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Interaction between Micro-Cracking, Cracking, and Reduced Durability of Concrete:
Developing Methods for Considering Cumulative Damage in Life-Cycle Modeling

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Interaction between Micro-Cracking, Cracking and Reduced Durability of Concrete: Developing Methods for Quantifying the Influence of Cumulative Damage in Life-Cycle Modeling

Introduction

Cracking may occur in concrete pavements or structures due to either mechanical or environmental loading. These cracks can reduce the overall strength and stiffness of concrete while permitting increased fluid ingress. As a result, these cracks can dramatically reduce the long-term durability performance of concrete.

The objective of this investigation was to quantify the role of cracking for the eventual incorporation of this information in life-cycle performance models. This research project included both a field condition assessment and a laboratory investigation to assess how cracking influences deterioration.

During the field condition assessment portion of this project samples were taken from five pavement sections that were selected based on

age, traffic, and overall performance to assess the existing damage that may be expected in the field. The cores were visually inspected, sectioned, and used for air void analysis, microscopic investigation, water sorptivity and absorption, and splitting tensile strength determination. These results were used as a baseline to assess the types of damage that merited laboratory investigation.

The second task began with the development of laboratory procedures to simulate the damage that may be expected in the field. After various levels of damage were introduced by freezing and thawing and mechanical loading, freezing and thawing, water absorption, and mechanical loading tests were performed to develop an understanding of how pre-existing damage accelerated the deterioration process.

Findings

The field condition assessment of concrete cores taken from five concrete pavement sections in Indiana demonstrated that field cracking exhibits a wide range of characteristics (i.e., cracks occur at the aggregate paste interface, through aggregates, and through the paste). In damaged pavements, the sorptivity near the surface of the pavement was substantially higher (up to 54%) than that the sorptivity near the middle or bottom of the pavement due to the presence of microcracking.

Laboratory studies have shown that water absorption is dependent on the characteristics of the cracks that developed. Freeze and thaw damage resulted in a well-distributed crack network which increased the water absorption in a fashion that was linearly proportion to the extent of damage (as assessed using dynamic modulus). Freeze and thaw damage appeared to result in the development of a well connected, coalesced crack network once the

relative dynamic modulus had degraded to 75% of its original value. Load induced damage does not appear to influence the water absorption substantially up to approximately 90% of peak load due to the discontinuous nature of the crack pattern. After the damage localized at approximately 90% of peak load, the water absorption increased but only locally due to the presence of a crack band (damage zone) region.

The presence of pre-existing load induced damage has been observed to influence the freeze and thaw durability of concrete however. In general, specimens with higher levels of pre-existing damage are less durable. Similarly the

presence of pre-existing freeze-thaw damage reduces the overall strength and stiffness of concrete with a knee point when the stiffness was reduced to approximately 75%. Freeze-thaw damage was found to alter the fracture behavior and damage development in concrete. Aggregate pull-out occurred when the dynamic modulus reduced to approximately 75% of its original value (i.e., below the knee point). In addition, non-linearity of the stress-strain response increased with an increase in freeze/thaw damage. The size of the damage zone increased with an increase in damage level (i.e., a decrease in the relative dynamic modulus).

Implementation

The following report has provided fundamental data to illustrate that microcracking can substantially alter the performance of concrete by increasing water ingress, reducing freeze-thaw resistance, and decreasing the mechanical performance of the concrete when it is tested in tension. It was found that distributed damage (like that associated with shrinkage or freeze-thaw) was more detrimental than mechanical loading on water ingress. Ideally this information would be incorporated into design and life-cycle prediction models to account for the role of cracking and damage in long-term

performance prediction. Specifically it appears that damage caused by freeze-thaw cycling can be more critical in terms of water absorption than mechanical loading. This report emphasizes the importance of controlling cracking caused by environmental effects. In addition it appears that once damage develops from freezing and thawing that this damage tends to accelerate water ingress leading to an increased saturation inside the concrete.

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16. Abstract It is becoming increasingly popular to utilize numerical simulation models to predict the long-term performance of concrete pavements and structures. The majority of these models have been developed using laboratory test data that considers concrete in an uncracked state. While uncracked concrete exists as the best case scenario, frequent cracking occurs in real structures that could have a profound impact on life cycle performance. Cracks from several sources may accumulate and interact thereby accelerating the deterioration of concrete. For example, the distributed cracking caused by freeze/thaw damage can substantially increases the rate of water absorption and reduces the load carrying capacity of concrete. To accurately simulate the performance of actual concrete facilities, the role of cracking and its cumulative effect on the changes of material properties should be accounted for in these models. The main goal of this investigation was to assess the influence of cumulative damage in concrete and to quantify its influence for use in life-cycle performance modeling. Samples were taken from five concrete pavement sections based on age, traffic, and overall performance to assess existing damage and to identify possible sources responsible for inducing the damage. These results were used as a baseline to assess the types of damage that merited laboratory investigation. After the field assessment, laboratory investigations were conducted to simulate the damage that may be expected in the field. After various levels of damage were introduced in laboratory specimens, durability tests (freezing and thawing and water absorption) and direct tensile test were performed to develop an understanding of how the pre-existing damage accelerated the deterioration process. Specifically, it was determined that cracks caused by freezing and thawing dramatically increase the rate and amount of water absorption while cracks caused by mechanical loading only increased the absorption in a local region. Further, freeze-thaw damage dramatically reduces the direct tensile strength and modulus of elasticity of concrete until the aggregates begin to pull out of the matrix. This results in a larger fracture process zone in the damaged concrete than in the undamaged concrete.			
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CHAPTER 1 INTRODUCTION

1.1 Background

Numerous portland cement concrete bridges and pavements have been constructed using either prescriptive or Quality Control/Quality Assurance (QC/QA) specifications which either prescribed initial mixture proportions or minimized material quality characteristics. However, as owners and public agencies begin to shift to specifications for pavements and structures which are based on anticipated long-term performance (thus requiring numerical simulations), it is essential that models used in these simulations provide an accurate representation of performance that can be expected in the field. As a result, the mechanisms responsible for the deterioration process must be clearly identified and quantified. With this information, simulation procedures can be developed to reward contractors (through the use of pay incentives) for practices and material that improve construction practices and thus result in longer lasting pavements and structures. By understanding the mechanisms that contribute to failure, it may be possible to better design transportation structures thereby preventing/delaying deterioration and increasing service-life performance.

Although some models [Thomas and Bentz, 2001, Ehlen, 1999, and Hoerner and Darter, 2000] have been proposed in the past that consider fundamental mechanisms responsible for deterioration, little work has been performed to understand how these mechanisms interact [Weiss and Olek, 1998]. For example, the interaction between freeze/thaw deterioration and fatigue cracking can result in cumulative damage that may act to accelerate deterioration of pavements or bridge decks. Micro-cracks due to

mechanical loading may permit the ingress of excessive water, thereby increasing the susceptibility to freeze/thaw damage. Recent studies [Krauss et al., 1995] have shown that even small cracks (0.05mm) can act to increase the permeability of the system and thereby accelerate deterioration. The chain reaction that can occur as a result of the interaction between damage caused by several sources is anticipated to be responsible for a more accelerated deterioration of the concrete structures than what would be generated by any one mechanism independently. Unfortunately, cracking which may significantly influence the life cycle (LC) behavior of the structure may not be accounted for in many of the existing life cycle cost (LCC) models. This research investigates the potential relationship between the development of damage and LC performance.

Despite the recent attention to durability performance in cementitious material, little information exists on the influence of less than ideal curing conditions on the durability of concrete structures. For example, durability parameters such as freeze/thaw resistance or chloride penetrability have typically been measured using pristine concrete samples that have been well cured. Considering that most field concrete has undergone drying and thermal cycling, it is also likely that such concrete may have developed microcracks. As such, it may be safe to say that in real life concrete pavements and structures are frequently less than pristine with respect to microcracks.

Durability parameters measured using well-cured undamaged specimens may be misleading and may overestimate performance [Weiss, 2003]. For example, recent investigations by Guse and Hilsdorf [1997] showed that surface cracking in concrete can

substantially increase the surface permeability even though these micro-cracks do not have a substantial influence on the mechanical behavior of concrete. In a similar study, Wiens et al. [1997] observed that fine, distributed micro-cracks permitted water penetration to a depth of 30mm. They also observed that water and deicing salts could penetrate these cracks and thereby cause a significant weight change during freeze and thaw cycling. Similarly, Fagerlund [1997] showed that during repeated freezing and thawing cycles, water absorption would become accelerated as a result of micro-cracking, thereby accelerating freeze and thaw damage. By using well-cured undamaged specimens as a baseline some models may predict an overly optimistic life cycle performance that is not representative of the behavior of the concrete actually used.

1.2 Objectives and Scope of Work

The objectives of this investigation was to quantify the interactions between damage mechanisms that may result in cumulative damage that can lead to accelerated deterioration with the goal of using this information for LCC models. The main tasks of this research project included literature review and field condition assessment, laboratory investigation of deterioration.

During the first task, a state-of-the-art literature review was performed to gather information on the current state of knowledge on several sources of damage most frequently encountered in transportation structures including freeze/thaw, drying shrinkage, and mechanical loading. Information on the techniques of detecting and

quantifying damage in concrete was also collected. In addition to the literature review, samples were taken from five pavement sections (including a 25 year old un-trafficked section) based on age, traffic, and overall performance to assess existing damage and to identify possible sources responsible for inducing damage. The cores were visually inspected, sectioned, and used for air void analysis, microscopic investigation of cracking, water sorptivity and absorption, and splitting tensile strength. These results were used as a baseline to assess the types of damage that merited laboratory investigation.

The second task began with the development of laboratory procedures that simulated the damage that may be expected in the field. After various levels of damage were introduced, durability tests including freezing and thawing and water sorptivity were performed on both pristine and damaged concrete to develop an understanding of how the pre-existing damage accelerated the deterioration process. During the testing process, non-destructive testing such as resonant frequency and acoustic emission was employed to monitor damage development while additional testing procedures (e.g., optical microscope, scanning electron microscope, vacuum epoxy impregnation, and image analysis) were used to assess the effect of damage at various times.

1.3 Organization of the Report

This report consists of seven chapters. Chapter 1 introduces the background, research objectives, and scope of work. Chapter 2 presents a brief up-to-date review of literature related to this study. It summarized the damage mechanisms that could occur at

concrete structures and the techniques of detecting and quantifying damage in concrete. Chapter 3 focuses on the field condition assessments including the procedure of taking samples and the results of laboratory evaluation. Chapter 4 provides the results and analysis on how the damage induced by tension or freezing and thawing influences the water transport and the electrical conductivity of concrete. Chapter 5 gives the information on how damage is developing while tensile load is applied and how load induced damage affects freeze and thaw durability of concrete. Chapter 6 presents the results and analysis on quantifying the freeze and thaw damage and its effects on direct tensile strength, modulus of elasticity, and fracture behavior of concrete. Chapter 7 contains the summary and conclusions of this study.

CHAPTER 2 LITERATURE REVIEW PERTAINING TO CAUSES, MECHANISMS, AND DETECTION OF CRACKING AND WATER TRANSPORT IN CONCRETE

2.1 Damage of Concrete by Freezing and Thawing Action

There are two basic forms of deterioration, including internal cracking due to freezing and thawing cycles, and surface scaling due to freezing in the presence of deicer salts. Volume change from water to ice when freezing may cause deterioration either of the hardened paste or of the aggregate, or both. Freezing and thawing damage mechanisms are not well understood, but the way to make concrete resistant to freezing and thawing is well recognized. Experimental data from both laboratory and field has shown very conclusively that well-distributed air voids can provide pressure release and improve the freeze and thaw resistance [Pigeon, 1997]. However, when concrete is critically saturated, even well air-entrained concrete may suffer from freeze and thaw damage. The following review focused on the damage mechanisms, damage patterns as well as how the freezing and thawing damage influenced the transport properties and accelerated the deterioration of concrete.

2.1.1 Freeze and Thaw Damage Mechanisms

Damage mechanisms by freezing and thawing in concrete are very complex, especially in the presence of deicing salts. However, various theories have been developed to explain the deterioration of concrete by freezing and thawing.

2.1.1.1 Hydraulic Pressure Theory

Water expands 9% by volume during freezing. When the pores are critically saturated, extra water is expelled from the freezing site into the matrix. The drainage of viscous water along pore walls will induce hydraulic pressure, the magnitude of which depends on the rate of freezing, degree of saturation, pore structure, and the length of flow path to the nearest void for the water to escape [ACI Committee 201, 1992]. When the pressure exceeds the tensile strength of concrete, cracking would occur. Powers and his coworkers [Penttala, 1998] also noticed that the freezing rate influenced the damage mechanisms. Hydraulic pressure governs the freezing deterioration at a fast freezing rate and induces cracks in the matrix.

2.1.1.2 Osmotic Pressure Theory

During freezing, most of water was found to diffuse toward, not away from, the sites of freezing, and the expansions decreased with the increase of cooling rate. This is contrary to the hydraulic pressure theory. Powers attributed this phenomenon to osmotic pressure [Powers, 1975]. The water in concrete pores contains some dissolved ions and salts, which causes an initial super-cooling. As temperature drops to its freezing point, ice crystals will form in the larger capillaries, thus increasing the alkali concentration of the unfrozen solution at the freezing sites, which raises the osmotic potential of the unfrozen solution causing water to be sucked from surrounding smaller pores into the freezing sites. The suction dilutes the solution allowing further growth of the ice body. When the pores are filled with ice and solution, any further crystallization will induce the pressure buildup [ACI Committee 201, 1992]. The osmotic pressure develops in the direction

toward the capillary pores. Therefore, the total pressure required to force the capillary water out of the pores will be a combination of the hydraulic pressure and osmotic pressure, and this combined pressure would contribute to stressing the concrete.

The use of deicing salts would increase the salt concentration in the pore solution, resulting in larger osmotic pressure. This may be one of the reasons why the concrete with high chloride content shows low freezing and thawing resistance. The mechanisms of scaling during freezing in the presence of deicing salts can be partially explained by osmotic theory. Pore water in the surface layer, which contains the highest concentration of salts, would freeze at low temperature because salt reduces the freezing point of water. A deep layer, where salt concentration is lower may freeze at the same time. The combined effects of temperature gradient and salt concentration gradient along the depth will probably keep the intermediate layer between the surface and the deep in the liquid state, and it would freeze upon intensive cooling. The dilative pressure resulted from this delayed freezing may cause the scaling of surface layer because it cannot be relieved through the frozen layers [Yamato et al., 1987].

2.1.1.3 Thermodynamics and Surface Force Considerations

Water in the largest pores freezes first, and water in small pores cannot freeze or would freeze at very low temperature due to the high surface forces in small pores [ACI Committee 201, 1992]. These surface forces result in high chemical potential of water, and thus hinder ice crystal formation. The unfrozen water tends to reduce its potential and migrates to locations where it is able to freeze, such as the larger pores. As a result,

drying shrinkage occurs in the gel structure while ice accumulates in large pores or cracks, leading to the formation of new cracks or extension of old ones. At a lower freezing rate the diffusion of gel water is the dominant feature causing drying shrinkage in the gel structure, whereas at a fast freezing rate diffusion of gel water does not have time to evolve and hydraulic pressure governs the freezing deterioration and induces crack in the matrix [Penttala, 1998].

2.1.2 Damage by Freezing and Thawing

2.1.2.1 D-Cracking

Series of vertical cracks visible on the surface and most closely spaced at pavement joints and edges are called D-cracking as shown in Figure 2.1.

Figure 2.1 D-Cracking in Concrete Pavement by Freezing and Thawing

In the midwest of US, D-cracking is due to the destruction of the aggregate by freezing and thawing [Schwartz, 1987]: “D-cracking is a form of portland cement concrete deterioration associated primarily with the use of coarse aggregates that

disintegrate when they become saturated and are subject to repeated cycles of freezing and thawing. It is defined by a characteristic crack pattern that appears at the wearing surface of the pavement as a series of closely spaced fine cracks adjacent and generally parallel to transverse and longitudinal joints and cracks and to the free edges of the pavement. It is generally accepted that pore size is the most important characteristic of coarse aggregate influencing its susceptibility to D-cracking. It is generally agreed that the brand or composition of cement does not significantly influence D-cracking.”

D-cracking occurs around joints and edges of pavements where concrete exposes to wet and dry cycles at both the top surface and sides of slabs. Also curling and warping would induce stress concentrations at corners and edges of the concrete slab, which make it more susceptible to D-cracking [Sawan, 1987].

In the northeast of US, D-cracking may occur as a result of improper air void system. Concrete near the joints and edges is more susceptible to reach its critical degree of saturation and upon freezing concrete matrix may be fractured. When D-cracking is present, both the aggregate and the air-void parameters should be determined.

2.1.2.2 Scaling

Scaling often manifests itself as a separation of a thin layer of the top surface from the body of the concrete, and this thin brittle layer then crumbles under traffic, leaving the aggregate exposed. Surface scaling of concrete is aggravated by the widespread use of de-icing salts. The areas showing the greatest deterioration are usually

where ponding of water and salt solutions or continuous wetting has occurred during repeated cycles of freezing and thawing [Walker, 1997, Jakobsen, 1998, and Bayomy, 2000].

2.1.2.3 Internal Paste Cracking

Internal Paste cracking in concrete is due to the lack of well distributed air void system when concrete is critically saturated. It frequently occurs at the paste-aggregate interface, around the air voids, as well as in the highly porous regions of paste [Jakobsen, 1998]. Figure 2.2 shows the typical pattern of internal paste cracking by freezing and thawing. The concrete is extensively cracked. In the outer part of the sample, cracks are observed perpendicular to the surface, whereas in the interior part they are parallel to the surface.

Figure 2.2 Typical Internal Paste Cracking of Concrete Observed from Epoxy-Impregnated Thin Section (45×75mm) [Jakobsen, 1998]

2.1.2.4 Early Age Damage by Freezing

Concrete can be seriously damaged by freezing during fresh state. Signs of damage may include: cracking, flaking or scaling, softness, weakness, friability, unusual whiteness, and emitting a dull sound when struck lightly with steel hammer [ACI, 1968].

Casts of ice crystals are evidence of early freezing. If the concrete is fragmented and casts of ice crystals are abundant on almost all of the fragments, this indicates that the concrete was forced apart by the expansion of the ice formed before the paste achieved final setting [Walker, 1997].

2.1.3 Air Entrainment

Air voids are entrained into concrete to improve its resistance to the freezing and thawing damage. However, the distribution of air is of utmost importance to the freezing and thawing resistance. The size and distribution of air voids can easily be observed using epoxy impregnation [Jakobsen, 1998]. Concrete with well sized and distributed air void can resist the freezing and thawing damage, and concrete with unevenly distributed and/or agglomerated air voids are susceptible to freezing and thawing attack. Entrapped air is typically rather large (e.g., 1-5 mm) and irregularly shaped, and unevenly distributed. Very large air void are often observed in improperly compacted concrete.

2.1.4 Effects of Freeze and Thaw Damage on Durability of Concrete

Concrete will crack or spall off when it is subjected to freezing and thawing cycles. The spalling or scaling directly affects the smoothness of concrete pavements, and the interplay among scaled surface, freezing and thawing cycles, and traffic will undoubtedly accelerate the further deterioration of the pavements. The increasing amount and interconnection of internal micro-cracks and cracks due to freezing and thawing will facilitate the transport of water and increase the chloride permeability. This will aggravate the deterioration and cause the damage accumulation in the next freeze and thaw cycle. Banthia and Mindess [1989] studied the influence of freezing and thawing cycles on the permeability of cement paste at an age of 4-72 hours after casting. They found that the permeability of specimens damaged by freezing and thawing increased by an order of magnitude as compared with the normally cured specimens. However, Moukwa [1989] showed that there was no significant difference in the penetration of chloride ions between the mortar specimens subjected to the freezing and thawing in seawater from -7°C to 12°C and those exposed to seawater at -1°C . The apparent discrepancy of these findings may arise from the complicated nature of the influence of freezing and thawing cycles on the water and chloride permeability of cementitious materials. Mitsuru et al. [1994] measured the chloride permeability of normal weight and light-weight concretes under freezing and thawing cycles. The results showed that the chloride permeability of the normal weight concrete with an air content of 5.3% changed little with the repeated cycles of freezing and thawing, but the lightweight concretes made with fully saturated expanded shale aggregates exhibited

extremely high chloride permeability at any air content, when they were subjected to a single freeze and thaw cycle.

2.2 Forms of Damage at Early Age

Although concrete deterioration is usually a medium to long term process, the onset of deterioration and its rate may relate to the damage that originates at the time of construction such as defects and flaws, or in the early age of the pavement life such as plastic shrinkage cracking and drying shrinkage cracking. Damage incurred at the early age not only facilitates the ingress of water and aggressive chemicals making concrete more susceptible to both physical and chemical attacks, but also reduces the loading carrying capacity of the structure, thus accelerating the deterioration in the presence of external loads. To accurately understand the life cycle performance of concrete structure, it is important to gain an understanding of the basic forms of damage that may occur at the early age.

2.2.1 Construction and Design Defects

Construction defects such as honeycombing due to low workability concrete or inadequate compaction, and joint deficiency may provide channels for water and chemicals to enter into the concrete and attack the matrix itself. Design may influence the performance of concrete in a number of ways. For example, the selection of well-graded aggregates and proper mix proportions may produce a dense concrete, which is more resistant to the freeze and thaw damage. Appropriate choice of aggregates and constituent

materials can produce more durable concrete with less susceptibility to the physical and chemical attacks, such as freeze and thaw attack and alkali-silica reactions. Over-batching of water and under-batching of cement are construction errors that may lead to more porous and less durable concrete.

2.2.2 Plastic Shrinkage Cracking

Plastic shrinkage is a form of drying shrinkage and occurs on the surface of fresh concrete within the first few hours after placement, while the concrete is still plastic and before it has attained any significant strength [Cabrera, 1992]. Plastic shrinkage cracking is caused by the excessive evaporation of the water from the concrete surface due to improper curing procedures for a concrete under the climatic conditions existing at the time of placement. Plastic shrinkage cracking is a big problem for large flat structures, such as pavements, in which the exposed surface area is high and the sub-base underneath concrete overlay offers a high degree of restraint [Banthia et al., 1994]. Plastic shrinkage cracks will facilitate the penetration of detrimental agents, and make concrete structure more permeable, thus less durable.

2.2.2.1 Mechanisms of Plastic Shrinkage Cracking

The top surfaces of concrete are subject to evaporation and consequently loss of the mix water. The rate of evaporation is dependent upon ambient conditions such as temperature, wind speed and relative humidity. The water lost by evaporation is usually replaced by water rising to the surface of concrete by bleeding. When the rate of

evaporation of water from the surface exceeds the rate at which the water can be replaced by bleeding, a local reduction in volume occurs. The movements associated with the reduction in the volume of top layer of concrete are restrained by the layer of concrete that lies immediately below and is not subjected to the volume change. The restraint from the lower layer of concrete causes tensile stresses to be built up in the surface layer and, because the material still has very low strength at this early age, cracks occur [Kay, 1992].

The mechanism of plastic shrinkage cracking can be described based on the action of capillary pressure. As evaporation continues, water is removed from between particles near the surface, leading to a formation of a curved surface. This tends to pull the capillary walls together, causing horizontal and vertical contraction. The contraction increases with the decrease of the capillary diameters, resulting in the development of surface tensile stress.

2.2.2.2 Cracking Patterns

Plastic shrinkage cracking is typically in a V shape, in which crack opening of the upper (2-3 mm) surface is wider, but drops rapidly along depth. Sometimes, plastic shrinkage cracks may be extended to the full depth of concrete members by interacting with other damaging mechanisms such as drying shrinkage or external loading. Plastic shrinkage cracks occurs when concrete matrix is weak, indicating that plastic shrinkage cracks tend to pass through the matrix rather than through the aggregate [Kay, 1992]. If the cracks go through the aggregate particles and cause them to break, the cracking

should not be considered plastic cracking. If any little bridges of paste are seen that tie the two sides of a crack together (Figure 2.3a), or any uncracked aggregate particle bridges the crack, or the crack plane exists all around the particle, or if the aggregate is firmly attached to the hardened paste on either side of the crack, it can be concluded that the crack was caused by plastic shrinkage. The path of a plastic shrinkage crack is usually quite tortuous with irregular forms and soft edges because it is deflected by each piece of aggregates [Jakobsen, 1998] as shown in Figure 2.3b.

Figure 2.3 Plastic Shrinkage Cracking, (a) a Small Bridge of Paste across a Crack, and (b) Tortuous Crack Path

Bleeding is a type of defect formed in concrete before setting. Bleeding is recognized by zones of paste with a higher capillary porosity at one side of aggregate grains and by a generally higher capillary porosity of the surface compared to the inside of the concrete. Extensive bleeding can form channels through the concrete recognized by bands of higher capillary porosity. Typically, adhesion cracks are also observed in concrete suffering from bleeding.

2.2.3 Drying Shrinkage Cracking

Concrete undergoes volumetric changes (shrinkage) when it is subjected to a drying environment, and the magnitude of the drying shrinkage depends mainly on relative humidity and temperature of the environment, concrete geometry, and proportioning. The presence of drying shrinkage cracking in concrete reduces the load carrying capacity of concrete, increases the moisture ingress, thus causing durability problems [Shah et al., 1998].

2.2.3.1 Causes of Drying Shrinkage Cracking

On exposure to the atmosphere, concrete loses some of its original water through evaporation and shrinks. If unrestrained, drying shrinkage results in shortening of the member without the build-up of shrinkage stress. If restrained, drying shrinkage will lead to stress build-up, the magnitude of which can be initially estimated using Hooke's law $\sigma = E\epsilon_{SH}$, where E is the elastic modulus of concrete and ϵ_{SH} is the difference between free shrinkage and restrained shrinkage displacement. It should however be noted that this estimate of stress is too high since concrete is an aging visco-elastic material and the stress build-up is lowered by stress relaxation (Weiss 1999). This reduction in stress can result in stresses that are 30 to 70% of the elastic stresses (Weiss 1999). If the stress build-up (σ) is higher than the tensile strength of the concrete, drying shrinkage cracking can be expected. Drying shrinkage cracking occurs as a result of the interaction between strength of concrete, tensile stresses induced by shrinkage and stress relief due to relaxation [Mehta and Monteiro, 1993].

2.2.3.2 Crack Patterns

Drying shrinkage cracking typically occurs as circumferential and radial cracking as shown in Figure 2.4. This may be due to the heterogeneity of concrete, such as the difference in the modulus of elasticity between hydrated cement paste (HCP) and clinker residue, HCP and fine aggregates, and mortar and coarse aggregates.

Figure 2.4 Conceptual Illustration of Drying Shrinkage Cracking [Hearn, 1999]

2.2.4 Effect of Early Age Damage on Durability of Concrete

Unlike load-induced micro-cracks, which have local effects on transport properties of concrete, shrinkage-induced micro-cracks are well distributed and affect transport properties of concrete universally. Shrinkage cracks tend to have better connectivity, thus dramatically contributing to the permeability [Hearn, 1999]. In some cases, a full depth crack can be induced by the combination of plastic shrinkage and

subsequent drying shrinkage which leads to the direct access of water and corrosive agents into the concrete.

2.3 Deterioration of Concrete under Mechanical Loads

Concrete is a heterogeneous material with pores and voids of various sizes. A number of flaws or crack-like voids can exist in concrete even before loading. Most of these prior-to-load defects are interfacial cracks and fractures in the coarse aggregate from crushing. Some of these interfacial cracks are caused by the settlement of coarse aggregates during the placing process and an accompanying upward migration of mixture water, known as bleeding. Water accumulates on the undersides of the aggregate particles, forming a gap, or at least a partial loss of bond, between the mortar and aggregate. Shrinkage of mortar during setting and hardening may also cause bond cracks at the mortar-aggregate interface [Li and Li, 2000]. As a result, physical and mechanical properties are not identical at each point within a concrete. When concrete is subjected to mechanical loading, micro-cracks are generally initiated from these flaws, which highly influence the mechanical behavior of concrete.

