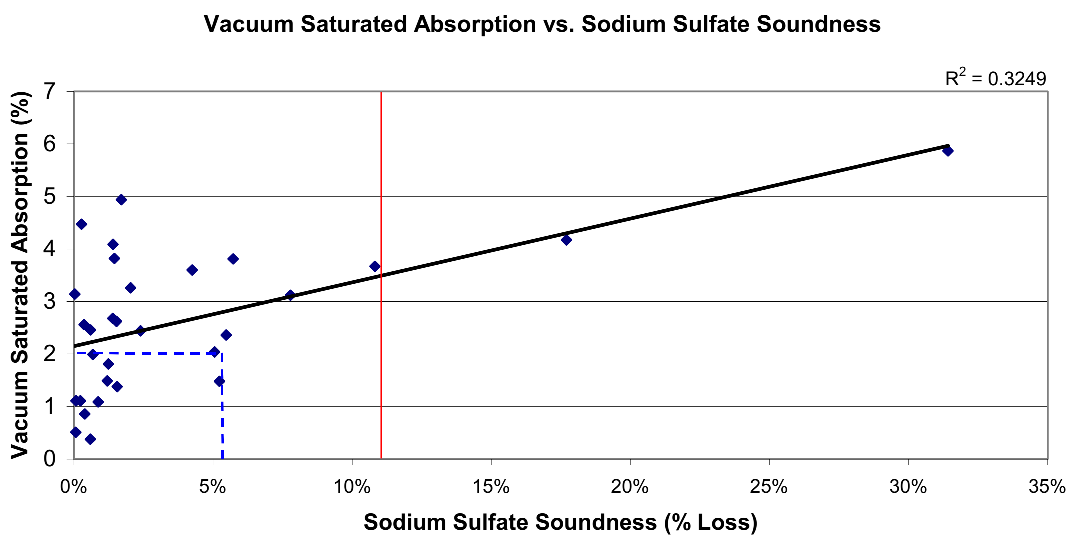


Figure 6.9.1 – Vacuum Saturated Absorption vs. Sodium Sulfate Soundness (VT)

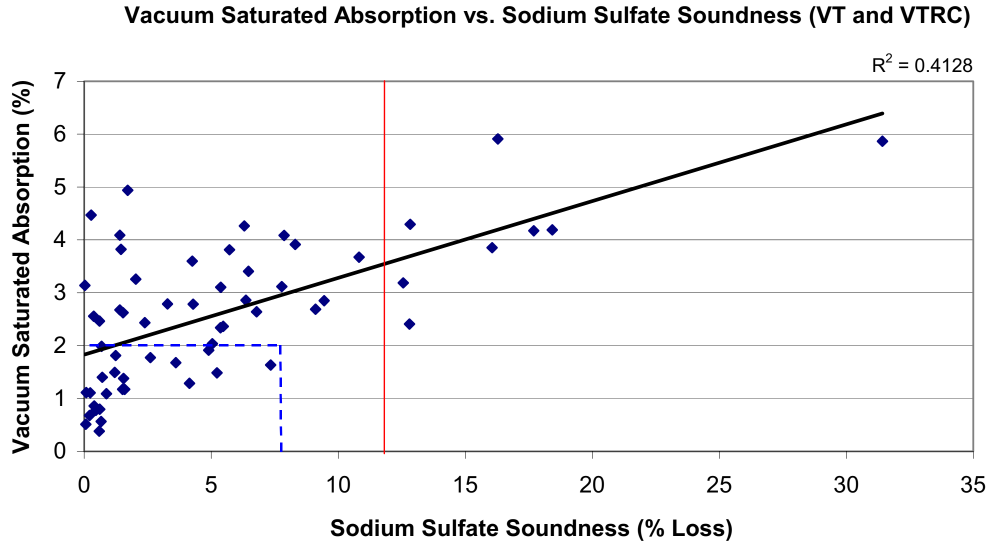


Figure 6.9.2 – Vacuum Saturated Absorption vs. Sodium Sulfate Soundness (VT and VTRC)

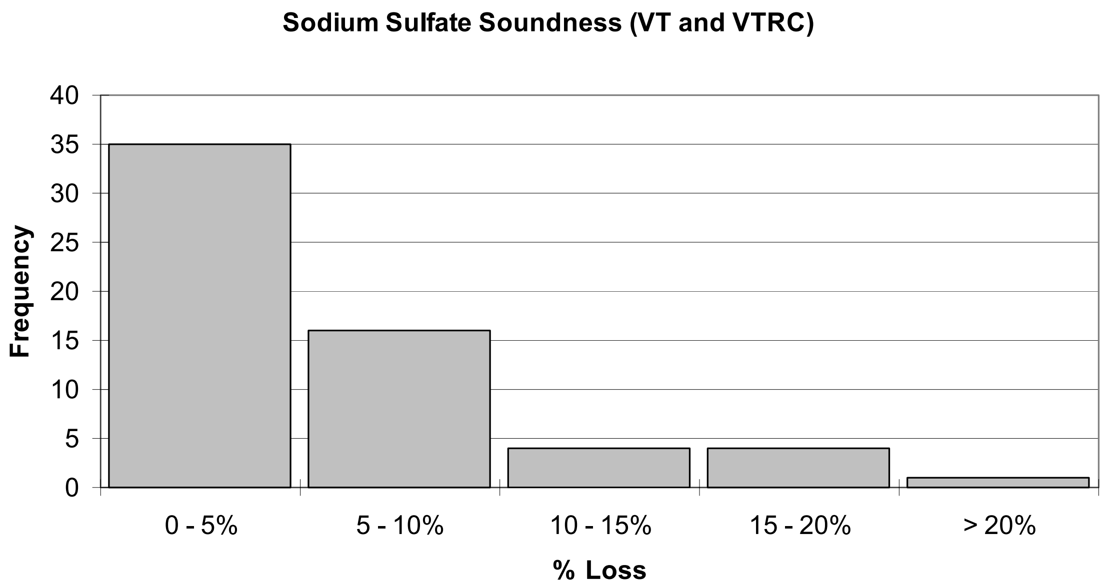


Figure 6.9.3 – Sodium Sulfate Soundness Distribution Histogram (VT and VTRC)

6.10 Unconfined Freezing and Thawing

Unconfined Freezing and Thawing of Aggregate tests were conducted in accordance to Canadian Standard A23.2-24A. The failure criterion is a 10% loss after five cycles of freezing and thawing, which is recommended by the Canadian Standards Association. Figures 6.10.1 and 6.10.2 present the relationship between VSA and freezing and thawing loss.

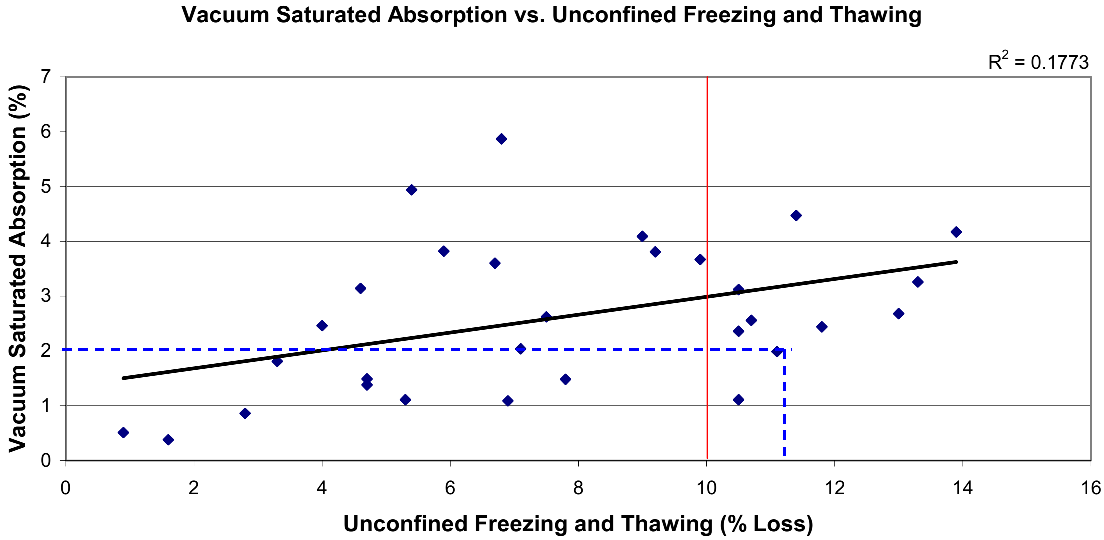


Figure 6.10.1 – Vacuum Saturated Absorption vs. Unconfined Freezing and Thawing

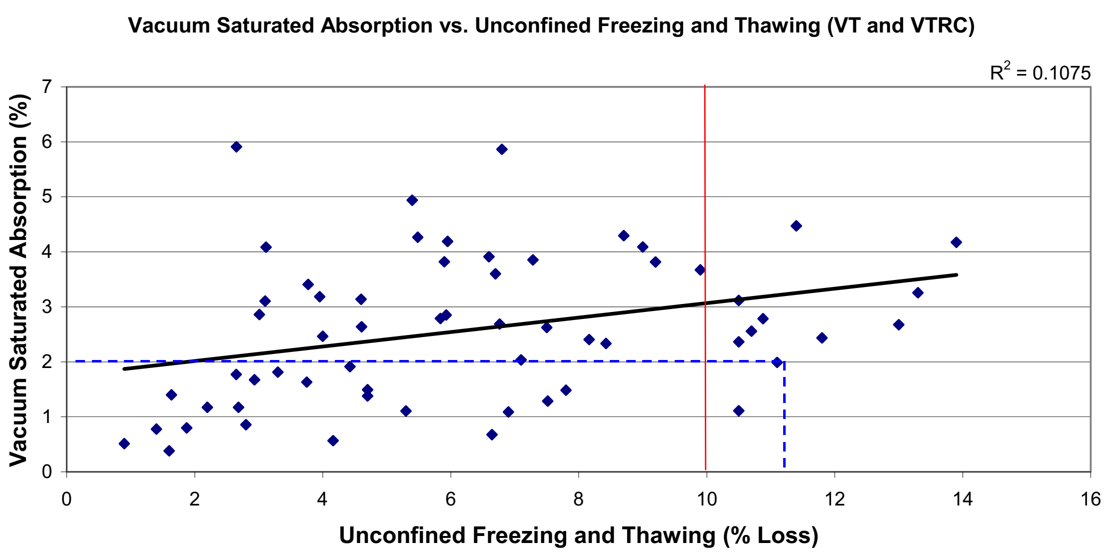


Figure 6.10.2 – Vacuum Saturated Absorption vs. Unconfined Freezing and Thawing (VT and VTRC)

There is no apparent relationship between VSA and unconfined freezing and thawing loss. It should also be noted that with a failure criterion of 10% loss the unconfined freezing and thawing test appears to be too discriminating. From an earlier report conducted on Wisconsin aggregate resources (Brown, 1999) it was determined that an unconfined freezing and thawing test (AASHTO T 103 – Procedure B) should be used instead of the sodium sulfate soundness test because it was "more sensitive" and "more reliable." The reason for using the unconfined freezing and thawing test was to correctly identify poor performing aggregates from the Sinnipee group. Therefore, a failure criterion of a 15% loss would be more appropriate for use in identifying aggregates with a poor performance record. Aggregates with a VSA of less than 2% have an unconfined freezing and thawing loss of less than 11%. The distribution histogram is presented in Figure 6.10.3.

