An Examination of Concrete Durability

By

Jahangir M. Abdoveis

Bachelor of Science in Civil Engineering
University of Oklahoma

Submitted to the Department of Civil and Environmental Engineering in Partial
Fulfillment of the Requirements for the Degree of

Masters of Engineering in Civil and Environmental Engineering

At the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

June, 2003

Signature of Author_____

Jahangir M. Abdoveis
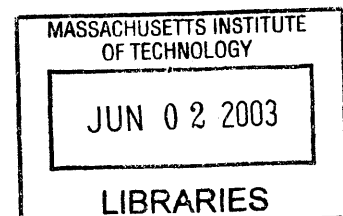Department of Civil and Environmental Engineeing
June, 2003

Certified By_____

Dr. Jerome Connor
Professor, Civil and Environmental Engineering
Thesis Supervisor

Accepted By_____

Dr. Oral Buyukozturk
Professor, Civil and Environmental Engineering
Chairman, Committee for Graduate Studies

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ABSTRACT

Recent trends in concrete durability design have favored the use of protective coatings. Although these coatings, if applied correctly, can totally inhibit degradation of the concrete member, these coatings are expensive. In the most severe conditions, the coatings are the only way to avoid extensive corrosion. In many cases, however, the coatings are used when less expensive means of avoiding concrete corrosion are available. If the type of degradation agents to which the concrete is to be exposed during its service life can be accurately predicted, the durability design requires only minor, inexpensive changes to the concrete mix proportions, the mix ingredients, or the structural detailing. This document provides a comprehensive guide to various types of concrete degradation and the mechanics involved with each type of degradation. For each of the degradation mechanisms discussed, several methods of designing concrete structural members, using only minor alterations in the concrete member, to resist degradation are provided in this document.

Thesis Supervisor: Dr. Jerome J. Connor

Title: Professor, Department of Civil and Environmental Engineering

Acknowledgements

I would like to thank Professor Connor, Lisa Grebner, and Paul Kassabian for their academic guidance.

I would like to thank Chris Lubke, Julie Smith, Zak Kostura, Chad Stevens, Diane Floresca, and Devin Schaffer for their friendship and support throughout the year.

Lastly, I would like to thank my family, Hamid, Jane, Elaine and Michael, along with Jill Walker for their unending love, support, and guidance.

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Chapter One: Introduction to Durability Design

Recent trends in concrete durability design have favored the use of protective coatings. Although these coatings, if applied correctly, can totally inhibit degradation of the concrete member, these coatings are expensive. In the most severe conditions, the coatings are the only way to avoid extensive corrosion. In many cases, however, the coatings are used when less expensive means of avoiding concrete corrosion are available. If the type of degradation agents to which the concrete is to be exposed during its service life can be accurately predicted, many times the durability design requires only minor, inexpensive changes to the concrete mix proportions, the mix ingredients, or the structural detailing. The first part of this document, Chapters 2-4, provides a comprehensive guide to various types of concrete degradation and the mechanics involved with each type of degradation. For each of the degradation mechanisms discussed, several methods of designing concrete structural members, using only minor alterations in the concrete member, to resist degradation are provided in the second part of this document, Chapters 6-8.

1.1　Durability Definition

Durability of concrete is the ability of a concrete to resist deterioration, particularly deterioration due to weather exposure, chemical exposure, or surface abrasion. According to Derucher, Korfiatis and Ezeldin, a durable concrete must maintain its form, quality, and serviceability for its prescribed service life.

Concrete durability is an issue of particular interest in concrete bridge structures and building exteriors, because of exposure to weather conditions, particularly wetting/drying cycles and freeze/thaw cycles, and exposure to harsh chemical environments, particularly the exposure to carbon compounds, acids, and de-icing salts.

