Concrete Performance Using Low-Degradation Aggregates

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CONCRETE PERFORMANCE USING LOW-DEGRADATION AGGREGATES

by

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16. ABSTRACT

The durability of Portland cement concrete (PCC) has long been identified as a concern by transportation communities around the United States. In this study, the long-term performance of two batches of concrete incorporating either low-degradation (LD) or normal (NM) aggregates subjected to freezing and thawing conditions was experimentally studied. The freezing and thawing test method (ASTM C666) was followed to condition all the test samples. Dynamic modulus and fracture energy for both groups of concrete samples after different numbers of freeze-thaw cycles were measured through nondestructive modal and cohesive fracture tests, respectively. Due to the higher air content in LD concrete, surface scaling was less severe under frost action. Test results showed that different aggregate degradation has an important effect on the freeze-thaw resistance of the concrete and the rate of decrease in fracture energy with number of freeze-thaw cycles for the LD concrete is higher than for the NM concrete. Compared to the dynamic modulus of elasticity, the fracture energy is a more sensitive parameter for evaluating concrete degradation caused by the frost action. Thus, the degradation of aggregate in concrete can be better evaluated by the cohesive fracture test. Additional research is needed to identify an appropriate threshold for when aggregate degradation properties become a concern. Until that time, it is recommended that WSDOT maintain their current restrictions on using low-degradation aggregates in concrete.

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EXECUTIVE SUMMARY

The durability of Portland cement concrete (PCC) has long been identified as a concern by transportation communities across the United States. Since quality gravel aggregate sources in Washington State are being used up and quarry sources are being used more, evaluating the potential to effectively use low-degradation aggregates in concrete could have significant economic benefits.

In this study, the long-term performance of two batches of concrete incorporating either low-degradation (LD) or normal (NM) aggregates subjected to freezing and thawing (F/T) conditions was experimentally studied. Both fresh and hardened material properties of these two groups of concrete were evaluated by ASTM standard tests. The Test Method for Resistance of Concrete to Rapid Freezing and Thawing (ASTM C666)-Procedure A was followed to condition all the test samples up to 1,500 F/T cycles. Dynamic modulus and fracture energy for both groups of concrete samples after different numbers of freeze-thaw cycles were measured through nondestructive modal and cohesive fracture tests, respectively.

Based on the experimental evaluation of surface scaling, natural transverse frequency, dynamic modulus, and fracture energy of samples prepared from the two batches concrete conducted in this study, the following conclusions are obtained: (1) Both the dynamic modulus and fracture energy of conditioned samples decrease with an increase in the number of F/T cycles, which reflects that frost damage and degradation in the concrete samples due to the freezing and thawing conditioning can be effectively accumulated (accelerated) by using the F/T conditioning protocol (ASTM C666). (2) Due to the higher air content in the concrete samples made with the LD aggregates, the surface

scaling following freeze-thaw conditioning was found to be less severe than that in those with the NM aggregates. Thus, surface scaling and loss of mass due to frost action significantly influences the dynamic modulus, and this type of frost damage is not able to characterize the degradation of aggregates. (3) The rate of decrease in the ratios of fracture energy as a function of the number of freeze-thaw cycles is greater in the LD concrete than those in the NM concrete. The difference in the rate of decrease indicates that different aggregate degradation properties have an effect on the freeze-thaw resistance of the concrete, and the fracture test can be an effective test method to evaluate the effects of aggregate degradation. (4) The greater rate of decrease in fracture energy with increasing freeze-thaw cycles obtained using the cohesive fracture test compared to the dynamic modulus test implies that the fracture energy is a more sensitive parameter for evaluating concrete degradation. (5) The greater rate of decrease with increasing freeze-thaw cycles in the fracture energy of LD concrete indicates that both different aggregates (LD vs. NM) and differences arising from preparing the concrete batches at different times (especially in air content values) have an important effect on the frost resistance of the concrete. (6) Compared to the dynamic modulus test data, surface scaling and air content seemed to have less influence on the fracture test data. Thus, the fracture test may be a more effective test method for examining degradation of aggregate and interfacial transition zones (ITZs) in the concrete. And (7), as demonstrated by the cohesive fracture test results, concrete with the LD aggregates (with a lower degradation value) degraded faster than concrete with the NM aggregates. Thus, degradation of aggregates is a contributing factor for overall concrete degradation.

Based on the experimental program conducted in this study, the following recommendations are made: (1) Based on the various test methods utilized in this project, the cohesive fracture test is an effective method to detect degradation in concrete (particularly in terms of its aggregate and ITZs), and it could be used to evaluate degradation properties for different sources of aggregates. (2) The dynamic modulus test is significantly influenced by surface scaling in the specimens, reflecting degradation at the specimen surface and a loss in mass. The degradation caused by the surface scaling in the dynamic modulus measurement may overshadow the effect of internal damage and aggregate degradation due to the freeze-thaw conditioning cycles. Thus, the dynamic modulus test may not be effective in characterizing the degradation of concrete with different aggregates. (4) Further correlations of material properties of concrete (fracture energy) with WSDOT Test Method T 113 Aggregate Degradation Factor data should be empirically established. (5) Since only two specific sources of aggregates were evaluated in this study (LD vs. NM with respective degradation values of 31 vs. 59), there is not the ability to confirm or suggest a new threshold degradation value for aggregates. It is recommended that further evaluations of concrete with aggregates from additional sources and with different degradation values be conducted to more rigorously establish a suitable threshold degradation value for aggregates. And (6) to reflect local or internal defects caused by the frost damage, additional tests capable of assessing and capturing local material properties are recommended to be performed.

Chapter 1 INTRODUCTION

Significant transportation infrastructure in North America is located in regions with severe environmental conditions, where alternate freezing and thawing can seriously affect the material and structural integrity. The durability of Portland cement concrete (PCC) has long been identified as a concern by transportation communities across the United States. Degradation of aggregates in concrete can be caused by erosion or fracture, and both cementitious materials and aggregates age over time. Extensive studies have been conducted on concrete mix designs and the composition of cementitious materials to improve the performance and durability of concrete. However, there is limited information in the literature on the long-term performance of concrete made with low-degradation aggregates. There is therefore a need to characterize the long-term performance of concrete considering the aggregate degradation process as good-quality gravel sources increasingly becoming exhausted.

1.1 Problem Statement

Aggregates are the major constituents of concrete and typically occupy between 60% to 80% of the concrete volume. As quality gravel aggregate sources in Washington State are being used up and quarry sources are being used more, evaluating the potential to effectively use low-degradation aggregates in concrete could have significant economic benefits. Degradation in aggregates can be caused by erosion or fracture. The erosion process produces a material of poorer quality compared to the parent aggregate. Degradation becomes more pronounced in marine basalt materials, especially for Eocene basalts found in the State of Washington (see Figure 1) (Minor, 1959). In marine basalt materials, aggregates can degrade into plastic fines. Failure of aggregates in a bituminous matrix may be caused by inferior mineral content and is manifested first by softening of the mixture followed soon afterward by actual disintegration of the matrix. Basalt rock of the Eocene age and gravels associated with the Eocene basalt are most apt to exhibit this harmful degradation. Minute portions of basaltic glass occur interstitially throughout the crystalline structure of the aggregate. This interstitial glass is responsible for harmful degradation of the Eocene basalts. Most of the interstitial basaltic glass is altered to magnesium-saturated montmorillonite clay. In other words, secondary alteration minerals from basaltic glass are the basic cause of the deterioration problems with Eocene basalt.

Figure 1.1. Typical Pavement Failure Primarily Caused by Degradation of Underlying Base Material (Minor, 1959).

The performance of aggregates is usually determined by the composition of minerals in the aggregate. Grinding of saturated aggregate or petrographic analysis can be used to determine whether the mineral composition will provide satisfactory durability and therefore whether the aggregate would be suitable for use in specific applications. Though aggregate degradation of marine basalt materials is well recognized and understood, the long-term performance of concrete made with low-degradation aggregates has not been investigated. Furthermore, the current WSDOT degradation requirements for aggregates in Portland cement concrete were largely adapted from those developed for asphalt (WSDOT Standard Specifications, 2008). Thus, there is a need to understand the long-term performance of concrete incorporating low-degradation aggregate and to develop appropriate requirements for the use of low-degradation aggregates for concrete applications as good-quality gravel sources are increasingly becoming exhausted.

1.2 Research Objectives

The goal of this study is to evaluate the durability and degradation of Portland cement concrete with low-degradation aggregates by using a combined accelerated conditioning and material characterization protocol. The research objectives of this project include:

1. To identify critical issues and mechanisms associated with the long-term deterioration of concrete and to recommend test methods for long-term performance characterization of concrete through a review of existing literature on the deterioration of concrete and the state-of-practice for using concrete incorporating low-degradation aggregates;

2. To understand the intergranular and intragranular deterioration and failure in concrete by conducting accelerated freeze/thaw (F/T) conditioning tests of

concrete samples with both normal and low-degradation aggregates;

3. To assess damage accumulation and identify failure mechanisms in the concrete samples caused by the accelerated F/T cyclic conditioning; and

4. To develop recommendations for test methods to characterize the long-term performance of concrete with low-degradation aggregates, specifications on performance criteria and service life predictions, and guidelines for DOTs to evaluate concrete performance and safety.

Chapter 2 LITERATURE REVIEW

This chapter presents a detailed overview of the work that has been done in the field of durability (primarily frost-related effect and damage) of concrete related to the current study. This literature review includes the mechanism of freezing and thawing damage in concrete, the main factors that influence the frost durability of concrete, identification of frost-susceptible aggregates, and test methods to evaluate the long-term performance of concrete subjected to frost damage.

2.1 Mechanism of freeze and thaw damage in concrete

Durability degradation due to cyclic freezing and thawing is one of the major damage aspects in cementitious materials and structures in cold regions. Since the 1930s, many theoretical and experimental studies have been conducted to characterize the frost resistance of concrete (Powers, 1945; Powers, 1949; Powers, 1955; Powers and Helmuth, 1953; Swenson, 1969; Beaudoin and MacInnis, 1974; Jacobsen et al., 1996; Cai and Liu, 1998; Coussy, 2005), and several studies recognized mechanisms that explained different forms of frost damage to concrete.

Powers (1949) indicated that harmful stresses could result from hydraulic pressure created by the volume increase when water transforms into ice. According to this theory, as the temperature decreases, ice is formed in the cement paste capillaries and, in order to accommodate the volume increase associated with ice formation, excess water is expelled from the freezing sites causing a hydraulic pressure. If the degree of saturation exceeds about 91%, ice formation with consequent increase in volume of about 9% will produce rupture, which may occur with very young concretes that still have very large voids and are still nearly saturated or in continuously soaked concretes of poor quality. If the matrix does not have enough tensile strength to resist the resulting pressure, cracks develop and the long-term performance of the structure is compromised. Under the effect of the concentration gradient, a water movement of the nonfrozen pores towards the sites of ice formation must take place in order to restore thermodynamic equilibrium between ice crystals and water. Due to this water movement, an osmotic pressure is also built up within the porosity (Powers and Helmuth, 1953).

Based on experimental observations, subsequent works by Powers and co-workers showed the limitations of the hydraulic model. According to Powers and Helmuth's study (Powers and Helmuth, 1953), if a given temperature below freezing is kept constant over a period of time, non-air-entrained pastes continue to expand while air-entrained pastes exhibit shrinkage. The hydraulic pressure model alone cannot explain these experimental observations and the authors thus proposed that the shrinkage was caused by the growth of ice in the air-voids resulting from the diffusion of liquid water from the matrix. Helmuth (1960) further criticized the hydraulic pressure model by pointing out that saturated flows do not occur in the process of freezing mature air-entrained pastes. Instead the accretion of ice in capillaries and air-voids are due to unsaturated flow resulting from suction and surface diffusion. Even though there are strong criticisms for the hydraulic pressure model (Chatterji, 2003), it is often used and quoted as it is the only major theory capable of providing an order of magnitude of the stress and of the critical spacing factor.

As revealed in Powers and Helmuth's study (Powers and Helmuth, 1953), water in cement paste is in the form of a weak alkali solution. When the temperature of the concrete drops below the freezing point, there is an initial period of super-cooling, after which ice crystals will form in the larger capillaries, leading to an increase in alkali content in the unfrozen portion of the solution in these capillaries, creating an osmotic potential that impels water in the nearby unfrozen pores to begin diffusing into the solution in the frozen cavities. The resulting dilution of the solution in contact with the ice allows further growth of the body of ice (ice-accretion). When the cavity becomes full of ice and solution, any further ice-accretion produces dilative pressure, which can cause the paste to fail. When water is being drawn out of unfrozen capillaries, the paste tends to shrink. As indicated by Powers and Helmuth (1953), when the paste contains entrained air and the average distance between air bubbles is not too great, the bubbles compete with the capillaries for the unfrozen water and normally win this competition. Many researchers now believe that stresses resulting from osmotic pressure cause most of the frost damage to cement paste.

According to Litvan's (1976) study, the water adsorbed on the surface or contained in the smaller pores cannot freeze due to the interaction between the surface and the water. Because of the difference in vapor pressure of this unfrozen and super-cooled liquid and the bulk ice in the surroundings of the paste system, there will be migration of water to locations where it is able to freeze, such as the larger pores or the outer surface. The process leads to partial desiccation of the paste and accumulation of ice in crevices and cracks. Water in this location freezes, prying the crack wider, and if the space fills with water in the next thaw portion of the cycle, further internal pressure and crack opening results. Failure occurs when the required redistribution of water cannot take place in an orderly fashion either because the amount of water is too large, that is, high water/cement ratio (w/cm) for the same level of saturation, the available time is too short (rapid cooling), or the path of migration is too long (lack of entrained air bubbles). In such cases the freezing forms a semi-amorphous solid (non-crystalline ice), resulting in great internal stresses. Additional stresses can be created by the non-uniform moisture distribution.

There is general agreement that cement paste of adequate strength and maturity can be made completely immune to damage from freezing by means of entrained air, barring unusual exposure conditions result in filling of the air voids. Air entrainment alone, however, does not preclude the possibility of damage of concrete due to freezing, because freezing in aggregate particles should also be taken into consideration.

2.2 Main factors influencing the frost durability of concrete

Four key factors that mainly govern the deleterious freezing and thawing behavior are the porosity/permeability, aggregate characteristics, moisture state and climatic conditions. In addition to the key factors discussed further in this section, it must be noted that good curing is also a prerequisite requirement for durable concrete (Page and Page 2007).

2.2.1 Porosity and permeability

Freezing and thawing durability of concrete has close relationship with its pore structure. The volume, radius, and size distribution of pores decide the freezing point of pore solution and the amount of ice formed in pores. Generally, within a certain temperature interval, more frozen concrete pore solution induces greater internal hydraulic pressure and, consequently, more severe frost damages (Cai and Liu, 1998).

Porosity and permeability are not proportionately linked. High porosity concrete has the advantage of providing safety valves for ice to expand into, assuming that the concrete is not saturated. This is the basis of air entrainment as a control measure. However, low permeability concrete, derived from low water/cement ratio mixes, is traditionally associated with durable concrete (Page and Page, 2007). Low permeability concretes are beneficial in two aspects: (1) the impermeability of the concrete reduces the amount of free water that can enter the pore structure of the hardened concrete from external sources; and (2) the hydrated concrete contains minimal amounts of water that is freezable at normally encountered temperature ranges or available as free water to feed ice formation. The water in the largest pores is the first to form ice at a given temperature. Further drops in temperature cause the water in the capillary network to freeze but the gel pore water remains unfrozen. The volume and proximity of spaces into which the expelled water may escape greatly influence the degree of resistance to damage. Thus, a balance is required between high porosity and low permeability. Traditionally, enhanced durability of concrete with high porosity and low permeability is associated with reducing the water/cement ratio by adding water-reducer and air entrainment.

Water reducer

High performance concrete is generally characterized by relatively low water-to-cement ratios and high cement contents, and it is typically produced with the aid of high-range water-reducers (Mehta and Aitcin, 1990). According to Philleo's study (Philleo, 1986; 1987), if high-strength concrete is to be durable without entrained air, it must contain no freezable water. In the hydrated cement paste, the gel pores are small enough that water will not freeze in them at normal winter weather temperatures.

However, the empty spaces between the gel particles, known as capillary pores, are of a size that water will freeze in them near the normal freezing point of water. Theoretically, freezable water can be eliminated from concrete in three ways. The first way is to eliminate the capillary pores by using water-to-cement ratios below 0.38 and providing moist-curing until all spaces are filled with hydration products; under these circumstances, there will be no unhydrated cement left or capillary pores present, and thus, the cement paste will actually run out of water when 90 percent of the cement is hydrated. This "running out of water" is termed self-desiccation (Philleo, 1986; 1987). The second way is to make the capillary pores too small to contain freezable water. The low water-to-cement ratio of high-strength concrete reduces the porosity of the cement paste in addition to reducing the amount of capillary water susceptible to frost action (if ambient conditions permit continuous hydration for a long period of time) (Detwiler et al., 1989). The third way is to produce hydrated cement pastes of such low permeability that, after self-desiccation has used the internal water, it is impossible for water to reenter.

The low water-to-cement ratios commonly used in high-strength concrete mixes are made possible by the addition of high-range water-reducers. Several studies have explored the effects of high-range water-reducers on the freezing and thawing durability of concrete. According to Attiogbe, et al. (1992), air-entrained concrete with low water-to-cement ratios and containing high-range water-reducers can have adequate frost resistance at the spacing factors greater than 0.008 in (0.20 mm). However, Robson (1987) showed that the addition of a high-range water-reducer changed the air-void system of the concrete by increasing the size of the air voids and might impact durability.

Air entraining

Air entraining of concrete is considered as one of the most important methods of enhancing the resistance of concrete to freezing and thawing damage. Air entrainment provides a controlled method of accommodating the water movement caused during ice formation. Entrained air bubbles protect the surrounding spheres of concrete (Powers, 1949). However, there are many factors that influence the effectiveness of air-entrainment on concrete durability, such as the air content, spacing factor and specific surface.