2.3.1 Fracture Mechanisms under Load

Tremendous amount of studies have been conducted to investigate the behavior of concrete under axial loading. It is found that the fracture process of concrete can be characterized as strain softening and fracture toughening due to the formation and branching of micro-cracking [Li et al., 1993, Jeng and Shah, 1991, Shum and

Hutchinson, 1990, Shah and Sankar, 1987, and Ueda et. al., 1994]. Some physical observations can be summarized as follows:

- Jeng and Shah [1991] described that micro-cracking initiates at the weakest point, where the tensile stress concentration is the highest, and it will propagate and may coalesce as load increases. However, some of the micro-cracks will be arrested by the aggregates and air voids. Crack branching may occur as a higher level of load is applied. When the load reaches the critical level, a macro-crack starts initiating and propagating, and finally breaks the concrete.
- Shum and Hutchinson [1990] discussed toughening resulting from micro-cracks. Crack deflection occurs when the path of least resistance is around a relatively strong particle or along a weak interface. Grain bridging occurs when the crack has advanced beyond an aggregate that continues to transmit stresses across the cracks until it ruptures or is pulled out. This causes energy dissipation through friction and some bridging across the crack. Another mechanism in the fracture process is crack-branching. The crack may propagate into several branches due to heterogeneity of concrete. More energy should be consumed to form new crack branches.

Generally, the crack surface is tortuous due to the toughening mechanism, in which crack generally branch around aggregates, causing random propagation in concrete. The roughness of the crack surface depends on the toughness and size of aggregates and the properties of matrix and interface.

Under load, two types of cracks may occur, which are simple bond cracks and combined cracks as shown in Figure 2.5 [Hearn, 1999]. Simple bond cracks refer to cracking at mortar-aggregate interface. These cracks occur at the early stage of loading as a result of debonding, and they are isolated and stable under constant load. As the load increases, cracks may initiate and propagate within mortar, and eventually connect to the bond cracks forming combined cracks, which can be grouped into Type I and Type II [Hearn, 1999]. Type I cracks consist of one bond crack and a mortar crack, or the combination of two bond cracks connected by a mortar crack. This type of cracks extends in a relatively stable manner, which means that if the applied load is held constant, cracks cease propagating. Type II cracks contain two bond cracks and two mortar cracks. This type of crack propagates spontaneously even under constant load leading to failure.

Figure 2.5 Types of Cracks under Loading [Hearn, 1999]

2.3.1.1 Deterioration of Concrete under Compressive Load

Micro-cracks created by compressive loadings are varied in shapes and length. This is due to the heterogeneity of the material. Shah and Sankar [1987] observed under a microscope that when compressive stress increases up to 85% of ultimate stress, micro-cracks primarily occurred at the interface between the coarse aggregate and mortar. They are uniformly distributed in both transversal and longitudinal directions. When stress reaches 95% of ultimate stress, bond cracks are connected by cracks through the mortar. Continuous cracks occur, which are essentially parallel to the loading directions. Hsu et al. [1963] reported that under microscope significant increase in bond cracks can be observed at the interface between the coarse aggregates and the paste at 30% of peak load, and mortar cracks start initiating at the stress level between 70% and 90% of peak stress.

Bascul [1996] reported that at 30% stress level, micro-cracks initiate at coarse aggregate and paste interfaces. When stress increases from 30% to 50% of peak stress, micro-cracks propagate around the interfaces, which are simple cracks on the order of 0.1mm in length. Between 50% and 70% stress levels, they extend to the paste where they remain very short with some branches. Above 70% stress level, new micro-cracks start initiating around fine aggregates and connect with the previous micro-cracks, leading to the failure of specimens.

2.3.1.2 Deterioration of Concrete under Direct Tensile Load

Concrete members and structures primarily fail by tension even under bending or compression. Understanding the damage mechanisms of concrete under direct tension is of great importance in both design and construction to insure proper control of cracking that may occur. Since concrete is relatively weak and brittle in tension, cracking will be developed when the tensile stress exceeds the tensile strength of concrete. Many factors affect the tensile strength of concrete, which can be categorized as aggregate-related and non-aggregate-related. Aggregate related factors include size and distribution, shape, roughness, strength and volume of aggregate itself, as well as the shape, position and size of defects on the aggregate surface. Non-aggregate-related factors include age, water cement ratio, specimen size, loading rate and direction, and the test method [Ueda et al., 1994]. Three main methods have been used for measuring the tensile strength of concrete: splitting tensile test, flexural test, and direct tensile test. The first two tests are indirect methods, in which the tensile stress is applied indirectly to specimens. As a result, the strength measured is not the true tensile strength. The direct tension test could yield a tensile strength closer to the true tensile strength of concrete assuming the pure uniaxial tension is applied [Zheng et al., 2001].

Over the past decades, several variations of the direct tensile test have been developed. The major problem of these tests is that the secondary stress is induced at the end of specimen. Such end effect leads to the uneven stress distribution in specimen, causing localized failure at the ends, and significant reduction in the measured tensile strength. It is realized that the most reliable method of applying direct tension without

inducing secondary stresses is by gluing steel plates or bars to the ends or sides of concrete specimens, and pulling the steel plates or bars so that the force is transmitted to the concrete in the form of tension. Gérard et al. [1996] conducted a parallel elastic load-shearing system to obtain a stable response in the post-peak region for a uniaxial tension test as shown in Figure 2.6. In this way, unstable failure near the peak load is avoided. The major drawback of the method is that the accuracy of the results obtained is limited. Load shared by concrete is obtained by subtracting the contribution of the load shearing system from the total load, implying subtraction of two relatively large numbers to obtain a small number.

Figure 2.6 Tension Test Setup for a Parallel Load-Shearing System [Gerard et al., 1996]

Another tensile test setup of using double end plates was proposed by the U.S. Bureau of Reclamation [1992] as shown in Figure 2.7. Each of the two inner plates is

bonded directly to one end of the specimen, and then bolts to the outer plate. This method can result in a more uniform stress distribution since a number of well distributed bolts are used, which help transmit the load evenly across the sections. However, this method is only specified for cylindrical specimens.

Figure 2.7 Tensile Test Setup and Stress Distribution at End of Specimen, (a) Friction Grips at Ends, and (b) Studs Embedded in Concrete at Ends [US Bureau of Reclamation, 1992]

In order to test prismatic specimens, Zheng et al. [2001] modified this method for square sections. It is reported that a more uniform tensile stress distribution at the plate-specimen interface was achieved based on the finite element analysis of the test assembly.

Another method for testing prismatic specimens was developed by Li et al. [1993] and involved in using steel loading plates to load the specimen as shown in Figure 2.8. For each end of the specimen, two plates were first glued on the two opposite sides of the specimen, and then connected to a fixture through two pins. To avoid a sudden change in

stiffness and consequent stress concentration, a gradual decrease in steel thickness was used. To reduce the eccentric load, a base plate with four aligning pins was used during gluing for alignment. This setup was reported [Li et al., 1993] to apply a reliable tension to the specimen. Attempts were also made to use closed-loop control to obtain post peak behavior. It is commonly accepted that under tension, there is brittle failure when load is applied, and failure occurs when an unstable macro-crack is developed. It is difficult to observe the post peak of strain-stress curve. It is reported [Li et al., 1993] however that gradual failure of an unnotched specimen can be obtained and instability near the peak load can be avoided if the opening of the crack can be controlled in a closed-loop controlled manner.

Figure 2.8 Tensile Test Setup with Loading Plates Glued to a Specimen [Li et al., 1993]

Macroscopic mechanical behavior under tension can be generally described by the stress-strain curve which contains three regions: elastic, non-linear range, and shift due to the formation and propagation of a macro-crack [Ueda et al., 1994]. The initiation of micro-cracking can occur at the aggregate interface that is most vulnerable as this is a region where stress tends to concentrate, or at voids caused by bleeding or insufficient compaction which exist in the concrete before loading. Studies [Ueda et al., 1994] also reveal that micro-cracking can occur on the side of aggregate interface facing the tensile loading direction if there is insufficient adhesion.

Li and Li [2000] studied the fracture process of both plain and fiber reinforced concrete using acoustic emission. It was observed that for plain concrete micro-cracking began to initiate at about 40% of peak stress. Between 40% and 80%, the amount of micro-cracking increased gradually. Above 80%, micro-cracking started propagating and extended to the matrix. As the tensile stress continued to increase up to 100% peak stress, a macro-crack formed. After peak stress, load started decreasing. From 100% to 80% of post peak stress, the propagation of the major crack dominated the fracturing process leading to ultimate failure.

The failure pattern of concrete under tension manifests itself by a single major crack or several parallel major cracks mainly perpendicular to the loading direction. The density of micro-cracks in plain concrete is larger than that in the high strength concrete,

indicating that the high strength concrete is more brittle than the plain concrete [Gérard et al., 1996]].

2.4 Damage Detection and Quantification in Concrete

Detection and quantification of damage in concrete has become an integral part of concrete technology. Diagnostic methods that have emerged over the last decades have led to improved understanding of fracture mechanisms and long-term performance of concrete.

There are mainly five types of techniques that have been used to evaluate damage in concrete: visual observation, X-ray examination, microscopy, and non-destructive testing, in which nondestructive testing (NDT) methods have been proved to be very valuable and practical, allowing for rapid inspection and evaluation of damage and deterioration in concrete structures.

2.4.1 Visual Observation

Visual observations are most widely used forms of nondestructive evaluation. A visual inspection is usually conducted at the outset of a condition assessment. This method can yield much information regarding cracking, corrosion, scaling, spalling, and efflorescence of concrete. However, some forms of destructive or nondestructive testing are required to determine the causes of observed deterioration and to assess the integrity of structure.

2.4.2 Nondestructive Testing

Nondestructive testing (NDT) is defined as any test or method, by which the quality of a structure can be evaluated without damaging its serviceability [Poston et al., 1995]. Generally speaking, visual examination can be considered as NDT. The use of NDT techniques to assess the damage in concrete has led to improved understanding of fracture mechanisms of concrete, especially in the area of how damage influences mechanical and durability related properties of concrete such as modulus of elasticity, peak load, toughness, permeability, and sorptivity. Due to the rapidity and ease, the NDT has been widely used both in field assessment and laboratory investigations. Most frequently used nondestructive testing techniques for detecting damage in concrete are based on acoustic phenomena, including ultrasonics and acoustic emission. The following review focused on the applications of these techniques.

2.4.2.1 Resonant Frequency Methods

The natural vibrational resonant frequency of a specimen is related to the elastic material constant, which is often called “dynamic” elastic material constant when it is determined by the low strain nondestructive test. The resonant frequency can be used to evaluate the relative damage level of concrete due to the fact that the dynamic elastic constant is an indication of the relative material quality and uniformity [Kolluru et al., 2000]. For example, fundamental resonant frequencies can be used to determine the dynamic modulus of elasticity based on the ASTM C215 [2002], and to evaluate the freeze and thaw durability based on ASTM C666 [2003].

There are two alternative procedures that can be used [ASTM C215, 2002]: the forced resonance method and the impact resonance method. In the forced resonance method, an electrical-mechanical driving unit is used to force the specimen to vibrate, and the specimen response is monitored by a lightweight pickup unit. The driving frequency is varied until the measured specimen response reaches the maximum amplitude. The value of frequency causing maximum response is the resonant frequency of specimen. In the impact resonance method, a small impactor is used to strike a supported specimen, and the specimen response is measured by a needle pickup unit (or an accelerometer) attached to the specimen. The output of the accelerometer is recorded, and the resonant frequency is determined by counting zero crossings in the recorded waveform or using digital signal processing methods.

For the above two test procedures, there are three natural vibration modes (transverse, longitudinal, and torsional) that can be used to measure various specimens of different shapes, including rectangular bars, thin disk specimens, and cylinders. The progressive changes of concrete properties with age, or freeze and thaw cycles, or chemical attack can be assessed using the resonant frequency method. Concrete cores with approximate $L/D = 2$ or nonstandard geometry are sometimes removed from structures for strength evaluation of in place concrete. Changes in relative strength can be easily evaluated using the longitudinal frequency mode if the control specimens are available.

The dynamic properties determined by the resonant frequency method generally depend on the size and shape of the specimen, and this method is typically limited to the laboratory test using relatively small dimensions. However, the ultrasonic pulse methods (described in the next section) are capable of measuring concrete specimens with a wide range of sizes and shapes from laboratory specimens to pavements and mass concrete.

2.4.2.2 Ultrasonic Wave Method

Of all the nondestructive methods, ultrasonic methods offer a distinct advantage of being able to detect the interior deterioration (even after the occurrence) at relatively low cost and without causing any new damage. Ultrasonic waves are mechanical waves generated by exciting a piezo-electric crystal inside transmitting transducers with a high voltage pulse. The frequencies generated are usually greater than 16 kHz [Suaris and Fernando, 1987]. After traveling through test material, the waves reach a receiving transducer, which will produce an output voltage. There are two types of waves: compressive waves and shear waves. For concrete, compressive waves are most frequently used for ultrasonic testing because of the difficulty of coupling shear transducers and the higher attenuation characteristics of shear waves.

2.4.2.2.1 Impact Echo Method

This technique can be used to detect damage or flaws that have limited sizes and locations in concrete. When the ultrasonic wave hits a crack or flaw, it will be reflected by the crack interfaces and be received by the detector. As a result, it is possible to locate

the flaw and quantify its size by analyzing the waveforms received from several transducers at different sites. However, it is impossible to use this technique for detecting a large number of distributed micro-cracks [Carino et al., 1986].

Another application of this method is to measure the P-wave speed and the thickness of concrete plates [ASTM C1383, 1998]. P-wave speed is determined by dividing the distance between two transducers by the time it takes for the P-wave generated by a short-duration, point impact to travel between two transducers. The thickness of the plate is calculated from the P-wave speed and the P-wave frequency reflected by the opposite surface of the plate.

2.4.2.2.2 Ultrasonic Pulse Transmission Methods

Ultrasonic Pulse Transmission Method has been widely used for assessing the extent of damage and the uniformity and the relative quality of concrete to indicate the presence of voids and cracks, to estimate the depth of cracks, and to evaluate the effectiveness of repairs. It may also be used to indicate changes in the properties of concrete, and to estimate the severity of deterioration or cracking. For example, ultrasonic pulse attenuation is used as a tool for assessing the crack growth during cyclic loading of concrete, indicating that the crack growth is not linearly proportional to the number of load cycles [Suaris and Fernando, 1987]. The most frequently used method is Ultrasonic Pulse Velocity (UPV) [ASTM C597, 1997] which measures the transmit time of the fastest wave between two transducers (the transmitting transducer and the

receiving transducer), attached to two opposite surfaces of the specimen. An electric timing circuit or a pulse meter is used to measure the time for a pulse to transverse a specimen of known length. The pulse meter initiates an internal clock when the sending transducer is shocked; this is disabled when the first disturbance is felt by the receiving transducer [Popovics et al., 1990]. The time in microseconds is digitally displayed, and the pulse velocity (m/s) can then be calculated by dividing the distance (m) between centers of transducer faces by the transmit time (s).

Many attempts have been made to correlate the pulse velocity with the strength of concrete. There are many factors such as constituent materials, curing conditions, and moisture that may strongly influence the relationship between the pulse velocity and the strength [Suaris and Fernando, 1987].

Effects of Frequency on Pulse Velocity

Concrete is a heterogeneous material, and the pulse frequency affects the pulse velocity. One reason is that geometry of the specimen affects the pulse velocity. The pulse velocity is independent of the dimension or geometry of the test object provided that the waves reflected from boundaries do not complicate the arrival time of the directly transmitted pulse. To satisfy this condition, smallest dimension of the test object must exceed the wavelength of the ultrasonic vibrations [Popovics et al., 1990]. For example, the wavelength for a 24kHz transducer is 6 in., which is about comparable to the size of a specimen (e.g., a 4"×6" cylinder) assuming a pulse velocity of 3500 m/s. This sets up a guided wave situation, such as bar wave, in the longitudinal direction that affects the

velocity readings. As the diameter/wavelength increases, the degree of dispersion decreases. This suggests the importance of picking a suitable frequency when testing a concrete specimen of specific geometry. Another reason is that constituent materials affect the pulse velocity. For example, the increase of pulse velocity with frequency is greater in concrete than in pastes, particularly in higher frequency range. This is because the paste is relatively homogeneous as compared with concrete. For a large specimen in a high frequency range, the influence of geometry is negligible, but the heterogeneity of concrete will affect the pulse velocity.

Effects of Stress Level on Pulse Velocity

It is expected that pulse velocity will gradually decrease with the increase in loading because the dependence of pulse velocity on the modulus of elasticity. However, experimental results showed that under uniaxial compressive stress there is an increase in velocity at small stress level (e.g. 25%), followed by a relative constant region up to 70% stress level, then a dramatic decrease in pulse velocity as the specimen approaches failure [Popovics and Popovics, 1991]. This suggests that concrete does not follow the expected pattern of gradual decrease in pulse velocity under increasing stresses. A hypothetical explanation for this unexpected response of pulse velocity to stresses may be that small loads do not produce sizable micro-cracking, but they do consolidate the specimen resulting in the increase in pulse velocity. As the load increases, the increase in pulse velocity levels off due to the propagation of micro-cracking. When the propagation of micro-cracking becomes dominant, the velocity drops dramatically [Popovics et al., 1990].

Other Effects on Pulse Velocity

Any factors that affect the internal flaw size or uniformity will affect the pulse velocity of concrete. For example, pulse velocity increases with age but at a decreasing rate; internal segregation results in a higher pulse velocity in the bottom part of the specimen as compared with the top [Popovics et al., 1990].

The degree of saturation of concrete also affects the pulse velocity. The pulse velocity of saturated concrete may be up to 5% higher than that of dry concrete. In addition, pulse velocity in saturated concrete is less sensitive to changes in its relative quality [ASTM C597, 1997].

2.4.2.3 Acoustic Emission

Acoustic Emission (AE) is a nondestructive testing method that is widely used for monitoring, characterizing and locating the initiation and growth of damage in concrete specimens. During loading, cracking occurs when the localized strain energy in concrete is spontaneously released leading to the formation of stress waves (ultrasonic waves). As the stress wave propagates to the surface of the material, it can be detected and registered by highly sensitive piezoelectric transducers as an AE event. As a result, the characterization of an AE event is influenced by the following three separate processes: the source (micro-crack) event, the resulting stress wave propagation through the material, and the measurement of the waveform on the specimen surface [Landis and Shah, 1995].

There are many ways to characterize the onset and extent of damage in concrete using acoustic emission techniques, such as AE event rate, cumulative AE event number, and cumulative AE energy during loading. The source location of an AE event can be determined by time differences among the recorded signals from an array of transducers, an approach that has been used to monitor micro-cracking localization and the fracture process zone in concrete under loading [Li and Li, 2000, Mihashi and Normura, 1992, Li and Shah, 1994, Ohtsu et al., 1998, and Ohtsu, 1999]. Traditionally, AE event rates, peak amplitudes, frequency spectra, and source locations are recorded to analyze the damage growth or other fracture behaviors [Hauwaert et al., 1999].

The most frequently used parameter is AE activity that can be characterized by cumulative AE event number and AE event rate since they are closely related to the crack initiation and propagation. By monitoring cumulative AE events or AE event rate, the points when new cracks are initiating or existing cracks are propagating due to mechanical loading or other equivalent loads such as shrinkage or expansion can be easily identified. Li et al. [1993] observed that the occurrence of AE events depends on the imposed stress level. Below 40% of the peak load, no AE events occurred. At around 40% of peak stress, AE activities became detectable, and the number of AE events gradually increased with the increase in stress level. Above the 80% stress level, the defects, voids and other existing micro-cracks began to propagate (due to the stress concentration) and emit acoustic waves with sufficiently high amplitudes to be detectable. This keeps AE events gradually increasing. The cumulative AE events can be

a valuable indicator of the level of deterioration in concrete structure. Yoon et al. [1999] found that the cumulative AE event related to the degree of corrosion in the reinforced concrete. As the corrosion increased, the total number of AE events dropped. This is attributed to the fact that parts of the energy might have been dissipated by the micro-cracking induced by corrosion.

AE behaviors have been studied during cracking processes in RC beams [Yuyama et al., 1999]. Table 2.1 shows the correlation of AE behavior with the fracture stage and damage level during the cyclic bending test of RC beams.

Table 2.1 Example of Evaluation Criterion for Damages Induced in RC Beams

Fracture Stage		Damage Level (Crack Width:W)	Amplitude (dB)	Concrete Beam Integrity Ratio	AE Activity during Unloading
I	Early Microcracks	Low $W < 0.12 - 0.2$ mm	40 - 60	> 1	Low
	Main Tensile Cracks		80 - 100		
II	Secondary Tensile Cracks	Medium $W > 0.12 - 0.2$ mm	40 - 80	0.8 - 0.9	Medium
	Internal Shear Cracks				
III	Slips between Reinforcement and Concrete	High $W > 0.5$ mm	40 - 60	< 0.8	High

The AE source locations can be determined based on the difference among the first P-wave arrival times at each of the AE transducers and the P-wave velocity in

concrete [Li and Li, 2000]. The map of AE source can be obtained showing the time or stress levels at which cracking initiates, propagates, and localizes. For example, Li and Shah [1994] observed that when the stress level was less than around 80% of peak stress, the AE events were randomly and relatively sparsely distributed in the specimen. The randomness of AE events (and therefore of the micro-cracking activities) is due to the random distribution of the pre-existing flaws and paste-aggregate interfaces. Above 80% stress level, the AE events tended to localize in a certain parts of the specimen. After the 80% post-peak load, AE events are exclusively localized in the portion of the major crack.

2.4.3 Microscopic Observations

In addition to the nondestructive testing, a variety of other techniques have been used to study micro-cracks in concrete. These include the direct observation of microstructure of concrete under an optical microscope and a scanning electron microscope (SEM).

2.4.3.1 Sample Preparation

Concrete slices for microscopic examination require a smooth lapped surface and its preparation involves cutting and polishing. Cutting is usually performed using a water-cooled diamond-edged rotary saw. Sometimes, oil-cooling is used so that specimens are not exposed to water that may dissolve certain components of concrete. To produce a fine surface, polishing is performed using laps and grinding compounds. For easy observation, impregnation techniques are used to highlight the microstructure.

Various impregnation methods have been developed including dyne impregnation and vacuum epoxy impregnation.

In dyne impregnation, a dye, either fluorescent or dark colored, dissolved in an organic solvent or low viscosity epoxy, is used to penetrate into the cracks or air voids to highlight them for easy observation. A fluorescent dye makes internal pores or cracks visible under ultraviolet light. A proper polishing operation is important as insufficient polishing will leave excessive colored dye on the surface and the cracks will not be easily discernible during microscopic observation. Excessive polishing will remove the colored dye from impregnated cracks, and negatively influence the subsequent quantifications [Gran, 1995 and Hornain et al., 1996].

For vacuum epoxy impregnation, a dried specimen is immersed in an epoxy solution while under a vacuum. The reason of drying the sample is due to the fact that pores in concrete are filled with water and it is very difficult for the epoxy to penetrate into the water-filled pores. The use of vacuum is to assist the impregnation so that a sufficient depth of impregnation can be achieved. This is needed to prevent the impregnated layers from being accidentally removed by the inaccurate grinding and to obtain the reproducible measurements. For example, an impregnation depth of 25 μm is required for a thin section. However, even after a successful impregnation process is performed, the penetration depth in concrete is only 50 μm , which gives little room for grinding and polishing [Gran, 1995].

Drying and vacuum are harsh treatment that may cause the formation of new cracking due to shrinkage and pressure. One way of avoiding drying and vacuum during the impregnation process is the use of sequential replacement, in which water is gradually replaced by an alcohol, and subsequently the alcohol is replaced by the dyed ultra low viscosity epoxy (e.g. the one used in the preparation of SEM samples) provided that the epoxy is miscible with the alcohol. This procedure can be used for SEM analysis or for thin section study [Struble and Stutzman, 1989]. The main drawbacks are the possible insufficient impregnation and the long time needed to impregnate the samples.

2.4.3.2 Optical Microscopy

Optical microscopic examination, also called petrography, is the primary tool for studying the concrete microstructure, crack patterns, and the distress mechanisms since it offers a larger field of view as compared with SEM. The combination of the results from optical microscopy, visual examination, information on the constituent materials and mix design, and environmental conditions is the fundamental process of identifying the mechanisms of distresses in the field. For example, crack patterns, air void system (entrained or entrapped), porosity of aggregate, segregation, and excessive bleeding can be easily identified using an optical microscope with fluorescent illumination. Secondary precipitates such as ASR gel and ettringite can be identified by an optical microscope with polarized light. Sometimes, the optical microscopy is supplemented with SEM, and X-ray analysis to respectively evaluate the microstructure of the distressed concrete under higher magnification and to identify chemical composition of the secondary precipitation.

Table 2.2 shows the information that can be obtained using the optic microscopic examination [Jakobsen et al., 1997].

2.4.3.3 Scanning Electron Microscopy (SEM)

The scanning electron microscope (SEM) is an effective tool to identify the microstructure, and (when equipped with energy dispersive X-ray elemental analysis (EDS)) to analyze the components in materials such as cement paste, mortar and concrete. It is frequently used in combination with the optical microscope and macro-examination to better understand the mechanisms of deterioration of concrete. For example, the presence of brucite in concrete detected by SEM with EDS can be a diagnostic feature of alkali carbonate reaction. Sometimes, SEM can be a powerful tool in observing the micro-cracks in concrete due to its high magnification. For example, the use of SEM and replica techniques can help in observing the growth of micro-cracking in concrete specimen [Toumi, 1998]. Two modes (secondary and backscattered) are often used. In secondary mode, a secondary electron detector is used to detect the electrons with energy of a few electron volts (secondary electrons) that are produced when an electron beam reacts with a specimen. In backscattered mode, a backscattered electron detector is used to detect the electrons that are reflected (backscattered) when the electron beam reacts with a specimen. Those electrons typically have higher energy that is just below the accelerating voltage of the electron beam. Generation of backscattered electrons is mainly dictated by the average atomic number of materials. For the materials composed of low atomic number elements (e.g., epoxy), most of incident electrons are absorbed within the specimen and a very small amount of backscattered electrons are

Table 2.2 Diagnostic Features Observed under an Optical Microscope

Features		Ordinary Light	Crossed Polarized Light	Fluorescent Light
Concrete Composition	Cement Paste	Composition, type	Composition, type	Amount, porosity
	Admixtures (e.g. SF, FA)	Composition, type	Composition, type	Homogeneity, cracks
	Aggregate	Size, shape	Type of minerals and rocks	Porosity
	Air Voids	Size, shape, and distribution		Size, shape, and distribution
	W/C	Density of cement grains		Estimation, variation
Defects and Cracks	Porosity			Variation, homogeneity
	Bleeding			Presence, shape
	Cracks	Fine, coarse		Micro, fine, coarse
Change of the Cement Paste	Carbonation		Depth, extension	Porosity changes
	Leaching		Depth, extension	Porosity changes
Secondary Mineral Precipitates	ASR Gel	Usually transparent, clear, occasionally muddy brownish	Black when amorphous, occasionally birefringence when crystalline (orange)	Homogeneous green, yellow shrinkage crack
	Ettringite	Clear needle shaped crystals, may appear massively, whitish filling voids and cracks	Weak birefringence, dark gray of 1. order	Some porosity between crystals, when massive it appears dark green
	Calcium Hydroxide	Clear crystals	High birefringence, blue-violet of 2. order	Homogeneous green
	Calcite	Clear, striations	Very high birefringence, higher order colors	
	Gypsum	Clear, often sawtoothed	Birefringence, light gray of 1. order	Homogeneous green
	Thaumasite	Fibrous	High birefringence, yellow orange of 2. order	
	Brucite	Clear, often layered	High birefringence, brown-yellow of 1. order	

produced leading to a darker region, while a material with higher average atomic number generates a greater portion of backscattered electrons resulting in a brighter region. As a result, phases in concrete (e.g., CSH, CH, and cracks and voids) can be easily identified based on the backscattered electron image intensity [Sahu et. al., 2004]

2.5 Transport Phenomena in Concrete

Although durability of concrete has been a key issue for many years due to its impact on the service life of concrete, it is being taken much more seriously now than ever before due in part to the high material and construction costs. It is much more economical to extend the service life of existing structures with little repair or maintenance than to construct a new structure. There are a wide variety of reactions and processes responsible for concrete deterioration, such as corrosion of the reinforcement, freeze and thaw damage, ASR, and sulfate attack. However, in each of these deterioration processes, water is involved in the mechanisms of expansion or chemical reactions. Also, water is the principal vehicle for the transportation of aggressive agents such as chloride ions and sulfate into concrete.