Employing the model proposed by Paeglitis, the deterioration process can be divided into two phases, initiation and propagation. Initiation is the deterioration of the concrete on a micro-elemental level. Propagation is the process where many micro-elemental deteriorations combine to compromise the overall structural, functional, or aesthetic purpose of the concrete. The difference between the two phases is illustrated by the following example. When examining alkali-silica reaction, discussed in further detail in Chapter 2, initiation is the formation of micro cracks around an aggregate, but these micro cracks are not severe enough to

affect the overall structural, functional, or aesthetic purpose of the concrete. Propagation is the process of these micro cracks connecting to each other and forming larger cracks that are severe enough to affect the overall performance of the concrete. The propagation phase continues until the concrete reaches the durability failure point, at which, the concrete is unable to fulfill its purpose.

The durability failure point, the end of the propagation phase, is determined by the function of the structure. Various types of structures have different durability failure points. For example, in a structure where aesthetics are a primary concern, the durability failure point may correspond to the formation of small surface cracks on the concrete. In contrast, in a structure such as an underground concrete foundation pile, the durability failure point corresponds to when the load-bearing capacity falls below a critical value.

The rates of initiation and propagation depend on the definition of the durability failure point, but commonly, the propagation phase proceeds at a much faster rate than the initiation phase (Paeglitis, 2001). Figure 1 presents a graph of concrete deterioration versus time, with the initiation and propagation phases shown. The ideal design situation would be that the initiation phase is longer than the service life of the structure.

Unfortunately, this ideal situation is not economical in many modern structures, and thus the propagation phase is critical to durability design.

Figure 1. Plot of Damage Vs. Time

The plot shows Damage on the vertical axis and Time on the horizontal axis. A line rises from the origin, passing through the Initiation phase and then the Propagation phase, crossing the Failure Point at the end of the Service Life span.

- Service Life (horizontal span near top)
- Failure Point (dashed horizontal line)
- Initiation (first horizontal segment along Time axis)
- Propagation (second horizontal segment along Time axis)

1.2 Durability Design for Concrete

Concrete durability design, for all types of deterioration, is based on two fundamental concepts: prolonging the initiation phase and reducing the rate at which the propagation phase occurs. Prolonging the initiation phase is reducing the rate at which micro-elemental deterioration occurs. By prolonging the initiation phase, the level of damage required for

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propagation to begin is attained at a later time, thus increasing the service life of the structure. By reducing the propagation rate, the level of damage that corresponds to the failure point is attained at a later time, again increasing the service life of the structure.

This document focuses on the mechanisms that improve the durability of concrete structures, and these mechanisms are categorized as either prolonging the initiation phase or reducing the propagation rate. Durability design techniques for concrete employ one or both of these mechanisms by altering the chemical reactivity of the concrete, altering the physical composition of the concrete structural members, or in some cases, by altering the chemical reactivity and the physical composition.

Overall, this document examines the different types of concrete deterioration in context of initiation and propagation, then secondly, provides various methods to design concrete structural members for durability in the context of altering the chemical reactivity or the physical composition of the concrete members.

Chapter Two: Types of Chemical Degradation

2.1 Introduction to Chemical Degradation

The durability, or degradation resistance, of a particular concrete depends on many factors, and these factors can be grouped into two main categories: the physical characteristics of the concrete and the chemical composition of the concrete. Following this logic, the causes of durability degradation can be grouped into two categories: physical degradation and chemical degradation (MacGregor,1997). Because chemical degradation often has physical results, the two types of durability degradation are often confused. The fundamental difference is that physical degradation involves no chemical reaction, and chemical degradation is the physical result of harmful chemical processes within the concrete. Physical degradation depends on the concrete's exposure to freeze/thaw cycles and to abrasive or erosive elements. Chemical degradation depends on the chemical composition of the concrete in regards to alkali-silica reaction, alkali-carbonate reaction, sulfate reactivity, acid reactivity, carbonate reactivity, and corrosion of the reinforcing steel.

This chapter examines the various types of chemical degradations resulting from reactions between different components of the concrete

and reactions between the concrete and corrosive environments. Chapter 3 examines the chemical degradation of the steel embedded in concrete structural members, and Chapter 4 examines degradation of concrete due to physical degradation. To begin the discussion of the various types of chemical degradation, the basic composition of concrete must be understood.