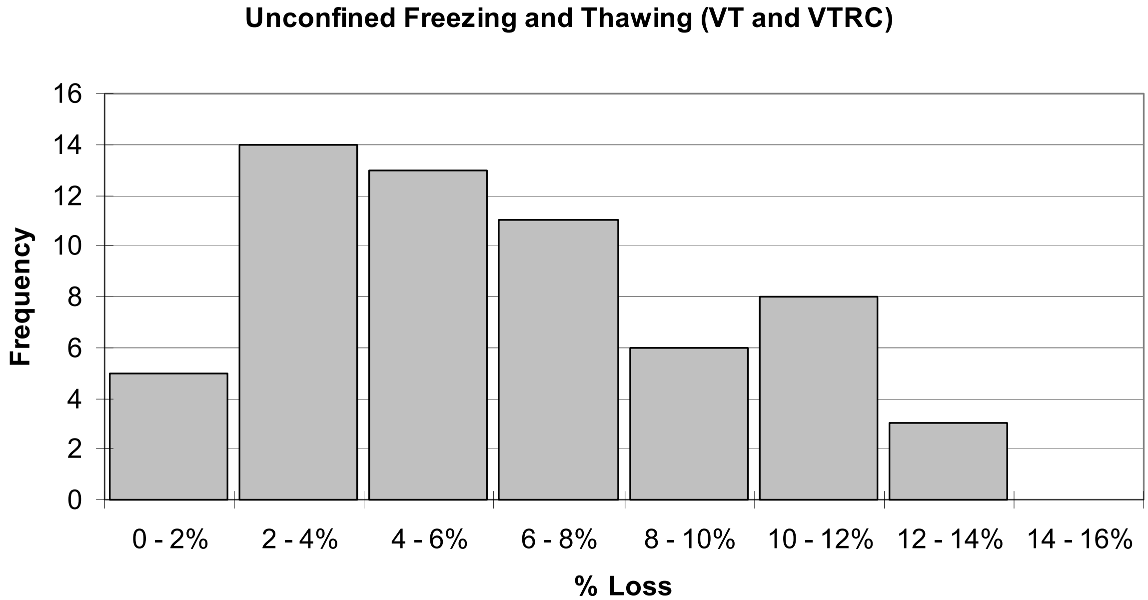


Figure 6.10.3 – Unconfined Freezing and Thawing Distribution Histogram (VT and VTRC)

6.11 Freezing and Thawing in Concrete

The freezing and thawing of concrete specimens containing Wisconsin aggregate was conducted in accordance with ASTM C 666 Method A (Resistance of Concrete to Rapid Freezing and Thawing). The aggregate was saturated by soaking it in water for a period of 24 hours prior to the mixing of the concrete. The concrete was then cured in lime-saturated water for 28 days prior to testing. Table 6.11.1 presents the results of the freezing and thawing testing and notes on the deterioration mode. Figure 6.11.1 illustrates the relationship between absorption and percent reduction in fundamental transverse frequency over 300 cycles and Figure 6.11.2 presents the absorption vs. initial fundamental transverse frequency data. All data points shown represent the average of three specimens. Batch quantities for the concrete specimens made are presented in Appendix B.

Sample	Fundamental Transverse Frequency (Hz)	% Reduction in FTF (after 300 freezing and thawing cycles)	Deterioration Details	% Lightweight	% Chert
12	1.569	30.0%	Large popouts and substantial transverse cracking of specimen resulting from chert particles.	2.4	0.6
22	1.554	20.4%	Large popouts and substantial transverse cracking of specimen resulting from chert particles.	5.2	1.8
31	1.560	8.3%	Small - Moderate size popouts resulting from chert particles.	8.4	1.8
36	1.699	0.0%	No Damage	0.0	0.0
39	1.817	5.1%	No Damage	0.0	0.0
42	1.473	5.8%	Very few popouts resulting from chert particles.	5.5	1.2
50	1.455	3.0%	Non-durable reddish-brown aggregates.	16.2	0.0
55	1.572	7.1%	Small popouts resulting from chert particles.	4.6	1.6
60	1.690	9.4%	Several large popouts and D-cracking resulting from chert particles.	3.1	2.5

Table 6.11.1 – ASTM C 666 Results

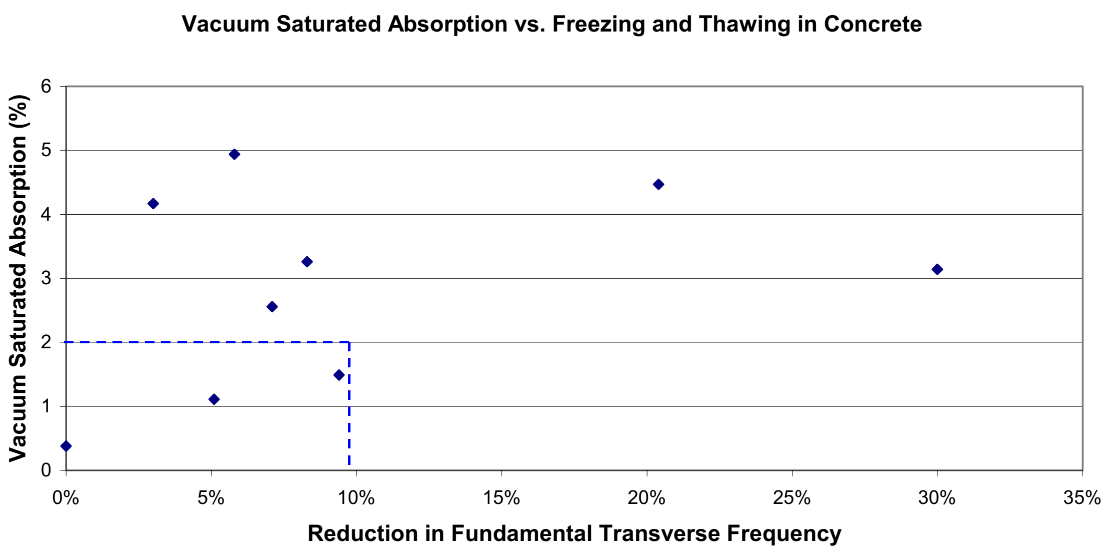


Figure 6.11.1 - VSA vs. Reduction in Fundamental Transverse Frequency

As shown in Table 6.11.1 it appears that the amount and type of deterioration depends directly on the amount of chert present in the sample. Those aggregate samples containing chert showed higher levels of deterioration than those not containing chert. There appears to be no relationship between VSA and the % reduction in fundamental transverse frequency for the aggregate samples tested. All of the specimens had %

reductions lower than the 40% reduction failure criterion, which is recommended in ASTM C 666. The conclusion can be made that for the aggregates tested, those with VSA of less than 2% will have % reductions of less than 10% and initial fundamental transverse frequencies greater than 1.675 Hz. This test does not reject any of the aggregates tested based on the recommended failure criterion, but the deterioration observed throughout the test i.e. popouts and disintegration of aggregate particles, may eliminate a particular aggregate for use in specific applications. Plots of the fundamental transverse frequency as a function of the number of cycles for each sample tested are presented in Appendix E.

6.12 Sodium Sulfate Soundness and Unconfined Freezing and Thawing Test Comparison

A comparison of Sodium Sulfate Soundness and Unconfined Freezing and Thawing test results presented in Figures 6.12.1 and 6.12.2 demonstrates that there is no apparent relationship between the two tests. This indicates that although the tests are designed to measure the freezing and thawing resistance of the aggregate, that in reality this may not be the case. The Sodium Sulfate Soundness test may be measuring some other soundness property.

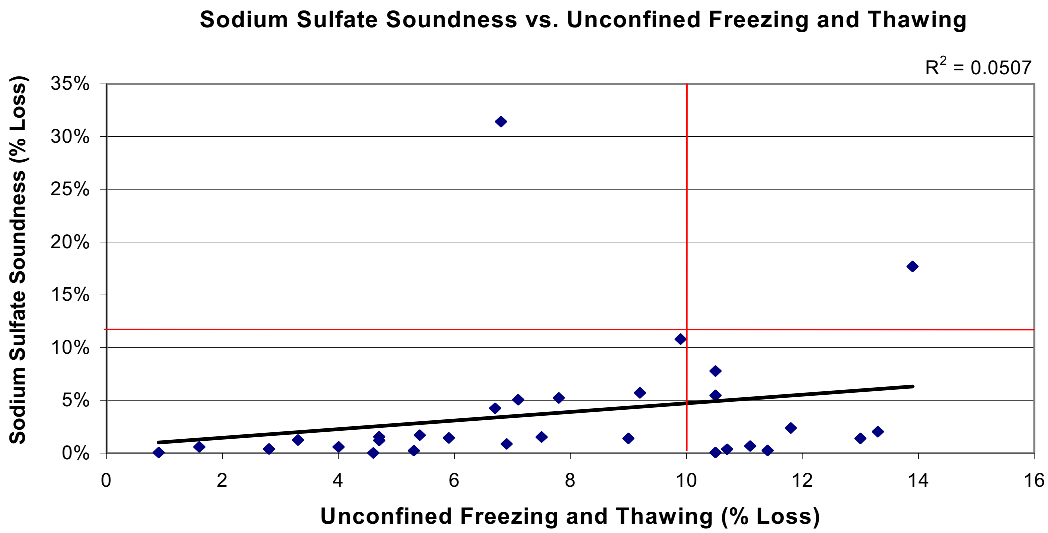


Figure 6.12.1 – Sodium Sulfate Soundness vs. Unconfined Freezing and Thawing (VT)

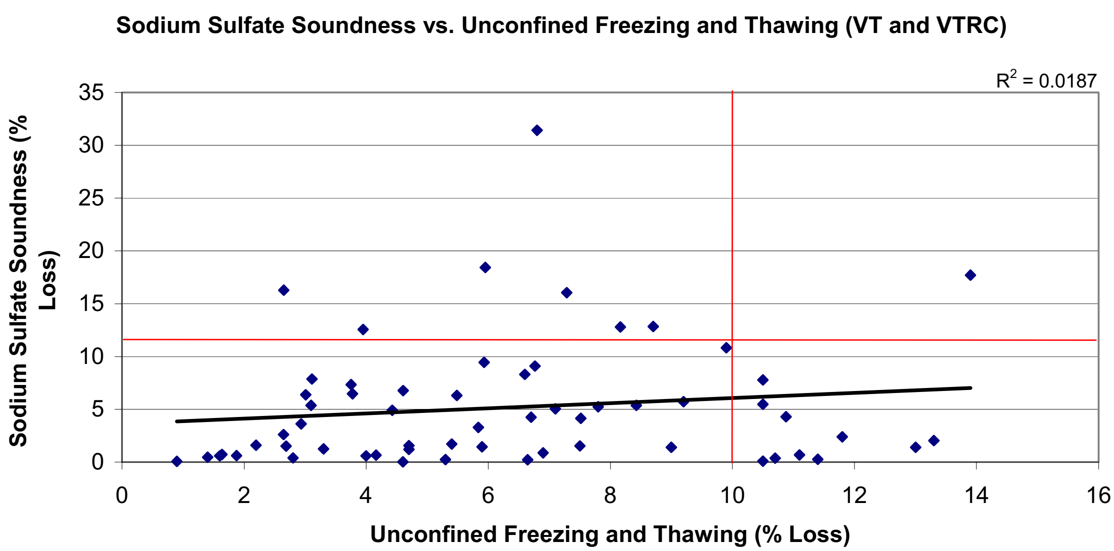


Figure 6.12.2 – Sodium Sulfate Soundness vs. Unconfined Freezing and Thawing (VT and VTRC)

6.13 Recycled/Reclaimed Aggregate Test Results

The results for the testing conducted on the recycled concrete and slag aggregates is presented in Table 6.13.1

Recycled/Reclaimed Aggregate Test Results							
Material	VSA (%)	VSSG	L.A. Abrasion (% Loss)	Micro-Deval (% Loss)	ACV (% Loss)	Sodium Sulfate Soundness (% Loss)	Unconfined Freezing and Thawing (% Loss)
RCA	5.62	2.42	38.8	19.0	24.1	24.1	5.7
Slag	1.06	2.74	26.8	7.7	---	1.2	4.6

Table 6.13.1 – Recycled/Reclaimed Aggregate Test Results

6.13.1 Recycled Concrete Pavement (RCP)

RCP shows higher levels of deterioration in regards to strength and abrasion testing, but it meets the acceptance criterion for every test except for the Micro-Deval test. The RCP sample failed the Micro-Deval test by a loss of only 1%. As expected, the RCP failed the Sodium Sulfate Soundness test by a considerable margin due to the fact that the sodium sulfate reacts with the cement paste and causes it to deteriorate. The unconfined and confined freezing and thawing tests indicate that this RCP is durable. The unconfined freezing and thawing and confined freezing and thawing tests would assess the durability of RCP as base course and in concrete, respectively. From the test data it appears that RCP would be acceptable for use in the unbound state or in a low-grade concrete. The VSA is high and the VSSG is low due to the structure of the hydrated cement paste that remains adhered to the coarse aggregate. These parameters must be accounted for when designing a concrete mixture containing RCP.