Concrete structure, when properly constructed, should be watertight, and have a long service life. However, cracks may occur due to mechanical loading or environmental attack during service, resulting in the loss of water-tightness. Concrete then becomes more susceptible to one or more processes of deterioration. Moreover, the interplay between water and cracks may result in the accelerated deterioration. As a result, the transport of water into concrete plays a key role in the durability of concrete structure.

2.5.1 Basic Laws Governing Transport Phenomena in Concrete

Concrete, depending on its degree of saturation, can be modeled as a two-phase porous media consisting of solid and liquid, or three-phase porous media consisting of solid, liquid and air [Bear and Buchlin, 1991, Isgor and Razaqpur, 2003, Bangert et al., 2003, Selih et al., 1996, and Monlouis-Bonnaire et al., 2003]. Transport phenomena in concrete are complicated in which more than one combined transport mechanisms such as permeation, absorption, and convection are involved. However, the basic laws or the fundamental equations governing the fluid flow (e.g. moisture and air movement, or mass transfer) inside concrete can be derived when the individual mechanism is considered. Generally, there are three basic laws (mass balance, force equilibrium, and energy conservation) and four main transport mechanisms (diffusion, permeation, capillary absorption, and convection) involved in describing the fluid flow in concrete. For the unsteady state flow, the binding properties are also required for solving the mass balance equation.

2.5.1.1 State Parameters for Air and Water in Concrete

Fundamental parameters used for defining the state of the transported substances include pressure p , mass m , volume V , and temperature T . Other parameters can be derived, for example, mass and volume gives concentration $C = m/V$.

Unsaturated concrete can be modeled as a multiphase system, where the pores of solid matrix are filled partially with water, and partially with gas. The gas phase can be

modeled as an ideal gas composed of dry air and vapor, which are regarded as two miscible species. The stress state of concrete can be described as gas pressure P_g , capillary pressure P_c , temperature T , and the displacement of solid matrix [Gawin et al., 1995].

Water content, or degree of saturation, S , is defined as the volume of voids and pores that are filled with water divided by total volume of voids and pores. It is assumed to be a function of capillary pressure and temperature: $S = S(P_c, T)$. Water pressure can be expressed as $P_l = P_g - P_c$.

For the gas phase, the ideal gas law when applied to dry air (ga), vapor (gw) and moist air (g) gives:

$$P_{ga} = \frac{\rho_{ga} TR}{M_{ga}}, \quad P_{gw} = \frac{\rho_{gw} TR}{M_{gw}}, \quad P_g = P_{ga} + P_{gw}, \quad \rho_g = \rho_{ga} + \rho_{gw} \quad 2-1$$

Where, ρ -mass concentration or density, M -molar mass, R - gas constant

For a gas-water interface, assuming that it is flat, the Kelvin equation can be obtained based on the thermodynamics [Jennings, 1996]:

$$P_g - P_l = \frac{\rho_l RT}{M_{gw}} \ln \frac{P_{gw}}{P_{gws}} \quad 2-2$$

Where, P_{gws} -saturated water vapor pressure in gas phase. The relative vapor pressure (P_{gw}/P_{gws}) is also called the relative humidity (RH). When $RH < 100\%$, water will evaporate from flat surface. But for concrete with small capillary pores, continuous evaporation of water will become more and more difficult due to the capillary action, where a curvature of meniscus separates the water phase from the gas phase inside the pore. At the interface between water with a pressure P_l and air with pressure P_g , the Laplace equation should be applied:

$$\Delta P = \frac{2\gamma \cos \theta}{r} \quad \text{or} \quad P_g - P_l = \frac{2\gamma \cos \theta}{r} \quad 2-3$$

Where, P_g -pressure of air; P_l -pressure of water; r -radius of meniscus; γ -surface tension; θ -contact angle

The force balance between the adhesive forces (water-concrete) and the internal cohesive forces of water (i.e. surface tension γ) will give rise to the radius of meniscus r during drying, and the pore water pressure will drop. As a result, evaporation will be slowed down assuming the same RH.

To describe the state of a solid skeleton which is partially saturated with water and subjected to stress and internal pressure, Biot constant and effective stress were introduced to account for the volumetric deformability of the CSH gel [Bangert et al., 2003 and Zuber and Marchand, 2000].

$$\sigma_e = \sigma_t + \alpha P \mathbf{I}, \quad \text{and} \quad \alpha = 1 - \frac{K_T}{K_S} \quad 2-4$$

Where, σ_e -effective stress tensor, σ_t -total stress tensor, $\mathbf{I}$ -unit tensor, α -Biot constant, K_T -bulk modulus of concrete, K_S -modulus of CSH gel, P -average pressure of air and water which can be evaluated assuming immiscible two phase flow:

$$P = S \cdot P_l + (1-S) \cdot P_g \quad 2-5$$

Where, S is the degree of saturation.

It is assumed that the pore fluid and CSH gel both contribute to the stress state of concrete, and the effective stress is the fraction of total stress carried by solid skeleton. The constitutive relationship between the effective stress and elastic strain for the solid matrix can be written as (assuming the infinitesimal deformation):

$$d\sigma_e = \mathbf{D} (d\epsilon - d\epsilon^{th} - d\epsilon^{ir}) \quad 2-6$$

Where, $\mathbf{D}$ -elastic tensor related to the modulus of elasticity and Poisson's ratio, ϵ -total strain, ϵ^{th} -thermal strain tensor, ϵ^{ir} - irreversible strain tensor

2.5.1.2 Mass Conservation Law

Mass is neither created nor destroyed, but may be changed in form or location. The Principle of mass conservation can be applied to a controlled region, and the mass balance can be expressed as:

$$\left[\begin{array}{c} \text{Net Accumulation} \\ \text{of Mass within CR} \end{array} \right] = \underbrace{\left[\begin{array}{c} \text{Mass Input} \\ \text{Rate to CR} \end{array} \right]}_{\text{Transport}} - \underbrace{\left[\begin{array}{c} \text{Mass Output} \\ \text{Rate from CR} \end{array} \right]}_{\text{Phenomena}} \pm \underbrace{\left[\begin{array}{c} \text{Production / Consumption} \\ \text{Rate within CR} \end{array} \right]}_{\text{Hydration Absorption etc.}} \quad 2-7$$

Where, CR – Control Region

The governing equation can be formulated as:

$$\frac{\partial m}{\partial t} = \Phi_{in} - \Phi_{out} \pm \gamma * V \quad 2-8$$

Where, m – Mass of substance (e.g. water) in CR, [Kg]

$\phi_{i, in}$ - Mass input rate to control region, [Kg/s]

$\phi_{i, out}$ - Mass output rate to control region, [Kg/s]

γ - Spatially averaged reaction rate within control region, [Kg/m³.s]

2.5.1.3 Force Equilibrium

The Navier-Stokes equation is the basis of convective flow fields for incompressible, constant viscosity fluids under laminar flow conditions. The fundamental nature of the equation is the momentum balance in which the force component, ΣF , acting on a fluid element is the result of a time rate of change of momentum (Newton's second law of motion). The total force on an element includes spatial forces resulting from gravitational fields and velocity fields, and the surface forces resulting from friction at a boundary. The Navier-Stokes equation can be expressed as [Weber JR and Digiano, 1996]:

$$\rho \frac{\partial \vec{V}}{\partial t} = \rho g - \nabla P + \mu_v \nabla^2 \vec{V} \quad 2-9$$

Where, $\vec{V}$ - velocity field, ρ – density of fluid, P – pressure, μ_v – viscosity of fluid

For incompressible fluids such as water, the continuity equation can be expressed as:

$$\nabla \bullet \vec{V} = 0 \quad 2-10$$

For incompressible fluids, the Navier-Stokes equation can be reduced to:

$$\underbrace{\rho g}_{\text{Gravity}} - \underbrace{\Delta p}_{\text{Pressure}} + \underbrace{\mu_v \nabla^2 \vec{V}}_{\text{Shear}} = \underbrace{\left(\frac{\partial}{\partial t} + \vec{V}_x \frac{\partial}{\partial x} + \vec{V}_y \frac{\partial}{\partial y} + \vec{V}_z \frac{\partial}{\partial z} \right) \vec{V}}_{\substack{\text{Accumulation} \\ \text{input, output of momentum}}} \quad 2-11$$

The distribution of velocity and pressure of the water system in concrete as a function of time and space can be obtained by solving the Navier-Stokes equation and continuity equation together using boundary conditions. Due to the nonlinear system of equations and the complexity of microstructure of porous concrete, exact solutions are almost impossible, and are therefore limited to a few special simplified situations. For example, for a steady and uniform flow in one or two dimensions, the acceleration term on the left hand side of Navier-Stokes equation can be ignored (equal to zero). However, if the space within the system is discretized, or divided into meshes, velocity fields can be obtained by using numerical methods such as finite element, to solve the Navier-Stokes equation.

2.5.1.4 Energy Conservation Law

Conservation of energy can also be applied to a controlled region (CR), and can be expressed as:

$$\left[\begin{array}{c} \text{Net Accumulation} \\ \text{of Energy within CR} \end{array} \right] = \underbrace{\left[\begin{array}{c} \text{Energy Input} \\ \text{Rate to CR} \end{array} \right] - \left[\begin{array}{c} \text{Energy Output} \\ \text{Rate from CR} \end{array} \right]}_{\substack{\text{Diffused by Conduction, Transported by Fluid Flow} \\ \text{(Convection)}}} \pm \underbrace{\left[\begin{array}{c} \text{Production / Consumption} \\ \text{Rate within CR} \end{array} \right]}_{\text{Hydration, Phase Change, bonding / debonding}} \quad 2-12$$

The differential equation for heat transfer can be formulated as [Tenchev et. al., 2001]:

$$\underbrace{(\rho C) \frac{\partial T}{\partial t}}_{\text{Accumulation}} = \underbrace{-\nabla \bullet (-K \nabla T)}_{\text{Conduction}} - \underbrace{(\rho C \vec{V}) \bullet \nabla T}_{\text{Transport}} - \underbrace{\lambda_E E_L}_{\text{Evaporation}} - \underbrace{\lambda_F E_S}_{\text{Freezing}} - \underbrace{\lambda_D \frac{\partial \rho_D}{\partial t}}_{\text{Sorption / Desorption}} \quad 2-13$$

Where, ρ – Density of liquid or solid phase [Kg/m³]

C – Specific heat of air, water vapor, liquid, or solid phase [J/(Kg. °C)]

$\vec{V}$ - Velocity of liquid or gas phase [m/s]

ρC - Effective heat capacity [J/ (m³ °C)]

$\rho C \vec{V}$ – Vector of energy transported by fluid flow [J/ (m² °C)]

K - Thermal conductivity of concrete [J/ (m² s °C/m)]

λ_D – Specific heat of desorption [J/Kg]

λ_E – Specific heat of evaporation [J/Kg]

λ_F – Specific heat of freezing [J/Kg]

E_L - Rate of evaporation of liquid water per unit volume of concrete [kg/(m³s)]

E_S - Rate of freezing of liquid water per unit volume of concrete [kg/(m³s)]

ρ_D - Mass of water due to sorption and desorption per unit volume of concrete [kg/m³]

2.5.2 Transport Mechanisms

2.5.2.1 Diffusion

Diffusion is the process that occurs as a result of concentration gradients. Moisture movement by diffusion may occur in unsaturated concrete, but it stops when the concrete is saturated. However, the aggressive agents that are dissolved in the water such as Cl^- , sulfate, and CO_2 can be transported wherever there is a concentration difference.

The steady state diffusion process is governed by the Fick's 1st law, and can be formulated as [Dullien, 1992]:

$$J_m = -D \cdot \nabla C \quad 2-14$$

Where, C – Concentration, D – Diffusion coefficient, and J_m– Mass flux

The time dependent concentration can be derived from the divergence of flux density:

$$\frac{\partial C}{\partial t} = -D \nabla^2 C \quad 2-15$$

This equation is also known as the diffusion equation or Fick's second law, and can be used to describe the unsteady state diffusion process.

The diffusion of ions is usually simplified as if the ions were diffusing independently. However, diffusion of ions is very complex and involves not only the interactions with counter-ion and/or other ions in the solution, but also the interaction with the electric field formed as a result of electric double layer.

The continuing hydration and pozzolanic reactions in concrete refine its pore structure as it matures, thereby reducing the connectivity of pore system and the diffusivity of concrete [Mehta and Monteiro, 1993]. The diffusion process is, thus, time and temperature dependent, and the use of short-term tests to characterize the diffusion process through concrete is misleading especially for the concrete with higher percentage of silica fume and slag.

2.5.2.2 Convection and Diffusion

Convection or advection is the process that describes the transport of a solute (e.g. chloride) as a result of the bulk moving fluid, which is often accompanied by diffusion.

This combined process can be described as [Boddy et al., 1999]:

$$\frac{\partial C}{\partial t} = D \nabla^2 C - \vec{V} \cdot \nabla C \quad 2-16$$

Where, $\vec{V}$ - Velocity vector of fluid flow

C – Concentration of a solute

$\vec{V} \cdot \nabla C$ – Convection term

2.5.2.3 Permeation

Permeation in concrete is the flow of a fluid as a result of gravity and hydraulic pressure gradients. Flux of a fluid through porous media (e.g. concrete) when saturated is governed by Darcy's Law. After studying the rate of flow of water through sand filter bed, Darcy found that the macroscopic flow velocity V was proportional to the hydraulic gradient or hydraulic head [Yong et al., 1992]:

$$V = \frac{Q}{At} = k \frac{\Delta h}{\Delta x} \quad 2-17$$

Where, Q - volume of water flow, A - cross-sectional area of bed, t - time of flow,

k - Constant defined as hydraulic conductivity or coefficient of permeability

Although Darcy's Law was obtained experimentally, it can be derived by applying Navier-Stokes equation to the flow through porous media [Weber JR. and Digiano, 1996].

2.5.2.4 Capillary Absorption

Uptake of water by unsaturated concrete is mainly attributed to capillary absorption, and the ability of a material to absorb or transmit water by capillarity is defined as sorptivity [Wilson et al., 1999]. It is increasingly used to evaluate the concrete resistance to exposure in aggressive environments. Although other mechanisms such as diffusion and permeation may be involved, they are negligible in terms of the amount of water being uptaken as compared with capillary absorption.

For a concrete with initial uniform water content, capillary absorption can be described mathematically by the nonlinear diffusion equation [Hall, 1989 and Lockington et al., 1999].

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x} \left(D(\theta) \frac{\partial \theta}{\partial x} \right) \quad 2-18$$

Where, θ -Water content

However, capillary diffusion differs from molecular diffusion in that the capillary flow in unsaturated concrete arises from the capillary forces or the gradient of capillary potential that correlates with both the pore structures and the water content.

The cumulative water absorption per unit area of the inflow surface at time t can be calculated by integrating the water content θ along the depth. For one dimensional case, defining $\phi = x t^{-1/2}$, θ is related to ϕ . The cumulative absorption i can be expressed as [Bear and Buchlin, 1991]:

$$i = \int_{\theta_0}^{\theta_1} x d\theta = t^{\frac{1}{2}} \int_{\theta_0}^{\theta_1} \phi d\theta = S \bullet t^{\frac{1}{2}} \quad 2-19$$

and,

$$S = \int_{\theta_0}^{\theta_1} \phi d\theta = (\theta_1 - \theta_0) \int_0^1 \phi d\theta_r \quad 2-20$$

Where : $\theta_r = \frac{\theta - \theta_0}{\theta_1 - \theta_0}$ is normalized water content

The initial condition is $\theta = \theta_0$, and for dry concrete θ_0 is close or equal to zero. The boundary condition at the wetted surface is $\theta = \theta_1$.

Assuming that concrete pores can be modeled as tubes perpendicular to the free water surface, capillary absorption of concrete pores can be evaluated based on the surface chemistry [Levine, 1988]. Generally, when two dissimilar materials such as two immiscible fluids or a fluid and a solid come into contact with each other, two different forces, cohesion and adhesion are developing. Cohesion is the attraction between like substances such as two molecules of a liquid, and adhesion is the attraction between unlike substances such as one molecule of the liquid and one molecule of the wall of the tube. Adhesion may be positive, as in the case of water wetting concrete pore wall, or may be negative, as in the case of mercury wetting glass. The adhesive forces cause a thin film of water to creep up the pore wall. When the water creeps up, the top surface of the

water expands creating new water-air interface. Due to the cohesive forces between water molecules, a certain amount of work is needed to form the new interface. The quantity of this energy is called surface tension. As a result, the expansion is opposed by the surface tension at the water air interface making the interface curved. The shape of the interface depends on the relative magnitudes of the adhesive forces between the water and the wall and the internal cohesive forces in water. At the point where the air-water interface meets the wall, a contact angle θ is defined as the angle between the plane tangent to the air-water interface and the water-solid interface. When the adhesive forces exceed the cohesive forces, θ lies in the range of 0° to 90° ; when the cohesive forces exceed the adhesive forces, then $90^\circ < \theta \leq 180^\circ$. The curved interface between air and water causes the equilibrium pressures in the bulk phase air and water to differ. The pressure drop is governed by Laplace equation (2-3). When r increases to infinity, the pressure difference goes to zero as it should be for a planar interface.

The pressure difference is significant when r is small. Assuming a vertical pore with gas phase connected to the atmosphere, the equilibrium height of the meniscus in a gravitational field (h) can be obtained as follows:

$$h = \frac{2\gamma \cos \theta}{(\rho_{\text{water}} - \rho_{\text{air}})gr}$$

Where: g is the gravitational constant, and ρ_{water} and ρ_{air} are density of water and air respectively. This can explain why concrete pores absorb water. For example, for concretes, typical size of a larger capillary pore is $1 \mu\text{m}$, and surface tension for water-air

interface at room temperature is $\gamma = 7.2 \times 10^{-2}$ N/m. Assuming perfect wetting, $\theta = 0^\circ$, then $h = 14$ m.

2.5.3 Equilibrium of Water in Concrete Pores

The distribution of both liquid and gas within concrete in equilibrium with its environment depends on the sizes and geometry of the pores and the external environmental conditions. It is proposed [Setzer, 1991] that pores smaller than 2 nm are classified as micro-pores. In such small pores the forces of surface interactions among solid matrix, water, and air are so dominant that the macroscopic physical laws are not applicable and the water is fully structured. Pores between 2 to 50 nm are meso-pores, in which a transition from surface physics to the physics of macroscopic bulk media occurs and condensate water is present. Pores between 50 nm to 2 μm are micro-capillaries, where bulk water exists but there is no macroscopic equilibrium. Pores between 2 μm to 50 μm are meso-capillaries, in which capillary absorption can take place from minutes to weeks. Pores between 50 μm to 2mm are macro-capillaries, where capillary water uptake occurs instantly. Generally, it is recognized that the following states of water exist in concrete due to different surface force fields [Levine, 1988 and Churaev, 2000]: adsorption layers, wetting films, pores filled with water as described in the following sections.

2.5.3.1 Adsorption Layers

In the region of relatively low vapor pressure, individual molecules are adsorbed on the pore surface with a thickness from a fraction of monolayer to several monolayers

by inter-molecular bonds. This water will not freeze during freeze and thaw cycles even at very low temperature.

2.5.3.2 Wetting Films

When the vapor pressure approaches saturation, water can be adsorbed to form a thin film with a thickness of 0.8-1nm, and this thin film is bound on one side by solid concrete and on another by air and vapor. At lower vapor pressure, the surface bonded water should be considered rather as adsorption films. When the film thickness grows to a thin liquid phase due to the increase of vapor pressure, it can be considered as wetting films. The condition for equilibrium between a wetting film and a meniscus is the equality of capillary pressure under the meniscus and disjoining pressure in the film.

2.5.3.3 Pores Filled with Water

The state of water depends on the capillary pressure of confining menisci and the surface force generated by pore walls. The gas phase in pore/air system of concrete typically consists of air and water vapor. The general gas laws are applicable except the very narrow pores that are commensurate with molecule sizes.

For general consideration water movement or water transfer into concrete may be divided into two particular systems: the saturated system where all the voids are filled with water; and the partially saturated system where both air and water are present. For partially saturated concrete, mechanisms of moisture transfer will depend on the degree of dryness or wetness within the concrete. For a relatively dry concrete, moisture

movement by vapor diffusion is greater than that by liquid flow, whereas when concrete is relatively wet, liquid transfer will outweigh vapor transfer. The water movement by viscous flow in unsaturated concrete is primarily due to the capillary potential or suction, which arises from the difference in water content, and from the capillary force of concrete due to its pore structure, but is not restricted to the capillary rise of water due to surface tension.

Since concrete pavements are always exposed to the environment, they are rarely fully saturated when in service. The use of permeability to characterize the fluid flow in concrete is misleading. The reason is clear. Permeability is a measure of steady state flow in response to imposed pressure gradient. To be in steady state, concrete should be completely saturated. A material property measured in saturated steady state such as permeability will tell nothing as far as the parameters needed for modeling of the unsaturated capillary flow in concrete. A better way to assess the water transport property in a concrete pavement is sorptivity, which reflects the ability of concrete to absorb and transport water by capillarity.

2.5.4 Effects of Cracking on Transport Properties of Concrete

It is well recognized that cracks in concrete structure provide flow channels for water, gases, and dissolved deleterious substances to diffuse, permeate, or absorb into the microstructure. Limited studies have been conducted to investigate the effects of cracking on permeability of concrete. Wang et al. [1997] studied the water permeability of cracked concrete cylinders subjected to splitting tensile loading and found that cracks with

openings greater than 50 microns would greatly increase the permeability of concrete. Aldea et al. [1999] studied how mixture types (paste, mortar and concrete) and specimen thickness influenced the permeability of concrete. It was found that water permeability greatly increased with the increase of crack width. In addition, normal strength concrete showed highest permeability while mortar had the lowest water permeability. The relationship between damage and the air permeability of concrete was also studied using concrete beam under flexural cyclic loading [Tawfiq et al., 1996]. Findings indicated that air permeability of concrete increased with the increase of loading levels as well as the number of cycles.

CHAPTER 3 FIELD CONDITION ASSESSMENT

The primary goal of this chapter is to describe a field condition assessment that was performed to identify the typical characteristics of damage in concrete pavements including type, extent and distribution of damage in cored specimens using both destructive and nondestructive testing. Sites were sampled from around the state of Indiana. Samples were analyzed to evaluate the development of cumulative damage as a result of the interaction of several unseen mechanisms, including freeze and thaw damage, early-age drying shrinkage damage, and damage induced by traffic load.

3.1 Pavement Survey

Four different sites located in the state of Indiana were selected for this investigation. The sites were selected to represent pavements that were showing signs of premature deterioration as well as older pavements that were providing reasonable performance. To determine which sites would be used, information was obtained from maintenance engineers, pavement condition reports from the Indiana Department of Transportation (INDOT), and the visual examination of PCC pavements.

Table 3.1 provides details of the location, age, thickness, traffic level, quality index of the pavements from which cores used in this study were obtained. The first site (I-465) represented a newly constructed pavement that had not been opened to traffic to provide a baseline of the typical concrete properties that could be expected from a newly constructed concrete pavement before the section experiences any weathering or exposure to traffic. The second site (I-70) that was chosen was a relatively young (approximately six years old) pavement that had shown signs of premature deterioration

and been rated poorly (i.e., less than 60) using the INDOT pavement quality index (PQI) as explained in Appendix I [Pavement Management Unit, INDOT, 2000]. The third site (I-469) consisted of a middle-aged pavement (14 years old) that had exhibited a low PQI and premature deterioration mainly in the form of faulting and cracking. The final site (I-65) was an older pavement (25 years old). The I-65 site was particularly well suited for this investigation since it provided a section that was trafficked (I-65-T) as well as a section that was constructed at the same time but that had remained essentially untrafficked (I-65-U), thus enabling the effects of traffic to be separated for environmental effects. It can be seen that the pavements investigated represent a wide range of traffic levels, ages, and performance.

Table 3.1 Pavement Sections Selected for Sampling and the Related Information

Pavement	Age (Year)	Thickness mm (in.)	Traffic Level		PQI ²	Location
			AADT ¹	%Truck		
I-465	0	356 (14.0)	None	None	–	Indianapolis, Indiana
I-70	6	363 (14.3)	96,000	10%	64	Senate Ave to 7.7 MI E of I-465 @ East St., Indianapolis
I-469	14	295 (11.6)	19,000	4%	61	I-69 to 0.7 MI W of SR 1, Fort Wayne
I-65-U	25	315 (12.4)	None	None	–	Intersection of I-65 and I-70 near North Split, Indianapolis
I-65-T	25	315 (12.4)	140,000	7%	66	Intersection of I-65 and I-70 near North Split, Indianapolis

1. AADT – Annual Average Daily Traffic.

2. PQI – See Appendix I

3.2 Coring of Pavement Sections

After arriving at each site, traffic was diverted from the section of the pavement that was investigated and a visual examination of the site was performed. Based on the

visual inspection, a typical slab was selected for core sampling. Five cores were taken from each slab using an approach that was similar to that used by Van Dam et al. [2001]. Two different size cores were taken with a 102 mm (4 in) diameter and a 152 mm (6 in) diameter, depending on the location of the core in the slab. The extraction of smaller cores at the joints was typically preferred to minimize the potential for intersecting a dowel bar. Figure 3.1 illustrates the coring locations in a typical slab and shoulder. This coring pattern was followed for all but one (newly constructed I-465) pavement sections. Since I-465 section was never exposed to traffic, the exact location of the cores is insignificant and therefore was not recorded. One core was taken from the center of the slab in the wheel path (W1), one core was taken from the wheel path near the joint (WJ2), one core was taken from the center of the slab (C3), one core was taken from the center of the shoulder (S4), and the last core was taken from the corner near the joint (CJ5). It is assumed that the wheel path and the center of the slab are exposed to both environmental attack and traffic loading, while the shoulder is primarily subjected to environmental exposure. By comparing these locations, the impact of traffic loading, joint and edge effects and environmental exposure can be assessed. After the cores were extracted, they were taken back to the laboratory for further analysis as described in the following section.

Figure 3.1 Illustration of Coring Locations from a Typical Slab

Core W1: Sample No. 1, taken from the center of the wheel path

Core WJ2: Sample No. 2, taken across the joint and between dowels of the wheel path

Core C3: Sample No.3, taken from the center of the slab

Core S4: Sample No. 4, taken from the shoulder

Core CJ5: Sample No. 5, taken from the corner and joint of the slab edge

3.3 Laboratory Analysis of Field Cores

After the cores were delivered to the laboratory, they were visually inspected and any visible damage was recorded. The core samples were then tested using ultrasonic pulse velocity to inspect for any significant internal flaws. After the non-destructive testing was completed, the cores were sectioned by alternately the cutting of two 13 mm (0.5 inch) sections and one 51 mm (2 inch) section as illustrated in Figure 3.2. The core sections were then used for microscopic evaluation, air void determination, water

absorption measurement, and splitting tensile strength determination. The following sections describe how the specimens were prepared for each of these test procedures.

3.3.1 Visual Inspection

The cores were first visually inspected to determine whether any visible damage, segregation, or construction defects could be observed. Any damage that was observed was noted and recorded.

3.3.2 Ultrasonic Pulse Velocity (UPV) Testing

Ultrasonic Pulse Velocity (UPV) test [ASTM C597, 1997] was used to evaluate the internal damage, uniformity and relative quality of concrete. In these tests, two transducers were used to measure the time it took to transmit an ultrasonic wave across the diameter of the specimen. The wave speed was calculated by dividing the distance between the transducers (nominally 152 mm or 100 mm) by the time that it took the wave to cross the specimen. The longer the wave transmission time (i.e., the slower the wave speed) the more internal damage could be expected in the core. A V-meter (manufactured by James Instrument) was connected to a pair of 54 kHz transducers that were coupled to the opposite sides of the core using petroleum jelly. One transmitter was used as the pulser while the other was used as the receiver and the time that it took the wave to transmit across the cylinder (in milli-seconds) was recorded. The UPV was measured in two perpendicular directions every 51 mm from the top of the core to bottom of the core since the cracks that form may have a specific orientation. The average wave speed that was measured in the two directions was reported.