2.2 Composition of Concrete

Typical concrete consists of five materials: coarse aggregate, fine aggregate, cement, water, and air. Coarse aggregate, commonly referred to as aggregate, is the larger stones, typically with a diameter ranging from 0.3 cm to 1.5 cm. Coarse aggregates typically account for 65 to 85% of concrete by volume (Derucher, Korfiatis, and Ezeldin, 1998). The type of coarse aggregate in concrete varies with geographic location, with most concrete producers using suitable local aggregates. The main function of coarse aggregate in concrete is reducing the overall cost of the concrete. Because coarse aggregates are both strong and relatively cheap compared to cement, coarse aggregates are used as a fill material in concrete to occupy space while not compromising the concrete's strength. Fine aggregates, typically sand, serve a similar purpose as coarse aggregates, to occupy space while not compromising strength. The strength, surface textures, water affinity, and chemical composition

of both the fine aggregate and the coarse aggregate greatly affect the concrete's durability.

The cement, typically Portland cement, is the binder for the concrete. When hardened, the cement provides cohesion between the components of concrete, and thus, provides the overall strength of the concrete. The cement is typically composed of lime, silica, iron oxide, and alumina (MacGregor,1997). Cements are classified as either hydraulic, meaning that they require water to harder, or non-hydraulic, meaning that they only require air exposure to harden. Portland cement, the most commonly used cement in concrete, is a hydraulic cement. The process of Portland cement hardening when exposed to water is called hydration, and the hydration process and its chemical products play an integral role in the durability of the concrete.

The water, as mentioned above, drives the hydration reaction that causes hydraulic cements to harden. The main factor that affects the overall concrete strength is the water to cement ratio. As the water–cement ratio decreases, the strength of the concrete increases. The water to cement ratio is limited by workability, because the workability decreases as the water–cement ratio decreases. In recent years, admixtures have been developed to allow very low water to cement ratios while maintaining acceptable workability, resulting in the ability to produce very high

strength concrete for construction use. The amount of water in concrete also has a large effect on the durability, particularly concerning freeze/thaw cycles.

The air in concrete is a product of the hydration reaction and other reactions between various ions in concrete with organic compounds (Swenson, 1969). Too much air results in a loss of strength of the concrete, but manipulating the amount and distribution of the air in concrete can greatly increase the durability.

Once the concrete is hardened, the concrete composition consists of two main components, the coarse aggregate and the cement paste. The cement paste is the mixture of sand, Portland cement, water, and air that surrounds the coarse aggregate particles. The cement paste, during hardening, bonds with the coarse aggregate, and the location of this bond is commonly known as the transition zone (Derucher, Korfiatis, and Ezeldin, 1998). The transition zone is critical for concrete strength and durability. A note must be made that after hardening, the coarse aggregate particles contain water and air in their voids, and this water and air in the voids of the coarse aggregate, along with the water and air in the cement paste, is a critical aspect of the concrete's durability.

The above presented information is critical in understanding the mechanics of concrete degradation, both due to chemical degradation and physical degradation.

2.3 Chemical Degradation Introduction

Chemical degradation is induced by chemical reactions between the cement paste and harmful chemicals from the environment, chemical reactions between the cement paste and reactive materials in the aggregate, and chemical reactions between the steel embedded in concrete and the chemical environment of the steel. The reactions critical to durability between the cement paste and harmful chemicals from the environment are sulfate reactions, acid neutralization reactions, and carbonation reactions. The reactions critical to durability between the aggregate and the cement paste are the alkali-silica reaction and the alkali-carbonate reaction. The critical reaction between the embedded steel and the steel's chemical environment is the cathodic corrosion of the steel. An examination of the chemical processes and physical consequences of each of these reactions is presented in the remainder of this chapter.

