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16. Abstract (Limit: 200 words)

Study goals included: 1) identify mechanisms causing premature failure in Minnesota concrete pavements; 2) evaluate the accuracy of existing tests of aggregate freeze-thaw durability using Minnesota aggregate sources and pavement performance records; 3) develop a new methodology for quickly and reliably assessing aggregate freeze-thaw durability; and 4) evaluate techniques for mitigating D-cracking.

Research results indicate that the poor durability performance of some Minnesota PCC pavement sections can often be attributed to aggregate freeze-thaw damage. However, secondary mineralization, embedded shale deposits, poor mix design and alkali-aggregate reactions were also identified as problems. Petrographic examination can help to differentiate between these failure mechanisms.

A reliable and universal method for quickly identifying D-cracking aggregate particles was not identified. A test protocol was developed for improved aggregate durability evaluation. It includes several tests which are selected for use based on aggregate geological origin and composition and the results of previous tests. Further validation of the proposed test protocol is recommended.

Several techniques appear to be effective in improving the freeze-thaw durability of concrete prepared using marginally durable aggregate: mix design modifications, reductions in aggregate top size, and the blending of durable and nondurable aggregates. Some chemical treatments showed promise, but may not be economical.

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MITIGATING CONCRETE AGGREGATE PROBLEMS IN MINNESOTA

Final Report

Prepared by:

Amir Koubaa
Mark B. Snyder
Karl R. Peterson

Department of Civil Engineering
University of Minnesota

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395 John Ireland Boulevard
St. Paul, MN 55155

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EXECUTIVE SUMMARY

It is believed that certain aggregate sources in southern Minnesota are responsible for the premature failure of large portions of Interstate 90. The freeze-thaw behavior associated with these aggregate sources has been suspected as the cause of these failures. As a result, the Minnesota Legislature, Department of Natural Resources and Department of Transportation commissioned this study to:1) positively identify the mechanisms causing premature failure in southern Minnesota concrete pavements; 2) identify aggregate sources that appear to be responsible for the premature failure of concrete pavement by D-cracking; 3) document the accuracy and reliability of existing tests of aggregate freeze-thaw durability using Minnesota aggregate sources and pavement performance records; 4) develop a new methodology for quickly and reliably assessing the freeze-thaw durability of a given aggregate source; and 5) identify and evaluate techniques for mitigating D-cracking, thereby allowing the increased use of local aggregate sources that are currently considered marginal or unacceptable in concrete construction.

The results of this research indicate that the poor durability performance of PCC pavement sections in southern Minnesota can, in many cases, be attributed to the susceptibility of coarse aggregates to freeze-thaw damage; however, secondary mineralization, embedded shale deposits, poor mix design and alkali-aggregate reactions were also identified as problems that can aggravate D-cracking or appear similar to it. Petrographic examination of pavement cores can help to differentiate between these different failure mechanisms.

A single quick, simple, economical and reliable method for identifying frost-susceptible aggregate particles was not identified, although there are quick tests that can be used to determine the frost resistance of one aggregate relative to others. Whether an aggregate can resist repeated cycles of freezing and thawing can sometimes be answered only by tests that simulate field exposure conditions (such as ASTM C 666).

A test protocol was developed to more quickly and accurately assess the freeze-thaw durability of Minnesota concrete aggregates. Several different tests are included in this protocol, and the selection of tests for use on the basis of aggregate minerology, as well as the results of some of the quick screening tests. The test protocol, in its current form, is not yet ready for adoption as a procedure for predicting the frost resistance of coarse aggregates; additional aggregate sources should be evaluated in order to establish and validate acceptance/rejection criteria. It is believed, however, that the test protocol developed under this project will serve as a model for the development of a reliable procedure for accurately assessing the freeze-thaw durability of many types of concrete aggregate.

It was also determined that several techniques are effective in improving the freeze-thaw durability of new concrete construction using marginally durable aggregate. These include mix design modifications (such as reduced water-cement ratio), reductions in the top size of nondurable aggregates, and the blending of durable and nondurable aggregates. Chemical aggregate treatments also showed some promise, but the economics of such treatments may not be favorable.

The research that was completed can be used to improve the performance of future concrete pavements throughout Minnesota while allowing the continued use of local Minnesota aggregate resources. Possible economic benefits include the reduction of Minnesota pavement life cycle costs and the increased utilization of Minnesota aggregate resources that were previously considered marginal performers.

Future research should include studies in a similar vein but wider in scope to provide the data needed for refining and implementing the durability test protocol that was developed under this study. The effect of deicing salts on carbonate and other types of rocks during freezing and thawing should also be investigated, as this study demonstrated that deicing salts have an extremely deleterious effect on many types of concrete aggregates.

CHAPTER 1

INTRODUCTION

1.1 Background

Portland cement concrete (PCC) durability is defined as its ability to withstand exposure to environmental conditions such as heating and cooling, freezing and thawing, wetting and drying, chemical attack or abrasion. Freeze-thaw deterioration is the most common and severe durability problem for PCC pavements in many states, and coarse aggregate durability cracking (also known as D-cracking) is a common manifestation of freeze-thaw deterioration in PCC.

D-cracking is a progressive distress associated primarily with the use of coarse aggregates that deteriorate when critically saturated and subjected to repeated cycles of freezing and thawing. The D-cracking mechanism typically begins when water enters into open joints and cracks. This moisture, along with water present beneath the pavement, may cause coarse aggregates within the slab to become critically saturated (i.e., degree of saturation exceeds about 91%). When these critically saturated aggregates are frozen, the expansion of ice within the aggregate pores may generate pressures that exceed the tensile strength of the aggregates, causing cracking of the aggregate and/or the surrounding mortar (see figure 1.1). With cycles of freezing and thawing, these cracks often widen and become additional channels for water migration and additional sites for ice formation, allowing further propagation and widening of these cracks.

D-cracking is observed most often in pavements and other slabs on grade, although it can occur in other concrete structures. D-cracking usually originates at the bottom of the slab (where the pavement is saturated most frequently) and progresses upward, although it can start at the pavement surface or in the middle of the slab. At the pavement surface, D-cracking almost always appears first along joints or cracks (where water is easily stored, allowing localized saturation). Signs of D-cracking include:

- a series of closely spaced cracks observed at the surface, adjacent and roughly parallel to transverse and longitudinal joints or cracks and free edges. These cracks are sometimes filled with black, blue, gray or white deposits, which consist of calcium carbonate and dirt (1).
- a series of cracks, observed in pavement cores, which propagate approximately parallel to the pavement surface in the bottom or the middle of the slab, where they usually develop before appearing at the surface of the pavement (1).

The first signs of D-cracking normally appear at the intersection of longitudinal and transverse joints, and at the outside corners of pavement slabs. As the cracks propagate, they form a continuous network along the peripheral areas of the slab, as shown in figure 1.2. When the cracks propagate further, they spread towards the central part of the slab of the pavement (2).

Freeze-thaw damage can also originate in the cement paste or at the interface between coarse aggregate particles and the mortar matrix as the water in concrete pore structure expands upon freezing (1, 3). This type of deterioration may resemble the cracking caused by nondurable coarse aggregates, but is not considered D-cracking.

1.2 Problem Statement

D-cracking of concrete pavements and other structures is a major distress that necessitates large annual expenditures for slab replacement or repair (1). It is believed that certain coarse aggregate sources in southern Minnesota are prone to D-cracking and are responsible for the premature failure of large parts of Minnesota's concrete pavement network, particularly large portions of I-90 in southern Minnesota.

In 1972, the Minnesota Department of Transportation (Mn/DOT) adopted the following acceptance criteria for coarse aggregate intended for use in concrete paving in an effort to reduce the incidence of D-cracking: 1) Class C materials (gravel) must have less than 30 percent of the particles (by weight) from carbonate origin and 2) Class B materials (carbonates) must have an absorption capacity of less than 1.75 percent. While these criteria seem to have eliminated most

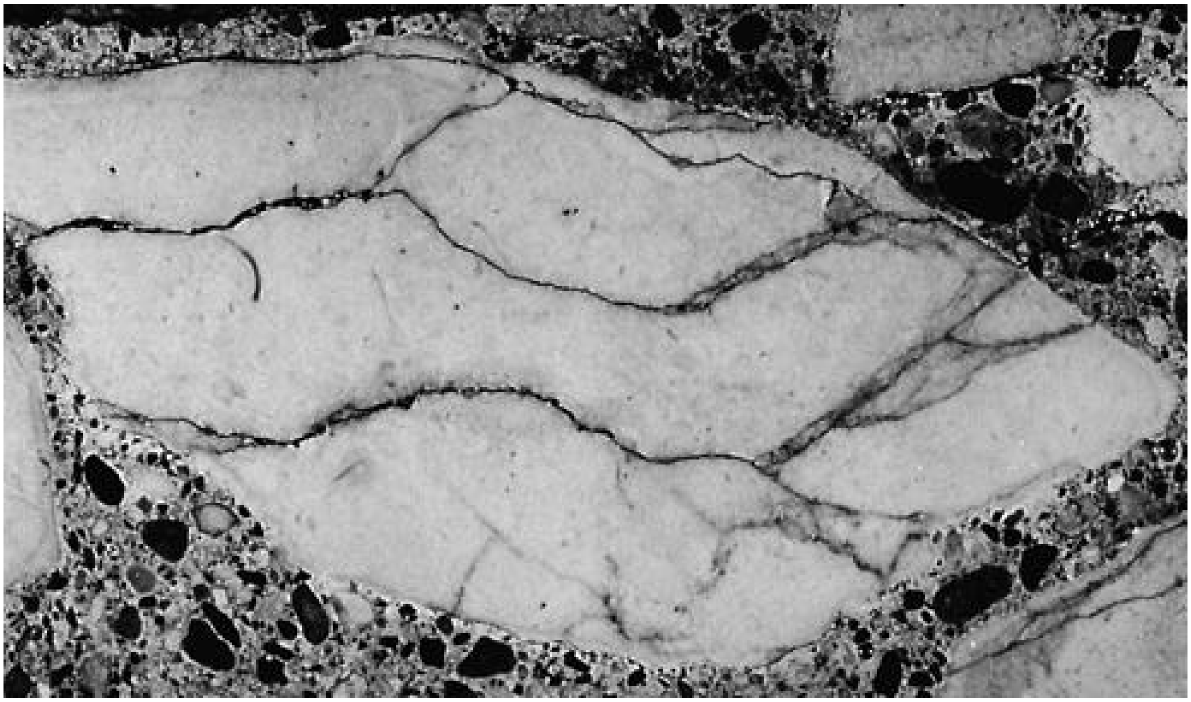

Figure 1.1. Fractured carbonate aggregate particle as a source of distress in D-cracking (4).

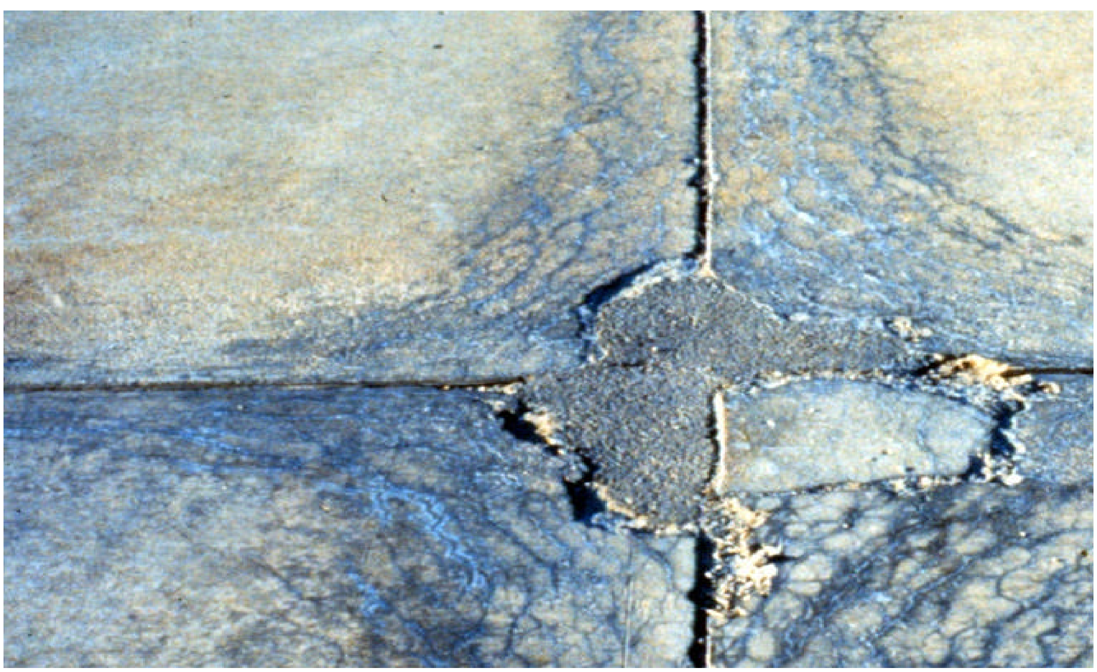

Figure 1.2. D-cracking at intersection of longitudinal and transverse joints.

(but not all) apparent incidences of D-cracking in Minnesota, they have also proven to be very restrictive, eliminating some potentially sound sources.

The positive identification of D-cracking-susceptible aggregates prior to their use in concrete construction is difficult. Many tests have been developed to determine D-cracking susceptibility, although their suitability has been found to vary among different researchers and state transportation agencies (1, 5, 6). Many tests identify D-cracking susceptibility only for certain types of aggregates and are not useful for other types of aggregates. Other more reliable testing methods require expensive equipment and are time-consuming to perform. It appears that no single test or acceptance/rejection criterion is currently available for predicting the freeze-thaw durability of coarse aggregate in PCC pavements (1, 5). A fast, reliable, reproducible, easily performed and inexpensive test or suite of tests is still needed for identifying aggregate susceptibility to D-cracking.

It is also desirable to identify techniques for mitigating the problem of D-cracking. This might allow increased use of local aggregate resources in areas where freeze-thaw damage potential is known to exist, which would produce economic benefits such as reduced initial cost, increased service life and lower life cycle costs.

1.3 Study Objectives

The objectives of this study were:

1. identify aggregate sources that appear to be responsible for the premature failure of concrete pavement in southern Minnesota by D-cracking;
2. document the accuracy and reliability of existing tests of aggregate freeze-thaw durability using Minnesota aggregate sources and pavement performance records;
3. develop a new methodology for quickly and reliably assessing the freeze-thaw durability of a given aggregate source; and
4. identify and evaluate techniques for mitigating D-cracking, thereby allowing the possible use of aggregates sources that are currently considered marginal or unacceptable in concrete construction.

1.4 Research Approach

The research approach adopted for this study included consideration of previous research efforts in this area (many of which are laboratory-based) and focused on relating the results of laboratory tests to field performance. The research work plan was developed in close cooperation with Mn/DOT representatives based on the findings of the literature review, and consisted of the tasks described below.

Task 1: Literature Review

A literature review was conducted to summarize the state of knowledge and practice concerning aggregate freeze-thaw problems. The primary purpose of this task was to identify those tests which seemed to bear the most promise and to eliminate from further consideration those which seemed poorly conceived or appeared to have little merit.

Task 2: Field Study Investigation

The Minnesota Department of Transportation (Mn/DOT) pavement management databases were examined to identify pavement sections around the state that represent a range of aggregate types (e.g., limestone, dolomite, gravel, etc.), climatic conditions and pavement durability-related performance (e.g., good, fair and poor) found in southern Minnesota. Field condition surveys were performed to validate the information contained in the Mn/DOT databases and to provide a quantitative assessment of the amount and severity of freeze-thaw distress produced by each aggregate source in response to the local climate.

Cores were retrieved (for laboratory testing and petrographic examination) from pavement sections representing a wide range of durability performance. Coarse aggregate samples were obtained from the sources that were originally used to construct the pavement study sections. Petrographic examinations were performed to ensure that the samples obtained were sufficiently similar to those used in the construction of the field sections many years ago.

Task 3: Perform Aggregate Freeze-Thaw Durability Testing:

Each coarse aggregate sample was subjected to the most promising tests identified in the literature review. The test results were correlated with the performance observed in the field study sections to validate existing acceptance/rejection criteria or to develop new criteria. The results of laboratory correlative tests were also compared with the results of simulative tests.

Task 4: Analyze Results

The geological and engineering properties of the aggregates were compared to the results of other tests performed on the aggregates to evaluate the potential of each test for accurately predicting the freeze-thaw damage potential of typical Minnesota coarse aggregates.

The cumulative results of these tests and evaluations of field sections were used to develop a suite of freeze-thaw durability tests for accurately assessing the probable field performance of any given aggregate as a function of its original geological origin and probable environmental exposure.

Task 5: Evaluate Beneficiation Techniques

Six of the aggregate sources used to develop the suite of tests for determining aggregate freeze-thaw durability were also used to test candidate beneficiation techniques. The treated aggregates and special mixtures of concrete were subjected to freeze-thaw testing to determine whether the beneficiation techniques produced improved freeze-thaw durability performance.

1.5 Scope

This study was undertaken to study the freeze-thaw performance of Minnesota aggregates. Fifteen pavement sections were selected for the field study to be representative of the range of freeze-thaw durability performance (i.e., good, fair and poor) observed in Minnesota. Thirteen of these pavement sections were in southern Minnesota and two pavement sections were in western Minnesota. The pavement sections in western Minnesota (gravel sources) were included because they showed very poor performance and because most of the severely D-cracked pavement sections in southern and central Minnesota had been overlaid and were unavailable for inclusion in this study.

The scope of the laboratory study was to perform the most promising durability tests on aggregate and concrete samples representative of the materials used in the study pavement sections. These tests were selected based upon the review of the literature and highway agency practices for accepting or rejecting the use of an aggregate source.

Fourteen aggregate samples (two gravels and twelve carbonates) were obtained; eleven of these sources closely matched the aggregates used in the study pavement sections. The origins and geological formations of these aggregates are typical of rock formations found in Minnesota and northern Iowa. The ability of each test to predict freeze-thaw performance was evaluated based upon correlations between the test results when the aggregate samples were used and the observed field performance of the same aggregates in the study pavement sections. The methodology developed for accepting or rejecting an aggregate source was based upon correlations between the tests that best predicted the freeze-thaw susceptibility and the performance of the study pavement sections.

Samples from six of the aggregate sources were used to test four different techniques for mitigating D-cracking. The effectiveness of these techniques was evaluated using a modification of ASTM C 666 (Standard Test for Freeze-Thaw Durability of Concrete).

CHAPTER 2

LITERATURE REVIEW

2.1 Historical Background of D-Cracking in Minnesota

D-cracking was first identified in Kansas in 1930 (1). By the 1960's and 70's, D-cracking was considered a serious problem in several states. D-cracking was originally considered to be related to the use of crushed stone coarse aggregate, especially limestone, but it was later found that coarse gravel aggregates were also associated with D-cracking (1). In a reply to a Bureau of Public Road's questionnaire, Carsberg associated D-cracking with heavy truck volumes, inadequate subgrade support of pavement, inadequate pavement thickness, low cement content in concrete and coarse aggregates with high percentages of limestone pebbles (i.e., very small and rounded particle deposits with a carbonate origin)(7).

Minnesota research efforts to mitigate D-cracking of concrete pavements started in 1939 (8). In 1958, the Mn/DOT adopted the following acceptance criteria based on aggregate particle freeze-thaw, absorption and concrete freeze-thaw tests (8):

- 25 percent maximum loss after 16 cycles of the Iowa aggregate particle freeze-thaw test in water;
- 3 percent maximum loss through the next smaller sieve in the Kansas aggregate particle freeze-thaw test in water;
- 3.5 percent maximum weighted average absorption of the 4.75 to 37.5-mm material; and
- 30 percent maximum reduction in the sonic modulus of elasticity of concrete after 150 cycles of freeze-thaw.

In 1972, Carsberg reported that the Minnesota Department of Transportation (Mn/DOT) had adopted the following acceptance criteria for coarse aggregate intended for use in concrete paving (9):

- Class C materials (gravel) must have less than 30 percent of the particles (by weight) from limestone origin.
- Class B materials (limestone) must have an absorption capacity of less than 1.75 percent.

These concrete aggregate durability acceptance criteria are still in effect today.

In 1977, Mn/DOT purchased a freeze-thaw machine to test concrete beams using the rapid freezing and thawing test (ASTM C 666). Its use was discontinued in 1982 because of poor correlation between test results and field performance, which was explained by improper testing procedures or faulty equipment (10). The criteria adopted by the Mn/DOT in 1972 have remained in effect, even though they have proven to be very restrictive, eliminating some potentially sound sources while failing to prevent some apparent freeze-thaw failures in the southern part of the state.

2.2 Conditions Necessary for D-Cracking

It is currently believed that D-cracking can occur only when: 1) the concrete contains a sufficient quantity of large aggregate particles that are susceptible to D-cracking; 2) these particles are allowed to become critically saturated (i.e., > 91 percent saturated), which often occurs near joints and cracks; and 3) the concrete is exposed to a sufficient number of freezing and thawing cycles (5). D-cracking usually appears in the field after 5 to 10 years, but may not develop for 20 years or more (5).

The prevention of D-cracking can be accomplished by eliminating one or more of the above conditions. Eliminating the use of unsound aggregates is not always feasible when local resources are inexpensive and the cost of transporting aggregate from a more durable source is high. Furthermore, the identification of aggregates susceptible to D-cracking often requires performing expensive and time-consuming tests, and reliable acceptance/rejection criteria are needed to avoid using nondurable materials. Elimination of freezing in the field is generally not feasible, even when thick overlays are used. Reducing the level of saturation of the concrete and coarse aggregate is generally the most feasible and preferred method of preventing D-cracking. The use of surface sealers and joint sealants can be effective in preventing the entry of moisture, thereby reducing the occurrence of D-cracking.

Although the air void system of the concrete is not related to the occurrence of D-cracking, inadequate air entrainment can accelerate the rate of D-cracking progression by allowing more moisture intrusion along the joints and cracks (5).

2.3 Mechanisms Involved in D-Cracking

It is generally accepted that D-cracking is caused by the expansion and deterioration of critically saturated coarse aggregate particles due to freezing and thawing. Critical saturation is typically defined as a saturation level exceeding 91.7 percent, since water expands by approximately 9 percent when frozen. The aggregate particles can become saturated by moisture that is introduced through open cracks and joints in the pavement as well as by moisture that collects or is present beneath the pavement. During freezing and thawing of a critically saturated, nondurable aggregate, the pore pressures generated in the aggregate particle exceed the tensile strength of the aggregate and cause cracking of the aggregate particle and the surrounding mortar. With additional freezing and thawing, these cracks become additional channels for the migration of moisture in the aggregate particles and become additional sites for the formation of ice (1).

The mechanism of damage to <u>concrete</u> from repeated cycles of freezing and thawing is still not well understood. Several theories have been proposed to describe the mechanism of frost action in concrete, and most of them were developed to explain the freeze-thaw damage in mortar or cement paste, although some of these theories can be used to explain freeze-thaw damage in aggregate particles. The theories that have gained widest acceptance are the ones proposed by Powers (11), Powers and Helmuth (12), Verbeck and Landgren (13), Dunn and Hudec (14), Larson and Cady (15) and Litvan (16). Other theories have also been proposed and research is still being conducted to more fully understand freeze-thaw action in concrete mortar and aggregate particles.

<u>2.3.1 Powers (1945)</u>

Powers first proposed the hydraulic pressure hypothesis, which is based on the expansion of water when frozen and the pressure developed in the unfrozen water (11). He proposed that hydraulic pressure is developed as unfrozen water is expelled by an ice front advancing through saturated pores, resulting in internal stresses that could exceed the tensile strength of the aggregate and cause the aggregate particle to rupture. As the temperature drops below 0^0C, ice starts to form in the largest pore spaces. As the temperature drops further, ice forms in the smaller pores, displacing water that must flow through the unfrozen part of the body to the nearest point of escape. The magnitude of hydraulic pressure developed is a function of the freezing rate, the distance that the water has to travel to escape, the permeability of the aggregate and the viscosity of water. Under certain combinations of these four factors, sufficient pressure can build up and cause the concrete to fracture.

2.3.2 Powers and Helmuth (1953)

Further studies by Powers and Helmuth indicated that the hydraulic pressure theory did not account for the continued dilation of PCC observed at constant freezing temperatures or for the shrinkage of air-entrained cement paste (12). Based upon these observations, they proposed a gel water diffusion mechanism to account for the damage of concrete due to frost action, as described below.

Water in the concrete capillary system is impure due to the presence of soluble substances such as alkalis, chlorides and calcium hydroxide. When water in the large capillary pores is frozen and the water in the smaller gel pores is unfrozen, the unfrozen water is attracted to the ice because of the differences in solute concentrations. The resulting generation of distending pressures (osmotic pressure) is responsible for the formation of some cracks in the concrete.

This theory was developed to explain phenomena observed in portland cement mortar and concrete, but can be extended to conditions that exist in aggregate particles as well.

2.3.3 Verbeck and Landgren (1960)

Verbeck and Landgren observed that the failure of large aggregate particles cannot be attributed to hydraulic pressure theory alone, and that the magnitude of the hydraulic pressure developed is significantly influenced by the size of the aggregate particle, as well as the permeability and air content of the paste (13). The following mechanisms were offered to explain the development of D-cracking:

- Some aggregate particles are not strong enough to withstand pressures developed during freezing and will fracture and cause distress in the surrounding paste. The effect of aggregate size is considered critical, since the pressure required to expel excess water from the frozen particles increases with the distance that the water must travel through the pores, which increases with increasing particle size.

- Some aggregates may possess enough elasticity to withstand considerable pressure without fracturing. As these pressures increase, the aggregate expands elastically. The surrounding mortar may be unable to accommodate this expansion and may fracture as a result.
- The expulsion of water from highly absorptive aggregates to the surrounding paste can generate highly disruptive pressures at the aggregate-paste interface.

On this basis, Verbeck and Landgren proposed three classes of concrete aggregates:

1. low-permeability and high-strength aggregates, which are capable of accommodating elastic strain without fracturing when the pore water freezes.
2. intermediate-permeability aggregates, where the development of pressure depends on the rate of temperature drop and the distance that water must travel to find an escape boundary in either an empty pore or at the aggregate surface.
3. high-permeability aggregates, which may permit easy entry and egress of water, but are usually responsible for durability problems because of the damage to the transition zone between the aggregate and the cement paste matrix that results when water is expelled from the aggregate.

2.3.4 Dunn and Hudec (1966)

Dunn and Hudec advanced the "ordered water theory", which states that the principal cause of aggregate particle deterioration is not the expansion of freezing water but is due to the expansive phase transition of the adsorbed water, which is similar to the water-to-ice transition (14). They suggested that the delineation between sound and unsound aggregate could be obtained from the relationship between unfilled pores of carbonate aggregates after saturation for 24 hours and the water sorbed at 85 percent humidity at 30 ^{o}C. Mather reported that tests of clay-bearing (argillaceous) limestone aggregates seemed to support this theory because they could be failed without freezing (17). However, Schwartz reported that the application of this theory presents a problem since it does not relate freezing conditions to D-cracking, which exists only in areas with freezing conditions (1).

2.3.5 Larson and Cady (1969)

Larson and Cady recognized the roles of both hydraulic pressure and sorptive mechanisms (15). Powers' hydraulic pressure mechanism was described previously (11). Secondary and post-freezing dilations were attributed to the adsorption of water to ice and rock surfaces. Adsorbed water is ordered water that creates an expansive phase transition similar to the water-to-ice transition. The second phase of hydraulic pressures is generated by an increase in the adsorption rate until all of the bulk water in the aggregate or paste has been adsorbed (i.e., until all water has undergone a change of state).

2.3.6 Litvan (1972)

Litvan's theory is similar to the theory presented by Powers except that it suggests that hydraulic pressure is built up in the pore system of the concrete as water is expelled in an attempt to come to equilibrium with the vapor at the air-water interface (16). Litvan's theory is based on the lowering of relative humidity with decreasing temperature through condensation in the form of ice. The migration of water (from the higher energy sites to the lower energy sites) and the resulting dilation is caused by hydraulic pressure, which is a direct result of the thermodynamic dis-equilibrium between the frozen water in the capillary pores (low energy) and the unfrozen water in the gel pores (high energy). As cooling progresses, a decrease in humidity is accomplished by condensation (in the form of ice) and desorption of the adsorbed water, which migrates to the nearest point of escape. The concrete ruptures when the hydraulic pressures generated exceed the tensile strength of the concrete.

2.4 Factors Affecting D-Cracking

It is currently believed that D-cracking can occur only when the concrete contains a sufficient quantity of large aggregate particles that are susceptible to D-cracking, the concrete is exposed to an amount of moisture sufficient to critically saturate the aggregates, and the concrete is exposed to repeated cycles of freezing and thawing (5). However, several other factors contribute to the rate of development and severity of D-cracking, as discussed below (1).

2.4.1 Environmental Effects

D-cracking is observed only in areas where freezing and thawing occurs, and structures that are not exposed to external sources of moisture do not exhibit this type of deterioration. Cyclic freezing and thawing and moisture are the only natural environmental factors believed to affect D-cracking.

Cyclic Freezing and Thawing

The number of freezing and thawing cycles applied to concrete pavements is an important factor in determining the rate of deterioration; repeated cycles of freezing and thawing are more severe than a single freezing and can increase the moisture content in the concrete (18). Stark reported that the intensity of D-cracking deterioration around joints increases as the number of freezing and thawing cycles increases (19). Schwartz (1) reported that 5 to 10 years or more of freezing and thawing are often sufficient for D-cracking appearance. Janssen and Snyder (5) reported that the depth of freezing in pavements has an effect on the development of D-cracking, with mild climates producing D-cracking resembling shallow spalls near joints rather than deterioration starting at the bottom of the concrete slab. They also noted that the number of freezing and thawing cycles often varies with the depth of the slab (i.e., fewer number of cycles may be observed at the bottom than at the surface of the slab).

The freezing temperature and cooling rate are also key factors in deterioration by freeze-thaw action. The freezing point of water decreases as the pore size decreases due to surface tension effects. Therefore, the further the temperature is decreased below freezing, the more ice ice forms in smaller aggregate particle and cement mortar pores.

The temperature at which freezing occurs can be depressed under the following conditions:

- presence of dissolved substances such as salt, hydroxides and alkalis (20);
- reduced capillary pore size, resulting in increased surface tension forces (20);
- supercooling of water in the absence of ice crystals (20); and
- presence of impurities, such as dust (21).

The freezing rate also appears to be a key factor in determining the severity of concrete freeze-thaw deterioration. Vanderhorst and Janssen reported that freezing rates in the field ranged from 0.8 to 0.9 ^{o}C/hour, and that the degree of severity of the freeze-thaw deterioration increased as this cooling rate is reduced (18). Lin and Walker found that slow cooling rates reduce the freeze-thaw durability by increasing the moisture content of the concrete quickly (22). However, Pigeon et al. reported that an increase in the cooling rate results in reduction of the freeze-thaw durability. The cooling rates used by Pigeon et al. are typical of those used for laboratory rapid freezing and thawing tests (23).

Moisture State

The development of D-cracking requires the presence of a sufficiently high moisture content in the concrete or the aggregate. Concrete pavement moisture can originate from many sources, including infiltration from the pavement surface, condensation or collection in the layers beneath the concrete, and from the unfrozen water in the concrete pores. Vanderhorst and Janssen reported that the degree of saturation for concrete in the field is typically around 90 percent, which is considered to be high and may be critical for concrete exposed to freezing and thawing conditions; for aggregates, a value of 91.7 percent is generally accepted as being critical (18). The movement of moisture within concrete is also considered an important factor in the development of freeze-thaw deterioration, as discussed earlier.

2.4.2 Coarse Aggregate

The freeze-thaw durability of aggregate particles is influenced by their mineralogy, pore structure, absorption and adsorption potential, particle size and specific gravity, as discussed in the following subsections.

Mineralogy

Materials of igneous origin (i.e., intrusive and extrusive rocks, such as granite and basalt, respectively) are not known to cause D-cracking. Similarly, rocks of metamorphic origin (e.g., gneiss, quartzite and marble) have usually performed well and are not generally associated with D-cracking. However, many sedimentary rocks (e.g., carbonates, silicates, friable sandstone and clay lumps) and metagraywacke are known to cause D-cracking (4, 24). Marks and Dubberke also pointed out that the use of river gravel can result in D-cracking if the carbonate fraction is frost-susceptible and present in sufficient quantities (25).

Gaynor and Meininger reported that the frost susceptibility of aggregates is increased by the presence of small percentages of chert, deleterious particles and lightweight particles (26). Friable sandstones, soft limestones and clay lumps affect the freeze-thaw durability of concrete because they fail to maintain their integrity. Pence reported that weathered chert and limestones containing clays are deleterious in concrete because they contain minerals with highly active surfaces which attract water molecules (24). These minerals produce disruptive expansive forces in the concrete when frozen (i.e., if frozen when they are saturated, they increase in volume and develop sufficient pressure to cause disintegration of the concrete). Pence also reported that shale particles are capable of attaining a high degree of saturation because of their high clay mineral content (which easily absorbs water) and their many interconnected voids; thus, the high hydraulic pressures that develop when they are frozen disrupt the bond between the aggregate particles and the paste (24).

Carbonate aggregates are sedimentary materials containing primarily calcite or dolomite minerals. They range from pure calcite ($CaCO_3$) or dolomite ($MgCO_3$) minerals to various blends of these materials. Limestones and dolomites in Minnesota usually contain both carbonate minerals and non-carbonate minerals, such as clay and sand. Some impurities, such as expansive clay minerals and opal, may increase the frost susceptibility of limestone aggregates. The presence of clay may affect the durability of carbonate aggregates because small amounts of clay in limestone may reduce freezing expansion, while large amounts can aggravate the D-cracking deterioration by the expansion of clay and the ordering of water molecules, which would expand the limestone (27).

Hudec suggested that the presence of deicing salt increases the potential for osmotic pressures, expansion and breakdown of the aggregate particles (28). Some fine-grained dolomites were found susceptible to D-cracking in the presence of deicing salts (28, 29). Dubberke and Marks studied the effects of deicing salt on carbonate rocks by boiling aggregate specimens in three different solutions: distilled water, calcium chloride and sodium chloride. Their study did not prove that the aggregate was weakened by deicing chemicals, but it suggested that deicing salts produce chemical and crystallographic changes within some aggregates that may lead to their deterioration (30).

Dubberke noted that trace constituents (e.g., magnesium, iron, sulfur and cryptocrystalline chert) may also contribute to the D-cracking deterioration of pavements subjected to deicing salts (31). Dubberke and Marks reported that trace elements of strontium or phosphorous also appear to influence the frost susceptibility of carbonate aggregates. They suggested that these trace elements contribute to chemical reactions and may alter and weaken the crystalline structure of the carbonate aggregate and the cement paste (32).

Pore structure

Pore structure is the most important factor influencing the susceptibility of coarse aggregates to D-cracking. The characteristics of aggregate pore structure include porosity, permeability and pore size distribution. Several researchers have reported that aggregate pore characteristics affect the durability of concrete by determining or influencing the aggregate absorption capacity, absorption rate and the ease of draining, internal surface area, and bulk volume occupied by solids, the quality of the bond with the cement matrix, the osmotic and hydraulic pressures developed by freezing and thawing, and their effects on the freezing temperature (20, 33, 34). Kaneuji reported that lower freeze-thaw durability is expected for aggregates with large pore volumes or small pore diameters (i.e., for pore sizes larger than 1 μm and not smaller than 45 Å) (35). Marks and Dubberke reported that almost all nondurable aggregates have a large proportion of pore diameters between 0.04 and 0.2 mm (36). Other researchers have concluded that the pore size range for nondurable coarse aggregates is from 0.008 to 8 microns, although no correlation was reported between pore size distribution alone and service records (37, 38, 39). Mehta and Montiero have reported that aggregates with very fine pore size distributions (i.e., < 1 μm in diameter) seem to be highly associated with D-cracking (40).

The degree of saturation of aggregate particles in concrete is influenced by their pore structure (and other factors). Dolch observed that the rate of increase in the degree of saturation and the ratio of absorption to permeability affect the frost susceptibility of concrete aggregates (33).

Absorption and Adsorption

Absorption is the assimilation of water into the pores of the aggregate and adsorption is the adherence of water to the surface of the aggregate. Materials that are impermeable are not susceptible to D-cracking, and absorptive but relatively permeable aggregates will not be disrupted when frozen if the freezing rate is slow enough to allow water movement through the particle to escape boundaries ahead of the freezing front (13).

Dolch reported that nondurable Indiana limestone aggregates had high absorption values and high rates of saturation; he concluded that the ratio of absorptivity to impermeability and the rate of saturation are two indices of frost susceptibility (33). Stark reported that an adsorption value of less than 0.1 percent identifies a nondurable aggregate, and that aggregates with high absorption and adsorption are susceptible to D-cracking and popouts (4). Durable aggregates with low absorption and high adsorption values might become saturated but do not fail because they cannot contain sufficient moisture to become overstressed during freezing (1).

Particle Size

Coarse aggregate particle size can influence aggregate susceptibility to D-cracking: the smaller the nominal maximum size, the better the freeze-thaw durability (1). The size of the aggregate is critical because it affects the length of the flow path that the water must travel when being expelled from an aggregate particle subjected to freezing. Smaller particle sizes generally have shorter paths and develop less hydraulic pressure. Stark and Klieger verified this, reporting that the durability of concrete pavement was improved when the nominal maximum size of crushed limestone aggregates was reduced, and that the rate of the development of D-cracking, as documented through both service records and freeze-thaw testing, was also reduced (41).

Bulk Specific Gravity

It has been reported that lower coarse aggregate bulk specific gravity values can be associated with higher susceptibility to D-cracking (35, 42, 43). Specific gravity might be a good indicator of freeze-thaw durability because it is an indicator of both aggregate porosity and particle strength. However, others have reported that bulk specific gravity is not as good as other characteristics for predicting aggregate freeze-thaw durability (44, 45).

2.4.3 Fine Aggregates

In 1974, Klieger et al. reported that the source of the fine aggregate doesn't affect the freeze-thaw durability of concrete, even if the fine aggregate is obtained by crushing a nondurable coarse aggregate source (2). However, in 1985 Dubberke and Marks reported that adding 5 percent dolomite fines reduced the durability of concrete when the coarse aggregates were treated with salt. They offered this phenomenon as evidence of the limitations of ASTM C 666 in identifying chemical problems under freeze-thaw conditions (46).

2.4.4 Structural and Thickness Design of Pavements

Schwartz has reported that pavement design has little influence on the occurence of D-cracking, but it can influence the rate of deterioration of a D-cracked section (1). For example, D-cracking is more serious in continuously reinforced concrete pavement (CRCP) than in jointed concrete pavement (JCP), because the many transverse cracks in CRCP provide additional channels for the ingress of water to the concrete and aggregates, thereby facilitating more rapid deterioration (1, 47).

Overlaying PCC pavements with a thick asphalt concrete overlay is generally not sufficient to prevent freezing in the pavement and can actually accelerate the rate of deterioration due to a reduction in the cooling rate and an increase in the degree of saturation of the concrete (5, 48).

2.4.5 Subsurface Drainage

Several researchers have reported that better drainage has no effect on the development of D-cracking, although it may be effective in reducing deterioration rates (1, 2, 41). However, Glass concluded that a combination of improved drainage (to reduce the moisture available to the aggregate and the concrete through the base layers) and control of the aggregate characteristics through appropriate specifications (i.e., 0.05 percent expansion limit after 350 cycles of freezing and thawing using ASTM C 666 procedure B) should be expected to increase the life of concrete pavements (49).

2.4.6 Traffic

Several researchers have reported that traffic has little influence on the development of D-cracking, but that higher traffic and heavier loads accelerate the rate of deterioration of a pavement once D-cracking has developed (1, 43, 45). These cracks become additional channels for water and the action of traffic can help to achieve higher levels of saturation in the mortar and aggregates.

2.4.7 Use of Deicing Salts and Chemicals

Salts generally lower the freezing point of water and can, therefore, reduce the freezing of water and the development of hydraulic pressure. However, salts are aggressive to the sorption process and ultimately cause more freeze-thaw deterioration (14). Deicing salts and chemicals increase the severity of freeze-thaw deterioration due to the osmotic pressure caused by increased water movement, the pressure generated when salt crystallizes in large pores, and increases of the temperature gradient and associated stresses in the concrete (50). Crumpton et al. observed that salt treatment "corroded" limestone aggregates and the cement paste and altered the clays in limestone aggregates (51).

In 1987, Hudec reported that the grain size, pore size and total internal surface area of coarse aggregates have a major influence on aggregate freeze-thaw durability in the presence of deicing chemicals. The grain size of rock minerals determines the surface area available for water in the pores, which are formed between these grains. Deterioration due to freezing is explained by the formation and expansion of ice in the pores (hydraulic pressures) and the osmotic differences generated by ice formation and the increased concentration of unfrozen fluids due to the effect of deicing salt (28).

2.4.8 Summary

Cycles of freezing and thawing are essential for the development of D-cracking in concrete pavements; the cooling rate and freezing temperature are also important and affect the mechanisms by which D-cracking occurs. Sufficient moisture in the concrete aggregates is essential to the formation of D-cracking, and the movememt of water during freezing produces internal pressures which can result in the formation of cracks.

In order for D-cracking to develop, the concrete must contain sufficient amounts of unsound aggregates of the proper size. The D-cracking susceptibility of coarse aggregate is related to its mineralogy (i.e., its origin, grain size, clay content, trace elements and carbonate content). The porosity characteristics of aggregates (i.e., pore content, pore size distribution, permeability and surface area) are believed to have the greatest effects on its freeze-thaw susceptibility. Absorption and adsorption have often been used to indicate the D-cracking susceptibility of aggregates, since they measure the amount of water potentially available for the D-cracking mechanisms. Coarse aggregates with low specific gravity appear to be susceptible to D-cracking, although a direct relationship is not proven. Reducing the maximum size of D-cracking susceptible aggregate particles appears to improve the freeze-thaw durability of concrete and slow the rate of development of D-cracking.

Fine aggregates, traffic, pavement design, and subsurface drainage do not seem to have any significant effect on the development of D-cracking in concrete, although these factors may affect the rate of D-cracking deterioration.

Deicing salts and chemicals are commonly on concrete pavements and seem to have a detrimental effect on the freeze-thaw durability of the concrete and coarse aggregates, even though they generally lower the freezing temperature of water. The generation of osmotic and hydraulic pressures due to water movement and pore water crystallization are influenced by the presence of deicing chemicals and their concentrations.

2.5 Frost Resistance Tests for Coarse Aggregates

A major concern for many testing engineers is how D-cracking susceptible aggregates can be positively and quickly identified. Many tests have been developed to measure D-cracking susceptibility, although acceptance of these tests varies widely among researchers and state transportation agencies (1, 5, 6). Many tests identify D-cracking susceptibility only for certain types of aggregates and are very restrictive for other types of aggregates. Other more reliable testing methods require expensive equipment and are time-consuming. It is apparent that no single test or acceptance/rejection criterion is currently available for predicting the frost resistance of coarse aggregate in PCC pavements (1, 5). A fast, reliable, reproducible, easily-performed and inexpensive test or suite of tests is needed for identifying aggregate susceptible to D-cracking.

Measuring the performance of an aggregate with respect to freeze-thaw durability is usually done by considering field experience and the results of laboratory tests that simulate exposure to certain field conditions. The most widely-used tests to identify the susceptibility of coarse aggregate to D-cracking are the rapid freezing and thawing test (ASTM C 666) and the Powers single-cycle slow freeze test (ASTM C 671). More rapid tests are often considered incapable of accurately determining the acceptability of a coarse aggregate with respect to frost susceptibility, but they can be correlated with the results of freeze-thaw tests to estimate coarse aggregate freeze-thaw durability.

Tests used to predict the freeze-thaw durability of coarse aggregates can be separated into two major groups. The first group includes tests that simulate the conditions to which the coarse aggregates will be exposed in the field, while the second group of tests correlates the results of tests of aggregate properties or characteristics with field performance and simulative test results. Some of the most common tests of concrete and aggregate freeze-thaw durability are categorized into one of these two groups and are described below.

2.5.1 Correlative Tests

Correlative tests (often called "quick-screening" tests) relate a particular aggregate property or behavior with predicted freeze-thaw durability (based on field performance or laboratory tests) These tests are preferred by many agencies because they require relatively little time to perform (a few days to 2 weeks) and are often less expensive and easier to perform than the simulative tests. Correlative tests include the absorption and specific gravity tests (ASTM C 127), absorption-adsorption test (PCA Method), acid-insoluble residue test (ASTM D 3024), Iowa pore index test, Washington hydraulic fracture test, petrographic examination, x-ray diffraction test, x-ray fluorescence test, thermogravimetric analysis, and determination of pore size and volume by mercury porosimeter (ASTM D 4044).

Absorption and Specific Gravity Test (ASTM C 127)

The absorption capacity is a measure of the quantity of water (as a percentage of the oven-dry aggregate weight) that the aggregates absorb under atmospheric pressure. The specific gravity is the relative density of the aggregate when compared to the density of water. It is an indicator of both aggregate porosity and particle density and provides a rough measure of the ability of aggregate particles to withstand internal pressure.

These parameters are determined by immersing a representative sample of aggregate in water for 24 hours, bringing it to a saturated, surface-dry condition, and weighing it. The sample is then weighed in a submerged condition in a wire basket. The sample is then oven-dried for 24 hours, or until no further decrease in weight is observed, to determine the oven-dry mass. The absorption capacity and bulk specific gravity are determined using the following formulas:

$$\text{Absorption Capacity (percent)} = (B-A) * 100 / A \qquad \text{(Eqn. 2.1)}$$

$$\text{Bulk Specific Gravity (at } 23\,^{o}\text{C)} = A / (B-C) \qquad \text{(Eqn. 2.2)}$$

$$\text{Bulk Specific Gravity (SSD)} = B / (B-C) \qquad \text{(Eqn. 2.3)}$$

where:

A : oven-dry mass of the sample, grams

B : saturated, surface-dry (SSD) mass of the sample, grams

C : submerged weight of the sample, grams

Kaneuji concluded that the absorption test is a rough measure of pore volume and that a highly absorptive aggregate is likely nondurable, but that absorption is not a direct measure of D-cracking susceptibility (35). In 1991, Folsom reported that the Missouri DOT restricts coarse aggregate absorption to a maximum of 1.5 percent to ensure good freeze-thaw durability and that the Iowa DOT noticed that if the absorption is below 0.5 percent or higher than 3.5 percent, the source of coarse aggregate is likely not susceptible to D-cracking (10). The absorption criteria adopted in Minnesota for coarse aggregates (i.e., 1.75 percent maximum for carbonate sources) did not eliminate D-cracking problems in southern Minnesota and may have eliminated the use of some sound aggregates. Wallace also reported that the absorption test failed to clearly predict the susceptibility of Kansas limestone sources to D-cracking (52). As a result, Folsom concluded that there is no direct correlation between absorption and D-cracking (10).

In 1971, Missouri officials reported that limestone aggregates with a bulk specific gravity of less than 2.65 were often frost-susceptible (53). Others also reported that low specific gravity values correlated with poor freeze-thaw performance (42, 43, 54). However, Meininger et al. have previously reported that bulk specific gravity is not as good as the direct porosity measurement technique for detecting frost-susceptible aggregates (44). Furthermore, Bukovatz and Crumpton reported that no correlation was found between bulk specific gravity and D-cracking deterioration for Kansas limestone sources (45).

In summary, these studies show that although aggregate absorption is related to apparent porosity and the frost susceptibility of aggregates, absorption alone is not an adequate measure of such characteristics (although highly absorptive aggregates are often nondurable). Aggregate absorption may be useful in combination with other test results to predict the frost susceptibility of an aggregate source. Bulk specific gravity is also not a direct measure of freeze-thaw durability, although aggregates with low specific gravity are often nondurable.

Absorption-Adsorption Test (PCA Method)

Considering absorption to be a rough measure of coarse aggregate pore volume and adsorption to be a rough measure of its surface area, Klieger et al. developed absorption-adsorption criteria based on service records to estimate aggregate frost susceptibility (2).

The sample preparation consists of obtaining aggregate slices with thicknesses of 1.6 mm and 32 mm for adsorption and absorption measurements, respectively. The slices are obtained from aggregates with a 19- to 38-mm particle size.

The slices used for absorption are oven-dried for 48 hours, moved to a desiccator to cool to room temperature ($23 \pm 1^{o}C$), and the oven-dry weight is then recorded. The slices are then vacuumsaturated in boiled distilled water (100°C). On alternate days, the slices are removed, brought to a saturated, surface-dry (SSD) condition, and then weighed to obtain the SSD weight. The weights are measured using a balance sensitive to 0.0001 gram. The SSD weight procedure is repeated until no change in absorption is observed. The absorption is the difference between SSD weight and oven-dry weight divided by the oven-dry weight.

The adsorption of coarse aggregates is obtained from the 1.6-mm aggregate slices, which are first crushed to 2.36- to 1.18-mm particle sizes, vacuum-oven-dried, cooled to room temperature in a desiccator containing calcium chloride, and finally weighed. The sample is then subjected to a humidity level of 92 percent for one day (using a saturated solution of KNO_3 at 23°C), after which the weight of the sample is obtained. The weight is measured on a weekly basis until no change in weight is observed. The adsorption (percentage) is the ratio of the total gain in weight to the oven-dry weight, multiplied by one hundred.

Klieger et al. reported that durable aggregates should have an absorption capacity below 0.3 percent or an adsorption capacity below 0.1 percent or both (2). Other researchers have suggested that the absorption-adsorption criteria for acceptance are very restrictive, classifying some sources with good field records as potentially nondurable (35, 55). Kaneuji reported that the adsorption measure is sensitive to the volume of small pores and may lead to erroneous prediction of the aggregate freeze-thaw durability (35). Folsom reported that when Mn/DOT used the absorption-adsorption criteria, very few sources passed it and some aggregate sources with good field performance were classified as nondurable (10).

The absorption-adsorption characteristics of aggregate do not appear to be directly related to its freeze-thaw durability. This test should not be used alone to accept or reject an aggregate source; however, it may be useful in combination or conjunction with other tests.

Acid-Insoluble Residue Test (ASTM D 3024)

The acid-insoluble residue test is used to determine the percentage of insoluble residue in carbonate aggregates by using a hydrochloric acid solution to dissolve the carbonate fractions. The test procedure consists of dissolving at least 500 grams of oven-dried coarse aggregate sample in hydrochloric acid. Hydrochloric acid is added until no reaction is observed with additional acid; the mixture is then heated for one hour. Any solution that passes the No. 200 sieve (0.074-mm opening) is then refiltered through a rapid filtering paper placed on the top of a number 200 sieve or a glass funnel. Residue that passes the No. 200 sieve usually consists of non-carbonated clay or silt-sized particles. Residue coarser than the No. 200 sieve is generally quartz, pyrite or chert.

Lemish et al. and Shakoor have suggested that the insoluble residue test is reliable for predicting the frost susceptibility of argillaceous carbonates and that it shows good correlation with their field performance (38, 42). Shakoor also suggested that argillaceous carbonate aggregates with more than 20 percent total acid-insoluble residue should be rejected (42). In 1991, Folsom reported that almost all Mn/DOT coarse aggregate sources with a minus-200 portion insoluble residue content of 8 percent or less met all other specifications for concrete aggregate acceptance (10).

Shakoor observed that the nature, amount and manner of distribution of insoluble materials strongly affect aggregate freeze-thaw durability. Shakoor noted that rocks containing insoluble clay, evenly distributed through the rock, are less durable than those containing even higher amounts of insoluble materials consisting of silty streaks and laminations (42). Dubberke and Marks also suggested that the size and chemical composition of the insoluble residue, rather than its percentage, control the durability deterioration (30).

Dubberke noted that the use of the acid-insoluble residue test method makes it very difficult to accurately measure the activity of the clay content (31).

In 1982, the Kansas DOT used the absorption capacity (described previously) and insoluble residue test results to develop a Pavement Vulnerability Factor (PVF), which is defined as:

$$PVF = \frac{100 * A}{A + \frac{B}{0.3846}} \qquad \text{(Eqn. 2.4)}$$

where:

A = percentage by weight of acid-insoluble residue

B = water absorption (percent)

When the PVF is less than 35, the coarse aggregate is considered durable; otherwise, further testing is needed to determine the acceptance or rejection of the source (56). In 1990, Wallace reported that the use of a combination of a maximum PVF value of 35, a maximum allowable acid-insoluble residue of 3.5 percent and a minimum freeze-thaw durability factor of 95 successfully eliminated all sources of nondurable aggregate (52).

In summary, the acid-insoluble residue can be used to determine the relative amounts of clay and silt in the aggregate particles. This test should not be used alone to determine the frost susceptibility of coarse aggregates, but it appears that it may be useful in combinations with other tests for the prediction of coarse aggregate freeze-thaw durability.

Iowa Pore Index Test

The Iowa pore index test is performed by placing a 9000-gram sample of oven-dried coarse aggregate in a pressure vessel that has been fitted with a calibrated stand pipe, which is then filled with water and pressurized to 240 kPa. After one minute of pressurization, the volumetric drop in the stand pipe water column (the "first reading" or "primary load") is measured. The primary load reflects the amount of water required to fill the aggregate macropores. A second reading is taken after 15 minutes of pressurization, and the difference between the two readings is referred to as the "secondary load" or "pore index." The secondary load reflects the amount of water required to fill the aggregate micropore system (pores in the 0.04 to 0.2 micron diameter size range). A secondary load greater than 27 ml is believed to indicate susceptibility to D-cracking. When the aggregate volume exceeds the capacity of the pot, a sample of 4500 grams is used and the measured primary and secondary loads are doubled (36).

Traylor reported that Iowa pore index test results were well-correlated with the durability performance of crushed stone aggregates, but not with that of gravel aggregates (6). In a study that compared the effectiveness of different types of aggregate durability tests, Shakoor and Scholer concluded that the Iowa pore index test is a simple, economical and reliable test for isolating unsound argillaceous carbonates and can be used with the insoluble residue test to predict the durability performance of coarse aggregates that contain more than 20 percent silt or clay (55). Glass reported that the majority of Kentucky aggregate sources with high pore indices failed the rapid freezing and thawing test, but that the Iowa pore index test was not effective in identifying the susceptibility to D-cracking of aggregates with a low pore index (49). Winslow reported that the Iowa pore index test does not accurately discriminate the durability of aggregate with relatively rapid rates of early absorption (57).

The quality number (QN) was introduced by Marks and Dubberke in 1982 as an additional tool for discriminating between durable and nondurable coarse aggregate (36). The quality number was defined by the following expression:

$$QN = 0.055 \times [SL \times (SL + PL)] / PL \qquad \text{(Eqn. 2.5)}$$

where:

QN = quality number,

SL = secondary load (ml), and

PL = primary load (ml).

High quality numbers (i.e., QN > 2.2) are generally considered to indicate nondurable aggregate, while low quality numbers (i.e., QN < 1.5) were correlated with good durability performance.

In summary, the Iowa pore index test predicts the incidence of D-cracking by providing a measure of the volume of microscopic pores in a given mass of aggregate particles, but it does not take into consideration environmental effects. Although, this test is simple and economical, the degree of reliability varies with aggregate type and origin (e.g., argillaceous carbonates, crushed limestones, gravel, etc.).

Washington Hydraulic Fracture Test (AASHTO TP 12)

The Washington hydraulic fracture test (WHFT) is a test that was originally developed at the University of Washington for identifying aggregates that are susceptible to D-cracking. This test was developed to simulate the hydraulic pore pressures developed during freezing by submerging a sample of oven-dried aggregates in water and subjecting them to high pressure to intrude the water into the aggregate pore structure and compress the air entrapped within those pores. The pressure is then released explosively, causing the entrapped air to expand and push the water through the pore structure. Aggregate fracture occurs if the pore pressure is not dissipated quickly and the aggregate is not able to resist the resulting high internal pressure (58). A more detailed description of the original test follows.

Prior to testing, aggregate particles ranging in size from 19 to 25 mm are washed, oven-dried for at least 12 hours, immersed for 30 seconds in a solution of water-soluble silane sealer, drained, and then oven-dried for at least 12 hours. A set of 3 samples, each weighing approximately 3200 grams and containing 150 to 225 particles, is used. Each sample is placed in a rock tumbler for at least one minute in order to fracture any particles weakened in the crushing process prior to testing. The initial weight and number of particles are then recorded. The sample is placed in a pressure vessel, which is then sealed. The vessel is then filled with water and a compressed nitrogen source is used to apply 7930 kPa of pressure for 5 minutes; the pressure is then released explosively. The chamber is then refilled with water, pressurized again for 2 minutes, and again depressurized explosively. The 2-minute pressurization-depressurization cycles are repeated until a total of 10 cycles of pressurization and release have been completed. The sample is then removed from the chamber, oven-dried for at least 12 hours, tumbled, weighed, sieved and counted. The sample is then subjected to the same procedure for an additional 4 days for a total of 50 cycles. After each 10 cycles, the material passing the 9.5-mm sieve is separated and excluded from any further pressure testing and is not included in the weights and counts (5).

The Hydraulic Fracture Index, HFI, is then computed to indicate the number of cycles needed to produce fracturing in 5 percent of the sample particles.

The "Percent Fracture", PF, was obtained using the following equation (58):

$$FP_i = 100 \times (n4_i / 2 + n_i - n_0) / n_0 \qquad \text{(Eqn. 2.6)}$$

where:

FP_i = the percent fracture after "i" cycles;

$n4_i$ = the cumulative number of pieces passing the 9.5-mm sieve and retained on 4.75-mm sieve after "i" cycles;

n_i = the cumulative number of particles retained on the 9.5-mm sieve after "i" cycles; and

n_0 = the initial number of particles tested.

The Hydraulic Fracture Index, HFI, was defined as the number of cycles needed to produce 5 percent fracture. When 5 percent fractures occurred in 50 or fewer cycles, HFI was calculated using the following equation:

$$HFI = A + 5 [(5 - FP_A) / FP_B - FP_A)] \qquad \text{(Eqn. 2.7)}$$

where:

A = the number of cycles prior to the 5 percent fracture,

FP_A = the percentage of fracture prior to the 5 percent fracture, and

FP_B = the percentage of fracture after the 5 percent fracture.

When 5 percent fracture did not occur within 50 cycles, HFI was determined as follows:

$$HFI = 50 \times (5 / FP_{50}) \qquad \text{(Eqn. 2.8)}$$

where:

FP_{50} = percentage fracturing after 50 cycles.

Aggregates with high HFI (e.g., >100) were considered not to be susceptible to D-cracking, while low HFI (e.g., < 50) values were believed to correlate strongly with poor frost resistance (5).