3.3.3 Microscopic Examination

After the UPV measurements, the cores were sectioned into smaller disks as shown in Figure 3.2. The 13 mm disks were selected for microscopic evaluation. One surface of the disks was polished using a bench-top lapping wheel. Water was used as a lubricant and a siliconcarbide (SiC) abrasive was applied with sizes ranging from No. 100 to No. 600. The final polishing was performed on a glass plate using No. 800 and No. 1000 SiC abrasives. The samples were then placed on a Roiswal traveling table and the volumetric percentages of aggregates, pastes, and air voids were determined based on the modified point count method [ASTM C457, 1998]. An optical microscope set at a magnification of 100X was used to examine the polished slices to assess the general condition and crack patterns of the concrete. Typical damage patterns were recorded using an image acquisition system.

Figure 3.2 Illustration of Sectioning a Typical Core for Laboratory Investigation

3.3.4 Water Absorption and Sorptivity Determination

Water absorption tests are being increasingly used to evaluate concrete durability in aggressive environments [Lockington et al., 1999, and Wilson et al., 1999]. In this study, the 51 mm (2 in) disks that were sectioned from either 152 mm (6 in) or 102 mm (4 in) diameter concrete cores were used to determine the water absorption properties of the concrete and the rate at which this water was absorbed. The disks were first air dried and their circumference was sealed with an epoxy to ensure that moisture exchange will only take place through the top and bottom faces of the disks. All the disks were then pre-conditioned in an environmental room with a controlled temperature of 23 °C and relative humidity of 50% for 14 days to enable the specimen to reach a relatively uniform state of moisture. It is anticipated that this drying would not lead to excessive additional cracking since this is a relatively mild drying condition and the field specimens would likely have experienced some similar drying during their life. A ponding dyke was placed on the top of the disks and the bottom was covered with plastic wrap to prevent moisture loss. Water was added to the inside of the dyke until it reached a depth of approximately 15 mm. Periodically (at 0, 0.1, 0.2, 0.5, 1, 5, 12, 24, 48, 96 hours, then at 96 hour intervals after that) water was removed from the surface of the disks, the surface was toweled dry (to achieve saturated surface dry conditions), and the mass of the specimen was recorded. After the mass was recorded, water was immediately reintroduced to the top of the specimen and remained on the surface until the next measurement was performed. This testing procedure was used to define the total amount of water that has been absorbed in the concrete as well as the rate of absorption (sorptivity) as described later in this chapter.

3.3.5 Strength Evaluation

After the water absorption tests were completed, splitting tensile tests were performed on the 51 mm (2 in) sections of the core following the approach outlined in ASTM C 496 [1996] with a loading rate of 0.69 MPa/min (100 psi/min). For each core, a total of three specimens were tested. These specimens were collected at an average depth of 38 mm (1.5 inches), 114 mm (4.5 inches), and 191 mm (7.5 inches).

3.4 Experimental Results and Discussions

3.4.1 Visual Inspection

No visible cracking or segregation was observed in the majority of the cored cylinders. It should however be noted that one specimen taken from I-65-U section was partially cracked. In addition, as shown in Figure 3.3, evidence of construction defects in the form of irregular voids up to 25 mm (1 in) in diameter were observed at the depth about 127 mm (5 inches) below the top surface in the I-465 pavement. These voids appear to be remnants of an unfilled vibrator trail. Further discussion of the I-65 and I-465 cores is provided in microscopic investigation section of this chapter.

Figure 3.3 Photograph Showing Construction Defects below Surface of a I-465 Pavement Section

3.4.2 Ultrasonic Pulse Velocity (UPV)

Figure 3.4a shows the result of UPV measurements for cores from each site as a function of pavement depth. The average UPV value of the five locations of the selected slab from each site was used. However, the cracked core from I-65-U section was not included in calculating the average. It can be seen that the UPV values range from 4600 m/s to 4900 m/s (251 ft/min to 268 ft/min) for the uncracked cores which is consistent with what would be expected in these concretes. Figure 3.4b shows a comparison of

UPV measurements for cracked and uncracked cores from I-65-U. It can be seen that the presence of visible cracks resulted in a dramatic reduction in the wave speed.

At this time it should be noted that many factors may affect the UPV measurements, including specimen dimension and geometry, properties of constituent materials, and degree of saturation. However, only large internal flaw or significant changes in uniformity will have noticeable influence on UPV [Suaris and Fernando, 1987, and Popovics et al., 1990]. Air voids, especially entrapped air, and presence of microcracks may reduce the UPV somewhat, but not significantly. This can be further confirmed by comparing Figure 3.4a and 3.4b. While some changes in the average values of UPV were observed for cores with no visible cracks (Figure 3.4a), mostly reflecting variability in age, air contents and microcracks, the truly significant drop in UPV was associated with the presence of visible cracks. This indicates that UPV techniques may be useful for locating the cracks in PCC pavement cores, however the size of the flaw that can be detected must be sufficiently large (i.e., of the size that it would approach being visible cracking).

3.4.3 Microscopic Examination

The parameters associated with the air void system evaluated during the microscopic examination of the specimens included the volumetric percentage of air voids, specific surface area, and spacing factor. In addition, the paste and aggregate content were also determined.

Figure 3.4 Variation of Ultrasonic Pulse Velocity with Pavement Depth, (a) Average of Five Cores for All Pavement Sections, and (b) Two Cores from I-65-U

The air void contents were found to vary from top to bottom of each pavement, and from one pavement to another. Figure 3.5a illustrates the variation in hardened air content as a function of pavement depth for the two cores from the newly constructed pavement section of I-465. It can be seen that the air contents vary from 4.9% to 13.1% with an average total air content of 9.8%. Very high air content was observed near the mid-depth of the pavement due to the presence of very large irregular entrapped-air voids ranging from 2.5 mm (0.1 inches) to 17 mm (0.7 inches) in diameter which may suggest that insufficient vibration occurred in this pavement. On the contrary, low air content near the top surface was observed (4.9 %), which may be attributed to the over-vibration

of some portions of the surface during construction. Paste and aggregate contents were also estimated for the same two cores. The volume of paste was found to be in the range from 18.9% to 23.8% and the volume of aggregate was in the range from 65.4% to 72.8%. It should also be noted that the distribution of the paste and aggregate fractions were relatively uniform throughout the depth of the pavement.

Figure 3.5b shows a comparison of the overall trend in the average value of air content for all the pavement sections investigated in this research. The air contents of three pavement sections (I-65-T, I-465, and I-65-U) are all relatively high. The sections from I-469 and I-70 appeared to have relatively typical air contents (5.0% to 7.6%). Concrete with an appropriate air void size and distribution can resist the freezing and thawing damage. Table 3.2 separates entrained air from entrapped air based on the size and shape of the air void using microscopic examination. It can be seen that the entrained air content varies from 3.4% to 7.3% in the pavement sections investigated. As the air content increases, the freeze-thaw resistance is typically thought to improve, however a reduction in strength is typically observed. Concrete with unevenly distributed air or large volumes of entrapped air may be susceptible to freezing and thawing attack. However, other than the sections that appeared to be over vibrated (I 465), the air void spacing was uniform and consistent with acceptable spacing [ASTM STP169B, 1978].

Figure 3.5 Variation in Total Hardened Air Contents in Pavements, (a) Two Cores (#1 and #2) from I-465 Section, and (b) Average of Five Cores from Five Pavement Sections

Table 3.2 Parameters of Air Void System

Pavement Section	Air Void System				
	Average Air Content (vol. %)			Specific Surface Area *	Spacing Factor
	Total	Entrained	Entrapped	mm ² /mm ³ (in. ² /in. ³)	mm (in.)
I-65-T	9.6	7.3	2.3	12.3 (313.1)	0.267 (0.0105)
I-469	7.3	4.1	3.2	9.5 (240.3)	0.356 (0.014)
I-70	4.6	3.4	1.2	14.5 (369.4)	0.330 (0.013)
I-465	11.5	7	4.5	8.3 (209.6)	0.244 (0.0096)
I-65-U	9.3	6	3.3	10.2 (258.7)	0.356 (0.014)

* Specific surface area was calculated based on the total air content (entrapped and entrained)

Typical cracking patterns showed cracks that developed at the interface between the aggregate and the paste, in the paste, or through the weak aggregates. Figure 3.6a shows examples of the cracked aggregate and paste. It can be seen that crack propagated

through both the fine aggregate and the paste implying that cracks occurred at later age, and was possibly induced by traffic or environmental attacks such as freeze and thaw cycles. Figure 3.6b provides an illustration of paste cracking, in which cracks extended to the void. Figure 3.6c illustrates interface cracking. This type of cracking is typically associated with damage that occurs due to loading as a result of dissimilar strains that develop between the aggregate phase and the paste phase.

(a) 60X

(b) 170X

(c) 60X

Figure 3.6 Examples of Typical Crack Patterns, (a) Crack through Both Aggregate and Paste (I-70, 6 Years), (b) Crack through Air Voids (I-65-T, 25 Years), and (c) Crack through Interface and Paste (I-65-U, 25 Years)

3.4.4 Water Absorption and Sorptivity

A typical plot of the change in mass of the samples due to water absorption per unit area of the surface that was exposed to water was plotted as a function of the square root of time and is shown in Figure 3.7. It can be seen that this graph contains four main regions: 1) the region of short-term water absorption, 2) transition region, 3) the region of long-term water absorption, and 4) the total absorption at a specific time (368 hours). The slopes of the curve in region 1 and 3 describe the rate of cumulative water absorption per unit area at short and long terms respectively. This water absorption was fitted using linear regression and the equation obtained was used to describe capillary actions at both short and long terms [Bentz et al., 2001].

Figure 3.7 Typical Result of Water Absorption as a Function of Square Root of Time

In this chapter the short-term rate of water absorption (i.e., initial sorptivity or short-term sorptivity) was determined based on the initial seven hour water absorption. It is thought to be a measure of how quickly the water fills the large pores [Marchand et al., 1997 and Martys and Ferraris, 1997] and cracks. It is dependent on the preconditioning (initial water content and drying) and pore size and distribution of concrete. Cracks may permit an increase in water ingress. As a result, concretes with higher degree of deterioration tend to have higher short-term sorptivity values.

The long-term rate of water absorption (i.e., long-term sorptivity) was determined based on the water absorption between 50 hours and 368 hours. It is commonly thought of as a measure of how quickly and easily the water fills the air voids [Wilson et al., 1999] while the total absorption simply describes the total volume (or mass) of water that has been absorbed by the concrete after 368 hours of water ponding. The long-term sorptivity and total absorption can play a key role in determining how long a concrete pavement may take to reach a critical degree of saturation that appears to be related to the onset of freeze and thaw deterioration.

Figure 3.8 presents the sorptivity results of one pavement section (I-65-T) to illustrate the comparison of sorptivity values that were measured at different locations in the slab. It is clearly seen that cores taken from the joint and edge of the slab behave differently from cores taken from the mid-panel of the slabs. Higher values of sorptivity (including both short-term and long-term) were observed at the middle depth of joints and edges and at the top surface of mid-panel suggesting that more damage may be

developing at these locations. The locations close to the joint demonstrate a higher overall sorptivity thereby suggesting that the damage was more uniform at the joints and corners. The reasons for the differences between the behavior of the slab at the center and at the corners may be explained by two considerations. First, at the joint the dowel bars transfer load more uniformly throughout the edge section and may transfer substantial stresses which may result in microcracks due to the restraint of moisture and thermal movements. Second, the joints and edges of the slab are more susceptible to water penetration during rainfall and may be more susceptible to the effects of freezing and thawing.

Figure 3.8 Changes of Sorptivity with Locations of the Core for One Pavement Section (I-65-T), (a) Short-Term Sorptivity, and (b) Long-Term Sorptivity

Figure 3.9a shows a comparison of the short-term sorptivity that was obtained by averaging the sorptivity of cores from the center, the wheel path, and the shoulder for pavements from various sections. It can be seen that two sections (I-65-T and I-70) demonstrate a substantially higher short-term sorptivity at the surface than in the middle or bottom of the specimen. This again is consistent with the damage development at the top of the pavement in sections that are showing signs of deterioration (i.e., microcracking was observed under microscopic examination).

A description of the influence of traffic loading on the cumulative damage in PCC pavements can be made by comparing the pavement sections on I-65 with the section that was exposed to traffic and the environment (I-65-T) and the section that was only exposed to the environment (I-65-U). In the section exposed to traffic (I-65-T) the short-term sorptivity at the top of the pavement is 54% higher than the short-term sorptivity at the bottom of the core. However, the untrafficked section only showed an increase of short-term sorptivity of 5%. This implies that the combination of traffic loading and environmental attack can accelerate the deterioration since both sections are of the same age and were under essentially the same environment.

In addition, the short-term sorptivity appears to be somewhat related to the age of the specimen. The oldest pavement section (I-65) shows the highest short-term sorptivity, while the short-term sorptivity of new pavement section (I-465) is the lowest. This may be due to an improvement in the mixture proportions/placement procedures over time that has resulted in a tighter, less porous system. In addition to pavement age,

damage may also play a part in increasing the short-term sorptivity. For example, although the cores taken from I-70 represent a pavement that is “younger” than the pavement I-469, it has higher sorptivity. This may be explained by the fact that pavement I-70 had started deteriorating based on the microscopic examination, whereas, no cracks were observed in pavement I-469.

Figure 3.9b shows the long-term sorptivity of all the pavement sections. Since long-term sorptivity is a measure of the slow saturation of air voids, it is closely related to characteristic of air void system such as air content, size and distribution, spacing, and specific surface area. The results reflect the combined effects of pores, air void system, and damage on the long time sorptivity. The interaction between capillary pores, air voids, and damage is complicated, and impacts the long-term transport behavior of water in concrete. Table 3.3 lists several of the main factors that were measured in these pavements that may be related to the long-term sorptivity. Damage is only identified qualitatively as cracked or uncracked based on microscopic examination. It is a hypothesis that cracks may reduce the resistance to diffusion and facilitate the transport of both air and water in concrete. Therefore, the pores in cracked concrete can be saturated in a shorter time while it will take a longer time for the pores of an undamaged concrete to be fully saturated. As such, the long-term sorptivity of a cracked concrete is primarily dictated by the gradual filling of air voids while both the saturation of pores and the filling of air voids are involved in the long-term water absorption of an uncracked concrete. This explains why a good concrete (uncracked I-469) showed higher long-term sorptivity than a cracked concrete (I-70) (Table 3.3).

Figure 3.9 Change of Sorptivity (Center, Wheel path, and Shoulder Cores Only) with Pavement Depth for All Pavement Locations, (a) Short-Term Sorptivity, and (b) Long-Term Sorptivity

Table 3.3 Factors That Contribute to Long-Term Sorptivity

Pavement Section	Long Time Sorptivity $\text{mm}^3/\text{mm}^2/\text{min.}^{0.5}$	Microscopic Investigation of Damage	Air Void System	
			Air Content (%Vol.)	Specific Surface Area * mm^2/mm^3 ($\text{in.}^2/\text{in.}^3$)
I-65-T	0.001	Cracked	9.6	12.3 (313.1)
I-469	0.0018	Uncracked	7.3	9.5 (240.3)
I-70	0.0009	Cracked	4.6	14.5 (369.4)
I-465	0.00025	Uncracked	11.5	8.3 (209.6)
I-65-U	0.0037	Cracked	9.3	10.2 (258.7)

3.4.5 Splitting Tensile Strength

The split cylinder test was used to provide an indication of the mechanical properties of the concrete pavements. Figure 3.10 shows the splitting tensile strength that was obtained from the cores taken from each of the pavement sections. It can be seen that the splitting tensile strength ranges from 3.1 MPa (450 psi) to 4.3 MPa (630 psi). The new pavement section (I-465) has the highest value, while the I-65-U that is 25 years old shows the lowest value. This may reflect the fact that the variation of splitting tensile strength with depth or pavements can be mainly linked to the age, air voids, and damage of the pavement sections. Similar results were reported by Graveen [2001] with an average splitting tensile strength of 3.16 MPa (28 days) and 3.63 MPa (103 days) using cylinder specimens (6"×12"). In addition, the average compressive strength was reported to be 44.8 MPa (28 days) and 57.8 MPa (103 days).

Figure 3.10 Splitting Tensile Strength of All Pavement Sections in This Study

3.5 Conclusions

This Chapter has reported the results of field condition assessment of selected concrete pavements in the state of Indiana. Specifically this work was intended to provide a better understanding of the damage that is present in existing pavements for use in life-cycle modeling. Based on the results obtained from cores taken from several pavements, the following conclusions can be drawn:

- Typical cracks observed in the pavements occur at the aggregate paste interface, through aggregates, and through the paste. While the exact cause of this cracking is not completely known, it is hypothesized that this cracking is due to a combination of loading and environmental effects.
- The average total hardened air contents for these pavements were found to vary widely from approximately 4.6% to 11.5%. Entrapped air may comprise relatively large portions of the total air (from 1.2% to 4.5%). In one pavement relatively large entrapped air voids (up to 25 mm in diameter) were found which could be a result of insufficient closing of the paste after vibration.
- In pavements that are known to be performing poorly (microcracking was observed under microscopic examination) it was observed that the short-term sorptivity near the surface of the pavement was substantially higher (up to 54%) than that from the middle or bottom of the pavement. This indicates that microcracking can occur at the surface of concrete pavements which results in more rapid water transport and absorption.

CHAPTER 4 WATER TRANSPORT IN CONCRETE DAMAGED BY FREEZE AND THAW AND TENSILE LOADING

In Chapter 3, it was shown that while in service, concrete can develop cracks or micro-cracks which can influence the transport of liquids and ions. This chapter describes an investigation of fluid transport in laboratory cured concrete that was damaged by tensile loading or freeze-thaw cycling. Acoustic emission (AE) was used to quantify the extent of damage in the specimens exposed to tensile loading while resonant frequency was used to quantify the extent of damage in the specimens exposed to freezing and thawing. Water absorption was measured in the damaged concrete. The presence of damage increased both the initial sorptivity and total water absorption. Electrical conductivity was measured and found to increase in a bi-linear fashion with freeze-thaw damage. A knee-point in the electrical conductivity versus freeze-thaw damage curve was observed when the dynamic modulus degraded to 75% of its initial value which appears to correspond with the coalescence of cracking.

4.1 Introduction

Some of the primary mechanisms for the deterioration of concrete facilities include the corrosion of reinforcing steel, cracking due to shrinkage, freezing and thawing, and chemical attack. In each of these processes, water is either the principal agent responsible for the deterioration or the principal medium by which aggressive agents (such as chloride or sulfate ions) are transported into the concrete. As a result, water ingress and aggressive agent transport are key issues that influence the long-term durability of concrete facilities.

The main transport processes in concrete include capillary absorption, diffusion, permeation, and convection. Capillary absorption describes water uptake in unsaturated concrete [Wilson et al., 1999] due to the capillary forces or a gradient of capillary potential. Diffusion describes the transport of moisture or dissolved ions as a result of a concentration gradient [Dullien, 1992]. Permeation describes the flow of a fluid (e.g., water or air) as a result of gravity or a hydraulic pressure gradient [Yong et al., 1992]. Convection (or advection) is the process that describes the transport of a solute (e.g., chloride or sulfate ions) as a result of the bulk moving water [Boddy et al., 1999]. The transport processes in most concrete structures can be complicated and may involve more than one of the referred transport mechanisms. Each of these mechanisms, however, has the potential to be influenced by cracking and micro-cracking because cracking or micro-cracking provides a less resistant, thus preferential, pathway for fluid to flow.

There is growing interest in attempting to predict the life-cycle performance of concrete pavements and structures [Bentz et al., 2001]. While most predictions consider concrete that is in an uncracked state [Fagerlund, 1997, Martys and Ferraris, 1997, Lockington et al., 1999, Hall, 1989, and Breysse and Gérard, 1997], recent studies have reported that cracks can dramatically influence the transport properties of concrete. For example, Wang et al. [1997] studied the permeability of cracked concrete and concluded that cracks generally accelerated water permeation in concrete. Specifically, the permeability was found to increase by an order of magnitude when the width of the crack opening increased from 50 μm to 200 μm . Hearn [1999] reported that cracks increased

the permeability and that the magnitude of such increase depended on the characteristics of cracks. Drying shrinkage cracking greatly increased the water transfer in concrete while the water permeability was insensitive to load induced damage. Similarly, Samaha and Hover [1992] investigated the relationship between rapid chloride permeability and compressive load and found that moisture and ion movement increased by 20% when load level was above 75% of ultimate strength. However, below this level the transport properties did not significantly change with loading. Jacobsen et al. [1996] also studied the effects of cracking on chloride transport in concrete and found that internal cracking increased the chloride penetration rate 2.5 to 8 times when compared with undamaged specimens.

In addition, recent studies have also indicated that cracks can accelerate the corrosion of reinforced steel. Yoon et al. [2000] studied the influence of pre-load, sustained load, and load levels on the initiation and rate of corrosion and found that corrosion initiated faster in pre-loaded specimens and that a higher load level resulted in a less initiation time and a faster rate of corrosion. In addition, sustained loading was found to increase the rate of corrosion as compared with pre-loading. Qi et al. [2003] studied the influence of plastic shrinkage cracking on the corrosion of reinforcing steel and found that an increase in crack width (0.25 to 0.5 mm) resulted in faster corrosion initiation (from 50 to 15 days) and wider surface cracking led to an increased amount of corrosion damage.

As a result, the use of properties obtained from undamaged concrete may overestimate the performance of a system resulting in an inaccurate prediction. To improve the accuracy, prediction models should be modified to properly account for changes in the transport process caused by cracking.

4.2 Experimental Program

The experiments used in this study included the use of freezing and thawing and tensile loading to induce damage in concrete. For freezing and thawing studies, the specimens were pre-conditioned to different degrees of saturation and then exposed to freezing and thawing cycles. The resonant frequency was used to monitor the damage development. After desirable damage levels (e.g., the resonant frequency reduced to 85%, 70%, or 40% of its original value) were reached, the specimens were sliced to evaluate their electrical conductivity and water absorption. For tensile loading, specimens were loaded to a specific load level (i.e., 60%, 80%, or 90% of the peak load), unloaded, and cut for the electrical conductivity and water absorption measurements. The following sections provide details regarding the materials, specimen geometry, and test programs used in this study.

4.2.1 Materials and Mixture Proportions

This study focused on using a concrete mixture that is typical of concrete pavements in the state of Indiana. An ASTM Type I portland cement ($C_3S=67.0\%$; $C_2S=7.2\%$; $C_3A=9.8\%$; $C_4AF=7.6\%$) was used. A coarse limestone aggregate that was

supplied by the Martin Marietta (96th Street, Indianapolis) with a bulk specific gravity (SSD) of 2.64 and the absorption capacity of 1.44% was used. Natural river sand was used as a fine aggregate with a bulk specific gravity (SSD) of 2.63, absorption of 1.56%, and fineness modulus of 2.71. The gradations of the aggregates were consistent with the recommendation of ASTM C-33 as shown in Tables 4.1 and 4.2. The mixture proportions are shown in Table 4.3. This mixture was proportioned at water to cement ratio (W/C) of 0.42, and contained 68% aggregate by volume, a mid-range water reducer (WRDA 82), and an air entraining admixture (Daravair 1400). The water reducer was adjusted to be slightly higher than typically used for slip form paving operations to facilitate lab placement with a slump of 51-76 mm (2-3in.). This adjustment typically resulted in the addition of 80 ml/m³ of the admixture. The air entraining admixture content was varied from 0.3 to 3.0 ml/kg cement to achieve fresh air volumes between 4% and 11%.

Table 4.1 Coarse Aggregate Gradation (IN #8)

Sieve Size	Cumulative Weight (Percent Passing)
25 mm (1 in.)	100
19 mm (3/4 in.)	90.5
12.5 mm (1/2 in.)	58.4
9.5 mm (3/8 in.)	38.7
4.75 mm (No. 4)	8.7
2.36 mm (No. 8)	1.9

Table 4.2 Fine Aggregate Gradation (IN #23)

Sieve Size	Cumulative Weight (Percent Passing)
9.5 mm (3/8 in.)	100
4.75 mm (No. 4)	99.9
2.36 mm (No. 8)	95.8
1.18 mm (No. 16)	66.8
600 µm (No. 30)	33.4
300 µm (No. 50)	9.6
150 µm (No. 100)	1.6
75 µm (No. 200)	0.3

Table 4.3 Mixture Proportions for Concrete Used in This Study

Materials	Batch Proportioning 1 m ³
Cement	354 Kg
Coarse Aggregate	1010 Kg
Fine Aggregate	840 Kg
Mixing Water	142 Kg
Free Water	6 Kg
W/C Ratio	0.42
Water Reducer	Adjusted to achieve slump of 50-75 mm
Air Entraining Agent	Adjusted to achieve fresh air contents of 4% - 11.5%

4.2.2 Specimen Preparation and Specimen Geometry

The materials were mixed in a horizontal pan mixer and the specimens were cast and cured following ASTM C192 [2002] with a typical batch size of 0.05 m³ (1.8 ft³). Slump and air content were measured in fresh concrete following ASTM C143 [2000] and ASTM C231 procedure B [1997] respectively. The fresh concrete was placed in the forms, which were externally vibrated on a vibration table for 20 seconds, and finished using a trowel. After finishing the specimens were covered with plastic sheet and stored wet at room temperature of 23°C. After 24 hours, the specimens were then de-molded and cured in moist room at 23°C until the time of test.

Two series of experiments were performed in this study. Freezing and thawing was used to induce damage in the first series, while the second series employed direct tensile loading to induce damage. For the first series of tests, four concrete prisms (76 × 102 × 406 mm) were cast for each mixture (i.e., fresh air content 5.5% and 9.5% by

volume respectively). After moist curing for at least 60 days, all the beams were cut in half ($76 \times 102 \times 203$ mm) for freeze and thaw testing as described in the following sections. Upon the completion of the freeze and thaw tests, each beam was cut into $76 \times 102 \times 51$ mm slices for individual sorptivity test. The second series of tests consisted of three concrete prisms ($76 \times 102 \times 864$ mm) that were prepared for each of the three different mixtures (4.7%, 6.2%, and 9.0% air content respectively). The specimens were cut to $76 \times 102 \times 432$ mm for tensile testing as described later in section 4.2.8. After tensile load was applied, the central portion of each specimen ($76 \times 102 \times 204$ mm) was cut to $76 \times 102 \times 51$ mm slices for sorptivity test. Detailed information on the specimens used in this study is summarized in Table 4.4.

Table 4.4 Type of Tests Performed and Specimens Used in This Study

Test	Specimen Geometry mm	Air Content (%)		# of Specimens	Average Tensile Strength (Standard Deviation) MPa	Modulus of Elasticity (standard deviation) GPa
		Fresh	Hardened			
Series I Freeze and Thaw Test	$76 \times 102 \times 203$	5.5	4.4	8	—	—
		9.5	6.9	8	—	—
Series II Tensile Test	$76 \times 102 \times 432$	4.7	3.3	6	3.8 (0.3)	42.5 (0.2)
		6.2	5.3	6	3.5 (0.3)	40.6 (2.3)
		9.0	6.8	6	3.1 (0.2)	38.1 (2.4)

4.2.3 Specimen Saturation

The specimens in the present study were saturated following the procedure similar to that described by Fagerlund [1977]. At an age of at least 60 days, concrete specimens ($76 \times 102 \times 203$ mm) were dried at 50°C for 4 weeks, and then evacuated to a residual pressure of 130-270 Pa for 4 to 6 hours. Water was introduced while the vacuum pump

was still running and the specimens were kept in water under vacuum for at least 2 hours. After removal of vacuum, the specimens were kept in water under atmospheric conditions for 1 week for further saturation. After saturation, each specimen was adjusted to a specific degree of saturation (e.g., 0.7, 0.8, 0.85, 0.9, or 0.95) by drying at 23°C and 50% relative humidity for a specific period typically from 2 hours to several days. After drying all specimens were sealed using plastic bags (e.g., Ziploc) for at least three days to allow the moisture to equilibrate. The specimens were then unsealed for the electrical conductivity measurement and sealed again for freezing and thawing test.

The degree of saturation in this study was calculated following a similar procedure as the one described by Fagerlund [1977]. Detailed information is provided in Appendix II.

4.2.4 Measurement of Electrical Conductivity

Electrical conductivity has been used to study microstructure development, permeability, diffusivity, drying, and crack growth [Christensen et al., 1994, Whittington et al., 1981, Snyder et al., 2000, Rajabipour et al., 2004, and Gu et al., 1993]. In the present work, impedance spectroscopy (IS) was used on concrete specimens to evaluate two properties. First, electrical conductivity was measured on the sound specimens (76×102×203mm) to describe the degree of saturation of concrete. Second, IS measurements were conducted on the slices (76×102×51 mm) cut from specimens damaged by tensile loading and freezing and thawing cycles to evaluate how the damage level influences the electrical conductivity of concrete.