The air content required in a particular situation depends on the volume of frozen water to be accommodated, and it is a function of the exposure conditions. The resistance of concrete to freezing and thawing damage is considerably enhanced by air-entrainment in the range of 2% - 6% (Mohamed and Rens, 2000). The target mean air content was used in the specifications in the past. Recently, Europe has more adopted a standard specification on minimum air content (European Committee for Standardization, 2000). The mean entrainment level is typically of the order of 4% - 6%, but it is contingent on the degree of exposure. The upper limit on air content is the specified minimum value plus 4%.

Spacing factor is also a key aspect of successful air entrainment. A small spacing factor will enhance the resistance of concrete to frost action (Mindess and Young 1981). Resistance to flow is proportional to flow path, and therefore, the path length to an air bubble needs to be short enough to dissipate hydraulic pressure. The osmotic pressure also dissipates if the path length is adequately short (Powers 1949; Fagerlund 1997). Powers noted that the critical spacing is proportional to tensile strength and permeability, while inversely proportional to the degree of saturation and freezing rate. A large number of small air voids is an effective control measure in freezing and thawing environments; whereas an equivalent air volume in the form of a small number of large voids is not. Specific surface is the surface area to volume ratio. The higher the specific surface, the smaller the air bubbles are. The entrained air bubbles have typical diameters of 50 μm; while the specific surface is of the order of 25 mm^{-1} (Neville, 1995).

It should be noted that although air-entrainment of concrete enhances its resistance to freezing and thawing damage, in the long-term, however, continuous exposure to freezing and thawing may result in freezing and thawing distress depending on the severity of weather or frequency of freezing and thawing cycles. Moreover, air entrainment does not enhance the performance of concrete made with aggregates susceptible to significant deterioration in freezing and thawing conditions (American Concrete Institute, 1994).

2.2.2 Aggregate characteristics

Aggregates are the major constituents of concrete, typically occupying 60% to 80% of the volume of concrete, and the concrete durability may be significantly influenced by the characteristics of the aggregate. Physical characteristics of aggregates, such as gradation, absorption, and specific gravity are important properties for mixture proportioning and general quality control and assurance. They can also be important to durability in several ways (Page and Page, 2007). For example, for some aggregate types, absorption and specific gravity values can serve as useful indices for frost susceptibility. Aggregate grading, shape, and texture have significant effects on water demand for fresh concrete, and any water added to the concrete to offset workability issues will increase the permeability of the concrete and exacerbate frost-related damage.

The mechanical properties of aggregates, such as strength, elastic modulus, abrasion resistance, and resistance to polishing have an essential impact on concrete durability (Page and Page, 2007). For high performance concrete, the aggregate strength is very important as the interfacial transition zone (ITZ) is high in strength and low in porosity such that failure can occur through the coarse aggregates. The elastic modulus of aggregates plays a significant role in early-age properties, especially with regard to cracking potential, and in long-term properties. Abrasion resistance may also be a factor in the breakdown of aggregates during handling, mixing, and placing concrete. The resistance of an aggregate to polishing is important, especially for pavements and other types of flatwork.

The behavior of the aggregate when exposed to freezing and thawing depends primarily on the properties related to the pore structure within the aggregate particles. When saturated aggregate particles are frozen, the increase in volume during the formation of ice must be accommodated either within the aggregate particle itself or be forced into the surrounding paste. As with the case of freezing cement paste, the stress in the aggregate is produced by hydraulic pressure from water flow (Powers 1975). Therefore, the important aggregate properties that may influence the frost durability are pore size and its distribution, absorption capacity, degree of saturation of the aggregate, as well as the size and type of coarse aggregates (Waugh 1961).

Pore size and its distribution

The single most important feature of an aggregate that affects frost resistance of concrete is the pore size and its distribution in aggregate (Kaneuji et al. 1980; Hudec 1989). Aggregates with a significant number of pores smaller than 5 μm tend to remain nearly saturated, even at moderate humidity levels. Aggregates with smaller pore sizes tend to exhibit lower permeability, thereby generating higher hydraulic pressures (Pigeon and Pleau 1995). In addition, the total porosity of aggregates plays a role. According to Kaneuji et al. (1980)'s study, for aggregates with similar pore size ranges, the aggregate with higher total porosity is less frost resistant, and durability was not a problem for concrete if the total porosity, regardless of pore size distribution, of a given aggregate was low enough (e.g., aggregates with absorption capacities less than 1.5%).

Thus, both the pore size distribution and total porosity of aggregates affect frost resistance of concrete. For convenience, based on total porosity, it is possible to group aggregates into three categories (Pigeon and Pleau, 1995), i.e., low porosity, intermediate porosity, and high porosity. Aggregates with low porosity are generally durable because the amount of freezable water is so low. Aggregates with intermediate porosity are the least durable because they contain appreciable freezable water upon full saturation and the pore structure is restrictive enough to impede the expulsion of water upon freezing. Aggregates with high porosity are generally durable because they tend to have a coarse pore system, which is easily drained as the relative humidity drops below 100%. When water does freeze within the aggregate, the porous structure (and inherent high permeability) allows for the freezing water to be expelled without causing significant hydraulic pressure (Pigeon and Pleau, 1995).

Generally, it is the coarse aggregate particles with relatively high porosity or absorption values, caused principally by medium-sized pore spaces in the range of 0.1 to 5 μm that are most easily saturated and contribute to deterioration of concrete pop-outs. Larger pores usually do not get completely filled with water, and therefore, damage is not caused by freezing. According to ACI 221R (1996), water in very fine pores may not freeze and fine aggregate is generally not a problem, because the particles are small enough to be below the critical size for the aggregate type and the entrained air in the surrounding paste can provide an effective level of protection (Gaynor, 1967).

Absorption capacity

The absorption capacity of aggregates is another important factor influencing the frost resistance. In the fresh concrete, bleeding causes water to collect underneath the aggregate particles, and water-rich layers form around the aggregate particles. As a result, non-absorbent aggregate forms a rather weak and porous zone around the aggregate while the interfacial transition zones around an absorbent aggregate may be less porous (Chatterji and Jensen, 1992). Porous aggregate can achieve excellent bond since the process of absorption will improve the contact between the paste and the aggregate. This relationship between bond and absorption may account, in part, for the poor correlation between the freezing and thawing durability of the concrete and the aggregate absorption, in view of the fact that the strength of bond tends to increase as aggregate absorption increases (for an unsaturated aggregate) whereas the durability of concrete tends to decrease as aggregate absorption increases (Rhoades, 1946).

Aggregate absorption data are frequently used in assessing the probable durability of concrete exposed to freezing and thawing. According to the ordered-water theory (Dunn and Hudec, 1965), the principal cause of deterioration of aggregate is not freezing but the expansion of adsorbed water (which is not freezable); specific cases of failure without freezing of clay-bearing limestone aggregates seemed to support this conclusion. However, this is not consistent with the observation (Helmuth, 1961) that the adsorbed water does not expand but actually contracts during cooling. Nevertheless, Helmuth (1961) agreed that the adsorption of large amounts of water in aggregates with a very fine pore structure can disrupt concrete through ice formation. Generally, it is believed that the coarse aggregate particles with relatively high absorption values are most easily saturated and they contribute to the deterioration of concrete.

Degree of saturation

The effect of the aggregate on concrete frost durability depends largely on the ability of the particle to become and stay highly saturated while enclosed in cement paste. In this respect, the degree of saturation of the aggregate is also a crucial factor influencing the concrete frost resistance. If aggregates are saturated at the time of concrete mixing with low water-to-cement ratio pastes, self-desiccation during hydration may decrease significantly the moisture content of the aggregate (Powers, 1975; Pigeon et al., 1991). When freezing occurs, most saturated aggregates tend to expel water. As the water is forced out of the aggregate, the hydraulic pressure increases; as a result, the aggregate may expand elastically to accommodate the increased volume. This expansion may cause distress to the surrounding paste. If the water expelled by the aggregate cannot be accommodated by the cement paste, failure will occur within the interfacial transition zones between aggregate and cement paste (Sawan, 1987).

Size of coarse aggregates

The size of the coarse aggregate has been shown to be an important factor in its frost resistance. Because hydraulic pressure is the prevalent mechanism governing aggregate-related distress, it is logical that the size of aggregates affects the development (and release) of pressure within aggregates as water is expelled upon freezing (Page and

Page, 2007). Verbeck and Landgren (1960) demonstrated that, when unconfined by cement paste, the ability of natural aggregate to withstand freezing and thawing without damage increases with a decrease in size, and that there is a critical size below which aggregates can be frozen without damage. According to Powers (1955), the extent of the hydraulic pressure is a function of several parameters, including permeability, tensile strength, freezing rate, and distance to escape boundary. When the source of distress is from within an aggregate, the distance to an escape boundary is directly related to the aggregate size.

For a given aggregate, there exists a critical size above which frost damage may occur because the distance to an escape boundary is too large for flow of water to relieve the internal stress generated around the freezing sites (Verbeck and Landgren, 1960). For fine-grained aggregates with low permeability, the critical particle size may be within the range of normal aggregate sizes (Kosmatka et al., 2002). In fact, many non-durable coarse aggregates can be rendered durable by crushing them, thereby reducing the aggregate dimension below the critical size. The critical particle size for coarse-grained aggregates or those with pore systems interrupted by numerous macropores (voids too large to hold moisture by capillary action) may be sufficiently high to be of no consequence for typical coarse aggregates used in construction (Kosmatka et al., 2002). Verbeck and Landgren (1960) also showed that the critical size of some aggregates can be as small as a 6 mm. The capacities of some aggregates (such as granite, basalt, diabase, quartzite, and marble) for freezable water is so low that they do not produce stress when freezing occurs under commonly experienced conditions, regardless of the particle size.

Aggregate type

Many concrete investigations have been conducted specifically to look at the effect of aggregate type on the frost durability of concrete. Mehta (1990)'s study indicated that aggregate type can play an important role in controlling the strength of the transition zone and the degree of microcracking. A weak transition zone is vulnerable to microcracking when the concrete is subjected to stress. Microcracking in the aggregate-cement paste transition zone of high-strength concrete increases permeability and reduces durability

Walker and Hsieh (1968) conducted the freezing and thawing tests on air-entrained concrete made with different coarse aggregates ranging from crushed limestone and trap aggregate to gravel from glacial and nonglacial sources in the United States and Canada. Their test results indicated that the aggregate producing frost resistant concrete had relatively low absorptions, while concrete with higher absorption aggregates exhibited worse freezing and thawing performance. However, Lane and Meininger (1987)'s study on air-entrained concrete made with four limestones showed that the highest absorption aggregate did not show the worst performance, which agrees with the concept that the pore structure, rather than the absorption of the aggregate, controls concrete frost durability.

The effect of aggregate type on frost durability for non-air-entrained high-strength concrete was studied by Pigeon et al. (1991). It was found that both the limestone and the granite concrete mixes exhibited excellent frost resistance. The effects of aggregate type on the freezing and thawing durability of non-air entrained Portland cement concrete were studied by Kriesel et al. (1998). Five different types of local coarse aggregates were used: round gravel, partially-crushed gravel, granite, a high-absorption limestone and a low-absorption limestone. Of the five aggregate types investigated, the granite aggregate was found to be unsuitable for the production of frost-resistant concrete; the partially-crushed gravel and the round gravel aggregate were found to be suitable aggregates for use in the production of durable concrete; and the concrete specimens containing limestone aggregates consistently exhibited the best performance in freezing and thawing durability testing, while the strong bond between the limestone aggregate and the cement paste was determined to be the reason for the exhibited frost durability.

Frost-susceptible aggregates tend to result in cracking (commonly called durability cracking, D-cracking or D-line cracking) and deterioration of concrete exposed to cyclic freezing and thawing. Prince (2001) declared that the mechanism governing the natural alteration of aggregate also leads to the development of alkali aggregate reaction (AAR). The high alkaline concrete pore solutions might react with certain composition in aggregates and generate some secondary minerals, which might have larger volumes resulting in the traditional AAR, or might have some weaker properties producing the "low-degradation aggregate" phenomena. Khatri et al. (2008) investigated the siderite-bearing aggregates used in concrete around Sydney in Australia and pointed out the aggregates showed the alteration of siderite secondary mineral in the Mt Gibraltar aggregate to iron oxides/hydroxides through oxidation process. Meanwhile, the secondary minerals with larger volumes, mostly reported as AAR, are not often viewed due to other mechanism like low-degradation aggregate. However, the studies on concrete with low-degradation aggregates are very limited in the literature.

2.2.3 Moisture state and climatic conditions

The volume expansion occurring on conversion of water to ice and associated water movement cannot be accommodated in fully saturated concrete. The drier the concrete, the more capable it is to absorb the pressures associated with ice formation and water flow under hydraulic and osmotic influences. In this sense, minimizing the degree of saturation of the pore structure is an important controlling parameter (Page and Page, 2007).

Concrete structures at the greatest risk of saturation are those either permanently exposed to water or occasionally exposed to water under pressure. Deicing agents also encourage saturation because the critical relative humidity below which drying is induced for a pore of given radius increases as the solute concentration of the pore liquid increases (Harrison et al., 2001). Free water in the concrete is not only influenced by the environment but also by the detailing of the elements of the structure. Drainage features that minimize the contact time between water and concrete are particularly beneficial.

Exposure conditions that represent the greatest threat to concrete in cold climates are those that combine a number of parameters. These include the frequency of freeze-thaw cycles, the absolute lowest temperature reached, the period of freezing, and the rate of cooling. The lower the temperature and the longer the freezing period, the more extensive the effect on the volume of pores exposed to potentially damaging conditions. Concrete has a low rate of heat transfer, and therefore, significant penetration of the ice front requires very low temperatures, long periods of freezing, or a combination of both. The rate of cooling can be a factor. The slower the rate of cooling, the greater the opportunity for dissipation of the pressures from expansion and expulsion of water from the capillaries. This may prevent the cumulative buildup of tensile stresses beyond the capacity of the concrete.

2.3 Identification of frost-susceptible aggregates

For most existing infrastructure across the U.S., the aggregates used in their construction are primarily from natural sources formed by geological changes occurring over millions of years. Due to different evolution and chemical composition, the long-term performance of different aggregates varies. Destructive alteration of aggregates, which usually arises from weathering or internal deep-seated and wide-scale geochemical processes, has been harmful for concrete with these aggregates. Previous studies reported several examples of unsound aggregates from different areas (Hosking and Tubey, 1969). Although satisfactory results could be obtained in initial routine laboratory tests, these aggregates are usually subjected to decomposition in stock piling and/or in use. Some of the most severe cases of this type of failure have been found in Washington State (Turner and Wilson, 1956; Minor, 1959), Virginia (Melville, 1948), Oregon (Scott, 1955) and Idaho (Day, 1962), which were all failure of aggregates in unbound bases and associated with physical breakdown of aggregate suffered from weathering.

A key step in designing durable concrete in cold weather applications is to identify whether an aggregate of interest may be susceptible to frost damage, which may manifest itself as internal distress within concrete or surface damage, such as D-cracking (especially for pavements) and pop-outs (Page and Page, 2007).

There are a variety of laboratory tests for evaluating the frost resistance of aggregates, ranging from those that test solely the aggregate to those that test concrete containing the subject aggregate. However, there does not exist a single test that can be used to clearly delineate a frost-resistant aggregate from a frost- susceptible aggregate, especially given the complexities of the underlying mechanisms of frost damage and the variety of manifestations of damage in field concrete, such as D-cracking and pop-outs. This difficulty in predicting aggregate performance from a single laboratory test is compounded with the fact that the specific exposure conditions to which concrete is subjected play a major role in determining the frost resistance of a given aggregate. Nevertheless, there are still several laboratory tests that can provide excellent insight into the physical, mineralogical, and chemical characteristics that govern the frost resistance of a given aggregate (NCHRP, 2003). Some of the direct aggregate tests do not actually evaluate frost resistance, but rather that they measure other relevant aggregate properties affecting frost resistance, such as absorption or porosity.

Aggregate absorption has an important impact on frost resistance of aggregates and its manifestation in field-damaged concrete. For certain aggregate types, there is a good correlation between absorption capacity and frost resistance. Pigeon and Pleau (1995) suggested that a maximum absorption capacity of 2% be imposed on aggregates to prevent aggregate-related damage from freezing and thawing cycles. Aggregates of higher porosities and absorption capacities may suffer damage upon freezing since water is expelled from the aggregate. Although certain aggregates exhibiting higher absorption may be prone to D-cracking or pop-outs, absorption should not be considered as an absolute index of frost resistance because many aggregates with relatively high absorptions are quite durable. Thus, the local experience and familiarity with a given aggregate type or source should be used to determine if absorption is related to the durability of that aggregate (NCHRP, 2003).

Standard aggregate absorption tests (e.g., ASTM C127 and C128) are typically used in specifications. More advanced tests, such as nitrogen absorption or mercury intrusion porosimetry, can be used to obtain information on internal porosity, pore structure, and absorption capacity of aggregates. Other general aggregate tests are specific gravity and gradation. As noted previously, the size of an aggregate tends to affect its frost resistance, and certain non-durable aggregates can be rendered durable by reducing the size below its critical value.

Besides measuring basic aggregate properties (e.g., absorption, specific gravity, grading), more specific aggregate testing that aims at triggering distress (e.g., typically measuring mass loss) is also widely used. Unconfined aggregate tests are fairly common measures for assessing frost resistance. These tests are quicker than testing of concrete, and they can be used more economically to track aggregate quality and performance. There are a variety of different unconfined aggregate tests. Most involve freeze-thaw cycles of aggregates in water or salt solutions. For example, AASHTO T 103-91 (Procedure B) uses immersion of the aggregates in an alcohol-water solution to increase the penetration of water during the soaking period, and the Canadian Standards Association test (CSA A23.2-24A -Test Method for the Resistance of Unconfined Coarse Aggregate to Freezing and Thawing) uses salt solution to improve the correlation between laboratory freezing and thawing results and D-cracking in concrete pavements (Rogers et al., 1989).