In a later study, Snyder, et al. proposed a rejection criteria of 2.5 percent fracture or more for gravel aggregates, acceptance of aggregate sources with a weight loss of 0.2 percent or less for carbonate sources, and rejection of all carbonate aggregate sources with a mass loss of 0.5 percent or higher (59). The percent mass loss, ML, is determined by the following equation:

$$ML_i = (100 / m_0) \times [\, m_0 - (m4_i - m_0)] \qquad \text{(Eqn. 2.9)}$$

where:

ML_i = the percent mass loss after "i" cycles;

m_0 = the initial mass of sample;

$m4_i$ = the cumulative mass of materials passing the 9.5-mm sieve and retained on the 4.75-mm sieve after "i" cycles; and

m_i = the cumulative mass of materials retained on the 9.5-mm sieve after "i" cycles.

In summary, the Washington hydraulic fracture test is a rapid and inexpensive test (when compared with freeze-thaw testing using ASTM C 666) for determining the freeze-thaw durability of coarse aggregates. At the time of this research study, the WHFT appeared to be a promising test, but further testing and validation was required.

[By the time that this report was being prepared for publication, additional developmental work had been performed on the WHFT to address concerns that arose as a result of nationwide round-robin testing. The results of this work are described in Reference 60. The basic test mechanism of pressurization and depressurization has not changed, but the tested aggregates are separated into several particle size ranges and the mass of the material retained on each sieve is correlated with the results of freeze-thaw testing through a regression equation. The Hydraulic Fracture Index (HFI) is no longer used. The hydraulic fracture testing used and described in later sections of this report was the original Washington Hydraulic Fracture Test (WHFT), not the most current HFT.]

Petrographic Examination (ASTM C 295)

Petrographic examination consists of examining aggregate to observe and analyze properties such as mineralogy, bond and texture, crystalline interlock, chemical characteristics and reactivity, and pore structure. Petrographic examination is often used to analyze the properties of aggregates that affect concrete durability or predict its freeze-thaw performance (61). Petrographic examination can also provide information about the existence of constituents deleterious to freeze-thaw durability, such as soil, clay, shale, weathered chert and argillaceous limestones. Tremper and Spellman concluded that petrographic examination can be a valuable tool in determining aggregate soundness when freeze-thaw test results are provided (62). The primary disadvantages of the petrographic examination are the need for a well-trained and experienced petrographer and the potential for error due to the subjective nature of the test and the extrapolation of results based on the use of small sample (5, 63, 64).

X-Ray Diffraction Test

The x-ray diffraction test is an easy, fast and convenient test for determining the major and minor compounds of an aggregate sample by measuring the spacing of the crystallographic planes. The test procedure and equipment used are described in detail in a paper by Dubberke and Marks, who reported in 1989 that dolomitic aggregates that yield a d-spacing greater than 2.899 are generally nondurable (32).

X-ray diffraction was used earlier by Mayo (54) and Tipton (3) to determine the amounts of dolomite, limestone and quartz, and the calcite-dolomite ratio. They reported no significant correlation between x-ray diffraction test results and freeze-thaw durability.

The potential of this method for predicting D-cracking appears to be very limited because of the high cost of the required equipment and the lack of broad-based evidence that mineralogy contributes significantly to the development of D-cracking.

X-Ray Fluorescence Test

This test consists of using a sequential x-ray spectrometer on an aggregate sample and measuring the x-rays and secondary electrons emitted from the sample to determine the elemental components of carbonate aggregates. It can also be used to quantify the minor, major, and selected trace elements present in aggregate samples. The test procedure and equipment used are described in detail in a paper by Dubberke and Marks (32), who concluded that Iowa limestone aggregates with large, open pore structures, and aggregates with a strontium content below 0.013 percent and a phosphorous content below 0.010 percent are not susceptible to D-cracking. Limestone with a strontium content of more than 0.050 percent was expected to perform poorly. Dubberke and Marks related freeze-thaw deterioration to the presence of these trace elements, which they felt might contribute to chemical reactions that alter and weaken the crystalline structure of the carbonate aggregate and the cement paste (32).

Thermogravimetric Analysis

Thermogravimetric analysis (TGA) consists of testing a 54- to 57-milligram ground (pulverized) aggregate sample placed in a platinum pan and suspended on a micro-scale. Mass loss is measured as the sample is heated to its transition temperature. The heating procedure consists of rapidly bringing the sample to a temperature of 300°C and then increasing the sample temperature at a rate of 40 degrees per minute until a significant mass loss occurs. Significant mass losses are typically obtained after calcite and dolomite transitions (i.e., burning off and loss of carbon dioxide). Dubberke and Marks reported that durable limestone generally exhibits little mass loss prior to calcite transition at 905°C; nondurable limestone loses significant mass starting at approximately 600°C (65). Durable dolomite begins to exhibit mass loss at approximately 570°C and continues to lose weight at a more rapid rate until it reaches its transition temperature (705°C). Nondurable dolomite loses little mass before reaching a temperature of about 700°C, after which a second mass loss, continuing at a greater rate, is observed from about 740°C to 905°C.

Dubberke and Marks reported that a relationship between aggregate grain size and chemical reactivity (prior to the freeze-thaw activity) might be used to explain freeze-thaw damage. They suggested that the slope of the mass loss from TGA prior to the burning of calcite or dolomite might be used as an indicator of the chemical reactivity of the aggregate (65).

Dubberke and Marks suggested that the TGA test is repeatable and very accurate if the sample size, rate of heating, and test method are held constant (65). They reported that TGA test results correlated well with the field performance of carbonate aggregates in Iowa. Non-carbonate fractions can also be tested using TGA and should correlate well with results of the acid-insoluble residue test. Thus, Dubberke and Marks concluded that TGA has good potential for determining freeze-thaw durability of carbonate aggregate and could be used in combination with other tests to predict their performance. Schlorholtz and Bergeson reported that TGA results correlated better with service records of concrete pavements than did the results of rapid freezing and thawing tests (ASTM C 666) and acid-insoluble residue tests (29).

Thermogravimetric analysis (TGA) is an easy test to perform and preliminary results of tests performed by the Iowa DOT are promising. However, it does not consider the particle strength, the effects of different aggregate types in one sample or the effects on the mortar-aggregate system. In addition, TGA requires very expensive equipment and the sample size tested is very small (requiring, in some cases, the analysis of several samples to accurately represent a single source of aggregate).

Pore Size and Volume by Mercury Porisometer (ASTM D 4044)

The pore size distribution is often considered the most important parameter affecting aggregate durability (3, 42). Several researchers attribute aggregate freeze-thaw durability to the presence or absence of certain pore size ranges (16, 35, 36, 37).

The mercury porisometer is used to measure the pore size distribution of coarse aggregates by measuring the amount of mercury that can be forced into aggregate pores at various pressures, with higher required pressures corresponding with smaller pore sizes. The test is described by ASTM D 4044 and consists of applying a mercury intrusion pressure of up to 41.4 MPa to determine the volume of pore sizes in the range of 0.0025 mm to 500 mm, which covers the range of pores typically present in coarse aggregate particles. The test is conducted by placing a 0.5- to 4-gram sample in a pressure chamber filled with mercury, selecting and applying a series of increasing pressures, measuring the intrusion at each step after intrusion equilibrium is reached, and calculating the pore volume and size at each step.

Several other pore parameters can also be measured or estimated using porisometry data, including effective porosity, total porosity, bulk density, specific surface area, average pore radius and pore geometry.

Kaneuji based his work on the assumption that the pressure required to fill a pore is a function of the geometry of the pore and the surface properties of the liquid and the solid (35). He developed a correlation between the pore size distribution and rapid freezing and thawing test results (ASTM C 666 procedure A) which is described by the Expected Durability Factor (EDF), which he defined as:

$$EDF = (0.579/PV) + 6.12*MD + 3.04 \qquad \text{(Eqn. 2.10)}$$

where:

PV = intruded volume of pores larger than 45 Å in diameter, cc/g; and

MD = median diameter of pores larger than 45 Å in diameter, μ, as measured by the mercury porosimeter.

Based on field performance, Kaneuji suggested the following criteria for discriminating between durable and nondurable aggregates (35):

EDF	**Predicted Durability**
up to 40	Nondurable
40 to 50	Marginal
over 50	Durable

Additional research was done by Winslow, et al. suggested an EDF acceptance criterion value of 50 with a condition that 90 percent of the coarse aggregate have an EDF value above 50 (57). Shakoor and Scholer reported that the mercury intrusion porosimetry and the Iowa pore index test provided similar indications of durability for 30 aggregate samples (55). They also pointed out that the Iowa pore index test is a less expensive and quicker test of larger aggregate samples. However, unlike mercury intrusion porosimetry, the Iowa pore index test does not provide any direct information about pore size distribution.

Janssen and Snyder reported that the mercury intrusion porosimetry has various drawbacks, including (5):

- the Washburn's equation used in the analysis of data assumes that the pores are cylindrical and interconnected, which is often not the case for aggregates;
- the contact angle and surface tension of the mercury are assumed;
- the samples are not necessarily representative because of their small size;
- the equipment is expensive and requires trained people and special handling; and
- the tested specimen is contaminated with mercury, which is a hazardous material, resulting in specimen disposal problems.

In addition, Illinois, Indiana and Ohio reported that the results of mercury intrusion porosimetry did not exhibit a good correlation with field performance (1, 3).

In summary, it seems likely that the mercury intrusion porosimetry test accurately measures pore size distribution, which is believed to be strongly correlated with D-cracking potential. However, it does not consider particle strength, environmental effects or the effects of the mortar-aggregate interface system. The need for a trained operator, the high cost of equipment, the hazard of working with mercury, and the lack of strong correlations with freeze-thaw test results have limited the use of this method.

2.5.2 Simulative Tests

Tests that simulate environmental freeze-thaw exposure conditions are generally considered to be better correlated with the field freeze-thaw performance of the coarse aggregates than the tests described previously; thus they are more widely used to reject or accept coarse aggregate sources for concrete applications (1). Simulative tests are often time-consuming, however, sometimes requiring months of testing after sample preparation and curing, and they often require expensive equipment (26). The following environmental simulative tests were selected for consideration in this study: the rapid freezing and thawing test (ASTM C 666), the aggregate particle freeze-thaw test (ASTM C 131), the Powers' slow cool test (ASTM C 671), the sulfate soundness test (ASTM C 88) and the Virginia Polytechnic Institute (VPI) single-cycle slow freeze test.

Rapid Freezing and Thawing Test (ASTM C 666)

Rapid freezing and thawing of concrete prisms is the test most commonly used for identifying aggregates that are susceptible to D-cracking. Many agencies and researchers believe this test to be the most reliable indicator of the relative durability of an aggregate (1). However, rapid freezing and thawing tests have been criticized because of their accelerated nature, inexact replication of field conditions (i.e., use of a rapid cooling rate, different moisture condition of the aggregates, limitations on the maximum particle size of the aggregates), duration of the test (up to five months from casting the concrete specimen to completion of the test), the high cost of purchasing, maintaining and operating the equipment, and the limited availability of guidance in establishing aggregate acceptance or rejection criteria (5, 23).

Two procedures are currently approved by the ASTM and both can be used to test beams, cores or cylinders of concrete, although concrete beams of 75 x 100 x 400 mm are widely used (1). ASTM C 666 procedure A consists of freezing and thawing in water, and procedure B consists of freezing in air and thawing in water. Both procedures consist of repeatedly lowering the temperature of the specimens from 4.4°C to -17.8°C and then bringing it back to 4.4°C within 2 to 5 hours. This temperature cycling is repeated for up to 300 cycles, or until the relative dynamic modulus of the PCC is reduced by 40 percent of its initial value, or until the specimen dilates by 0.10 percent, whichever occur first. The damage done to the concrete specimens in terms of length change, loss of mass and loss in the stiffness or relative dynamic modulus of elasticity (ASTM C 215) is assessed after every 36 (or fewer) cycles.

Procedure A is preferred by some agencies because 1) the concrete specimens are kept saturated when frozen and 2) the specimens are not allowed to dry during freezing, which might slow the accumulation of freeze-thaw damage. However, several problems have been reported when procedure A was used, such as the physical confinement of specimens by ice within rigid specimen containers (which may cause damage to the specimens), difficulties in maintaining a uniform water layer thickness around the specimen, and damage to the containers and specimen gage pins (used to measure expansion of the specimen).

Procedure B is preferred by many agencies because 1) there is reduced potential for specimen damage, which can be induced by the containers typically used in procedure A, and 2) the time required to perform each test cycle is generally shorter for procedure B.

In 1985, Dubberke and Marks developed a modification of ASTM C 666 procedure B to investigate the effect of salt treatment of aggregates prior to rapid freezing and thawing test (46). The salt treatment consists of subjecting the coarse aggregates to five cycles of oven-drying for 24 hours at 110°C and immersing them in a saturated solution of pure reagent sodium chloride for 24 hours at a temperature of 23°C (the salt brine solution is poured over the aggregate immediately after it is removed from the oven). After the final salt treatment, the coarse aggregates are rinsed with clean tap water. The salt treatment of coarse aggregates was proposed by Dubberke and Marks to simulate and account for the exposure of the PCC concrete and coarse aggregates to deicing salts. They reported that the use of salt-treated aggregates and ASTM C 666 procedure B better simulated the detrimental effect of salt on freeze-thaw performance of concrete and yielded better correlation with service records than either Procedures A or B alone.

In 1994, Janssen and Snyder proposed a new procedure (dubbed Procedure C) in an attempt to better simulate field exposure conditions and in response to the major criticisms to procedures A and B as described above (5). Procedure C is the same as procedure B except that the specimens are subjected to freezing and thawing while wrapped in absorbent cloth wraps to keep the specimens wet during freezing without the confining effects of expanding ice in a rigid container. This procedure was developed to overcome the perceived shortcoming of procedures A (i.e., the confining effects of ice and the specimen containers, the longer freeze-thaw cycle time, etc.) and B (surface drying before freezing) and provide more reproducible results.

In any of the procedures described, the relative dynamic modulus, length change and/or mass change of the specimens is determined after every 36 cycles of freezing and thawing (or more frequently). The following performance measures can be computed:

DF_F: durability factor using relative dynamic modulus criteria;
DF_L: durability factor using dilation criteria; and
d_L : percent dilation after failure (RDM = 60 percent) or 300 cycles.

The dilation, d_L, can be calculated as:

$$d_L = 100*(L_2 - L_1)/(L_1 - 2g) \qquad \text{(Eqn. 2.11)}$$

where:
d_L : percent change in length of specimen after cycle c;
L_1 : length reading at cycle 0;
L_2 : length reading at cycle c; and
g: the length of each embedded gage stud.

The relative dynamic modulus of elasticity, P_c , is determined using ASTM C 215 procedures and is calculated as follows:

$$P_c = 100*(n_c^2/ n_0^2) \qquad \text{(Eqn. 2.12)}$$

where:
n_c : fundamental transverse frequency at cycle c, and
n_0 : fundamental transverse frequency at cycle 0.

The Durability Factor (DF) is calculated as:

$$DF = P*N/M \qquad \text{(Eqn. 2.13)}$$

where:

P: relative dynamic modulus of elasticity at N cycles, percent;

N: number of cycles at which the test ended (RDM = 60 percent or 300 cycles, whichever occurs first);

M: specified number of cycles at which test procedure is terminated (usually 300 cycles).

Some highway agencies have established failure criteria based upon correlations with field performance. For example, the Ohio DOT uses a failure criteria of 0.032 to 0.035 percent dilation per hundred cycles after 350 cycles (1); others reported that both Indiana and Illinois use a failure criteria of 0.06 percent expansion after 350 cycles (47, 66), and Glass reported that Kentucky uses a failure criteria of 0.05 percent dilation (49). Mayo (54) reported that a durability factor above 80 can be considered to be good and Tipton (3) reported that an aggregate with a durability factor of 60 or less should be considered nondurable.

In summary, the rapid freezing and thawing test (ASTM C 666) is the most common, fully developed and reliable test for predicting aggregate or concrete susceptibility to freeze-thaw damage. It has been used successfully for a large variety of aggregates and environments, but is a very time-consuming and expensive test.

Unconfined Aggregate Particle Freeze-thaw Test (ASTM C 131)

This method consists of freezing and thawing of aggregates in water, an alcohol-water mixture (usually 5 percent), a water-salt mixture (usually 3 to 5 percent), or in air (freezing) and water (thawing) (3, 24). The aggregate samples are placed in plastic bags and are totally covered with the test solution. Fifty cycles of freezing and thawing are used when testing in water or water-salt mixtures, and 16 cycles are used when testing in water-alcohol mixtures. Alcohol is believed to increase the deleterious effects of freezing and thawing of aggregates, thus reducing the number of cycles required to accomplish the same results (10). Mass loss is measured in terms of the percent mass or weight of sample that will pass a sieve smaller than the size upon which it was originally retained. The material passing the sieves is an indicator of the deterioration of each size fraction. A 10 percent loss (by weight) was often used as the borderline between durable (i.e., loss < 10 percent) and nondurable (i.e., loss > 10 percent) sources (10).

The unconfined aggregate particle freezing and thawing test has been criticized because of poor correlation with field freeze-thaw performance for carbonate aggregates, the length of time required for testing, failure to simulate the effects of aggregate confinement by mortar, and the difficulty in replicating test results due to test variables such as cooling rate, final temperature, rate of thawing, moisture condition prior to freezing and the duration of the freezing and thawing period (5, 24, 39, 42, 67).

Folsom reported that some agencies think that this method is very effective in measuring the "dirtiness" of aggregates (10). He also reported that Mn/DOT used this technique in the past; however, it was discontinued because the magnesium sulfate test gave comparable results and was easier and quicker to perform.

In summary, although the unconfined aggregate particle freeze-thaw test subjects aggregates directly to freeze-thaw actions, the results of this test do not correlate well with service records. This test should not be used alone to accept or reject a source of aggregates; however, it may be useful when used in combination or conjunction with other tests.

Powers' Slow Cool Test (ASTM C 671)

In 1955, Powers considered rapid freezing and thawing tests very severe when compared to field conditions because they do not account for drying conditions and might reject a durable aggregate source (68). He developed the slow cool test (ASTM C 671), which consists of maintaining concrete specimens in a water bath at a constant temperature of 2°C, and then submerging them in a water-saturated kerosene bath where they undergo a change of temperature from 2°C to - 9.5°C at a rate of -15°C per hour. The temperature and length change are measured during the cooling cycle, after which they are put back in the constant temperature water bath. The procedure is repeated every other week. The critical dilation is the dilation that occurs during the last cycle before it begins to increase sharply (i.e., when the expansion rate has increased by a factor greater than or equal to two). The period of frost immunity is determined by the number of cycles for which the dilation has remained constant. The test is terminated when the critical dilation is reached. Some highly frost-resistant aggregates may never produce critical dilations in this test.

Verbeck, et al. reported that the rapid freeze-thaw test correlated better with field conditions than the critical dilation test (69). Kaneuji reported that this test correlated very well with the field performance, but that the condition of the aggregate and the sample, cooling rate and the curing time are not representative of field conditions (35). In addition, this test requires a long testing time and expensive equipment (5).

In summary, the Powers' slow cool test was developed to overcome some of the deficiencies of the rapid freezing and thawing test. However, this test also presents some disadvantages such as expensive equipment, long testing time and questionable correlation with field performance.

Sulfate Soundness Test (ASTM C 88)

The sodium or magnesium sulfate soundness test is a correlative test that uses the growth of sulfate crystals in the aggregate pore structure to simulate the growth of ice crystals. The test consists of alternately saturating a uniformly graded aggregate sample in a magnesium or sodium sulfate solution and drying it in an oven. After each 5 or 10 cycles, the sample is weighed and sieved. Mass loss is measured in terms of the amount of sample that will pass a sieve smaller than the size upon which it was originally retained; this material is considered an indicator of the potential freeze-thaw deterioration of the sample. Five to ten cycles are usually performed to complete this test. A 15 percent loss (by weight) is used by the Minnesota and Illinois DOTs as the borderline between durable (i.e., loss < 15 percent) and nondurable (i.e., loss > 15 percent) sources (10).

This test is favored among many agencies because of the simplicity of the equipment needed and the relatively short amount of time required to run it (24). The deterioration of the aggregate particle is caused by the growth of salt crystals in the pores, which simulates the effect of ice growth and the resulting expansive forces within the aggregate particles. However, the growth of salt crystals in pores is not analogous to the development of hydraulic pressure produced when the water attempts to leave the zone of freezing (11, 13). Several researchers have reported that sodium sulfate soundness is not a good indicator of aggregate freeze-thaw durability (42, 67, 70, 71, 72). Others have criticized the test because it does not account for the confined state of aggregate in concrete, which would resist unconfined expansion (5, 62).

In summary, the sodium or magnesium sulfate soundness test is a correlative test that uses the growth of sulfate crystals in the aggregate pore structure to simulate the growth of ice crystals. Lack of correlation with service records and freeze-thaw test results makes the use of this test questionable and its use has been discontinued by many state agencies.

Virginia Polytechnic Institute Single-Cycle Slow Freeze Test

This test consists of subjecting 75- x 100- x 400-mm (nominal size) concrete beams containing the subject coarse aggregates to a single cycle of freezing while measuring the dilation of the specimens. After 14 days of moist curing at 23°C, the transverse fundamental frequency, mass and length are recorded. The specimens are then quickly placed in a conventional freezer for a 3-hour cooling period. Strain measurements are taken using a multi-position strain gage to measure the length change between two gage studs located 250 mm apart. Length measurements are taken every 15 minutes while the specimen temperature drops from 21°C to 4.5°C, and every 5 minutes while the specimen temperature is between 4.5°C and - 9.5°C.

The cumulative change in length and the temperature change between 4°C and -6°C are used to determine the minimum temperature slope, b_l, which occurs when hydraulic pressures cause expansion due to ice formation. This expansion can counteract the natural contraction of the beam due to cooling. The units of b_l are in µm/°C. A temperature slope of zero or less implies that the aggregate used is susceptible to D-cracking, while a temperature slope above zero is considered inconclusive (73).

The time slope, b_t, is the minimum slope that occurs in a time interval of 20 to 60 minutes and has units of µm/hour. Faulkner and Walker reported that there is a relationship between the dilation which takes place over a period of time and the durability factor (73). Walker, et al. reported that when the time slope is less than -10.2, the freeze-thaw durability of the aggregate is questionable, but that when it is higher than 2.5, no further testing is required and the aggregate is durable. When the time slope is between 10.2 and 2.5, further testing is required to determine susceptibility to freeze-thaw damage (61).

Although no agency reports regular use of this test, results reported in the literature correlate very well with rapid freezing and thawing tests and field performance for known durable and nondurable sources (1, 24, 61, 73). Additional research is needed to investigate the frost susceptibility of sources with questionable performance. The time slope (b_t) and temperature slope (b_l) criteria can be used to determine the durability of aggregate sources in two to three weeks. Accelerated curing of the PCC specimens can significantly reduce the testing time.

2.5.3 Summary

This section summarizes available literature concerning various test methods and procedures for assessing the freeze-thaw damage potential of coarse aggregate intended for use in Portland cement concrete and discusses the relative abilities of these tests to accurately predict concrete aggregate D-cracking performance potential.

Many tests have been developed to assess aggregate susceptibility to D-cracking. These tests can be separated into two major groups. The first group (sometimes call simulative tests) consists of tests that simulate the conditions to which the coarse aggregates will be exposed in the field, while the second group of tests (sometimes called correlative tests) correlates the results of aggregate property characteristic tests with field performance and simulative test results.

The simulative tests are generally considered to be better indicators of coarse aggregate durability performance in the field and they are widely used to reject or accept coarse aggregate sources for concrete applications. Unfortunately, they generally require long test periods, expensive equipment, and highly skilled operators.

The rapid freezing and thawing test (ASTM C 666) appears to be the most common, reliable and fully developed test currently available for predicting aggregate susceptibility to D-cracking. It has been used successfully to test a wide range of aggregates that are used in environments representing different degrees of exposure. Local highway agencies have developed varying acceptance criteria based on field correlations; some agencies use this test in conjunction with other tests to accept or reject coarse aggregate sources. The principal drawback to this test is that it typically requires several weeks to complete.

The unconfined aggregate freeze-thaw test subjects aggregates directly to freeze-thaw conditions, but several agencies have reported that the results of this test do not correlate well with service records. There are also concerns about the reproducibility of results and differences between the test conditions and the field conditions. However, this test may still be useful in combination or conjunction with other tests.

Powers' slow cool test was developed to overcome some of the deficiencies of the rapid freezing and thawing test. Good correlation with field performance has been reported, but this test also presents some disadvantages, such as the need for expensive equipment, a potentially long test duration, and the need for a highly skilled operator.

The sodium or magnesium sulfate soundness simulates the effects of ice crystal formation in the pores using salt crystals. Although this test has been favored by many agencies because of the simplicity of the equipment and the short amount of time required, poor correlations between service records and freeze-thaw tests results were often reported. Detractors generally believe that the formation of salt crystals test does not adequately simulate the effects of freezing and thawing and that additional mechanisms are present in freeze-thaw deterioration.

The VPI single-cycle slow freeze test subjects concrete beams containing the test aggregate to a single, slow freeze cycle while length measurements are made to determine whether the aggregate particles will undergo destructive volume changes. Literature indicates that VPI single-cycle slow freeze test results have correlated well with field durability observations for known durable and nondurable aggregate sources, but additional research is needed to investigate the suitability of this test for aggregate sources with questionable durability. This test requires inexpensive equipment and a few weeks to perform.

Correlative tests are preferred by many agencies because they require less time to perform and are often less expensive and easier to perform than simulative tests. The reported accuracy of correlative tests results has varied widely among highway agencies and researchers, however.

The measurement of absorption capacity has been the simplest test for assessing the frost susceptibility of concrete aggregates. However, absorption capacity alone is not a measure of D-cracking potential and predictions based solely on absorption have been inconsistent and unreliable. Aggregate absorption capacity might be useful in combination with other test results for predicting the frost susceptibility of an aggregate source.

Bulk specific gravity is also not a direct measure of aggregate freeze-thaw durability, although aggregates with low specific gravity are often nondurable. Elimination of aggregate particles with low specific gravity is generally beneficial to the frost resistance of concrete.

The absorption-adsorption test measures two aggregate characteristics that are believed to be related to D-cracking potential. Unfortunately, the test results do not appear to consistently predict field observations of D-cracking, and the acceptance-rejection criteria have been reported to be conservative. Therefore, this test should not be used alone to accept or reject a source of aggregate; however, it may be useful in conjunction with other tests.

The acid-insoluble residue test is used to determine the amounts of clay and silt (non-carbonates) in the aggregate particles. Several researchers have found that these materials have a detrimental effect on frost resistance. This test should be used in correlation or combination with other tests for the prediction of coarse aggregate freeze-thaw durability.

The Washington hydraulic fracture test (WHFT) is a recently-developed, rapid and relatively inexpensive test for determining the frost susceptibility of coarse aggregates in which aggregate samples are subjected to simulated cycles of internal pressure similar to those developed under freeze-thaw action. Early studies reported that WHFT results can differentiate between many nondurable and durable aggregates, but that further testing might be required for marginal aggregates. More recent studies (completed after the laboratory portion of this study were performed) resulted in significant modifications to the test and strong correlation of the test results with the results of ASTM C 666 (Rapid Freeze-Thaw Test).

Visual inspection of aggregates (in term of lithology and individual particle properties) might be beneficial in determining the detrimental effects of these properties on frost resistance. Petrography is a useful tool for studying these effects and predicting the freeze-thaw durability of similar or different concrete aggregates.

X-ray analysis (i.e., x-ray diffraction and x-ray fluorescence) reveals the compounds and elemental composition of concrete aggregates. The presence of impurities or trace elements might affect the freeze-thaw durability can be determined from x-ray analysis.

Thermogravimetric analysis (TGA) is a very effective method for determining the presence of aggregate minerals that may contribute to D-cracking potential based on their mass loss characteristics as they are heated through various transition temperatures. TGA requires the use of very expensive equipment and trained personal, but the ability to rapidly identify mineral properties and impurities is believed to be potentially beneficial in predicting the frost resistance of concrete aggregate sources.

The mercury intrusion porosimetry test is a good test for measuring the pore size distribution of aggregate particles. D-cracking has been strongly associated with the presence of aggregate pores in a specific size range. The widespread use of this test has been limited by the need for a trained operator, the high cost of the equipment, the hazards associated with the use of mercury, and the relatively weak observed correlations with freeze-thaw test results and field observations of D-cracking.

2.6 Mitigation of D-Cracking

2.6.1 Background

D-cracking in PCC pavements can occur only when freeze-thaw cycles, moisture and a sufficient amount of unsound coarse aggregate are present. Mitigation of D-cracking requires the elimination of one or more of these conditions. In existing concrete pavements, the mitigation of D-cracking may require the full-depth repair of sections of the concrete pavement (to replace badly D-cracked areas) and the elimination of freeze-thaw conditions or the reduction of moisture (to levels below critical saturation) (5). In new construction, the mitigation of D-cracking may be possible by eliminating the use of susceptible aggregates or modifying the aggregates to improve their resistance to freezing and thawing.

This research project sought to develop mitigation methods for improving the frost resistance of concrete constructed using existing aggregate sources. The accomplishment of this goal would allow the continued use of local Minnesota aggregates (especially in southern Minnesota) and would reduce the associated pavement life cycle costs.

2.6.2 Identification of Nondurable Aggregate Characteristics

Previous sections discussed factors that affect D-cracking and frost-resistance tests. Eliminating or mitigating D-cracking in new construction may require one or more of the following:

- *Documentation of field experience and quarry site investigations.* This is accomplished by providing detailed information about the service records of each aggregate source and by establishing and maintaining a permanent petrographic record for each quarry bed or pit (52).
- *Identification of physical and mechanical properties of the aggregate particles that affect D-cracking*, such as mineralogy, porosity, absorption, size, and specific gravity. This information is often useful for selecting the most appropriate mitigation technique.
- *Use improved materials and construction specifications.* This may include specifications concerning the aggregate, the mix, the maximum size of the aggregate, and testing to determine the acceptance or rejection of the aggregate source. This should be performed on a source-by-source basis for gravel and on a ledge-by-ledge basis for quarries of crushed stone.

2.6.3 Improvement

Several aggregate beneficiation techniques may be useful in mitigating D-cracking, including:

- reducing the maximum nominal size of coarse aggregate,
- selective quarrying,
- mechanical separation,
- blending,
- aggregate heat treatment,
- coating or impregnation, and
- improved concrete mix proportioning.

Reducing the Maximum Nominal Coarse Aggregate Size

Both laboratory and field observations have indicated that reducing the maximum size of coarse aggregate is a feasible technique for reducing D-cracking susceptibility (1, 2, 5, 13, 41). Some researchers recommend testing to determine the degree of frost susceptibility for individual sources of coarse aggregates and the potential benefits and size reduction necessary to produce a durable aggregate (1, 41). The Kentucky DOT uses freeze-thaw testing procedure ASTM C 666 Procedure B to determine the size reduction needed to meet the specification on a source-by-source basis and for individual production benches (49).

Schwartz reports that reducing the maximum aggregate size is not always effective, and some exceptions were recorded where the reduction resulted in poorer pavement performance when other modifications were not made (e.g., structural design, mix design, reinforcing design, etc.) (1). Some researchers reported that reducing the maximum coarse aggregate size tended to increase the frequency of transverse cracks, increase the severity of faulting, and reduce the grain interlock (74, 75).

Selective Quarrying

This technique involves selecting concrete aggregates by ledges (rather than by quarries) in order to obtain more consistent and uniform materials and to reduce the probability of intermixing nondurable rocks, such as clays and argilleous materials. Kaneuji reported that the pore size distribution of rocks, and hence their frost susceptibility, varied significantly from ledge to ledge within a given quarry (35). Selective quarrying requires sampling of the ledges, as well as monitoring of crushing and screening techniques. Wallace recommended maintaining the concept of bed approval to improve concrete durability and to ensure the "sameness" of the delivered concrete aggregates (52).

Mechanical Separation

This method separates coarse aggregates on the basis of specific gravity to eliminate materials with a specific gravity lower than a specified value. This is generally acheived by passing the aggregate stream through a heavy media bath. This beneficiation technique is based on the assumption that aggregate specific gravity is often correlated with freeze-thaw durability, and that the removal of low specific gravity particles (e.g., shales, cherts, sandstone, etc.) often eliminates the most nondurable particles. It has been reported that overall freeze-thaw durability improves when heavy media separation is used (1). However, Stark found this technique unreliable, and Arnold (1990) reported that the results varied significantly and the quality control of plant production was quite variable (4, 74).

Blending

This method effectively dilutes the effects of nondurable aggregate sources by blending a durable source with the nondurable. This reduces the amount of nondurable aggregate in the concrete mixture and often improves concrete freeze-thaw durability. Blending durable and nondurable concrete aggregates can be safe, economical, and beneficial if suitable proportions are used (42). Schwartz found that blending nondurable aggregate with more durable aggregate can upgrade the quality of coarse aggregates, and that laboratory freeze-thaw testing is a valuable method to investigate the potential benefits of this technique (1).

It should be noted that durable aggregate blends generally contain only small amounts of nondurable material, so successful blending may require much larger proportions of durable source material than nondurable source material. For example, Stark reported that blending 25 percent aggregate from a source considered nondurable with 75 percent from a source considered durable produced a blend with durability that was not improved from that of the nondurable source (4). Lindgren reported that blending 10 to 15 percent nondurable aggregate with durable aggregate results in poor concrete performance (76). Bukovatz and Crumpton found that blending more than 35 percent nondurable coarse limestone aggregates with durable aggregates is more likely to produce D-cracking than when blending less than 35 percent (45). Marks and Dubberke and Arnold reported that blending more that 10 percent of nondurable coarse limestone aggregates with durable aggregates is enough to produce D-cracking in concrete pavements (36, 74).

Aggregate Heat Treatment

This method heats coarse aggregates from a temperature of 450°C to 800°C in a rotary kiln, which dries the aggregate and provides a ceramic surface that prevents water absorption while promoting the bond between the aggregates and the cement matrix and improving concrete durability. For this treatment to be effective, the aggregate should not be susceptible to heat treatment; otherwise, fluxing agents should be used to provide the ceramic surface when heated. Heating the aggregates will produce a waterproof, insoluble, and weather-resistant surface coating for the aggregates (77).

Coating or Impregnation

This method consists of coating coarse aggregate surfaces with a thin film or impregnating it with a polymer, thin plastic films of thermosetting or thermoplastic materials (77). Aggregates can be coated by physical, chemical, thermal, or combined processes to prevent the intrusion of water or harmful materials into the aggregates, improve resistance to weathering, seal off penetrable pores after drying, or improve the bond between the aggregates and the cement paste. Some coatings and impregnants improved concrete frost resistance (e.g., epoxy and linseed oil emulsion coatings, epoxy, methyl methacrylate, boiled linseed oil, and polyethylene glycol impregnants)(78). Janssen and Snyder reported that treating PCC pavement cores with silane improved the durability factor for freeze-thaw tests; it also seemed that sealing concrete with silane reduced the D-cracking deterioration rate (5).

Concrete Mix Proportioning

Schwartz reported that the type and quantity of the cement does not appear to influence concrete durability (1); adequate air entraining should be used in concrete exposed to freezing and thawing, although it is very well known that this does not prevent D-cracking when enough unsound aggregates are used. However, increasing the fines content is also a mitigation option because it reduces the amount of coarse aggregates in the mix, which are primarily responsible for D-cracking.

Several researchers are now studying the effects of variation in cement content and composition, such as the inclusion of silica fume, fly ash and ground slag. Sabir and Kouyiali reported that the freeze-thaw performance of concrete containing condensed silica fume was somewhat inferior to that of concrete without it (79). Using a superplasticizer improved concrete freeze-thaw durability because of the resulting increase in air content, more favorable size and distribution of the air bubbles in the concrete, and the improved morphology of the hydration products (80). Adequate air entrainment, low water-cement ratio, and adequate curing are believed to yield an immune freeze-thaw concrete when sound aggregates are used (81, 82, 83). In 1994, Janssen and Snyder performed a broad-based study of the effects of various mix design parameters on the durability of concrete containing durable aggregate.

2.6.4 Prevention

D-cracking can generally be prevented by eliminating the use of unsound coarse aggregates, cycles of freezing and thawing, or the presence of moisture in the concrete. Eliminating nondurable aggregates requires expensive and time-consuming tests as well as the use of specifications to avoid using such materials from a given source. This approach, along with special consideration to concrete mixture proportioning and pavement, should be applied on a project-by-project basis to minimize D-cracking problems (1).

Preventing freezing in concrete pavements is not generally a cost-effective mitigation technique, even when accomplished using thick overlays. Preventing moisture infiltration into concrete pavements using sealers does reduce moisture in the concrete and reduces the rate of D-cracking development (5).

Other techniques are used to minimize D-cracking development by increasing the development time and reducing the rate of deterioration. Such techniques involve concrete mix proportioning, and modifications to pavement design parameters, including pavement type, base and drainage characteristics.

Testing

Nondurable aggregates can probably be eliminated from further use by using reliable tests of their freeze-thaw durability on a ledge-by-ledge basis for crushed aggregates and a project-by-project basis for gravel. Thus, preliminary information about the frost susceptibility of coarse aggregates should be compiled for initial consideration. Different mitigation techniques should also be tested to maximize the use of locally available sources and minimize production costs.

Material and Construction Specifications

Material and construction specifications guard against the use of D-cracking susceptible aggregates and help in designing the mixture for frost resistance. Coarse aggregate specifications may dictate the maximum size that can be used, the allowable presence and quantities of certain components in the coarse aggregates, the elimination of certain types of coarse aggregates, and the restriction or elimination of certain aggregate sources using physical characteristics such as specific gravity, absorption and frost resistance test results (1).

Concrete mixture design specifications should include the requirement of adequate air entrainment, consideration of the use of pozzolans and admixtures, fine aggregate content requirements, and the blending of durable and nondurable coarse aggregates (1).

Pavement Type

The type of portland cement concrete pavement is not generally considered to affect the development of D-cracking; however, it may influence the magnitude of the problem posed by full-developed D-cracking (i.e., D-cracking of continuously reinforced concrete pavements poses a great problem due to the relatively close spacing of transverse cracks). In addition, some mitigation techniques may produce other performance-related problems (e.g., reduced grain interlock potential and increased crack faulting associated with the use of reduced aggregate top size).

Base Type and Drainage

Selecting a proper pavement base will probably not eliminate the development of concrete pavement D-cracking; however, a good base can reduce the rate at which D-cracking develops and may also prevent other types of pavement distress due to loss of support and or heavy traffic load applications. The inclusion of a positive subsurface drainage system should increase the amount of time required for D-cracking to develop and may reduce the rate of continued deterioration of existing D-cracked pavements.

2.6.5 Summary

D-cracking can generally be prevented by eliminating unsound coarse aggregates or reducing the moisture available to the concrete or coarse aggregates. Nondurable coarse aggregates can often be eliminated by the use of reliable freeze-thaw durability tests and specifications that dictate the appropriate amounts, size, and types of coarse aggregates to use. Moisture is often prevented from reaching concrete and aggregates by controlling mix parameters and incorporating adequate pavement design features (drainage and base type). Concrete mix proportioning can be adjusted to produce a frost-resistant concrete when sound aggregates are used. This technique includes reduction of the water-cement ratio, the use of pozzolans and admixtures, and provision of adequate air entrainment and curing. The ability of this approach to mitigate D-cracking in the presence of nondurable aggregate has not been adequately investigated.

The D-cracking potential of a given aggregate source can be reduced by either improving the quality of the aggregate or by reducing the severity of the conditions that cause D-cracking by improving the mix design of the concrete and/or reducing the availability of moisture (as described above). Knowledge of the coarse aggregate characteristics and properties that affect D-cracking is essential for selecting the aggregate and mitigation method. Candidate techniques for decreasing the potential for D-cracking include reducing the maximum size of coarse aggregate, selective quarrying, mechanical separation, blending, heat treatment, coating, or impregnation.

While reducing the maximum size improved the freeze-thaw performance, an increase in the severity of transverse cracking was reported by some researchers. Selective quarrying offers the potential for more uniform coarse aggregate materials and can eliminate the inclusion or use of nondurable coarse aggregate particles. Blending durable and nondurable aggregates can produce a material with improved freeze-thaw resistance, but the allowable percentage of nondurable coarse aggregate is subject to debate. Heat treatment, coating, and impregnation of coarse aggregates were reported to improve their frost resistance; however, the practicality and feasibility of these techniques are questionable.

CHAPTER 3
EVALUATION OF FIELD STUDY SECTIONS

3.1 Identification and Selection of Field Study Sections

The Mn/DOT pavement management system was used to identify 38 Minnesota concrete pavement sections as candidates for this study. These pavements were grouped according to their aggregate sources and D-cracking field performance to date (i.e., good, fair or poor). Visual condition surveys of the PCC pavement sections were performed in accordance with the 1993 SHRP-P-338 manual and included an evaluation of the severity of any D-cracking present, the drainage condition, the joint seal condition and identification of the presence of other pavement distress or rehabilitation work done to the pavement section (84). Tables 3.1, 3.2 and 3.3 list the pavement sections surveyed in this study in the good, fair and poor categories, respectively, and summarize their performances to date.

For each aggregate source used in the 38 sections, one pavement section was selected for further studies when the D-cracking field performance of the aggregate source was consistent. For aggregate sources that appeared to exhibit different degrees of freeze-thaw durability, two pavement sections exhibiting different D-cracking performance (preferably at the same site) were selected. Pavement sections that exhibited durability distresses other than D-cracking (e.g., high steel damage) were not included in this study. In this way, fifteen of the 38 original field sites (featuring twelve different aggregate sources) were selected for further use in this study.

The fifteen field test sections were divided into three groups based upon records of their freeze-thaw durability. Group I consisted of pavements constructed using aggregate sources that have been consistently associated with D-cracking pavements in Minnesota, while group II included only sections constructed using aggregate sources that have been considered durable with respect to D-cracking. Group III consisted of pavements constructed using aggregate sources that have apparently exhibited different degrees of frost resistance at various construction sites. Table 3.4 lists the pavement sections and aggregate sources included in this study and separates them into the three groups described above.

Table 3.1. Field observations for study sections in good condition.

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 52 I	79.170 to 79.380	155037	32	10	None.	Clean, overall good condition.	Low-severity damage.	Low- to-medium-severity transverse cracks.	None.	
TH 52 I	56.993 to 64.945	155037	51	34	None.	Drains OK, good condition.	Low-to-medium-severity damage.	Low-to-high-severity transverse cracks, low-to-medium-severity corner cracks.	Full- and partial-depth repairs, crack sealing, corner crack patching.	
TH 42	0.000 to 3.500	155037	51	28	None.	Clean, overall good condition.	Very low-severity damage.	Medium-severity longitudinal cracking; very low-severity transverse cracks, low-to-medium-severity faulting, slight spalling.	Partial bituminous overlay, partial patching, longitudinal crack sealing.	
TH 52 D	76.476 to 79.530	125009	51	33	Very light at transverse joints.	Clean, overall good condition.	Low-to-medium-severity damage.	Medium-to-high-severity midpanel cracks, medium-severity faulting, high-severity corner breaks.	Patches around joints and cracks; sealing of transverse cracks.	
TH 52 D	70.854 to 72.039	125009	51	33	None.	Clean, overall good condition.	No damage.	Medium-to-high-severity midpanel cracking; low-to-medium-severity corner cracks.	Patching around joints; sealing of mid-panel cracks.	
TH 52 I	69.460 to 79.530	125009	51	33	None.	Drains OK, good condition.	Low-to-high-severity damage.	Light staining; low-to-high-severity transverse cracks, medium-to-high-severity holes near joints, low-to-medium-severity corner cracks, low-severity faulting.	Low-severity partial-depth patching; joint patching, some maintenance overlay work.	

Table 3.1. Field observations for study sections in good condition (continued).

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 14	212.798 to 213.481	179036 & 155037	51 (& 19-)	27	None.	Clean, overall good condition.	Low-severity damage for joints, high-severity damage transverse cracks.	Medium-severity transverse cracks, low-to-medium-severity corner cracks, medium-severity high steel damage.	Transverse cracks sealed, hole patching.	
TH 90 D	172.400 to 175.771	193016	51	30	None but minor staining.	Good condition.	No damage.	High-severity midpanel cracks, medium-severity faulting, low-severity corner breaks; high-severity pumping but occurrence is not frequent.	Patching of pumped areas, sealing of mid-panel cracks.	
TH 90	138.775 to 145.900	193016	51	18	None, but minor staining.	Good condition.	No damage.	Low-to-medium-severity faulting; very low-severity midpanel cracks.	None.	
TH 35	13.69 to 19.264	193016	51	24	None, but staining.	Good condition, wet ditches.	No damage.	Very low-severity corner breaks, medium-to-high-severity faulting, medium-to-high-severity midpanel cracks .	None.	
TH 35	19.595 to 25.050	193016	51	30	None, but staining.	Good condition.	No damage.	Very low-severity corner breaks, very low-severity pumping, medium-severity midpanel cracks.	Repairing of mid-panel cracks by full-depth patching; sealing of transverse cracks; Grinding, partial patching around joints.	
TH 35 I	8.450 to 12.920	193016	51	24	None, but minor staining.	Good condition.	No damage.	Lots of very low-severity corner breaks, very low-severity popouts, minor-to-high-severity midpanel cracks with very few high-severity ones; medium-severity high steel damage.	Sealing of high-severity midpanel cracks.	

Table 3.1. Field observations for study sections in good condition (continued).

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 90 D	249.444 to 266.509	185007	51	23	None.	Wet edges (ditches); transverse pipe blocked at the ends.	Medium-severity damage.	Medium-severity pumping, medium-severity faulting, low-to-medium-severity corner breaks; low-to-medium-severity midpanel cracks.	Partial- and full-depth patching of joints.	
TH 35 I	5.000 to 7.000	193011	51	23	None, but light staining.	Good condition.	No damage.	Very low-severity corner breaks, very low-severity pumping, high-severity shoulder distress; high-severity midpanel cracks but very few occurrences.	Patching of shoulder.	
TH 35 D	0.000 to 7.000	193011	51	23	None, but light staining.	Good condition.	No damage.	Very low-severity corner breaks, very low-severity pumping, high-severity shoulder distress; high-severity midpanel cracks but very few occurrences.	Patching of shoulder.	
TH 35 D	11.674 to 12.000	193011	51	24	None, but minor staining.	Good condition.	No damage.	Very low-severity corner breaks, medium-severity faulting, high-severity midpanel cracks but very few occurrences.	Patching of shoulder; sealing of high-severity midpanel cracks.	

Table 3.1. Field observations for study sections in good condition (continued).

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 3	44.986 to 47.909	182002	*n/a*	*n/a*	None.	Good condition.	Good Condition.	Low-to-medium-severity, medium-severity spalling of transverse joints, low-severity corner breaks, few low-severity midpanel cracks, some medium-severity popouts.	Partial-depth patching of transverse and longitudinal joints; full-depth patching of joints, repair of corner breaks.	
TH 90 Ramp to TH 91	Ramp	167001	*n/a*	*n/a*	None.	Good condition.	Low-severity damage .	Occasional medium-severity popouts, low-severity transverse spalling.	None.	
TH 90 Ramp to TH 266	Ramp	167001	*n/a*	*n/a*	None, but staining.	Good condition.	Low-severity damage.	Low-severity corner breaks, high-severity longitudinal cracks.	Overlaying of east bound ramps, sealing of longitudinal cracks, full-depth repairs along cracks.	
TH 90 Ramp to TH 59	Ramp	167001	*n/a*	*n/a*	None.	Standing water and cattails in ditches.	Low-severity damage.	High-severity longitudinal cracks, low-severity transverse cracks.	Majority of ramps are overlaid.	
TH13	1.350 to 3.020	193017	51	22	None, but light staining.	Good condition.	No damage.		Diamond ground, new shoulder.	

Table 3.2. Field observations for study sections in fair condition.

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 52 D	56.077 to 61.929	155037	51	35	Very light along longitudinal joints.	Drains OK, good condition.	Low-to-medium-severity damage.	Low-to-high-severity midpanel cracks, low-severity corner cracks, medium-severity faulting.	Full- and partial- depth patching; crack sealing, corner crack patching.	
TH 90	222.738 to 249.444	155051 & 155037	51	23	Minor to low and slight to none.	Water drains under pavement near connection with TH42.	Low-severity damage to longitudinal joint sealing.	Loss of support cracks, low-to-medium-severity faulting, low-to-medium-severity midpanel cracks, low-to-medium-severity corrosion damage.	Taped longitudinal cracks in good condition.	
TH 56	35.500 to 36.000	155011	32	21	Minor cracking near joint, probably D-cracking.	Clean.	Minor damage.	Low-severity corner cracks; medium-to-high-severity midpanel cracks; medium-severity faulting.	Full-depth joint patching with concrete; transverse cracks sealed.	Cracking might be caused by ASR or corrosion.
White Bear Lake	Highway 694 to County E Road	182002	*n/a*	*n/a*	None to very low plus some staining.	Good condition.	Lots of medium-severity damage.	Medium-severity midpanel cracks with some staining; medium-severity corner breaks; lots of joint spalling cracks; few low-severity high steel damage.	Patching; joint spalling repair; repair of corner breaks; partial-depth patching of joints.	Medium-severity damage near joints probably caused by damage to bond between aggregate and cement matrix.
TH 90 Ramp	Adrian Rest Area: East and West Directions	167001	*n/a*	*n/a*	None to minor plus staining.	Standing water and cattails in ditch.	Low-severity damage.	Low-severity corner breaks, low-severity longitudinal cracks.	Repairs made at longitudinal and transverse joints, full-depth patching around joints and along longitudinal section, very few corner patches.	

Table 3.3. Field observations for study sections in poor condition.

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 42	3.500 to 4.900	155037	51	28	Severe	Clean, overall good condition.	High-severity damage to transverse joint sealant; medium-severity damage to longitudinal joint sealant.	High-severity faulting, medium-severity longitudinal cracks.	Patching and overlaying after MP 4.	
County Rd 7	Highway 52 to Highway 90	155051	51	22	Medium-to-severe	Good with some water in the edges due to failure of shoulders.	High-severity damage.	High-severity longitudinal cracks, medium-to-high-severity faulting, medium-to-high-severity corner breaks, high-severity midpanel cracks but few occurrences.	Partial-depth patching of joints, sealing of longitudinal and mid-panels cracks, patching of corner breaks, overlay of about 1 mile.	
TH 52	52.259 to 54.297	179036	51	27	Low-to-medium	Drains OK and source of water in middle and edges.	Low-to-high-severity damage .	High-severity corrosion damage, medium-to-high-severity corner breaks, medium-to-high-severity midpanel cracks.	Corner repair, full- and partial-depth patching of joints.	High steel cracks probably looks like D-cracking. No D-cracking.
TH 90 D	166.217 to 172.400	193016	51	30	Medium and staining		No damage.	Lots of medium-severity high steel damage; low-severity corner breaks; high-severity pumping damage near joint but not very often; low-to-medium-severity scaling damage.	Full-depth patching; full-depth joint patching.	

Table 3.3. Field observations for study sections in poor condition (continued).

Pavement Section	Mile Post: From - To	Aggregate Source	Aggregate Top Size (mm)	Age of Pavement	D-Cracking Condition	Drainage Condition	Joint Sealant Condition	Other Distress	Rehabilitation	Additional Observations
TH 13	3.020 to 3.660	193011	51	26	Low-to-medium plus staining.	Good condition.	No damage.	Medium-severity high steel damage; medium-to-high-severity midpanel cracks; low-to-high-severity medium corner breaks.	Milling or grinding, partial patching.	Test section 6013 R.
TH 35 I	0.000 to 2.000	193011	51	23	Low-to-medium plus staining.	Good condition.	No damage.	Very low-to-medium-severity corner breaks; low-to-high-severity midpanel cracks.	Patching of corner breaks.	
TH 175	0.000 to 10.500	135001	51	24	Low- to-medium.	Overall good condition, road is overlaid when in poor drained areas.	Low-to-high-severity damage.	High-severity longitudinal cracks, medium-severity corner breaks.	Partial overlaying of the road in poor drained areas, sealing of cracks.	
TH 212	137.69 to 140.7	170006	19	20	Medium-to-severe.	Overall good.	Medium-severity damage.	High-severity faulting; medium-severity corner breaks.	Grinding.	

Table 3.4. Observed durability of selected field study sections.

Source	Source Number	Type	Mn/DOT Durability History	Route	Location (Mile Post, etc.)	Observed Field Durability	Nominal Top Size (mm)	Years in Service
Group I: Nondurable Sources								
A: Grand Meadows	155011	Carbonate	Poor	TH 56	35.5 - 36.0	Fair	32	21
E: St Paul Park	182002	Gravel	Fair to Poor	White Bear Lake Ave	Dell Street	Fair	50	*N/A*
F: Luverne	167001	Gravel	Poor	Rest Area	I-90, near M.P. 7	Fair	50	27 or less
G: Bryan Rock	170006	Carbonate	Fair	TH 212	137.7 - 140.7	Poor	19	20
H: Halma	135001	Gravel	Poor	TH 175	0 - 10.5	Poor	50	24
Group II: Durable Sources								
C: Hammond	179036	Carbonate	Good	TH 14	212.8 - 213.5	Good	50	27
D: Wilson, Winona	185007	Carbonate	Fair to Good	TH 90	227.0 - 249.4	Fair	50	23
I: Harris	193017	Carbonate	Good	TH 13	1.35 - 3.02	Good	50	22
L1: Zumbrota	125009	Carbonate	Fair to Good	TH 52	76.5 - 79.5	Good	50	33
Group III: Poor to Good Sources								
B: Rochester	155037	Carbonate	Fair to Poor	TH 42	0.0 - 3.5	Good	50	28
				TH 42	3.5 - 4.9	Poor	50	28
M: Stewartville	155051	Carbonate	Fair to Poor	TH 90	222.7 - 249.4	Fair	50	23
				Cnty Rd 7	U.S. 52 - I-90	Poor	50	22
N: Kuennens	193016	Carbonate	Good to Poor	TH 90	172.4 - 175.8	Good	50	30
				TH 90	166.2 - 172.4	Poor	50	31

3.2 Tests of Pavement Cores

Cores were retrieved from each of the fifteen selected pavement sections for laboratory testing as described below:

- five 150-mm diameter cores from mid-panel areas of the slab (away from any cracks or joints) for compressive strength tests (3 cores), split tensile tests (1 core) and microscopic examinations (linear traverse test and thin film test, 1 core);
- three 100-mm diameter cores from mid-panel areas (away from any cracks or joints) for freeze-thaw testing (ASTM C 666 procedure C);
- two sets of three 100-mm diameter cores (one set taken from the wheel path and the other from the middle of the panel) at 0, 300 and 600 mm away from the transverse joint to determine the presence and extent of D-cracking; and
- two additional sets of three 100-mm diameter cores from the wheel path and the middle of the panel at 0, 300 and 600 mm from a typical crack to determine the extent of D-cracking, if any.

Tables 3.5 and 3.6 present summaries of the test results and observations made from the drilled cores and pavement surveys.

3.2.1 Visual Inspection of Cores

Examination of full-depth cores is the only technique currently available to positively identify the development of D-cracking before it appears at the surface. Visual identification of D-cracking was performed in accordance with the SHRP-P-338 manual (84). Reports documented the conditions of the cores and the evidence, extent and location of any D-cracking. The results of the visual inspection are presented in tables 3.5 and 3.6.

For the pavements that showed D-cracking, slab deterioration was generally limited to within 300 mm of the pavement joints. An exception was the pavement containing source G aggregate in the concrete. This pavement was also severely faulted (a distress that develops in the presence of free water beneath the pavement slab), which suggests that increased levels of moisture near the joints may have allowed extended crack formation. In most cases, the extent of medium- or high-

Table 3.5. Results of tests on cores (Groups I and II).

	Group I					Group II			
Aggregate Source:	**A**	**E**	**F**	**G**	**H**	**C**	**D**	**I**	**L1**
Aggregate Type	Carbonate	Carbonate	Gravel	Carbonate	Gravel	Carbonate	Carbonate	Carbonate	Carbonate
Route	TH 56	White Bear	Rest Area	TH 212	TH 175	TH 14	TH 90	TH 13	TH 52
Location (milepost,etc.)	35.5-36	Dell Street	Hwy 90	137.7-140.7	0-10.5	212.8-213.5	227-249.4	1.35-3.02	76.5-79.5
Years in Service	21	*(a)*	27 or less	20	24	27	22	22	33
Field Performance	Fair	Fair	Poor	Fair	Poor	Good	Fair	Good	Good
Linear Traverse Test Results									
Average Chord Intercept (mm)	0.21	0.15	0.17	0.19	0.12	0.24	0.20	0.21	0.18
Voids per cm	3.9	3.3	3.1	2.1	3.9	2.3	1.9	3.9	3.2
Specific surface (mm^2/mm^3)	*18.7*	25.7	*23.6*	*20.7*	33.0	*16.6*	*20.3*	*18.7*	*22.2*
Paste-to-air ratio	3.59	5.89	5.79	7.51	6.41	5.47	7.94	3.59	5.23
Air content (%)	8.36	5.1	5.18	*3.99*	*4.68*	5.48	*3.78*	8.36	5.73
Spacing Factor (mm)	0.19	0.19	*0.21*	*0.27*	0.16	*0.29*	*0.28*	0.19	*0.21*
Compressive Strength Test Results									
Number of cores tested	3	3	3	3	3	*(b)*	*(b)*	3	*(b)*
Strength (kPa)	46,710	44,300	48,490	62,480	45,950			43,900	
Split Tensile Strength Test Results									
Number of cores tested	1	1	1	1	1	3	3	1	3
Strength (kPa)	4,840	5,820	5,100	6,730	5,910	3,600	5,180	5,370	4,630
Visual Inspection of Cores									
Evidence of D-cracking	minor	low	severe	low to minor	low to medium	none	low	none	minor
Extent of Cracking	joint	joint	± 300 mm	± 600 mm	± 300 mm	*N/A*	joint	*N/A*	joint
Location	between wheel paths	between wheel paths	in and between wheel paths	in and between wheel paths	in and between wheel paths	*N/A*	in and between wheel paths	*N/A*	between wheel paths

(*a*): Data unavailable

(*b*): Cores contained embedded steel and could not be tested in compression.

N/A: Not applicable

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

Table 3.6. Results of tests on cores (Group III).

	Group III					
Aggregate Source:	**B**		**M**		**N**	
Aggregate Type	Carbonate		Carbonate		Carbonate	
Route	TH 42	TH 42	TH 90	Cnty Rd 7	TH 90	TH 90
Location (milepost,etc.)	0.0-3.5	3.5-4.0	220.7-249.4	Rochester	172.4-175.8	166.2-172.4
Years in Service	28	28	23	22	31	31
Field Performance	Fair	Poor	Good	Poor	Good	Poor
Linear Traverse Test Results						
Average Chord Intercept (mm)	0.2	0.2	0.2	0.2	0.2	0.2
Voids per cm	2.5	4.1	3.7	3.7	2.8	2.2
Specific surface (mm^2/mm^3)	*22.5*	24.3	25.1	*18.5*	*19.3*	*19.7*
Paste to air ratio	3.59	5.79	6.41	7.51	5.89	7.94
Air content (%)	8.36	5.18	*4.68*	*3.99*	5.1	*3.78*
Spacing Factor (mm)	*0.24*	0.18	0.19	0.20	*0.24*	*0.27*
Compressive Strength Test Results						
Number of cores tested	3	3	*(b)*	3	*(b)*	*(b)*
Strength (kPa)	55,140	45,820		35,280		
Split Tensile Strength Test Results						
Number of cores tested	1	1	3	1	3	4
Strength (kPa)	5,600	4,900	3,330	3,960	4,110	4,130
Visual Inspection of Cores						
Evidence of D-cracking	minor	low to severe	none	low to medium	none	medium
Extent of Cracking	joint	± 300 mm	*N/A*	joint and cracks	*N/A*	± 300 mm
Location	wheel path	wheel path	*N/A*	in and between wheel paths	*N/A*	in and between wheel paths

(*b*): Cores contained embedded steel and could not be tested in compression.