The experimental geometry used for electrical conductivity testing is shown in Figure 4.1. Two 76×102×5mm electrodes (stainless steel plates) were clamped on the two ends of the specimen (normally 76×102×203 (or 51) mm). A water saturated sponge pad was placed between an electrode and one end of the specimen to provide sufficient electrochemical contact. Each specimen was subjected to a sinusoidal AC voltage with amplitude of 250 mV over a wide frequency range of 100 to 10⁷ Hz. The bulk resistance of concrete was determined as the real component (Z') of the impedance at the lowest value of imaginary component ($-Z''$). The electrical conductivity (σ) was computed from the bulk resistance using equation 4-6 [Whittington et al., 1981]:

$$\sigma = \frac{1}{\rho} = \frac{L}{R_B A} \quad 4-6$$

Here R_B is the bulk resistance of concrete, ρ is the bulk resistivity of concrete, L is the distance between electrodes, and A is the transfer area of the current (area of electrode).

Figure 4.1 Impedance Spectroscopy Test Setup

4.2.5 Freeze and Thaw Test and Resonant Frequency Method

After the specimens were saturated and electrical conductivity measurements were performed, freezing and thawing was conducted on the sealed specimens (76×102×203mm) in an environmental room that enables one freeze and thaw cycle per day. The room temperature was varied from -16 °C to 23 °C with a freezing rate of 4.2 °C/hour and a thawing rate of 5.8 °C/hour. In each cycle, the temperature was kept at -16°C for 5 hours to promote freezing and at 23°C for 4.3 hours for complete thawing.

The relative dynamic modulus (E/E_0 , a ratio of the dynamic modulus at n^{th} cycle over the dynamic modulus at 0^{th} cycle) was used to assess the damage level of the specimens as a result of freezing and thawing. ASTM E1876 [2002] method was followed to measure the flexural resonant frequency (out-of-plane) of the specimens after every freeze and thaw cycle. A constant Poisson's ratio (0.22) was used in all calculations due to the fact that changes in Poisson's ratio during freezing and thawing does not significantly influence the value of the dynamic modulus (a 10% reduction in Poisson's ratio resulted in a decrease of 0.4% in the dynamic modulus).

4.2.6 Water Absorption and Sorptivity Test

A ponding absorption test was performed using an approach that is similar to that described by Bentz et al. [2001], to evaluate the effects of damage on water ingress in unsaturated concrete. Internal damage in concrete was either generated by tensile loading or freezing and thawing. For the tensile loading, the specimens (76×102×432 mm) were

unloaded after a desirable level of tensile load applied, and the central portion (76×102×203 mm) of each specimen was cut to slices (76×102×51 mm). For the freeze and thaw testing, specimens were taken out of the chamber after a desirable drop in dynamic modulus of elasticity (e.g., $E/E_0=0.85$, 0.70, or 0.40) was achieved, and cut to slices (76×102×51 mm). All slices were then pre-conditioned in an oven with a controlled temperature of 50 °C for 14 days. It should be noted that this oven drying may introduce some microcracking damage that may not occur in an undried specimen [Bentz et al. 2001]. The sides of each slice were coated with an epoxy and a ponding dyke (about 25 mm deep) was prepared on the top of each specimen slice. Water was added to the pond (approximately 15 mm deep), and the mass change of each slice was recorded regularly (at 0, 0.1, 0.2, 0.5, 1, 5, 12, 24, 48, 96 hours, then at 96 hour intervals after that) after water was removed and the top surface of concrete was toweled to a condition of saturated surface dry. Characteristics of water absorption can be obtained by analyzing the change of cumulative water absorption with the square root of time as described later in section 4.3.2.2.

4.2.7 Microscopy

A stereo optical microscope was used to determine the hardened air content of concrete using 25 mm thick polished slices. The modified point count method [ASTM C457, 1998] was used. The general condition and crack patterns of the concrete were also examined and recorded.

A scanning electron microscope was employed in combination with the optical microscope to examine the microstructure and crack patterns under higher magnification. Concrete specimens were cut to small pieces (approximately 13 mm cubes) and dried at 50°C for 2 days. After drying, all the pieces were embedded into a low viscosity epoxy that is specially designed for SEM using cylindrical molds (32x32 mm), and kept under vacuum for approximately 3 hours. Specimens were removed from the molds after being cured in an oven at 72°C for 8 hours. Then, they were re-cut to a proper size to remove the top layer of the embedded medium using ISOMET low speed saw. After re-cutting, the specimens were polished using lapping wheels (45 μm to 0.25 μm). After polishing, specimens were coated with a film of conductive metals using LADD H-6.2 SPUTTER COATER. Finally, a strip of metal was bound to the edge of each specimen to avoid the build up of electrical charge within the specimen. Backscattered mode was used to acquire images at 15, 50, 100, and 1200X respectively. Observations were performed by scanning the surface of the specimen. A representative view was selected to illustrate the microstructure and crack patterns.

4.2.8 Direct Tensile Test and Acoustic Emission

A uniaxial tensile test was conducted to generate desirable degrees of damage (i.e., by applying 60%, 80%, or 90% of the peak load). The test setup is similar to that described by Li et al. [1993] as shown in Figure 4.2. It can be seen that steel plates were glued to the surface of concrete specimen for applying tensile load. The steel plates were sandblasted to remove surface oil and rust before the specimens were bonded to the plates using a high strength and high modulus epoxy. The steel plate thickness at one end

gradually decreased from 12.7 mm to 1.5 mm to avoid stress concentration when loading. Two LVDTs with working range of ± 5 mm, and gage length 200 mm were glued on the two sides of the specimen. Displacement control modes were employed in all the tests with the rate of 0.05 mm per minute.

Acoustic emission was employed to determine the extent and distribution of damage along specimens [Li and Li, 2000, Yoon et al., 2000, and Kim and Weiss, 2003]. Whenever damage occurred due to the deformation or deterioration of concrete under loading, it created a sound wave that was registered by transducers as electrical signals. Four AE transducers that were evenly attached to the front face of the specimen were used in this study (Figure 4.2). These transducers are highly sensitive with a frequency range of 250-700 KHz and they are convenient to use due to their small size (20.5 mm in diameter and 14 mm high) and light weight (12 g). The recorded signals were amplified by preamplifiers and then recorded by the system. AE source location analysis was performed using the location analyzer and one-dimensional approximation and the acoustic emission energy defined as the integration of the absolute value of recorded voltage was used to measure the internal damage level in this study.

Figure 4.2 Direct Tensile Test Setup

4.3 Experimental Results and Discussions

4.3.1 Degree of Saturation and Conductivity

The effect of degree of saturation on the connectivity of water phase in concrete has been experimentally examined based on the electrical conductivity of concrete using impedance spectroscopy. Figure 4.3 shows the typical result of how degree of saturation

influences the electrical conductivity of concrete. Many factors may influence the electrical conductivity of concrete including porosity, degree of saturation, composition of the pore solution, and pore size and connectivity [[Rajabipour et al., 2004](#)]. However, for each series of specimens in this study, the only factor that was varied was the degree of saturation. As expected, an increase in the degree of saturation resulted in an increased electrical conductivity. This is due to the fact that the increase in the degree of saturation results in both an increase in the volume of water (a conductive phase) and an increase in the interconnectivity of the water phase. A knee point occurred at approximately 0.8 of degree of saturation, below which the electrical conductivity increased rapidly with the degree of saturation. However, when the degree of saturation is above 0.8, it did not influence the electrical conductivity significantly. This suggests that once concrete reached a certain degree of saturation (i.e., 0.8), the water phase becomes well connected and increasing the amount of pore water does not influence the connectivity of water phase significantly.

Figure 4.3 Relationships between Degree of Saturation and Conductivity of Concrete

4.3.2 Effects of Freeze and Thaw Damage on Water Absorption and Electrical Conductivity of Concrete

4.3.2.1 Freeze and Thaw Damage Characterization

It was noticed that freeze and thaw damage (internal cracking) was well distributed across the specimen and frequently occurred at aggregate-paste interfaces and around air voids. Figures 4.4 and 4.5 illustrate the typical microstructural characteristics and the crack patterns obtained by examining the freeze and thaw damaged specimens by the SEM. It can be seen that crack width, density, and connectivity increased with the decrease in the relative dynamic modulus. A dense microstructure was observed under high magnification (100X) for a sound concrete ($E/E_0=1.0$, Figure 4.5a). However, in the specimen with a relative dynamic modulus of 0.86 micro-cracks became visible even at

low magnification (15X) (Figure 4.4a) and isolated micro-cracks were observed in the paste and the aggregate-paste interfaces under high magnification (Figure 4.5b). As the relative dynamic modulus dropped (i.e., $E/E_0=0.69$), micro-cracks were visible in the paste, the aggregate-paste interfaces as well as the regions surrounding the air voids (Figure 4.5c). These micro-cracks are continuous and partially connected. As the relative dynamic modulus was further reduced (to 0.40), cracks up to 100 μm in width were seen under both low and high magnifications (Figures 4.4a and 4.5d respectively) through paste, aggregate paste interfaces and air voids, and a well connected crack network was formed. It should be noted that visible cracks were observed in the specimens with a relative dynamic modulus of 0.40 during visual inspection in this study. The characteristics (e.g., width and connectivity) of these cracks will contribute significantly to the transport properties and conductivity of concrete which are described in the following sections.

Figure 4.4 Microstructures and Crack Patterns of Damaged Specimens by Freezing and Thawing at Low Magnification (SEM-backscattered, 15X).

Figure 4.5 Microstructures and Crack Patterns of Damaged Specimens by Freeze and Thaw Cycles at High Magnification (SEM-backscattered, 100X).

4.3.2.2 Water Absorption

The typical result of the cumulative water absorption (normalized per surface area) as a function of the square root of time is shown in Figure 4.6. Each curve represented a specific freeze and thaw damage level (E/E_0). It can be seen that the cumulative volume of water absorbed per surface area (mm^3/mm^2), also called water front (mm), in the specimens increased with the square root of time. However, the water front differed as the freeze and thaw damage level increased. The concrete specimen with the highest level of freeze and thaw damage in this study ($E/E_0=0.40$) has the highest rate of water absorption (i.e., slope of the water absorption curve or sorptivity). The concrete specimen with no freeze and thaw damage ($E/E_0=1.0$) has the slowest rate of water absorption. This indicates that freeze and thaw damage facilitated the water ingress in concrete. In particular, for the highly damaged concrete ($E/E_0=0.40$), the initial rate of water absorption was very fast thereby implying that the cracks and capillary pores were saturated in a very short time.

Figure 4.6 Comparison of Water Absorption as a Function of Square Root of Time

In this study, two parameters were used for describing the transport properties of unsaturated concrete. The first parameter was the total water absorption defined as the total weight of water absorbed after 45 days divided by the dry weight of the concrete specimen (expressed in percents). The second parameter, termed as initial sorptivity, described the initial rate of water absorption within 24 hours as obtained from the initial slope of the absorption curve.

The relationship between the initial sorptivity and the relative dynamic modulus is shown in Figure 4.7. The initial sorptivity appears approximately linearly proportional to the damage levels (measured as the relative dynamic modulus). A decrease in relative dynamic modulus of 10% results in a 3% increase in the initial sorptivity. Figure 4.8 shows how the total water absorption changes with the change in the relative dynamic modulus. Similarly to what was previously observed for sorptivity, the total water absorption also increases linearly with a decrease in the relative dynamic modulus. In this case, a 10% decrease in the relative dynamic modulus resulted in a 4.5% increase in the water absorption.

Figure 4.7 Relationships between Initial Sorptivity and Freeze and Thaw Damage (E/E₀)

Figure 4.8 Relationships between Total Water Absorption and Freeze and Thaw Damage (E/E₀)

4.3.2.3 Electrical Conductivity

The electrical conductivity of damaged concrete provides a method to characterize the relevant transport properties and connectivity of crack networks. A Nyquist plot is shown in Figure 4.9a illustrating the response of samples with the varying levels of damage. The value of the real component (Z') at the minimum value of imaginary component ($-Z''$) is defined as the bulk resistance of concrete [Snyder et al., 2000]. It can be seen that an increase in damage results in a reduction in the bulk resistance.

Figure 4.9b gives the relationship between the electrical conductivity and the relative dynamic modulus. The electrical conductivity demonstrated bi-linear relationship with the increase in the relative dynamic modulus. For low levels of damage (i.e., when the relative dynamic modulus was higher than 0.75), a 10% decrease in the relative dynamic modulus results in 20% reduction in the electrical conductivity. The change in electrical conductivity begins to become much less dramatic after the stiffness dropped below 75% of its original value. This suggests that at low levels of damage (i.e., $E/E_0 > 0.75$) the development of small microcrack pockets resulted in “small cracks” that created short circuit in the microstructure allowing current flow. However, as the cracks coalesced further, an interconnected network of cracks was formed that allowed for current flow. A subsequent increase in freeze and thaw damage may increase the crack width and density, but the change in interconnectivity was insignificant. This is validated

by the microscopic examination. When E/E_0 dropped below 0.75, microcracks coalesced (Figures 4.5c and 4.5d) and formed a well connected crack network.

Figure 4.9 Results of Impedance Spectroscopy Study, (a) Typical Nyquist Plot for Calculating Bulk Resistance, and (b) Electrical Conductivity and Relative Dynamic Modulus

4.3.3 Effects of Loading Induced Damage on Water Absorption and Electrical Conductivity

4.3.3.1 Loading Induced Damage Characterization

Figure 4.10 provides a typical result describing the relationship between the acoustic energy release rate and the stress-strain response. The extent of damage induced by tensile loading can be divided into three regions based on the stress-strain response: (1) the linear portion, (2) non-linear range, and (3) highly non-linear region. At lower stress level, a linear elastic behavior of concrete was observed. The linear response began to deviate at the time when the tensile stress reached 60% of the peak stress

indicating the development of micro-cracking. At approximately 90% of peak stress the non-linearity increased greatly presumably due to the formation of a localized crack band.

The extent of damage was characterized using the acoustic emission energy release rate that was measured at various intervals during the test (Figure 4.10). It can be seen that at 60% of peak stress the AE energy release rate increased which is consistent with the onset of substantial microcracking and non-linearity in the curve. An additional increase in AE energy release rate occurred at 80% of peak stress. It can also be seen that at 90% of ultimate stress, a great increase in AE energy release rate occurred due to the formation of a cohesive crack or a coalesced damage zone.

Figure 4.10 Relationships between Acoustic Emission Energy Release Rate and Stress-Strain Response (Hardened Air Content 6.6%)

After specimens were loaded to a specific loading level (60%, 80%, or 90%) and unloaded, they were examined under microscopes and two types of cracks were identified. One type was the bond crack through the aggregate-paste interface and the other type was the paste crack. At low stress levels (up to 80% of ultimate strength), single discontinuous cracks were found at the interface or in the paste but these were few in number. As load increased to 90% of peak load, coalesced cracks (bond cracks connected with paste cracks) were observed. Figure 4.11 is an example of typical crack pattern induced by tensile loading at 90% stress level. Micro-cracks present in the paste combined with the cracks present at the aggregate-paste interfaces forming a more continuous crack network. However, it should be noted that load induced cracks were not well distributed and linked even at 90% stress level.

Figure 4.11 Crack Patterns of Concrete Subjected to Tension at the Stress Level of 90% of Ultimate Strength

4.3.3.2 Damage Localization

Locations of cracks and microcracks developed by loading were characterized using AE source location analysis. It can be seen that the AE energy generation during loading was not evenly distributed along the specimen (Figure 4.12). The highest cumulative AE energy was registered at the left portion of the specimen between -5 cm and -7.5 cm from the center, while very low cumulative AE energy was recorded at other locations. This indicates that damage induced by tension was localized at a very narrow portion of the specimen where the major crack actually occurred.

Figure 4.12 Typical Results from Damage Location Analysis

4.3.3.3 Water Absorption

The effect of damage on water ingress into concrete structures depends not only on the extent of damage but also on the location of damage. Sections with higher localized damage may have higher water absorption, while sections with low, randomly-distributed damage may behave as a sound concrete, or absorb water at much lower rate than the sections with localized damage.

Sorptivity tests were conducted to determine how loading induced localized damage affects the water absorption of concrete. Figure 4.13 provides an example of a specimen loaded to the 90% stress level. It contains the distribution of cumulative AE energy along the specimen and the initial sorptivity and total water absorption of each slice that was cut from a specific location. It can be seen that the locations that have higher cumulative AE energy have higher initial sorptivity and total water absorption. Since the cumulative AE energy correlates with the extent of damage in concrete, a higher cumulative AE energy level represents a higher internal damage level.

The amount and rate of water absorption was found to be insensitive to the applied stress levels up to 90% of ultimate strength. This can be attributed to the discrete and localized features of load induced cracking. Increasing the stress level (e.g., to 90%) may result in a coalescence of bond and paste cracking, however this jointed cracking mainly occurred in a localized zone based on location analysis using acoustic emission (Figure 4.12) and would not significantly influence the overall water transport in

concrete. On the other hand, this locally jointed cracking was still not well connected (Figure 4.11) and the water flow was primarily controlled by the continuous intact area.

Figure 4.13 Relationships between Acoustic Emission Energy and Water Absorption for Specimen Loaded to a Stress Level of 90% Peak Stress

Similarly, tensile loading up to 90% stress level does not affect conductivity of concrete (Figure 4.14). This, again, can be attributed to the fact that load induced cracks are discontinuous, and few in number. An increase in load level from 60% to 90% does not significantly increase the overall density and connectivity of cracking.

Figure 4.14 Relationships between Electrical Conductivity and Tensile Loading Levels

4.4 Conclusions

Quantitative information was obtained between damage caused by freezing and thawing, water absorption, and electrical conductivity of concrete.

- Both the rate and the total amount of water absorption increased linearly with the increase in freeze and thaw damage. A decrease in relative dynamic modulus of 10% resulted in a 3% increase in the rate of water absorption and a 4.5% increase in the total amount of water absorbed. The electrical conductivity of concrete increased with the increase in the freeze and thaw damage and showed a bi-linear relationship with the relative dynamic modulus. The knee point in this relationship appears to correspond to about 75% reduction in the relative dynamic modulus. Below that point, the cracks are well connected and a dramatic decrease in the performance of

concrete could be expected since a well established crack network will provide preferential flow paths for water ingress.

- The localized load-induced damage at 90% stress level also increased to a much less degree the rate and the amount of water absorption. However, water absorption was insensitive to stress levels up to 90% of ultimate strength. Similarly, the electrical conductivity was not correlated well with the load induced damage. This is partly due to the fact that the load induced cracks or micro-cracks in concrete specimens up to 90% stress level are not well connected based on microscopic examination and are strongly localized based on acoustic emission analysis.
- Water absorption and electrical conductivity greatly depend on the characteristics of cracking (i.e., connectivity and distribution). Freezing and thawing process generates a network of well-distributed and uniform cracks. As a result, the rate of water absorption and electrical conductivity increase 2-3 times with the increase in freeze and thaw damage. However, load induced cracks are discrete and have local effects, and thus do not significantly influence the overall transport and electrical properties of concrete.

CHAPTER 5 ASSESSING DAMAGE DEVELOPMENT CAUSED BY UNIAXIAL TENSILE LOADING AND QUANTIFYING ITS INFLUENCE ON FREEZE AND THAW DURABILITY OF CONCRETE

When concrete is loaded in tension, damage develops in the form of cracking. It is hypothesized that this damage can act to accelerate the freeze and thaw deterioration. Acoustic emission was used to better understand and quantify the damage as it develops under mechanical loading. Acoustic events were analyzed in an attempt to use AE event amplitude, event duration, and energy to evaluate the extent of damage that occurred during tensile loading. Cumulative AE energy was found to correlate with damage. In addition, wavelet analysis revealed four primary types of AE signals that occurred during tensile loading. These signals are believed to be associated with different damage mechanisms. Internal damage was found to increase when the load reached 60%, 80%, and 90% of peak load. These loading levels appear to correspond with the microcracking, crack coalescence, and a formation of a localized crack band. A series of specimens were loaded in tension, to well defined damage levels, before being subjected to freezing and thawing. Results showed that the specimens with higher damage levels appeared to have higher water absorption and were less resistant to the freeze and thaw cycles.

5.1 Introduction

Cracks can form in concrete as a result of either mechanical or environmental loading. Recent studies have shown that cracking can increase the ingress of water and

aggressive agents in concrete, thereby increasing its susceptibility to deterioration [Soroushian and Elzafraney, 2003, Gowripalan et al., 2000, and Mohammed et al., 2003]. Work at Cornell University in the 1960's looked at quantifying the development of micro-cracking and its effects on the mechanical response of concrete. Over the last two decades, researchers have begun to make a more concentrated effort on understanding how cracking influences the water transport and durability of concrete. For example, work by Wang et al. [1997] has shown that tensile splitting cracks can increase permeability by several orders of magnitude. Samaha and Hover [1992] showed that micro-cracking caused by compressive loading increased water absorption and that highly stressed concrete appeared to be more absorptive. However, work by Hearn [1999] indicated that water permeability is insensitive to load induced cracking while drying shrinkage cracking increased the water permeability by several orders of magnitude.

In addition to changes in water absorption, recent studies have indicated that damage (i.e., cracking) can result in accelerating various types of deterioration [Yoon et al., 2000, Černý et al., 2001, Zhou et al., 1994, Sun et al., 1999, and Mu et al., 2002] Yoon et al. [2000] and Mohammed et al. [2003] reported that pre-loading of undamaged reinforced concrete beams resulted in a more rapid initiation and propagation of corrosion. Yoon et al. [2000] also showed that presence of a sustained load resulted in faster corrosion than observed in a beam that was loaded and unloaded suggesting that a wide crack would have a higher rate of ingress of corrosive agents.

Zhou et al. [1994] and Zhou [1994] reported that preloaded (by flexure) specimens were less resistant to freezing and thawing. Sun et al. [1999] and Mu et al. [2002] studied the effects of sustained flexural loading on the freeze and thaw durability and concluded that the higher the stress ratio, the faster the damage developed during freezing and thawing.

Recently, interest has been developed in using life cycle models to predict the serviceability of existing concrete structures. Accurate simulation of performance of the material requires the ability to consider the cumulative effects of cracking on various deteriorations that may occur in the field.

5.2 Experimental Program

This study used mechanical loading (direct tension) to induce damage (cracking) in the test specimens. Once the specimens were pre-cracked, freezing and thawing was used to simulate weathering. Figure 5.1 provides a general schematic of test programs used in this study, details of which are described in sections below.

5.2.1 Specimen Preparation

A typical low slump paving concrete mixture was used in this investigation with a water to cement ratio of 0.42. This mixture contained limestone aggregate with a maximum size of 25 mm (1 in.) while the fine aggregate consisted of sand with fineness modulus of 2.71. A water reducer (WRDA 82) was used with a typical dosage from

60ml/m³ to 80 ml/m³ to achieve a slump of about 60mm. The air content was varied from 4.7% to 11.5% by varying the dosage of an air entraining admixture (Daravair 1400) from 0.3 to 3.0 ml/kg cement. Materials were mixed using a rotary pan mixer with a capacity of 0.05m³ following ASTM C192 [2002]. The mixture was cast into metal molds and consolidated on a vibrating table for 20 seconds. After finishing, all specimens were covered with wet burlap, stored at 23°C for 24 hours and then demolded. After demolding, the specimens were moist cured for at least 60 days at 23°C. Information on the mixture proportions and air content is provided in Table 5.1. A total of six batches were prepared. For each batch, ten (76×102×406mm) specimens were prepared and the tensile strength was determined using 2 to 4 of the beams. Three batches were selected and used to determine the impact of load induced damage on freeze and thaw durability.

Figure 5.1 Main Testing Programs Used in This Study

Table 5.1 Air Contents and Mixture Proportioning of Specimens

Batch No.	Air Contents (%)		Mixture Proportioning (0.05 m ³)
	Fresh*	Hardened**	
#1***	11.5	10.2	Cement: 17.6 kg Water: 8.6 kg Sand: 40.9 kg Coarse Aggregate: 50 kg Air entraining admixture: adjusted Water reducer: adjusted
#2	9.5	6.8	
#3***	7.4	5.5	
#4	6.6	6.6	
#5	5.5	4.4	
#6***	4.7	3.3	

* Fresh air content was measured following ASTM C231 procedure B [1997]

** Hardened air content was determined following ASTM C 457 [1998]

*** Batches #1, #3, #6 were only tested to determine how air content influence the tensile strength of concrete

5.2.2 Direct Tensile Test

The direct tensile test was used in this study to generate a specific level of damage in concrete. After the initial strength determination, each of the remaining specimens from the selected batch was then loaded to a specific stress level (e.g. 60%, 80%, or 90%) and then unloaded. The test setup was based on a previous study by Li et al. [1993] using the geometry shown in Figure 5.1. Each end of the specimen was bonded to two loading plates and the plates were then mounted to a loading fixture that was subsequently used to apply load to the specimen by connecting to either the actuator or the load cell. Two LVDTs with a gage length of 203mm and a working range of ± 5 mm were used. One LVDT was attached to either side of the specimen, while another LVDT (the stroke LVDT) was used as the feedback to obtain a displacement rate of 0.05 mm/min.

5.2.3 Acoustic Emission

Acoustic emission (AE) has been used in the present study to characterize and locate the damage in concrete under loading [Kim and Weiss, 2003, Li and Shah, 1994, and Yuyama et al., 2001]. When a crack develops in concrete, an elastic wave is generated that propagates and is detectable by acoustic emission transducers attached to the surface of specimens. Since AE signals contain information about the damage processes, the type, extent, and distribution of internal damage can be estimated by analyzing the AE behavior.

Four broadband transducers (250-700 kHz, 20.5 mm in diameter, and 14 mm high) were used in this investigation. The preamplifiers had a wideband response (2.5 kHz to 3 MHz) and a gain setting of 34 dB. Signals were recorded and amplified by preamplifiers and the digitalized signals and waveforms were registered by the system. AE analysis was performed using Visual-AE®, Visual-TR®, and AGU-Wavelet® software. In addition, the AE source location analysis was performed using the location analyzer and one-dimensional approximation.

5.2.4 Freezing and Thawing Test

After a specific tensile load was applied, specimens were unloaded and the middle portion (76x102x203 mm) of each specimen was sectioned and placed in a specimen container like that used in ASTM C666 [2003]. The container was designed in such a way that when water was introduced, it surrounded the specimen with a thickness of about 3 mm. The specimens were exposed to freezing and thawing test using one cycle

per day and varying the temperature from -16°C to 23°C . For each specimen, the fundamental flexural resonant frequency was measured following ASTM E1876 [2002] and the mass gain was recorded during freezing and thawing. The dynamic modulus of elasticity (E) was calculated following ASTM E1876 [2002] and the relative dynamic modulus (E/E_0) was used to assess the internal damage developed during freezing and thawing.

5.3 Experimental Results and Analysis

5.3.1 Stress and Strain Relationship

A typical tensile stress (σ)-strain (ϵ) relationship (Figure 4.10) of concrete shows two interesting points which can be used to divide the σ - ϵ curve into three regions: a linear portion, a non-linear range, and a highly nonlinear region. The first region of interest (the linear region) occurred between 0% and 60% of the ultimate stress. The second region was between 60% and 90% of the ultimate stress. The final region consisted of the intervals between 90% and 100% of ultimate stress. Qualitative information on the damage development can be obtained by analyzing the stress-strain response since the deviation of the curve is primarily caused by the crack initiation and growth. The non-linearity of the curve started at approximately 60% of ultimate stress suggesting that significant micro-cracking initiated after this point. High nonlinearity occurred at about 90% of ultimate stress, implying that a localized crack band formed. These observations agreed well with the results of acoustic emission that will be discussed later in this chapter.

5.3.2 Air Content and Tensile Strength

The effects of air voids on concrete properties are twofold: the air voids make concrete more resistant to the freeze-thaw attacks but less resistant to the mechanical loading. Figure 5.2 shows the relationship between the direct tensile strength and the air content. The analysis of these data indicates that an increase in air content by one percent reduced the direct tensile strength by approximately four percent.