6.13.2 Foundry Slag Aggregate

The L.A. Abrasion, Micro-Deval, Sodium Sulfate and Unconfined Freezing and Thawing tests were conducted on a 40/60, slag/gravel aggregate mixture. The Confined Freezing and Thawing test was conducted with a saturated 40/60, slag/crushed stone mix with a crushed stone known to be of good quality. This was done to ensure that any degradation that occurs during the freezing and thawing testing is the result of the slag aggregate. The VSA and VSSG test results presented are for the slag only. From the test results obtained it appears that slag aggregate will perform at least as well as natural aggregate and in many cases better than natural aggregate.

The slag used is crushed to a small size, approximately 3/8 in., therefore requiring that it be mixed with a larger natural aggregate. It appears that the performance of a slag/gravel coarse aggregate mixture will depend primarily on the durability of the natural aggregate fraction.

6.13.3 Recycled Asphalt Pavement

The Tensile Strength Ratio's (TSR) of the RAP samples were determined using AASHTO T 283. An analysis of the physical properties of the aggregate was also conducted for use in determining the susceptibility of the asphalt pavement to rutting. The test results are presented in Tables 6.13.2 and 6.13.3. The only data that is recommended for use as an aggregate durability indicator is the TSR data. The TSR of the RAP mix should be greater than 0.80 to ensure durable pavements. The test accounts for the effects of the RAP aggregate as well as the binder on the total mix. If a non-durable aggregate is present in the RAP it will be reflected in a low TSR value.

						Tensile Strength Ratio		
Sample #	% Flat and Elongated Particles (3 to 1)	% Flat and Elongated Particles (5 to 1)	Fine Agg. Angularity	Coarse Agg. Angularity (1 or more)	Coarse Agg. Angularity (2 or more)	Virgin Mix	RAP Mix	RAP Asphalt Content
72	9.6	0.7	42.9	90.1	87.9	0.74	0.86	5.64
74	3.8	0.2	49.1	83.8	74.3	0.74	0.87	7.81

Table 6.13.2 – Recycled Asphalt Pavement Test Results

As shown in Table 6.13.2, both RAP samples have TSR values greater than 0.8. Therefore, it can be concluded that both of the samples consist of durable aggregates.

	Sieve Size (Percent Passing)										
Sample #	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
72	100	100	100	96.3	88.2	68.2	54.3	44.9	34.9	19.5	11.5	7.7
74	100	100	97.3	82	70.5	50.7	38.5	30.5	24.2	19.2	14.7	9.1

Table 6.13.3 – Recycled Asphalt Pavement Gradations

CHAPTER 7: CONCLUSIONS

1. The absorption of an aggregate, while not directly related to the quality of the aggregate, can still be used as a preliminary indicator of durability. It is further noted that aggregates with vacuum saturated absorptions of less than 2% will meet the durability requirements of the L.A. Abrasion test, Micro-Deval test, Sodium Sulfate test, and Unconfined Freezing and Thawing test.
2. ASTM C 123 (Lightweight Particles in Aggregate) is not able to characterize the durability of an entire aggregate sample, but it is able to determine the percentage of lightweight particles present in the aggregate. Lightweight particles tend to be non-durable and will lead to the early deterioration of concrete and bituminous pavements. Most notably, low-density chert, a lightweight aggregate will cause popouts in concrete.
3. The L.A. Abrasion test was only able to identify the very worst aggregate sample as being poor. This indicates that the L.A. Abrasion test does have some ability to predict field performance, but that the loss limitations may not be stringent enough. From this data it can be concluded that the L.A. Abrasion test cannot directly predict the overall performance of an aggregate, but it can accurately estimate a key parameter, aggregate strength.
4. The Micro-Deval test (AASHTO TP 58) with the recommended maximum allowable loss limit of 18% rejects nearly 50% of the aggregates tested. For Wisconsin aggregate in general it appears that a maximum allowable loss limit of 25-30% is more reasonable. However, for some applications where highly durable aggregates are essential, i.e. concrete pavement, a more discriminating limit of 13-18% would be appropriate.
5. The Aggregate Crushing Value test results (BS 812-110) are related to the L.A. Abrasion test. This suggests that the tests are measuring the same property, aggregate strength.
6. The Sodium Sulfate Soundness test (ASTM C 88) is able to identify several poor performing aggregates, but it has been suggested that it cannot accurately identify a poor performing group of aggregates, that being the Sinnipee group. The Sodium Sulfate Soundness test is also highly variable. The multi-laboratory coefficient of variation is 41% and the single-operator CV is 24%.
7. The Unconfined Freezing and Thawing test (A23.2-24A) data has no correlation with Sodium Sulfate Soundness results or VSA. The test rejected 30% of all aggregates tested. Therefore, it is more appropriate to set a failure criterion of 15% loss rather than a 10% loss recommended by the Canadian unconfined freezing and thawing test, which will guarantee that only very non-durable aggregates will fail the test.
8. The Resistance of Concrete to Rapid Freezing and Thawing test did not reject any of the aggregate samples tested, but by observing the deterioration modes of the concrete specimens conclusions can be drawn about the aggregate used. Most notably, certain aggregate used disintegrated, caused popouts, or cracking in the concrete in which they are contained.
9. Recycled Concrete Pavement appears to be able to be tested using the same durability tests used for natural aggregates. The Sodium Sulfate test is unacceptable for use as a

durability indicator for RCP because the sulfate reacts with the cement paste to cause excessive deterioration.

10. Foundry slag aggregate performed well in all of the testing that was conducted, although it is noted that the test results may vary widely depending on the quality of the natural aggregate with which it is mixed.
11. Recycled Asphalt Pavement Aggregates can be tested using AASHTO T 283.Those aggregate samples with TSR values greater than 0.80 should result in durable pavements. This test can be used for aggregates that will be used as a base course material as well aggregates that will be used in bituminous pavements. To ensure the overall durability and resistance to rutting of a bituminous pavement, additional aggregate testing is required.

CHAPTER 8: RECOMMENDATIONS

1. Aggregates with vacuum saturated absorptions of less than 2% do not need to be tested for L.A. Abrasion loss, Micro-Deval loss, Unconfined or Confined Freezing and Thawing tests. They will still, however, need to be tested for Lightweight Pieces in Aggregate.
2. The inclusion of ASTM C 123 (Lightweight Pieces in Aggregate) in the WisDOT aggregate durability testing protocol is necessary in order to quantify non-durable lightweight aggregate percentages. This is important particularly for gravel resources. A limit of 5% lightweight material is recommended, although it is not recommended that an aggregate sample be immediately rejected if it has percentages of lightweight material greater than 5%. If aggregates have greater than 5% lightweight material and they are intended for use in the bound state they should be tested for durability in the appropriate material, concrete or asphalt. Aggregates to be used in concrete pavements or structures should be tested using ASTM C 666 (Freezing and Thawing in Concrete). A failure criterion of a 40% reduction in the natural transverse frequency should be used. Additionally, concrete samples should be inspected visually for popouts and aggregate deterioration. Excessive popouts or aggregate deterioration may be reason for rejection of the aggregate at the engineer's discretion. Aggregates to be used in asphalt should be tested in accordance with AASHTO T 283 (Resistance of Asphalt to Moisture Induced Damage). A failure criterion of asphalt samples with TSR's less than 0.80 is recommended.
3. The L.A. Abrasion test should continue to be used to evaluate aggregate strength. Noting that there is a linear relationship between L.A. Abrasion loss at 100 cycles and loss at 500 cycles it is recommended that the test be run only for 500 cycles and then any necessary information interpolated from that data. Realizing that WisDOT has historically used the L.A. Abrasion test, and that there appears to be no value in replacing the L.A. Abrasion test with the Aggregate Crushing test, there is no reason to change testing procedures.
4. The Micro-Deval test should be added to WisDOT testing protocol to evaluate the abrasion resistance of aggregate. This test more accurately models the degradation that occurs during handling and mixing.
5. The Sodium Sulfate Soundness test should be replaced by the Unconfined Freezing and Thawing test.   A failure criterion of 15% loss is recommended for the Unconfined Freezing and Thawing test.
6. The Freezing and Thawing of Concrete test is recommended for aggregate that are to be used in the bound state. This test helps to identify non-durable aggregates that may result in aggregate disintegration, popouts, or cracking of concrete.
7. RCP should be tested using the same testing procedures used for natural aggregates. The high VSA and low VSSG of RCP must be considered if it is intended for use in the bound state.
8. Foundry slag aggregate should be tested using the same testing procedures used for natural aggregates. Additionally, it is recommended that slag sources be tested for iron unsoundness and calcium disilicate unsoundness prior to initial acceptance. These additional tests are not recommended for use on a regular basis due to the rarity of the unsoundness problems, but if there is a significant change in the slag

production process or evidence of unsoundness is found, the tests should be conducted.

9. RAP samples should be tested in accordance with AASHTO T 283 (Resistance of Asphalt to Moisture Induced Damage). RAP samples with TSR's less than 0.80 should be rejected due to durability issues.
10. Proposed testing protocols for unbound and bound aggregates are presented in Figures 8.1, 8.2, and 8.3. These protocols, if followed, would greatly reduce the aggregate durability testing required by WisDOT.
11. It is recommended that further research be conducted relative to the use of the Canadian unconfined freezing and thawing test. Increasing the number of freezing and thawing cycles from 5 to 15 or 25 may improve the ability of the test to differentiate between materials of different quality.