N/A : Not applicable

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

severity D-cracking was limited to areas within 300 mm of the joints. D-cracking severity generally decreased with distance from the joint.

The pavement section that contained source F aggregates exhibited severe D-cracking at the bottom of the slab while the surface of the slab was in fair condition. This might be explained by the fact that this pavement section was located in a rest area where the rate of D-cracking deterioration was not accelerated by the high traffic levels and deicing salt applications to which major interstates are subjected. It seems likely that severe D-cracking will be exhibited at the pavement surface within a few years because the D-cracking at the bottom of the slab is so severe.

3.2.2 Strength Tests

Compressive and indirect tensile strength tests were performed in accordance with ASTM C 39 and C 496, respectively. Cores were prepared for both tests in accordance with ASTM C 42. Three cores were used for compressive strength tests, although compressive strength could not be determined for all projects in this study because embedded reinforcement was present in some cores. Trimming the cores to eliminate the steel would have produced a length-to-diameter ratio (L/D) less than 1, which is not allowed by ASTM C 39. In these cases, three cores were used (instead of one) to determine the indirect tensile strength, provided that no steel was present in the tensile fracture plane.

Compressive and split-tensile strength test results are also presented in tables 3.5 and 3.6. Compressive strengths ranged from 45.8 to 62.5 MPa, and split tensile strengths ranged from 3.3 to 6.7 MPa. Ratios of tensile to compressive strength varied between 0.10 and 0.13, which is typical for moderate-strength concrete.

3.2.3 Linear Traverse Test

The linear traverse test is a microscopic measurement of the concrete air void system and is typically performed on samples of concrete obtained from near midpanel of the slab. Air void system parameters provide a means of predicting the frost resistance of the mortar and can provide an indication of whether observed freeze-thaw durability problems are attributable to the mortar of the concrete.

The air void system parameters that are commonly used to evaluate the freeze-thaw-durability of concrete mortar are the spacing factor, L, and the specific surface, α. The spacing factor is the average minimum distance of any point in the cement paste from the periphery of an air void. The spacing factor should be small enough to allow unfrozen water to escape to an air void during freezing. ACI 211.1-89 (1989) recommends a spacing factor of 0.2 mm or less to protect the concrete against freeze-thaw damage. The specific surface is the ratio of the surface area of the voids divided by their volume. The specific surface area is an indicator of the size of the air bubbles introduced by air entrainment. A minimum specific surface of 24 mm^2/mm^3 is recommended by ACI 211.1-89. ACI 211.1-89 also recommends a total air content of 5.0 percent or more.

The linear traverse test was performed in accordance with ASTM C 457 on slices of concrete obtained from 150-mm cores retrieved from the midpanel of the slab, away from any cracks. A summary of linear traverse test results for the cores obtained from pavement sections included in this study is presented in tables 3.5 and 3.6, and the values that fail to meet the aforementioned ACI criteria are italicized. Only two study sections met the acceptance criteria for all three parameters (i.e., spacing factor, specific surface and total air content): the source E section and one section constructed using source B. Both sections exhibited fair-to-poor durability in the field, indicating probable aggregate-related distress. The source A section failed only the specific surface criteria, but also exhibited D-cracking within the study section; this aggregate source is also widely associated with freeze-thaw problems in Minnesota.

Samples from several sections (i.e., B, D, F, G, M and N) failed 2 or more criteria and exhibited fair-to-poor freeze-thaw durability. Frost resistance problems in these sections could be based in the mortar, the aggregate, the transition zone, or some combination of the three. There were also several sections that exhibited no freeze-thaw damage, in spite of apparently deficient mortar air systems (i.e., C and L1), and almost all of the "good" pavements failed at least one of the ACI air void system criteria.

This suggests that saturation levels were not high enough to produce damage in these cases or that the ACI guidelines are conservative.

In summary, the spacing factor, specific surface and other air void parameters were determined using microscopic examination (linear traverse test conducted in accordance with ASTM C 457). These air void parameters were used to determine the absence or presence of a well-distributed air void system in the paste, which would aid in determining whether any observed freeze-thaw damage was due to deficiencies in the mortar or the aggregate. Although several pavement sections failed one or more of the American Concrete Institute recommendations for durable mortar, some of these pavement sections showed no apparent signs of freeze-thaw damage.

3.2.4 Petrographic Examination of Cores

Petrographic examination consists of a visual inspection of aggregate particles to determine their lithology and individual particle properties. The petrographic examination for this study involved microscopic examination of aggregate and polished concrete sections to determine the mineralogy and condition of the concrete components, especially the coarse aggregates.

The petrographic examination was performed on thin sections obtained from the 150-mm cores retrieved from the centers of slabs, away from any cracks or joints, in each pavement section. Thin sections were made from slices taken from the middle of the cores, were polished to a thickness of approximately 25 μm and were examined using a petrographic microscope (25x, 100x and 400x magnifications). Hand samples were also examined using an optical stereo dissecting microscope.

Geology of Coarse Aggregates

Results of the petrographic examination and the geological descriptions of each aggregate source used in the selected pavement sections are presented in tables 3.7, 3.8 and 3.9 and in Appendix A. Visual determination of coarse aggregate types and their physical condition was conducted for each project. All of the carbonate aggregates included in this study consisted primarily of dolomite.

Table 3.7. Geologic description of carbonate sources (Group I).

Aggregate Source	A: Grand Meadows	F: Luverne	H: Halma	G: Bryan Rock	E: St. Paul Park
Type	Carbonate	Gravel	Gravel	Carbonate	Carbonate
Origin	TH 36 MP 36	Adrian Rest Area	TH 175 MP 10	TH 212 MP 140	White Bear Lake Avenue, Maplewood
Observed Field Performance	Fair	Poor	Poor	Fair	Fair to Poor
Formation	Cedar Valley: Upper Solon Member	Quaternary: alluvium	Quaternary: lake washed till	Shakopee: Prairie du Chien group	Shakopee: Prairie du Chien group
Composition	biolithic dololutite ; coarse-grained calcite sparite filling biolithic fragments	biolithic dololutite (30%); igneous-metamorphic rocks (66%); and mafic rock (4%)	calcitic dololutite (60%) and igneous-metamorphic rocks (40%)	oolitic quartzose dolarenite: euthdral dolomite sparite/rhombs (50%); dolomite pseudosparite (15%); and dolomite pseudosparite (35%)	dololutite and dolarenite
Folk's Textural Classification of Carbonates	dolobiomicrosparite	dolobiomicrosparite	dolosparite	sandy doloointrasparite	sandy doloointrapel sparite
Grain Size of Carbonates	< 70 μm	1 - 70 μm	10 - 50 μm	5 - 400 μm	25 - 500 μm
Grain Size Classification of Carbonates	very fine - fine	very fine - fine	very fine - fine	very fine - medium	very fine - coarse
Median Grain Size of Carbonates	60 μm	5 μm	10 μm	140 μm	140 μm

Table 3.8. Geologic description of carbonate sources (Group II).

Aggregate Source	C: Hammond	D: Wilson, Winona	I: Harris	L1: Zumbrota
Type	Carbonate (contain 19mm-minus Source B aggregates)	Carbonate	Carbonate	Carbonate
Origin	TH 14 MP 213	I-T390 MP 252	TH 175 MP 10	TH 52 MP 78
Observed Field Performance	Good	Fair	Good	Good
Formation	Shakopee: Prairie du Chien group	Shakopee: Prairie du Chien group	Shellrock: Devonian	Shakopee: Prairie du Chien group
Composition	dolarenite	dolarenite	biolithic dolarenite	quartzose oolitic dolarenite
Folk's Textural Classification of Carbonates	dolosparite	dolosparite	dolobiosparite	sandy doloointrapel sparite
Grain Size of Carbonates	50 - 600 μm	50 - 350 μm	50 - 400 μm	15 - 500 μm
Grain Size Classification of Carbonates	fine - coarse	fine - medium	fine - medium	very fine - coarse (25% are very fine-grained)
Median Grain Size of Carbonates	200 μm	250 μm	140 μm	140 μm

Table 3.9. Geologic description of carbonate sources (Group III).

Aggregate Source	B: Rochester		M: Stewartville		N: Kuennens	
Type	Carbonate	Carbonate	Carbonate (Contains 19-mm minus Source B aggregate)	Carbonate	Carbonate	Carbonate
Origin	TH 14 MP 1	TH 14 MP 3.5	I-90 MP 225	County Road 7, Rochester	I-90 MP 174	I-90 MP 168
Observed Field Performance	Fair	Poor	Fair	Poor	Good	Poor
Formation	Shakopee: Prairie du Chien group	Shakopee: Prairie du Chien group	Stewartville: Ordovician	Stewartville: Ordovician	Shellrock	Shellrock
Composition	quartzose dolarenite	biolithic dolarenite	biolithic dolarenite	dolarenite	dolarenite	dolarenite
Folk's Textural Classification of Carbonates	sandy dolintrasparite	dolobiosparite	dolosparite	dolobiosparite	dolosparite	dolosparite
Grain Size of Carbonates	25 - 1000 μm	25 - 1000 μm	30 - 500 μm	25 - 250 μm	15 - 500 μm	10 - 100 μm
Grain Size Classification of Carbonates	fine - coarse	fine - coarse	fine - coarse	very fine - medium	very fine - coarse	very fine - medium
Median Grain Size of Carbonates	120 μm	120 μm	190 μm	150 μm	150 μm	160 μm

Several researchers recommend avoiding dolomites which contain very fine grains or large amounts of silt or clay (29, 85, 86). The poor performance of fine-grained dolomites is attributed to the increase in grain surface area, which provides more space for enclosed void spaces for water. Hudec and Achampong reported that grain size alone is not a good indicator of freeze-thaw resistance, although their experimental results suggested that fine-grained rocks are more prone to freeze-thaw damage. They explained that the fine-grained materials contain finer pore sizes which, in turn, produce larger amounts of internal surface areas for water and ion sorption. Therefore, the increase in small pore volume is responsible for the expansive forces that expand and damage the rock. They also reported that salted specimens adsorbed higher amounts of water (by as much as 10 percent), implying that either 10 percent more pores are filled or smaller pores are filled. This increase explains the increased damage often observed in the presence of salts, even though salts depress the freezing point of water in the rock (85). Tables 3.7 and 3.8 show that the coarse aggregates in group I had more very fine-to-fine-grained dolomites than group II, which may help to account for the performance differences between the two groups.

Correlation of Petrographic Test Results and Field Performance

The PCC pavement sections which contained very fine-grained dolomites (A, E, F, G, H and N) performed poorly when compared to PCC pavement sections which contained more coarsely grained dolomites (C, D and I). These observations are consistent with the results of other studies which relate fine-grained crystalline dolomite and the application of deicing salt to frost resistance problems, as described previously (29, 85, 86).

Microscopic examination of cores from PCC pavement sections which contained coarse aggregate from sources B and G revealed mineral growth in the air voids, similar to that shown in figure 3.1. The mineral growth in some of the entrained air voids was identified as ettringite through the use of a scanning electron microprobe. The energy dispersion spectrum (EDS) for embedded ettringite is shown in figure 3.2. This spectrum shows major peaks at locations which correspond to calcium, sulfur and aluminum. The abundance of these elements, which are the principal components of ettringite, $(CaO).(Al_2O_3).3(SO_3).32(H_2O)$, supports the conclusion that the mineral in the voids is very likely ettringite. The observed crystalline shape (fibrous or needle-like) also supports this conclusion.

Since entrained air plays a major role in the durability of concrete, the reduction or the obstruction of these voids by mineral growth may have a detrimental effect. In 1995, Marks and Dubberke documented the growth of ettringite in entrained air voids and proposed a mechanism of deterioration due to the expansion that follows the dissolution of ettringite when exposed to NaCl brine. They reported that cracks were radiating from some of the air voids, which indicated that the air voids were filled with ettringite prior to cracking and that they were centers of pressure for the development of these cracks (87).

The greater extent of surface distresses that resemble D-cracking in the pavement section which contained aggregate from source G might be explained by the secondary mineralization present in the mortar air voids, which could aggravate the deterioration of the pavement caused by nondurable aggregates.

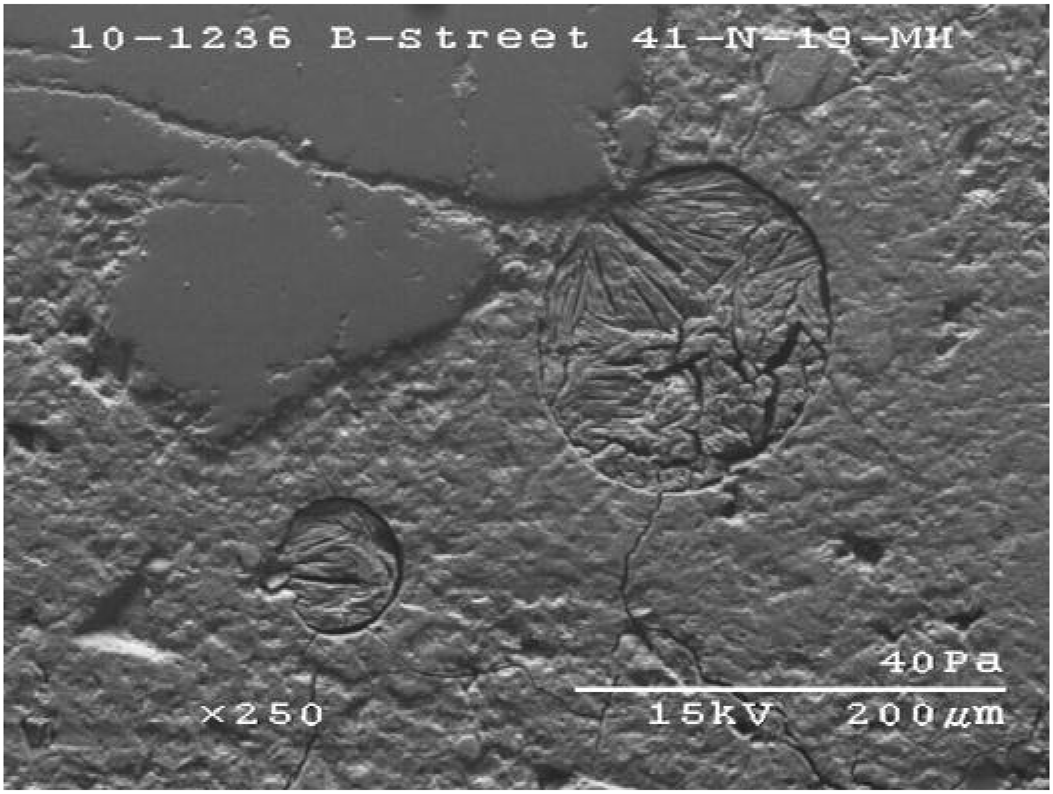

Figure 3.1. Photomicrograph of ettringite-filled air voids in concrete (Courtesy of American Petrographic Services).

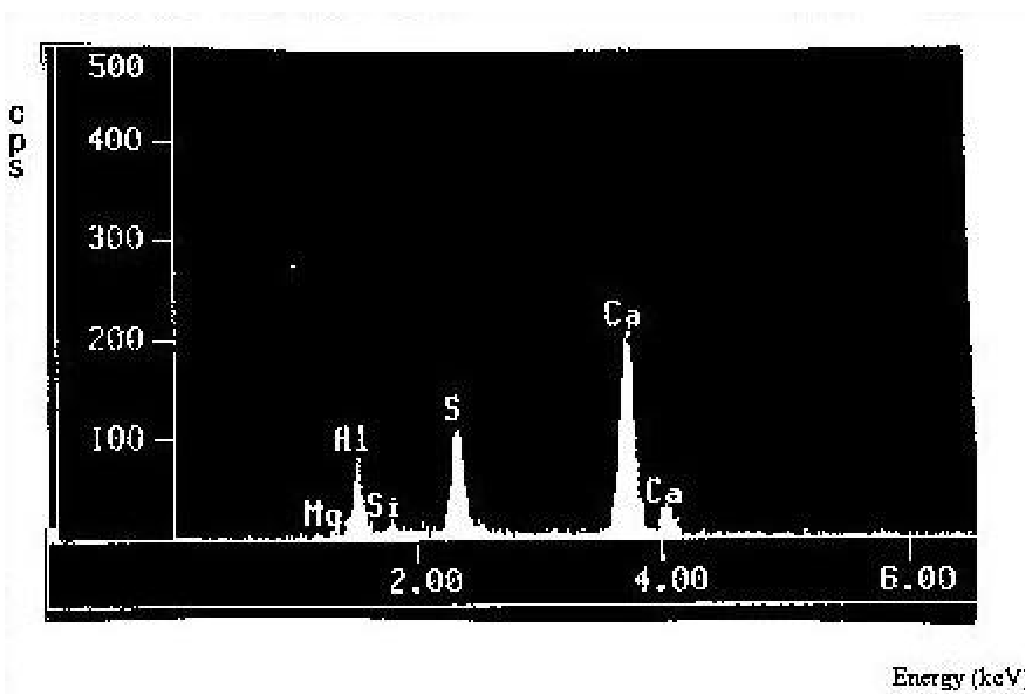

Figure 3.2. Typical energy dispersion spectrum (EDS) for embedded ettringite.

Secondary mineralization was also found in both pavement sections that contained source B coarse aggregates, but was more severe in the pavement section that performed badly, especially near joints, where air voids smaller than 200 μm were filled. The concrete pavement section which showed significant secondary mineralization in air voids exhibited a distress that appeared similar to D-cracking in the field. Although the aggregates found in both sections are lithologically distinct, as shown in table 3.9, there is no evidence that the cause of the apparent freeze-thaw problem in the badly damaged pavement was due to the aggregates. Therefore, it is believed that the pavement deterioration observed in the source B sections is due to the effects of secondary mineralization of the air void system, which was exacerbated by cycles of freezing and thawing and deicing salt applications.

The two PCC pavement sections which contained source N coarse aggregates performed differently. The pavement section that performed poorly contained 30 percent fine-grained dolomite particles, while the section that performed fairly contained only 5 percent fine-grained dolomite particles. Cracked shale particles were also present in greater quantities in the poor section than in the fair section, and dark reaction rims were noticed around the shale and dolomite particles in the poor section. Kosmatka and Panarese classified shale as a harmful substance that may be present (with other deleterious materials) in aggregates, causing popouts by swelling and/or freezing after absorbing water. They reported that most specifications limit the permissible amount of shale particles in aggregates. For example, ASTM C 33 requires that the amount of shale shall not exceed 5 percent by weight (86). Mn/DOT limits the amount of shale in the coarse aggregate sample to 0.4 percent when the maximum particle size is 12.5 mm and 0.7 percent when the maximum size is 4.75 mm. Shale in the sand-sized portion is limited to 2.5 percent (88).

Dark reaction rims are often associated with alkali-aggregate reactions (86). The alkali-silica reaction (a specific type of alkali-aggregate reaction) refers to a PCC distress resulting from chemical reactions involving alkali ions from the portland cement (Na^+ and K^+), hydroxyl ions and certain siliceous constituents that may be present in the aggregates. The mechanism of deterioration involves the depolymerization or breakdown of the silica structure of the aggregate by the hydroxyl ions, followed by the absorption of the alkali ions to form an alkali-silica gel. When this gel is formed and later comes in contact with water, it expands and generates hydraulic pressures, which leads to the expansion and cracking of the aggregates and cement paste matrix surrounding it. The mobility of the alkali-silica gel (from the aggregate interior and the surrounding transition zone to micro-cracked regions within the aggregates and the cement paste) results from its solubility in water. The further expansion and cracking of the concrete are results of the continued availability of water to the concrete (40).

Kosmatka and Panarese reported that, although many carbonate rocks react with cement hydration products, expansive reactions are rarely produced (86). Expansive alkali-carbonate reactivity is suspected only in extremely fine-grained dolomitic limestones with large amounts of calcite, clay, silt or dolomite rhombs found in a matrix of clay and fine calcite.

The durability of the concrete pavement section which contained both cracked shale particles and more finely grained dolomites (i.e., the section which contained aggregate from source N and performed poorly) produced distress that appeared similar to D-cracking. The presence of the cracked shale particles and the possible alkali-silica reaction are believed to have contributed to the reduced durability of this pavement section.

The two PCC pavement sections which contained source M coarse aggregates also performed differently. The section that performed poorly contained aggregate particles with tightly packed fine-to-medium-grained dolomite rhombs within a very fine-grained calcite pseudosparite matrix. The section that performed well contained mostly aggregate particles that are composed of medium-to-coarse-grained dolomite sparite. The poor performance of the section which contained fine-grained dolomites is consistent with the field performance of group I aggregate sources and with other studies (29, 85, 86) which relate fine-grained crystalline dolomite and deicing salt-applications to frost resistance problems, as described previously.

The maximum aggregate particle size used in the pavement sections selected for this study was 50 mm, with the exception of sections constructed using sources A and G (Grand Meadows and Bryan Rock, respectively, which had top sizes of 32 mm and 19 mm), as shown in table 3.4. In previous studies, a reduction in the maximum top size of coarse aggregates was found to improve the durability performance in both the laboratory and the field (1). Source A performed fairly in the field section included for this study even though this source is generally considered nondurable; this may be attributed to the reduced maximum top size of the coarse aggregate (32 mm).

In summary, comparisons of field performance with the results of petrographic examinations of the aggregates and the hardened concrete showed that pavement sections which contained very fine-grained dolomite performed more poorly than comparable pavement sections containing more coarsely grained dolomites. These examinations also identified two possible sources of durability problems other than D-cracking in some pavement sections. The first consists of mineral growth in the air voids and the second consists of the presence of cracked shale particles and dark reaction rims, which are known to be associated with alkali-aggregate reactions. Either of these mechanisms might reduce the durability of the hardened concrete and could produce distresses on the pavement surface that appear similar to D-cracking. In addition, the rate of deterioration of these two mechanisms is also affected by cycles of freezing and thawing.

3.2.5 Rapid Freeze-Thaw Test (ASTM C 666 Procedure C)

Three 100-mm diameter cores were obtained from each pavement section included in this study for rapid freezing and thawing testing in accordance with ASTM C 666 procedure C, as described previously. The rapid freezing and thawing test subjects concrete specimens to repeated cycles of freezing and thawing while monitoring changes in the length and/or stiffness of the specimens as evidence of structural damage. Procedure C subjects the specimen to freezing in air and thawing in water while the specimens are cloth-wrapped to maintain specimen moisture during freezing. Procedure C was selected to test the cores retrieved based on recommendations by Janssen and Snyder (5) and results of the laboratory phase of this study, which are discussed later in this report.

The freeze-thaw test is typically continued until each specimen expands by 0.10 percent of its initial length, the relative dynamic modulus is reduced by 40 percent of its initial value, or 300 cycles of freezing and thawing have been completed without failure, whichever occurs first. The three 100-mm diameter cores were shortened to a length of 190 mm or less so that length measurements could be taken after each 36 cycles using a 200-mm digital caliper. The length measurements are reported as the average of three readings. The core sides were also sawed to produce specimens with a width of 75 mm in one direction to fit in racks in the freeze-thaw machine. A hole was drilled in the center of each end of the cores, and gage studs were anchored in these holes using an epoxy-vinylester resin. This was done to allow the use of a comparator for length measurements. However, measurements taken along the sides of the core using a digital caliper were found to be more reliable and reproducible.

The relative dynamic modulus was also determined after every 36 cycles of freezing and thawing. For each core, the following parameters were determined:

- DF_F : durability factor using relative dynamic modulus (RDM) criteria;
- DF_L : durability factor using dilation criteria; and
- d_L : percent dilation or length change after "c" cycles of freezing and thawing

The percent dilation, d_L, was calculated as:

$$d_L = 100 \times (L_2 - L_1) / (L_1 - 2g) \qquad \text{(Eqn. 3.1)}$$

where:

- d_L : length change of specimen after cycle c, percent;
- L_1 : length reading at cycle 0;
- L_2 : length reading at cycle c; and
- g : length of the embedded gage studs.

The relative dynamic modulus was determined using a modification of the ASTM C 215 procedure (modified to measure longitudinal frequency response rather than transverse frequency response). The modifications consisted of 1) placing the specimen on a flat horizontal surface instead of placing it on two parallel support wires; 2) attaching the accelerometer to the center of the core end instead of on the top face of the specimen; and 3) impacting the core horizontally on the end of the core opposite the accelerometer instead of impacting it vertically. The relative dynamic modulus of elasticity, P_c , was calculated as:

$$P_c = (n_c^2 / n_0^2) \times 100 \qquad \text{(Eqn. 3.2)}$$

where:

n_c : fundamental longitudinal frequency at cycle c, and

n_0 : fundamental longitudinal frequency at cycle 0.

The durability factor (DF) was calculated as:

$$DF = P \times N / M \qquad \text{(Eqn. 3.3)}$$

where:

P : relative dynamic modulus of elasticity at N cycles, percent;

N : number of cycles at which the test ended (RDM = 60 percent or 300 cycles, whichever occurs first);

M : specified number of cycles at which the test procedure is terminated (usually 300 cycles).

When the concrete specimen failed by reduction of the relative dynamic modulus, RDM, or has been exposed to 300 cycles of freezing and thawing without failure, the durability factor is referred to as the durability factor using relative dynamic modulus criteria, DF_F. When the concrete specimen failed by dilation criteria, the frequency at which the dilation criterion was reached was used to determine the relative dynamic modulus and the durability factor, DF_L.

Mayo (54) reported that a durability factor above 80 can be considered to be good and Tipton (3) reported that an aggregate with a durability factor of 60 or less should be considered nondurable. Some state agencies allow the use of aggregate sources with a lower limit for the durability factor (e.g., the Michigan Department of Transportation allows the use of aggregate sources with durability factors of 20 or higher in some highway applications).

The results of the freeze-thaw testing performed for this study are reported in tables 3.10 and 3.11. In this study, the differences between durability factors computed using the dynamic

Table 3.10. Results of freeze-thaw tests on cores (Groups I and II).

	Group I					**Group II**			
Aggregate Source:	**A**	**E**	**F**	**G**	**H**	**C**	**D**	**I**	**L1**
Aggregate Type	Carbonate	Carbonate	Gravel	Carbonate	Gravel	Carbonate	Carbonate	Carbonate	Carbonate
Route	TH 56	White Bear	Rest Area	TH 212	TH 175	TH 14	TH 90	TH 13	TH 52
Location (milepost,etc.)	35.5-36	Dell Street	Hwy 90	137.7-140.7	0-10.5	212.8-213.5	227-249.4	1.35-3.02	76.5-79.5
Years in Service	21	*(a)*	27 or less	20	24	27	22	22	33
Field Performance	Fair	Fair	Poor	Fair	Poor	Good	Fair	Good	Good
Freeze-Thaw Test Results (Procedure C)									
Durability Factor (RDM failure), DF_F	*35*	*59*	*35*	*17*	*41*	80	92	82	73
Durability Factor (length failure), DF_L	*30*	*36*	*32*	*17*	*31*	(b)	(b)	(b)	(b)
Dilation, d_L (%)	*0.139*	*0.134*	0.087	*0.102*	*0.180*	0.053	0.056	0.040	0.065
Visual Inspection of Cores									
Evidence of D-cracking	minor	low	severe	low to minor	low to medium	none	low	none	minor
Extent of Cracking	joint	joint	± 300 mm	± 600 mm	± 300 mm	*N/A*	joint	*N/A*	joint
Location	between wheel paths	between wheel paths	in and between wheel paths	in and between wheel paths	in and between wheel paths	*N/A*	in and between wheel paths	*N/A*	between wheel paths

(*a*): Data unavailable

(b): 0.10% dilation was not reached

N/A : Not applicable

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

Table 3.11. Results of freeze-thaw tests on cores (Group III).

	Group III					
Aggregate Source:	**B**		**M**		**N**	
Aggregate Type	Carbonate		Carbonate		Carbonate	
Route	TH 42	TH 42	TH 90	Cnty Rd 7	TH 90	TH 90
Location (milepost,etc.)	0.0-3.5	3.5-4.0	220.7-249.4	Rochester	172.4-175.8	166.2-172.4
Years in Service	28	28	23	22	31	31
Field Performance	Fair	Poor	Good	Poor	Good	Poor
Freeze-Thaw Test Results (Procedure C)						
Durability Factor (RDM failure), DF_F	74	78	88	*53*	79	80
Durability Factor (length failure), DF_L	(a)	(a)	(a)	(a)	(a)	(a)
Dilation, d_L (%)	0.050	0.065	0.040	*0.111*	0.009	0.047
Visual Inspection of Cores						
Evidence of D-cracking	minor	low to severe	none	low to medium	none	medium
Extent of Cracking	joint	± 300 mm	*N/A*	joint and cracks	*N/A*	± 300 mm
Location	wheel path	wheel path	*N/A*	in and between wheel paths	*N/A*	in and between wheel paths

(a): 0.10% dilation was not reached

N/A : Not applicable

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

modulus and dilation criteria were insignificant for pavements in group I, although most of the specimens failed first by dilation. The specimens in group II and most of the specimens of group III did not reach the dilation failure criteria.

Specimens retrieved from pavement sections that exhibited D-cracking (A, E, F, G and H) showed low durability factors (60 or less), high dilations (0.1 percent or more), or both. The low durability factors and high dilations for these aggregate source specimens (group I) correlated with their poor field performance and the fine-grained crystalline structure of their dolomites.

A low durability factor was also observed for cores obtained from the pavement section that contained aggregate from source M and exhibited D-cracking. This pavement section contained fine-grained crystalline dolomite, as described previously, which is consistent with its field performance. The cores obtained from the pavement section that contained aggregate from source M and exhibited no D-cracking exhibited a high durability factor and low dilation, indicating good resistance to frost damage. The cores from pavement sections which contained aggregate sources B and N and exhibited durability problems showed high durability factors (70 or higher) and low dilation (0.07 percent or less), which indicates that the source of their durability problems is probably not D-cracking.

Samples obtained from pavement sections that exhibited no D-cracking (group III) exhibited high durability factors (> 70) and low dilations ($\leq$ 0.065), which is consistent with their good field performances.

Figures 3.3 and 3.4 present plots of durability factor and dilation test results for groups I and II. These figures show that the rapid freezing and thawing test results for group I specimens were generally significantly different from the results for group II specimens, and that the differences in test results correlate well with the differences observed in their field performances. Pavement sections that exhibited D-cracking (group I) and contained very fine-grained dolomites or dolomitic limestones showed durability factors lower than 60, dilation higher than 0.1 percent or both. On the other hand, pavement sections that exhibited no D-cracking (Group II) and

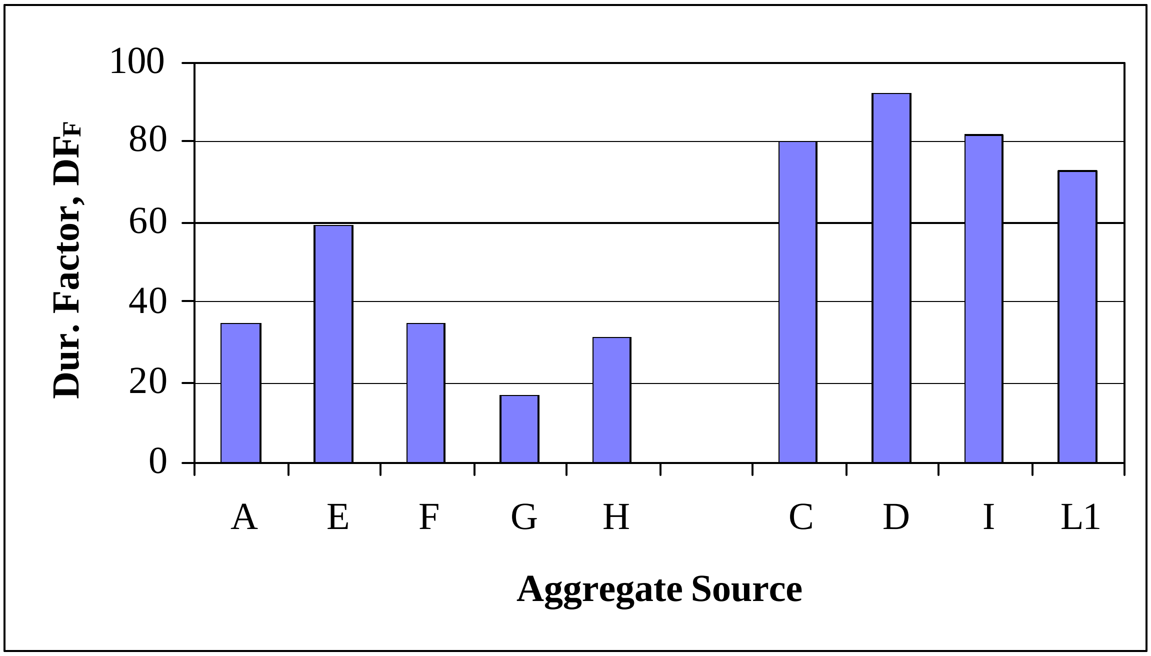

Figure 3.3. Durability factor values for Group I and II aggregate sources.

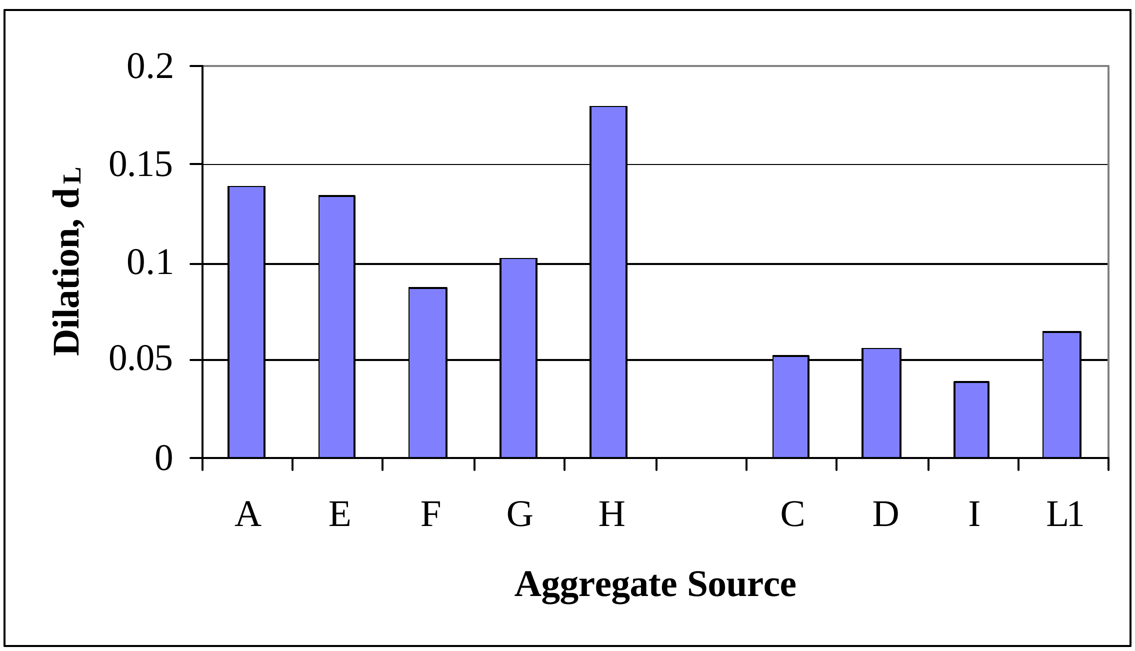

Figure 3.4. Freeze-thaw dilation test values for Group I and Group II aggregate sources.

contained more coarsely grained dolomites or dolomitic limestones showed durability factors higher than 70 and dilation lower than 0.065 percent. These observations are consistent with the results of other studies that relate fine-grained crystalline dolomite content to freeze-thaw durability problems (29, 85).

3.3 Correlation of Aggregate and PCC Properties and Field Performance

The field performance study indicated that two sources of durability problems other than D-cracking were present in some pavement sections. Samples obtained from three pavement sections showed evidence of secondary mineralization (ettringite formation in the air voids) and one pavement section contained cracked shale particles and dark reaction rims, which are often associated with alkali-aggregate reactions. These mechanisms probably reduced the durability of the concrete in these sections and resulted in rapid deterioration under the action of freezing and thawing cycles with distresses on the pavement surface that appear similar to D-cracking. The high durability factors and/or low dilation (0.07 percent or less) for cores obtained from these pavement sections indicate that D cracking is not present. Petrographic examination of the cores was useful in differentiating between these different failure mechanisms.

Several pavements sections showed evidence of D-cracking and petrographic examination of core samples from these sections indicated that very fine-grained dolomites were more susceptible to D-cracking than more coarsely grained dolomites. Fine-grained dolomites or dolomitic limestones also exhibited poor freeze-thaw durability in the laboratory under the rapid freezing and thawing test (ASTM C 666 procedure C), which confirmed that the very fine-grained dolomite is more susceptible to freeze-thaw action. Table 3.12 lists the pavement sections that were included in this study and summarizes the suspected causes and severity of deterioration observed in each section.

3.4 Summary

Field condition surveys were performed on 38 concrete pavement sections in Minnesota. These surveys were conducted to visually determine the extent of durability-related distresses and the presence of contributing factors. Cores were retrieved from fifteen of the surveyed pavement sections to determine the extent of their D-cracking and to use in laboratory testing.

The cores were inspected visually to positively identify the development of D-cracking. Slab deterioration in pavement sections that exhibited D-cracking was generally limited to within 300 mm of pavement joints and cracks with the exception of one pavement section, which also exhibited severe faulting and secondary mineralization.

Table 3.12. Suspected causes of deterioration for study sections.

Source	**Source Name**	**Field Performance**	**Probable Cause of Deterioration**
Group I: Nondurable Sources			
A	Grand Meadows	Fair	D-cracking
E	St. Paul Park	Fair	D-cracking
F	Luverne	Fair	D-cracking
G	Bryan Rock	Poor	D-cracking Secondary mineralization Alkali-silica reaction
H	Halma	Poor	D-cracking
Group II: Durable Sources			
C	Hammond	Good	
D	Wilson, Winona	Fair	Deficient mortar air system
I	Harris	Good	
L1	Zumbrota	Good	
Group III: Variable Sources			
B	Rochester	Good	Secondary mineralization
		Poor	Secondary mineralization
M	Stewartville	Fair	
		Poor	D-cracking
N	Kuennen's	Good	
		Poor	D-cracking Deficient mortar air system Alkali-silica reaction

The compressive strength of cores retrieved from the selected pavement sections ranged from 45.8 to 62.4 MPa; the split tensile strengths ranged from 3.3 to 6.7 MPa. These values are considered typical for moderate-strength paving concrete tested several years after placement.

The linear traverse test was used to estimate the air void system parameters which are considered to be strongly related to the frost resistance of the mortar and provide an indication of whether observed durability problems are attributable to the mortar or the aggregates. Samples obtained from most of the pavement sections failed two or more of the American Concrete Institute (ACI) recommended limits on air void system measurements, and some of them exhibited no freeze-thaw damage, suggesting that these recommendations are conservative or that the saturation levels in the concrete were not high enough to produce freeze-thaw damage.

Petrographic examinations were performed on polished concrete and aggregate sections to determine their mineralogy and examine the concrete components, especially the coarse aggregate. Pavement sections containing very fine-grained dolomites or dolomitic limestones were generally more highly correlated with freeze-thaw durability problems in the field than comparable pavement sections containing more coarsely grained dolomites. Poor performance of some of the selected pavement sections may be attributed to factors other than D-cracking, such as secondary mineralization, the presence of cracked shale particles, and alkali-aggregate reaction.

The rapid freezing and thawing test subjects concrete specimens to repeated cycles of freezing and thawing and was performed on cores retrieved from the field study sections. Cores obtained from pavement sections that exhibited no D-cracking were generally resistant to this test, exhibiting durability factors of 80 or more, dilation of 0.065 or less, or both. Specimens obtained from pavements that exhibited D-cracking showed large dilations (higher than 0.07) and low durability factors (60 or less).

CHAPTER 4

EVALUATION OF AGGREGATE FREEZE-THAW DURABILITY TESTS

4.1 Introduction

The laboratory test program consisted of two distinct parts with separate goals. The objectives of the first part of the laboratory portion of this study were to determine the sources of durability problems on selected pavement sections in southern Minnesota, evaluate the effectiveness of various tests of coarse aggregate and concrete freeze-thaw durability, and to develop a suite of tests for more quickly and accurately determining the freeze-thaw durability of coarse aggregate samples. This was achieved by performing durability tests on aggregate and concrete samples that are representative of the materials used in the study pavement sections and evaluating their effectiveness in predicting the freeze-thaw durability performance of the study sections.

The second portion of the laboratory test program involved the evaluation of techniques for mitigating D-cracking. This is described in Chapter 5.

4.2 Selection and Description of Study Coarse Aggregates

Aggregate samples obtained today from a given aggregate source might be expected to be at least somewhat different from the coarse aggregates produced at the same quarry or pit 20 to 30 years ago. In the case of gravel aggregates, for example, the aggregate composition and fractions are affected by the erosion of glacial deposits and carbonate or metamorphic bedrock by rivers and streams. Since river features often change rapidly, varying amounts and compositions of mineral deposits are expected in gravel samples. The proportion of different rocks in the gravel samples is a record of what the glacial river was eroding from the glacial deposits; this changes as the input to and features of the river change. Various deposits often overlie one another, so different materials may also be produced as excavation depth changes.

Some aggregate sources produce relatively uniform products over time. For example, dolomites represent sedimentary environments that were deposited about four hundred million years ago and are affected by the presence of ancient stacked sea floors and changes in the deposits by ground waters. Since dolomite deposits are generally extensive laterally, it is not unreasonable to expect some continuity and similarities between aggregates taken 20 to 30 years ago and aggregates taken recently if the same ledges and beds are being mined.

The coarse aggregate samples selected for use in this study were intended to match the aggregates used in the PCC pavement sections included in the field study as closely as possible.
Fourteen different aggregate sources were sampled for this study, including eleven that were obtained from the same sources used in the field study pavement sections and three sources that closely matched those used in the field study sections (for use when the original sources were no longer available).

Petrographic examinations of aggregate and polished concrete sections were performed on pavement samples to determine the mineralogy and condition of the coarse aggregate components. Concrete beams were prepared using coarse aggregate samples obtained from each of the selected sources, and thin slices were cut from these beams. (The quantity of aggregate remaining at Kuennen's quarry (source N) was not sufficient for casting concrete beams, so a thin section was cut from a small epoxy/aggregate conglomerate instead.) The thin sections were then polished to a thickness of 25 μm and examined using a petrographic microscope. Hand samples were examined using an optical stereo dissecting microscope.

The purpose of this petrographic examination was to determine whether the samples obtained for the laboratory testing portion of this study were comparable to those used in the construction of the study pavement sections decades ago and to determine whether some of todays products were suitable surrogates for the original aggregate sources in the evaluation of freeze-thaw durability tests. The three sources that were selected as "close matches" included:

- the Glenville quarry (source J), which was selected as a replacement for Kuennen's quarry (source N), which had been cleaned out and from which only a small sample (12 kg) was available;

- the Zumbrota quarry (source L2) was selected as a replacement for Zumbrota quarry (source L1), which was also cleaned out; and
- samples that were obtained from two different ledges of the Hammond quarry (sources C and K).

A summary of the geological descriptions of each aggregate source sample, as determined by petrographic examination, is presented in tables 4.1, 4.2 and 4.3. The results of the petrographic examinations and geological descriptions of each aggregate source and coarse aggregate in each pavement study section are also presented in Appendix A.

The following comparisons are presented to discuss the differences, if any, between the coarse aggregate used in the pavement study sections and the coarse aggregate samples obtained recently from the quarries or pits.

Source A: Grand Meadow

The coarse aggregates obtained from the quarry and those present in the Source A pavement section presented some lithological differences; however, both consisted of fine-grained dolomite microsparites, which were associated with poor durability in the field.

Source B: Rochester

The quarry sample aggregates were similar to those used in the Source B pavement section, which exhibited good field durability. Both samples contained fine-to-medium-grained dolomites with vugs and calcite-filled vugs.

Sources C and K: Hammond

Quarry samples C and K exhibited lithological similarities and were similar to the aggregates used in the pavement section which contained source C aggregates. Sample K contained more fine- to medium-grained dolomites than sample C. Sample K was classified as “DOT-Fail” because it was obtained from a ledge that produced some absorptive aggregates and, therefore, was not approved for paving in

Minnesota. Sample C was classified as “DOT-Pass” and contained mostly medium- to coarse-grained dolomites with fine-grained crystals intermixed.

Table 4.1. Geologic description of carbonates (Group I).

Aggregate Source	A: Grand Meadows	F: Luverne	H: Halma	G: Bryan Rock	E: St. Paul Park	L2: Zumbrota
Type	Carbonate	Gravel	Gravel	Carbonate	Carbonate	Carbonate
Formation	Cedar Valley: upper and lower Solon Member	Quaternary: alluvium	Quaternary: lake washed till	Shakopee: Prairie du Chein	Shakopee: Prairie du Chein	Shakopee: Prairie du Chein
Composition	biolithic dololutite	biolithic dololutite (35%) and igneous-metamorphic rocks (65%)	calcitic dololutite (60%) and igneous-metamorphic rocks (40%)	oolitic dolarenite: euthdral dolomite sparite/rhombs (55%); dolomite pseudosparite (10%); and dolomite pseudosparite (35%)	dololutite and dolarenite	quartzose dolarnite
Folk's Textural Classification of Carbonates	dolobiomicrosparite	dolobiomicrosparite	dolosparite	doloointrasparite	dolomicrosparite	sandy dolosparite
Carbonate Grain Size	< 60μm	< 10 μm	20 - 100 μm	5 - 400 μm	<25 - 500 μm	20 - 300 μm
Carbonate Grain Size Classification	very fine - fine	very fine - fine	very fine - fine	very fine - medium	very fine - coarse	very fine - medium
Median Grain Size of Carbonates	50 μm	5 μm	20 μm	140 μm	120 μm	140 μm (50% are very fine-grained)

Table 4.2. Geologic description of carbonates (Group II).

Aggregate Source	**C: Hammond (DOT Pass)**	**K: Hammond (DOT Fail)**	**D: Wilson, Winona**	**I: Harris**	**J: Glenville**
Type	Carbonate	Carbonate	Carbonate	Carbonate	Carbonate
Formation	Shakopee: Prairie du Chein	Shakopee: Prairie du Chein	Oneota: Prairie du Chein	Shellrock: Devonian	Cedar Valley: Devonian
Composition	dolarenite	dolarenite	dolarenite	dololutite and calcitic dolarenite	dololutite and dolarenite
Folk's Textural Classification of Carbonates	dolosparite	dolosparite	dolosparite	dolopseudosparite and calcitic dolosparite	dolosparite
Carbonate Grain Size	50 - 500 μm	50 - 500 μm	50 - 500 μm	1 - 400 μm	50 - 500 μm
Carbonate Grain Size Classification	fine - coarse	fine - coarse	fine - coarse	very fine - medium (60% very fine dolopseudosparite)	fine - coarse
Median Grain Size of Carbonates	200 μm	200 μm	250 μm	50 μm	150 μm

Table 4.3. Geologic description of carbonates (Group III).

Aggregate Source	B: Rochester	M: Stewartville	N: Kuennens
Type	Carbonate	Carbonate	Carbonate
Formation	Shakopee: Prairie du Chein	Stewartville: Ordovician	Shellrock
Composition	dolarenite	dolarenite	dolarenite
Folk's Textural Classification of Carbonates	dolosparite	dolobiosparite	dolosparite
Carbonate Grain Size	25 - 500 μm	50 - 200 μm	60 - 200 μm
Carbonate Grain Size Classification	fine - coarse	fine - medium	fine - medium
Median Grain Size of Carbonates	120 μm	150 μm	140 μm

Source D: Wilson quarry in Winona

The quarry sample was very similar to the coarse aggregate used in the source D pavement section. The quarry sample was composed of medium- to coarse-grained dolomites. The aggregate in the corresponding pavement study section was composed of medium-grained dolomites.

Source E: St. Paul Park (Shiely)

The quarry sample aggregate is lithologically similar to the aggregate used in the source E pavement section, although the quarry sample showed a slightly higher proportion of fine- to very fine-grained dolomite aggregate particles.

Source F: Luverne

The non-carbonate fractions of the pit sample and the coarse aggregate used in the source F pavement section exhibited similar lithological composition. The carbonate fractions were composed of very fine- to fine-grained dolomite and were similar with the exception that few dolomite particles in the pavement section were filled with coarse-grained calcite. In addition, a slightly higher proportion of carbonate material was found in the pit sample than in the pavement section.

Source G: Bryan Rock (Shakopee)

The quarry sample and the coarse aggregate used in the source G pavement section exhibited the same lithological composition. Both coarse aggregate samples showed approximately the same amount of medium-grained and fine- to very fine-grained dolomites.

Source H: Halma/Forester

The non-carbonate fraction of the pit sample and the coarse aggregate used in the source H pavement section exhibited similar lithological compositions. The carbonate fractions were composed of very fine- to fine-grained dolomites (pavement samples) or fine- to medium-grained dolomites (quarry sample). A few dolomite particles in the quarry sample were filled with coarse-grained calcite. The proportion of carbonate aggregates in the pit sample was significantly lower than the proportion observed in the pavement section.

Source I: Harris

The quarry aggregate sample differed significantly from the coarse aggregate used in the source I pavement section in that the quarry sample contained primarily very fine-grained dolomites and calcitic dolomites, whereas the coarse aggregate in the pavement section contained fine- to medium-grained dolomites.

Source L: Zumbrota

The original Zumbrota quarry (source L1) is no longer in business and has been cleaned out. A sample of supposedly similar material was obtained from a nearby quarry (designated source L2). The aggregate contained in the source L2 quarry sample and the source L1 pavement study section both contained quartzose dolomites; however, the quarry sample contained a larger proportion of very fine-grained dolomites than did the pavement section sample. The quarry sample also contained a higher proportion of insolubles (i.e., clay, silt and very fine-grained quartz).

Source M: Stewartville

The quarry sample contained fine- to medium-grained dolomites filled with coarse-grained calcite sparite. The quarry sample is similar to the aggregate used in the source M pavement study section that exhibited poor performance in the field.

Source N: Kuennen

Kuennen's quarry is currently a recreational area and the former quarry site is filled with water. Only a small sample of aggregates could be obtained from an old stockpile. The aggregate sample obtained and the coarse aggregate in the contained in the source N pavement study sections contained approximately the same proportions of fine- to medium-grained dolomites.

Source J: Glenville

The Glenville quarry is located within few miles of the Kuennen's quarry (source N), which was cleaned out. The Glenville sample was obtained for examination to determine whether it might produce aggregate similar to that formerly produced at Kuennen's quarry. The Glenville quarry sample was lithologically similar to the Kuennen's quarry sample, suggesting that the Glenville quarry may also draw from the Shellrock formation (like Kuennen's quarry) rather than the Cedar Valley formation. The Glenville quarry sample is composed mainly of fine- to medium-grained dolomites.

All of the carbonate aggregates included in this study consisted primarily of dolomites. The quarry and pit samples obtained for this study were generally similar to the coarse aggregates used in the pavement sections associated with the same sources with the exceptions of sources I and L, which exhibited some variation in the grain size of dolomites and the proportion of carbonate particles in the gravel samples.

4.3 Testing To Predict Freeze-Thaw Durability

Estimating the performance potential of an aggregate with respect to freeze-thaw durability is usually done through laboratory testing and consideration of service records. In this study, samples were obtained from each aggregate source and were used to evaluate the relative abilities of different aggregate and concrete durability tests to accurately predict the performance observed in the field study sections.

There are many tests for assessing coarse aggregate susceptibility to freeze-thaw damage and they can be separated into two major groups: 1) tests that simulate the conditions to which the coarse aggregates will be exposed in the field, and 2) tests that correlate aggregate properties and characteristics important to freeze-thaw durability with field performance and simulative test results.

The environmental simulative tests are generally considered to be better correlated with field performance and are widely used to reject or accept coarse aggregate sources for concrete applications (1). They are often time-consuming, however, sometimes requiring months of testing, and they often require expensive equipment. The environmental simulative tests selected for consideration in this study were the rapid freezing and thawing tests (variations of ASTM C 666) and the VPI single-cycle slow-freeze test.

Correlative tests (often called "quick-screening" tests) are preferred by many agencies because they require less time to perform (from a few days to two weeks) and are generally less expensive and easier to perform than the environmental simulative tests. The correlative tests selected for use in this study were the absorption and bulk specific gravity tests, PCA absorption and adsorption tests, Iowa pore index test, acid insoluble residue test, X-ray diffraction analysis, X-ray fluorescence analysis, thermogravimetric analysis and the Washington hydraulic fracture test.

The objectives of this part of study were to perform tests commonly used for assessing freeze-thaw durability and to correlate their results with observed field performance in order to evaluate their effectiveness in differentiating between frost resistant and non-resistant aggregates. The ultimate objective of this part of the study was to establish freeze-thaw durability rejection or acceptance criteria for using a particular coarse aggregate in PCC pavements.

4.3.1 Simulative Tests

The rapid freezing and thawing test (ASTM C 666) and the VPI single-cycle slow-freeze test were selected for inclusion in this study based on the literature review presented earlier. Test specimens were made from each aggregate source included in this study with the exception of source N (Kuennen's quarry), which is no longer producing aggregate.

Sample Preparation

Concrete beams were prepared using a type I-II portland cement and a natural coarse sand with a fineness modulus of 2.71, a saturated surface-dry specific gravity of 2.544 and an absorption capacity of 1.02 percent. All coarse aggregates were separated into component size fractions and then re-blended in appropriate proportions to produce identical gradations for each source. Mn/DOT gradation CA35 (maximum particle size of 38 mm), a common gradation for concrete paving, was used for most of the mixtures; however, some mixtures (sources E, K and M) were prepared using a CA60 gradation (maximum particle size of 20 mm) because only small particle sizes were available at the sources. The specified distribution of aggregate sizes by weight for the CA35 and CA60 gradations are presented in table 4.4. The cementitious content, coarse aggregate content, water-cement ratio, and coarse-to-fine aggregate volume ratio were all held constant at 337 kg/m^3, 0.41 m^3/m^3 PCC, 0.39 and 1.46, respectively. The laboratory test mixtures were prepared to meet Mn/DOT requirements for a 3A21 (air-entrained, slip-form consistency) mixture using saturated, surface-dry coarse aggregates and oven-dried fine aggregates. The exact mix designs used in the field pavement sections selected for this study were not available. Mixing and curing of the laboratory concrete was performed in accordance with ASTM C 192.

Table 4.4. Coarse aggregate gradation used in PCC mix designs.

	Gradation	
	CA 60	CA 35
Sieve Opening (mm)	Percent Passing (by Weight)	
37.5	100	100
32	100	95 – 100
19	100	55 – 85
16	85 – 100	N/A
9.5	40 – 70	20 – 45
4.76	0 - 12	0 – 7

Nine beams and four cylinders were cast from each test mixture: three sets of three 76- by 102- by 406-mm beams for the VPI single-cycle slow-freeze test and ASTM C 666 procedures B and C; three 100- by 200-mm cylinders for compression tests, and one 100- by 200-mm cylinder for split tensile testing. The 28-day compressive strength and indirect tensile strength of each mix are presented in tables 4.5 and 4.6.

Table 4.5. Results of laboratory tests (Group I).

Aggregate Source:	A	E	F	P (source F, carbonate only)	G	H	O (source H, carbonate only)	L2
28-day Strength Test Results								
Compressive Strength (kPa)	52,640	54,800	41,650	*(a)*	47,550	42,130	*(a)*	45,540
Tensile Strength (kPa)	5,850	7,140	5,590	*(a)*	5,420	6,380	*(a)*	4,230
Tensile Strength (kPa) with Salt-Treated Aggregates	4,880	4,970	4,630	*(a)*	6,140	4,810	*(a)*	4,590
Freeze-Thaw Test Results (Procedure B)								
Durability Factor (RDM failure) DF_F	82	95	98	*(a)*	95	95	*(a)*	80
Dilation, d_L (%)	0.059	0.007	0.015	*(a)*	0.078	0.008	*(a)*	0.060
Freeze-Thaw Test Results (Procedure C)								
Durability Factor (RDM failure) DF_F	75	90	94	*(a)*	66	91	*(a)*	68
Dilation, d_L (%)	0.050	0.018	0.032	*(a)*	0.121	0.013	*(a)*	0.057
Freeze-Thaw Test Results (Procedure B with Salt-Treated Aggregates)								
Durability Factor (RDM Failure) DF_F	*32*	88	79	*(a)*	*22*	84	*(a)*	*58*
Dilation, d_L (%)	0.086	0.054	0.043	*(a)*	0.084	0.035	*(a)*	0.061
VPI Single-Cycle Freeze Test Results								
Temperature Slope, b_t (mm/^{o}C)	*-1.3*	*-0.9*	0.2	*(a)*	1.7	*-0.4*	*(a)*	*-0.1*
Time Slope, b_t (mm/hour)	*-21.3*	*-11.2*	*-10.7*	*(a)*	2.1	-7.6	*(a)*	-7.6
Absorption and Specific Gravity Test Results								
Bulk Specific Gravity at 23C	2.518	2.695	2.625	*(a)*	2.576	2.645	*(a)*	2.588
Absorption (%)	3.028	1.456	1.547	*(a)*	2.751	0.968	*(a)*	2.237
Iowa Pore Index Test Results								
Primary Load (ml)	153.5	55.6	46.3	86.7	85.3	31.3	60.8	104.8
Secondary Load (ml)	*38*	23	21	*31*	*58*	19	*39*	*29*
Quality Number	2.62	1.75	1.69	2.31	5.29	1.65	3.46	2.01
PCA Absorption & Adsorption Results								
Absorption (%)	4.938	1.581	1.513	3.616	2.333	1.695	3.099	2.080
Adsorption (%)	0.582	0.895	0.755	1.010	0.266	0.315	0.581	0.228
Acid Insoluble Residue Test Results								
Total Residue (%)	5.2	9.3	45.4	*(a)*	12.9	55.5	*(a)*	26.6
Silt and Clay Residue (%)	4.1	4.5	3.6	*(a)*	7.5	3.7	*(a)*	9.5
Pavement Vulnerability Factor, PVF	40	71	92	(a)	64	96	(a)	82
Washington Hydraulic Fracture Test Results								
Hydraulic Fracture Index, HFI	65	1480	*24*	*19*	*19*	66	70	*21*
Percent Fracture, PF (%)	3.8	0.2	*8.8*	*13.2*	8.5	3.8	3.6	17.1
X-ray Diffraction Results								
Dolomite D-Spacing Factor	2.8910	2.8909	2.8885	2.8884	2.8911	2.8906	2.8893	2.8892
X-ray Fluorescence Results								
Percent Strontium (%)	0.013	0.014	0.044	0.024	0.017	0.055	0.017	0.013
Percent Phosphorous (%)	0.036	0.030	0.063	0.034	0.046	0.121	0.035	0.024
Thermogravimetric Analysis Results								
Dolomite Loss Rate (%/^{o}C)	0.0153	0.0425	0.0217	0.0262	0.0231	0.0269	0.023	0.0459
Limestone Loss Rate (%/^{o}C)	0.0225	*N/A*	*N/A*	0.0250	*N/A*	0.0250	0.0250	*N/A*
Insoluble Residue (%)	3.1	8.2	46.2	4.5	3.6	36.4	3.2	23.1

(a) : Only limited testing was performed on the carbonate fraction samples of gravel sources

N/A : Not applicable or not available

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

Table 4.6. Results of laboratory tests (Groups II and III).