Sulfate soundness tests (e.g., ASTM C88) are also common methods used in routine quality control of aggregates. These tests involve subjecting aggregate samples to repeated soaking in either magnesium or sodium sulfate solution, followed by oven drying, and recording of the mass loss after various test cycles. Rather than inducing freeze-thaw cycles within the aggregates, these tests trigger crystallization and/or hydration pressures in the pores of aggregates, which can lead to significant damage. Although these tests are not regarded as reliable indicators of frost resistance, they can provide a rapid means of acquiring comparative data for a given aggregate source.

It was found that a good correlation exists between mass loss in the Micro-Deval test (ASTM D6928) and magnesium sulfate soundness testing (Rogers et al., 1991). However, due to lower sensitivity to aggregate grading, it is much easier to reproduce the Micro-Deval test. Thus, the wet attrition of fine and coarse aggregates has gained in popularity in recent years, with the Micro-Deval test emerging as the most common method (NCHRP, 2003). According to the CSA specifications, the loss for fine aggregates is limited to 20%, while the loss for coarse aggregates is limited to between 14 and 17% (CSA 2000).

In addition to the above mentioned laboratory test methods, two tests have been developed specifically to address D-cracking potential of aggregates: the Iowa Pore Index Test (IPIT) and the Washington Hydraulic Fracture Test (WHFT). The IPIT was developed in an attempt to quantify the volume of micropores in aggregates, and it has been found that the IPIT correlated well with the potential for D-cracking (Dubberke and Marks, 1992). It was also reported that the IPIT test results correlate well with field performance of aggregates in different localities (NCHRP, 2003; Winslow, 1987; Rogers and Senior, 1994). The Washington Hydraulic Fracture Test (WHFT) was developed by

Jannsen and Snyder (1994) and later modified (Embacher and Snyder, 2001) to assess aggregate frost resistance. In this test method, aggregates are put under pressure but, rather than measuring the volume of penetrating water, the degradation of the aggregate sample is measured as the pressure is removed from the saturated aggregate, dispelling the internal water and causing hydraulic pressures. The damage to the aggregate sample is estimated by the change in gradation of the aggregates as larger particles are fractured to form smaller ones.

In the cases mentioned above, the aggregate types were found to be black basalts from the Eocene age. Other types of aggregate, such as Tertiary sandstones and shales and gravels of Mesozoic age were also found to be involved. It was believed (Scott, 1955; Day, 1962; Weinert, 1968; Fookes, 1988; Woodside and Woodward, 1989; and Wylde, 1976) that it is the secondary minerals formed from the primary minerals under some certain action of weathering and disintegration, rather than the primary minerals, that are the principal reasons for these failures in pavements. Smith and Collis (1993) summarized a large number of reported failures where in-service deterioration had occurred within an engineering time scale in different countries and confirmed that the failure usually happened under two conditions: (1) the use of igneous (usually basic igneous) aggregate, and (2) the presence of secondary minerals in such aggregates. They also noted that the sedimentary aggregates employed as aggregates, such as the sandstones, shales, and gravels, were the product of weathering process and could not suffer from mineral alteration. Thus, they could not be the causes for the failures.

The weaker secondary mineral formed in aggregate degradation would easily break down into dust, which will change the gradation of the mix and create the need for increased asphalt cement to be introduced to the HMA mixture. Since Minor (1959) reported the phenomenon of degradation of low-degradation aggregates, the aggregate degradation of marine basalt materials in the state of Washington has been a recognized problem, and specification requirements for the degradation of aggregates have been established for hot mix asphalt and for most aggregate products (WSDOT Standard Specifications, 2010). WSDOT employs its own degradation test (WSDOT Test Method T 113) to determine a "degradation value" to replace the standard aggregate soundness, which can range from 0 to 100 with higher values representing less degradation. The degradation value represents the susceptibility of the certain aggregate to disintegrate into plastic fines when abraded in the presence of water. Aggregates with a degradation value below 35 are currently considered as low-degradation aggregates (WSDOT Y-8694, 2005). The low-degradation aggregates used in this study have a degradation factor of 31.

2.4 Test methods for frost durability and long-term performance of concrete

Concrete durability has been a key issue for long-term performance due to its high significance in the serviceability of concrete structures as well as to its tremendous economic impact on the construction sector. The degradation of concrete is a slow process, and field testing alone cannot satisfy the research needs on concrete durability. In order to assess the durability performance of concrete, test methods that accelerate the process are often needed, to allow that the results of the tests could be obtained in a reasonable time.

Several standardized laboratory tests for evaluation of concrete durability have been in use for a long time. These testing methods are standardized by various organizations, such as ASTM International, American Association of State and Highway Transportation Officials (AASHTO), American Concrete Institute (ACI), and some other standardized methods used in the U.S. as well as other countries. It is not evident, however, that durability tests are representative of the extremely variable exposure conditions to which concrete can be subjected in the field. Moreover, for each test method, the procedure has some advantages and disadvantages. Thus, it is important to understand the state of the research on the test methods used for long-term performance and durability of concrete. To this end, a thorough review on the conventional test methods, such as freeze and thaw resistance, chloride penetration, abrasion and alkali silica, and their potential correlation with practice and field data will be conducted in this section.

2.4.1 Freeze and thaw test methods

Freezing and thawing damages may appear as internal cracking or surface scaling (Ferreira, 2004; Jacobsen et al., 1997; Valenza II and Scherer, 2007; Tang and Petersson, 2004; Penttala, 2006). The former one may be due to the mechanisms of internal ice formation and result in strength loss, but it may not be surface visible. The latter one occurs on both the horizontal and vertical surfaces where water or snow can naturally deposit, and surface remains wet for periods of time. The susceptibility to surface scaling will increase in the presence of deicing salts. Due to the distinct nature of the two actions, there are different testing methods to evaluate the resistance of concrete to these actions, respectively.

2.4.2 Test methods for internal damage

ASTM C666 "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing" is the most commonly used test method for determining the freezing and thawing resistance of concrete, especially the internal damage.

ASTM C666 presents two procedures (Procedure A- Rapid Freezing and Thawing in Water, and Procedure B- Rapid Freezing in Air and Thawing in Water) that may be used. Both procedures begin with the preparation of concrete specimens, which are typically of dimension 3×4×16 in. (75×100×400 mm) prisms and moist-cured for 14 to 28 days at room temperature after casting. After the moist curing is complete, the specimens are cycled by alternately lowering the temperature from 40 to 0 °F (4 to -18℃) and raising it from 0 to 40 °F (-18 to 4℃) in not less than 2 hours and not more than 5 hours. At intervals not to exceed 36 cycles of freezing and thawing, the specimens are removed from the water bath and a measurement of the dynamic modulus of elasticity of the specimen is taken. In other freezing and thawing tests, the temperature ranges are different, such as -20 to 20℃ in Pospichal et al. (2010) and Tang and Petersson (2004).

The dynamic modulus of elasticity was determined by measuring the fundamental transverse frequency of the specimens at each test interval, and the test method was stated in ASTM C215. ASTM C666 provides an equation to convert the fundamental transverse frequency to the dynamic modulus of elasticity. The procedure is repeated at the specified minimum interval until the specimen has been subjected to 300 cycles of freezing and thawing, or until the relative dynamic modulus of elasticity falls below 50%. The relative dynamic modulus of elasticity is the ratio of the dynamic modulus determined at the particular test interval to the initial dynamic modulus at the beginning of the test, as a way to detect deterioration. The test continues until the specimen has undergone either the desired number of cycles (usually 300 cycles) or the number of cycles after which the critical dilation has occurred.

Besides the standard measurement using the ASTM C666, some other methods to interpret the tests also have been used. Since the degree of saturation is considered as very important for the performance of concrete in a frost test, absorption during the test sometimes has also been measured. Tang and Petersson (2004) used the ultrasonic pulse transmission time (UPTT) measurement for measurement of internal damage. Pospichal et al. (2010) employed the UPTT and resonance method to determine the degradation of specimens.

2.4.3 Discussion

It is usually accepted that there remains an inadequate correlation between the accelerated laboratory test results and the observed behavior of mature concrete exposed to natural freezing and thawing (Pigeon and Pleau, 1995; Mao and Ayuta, 2008; ACI 213, 2003). Therefore, some discussion about the characteristics of the tests are beneficial for the improvement and application.

In the ASTM C666, Procedure A is the most commonly used method, which means that the concrete specimens are fully saturated during the entire cycling period and are not allowed to dry before the beginning of the tests, which is contrary to the natural exposure conditions where concrete is generally exposed to frequent wetting and drying cycles. Because the degree of saturation of the concrete will increase dramatically, damage may accumulate much faster in the laboratory than in most real exposure situations (Jacobsen, 2005), which is similar to the criticism raised in NCHRP Report No. 129 (1986) (Williamson, 2005). On the other hand, Procedure B was criticized for allowing the specimens to dry (Williamson, 2005). To better simulate the real field conditions, a third procedure was developed by Jansen and Snyder (1994), which is referred to as Procedure C: Rapid Freezing and Thawing while wrapped in terry cloth.

In addition, for the ASTM C666, the rate of temperature change has also been questioned for being too harsh (Williamson, 2005). The cooling rate is one of the most important parameters governing the frost durability (Jacobsen et al., 1997). The test conditions of ASTM C666 are more severe than those occurring in practice since the prescribed heating and cooling cycle is between 4.4 and -17.8 ℃ (40 and 0 °F) at a rate of cooling of up to 14 ℃ per hour (26 °F per hour). In most parts of the world, a rate of 3℃ per hour (5°F per hour) is rarely exceeded (ASTM C666). The results from the reference showed that the internal cracking was increased by increasing the rate of cooling (Jacobsen et al., 1997). Pigeon and Pleau (1995) showed that the higher cooling rates were generally detrimental to the frost resistance, suggesting that certain type of concrete exhibiting significant deterioration during laboratory tests might be able to resist, without any damage, the natural freezing and thawing cycles. It means that one tested concrete may be viewed as frost resistant in one laboratory, but as frost-susceptible in another laboratory, if different cooling rates are used.

Meanwhile, for the scaling tests, ASTM C672 prescribes with one cycle every one day, which is slower than the one in the ASTM C666. The different setting of freezing rate agrees with the statement by Jacobsen et al. (1997), i.e., the difference of crack susceptibility to the freezing rate of different cracking type leads to different freezing rate in these two test methods. A slow cooling rate produced more scaling. However, the test results by other researchers (Valenza II, 2007a, b; Marchand et al., 1997) indicates that the freezing rate has very little influence on the amount of damage. It was also demonstrated that frost damage would be reduced when the minimum temperature of the cycle is increased, and the longer time at the minimum temperature, the more frost damage would be introduced in concrete (Valenza II, 2007a). Thus, it is highly possible that the slow freezing rate leads to longer time at the lower temperature, which results in more damage. In addition, the maximum amount of scaling damage is achieved with a moderate amount of salt, which is widely accepted at the concentration of approximately 3% by weight (Valenza II, 2007a).

Chapter 3 MATERIALS AND TEST PROGRAMS

The primary goals of this study are to evaluate the long-term durability of concrete incorporating two aggregate sources (low-degradation (LD) and normal (NM)) and to develop corresponding accelerated testing procedures to evaluate the long-term performance of concrete. In the following sections, the materials used in this study and the testing procedures selected to evaluate the long-term deterioration of concrete are presented.

3.1 Materials and mix design

The cement used to prepare the samples in this study was Portland cement, Type I-II. Coarse aggregates (both LD and NM) were provided by the Washington State Department of Transportation (WSDOT). The specific gravities and water absorption of the LD aggregates are 2.68 and 1.2%, respectively; and the similar values for the NM aggregates are 2.62 and 1.3%, respectively. The specific gravity of the fine aggregate used in the concrete mixes is 2.65. According to the aggregate degradation review conducted by KBA, Inc. (WSDOT Y-8694, 2005), aggregates with a degradation factor less than a value of 35 are commonly considered as low-degradation aggregate. In this study, two identified aggregate sources, the LD aggregates with degradation factor of 31 and the NM aggregates with a degradation factor of 59, were used. The degradation factors for the LD and NM aggregates were obtained by the WSDOT using WSDOT Test Method T 113. The only differences between the LD-WSDOT and NM-WSDOT concrete batches were the type of aggregates and being mixed at different times. The natural fine aggregates (sand) and mix designs used in both the LD-WSDOT and

NM-WSDOT concrete were the same.

The gradations of the coarse aggregates (LD and NM) and fine aggregate are presented in Tables 3.1 and 3.2, respectively,

Table 3.1 Coarse Aggregate Gradations (Sieve Analysis)

	LD aggregate	NM aggregate
Sieves	Cumulative % Passing	Cumulative % Passing
1/2"	100	100
3/8"	98.5	94.8
1/4"	67.8	64.4
#4	37.3	11.4
#8	3.0	2.8
#16	0.4	1.3

Table 3.2   Fine Aggregate Gradations (Sieve Analysis)

	Fine Aggregate	
Sieves	Individual % Retained	Cumulative % Passing
3/8"	0	100
1/4"	0.5	99.5
#4	1.8	97.7
#8	13.4	84.3
#16	23.3	61
#30	18.8	42.2
#50	24.5	17.7
#100	13.6	4.1
#200	1.9	2.2

The mix design used for the test samples was based on WSDOT guidelines for a 4000 psi mix (WSDOT Concrete Mix Performance Guideline (4000D)) and is given in Table 3.3. This mix design was used for both the LD-WSDOT and NM-WSDOT concrete samples.

Table 3.3 WSDOT Mix Designs

Mixtures	Cement (lb/yd^3)	Aggregate (lb/yd^3)	Sand (lb/yd^3)	Water/Cement ratio
LD-WSDOT	564	1830	1270	0.48
NM-WSDOT	564	1830	1270	0.48

3.2 Sample preparations

To prepare all the test samples with as close material properties as possible, two batches of concrete, each with a volume of approximately 3.5 yd^3, were mixed by a commercial ready-mix concrete company and transported to WSU's concrete laboratory by a concrete mixer truck. In accordance with ASTM C192/C192M, the fresh concrete was cast in the oiled wood molds to form beams with dimensions of 3×4×16 in. Prisms with dimensions of 4×4×16 in. and cylinders with dimensions of 6 ×12 in. were cast for the flexural strength and compression strength test, respectively. Immediately after casting, all samples were externally vibrated for approximately 15 seconds and finished using a metal trowel. All specimens were demolded after 24 hours and then cured in lime-saturated water at room temperature for at least 28 days before testing. In the first batch of concrete samples with LD aggregates, 36 cylinders, 180 beams and 9 prisms were cast; while in the second batch of samples with NM aggregates, 9 cylinders, 60 beams and 9 prisms were cast.

3.3 Experimental testing program

In order to evaluate the properties of the concrete incorporating different aggregates, a number of tests were conducted. According to the condition of the concrete when it is being tested, these tests are grouped into two categories: (1) fresh concrete tests, and (2) hardened concrete tests. Fresh concrete property tests were conducted to evaluate the air content and slump of the concrete. The hardened concrete property tests were divided into two sub-categories. The first category pertains to the mechanical properties at different ages, such as compression strength, flexural strength, and modulus of elasticity of the concrete. The second category relates to the long-term durability of concrete due to freezing and thawing (F/T) exposure and the corresponding tests to screen the frost damage after different numbers of F/T cycles, including dynamic modulus of elasticity test and cohesive fracture test. The procedures for each test are briefly discussed in the following sections, and the tests considered in this study are summarized in Table 3.4 along with their ASTM and/or AASHTO standard test method designations.

Table 3.4 Experimental Testing Program

Properties of Concrete	Test Methods
Fresh Properties of Concrete	
Air content	ASTM C231/AASHTO T152
Slump	ASTM C143/AASHTO T119
Hardened Properties of Concrete	
Compression Strength of Concrete	ASTM C39/AASHTO T22
Flexural Strength of Concrete	ASTM C78/AASHTO T97
Modulus of Elasticity of Concrete	ASTM C496
Concrete Durability	
Freezing and thawing	ASTM C666
Dynamic Modulus of Elasticity	ASTM C215
Fracture Energy	RILEM 50-FMC (1985)

3.3.1 Property tests of fresh concrete

Slump and air content tests were performed on fresh concrete to evaluate the workability and durability properties. The slump test was conducted following the procedures of ASTM C143/AASHTO T119 “Slump of Hydraulic Cement Concrete” (see Figure 3.1). The air content test was conducted following the procedures of AASHTO T152/ASTM C231 “Air Content of Freshly-mixed Concrete by the Pressure Method” (see Figure 3.2).

Figure 3.1   Slump Test

Figure 3.2   Air Content Test

3.3.2 Property tests of hardened concrete

Three basic mechanical properties were evaluated for the hardened concrete: compressive strength, modulus of elasticity and flexural strength. The compressive strength test (Figure 3.3) was conducted following the procedures of ASTM C39/AASHTO T 22 “Compressive Strength of Cylindrical Concrete Specimens”. The modulus of elasticity test (Figure 3.3) was conducted following the procedures of ASTM C469 “Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression”. The flexural strength test (Figure 3.4) was performed following AASHTO T97/ASTM C78 “Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)”.