Figure 5.2 Influence of Air Content on Direct Tensile Strength of Concrete

5.3.3 Characteristics of AE Sources in Concrete

Parametric AE analysis has been used to describe the overall damage in concrete elements [Li and Shah 1994, Ohtsu 1999, Ohtsu 1998, and Landis 1999]. Due to the

random size, shape, and distribution of aggregates, AE sources in concrete are rather difficult to describe. However, some qualitative and quantitative information can be obtained through the AE parametric analysis.

A typical set of parametric AE results from one specimen is illustrated in Figure 5.3 where AE event amplitude and duration were considered as a function of tensile strain and stress level. Low amplitude (40-60 dB) and short duration ($<200\mu\text{s}$) events occurred at low load levels. It is believed (based on limited microscopy work performed as a part of this study) that these events originated from the opening and growth of micro-cracks at the interface of paste and aggregate. Higher amplitude (60-70 dB) longer duration (400-600 μs) events occurred as the stress approached 60% of the peak stress which accompanied the deviation from linearity of stress-strain curve. Those events might relate to the nucleation of new micro-cracks in paste or the propagation of existing ones. With the increase of stress level from 60% to 80%, the shift to lower amplitude and longer duration implied that micro-cracking started coalescing. Very high amplitude (>70 dB) and long duration ($>2000\mu\text{s}$) events began at about 90% stress level which corresponded to the beginning of highly nonlinear region of stress-strain curve. This group of events can be attributed to the formation and propagation of a single localized crack band like that described by Bažant and Oh [1993].

Figure 5.3 Change of Classified AE Event Amplitude (a) and Duration (b) with Direct Tensile Strain and Stress

In addition, the digitized acoustic waveforms were analyzed using AGU-Wavelet software. Frequency spectra were calculated by the software using a Fast Fourier Transformation (FFT) and additional parameters such as the peak frequency were extracted (Figure 5.4). In addition, wavelet transforms were performed allowing the energy of each AE signal to be displayed in its time and frequency domain (Figure 5.5) [Hamstand et al., 2002 part I, and Hamstand et al., 2002 part II].

By reviewing the shape of frequency spectra, the peak frequency, and the AE signal energy distribution, the AE sources were classified into different categories to better discriminate the damage mechanisms. Four typical types of AE signals were observed as shown in Figure 5.4. The Type I signal had a short duration, short to medium arrival time, and low to medium amplitude. Its frequency spectrum had a large peak occurring approximately at 350 kHz (Figure 5.4a). The signal energy was concentrated in a frequency range of 300 kHz to 400 kHz (Figure 5.5a). The Type II signal was similar to Type I signal. However, it had a medium to long duration (Figure 5.4b). The Type III signal had a peak frequency of 100 kHz, medium to long arrival time, and very long duration (Figure 5.4c). The signal energy was intensively concentrated in a very narrow frequency range (Figure 5.5b). The type IV signal was of high amplitude and long duration and medium to long arrival time. Its frequency spectrum only contained one large peak at about 100 KHz (Figure 5.4d). It should be pointed out that while the aforementioned types of waveforms accounted for a substantial portion of the AE signals, some signals could not be classified due to the complexity of the wave.

(a)

(b)

(c)

(d)

Figure 5.4 Characteristics of AE Waveforms and Frequency Spectra: (a) Type I, (b) Type II, (c) Type III, and (d) Type IV

Figure 5.5 Examples of AE Signal Energy Distribution in Time and Frequency Domain, (a) Type I, and (b) Type III

The classified AE events changed as a function of stress level as shown in Figure 5.6. The events that can be classified as Type I signal occurred at very low stress levels and increased slowly as the load increased. At about 55% of the peak stress, a rapid increase in the number of Type I events was observed. Above this stress level, it continued to increase at a high rate until failure. The Type II signal initiated at about 20% of ultimate stress and started to increase considerably at about 60% of ultimate stress. Type III signal started to accumulate shortly after the Type II signal. Both of Type II and Type III signals accumulated at the same rate until 95% ultimate stress. However, Type III is less numerous than Type II. The type IV signal occurred at about 95% of peak stress and continued to increase until failure.

Figure 5.6 Histories of Classified AE Events as a Function of Stress Level (Hardened Air Content 6.6%Vol.)

It is proposed herein that these four types of signals could be correlated with different damage sources. It is commonly accepted that damage sources in concrete mainly include paste cracking, de-bonding between paste and aggregates, and physical failure of aggregates. Comparison of the history of the classified AE signals with the commonly accepted damage mechanisms in concrete suggests that the Type I and Type II signals may be related to the paste cracking (micro-crack initiation and micro-crack propagation respectively), Type III to de-bonding, and Type IV to physical failure of aggregates. Further studies would be needed to validate these hypotheses.

5.3.4 AE Energy and Internal Damage Development

The AE energy was calculated in this study by integrating the recorded voltage (from the acoustic sensor) over time, which was actually a measure of the magnitude of an acoustic electrical signal registered. Therefore, this rough measurement of the cumulative AE energy provides a reasonable estimate of the extent of damage in concrete. The AE energy release rate was calculated by dividing the average value of cumulative AE energy increments (from four transducers) by the tensile strain increments. By monitoring AE energy release rate, the critical points at which rapid damage development occurred can be easily identified.

The relationship between the cumulative AE energy release rate and the stress levels is provided in Figure 5.7. It can be seen that below 60% of peak stress very low levels of cumulative AE energy and AE energy release rate were recorded, implying that damage developed very slowly in this stage. When the stress increased to approximately 60% of peak stress, a significant increase in AE energy release rate was observed due to the microcracking propagation and the beginning of debonding as illustrated in Figure 5.6. Once the stress level reached about 80% of peak stress significant changes in cumulative AE energy were detected due to an increased occurrence of debonding and paste cracking. When stress level increased to 90%, dramatic increase in AE energy release rate was observed. It should be noted that the AE energy release rate increased and decreased, but not continuously from 60% to 90% stress level implying that cracks were arrested and prevented from further propagation. After the stress level reached 90% of peak stress, high AE energy release rates were recorded due to the formation of a

localized macro-cracking region (including cracking of aggregates). As stress approached peak stress, impending failure occurred resulting in the saturation of the AE transducers.

Figure 5.7 Changes of Cumulative AE Energy Release Rate with Direct Tensile Strain and Direct Tensile Stress Level (Hardened Air Content 6.6% Vol.)

The cumulative AE energy was correlated with the tensile stress level based on the classified AE signals (Figure 5.8). It can be seen that below 90% of peak stress, the majority of AE energy was contributed by Type I and Type II signals. Type III signal played a less significant role. Between 90% and 95% ultimate stress, the AE energy generated by Type II and Type III signals increased very rapidly. Above 95% stress level, AE energy was mainly dictated by Type III and Type IV signals. This suggests that up to 90% of peak stress damage was mainly dominated by the micro-cracking

initiation, propagation, and some debonding. Between 90% and 95% of peak stresses, the damage mainly developed as a result of the crack propagation and debonding. Above 95% ultimate stress, the damage predominantly developed in the form of macro-cracking propagation (including fracturing of aggregates) as well as debonding.

Figure 5.8 Histories of Classified AE Energy as a Function of Tensile Stress Level (Hardened Air Content 6.6% Vol.)

5.3.5 Evaluation of Controlled Damage in Concrete

Stress (or strain) level is commonly used as an indicator of internal damage level of concrete during mechanical testing. However, due to the heterogeneity of concrete and its complicated local responses to mechanical loading, overall specimen stress and strain may not provide sufficient indication on the level of damage that may exist in a pavement. This occurs since the actual strength of the specimen at failure may differ from the average strength or average failure strain of the specimens at failure. Rather the

ultimate strength is likely more related to the extent of damage that has developed in a specimen. To try to capture this behavior Table 5.2 shows different damage levels in terms of the stress level, strain level, and cumulative AE energy level. Four specimens (a, b, c, and d) with same mixture proportions and curing conditions were loaded under tension until failure. The cumulative AE energy level was calculated by dividing the cumulative AE energy at the stress level of interest by the maximum cumulative AE energy registered before the saturation of AE transducers. For example, when stress level reached 90% of ultimate stress, the average strain level was only 82.4% of the way to failure and the average cumulative AE energy level was only 45% of the way to failure. As a result, using the stress or strain level alone may not be the best way to represent the extent of internal damage in concrete.

Table 5.2 Comparison of Different Damage Levels

Damage Levels Specimen ID	95% Stress level		90% Stress Level		80% Stress Level		60% Stress Level	
	Strain Level (%)	Cum. AE Energy Level (%)	Strain Level (%)	Cum. AE Energy Level (%)	Strain Level (%)	Cum. AE Energy Level (%)	Strain Level (%)	Cum. AE Energy Level (%)
(a)	86.4	45.4	80.3	28.3	72.4	22.9	51.2	12.1
(b)	88.3	53.2	83.0	36.1	71.8	26.0	52.6	8.0
(c)	89.7	82.7	83.3	58.4	72.1	35.6	52.5	21.9
(d)	89.0	72.9	83.0	56.9	73.6	40.2	54.5	24.9
Average	88.4	63.5	82.4	45.0	72.5	31.2	52.7	16.7

The cumulative AE energy was approximated as a cubic polynomial equation with strain as the variable. Figure 5.9 provides a typical illustration of a specimen with 6.6% fresh air by volume. This suggests that the cumulative AE energy level (or damage) can now be estimated based on the measured tensile strain.

5.3.6 Damage Location Analysis

The effects of damage on the durability of concrete structures depend not only on the extent of damage, but on the distribution of damage as well. Sections with higher localized damage may be less durable and deteriorate more rapidly during environmental attack, while sections with low randomly distributed damage may be as durable as sound concrete, or deteriorate much slower than the sections with localized damage.

Figure 5.10 provides two examples of the AE source location analysis. In this case, specimens were pulled to failure to determine how the overall cumulative AE energy was distributed in the specimens. It can be seen that the cumulative AE energy is not evenly distributed in the specimen during tension, implying that load induced damage is localized at certain sections. Figure 5.11 shows in detail the damage distribution at different loading stages based on cumulative AE energy at each stage. When the stress level was less than 90% to 95% of the peak stress, the damage was randomly distributed along the concrete beam. The randomness of distribution of the cumulative AE energy at low stress levels implies random activities of micro-cracking and de-bonding. The AE energy registered above 90% to 95% of peak stress (95% in this case) shows a trend to localize on a narrow band of the specimen suggesting that this is mainly related to the development and propagation of the localized crack band.

Figure 5.10 Illustration of Damage Location Analysis Based on Cumulative AE Energy (Two Specimens with Same Air Content 6.6%)

Figure 5.11 Damage Development and Distribution in Concrete at Different Stress Levels (Hardened Air Content 6.6% Vol.)

5.3.7 Load Induced Damage and Freeze and Thaw Durability

Figure 5.12 shows that the dynamic modulus of elasticity of pre-cracked specimens (hardened air content of 6.6% by vol.) degraded with an increase in the number of freeze and thaw cycles. It can be seen that the specimens with higher estimated stress levels tended to deteriorate at an earlier age. The first specimen that failed in freezing and thawing test was of 90.2% stress level, followed by specimens with a stress level of 80.7%, 81.1%, 59.6%, 89.8%, 60.6%, and 0% of peak stress respectively. It should be noticed, however, that the specimens that were loaded to the same estimated stress level did not fail at the same time. In particular, one specimen that was pre-loaded to an estimated 89.8% stress level failed after 170 cycles of freezing and thawing. This particular specimen showed higher freeze-thaw durability than the specimens that were exposed to the lower stress levels. It is anticipated that this discrepancy occurs because the higher loading in this particular specimen was not sufficient to cause damage development in the form of localized cracking which would lead to lower freeze and thaw durability. As a result, the stress level calculated by dividing the applied load by the average strength did not accurately represent the actual damage level in concrete specimens. It could be hypothesized that this may simply be due to the fact that this particular specimen was stronger than the average specimen. For this reason, the cumulative AE energy level was used to provide supplementary information for better evaluation of the pre-existing damage in concrete. The damage index (DI) that combines the stress level, strain level, and cumulative energy level (Equation 5-1) may better describe the extent of damage in concrete. It can be seen from Figure 5.12 that the specimens that have higher DI value are less durable to freeze and thaw cycling. It

should also be noted that for the particular specimen (89.8% stress level) discussed earlier, the AE energy level and DI value are very low (11.3%). This indicates that the extent of pre-loaded damage in this specimen was very low despite of a higher estimated stress level. It is therefore suggested that the combination of stress level, strain level, and cumulative AE energy level may provide a more reliable way to evaluate the load induced damage in concrete than using stress alone, by which the influence of pre-existing damage on the freeze and thaw durability can be accurately assessed.

Figure 5.12 Degradation of Dynamic Modulus of Elasticity of Concrete with Different Load Induced Damage Undergoing Freezing and Thawing Test (Hardened Air Content 6.6% Vol.)

Figure 5.13 shows the weight change of specimens undergoing freezing and thawing test. It can be seen that the weight gain was related to pre-existing damage. Damage leads to more water absorption, and thus more weight gain. This is consistent with the fact that micro-cracks and cracks provide extra space and preferential pathways for water

transport. Accelerated deterioration can be anticipated since the concrete specimen would become more and more permeable. This again explains why concrete that has higher pre-existing damage is less durable.

Figure 5.13 Weight Change of Concrete with Different Load Induced Damage Undergoing Freezing and Thawing Test (Hardened Air Content 6.6% Vol.)

5.4 Conclusions

This chapter described an experimental program to investigate how cracking influences the freeze and thaw durability of concrete. In addition to the standard mechanical measurements, acoustic emission was used to evaluate the damage development in concrete. Based on this study, the following can be concluded:

- The stress-strain response began to deviate from linearity at approximately 50 to 60% of the peak stress. This is generally thought to correspond to the onset of micro-cracking. As the stress level increased, an increase in the rate of acoustic activity was also observed. When the load level reached 80% of the peak value, higher amplitude, longer duration AE events were observed which are believed to be due to the crack coalescence. As the stress level increased to approximately 90% of the peak load, it is believed that a crack band formed (as indicated by the high non-linearity in stress-strain response and high AE energy release rate) which ultimately leads to failure.
- Wavelet analysis was performed and four types of AE events were identified that may be related to different damage mechanisms (paste cracking, debonding, and aggregate fracture).
- Examination of the cumulative AE energy, source type, and location of AE events revealed that the load induced damage was randomly distributed in the specimens in the form of micro-cracking and de-bonding as the stress was below 90% to 95% of peak stress. Above 90% to 95% of peak stress, damage started localizing at a specific section and was mainly attributed to macro-cracking propagation and de-bonding.
- The freeze and thaw tests indicate that specimens with higher pre-existing damage are less durable. A damage index was used to illustrate that stress and strain may not be the best measures of the extent of damage that has occurred in a specimen. This indicates that the use of a cumulative AE energy measurement in combination of the

knowledge of the existing stress and strain levels may provide a better indication of internal damage development in concrete during loading.

It is proposed that a combination of stress level, strain level, and cumulative energy level could be used as damage index to indicate the extent of damage in concrete as shown in equation 5-1 where damage index (DI) is defined as:

$$DI = \frac{\Omega}{\Omega_0} \sqrt{\frac{\sigma}{\sigma_0}} \sqrt{\frac{\varepsilon}{\varepsilon_0}} \quad 5-1$$

where, Ω – cumulative AE energy measured at the time the stress was applied, Ω_0 – cumulative AE energy predicted using the cubic polynomial equation and the estimated failure strain, and σ_0 and ε_0 are the estimated average tensile strength and strain at failure respectively. While further research is needed to more completely develop the optimal form of the damage index that should be used (i.e., equation 5-1), it can be seen that this equation is essentially a combination of the normalized relative proportions of the energy at failure, strain at failure and stress at failure and it appears to work reasonably well.

Figure 5.9 Cumulative AE Energy Variation as a Function of Direct Tensile Strain (Hardened Air Content 6.6% Vol.)

CHAPTER 6 INFLUENCE OF FREEZE AND THAW DAMAGE ON MECHANICAL RESPONSE AND DAMAGE DEVELOPMENT OF CONCRETE UNDER TENSILE LOADING

This chapter discusses how the damage that is caused by freezing and thawing influences the mechanical properties of concrete when it is tested under tensile loading. A series of experiments were conducted in which saturated concrete was exposed to slow freezing and thawing (i.e., 1 cycle per day) to generate predetermined levels of freeze-thaw damage (quantified using image analysis, optical microscopy, and relative dynamic modulus). After a predetermined level of freeze-thaw damage was introduced in the concrete specimen, the mechanical response of the concrete was assessed through direct tensile loading.

The tensile strength and elastic modulus demonstrated a bi-linear relationship with freeze-thaw damage (as assessed through the relative dynamic modulus). The behavior of the lightly damaged concrete ($E/E_0 > 0.85$) was similar to that of an undamaged concrete when low levels of load were applied. It should however be noted that when a higher stress level was applied to lightly damaged concretes, the micro-cracking and non-linear response increased as the damage level increased. However, as the extent of freeze-thaw damage increased (i.e., $E/E_0 < 0.85$), the non-linearity of the stress and strain response increased significantly, even at low stress levels and the fracture surfaces showed a substantial increase in the percentage of aggregate that was pulled out as opposed to fracturing.

Acoustic emission was used to quantify both damage development and the size of the failure region. It was noted that in specimens with low levels of damage (i.e., $E/E_0 > 0.85$) a “single” major crack initiated and propagated to cause failure while specimens with extensive freeze and thaw damage (i.e., $E/E_0 < 0.75$) showed a damaged zone that occupied a broader region of specimen during failure.

6.1 Introduction

In service, cracks can develop in concrete due to mechanical loading or environmental loading. Damage in the form of microcracking in concrete can reduce strength and stiffness while permitting the accelerated ingress of water and aggressive chemicals. Recent studies have indicated that cracks in concrete increase the water flow rate [Wang et al., 1997 and Hearn, 1999], reduce the corrosion resistance of reinforcement [Yoon et al., 2000 and Qi, 2003], and lower the load carrying capacity of concrete [Tinic and Brühwiler, 1985 and Soroushian and Elzafraney, 2003]. It should be noted that damage from various deterioration mechanisms (mechanical and environmental) may add together to be more severe than either mechanism alone [Sun et al., 1999 and Mu et al., 2002]. For example, the increase in water ingress due to cracking may increase the susceptibility for further deterioration. A decrease in strength and stiffness would facilitate crack propagation during subsequent loading thus resulting in a more permeable and absorptive concrete.

It is becoming increasingly common to utilize numerical simulations to predict the long-term performance of concrete structures. Several service-life prediction models

have been proposed for pavements and concrete structures including LIFE-365 [Thomas and Bentz, 2001], Bridge LCC [Ehlen, 1999], and PaveSpec [Hoerner and Darter, 2000]. Unfortunately, cracking may occur in many structures which may not be explicitly accounted for in the deterioration modules of many of the existing models. Despite not being included in the models, cracking can have a profound impact on life cycle performance. Therefore, to accurately simulate the performance of actual concrete facilities, procedures that consider and quantify the role of cracking are required.

Although the fracture of undamaged concretes has been studied extensively to better understand how concrete structures will respond to loading [Jenq and Shah, 1991, Li and Shah, 1994, Li and Li, 2000, and Ueda et al., 1994], much less information is available on how existing cracking influences the life cycle performance. The role of pre-existing cracks was studied in this research program to better understand how cracking can influence the performance of actual field concrete.

The present study used freezing and thawing to induce a controlled level of damage in concrete. The freezing and thawing was followed by direct tensile testing to determine how the pre-existing damage influenced the mechanical properties and fracture behavior of concrete. Acoustic emission was used to monitor damage evolution and to identify the cracking that occurs in concrete. A vacuum epoxy impregnation technique was used to stabilize the damage before sectioning and polishing while image analysis was used to characterize the damage patterns and quantify the overall damaged area in concrete.

6.2 Experimental Approach and Procedures

Figure 6.1 illustrates an overview of the testing program that was undertaken in this research. The concrete specimens were conditioned to achieve different degrees of saturation (for further information, see section 4.2.3). Once the required level of saturation was reached, freezing and thawing was conducted to induce damage and the damage level was evaluated non-destructively using resonant frequency and the relative dynamic modulus (E/E_0). Two series of tests were conducted on the freeze-thaw damaged concrete. The first series of tests was performed to detect the crack patterns and to quantify the damaged area using a stained epoxy, vacuum impregnation, and image analysis. The second series of tests was performed to measure direct tensile response of damaged concrete and to determine the effects of existing damage on the mechanical properties in concrete. The mechanical response was evaluated based on the stress and strain relationship and visual examination of the fracture surfaces. Acoustic emission was used to characterize the damage initiation, propagation, and localization in a concrete, which help to better understand the mechanical behavior of the concrete.

6.2.1 Specimen Preparation

The specimens used in this study were made using an ordinary type I portland cement and a water to cement ratio (w/c) of 0.42. A crushed limestone was used as the coarse aggregate (maximum size of 25 mm) and river sand was used as the fine aggregate (fineness modulus of 2.71). The concrete mixture proportions were 1:2.85:2.37:0.42

(cement: coarse aggregate: fine aggregate: water by mass). A water reducer (WRDA 82) and an air-entraining admixture (Daravair 1400) were used to achieve a slump between 50 mm and 75 mm and the desired fresh air content (i.e., 4.7%, 6.2%, 6.6%, or 9.0%).

Figure 6.1 Experimental Program Used in This Study

The materials were mixed in a horizontal pan mixer with a typical batch size of 0.05 m^3 (1.8 ft^3). The specimens were cast and cured following ASTM C192 [2002]. Slump was measured in accordance with ASTM C143 [2000] while air content was determined following ASTM C231 [1997]. The fresh concrete was placed in the forms, externally vibrated for 20 seconds, screeded, and finished using a magnesium trowel. The concrete was then covered by a plastic sheet and stored under wet conditions at room temperature (23°C). After 24 hours, the specimens were then demolded and cured in a moist room (i.e., $\text{RH}>98\%$) at 23°C .

Four batches of specimens were prepared with fresh air contents of 4.7%, 6.2%, 6.6%, and 9.0% by volume. Six concrete prisms ($76 \times 102 \times 406$ mm) were cast for each batch (i.e., air content). After moist curing for at least six months, all the beams were cut in half ($76 \times 102 \times 203$ mm) and then used for freeze and thaw testing. Upon the completion of the freeze and thaw test, one batch (6.6% air) of 12 specimens were sliced for vacuum impregnation to quantify the damaged area, while the other three batches (36 total specimens) were used for direct tensile testing.

6.2.2 Specimen Saturation

The specimens in the present study were saturated following the procedure similar to that described by Fagerlund [1977]. The concrete specimens ($76 \times 102 \times 203$ mm) were dried at 50°C for 4 weeks, and then evacuated to a residual pressure of 130-260 Pa for 4-6 hours. Water was introduced while the vacuum pump was still running and the specimens were kept in water under vacuum for a minimum of 2 hours. After removal of vacuum, the specimens were kept in water under atmospheric conditions for 1 week for further saturation. After saturation, the specimens were adjusted to different saturation levels by allowing the specimens to dry at 50% relative humidity at 23°C for different length of time (typically 1-3 days) and monitoring the weight loss of each specimen. After a specific weight loss (corresponding to a specific saturation) was reached, the specimens were sealed to avoid further moisture loss and kept at room temperature for a week to enable the specimen to achieve a more uniform distribution of moisture.

6.2.3 Freeze and Thaw Test

After achieving the desired degree of saturation (e.g., 0.8, 0.85, 0.9, 0.95), each specimen was put into a rectangular specimen container similar to that described in ASTM C666 [2003]. Water was introduced into the container that surrounded the specimen providing a layer of water around the specimen that was approximately 3 mm thick. Freezing and thawing was performed in an environmental chamber that enabled one freeze and thaw cycle per day to introduce a desired level of internal damage (e.g., the relative dynamic modulus reached 0.9, 0.85, 0.8, 0.7, 0.6, 0.5, 0.4 etc. as described later). The temperature of the chamber varied from -16°C to 23°C as shown in Figure 6.1. The freezing rate was $4.2^{\circ}\text{C}/\text{hour}$ while the thawing rate was $5.8^{\circ}\text{C}/\text{hour}$.

The fundamental transverse resonant frequency described by ASTM E1876 [2002] was measured after each cycle to monitor the damage development during freezing and thawing. The dynamic modulus of elasticity was calculated following the procedures provided in ASTM E 1876 [2002] and the relative dynamic modulus (ratio of the dynamic modulus at n^{th} cycle over the dynamic modulus before freeze and thaw began (i.e., the 0^{th} cycle), E/E_0) was used as an index to indicate the extent of damage at that particular cycle.

In addition, water absorption was monitored to help evaluate the damage development in concrete. After a specified damage level was achieved, the specimen was removed from the chamber and prepared for either vacuum epoxy impregnation or direct tensile testing. It should be noted that when a specimen was heavily damaged (e.g., E/E_0

is below 0.40) and had absorbed a significant amount of water, no resonant frequency readings could be obtained due to the damping effect of water. For such a specimen, the resonant frequency was measured in this study by letting the specimen dry at 50% RH at 23°C (normally 2-10 hours) until a first stable reading was obtained.

6.2.4 Vacuum Epoxy Impregnation and Image Analysis

To quantify the percentage of damage in concrete (i.e., by area) a procedure was implemented in which the specimens were impregnated with epoxy under a vacuum before an image analysis procedure was performed [Puri and Weiss, 2004]. The benefit of using epoxy impregnation was that it could be used to stabilize the damaged area in order to minimize disruption and introduction of new damage during the cutting and polishing of the specimen. The following section describes how the specimens were prepared and analyzed.

After the specimens were exposed to the freezing and thawing, a wet-saw with a diamond blade was used to cut 76×102×60 mm slices from the original specimen (76×102×204 mm). These specimens were air-dried at 23 °C and 50% relative humidity for 15 days before being placed in a cylindrical mold that would eventually be filled with epoxy. Before introducing the epoxy however, the empty space between the specimen and the mold wall was filled with coarse aggregate to minimize the amount of epoxy to be added, thereby avoiding excessive heat generation. After placing the samples in the mold, the two ends of the mold were capped and sealed. The cap on the top of the mold contained two openings, one of which could be connected to the vacuum pump, while the

other port was used to introduce the epoxy. The epoxy used in this study was a two component low viscosity epoxy. Carbon black was dissolved into the epoxy before it hardened to aid in highlighting the cracks and damaged area.

To evacuate the air in the mold, a vacuum (a residual pressure of 650 Pa) was applied for one hour to the mold containing the specimens and coarse aggregates. The epoxy was introduced into the mold while the pump was running. The vacuum was kept running for 10 minutes. The mold was subsequently disconnected from the vacuum pump and cured at room temperature. After the epoxy hardened, the mold that contained concrete slices was sectioned using a diamond tipped saw. The surface of concrete was polished using a lapping wheel and polishing compound (silicon carbide grit from 150 to 800). A color image of each polished surface was captured by a flat-bed scanner with a typical size of 76×102 mm and a resolution of 12 pixels/mm. Additional microscopy was performed to study the structure of the material, crack patterns, and crack connectivity in concrete.

Image analysis was used to quantify the extent of damage in each specimen. The original color image was converted into a gray scale image as shown in Figure 6.2a. To segment the damaged areas from undamaged areas, a thresholding operation was performed in which the pixels were categorized based on the gray level intensity. The threshold value was defined by gradually increasing or decreasing the boundary value until the damaged area was highlighted as shown in Figure 6.2b. In some cases, since some aggregates or air voids might have the gray level that is close to the damaged

regions, these aggregates or voids would induce noise to the image. It can be noticed that aggregates or voids could be differentiated from cracks and damaged portion of the paste by their predominant shape. The air voids and aggregates tend to be round while the damaged areas were irregular and tortuous. As a result, the image can be cleaned by removing the circular objects (i.e., objects where aggregates and air voids occurred) using Adobe Photoshop as shown in Figure 6.2c. Once the damaged region was segmented and cleaned, the total area (or the area percentage of damaged regions) was measured. For example, the relative resonant frequency for the specimen shown in Figure 6.2 was 0.69 and the damaged area was measured to be 25%.