	Group II					Group III		
Aggregate Source:	**C**	**D**	**I**	**J**	**K**	**B**	**M**	**N**
28-day Strength Test Results								
Compressive Strength (kPa)	50,510	45,580	52,620	45,910	49,080	53,330	*(b)*	*(b)*
Tensile Strength (kPa)	6,060	7,090	6,840	8,040	7,750	5,060	*(b)*	*(b)*
Tensile Strength (kPa) with Salt Treated Aggregates	4,640	6,060	5,170	4,900	5,410	5,220	*(b)*	*(b)*
Freeze-Thaw Test Results (Procedure B)								
Durability Factor (RDM failure) DF_F	99	101	97	100	101	97	101	*(b)*
Dilation, d_L (%)	-0.007	-0.018	0.008	0.003	-0.001	0.004	0.001	*(b)*
Freeze-Thaw Test Results (Procedure C)								
Durability Factor (RDM failure) DF_F	98	99	89	99	100	97	*(b)*	*(b)*
Dilation, d_L (%)	0.012	-0.014	0.018	0.006	0.003	0.012	*(b)*	*(b)*
Freeze-Thaw Test Results (Procedure B with Salt-Treated Aggregates)								
Durability Factor (RDM Failure) DF_F	99	103	90	99	99	98	98	*(b)*
Dilation, d_L (%)	-0.003	0.008	0.054	-0.005	0.006	0.005	0.009	*(b)*
VPI Single-Cycle Freeze Test Results								
Temperature Slope, b_l (mm/°C)	0.1	0.3	0.8	0.5	0.9	0.7	*(b)*	*(b)*
Time Slope, b_t (mm/hour)	-5.1	-5.1	-5.1	-2.5	3.8	0.0	*(b)*	*(b)*
Absorption and Specific Gravity Test Results								
Bulk Specific Gravity at 23C	2.703	2.649	2.666	2.729	2.686	2.672	2.573	2.74
Absorption (%)	1.126	1.605	1.363	0.932	1.422	1.293	2.534	0.802
Iowa Pore Index Test Results								
Primary Load (ml)	55.1	73.5	41.5	25.9	53.9	75.0	106.3	30.7
Secondary Load (ml)	22	18	17	17	20	24	23	22
Quality Number	1.70	1.27	1.29	1.58	1.50	1.72	1.50	2.02
PCA Absorption & Adsorption Results								
Absorption (%)	1.037	1.737	1.288	0.931	1.381	2.364	3.300	0.767
Adsorption (%)	0.459	0.359	0.277	0.393	0.605	0.325	0.257	0.520
Acid Insoluble Residue Test Results								
Total Residue (%)	6.2	7.6	5.4	5.3	7.6	9.9	*(b)*	*(b)*
Silt and Clay Residue (%)	3.3	4.0	5.1	4.8	3.3	4.8	*(b)*	*(b)*
Pavement Vulnerability Factor, PVF	68	65	61	69	67	75	*(b)*	*(b)*
Washington Hydraulic Fracture Test Results								
Hydraulic Fracture Index, HFI	54	54	146	224	166	54	790	206
Percent Fracture, PF (%)	4.6	4.6	1.7	1.1	1.5	4.6	0.3	1.2
X-ray Diffraction Results								
Dolomite D-Spacing Factor	2.8896	2.8891	2.8927	2.9016	2.8905	2.8888	2.8911	2.9005
X-ray Fluorescence Results								
Percent Strontium (%)	0.018	0.017	0.020	0.025	0.015	0.013	0.016	0.026
Percent Phosphorous (%)	0.015	0.009	0.030	0.016	0.014	0.019	0.066	0.010
Thermogravimetric Analysis Results								
Dolomite Loss Rate (%/°C)	0.0729	0.0488	0.0184	0.0227	0.0735	0.025	0.0171	0.0294
Limestone Loss Rate (%/°C)	*N/A*	*N/A*	0.0212	*N/A*	*N/A*	*N/A*	0.0250	*N/A*
Insoluble Residue (%)	0.7	8.9	4.1	3.8	1.1	9.9	3.1	3.7

(b) : Quarry closed; small sample did not permit completion of full battery of tests.

N/A : Not applicable or not available

Note: *Italicized values* do not meet generally accepted criteria for acceptance.

Three beams were also prepared using salt-treated aggregates to investigate the effect of deicing salt on freeze-thaw durability using ASTM C 666 procedure B. The salt treatment consisted of five cycles of drying the aggregate in an oven at 110^{o}C for 24 hours, followed by immersion in a 23^{o}C saturated solution of pure reagent sodium chloride for 24 hours (the salt brine solution is poured over the aggregate immediately after it is removed from the oven). After the final salt treatment, the coarse aggregates were rinsed with clean tap water. The salt treatment of coarse aggregates was proposed by Dubberke and Marks to simulate and account for the exposure of the PCC concrete and coarse aggregates to deicing salts (46).

Standard Test for the Resistance of Concrete to Freezing and Thawing (ASTM C 666)

The rapid freezing and thawing test (ASTM C 666) subjects concrete beams to cycles of freezing and thawing. Three procedures were used:

- procedure B, which consists of freezing in air and thawing in water (no containers);
- procedure C, which is the same as procedure B except that the specimens are subjected to freeze-thaw cycles in cloth wraps to maintain specimen moisture during the freezing portion of the cycle; and
- procedure B using beams made with salt-treated aggregates.

Procedure C was introduced to keep the specimens wet during freezing without confining the expansion of ice and saturated concrete in containers (5). The five-cycle salt treatment of coarse aggregates (Iowa salt treatment) was introduced by Dubberke and Marks to investigate the effects of deicing salts on aggregate freeze-thaw durability after fine-grained dolomites were found to deteriorate significantly in the presence of deicing salts (46).

For each aggregate source and set of beams, the durability factors were determined using relative dynamic modulus (RDM) criteria and dilation failure criteria (i.e., DF_F and DF_L; respectively). Percent dilation after RDM failure or 300 cycles (i.e., d_L) was also determined.

The results of the freeze-thaw testing performed for this study are reported in tables 4.5 and 4.6. In this study, the differences between durability factors computed using the dynamic modulus and dilation criteria were insignificant and few beams failed by dilation. Sources A, G and L2 exhibited large dilations when procedure B was used. However, the durability factors associated with all aggregate sources were high (> 80), even for sources believed to be frost-susceptible on the basis of field performance (e.g., F and H). Procedure C yielded lower durability factors than procedure B for historically nondurable aggregates, but even these durability factors were still higher than 60. In addition, procedure C results generally provided the best precision.

Aggregates treated using the Iowa salt procedure produced low durability factors for sources A, G and L2 and resulted in large dilations for sources E, H and I, as shown in figures 4.1 and 4.2. It should be noted that the source I aggregates were significantly different from those in the source I pavement core; the fine-grained quarry sample produced marginal performance when salt-treated aggregates were tested using ASTM C 666 procedure B (0.042 percent dilation), as would be expected for fine-grained dolomites.

In summary, ASTM C 666 procedure C provided better correlation with field performance than procedure B. However, the durability factors for known nondurable aggregate sources were higher than 60. Most nondurable sources tested (all but H) dilated significantly. The use of procedure B with salt-treated aggregates showed the best correlation with accepted durability factor failure criteria by providing low durability factors (below 60) or high dilation for historically nondurable aggregates.

VPI Single-Cycle Slow-Freeze Test

A set of three 75- by 100- by 400-mm (nominal size) concrete beams were used for this test. This test was performed after 28 days of moist curing at a temperature of 23^{o}C. Length measurements were taken every 15 minutes while the specimen temperature dropped from 21 to 4.5^{o}C, and every 10 minutes while the specimen temperature was below 4.5^{o}C. The test procedure calls for recording the temperature every 5 minutes between 4.5 and -9.5^{o}C; however, this measurement frequency was not accomplished in this study because the frequent opening of the freezer to measure strain produced specimen temperature fluctuations.

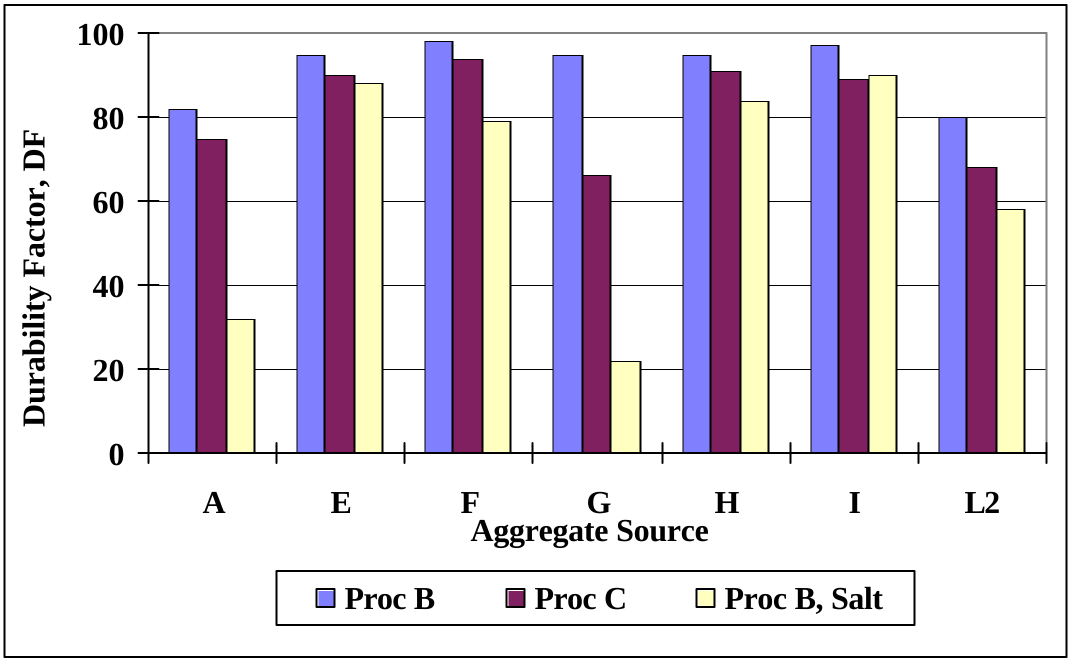

Figure 4.1. Durability factors for nondurable aggregate sources (Group I) and Source I.

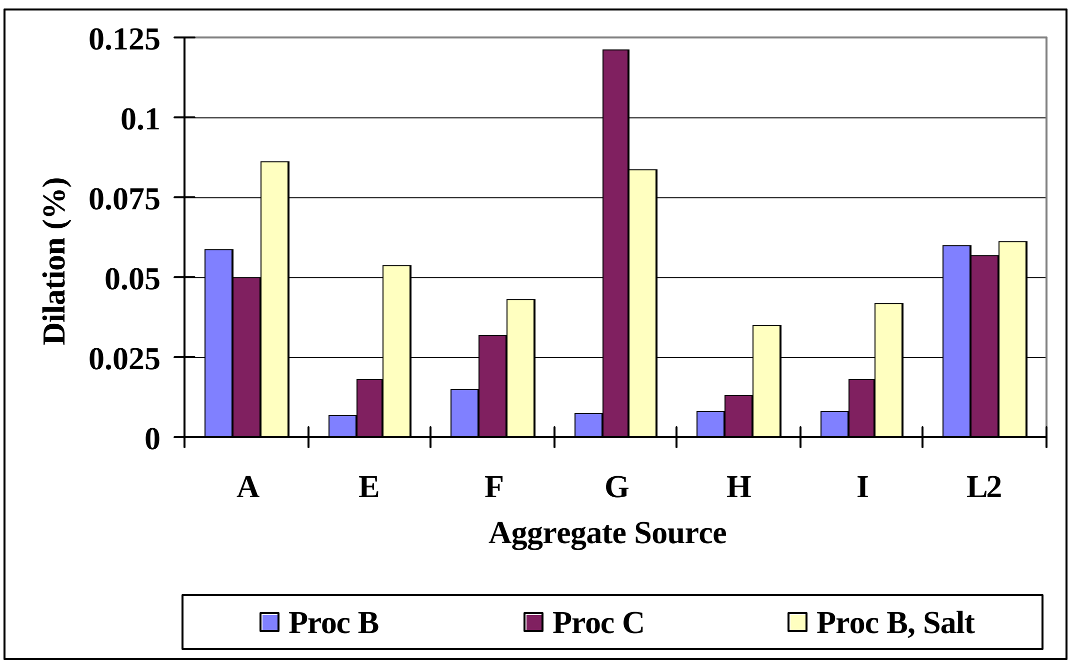

Figure 4.2. Percent dilations for nondurable aggregate sources (Group I) and Source I.

For each aggregate source, the minimum temperature slope, b_l, and the time slope, b_t, are given in tables 4.5 and 4.6. Sources A, E and H had temperature slopes less than zero, which indicates nondurable aggregate and is consistent with their field performance and their low durability factors and/or high expansions observed when tested using ASTM C 666 procedure B with salt-treated aggregates. Source L2 also exhibited a negative temperature slope, low durability factor and high expansion.

Based on the time slope criteria (i.e., when the time slope is less than -10.2, the freeze-thaw durability of the aggregate is questionable, but that when it is higher than 2.5, no further testing is required and the aggregate is durable), poor durability was predicted for sources A, E and F (time slopes of -21.4, -11.2 and -10.7, respectively). Source H showed a time slope of -7.6, which is close to the threshold for classification as a nondurable aggregate source; the field performance and the results of ASTM C 666 procedure B using salt-treated aggregates also indicate that this material is nondurable. Conversely, source K is considered a durable aggregate source in Minnesota pavements and showed a time slope of 3.8, which implies good performance.

The VPI test method successfully identified some very durable and nondurable aggregate sources (six sources), but further testing was indicated for six other sources. The only source that the test failed to correctly identify as nondurable was the highly porous source G, which may be failing at the aggregate-matrix interface rather than in the aggregate itself. Thus, it appears that this test is generally a reliable method for identifying very freee-thaw susceptible aggregate sources in only few weeks.

4.3.2 Correlative Tests

Correlative tests and analyses are performed to determine aggregate properties that usually affect the freeze-thaw durability of concrete. Selected correlative tests included the aggregate absorption and bulk specific gravity tests, PCA absorption and adsorption tests, Iowa pore index test, acid insoluble residue test, x-ray diffraction analysis, x-ray fluorescence analysis, thermogravimetric analysis and the

Washington hydraulic fracture test. Tables 4.5 and 4.6 present the results of correlative tests of the study aggregate sources and table 4.7 presents the linear correlation matrix between simulative and correlative test results.

Absorption Capacity and Bulk Specific Gravity

The absorption capacity and specific gravity of the coarse aggregates were determined in accordance with ASTM C 127. A summary of the results of absorption and specific gravity testing is presented in tables 4.5 and 4.6.

The absorption of coarse aggregates included in this study ranged from 0.8 to 3.03 percent. The fine-grained carbonate aggregates (sources A, G and M) had the highest absorption capacities due, in part, to the large surface area for water and ion sorption produced by the fine pore size (85). Good correlation was observed between the ASTM C 666 procedure B results using salt-treated aggregates and absorption (r^2 = 0.85 with durability factors and 0.71 with dilations, as shown in figure 4.3). Good correlation is expected because freeze-thaw damage is related to the expansion of water in the pores (hydraulic pressures) and the osmotic differences generated by the ice formation, while the use of salt-treated aggregate affects on the sorption and porosity of the aggregate and increases the potential for water to penetrate the pores.

In general, absorption capacities below 1.5 percent were associated with durability factors of 80 or more, while absorption capacities above 2 percent were linked with durability factors of 60 or less.

The bulk specific gravity for the aggregate sources tested in this study varied from 2.52 to 2.74. Figure 4.4 illustrates the relationship between specific gravity and durability factor or dilation for the study aggregates. There is a strong correlation ($|r| > 0.8$) between specific gravity and durability factor (Procedure B with salt-treated aggregates). Figure 4.4 also shows a strong correlation between specific gravity and dilation. This figure suggests good durability for specific gravities greater than 2.6. This might be explained by the fact the specific gravity is an indicator of both aggregate porosity and particle strength. The porosity of coarse aggregate of a given

Table 4.7. Correlation matrix for selected tests of durability and aggregate properties.

	BSG	ABS	IPIT SL	ACIR TR	DF-C	DF-S	DL-C	DL-S
BSG	1.00	-0.93	-0.61	-0.19	0.81	0.88	-0.67	-0.89
ABS	-0.93	1.00	0.73	-0.16	-0.84	-0.92	0.76	0.84
IPIT SL	-0.61	0.73	1.00	-0.11	-0.80	-0.90	0.93	0.70
PCA ABS	-0.87	0.84	0.55	-0.11	-0.53	-0.69	0.39	0.63
PCA ADS	0.24	-0.16	0.00	0.00	0.27	-0.01	-0.13	0.00
ACIR TR	-0.19	-0.16	-0.11	1.00	-0.05	-0.01	0.03	0.14
ACIR SCR	-0.41	0.49	0.51	0.00	-0.76	-0.51	0.63	0.47
HFI	0.31	-0.13	-0.15	-0.22	0.44	0.43	-0.36	-0.53
XRD D	0.51	-0.39	-0.23	-0.29	0.18	0.15	-0.13	-0.24
XRD PS	0.13	-0.43	-0.29	0.87	0.28	0.21	-0.20	-0.09
XRD PP	-0.39	0.11	0.05	0.86	-0.13	-0.18	0.14	0.30
TGA DL	0.38	-0.29	-0.27	-0.22	0.23	0.36	-0.27	-0.48
TGA LL	-0.11	-0.06	0.18	0.94	0.29	0.10	-0.30	-0.53
DF-B	0.78	-0.71	-0.41	-0.08	0.77	0.67	-0.46	-0.68
DL-B	-0.81	0.86	0.86	0.03	-0.97	-0.96	0.93	0.87
VPI-TEMP	0.21	-0.09	0.22	-0.17	-0.03	0.04	0.32	-0.08
VPI-TIME	0.50	-0.36	-0.01	-0.18	0.17	0.30	0.06	-0.41

BSG : Bulk Specific Gravity at 23C
ABS : Absorption (%)
IPIT SL : IPIT Secondary Load (ml)
PCA ABS : PCA Absorption (%)
PCA ADS : PCA Adsorption (%)
ACIR TR : Total Acid Insoluble Residue (%)
ACIR SCR : Silt and Clay Residue (%)
HFI : Hydraulic Fracture Index
XRD D : X-ray D-Spacing Factor
XRD PS : Strontium Content from X-ray Diffraction
XRD PP : Phosphorous Content from X-ray Diffraction
TGA DL : TGA Dolomite Percent Loss Rate
TGA LL : TGA Limestone Percent Loss Rate
DF-B : Durability Factor (ASTM C 666 Procedure B)
DF-C : Durability Factor (ASTM C 666 Procedure C)
DF-S : Durability Factor (ASTM C 666 Procedure B and Salt-Treated Aggregates)
DL-B : Dilation (ASTM C 666 Procedure B)
DL-C : Dilation (ASTM C 666 Procedure C)
DL-S : Dilation (ASTM C 666 Procedure B and Salt-Treated Aggregates)
VPI-TEMP : Temperature Slope from VPI Single-Cycle Freeze Test
VPI-TIME : Time Slope from VPI Single-Cycle Freeze Test

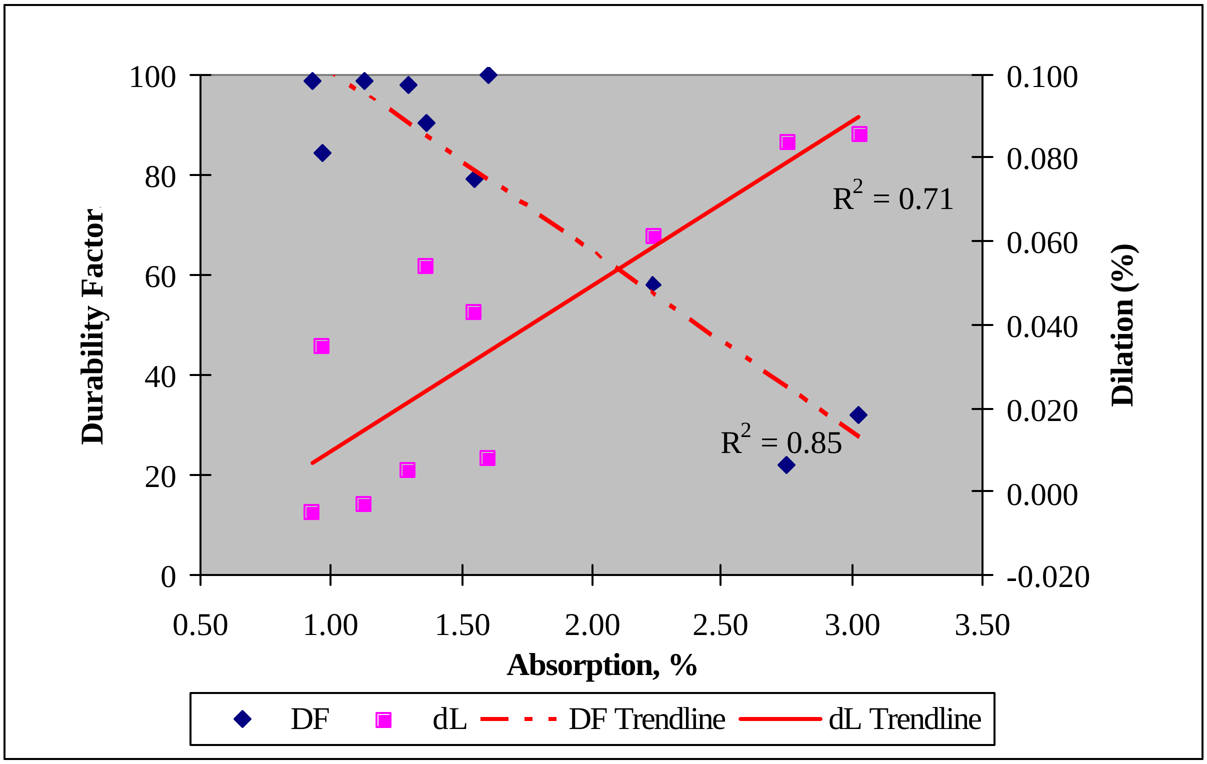

Figure 4.3. Absorption vs. ASTM C666 results (Procedure B and salt-treated aggregates).

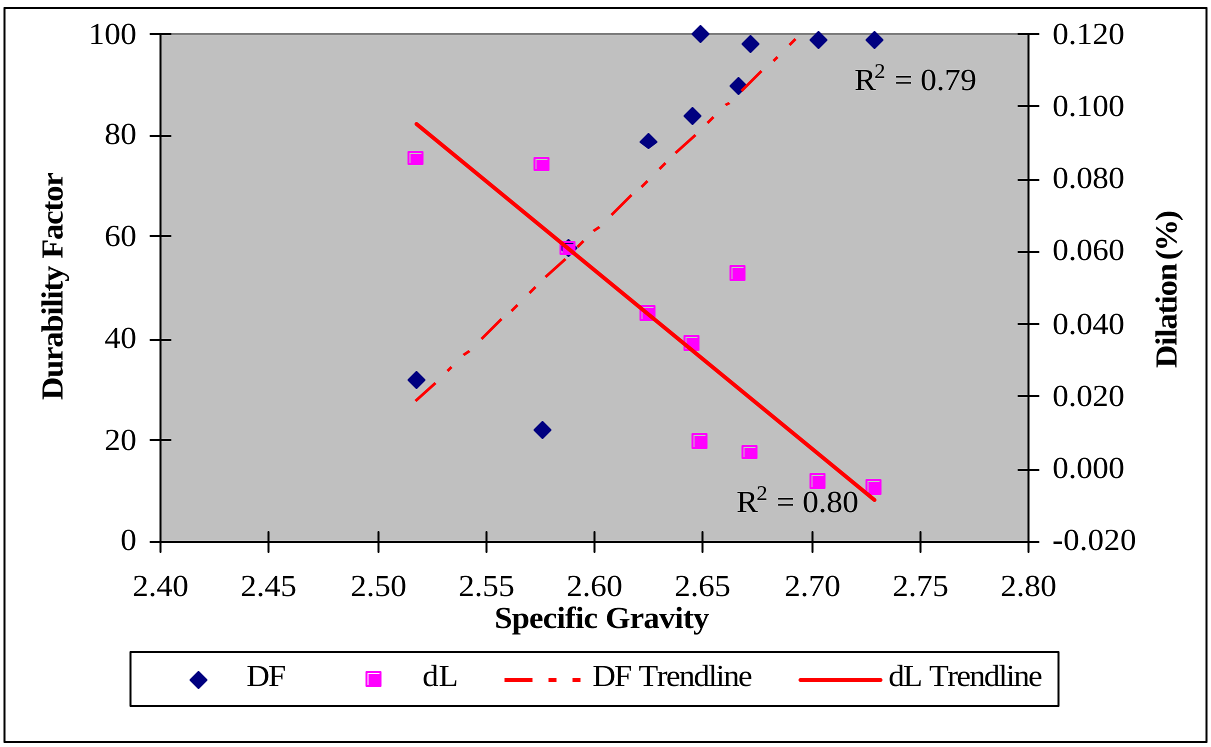

Figure 4.4. Specific gravity vs. ASTM C666 results (Procedure B and salt-treated aggregates).

mineralogy is generally expected to decrease when the bulk specific gravity is increased and thus the freeze-thaw durability is improved.

Absorption / Adsorption (PCA Method)

Absorption/adsorption test samples were prepared for each aggregate source using the test method described by Klieger, et al. (2). Two additional samples, P and O, were obtained for this testing by sampling the carbonate fractions of the source F and H gravel sources. An adsorption value of less than 0.1 percent identifies a durable aggregate while aggregates with high absorption (i.e., 0.3 percent or higher) and high adsorption (i.e., 0.1 percent or higher) are considered susceptible to D-cracking and popouts.

The aggregate samples included in this study were characterized by absorption capacities between 0.9 and 4.9 percent, and adsorption capacities between 0.23 and 1.01 percent, as shown in tables 4.5 and 4.6. These aggregates are all outside the PCA criteria for the absorption and adsorption of durable aggregate. This observation is similar to those of other researchers, all of whom concluded that the absorption-adsorption criteria for aggregate acceptance are very restrictive and may classify sources with good field records as nondurable (10, 35, 36, 42).

In this study, no correlation was observed between PCA absorption and adsorption values and field performance or rapid freeze-thaw test results.

Acid Insoluble Residue Test

The acid insoluble residue test is used to determine the percentage of insoluble residue in carbonate aggregates by using hydrochloric acid solution to dissolve the carbonate material. Test values lower than 30 percent are generally considered to indicate good freeze-thaw durability (42).

Tests of aggregates included in this study showed total acid insoluble residue contents between 5.2 and 26.6 percent for the carbonate rocks, and silt and clay residue contents ranging from 2.8 to 9.5 percent for the carbonate rocks and gravels. Although these total acid insoluble residue test values were all lower than the 30 percent threshold value, field performances varied widely. Some nondurable (Group I) sources exhibited relatively high total residues (e.g., 26.6 and 12.9 percent for sources L2 and G, respectively), while others had very low values (e.g., 5.2 percent for source A) and comparable insoluble residue values were often associated with very different durability factors and field performances (e.g., 5.2 percent for source A and 5.3 percent for source J), as shown in figure 4.5. In general, the correlation between total insoluble residue and freeze-thaw test results was very weak (|r| < 0.15). The correlation of silt and clay residue and freeze-thaw results was moderate to weak (|r| < 0.78).

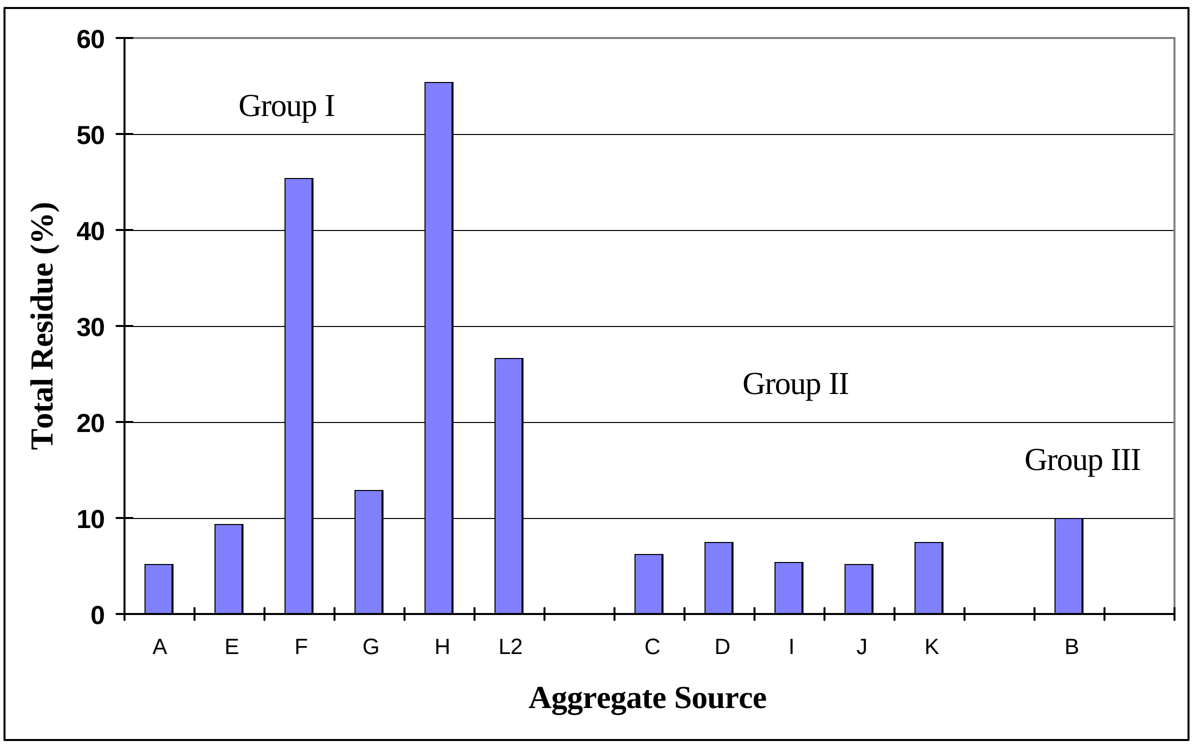

Figure 4.5. Acid insoluble residue test results (Total Residue, %).

The Pavement Vulnerability Factor (PVF) for the aggregates used in this study ranged from 39.8 to 82.1. Since the PVF failed to predict the good field performance associated with several of the study aggregates without the benefit of other tests, it was found to be of little use in this study.

Iowa Pore Index Test

The primary and secondary loads are believed to reflect the amount of water required to fill the aggregate macropores and micropores, respectively. A secondary load greater than 27 ml is believed to indicate susceptibility to D-cracking (36).

The results obtained for the aggregates included in this study indicated that sources A, G and L2 are susceptible to D-cracking. The two gravels included in this study (sources F and H) showed secondary loads lower than 27, but their carbonate fractions showed secondary loads of 39 and 31 ml, respectively, which indicates that the carbonate fractions are susceptible to D-cracking.

The secondary load values showed a good correlation with field performance for the carbonate aggregates, as shown in figure 4.6. Although the gravels passed the 27 ml criteria, their carbonate fractions (which comprise more than 30 percent of each gravel source) failed the same criteria, and the presence of as little as 10 percent nondurable aggregates in a PCC pavement has been determined to be enough to produce frost damage (33, 36). Although source A (secondary load = 38 ml and a known nondurable source) was observed to perform fairly in the project observed for this study, this may be attributed to the reduced maximum top size of the coarse aggregate (25 mm). This suggests that the Iowa pore index test is not sensitive to aggregate particle size and that it might be appropriate to vary the pore index failure criteria with particle size.

Overall, the test results confirm earlier results reported by Marks and Dubberke (36), Traylor (6) and Glass (49), who stated that Iowa pore index test results higher than 27 correlate strongly with the frost susceptibility for crushed stone aggregates. Very strong correlation ($r^2 = 0.81$) was observed between the secondary load and freeze-thaw durability factor when ASTM C 666 procedure B and salt-treated aggregates were used, as shown in figure 4.7. A very strong correlation ($r^2 = 0.87$) was also observed between secondary load and percent dilation when ASTM C 666 procedure C was used, as shown in figure 4.8.

The primary loads for the aggregate sources included in this study varied from 26 to 154. The primary loads for group I were slightly higher than for group II, as shown in figure 4.9, and the

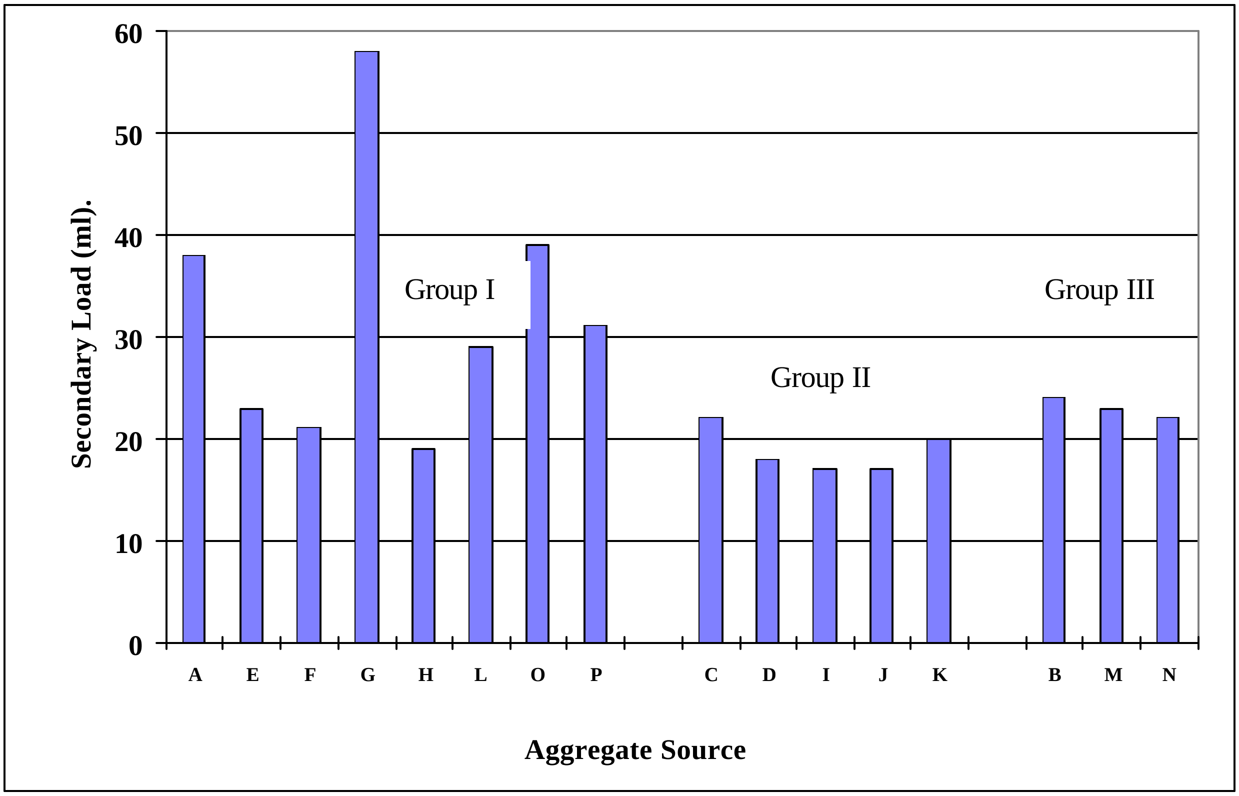

Figure 4.6. Iowa pore index secondary load test results.

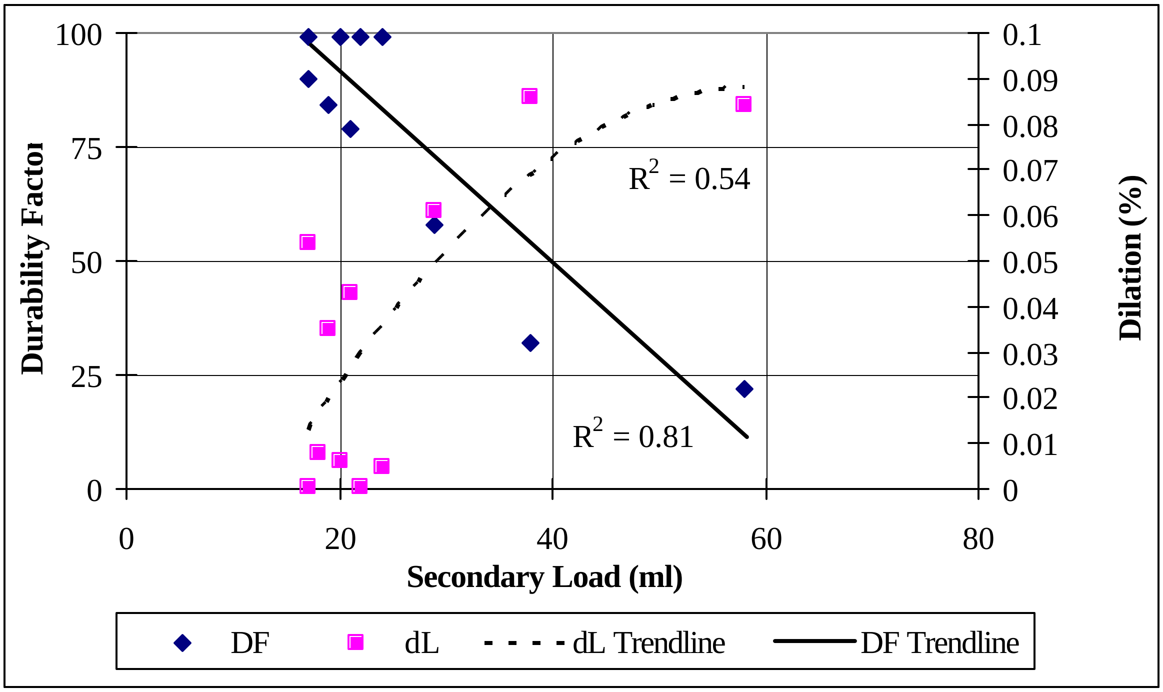

Figure 4.7. Secondary loads vs ASTM C 666 test results
using Procedure B and salt-treated aggregate.

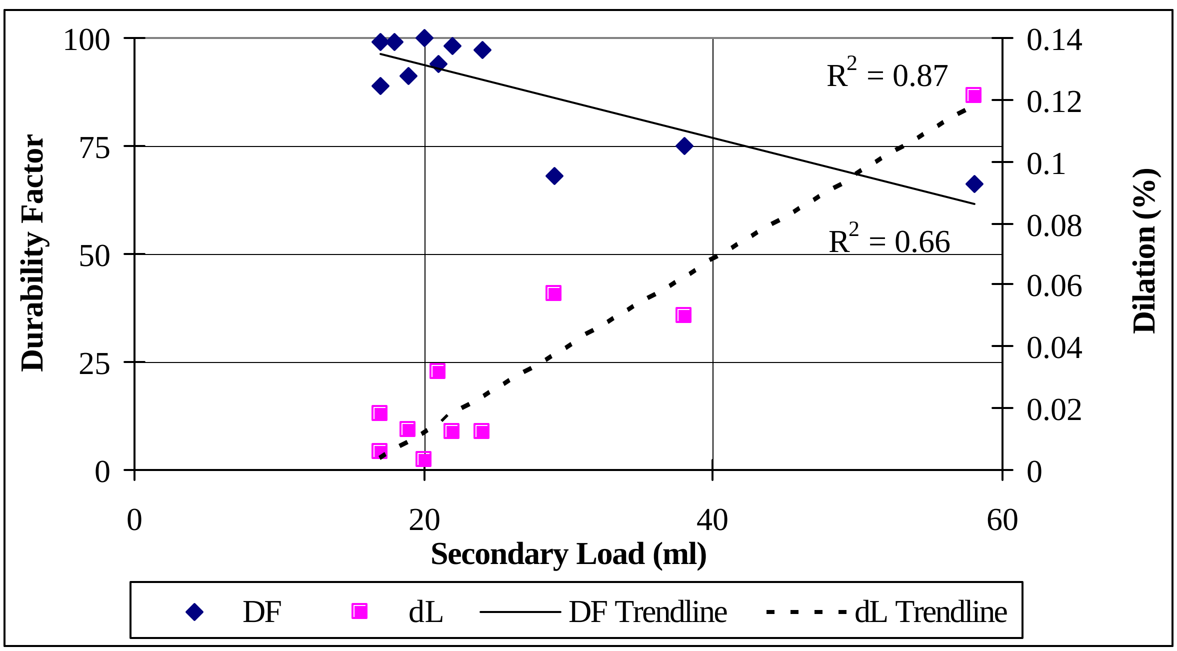

Figure 4.8. Secondary loads vs ASTM C 666 test results using Procedure C.

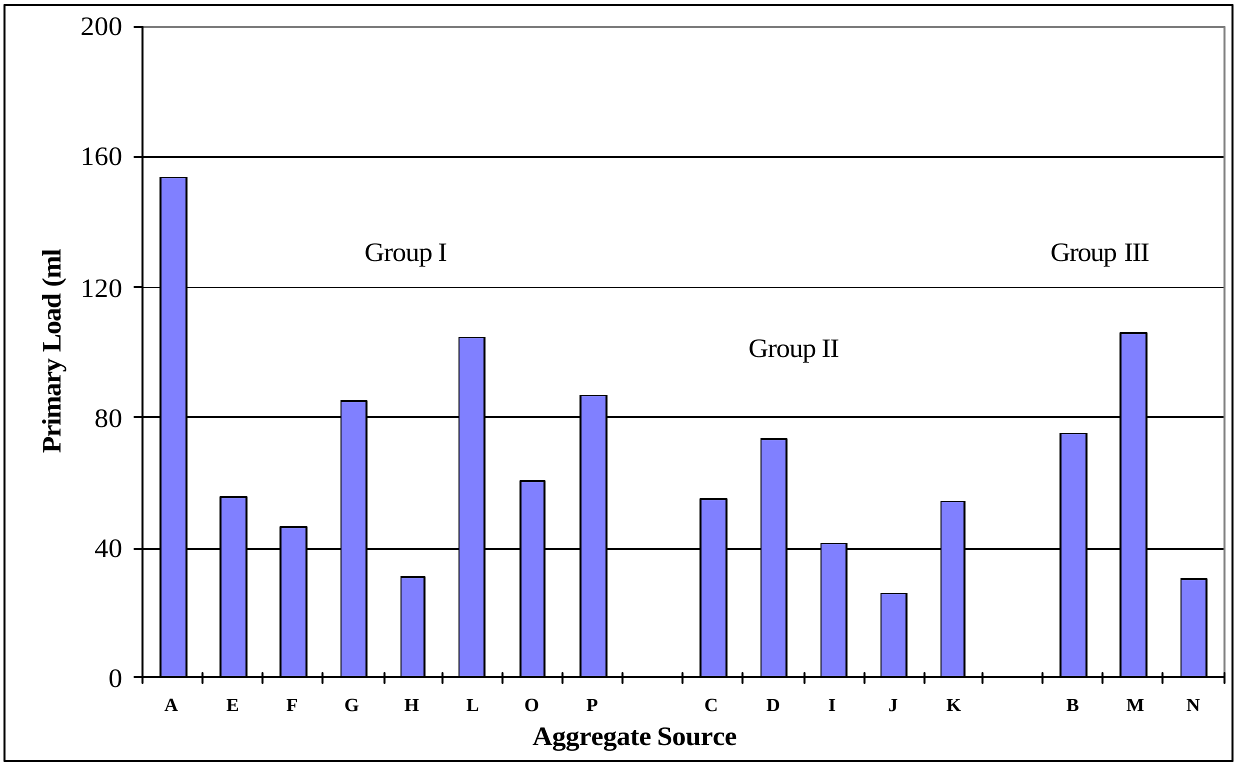

Figure 4.9. Iowa pore index primary load test results.

primary loads for all aggregate sources showed strong correlation with the absorption values (r^2 = 0.82) as shown in figure 4.10. The trend for higher primary loads for nondurable aggregates is explained by the fact that the primary load is closely and directly related to absorption capacity and increased availability of water increases the potential for freeze-thaw damage.

The quality numbers for the nondurable aggregate sources (group I) were higher than for durable aggregate sources (group II) with the exception of the gravel sources (F and H) and the source E carbonate, as shown in figure 4.11. While the gravels showed low quality numbers, their carbonate fractions showed high quality numbers similar to the rest of the nondurable sources. The coarse aggregates obtained from source E had a top size of 19 mm, which may explain the low quality number obtained and brings into question the sensitivity of the Iowa pore index test results to the top size of the coarse aggregates. Strong correlation (i.e., $|r| > 0.8$) was also observed between the quality number and freeze-thaw test results when ASTM C 666 procedure B was used with salt-treated aggregates (as shown in figures 4.12 and 4.13).

In summary, the Iowa pore index secondary load was highly correlated with field performance for the carbonate aggregates and the carbonate fraction of the gravel samples included in this study. It was noted that the nondurable gravel samples considered had secondary load values that suggested they were durable; however, results of their carbonate fractions failed the test, suggesting that the durability problems associated with these sources can be attributed primarily to the nondurable carbonate fraction. Strong correlations were also observed between the secondary load and the results of rapid freezing and thawing tests (procedure B using salt-treated aggregates and procedure C).

Washington Hydraulic Fracture Test

The results of the hydraulic fracture tests performed on the aggregates included in this study are presented in tables 4.5 and 4.6. Gravel source F included in this study showed a low HFI and more than 5 percent fracturing after 50 cycles, indicating its susceptibility to frost damage. Carbonate sources G and L2 also showed low HFI values and more than 5 percent fractured

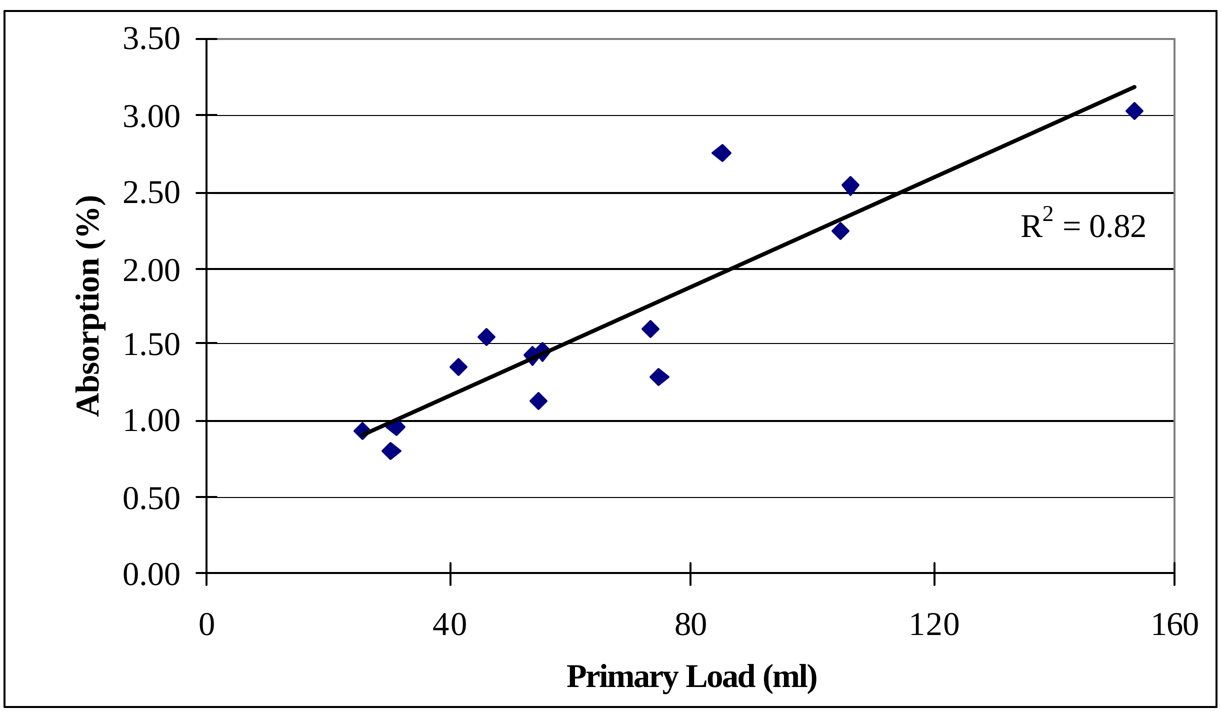

Figure 4.10. Absorption capacity vs. Iowa pore index primary load.

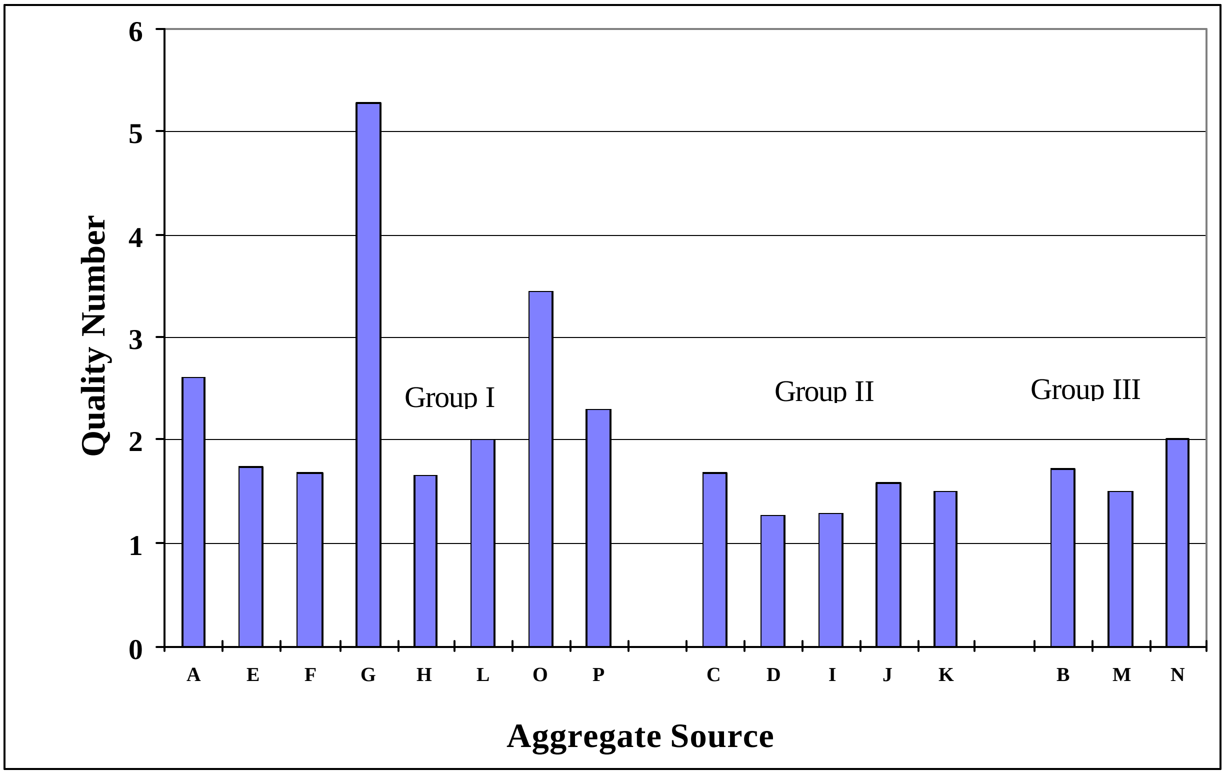

Figure 4.11. Quality number results for Iowa pore index testing.

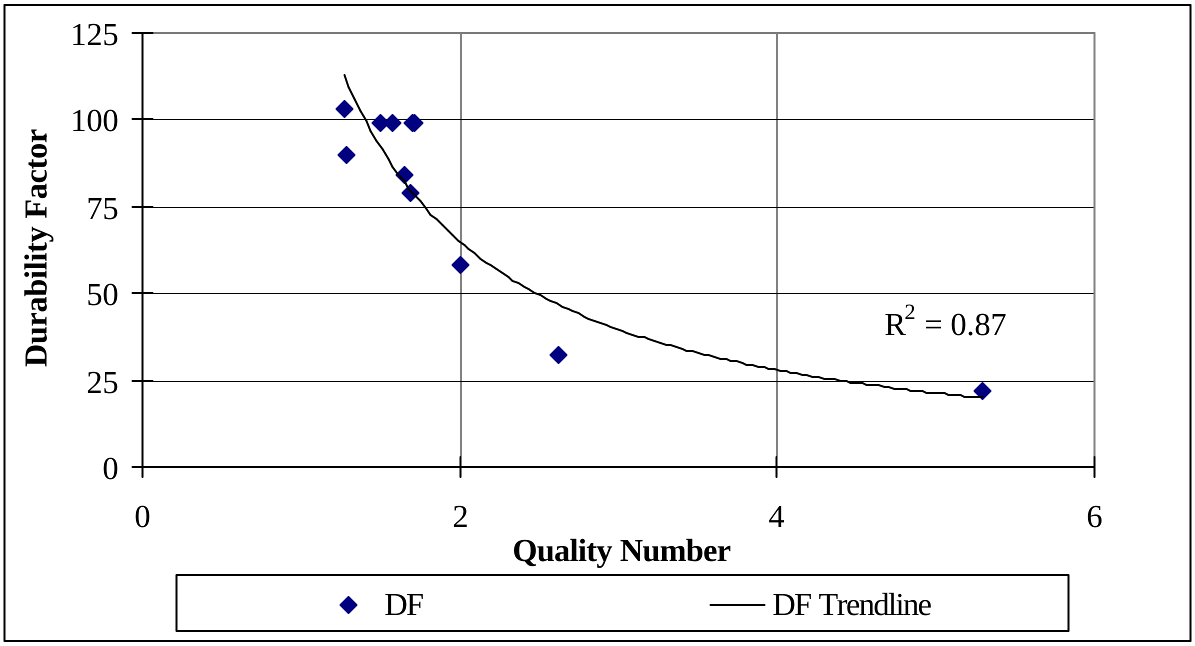

Figure 4.12. Durability factor (ASTM C666 Procedure B with salt-treated aggregate) vs. Iowa pore index quality number.

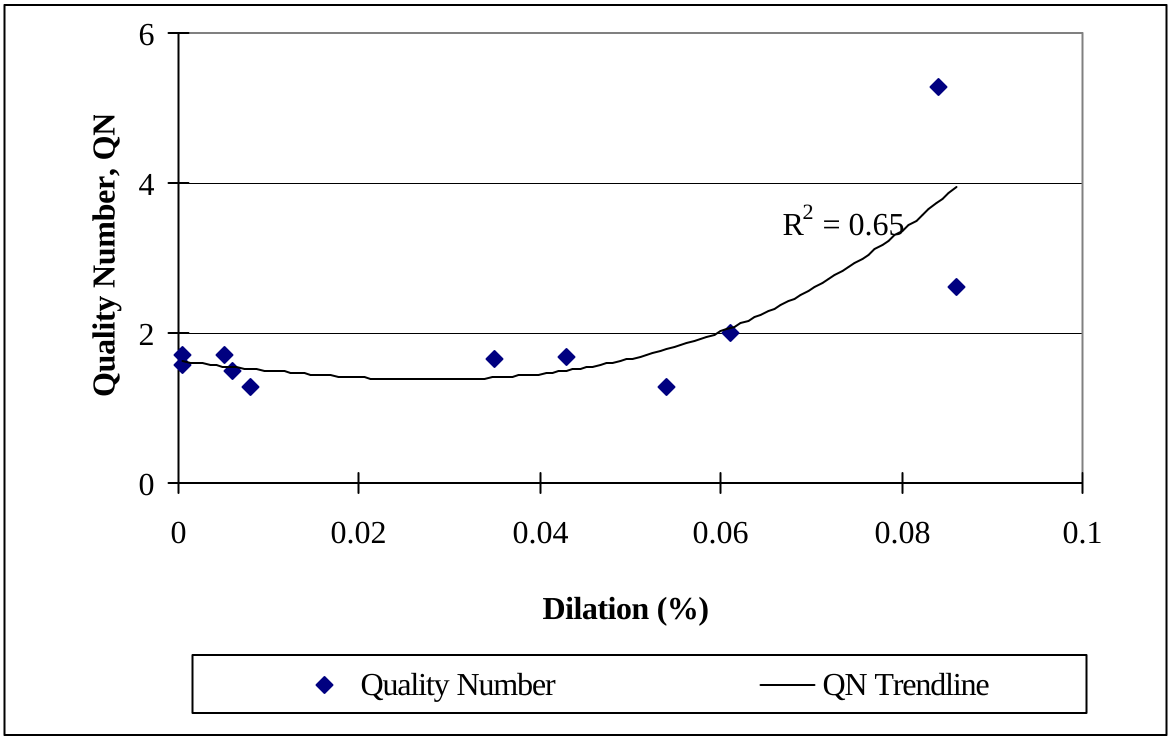

Figure 4.13. Iowa pore index quality number vs. dilation (ASTM C 666 Procedure B with salt-treated aggregate).

particles after 50 cycles. These indications of nondurability correlated very well with their field performance and the results of rapid freeze-thaw tests using procedure B and salt-treated aggregates, which showed either a low durability factor (e.g., DF < 60) or high dilation (e.g., d_L > 0.050 for carbonates, or > 0.035 for gravels).

Source E presented a very high HFI (> 500), but is usually considered to be of marginal durability. This may be explained by the reduced particle sizes in the test sample. Aggregate samples from sources K, J, M and N exhibited high HFIs, which indicates their resistance to frost damage, and is consistent with their field performance and the results of rapid freeze-thaw testing. Source I had a HFI of 146, which indicates a durable source and is validated by freeze-thaw results when procedure B and salt-treated aggregate were used (i.e., DF = 90 and dilation = 0.042 percent). This relatively high dilation may have been due to the presence of fine-grained dolomites in the quarry sample that was tested.