Figure 3.3 Compressive and Modulus of Elasticity Test

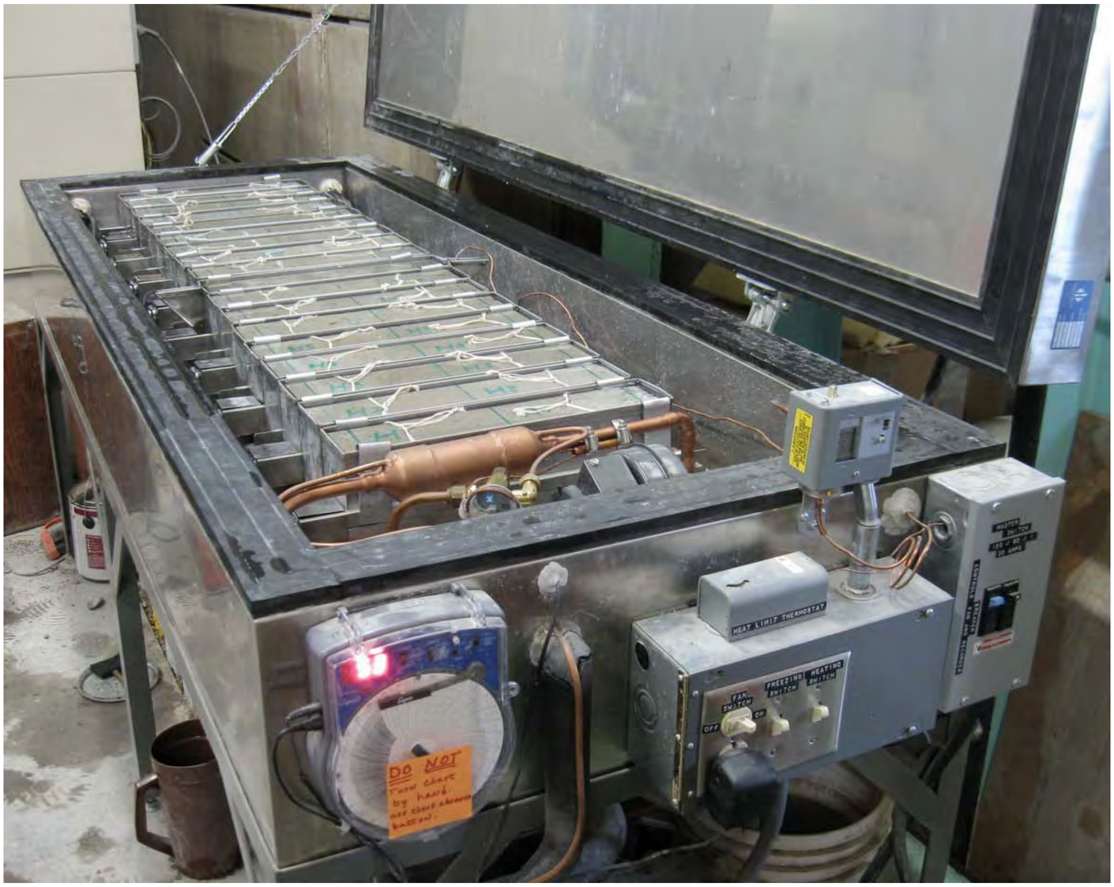
Figure 3.4 Flexural Strength Test

3.3.3 Rapid freeze and thaw test

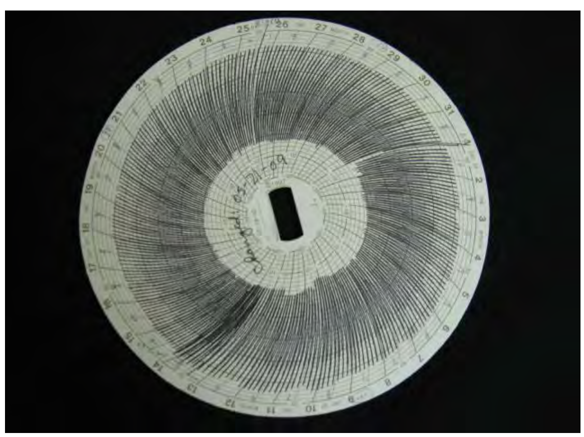
Test Method for Resistance of Concrete to Rapid Freezing and Thawing (ASTM C666), Procedure A, which was designed to provide an indication of the potential freezing and thawing durability of concrete in a freezing and thawing environment, was adopted in this study to condition all the concrete samples. It provides a relative assessment of the frost resistance of the concrete after a given number of F/T cycles in comparison to the specimens which have not undergone freezing. As aforementioned, the rapid cyclic freezing and thawing test is capable of introducing severe conditions in the concrete specimens, and this test can thus be viewed as an accelerated conditioning protocol to condition concrete. Combined with other test methods, it can be used to evaluate the long-term durability of concrete.

The freezing and thawing machine used in this study consists of 18 compartments (see Figure 3.5 (a)), into which 17 samples can be placed and subjected to specified freezing and thawing cycles, along with another particular compartment for a control sample, which serves to produce continuously, automatically and reproducible cycles with the specified temperature. The control sample, drilled with two holes at each end such that probes could be inserted into its center, was positioned near the center of the machine. A thermometer was placed in one end of the specimen to measure temperature, while another probe was inserted into the opposite end of the control sample to regulate the cyclic freezing and thawing. When this probe measures that the center of the control sample has reached the specified freezing temperature, it triggers the temperature control unit to switch to thaw mode and then back to freezing when the thawed state is reached. All concrete samples were cured in limewater for 28 days according to ASTM C192.

An "S"-shaped stainless steel wire with a diameter of 3/32 in. is placed in the bottom of the containers before placing the samples so that: 1) each sample will be always completely surrounded by enough water while it is being subjected to cyclic freezing and thawing actions, and 2) the temperature of the heat-exchanging medium will not be transmitted directly through the bottom of the container to the full area of the bottom of the samples. During the conditioning period, each group of samples were rotated in their conditioning positions in the freezing and thawing machine so that all samples in the same group were conditioned in as consistent a manner as possible.



(a) Freezing and thawing machine



(b) Temperature reading sheet

Figure 3.5 Freezing and Thawing Conditioning.

The temperature range of every freezing and thawing cycle in this study was set to cycle between 0 and 40°F as specified in ASTM C666. The difference between the temperatures at the center of a specimen and that at its surface was continuously monitored and controlled not to exceed 50^0F. Usually, temperature range in the samples was from 0 to 40^0F with an error of less than 2^0F and a cycle frequency of 6 to 9 cycles per day. In a few cases, damage to the control concrete specimen resulted in greater fluctuations in the temperature range and cycle frequency for the freezing and thawing samples, as shown in Figure 3.5(b). Whenever the temperature reaches either lower than -3^0F or higher than 43^0F, the F/T machine was turned off and the controlling sample was replaced by a new one to help maintain the temperature range in the standard range.

Within this temperature range, two groups of samples were conditioned at the different time periods. The first group consists of 16 samples all with the LD aggregate sources, while the second group consists of 8 samples with the LD aggregate sources and 8 samples with the normal aggregate sources. Both these two groups of samples were conditioned up to 1,500 F/T cycles. Due to the limit on the number of samples (i.e., a maximum of 16 samples can be conditioned at a time) in the freeze/thaw conditioning machine, in the first sample group all with the LD aggregate source, sub-groups (i.e., 6, 6 and 5 samples for three sub-groups) of samples were conditioned at each time. Each sub-group of samples was randomly selected from the total 200 samples cast at one time. A total of nine sub-groups samples were conditioned for the first group of LD-WSDOT concrete samples, and they were then taken out for a fracture test at 0, 60, 120, 180, 240, 300, 500, 700, 900, and 1,500 cycles. In the second group, 8 LD-WSDOT and 8 NM-WSDOT concrete samples were simultaneously conditioned up to 1,500 cycles. For both groups of samples, approximately at every 30 F/T cycles the specimens were removed from the conditioning machine for weighing, measuring the dynamic modulus of elasticity, and visually examining the surface scaling. Specimen deterioration was evaluated by measuring the variance of the fundamental transverse frequency used to determine the dynamic modulus of elasticity. More detailed discussion of the method used to measure the fundamental frequency is given in the following section.

3.3.4 Dynamic modulus test

Dynamic modal testing is a nondestructive method for assessing the dynamic response of structures. This method uses sinusoidal or impact excitation for the input signal and forces the specimen to vibrate at some frequency as the response of the specimen is monitored with an accelerometer. A natural frequency of vibration is a characteristic (dynamic property) of the tested elastic system. Assuming the concrete samples are homogeneous, isotropic, and comprised of an elastic material, the dynamic modulus of concrete will be related to the resonant frequency and density. According to ASTM C215, two methods are available for measuring resonant frequency are sinusoidal excitation (forced oscillation) and impact excitation. The classic forced resonance setup uses either transverse or longitudinal resonance. In the longitudinal mode, the oscillator is placed at one end and the pickup is at the other. In the transverse mode, the oscillator is placed in the middle of the top surface, and the pickup is at one end of the top surface. The ASTM C215 impact method uses a modally-tuned impact hammer to excite vibrations in the tested beam and an accelerometer attached to the beam to record the response. Modal tuning enables the isolation of the hammer’s response from the structural response, thus providing an accurate measurement of the specimen response, rather than the combined system (impact hammer and structure) response.

In this study, the dynamic modulus test was conducted following ASTM C215. The fundamental transverse frequency was used to calculate the dynamic modulus of elasticity of the samples under certain F/T cycles. In this test, an accelerometer (output signal) is attached to one end of the concrete beam using vacuum grease. The test sample is slightly impacted by the hammer at the approximate middle of the sample. The time domain response data (impulse versus time and response versus time) were recorded automatically by the commercial software "Control Desk". Typical time domain impulse and response data are shown in Figure 3.6 and Figure 3.7, respectively. The frequency domain response data are shown in Figure 3.8.

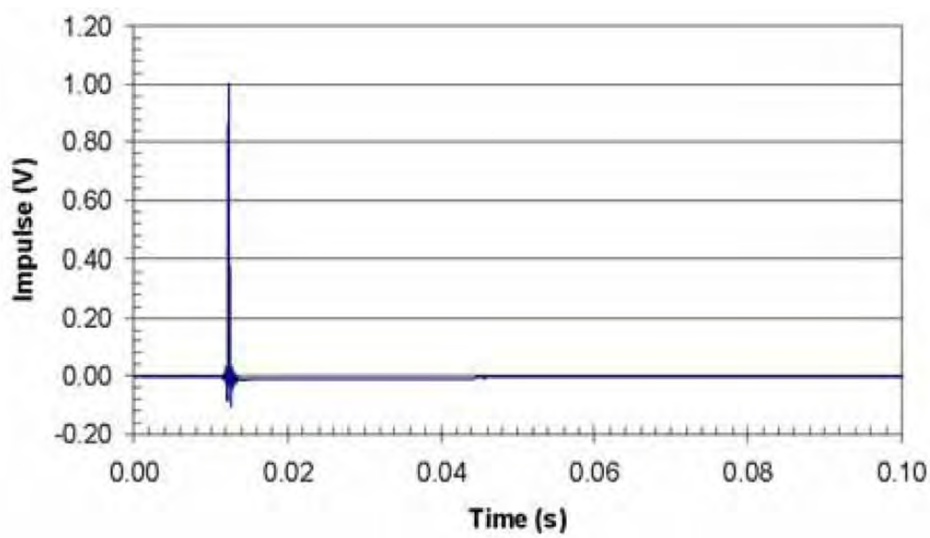


Figure 3.6 Time Domain Impulse Signal.

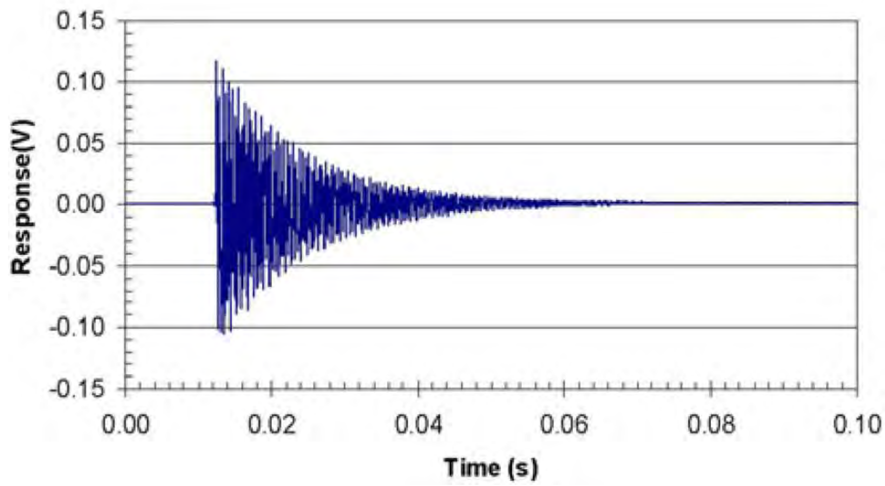


Figure 3.7 Time Domain Response Data.

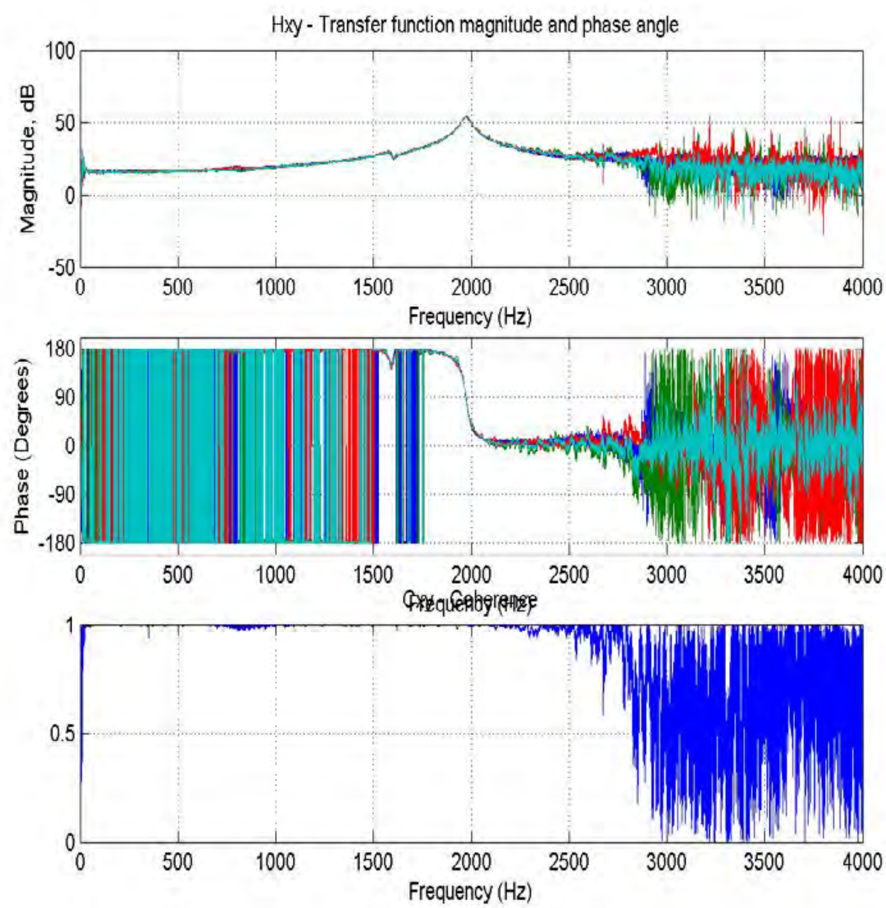
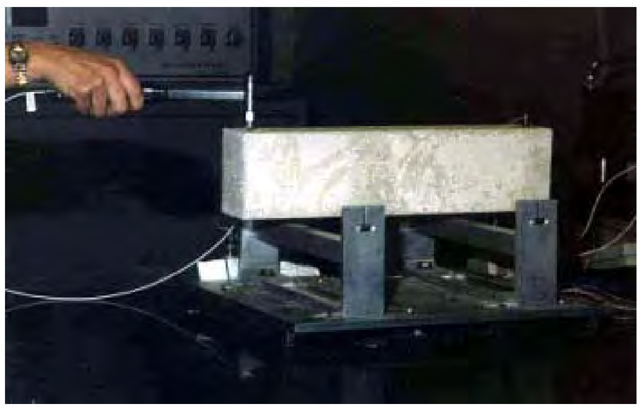


Figure 3.8 Frequency Domain Response Data.

According to ASTM C215, the test samples should be supported at locations with distance of about 0.224 of the length of the sample from each end so that it is able to vibrate freely in the transverse mode, as shown in (Figure 3.9a). However, test data obtained using an alternative support approach where the sample was placed on a thick pad of soft sponge (as shown in Figure 3.9(b)) were compared to results obtained using the standard supporting method and found to differ by less than 0.05%. Considering the simplicity of the second support method, the thick pad support approach was used in this study for the transverse frequency measurement.



(a) Test support suggest by ASTM C215

(b) Thick pad support approach used in this study

Figure 3.9 Set-up for Dynamic Modulus Test.

The dynamic Young's modulus of elasticity, E, in Pascal can be determined from the fundamental transverse frequency, mass, and dimensions of the test sample using the following equation:

$$E = CMn^2 \tag{3.1}$$

where M is the mass of the sample; n is the fundamental transverse frequency; $C = 0.9464T\frac{L^3}{bt^3}$ for a prism; L is the length of the sample; t and b are the thickness and width of the sample, respectively. T is a correction factor that depends on the ratio of the radius of gyration to the length of the specimen and on the Poisson's ratio. Using the ASTM C215 standard and combining with the samples in this study, the value of T was determined as 1.41.