2.4 Aggregate-Paste Reactions

The trend to reduce the price of concrete has led to the use of low quality aggregates and cements with high alkali contents. First, the lower quality aggregates are often more reactive, containing high amounts of carbonate and silica typically found in types of sedimentary, volcanic, and metamorphic rocks (Derucher, Korfiatis, and Ezeldin, 1998). Secondly, less costly methods of cement production often result in cements with higher alkali content. The two main detrimental reactions between alkalis in the cement paste and aggregates are the alkali-silica reaction and the alkali-carbonate reaction. See Figure 2 Below for an example of concrete damage due to an aggregate-paste reaction.
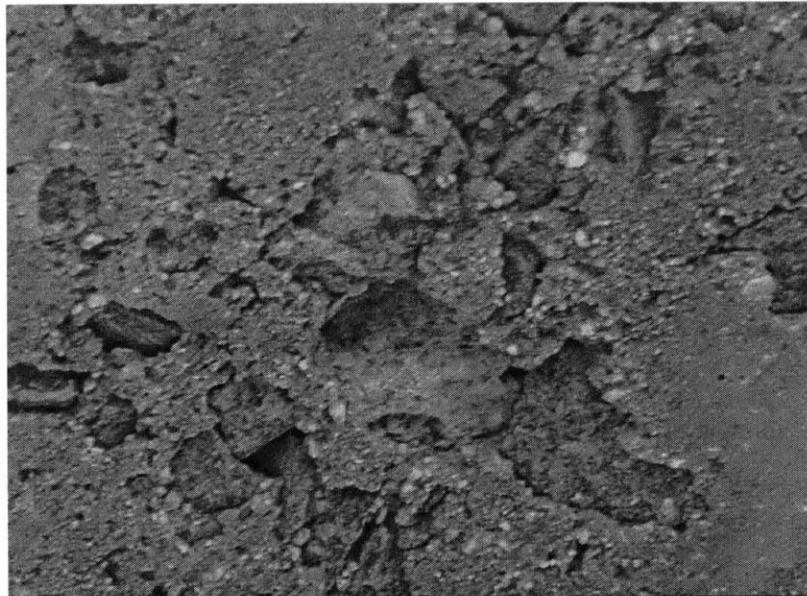
Figure 2. Concrete Damage Due to Aggregate-Paste Reactions

2.4.1 Alkali-Silica Reaction

The alkali-silica reaction is the most detrimental and the most important of the alkali-aggregate reactions. When aggregates with high silica contents, such as volcanic, sedimentary, and metamorphic rocks, are used in concrete, these silica particles react with the alkalis in the cement paste. This reaction is detrimental to the concrete in two ways: this reaction results in a volumetric expansion by creating a gel layer around the aggregate particles, and this gel layer attracts water and thus induces fractures due to osmotic pressures.

According to Derucher, Korfiatis, and Ezeldin, the reaction proceeds as follows. The reaction begins during the mixing of the concrete. Alkalis, mainly sodium oxide and potassium oxide, from the cement dissolve in the water. As hydration occurs, silicates in the cement extract water from the solution so that the silicates can harden. This extraction results in a decrease in the volume of water, resulting in an increase in the concentration of the alkalis in the water. If the alkali content of the cement is high before mixing, the alkali concentration in the water will be high enough that the alkalis begin to react with silica in the aggregates. This reaction results in the formation of a silica gel around the aggregate. This silica gel formation requires a net increase in volume, thus creating pressures around the silica gel layer. These pressures are sometimes

large enough to form micro cracks around the aggregate. The silica gel layer also has a very high affinity for water, and the gel layer draws water from the surrounding cement paste. This process results in large osmotic pressures that often cause micro cracking around the aggregate.

The alkali-silica reaction proceeds at a faster rate as the temperature increases, but at very high temperatures, the reaction rate begins to decrease insignificantly. The alkali-silica reaction can be easily detected by petrography analysis of the concrete. The silica gel layer can be detected by visual examination of the petrography sample.