The gravel sample from source F showed lower HFI and higher percent fracture when only the carbonate fraction was used. This observation might suggest that the carbonate fraction is primarily responsible for the freeze-thaw susceptibility of this material. However, the gravel sample from source H showed the same trends and approximately the same results when either the full sample or only the carbonate fraction was tested. It was noted that particles other than carbonate particles fractured during the testing of the full source H sample.

Aggregate sources G and L2 showed significant fracturing (and had the lowest HFI values) and at the same time had the highest amount of material passing the number 200 sieve (also called silt and clay residue). The residue passing the number 200 sieve from sample L2 was mostly silt and fine quartz. The presence of silt and clay in carbonate particles adversely affects their freeze-thaw durability by weakening the bonds between carbonate crystals, attracting water molecules, and expanding disruptively or producing expansive forces when frozen in concrete.

The WHFT test successfully discriminated between very durable and nondurable aggregate for eight sources, but the need for further testing ($50<HFI<100$) was indicated for five other sources. The test failed to identify the highly porous source A and the gravel source H as nondurable. Source A is highly porous and the fracture of aggregate particles when subjected to the WHFT was probably prevented by the ease of draining due to the interconnection between the pores. A significant variance was observed in the results of tests for aggregate sources with inconclusive results, which may suggest that more replicates or a larger sample size (i.e., more aggregate particles) should be used in this test. In general, however, it appears that this test is a reliable method for discriminating between very nondurable and durable aggregate sources in a relatively short time (only one to two weeks).

In general, aggregate sources with low HFI or fracture percentages higher than 5 percent correlated well with low durability factor or high dilation test results from ASTM C 666 procedure B when salt-treated aggregates were used, as shown in figures 4.14 and 4.15. Moreover, Group II (durable aggregates) showed high HFI, which is consistent with their field performance resistance records and ASTM C 666 test results using procedure B and salt-treated aggregates.

X-Ray Diffraction (XRD)

X-ray diffraction was used to identify aggregate minerology by measuring the spacing of the crystallographic planes. When the carbonate aggregates contains at least 1.5 percent dolomite, a dolomite d-spacing factor will be obtained from the x-ray diffraction results. All of the aggregates included in this study are dolomitic and had d-spacing factors between 2.8884 to 2.9016, as shown in tables 4.5 and 4.6.

The dolomite d-spacings for group I were not significantly different from those of group II. Sources A and K showed essentially the same d-spacing, but their field performances and rapid freeze-thaw test results (using procedure B and salt-treated aggregates) were significantly different. Furthermore, samples from sources J and N showed d-spacing factors of 2.9016 and 2.9005, respectively, which implies susceptibility to D-cracking, as suggested by Dubberke and

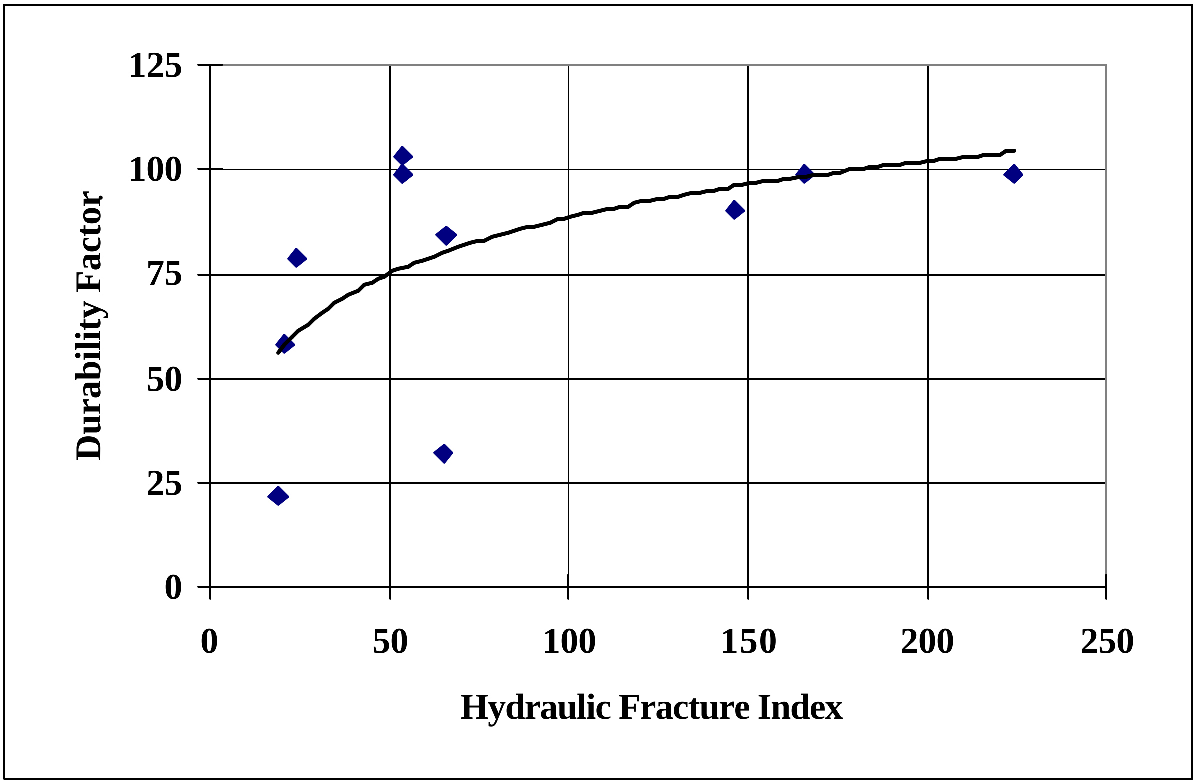

Figure 4.14. Durability factor (ASTM C666 Procedure B with salt-treated aggregate) vs. hydraulic fracture index.

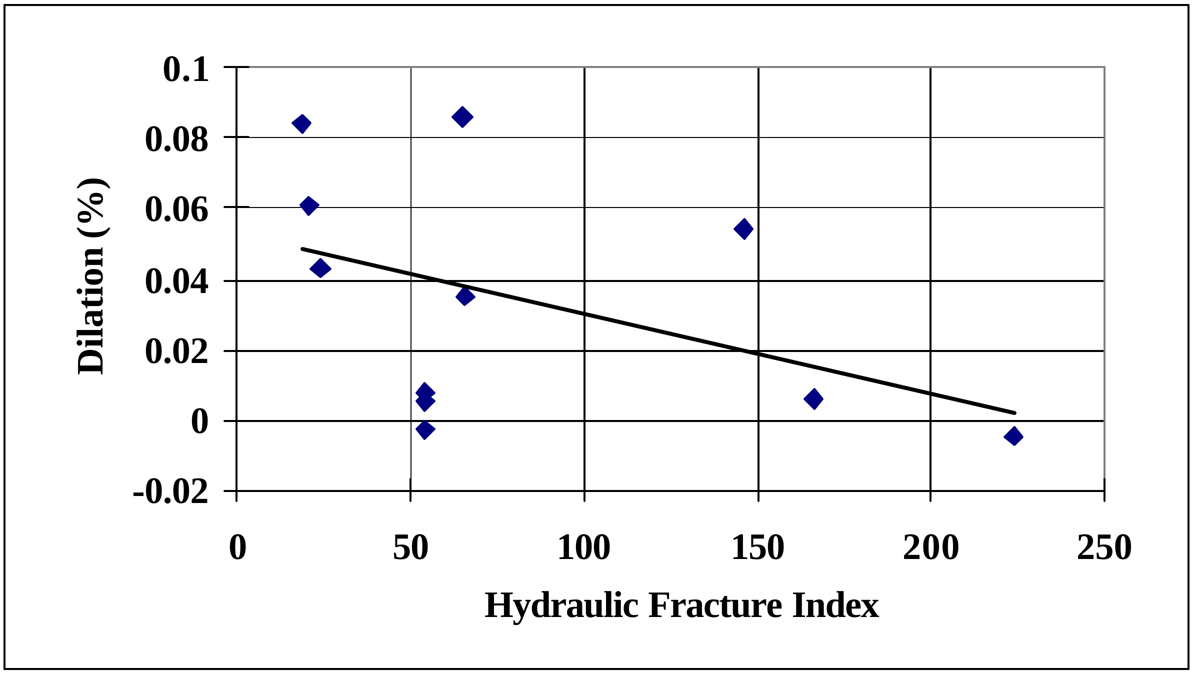

Figure 4.15. Dilation (ASTM C666 Procedure B with salt-treated aggregate) vs. hydraulic fracture index.

Marks (32). However, their historic field performance and results from freeze-thaw durability indicated that they are durable aggregate sources. The dolomite d-spacing was found to be of rather limited usefulness in this study.

X-Ray Fluorescence (XRF)

The elemental components of carbonate aggregates can also be determined by using a sequential x-ray spectrometer and measuring the x-rays and secondary electrons emitted from the sample (32). Test results for the coarse aggregates included in this study are presented in tables 4.5 and 4.6. The carbonate aggregates (or carbonate fractions of gravel) included in this study are dolomitic limestones with dolomite fractions ranging from 25.2 to 42.2 percent and limestone fractions between 49.3 and 70.3 percent. The carbonate fractions indicated percentages of strontium between 0.013 and 0.026, percentages of phosphorous between 0.009 and 0.120, and most measurements were higher than the 0.013 and 0.01 limits for strontium and phosphorous, respectively, given by Dubberke and Marks (32), indicating that the results of the x-ray fluorescence are inconclusive for the aggregate samples tested.

The phosphorus content was generally higher for nondurable coarse aggregates than for durable aggregates with the exception of source M, as shown in figure 4.16. For example, source D had the lowest phosphorous content (0.006 percent) and is known for its good field performance in southern Minnesota.

In general, phosphorous contents of 0.030 percent or higher correlated very well with poor field and laboratory freeze-thaw performance. Phosphorous contents of 0.020 percent or lower correlated very well with good performance in the field and rapid freeze-thaw testing.

Themogravimetric Analysis (TGA)

Thermogravimetric analysis (TGA) consists of measuring the mass loss of a ground aggregate sample as the sample is heated to its transition temperature. Durable limestone generally exhibits little mass loss prior to calcite transition at 905°C; nondurable limestone loses significant mass starting at approximately 600°C. Durable dolomite exhibits a mass loss of carbon dioxide at approximately

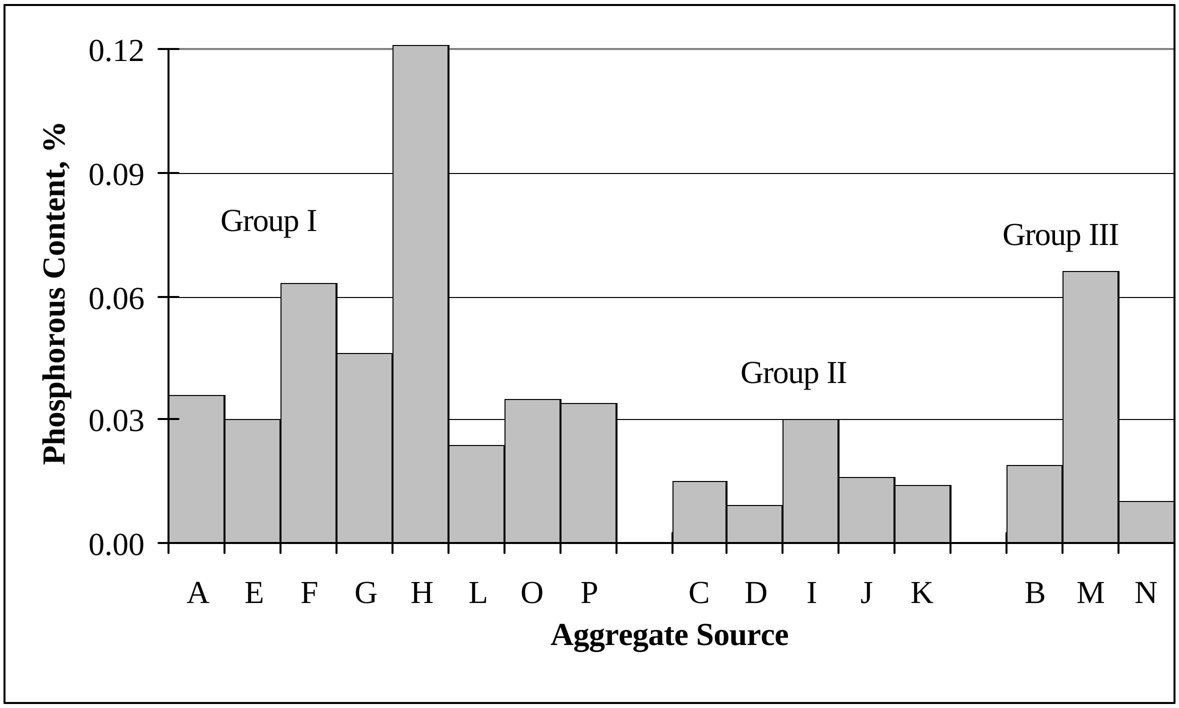

Figure 4.16. Phosphorus content (from X-Ray fluorescence analysis).

570°C and continues to lose weight at a more rapid rate until it reaches its transition temperature (705°C). Nondurable dolomite loses little mass before reaching a temperature of about 700°C, after which a second mass loss, continuing at a greater rate, is observed from about 740°C to 905°C.

As stated previously, all of the carbonate aggregates included in this study are dolomitic. Tests results for these aggregates are presented in tables 4.5 and 4.6.

The slope of the mass loss-temperature curve prior to the transition of magnesium carbonate to magnesium oxide varies from 0.0153 to 0.0735 percent mass loss/°C. Sources K and D showed steep burning slopes prior to dolomite burning (0.0735 and 0.0488 percent mass loss/°C, respectively), which is consistent with their excellent frost resistance and freeze-thaw test results. Sources A, G and I showed low burning slopes (0.0153, 0.0231 and 0.0184 percent mass loss/°C, respectively), which is consistent with their relatively poor freeze-thaw test results (high dilation). Source A exhibited a low burning slope prior to dolomite transition (0.0153 percent mass loss/°C), which is consistent with the low durability factor (DF = 32) obtained when tested using ASTM C 666 procedure B with salt-treated aggregates. However, source L2 showed a high burning slope (0.0459 percent mass loss/°C) prior to dolomite transition (indicating good durability), while the sample obtained failed other durability tests (e.g., ASTM C 666 procedure B with salt-treated aggregates, Iowa pore index test and the Washington hydraulic fracture test) and is not approved as a concrete aggregate in Minnesota. Moreover, source J exhibited a low burning slope (0.0227 percent mass loss/°C) prior to dolomite transition, although it is considered a durable source.

Dubberke and Marks reported that a nondurable limestone will have a high burning slope prior to calcite transition (0.014 percent mass loss/°C or higher), and that a durable limestone will show a low burning slope prior to calcite transition (0.0008 percent mass loss/°C or lower)(65). The aggregate sources included in this study which have calcite fractions of 20 percent of more (A, I, and the carbonate fractions of H and F) showed high burning slopes prior to the calcite transition (e.g., > 0.014 percent mass loss/°C), which is consistent with both the poor field performance and freeze-thaw results when salt-treated aggregates were used with procedure B (e.g., d_L >0.05 percent for carbonates and 0.035 percent for gravels).

In summary, TGA test results showed good correlation between high burning slope prior to calcite transition and poor field performance and high freeze-thaw dilation for carbonate fractions with calcite contents of 20 percent or more. These observations are consistent with the conclusions drawn by the Iowa DOT (65). However, the correlation between burning slope prior to dolomite transition and freeze-thaw test results or field performance was not as consistent.

4.3.3 Summary

Samples of coarse aggregates were obtained from fourteen different quarries or pits which were identified as sources that closely matched those used in the field study pavement sections. Samples were obtained and used to evaluate the relative abilities of different aggregate and concrete durability tests to accurately predict the performance that had been observed in the field.

The rapid freezing and thawing test (ASTM C 666) was used to subject concrete specimens to repeated cycles of freezing and thawing. The best correlation with field performance was obtained when ASTM C 666 procedure B and salt-treated aggregates were used.

The VPI single-cycle slow-freeze test subjects beams to a single cycle of freezing while monitoring the temperature and dilation of the specimens. The VPI test method successfully identified very durable and nondurable aggregate sources (six sources), but the need for further testing was indicated for others (six sources). This test identified only one durable aggregate source and failed to identify only one nondurable aggregate source (a highly porous source).

The absorption capacity of the aggregate samples included in this study ranged from 0.8 to 3.03 percent. Good correlation was observed between the results of absorption capacity tests and rapid freezing and thawing test results. Good performance was indicated for carbonate aggregates with absorption capacities below 1.5 percent and poor performance was indicated for aggregates with absorption capacities higher than 2 percent.

The bulk specific gravity of the aggregate sources considered in this study varied from 2.52 and 2.74. This property correlated well with the rapid freezing and thawing test results and good durability was observed for sources with specific gravity greater than 2.65.

The PCA absorption and adsorption criteria were found to be very restrictive. Test results classified all of the test aggregate sources as nondurable, regardless of field performance. No correlation was observed between the PCA absorption and adsorption values and the either field performance or the rapid freezing and thawing test results.

The acid insoluble residue test is used to determine the percentage of insoluble residue in carbonate aggregates by dissolving the carbonate fraction in hydrochloric acid. The carbonate aggregates included in this study showed acid insoluble residue values between 5.2 and 26.6 percent. Comparable insoluble residue values were often associated with very different performance and only a weak correlation was observed with the results of rapid freezing and thawing tests.

The Iowa pore index test is performed to estimate the volume of aggregate micropores and macropores. The amount of water absorbed by the micropores, as indicated by the secondary load, was highly correlated with the field performance and rapid freezing and thawing test results for the carbonate aggregates. The nondurable gravel sources considered had secondary load values that suggested they were durable; however, the secondary loads of their carbonate fractions indicated their poor performance and suggested that the D-cracking associated with these sources can be attributed primarily to the nondurable carbonate fractions.

The Washington hydraulic fracture test (WHFT) simulates the pore pressures developed during freezing by exposing coarse aggregates, submerged in water, to high pore pressures, which are then released explosively. The hydraulic fracture index, HFI, is computed as the number of pressurization-release cycles required to produce fracture in five percent of the tested particles. HFI exceeding 100 are generally associated with durable sources, while HFI less than 50 are believed to indicate nondurable aggregate sources. Good correlation was observed for durable and nondurable aggregates and the WHFT accurately predicted the performance of seven sources (i.e., three nondurable sources and four durable sources). The need for additional testing was indicated for five aggregate sources.

X-ray diffraction was used to identify aggregate components by measuring the spacing of the crystallographic planes. The dolomite d-spacing factors of durable aggregate sources were not significantly different from those of nondurable sources. Therefore, the results of x-ray diffraction testing were found to be of limited usefulness in this study.

X-ray fluorescence was used to identify the elemental components of the carbonate aggregates by measuring the X-rays and secondary electrons emitted from the samples (32). The carbonate fractions of the aggregate sources included in this study exhibited strontium contents between 0.013 and 0.026 percent and phosphorous contents between 0.009 and 0.066, with most measurements above the

0.013 and 0.01 limits, respectively, that are suggested by Marks and Dubberke as indicators of freeze-thaw durability (32). However, it was observed that phosphorous contents higher than 0.030 percent correlated well with poor field and laboratory freeze-thaw performance, while phosphorous contents of 0.020 percent or less correlated well with good field and laboratory freeze-thaw performance.

The thermogravimetric analysis, TGA, subjects a sample of pulverized aggregate to extremely high temperatures. Durable limestones generally exhibit little mass loss before reaching the calcite transition temperature (approximately 905°C) and nondurable limestones typically show significant weight loss starting at about 600°C. Durable dolomites exhibit a loss of carbonate dioxide at an approximately 570°C and continue to lose mass at a more rapid rate until reaching a transition temperature (i.e., 705°C). Nondurable dolomites lose little weight before reaching a temperature of about 700°C. TGA test results showed that high burning slopes prior to calcite transition correlated well with poor field performance and high freeze-thaw dilation for carbonate fractions with calcite contents of 20 percent or more. However, the correlations between burning slope prior to dolomite transition and freeze-thaw results or field performance were not as consistent.

4.4 Analysis and Development of Methodology for Assessing Freeze-Thaw Resistance

4.4.1 Introduction

A major concern for many engineers has been to positively identify D-cracking susceptible aggregates before they are placed in field concrete. Previous portions of this report document that many tests have been proposed for identifying coarse aggregates that are susceptible to D-cracking. However, researchers are not in complete agreement concerning the usefulness of many of these tests and it appears that many of these tests identify D-cracking susceptibility only for certain types of aggregates and may restrict the use of some durable aggregates. Furthermore, some of the more reliable testing methods require expensive equipment and a long time to conduct. None of the tests and procedures evaluated in this study can be considered to be fast, reliable, reproducible, easy performed and inexpensive approaches for identifying aggregate susceptibility to D-cracking. It is apparent also from the literature review and testing performed under this study that no single test or acceptance/rejection criterion is currently available for predicting the frost resistance of coarse aggregate in PCC pavements (1, 5).

One of the principal objectives of the field study and laboratory test program was to develop a suite of tests that will guide engineers and technicians in accurately assessing the probable field performance of any given aggregate as a function of its petrography and probable environmental exposure. The potential benefit of achieving this goal would be the elimination of D-cracking aggregates from highway and airfield pavement construction, which would reduce pavement maintenance and rehabilitation costs, as well as user costs associated with travel delays and vehicle repairs necessitated by rough roads. Improved ride quality and extended pavement life with fewer lane closures would also result in a safer highway network.

4.4.2 Correlation of Test Results with Field Performance

The results of the field performance study and the petrographic examinations suggest that petrography is an effective tool in determining the potential performance of an aggregate based on its mineralogical composition and properties. For example, it was observed that: 1) fine-grained dolomites and dolomitic limestones exhibited poor performance in the field and in laboratory testing (ASTM C 666 procedure B when salt-treated aggregates were used); and 2) the presence of cracked shale particles contributed to the poor durability performance of at least one pavement section. Consideration should be given to making petrographic examination the first step in any evaluation of frost resistance.

The identification of aggregate composition and origin would provide a basis for better selection of subsequent durability tests. For example, aggregates containing significant quantities of shale would be directed to rapid freezing and thawing tests or some other durability screening test, but not to the absorption, Iowa pore index test or Washington hydraulic fracture test, which fail to indicate the nondurability of shale particles.

The laboratory study showed that 1) tests that provided the best correlation with field performance included a modification of ASTM C 666 procedure B (specimens prepared using salt-treated aggregates), the VPI single-cycle slow-freeze test and the Washington hydraulic fracture test; 2) absorption, specific gravity, Iowa pore index and x-ray fluorescence test results were correlated with field performance to lesser extents; 3) fine-grained dolomites and dolomitic limestones exhibited poor performance in laboratory testing, especially when salt-treated aggregates were used with ASTM C 666 procedure B; and 4) it may be appropriate to restrict the use of highly porous coarse aggregates in PCC pavements because it appears that this type of aggregate may produce a failure at the aggregate-matrix boundary that is very difficult to predict or mitigate.

4.4.3 Statistical Analysis and Multiple Correlations

Statistical analyses were performed to examine correlations between the results of different durability tests (i.e., simulative and correlative tests). Correlation of durability tests with field performance was performed using criteria established in previous research studies and highway agency experience. Analysis consisted of assessing the ability of each test to predict freeze-thaw performance by comparing the correlative test results to field performance and to the results of simulative tests. Data from individual correlative tests were correlated with environmental simulation tests, which are generally used as a standard in ascertaining the potential freeze-thaw durability of aggregates. The objective of these correlations was to develop a protocol for rapidly determining the frost susceptibility of a coarse aggregate. This protocol may include the use of a single test or a group of tests which show good-to-strong correlation with the field performance and the simulative tests.

Correlation analysis is usually performed assess the strength of the relationship between two independent variables by measuring the scatter of data points around a best-fit line or curve. The strength of this relationship is then characterized by Pearson's correlation coefficient, r. The correlation coefficient, r, for a simple linear regression can vary between -1 and +1, with a negative value for an inverse relationship and a positive value for a direct relationship. The stronger the relationship, the more closely r approaches -1 or +1. The correlation coefficient approaches 0 when the variables have little or no interrelationship. Descriptive scaling of correlation is usually performed using the following scale (3) :

- Very strong when absolute value of r (i.e., | r |) is between 1.00 and 0.90;

- Strong when | r | is between 0.899 and 0.80;
- Moderate when | r | is between 0.799 and 0.70;
- Weak when | r | is between 0.699 and 0.60;
- Very weak when | r | is between 0.599 and 0.50; and
- Not significant when | r | is less than 0.499.

The correlation coefficients obtained through simple linear regression between pairs of data sets for the different tests performed for this study are presented in table 4.7. These correlations were analyzed throughout the discussion of the results of frost resistance tests. Absorption capacity, specific gravity and secondary load of the Iowa pore index test were found to be strongly correlated with rapid freezing and thawing test results.

Test results which showed poor correlations with field performance were eliminated from further consideration, as were the results of tests for which slight modifications produced better correlation (i.e., ASTM C666 procedure B with salt-treated aggregate versus either procedures B or C without salt-treated aggregate). On this basis, the following tests were eliminated from further consideration: the total acid insoluble residue, silt and clay residue, x-ray diffraction results, x-ray fluorescence results, thermogravimetric analysis results, primary load from the Iowa pore index, PCA absorption and adsorption, durability factor and dilation from procedures B and C of the rapid freezing and thawing test ASTM C666 and temperature slope from the VPI single-cycle slow-freeze test.

The results of the VPI single-cycle slow-freeze tests showed poor linear correlation with other tests, although the only nondurable source that the test failed to identify was the highly porous source G. Analysis of residuals and diagnostic statistics were performed for correlations between the VPI single-cycle slow-freeze test results and the durability factors obtained from ASTM C 666 using salt-treated aggregates. The aggregate source G was confirmed as an unusual observation using the most common measure of influence, known as Cook's Distance. The Cook's distance is a function of the sum squared changes in the expected response value when the outlier observation is removed from the data (89). When the durability factor is used as a predictor for the simple linear-regression model, the

Cook's distances for source G is 3.24 for the time slope. When the dilation is used as a predictor, the Cook's distances for source G is 1.35 for the time slope. These Cook's distances are significantly high, indicating an unusual observation values or case. The Cook's distances for source A (1.45 and 1.27 when the durability factor and dilation were used as predictors; respectively) were also higher than 1, but to a lesser extent; therefore the data for this source were not considered to be outliers.

When data from source G were eliminated from the analyses of test result pairs, the Pearson correlation coefficients improved significantly for the aggregate sources included in the regression models, as shown in table 4.8. (In addition to source G, data for sources E and K were discarded because their top sizes were different and source M was eliminated because it had a different coarse aggregate gradation).

The results of the Washington hydraulic fracture tests showed poor linear correlation with other test results, which is expected because the HFI scale is highly nonlinear. However, this test correlated very well with field performance and rapid freeze-thaw test results using procedure B and salt-treated aggregates by accurately predicting the freeze-thaw performance of 8 out of 13 aggregate sources while recommending additional testing for the other 5 sources.

Correlations between the results of simulative tests and correlative tests were performed using linear regression analysis and linear predictors. Results from the rapid freezing and thawing (i.e., ASTM C 666 procedure B using salt-treated aggregates) and VPI single-cycle slow-freeze test were used as the model responses. The model predictors included the absorption, bulk specific gravity, secondary load and the quality number (from the Iowa pore index test). The results of the Washington hydraulic fracture test (i.e., the hydraulic fracture index, HFI, and the percent of particle fractures) were eliminated from the regression models because they were poorly correlated with the results of other simulative tests.

Table 4.8. Correlation matrix (r coefficient) for the VPI single-cycle slow freeze test.

	Aggregate Source G Included		Aggregate Source G Not Included	
	VPI Temperature Slope	**VPI Time Slope**	**VPI Temperature Slope**	**VPI Time Slope**
BSG	0.16	0.48	0.78	0.87
ABS	0.35	0.29	-0.69	-0.81
IPIT PL	0.01	-0.47	-0.67	-0.68
IPIT SL	0.75	-0.18	-0.77	-0.75
IPIT QN	0.86	0.33	-0.83	-0.77
PCA ABS	-0.07	-0.58	-0.76	-0.78
PCA ADS	-0.35	-0.60	-0.35	-0.55
ACIR TR	-0.14	-0.16	-0.21	-0.13
ACIR SCR	0.44	0.32	0.06	0.09
HFI	-0.20	0.07	0.36	0.30
PF	0.15	0.01	-0.17	-0.13
DF-B	0.18	0.54	0.67	0.64
Dil-B	0.48	-0.09	-0.63	-0.68
DF-C	-0.43	0.10	0.58	0.61
Dil-C	0.74	0.18	-0.54	-0.64
DF-S	-0.49	0.18	0.82	-0.89
Dil-S	0.35	-0.31	-0.60	-0.81

BSG : Bulk Specific Gravity at 23C
ABS : Absorption (%)
IPIT PL : IPIT Primary Load (ml)
IPIT SL : IPIT Secondary Load (ml)
IPIT QN : IPIT Quality Number
PCA ABS : PCA Absorption (%)
PCA ADS : PCA Adsorption (%)
ACIR TR : Total Acid Insoluble Residue (%)
ACIR SCR : Silt and Clay Residue (%)
HFI : Hydraulic Fracture Index
PF : Percent Fracture
DF-B : Durability Factor (ASTM C 666 Procedure B)
Dil-B : Dilation, % (ASTM C 666 Procedure B)
DF-C : Durability Factor (ASTM C 666 Procedure C)
Dil-C : Dilation, % (ASTM C 666 Procedure C)
DF-S : Durability Factor (ASTM C 666 Procedure B and Salt-Treated Aggregate)
Dil-S : Dilation, % (ASTM C 666 Procedure B and Salt Treated Aggregate)

The correlation coefficient, r, is usually used to describe the strength of the relationship between two variables; however, this coefficient should not used to describe correlations for regression models that have more than one predictor (89). The square of the correlation coefficient, r^2, is usually used as a goodness-of-fit statistic and to compare the validity of alternative models. However, the following problems can be encountered when using this coefficient:

- r^2 pertains to explained and unexplained variables and does not account for the number of degrees of freedom (89); and
- r^2 is very sensitive to the number of variables, and adding variables does not necessarily reduce it.

The adjusted correlation coefficient, r^2_{adj}, is usually used and is more desirable because it takes into account the number of degrees of freedom and it can decrease or increase as variables are added. The adjusted correlation coefficient, r^2_{adj}, is given by:

$$r^2_{adj} = 1 - \frac{(1 - r^2)\ (N - 1)}{(N - p - 1)} \qquad \text{(Eqn. 4.1)}$$

where:

r : Pearson's correlation coefficient,

N : number of observations, and

p : number of predictors.

The regression models in this study were developed using backward elimination techniques. Backward elimination involves starting with a model that contains p potential predictors and then deleting them one by one, each time eliminating the predictor that contributes the least to the model based on the t-test statistic of the regression coefficient. This process continues until all of the remaining predictors appear to be significant. For this study, the backward elimination process was performed until the absolute values of t-test values for all regression coefficients were higher than 2.0. It is customary to use a value of 2.0 for the t-test value since many critical values for a two-tailed test with level of significance 0.05 are close to this value (90).

The t-value for each estimate has N - p - 1 degrees of freedom and is given by:

$$t = b_i / \sigma_i \qquad \text{(Eqn. 4.2)}$$

where:

b_i : regression coefficient for predictor I, and

σ_i : standard deviation of b_i .

The larger model in this process (which initially contains all of the variables under consideration) can be called the "alternative hypothesis" or AH, while the smaller model can be called the "null hypothesis" or NH.

The two hypotheses (NH) and (AH) can be generalized as follows (91):

$$\text{NH:} \quad Y = \beta_{0\text{ - NH}} + \beta_{1\text{ - NH}} X_1 + \beta_{2\text{ - NH}} X_2 + \beta_{3\text{ - NH}} X_3 + e \qquad \text{(Eqn. 4.3)}$$

$$\text{AH:} \quad Y = \beta_{0\text{ - AH}} + \beta_{1\text{ - AH}} X_1 + \beta_{2\text{ - AH}} X_2 + \beta_{3\text{ - AH}} X_3 + \beta_{4\text{ - AH}} X_4 + e \qquad \text{(Eqn. 4.4)}$$

where:

Y : model response,

X_i : model predictor I,

β_i : estimation coefficient or regression coefficient for predictor X_i,

β_0 : intercept, and

e : regression error.

A comparison of the suitability of the two models begins by computing the residual sum of squares (RSS) and the number of degrees of freedom (d.f.) for each model. The F-test is usually used to test the significance of predictors to the regression models. The F-test statistic is computed using the following equation (91):

$$F = \frac{(RSS_{NH} - RSS_{AH}) / (d.f._{NH} - d.f._{AH})}{(RSS_{AH} / d.f._{AH})} \qquad \text{(Eqn. 4.5)}$$

where:

RSS_{NH} : residual sum of squares for the null hypothesis model,

RSS_{AH} : residual sum of squares for the alternate hypothesis model,

$d.f._{NH}$: degrees of freedom for the null hypothesis model, and

$d.f._{AH}$: degrees of freedom for the alternate hypothesis model.

The F-test statistic is then compared with the standard tabulated value for $F(d.f._{NH}-d.f._{AH,}\ d.f._{AH})$ at some appropriate significance value, α (typically, $\alpha = 0.05$). F_{α} has a probability distribution based on a numerator degree of freedom equal to $(d.f._{NH}-d.f._{AH})$ and a denominator degree of freedom equal to $d.f._{AH}$ (90). When the F value from equation 4.5 is higher than the standard tabulated F distribution value, the null hypothesis is rejected. These analyses are performed assuming that the model response and the residual errors are normally distributed.

Summaries of the statistical analyses performed for the models evaluated in this study are presented in tables 4.9 and 4.10. The original regression models and the final models used to predict results of the simulative tests from the results of correlative tests are presented in table 4.11. The r_{adj} value and F-test results are also included in table 4.11. The final models developed for the three different selected responses are:

<u>Model 1</u>: $DF = -500.76 + 231.26 \times BSG - 1.25 \times IPISL$ (Eqn. 4.6)

<u>Model 2</u>: $DL = 1.284 - 0.473 \times BSG$ (Eqn. 4.7)

<u>Model 3</u>: $B_t = -116.57 + 41.74 \times BSG + 1.99 \times QN$ (Eqn. 4.8)

where:

DF	: durability factor (ASTM C 666 procedure B and salt-treated aggregates),
BSG	: bulk specific gravity,
IPISL	: secondary load from the Iowa pore index test, and
QN	: quality number from the Iowa pore index test.

Figures 4.17, 4.18 and 4.19 present the predicted values vs. the actual values for each model and demonstrate how close the data lie relative to the line of equality for each model.

Table 4.9. Regression models for ASTM C 666 Procedure B using salt-treated aggregate.

Regression Models for Durability Factor

Variable	Estimate and (t-value) at Step Number 0	1	2
Intercept	-545.23 (-1.523)	-573.73 (-2.471)	-500.76 (-3.367)
BSG	-245.52 (1.910)	255.96 (3.133)	231.26 (4.241)
IPISL	-0.75 (-0.548)	-0.83 (-0.816)	-1.25 (-4.720)
QN^2	-0.65 (-0.405)	-0.58 (-0.429)	
ABS	-1.70 (-0.113)		
d.f.	5	6	7
r	0.975	0.975	0.974
r_{adj}	0.954	0.962	0.966
RSS	373.16	374.12	385.62

Variable	Estimate and (t-value) at Step Number 0	1	2	3
Intercept	1.118 (1.670)	1.402 (3.188)	1.053 (3.144)	1.284 (5.551)
BSG	-0.437 (-1.767)	-0.543 (-3.440)	-0.407 (-3.662)	-0.473 (-5.389)
IPISL	0.349 (1.438)	0.283 (1.392)	0.050 (0.957)	
QN^2	-0.129 (-1.244)	-0.092 (-1.183)		
ABS	0.018 (0.590)			
d.f.	5	6	7	8
r	0.925	0.919	0.899	0.885
r_{adj}	0.860	0.876	0.868	0.870
RSS	0.00147	0.00157	0.00193	0.00218

Table 4.10. Regression models for VPI single-cycle slow freeze test results.

Regression Models for Time Slope (Case G Included)

	Estimate and (t-value) at Step Number		
Variable	0	1	2
Intercept	-106.855 (-1.397)	-107.41 (-1.534)	-116.569 (-4.240)
BSG	38.236 (1.384)	38.45 (1.520)	41.74 (4.099)
QN	1.33 (0.339)	2.05 (2.873)	1.99 (3.705)
ABS	-0.82 (-0.223)	-0.36 (-0.144)	
IPISL	0.09 (-0.187)		
d.f.	5	6	7
r	0.861	0.860	0.860
r_{adj}	0.731	0.780	0.815
RSS	17.709	17.833	17.895

Linear Regression Models for Time Slope (Case G Not Included)

	Estimate and (t-value) at Step Number			
Variable	0	1	2	3
Intercept	-55.5 (-1.090)	-96.218 (-2.689)	-67.343 (-2.071)	-90.806 (-4.702)
BSG	21.8 (-1.200)	36.267 (2.829)	25.4338 (2.231)	33.269 (-4.555)
QN	-8.845 (-2.100)	-6.399 (-1.745)	-1.591 (0.904)	
IPISL	0.63125 (-1.844)	0.375 (-1.459)		
ABS	-2.605 (-1.100)			
d.f.	4	5	6	7
r	0.969	0.939	0.921	0.930
r_{adj}	0.872	0.839	0.844	0.844
RSS	5.54200	7.229	10.311	11.714

Table 4.11. Summary of regression model testing results.

Hypothesis	Model	F	F_a	r_{adj}	Decision
Regression Models with Source G included					
Durability Factor (Procedure B w/Salt-Treated Aggregate)					
NH	$DF = \beta_0 + \beta_1\ BSG + \beta_2\ IPISL + e$	0.11	5.79	0.966	Accept NH
AH	$DF = \beta_0 + \beta_1\ BSG + \beta_2\ QN^2 + \beta_3\ ABS + \beta_4\ IPISL + e$				
Dilation (Procedure B w/Salt-Treated Aggregate)					
NH	$DL = \beta_0 + \beta_1\ BSG + e$	0.87	5.41	0.87	Accept NH
AH	$DL = \beta_0 + \beta_1\ BSG + \beta_2\ QN^{0.25} + \beta_3\ ABS + \beta_4\ IPISL^{0.33} + e$				
VPI Single-Cycle Slow Freeze Time Factor					
NH	$VPI\text{-}TIME = \beta_0 + \beta_1\ BSG + e$	0.030	5.79	0.815	Accept NH
AH	$VPI\text{-}TIME = \beta_0 + \beta_1\ BSG + \beta_2\ QN + \beta_3\ ABS + \beta_4\ IPISL + e$				

Regression Models with Source G not included					
VPI Single-Cycle Slow Freeze Time Factor					
NH	$VPI\text{-}TIME = \beta_0 + \beta_1\ BSG + e$	1.48	6.59	0.844	Accept NH
AH	$VPI\text{-}TIME = \beta_0 + \beta_1\ BSG + \beta_2\ QN + \beta_3\ ABS + \beta_4\ IPISL + e$				

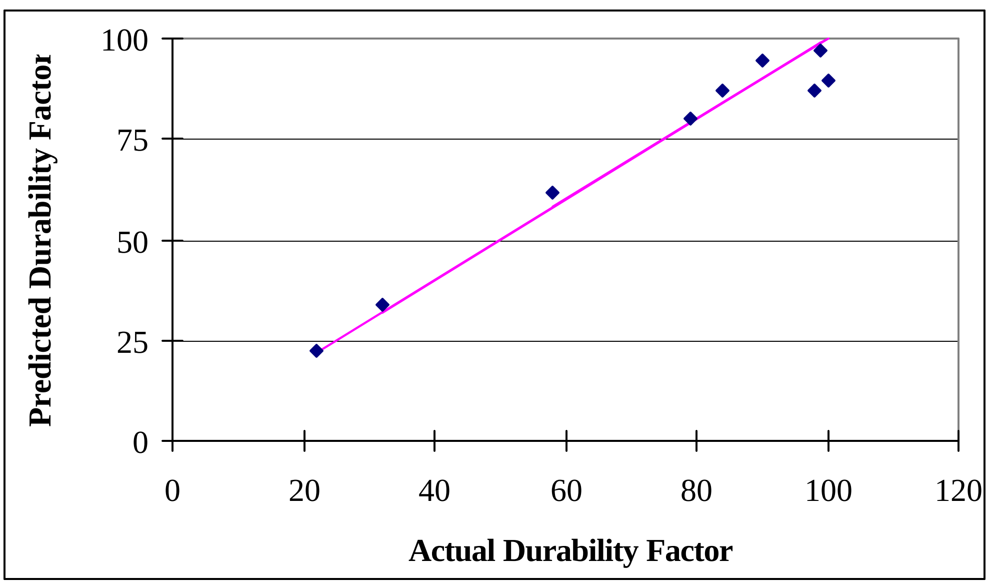

Figure 4.17. Plot of predicted vs. actual durability factors.

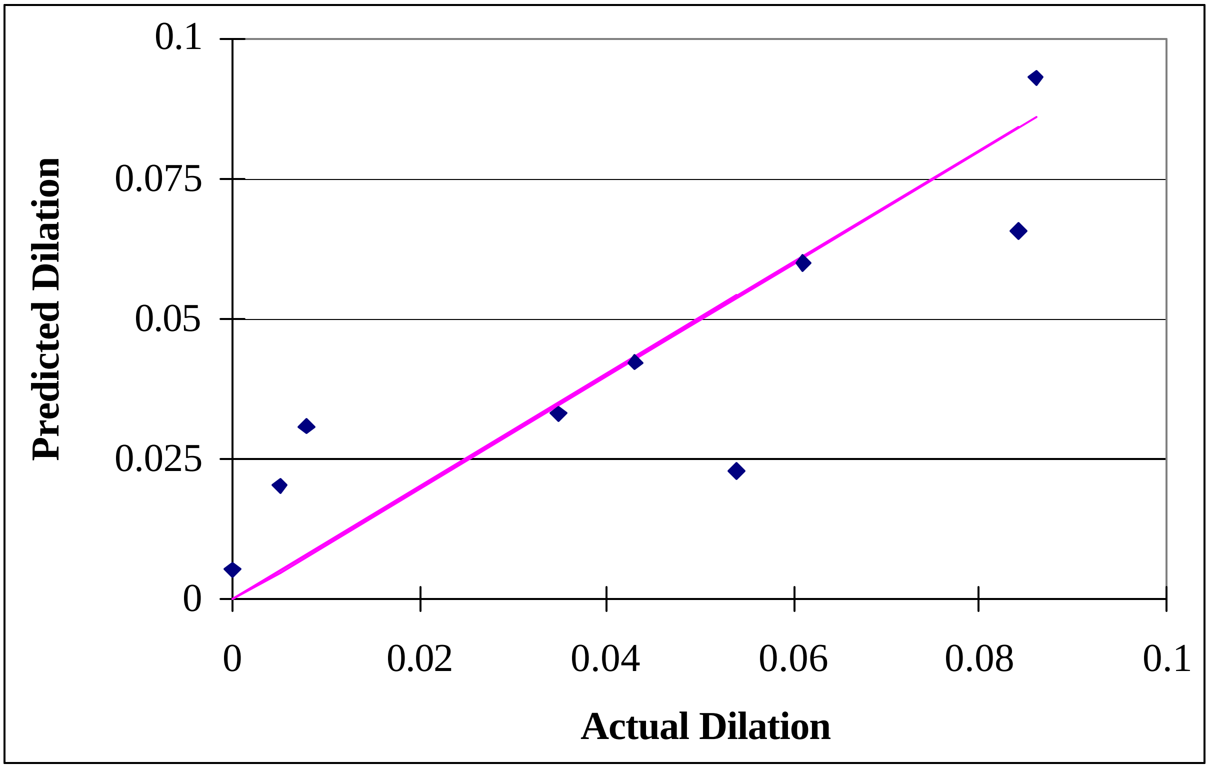

Figure 4.18. Plot of predicted vs. actual dilations.

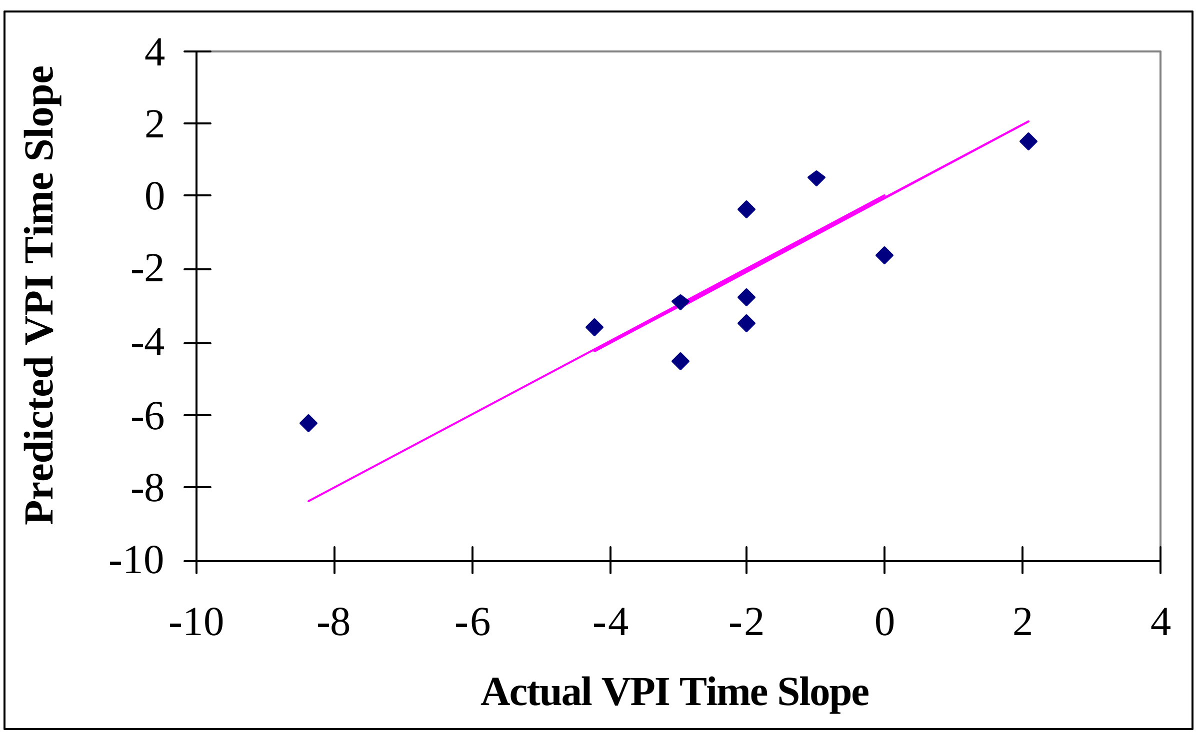

Figure 4.19. Plot of predicted vs. actual VPI time slope values.

The sample number (i.e., number of cases) in these regression models is considered small. One of the objectives of this study was to validate the effectiveness of frost resistance tests and only 13 aggregate sources were used and only small quantities of aggregate were obtained from some of them. The results of these regression models should be considered with precaution; additional aggregate sources are needed to validate these models or different models.

For the linear regression models with the VPI time slope as the dependent variable, two cases are presented: one with source G included and with source G excluded from the analysis. These two cases were used because source G was found to have a significant influence on the results of the analysis. However, the sample size included in these models is relatively small (i.e., a small number of degrees of freedom are present with which to analyze residual errors, given the number of predictors included). Additional sources are required for future research, as proposed earlier, to investigate these models and others.

4.4.4 Development of Methodology for Assessing Freeze-Thaw Durability

The field investigation showed that dolomite grain size significantly affected concrete freeze-thaw performance, with fine-grained dolomite coarse aggregate being associated with poor performance. The laboratory study also showed that concrete containing very fine-grained dolomites exhibited low durability factors and high dilations when compared to concrete containing coarse-grained dolomites.

The field investigation showed that PCC pavement sections which contained cracked shale particles performed poorly, and laboratory rapid freezing and thawing tests of these mixtures showed high dilation. However, it should be noted that the following additional factors may also have contributed to the high dilations: 1) alkali-silica reaction signs observed in these cores; and 2) the presence of a deficient air void system in the concrete mortar, as indicated by the linear traverse examination (i.e., failing to comply with ACI 211.1-1989 recommendations).

The VPI single-cycle slow-freeze test accurately predicted the poor performance of nondurable aggregates and can be used as a screening test for very fine-grained dolomites when their performance is not predicted by the Washington hydraulic fracture test. For example, the VPI single-cycle slow-freeze test accurately predicted the poor performance of source E (which the WHFT did not), in spite of its reduced top size (19 mm).

The rapid freezing and thawing test results (ASTM C 666 procedure C) of the specimens retrieved from the field study pavement sections correlated very well with field performance and were consistent in predicting the poor performance of D-cracked pavements, while showing no freeze-thaw resistance problems in pavements that exhibited distresses other than D-cracking. The rapid freezing and thawing test (ASTM C 666 procedure B using salt-treated aggregates) predicted very well the performance of the aggregate sources included in the laboratory study and showed good correlations with field performance.

For accurately assessing the probable field performance of any given aggregate, the original geological characteristics and an appropriate suite of freeze-thaw durability tests should be used. These tests should probably include petrographic examination, the rapid freezing and thawing test (procedure B using salt-treated aggregates), the VPI single-cycle slow-freeze test and the Washington hydraulic fracture test.

One possible procedure for determining whether to accept or reject an aggregate source on the basis of freeze-thaw durability is shown in figure 4.20. The steps proposed for assessing coarse aggregate freeze-thaw durability are listed below:

1. Determine the field performance of the aggregate source through service records and field condition surveys. This investigation should provide detailed information concerning the service records of the aggregate source and the performance of each ledge or area of the quarry or pit if the field performance varies significantly.
2. Perform a petrographic examination of aggregate samples obtained from each stockpile, site or ledge. The material characterization should be based on the rock type (mineralogy), composition, grain size and presence of shale particles for carbonate aggregate sources. When the aggregate source is gravel, the petrographic examination should include the rock type and

grain size of the carbonate fraction, and should document the presence of shale particles in the gravel samples.

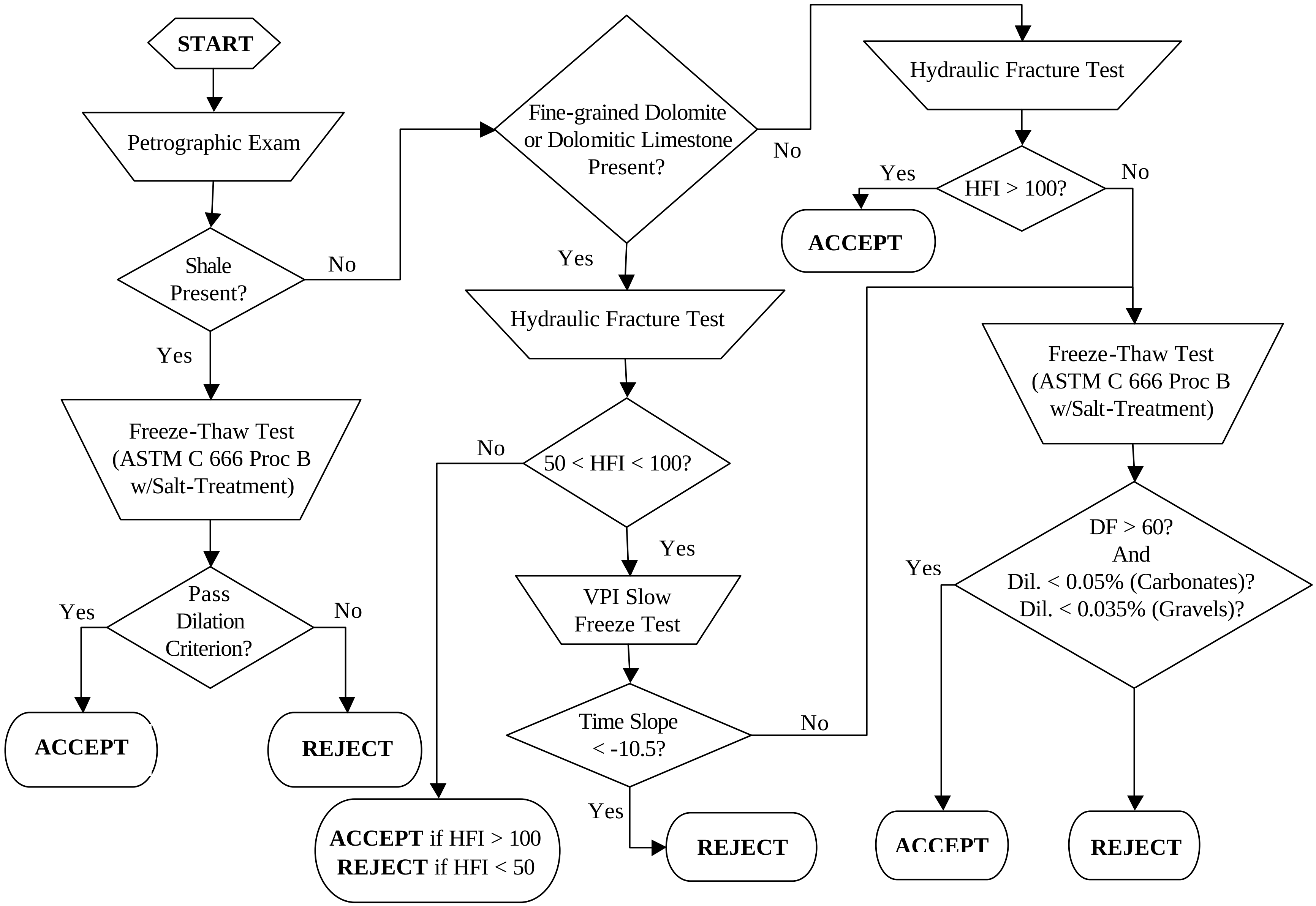

Figure 4.20. Example procedure for concrete aggregate acceptance based on freeze-thaw durability.

3. When shale particles are present, the rapid freezing and thawing test (ASTM C 666) should be performed using salt-treated aggregates to determine the potential freeze-thaw durability of the aggregate source as well as the effect of the shale particle presence on dilation.
4. When no shale particles are present, the aggregate source should be categorized based on the presence of very fine-grained dolomites or dolomitic limestones.
5. When very fine-grained dolomites or dolomitic limestones are present in the aggregate sample, poor performance might be expected. The Washington hydraulic fracture test should be performed first to eliminate the very durable (i.e., HFI > 100) and nondurable (i.e., HFI < 50) aggregate sources. If this test fails to predict the performance of the aggregate sample (i.e., 50 < HFI < 100), the VPI single-cycle slow-freeze test should be performed to eliminate aggregate sources that are nondurable (i.e., those with time slopes less than -10.2). The rapid freezing and thawing test (procedure B using salt-treated aggregates) should be performed for the sources whose durability can not be accurately predicted by the Washington hydraulic fracture test or the VPI single-cycle slow-freeze test. In this case, the durability factor and/or dilation should be used to reject or accept the aggregate source.
6. When the coarse aggregate sample does not contain very fine-grained dolomites, the Washington hydraulic fracture test should be performed first. When the HFI is higher than 100, the aggregate sample should be classified as durable. If the HFI is less than 100, the rapid freezing and thawing test (procedure B using salt-treated aggregates) should be performed to determine the potential freeze-thaw durability of the aggregate source.

These steps are presented graphically in the form of a test flow chart or decision tree in figure 4.20. The results of the field investigation and laboratory study validate this procedure for accepting or rejecting the aggregate source. Figure 4.21 shows how this protocol would correctly determine the durability of the aggregate sources included in this study.

The results of this study could have significant impacts in both economics and pavement engineering. The economic importance relates to the cost of D-cracking to state highway departments. A significant amount of money has been diverted from planned maintenance and repair activities to the repair of prematurely deteriorated D-cracked pavements. When pavements

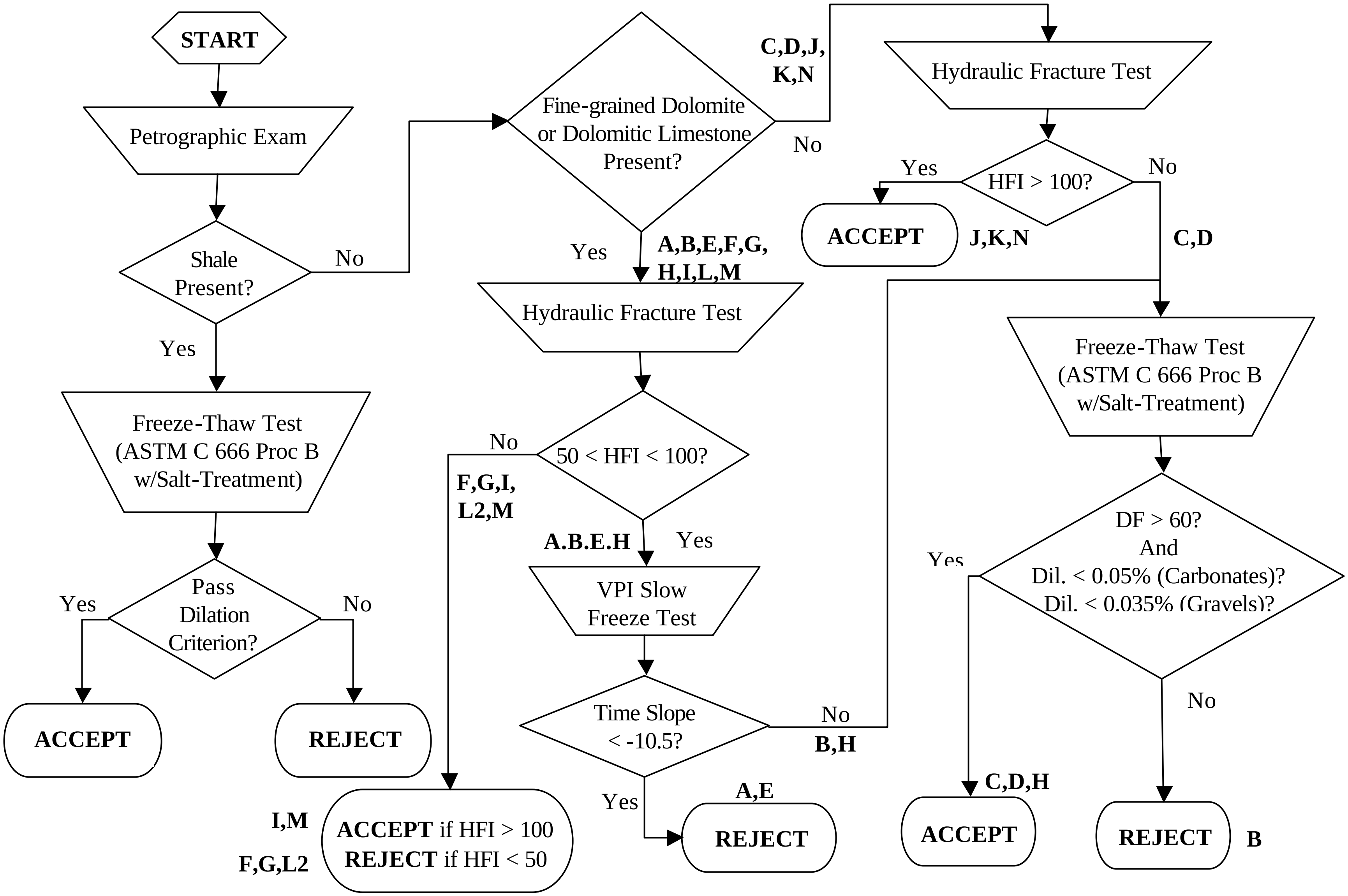

Figure 4.21. Concrete aggregate durability screening procedure with study aggregate test results.

exhibit premature failures, the tax-paying public does not receive the best value for its money and the economic costs of concrete pavements cannot be properly calculated and compared with the costs of other types of pavements.