3.3.5 Cohesive fracture test

Fracture tests for both groups of concrete samples with LD and NM aggregates were conducted to evaluate the fracture energy of samples with different F/T cycles. The fracture energy is a material property that may be as important as normal strength properties. The general idea of this type of destructive test is to measure the amount of energy a notched sample absorbs when it is fractured into two halves. The determination of fracture energy was performed using the procedures given in RILEM 50-FMC (1985) and Hillerborg (1985), where a three-point bending beam (3PBB) method is recommended, as shown in Figure 3.10(a). The fracture energy is obtained by dividing the area contained by the load-displacement curve by the fractured area projected on a plane perpendicular to the tensile stress direction. A typical measured load-deflection curve is shown in Figure 3.10(b), in which the remaining parts of a hypothetical complete load-deflection curve are shown in dashed lines. The additional load P_1 is the self-weight of the sample and was excluded from the measured load P. Accordingly, the total amount of absorbed energy W can be determined as:

$$W = W_0 + W_1 + W_2 \tag{4.2}$$

where W_0 is the area within the measured load-deflection curve; $W_1 = P_1\ \delta_0$ is the energy absorbed by the tested sample self-weight, where δ_0 is the deflection when $P = 0$ and when the sample breaks; and W_2 is the residual energy that need to fully separate the fractured sample into two halves after the measured load drops to zero. It was demonstrated that W_2 is approximately equal to W_1. Therefore, the fracture energy can be determined by:

$$G_F = \frac{W_0 + 2P_1\delta_0}{A_{lig}} \tag{4.3}$$

where $A_{lig} = b\left(t - a_0\right)$ is the fractured area of the sample.

Before conducting the fracture test, notches were cut in the samples at the mid-span of the sample using a diamond saw (see Figure 3.11(a)). In order to minimize the shear effect during the fracture test, a deep notch is recommended by the RILEM 50-FMC (1985). Thus, the depth of the notch was selected as half of the depth of the sample in this study. A typical notched sample ready for the fracture test is shown in Figure 3.11(b).

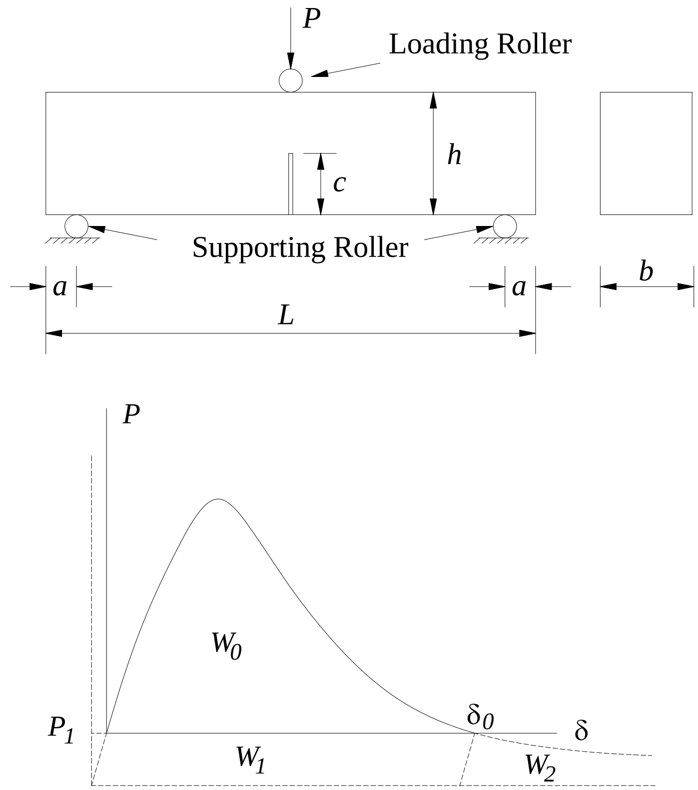


Figure 3.10 Fracture Test under Three Point Bending: (A) Specimen Sketch; and (B) Typical Load-Deflection Curve.

All the fracture tests were performed using an MTS servo-hydraulic testing machine (see Figure 3.12(a)). The tests were conducted using displacement-controlled mode and with a loading rate of 0.0236 in./min. In this test, a high resolution linear variable differential Transducer (LVDT) was used to measure the loading point deflection of the test sample, where the tip of the LVDT's core was placed on a short “L” shaped steel strip which was tightly clamped to the concrete sample at the loading position. The measurements of applied load and mid-span displacement (MSD) were automatically recorded through the commercial data acquisition software “LabVIEW”. The loading setup for the test sample is shown in Figure 3.12. It was found that flat supporting (bottom) and loading (top) surfaces of the test sample are critical in order to eliminate effects of instability and to obtain consistency in the deformation for the tested sample during loading process since the supporting and loading rollers are of finite width rather than the idealized “point” load. To address this situation, the surfaces of the notched samples, especially those near the supporting and loading positions, were ground smooth. The test was terminated when the applied load decreased to a value around 15.0 lbs, which is less than the self-weight of the samples. The reason for applying this termination loading criterion is that all the samples will have completely fractured at this point, and continued acquisition of data is without value (Hillerborg, 1985). A typical fractured sample and its fractured section are shown in Figure 3.13.

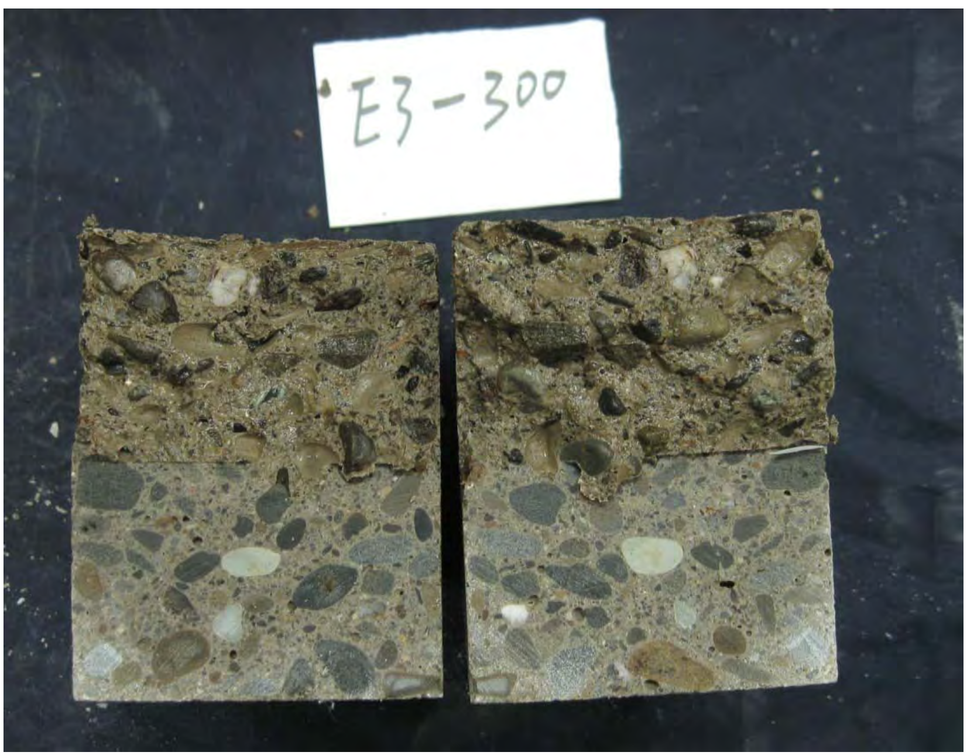
(a) Notch-cutting by diamond saw

(b) Notched sample cut by diamond saw

Figure 3.11 Sample Preparation for Fracture Test.

Figure 3.12 Testing Equipment Setup for Fracture Test.

(a) Fractured sample



(b) Fractured section

Figure 3.13 Typical Fractured Sample

Chapter 4 TEST RESULTS AND ANALYSIS

In this chapter, results from the tests discussed in the previous chapter are presented and analyzed.

4.1 Test results of fresh and hardened concrete

Slump and air content tests were performed on fresh concrete to evaluate workability and durability properties. Both the slump and air content tests for each concrete batch were conducted three times. The average slump and air content for the NM-WSDOT and LD-WSDOT are listed in Table 4.1. Based on WSDOT's concrete mix performance guidelines for 4000D concrete, the specified maximum slump and the range of air content are 3.5 in. and 8 to 6%, respectively. As shown in Table 4.1, the air content of NM-WSDOT batch is lower than that of LD-WSDOT batch. To better characterize the effect of aggregate degradation on the frost durability of concrete materials, the air content of the two concrete batches incorporating the different aggregates should be as close in value as possible. However, in this study, the two concrete batches were mixed at the ready-mix plant at two different times and then delivered to WSU's campus for casting of the samples, and their air contents were not controlled to produce the same value. As discussed later, the NM-WSDOT concrete with the lower air content exhibited greater scaling and deterioration of the concrete paste after undergoing F/T cycles when compared to its counterpart of LD-WSDOT concrete.

Table 4.1 Material Properties of Fresh Concrete

Mixtures	Slump (in.)	Air Content (%)
LD-WSDOT	4.0	4.8
NM-WSDOT	2.5	3.0

Young's modulus, compressive strength and flexural strength for each concrete batch were measured after aging for 7 and 28 days, respectively. For each property, three samples were tested. The average values for Young's modulus, compressive strength and flexural strength for the NM-WSDOT and LD-WSDOT are shown in Table 4.2.

Table 4.2 Material Properties of Hardened Concrete

Mixtures	Young's Modulus ($\times 10^6$ psi)		Compressive strength (psi)		Flexural Strength (psi)	
	7 days	28 days	7 days	28 days	7 days	28 days
LD-WSDOT	2.95	3.4	3461	4432	499	748
NM-WSDOT	2.87	3.3	3655	4844	473	691

4.2 Test results of conditioned samples by freezing and thawing damage

Two groups of concrete samples made with the two aggregates sources (LD-WSDOT and NM-WSDOT) were conditioned to investigate the long-term performance and durability of concrete subject to freezing and thawing exposure. The measured data, including mass reduction, natural frequency, specimen dimensions, visual inspection of specimen condition, and fracture energy, are given in the following sections.

4.2.1 Results of concrete with LD-WSDOT aggregate source

4.2.1.1 Mass variance due to surface scaling

It was observed that the mass loss due to freezing and thawing damage in concrete samples made with LD aggregates mainly resulted from the scaling of paste and small mortar from the surfaces, as shown in Figure 4.1. The mass reduction of the first group of concrete samples made with LD aggregate with respect to the F/T cycles is shown in Figure 4.2, where the circles and bars denote the average values and error variances, respectively.

Figure 4.1 Surface Scaling of Concrete Samples with LD-WSDOT Aggregate Sources at 1,500 F/T Cycles.

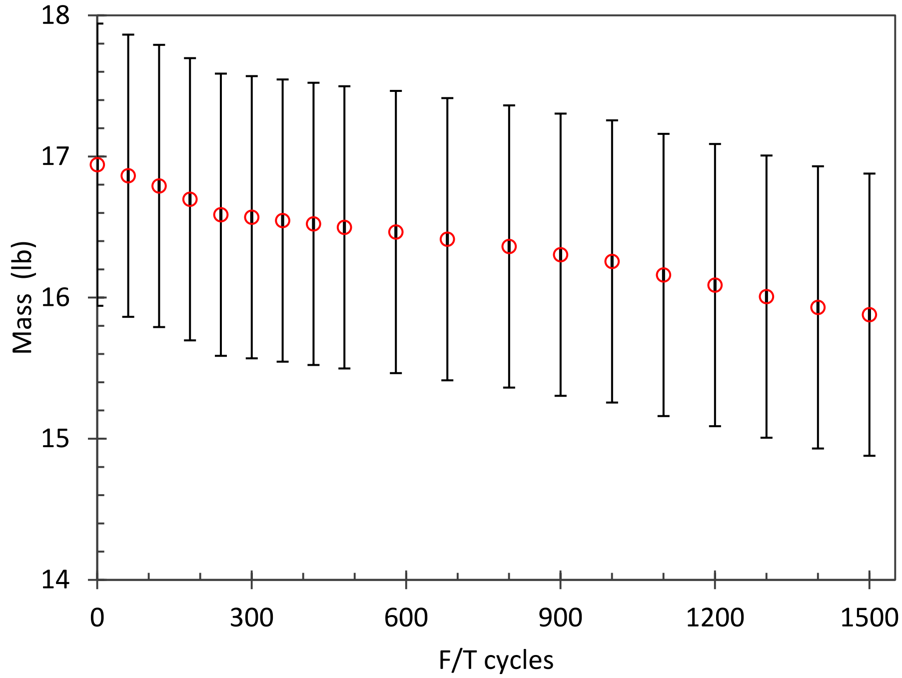


Figure 4.2 Mass Variance of Concrete Samples with LD Aggregate with Respect to F/T Cycles.

4.2.1.2 Transverse frequency

The dynamic modulus of elasticity was determined based on three repeated frequency measurements of six replicate concrete prisms. The effects of the number of F/T cycles on the transverse frequency are shown in Figure 4.3. As shown in Figure 4.3, the natural transverse frequency keeps decreasing due to the frost damage as the F/T cycles increase. With the accumulation of frost damage, the larger variance in the measured frequency among different samples becomes larger for conditioned samples with more than 800 F/T cycles.

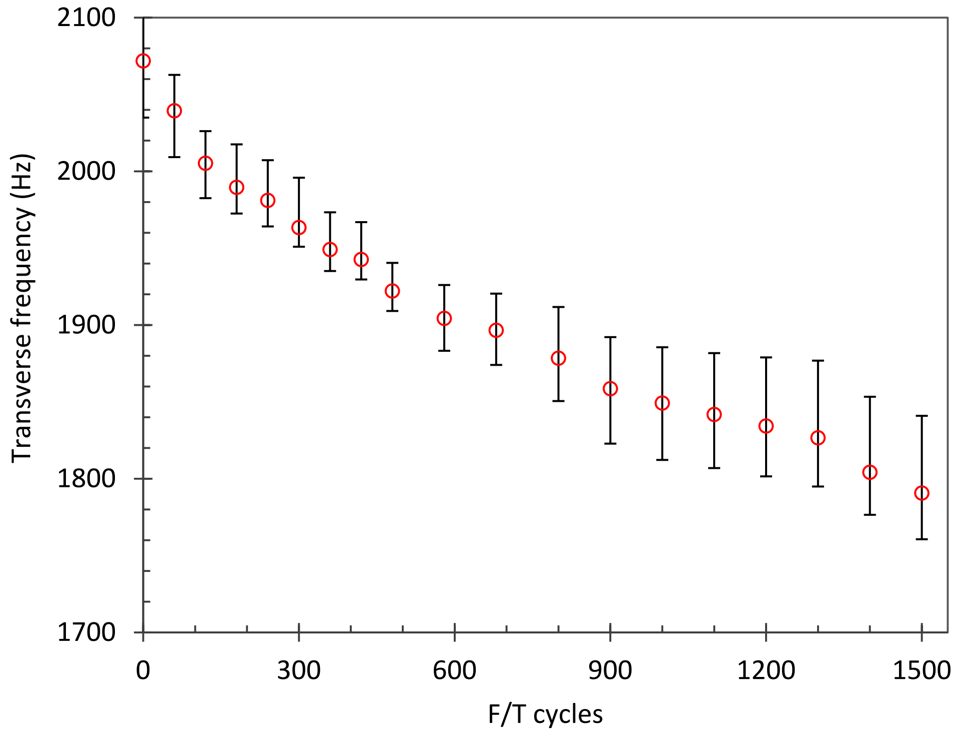


Figure 4.3 Variance of Fundamental Transverse Frequency of Concrete Samples with LD Aggregate with Respect to F/T Cycles.

4.2.1.3 Dynamic modulus of elasticity

The effects of the number of F/T cycles on the dynamic modulus of elasticity of the conditioned concrete samples are shown in Figure 4.4. Similar to the natural frequency, the dynamic modulus of elasticity decreases consistently as the F/T cycles increase. Nevertheless, the variance in the measured dynamic modulus among different samples is not as great as compared to that of transverse frequency since the mass reduction of the sample due to the frost damage has been accounted for as shown in Eq. (3.1). Thus, compared to the natural frequency, the dynamic modulus of elasticity is likely to be a more promising tool to reveal the frost damage in concrete materials.

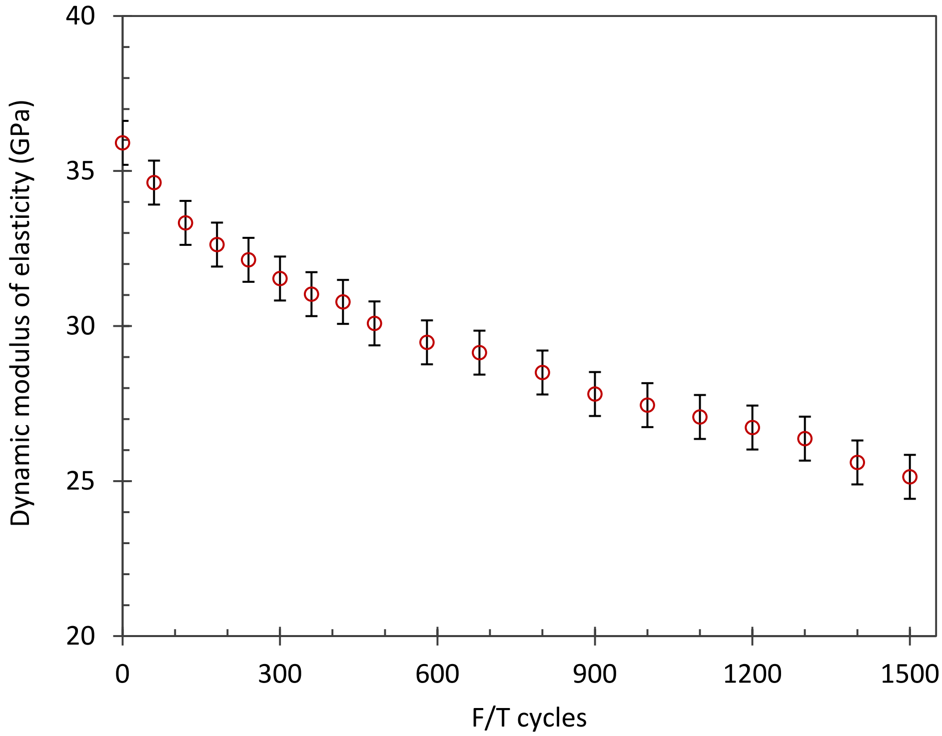


Figure 4.4 Dynamic Modulus of Concrete Samples with LD Aggregate with Respect to F/T Cycles.