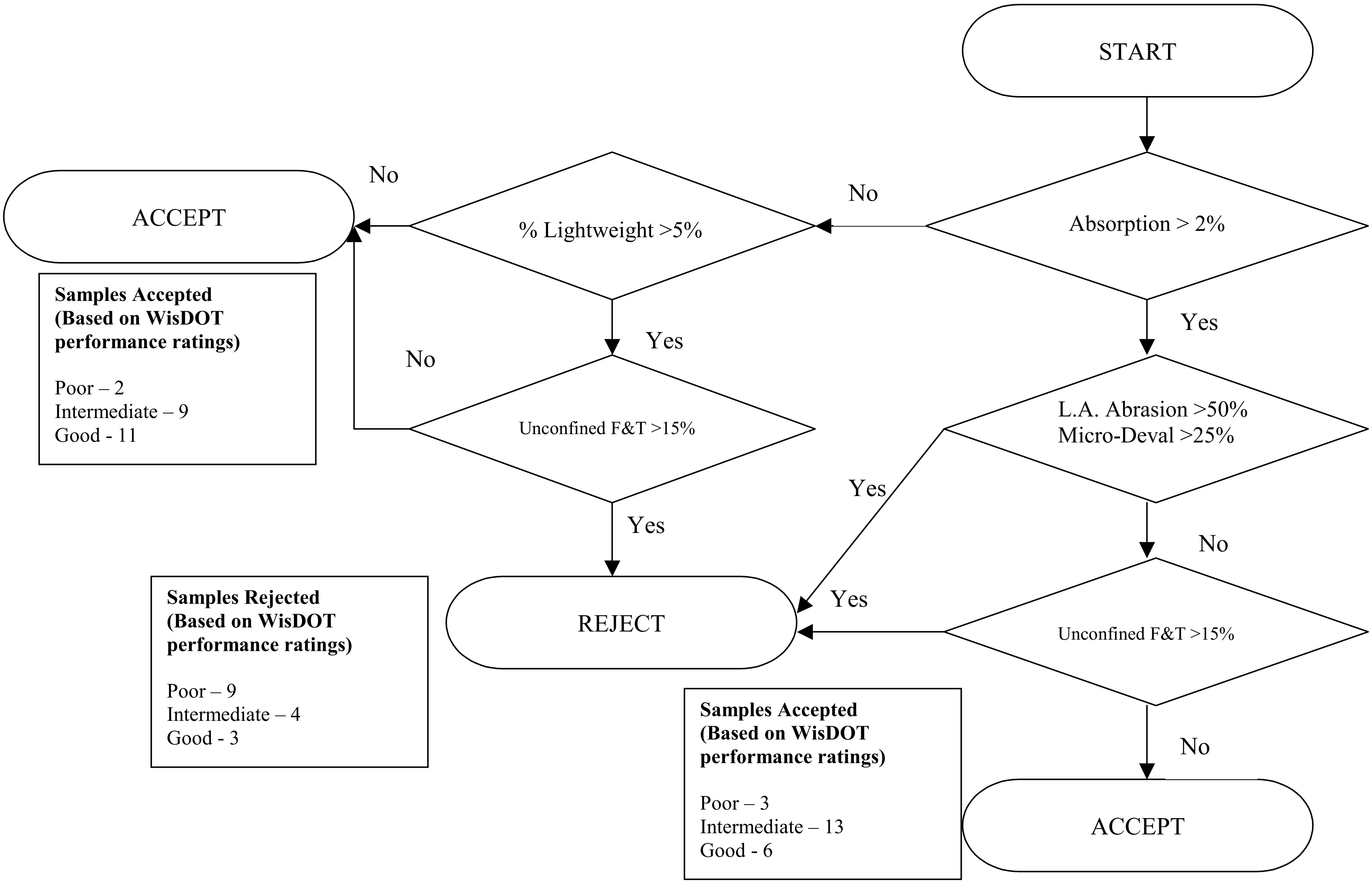


Figure 8.1 – Aggregate Durability Testing Flowchart for Unbound Aggregate

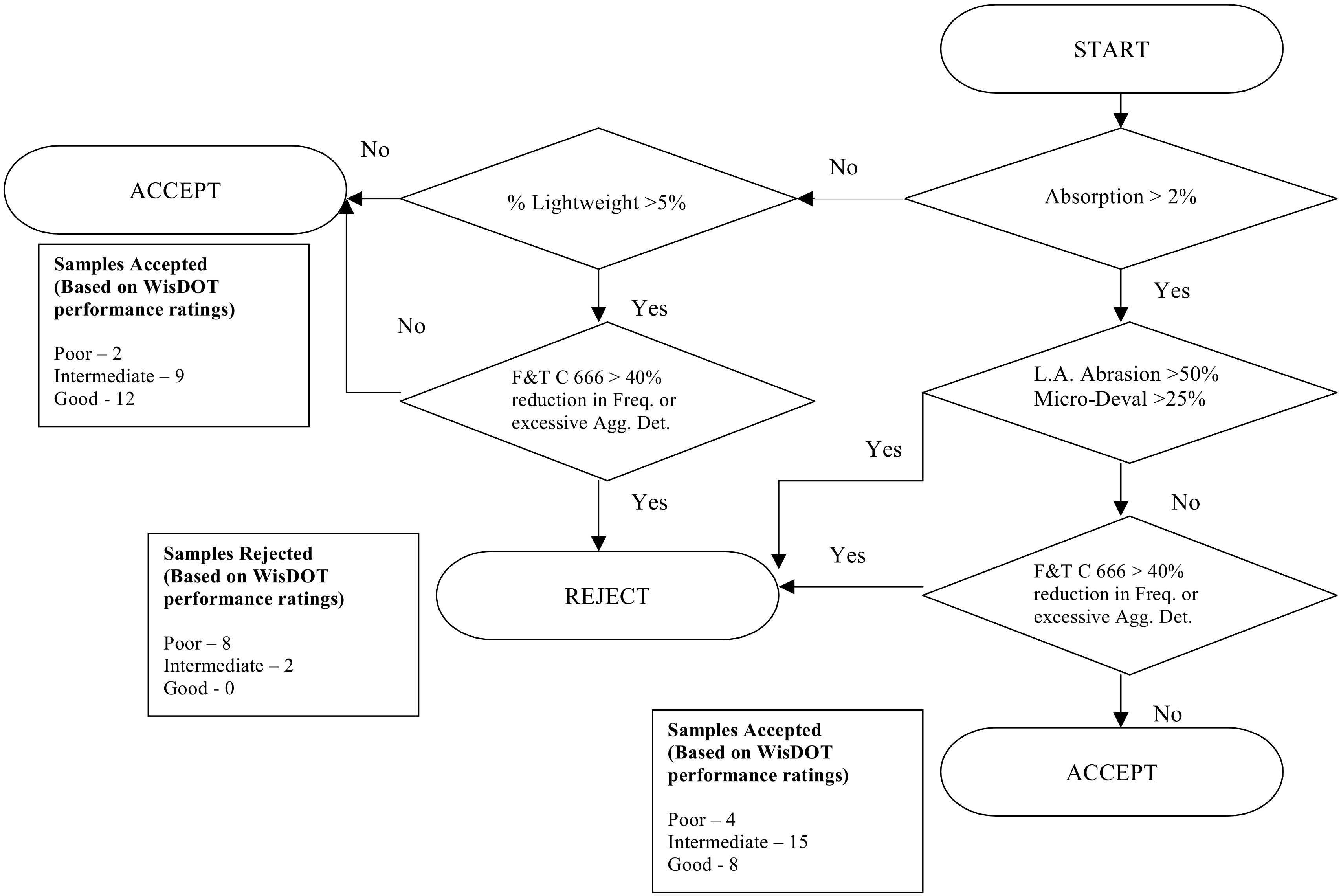


Figure 8.2 – Aggregate Durability Testing Flowchart for Concrete Aggregates

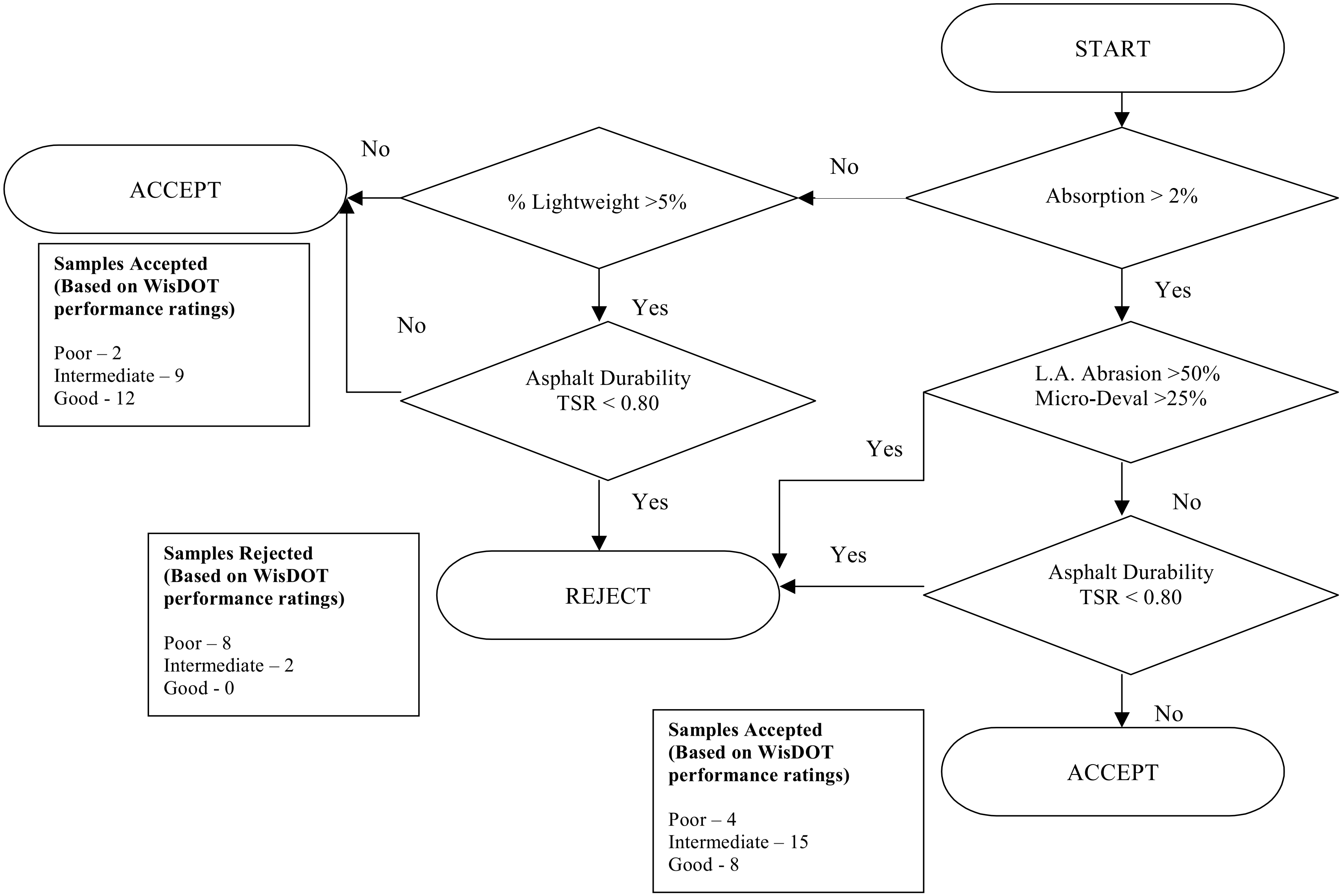


Figure 8.3 –Aggregate Durability Testing Flowchart for Bituminous Aggregates

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APPENDIX A

Analysis of WisDOT Aggregate Test Results Database

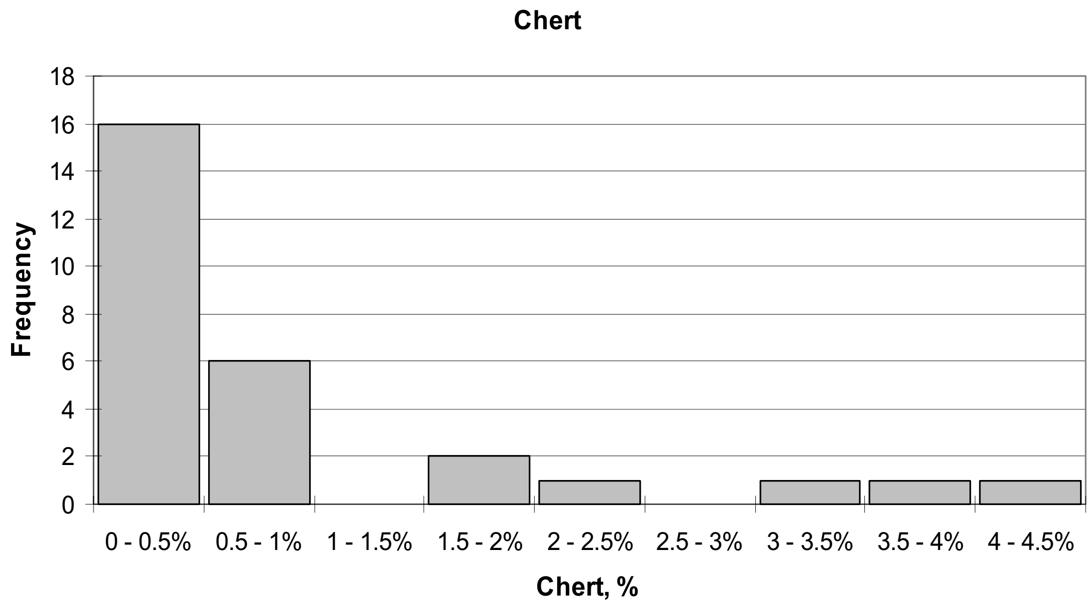

Figure A-1 – % Chert Distribution Histogram (WisDOT Database)

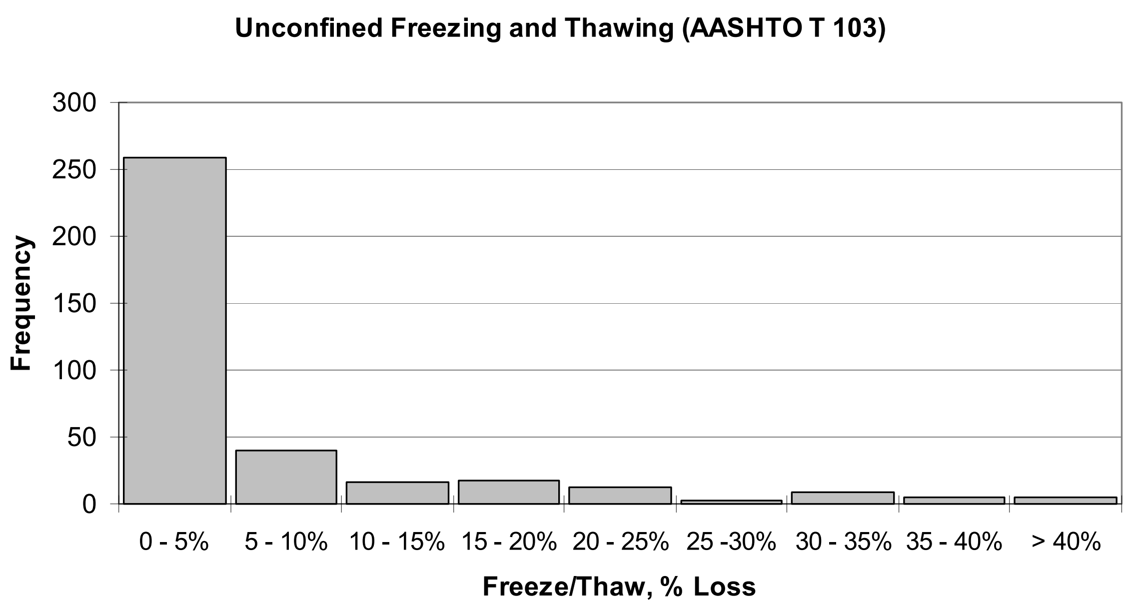

Figure A-2 – Unconfined Freezing and Thawing (AASHTO T 103) Distribution Histogram (WisDOT Database)