Figure 6.2 Illustration of Image Analysis Procedures; (a) Gray Level Image, (b) Binary Image after Thresholding, (c) Cleaned Image

6.2.5 Direct Tensile Test and Acoustic Emission

After a specific damage level was reached, two loading plates were glued to each end of the specimen using a high modulus, high strength epoxy. The test setup was illustrated in Figure 6.1. Two loading fixtures were used to connect the plates to the mechanical testing system. One LVDT with a working range of ± 5 mm was glued on each side of the specimen so that it had a gage length of 100 mm. Direct tensile testing was performed using a displacement control mode with a loading rate of 0.05 mm/min. Three piezoelectrical AE sensors were evenly placed on the front surface of the specimen to register the elastic waves caused by the release of strain energy in concrete. A Vallen AMSY4 AE system was used to record the signals and AE analysis was performed by Visual-AE, Visual-TR, and AGU-Vallen Wavelet software.

6.3 Test Results, Observations, and Analysis

6.3.1 Relate Damaged Area to Relative Dynamic Modulus

Figure 6.3 shows an example of two polished sections that correspond to two different levels of freeze and thaw damage. The damaged regions were well impregnated and clearly discernible. These regions may either represent porous deteriorated zones or small cracks that might have formed due to freezing and thawing. In the damaged specimen the paste can be seen to contain dark-colored regions that were filled with epoxy and these regions increased with the decrease of relative dynamic modulus (E/E_0).

Figure 6.3 Characterization of Crack Patterns and Damaged Area due to Freezing and Thawing; (a) $E/E_0=0.81$; (b) $E/E_0=0.76$

Figure 6.4 provides a quantitative relationship between the amount of damage observed from image analysis and the relative dynamic modulus. Upon further inspection it was seen that as the relative dynamic modulus (i.e., E/E_0) decreased from 1.0 to 0.9 the damage that developed was in the form of discontinuous micro-cracks. Microscopic examination revealed the presence of small discontinuous cracks at the paste-aggregate interface that are too small to be accessible for epoxy at the resolution of the flat-bed scanner. As the relative dynamic modulus reduced from 0.9 to 0.8, the damaged area increased suggesting that at this stage, micro-cracks may be beginning to coalesce to form isolated damage regions. As the dynamic modulus degraded further

(i.e., E/E_0 reduced from 0.8 to 0.7) a well-connected crack network was observed to form. As E/E_0 continued to decrease below 0.7, the damaged area continued to increase but at a much slower rate.

Figure 6.4 Quantification of Damaged Area due to Freezing and Thawing

6.3.2 Relate Existing Damage to Aggregate Pullout

After tensile testing, the failure surface was visually examined and the number of aggregates that fractured was compared to the number of aggregates that were observed to pull out of the paste. In specimens that were heavily damaged (i.e., $E/E_0 < 0.75$) the aggregates appeared to pull out of the paste while concrete without extensive damage (i.e., $E/E_0 > 0.75$) showed fractured aggregate. Figure 6.5 illustrates the relative number of aggregates that pulled-out of the paste as the freeze and thaw damage in the specimen

increased. A bi-linear relationship was found between percentage of pullout aggregate and relative dynamic modulus with a “knee point” that occurred when the dynamic modulus was reduced 25% (i.e., $E/E_0 = 0.75$). At low levels of damage (i.e., $E/E_0 > 0.75$) the percentage of aggregates that pulled out increased rapidly as the stiffness of the concrete degraded. Nearly 80% of the coarse aggregate was observed to pull out by the time the dynamic modulus reduced to 75% (i.e., $E/E_0 = 0.75$). After the stiffness was reduced to 75% of its original value, a slower increase in the number of pullout aggregates was observed.

Figure 6.5 Influence of Pre-existing Freeze and Thaw Damage (E/E_0) on Percentage of Aggregate Pull-out During Tension

The observation of aggregate pullout is consistent with the observation of distributed paste cracking in the freeze-thaw damaged specimen that enables aggregate debonding. For highly damaged specimens the low strength of the binder enables de-

bonding to gradually proceed as the load increases until the aggregates are pulled out. Less damaged specimens have a stronger paste (and paste and aggregate interface) that is able to transfer a higher stress to the aggregate which ultimately results in the failure of the aggregates.

6.3.3 Mechanical Response of Concrete with Existing Freeze-Thaw Damage

Typical stress-strain responses of specimens with increasing freeze-thaw damage are provided in Figure 6.6. For a concrete without pre-existing damage, the mechanism is well understood and commonly accepted [[Shah, 1997](#)]. Linear elastic behavior was observed up to 60% of peak stress. After 60% of the peak stress was achieved, the curve then deviated slightly due to the initiation of micro-cracking. As load increased, the micro-cracks grow and coalesce making the curve deviate from linearity considerably. The subsequent propagation of the crack leads to even more excessive deformation after the stress exceeds 90% of the peak stress which is followed by an abrupt failure of concrete. Conversely, concrete with pre-existing damage exhibits more non-linear behavior in the pre-peak region. The initial non-linearity may be attributed to the opening, closing, and sliding of pre-existing cracks as well as the contribution of new crack formation. Similarly, the post-peak response appears more ductile and the tensile strength and the stiffness decreased with the increase of freeze-thaw damage.

Figure 6.6 Comparison of Stress-Strain Relationship for Specimens with Different Levels of Freeze and Thaw Damage

The direct tensile strength decreased with an increase in the amount of pre-existing freeze-thaw damage. A bi-linear relationship was found with the “knee point” occurring when the stiffness was reduced by 25% (i.e., $E/E_0 = 0.75$) (Figure 6.7). The direct tensile strength of concrete is influenced by the presence of defects or cracks. The direct tensile strength of a concrete without pre-existing damage ($E/E_0 = 1.0$) was approximately 3.5 MPa. For this concrete a 10% reduction in dynamic modulus resulted in a 30% reduction in tensile strength. After the dynamic modulus reduces to approximately 75% of the undamaged concrete, the direct tensile strength continued to decrease but at a slower rate. This reduction in strength can be attributed to the presence of pre-existing cracks. Micro-cracks that developed as a result of freezing and thawing (E/E_0 is from 1.0 to 0.75) are mostly uniformly distributed and remain discontinuous.

The rapid decrease in direct tensile strength can be attributed to the rapid growth in size and number of these cracks.

Figure 6.7 Influences of Freeze and Thaw Damage on Tensile Strength of Concrete

The modulus of elasticity for the damaged specimens was calculated based on the slope of stress-strain curve between 10% and 40% peak stress. Figure 6.8 again shows a bi-linear relationship between modulus of elasticity and relative dynamic modulus (E/E_0) with a knee point at 0.75 of E/E_0 . It can be seen that above the knee point, the modulus of elasticity decreased at a higher rate. When E/E_0 decreased by 10%, the modulus of elasticity decreased by about 9.4 GPa (20%). Below the knee point, the modulus of elasticity would decrease at a lower rate. Every 10% decrease in E/E_0 would result in 2.5 GPa (0.56%) reduction in the modulus of elasticity.

Figure 6.8 Influences of Freeze and Thaw Damage on Modulus of Elasticity of Concrete

It should be noted that the elastic modulus degraded at a slower rate than tensile strength. This can be explained by the fact that pre-existing cracks result in stress concentrations that greatly influence tensile strength while elastic modulus is mainly dictated by the overall structure of the material.

6.3.4 Damage Development

The behavior of pre-damaged concrete was characterized using acoustic emission according to signal characteristics, event distribution, and the rate of event occurrence. Figure 6.9 shows one AE event characteristic (i.e., amplitude) for undamaged and damaged concrete (i.e., $E/E_0 = 1.0$ and 0.6 respectively). Low amplitude events are associated with micro-crack initiation, while higher amplitude events are related to the

crack propagation [Puri and Weiss, 2004]. In damaged concrete, (here $E/E_0 = 0.6$) various event amplitudes are observed even at the beginning of stress application, implying that crack initiation and propagation occurred simultaneously upon loading. It can also be seen that a higher number of AE events occurred indicating that substantial micro-cracking occurred during loading.

Figure 6.9 Change of Classified AE Event Amplitude with Normalized Tensile Strain and Tensile Stress, (a) $E/E_0=0.6$, and (b) $E/E_0=1.0$

When compared with the response of concrete without pre-existing damage, it can be seen that in the damaged concrete (i.e., $E/E_0 < 0.75$) there were no critical stress levels that corresponded to different damage mechanisms such as micro-cracking initiation, propagation, and coalescence and all these mechanisms could occur at any stress levels. It can also be seen that high amplitude (>80dB) events occurred at 80% pre-peak stress

in damaged concrete while in undamaged concrete these AE events occurred at about 90% stress level, thereby implying that macro-cracking formed at an earlier stress level in a pre-damaged concrete.

It has been shown that AE parameter (e.g., amplitude) analysis provided useful information on the damage development. It should however be noted that parameter analysis can be difficult in concrete due to the overlapping waves and filtrating effects. As a result, the AE parameters that are detected may differ based on the source, or two similar AE source events may appear different by the time the events reach the transducer. As a result, a more in-depth analysis may be needed. In this work a wavelet-based analysis was performed, in which the digitized waveform was analyzed and characteristic features such as peak frequency were extracted. In this study, the wavelet transform program (i.e., AGU-Vallen Wavelet) was used. Detailed information on this software was provided by Hamstad et al. [2002 part I and 2002 part II].

Three characteristic AE waveforms (Type I, II, and III) were identified in this study (e.g., $E/E_0=0.6$). These waveforms were correlated with possible damage mechanisms during tensile loading (see section 5.3.3). The Type I waveform (350 kHz peak frequency) was related to the paste crack initiation, the Type II waveform (350 kHz peak frequency) to the paste crack propagation, and the Type III waveform (100 kHz peak frequency) to the interfacial de-bonding. It is seen that the overall AE events were dominated by de-bonding (Type III) followed by paste cracking initiation (Type I) and propagation (Type II). A small quantity of paste cracking and de-bonding occurred

simultaneously at early stage of loading ($<70\%$ peak stress). A substantial increase was observed in de-bonding and paste cracking when the stress level reached approximately 90% of the peak stress. Figure 6.10a provides a description of how the cumulative AE energy changes as a function of these types of AE events along with a normalized strain and stress response. The AE energy associated with de-bonding increased from the beginning of load application, while a significant increase in the cumulative AE energy was observed at about 90% peak stress for paste cracking. As a result, de-bonding was the main damage mechanism that consumed the majority of applied energy and contributed to the toughening of concrete. This is different from the result of concrete without pre-existing damage as shown in Figure 6.10b, in which both paste cracking and de-bonding almost equally contributed to the damage development.

Figure 6.10 Changes of Cumulative AE Energy of Classified AE Events as a Function of Normalized Tensile Strain, (a) $E/E_0=0.6$, and (b) $E/E_0=1.0$

It should be noted that Type IV events (Figure 6.10b) that were observed to occur around the time of abrupt failure in concrete that may correspond to the fracture of aggregates. This type of events did not occur in the damaged specimen (Figure 6.10a). This is consistent with the results shown in Figure 6.5 that indicate that when the relative dynamic modulus was below 0.75 aggregate predominantly pulled out.

6.3.6 Location Analysis

The failure of concrete with limited damage (i.e., $E/E_0 > 0.75$) is characterized by a single major crack while the failure pattern for concrete with more substantial damage (i.e., $E/E_0 < 0.75$) exhibits multiple crack paths and distributed damage throughout the specimen based on the visual observation during tensile testing. This is consistent with the results from AE location analysis. Figure 6.11a compares the damage distribution of two specimens with relative dynamic modulus of 1.0 and 0.29 respectively. For concrete without pre-existing damage ($E/E_0 = 1.0$), AE energy was observed to concentrate at a narrow section (-15 mm to -35 mm) implying that load induced damage is localized on a small zone. For a heavily pre-damaged concrete ($E/E_0 = 0.29$), the AE energy is seen to be widely distributed along the specimen (40 mm to -40 mm). This suggests that a significant amount of damage was distributed along a wider range of the specimen when freeze-thaw damage was present.

Figure 6.11 Quantifying Size of Damage Zone, (a) Acoustic Emission Energy Distribution along Specimen, and (b) Size of Damage Zone Increases with Freeze-Thaw Damage

Figure 6.11b shows in detail how the load induced damage zone changes with the extent of pre-existing damage (i.e., the relative dynamic modulus). The damage zone in this plot is defined as the sum of all the locations along the specimen in which the cumulative AE energy exceeds a certain threshold value (e.g., 1.0, or 2.0 $\mu\text{V.s}$ as described in Figure 11b). The size of the damage zone increased with the decrease of relative dynamic modulus and a considerable increase in the damage zone occurred when the relative dynamic modulus decreased from 0.8 to 0.6. This can again be explained by the findings in Figure 6.4 that great increase in pre-existing damage occurred in this range. As a result, the degree of relaxation was dictated by the extent of pre-existing damage.

6.4 Conclusions

The pre-existing damage in concrete generated by freezing and thawing was evaluated based on the relative dynamic modulus. Damage patterns were detected using vacuum epoxy impregnation, and quantified using image analysis. The influence of the pre-existing damage was correlated with the mechanical properties using direct tensile testing, acoustic emission, and visual examination. Experimental results support the following conclusions:

- Freeze and thaw damage is manifested primarily as paste cracking and interfacial cracking. Isolated micro-cracks at paste or interface were observed in concrete with low damage levels ($E/E_0 > 0.8$). The extent and connectivity of the cracks increased rapidly as the relative dynamic modulus decreased (E/E_0 from 0.8 to 0.7). When the relative dynamic modulus reached 0.7, a well-connected crack network was seen.
- Visual observation indicated that the fracture section of a pre-damaged concrete is much more tortuous than that of a concrete without pre-existing damage. Aggregate pull-out occurred in a pre-damaged concrete. The percentage of pull-out aggregate showed a bilinear response with a knee point that occurred when the relative dynamic modulus reached approximately 0.75.
- For a specimen without pre-existing damage, linear elastic behavior was observed up to 60% of peak stress and a brittle failure was observed. For pre-damaged specimens however, the non-linearity occurred at an earlier stage of loading. Ductile failure was

observed and the descending branch became more and more obvious when relative dynamic modulus decreased.

- Observations from acoustic emission indicated that specimens with pre-existing damage have substantially different fracture behavior from undamaged concretes. For concrete without pre-existing damage, micro-cracking initiated at a critical stress level (e.g., 60% peak stress) and propagated at a higher critical stress level (e.g., 80% peak stress), while for a pre-damaged concrete crack initiation and propagation may occur simultaneously at very low stress level, or at the beginning of load application as the relative dynamic modulus decreased to a very low level (e.g., 0.6). The pre-existing damage relaxed the damage localization during loading and the size of load damaged zone increased with the decrease of relative dynamic modulus.
- The presence of pre-existing damage was observed to alter the mechanical properties of concrete. A relationship was observed between the mechanical properties and the relative dynamic modulus. The direct tensile strength and stiffness were found to decrease bi-linearly with the decrease of the relative dynamic modulus. The knee point was around 0.75 of the relative dynamic modulus. Above the knee point, a 10% decrease in the relative dynamic modulus resulted in 30% decrease in tensile strength and 20% decrease in modulus of elasticity. Below the knee point, 10% decrease in relative dynamic modulus led to 2.6% reduction in tensile strength and 0.56% reduction in modulus of elasticity.

CHAPTER 7 SUMMARY AND CONCLUSIONS

7.1 Introduction

Cracks may occur due to either mechanical loading or environmental loading which can reduce the overall strength and stiffness of concrete. The hypothesis of this report, that these cracks can permit increased fluid ingress and thereby these cracks can dramatically reduce the durability of concrete, was shown to be true. The accurate prediction of the performance of concrete structures should therefore account for the presence of cracking. This study attempts to provide quantitative information on how damage influences water absorption, freeze-thaw behavior, and mechanical properties of concrete. This research should be implemented in the pavement design and life-cycle performance modeling processes.

7.2 Findings and Conclusions

In this study, a field condition assessment of concrete pavements was conducted to assess the types of damage that occur in the field. Cores were taken from five concrete pavement sections based on age, traffic, and overall performance. It has been observed that the cracks that develop in the field typically have a wide range of characteristics (i.e., cracks occur at the aggregate paste interface, through aggregates, and through the paste). In damaged pavements, the sorptivity near the surface of the pavement was substantially

higher (up to 54%) than that near the middle or bottom of the pavement due to the presence of microcracking.

After the field investigation, laboratory studies were conducted to assess how cracking (cracking that was carefully and systematically introduced) from loading or freezing and thawing influences fluid transport, electrical conductivity, freeze and thaw durability, and mechanical response of concrete. These procedures are described in detail in the chapters of this report. Specimens were prepared that were exposed to freezing and thawing and mechanical loading to induce a specific level of damage. After damage was introduced, additional mechanical and durability testing was performed, the results of which are described below.

It has been shown that water absorption and electrical conductivity are dependent on the characteristics of the cracking that was developed in the concrete. Freeze and thaw damage results in a well-distributed crack network that increased the water absorption in a linear proportion to the extent of damage (i.e., E/E_0). A 10% decrease in E/E_0 led to a 3% increase in sorptivity and a 4.5% increase in total water absorbed. Freeze and thaw damage increased the electrical conductivity in a bi-linear fashion with a knee point when the stiffness was reduced to 75% of its original value (i.e., $E/E_0=0.75$), which corresponds with a well connected, coalesced crack network. Load induced damage does not appear to influence the water absorption or electrical conductivity at loading levels up to approximately 90% of peak load due to the discontinuous nature of the crack pattern. After the damage localized at approximately 90% of peak load, the

water absorption increased locally due to the presence of a crack band (damage zone) region.

The pre-existing load induced damage has been observed to influence the freeze and thaw durability of concrete. In general, specimens with higher levels of pre-existing damage are less durable.

The pre-existing damage in concrete generated by freezing and thawing has been bi-linearly related to the reduction in the overall strength and stiffness of concrete with a knee point when the stiffness was reduced to approximately 75% of the original value (i.e., $E/E_0 = 0.75$). Above the knee point, a 10% decrease in the relative dynamic modulus resulted in a 30% decrease in the tensile strength and a 20% decrease in the elastic modulus. Below the knee point, a 10% decrease in relative dynamic modulus led to a 2.6% reduction in the tensile strength and a 0.56% reduction in the modulus of elasticity. Also, freeze-thaw damage was found to alter the fracture behavior and damage development in concrete. Aggregate pull-out occurred when the dynamic modulus reduced to 75% of its original value (i.e., below the knee point). In addition, non-linearity of the stress-strain response increased with the freeze-thaw damage. Observations from acoustic emission indicated that the size of the damage zone increased with an increase in damage level (i.e., a decrease in the relative dynamic modulus). Isolated micro-cracks at paste or interface were observed in less damaged concrete ($E/E_0 > 0.8$). The amount and connectivity of cracks increased very fast with the decrease of E/E_0 from 0.8 to 0.7. When the relative dynamic modulus reached 0.7, well-connected

crack network was seen. As a result, cracking connectivity dramatically influenced the mechanical response and damage development in concrete.

7.3 Influence on Life-Cycle Modeling

This report has shown relationships between stress, strain, and the cumulative energy (measured using acoustic emission). Figure 7-1a shows that the relative strain level (i.e., the strain is relatively linear with the stress level at stresses less than approximately 80 to 90%). The strain begins to increase more rapidly at higher stress levels demonstrating the material non-linearity and rapidly increases after the stress level reaches approximately 80 to 90% of the peak load. This excessive non-linearity corresponds to the coalescence of microcracks into a damage zone. In addition, Figure 7-1a shows that the relationship between tensile stress and acoustic energy (i.e., the energy associated with damage) is even more non-linear. This corresponds to the development of distributed microcracking through concrete.

Figure 7-1a appears to suggest that if the stresses are kept below 50% of the peak strength the damage can be kept below 15 to 20% of the damage that is required to cause failure. In examining Figure 7-1b it appears that there is some threshold of damage that may exist (i.e., in the range of 15% of the energy to failure) and if the damage is kept below this level rapid degradation from freezing and thawing may not be expected.

It was also be noted that the ‘geometry’ of the damage that develops differs widely depending on the source of the damage. For example, mechanical loading in an

undamaged concrete results in a relatively narrow band of damage. This may be considered appropriately in life-cycle models by incorporating a local band of damage. Damaged caused by freezing and thawing (and it is hypothesized hygral and thermal shrinkage) is much more uniform throughout the specimen and consequently should be modeled as being distributed throughout the pavement.

It should also be noted that the use of models that consider only the initial material properties for predicting durability may provide an unrealistic expectation of the service life that can be expected. It is suggested that before the INDOT chooses to implement a durability related model that they consider how this model will account for the effect of damage that may occur over time.

Figure 7.1 (a) Summary of the Relationship between Stress, Strain, and Percentage of Cumulative Acoustic Emission Energy Level and (b) a Summary of the Relationship Between the Cumulative Acoustic Emission Energy Level and the Number of Cycles Required to Fail a Specimen in Freezing and Thawing Loading

Despite the recent attention to durability performance in cementitious material, little information exists on the influence of less than ideal curing conditions on the durability of concrete structures. For example, lack of sufficient curing can result in the development of microcracking a material. These results have shown field concrete has undergone drying and thermal cycling, may have developed microcracks. As such, it may be possible to relate insufficient curing to changes in the material that will reduce the long-term performance of concrete.

7.4 Suggestions for Implementation

There are several opportunities for implementing the results of this study. The following section illustrates some suggestions.

First, chapter 3 reported the results of field condition assessment of selected concrete pavements in the state of Indiana. This chapter can be reviewed by the pavement design committee to more fully appreciate how the concrete pavements constructed in Indiana are actually performing. The committee should take note of the correlation between PQI and the increase in sorptivity. It was observed that cracking in concrete pavements occurs and is due to a combination of mechanical loading and environmental effects. Microcracking was observed in pavements that are known to be performing poorly and these microcracks would be consistent with distributed damage occurring from freeze-thaw, thermal, or hygral shrinkage. This suggests in addition to mechanical loading additional sources of residual stress and environmental should be

considered in pavement design, especially in pavements designed with a longer service life.

Second, pavements with microcracking showed short-term sorptivity near the surface of the pavement that was substantially higher (up to 54%) than that from the middle or bottom of the pavement. This indicates that microcracking can occur at the surface of concrete pavements that results in more rapid water transport and absorption. It is suggested that as pavement design and analysis models (e.g. CONCLIFE) are implemented that use sorptivity to predict long-term performance a higher sorptivity value is used than that obtained from laboratory tests of pristine concrete to account for the presence of microcracking (or damage) with a change in pavement stiffness (elastic modulus). This increased sorptivity could be between 20 and 50% higher than the pristine condition to provide a conservative prediction that accounts for microcracking in life-cycle performance predictions.

Third, this work suggests that a correlation may be able to be established between results of the Pavement Quality Index (PQI) and the rate of water absorption. As the PQI indicated reduced quality in pavement the rate of water increased. Further, as relationship exists between sorptivity and change in elastic modulus is may be possible to utilize changes in concrete stiffness as detected using the Falling Weight Deflectometer (FWD) to indicate when absorption rates are rapidly increasing. This information could be able to be used to better estimate remaining service life of a concrete pavement section.

Fourth, the results of this study indicate that keeping the damage level in the concrete low is essential for providing long-term performance. It can be seen from Figure 7-1 that the cumulative acoustic emission (AE) energy level is highly non-linear with the applied level of stress. This indicates that the permanent damage (i.e., cracking as determined from acoustic emission energy) is highly non-linear with relation to stress. Therefore even though a visible crack may not occur, microcracking may occur in concrete during loading that could impact long-term performance. Figure 7-1b shows that the damage level (as introduced by loading) should be maintained below approximately 20% of that required for failure. This suggests that the stress level in use should be less than 60% (the minimum stress level tested in this study) and it can be estimated that the design stresses should be below the 50% strength to avoid the rapid increase in microcracking that occurs when the stress reaches approximately 50% of the tensile strength.

Fifth, electrical conductivity measurements demonstrate a substantial increase at the time the cracks become connected. This suggests that at some time in the future rapid field tests may be able to be developed to utilize electrical conductivity measurements to assess if the crack network is connected in a concrete pavement. This could indicate when a rapid increase in freeze-thaw damage may be imminent and when a repair procedure should be implemented.

7.5 Suggestions for Further Study

It is suggested that the INDOT consider conducting an additional study to assess the impact of improper curing on the potential for damage development in a concrete pavement. The long-term cost of improper curing (in terms of reduced service life) should be quantified. This could provide the INDOT with information that can be used to provide justification for incentives and disincentives related to the level of curing supplied by a contractor.

It is also suggested that the INDOT consider a follow up study to develop analytical procedures to quantify how the sorptivity of concrete pavements change over time. This study could consist of a long-term monitoring project for pavements representative of those constructed throughout the state of Indiana. As the analytical models are developed that could be implemented in life-cycle models like CONCLIFE to provide information for pavement design that could illustrate how residual stresses, fatigue, and overloading may be related to reduced service life.