4.2.1.4 Relative dynamic modulus of elasticity

The relative dynamic modulus of elasticity was calculated as the ratio of the dynamic modulus at a certain number of F/T cycles to the initial value before conditioning began. The change in the relative dynamic modulus of elasticity with respect to the F/T cycles is shown in Figure 4.5. Again, a larger variance among different samples occurs with the accumulation of frost damage for conditioned samples with more than 800 F/T cycles. According to ASTM C666, the samples are assumed to be failed when their relative dynamic modulus of elasticity has fallen below 50% of its initial modulus, and it is not recommended that specimens be continued in the test. However, as shown in Figure 4.5, even after 1,500 F/T cycles the relative dynamic modulus of elasticity is about 74.7%, which is still much larger than the benchmark 50% as defined by ASTM C666.

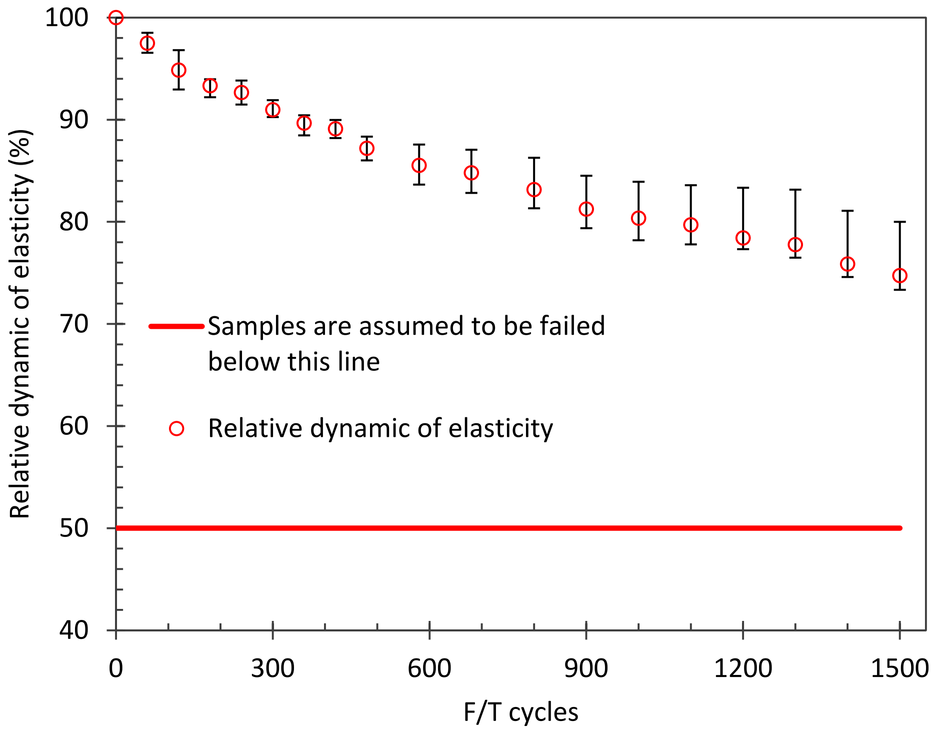


Figure 4.5 Relative Dynamic Modulus of Concrete Samples with LD Aggregate with Respect to F/T Cycles.

4.2.1.5 Fracture test results

The cohesive fracture energy is defined as the amount of energy a notched sample absorbs when it is broken into two halves. During the fracture test, two important parameters, peak load and fracture energy, can be simultaneously obtained, which can be considered as two inherent material properties to screen the material degradation of a concrete samples subjected to frost damage. The determination of fracture energy was performed following the procedures given by RILEM 50-FMC (1985).

Typical fracture test results for the notched fracture concrete sample are obtained from the load vs. displacement curves as presented in Figure 4.6. Initially, the displacement increases linearly with the applied load. During this period, the elastic strain energy stored in the specimen increases until the stored elastic energy is equal to the energy required to initiate the crack. The crack initiation begins following attainment of the peak load. The subsequent drop in load with the extension of the crack represents the softening region. During this softening period, the stored elastic strain energy decreases along with the applied load with respect to the crack extension until the test sample is fully broken into two halves.

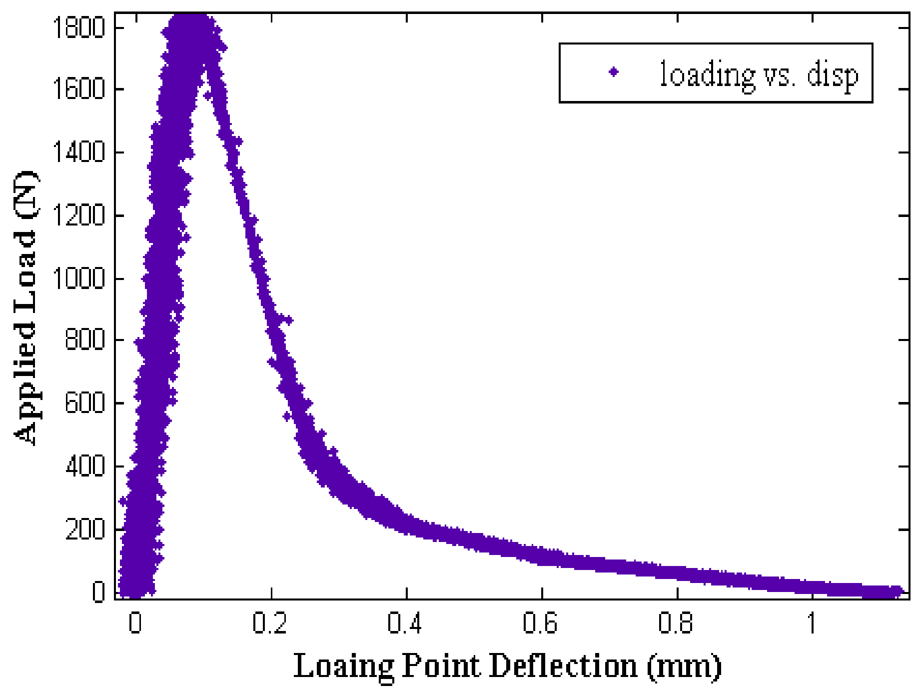
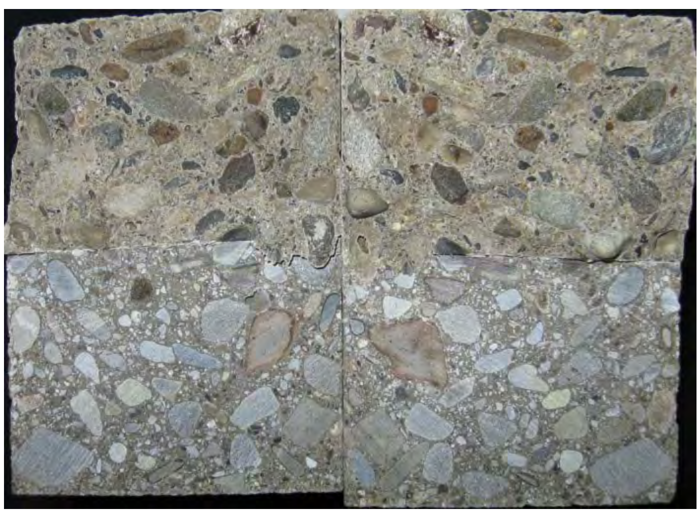


Figure 4.6 Load-displacement Curve of Notched Fracture Specimen under 3-point Bending.

The path of crack propagation in a notched sample during fracturing is shown in Figure 4.7(a), and the fractured surface after the sample was fully fractured into two halves is shown in Figure 4.7(b). It can be seen that after the concrete sample fractured, some aggregates themselves fractured, while others pulled out from the mortar. Based on careful examination of the fractured surfaces for different samples with different F/T cycles, the number and location of the fractured and pulled-out aggregates are random. The random distributions of the fractured and pulled-out aggregates reflect the anisotropic nature of the concrete material.

(a) Crack propagating path during fracture test



(b) Cross section of fractured samples

Figure 4.7 Crack Propagating Path and Fractured Cross Section of Conditioned Samples.

The peak load and fracture energy of ten sub-group samples conditioned at 0, 60, 120, 180, 240, 300, 500, 700, 900 and 1,500 F/T cycles are compared in Figure 4.8 and Figure 4.9, respectively. Based on the data reduction technique given in Section 3.3.5, the fracture energy for different samples with different F/T cycles are obtained and given in Table 4.3, where the sequence of the data for each sub-group is arranged from the minimum to the maximum for the convenience of later data analysis. Due to the sample limit of the freezing and thawing conditioning machine, three unequal size groups (6, 6, and 5 samples for each sub-group) of samples were conditioned each time. Blanks in Table 4.3 are associated with samples which were accidently failed during the notch preparations as well as those undergoing the catastrophic failure without the softening stage being recorded during the fracture test. In Table 4.3, the relative decreasing ratio of fracture energy (= $\Delta G_F/G_{F0}$) was calculated as well, and it is used as an indicator for degradation or decay rate of the material.

It is noteworthy that both the peak load and fracture energy increase at 60 F/T cycles compared to their initial values, which might be due to toughening effects associated with the ingress of moisture. However, after 60 F/T cycles, both the peak load and the fracture energy decrease with an increase in F/T cycles, indicating that the degradation due to the frost damage becomes more significant. As shown in Figure 4.8(b) and Figure 4.9(b), after 900 F/T cycles, the flexural peak load and fracture energy of the first group of concrete samples made with LD aggregate are 69.5% and 43.5%, respectively, compared to those of virgin samples; after 1,500 F/T cycles, the flexural peak load and fracture energy of the second group of concrete samples made with NM aggregate are 49.9% and 36.2%, respectively, compared to those of virgin samples.

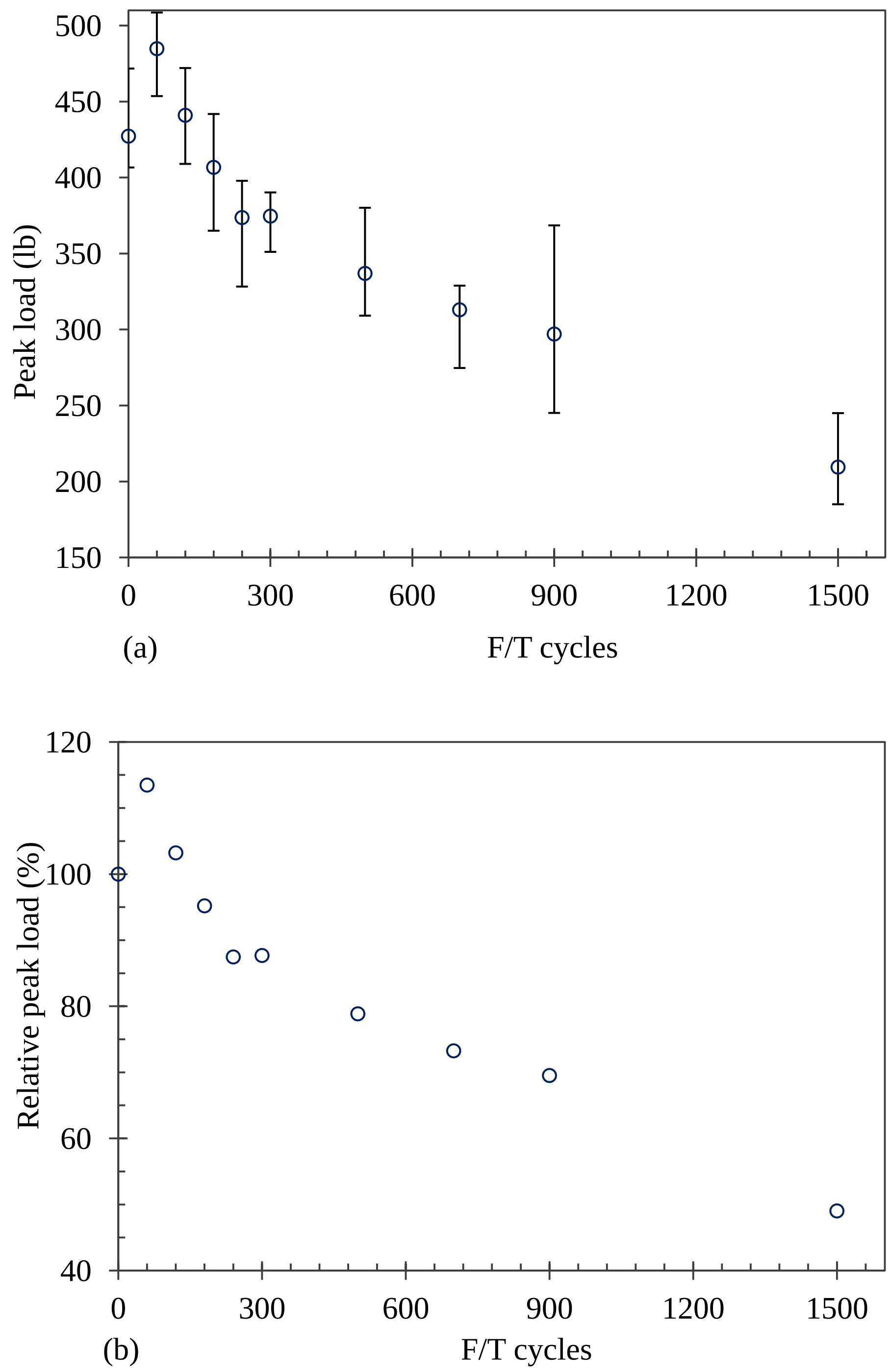
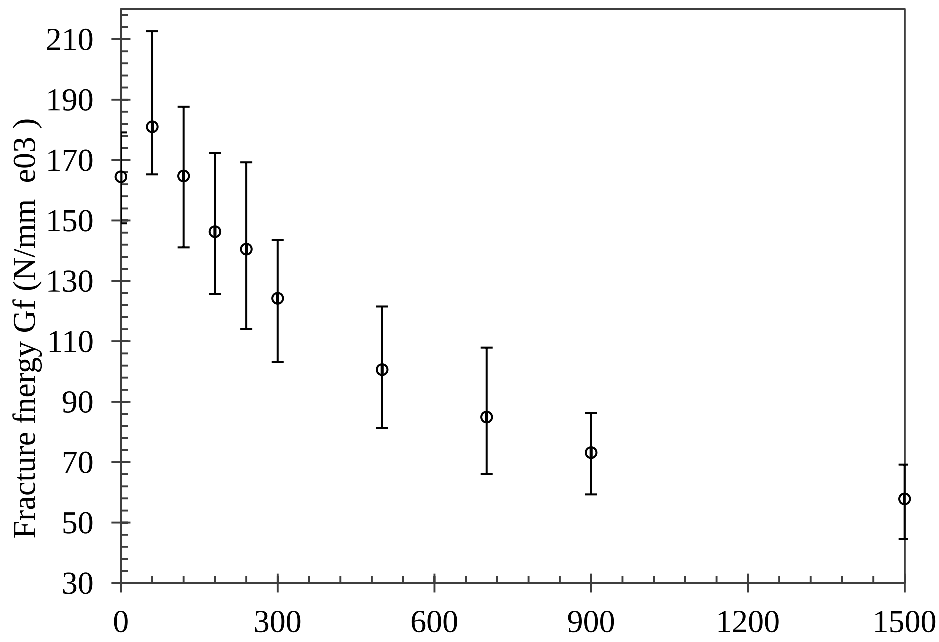
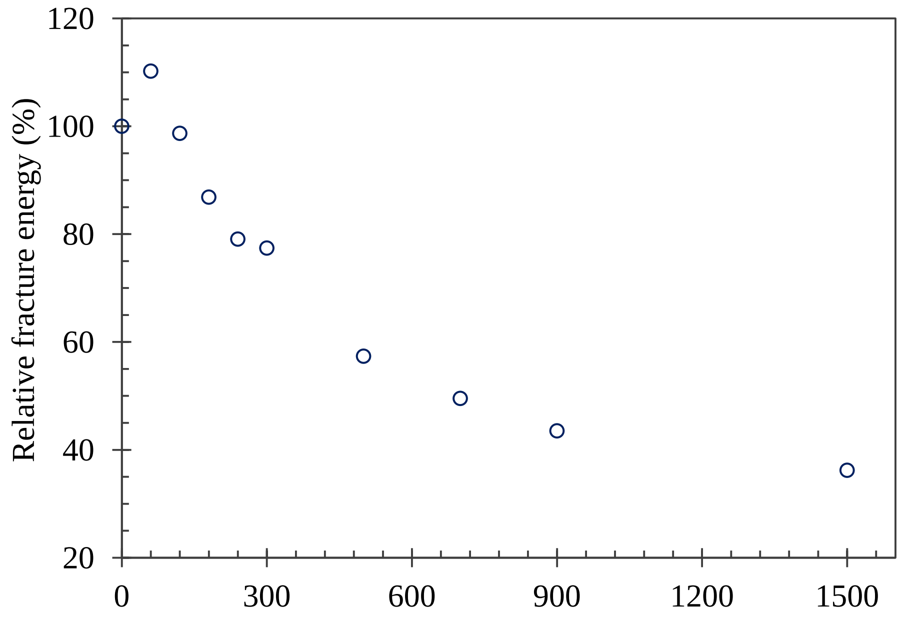


Figure 4.8 Peak Load of the Fractured Samples of Concrete with LD Aggregate with Respect to F/T Cycles.



(a)



(b)

Figure 4.9 Fracture Energy of the Fractured Samples of Concrete with LD Aggregate with Respect to F/T Cycles.