In summary, the alkali-silica reaction is detrimental to concrete in two ways: creating micro cracks due to the pressures induced by volumetric expansion and creating micro cracks due to osmotic pressures induced by the water affinity of the gel layer. The creation of these micro cracks constitutes the initiation phase of deterioration, and the connection of these micro cracks with each other resulting in larger cracks constitutes the propagation phase. When the larger cracks become large enough so that they compromise the ability of the concrete to fulfill its purpose, structural and/or aesthetic, the concrete reaches its failure point.

2.4.2 Alkali-Carbonate Reaction

The alkali-carbonate reaction occurs between aggregates with high carbonate contents, such as dolomitic limestone aggregates, and the alkalis in the cement paste. This reaction is expansive, meaning that the volume of the products is larger than the volume of the reactants, and thus, this reaction causes micro cracks due to volumetric expansion.

According to Derucher, Korfiatis, and Ezeldin, the reaction proceeds as follows. The reaction begins in the same manner as the alkali-silica reaction. As a result of the mixing process, the concentration of alkalis in the water-alkali solution increases. If this concentration is large enough as a result of a high initial alkali content of the cement, the alkalis begin to react with the carbonate in the aggregate. This reaction proceeds according to one of the following equations, depending on the type of alkali present.

$$CaMg(CaO_3)_2 + 2KOH \rightarrow Mg(OH)_2 + CaCO_3 + K_2CO_3 \quad \text{(eq. 2.1)}$$

$$CaMg(CaO_3)_2 + 2NaOH \rightarrow Mg(OH)_2 + CaCO_3 + Na_2CO_3 \quad \text{(eq. 2.2)}$$

$$CaMg(CaO_3)_2 + 2LiOH \rightarrow Mg(OH)_2 + CaCO_3 + Li_2CO_3 \quad \text{(eq. 2.3)}$$
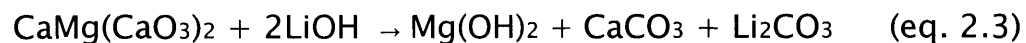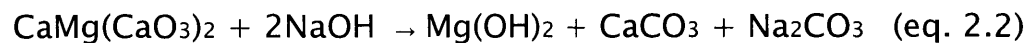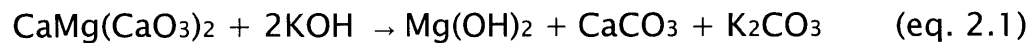
In addition to these reactions, the alkali carbonate product of the above reactions reacts with the calcium hydroxide products of Portland cement

hydration to produce an alkali according to the following equations, depending on the alkali present.

$$K_2CO_3 + Ca(OH)_2 \rightarrow 2KOH + CaCO_3 \quad \text{(eq. 2.4)}$$

$$Na_2CO_3 + Ca(OH)_2 \rightarrow 2NaOH + CaCO_3 \quad \text{(eq. 2.5)}$$

$$Li_2CO_3 + Ca(OH)_2 \rightarrow 2LiOH + CaCO_3 \quad \text{(eq. 2.6)}$$

This newly produced alkali then reacts with the carbonate again according to eq. 2.1, 2.2, or 2.3. This cycle continues until the alkali or the carbonate supply is depleted. With each cycle, a volumetric increase is incurred, and this volumetric increase causes micro cracks.

In summary, the alkali carbonate reaction is detrimental to concrete through the forming of micro cracks due to the volumetric expansion caused by the reaction. The formation of the micro cracks constitutes the initiation phase of deterioration, and these micro cracks connecting to each other to form larger cracks constitutes the propagation phase. When the larger cracks become large enough so that they compromise the ability of the concrete to fulfill its purpose, structural and/or aesthetic, the concrete reaches its durability failure point.