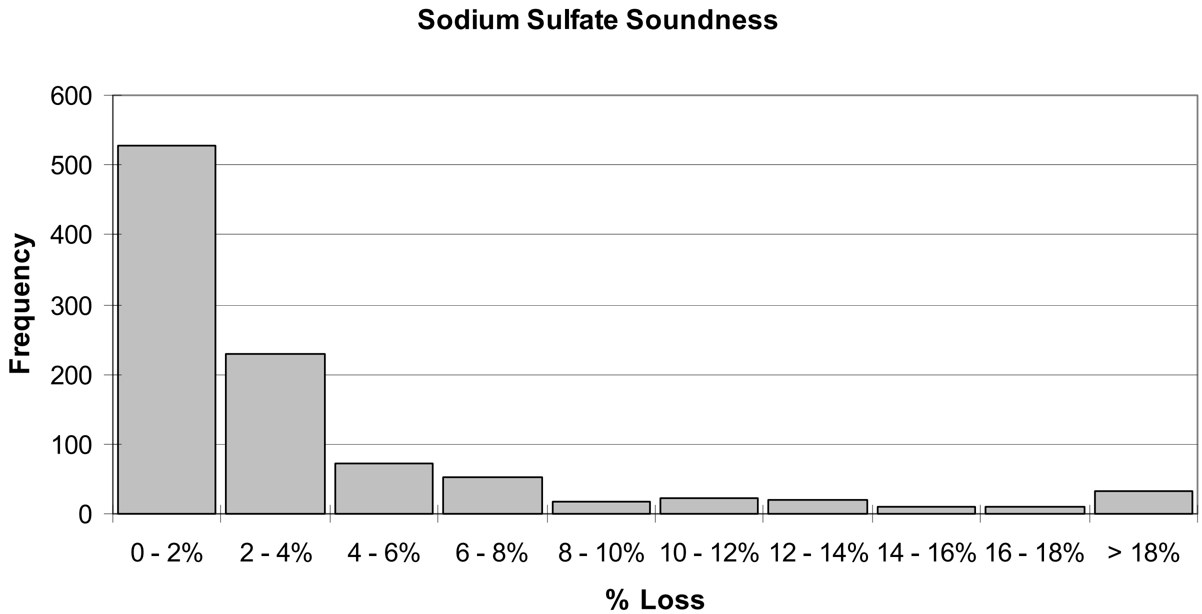

Figure A-3 – Sodium Sulfate Soundness Distribution Histogram (WisDOT Database)

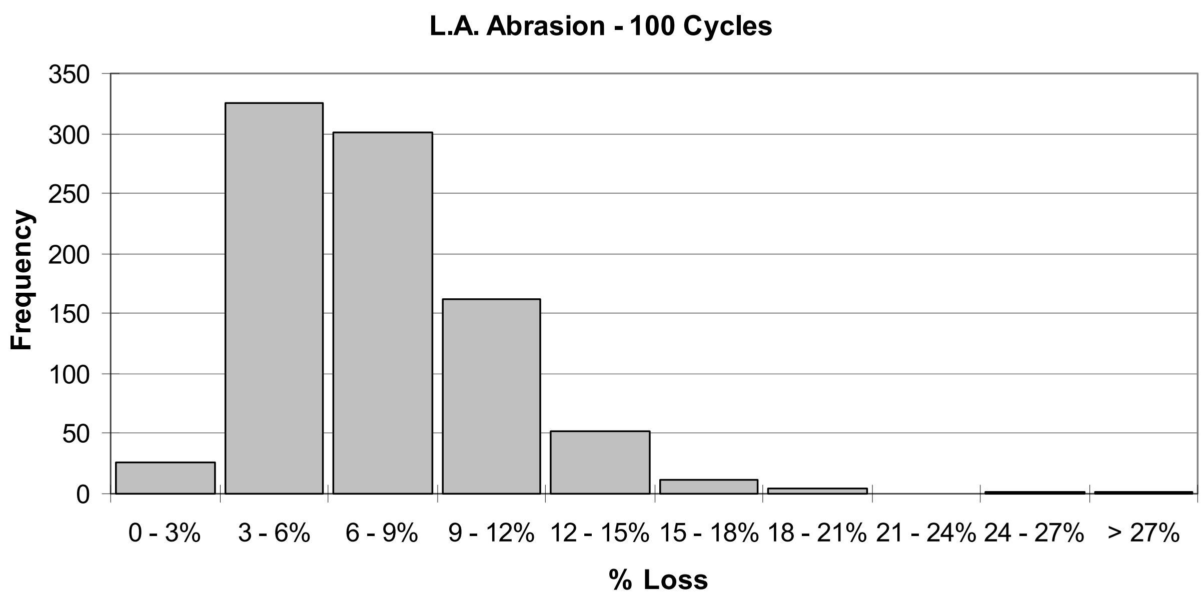

Figure A-4 – L.A. Abrasion Distribution Histogram – 100 Cycles (WisDOT Database)

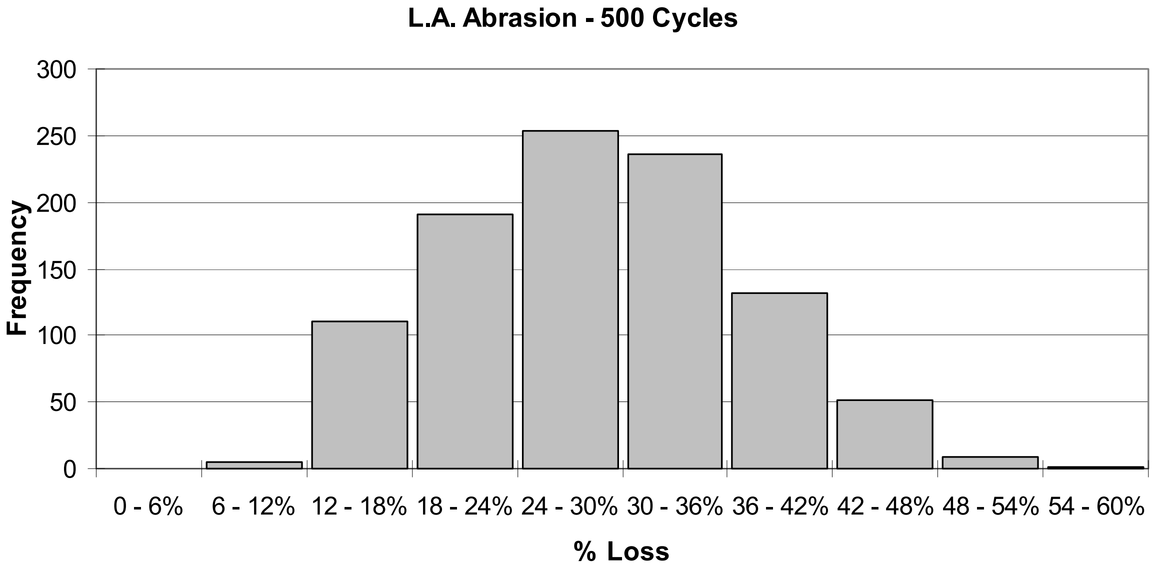

Figure A-5 – L.A. Abrasion Distribution Histogram – 500 Cycles (WisDOT Database)

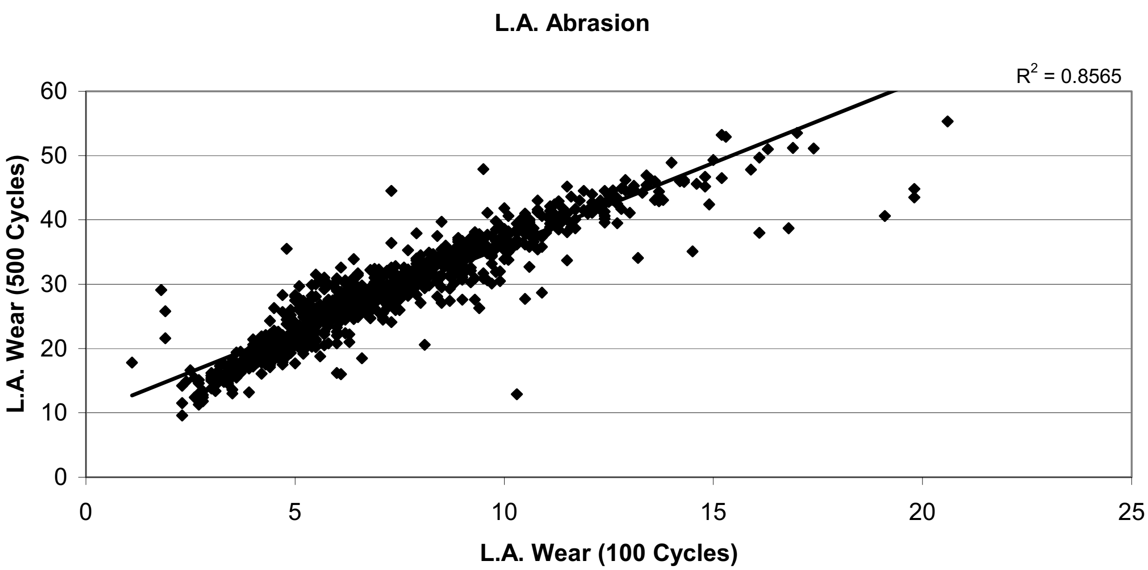

Figure A-6 – L.A. Abrasion 500 Cycles vs. L.A. Abrasion 100 Cycles (WisDOT Database)

APPENDIX B

Concrete Batch Quantities

Batch Quantities per cubic yard											
Sample No.	**36**	**42**	**12**	**31**	**22**	**60**	**55**	**50**	**39**	**71**	**73**
Maximum Nominal Size, in	1"	1"	3/4"	3/4"	¾"	1"	3/4"	3/4"	1"	1.5"	3/4"
Coarse Aggregate, #	1715	1751	1721	1713	1658	1282	1760	1777	1822	1479	1068
Slag Aggregate, #	0	0	0	0	0	0	0	0	0	0	712
Fine Aggregate, #	1195	1050	1112	1092	1122	1823	1277	1183	1168	1297	1125
Cement, #	377	377	391	391	391	328	328	328	377	349	391
Slag Cement, #	251	251	260	260	260	219	219	219	251	243	260
Water, #	302	254	280	281	280	262	253	256	270	275	288
AEA, ml	11.6	11.6	12	12	12	10	10	10	11.6	11.6	12
w/c ratio	0.48	0.41	0.43	0.43	0.43	0.48	0.46	0.47	0.43	0.47	0.44