Table 4.3 Fracture Energy G_F (N/mm x 10^3) of Concrete Samples with LD Aggregate at Different F/T Cycles

F/T cycles		0	60	120	180	240	300	500	700	900	1500
Samples	1	149.1	165.3	141.1	125.6	114.0	103.2	81.4	66.1	59.3	44.7
	2	153.1	168.4	151.5	132.4	118.6	114.1	89.0	77.4	64.0	49.5
	3	160.0	174.3	164.0	143.2	138.9	128.3	100.6	81.5	73.5	55.1
	4	167.4	184.6	179.5	157.9	144.2	132.1	101.0	86.8	82.8	62.9
	5	178.2	212.6	187.7	172.4	158.1	143.6	110.4	89.9	86.2	65.8
	6	179.2				169.2		121.5	107.9		69.2
Average		164.5	181.0	164.7	146.3	140.5	124.2	100.6	84.9	73.2	57.9
COV*		7.7	10.6	11.7	13.0	15.4	12.7	14.3	16.5	15.9	16.7
Relative decreasing ratio (%)		0	-10.0	-0.12	11.1	14.6	24.5	38.9	48.4	55.5	64.8

* Coefficient of Variation

4.2.2 Results of concrete with NM-WSDOT aggregate source

4.2.2.1 Mass variance due to surface scaling

The surface scaling for the second group of samples made with NM-WSDOT aggregate source at their initial state, 900 F/T cycles and 1,500 F/T cycles are shown in Figure 4.10. From Figure 4.1 and Figure 4.10(c), it can be seen that the surface scaling in samples with the NM aggregates is more severe than that in those with the LD aggregate. This is likely due to the lower air content in the NM aggregate concrete than that in the LD aggregate concrete, leading to lower durability in the concrete samples with NM aggregates in terms of surface scaling.

(a) Virgin samples (prior to F/T testing)

(b) Samples after 900 F/T cycles

(c) Samples after 1,500 F/T cycles.

Figure 4.10 Comparison of the Surface Scaling of Samples with NM-WSDOT Aggregate Sources at Different F/T Cycles.

The change in mass for all eight concrete samples with the NM aggregate sources and their average values with respect to the F/T cycles are shown in Figure 4.11(a) and (b). As shown in Figure 4.11, at the first 300 F/T cycles, the mass reductions for all eight samples due to the frost damage are slight; after 300 F/T cycles, however, the mass reduction becomes greater and the mass for all the samples significantly decreases as the F/T cycles increases.

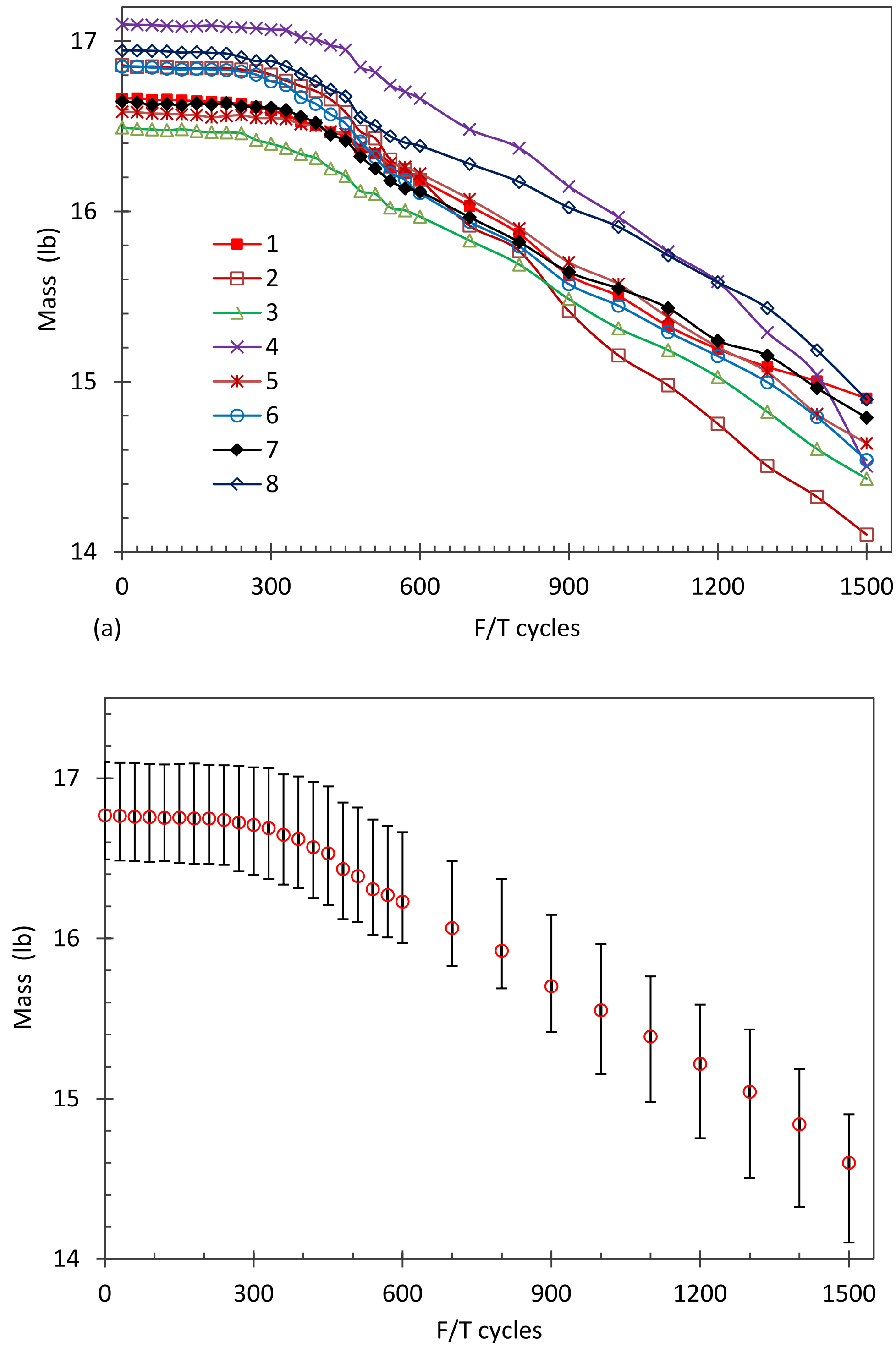
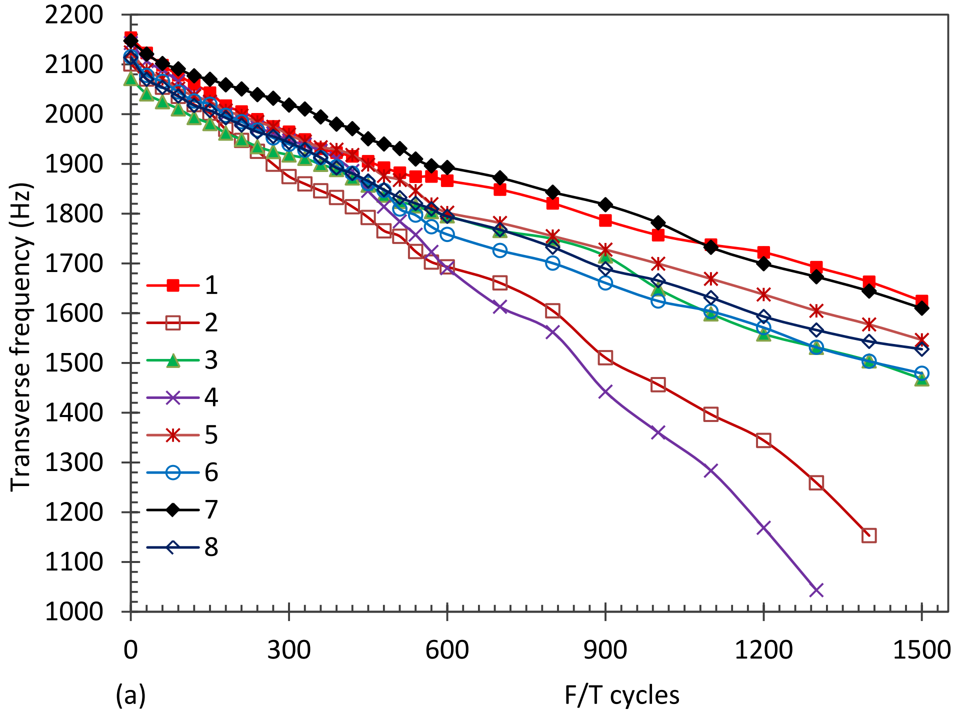


Figure 4.11 Mass Variance of Concrete Samples with NM Aggregate with Respect to F/T Cycles.

4.2.2.2 *Transverse frequency*

The variance of transverse natural frequency of concrete samples with NM aggregate with respect to F/T cycles is shown in Figure 4.12. Different from the mass reduction shown in Figure 4.11, the natural transverse frequency keeps decreasing in a similar ratio as the F/T cycles increasing. By comparing Figure 4.11 and Figure 4.12, it can be observed that the internal damage, which is responsible for the natural frequency reduction, exists as long as the concrete samples are subjected to frost action; in contrast the surface scaling, which is responsible for the mass loss, only starts at F/T cycles after a certain period of frost damage accumulation (300 F/T cycles in this test for concrete samples with the NM aggregate source).

As shown in Figure 4.12(a), after 600 F/T cycles, a larger variance of frequency among different concrete samples becomes more obvious, especially for Samples 2 and 4 which diverged from the remaining six samples. Extensive damage occurred in Samples 2 and 4 after 1,300 F/T cycles, and they were broken into two halves at 1,400 and 1,300 F/T cycles, respectively (see Figure 4.13). Thus, to better present the test data in a statistical way, the measured data of these two samples were removed when calculating the averaged value of natural frequency as shown in Figure 4.12(b).



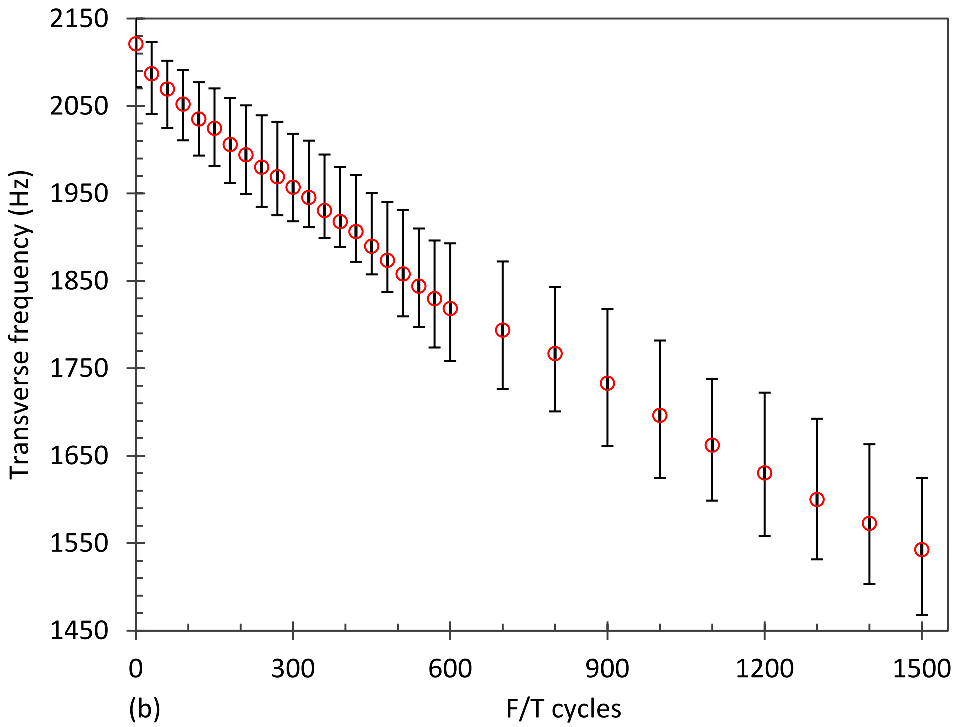


Figure 4.12 Fundamental Transverse Frequency of Concrete Samples with NM Aggregate with Respect to F/T Cycles.

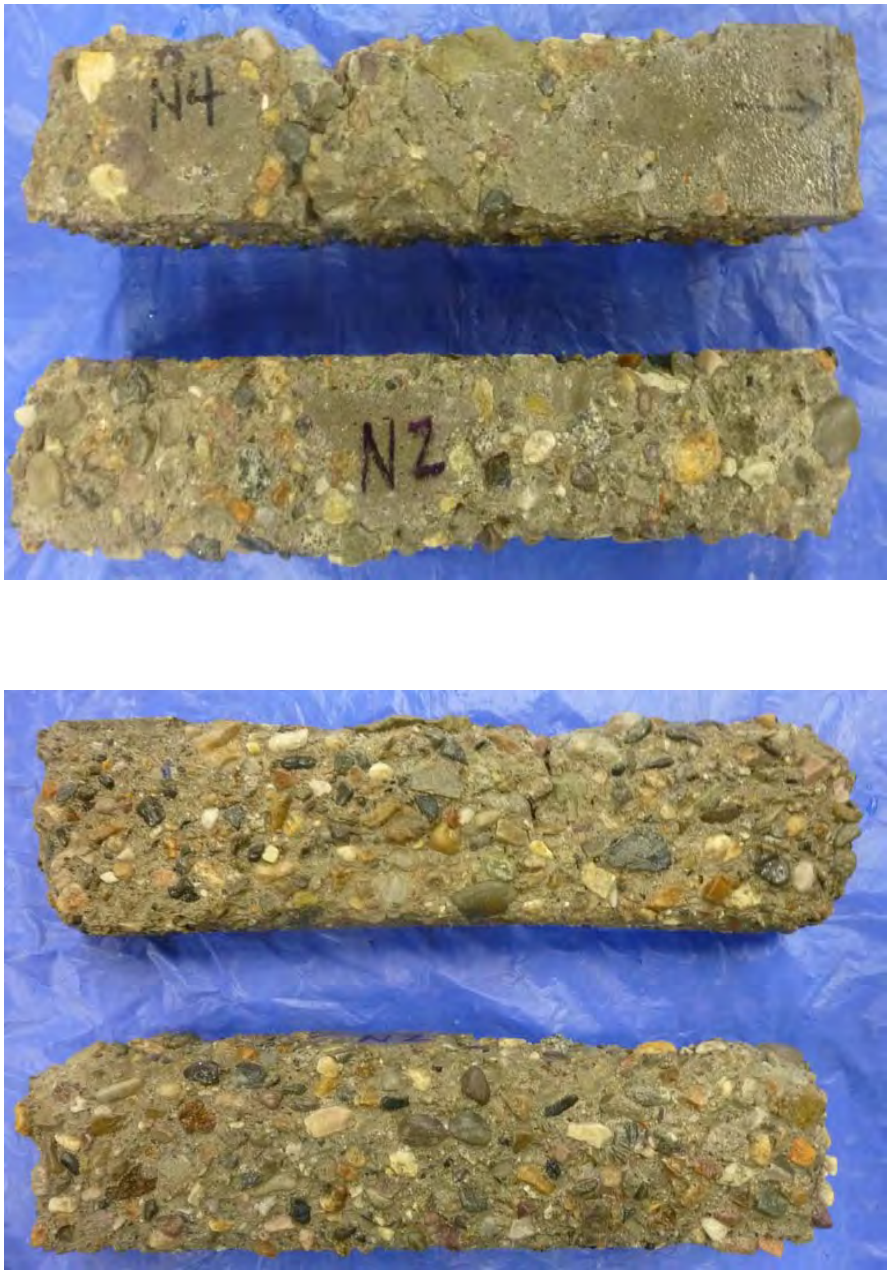


Figure 4.13 Failed Samples in the Second Batch of Concrete with NM Aggregate Sources Due to F/T Damage.

4.2.2.3 Dynamic modulus of elasticity and relative dynamic modulus

The variances of dynamic modulus of elasticity and the relative dynamic modulus of the conditioned samples with the NM aggregate sources with respect to the F/T cycles are shown in Figure 4.14 and Figure 4.15, respectively. Similar to the natural frequency, the dynamic modulus and the relative dynamic modulus of the two broken samples due to the frost damage significantly diverge from those of the remaining samples after 600 F/T cycles, and therefore they were discarded when calculating the averaged value as shown in Figure 4.14(b) and Figure 4.15(b). It is important to note that the relative dynamic modulus for all the samples with the NM aggregate source drops below 50% after 1,500 F/T cycles as shown in Figure 4.15(a), which implies, according to ASTM C666, that all the samples were failed.

By comparing Figure 4.15 and Figure 4.5, it can be observed that after 1,500 F/T cycles the relative dynamic modulus of the NM aggregate concrete samples is much lower than for the NM aggregate concrete samples. This phenomenon on one hand may be due to the fact that both the flexural strength and fracture energy of virgin NM concrete samples are lower than those of virgin LD concrete samples, while on the other hand it may also be attributable to the lower air content of the NM aggregate concrete samples.