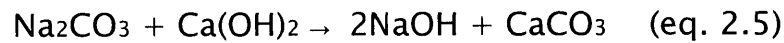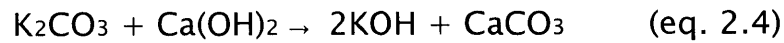## 2.5 Reactions Between Concrete and Corrosive Environments

Because of the high concentrations of Na+ and OH– ions, the pH of concrete is commonly between 12.5 and 13.5 (MacGregor, 1997), implying that concrete is a strong base. Because of this high pH, any environment with a pH less than about 12.5 would destabilize the alkalis in the pore fluid and, as a result, destabilize the cementious products of hydration that give concrete its strength. Therefore, any environment with a pH of 12.5, which includes most environments, must be considered corrosive to concrete (Derucher, Korfiatis, and Ezeldin, 1998). Common corroding agents for concrete include natural waters, waters containing industrial wastes, air, and soils. The three main corroding mechanisms for concrete are sulfate attack, acid attack, and carbonation. See Figure 3 below for an example of concrete damage to harmful chemical environments.

Figure 3. Concrete Damage Due to a Corrosive Environment

2.5.1 Sulfate Attack

Concrete is often exposed to sulfates occurring in the forms of sodium sulfate, potassium sulfate, and magnesium sulfate. Many of these substances are found in soil and come into contact with concrete either through direct contact between concrete and soil or through contact between concrete and groundwater containing dissolved sulfates. When dissolved sulfates come into contact with concrete, evaporation can magnify the detrimental effects of sulfate attack, because evaporation causes the concentration of the sulfates in solution to increase. According to Derucher, Korfiatis, and Ezeldin, the sulfate attack reactions follow the following equations.

Step 1:

$$Ca(OH)_2 + MgSO_4 \bullet 7H_2O \rightarrow CaSO_4 \bullet 2H_2O + Mg(OH)_2 + 5H_2O \qquad \text{(eq. 2.7)}$$

$$Ca(OH)_2 + Na_2SO_4 \bullet 7H_2O \rightarrow CaSO_4 \bullet 2H_2O + 2NaOH + 5H_2O \qquad \text{(eq. 2.8)}$$

$$Ca(OH)_2 + K_2SO_4 \bullet 7H_2O \rightarrow CaSO_4 \bullet 2H_2O + 2KOH + 5H_2O \qquad \text{(eq. 2.9)}$$

Step 2:

$$3CaO \bullet Al_2O_3 + 3CaSO_4 + XH_2O \rightarrow 3CaO \bullet Al_2O_3 \bullet 3CaSO_4 \bullet 31H_2O \quad \text{(eq. 2.10)}$$
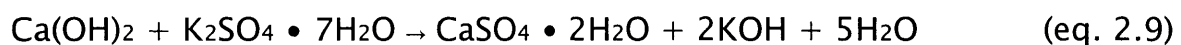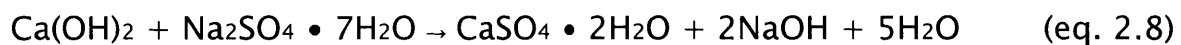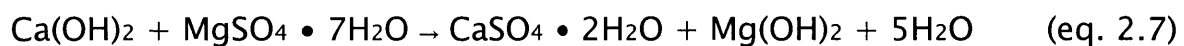
The X denotes a large, seemingly infinite, supply of water molecules. During the first step, eq. 2.7, 2.8, and 2.9, the sulfate combines with

calcium ions to form gypsum ($CaSO_4 \cdot 2H_2O$) and a hydroxide of magnesium, sodium, or potassium. In the second step, eq. 2.10, the gypsum combines with hydrated calcium aluminate to form calcium sulfoaluminate ($3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 31H_2O$).

The reactions in both step one and step two result in a volumetric expansion, and therefore, cause micro cracking. This micro cracking is usually near the surface, and is easily detectable by visual inspection. Because this micro cracking is near the surface, the micro cracking increases the permeability of the concrete, and therefore allows accelerated rates of deterioration. The formation of the micro cracks constitutes the initiation phase of deterioration, and these micro cracks connecting to each other to form larger cracks constitutes the propagation phase. When the larger cracks become large enough so that they compromise the ability of the concrete to fulfill its purpose, structural and/or aesthetic, the concrete reaches its durability failure point. When sulfate attack is the dominant factor in the concrete's deterioration, the failure point is often defined as the point at which the permeability coefficient rises above a critical value.