Table B-1 – Concrete Batch Quantities

APPENDIX C

Test Results

TEST RESULTS							
Sample #	Absorption	Micro-Deval	L.A. Abrasion	ACV	Sodium Sulfate	Unconfined Freezing and Thawing	% Lightweight
1	2.69	15.76	19.14	19.36	9.10	6.8	0.5
2	5.87	39.98	56.88	26.96	31.42	6.8	12.9
3	3.85	21.62	28.19	21.26	16.05	7.3	0.0
4	0.51	5.09	29.96	19.18	0.06	0.9	0.0
5	4.19	28.66	44.47	29.46	18.43	5.9	0.4
6	1.38	8.88	21.37	15.03	1.55	4.7	0.0
7	4.30	23.70	44.10	25.85	12.83	8.7	0.7
8	3.67	24.92	35.87	20.14	10.82	9.9	2.3
9	3.91	26.08	37.84	28.22	8.30	6.6	2.1
10	2.46	------	------	------	0.60	4.0	0.0
50	4.17	30.52	35.07	18.20	17.70	13.9	16.2
51	5.91	38.18	38.89	29.32	16.28	2.7	4.4
52	3.81	25.25	28.01	------	5.72	9.2	7.8
53	2.41	22.75	18.67	19.38	12.80	8.2	7.1
11	3.82	19.62	32.58	24.91	1.45	5.9	3.4
12	3.14	14.38	28.11	17.51	0.03	4.6	2.4
13	2.04	16.03	34.71	22.04	5.05	7.1	0.5
14	0.86	6.39	29.14	21.32	0.39	2.8	0.5
15	0.68	3.42	15.21	21.43	0.21	6.6	0.0
16	1.48	------	27.70	19.34	5.23	7.8	0.5
17	1.29	14.35	19.95	16.49	4.15	7.5	0.0
18	2.64	22.97	28.47	18.54	6.78	4.6	4.9
19	2.68	16.77	31.24	19.06	1.40	13.0	1.3
20	2.85	20.00	21.57	24.88	9.45	5.9	0.1
21	1.11	9.31	21.77	15.65	0.24	5.3	0.5
22	4.47	26.50	45.10	26.82	0.27	11.4	5.2
23	3.19	18.26	38.06	27.73	12.56	4.0	9.4
24	4.09	23.84	40.33	26.87	1.40	9.0	2.1
25	4.26	18.48	23.05	21.70	6.30	5.5	2.9
26	2.79	13.71	25.25	24.84	3.28	5.8	1.8
27	2.79	16.11	18.67	20.77	4.29	10.9	3.9
28	1.91	16.77	26.52	23.41	4.90	4.4	2.8
29	3.41	21.68	29.25	24.81	6.46	3.8	2.6
30	2.86	16.32	23.07	21.51	6.37	3.0	1.2
55	2.56	15.35	30.84	18.67	0.37	10.7	4.6
57	3.12	25.54	22.81	------	7.78	10.5	9.1
58	1.68	11.98	17.70	17.32	3.61	2.9	3.1
59	2.62	16.61	25.85	17.32	1.53	7.5	9.2
60	1.49	8.98	23.34	14.50	1.20	4.7	3.1
61	1.63	11.08	25.67	17.50	7.34	3.8	2.5
31	3.26	20.53	40.16	22.81	2.03	13.3	8.4
32	1.17	8.15	19.10	13.52	1.60	2.2	0.6
33	2.36	14.61	19.25	15.70	5.47	10.5	0.5
34	0.57	4.53	9.89	20.42	0.67	4.2	0.0
35	1.40	10.48	21.80	18.84	0.71	1.6	0.0
36	0.38	3.76	13.57	11.39	0.59	1.6	0.0
37	0.80	5.47	13.20	15.85	0.61	1.9	0.0
39	1.11	8.34	22.00	15.56	0.08	10.5	0.0
40	1.99	10.19	28.31	14.98	0.68	11.1	3.9
41	3.10	18.40	35.24	24.99	5.38	3.1	0.2
42	4.94	22.52	41.56	24.41	1.71	5.4	5.5
43	4.08	23.13	37.56	24.72	7.86	3.1	0.9
44	0.78	5.76	13.58	15.55	0.45	1.4	0.3
46	2.34	13.69	17.45	21.29	5.36	8.4	0.2
48	3.60	23.57	38.83	24.44	4.25	6.7	4.8
62	1.77	7.26	15.88	17.30	2.61	2.6	2.5
63	1.17	9.53	12.47	15.49	1.51	2.7	1.4
64	1.81	11.24	19.57	14.58	1.24	3.3	5.4
68	1.09	10.20	33.46	18.73	0.87	6.9	1.0
69	2.44	21.16	35.65	19.13	2.39	11.8	0.5

Table C-1 – Test Results

APPENDIX D

Sieve Analyses

Sieve Anayses						
	Percent Retained on Sieve Sizes					
Sample #	> 1 1/2"	1 1/2" - 1"	1" - 3/4"	3/4" - 1/2"	1/2" - 3/8"	3/8" - #4
1	0.0	0.0	1.6	36.0	24.0	38.4
2	0.0	0.2	17.2	43.2	24.0	15.4
3	0.0	0.0	10.3	42.6	23.9	23.2
4	20.5	61.6	17.3	0.6	0.0	0.0
5	0.0	3.6	15.2	33.0	18.4	29.8
6	0.0	0.3	12.9	28.5	17.4	40.8
7	0.0	5.3	21.6	36.1	15.3	21.7
8	0.0	0.3	21.5	49.8	15.6	12.7
9	0.0	6.0	17.3	31.5	16.5	28.7
10	1.7	40.1	42.3	12.6	1.6	1.7
11	0.0	0.0	1.5	29.7	27.5	41.4
12	0.0	0.0	12.0	84.5	2.6	1.0
13	0.0	4.5	21.5	31.5	17.5	25.0
14	0.0	0.4	10.6	40.0	22.5	26.5
15	0.0	0.0	7.0	54.0	22.6	16.3
16	0.0	0.0	58.8	39.5	1.5	0.2
17	0.0	0.0	1.0	55.9	26.5	16.5
18	0.0	39.1	17.0	17.5	10.2	16.2
19	0.0	1.8	15.8	45.5	19.3	17.7
20	0.0	0.0	6.7	26.1	18.0	49.3
21	0.0	17.4	27.1	48.1	6.8	0.6
22	0.0	0.0	4.7	45.5	23.6	26.1
23	0.0	7.9	28.7	34.3	14.1	15..1
24	0.0	2.8	21.0	41.0	16.9	18.4
25	0.0	1.7	12.0	52.1	22.1	12.1
26	0.0	33.6	52.6	11.6	1.1	1.0
27	0.0	1.0	10.7	28.3	25.2	34.9
28	0.0	2.1	21.7	38.9	15.1	22.2
29	0.0	14.7	31.3	31.3	9.9	12.8
30	0.0	0.0	10.8	34.0	17.7	37.4
31	0.0	0.0	3.8	68.8	21.4	6.1
32	0.0	0.0	1.9	40.1	44.9	13.1
33	0.0	0.7	15.4	39.8	18.7	25.4
34	0.0	26.9	8.4	35.3	16.3	13.0
35	0.0	7.9	22.7	26.7	13.9	28.8
36	0.0	0.5	23.6	49.0	19.9	7.0
37	0.0	0.0	6.4	42.9	23.9	26.8
39	0.0	1.5	9.5	49.8	17.6	21.6
40	0.0	0.0	0.1	27.9	26.5	45.4
41	0.0	10.2	15.6	36.7	15.6	21.8
42	0.0	3.7	32.2	37.0	14.1	13.0
43	0.0	7.6	23.5	32.3	14.7	21.9
44	0.0	1.0	10.9	36.3	20.4	31.4
46	0.0	5.7	18.4	27.3	15.1	33.4
48	0.0	5.3	13.1	27.1	18.4	36.1
50	0.0	0.0	0.0	42.9	33.5	23.6
51	0.0	0.0	12.0	31.8	19.2	37.0
52	0.0	0.0	14.2	33.4	18.3	34.2
53	0.0	0.0	30.9	47.9	11.4	9.9
55	0.0	0.0	0.0	34.9	51.7	13.4
57	0.0	0.0	16.6	33.0	18.9	31.5
58	0.0	0.0	13.5	50.0	16.0	20.4
59	0.0	0.0	18.7	37.0	17.4	26.9
60	0.0	0.0	14.2	79.5	5.0	1.2
61	0.0	0.0	12.2	31.9	21.9	34.0
62	0.0	0.0	9.5	48.8	22.7	19.0
63	0.0	0.0	6.8	54.5	17.1	21.6
64	0.0	1.7	23.8	36.0	16.4	22.1
68	0.0	7.3	20.3	38.8	16.2	17.4
69	0.0	6.6	25.6	35.6	17.0	15.2
71	0.0	11.1	50.9	35.4	2.0	0.5
73	0.0	0.0	3.7	27.2	20.3	48.7

Table D-1 – Sieve Analyses

APPENDIX E

Resistance of Concrete to Rapid Freezing and Thawing Test Results

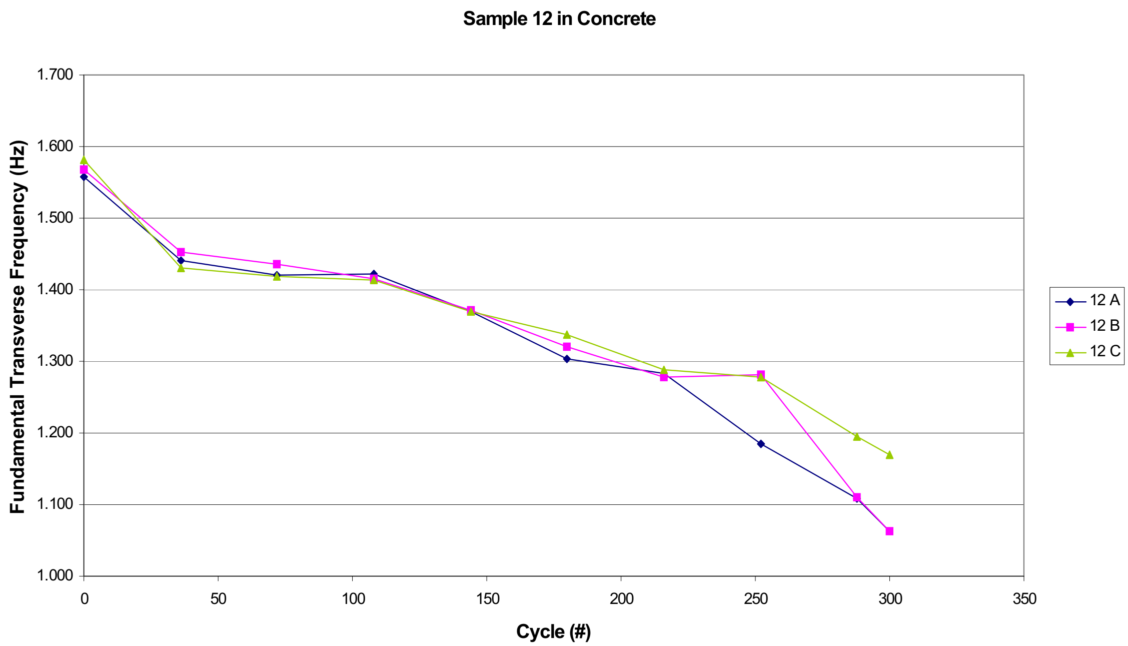

Figure E-1 – Sample 12 (Transverse Frequency vs. Freezing and Thawing Cycle)

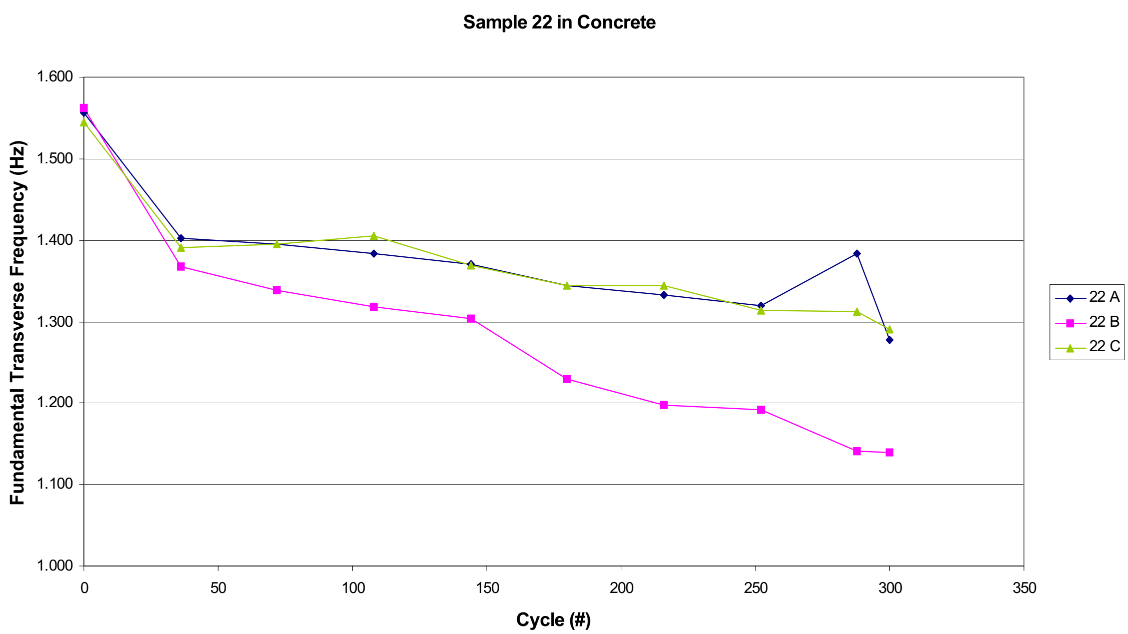

Figure E-2 – Sample 22 (Transverse Frequency vs. Freezing and Thawing Cycle)