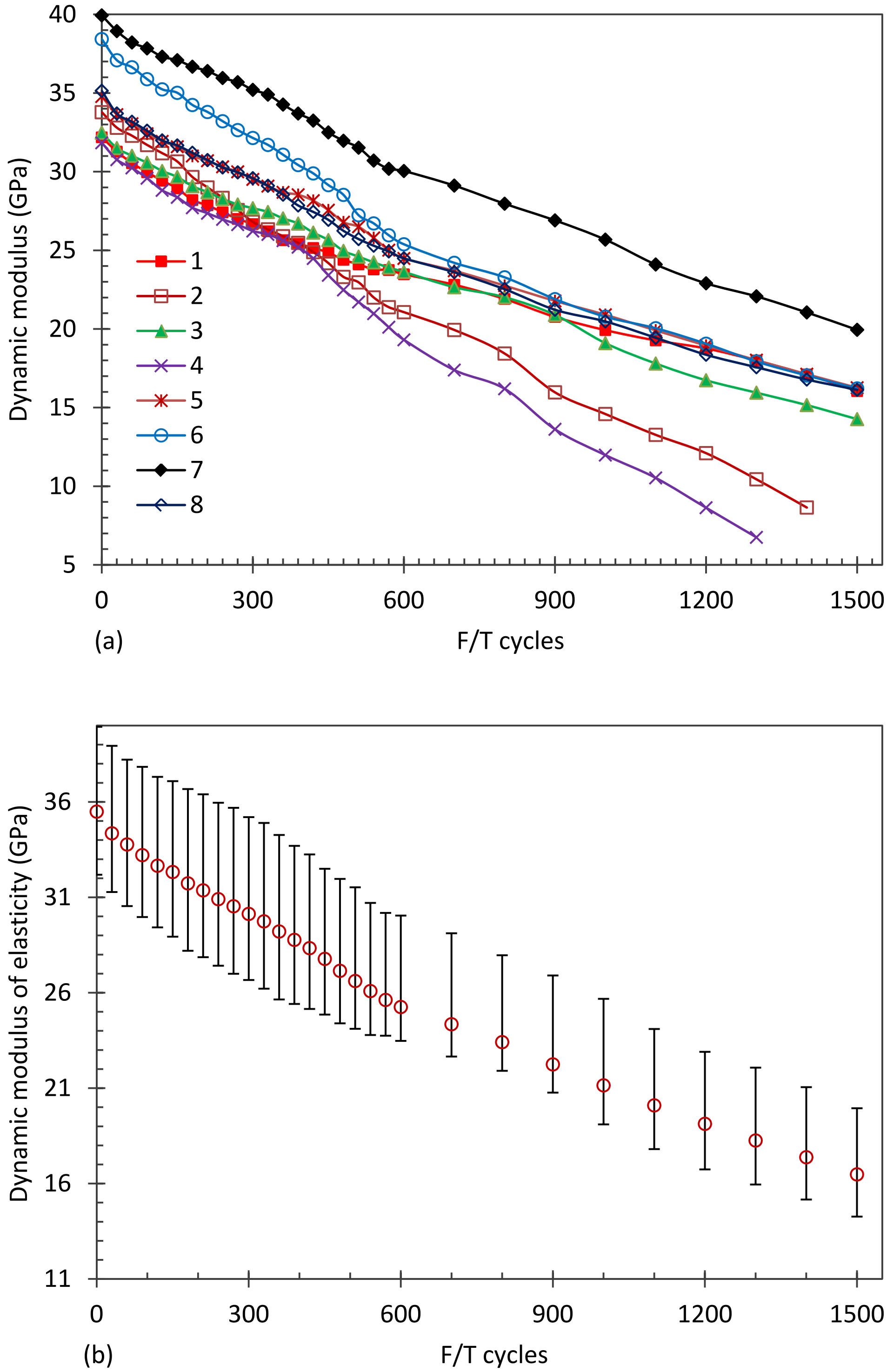


Figure 4.14 Dynamic Modulus of Concrete Samples with NM Aggregate with Respect to F/T Cycles.

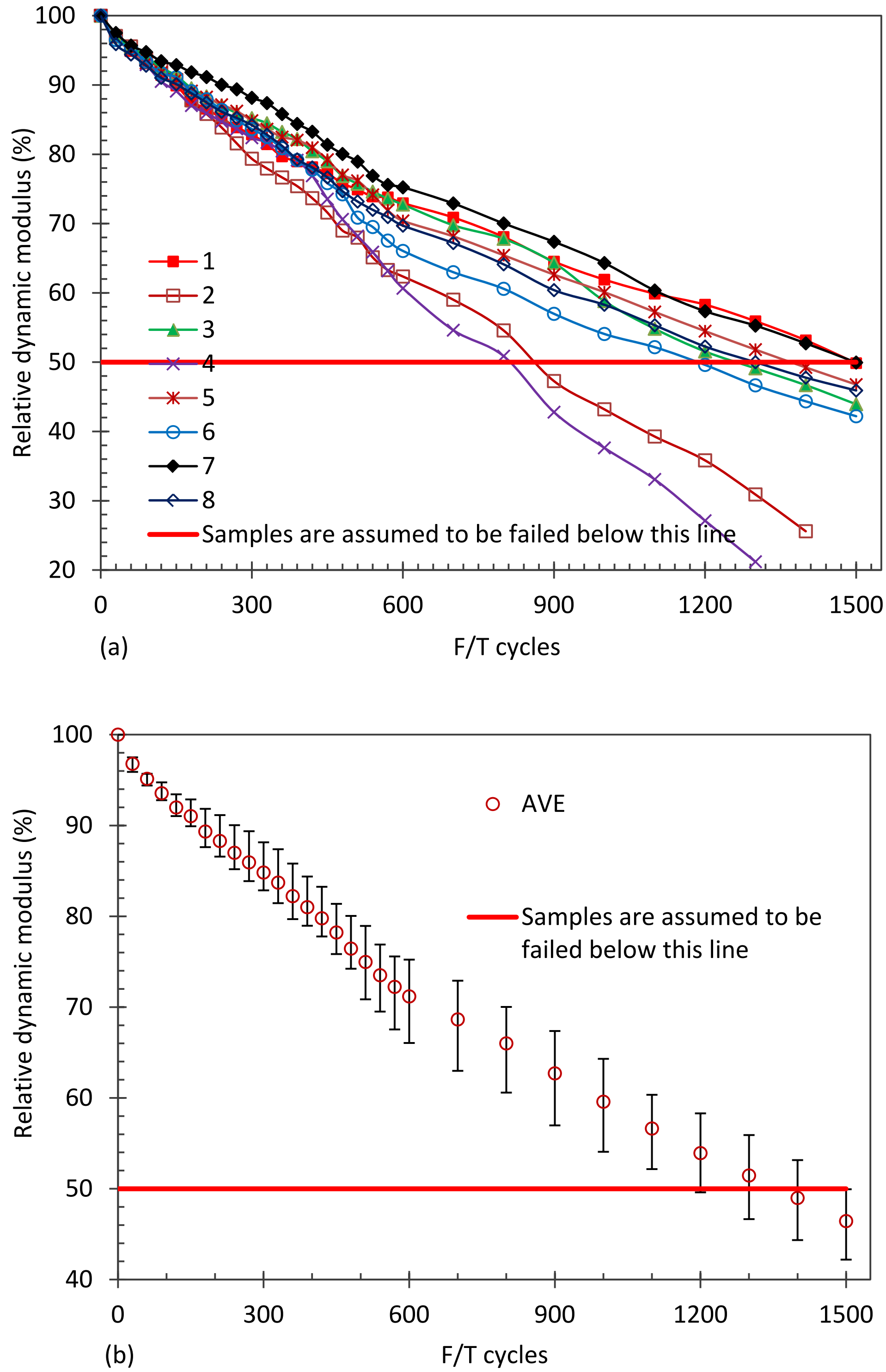


Figure 4.15 Relative Dynamic Modulus of Concrete Samples with NM Aggregate with Respect to F/T Cycles.

4.2.2.4 Fracture test results

For the second group of concrete samples with the NM aggregates, the fracture peak load and fracture energy of three sub-groups of samples with 0, 900, and 1,500 F/T cycles are summarized in Tables 4.4 and 4.5, respectively. Similarly, the sequence of the data for each group was arranged from the minimum to the maximum. As shown in Tables 4.4 and 4.5, after 900 F/T cycles the flexural peak load and fracture energy of the second group of concrete samples with the NM aggregates are 75.6% and 68.1%, respectively, compared to those of virgin samples. After 1,500 F/T cycles, the flexural peak load and fracture energy of the second batch of concrete samples with the NM aggregate are 50.5% and 42.8%, respectively, compared to those of virgin samples.

Table 4.4 Fracture Peak Load (lbs) for Samples with NM Aggregate Source at Different F/T Cycles

F/T cycles	Samples						Average	Relative decreasing	COV
	1	2	3	4	5	6			
0	375.7	392.6	403.7	415.2	430.8	444.3	410.4	0.0	5.6
900	263.7	287.6	309.5	311.4	335.6	352.7	310.1	24.4	9.0
1500	174.5	182.6	188.4	204.3	231.6	261.5	207.2	49.5	6.1

Table 4.5 Fracture Energy G_F (N/mm x 10^3) for Samples with NM Aggregate Source at Different F/T Cycles

F/T cycles	Samples						Average	Relative decreasing	COV
	1	2	3	4	5	6			
0	115.7	129.6	134.8	140.9	143.4	156.8	136.9	0.0	10.1
900	79.6	85.3	87.6	95.5	100.4	110.2	93.1	31.9	12.0
1500	43.8	49.7	57.5	62.2	65.4	73.7	58.7	57.2	18.4

4.2.3 Discussion of test results on frost damage caused by F/T cycles

Although the relative dynamic modulus of the first group of concrete with LD aggregates are higher than that of the second group with NM aggregates after 1,500 F/T cycles, the relative decreasing ratio of fracture energy of the LD aggregate concrete is much higher than that in the NM aggregate concrete as shown in Table 4.5 (i.e., 55.5% of LD vs. 31.9% of NM at 900 F/T cycles and 64.8% of LD vs. 57.2% of NM at 1,500 F/T cycles). If the air content of the two studied concrete were closer, this discrepancy might be even larger. The large discrepancies of the relative decreasing ratios of fracture energy between the LD and NM aggregate concrete with respect to the F/T cycles indicate that different aggregate degradation properties have an important effect on the freeze-thaw resistance of the concrete.

It should be noted that the basic assumption behind the dynamic modulus test is that the concrete material behaves homogeneously; thus, this global perturbation test might overlook some local defects caused by the frost damage, such as intergranular debonding in the interface interfacial transition zone (ITZ) between the cement paste and aggregates/quartz sands or intragranular defects and material decay (e.g., microcracks in both cement paste and aggregates). It was shown that the transition zone properties govern the damaging processes of concretes by enhancing transport processes within the concrete and facilitating ingress and penetration of the freezing medium and aggressive agents into the concrete microstructure (Cwirzen and Penttala, 2005; Vancura et al., 2011). In reality, concrete is a heterogeneous composite material rather than a homogeneous material. The nonlinear behavior of concrete is mainly attributed to the initiation, propagation, accumulation and coalescence of microcracks within the internal material structure such as aggregate/quartz sands, cement paste and ITZs, and the global material behavior of concrete (such as the dynamic modulus of elasticity) is influenced to a large extent by the material properties of the individual material constituents and their mutual interaction at the micro level.

By comparing Figure 4.5 with Table 4.3 or Figure 4.15 with Table 4.5, it can be seen that the relative decreasing ratio of fracture energy for concrete samples with either LD or NM aggregate is larger than the relative decreasing ratio of dynamic modulus in terms of relative dynamic modulus measured by the dynamic modulus tests, which implies that the fracture energy is a more sensitive parameter and therefore may be more effective at evaluating concrete degradation caused by the F/T damage. On the other hand, the larger magnitudes of the relative decreasing ratio of fracture energy for concrete with the LD aggregates at different F/T cycles than those for concrete with the NM aggregates indicate that the degradation of aggregates can be detected by the fracture test (i.e., fracture through aggregate or aggregate dominated ITZs).

Chapter 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

In this study, the long-term performance of concrete with both low-degradation (LD) and normal (NM) aggregates subjected to freezing and thawing damage was experimentally studied. An extensive literature review was conducted to better understand the mechanism of freezing and thawing damage in concrete, major factors that influence the frost durability of concrete, and common test methods to evaluate the long-term performance of concrete subjected to frost damage.

Two groups of concrete samples with low-degradation (LD) and normal (NM) aggregates were cast. Material properties of the fresh concrete and hardened concrete were evaluated and compared. Test Method for Resistance of Concrete to Rapid Freezing and Thawing (ASTM C666) - Procedure A was followed to condition all the test samples. Dynamic modulus of elasticity and fracture energy for both groups of samples at different freeze-thaw (F/T) cycles were measured through nondestructive modal and cohesive fracture tests, respectively.

Based on evaluations of surface scaling, natural transverse frequency, dynamic modulus and fracture energy for samples with the two different aggregate sources, the following findings/conclusions are obtained.

1. The dynamic modulus and fracture energy of the conditioned samples decrease with an increase in the number of F/T cycles, which reflects that frost damage and degradation in the concrete samples due to F/T conditioning can be effectively accumulated (accelerated) by using the F/T conditioning protocol (ASTM C666). The measured decrease also demonstrates that the dynamic modulus of elasticity

test is able to account for concrete degradation through the F/T conditioning. However, the change of dynamic modulus is sensitive to the surface scaling, which may not represent the degradation of concrete samples as a whole. In contrast, the cohesive fracture test is effective for evaluating the concrete material degradation through the F/T conditioning cycles.

2. Due to the higher air content (4.8%) in the concrete samples made with the LD aggregates, the surface scaling following F/T conditioning was found to be less severe than that in those with the NM aggregates, which had an air content of 3.0%. The cement paste near the specimen surface seemed to degrade faster than the inner parts of the specimen, as demonstrated physically by the surface scaling and loss of mass. Thus, loss of material (primarily cement paste) from the specimen surface, rather than degradation (local defects due to frost damage) throughout the whole specimen, has a larger effect on the measured dynamic modulus. In the NM aggregate concrete, with the lower air content of 3.0% which led to less resistance to surface scaling, it was found that both the surface scaling and internal damage, which are responsible for the natural frequency reduction, exist as long as the concrete samples are subjected to frost action. Thus, surface scaling and loss of mass due to frost action significantly influences the measured dynamic modulus, and this type of frost damage is not able to characterize the degradation of aggregates.
3. The rate of decrease in the ratios of fracture energy as a function of F/T cycles is greater in the LD concrete than in the NM concrete. The difference in the rate of decrease indicates that different aggregate degradation properties have an effect

on the freeze-thaw resistance of the concrete. It also indicates that the fracture test can be an effective method for evaluating aggregate degradation due to the F/T conditioning.

4. After 1,500 F/T cycles, the fracture energy of concrete samples with the LD and NM aggregates decreased by 64.8% and 57.2%, respectively, compared to those of virgin samples. The larger relative decreasing ratio of fracture energy measured by the cohesive fracture test than those by the dynamic modulus test implies that the fracture energy is a more sensitive parameter for evaluating concrete degradation. Since local defects are caused by frost damage, including intergranular debonding in the interfacial transition zone (ITZ) between the cement paste and aggregates/quartz sands and intragranular defects (microcracks) in both the cement paste and aggregates, the fracture test method is more effective in characterizing the accumulated damage in concrete than the dynamic modulus test, which is more of a global perturbation technique and largely affected by the surface scaling.
5. Considering that both the flexural strength and fracture energy of virgin NM concrete samples are lower than those for the virgin LD concrete samples, the higher rate of decrease in the ratio of fracture energy with increasing F/T cycles for the LD concrete than for the NM concrete indicates that: (1) both different aggregates (LD vs. NM) and differences arising from preparing the concrete batches at different times (especially in air content values) have an important effect on the frost resistance of the concrete; (2) aggregate degradation can be effectively evaluated by the cohesive fracture test (fracture through aggregate or

aggregate-dominated ITZs); and (3) the rate of degradation in the LD concrete samples was higher than that for the NM concrete samples.

6. Compared to the dynamic modulus test data, surface scaling and air content seemed to have less influence on the fracture test data. The fracture energy of the virgin concrete samples with NM aggregates and possessing a lower air content is smaller than that for the virgin concrete samples with LD aggregates and higher air content, demonstrating the lesser effect of air content (or air voids) on the fracture test. However, as the F/T conditioning cycle increase, the rate of decrease in the ratio of fracture energy of the NM concrete is smaller than that for the LD concrete, indicating the fast degradation of aggregate and ITZs in the LD concrete. Thus, the fracture test may be a more effective test method for examining the degradation of aggregate and ITZs in the concrete.
7. As demonstrated in the cohesive fracture test results, concrete with LD aggregates (with a degradation value of 31) degrades faster than the concrete with NM aggregates (with a degradation value of 59). Thus, degradation of aggregates is a contributing factor for overall concrete degradation.

5.2 Recommendations

The results of this study are limited to the specific aggregates and tests conducted. Based on the experimental program in this study, the following recommendations are suggested:

1. Based on the various test methods utilized in this project, the cohesive fracture test is an effective method to detect degradation in concrete (particularly in terms

of its aggregate and ITZs), and it could be used to evaluate degradation properties for different sources of aggregates.

2. The dynamic modulus test is significantly influenced by surface scaling in the specimens, reflecting degradation at the specimen surface and a loss in mass. The degradation caused by the surface scaling in the dynamic modulus measurement may overshadow the effect of internal damage and aggregate degradation due to the F/T conditioning cycle. Thus, the dynamic modulus test may not be effective in characterizing the degradation of concrete and depicting the local defects and internal damage caused by the F/T conditioning cycle. This test method should not be used to evaluate degradation properties for different sources of aggregates.
3. Further correlations of material properties of concrete (fracture energy) with WSDOT Test Method T 113 Aggregate Degradation Factor data should be empirically established.
4. Since only two specific sources of aggregates were evaluated in this study (LD vs. NM with respective degradation values of 31 vs. 59), the ability to confirm or suggest a new threshold degradation value for aggregates does not exist. It is recommended that further evaluation of concrete with aggregates from additional sources and with different degradation values be conducted to more rigorously establish a suitable threshold degradation value for aggregates.
5. To reflect local or internal defects caused by the frost damage, such as intergranular debonding in the interfacial transition zone (ITZ) between cement paste and aggregates/quartz sands or intragranular defects (microcracks) in both

cement paste and aggregates, additional tests capable of assessing and capturing local material properties, such as Vicker's indentation or nanoindentation tests, are recommended to be performed.

6. To better characterize the effect of aggregate degradation on the frost durability of concrete materials, other material properties of aggregates, such as pore size and its distribution, moisture content, and differences in absorption capacity among investigated aggregates, need to be chosen as closely as possible for the tests since each of them has significant effect on the concrete frost durability and test results. While for concrete casting, the air content and mix proportion among investigated aggregate concrete should be kept as close as possible, since they have a significant impact on the durability (e.g., surface scaling) of materials as demonstrated in this study.

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