2.5.2 Acid Attack

Concrete, because of its high pH, is very vulnerable to acids. Concrete often is exposed to acids through contact with soils, groundwater runoff, industrial runoff water, and rainwater containing acids. According to Derucher, Korfiatis, and Ezeldin, the reaction typically proceeds as follows.

$$2(acid) + Ca(OH)_2 \rightarrow 2H_2O + (hydrated\ calcium\ compound) \qquad (eq.\ 2\text{-}11)$$
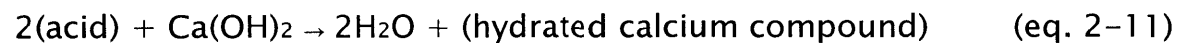
The acid combines with calcium hydroxide, a product of Portland cement hydration, to produce water and a hydrated calcium compound. The hydrated calcium compound produced depends on the type of acid encountered, and these hydrated calcium compounds are water soluble. Because acids are encountered on the surfaces of concrete members, the water soluble hydrated calcium compounds are removed from the concrete by exposing the concrete to water. The end result is a wearing away of the concrete surface, and the wearing away of the surface is detrimental to the concrete's durability.

Acid attack can be diagnosed by visual inspection by noticing the overall wearing away of the concrete surface. The initiation phase of deterioration consists of the exposure of the concrete member to acids,

and resulting reactions and wearing away of the concrete surface is the propagation phase of deterioration. When the surface is damaged to the extent that the ability of the concrete to fulfill its purpose is compromised, structurally and/or aesthetically, the concrete reaches its failure point.

2.5.3 Carbonation

According to Derucher, Korfiatis, and Ezeldin, carbonation results from concrete exposure to carbon dioxide, either from exposure to air or exposure to water containing dissolved carbon dioxide. The exact reaction that occurs is unknown, but the reaction can be generalized by carbon dioxide reacting with calcium hydroxide compounds to produce a carbonate and a salt. This reaction results in a net decrease in volume. If excessive carbonation occurs, enough carbonate will be produced to induce a reaction between the carbonate and the alkali components of the hydrated Portland cement paste. As discussed in Section 2.4.2, the alkali-carbonate reaction results in a net volume increase, and this net volume increase causes micro cracking. Carbonation results in the formation of a network of micro cracks along the surface of a concrete member. These micro cracks increase the permeability coefficient of the concrete, leading to further deterioration, and may cause pieces of the concrete surface to spall.

For carbonation, the initiation phase of deterioration consists of the formation of micro cracks, and these micro cracks connecting to each other to form larger cracks constitutes the propagation phase of deterioration. When the surface is damaged to the extent that the ability of the concrete to fulfill its purpose is compromised, the concrete reaches its failure point. A note should also be made that carbonation also results in a decrease in the overall decrease in the pH of concrete, and this overall decrease in pH causes the reinforcement to be susceptible to corrosion.

2.6 Chemical Degradation Summary

As a result of the hydration process, Portland cement paste contains many ions. These ions, if their concentration is high enough, can react with either the aggregate or with chemicals introduced into the concrete from the concrete's environment. Although many reactions between the concrete and its environment do not affect the durability of a concrete member, but the reactions discussed in Chapter 3 and 4 have a detrimental effect on the concrete's durability. The critical reactions between constituents of the concrete, mainly the cement paste and the aggregate, are the alkali silica reaction and the alkali-carbonate reaction. The critical chemical processes between the concrete and its environment

are sulfate attack, acid attack, and carbonation. These different chemical processes often occur simultaneously in concrete, and their effects must be limited in order to maintain durability throughout the service life of the structure.