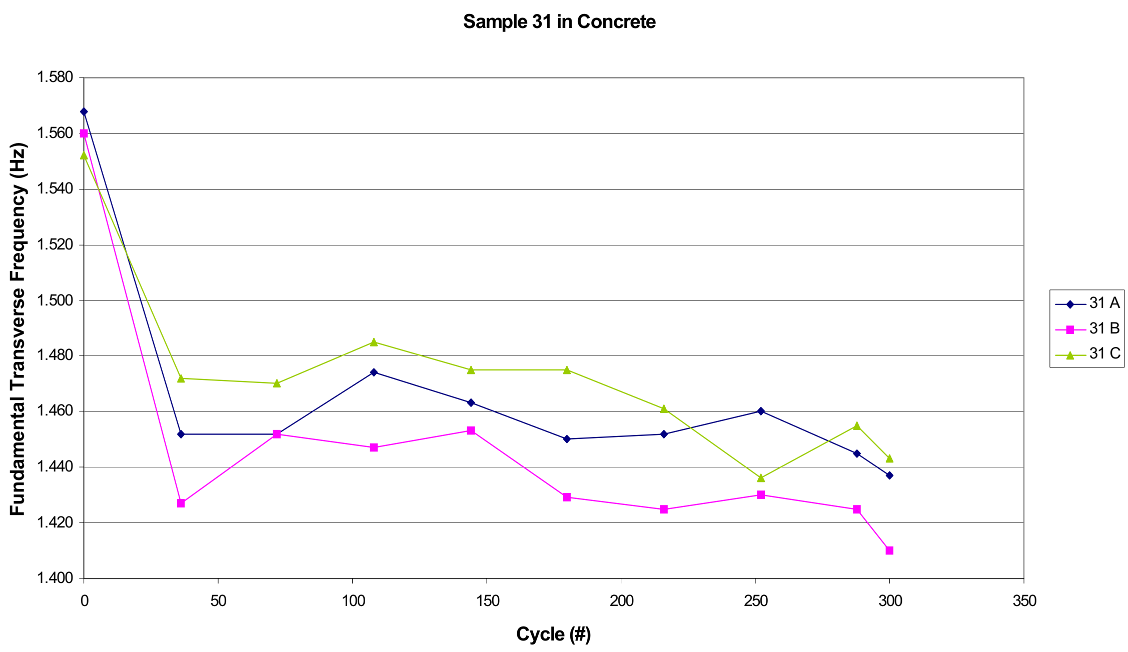

Figure E-3 – Sample 31 (Transverse Frequency vs. Freezing and Thawing Cycle)

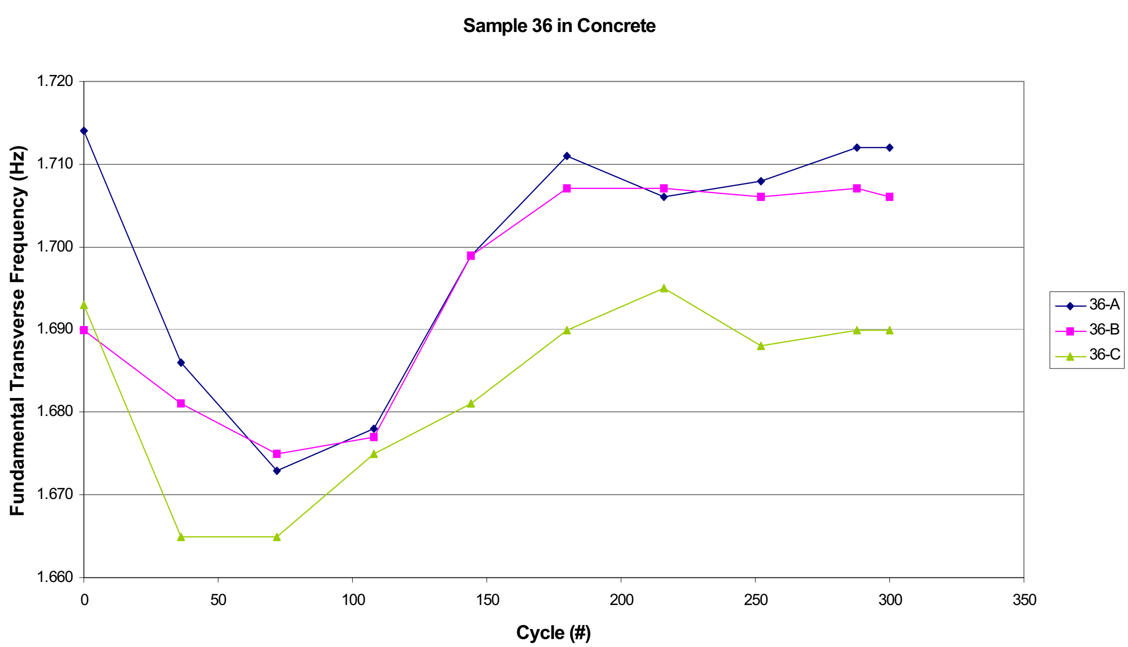

Figure E-4 – Sample 36 (Transverse Frequency vs. Freezing and Thawing Cycle)

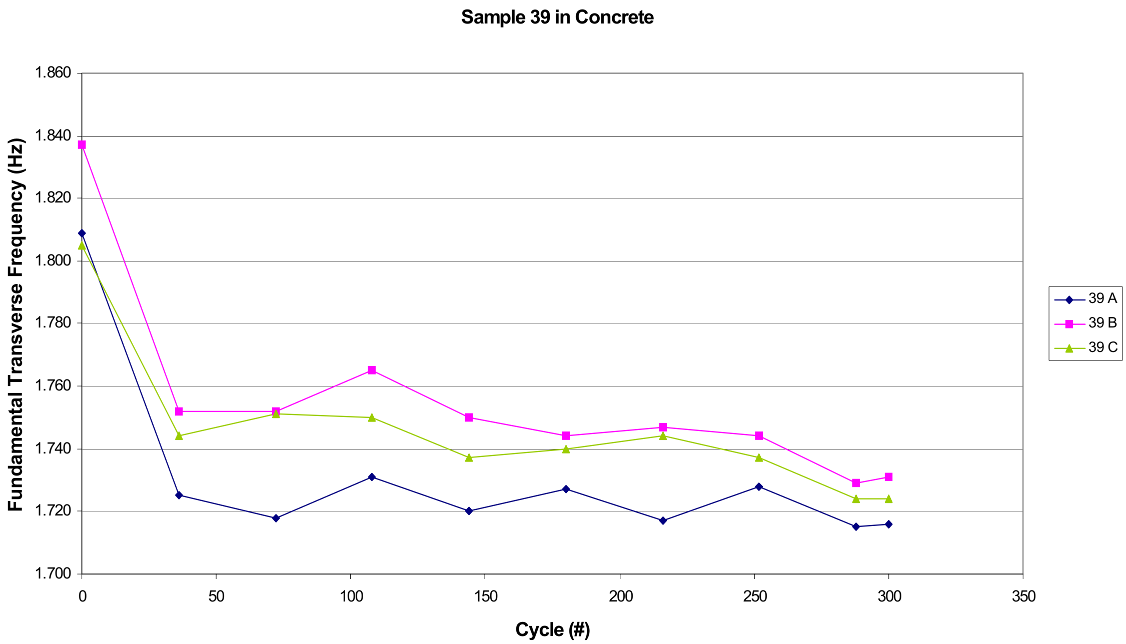

Figure E-5 – Sample 39 (Transverse Frequency vs. Freezing and Thawing Cycle)

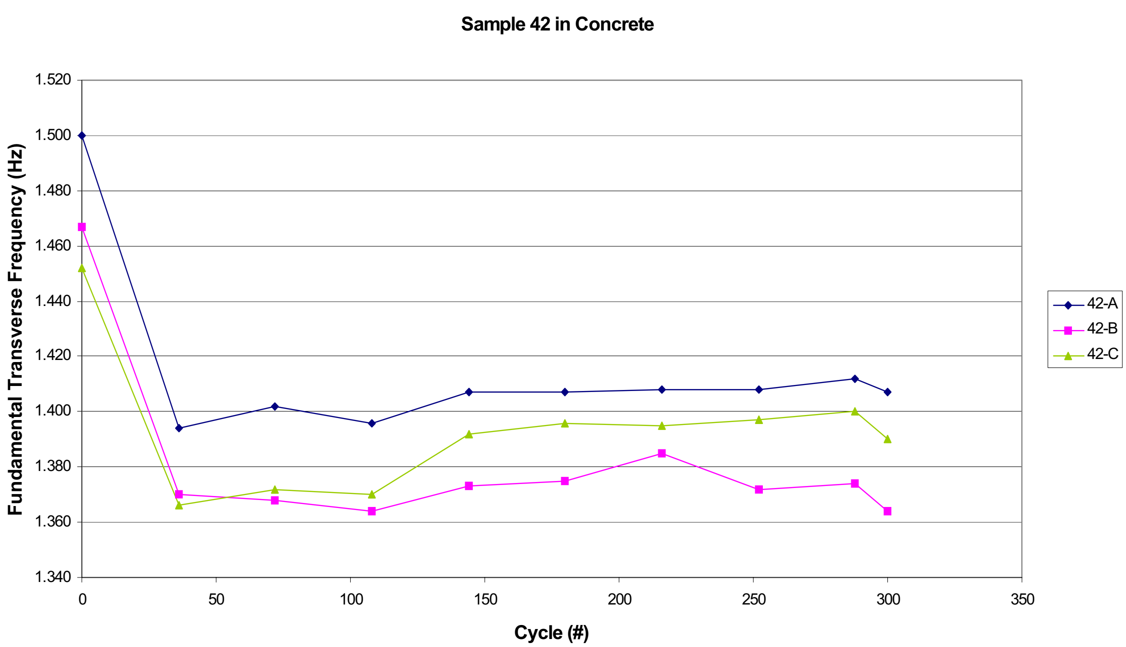

Figure E-6 – Sample 42 (Transverse Frequency vs. Freezing and Thawing Cycle)

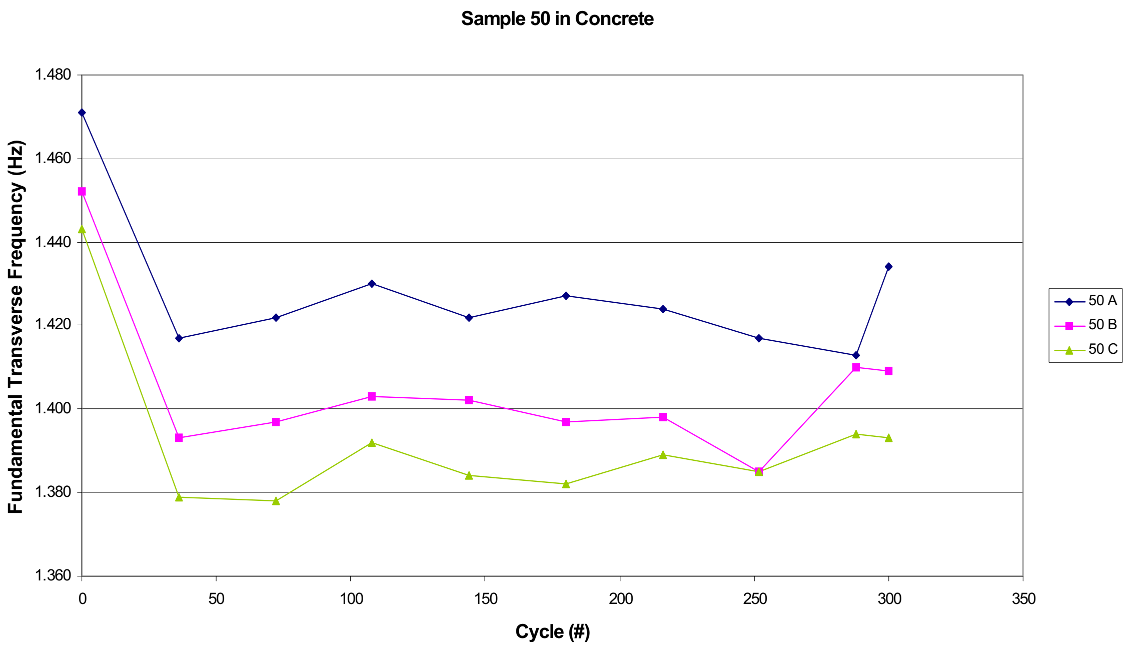

Figure E-7 – Sample 50 (Transverse Frequency vs. Freezing and Thawing Cycle)