The development of this test protocol represents a step forward in the rapid and accurate prediction and control of D-cracking because it is apparent that no single test or acceptance/rejection criterion is currently available for predicting the frost resistance of coarse aggregate in PCC pavements (1, 5). With further validation testing, this suite of tests and the adoption of appropriate rejection/acceptance criteria will aid Mn/DOT and other highway agencies in eliminating the premature failure of concrete pavements by D-cracking.

4.4.5 Summary

From the results of this study, it is apparent that no single test has the attributes of speed, simplicity and accuracy for predicting the durability of coarse aggregate in PCC pavements that are subjected to freezing and thawing exposure. A suite of tests was developed to guide engineers and technicians in accurately assessing the probable field performance of any given aggregate as a function of its minerology and probable environmental exposure.

Petrographic examination should be the first step in evaluating frost resistance because identification of aggregate composition and origin seems to provide a basis for better selection of subsequent durability tests. The proposed battery of tests includes the Washington hydraulic fracture test, VPI single-cycle slow-freeze test and the rapid freezing and thawing test using ASTM C 666 procedure B and salt-treated aggregates.

CHAPTER 5

D-CRACKING MITIGATION TECHNIQUES

5.1 Introduction

D-cracking in PCC pavements can only occur when a sufficient quantity of unsound aggregates are critically saturated and exposed to a sufficient number of cycles of freezing and thawing. The first requirement (unsound aggregate) suggests that enough D-cracking aggregate need be present to damage the concrete as a whole, rather than cause localized damage (5). The second requirement (critical saturation) is most frequently observed near transverse and longitudinal joints and cracks where moisture easily trapped and not easily removed. The time required for the final condition (exposure to a sufficient number of freezing and thawing cycles) varies with climatic conditions, but generally requires 5 to 10 (or more) years before D-cracking begins to become noticeable and apparent in the field.

Preventing D-cracking requires the elimination of at least one of these conditions. In new construction, mitigating D-cracking may require preventing the use of unsound aggregates or beneficiating them (e.g., removing or treating the nondurable fractions) to improve their performance. Mitigation of D-cracking in existing concrete pavements may necessitate the full-depth repair of the concrete pavement (to remove previously damaged material) and the elimination of either freezing (through the use of thick overlays) or moisture (5).

Although the concrete mortar and mortar air void system do not cause D-cracking, an inadequate air void system may result in freeze-thaw deterioration of the paste (which may resemble D-cracking), allowing additional available moisture to be absorbed by the aggregate and thus accelerate D-cracking progression when unsound aggregates are present in the concrete. Even properly air-entrained concrete may develop D-cracking when a sufficient amount of unsound aggregate particles exist in the concrete (5).

This research sought to develop mitigation methods that improve the performance of the concrete prepared using local aggregate sources that might be considered nondurable without beneficiation. The general approach to accomplishing this goal involved:

- identifying the most promising techniques for mitigating D-cracking;
- selecting D-cracking susceptible aggregate sources and appropriate durability tests for inclusion in the study;
- subjecting these aggregates or concrete mixtures made using them to the most promising beneficiation techniques identified during the literature review; and
- evaluating the effects of beneficiation techniques on D-cracking potential.

Details concerning the research approach are provided in later sections of this chapter.

5.2 Selection of D-Cracking Mitigation Techniques

Several methods for D-cracking mitigation were identified through the literature review and during meetings with Mn/DOT researchers. The following approaches to D-cracking mitigation were selected for study:

- changes in mix proportioning to reduce the concrete permeability, thereby reducing the infiltration of water and deicing salts (i.e., reductions in water-to-cement ratio and the use of silica fume or other pozzolanic admixtures);
- the use of various aggregate treatments intended to reduce water adsorption and absorption (i.e., silane and linseed oil treatment);
- the blending of sound and unsound coarse aggregates;
- changes in coarse aggregate gradation to reduce the aggregate particle top size; and
- the use of both blended aggregate and reduced top size.

5.3 Selection of Test Aggregate Sources

The following six aggregate sources were selected for use in the D-cracking mitigation portion of the study to represent nondurable gravels, durable and nondurable limestones, and salt-susceptible aggregate:

- Halma (source H), a D-cracking susceptible gravel;

- Luverne (source L), another D-cracking gravel;
- Shakopee (source G), a D-cracking dolomitic limestone;
- Grand Meadows (source A), another D-cracking dolomitic limestone;
- Harris (source I), a fine-grained dolomitic limestone that experienced large dilations when subjected to the Iowa salt treatment and freeze-thaw testing; and
- Hammond (source C), a durable limestone to be used for blending with nondurable sources.

5.4 Selection of Freeze-Thaw Durability Tests

The following environmental simulation and correlative tests were identified as most useful in the first part of this laboratory study and were used to evaluate treated aggregate samples and modified concrete mixtures:

- the Washington hydraulic fracture test (used to evaluate the effectiveness of aggregate coating techniques);
- the Iowa pore index test (also used to evaluate the effectiveness of aggregate coating techniques),
- absorption capacity test (ASTM C 127);
- specific gravity test (ASTM C 127);
- ASTM C 666, Standard Test for Resistance of Concrete to Freezing and Thawing (used to evaluate concrete specimens made with treated and untreated aggregates or modified mix designs); and
- the VPI single cycle slow freeze test (used for the same purposes as ASTM C 666).

The rapid freezing and thawing and VPI single-cycle slow-freeze tests were used to investigate the effectiveness of each D-cracking mitigation technique. Two procedures were used for the rapid freezing and thawing test: procedure C was used to evaluate the effectiveness of modified mix designs (i.e., reduced water content and use of silica fume); and procedure B using salt-treated aggregates to evaluate the effectiveness of other treatments (i.e., aggregate pretreatment with silane and linseed oil, reduction of the maximum top size of coarse aggregate, blending of sound and unsound coarse aggregates, and blending of sound and unsound aggregates with a reduction of the top size of nondurable aggregates). When used for modified mix designs, procedure C helped address the theory that less permeable concrete reduces water and salt intrusions to the aggregates. The VPI single-cycle slow-freeze tests were performed on the same mix of specimens used for the freezing and thawing tests, including salt-treated aggregate specimens.

The Washington hydraulic fracture test (WHFT) was used to evaluate the relative D-cracking potential of beneficiated aggregate samples and salt-treated aggregates. Linseed oil-treated aggregates were not subjected to the WHFT because other tests indicated that this treatment was not effective before the hydraulic fracture testing began. It is also worth noting that silane treatment of aggregate sources is a part of the normal WHFT test procedure, so there were no comparisons of silane-treated versus nontreated aggregates.

The Iowa pore index test was used to explore the potential D-cracking performance of beneficiated aggregate samples (including silane-treated aggregates) and salt-treated aggregates. Linseed oil-treated aggregates were not subjected to the Iowa pore index test because other tests indicated that this treatment was not effective before the pore index testing began.

The absorption and specific gravity tests (ASTM C 127) were performed on both treated and untreated aggregate samples to investigate the effects of various treatments on the selected aggregates and to provide mix design data for use in preparing concrete beams.

Table 5.1 summarizes the tests that were included in this portion of the laboratory study.

5.5 Preparation of Test Specimens

All coarse aggregate samples were sieved into their component size fractions and then reblended in appropriate proportions to produce identical gradations for each aggregate source. Mn/DOT gradation CA-35 (38-mm top size) was used for all mixtures except for the two mitigation techniques which called for a reduction of the top size (i.e., reduction of the maximum top size of coarse aggregate, and blending of sound and unsound aggregates with a reduction of the top size of nondurable aggregates). Reduced top size aggregates were graded to meet Mn/DOT CA-60 gradation (19-mm top size).

Table 5.1. Mitigation treatments and evaluation tests used.

Mitigation Treatment	Evaluation Tests
Silane Treatment	- Absorption and Specific Gravity Tests - Washington Hydraulic Fracture Test - Iowa Pore Index Test - Rapid Freezing and Thawing Test (using Procedure B and Salt-Treated Aggregates) - VPI Single-Cycle Slow-Freeze Test
Size Reduction	- Absorption and Specific Gravity Tests - Rapid Freezing and Thawing Test (using Procedure B and Salt-Treated Aggregates) - VPI Single-Cycle Slow-Freeze Test
Blending	- Rapid Freezing and Thawing Test (using Procedure B and Salt-Treated Aggregates) - VPI Single-Cycle Slow-Freeze Test
Blending with Size Reduction of Nondurable Materials	- Rapid Freezing and Thawing Test (using Procedure B and Salt-Treated Aggregates) - VPI Single-Cycle Slow-Freeze Test
Mix Proportioning	- Rapid Freezing and Thawing Test (Procedure C) - VPI Single-Cycle Slow-Freeze Test

Concrete specimens were prepared using a type I-II portland cement and a natural coarse sand with fineness modulus, specific gravity and absorption capacity values of 2.71, 2.544, and 1.02 percent, respectively.

All concrete specimens prepared for use with ASTM C 666 procedure B using salt-treated aggregates had the following mix design parameters:

- cementitious content: 330 kg/m^3
- coarse aggregate content: 0.42 m^3/m^3
- water-cement ratio: 0.42
- coarse-to-fine aggregate volume ratio: 1.52.

Each mix met Mn/DOT requirements for a 3A21 mixture (air-entrained, consistency appropriate for slip-form paving) using saturated, surface-dry coarse aggregates and oven-dried fine aggregates. Mixing and curing were performed in accordance with ASTM C192.

For the modified mix designs (e.g., reduced water content and addition of silica fume), the cementitious content, coarse aggregate content, and coarse-to-fine aggregate volume ratio were held constant at 330 kg/m^3, 0.42 m^3/m^3, and 1.52, respectively. The water-to-cementitious ratios were 0.44, 0.40, and 0.40 for the control mix, reduced water content mix, and the mix with addition of silica fume, respectively. The class F fly ash content was 62 kg/m^3 and represented a 25 percent replacement of cement (by volume). The silica fume content was 25 kg/m^3 and represented a 10 percent replacement of cement (by volume). The silica fume content added was an 8 percent replacement (by weight) of the cementitious content typically used by Mn/DOT. Each mix met Mn/DOT requirements for a 3A21 mixture (air-entrained, consistency appropriate for slip-form paving) using saturated, surface-dry coarse aggregates and oven-dried fine aggregates. Mixing and curing were performed in accordance with ASTM C192.

The following schedule of test specimens was produced for each aggregate sample and treatment:

- three 76- by 102- by 406-mm freeze-thaw test prisms for use with ASTM C 666 procedures B or C;
- three 100- by 200-mm cylinders for the VPI single-cycle slow-freeze test;
- three 100- by 200-mm cylinders for 28-day compression strength tests; and
- one 100- by 200-mm cylinder for indirect tensile testing.

Compression and indirect tensile strength tests were performed to investigate the effects of treatment processes on the strength of the concrete mixtures.

Cylinders were made for use with the VPI single-cycle slow-freeze test (instead of using freeze-thaw beams) in an attempt to improve the accuracy of dilation measurements by using a linear variable differential transformer (LVDT) to measure the change in length instead of a regular dial indicator gage. However, the LVDT did not function properly in temperatures below freezing (i.e., 0°C), so two gage studs were mounted 152 mm apart on the cylindrical surface of each specimen, and a multiposition strain gage was used to measure the change of length between the embedded gages.

5.6 Evaluation of D-Cracking Mitigation Techniques

5.6.1 Concrete Mix Proportioning

It has been hypothesized that concrete durability can be improved, even in the presence of nondurable coarse aggregate, if the concrete is made less permeable, thereby making it harder to critically saturate the aggregate (81, 82, 83). Mix design modifications that might effect such changes in mortar and paste permeability include: decreases in water content, increases in cement content, and the use of pozzolanic admixtures such as silica fume, fly ash, and ground slag. The more economical of these options are decreased water content and the use of pozzolanic admixtures; the effects of these two approaches were investigated in this study.

Reduction of Water/Cement Ratio

The proportion of mixing water used strongly affects the permeability of the hydrated cement paste because it determines both the total volume of voids in the paste and the volume of water-filled or unfilled space at any stage of cement hydration or curing (i.e., the proportion of mix water relative to the volume of cementitious material and the degree of hydration determine the porosity of the mix). As the water-cement ratio increases, porosity increases, progressive weakening of the matrix occurs, and the capacity for freezable water increases (40). Reducing the water-cement ratio densifies the cement paste, thereby reducing its permeability and making it more difficult for externally supplied water to reach the embedded aggregates.

Five aggregate sources (i.e., A, F, G, H, and I) were used to prepare concrete mixes using two different water-to-cement ratios (i.e., 0.44 and 0.40). In addition, concrete was prepared using aggregates from Source C (Hammond) at water-to-cement ratios of 0.42 and 0.40; the control mix featured a water-to-cement ratio of 0.42 instead of 0.44 after an unacceptably high slump was obtained when other mix parameters (i.e., cement, air, and aggregate contents) were held constant. Table 5.2 presents a summary of the results of tests performed on these mixtures.

Table 5.2. Laboratory test results for alternate mix designs.

		Aggregate Source:	A	C	F	G	H	I
Water-to-Cementitious Ratio		**Mix Design**						
	Ratio by Weight	Control	0.44	0.42	0.44	0.44	0.44	0.44
	Ratio by Weight	Reduced W/C Ratio	0.40	0.40	0.40	0.40	0.40	0.40
	Ratio by Weight	Replacement with Silica Fume	0.40	0.40	0.40	0.40	0.40	0.40
Properties of Fresh Concrete								
	Slump (mm)	Control	44	95	57	76	83	70
		Reduced W/C Ratio	19	51	83	25	51	38
		Replacement with Silica Fume	76	32	70	19	63	25
	Air Content (%)	Control	4.50	5.25	3.00	4.00	4.50	4.00
		Reduced W/C Ratio	5.00	4.25	4.00	4.50	4.75	3.50
		Replacement with Silica Fume	9.50	7.75	6.00	6.50	9.00	5.50
28-day Strength Test Results								
	Compressive Strength (kPa)	Control	41,740	39,490	35,570	40,740	37,350	40,890
		Reduced W/C Ratio	49,510	41,090	37,820	47,040	39,130	42,680
		Replacement with Silica Fume	37,710	42,730	40,770	46,940	36,600	54,790
	Tensile Strength (kPa)	Control	2,400	1,930	2,160	2,240	2,100	2,540
		Reduced W/C Ratio	2,570	1,900	2,220	2,340	2,210	2,150
		Replacement with Silica Fume	2,030	2,020	1,950	2,190	1,930	2,640
Freeze-Thaw Test Results (ASTM C 666 Procedure C)								
	Durability Factor (RDM Failure) DF_F	Control	58	98	85	66	88	96
		Reduced W/C Ratio	86	98	94	76	97	96
		Replacement with Silica Fume	74	91	77	45	87	91
	Dilation, d_L (%)	Control	0.078	0.004	0.012	0.046	0.017	0.005
		Reduced W/C Ratio	0.036	0.001	0.012	0.035	0.004	0.000
		Replacement with Silica Fume	0.038	0.002	0.034	0.055	0.023	0.019
VPI Single-Cycle Slow-Freeze Test Results								
	Temperature Slope, b_l (mm/°C)	Control	-1.70	-0.29	-1.09	-0.80	-0.39	-0.84
		Reduced W/C Ratio	-0.73	-0.30	-0.59	-0.23	-0.74	-0.30
		Replacement with Silica Fume	0.10	0.01	0.05	-0.08	0.01	-0.14
	Time Slope, b_t (mm/hour)	Control	-22.5	-8.4	-14.3	-16.1	-16.5	-16.3
		Reduced W/C Ratio	-14.8	-3.9	-13.0	-10.0	-17.0	-5.0
		Replacement with Silica Fume	-2.5	-2.5	-4.1	-5.3	-4.6	-3.0

The rapid freezing and thawing test results showed that reducing the water-cement ratio improved the durability factor and reduced the dilation for most concrete specimens prepared using each aggregate source, as shown in figures 5.1 and 5.2. The control mixture for source A exhibited the highest dilation (0.078 percent), and reducing the water-cement ratio by 0.04 reduced the dilation by more than half to 0.036 percent.

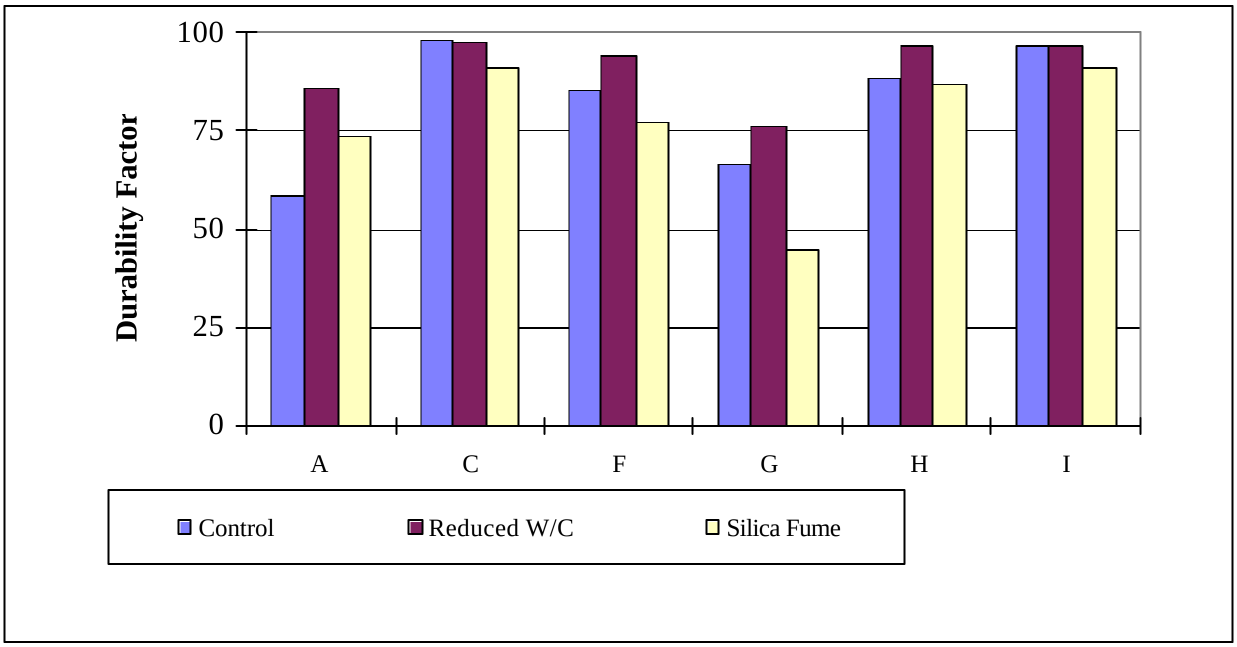

Figure 5.1. Rapid freeze-thaw test results (durabilty factor) for varying mix proportions.

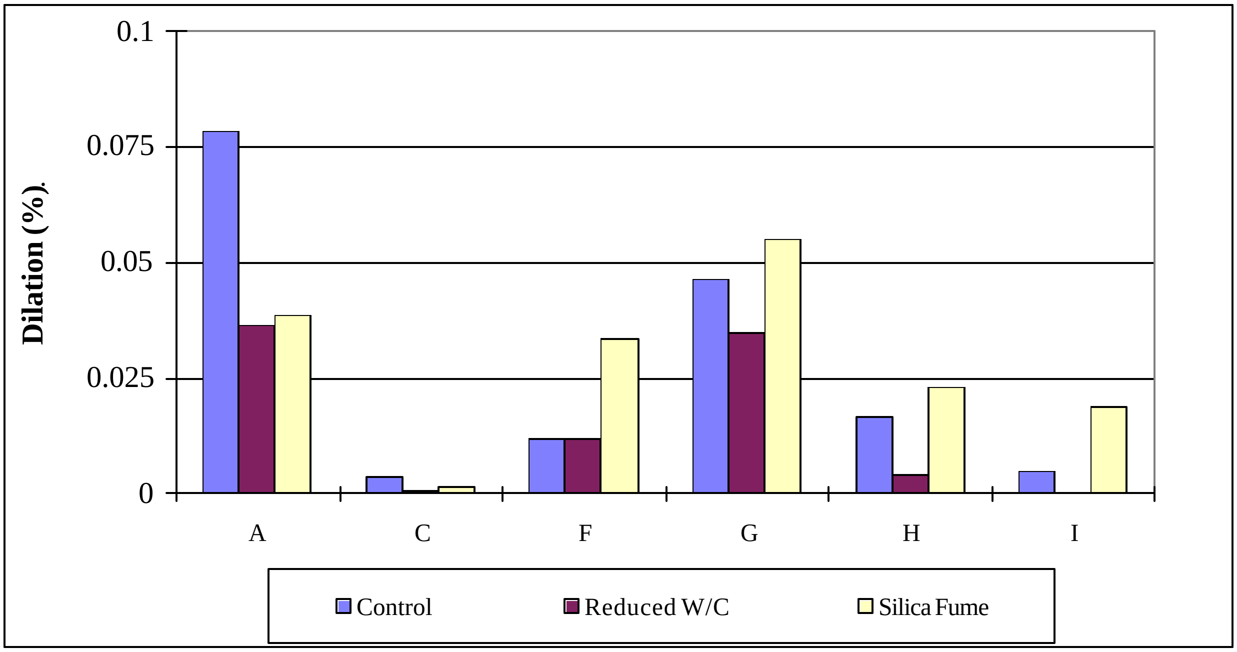

Figure 5.2. Rapid freeze-thaw test results (dilation) for varying mix proportions.

The VPI single-cycle slow freeze test also showed significant improvements in concrete durability with reduced water-to-cement ratio, as indicated by the changes in temperature and time slopes (i.e., b_l and b_t, respectively) and shown in figures 5.3 and 5.4. The temperature slope, b_l, was negative for all of the mixtures, indicating nondurable mixtures -- even for those prepared using the durable aggregate source C (Hammond). These results suggest that the temperature slope should not be used as the sole rejection criterion for concrete aggregate durability.

With a reduced water-to-cement ratio, the time slope, b_t, decreased by 9 to 69 percent for all aggregate sources except for source H, although the measured time slopes still classified sources A and F as nondurable. Source H aggregates exhibited a very slight increase in time slope with the reduced water-cement ratio, and the value (-17.0 mm/hour) indicated a nondurable aggregate source.

The compressive strengths for the mixtures with reduced water-cement ratios were 4 to 19 percent higher than those of the control mixtures. Indirect tensile strengths were also generally slightly higher for the reduced water-to-cement mixtures except for the mixture made with aggregates from source I. The increased strength of the reduced water-cement ratio mixtures probably helped to increase the durability of those mixtures.

Silica Fume Addition

Sometimes referred to as condensed silica fume or microsilica, silica fume is a pozzolanic admixture that is used as a partial replacement for or in addition to portland cement, typically at rates between 5 and 30 percent (by weight). Mixtures made using silica fume are generally considered to possess higher strength than normal concrete with good freeze-thaw durability due to the effects of pore refinement by the small silica fume particles and the production of additional hydration products (86, 92).

This study used an 8 percent (by weight) replacement of cement with silica fume to evaluate its effectiveness in reducing frost damage. A water-to-cement ratio of 0.40 and a high-range water

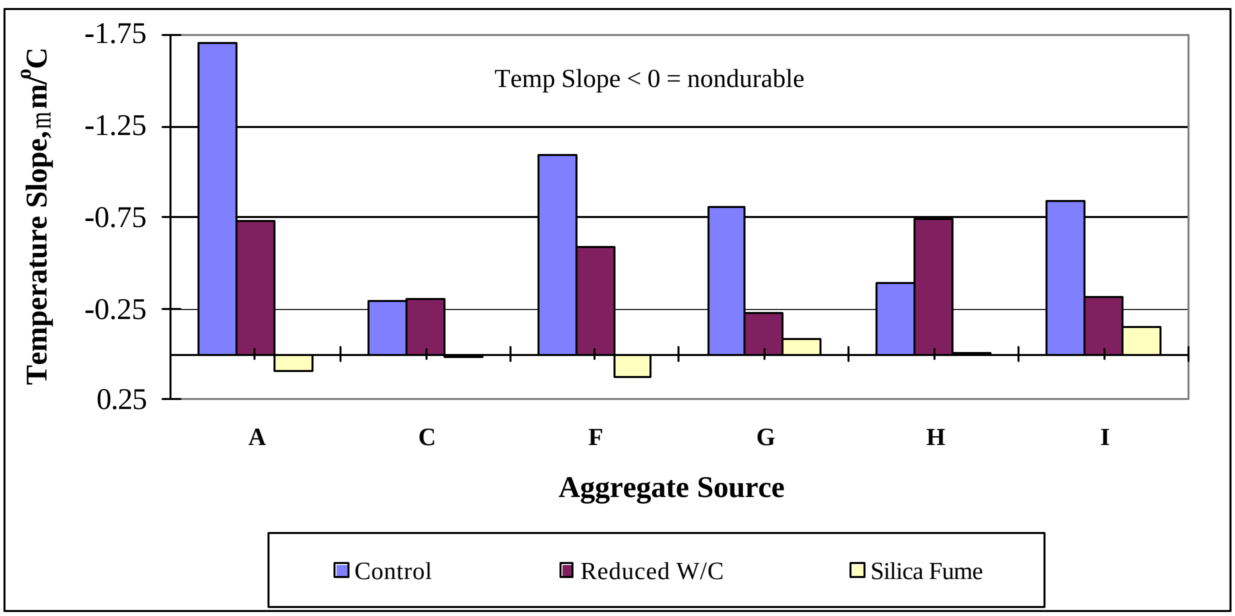

Figure 5.3. VPI single-cycle slow-freeze test results (temperature slope) for varying mix proportions.

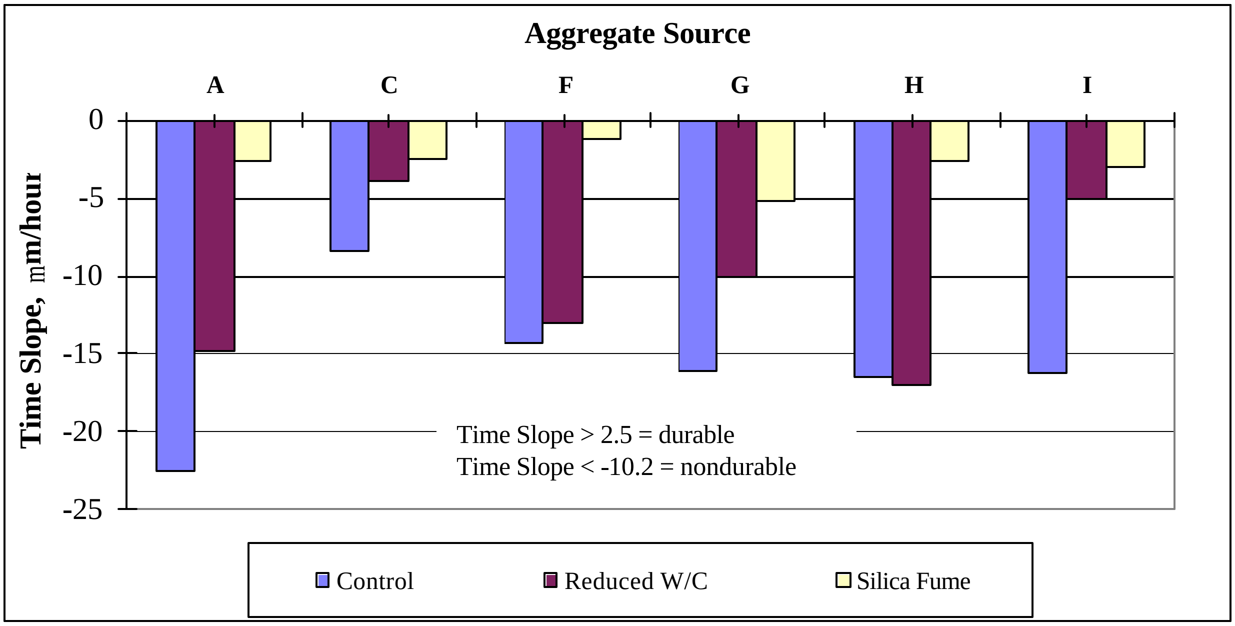

Figure 5.4. VPI single-cycle slow-freeze test results (time slope) for varying mix proportions.

reducer were also used. Table 5.2 presents the results of tests performed on concrete specimens prepared using this mitigation technique.

Figures 5.1 and 5.2 show that, with a water-cement ratio of 0.4, the replacement of small amounts of cement with silica fume caused a reduction in the durability factors and an increase in the dilations for all aggregate sources. This indicates that silica fume was not effective in reducing freeze-thaw damage when nondurable aggregates are included in the concrete mixture.

The VPI single-cycle slow-freeze test results improved significantly when silica fume was used as a partial replacement for cement, as shown in figures 5.3 and 5.4. The effect of silica fume in densifying the cement paste and increasing the compressive strength of the concrete may be partially responsible for the significant decreases in the time slope and the temperature slope.

The specimens prepared using silica fume showed increased compressive strengths for sources C, F, G, and I. Slight decreases in strength were observed for sources A and H, which might be explained, at least in part, by the high air contents of these mixes. The increases in compressive strength ranged from 10 to 34 percent from the control mixes and between -24 and 28 percent from the reduced water-cement ratio mixtures. The split tensile strengths for the mixtures made with silica fume ranged from 27 percent lower to 28 percent higher than the strengths of the control and reduced water content mixes.

5.6.2 Silane Treatment

Silane treatment of the concrete reportedly improved the freeze-thaw durability of PCC pavement cores and reduced the rate of D-cracking deterioration in previous studies (5). Silane treatment was used in the current study in an attempt to prevent the critical saturation of coarse aggregates with water, thereby improving their resistance to freeze-thaw damage.

Samples of the six coarse aggregate sources were treated with silane using the following procedure:

- the samples were sieved to separate particles smaller than 20 mm;
- the sample particles larger than 20 mm were washed and then placed in an oven at a temperature of 101 ^{o}C for 24 hours;
- a pan was filled with a water-based silane solution (Hydrozo Enviroseal 40) to a level that would cover the aggregate sample;
- the sample was placed into the strainer and the strainer was placed into the pan for 30 seconds;
- the strainer was removed from the pan and excess sealer was allowed to drain for 5 minutes;
- the sample was placed in an oven for 24 hours at a temperature of 101°C; and
- the fine aggregate (particles smaller than 20mm) were reblended with the treated coarse aggregate.

The silane-treated samples were then salt-treated in using the Iowa salt treatment procedure, and the aggregates were tested using the Washington hydraulic fracture, the Iowa pore index test, the VPI single-cycle slow-freeze test, and the rapid freezing and thawing test (ASTM C666 procedure B). Table 5.3 shows the results of tests on the silane-treated aggregate.

Absorption capacities of silane- and salt-treated aggregates were significantly lower than for their untreated counterparts, with the exception of source C (Hammond), which showed a slight increase in absorption when treated. The reduction in absorption varied from 31 to 39 percent for sources A, F, G, H, and I when compared to untreated aggregates and from 15 to 46 percent when compared to aggregates that were salt-treated without silane treatment. The reduction of absorption in these nondurable-to-marginal durability aggregate sources (i.e., A, F, G, H, and I) showed the effectiveness of silane in preventing water intrusion in the coarse aggregates, thus helping to prevent their saturation before freezing. Aggregate specific gravities did not vary significantly between treated and untreated aggregates, as expected.

The silane-treated aggregates exhibited a slight decrease in the Iowa pore index test secondary load, but not enough of a decrease to indicate a change in performance from nondurable to durable. The total amount of water taken by the aggregate pores (primary load plus secondary load) was higher for salt- and silane-treated aggregates than for silane-treated aggregates,

Table 5.3. Results of laboratory tests for silane-treated aggregate.

		Aggregate Source: / Type of Treatment	A	C	F	G	H	I
Absorption and Specific Gravity Test Results								
	Bulk Specific Gravity at 23C	Untreated	2.518	2.703	2.625	2.576	2.645	2.666
		Salt-Treated	2.495	2.693	2.612	2.565	2.639	2.659
		Silane- and Salt-Treated	2.511	2.698	2.619	2.567	2.651	2.669
	Absorption (%)	Untreated	3.028	1.126	1.547	2.751	0.968	1.363
		Salt-Treated	2.826	1.355	1.807	2.805	0.891	1.400
		Silane- and Salt-Treated	2.088	1.153	0.982	1.862	0.591	0.899
Iowa Pore Index Test Results								
	Primary Load (ml)	Untreated	153.5	55.1	46.3	85.3	31.3	41.5
		Silane-Treated	198.0	47.0	48.0	95.0	31.0	57.0
		Silane- and Salt-Treated	197.0	68.0	80.0	104.0	63.0	92.0
	Secondary Load (ml)	Untreated	38	22	21	58	19	17
		Silane-Treated	41	17	19	41	16	14
		Silane- and Salt-Treated	38	16	23	43	16	16
	Quality Number	Untreated	2.62	1.70	1.69	5.29	1.65	1.29
		Silane-Treated	2.73	1.29	1.48	3.26	1.36	0.93
		Silane- and Salt-Treated	2.46	1.13	1.59	3.34	1.08	1.07
Washington Hydraulic Fracture Test Results								
	Hydraulic Fracture Index, HFI	Silane-Treated	65	54	24	19	66	146
		Silane- and Salt-Treated	139	206	57	56	> 500	130
	% Fracture	Silane-Treated	3.8	4.6	8.8	8.5	3.8	1.7
		Silane- and Salt-Treated	1.8	1.2	4.4	4.4	2.5	1.9
Properties of Fresh Concrete								
	Slump (mm)	Silane-Treated	64	89	101	83	70	76
		Silane- and Salt-Treated	44	83	127	64	70	76
	Air Content (%)	Silane-Treated	4.00	4.75	4.50	4.50	3.25	4.75
		Silane- and Salt-Treated	3.25	5.50	4.50	4.50	3.00	4.50
28-day Strength Test Results								
	Compressive Strength (kPa)	Silane-Treated	45,830	39,560	33,890	41,110	36,110	38,220
		Silane- and Salt-Treated	46,020	40,610	33,010	40,260	37,750	38,340
	Tensile Strength (kPa)	Silane-Treated	2,290	2,280	1,580	2,450	2,110	2,080
		Silane- and Salt-Treated	1,910	2,080	1,680	2,200	1,800	2,200
Freeze Thaw Test Results (Procedure B)								
	Durability Factor (RDM Failure), DF_F	Silane-Treated	75	100	73	57	84	98
		Silane- and Salt-Treated	69	99	91	83	88	94
	Dilation, d_L (%)	Silane-Treated	0.072	0.015	0.052	0.067	0.057	0.005
		Silane- and Salt-Treated	0.092	0.011	0.041	0.054	0.033	0.018
VPI Single-Cycle Slow-Freeze Test Results+B9								
	Temperature Slope, b_l (mm/^{o}C)	Silane-Treated	0.0	-0.1	-0.2	0.0	-0.1	-0.3
		Silane- and Salt-Treated	-0.2	-0.4	-0.2	-0.5	-0.2	-0.1
	Time Slope, b_t (mm/hour)	Silane-Treated	-3.5	-3.5	-10.5	-6.5	-6.0	-4.5
		Silane- and Salt-Treated	-14.8	-8.0	-5.0	-7.3	-6.3	-4.3

(a): Only limited testing was performed on the carbonate fraction samples of gravel sources

primarily because the salt- and silane-treated aggregates showed higher absorption at early stages of the test (i.e., higher primary loads). These increases in the primary load may be explained by the effects of salt on the viscosity of water, the surface tension properties of water going in the pores, and the affinity of the pore system for water.

The Washington hydraulic fracture test was used to determine the effect of salt treatment on the hydraulic fracture index and the percent fracture. The effectiveness of silane treatment could not be evaluated because the treatment itself is part of the normal test procedure. The salt-treated aggregates exhibited increased hydraulic fracture indexes and significantly decreased rates of fracture for all aggregate sources but source I (Harris). The reduced rate of fracture and increased fracture index are probably due to the more rapid sorption of water into the aggregate pore system before pressurization (due to the salt treatment process), which would reduce the internal hydraulic pressure caused by the compression of air within the aggregate pore structure, thereby producing less fracturing of the aggregates. This suggests that the hydraulic fracture test would not provide a good indication of the deleterious effects of deicing salts on aggregate durability.

The rapid freezing and thawing test results (ASTM C 666 procedure B using salt-treated aggregates) showed slight improvements in the freeze-thaw durability of some nondurable silane-treated aggregates (sources F, G, and H) as indicated by either an increase in the durability factor (as shown in figure 5.5), a reduction in dilation (as shown in figure 5.6), or both. These improvements in freeze-thaw durability were not sufficient to classify these aggregates as durable, however. A slight decrease in durability factor and a small increase in dilation were observed for aggregate from sources A and I. Source A is a highly porous dolomite, and source I performed fairly well in the field and when tested using ASTM C666, even though it contained some fine-grained dolomites. The results of these tests indicate that silane treatment was not effective in providing significant improvements to the freeze-thaw resistance of nondurable aggregates.

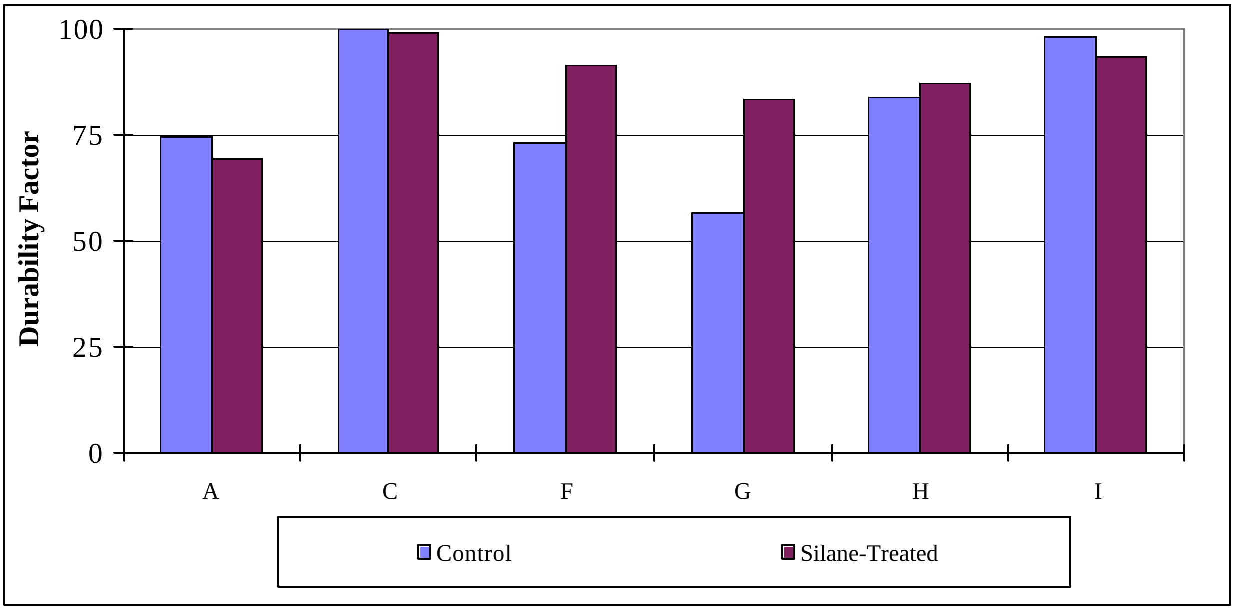

Figure 5.5. Freeze-thaw durability test results (durability factor) for silane-treated aggregate (ASTM C 666 Procedure B with salt-treated aggregate).

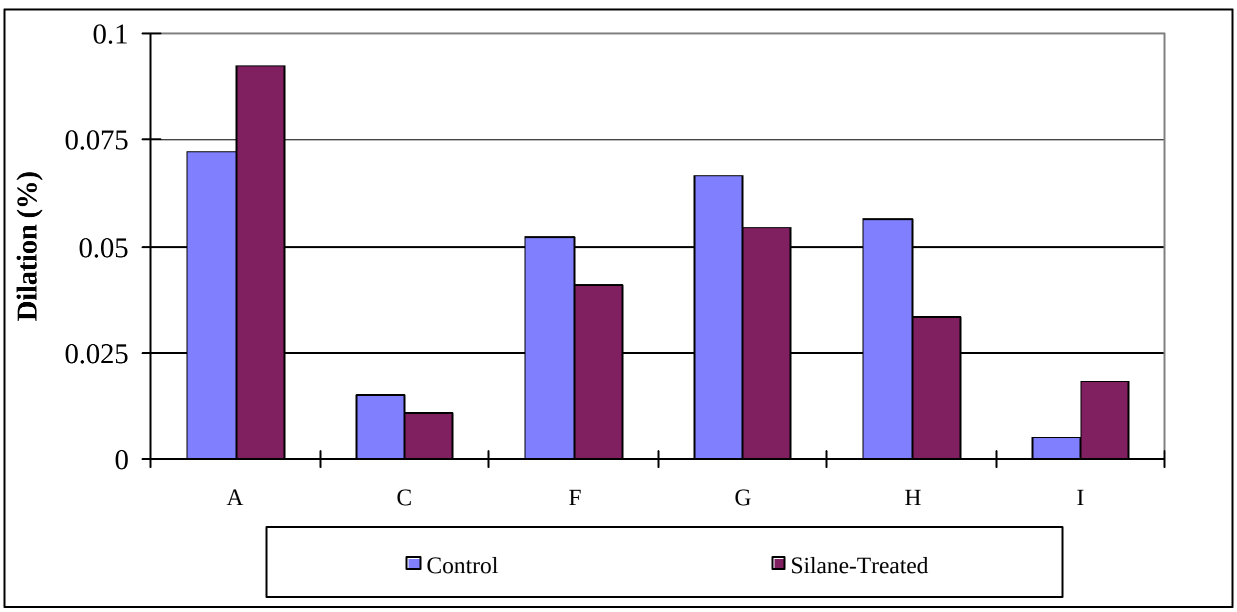

Figure 5.6. Freeze-thaw durability test results (dilation) for silane-treated aggregate (ASTM C 666 Procedure B with salt-treated aggregate).

The VPI single-cycle slow freeze test gave mixed results for the silane treatment. The mixed results might be explained by the effects of salt on concrete when freezing: although the salt lowers the freezing temperature of water, the amount of water available for freezing increases in the presence of salts.

5.6.3 Linseed Oil Treatment

Cady, et al. reported that impregnating aggregate with boiled linseed oil improved the frost resistance of concrete (78). The treatment procedure with boiled linseed oil consisted of the following:

- the aggregate samples were placed in an oven to reach 105°C;
- the hot aggregate samples were then soaked in the boiled linseed oil solution;
- the aggregate samples were then allowed to cool to room temperature for 24 hours; and
- they were then oven-dried for 24 hours.

The linseed oil-treated samples were then salt-treated in accordance with the Iowa salt treatment procedure. The development of a thick, dark coating was noted on the surface of the aggregate particles during the salt treatment process. This coating developed when the linseed oil surface treatment of the aggregate particles burned during the oven-drying cycles required for the salt treatment procedure. This resulted in a reduction in aggregate particle surface texture, which was believed to the reduce the bond between the aggregate and the cement paste, as well as introduce an organic carbon-based substance to the concrete mixture, which would tend to retard the setting of the concrete and reduce concrete strength -- a similar effect to that of adding sugar to the mixture.

The linseed oil treatment was discontinued because it resulted in poor concrete quality, as explained by the retardation of the cement reactions due to the presence of organic materials and oils and the poor bond between the aggregate and the cement paste.

5.6.4 Blending

This method consists of blending durable coarse aggregates with nondurable ones in an effort to allow the use of nondurable materials without compromising concrete durability. Four blend percentages were used to evaluate the effectiveness of this method: 10, 20, 30, and 40 percent nondurable material, by volume. Grand Meadows and Hammond (i.e., sources A and C) were used as the nondurable and durable materials, respectively.

The rapid freezing and thawing test results showed that durability factor decreased and the dilation increased with increased unsound aggregate content, as expected, and shown in table 5.4 and figures 5.7 and 5.8. When using 10 percent unsound aggregate, the durability factor and the dilation did not differ significantly from that of mixtures prepared using only sound aggregates. With 40 percent unsound aggregates in the concrete mix, the durability factor and dilation were similiar to those observed when only nondurable aggregates are used. The durability factors were high (indicating durable concrete) for all proportions of unsound aggregate used in the blends; however, the addition of 30 percent or more of unsound aggregate produced dilations indicating susceptibility to frost damage (i.e., greater than 0.04 percent). This indicates that, although blending unsound aggregate with 60 percent or more sound aggregate produced improvements in concrete freeze-thaw performance, this mitigation technique is probably not economical and may not be effective when the unsound aggregate content is 30 percent or higher.

The VPI single-cycle slow freeze test results were not conclusive for blends that contained 10 and 20 percent of unsound aggregates, as shown in figures 5.9 and 5.10. The blend that contained 30 percent unsound aggregates showed a temperature slope, b_t, of -10.3, which indicated its susceptibility to freeze-thaw damage. Overall, the VPI single-cycle slow freeze test did not indicate significant improvements in freeze-thaw durability as a result of aggregate blending.

Table 5.4. Results of laboratory test results for blended aggregate samples.

		Aggregate Blend by Source: / Aggregate Gradation (Top Size, Treatment)	C: 100 % A: 0 %	C: 90 % A: 10 %	C: 80 % A: 20 %	C: 70 % A: 30 %	C: 60 % A: 40 %	C: 0 % A: 100 %
Absorption & Specific Gravity Test Results								
	Bulk Specific Gravity at 23C	CA 35 (38 mm, None)	2.703	*2.685*	*2.666*	*2.648*	*2.629*	2.518
		CA 35 (38 mm, Salt)	2.693	*2.673*	*2.653*	*2.634*	*2.614*	2.495
	Absorption (%)	CA 35 (38 mm, None)	1.126	*1.303*	*1.484*	*1.666*	*1.852*	3.028
		CA 35 (38 mm, Salt)	1.355	*1.492*	*1.632*	*1.773*	*1.917*	2.826
Properties of Fresh Concrete								
	Slump (mm)	CA 35 (38 mm, Salt)	89	83	51	76	76	64
	Air Content (%)	CA 35 (38 mm, Salt)	4.75	5.50	4.25	5.50	5.00	4.00
28-day Strength Test Results								
	Compressive Strength (kPa)	CA 35 (38 mm, Salt)	39,560	40,300	41,660	46,820	42,170	45,830
	Tensile Strength (kPa)	CA 35 (38 mm, Salt)	2,280	1,920	2,170	2,250	2,240	2,290
Freeze Thaw Test Results (ASTM C 666 Procedure B)								
	Durability Factor (RDM Failure) DF_F	CA 35 (38 mm, Salt)	100	99	97	93	82	75
	Dilation, d_L (%)	CA 35 (38 mm, Salt)	0.015	0.017	0.024	0.043	0.076	0.072
VPI Single-Cycle Slow-Freeze Test Results								
	Temperature Slope, b_l (mm/^{o}C)	CA 35 (38 mm, Salt)	-0.1	-0.2	-0.2	-0.5	-0.1	-0.1
	Time Slope, b_t (mm/hour)	CA 35 (38 mm, Salt)	-3.5	-8.5	-6.3	-10.3	-4.5	-3.5

Note: *Italicized Values* for BSG and Absorption are interpolated from measured values for Sources A and C.

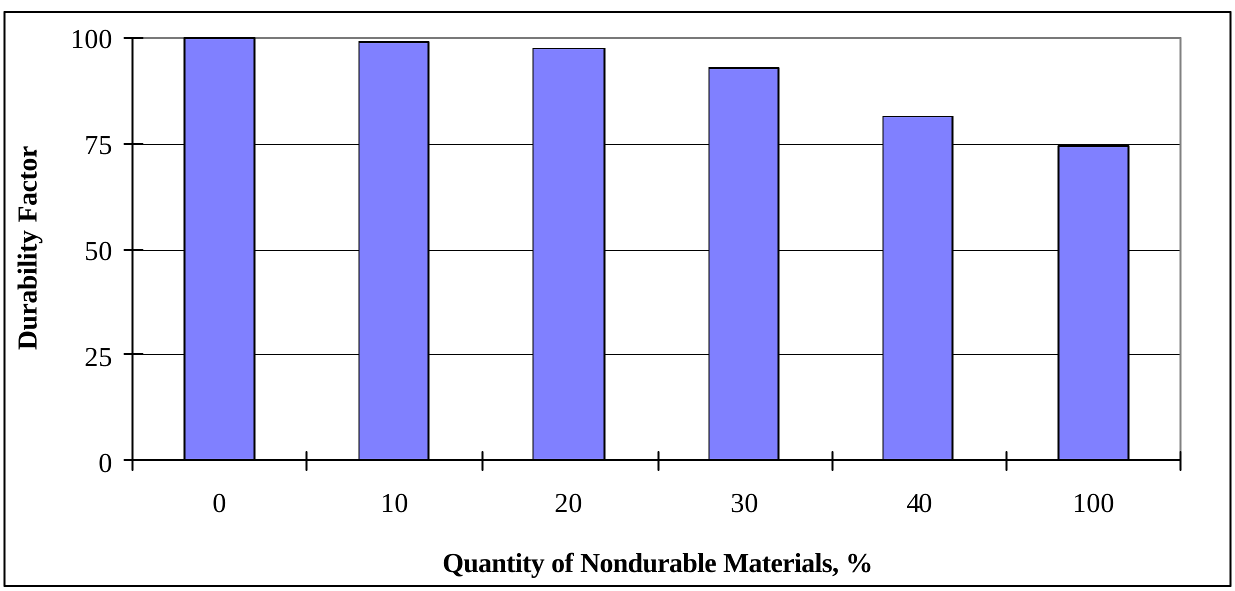

Figure 5.7. Rapid freeze-thaw test results (durability factor) for aggregate blending.

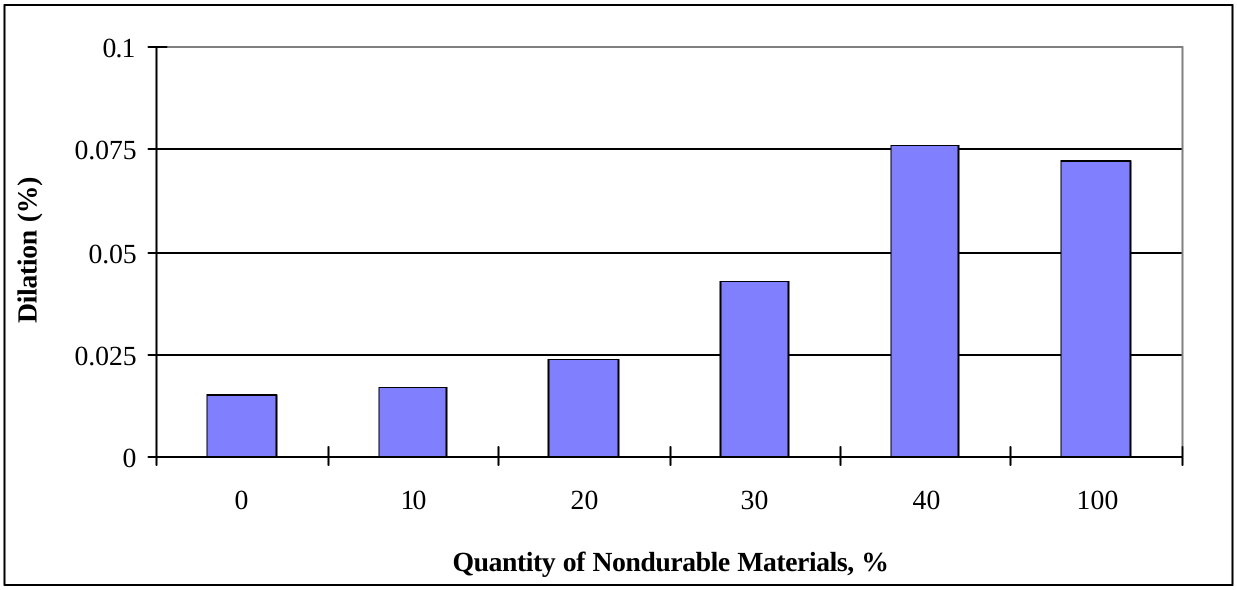

Figure 5.8. Rapid freeze-thaw test results (dilation) for aggregate blending.

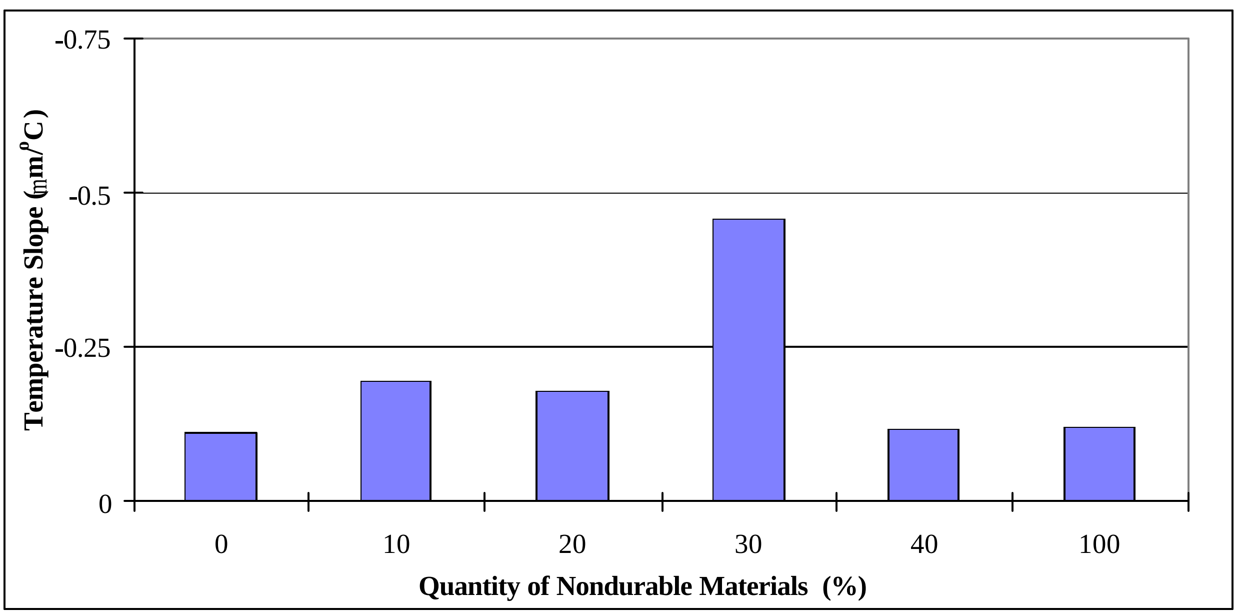

Figure 5.9. VPI single-cycle slow freeze test results
(temperature slope) for aggregate blending.

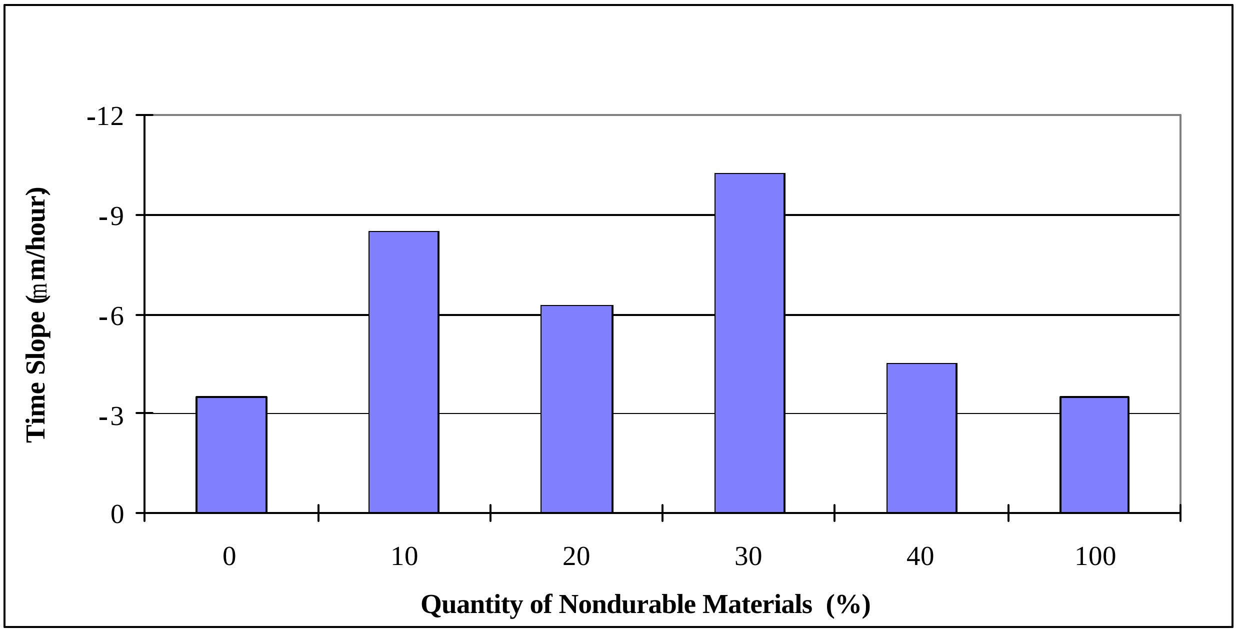

Figure 5.10. VPI single-cycle slow freeze test results (temperature slope) for aggregate blending.

5.6.5 Size Reduction

Laboratory and field observations point to the reduction of coarse aggregate top size as a feasible and effective method for reducing the incidence and severity of D-cracking (5). In this study, the top size of the coarse aggregates was reduced from 38 mm to 19 mm, and concrete specimens made with these aggregates were tested using the VPI single-cycle slow-freeze test and the rapid freezing and thawing test (ASTM C 666 procedure B). Table 5.5 presents the results of these tests for this mitigation technique.