REFERENCES

- ACI Committee 201, 1992, Guide to durable concrete, Report of ACI Committee 201, ACI 201.2R-92, Detroit
- ACI, 1968, Chapter 3 Freezing and Thawing, In Durability of Concrete Construction, American Concrete Institute Monograph, No.4, pp.15-35
- Aldea, C.-M., S.P. Shah, and A. Karr, 1999, Permeability of Cracked Concrete, Materials and Structures, Vol. 32, June, pp.370-376
- Aldea, C.-M., S. P. Shah, and A. Karr, 1999, Effect of Microcracking on Durability of High-Strength Concrete, Transportation Research Record, 1668, pp.86-90
- ASTM C 127, 2001, Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate
- ASTM C 192/C 192 M, 2002, Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory
- ASTM C 143/C-143 M, 2000, Standard Test Method for Slump of Hydraulic-Cement Concrete
- ASTM C215, 2002, Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Resonant Frequencies of Concrete Specimens
- ASTM C231, 1997, Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
- ASTM C457, 1998, Standard Test Method for Microscopic Determination of Parameters of the Air-Void System in Hardened Concrete
- ASTM C496, 1996, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens

- ASTM C597, 1997, Standard Test Method for Pulse Velocity through Concrete
- ASTM C 666/C 666M, 2003, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
- ASTM C1383, 1998, Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method
- ASTM E1876, 2002, Standard Test Method for Dynamic Young Modulus, Shear Modulus, and Poisson's Ratio by Impulse Excitation of Vibration
- ASTM STP 169B, 1978, Significance of Tests and Properties of Concrete and Concrete-Making Materials
- Bangert, F., S. Grasberger, D. Kuhl, and G. Meschke, 2003, Environmentally Induced Deterioration of Concrete: Physical Motivation and Numerical Modeling, Engineering Fracture Mechanics 70, pp. 891-910
- Banthia, N., and Mindess, S., 1989, Effect of early freezing on permeability of cement paste, Journal of Materials in Civil Engineering, 1, pp.119-132
- Banthia, N., Azzabi, M., and Pigeon, M., 1994, Restrained Shrinkage Test on Fiber Reinforced Cementitious Composites, ACI SP-155-7, Testing of Fiber Reinforced Concrete
- Bascoul, A., 1996, State of the art report – Part 2: Mechanical micro-cracking of concrete, Materials and Structures, Vol. 29, March, pp. 67-78
- Bayomy, F., 2000, Distresses for Pavements with Jointed Portland Cement Concrete Surfaces, Presentation for the Pavement Design Class, CE475, Fall 2000, University of Idaho

- Bažant, Z. P., and B. H. Oh, 1993, Crack Band Theory for Fracture of Concrete, *Matériaux et Constructions*, Vol. 16, pp.155-177
- Bear J., and J-M Buchlin (eds.), 1991, Modeling and Applications of Transport Phenomena in Porous Media, Kluwer Academic Publishers, pp.195-211
- Bentz, D.P., M. A. Ehlen, C. F. Ferrais, and E. J. Garboczi, 2001, Sorptivity-Based Service Life Predictions for Concrete Pavements, 7th International Conference on Concrete Pavements – Orlando, Florida, USA, Sept. 9-13
- Boddy, A., E. Bentz, M.D.A. Thomas, R.D. Hooton, 1999, an Overview and Sensitivity Study of a Multimechanistic Chloride Transport Model, *Cement and Concrete Research* 29, pp. 827-837
- Boulfiza, M., K. Sakai, N. Banthia, and H. Yoshida, 2003, Prediction of Chloride Ions Ingress in Uncracked and Cracked Concrete, *ACI Materials Journal*, Jan.-Feb., pp. 38-48
- Breysse, D., and B. Gérard, 1997, Modeling of Permeability in Cement-Based Materials: Part I – Uncracked Medium, *Cement and Concrete Research*, Vol. 27, No. 5, pp.761-775
- Cabrera, J.G., 1992, Effect of Superplasticizers on the plastic shrinkage of concrete, *Magazine of Concrete Research*, 44, No. 160, Sept., pp.146-155
- Carino, N. J., M. Sansalone, and N.N. Hsu, 1986, a Point Source-Point Receiver, Pulse-Echo Technique for Flaw Detection in Concrete, *ACI Materials Journal*, Vol. 83, No. 2, March-April, pp.199-208
- Christensen, B.J., R.T. Coverdale, R.A. Olson, S. J. Ford, E. J. Garboczi, H.M. Jennings, and T. O. Mason, 1994, Impedance Spectroscopy of Hydrating Cement-Based

- Materials: Measurement, Interpretation, and Application, Journal of American Ceramic Society, Vol. 77, No. 11, pp.2789-2801
- Churaev, N.V., 2000, Liquid and Vapor Flows in Porous Bodies, Surface phenomena, Gordon and Breach Science Publishers
- Graveen, C., 2001, Nondestructive Test Method to Assess Pavement Quality for Use in a Performance-Related Specification, MS Thesis, Purdue University, May
- Dullien, F.A.L., 1992, Porous Media: Fluid Transport and Pore Structures, Second Edition, Academic Press, Inc.
- Ehlen, M. A., 1999, Life-cycle Costing Software For Preliminary Bridge Design, BridgeLCC 1.0 Users Manual, National Institute of Standards and Technology, Gaithersburg, Maryland
- Fagerlund, G., 1997, Effect of Self-Desiccation on the Internal Frost Resistance of Concrete, Self-Desiccation and Its Importance on Concrete Technology, Ed. B. Persson, G. Fagerlund, Lund, Sweden, pp.227-238
- Fagerlund, G., 1997, On the Service Life of Concrete Exposed to Frost Action, Freeze and Thaw Durability of Concrete, J. Marchand, M. Pigeon, and M. Setzer (eds.), E & FN Spon, pp.24-41
- Fagerlund, G., 1977, The International Cooperative Test of the Critical Degree of Saturation Method of Assessing the Freeze/Thaw Resistance of Concrete, Materials and Structures, Vol. 10, No. 58, pp.231-253.
- Gawin, D., P. Baggio, and B.A. Schrefler, 1995, Coupled Heat, Water and Gas Flow in Deformable Porous Media, International Journal for Numerical Methods in Fluids, Vol. 20, pp.969-987

- Gérard, B., Breysse, D., Ammouche, A., Houdusse, O., and Didry, O., 1996, Cracking and Permeability of Concrete under Tension, *Materials and Structures*, Vol. 29, April, pp.141-151
- Gran, H.C., 1995, Fluorescent Liquid Replacement Technique, A Means of Crack Detection and Water : Binder Ratio Determination in High Strength Concrete, *Cement and Concrete Research*, Vol. 25, No. 5, pp.1063 – 1074
- Graveen, C., 2001, Nondestructive Test Methods to Assess Pavement Quality for Use in a Performance-Related Specifications, MS Thesis, Purdue University, May
- Gu, P., Z. Xu, P. Xie, and J. J. Beaudion, 1993, An A.C. Impedance Spectroscopy Study of Micro-cracking in Cement-Based Composites during Compressive Loading, *Cement and Concrete Research*, Vol. 23, pp. 675-682
- Guse, U., and Hisdorf, K., 1997, Surface Cracking of High-Strength Concrete – Reduction by Optimization of Curing, Self-Desiccation and Its Importance on Concrete Technology, Ed. Persson, B., Fagerlund, G., Lund, Sweden, pp.239-249.
- Hall, C., 1989, Water sorptivity of mortars and concretes: a review, *Magazine of concrete research*, 41, pp.51-61
- Hamstad, M. A., A. O’Gallagher, and J. Gary, 2002, A Wavelet Transform Applied to Acoustic Emission Signals: Part I: Source Identification, *Acoustic Emission*, V. 20, pp. 39-61
- Hamstad, M. A., A. O’Gallagher, and J. Gary, 2002, A Wavelet Transform Applied to Acoustic Emission Signals: Part II: Source Location, *Acoustic Emission*, V. 20, pp. 62-82

- Hauwaert, A.V., F. Delannay, and J. Thimus, 1999, Cracking Behavior of Steel Fiber Reinforced Concrete Revealed by Means of Acoustic Emission and Ultrasonic Wave Propagation, *ACI Materials Journal*, V.96, No. 3, May-June, pp.291-296.
- Hearn, H., 1999, Effect of Shrinkage and Load-Induced Cracking on Water Permeability of Concrete, *ACI Material Journal*, March-April, pp.234-240
- Hsu, T.T.C., Slate, F., Sturman, G., and Winter, G., 1963, Microcracking of Plain Concrete and the Shape of the stress-Strain Curve, *Journal of American Concrete Institute*, 60, pp.209-224.
- Hoerner, T. E., and M. I. Darter, 2000, Improved Prediction Models For PCC Pavement Performance-Related Specifications, Volume II: PaveSpec 3.0 User's Guide, FHWARD-00-131, Federal Highway Administration, Washington D. C., Sept.
- Hornain, H., Marchand, J., Ammouche, A., Commene, J.P., and Moranville, M., 1996, Microscopic Observation of Cracks in Concrete- a New Sample Preparation Technique Using Dye Impregnation, *Cement and Concrete Research*, Vol. 26, No. 4, pp.573-583
- Isgor, O. B., and A. G. Razaqpur, 2003, Finite Element Modeling of Coupled Heat Transfer, Moisture Transport and Carbonation Processes in Concrete Structures, *Cement and Concrete Composites*, 25, pp.1-17
- Jakobsen, U., H., 1998, Understanding the Features Observed in Concrete Using Various Fluorescence Impregnation Techniques, *Proceedings on the International Conference on Cement Microscopy*, Guadalajara, Mexico, April 19-23

- Jakobsen, U.H., Johansen, V., and Thaulow, N., 1997, Optical Microscopy – A Primary Tool in Concrete Examination, Proceedings of the Nineteenth International Conference on Cement Microscopy, April 27 – May 1, Cincinnati, Ohio, USA
- Jacobsen, S., J. Marchand, and L. Boisvert, 1996, Effect of Cracking and Healing on Chloride Transport in OPC concrete, Cement and Concrete Research, Vol. 26, No. 6, pp.869-881
- Jenq, Y., Shah, S.P., 1991, Features of Mechanics of Quasi-Brittle Crack Propagation in Concrete, International Journal of Fracture 51: 103-120
- Jennings, H., 1996, The Modeling of Microstructure and Its Potential for Studying Transport Properties and Durability, Kluwer Academic Publishers, pp.289-311
- Kay, T., 1992, Deterioration Processes, In Assessment & Renovation of Concrete Structure, Kong, F. K.(eds), John Wiley & Sons, pp.25-31
- Kim, B., and W. J. Weiss, 2003, Using Acoustic Emission to Quantify Damage in Restrained Fiber-Reinforced Cement Mortar, Cement and Concrete Research, 33, pp.207-214
- Kolluru, S.V., Popovics, J.S., and Shah, S.P., 2000, Determining Elastic Properties of Concrete Using Vibrational Resonant Frequencies of Standard Test Cylinders, Cement , Concrete, and Aggregates, Vol. 22, No. 2, Dec., pp.81-89
- Krauss, P. D., Rogalla, E.A., Sherman, M.R., McDonald, D.B., Osborn, A.E.N., and Pfeifer, D.W., 1995, NCHRP Report 380-Transverse Cracking in Newly Constructed Bridge Decks, pp.12-37.

- Landis, E.N., and Shah, S.P., 1995, the Influence of Micro-cracking on the Mechanical Behavior of Cement Based Materials, *Advanced Cement Based Materials*, 2:105-118
- Levine, I. N., 1988, *Physical Chemistry*, Third edition, McGraw-Hill, Inc., pp.357-364
- Li, F., and Li, Z., 2000, Acoustic Emission Monitoring of Fracture of Fiber-Reinforced Concrete in Tension, *ACI Materials Journal*, Vol.97, No.6, Nov.-Dec., pp.629-636
- Li, Z., Kulkarni, S.M., and Shah, S.P., 1993, New Test Method for Obtaining Softening Response of Unnotched Concrete Specimen under Uniaxial Tension, *Experimental Mechanics*, Vol. 33, No. 3, pp.181-188
- Li, Z., and Shah, S.P., 1994, Localization of Microcracking in Concrete under Uniaxial Tension, *ACI Materials Journal*, Jul. – Aug., pp. 372-381
- Lockington, D., Parlange, J.Y., and Dux, P., 1999, Sorptivity and the estimation of water penetration into unsaturated concrete, *Materials and Structures*, Vol. 32, June, pp.342-347
- Marchand, J., M. Pigeon, and M. Setzer, 1997, *Freeze-Thaw Durability of Concrete*, E & FN Spon, London, UK
- Martys, N. S., and C. F. Ferraris, 1997, Capillary Transport in Mortars and Concrete, *Cement and Concrete Research*, Vol. 27, No. 5, pp. 747-760
- Mehta, P.K. and Monteiro, P.J.M., 1993, *Concrete: Microstructure, Properties, and Materials*, 2nd Edition, Prentice Hall, Englewood Cliffs, NJ
- Mihashi, H., N. Normura, 1992, Micro-cracking and Tension-Softening Properties of Concrete, *Cement and Concrete Composites*, 14, pp.91-103

- Mitsuru, S., Ohta, M., and Ishimori, H., 1994, Chloride Permeability of Concrete Subjected to Freeze-Thaw Damage, *Cement and Concrete Composites*, 16, pp.233-239
- Monlouis-Bonnaire, J.P., J. Verdier, and B. Perrin, 2003, Prediction of the Relative Permeability to Gas Flow of Cement-based Materials, *Cement and Concrete Research* 2333, pp.1-8, in press
- Moukwa, M., 1989, Penetration of Chloride Ions from Seawater into Mortars under Different Exposure Conditions, *Cement and Concrete Research*, Vol. 19, pp.894-904
- Mu, R., C. Miao, X. Luo, W. Sun, 2002, Interaction between loading, freeze-thaw cycles, and chloride salt attack of concrete with and without steel fiber reinforcement, *Cement and Concrete Research*, V. 32, pp. 1061-1066
- Ohtsu, M., 1999, Estimation of Crack and Damage Progression in Concrete by Quantitative Acoustic Emission Analysis, *Material Evaluation*, May, p521-525
- Ohtsu, M., M. Shigeishi, and Y. Sakata, 1998, Nondestructive Evaluation of Defects in Concrete by Quantitative Acoustic Emission and Ultrasonics, *Ultrasonics*, 36, pp.187-195
- Pavement Management Unit, INDOT, 2000, 2000 Pavement Surface Report, December
- Penttala, V., 1998, Freezing-Induced Strains and Pressures in Wet Porous Materials and Especially in Concrete Mortars, *Advanced Cement Based Materials*, 7:8-19
- Pigeon, M., Frost Resistance, a Critical Look, *Concrete Technology, Past, Present, and Future*, Proceedings of V. Mohan Malhotra Symposium, American Concrete Institute, Detroit, SP-144

- Popovics, S., and Popovics, J., 1991, Effect of Stresses on the Ultrasonic Pulse Velocity in Concrete, *Materials and Structure*, 24, pp.15-23
- Popovics, S., Rose, J.L., and Popovics, J.S., 1990, the Behavior of Ultrasonic Pulse in Concrete, *Cement and Concrete Research*, Vol. 20, pp.259-270
- Poston, R.W., Whitlock, A. R., and Kesner, K.E., 1995, Condition Assessment Using Nondestructive Evaluation, *Concrete International*, July, pp.36-42
- Powers, T.C., 1975, Freezing Effects in Concrete, *Durability of Concrete*, ACI Special Publication SP-47, pp.1-11
- Puri, S., and J. Weiss, 2005, Assessing Damage Localization in Concrete Cylinders Tested in Compression, to appear in the *ASCE Journal of Civil Engineering Materials*
- Qi, C., 2003, Quantitative Assessment of Plastic Shrinkage Cracking and Its Impact on the Corrosion of Steel Reinforcement, Ph.D. Dissertation, Purdue University, West Lafayette, IN, May
- Rajabipour, F, J. Weiss, and D. M. Abraham, 2004, Insitu Electrical Conductivity Measurements to Assess Moisture and Ionic Transport in Concrete, *Proceedings of the International RILEM Symposium (CD-ROM)*, March 21-26, Northwestern University, Evanston, IL, USA
- Samaha, H. R. and K. Hover, 1992, Influence of Microcracking on the Mass Transport Properties of Concrete, *ACI Materials Journal*, July-August, pp.416-424
- Sahu, S., S. Badger, N. Thaulow, and R.J. Lee, 2004, Determination of Water-Cement Ratio of Hardened Concrete by Scanning Electron Microscopy, *Cement and Concrete Composites*, in press

- Sawan, J., 1987, Cracking Due to Frost Action in Portland Cement Concrete Pavements- A literature Survey, Concrete Durability, Katharine and Bryant Mather International Conference, ACI Sp-100 Vol.1, pp.781-802
- Schwartz, Donald R., 1987, D-Cracking of concrete pavements (NCHRP Synthesis No.134), Washington DC, Transportation Research Board
- Selih, J., A. C.M. Sousa, and T. W. Bremner, 1996, Moisture Transport in Initially Fully Saturated Concrete During Drying, Transport in Porous Media 24, pp.81-106
- Setzer, M.J., 1991, Interaction of Water with Hardened Cement Paste, In Advances in Cementitious Materials, Ceramic Transactions Volume 16, Edited by Sidney Mindess, The American Ceramic Society, Inc., pp. 415-439
- Shah, S. P., 1997, An Overview of the Fracture Mechanics of Concrete, Cement Concrete and Aggregates, CCAGDP, Vol. 19, No. 2, pp. 79-86
- Shah, S. P., and Sankar, R., 1987, Internal Cracking and Strain Softening Response of Concrete under Uniaxial Compression, Journal of American Concrete Institute, pp.200-212
- Shah, S.P., Weiss, W.J. and Yang, W., 1998, Shrinkage Cracking – Can It be Prevented, Concrete International, April, pp 51-55
- Shum, K.M., and Hutchinson, J.W., 1990, On Toughening by Micro-Cracks, Mechanics of Materials, Vol. 9, pp.83-91
- Snyder, K.A., C.F. Ferraris, N.S. Martys, and E.J. Garboczi, 2000, Using Impedance Spectroscopy to Assess the Viability of the Rapid Chloride Test for Determining Concrete Conductivity, Journal of Research of the National Institute of Standards and Technology, July-August, Vol. 105, No. 4, pp.497-509

- Soroushian, P., and M. Elzafraney, 2003, Damage effects on concrete performance and microstructure, Cement and Concrete Composites, In press
- Struble, L., and Stutzman, P., 1989, Epoxy Impregnation of Hardened Cement for Microstructural Characterization, Journal of Materials Science Letters, 8, pp.632-634
- Suaris, W., and Fernando, V., 1987, Ultrasonic Pulse Attenuation as a Measure of Damage Growth during Cyclic Loading of Concrete, ACI Materials Journal, May-June, pp.185-193
- Sun, W., Y. M. Zhang, H. D. Yan, and R. Mu, 1999, Damage and damage resistance of high strength concrete under the action of load and freeze-thaw cycles, Cement and Concrete Research, V. 29, pp. 1519-1523
- Tawfiq, K., J. Armaghani, and J. R. Vysyaraju, 1996, Permeability of Concrete Subjected to Cyclic Loading, Transportation Research Board 1532, pp.51-59
- Tenchev, R.T., L.Y. Li, and J.A. Purkiss, 2001, Finite Element Analysis of Coupled Heat and Moisture Transfer in Concrete Subjected to Fire, Numerical Heat Transfer, Part A, Vol. 39, pp. 685-710
- Thomas, M. D. A., and C. E. Bentz, 2001, LIFE-365TM Service Life Prediction Model, users Manual, Silica Fume Association
- Tinic, C. and E. Brühwiler, 1985, the Effect of Compressive Loads on the Tensile Strength of Concrete at High Strain Rates, International Journal of Cement Composites and Light Weight Concrete, 7(2), pp.103-108
- Toumi, A., Bascoul, A., and Turatsinze, A., 1998, Crack Propagation in Concrete Subjected to Flexural Cyclic Loading, Materials and Structures, Vol. 31, August-September, pp.451-458

- Ueda, M., Hasebe, N., Sato, M., and Okuda, H., 1994, Fracture Mechanism of Plain Concrete Under Uniaxial Tension, Concrete Library of JSCE, No. 24, Dec., pp.31-45
- U.S. Bureau of Reclamation, 1992, Procedures for Direct Tensile Strength, Static Modulus of Elasticity, and Poisson's Ratio of Cylindrical Concrete Specimens in Tension (USBR 4914-92), Concrete Manual, part 2, 9th edition, U.U Bureau of Reclamation, Denver, p726-731
- Van Dam, T. J., N. J. Buch, K. F. Hanson, J. E. Hiller, L. L. Sutter, R. Muethel, 2001, Michigan's Approach to a State-Wide Investigation of Material-Related Distress (MRD) in Concrete Pavements, Transportation Research Record 1775, pp.1-9
- Walker, H., N., 1997, Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual, Publication No. FHWA-RD-97-146, Nov.
- Wang, K., D.C. Jansen, and S. P. Shah, 1997, Permeability Study of Cracked Concrete, Cement and Concrete Research, Vol. 27, No.3, pp.381-393
- Weber JR, W .J., Digiano, F.A., 1996, Process Dynamics in Environmental Systems, John Wiley & Sons, Inc.
- Weiss, W. J. and J. Olek, 1998, Interaction between Micro-Cracking, Cracking and Reduced Durability in Concrete: Developing Methods for Considering Cumulative Damage in Life-Cycle Modeling, Research Proposal SPR-2474, Sponsor: FHWA/JTRP/INDOT
- Weiss, W. J., 2003, Linking Insitu Monitoring with Damage Modeling for Life-Cycle Performance Simulations in the Concrete Infrastructure, Career Proposal, Sponsor: National Science Foundation

- Whittington, H.W., J. McCarter, and M.C. Forde, 1981, The Conduction of Electricity through Concrete, Magazine of Concrete Research, Vol. 33, No. 114, pp.48-60
- Wiens, U., Meng, B., Schroeder, P., and Schiessl, P., 1997, Micro-cracking in High-Performance Concrete- From Model to the Effect on Concrete Properties, Self-Desiccation and Its Importance on Concrete Technology, Ed. B. Persson, G. Fagerlund, Lund, Sweden, pp193-208.
- Wilson, M.A, Carter, M.A., and Hoff, W.D., 1999, British Standard and RILEM Absorption Tests: a Critical Evaluation, Materials and Structures, Vol. 32, October, pp.571-578
- Yamato, T., Emoto, Y., Soeda, M., 1987, Freezing and Thawing Resistance of Concrete Containing Chloride, Concrete Durability, Katharine and Bryant Mather International Conference, ACI SP-100, pp.901-917
- Yong, R.N., Mohamed, A.M.O., and Warkentin, B.P., 1992, Principle of Contaminant Transport in Soils, Elsevier Science Publishers B.V.
- Yoon, D.J, Weiss, W.J., and Shah, S.P., 1999, Detecting the Extent of Corrosion with Acoustic Emission, Transportation Research Board 1698, pp.54-60
- Yoon, S., K. Wang, W. J. Weiss, and S. Shah, 2000, Interaction between Loading, Corrosion, and Serviceability of Reinforced Concrete, ACI Materials Journal, November-December, pp.637-644
- Yuyama, S., Okamoto, T., Shigeishi, M., Ohtsu, M., Kishi, T., 1999, A Proposed Standard for Evaluating Structural Integrity of Reinforced Concrete Beams by Acoustic Emission, Acoustic Emission: Standard and Technology Update, Ed., S. Vahaviolos, pp25-40

- Zheng, W., Kwan, A.K.H., and Lee, P.K.K., 2001, Direct Tension Test of Concrete, ACI Materials Journal, V. 98, No. 1, Jan.-Feb
- Zhou, Y., M. D. Cohen, and W. L. Dolch., 1994, Effect of External Loads on the Frost-Resistant Properties of Mortar with and without Silica Fume, ACI Materials Journal, Nov. – Dec., pp. 595-601
- Zhou, Y., 1994, Frost Resistance of Concrete with and without Silica Fume and the Effects of External Loading. Ph. D. Dissertation, Purdue University, May
- Zuber, B., and J. Marchand, 2000, Modeling the Deterioration of Hydrated Cement Systems Exposed to Frost Action Part I: Description of the Mathematical Model, Cement and Concrete Research 30, pp.1929-1939

APPENDIX I - PAVEMENT QUALITY INDEX (PQI)

PQI is a composite index to indicate the surface quality of pavement with a 100 point scale: excellent (100-90); good (90-80); fair (80-70); poor (<70). It is determined by the following formula:

$$PQI = 10*PSI + 0.5*PCR - 25 \text{ Rut.}$$

Where,

$PSI = 9.0 * e^{0.08747 * IRI}$ with an upper limit of 5 and IRI is international roughness index which measures the ride (bumpiness) of a pavement in terms of inches per mile with good pavements from 100 to 150, fair pavements from 150 to 200, and poor pavements above 200.

PCR is pavement condition rating which measures the distress on a pavement surface. These distresses include transverse cracking, longitudinal cracking, block cracking, etc. The pavement is reviewed at each reference post for 500 inches and the distresses are rated for severity and quality. A value is determined from 100 to 0 with excellent pavements from 100 to 90, good pavements from 90 to 80, fair pavements from 80 to 70, and poor pavements below 70.

Rut measures the average depth of ruts in the wheel path of a pavement. Rutting commonly occur on asphalt pavements, in which more than 0.25' ruts are rated severe. Rutting rarely occurs on a concrete pavement unless it is very old (>25 years).

APPENDIX II - DETERMINATION OF DEGREE OF SATURATION

Symbols

f_i - volume fraction of pores and voids in slice i

$\bar{f}$ - average volume fraction of pores and voids in a specimen

S - degree of saturation

V_i - total volume of slice i

V_T - total volume of a test specimen

V_v - total volume of pores and voids in a specimen

V_w - volume of pores and voids in a specimen that is filled with water

W – weight of a specimen saturated to a required degree of saturation

W_{di} - weight of slice i dried at 105°C

W_{d50} - weight of a specimen dried at 50°C before vacuum saturation

W_{d50}^i - weight of specimen i dried at 50°C

W_{d105}^i - weight of specimen i completely dried at 105°C

W_{si} - weight of saturated slice i in air

$W_{si,water}$ - weight of saturated slice i suspended in water

W_s - weight of a saturated specimen in air

$W_{s,water}$ -weight of a saturated specimen suspended in water

ρ_{water} - density of water under 1 atm and at 23°C

ΔW_i - free water content of specimen i dried at 50°C

ΔW - average free water content of specimens dried at 50°C

The degree of saturation was calculated using equation A2-1 showing below:

$$S = \frac{V_w}{V_v} \quad A2-1$$

1. Determining Total Volume of Pores and Voids in a Specimen (V_v)

V_v was determined using a control specimen (76×102×203 mm) using the following procedure. First, the control specimen was cut to 4 slices (76×102×51 mm) and each slice was then dried at 105°C for at least 3 days. After cooling in desiccators for 12 hours, all slices were vacuum-saturated with a residual pressure of 130-270 Pa. f_i was calculated using equation A2-2:

$$f_i = \frac{W_{si} - W_{di}}{\rho_{water} V_i} \quad A2-2$$

V_i was calculated using equation A2-3:

$$V_i = \frac{W_{si} - W_{si,water}}{\rho_{water}} \quad A2-3$$

$W_{si,water}$ was measured using a balance similar to that described in ASTM C127 [2001].

The balance was equipped with a wire basket and a water tank by which the specimen can be weighed while suspended in water.

The total volume of pore and voids (V_v) in a test specimen can be calculated as:

$$V_v = f * V_T \quad A2-4$$

$$f = \frac{1}{4} \sum_{i=1}^4 f_i \quad A2-5$$

V_T was calculated using equation A2-6:

$$V_T = \frac{W_s - W_{s,water}}{\rho_{water}} \quad A2-6$$

2. Determining Volume of Water-Filled Pores and Voids in a Specimen (V_w)

V_w was calculated based on equation A2-7:

$$V_w = \frac{W - W_{d50} + \Delta W}{\rho_{water}} \quad A2-7$$

The value of ΔW was determined using four control specimens (76×102×203 mm) that were made under the same conditions (materials, proportions, and curing) as test specimens.

$$\Delta W = \frac{1}{4} \sum_{i=1}^4 \Delta W_i \quad A2-8$$

$$\Delta W_i = W_{d50}^i - W_{d105}^i \quad A2-9$$

Figure A2.1 shows in detail the procedures of determining the degree of saturation of test specimens.

Figure A2.1 Schematic Representation of Procedures to Calculate Degree of Saturation

APPENDIX III - TEST DATA AND ANALYSIS (CHAPTER 5)

Assessing Damage Development Caused by Uniaxial Tensile Loading and Quantifying
Its Influence on Freeze and Thaw Durability of Concrete

Specimen ID: 06-27-16

W/C: 0.42
 Age: 80 days
 Fresh Air: 6.6%(Vol.)
 Hardened Air: 6.6% (Vol.)
 Peak Load: 28608 N (6436 LBs)
 Dimension: 76.2mmx101.6mmx406 mm
 (3"x4"x16")

APPENDIX IV - TEST DATA AND ANALYSIS (CHAPTER 6)

Influence of Freeze and Thaw Damage on Mechanical Response and Damage
Development of Concrete under Tensile Loading

Specimen ID: Crit-06-26-1-1
 E/E_0 : 0.899

Specimen ID: Crit-06-26-1-2
 E/E_0 : 0.942

Specimen ID: Crit-06-26-2-2
 E/E_0 : 0.854

Specimen ID: Crit-06-26-3-1
 E/E_0 : 0.706

Specimen ID: Crit-06-26-3-2
 E/E_0 : 0.789

Specimen ID: Crit-06-26-4-1
 E/E_0 : 0.262

Specimen ID: Crit-06-26-4-2
 E/E_0 : 0.735

SPecimen ID: Crit 06-26-5-2

E/E_0 : 0.61

Crit-06-26-6-2
 $E/E_0: 0.122$

Specimen ID: Crit-08-05-1-1
E/E₀: 0.452

Specimen ID: Crit-08-05-1-2
 E/E_0 : 0.635

Specimen ID: Crit-08-05-2-2
 E/E_0 : 0.475

Specimen ID: Crit-08-05-3-1
 E/E_0 : 0.1

Specimen ID: Crit-08-05-3-2
 E/E_0 : 0.81

Specimen ID: Crit-08-05-4-1
 E/E_0 : 0.76

Specimen ID: Crit-08-05-4-2
 E/E_0 : 0.52

Specimen ID: Crit-08-05-5-1
 E/E_0 : 0.886

Specimen ID: Crit-08-05-6-1
 E/E_0 : 0.294

Specimen ID: Crit-08-05-6-2
 E/E_0 : 0.957

Specimen ID: Crit-08-07-1-1
 E/E_0 : 0.03

Specimen ID: Crit-08-07-1-2
 E/E_0 : 0.33

Specimen ID: Crit-08-07-2-1
 E/E_0 : 0.71

Specimen ID: Crit-08-07-2-2

E/E_0 : 0.71

Tensile Strength: 0.885 MPa (128.3psi)

Specimen ID: Crit-08-07-3-1

E/E₀: 1.0

Tensile Strength: 3.26 MPa (473.3 psi)

Specimen ID: Crit-08-07-3-2

E/E_0 : 0.6

Tensile Strength: 0.8 MPa (116 psi)

Specimen ID: Crit-08-07-4-2

E/E_0 : 0.87

Tensile Strength: 2.0 MPa (295 psi)

Specimen ID: Crit-08-07-5-1

E/E_0 : 0.95

Tensile Strength: 2.1 MPa (303.9 psi)

Specimen ID: Crit-08-07-5-2

E/E_0 : 0.74

Strength: 1.03 MPa (149.4 psi)

Specimen ID: Crit-08-07-6-1

E/E_0 : 0.96

Tensile Strength: 3.89 MPa (564 psi)

Specimen ID: Crit-08-07-6-2

E/E_0 : 0.79

Tensile Strength: 1.33 MPa (192.8 psi)