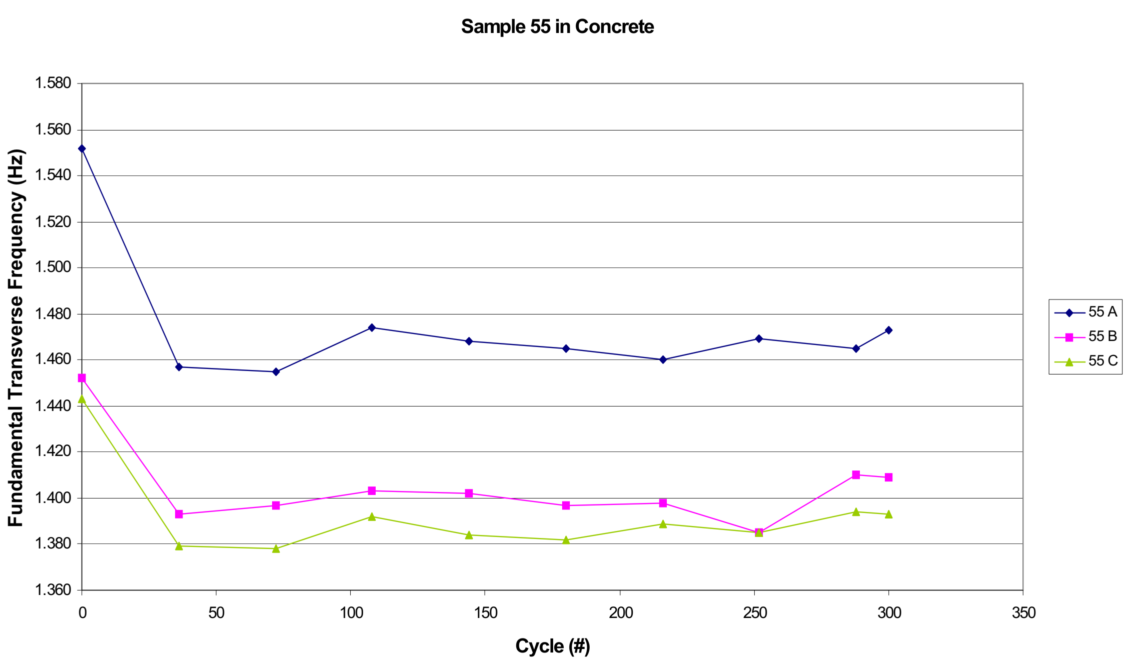

Figure E-8 – Sample 55 (Transverse Frequency vs. Freezing and Thawing Cycle)

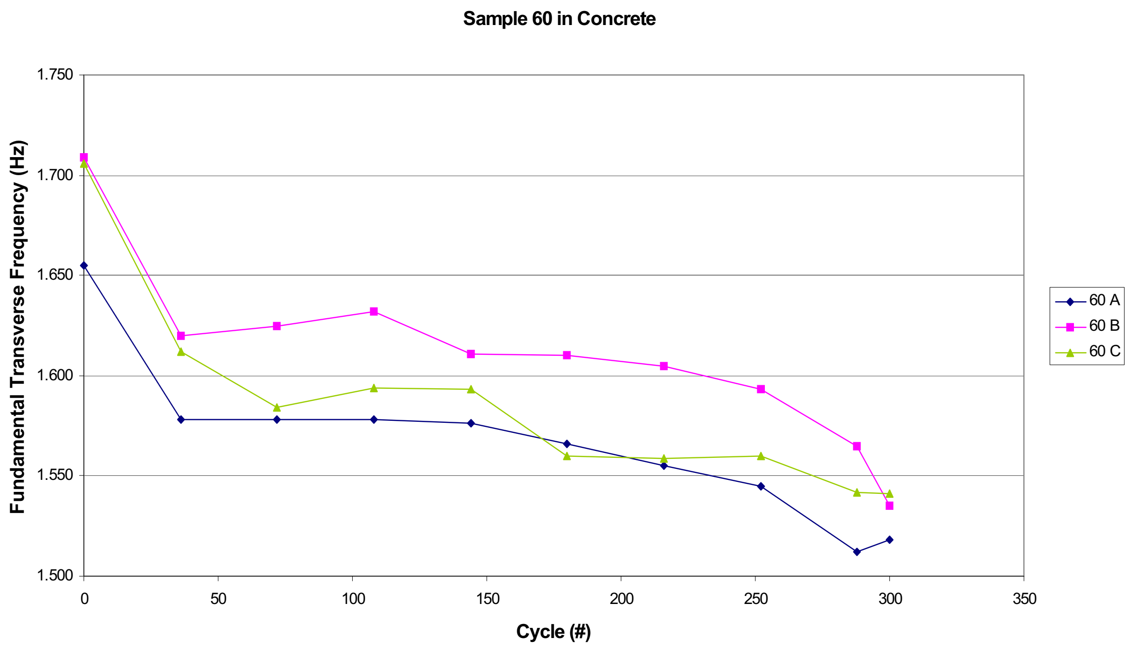

Figure E-9 – Sample 60 (Transverse Frequency vs. Freezing and Thawing Cycle)

APPENDIX F

Petrographic Analysis

Petrographic Analysis				
Sample No.	**Predominate Type**		**Sample No.**	**Predominate Type**
	Carbonate, hard to slightly weathered			**Granitic**
1			4	pink
6			14	pink
10	50:50 w/ more weathered		15	pink
11			36	
12	50:50 w/ more weathered		37	some foliation
13	80:20 w/ more weathered		53	weathered, 20% trap, 30% misc.
16			57	30% sandstone, quartzite, 5% chert
17				
18	w/ some sandstone (calc cemented0			**Trap**
19			33	
20	very slightly weathered		34	
21	very slightly weathered		35	
23			54	25% sandstone, 15% granite, 10% schist
26			58	20% weathered granite
27			59	20% granite, 20% carbonate
28	20% slightly weathered			
32				**Quartzite**
38			44	
39			45	
40	some weathered			
42	some weathered/sandy/soft			**Sandstone**
46			50	20% fair, 20% friable, 20% trap
49	15% Quartzite, 5% trap		52	30% trap, 15% granite, 5% gneiss
55	40% weathered, 10% sandstone, 5% trap, 5% chert			
56	slightly weathered, 25% chert, 5% granite			**RAP**
			74	Strong gas/kerosene smell, possible contamination
	Carbonates slightly weathered			
2	w/ some sandy (rounded quartz)			
3				
5	w/ 10% sandy (rounded quartz)			
7	w/ some sandy (rounded quartz)			
8	w/ 10% hard, less weathered			
9	some dense chert			
22				
24	1 pc each chert and RAP in 4			
25	w/ 10% sandy (rounded quartz)			
29				
30	20% sandy/soft			
31	w/ some sandy/soft			
41	75% sandy/soft, 25% hard			
43	slightly weathered sandy-soft			
47	10% sandy/soft			
48	tan-light brown, 75% soft, weathered			
51	deeply weathered			

Table F-1 – Petrographic Analyses
(Analyses conducted by Daniel S. Lane, Virginia Transportation Research Council)

APPENDIX G

Test Precision Statements

Specific Gravity and Absorption (ASTM C 127 - 01)

	Standard Deviation (1s)[A]	Acceptable Range of Two Results (d2s)[A]
Single-Operator Precision:		
Density (OD), kg/m^3	9	25
Density (SSD), kg/m^3	7	20
Apparent density, kg/m^3	7	20
Relative density (specific gravity) (OD)	0.009	0.025
Relative density (specific gravity) (SSD)	0.007	0.020
Apparent relative density (apparent specific gravity)	0.007	0.020
Multilaboratory Precision:		
Density (OD), kg/m^3	13	38
Density (SSD), kg/m^3	11	32
Apparent density, kg/m^3	11	32
Relative density (specific gravity) (OD)	0.013	0.038
Relative density (specific gravity) (SSD)	0.011	0.032
Apparent relative density (apparent specific gravity)	0.011	0.032

[A] These numbers represent, respectively, the (1s) and (d2s) limits as described in Practice C 670. The precision estimates were obtained from the analysis of combined AASHTO Materials Reference Laboratory proficiency sample data from laboratories using 15 h minimum saturation times and other laboratories using 24 ± 4 h saturation times. Testing was performed on normal-weight aggregates, and started with aggregates in the oven-dry condition.

Table G-1 – Specific Gravity and Absorption Precision Statement

L.A. Abrasion (ASTM C 131-01)

For nominal 19.0-mm (3/4-in.) maximum size coarse aggregte with percent losses in the range of 10-45%, the multilaboratory coefficient of variation has been found to be 4.5%. Therefore, results of two properly conducted tests from two different laboratories on samples of the same coarse aggregates should not differ from each other by more than 12.7% of their average. The single-operator coefficient of variation has been dound to be 2.0%. Therefore, results of two properly conducted tests by the same operator on the same coarse aggregate should not differ from each other by more than 5.7% of their average.

Micro-Deval (AASHTO TP 58)

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%) and (d2s%) limits as described in ASTM C670.

Table G-2 – Micro-Deval Precision Statement

Sodium and Magnesium Sulfate Soundness (ASTM C 88-99a)

	Coefficient of Variation (1S %), %[A]	Difference Between Two Tests (D2S %), % of Average[A]
Multilaboratory:		
Sodium sulfate	41	116
Magnesium sulfate	25	71
Single-Operator:		
Sodium sulfate	24	68
Magnesium sulfate	11	31

[A] These numbers represent, respectively, the (1S %) and (D2S %) limits as described in Practice C 670.

Table G-3 – Sodium and Magnesium Sulfate Soundness Precision Statement

Unconfined Freezing and Thawing (CSA A 23.2-24A)

Single-operator precision		
20–5 mm weighted % average loss	**Standard deviation**	**Acceptable range for two results (D2S)***
25	1.5	4.3
19	1.6	4.5
6	0.9	2.6
5	0.8	2.3
1	0.3	0.9
Multi-operator precision		
25	2.2	6.2
19	1.7	4.9
6	1.2	3.5
5	1.1	3.1
1	0.5	1.4

**Figures given for D2S are the limits of the difference between the results of two properly conducted tests on samples of the same material that should only be exceeded one time in 20.*

Note: *See C.A. Rogers, S.A. Senior, and D. Boothe,* Development of an unconfined freeze-thaw test for coarse aggregate, *Engineering Materials Report EM-87, Ontario Ministry of Transportation, Downsview, Ontario, July 1989, p. 21.*

Table G-4 – Unconfined Freezing and Thawing Precision Statement

Freezing and Thawing in Concrete (ASTM C 666-97)

Within-Laboratory Durability Factor Precision for Averages of 3 Beams		
Range of Average Durability Factors	**Standard Deviation (1S)**	**Acceptable Range of Two Results (d2s)**
0 to 5	0.6	1.8
5 to 10	2.3	6.6
10 to 20	4.7	13.2
20 to 30	6.1	17.2
30 to 50	8.9	25.1
50 to 70	11.6	32.9
70 to 80	9.9	27.9
80 to 90	5	14.4
90 to 95	2.3	6.4
Above 95	1.2	3.3

Table G-5 – Freezing and Thawing in Concrete Precision Statement