The absorption capacity of the coarse aggregate samples generally increased when the top size was reduced. These increases were highly significant for sources A, F and H (i.e., 22, 22.5 and 38 percent, respectively). Specific gravity values were slightly reduced.

Rapid freezing and thawing test results (ASTM C 666 Procedure B) showed improved durability factors for sources F and H; however, a decrease in the durability factor was observed for source A, a porous aggregate source.

Table 5.5. Results of laboratory tests of aggregates with reduced top size.

		Aggregate Source: / Aggregate Gradation (Top Size, Treatment)	A	F	H	I
Absorption & Specific Gravity Tests						
	Bulk Specific Gravity at 23C	CA 35 (38 mm, None)	2.518	2.625	2.645	2.666
		CA 35 (38 mm, Salt)	2.495	2.619	2.639	2.659
		CA 60 (19 mm, None)	2.500	2.620	2.648	2.665
		CA 60 (19 mm, Salt)	2.501	2.622	2.641	2.662
	Absorption (%)	CA 35 (38 mm, None)	3.0	1.5	1.0	1.4
		CA 35 (38 mm, Salt)	2.8	1.6	0.9	1.4
		CA 60 (19 mm, None)	3.4	1.8	1.1	1.5
		CA 60 (19 mm, Salt)	3.5	1.9	1.2	1.4
Properties of Fresh Concrete						
	Slump (mm)	CA 35 (38 mm, Salt)	64	101	70	76
		CA 60 (19 mm, Salt)	25	57	76	51
	Air Content (%)	CA 35 (38 mm, Salt)	4.0	4.5	3.3	4.8
		CA 60 (19 mm, Salt)	4.0	4.5	3.3	4.8
28-day Strength Test Results						
	Compressive Strength (kPa)	CA 35 (38 mm, Salt)	45,830	33,890	36,110	38,220
		CA 60 (19 mm, Salt)	46,950	39,630	41,400	40,700
	Tensile Strength (kPa)	CA 35 (38 mm, Salt)	2,290	1,580	2,110	2,080
		CA 60 (19 mm, Salt)	2,360	2,090	2,050	1,720
Freeze Thaw Test Results (ASTM C 666 Procedure B)						
	Durability Factor (RDM Failure) DF_F	CA 35 (38 mm, Salt)	75	73	84	98
		CA 60 (19 mm, Salt)	59	84	93	98
	Dilation, d_L (%)	CA 35 (38 mm, Salt)	0.072	0.052	0.057	0.005
		CA 60 (19 mm, Salt)	0.151	0.047	0.023	0.012
VPI Single-Cycle Slow-Freeze Test Results						
	Temperature Slope, b_t (mm/^{o}C)	CA 35 (38 mm, Salt)	-0.1	-0.2	-0.1	-0.3
		CA 60 (19 mm, Salt)	0.1	0.1	0.0	-0.1 (a)
	Time Slope, b_t (mm/hour)	CA 35 (38 mm, Salt)	-3.5	-10.5	-6.0	-4.5
		CA 60 (19 mm, Salt)	-2.5	-1.3	-2.5	-3.0 (a)

(a) : Only one specimen was tested due to limited aggregate quantities.

The dilation after 300 freeze-thaw cycles for specimens that contained source A increased significantly, which indicated increased freeze-thaw damage when reducing the top size of the coarse aggregates. This increase in the dilation and reduction of the durability factor for source A when reducing the top size might be explained by the high porosity and increased absorption of this material, which may cause cracking to originate in the area surrounding the coarse aggregate particles (i.e., the transition zone). The reduction in top size for this material is accompanied by an increase in coarse aggregate surface area, which may provide additional sites for transitional zone cracking.

The dilations for sources F and H were reduced by 9 and 60 percent, respectively, and dilations for both top sizes of source I were too low (less than 0.012) to suggest susceptibility to frost damage.

Figures 5.11 and 5.12 illustrate the changes in durability factors and dilations when reducing the top size. In summary, freeze-thaw testing seemed to indicate that the use of smaller aggregate particles generally improves aggregate freeze-thaw durability. However, the reduction of top size for aggregates with high porosity may cause greater amounts of disruption to the surrounding paste when water contained in the aggregate pores is expelled into the transition zone area, causing debonding of the aggregate and mortar.

The VPI single-cycle slow freeze test results showed that reducing the top size of the coarse aggregate significantly improved the temperature slope, b_l, for all aggregate sources (as shown in Figure 5.13). The time slope, b_t, did not indicate a frost susceptibility problem for sources A and H for either aggregate top sizes, but a 29 to 58 percent increase in the time slope values was observed, as shown in Figure 5.14. The time slope for source F did not show a frost resistance problem when the smaller top size was used, but did indicate the presence of a freeze-thaw durability problem when the top size reached 38 mm. Thus, the VPI single-cycle slow freeze test indicated improvements in concrete freeze-thaw durability when aggregate top size was reduced, although the indicated improvements were not always enough to qualify the beneficiated materials as “durable.”

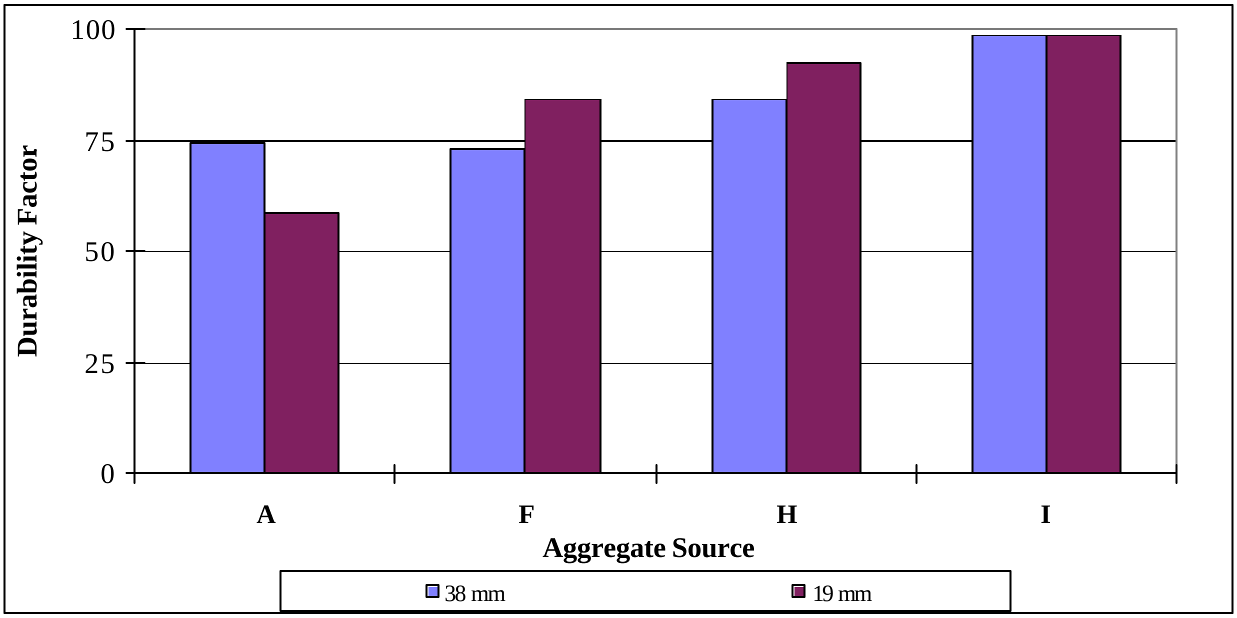

Figure 5.11. Rapid freeze-thaw test results (durability factor) for reduced aggregate top size.

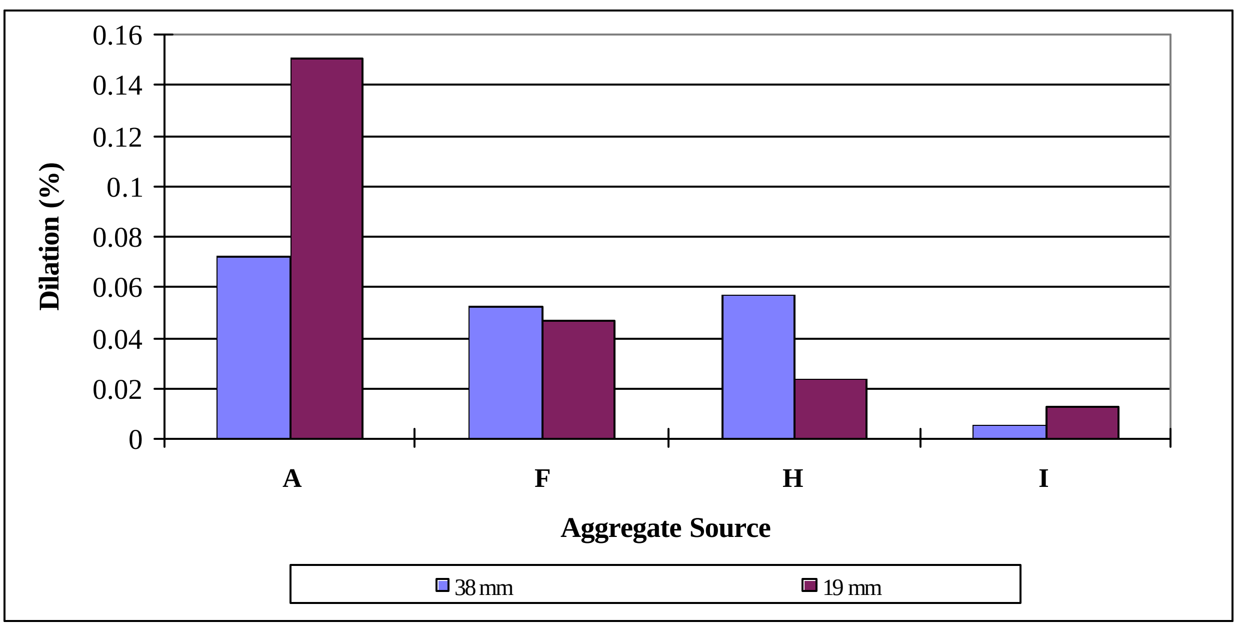

Figure 5.12. Rapid freeze-thaw test results (dilation) for reduced aggregate top size.

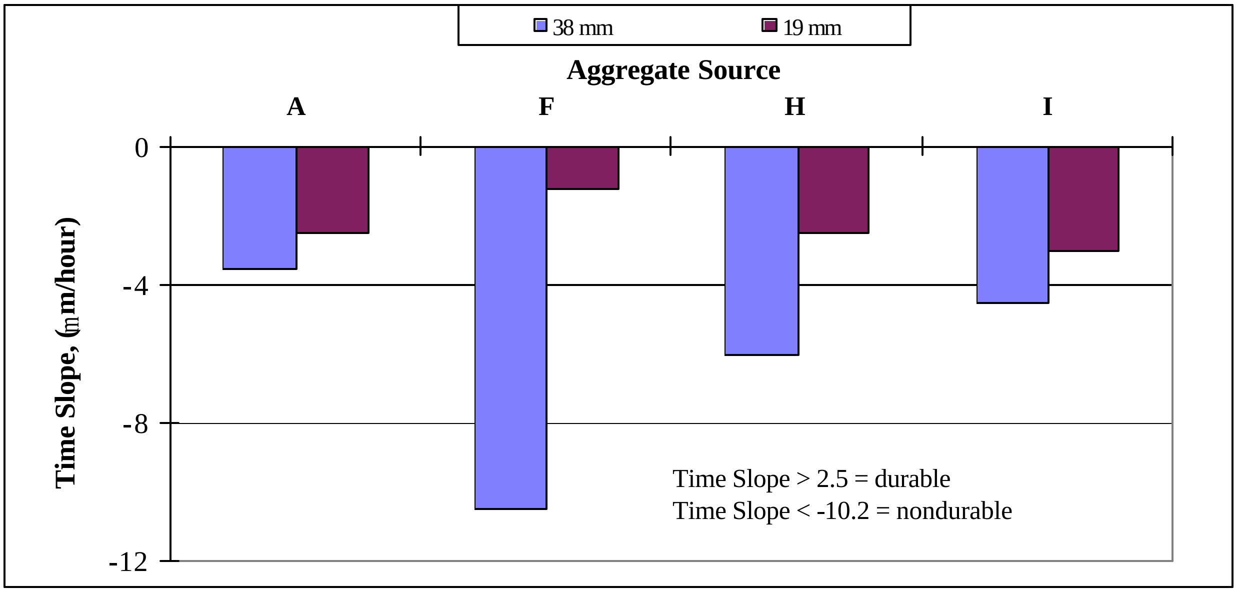

Figure 5.13. VPI single-cycle slow freeze test results
(time slope) for reduced aggregate top size.

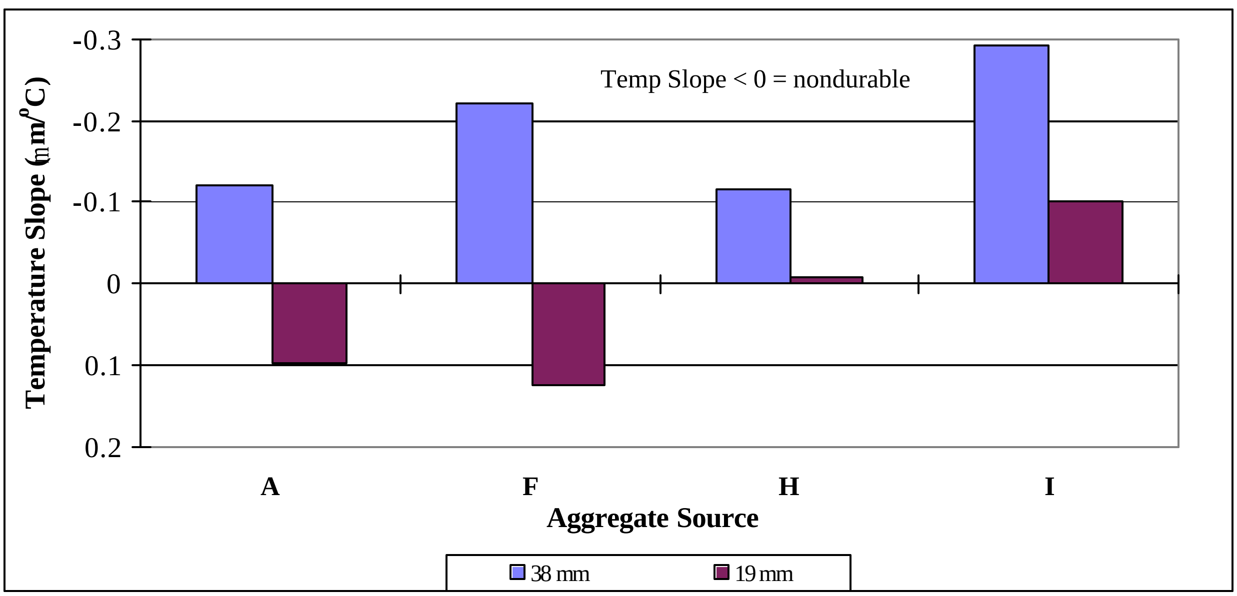

Figure 5.14. VPI single-cycle slow freeze test results
(temperature slope) for reduced aggregate top size.

5.6.6 Size Reduction and Blending (Mn/DOT Practice)

For D-cracking to occur, the concrete must possess a sufficient quantity of critically sized (i.e., large) unsound aggregate particles (5). Both aggregate blending and aggregate particle size reduction have been shown to produce improved freeze-thaw durability, so it is reasonable to assume that the concurrent use of both techniques should produce even greater improvements in durability than can be achieved by either single technique.

Mn/DOT blends coarse aggregates of at least two sizes whenever the size of coarse aggregate selected for use has less than 100 percent passing the 25-mm sieve. The 19-mm sieve is typically the sieve that divides these two size fractions (i.e., a 19-mm-plus source is blended with a 19-mm-minus source). Since 1988, Mn/DOT has allowed the use of coarse aggregate materials that do not meet Mn/DOT requirements for use in paving concrete to be used as the coarse aggregate size fraction that is smaller than 19 mm (88). This approach was also evaluated in the lab study (i.e., blending of two coarse aggregate fractions to evaluate the effect of blending sound and unsound materials using a reduced size for the unsound coarse aggregates).

An equal volume of sound and unsound aggregates were used for these blends, with source C (Hammond) being used as the aggregate source for the larger-sized particles. Sources A, F and H (Grand Meadows, Luverne and Halma, respectively) were used as nondurable sources. Mn/DOT gradations CA-35 and CA-60, with maximum top sizes of 38 and 19 mm, respectively, were used for the durable and nondurable aggregate sources. The 1988 Mn/DOT specification required an increase of cement content to 374 kg/m3; however, the cementitious content was not increased for these blends in order to make the mix parameters comparable to those used in the studies of other mitigation techniques.

Table 5.6 summarizes the results of tests selected to evaluate this mitigation technique and to determine the properties of the mixtures and aggregates. The rapid freezing and thawing test showed that this method significantly improved the durability factors for all three blends that contained the nondurable aggregates (i..e., A, F and H), as shown in figure 5.15. Dilations were also significantly reduced (as shown in figure 5.16), especially for sources F and H. The dilation

Table 5.6. Results of laboratory tests of mitigation by
blending durable aggregates and nondurable aggregates of reduced top size.

Aggregate Source(s):	C	A	A and C	F	F and C	H	H and C
Absorption & Specific Gravity Test Results							
Bulk Specific Gravity at 23C	2.703	2.495	*2.634*	2.612	*2.653*	2.639	*2.673*
Absorption (%)	1.126	2.826	*2.061*	1.807	*1.449*	0.891	*1.615*
Properties of Fresh Concrete							
Slump (mm)	89	64	70	101	83	70	76
Air Content (%)	4.75	4.00	4.50	4.50	3.25	3.25	3.00
28-day Strength Test Results							
Compressive Strength (kPa)	39,560	45,830	49,990	33,890	40,390	36,110	41,080
Tensile Strength (kPa)	2,280	2,290	1,810	1,580	1,940	2,110	2,070
Freeze Thaw Test Results (ASTM C 666 Procedure B)							
Durability Factor (RDM Failure) DF_F	100	75	90	73	96	84	97
Dilation, d_L (%)	0.015	0.072	0.044	0.052	0.022	0.057	0.018
VPI Single-Cycle Slow-Freeze Test Results							
Temperature Slope, b_l (mm/°C)	-0.1	-0.1	-0.1	-0.2	-0.1	-0.1	0.0
Time Slope, b_t (mm/hour)	-3.5	-3.5	-3.5	-10.5	-1.3	-6.0	-2.5

Note: *Italicized Values* for combined BSG and Absorption are interpolated from measured values for individual sources.

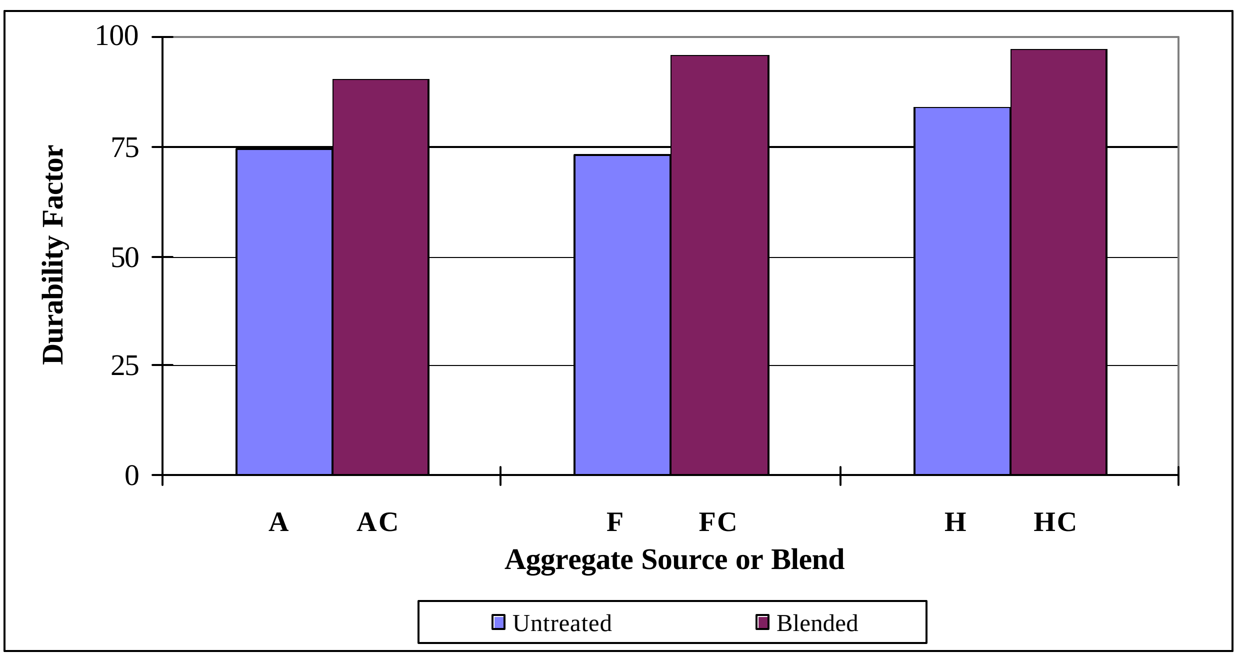

Figure 5.15. Rapid freeze-thaw test results (durability factor)

for blending durable aggregates and nondurable aggregates of reduced top size.

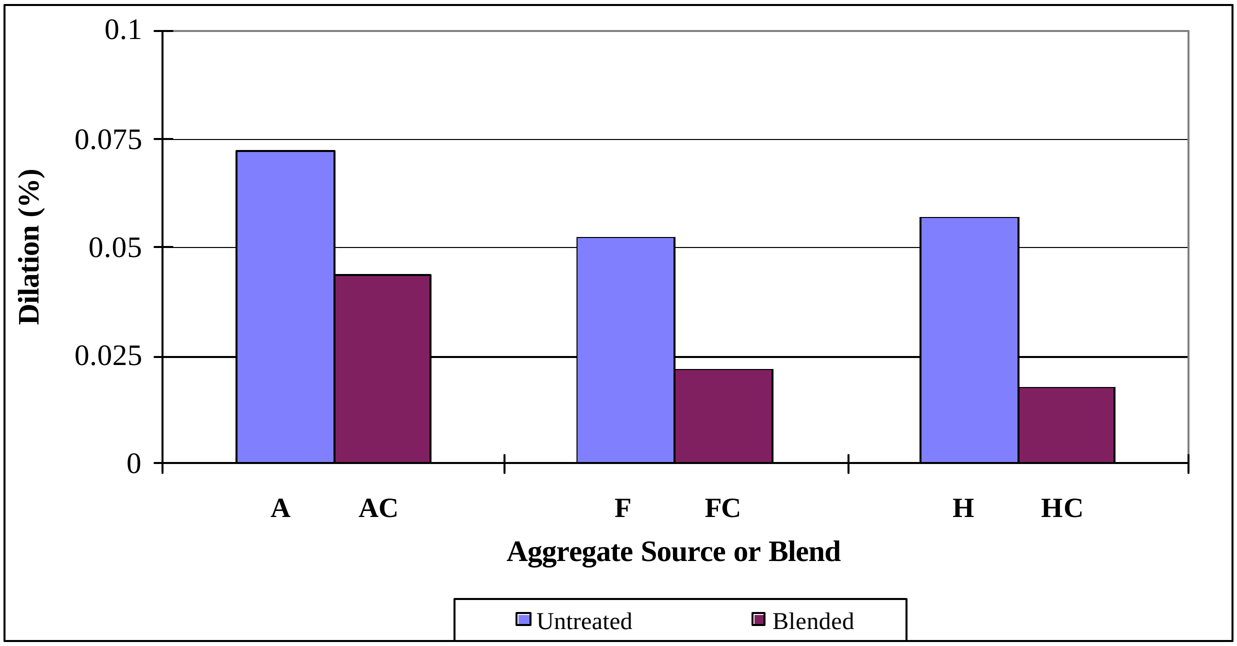

Figure 5.16. Rapid freeze-thaw test results (dilation)
for blending durable aggregates and nondurable aggregates of reduced top size.

for the blend that contained source A aggregate, which is highly porous, was reduced by 39 percent but was still higher than acceptable (i.e., 0.044 percent).

The VPI single-cycle slow freeze test results did not indicate a frost susceptibility problem with these mixtures, even when using only nondurable aggregates. The time slope values, b_t, were reduced for the blends that contained sources F and H (as shown in figure 5.17). The time slope, b_t, for the blend that contained source A did not differ from the slope observed when using sources A and C separately. The temperature slope, b_l, was less than 0 mm/^{o}C for all blends and did not show any significant difference when using the blends, as shown in figure 5.18.

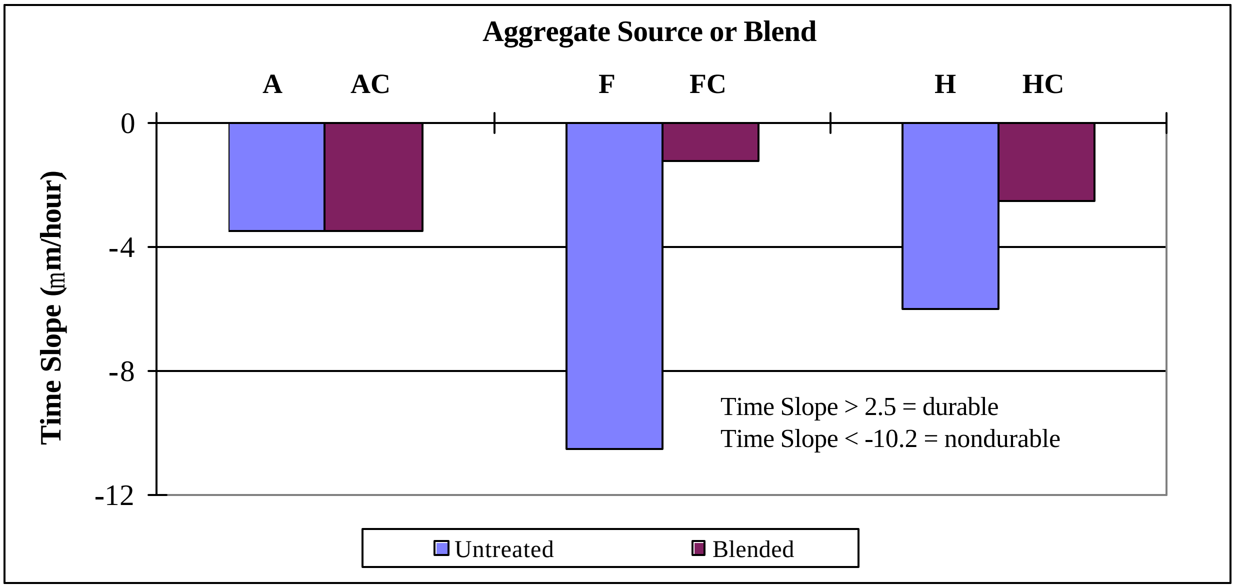

Figure 5.17. VPI single-cycle slow freeze test results (time slope) for blending durable aggregates and nondurable aggregates of reduced top size.

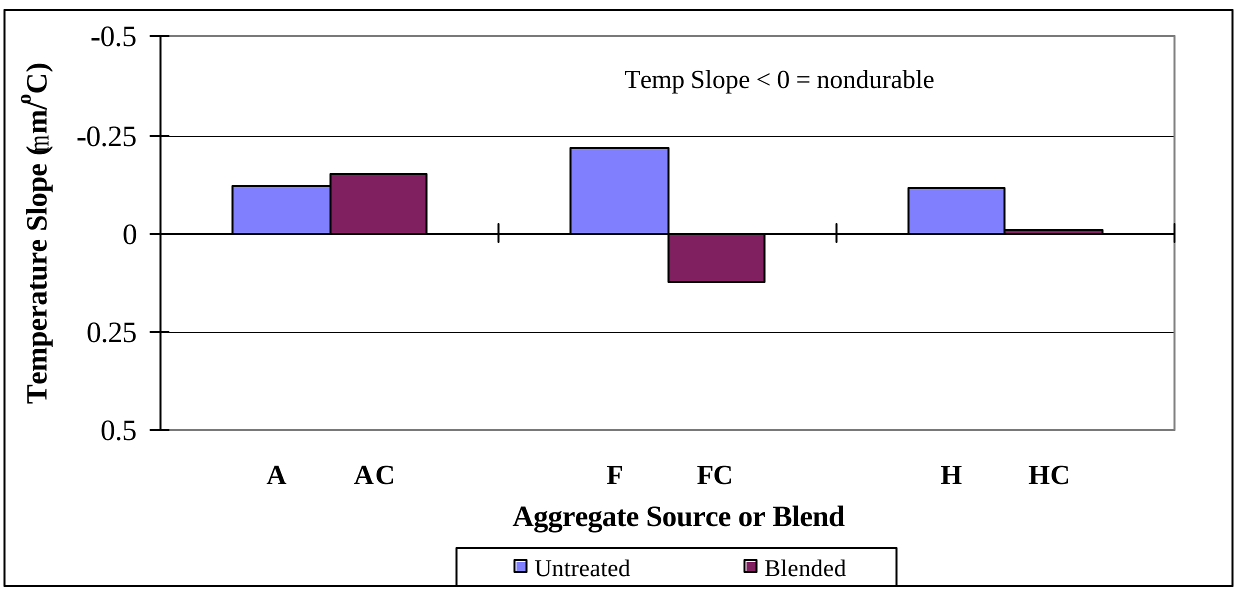

Figure 5.18. VPI single-cycle slow freeze test results (temperature slope) for blending durable aggregates and nondurable aggregates of reduced top size.

5.7 Summary

This part of the study investigated techniques that would reduce or eliminate the D-cracking potential of aggregate sources that are currently considered unacceptable for use in PCC. The study investigated several aggregate beneficiation techniques including: changes in concrete mixture proportioning (i.e., using a reduced lower water-cement ratio and the use of silica fume); silane and linseed oil treatment of coarse aggregates; blending of durable and nondurable aggregates; reduction of coarse aggregate top size; and blending of durable aggregate with nondurable aggregates of reduced size. The following tests were used to evaluate the effectiveness of these mitigation techniques: rapid freezing and thawing test, the VPI single-cycle slow freeze test, the Iowa pore index test and the Washington hydraulic fracture test.

Two water-cement ratios (0.44 and 0.40) were used to evaluate the effectiveness of densifying the cement paste, thereby reducing its permeability and porosity and making it harder to supply the aggregate with water. Reducing the water-cement ratio resulted in increased durability factors and reduced dilations, especially for aggregate sources with high absorption values. The VPI single-cycle slow freeze test did not indicate a significant improvement in the freeze-thaw durability. It is believed that reducing the water-cement ratio was effective in improving the durability of the concrete.

The use of 8 percent silica fume as a partial replacement for cement was considered in an attempt to evaluate the effectiveness of reducing the permeability of the cement paste on the concrete freeze-thaw durability. The rapid freezing and thawing test indicated that this method did not significantly improve the durability factor or significantly reduce the dilation. The VPI single-cycle slow freeze test indicated a significant improvement in the frost resistance, presumably due to the higher strength and/or density of silica fume concretes. Based on the results of the freeze-thaw tests, it is believed that the use of 8 percent silica fume did not produce significant improvements in the freeze-thaw durability of concrete produced using D-cracking susceptible aggregate.

Aggregate samples from selected nondurable sources were treated with a water-based silane solution. Iowa pore index test results did not show a significant difference in the absorption of water by the micropores (i.e., secondary load), although macropore absorption increased significantly (i.e., primary load increased). The Washington hydraulic fracture test showed an increase of hydraulic fracture index for the silane- and salt- treated aggregate, which might be explained by the effect of salt on the rapid sorption and quick saturation of the aggregates. Rapid freeze-thaw testing indicated a slight improvement in durability factor, a slight reduction of dilation, or both for all aggregate sources except for one highly porous aggregate source. Results from the VPI single-cycle slow freeze test were mixed, indicating that the silane treatment was effective for one aggregate source and detrimental for another (the porous limestone). Overall, it appears that the silane treatment was generally somewhat effective in improving the freeze-thaw durability of otherwise nondurable coarse aggregates.

Samples of each aggregate source included in the mitigation study were treated with boiled linseed oil. Researchers discontinued testing of these samples because linseed-treated aggregate samples resulted in poor concrete quality. The use of linseed oil resulted in apparent retardation of the cement hydration in the presence of organic materials and oils and an apparent decrease in bond between the aggregate particles and cement mortar.

Durable and nondurable coarse aggregates were blended to determine the feasibility of using limited quantities (i.e., 10, 20, 30 and 40 percent by volume) of nondurable material without compromising concrete durability. Freeze-thaw durability factors generally decreased and dilations generally increased as the quantity of nondurable material increased, with blends containing 30 percent or more of nondurable aggregates producing the most significant dilations (greater than 0.04 percent). The VPI single-cycle test results also indicated potential D-cracking problems when using 30 percent or more of nondurable aggregates.

Reducing the coarse aggregate top size from 38 mm to 19 mm resulted in increased freeze-thaw durability factors and decreased dilation for all aggregates tested except for one highly porous limestone. The VPI time slope and temperature slopes also indicated significantly improved resistance to freeze-thaw damage. Therefore, it was determined that reductions in coarse aggregate top size are effective in reducing D-cracking potential.

The use of both aggregate blending and reduced top size of nondurable materials was evaluated using concrete mixes containing equal volumes of durable and nondurable aggregates. High durability factors were obtained for all aggregate sources tested; low dilations also were obtained for all aggregate sources tested except the one highly porous limestone source, which appears to produce failures at the aggregate-mortar interface as water is expelled freely from the aggregate. VPI single-cycle slow freeze test results did not indicate a freeze-thaw damage potential for any of the mixtures tested, even when using only nondurable aggregates, so it was of little use in assessing the improvement due to blending and reduced aggregate size. In general, it appeared that Mn/DOT's current aggregate blending practice is effective in improving the durability of concrete constructed using some nondurable aggregate.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The objectives of this study were to:

1. identify aggregate sources that appear to be responsible for the premature failure of concrete pavement in southern Minnesota by D-cracking;
2. document the accuracy and reliability of existing tests of aggregate freeze-thaw durability using Minnesota aggregate sources and pavement performance records;
3. develop a new methodology for quickly and reliably assessing the freeze-thaw durability of a given aggregate source; and
4. identify and evaluate techniques for mitigating D-cracking, thereby allowing the possible use of aggregates sources that are currently considered marginal or unacceptable in concrete construction.

The research approach adopted for this study included:

- a literature review to summarize the state of knowledge and practice concerning aggregate freeze-thaw problems, identify durability tests that appear to be most useful in predicting D-cracking, and identify potential techniques for D-cracking mitigation;
- condition surveys of pavement sections in Minnesota that represent the range of aggregate types, climatic conditions and pavement durability-related performance found in southern Minnesota;
- laboratory testing and petrographic examination of cores retrieved from the pavement sections;
- laboratory testing and petrographic examination of coarse aggregate samples obtained from the sources used to construct the pavement study sections to ensure that the samples obtained are sufficiently similar to those used in the construction of the field sections;
- testing of coarse aggregate samples using the durability tests identified in the literature review;
- correlation of the durability test results with the observed field performance;
- development of a durability test protocol to allow more rapid and reliable evaluation of aggregate D-cracking potential; and
- evaluation of the effectiveness of selected aggregate beneficiation and D-cracking mitigation

techniques on improving the freeze-thaw durability of concrete aggregate.

The most important findings of this research can be summarized as follows:

1. Fine-grained dolomites and dolomitic limestones exhibited poor performance in the field and in laboratory testing, especially when salt-treated aggregates were used with ASTM C 666 procedure B.
2. The laboratory test that provided the best correlation with field durability performance was ASTM C 666 procedure B (rapid freeze-thaw testing) using salt-treated aggregates and either dynamic modulus of elasticity or dilation failure criteria.
3. The VPI single-cycle slow freeze test accurately predicted the frost susceptibility of some aggregates; however, test results were inconclusive for other sources, indicating the need for additional testing.
4. The secondary load from the Iowa pore index test was found to correlate well with the durability of carbonate rocks and the carbonate fraction of gravels.
5. Good correlation was generally observed between field performance and the original Washington hydraulic fracture test, although results indicated the need for further testing for some sources.
6. The results of PCA absorption, PCA adsorption, x-ray diffraction and TGA mass loss slope prior to dolomite burning were not well-correlated with aggregate freeze-thaw durability.
7. The results of x-ray fluorescence tests suggest that aggregates with relatively high phosphorous contents are generally nondurable.
8. Thermogravimetric analysis showed that nondurable carbonates with a calcite fraction of 20 percent or more showed a high burning slope prior to calcite transition.
9. Regression analyses were used to develop good models of freeze-thaw durability factor (ASTM C 666 using procedure B and salt-treated aggregate) and VPI single-cycle slow freeze time slope using the results of the Iowa pore index and the bulk specific gravity as predictors. Tests of additional aggregate sources are needed to validate these and other regression models. With further validation, these models might offer a rapid means of evaluating aggregate freeze-thaw durability without preparing concrete test specimens and performing lengthy freeze-thaw tests.
10. A test protocol was developed to accurately assess the probable field performance of any given

aggregate as a function of its minerology and probable environmental exposure. The battery of tests includes petrographic examination, the original Washington hydraulic fracture test, VPI single-cycle slow-freeze test and the rapid freezing and thawing test using procedure B and salt-treated aggregates.

11. Reducing the coarse aggregate top size seems to be effective in reducing the potential for concrete freeze-thaw damage.
12. The study found that blending of durable and nondurable aggregate particles without reducing the top size of the nondurable material is somewhat effective in reducing potential concrete freeze-thaw damage.
13. Blending of durable and nondurable aggregate particles with a reduction of the top size of the unsound aggregates effectively reduces freeze-thaw damage potential, especially for porous aggregate sources.
14. Silane treatment of coarse aggregate does not effectively reduce freeze-thaw damage potential, especially for porous aggregate sources.
15. Reductions in water-cement ratio significantly improved the frost resistance of concrete prepared using nondurable coarse aggregate.
16. The addition of silica fume to the mix (as a partial substitute for Portland cement) did not significantly improve concrete freeze-thaw damage potential (as measured using ASTM C 666 Procedure B). However, the results of the VPI single-cycle slow-freeze test did improve signficantly.

6.2 Conclusions

Based on the findings listed above and the analyses of the test data obtained in this investigation, the following conclusions can be drawn:

1. Poor durability performance of PCC pavement sections in southern Minnesota can, in many cases, be attributed to the susceptibility of coarse aggregates to freeze-thaw damage; however, secondary mineralization, embedded shale deposits, poor mix design and alkali-aggregate reactions were also found. These other problems can aggravate D-cracking or appear similar to it. Petrographic examination of cores can help to differentiate between these different failure mechanisms.
2. The original Washington hydraulic fracture test (WHFT) and VPI single-cycle slow freeze test are

recommended as screening tests of coarse aggregate freeze-thaw resistance. Rapid freezing and thawing tests (ASTM C 666 procedure B) with salt-treated aggregates should be used to determine the frost susceptibility of coarse aggregates not assessable by the WHFT and the VPI single-cycle slow-freeze tests.

3. The elimination of the use of highly porous coarse aggregates in PCC concrete should be considered because it appears that this type of aggregate produces a failure at the aggregate-matrix boundary that is very difficult to mitigate.
4. A single quick, simple, economical and reliable method for identifying frost-susceptible aggregate particles was not found. Freeze-thaw durability is not a fundamental property of coarse aggregate and a single rapid test is unlikely to provide a sound basis for accepting or rejecting aggregates with respect to their frost susceptibility. However, quick tests can be used to determine the frost resistance of one aggregate relative to several others. Whether an aggregate can resist repeated cycles of freezing and thawing can sometimes be answered only by tests that simulate field exposure conditions (such as ASTM C 666).
5. Tests that provided the best correlation with field performance included a modification of ASTM C 666 procedure B (specimens prepared using salt-treated aggregates), the VPI single-cycle slow-freeze test, and the Washington hydraulic fracture test. Other test procedures were correlated with field performance to lesser extents.
6. A test protocol was developed to assess the frost resistance of Minnesota concrete aggregates. The tests included in this protocol (petrographic examination, a modification of the ASTM C 666 procedure B (to use specimens prepared using salt-treated aggregates), the VPI single-cycle slow-freeze test and the original Washington hydraulic fracture test) were selected for use on the basis of minerology and composition, as well as the results of quick screening tests.
7. The test protocol, in its current form, is not yet ready for adoption as a tool for consistently and reliably predicting the frost resistance of coarse aggregates. Additional aggregate sources should be evaluated using this protocol and other tests (absoption, specific gravity, Iowa pore index test and acid insoluble residue), which correlated well with field performance to establish and validate the acceptance/rejection criteria used in the protocol. It is believed, however, that this battery of tests can eventually be successfully developed into a reliable procedure for accurately assessing the

freeze-thaw durability of many types of concrete aggregate.

8. The mitigation methods that showed the best improvement to the frost resistance of concrete containing frost-susceptible aggregates included the mix proportioning method using a reduced water-cement ratio (0.40), reduction of the top size of nondurable aggregates, and blending of durable materials with nondurable materials with a reduction of the top size of the nondurable aggregates.

6.3 Recommendations

On the basis of the results of this study, the following section highlights some recommendations drawn from this study to provide some direction for future research. A specific work plan for future research is not provided because such a plan should be developed in the context of the results of this study and other ongoing studies.

1. It is believed that a study in a similar vein but broader in scope is needed as the next step for designing optimal procedures for evaluating the frost resistance of coarse aggregates and the freeze-thaw durability of Portland cement concrete. A wider range of aggregate samples, in terms of both composition and grain size, should be employed. The test matrix should include petrographic examination, determination of aggregate absorption capacity and specific gravity, rapid freezing and thawing tests (i.e., ASTM C 666 procedures B using salt-treated aggregates and procedure C), the VPI single-cycle slow-freeze test, the Washington hydraulic fracture test (especially the improved version described in reference 60), the Iowa pore index test and the acid insoluble residue test.

2. Highway agencies and DOTs should make use of the results of this study and the results of other studies related to assessing the frost resistance of coarse aggregates to complete the development of acceptance/rejection criteria by conducting frost resistance tests which correlate with field performance on additional aggregate sources.
3. This study indicates that the mechanism of freeze-thaw failure in concrete beams containing porous aggregates probably originates in the transition zone between aggregate and paste (caused by the

expulsion of water from the saturated aggregate during freezing to the surrounding cement matrix), not within the aggregate particle. Therefore, similar studies should be performed for porous carbonates to determine the mechanism of failure in each.

4. A standard petrographic examination should be included as the first step in any evaluation of aggregate freeze-thaw durability. Identification of aggregate composition and origin provides a rational basis for the selection of subsequent durability tests. For example, concrete aggregates containing significant quantities of shale particles might be directed to the rapid freezing and thawing test (ASTM C 666) or some other durability screening test, but not to the hydraulic fracture test, the absorption test or the Iowa pore index test, which fail to predict the nondurable performance of shale particles. Similarly, carbonates containing some highly porous materials should be evaluated by rapid freezing and thawing test (ASTM C 666).
5. The effects of deicing salts on carbonate and other types of rocks during freezing and thawing should be investigated. Future studies should also be conducted to investigate the different mechanisms that contribute to the deterioration of aggregates or concrete due to freezing and thawing.
6. The mitigation study should be pursued further using a wider range of concrete aggregates. The mitigation methods should include the mix proportioning method using a wider range of water-cement ratios, reduction of the top size of nondurable aggregates, and blending of durable materials with nondurable materials with a reduction of the top size of nondurable aggregates.
7. Mn/DOT should make use of this study's results and the results of other studies concerning the mitigation techniques that might result in use of nondurable aggregates without compromising the quality and durability of PCC concrete pavements.

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APPENDIX A

RESULTS OF PETROGRAPHIC EXAMINATIONS

Quarry A, aggregate source #155011
Osmundson, Grand Meadow: T103N, R14W, Sec. 9

Highway 56, 1973
from 35.5 to 36.0, increasing
observed performance: fair

Cedar Valley formation
Upper Solon member
biolithic dololutite
Folk's texture: dolobiomicrosparite

Brown (10%) to tan (20%) to grey (40%) to buff (30%), reflecting a corresponding decrease in iron content. All aggregate particles composed of fine to very fine grained (60 µm>) euhedral dolomite microsparite martrix with coarse grained calcite sparite filled biolithic fragments: brachiopod shell fragments (2 mm>), and crinoid stem segments (0.5 mm>), and occasional calcite sparite filled intercrystal porosity.

Comments:
Brown particles commonly exhibit a rusty stain which penetrates into the paste. The iron associated staining is unremarkable, except in that it demonstrates the extent to which aggregate reacts chemically with the paste. A "radius effect" of chemical interaction is clearly defined by the staining, and is within a range of 1 to 5 millimeters. Smaller grey shale particles (3 mm>) frequently exhibit a dark reaction rim extending into the aggregate, and are thoroughly cracked.

1994 quarry sample
thermogravimetric analysis:

82.5%	13.7%	3.75%
dolomite	calcite	insoluble

Cedar Valley formation
Upper and Lower Solon members
bioithic dololutite
Folk Texture: dolobiomicrosparite

(90%) Yellow to buff aggregate particles composed of fine to very fine grained (50 µm >) dolomite microsparite with biolithic fragments frequently expressed as dissolved molds. **(10%)** Grey aggregate particles composed of fine to very fine grained (~50 µm) mottled subhedral dolomite microsparite matrix with coarse grained calcite sparite filled biolithic fragments: brachipod shell fragments (mm scale), horn coral fragments (cm scale), and crinoid stem segments (mm>).

Comparison/Comments

The yellow and buff particles are more consistant with descriptions of the Lower Solon Member in that they contain fossil molds, leaving empty cavities in the rock. The grey particles are more consistant with descriptions of the Upper Solon Member in that they contain calcite sparite replaced biolithic fragments (Kohls, 1961). In spite of their lithological differences, both the highway core and the 1994 quarry sample consist of very fine grained dolomite microsparite, and both performed poorly, which is consistant with the "fine grained bad" "coarse grained good" generalization.

References:
Kohls, Donald W., 1961, *Lithostratigraphy of the Cedar Valley Formation in Minnesota and Northern Iowa*, a thesis submitted to the faculty of the Graduate School of the University of Minnesota.

Quarry B, aggregate source #155037
Goldberg, Rochester: T108N, R14W, Sec. 36

Highway 42, 1966
from 0.0 to 3.4, increasing
observed performance: fair

Prairie du Chein group
Shakopee formation
quartzose dolarenite
Folk's Texture: sandy dolintrasparite

(80%) Yellow to light grey to buff aggregate particles composed of inequigranular fine to coarse grained (25 - 500 µm) subhedral dolomite sparite with vuggy porosity, void spaces frequently filled with coarse grained calcite sparite, sometimes with massive (cm scale) calcite crystals. **(20%)** Yellow to light grey to buff aggregate particles composed of fine to medium grained subhedral dolomite matrix with coarse to fine grained (1 mm >) well rounded spherical quartz sand and fine grained dolomite pseudosparite intraclasts (2 mm>).

Comments:
Frequent secondary mineralization in entrained air voids smaller than 0.15 mm.

Highway 42, 1966
from 3.4 to 4.0, increasing
observed performance: poor

Prairie du Chein group
Shakopee formation
biolithic dolarenite
Folk's Texture: dolobiosparite

(70%) Yellow to grey to buff aggregate particles composed of inequigranular fine to medium grained (25 - 300 µm) subhedral dolomite sparite with vugs, occasionally calcite sparite filled, and with bioliths of crescent shaped brachiopod shell fragments (4 mm >) and donut shaped crinoid stem segments (0.5 mm>) filled with coarse grained calcite sparite. **(25%)** Yellow to grey to buff aggregate particles composed of inequigranular fine to medium grained (25 - 300 µm) subhedral dolomite sparite with dolomite pseudosparite intraclasts (2 mm >), and with fine to coarse grained (100 - 500 µm) subangular to well rounded quartz sand. **(5%)** Yellow to light grey to buff aggregate particles composed of a fine to medium grained subhedral dolomite matrix with coarse to fine grained (1 mm>) well rounded spherical quartz sand.

Comments:
Moderate D-cracking present near pavement joints.

1994 quarry sample
thermograv. analysis:
89.7% dolomite
0.4% calcite
9.93% insoluble

Prairie du Chein group
Shakopee formation
dolarenite
Folk's Texture: dolosparite

(95%) Light grey to buff aggregate particles composed of inequigranular fine to coarse grained (25 - 500 µm) subhedral to euhedral dolomite sparite/rhombs with occasional vugs and intercrystal porosity. **(5%)** Light grey to buff to light green aggregate particles composed of fine grained quartz sand in a dolomite matrix, with some glauconite present.

Comparison/Comments

The contrast in performance between the two highway cores is not easily attributable to differences within the coarse aggregate. Although the aggregates from the two sections are lithologically distinct (one core has biolithic fragments and the other does not), they both posses the same characteristic texture of an inequigranular subhedral assortment of fine-to-coarse-grained dolomite sparite. However, both sections exhibited frequent secondary mineralization in the entrained air voids. A mechanism of PCC deterioration proposed by Marks and Dubberke involves the saturation of ettringite-filled entrained air voids with NaCl brine, resulting in expansion followed by dissolution. The secondary minerals of the highway 42 cores look similar in thin section to the EDS-identified ettringite-filled air voids of the Shakopee highway 212 core. The possibility of ettringite-related deterioration in the highway 42 sections should be investigated further.

References:
Marks, V.J. and W.G. Dubberke. "Investigation of PCC Pavement Deterioration - A Few Facts are Worth More Than 100 Opinions." Interim Report: HR-337. Office of Materials Division, Iowa Department of Transportation. Ames, Iowa. 1995.

Quarries C and K, aggregate source #179036
Hammond: T109N, R13W, Sec. 32

Highway 14, 1967
from 212.8 to 137.7, direction: I
observed performance: good

Prairie du Chein group
Shakopee formation
dolarenite
Folk's Texture: dolosparite

(95%) Grey to buff aggregate particles composed of inequigranular fine to coarse grained (50 - 600 µm) anhedral to subhedral dolomite sparite matrix with occasional rounded quartz grains (500 µm>), and occasional vugs (2 mm>) sometimes filled with massive (mm scale) calcite crystals. (**5%**) Buff colored aggregate particles composed of fine to medium grained (50 - 300 µm) anhedral to subhedral dolomite sparite with medium to fine grained (500 µm >) well-rounded quartz sand, and millimeter scale intraclasts composed of very fine grained dolomite pseudosparite.

comments:
3/4"- particles originated from aggregate source #155037, Goldberg, Rochester.

1994 quarry sample (C)
DOT pass
thermogravimetric analysis:

99.3%	0.0%	0.7%
dolomite	calcite	insoluble

Prairie du Chein group
Shakopee formation
dolarenite
Folk's Texture: dolosparite

(90%) Grey colored aggregate particles composed of inequigranular fine to coarse grained (50 - 500 µm) anhedral dolomite sparite, with occasional vugs. **(10%)** Grey colored aggregate particles composed of fine to medium grained (50 - 150 µm) anhedral dolomite sparite with occasional chert nodules.

1994 quarry sample (K)
DOT fail
thermogravimetric analysis

98.9%	0.0%	1.12%
dolomite	calcite	insoluble

Prairie du Chein group
Shakopee formation
dolarenite
Folk's Texture: dolosparite

(80%) Grey colored inequigranular fine to coarse grained (50 - 500 µm) subhedral dolomite sparite with occasional vugs (1mm>). **(20%)** Yellow to grey colored fine to medium grained (30 - 150 µm) subhedral dolomite sparite with occasional vugs (0.5mm>).

comments:
Occasional hematite nodules. Sample K is classified "DOT fail" because a ledge containing absorptive aggregate was mixed in with the "DOT pass" aggregate.

Comparison/Comments

The Hammond highway core and the freeze/thaw test beams all performed well. Hammond aggregates possess variable grain sizes, from fine- to coarse-grained, all within the same particle. However, the grain size tends to be primarily medium-to-coarse with fine-grained crystals intermixed. The good performance correlates with the "fine-grained bad, coarse-grained good" generalization.

Quarry D, aggregate source #185007
Winona: T106N, R07W, Sec. 16

Interstate 90, 1971
from 249.4 to 266.5, decreasing
observed performance: good

Top of Jordan Sandstone
dolarenite
Folk's Texture: dolosparite

(98%) Light grey to buff to white aggregate particles composed of medium grained (50 - 250 µm) subhedral to euhedral dolomite sparite/rhombs with occasional vuggy (2 mm - 30 mm) or intercrystal porosity. Infrequent (2%) massive (cm scale) coarse grained calcite inclusions with molds of biolithic fragments.

1994 quarry sample
thermogravimetric analysis:

--%	--%	--%
dolomite	calcite	insoluble

Top of Jordan sandstone
dolarenite
Folk's Texture: dolosparite

Light grey to buff to white aggregate particles composed of medium to coarse grained (50 - 500 µm) euhedral dolomite sparite/rhombs with occasional vugs, intercrystal porosity, and massive (cm scale) calcite inclusions. Occasional (rare) hematite nodules.

Comparison/Comments:

The Winona aggregate performed very well historically and in the 1994 durability testing, a good example of the "fine-grained bad, coarse-grained good" generalization.

Quarry E, aggregate source #182002
Shiely, Grey Cloud, St. Paul Park: T027N, R22W, Sec. 26

White Bear Lake Avenue, 19--
from HW 694 to 61 (co. rd. E), southbound
observed performance: good to fair

Prairie du Chein group
Shakopee formation
dololutite and dolarenite
Folk's Texture: sandy dolointrapel(pseudo)sparite

Yellow aggregate particles composed primarily of fine grained (25 - 60 µm) subhedral dolomite sparite intermixed with medium to coarse grained (100 - 500 µm) patches of dolomite sparite, and patches of very fine grained dolomite pseudosparite. Aggregate particles have a wide variety of allochems: vugs, ooids (with well preserved radial structure) (0.5 mm>), peloids (0.2 mm>), subangular to well rounded spherical medium grained (1 mm>) quartz sand, and/or fine grained dolomite intraclasts (7 mm>).

Comments:
Minor amounts of secondary mineralization in entrained air voids.

1994 quarry sample
thermogravimetric analysis:

90.9%	0.8%	8.24%
dolomite	calcite	insoluble

Prairie du Chein group
Shakopee formation
dololutite and dolarenite
Folk's Texture: dolo(micro)sparite

(50%) Yellow aggregate particles composed of fine to very fine grained (25 µm >) euhedral dolomite microsparite to pseudosparite with occasional medium to fine grained (500 µm >) subangular to well-rounded quartz sand.

(50%) Yellow aggregate particles composed of inequigranular fine to coarse grained (25 - 500 µm) anhedral dolomite sparite, occasionally with medium to fine grained (500 µm >) subangular to well-rounded quartz sand.

Comparison/Comments

The increase in the proportion of fine to very fine grained dolomite aggregate particles in the 1994 quarry sample may be related to the relatively poor performance of the freeze/thaw test beams.

Quarry F, aggregate source #167001
Northern Concrete, Luverne: T103N, R44W, Sec. 31

Interstate 90, ~1967
rest area, MP 24, increasing
observed performance: fair to poor

Quaternary alluvium
River gravel including (30%) biolithic dololutite (dolobiomicrosparite) and (70%) Precambrian rocks.

Major Precambrian representatives:
(49%) Medium to coarse grained pink granitic to gneissic rocks containing (in decreasing order of abundance) plagioclase, K-feldspar, quartz, biotite, hornblende.
(17%) Medium to coarse grained grey gneissic to granitic rocks containing primarily plagioclase, and varying amounts of quartz, pyroxene, biotite, hornblende, K-feldspar, and garnet.
(4%) Fine to medium grained mafic rocks containing plagioclase and varying amounts of hornblende, biotite, and pyroxene.

Major carbonate representatives:
(12%) Yellow to buff to white aggregate particles composed of very fine grained (5 µm>) dolomite micropseudosparite with occasional manganese dendrites.
(8%) Yellow to buff aggregate particles composed of very fine to fine grained (1 - 70 µm) dolomite micropseudosparite with bioliths filled with coarse grained calcite sparite, or molds of brachiopod shell fragments (2 mm>) and crinoid stem segments (0.5 mm>).

1994 quarry sample
thermogravimetric analysis:

41.9%	11.9%	46.2%
dolomite	calcite	insoluble

Quaternary alluvium
River gravel including 80% dololutite (dolopseudosparite) and 20% Precambrian igneous/metamorphic rocks.

Major Precambrian representatives:
(10%) Fine to medium grained mafic rocks containing plagioclase and varying amounts of hornblende, biotite, and pyroxene.
(5%) Medium to coarse grained pink granitic to gneissic rocks containing (in decreasing order of abundance) plagioclase, K-feldspar, quartz, biotite, hornblende.
(5%) Medium to coarse grained grey gneissic to granitic rocks containing primarily plagioclase, and varying amounts of quartz, pyroxene, biotite, hornblende, K-feldspar, and garnet.

The carbonate representative:
(80%) Yellow to buff to white aggregate particles composed of very fine grained (10 µm >) dolomite micropseudosparite with occasional vugs (2 mm>) and frequent manganese dendrites.

Comments:
Carbonate rocks account for most (~90%) of the 3/4"+ aggregate particles, and less than half of the 3/4"- aggregate particles.

Comparison/Comments

Carbonate fraction of the 1967 highway core accounted for 30% of the total coarse aggregate, while the carbonate fraction of the 1994 quarry sample accounted for 80% of the total coarse aggregate. Minnesota Department of Transportation specifications allow for a maximum of 30% carbonate particles to be present in river gravels. Although the 1967 core met these specifications, it still performed poorly. The carbonate particles from the Luverne pit consist of very fine grained calcitic dolomite, a recognized poor aggregate performer (Kosmatka and Panarese, 1992). The large difference in carbonate percentage between the 1967 core and the 1994 sample could also be reflected in the relatively poor performance of the Luverne quarry sample freeze/thaw test beams.

References:
Kosmatka, S.H., and W.C. Panarese. *Design and Control of Concrete Mixtures*, 13th edition. Portland Cement Association. Skokie, Illinois. 1992.

Weiblin, Paul W. Fieldtrip Guidebook for the Precambrian Terrain of the Minnesota River Valley. Minnesota Geological Society. St. Paul, Minnesota.

Quarry G, aggregate source #170006
Bryan Rock, Shakopee: T115N, R22W, Sec. --

Highway 212, 1974
from 137.8 to 140.7, decreasing
observed performance: poor

Prairie du Chein group
Shakopee formation
oolitic quartzose dolarenite
Folk's Texture: sandy doloointrasparite

(50%) Pink aggregate particles composed of medium grained (100 - 400 µm) euhedral dolomite sparite/rhombs (occasionally zoned) with occasional vugs and intercrystal porosity. **(15%)** Pink to yellow aggregate particles composed of very fine grained dolomite pseudosparite with occasional manganese dendrites and fine grained (100 µm >) subangular spherical quartz sand. **(35%)** Pink to yellow aggregate particles composed of a very fine grained dolomite pseudosparite matrix with fine grained (25 - 50 µm) euhedral dolomite rhombs, ooids (0.5 mm>), mouldic ooid porosity, fine grained dolomite pseudosparite intraclasts (8 mm>), and well rounded to subangular fine to medium grained (500 µm >) quartz sand.

Comments:
Secondary mineralization very common in entrained air voids smaller than 100 µm.

1994 quarry sample
thermogravimetric analysis

96.4%	0.0%	3.63%
dolomite	calcite	insoluble

Prairie du Chein group
Shakopee formation
oolitic dolarenite
Folk's Texture: doloointrasparite

(55%) Pink aggregate particles composed of medium grained (100 - 400 µm) euhedral dolomite sparite/rhombs (occasionally zoned or with a mottled or stained appearance) with occasional vugs, intercrystal porosity, and patches of very fine grained dolomite pseudosparite. **(10%)** Pink aggregate particles composed of very fine grained dolomite pseudosparite. **(35%)** Pink aggregate particles composed of a very fine grained dolomite pseudosparite matrix with fine grained (25 - 50 µm) euhedral dolomite rhombs, ooids (0.5 mm>), mouldic ooid porosity, fine grained dolomite pseudosparite intraclasts (8 mm>), and well rounded to subangular fine to medium grained (500 µm >) quartz sand.

Comparison/Comments

The Shakopee aggregates contain a relatively high percentage of very fine grained dolomite pseudosparite, which may be related to the poor performance of both the highway section and the freeze/thaw test beams. Of further interest is the abundance of secondary mineralization in the entrained air voids in the highway core. Since entrained air plays a major role in the durability of concrete pavements, the reduction or obstruction of these voids by mineral growth may have a detrimental effect. Furthermore, the growth of expansive secondary minerals may also affect concrete durability. A study conducted by Dubberke and Marks at Iowa State University and at the Iowa Department of Transportation has documented the growth of ettringite $(CaO)\cdot(Al_2O_3)\cdot3(SO_3)\cdot32(H_2O)$ in PCC entrained air voids. Furthermore, a PCC deterioration mechanism has been proposed, due to the resultant expansion followed by dissolution of ettringite when exposed to NaCl brine. The Shakopee highway core was found tp contain ettringite in the air void system.

References:
Marks, V.J. and W.G. Dubberke. "Investigation of PCC Pavement Deterioration - A Few Facts are Worth More Than 100 Opinions." Interim Report: HR-337. Office of Materials Division, Iowa Department of Transportation. Ames, Iowa. 1995.

Quarry H, aggregate source #135001
Forester, Halma: T160N, R46W, Sec. 17

Highway 175, 1970

from 0.0 to 10.02, increasing

observed performance: poor

Quaternary lake washed till

Glacial Lake Agassiz shoreline washed glacial till composed of ~60% calcitic dololutite (dolosparite), and ~40% igneous/metamorphic rocks.

Igneous/metamorphic representative:

(40%) A varied assortment of rocks from the Canadian cambrian shield.

The carbonate representative:

(60%) Yellow to buff to white aggregate particles composed of either very fine grained (12 μm >) dolomite microsparite, or fine grained (~50 μm) euhedral dolomite sparite/rhombs, both with occasional manganese dendrites.

Comments:

Occasional thoroughly cracked dark grey shale aggregate particles (5 mm>) with dark reaction rims.

1994 quarry sample

thermogravimetric analysis:

37.1%	30.5%	36.40%
dolomite	calcite	insoluble

Quaternary lake washed till

Glacial Lake Agassiz shoreline washed glacial till composed of ~60% calcitic dololutite (dolosparite), and ~40% igneous/metamorphic rocks.

Igneous/metamorphic representative:

(40%) Aggregate particles consist of a varied assortment of rocks from the Canadian cambrian shield.

The carbonate representative:

(60%) Yellow to buff to white aggregate particles composed of medium to fine grained (100 - 200 μm) dolomite microsparite or sparite/rhombs with occasional millimeter scale biolith fragments filled with coarse grained sparry calcite, and occasional manganese dendrites.

Comparison/Comments

The Forester/Halma and Northern Concrete/Luverne pit both have poor performance records. Both quarries consist of reworked Quaternary glacial till and, not surprisingly, contain a wide variety of rock types. As a result, pavements made from these sources contain a heterogeneous conglomerate of pebbles with varying strengths and compositions.

A study conducted by Dr. Catherine French and Roxanne Kriesel at the University of Minnesota (1995) documents the unexpected poor performance of high-strength concrete beams cast with river gravel, partially crushed river gravel, or crushed granite, when subjected to freeze/thaw testing (ASTM 666), while high-strength concrete beams cast with quarried dolomite performed considerably better. After freeze/thaw testing, the beams were cut and polished for linear traverse and microscopic analysis. Examination revealed that the high-strength aggregates (river gravels and granites) exhibited a greater incidence of cracking at the aggregate/paste interface as compared to the dolomitic aggregates. It was theorized that the cracking patterns were related to the differences in strength between the aggregate and the cement paste. When concrete cylinders containing stronger aggregates are put into compression, cracks form primarily at the aggregate/paste interface and through the cement paste matrix, leaving the aggregate intact. When concrete cylinders containing weaker aggregates are put into compression, cracks form through the cement paste and through the coarse aggregate. A concrete made with relatively weak aggregate, such as dolomite, may have a more homogeneous structure, resulting in more uniform distribution of internal stresses. Concrete made with strong aggregate, such as river gravel or granite, often has a more heterogeneous structure, resulting in more nonuniform distribution of stress and greater build-up of pressure at the aggregate/paste interfaces. Perhaps a mechanism of greater pressure at the aggregate/paste interface is responsible for the premature deterioration of the Halma and Luverne pavements.

References:

French, C.W. and R.C. Kriesel. *Durability of High Performance Concrete.* University of Minnesota. Minneapolis, Minnesota. 1995.

Quarry I, aggregate source #193017
Harris, Northwood Iowa: T100N, R20W, Sec. 29

Highway 13, 1972
from 1.35 to 3.02, decreasing
observed performance: good

Shellrock Formation
biolithic dolarenite
Folk's Texture: dolobiosparite

(50%) Dark brown, brown, to tan aggregate particles consisting of fine to medium grained (50 - 450 µm) euhedral dolomite sparite/rhombs with moderate intercrystal/vug porosity, occasionally filled or partially filled with coarse grained calcite sparite. **(35%)** Brown to tan aggregate particles consisting of fine to medium grained (50 - 400 µm) euhedral dolomite sparite/rhombs with crescent shaped biolithic fragments (1 mm>) replaced with coarse grained calcite sparite. **(15%)** Grey to dark grey aggregate particles consisting of fine to medium grained (50 - 200 µm) euhedral dolomite sparite/rhombs, occasionally with dissolved coarse grained (1 mm>) dolomite rhombs.

Comments:
Grey to dark grey dolomite particles frequently exhibit a dark grey reaction rim extending into the aggregate. Grey to dark grey shale particles (5 mm>) are present and thoroughly cracked. D-cracking present in ~10% of the aggregate particles.

1994 quarry sample
thermogravimeteric analysis:

58.7%	37.3%	4.05%
dolomite	calcite	insoluble

Shellrock Formation
dololutite and calcitic dolarenite
Folk's Texture: dolopseudosparite and calcitic dolosparite

(60%) Grey to white aggregate particles composed of very fine grained (10 µm >) dolomite pseudosparite matrix with centimeter to millimeter scale angular intraclasts composed of very fine grained (1 µm >) dolomite pseudosparite. **(40%)** Tan aggregate particles composed of fine to medium grained (50 - 400 µm) dolomite sparite with coarse grained calcite sparite filled vugs, intercrystal void spaces and biolithic fragments.

Comparison/Comments

The 1994 quarry aggregate sample differs significantly from the 1972 highway core aggregate in that it consists primarily (60%) of dolopseudosparite, whereas the highway core consists completely of fine-to-medium-grained dolomite sparite/rhombs. The change in quarry texture from 1972 to 1994 reflects the wide variety of sedimentary and groundwater conditions often found in many quarries.

Although the highway pavement had exhibited good durability to date, the 1994 Harris quarry sample performed poorly in freeze-thaw testing when compared to its other good-performing peers (i.e., Winona, Hammond and Ulland/Glenville). The abundance of dolopseudosparite in the 1994 sample could be a source of the poor performance of the freeze-thaw beams. A study by Schorholz and Bergeson at Iowa State University relates dolomite crystallite size to durability, with concrete pavements containing fine-grained crystallite dolomite aggregate tending to perform more poorly.

References:
Schlorholtz, S., and K.L. Bergeson. "Investigation of Rapid Thermal Analysis Procedures for Prediction of the Service Life of PCCP Carbonate Coarse Aggregate." *Final Report: HR-337*. Iowa Department of Transportation. Ames, Iowa. 1993.

Quarry J, aggregate source #193018
Ulland, Glenville: T099N, R20W, Sec. 10

1994 quarry sample

thermogravimetric analysis:

82.5%	13.7%	3.75%
dolomite	calcite	insoluble

Cedar Valley Formation

dololutite and dolarenite

Folk's Texture: dolosparite

Dark to light grey to tan aggregate particles composed of fine-to-medium-grained (50 - 200 µm) euhedral dolomite sparite/rhombs with occasional patches of medium grained (100 - 500 µm) of subhedral calcite sparite.

Comparison/Comments

The Glenville quarry sample consists of fine-to-medium-grained dolomite sparite rhombs (similar to the Shellrock formation), whereas the Grand Meadow quarry sample consists of very fine grained dolomite microsparite (consistant with descriptions of the Cedar Valley formation). It has been proposed by Kohls (1961) that outcrops of the Shellrock formation could exist within the southeastern section of Freeborn County, which seems to be the case for the Cedar Valley-classified Glenville quarry. No highway cores were drilled containing Glenville aggregate.

References:

Kohls, Donald W., 1961, *Lithostratigraphy of the Cedar Valley Formation in Minnesota and Northern Iowa.* A thesis submitted to the faculty of the Graduate School of the University of Minnesota.

Quarry L, aggregate source #125009
Goodhue, Zumbrota: T110N, R15W, Sec. 34

Highway 52, 1961

from 76.5 to 79.5, decreasing

observed performance: good to fair

Prairie du Chein group
Shakopee formation
quartzose oolitic dolarenite
Folk's Texture: sandy doloointrasparite

(75%) Yellow to grey aggregate particles composed of fine to coarse grained (30 - 500 µm) euhedral dolomite sparite/rhombs (occasionally zoned) with occasional vugs and frequent medium to fine grained (500 µm >) well-rounded to subangular quartz sand, and high intercrystal porosity. **(25%)** Yellow to grey aggregate particles composed of a very fine grained (15 µm >) euhedral dolomite microsparite matrix with patches of medium grained (50 - 300 µm) subhedral dolomite sparite and frequent medium to fine grained (500 µm >) well-rounded to subangular quartz sand. Leisegang banding present in some aggregate particles.

Comments:
Occasional oolitic aggregate particles, frequently represented by dissolved ooid molds. Leisegang banding present in some aggregate particles.

1994 quarry sample

thermogravimetric analysis:

71.6%	5.3%	23.1%
dolomite	calcite	insoluble

Prairie du Chein group
Shakopee formation
quartzose dolarenite
Folk Texture: sandy dolosparite

(50%) Yellow to grey aggregate particles composed of fine to medium grained (30 - 300 µm) subhedral to anhedral dolomite sparite with occasional medium to fine grained (500 µm >) well-rounded quartz sand. **(50%)** Yellow to grey aggregate particles composed of very fine grained (20 µm >) euhedral dolomite microsparite with frequent medium to fine grained (500 µm >) well-rounded quartz sand.

Comments:
The 1994 quarry sample was not taken from aggregate source #125009. Aggregate source #125009 is inactive, so aggregate was taken from a pile in a nearby quarry.

Comparison/Comments

Although the highway section performed well, the 1994 quarry sample performed poorly on a number of aggregate durability tests. The contrast in performance may be related to the larger proportion of very fine-grained dolomite microsparite in the 1994 quarry sample. The contrast may also be due to the increased quantity of insolubles (clay, silt, and very fine-grained quartz) in the 1994 quarry sample.

Quarry M, aggregate source #155051
Quarve Anderson, Stewartville: T105N, R14W, Sec. 5

Interstate 90, 1971
from 220.74 to 249.44, increasing
observed performance: fair

Stewartville formation
dolarenite
Folk's Texture: dolosparite

Grey to buff aggregate particles composed of inequigranular fine to coarse grained (30 - 500 µm) anhedral to subhedral dolomite sparite with vugs (2mm>), occasionally filled with coarse grained calcite sparite, and with occasional intercrystal porosity, sometimes filled with coarse grained calcite sparite intergrowth.

comments:
Infrequent (rare) white chert nodules and peloidal aggregate particles. 3/4"- aggregate particles originated from aggregate source #155037, Goldberg, Rochester.

County Road 7, 1972
from 0.0 to 0.6, direction: I
observed performance: poor

Stewartville formation
biolithic dolarenite
Folk's Texture: dolobiosparite

(~75%) Grey to buff aggregate particles composed of fine to medium grained (25 - 200 µm) inequigranular subhedral to euhedral dolomite sparite with coarse grained calcite sparite filled biolithic fragments consisting of crescent shaped brachiopod shell fragments (2 mm>) and donut shaped crinoid stem segments (0.5 mm>). **(~25%)** Grey to buff aggregate particles composed of fine to medium grained (25 - 250 µm) tightly packed dolomite rhombs in a matrix of very fine grained (5 µm >) calcite micro/pseudosparite, with coarse grained calcite sparite filled biolithic fragments of brachiopods and crinoid stems.

1994 Quarry Sample
thermogravimetric analysis:

73.7%	23.2%	3.08%
dolomite	calcite	insoluble

Stewartville formation
dolarenite
Folk's Texture: dolobiosparite

(~70%) Yellow to grey aggregate composed of fine to medium grained (50 - 150 µm) euhedral dolomite sparite/rhombs, with occasional intercrystal/vuggy porosity, commonly filled with coarse grained calcite sparite intergrowths. **(~30%)** Grey aggregate particles composed of fine to medium grained (25 - 200 µm) dolomite rhombs and coarse grained calcite sparite filled biolithic fragments (brachiopod shell fragments and crinoid stem segments) in a calcite pseudosparite matrix.

Comparison/Comments

The relatively poor performance of county road 7 may be related to the presence of aggregate particles containing tightly packed fine-to-medium-grained dolomite rhombs within a very fine-grained grained calcite pseudosparite matrix, an aggregate characteristic not shared with the Interstate 90 highway core. The 1994 quarry sample aggregate is similar to the county road 7 highway core aggregate in texture and in biolithic content.

Quarry N, aggregate source #193016
Kuennen's Park, Northwood Iowa: T099N, R20W, Sec. 31

Interstate 90, 1964
from 166.22 to 172.40, decreasing
observed performance: poor

Shellrock Formation
dolarenite
Folk's Texture: dolosparite

(70%) Dark brown, brown, tan, to buff aggregate particles consisting of fine to medium grained (40 - 500 µm) euhedral dolomite sparite/rhombs, with occasional intercrystal porosity and vugs (1 mm>) some filled with large calcite crystals. **(30%)** Grey to dark grey aggregate particles consisting of fine grained (10 - 100 µm) euhedral dolomite sparite/rhombs with occasional euhedral coarse grained calcite sparite replaced coarse grained (1 mm>) dolomite rhombs.

Comments:
Grey to dark grey dolomite particles frequently exhibit a dark reaction rim extending into the aggregate. Grey to dark grey shale particles (5 mm>) frequently exhibit dark reaction rims and are thoroughly cracked.

from 172.40 to 175.77, decreasing
observed performance: good

Shellrock Formation
dolarenite
Folk's Texture: dolosparite

(55%) Dark brown, brown to tan aggregate particles consisting of fine to medium grained (15 - 150 µm) euhedral dolomite sparite/rhombs, sometimes vuggy, and with occasional large calcite crystals (5 mm>). **(40%)** Light grey to buff aggregate particles consisting of medium grained (50 - 500 µm) euhedral dolomite sparite/rhombs, with high intercrystal to vuggy porosity and occasional calcite intergrowths (2.5 mm>). **(5%)** Dark grey to blue grey aggregate particles composed of fine grained (10 - 100 µm) euhedral dolomite sparite/rhombs.

Comments:
Moderate D-cracking in ~5% of the 3/4"- particles. Pronounced brownish tinge to cement paste. Small (5 mm>) grey shale particles are present and thoroughly cracked.

thermogravimetric analysis:

77.1%	19.3%	3.6%
dolomite	calcite	insoluble

Shellrock formation
dolarenite
Folk's texture: dolosparite

Dark brown, brown, to tan to grey aggregate particles composed of fine to medium (60 - 200 µm) grained euhedral dolomite sparite/rhombs which are occasionally mottled and stained in appearance, occasional fenestral vugs (mm scale in length).

Comments:
Kuennen's pit is now a recreational area. The quarried pits are filled with water. Only two bags were filled with aggregate for the 1994 sample, and were taken from the side of the road leading into the park.

Interstate 90, 1964 **1994 quarry sample**

Comparison/Comments

The 1964 highway cores are from the same stretch of Interstate 90, have identical mix designs, contain aggregate from the same source, and yet they performed differently. However, there is a recognizable difference within the coarse aggregate of the two cores. The core from the poor-performing pavement contains a larger quantity of grey to dark grey coarse dolomite aggregate composed of fine-grained dolosparite (approximately 30%) than the core from the better-performing pavement (only 5%). Furthermore, the grey to dark grey dolomite aggregate in the poor-performing pavement core frequently exhibited a dark reaction rim extending into the aggregate. Similar in appearance, but smaller in scale, the poor-performing pavement core also exhibited a higher occurrence of thoroughly cracked grey to dark grey shale particles (5mm>) with dark reaction rims. Reactions of this type are commonly attributed to alkali-aggregate reactions. The Portland Cement Association *Design and Control of Concrete Mixtures* handbook lists "extremely fine grained dolomitic limestones with large amounts of calcite, clay, silt, or dolomite rhombs found in a matrix of clay and fine calcite" as especially alkali-carbonate reactive, an aggregate description which could be applied to many of the quarries included in this study.

APPENDIX B

TABULATED TEST RESULTS

Table B.1. Results of rapid freezing and thawing tests (ASTM C666 Procedure B) on laboratory specimens.

	Durability Factor (RDM Failure)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	77	80	90	82
B: Rochester	97	95	98	97
C: Hammond (DOT Pass)	99	98	99	99
D: Wilson, Winona	101	100	102	101
E: St. Paul Park (Shiely)	96	95	95	95
F: Luverne	99	97	97	98
G: Bryan Rock, Shakoopee	97	97	91	95
H: Forester, Halma	98	90	97	95
I: Harris	98	97	97	97
J: Glenville	100	100	100	100
K: Hammond (DOT Fail)	100	102	102	101
L2: Goodhue, Zumbrota	82	84	75	80
M: Stewartville	101	*(a)*	*(a)*	101
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

	Percent Dilation (%)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	0.083	0.057	0.037	0.059
B: Rochester	0.003	0.002	0.006	0.004
C: Hammond (DOT Pass)	-0.014	-0.003	-0.005	-0.007
D: Wilson, Winona	-0.015	-0.027	-0.012	-0.018
E: St. Paul Park (Shiely)	0.007	0.006	0.008	0.007
F: Luverne	-0.025	0.066	0.004	0.015
G: Bryan Rock, Shakoopee	0.047	0.095	0.093	0.078
H: Forester, Halma	0.009	0.009	0.005	0.008
I: Harris	0.006	0.012	0.007	0.008
J: Glenville	0.003	0.005	0.001	0.003
K: Hammond (DOT Fail)	0.003	-0.001	-0.005	-0.001
L2: Goodhue, Zumbrota	0.055	0.041	0.083	0.060
M: Stewartville	0.001	*(a)*	*(a)*	0.001
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

Table B.2. Results of rapid freezing and thawing tests (ASTM C666 Procedure C) on laboratory specimens.

	Durability Factor (RDM Failure)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	70	75	81	75
B: Rochester	97	97	98	97
C: Hammond (DOT Pass)	101	97	96	98
D: Wilson, Winona	100	98	99	99
E: St. Paul Park (Shiely)	92	89	89	90
F: Luverne	88	94	99	94
G: Bryan Rock, Shakoopee	70	79	53	68
H: Forester, Halma	91	93	88	91
I: Harris	92	84	90	89
J: Glenville	99	99	99	99
K: Hammond (DOT Fail)	100	99	100	100
L2: Goodhue, Zumbrota	53	64	88	68
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

	Percent Dilation (%)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	0.083	0.057	0.037	0.059
B: Rochester	0.003	0.002	0.006	0.004
C: Hammond (DOT Pass)	-0.014	-0.003	-0.005	-0.007
D: Wilson, Winona	-0.015	-0.027	-0.012	-0.018
E: St. Paul Park (Shiely)	0.007	0.006	0.008	0.007
F: Luverne	0.025	0.066	0.004	0.032
G: Bryan Rock, Shakoopee	0.047	0.095	0.093	0.078
H: Forester, Halma	0.009	0.009	0.005	0.008
I: Harris	0.006	0.012	0.007	0.008
J: Glenville	0.003	0.005	0.001	0.003
K: Hammond (DOT Fail)	0.003	-0.001	-0.005	-0.001
L2: Goodhue, Zumbrota	0.055	0.041	0.083	0.060
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

Table B.3. Results of rapid freezing and thawing test (ASTM C666 Procedure B using salt-treated aggregates) on laboratory specimens.

	Durability Factor (RDM Failure)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	36	23	37	32
B: Rochester	98	99	99	98
C: Hammond (DOT Pass)	100	99	98	99
D: Wilson, Winona	103	104	103	103
E: St. Paul Park (Shiely)	91	80	93	88
F: Luverne	60	89	89	79
G: Bryan Rock, Shakoopee	21	18	26	22
H: Forester, Halma	82	97	73	84
I: Harris	91	86	91	90
J: Glenville	98	99	98	99
K: Hammond (DOT Fail)	99	99	99	99
L2: Goodhue, Zumbrota	94	24	57	58
M: Stewartville	98	*(a)*	*(a)*	98
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

	Percent Dilation (%)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	0.083	0.057	0.037	0.059
B: Rochester	0.003	0.002	0.006	0.004
C: Hammond (DOT Pass)	-0.014	-0.003	-0.005	-0.007
D: Wilson, Winona	-0.015	-0.027	-0.012	-0.018
E: St. Paul Park (Shiely)	0.007	0.006	0.008	0.007
F: Luverne	-0.025	0.066	0.004	0.015
G: Bryan Rock, Shakoopee	0.047	0.095	0.093	0.078
H: Forester, Halma	0.009	0.009	0.005	0.008
I: Harris	0.006	0.012	0.007	0.008
J: Glenville	0.003	0.005	0.001	0.003
K: Hammond (DOT Fail)	0.003	-0.001	-0.005	-0.001
L2: Goodhue, Zumbrota	0.055	0.041	0.083	0.060
M: Stewartville	0.009	*(a)*	*(a)*	0.009
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

Table B.4. VPI single-cycle slow freeze test results.

Project	Temperature slope (μm/ºC)			
	Specimen 1	Specimen 2	Specimen 3	Average
A: Grand Meadows	-0.104	-0.120	-0.600	-0.275
B: Rochester	0.143	0.000	0.292	0.145
C: Hammond (DOT Pass)	0.000	-0.165	0.211	0.015
D: Wilson, Winona	-0.130	0.265	0.027	0.054
E: St. Paul Park (Shiely)	-0.096	0.079	-0.945	-0.202
F: Luverne	-0.280	0.414	-0.032	0.034
G: Bryan Rock, Shakoopee	0.480	0.339	0.287	0.369
H: Forester, Halma	-0.150	-0.050	-0.051	-0.084
I: Harris	0.055	0.360	0.130	0.182
J: Glenville	0.172	0.129	0.000	0.100
K: Hammond (DOT Fail)	0.269	*(a)*	0.128	0.199
L2: Goodhue, Zumbrota	-0.078	0.113	-0.106	-0.024
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

*: a fourth beam was included

(a) : Quarry closed; sample did not permit completion of full battery of tests

Project	Time slope (μm/hour)			
	Specimen 1	Specimen 2	Specimen 3	Average
A: Grand Meadows	-8.400	-3.600	-13.200	-8.400
B: Rochester	0.000	0.000	0.000	0.000
C: Hammond (DOT Pass)	0.000	-6.000	0.000	-2.000
D: Wilson, Winona	-3.000	0.000	-3.000	-2.000
E: St. Paul Park (Shiely)	-3.000	-3.000	-11.657	-4.414
F: Luverne	-6.000	-3.000	-3.661	-4.220
G: Bryan Rock, Shakoopee	2.400	0.000	0.000	0.800
H: Forester, Halma	-3.000	-3.000	-3.000	-3.000
I: Harris	-3.000	0.000	-3.000	-2.000
J: Glenville	-3.000	0.000	0.000	-1.000
K: Hammond (DOT Fail)	3.000		0.000	1.500
L2: Goodhue, Zumbrota	-6.000	0.000	-3.000	-3.000
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

*: a fourth beam was included

(a) : Quarry closed; sample did not permit completion of full battery of tests

Table B.5. Specific gravity test results.

	Specific Gravity				
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**	**Average**
A: Grand Meadows	2.515	2.521	2.526	2.511	2.518
B: Rochester	2.674	2.670	2.671	2.673	2.672
C: Hammond (DOT Pass)	2.705	2.706	2.702	2.697	2.703
D: Wilson, Winona	2.655	2.644	2.650	2.647	2.649
E: St. Paul Park (Shiely)	2.696	2.698	2.699	2.688	2.695
F: Luverne	2.624	2.627	2.630	2.618	2.625
G: Bryan Rock, Shakoopee	2.582	2.571	2.577	2.573	2.576
H: Forester, Halma	2.652	2.642	2.646	2.641	2.645
I: Harris	2.670	2.665	2.666	2.661	2.666
J: Glenville	2.727	2.733	2.734	2.720	2.729
K: Hammond (DOT Fail)	2.692	2.684	2.687	2.680	2.686
L2: Goodhue, Zumbrota	2.584	2.589	2.593	2.585	2.588
M: Stewartville	*(a)*	(a)	(a)	(a)	
N: Kuennens	2.740	2.740	2.738	2.741	2.740

(a) : Quarry Closed; small sample did not permit testing of a gradation sample;
specific gravity was obtained from testing the size fractions.

Table B.6. Absorption capacity test results.

	Absorption (Percent)				
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**	**Average**
A: Grand Meadows	3.190	2.831	3.080	3.012	3.028
B: Rochester	1.290	1.301	1.297	1.294	1.296
C: Hammond (DOT Pass)	1.177	1.096	1.140	1.091	1.126
D: Wilson, Winona	1.607	1.581	1.609	1.621	1.605
E: St. Paul Park (Shiely)	1.397	1.431	1.386	1.610	1.456
F: Luverne	1.606	1.503	1.463	1.614	1.547
G: Bryan Rock, Shakoopee	2.726	2.727	2.727	2.824	2.751
H: Forester, Halma	0.892	0.999	0.970	1.011	0.968
I: Harris	1.354	1.365	1.348	1.385	1.363
J: Glenville	2.800	2.805	2.805	2.790	2.800
K: Hammond (DOT Fail)	1.411	1.447	1.332	1.499	1.422
L2: Goodhue, Zumbrota	2.291	2.271	2.251	2.136	2.237
M: Stewartville	*(a)*	(a)	(a)	(a)	
N: Kuennens	0.790	0.755	0.880	0.782	0.802

(a) : Quarry Closed; small sample did not permit testing of a gradation sample;
specific gravity was obtained from testing the size fractions.

Table B.7. PCA adsorption test results.

Aggregate Source	Sample 1	Sample 2	Sample 3	Average
A: Grand Meadows	0.515	0.692	0.539	0.582
B: Rochester	0.343	0.307	0.326	0.325
C: Hammond (DOT Pass)	0.436	0.435	0.505	0.459
D: Wilson, Winona	0.349	0.376	0.351	0.359
E: St. Paul Park (Shiely)	0.811	0.972	0.902	0.895
F: Luverne	0.824	0.721	0.721	0.755
G: Bryan Rock, Shakoopee	0.259	0.270	0.270	0.266
H: Forester, Halma	0.318	0.325	0.302	0.315
I: Harris	0.221	0.291	0.320	0.277
J: Glenville	0.358	0.452	0.369	0.393
K: Hammond (DOT Fail)	0.574	0.607	0.633	0.605
L2: Goodhue, Zumbrota	0.242	0.215	0.227	0.228
M: Stewartville	0.237	0.254	0.281	0.257
N: Kuennens	0.530	0.551	0.478	0.520
O: Halma, Carbonate only	0.636	0.517	0.589	0.581
P: Luverne, Carbonate only	1.036	1.264	0.730	1.010

Table B.8. PCA absorption test results.

Aggregate Source	Sample 1	Sample 2	Sample 3	Sample 4	Average
A: Grand Meadows	4.862	4.966	4.985	*(a)*	4.938
B: Rochester	2.128	2.599	2.364	*(a)*	2.364
C: Hammond (DOT Pass)	0.897	0.919	1.294	*(a)*	1.037
D: Wilson, Winona	1.686	1.942	1.582	*(a)*	1.737
E: St. Paul Park (Shiely)	1.589	1.514	1.641	*(a)*	1.581
F: Luverne	1.600	1.643	1.295	*(a)*	1.513
G: Bryan Rock, Shakoopee	2.302	2.159	2.538	*(a)*	2.333
H: Forester, Halma	1.840	1.691	1.555	*(a)*	1.695
I: Harris	1.296	1.170	1.442	1.242	1.288
J: Glenville	0.789	1.072	0.795	1.067	0.931
K: Hammond (DOT Fail)	1.337	1.345	1.460	*(a)*	1.381
L2: Goodhue, Zumbrota	2.183	1.823	2.234	*(a)*	2.080
M: Stewartville	3.100	3.489	3.310	*(a)*	3.300
N: Kuennens	0.673	0.807	0.821	*(a)*	0.767
O: Halma, Carbonate only	3.286	2.911	*(b)*	*(a)*	3.099
P: Luverne, Carbonate only	3.529	3.703	*(b)*	*(a)*	3.616

(a) An additional sample was used for sources I and J only.
(b) : Only limited testing was performed on the carbonate fraction samples of gravel sources

Table B.9. Results of the acid insoluble residue test (total residue).

	Total Acid Insoluble Residue (Percent)			
Aggregate Source	Sample 1	Sample 2	Sample 3	Average
A: Grand Meadows	5.52	3.98	6.11	5.20
B: Rochester	8.43	10.04	11.30	9.92
C: Hammond (DOT Pass)	3.70	8.16	6.68	6.18
D: Wilson, Winona	6.48	9.98	6.35	7.60
E: St. Paul Park (Shiely)	9.07	8.85	9.82	9.25
F: Luverne	42.51	45.39	48.21	45.37
G: Bryan Rock, Shakoopee	11.37	15.00	12.29	12.89
H: Forester, Halma	52.56	58.38	55.51	55.48
I: Harris	5.69	5.52	5.10	5.44
J: Glenville	5.74	5.41	4.67	5.27
K: Hammond (DOT Fail)	6.48	9.98	6.35	7.60
L2: Goodhue, Zumbrota	27.62	23.80	28.34	26.59
M: Stewartville	*(a)*	(a)	(a)	
N: Kuennens	(a)	(a)	(a)	

(a) : Quarry Closed; small sample did not permit testing of a gradation sample

Table B.10. Results of the acid insoluble residue test (silt and clay residue).

	Silt and Clay Residue (Percent)			
Aggregate Source	Sample 1	Sample 2	Sample 3	Average
A: Grand Meadows	3.75	3.74	4.87	4.12
B: Rochester	2.59	5.68	6.07	4.78
C: Hammond (DOT Pass)	2.43	2.62	4.76	3.27
D: Wilson, Winona	2.93	5.19	1.84	3.32
E: St. Paul Park (Shiely)	4.46	4.12	4.79	4.46
F: Luverne	2.94	2.45	5.30	3.56
G: Bryan Rock, Shakoopee	6.79	9.15	6.44	7.46
H: Forester, Halma	5.24	2.59	3.19	3.67
I: Harris	5.39	5.10	4.83	5.11
J: Glenville	5.27	5.11	4.15	4.84
K: Hammond (DOT Fail)	2.93	5.19	1.84	3.32
L2: Goodhue, Zumbrota	10.86	6.89	10.87	9.54
M: Stewartville	*(a)*	(a)	(a)	
N: Kuennens	(a)	(a)	(a)	

(a) : Quarry Closed; small sample did not permit testing of a gradation sample

Table B.11. Primary load (mm) from the Iowa pore index test.

Aggregate Source	Sample 1	Sample 2	Sample 3	Sample 4	Average
A: Grand Meadows	157.2	144.0	159.2	*(a)*	153.5
B: Rochester	72.5	76.6	74.0	*77.1*	75.0
C: Hammond (DOT Pass)	61.3	52.7	54.2	*52.2*	55.1
D: Wilson, Winona	74.5	70.0	76.1	*(a)*	73.5
E: St. Paul Park (Shiely)	58.3	55.3	53.2	*(a)*	55.6
F: Luverne	51.2	42.1	45.6	*(a)*	46.3
G: Bryan Rock, Shakoopee	81.1	73.0	92.3	94.8	85.3
H: Forester, Halma	37.0	24.3	32.4	*(a)*	31.3
I: Harris	55.8	58.3	54.2	53.2	55.4
J: Glenville	30.9	25.4	21.3	*(a)*	25.9
K: Hammond (DOT Fail)	61.9	50.7	49.2	*(a)*	53.9
L2: Goodhue, Zumbrota	107.5	96.3	110.5	*(a)*	104.8
M: Stewartville	99.4	105.5	114.1	*(a)*	106.3
N: Kuennens	34.0	27.4	*(b)*	*(a)*	30.7
O: Halma, Carbonate only	60.8	*(c)*	*(c)*	*(c)*	60.8
P: Luverne, Carbonate only	86.7	*(c)*	*(c)*	*(c)*	86.7

(a) An additional sample was used for sources B, C, G and I only.

(b) : Quarry closed; sample did not permit completion of full battery of tests

(c) : Only limited testing was performed on the carbonate fraction samples of gravel sources

Table B.12. Secondary load (mm) from the Iowa pore index test.

Aggregate Source	Sample 1	Sample 2	Sample 3	Sample 4	Average
A: Grand Meadows	37.5	37.5	39.5	*(a)*	38.2
B: Rochester	24.3	23.8	21.8	25.4	23.8
C: Hammond (DOT Pass)	21.3	22.3	21.8	22.8	22.1
D: Wilson, Winona	18.8	18.3	18.3	*(a)*	18.4
E: St. Paul Park (Shiely)	22.8	22.8	22.3	*(a)*	22.6
F: Luverne	24.3	19.3	19.8	*(a)*	21.1
G: Bryan Rock, Shakoopee	60.8	57.8	54.8	56.8	57.6
H: Forester, Halma	16.7	20.8	18.8	*(a)*	18.8
I: Harris	28.9	25.4	20.8	20.8	24.0
J: Glenville	18.3	17.7	15.7	*(a)*	17.2
K: Hammond (DOT Fail)	18.8	20.8	20.3	*(a)*	20.0
L2: Goodhue, Zumbrota	30.4	26.9	28.9	*(a)*	28.7
M: Stewartville	25.4	20.3	21.8	*(a)*	22.5
N: Kuennens	22.8	20.3	*(b)*	*(a)*	21.6
O: Halma, Carbonate only	38.5	*(c)*	*(c)*	*(c)*	38.5
P: Luverne, Carbonate only	30.9	*(c)*	*(c)*	*(c)*	30.9

(a) An additional sample was used for sources B, C, G and I only.

(b) : Quarry closed; sample did not permit completion of full battery of tests

(c) : Only limited testing was performed on the carbonate fraction samples of gravel sources

Table B.13. Quality number from the Iowa pore index test.

Aggregate Source	Sample 1	Sample 2	Sample 3	Sample 4	Average
A: Grand Meadows	2.56	2.60	2.72	*(a)*	2.62
B: Rochester	1.79	1.72	1.55	1.85	1.72
C: Hammond (DOT Pass)	1.58	1.75	1.68	1.80	1.70
D: Wilson, Winona	1.29	1.27	1.24	*(a)*	1.27
E: St. Paul Park (Shiely)	1.75	1.77	1.74	*(a)*	1.75
F: Luverne	1.97	1.54	1.56	*(a)*	1.69
G: Bryan Rock, Shakoopee	5.86	5.70	4.80	4.99	5.34
H: Forester, Halma	1.34	2.12	1.63	*(a)*	1.70
I: Harris	2.41	2.00	1.58	1.59	1.90
J: Glenville	1.56	1.66	1.50	*(a)*	1.58
K: Hammond (DOT Fail)	1.34	1.61	1.58	*(a)*	1.51
L2: Goodhue, Zumbrota	2.15	1.89	2.01	*(a)*	2.01
M: Stewartville	1.75	1.33	1.43	*(a)*	1.50
N: Kuennens	2.10	1.94	*(b)*	*(a)*	2.02
O: Halma, Carbonate only	3.46	*(c)*	*(c)*	*(c)*	3.46
P: Luverne, Carbonate only	2.31	*(c)*	*(c)*	*(c)*	2.31

(a) An additional sample was used for sources B, C, G and I only.

(b) : Quarry closed; sample did not permit completion of full battery of tests

(c) : Only limited testing was performed on the carbonate fraction samples of gravel sources

Table B.14. Results of the Washington hydraulic fracture test.

	Initial Mass in grams				Initial Particle Count			
Aggregate Source	Sample 1	Sample 2	Sample 3	Total	Sample 1	Sample 2	Sample 3	Total
A: Grand Meadows	2990.8	2756.3	2599.7	8346.8	137	145	160	442
B: Rochester	3365.2	3279.6	3258.5	9903.3	188	172	171	531
C: Hammond (DOT Pass)	3385	3358.2	2950.7	9693.9	144	149	120	413
D: Wilson, Winona	3064.9	3166.2	3143	9374.1	113	158	131	402
E: St. Paul Park (Shiely)	2863.6	*(a)*	*(a)*	2863.6	592	*(a)*	*(a)*	592
F: Luverne	3263.2	3185.4	2975.7	9424.3	145	130	130	405
G: Bryan Rock, Shakoopee	3162.5	2938.8	2764.4	8865.7	108	97	101	306
H: Forester, Halma	3372.2	3089.9	2945.8	9407.9	93	90	109	292
I: Harris	3183.9	2960.5	3090	9234.4	203	200	211	614
J: Glenville	3399.5	2832.3	2896.8	9128.6	168	164	160	492
K: Hammond (DOT Fail)	3456.7	3376.3	2818.9	9651.9	286	280	229	795
L2: Goodhue, Zumbrota	3168	3030.6	3047.8	9246.4	166	165	156	487
M: Stewartville	3109.6	3064	2945.9	9119.5	265	265	260	790
N: Kuennens	3293.5	3133.7	3290.3	9717.5	131	150	131	412
O: Halma, Carbonate only	3038.7	3076.6	3186	9301.3	137	107	121	365
P: Luverne, Carbonate only	3045.4	3132.8	3160.2	9338.4	134	133	154	421

(a): Test not performed because aggregate particle larger than 4.95 mm were not available

	Final Mass (After 50 Cycles), grams				Final Particle Count			
Aggregate Source	Sample 1	Sample 2	Sample 3	Total	Sample 1	Sample 2	Sample 3	Total
A: Grand Meadows	2973.1	2743.6	2585.6	8302.3	148	145	160	453
B: Rochester	3338.9	3267.2	3226	9832.1	196	174	174	544
C: Hammond (DOT Pass)	3367.7	3345.3	2936.8	9649.8	155	149	120	424
D: Wilson, Winona	3051.8	3151.3	3119.1	9322.2	114	160	138	412
E: St. Paul Park (Shiely)	2842	*(a)*	*(a)*	2842	595	*(a)*	*(a)*	595
F: Luverne	3244.7	3170.4	2964.3	9379.4	157	132	136	425
G: Bryan Rock, Shakoopee	3133.5	2926.3	2750	8809.8	116	102	101	319
H: Forester, Halma	3360	3085.6	2932.1	9377.7	94	92	111	297
I: Harris	3172.2	2946.1	3076	9194.3	204	200	212	616
J: Glenville	3384.2	2822.6	2887.4	9094.2	170	164	161	495
K: Hammond (DOT Fail)	3444	3364.7	2805.2	9613.9	292	283	230	805
L2: Goodhue, Zumbrota	3121.9	2974.4	3013	9109.3	178	183	162	523
M: Stewartville	3097.3	3052.3	2932.5	9082.1	265	267	259	791
N: Kuennens	3288.2	3131.7	3281.1	9701	132	150	133	415
O: Halma, Carbonate only	3033	3070.2	3174.5	9277.7	137	109	127	373
P: Luverne, Carbonate only	3015.5	3103.3	3139.1	9257.9	140	137	163	440

(a): Test not performed because aggregate particle larger than 4.95 mm were not available

Table B.15. Results of compressive strength tests of laboratory specimens.

	Compressive Strength (kPA)			
Aggregate Source	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
A: Grand Meadows	50,807	49,057	58,041	52,635
B: Rochester	53,260	53,391	*(b)*	53,325
C: Hammond (DOT Pass)	48,609	49,229	53,680	50,506
D: Wilson, Winona	44,799	45,088	46,859	45,582
E: St. Paul Park (Shiely)	52,771	54,755	56,877	54,801
F: Luverne	40,513	40,272	44,179	41,655
G: Bryan Rock, Shakoopee	45,398	50,214	47,038	47,550
H: Forester, Halma	42,132	41,588	42,684	42,135
I: Harris	53,032	53,177	51,647	52,619
J: Glenville	45,612	46,397	45,736	45,915
K: Hammond (DOT Fail)	46,604	50,077	50,573	49,084
L2: Goodhue, Zumbrota	45,570	46,156	44,902	45,543
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*
O: Halma, Carbonate only	*(b)*	*(b)*	*(b)*	*(b)*
P: Luverne, Carbonate only	*(b)*	*(b)*	*(b)*	*(b)*

(a) : Quarry closed; sample did not permit completion of full battery of tests

(b) : Only two samples were tested

(c) : Only limited testing was performed on the carbonate fraction samples of gravel sources

Table B.16. Results of compressive strength tests of cores.

Aggregate Source	Compressive Strength (kPA) Core 1	Core 2	Core 3	Average
A: Grand Meadows	6,513	6,356	7,184	6,684
B: Rochester	7,175	7,338	5,102	6,538
B: Rochester	7,494	7,986	8,113	7,865
C: Hammond (DOT Pass)	*(a)*	*(a)*	*(a)*	*(a)*
D: Wilson, Winona	*(a)*	*(a)*	*(a)*	*(a)*
E: St. Paul Park (Shiely)	5,138	6,955	7,101	6,398
F: Luverne	6,944	6,895	7,163	7,001
G: Bryan Rock, Shakoopee	9,504	8,749	8,811	9,021
H: Forester, Halma	6,475	7,087	6,241	6,601
I: Harris	5,807	6,504	6,684	6,332
L1: Goodhue, Zumbrota	*(a)*	*(a)*	*(a)*	*(a)*
M: Stewartville	*(a)*	*(a)*	*(a)*	*(a)*
M: Stewartville	5,608	4,398	5,166	5,057
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*
N: Kuennens	*(a)*	*(a)*	*(a)*	*(a)*

(a): Core contain reinforcement steel

Table B.17. Results of X-ray fluorescence (XRF) tests.

Aggregate Source	SRO	$MGCO_3$	FE_2O_3	S	TIO_2	MNO	SIO_2	$CACO_3$	K_2O	P_2O_5	AL_2O_3
A: Grand Meadows	0.013	27.47	0.622	0.043	0.02	0.05	1.48	69.76	0.17	0.036	0.325
B: Rochester	0.013	40.62	1.196	0.025	0.02	0.04	5.87	51.39	0.33	0.019	0.469
C: Hammond (DOT Pass)	0.018	42.21	1.511	0.061	0.02	0.08	2.48	53.23	0.17	0.015	0.213
D: Wilson, Winona	0.017	41.58	0.983	0.031	0.02	0.05	3.36	53.43	0.23	0.009	0.283
E: St. Paul Park (Shiely)	0.014	39.96	2.481	0.023	0.02	0.13	5.41	51.29	0.28	0.03	0.361
F: Luverne	0.044	25.25	2.337	0.012	0.1	0.06	22.5	44.78	1.08	0.063	3.78
G: Bryan Rock, Shakoopee	0.017	37.84	3.147	0.027	0.04	0.17	5.05	52.21	0.67	0.046	0.784
H: Forester, Halma	0.055	14.74	2.093	0.046	0.14	0.03	18.8	59.61	1.26	0.121	3.148
I: Harris	0.02	25.73	0.636	0.079	0.03	0.02	2.21	70.3	0.39	0.03	0.56
J: Glenville	0.025	36.44	0.797	0.148	0.03	0.04	2.2	59.56	0.28	0.016	0.47
K: Hammond (DOT Fail)	0.015	39.74	2.049	0.044	0.02	0.09	5.6	52.03	0.17	0.014	0.221
L2: Goodhue, Zumbrota	0.013	35.47	1.586	0.024	0.02	0.06	12.9	49.25	0.31	0.024	0.31
M: Stewartville	0.016	29.62	0.591	0.09	0.02	0.03	1.99	66.92	0.28	0.066	0.365
N: Kuennens	0.026	33.22	0.726	0.103	0.03	0.02	2.06	63.06	0.31	0.01	0.437
O: Halma, Carbonate only	0.017	28.83	0.655	0.045	0.02	0.02	1.66	68.22	0.21	0.035	0.28
P: Luverne, Carbonate only	0.024	32.25	0.603	0.033	0.02	0.03	2.45	64.13	0.2	0.034	0.244

Table B.18. Results of the thermogravimetric analysis (TGA) tests.

		Dolomite						Limestone						CO3		
Aggregate Source	**Sample Weight (mg)**	**Trans. Temp**	**Loss**	**Loss Adjusted (%)**	**Residue**	**Percent**	**Quality Number**	**Trans. Temp**	**Loss**	**Loss Adjusted (%)**	**Residue**	**Percent**	**Quality Number**	**PCT**	**Quality**	**Insol. Res., Calc. (%)**
A: Grand Meadows	55.61	740	0.0100	0.0153	84.4	65.4	8.5	909	0.0071	0.0225	54.4	31.5	9.0	96.9	8.7	3.14
B: Rochester	55.52	740	0.0224	0.0250	78.6	89.7	5.7	909	0.0064		56.3	0.4		90.1	5.7	9.93
C: Hammond (DOT Pass)	55.71	738	0.0724	0.0729	76.3	99.3	0.0	911	0.0108		52.9	0.0		99.3	0.0	0.70
D: Wilson, Winona	55.60	738	0.0532	0.0488	74.0	108.9	0.0	911	0.0091		51.1	0.0		108.9	0.0	-8.94
E: St. Paul Park (Shiely)	55.53	734	0.0386	0.0425	78.3	90.9	0.7	910	0.0149		55.5	0.8		91.8	0.7	8.24
F: Luverne	55.54	754	0.0091	0.0217	90.0	41.9	6.7	912	0.0075		74.4	11.9		53.8	6.7	46.15
G: Bryan Rock, Shakoopee	55.61	742	0.0223	0.0231	77.0	96.4	6.2	910	0.0122		54.6	0.0		96.4	6.2	3.63
H: Forester, Halma	55.67	747	0.0089	0.0269	92.1	33.1	5.2	910	0.0162	0.0250	70.5	30.5	10.0	63.6	7.5	36.40
I: Harris	55.56	744	0.0108	0.0184	86.0	58.7	7.6	910	0.0079	0.0212	55.1	37.3	8.5	96	7.9	4.05
J: Glenville	55.64	738	0.0187	0.0227	80.3	82.5	6.4	912	0.0099		53.9	13.7		96.2	6.4	3.75
K: Hammond (DOT Fail)	55.41	735	0.0727	0.0735	76.4	98.9	0.0	911	0.0124		54.2	0.0		98.9	0.0	1.12
L2: Goodhue, Zumbrota	55.97	732	0.0329	0.0459	82.9	71.6	0.0	912	0.0113		62.9	5.3		76.9	0.0	23.07
M: Stewartville	55.67	738	0.0126	0.0171	82.4	73.7	8.0	912	0.0070	0.0250	54.0	23.2		96.9	8.0	3.08
N: Kuennens	55.75	733	0.0227	0.0294	81.6	77.1	4.4	908	0.0109		54.1	19.3		96.3	4.4	3.65
O: Halma, Carbonate only	55.71	736	0.0156	0.0230	83.8	67.9	6.3	911	0.0085	0.0250	54.3	29.0		96.8	6.3	3.17
P: Luverne, Carbonate only	55.66	735	0.0188	0.0262	82.9	71.6	5.4	911	0.0089	0.0250	54.7	23.9		95	5.4	4.46

	Limestone						CO3		
Aggregate Source	**Trans. Temp**	**Loss**	**Loss Adjusted (%)**	**Residue**	**Percent**	**Quality Number**	**PCT**	**Quality**	**Insol. Res., Calc. (%)**
A: Grand Meadows	909	0.0071	0.0225	54.4	31.5	9.0	96.9	8.7	3.14
B: Rochester	909	0.0064		56.3	0.4		90.1	5.7	9.93
C: Hammond (DOT Pass)	911	0.0108		52.9	0.0		99.3	0.0	0.70
D: Wilson, Winona	911	0.0091		51.1	0.0		108.9	0.0	-8.94
E: St. Paul Park (Shiely)	910	0.0149		55.5	0.8		91.8	0.7	8.24
F: Luverne	912	0.0075		74.4	11.9		53.8	6.7	46.15
G: Bryan Rock, Shakoopee	910	0.0122		54.6	0.0		96.4	6.2	3.63
H: Forester, Halma	910	0.0162	0.0250	70.5	30.5	10.0	63.6	7.5	36.40
I: Harris	910	0.0079	0.0212	55.1	37.3	8.5	96	7.9	4.05
J: Glenville	912	0.0099		53.9	13.7		96.2	6.4	3.75
K: Hammond (DOT Fail)	911	0.0124		54.2	0.0		98.9	0.0	1.12
L2: Goodhue, Zumbrota	912	0.0113		62.9	5.3		76.9	0.0	23.07
M: Stewartville	912	0.0070	0.0250	54.0	23.2		96.9	8.0	3.08
N: Kuennens	908	0.0109		54.1	19.3		96.3	4.4	3.65
O: Halma, Carbonate only	911	0.0085	0.0250	54.3	29.0		96.8	6.3	3.17
P: Luverne, Carbonate only	911	0.0089	0.0250	54.7	23.9		95	5.4	4.46

Table B.19. Results of X-ray diffraction (XRD) analysis tests.

Aggregate Source	Dolomite	Zirconia	Limestone	K	Feldspar	Quartz
A: Grand Meadows	2.8910	3.17	3.0371			3.3497
B: Rochester	2.8888	3.17	3.0385	3.252		3.3475
C: Hammond (DOT Pass)	2.8896	3.17	3.0344	3.236		3.3489
D: Wilson, Winona	2.8891	3.17	3.0384	3.254		3.3494
E: St. Paul Park (Shiely)	2.8909	3.17	3.0329	3.253		3.3491
F: Luverne	2.8885	3.17	3.0352	3.254	3.198	3.3479
G: Bryan Rock, Shakoopee	2.8911	3.17	3.0329	3.244		3.3488
H: Forester, Halma	2.8906	3.17	3.0363	3.254	3.195	3.3490
I: Harris	2.8927	3.17	3.0373			3.3497
J: Glenville	2.9016	3.17	3.036	3.236		3.3485
K: Hammond (DOT Fail)	2.8905	3.17	3.0351			3.3477
L2: Goodhue, Zumbrota	2.8892	3.17	3.0322	3.246		3.3477
M: Stewartville	2.8911	3.17	3.0385			3.3425
N: Kuennens	2.9005	3.17	3.0347			3.3480
O: Halma, Carbonate only	2.8893	3.17	3.0363			3.3489
P: Luverne, Carbonate only	2.8884	3.17	3.0351			3.3482

Table B.20. Results of rapid freezing and thawing tests on cores.

	Durability Factor				Dilation (Percent)			
Aggregate Source	Core 1	Core 2	Core 3	Avg.	Core 1	Core 2	Core 3	Avg.
A: Grand Meadows	28.0	33.4	42.8	34.7	0.0678	0.1460	0.2020	0.1386
B: Rochester	67.9	73.7	79.1	73.6	0.0243	0.0736	0.0524	0.0501
B: Rochester	69.4	86.2	77.9	77.8	0.1129	0.0319	0.0505	0.0651
C: Hammond (DOT Pass)	85.2	76.8	79.3	80.5	0.0303	0.0528	0.0755	0.0529
D: Wilson, Winona	94.2	88.4	93.3	92.0	0.0792	0.0098	0.0697	0.0529
E: St. Paul Park (Shiely)	46.4	64.4	66.1	59.0	0.1680	0.1386	0.0958	0.1341
F: Luverne	28.4	57.9	18.8	35.0	0.0339	0.0904	0.1368	0.0870
G: Bryan Rock, Shakoopee	10.3	16.3	25.2	17.3	0.1193	0.0895	0.0985	0.1024
H: Forester, Halma	45.0	33.1	46.0	41.4	0.2579	0.1234	0.1592	0.1802
I: Harris	85.4	66.4	91.4	81.1	0.0196	0.0611	0.0430	0.0412
L1: Goodhue, Zumbrota	79.4	80.2	60.7	73.4	0.0310	0.0872	0.0775	0.0652
M: Stewartville	89.4	87.7	87.8	88.3	0.0388	0.0405	0.0398	0.0397
M: Stewartville	49.0	68.2	42.8	53.3	0.1668	0.0861	0.0795	0.1108
N: Kuennens	93.9	96.5	46.2	78.9	0.0033	0.0204	0.0025	0.0087
N: Kuennens	74.2	84.3	80.6	79.7	0.0649	0.0406	0.0361	0.0472

(a): Core contain reinforcement steel

Table B.21. Freeze-thaw test results for mixes comprising blended durable and nondurable aggregate.

	Average			Standard Deviation		
	DF	**Mass Change %**	**Dilation %**	**DF**	**Mass Change %**	**Dilation %**
BA: 10%	99.2	-0.30	0.0168	0.6913	0.0543	0.0045
BB: 20%	97.4	-0.18	0.0237	0.8677	0.0364	0.0059
BC: 30%	93.1	0.01	0.0425	3.2504	0.0487	0.0071
BD: 40%	81.5	0.08	0.0758	10.4511	0.1392	0.0132

Table B.22. Freeze-thaw test results for mixes comprising blends with reduced size (Mn/DOT practice) salt-treated aggregates.

	Average			Standard Deviation		
	DF	**Mass Change %**	**Dilation %**	**DF**	**Mass Change %**	**Dilation %**
AC	90.1	-0.16	0.0437	4.6943	0.1527	0.0124
FC	95.8	-0.12	0.0219	2.6460	0.0957	0.0107
HC	96.9	-0.10	0.0177	2.3706	0.0203	0.0037

Table B.23. Freeze-thaw test results for mixes with silica fume.

	Average			Standard Deviation		
	DF	**Mass loss %**	**Dilation %**	**DF**	**Mass loss %**	**Dilation %**
A	73.8	0.63	0.0383	8.9023	0.0227	0.0165
C	91.1	0.05	0.0018	1.2029	0.0314	0.0104
F	83.3	0.10	0.0254	3.7672	0.0789	0.0208
F2	77.0	0.21	0.0336	6.5096	0.0344	0.0092
G	44.9	0.32	0.0551	4.7136	0.0793	0.0487
H	86.5	-0.10	0.0273	9.0274	0.0862	0.0138
H2	86.9	0.20	0.0231	4.0359	0.0425	0.0097
I	91.2	0.03	0.0188	2.5951	0.0584	0.0038

Table B.24. Freeze-thaw test results for control mix.

	Average			Standard Deviation		
	DF	Mass loss %	Dilation %	DF	Mass loss %	Dilation %
A	58.3	0.25	0.0783	3.0963	0.0078	0.0080
C	97.8	0.06	0.0039	1.2439	0.0277	0.0031
F	85.2	0.10	0.0117	9.4577	0.0408	0.0052
G	66.3	0.04	0.0463	18.9050	0.1007	0.0202
H	88.5	0.02	0.0169	12.5004	0.0569	0.0045
I	96.4	-0.08	0.0051	1.4395	0.0707	0.0082

Table B.25. Freeze-thaw test results for control mixes with salt-treated aggregates.

	Average			Standard Deviation		
	DF	Mass loss %	Dilation %	DF	Mass loss %	Dilation %
A	74.6	0.59	0.0721	6.8	0.0722	0.0084
C	100.0	-0.21	0.0153	0.0	0.0308	0.0049
F	73.4	0.11	0.0523	18.0	0.2901	0.0172
G	57.0	-0.10	0.0665	23.4	0.3822	0.0246
H	83.9	0.05	0.0566	11.4	0.3555	0.0271
I	98.3	-0.05	0.0049	0.7	0.1359	0.0026

Table B.26. Freeze-thaw test results for reduced size aggregates (salt-treated).

	Average			Standard Deviation		
	DF	Mass loss %	Dilation %	DF	Mass loss %	Dilation %
A	58.9	0.51	0.1505	5.0	0.037	0.0167
F	83.8	0.04	0.0470	16.2	0.173	0.0231
H	92.2	-0.01	0.0233	0.9	0.074	0.0024
I	98.3	-0.16	0.0124	0.3	0.070	0.0014

Table B.27. Freeze-thaw results for control mixes with salt- and silane-treated aggregates.

	Average			Standard Deviation		
	DF	Mass loss %	Dilation %	DF	Mass loss %	Dilation %
A	69.4	0.45	0.0922	12.1	0.0176	0.0089
C2	99.0	-0.11	0.0109	1.7	0.0497	0.0027
F	69.0	0.42	0.0680	6.1	0.0909	0.0191
F2	91.4	0.16	0.0408	1.9	0.0662	0.0047
G	83.5	-0.31	0.0543	8.9	1.1763	0.0063
H	87.5	0.19	0.0332	8.0	0.0241	0.0071
I	93.6	-0.01	0.0180	3.2	0.0624	0.0032

Table B.28. Freeze-thaw test results for reduced water/cement ratio mixes (w/c = 0.40).

	Average			Standard Deviation		
	DF	Mass loss %	Dilation %	DF	Mass loss %	Dilation %
A	85.7	0.15	0.0364	1.9501	0.0461	0.0028
C	97.7	0.06	0.0007	0.2322	0.0195	0.0006
F	94.0	0.00	0.0119	3.2737	0.0355	0.0021
G	75.9	0.12	0.0347	14.0623	0.1251	0.0124
H	96.5	0.02	0.0039	1.1071	0.0867	0.0017
I	96.4	-0.05	-0.0008	1.4516	0.1584	0.0047