Chapter Three: Corrosion of Embedded Steel

3.1 Introduction to Embedded Steel Corrosion

The corrosion of the embedded steel in concrete is a chemical degradation in that the corrosion depends on the chemical reactions involved with rusting. The corrosion process of the embedded steel is often the most critical of all the chemical degradation processes affecting concrete durability for two reasons. First, the corrosion of the embedded steel results in the largest volumetric expansion due to chemical reactions. This large volumetric expansion causes the most concrete cracking of any chemical degradation process, and as the cracking continues, the corrosion of the embedded steel accelerates. Second, the strength of the steel is a critical factor in determining the durability failure point. As the steel corrosion process progresses, the strength capacity of the steel is reduced. For concrete structural members, the bending capacity, shear capacity, and torsion capacity are highly dependent on the steel to resist the induced loadings (MaGregor, 1997). If the steel experiences a reduction in its cross-sectional area due to corrosion, the load-bearing capacity of the reinforcement is greatly reduced. The durability failure point for structural capacity corresponds to the point where the concrete member is no longer to resist the applied loadings due to degradation of the reinforcing steel. When the reinforcing

steel continually loses its load-bearing capacity due to corrosion, the load resisting capacities of the entire member are drastically reduced, and thus, the durability failure point is reached.

The concrete, with a pH around 13, usually protects the steel from extensive corrosion by inhibiting the reaction mechanisms. Also, the steel is usually covered with an initial layer of rust that aids in the protection of the steel from corrosion. When the permeability coefficient of the concrete increases, the initial layer of rust is not sufficient to protect the steel from corroding. As a general trend, as concrete ages, the permeability increases and the pH decreases, thus making the embedded steel more susceptible to corrosion. See Figures 4, 5, and 6 for examples of reinforcement corrosion.

Figure 4. Cracking Due to Reinforcement Corrosion
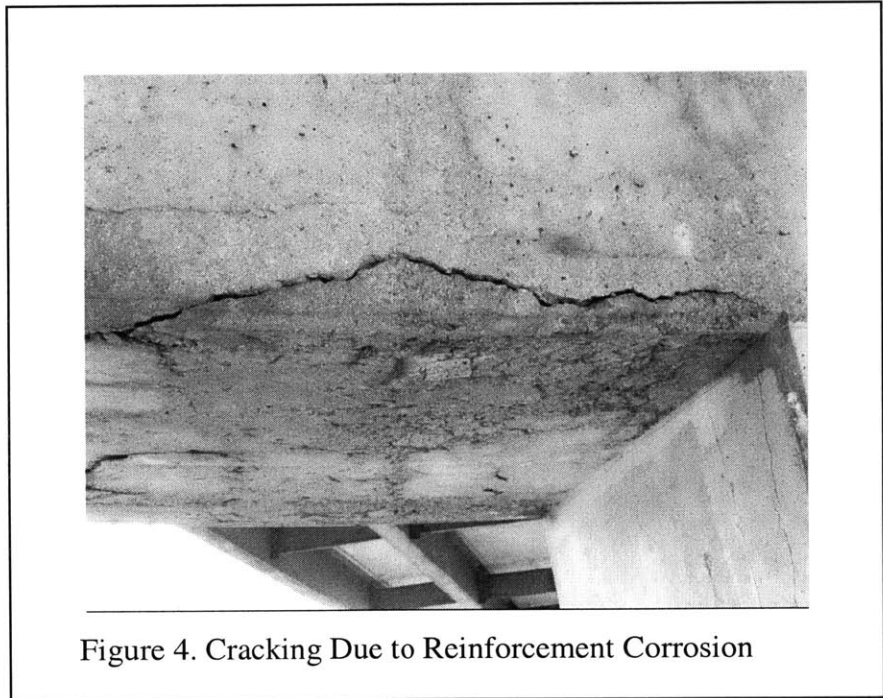
Figure 5. Spalling Due to Reinforcement Corrosion

Figure 6. Extensive Damage Due to Reinforcement Corrosion

3.2 Permeability

The main factor that affects the rate at which the steel corrodes is the concrete permeability. Permeability is the rate of flow of a fluid into a solid. Using a steady state model, the permeability coefficient, k, can be found by solving the following differential equation (Derucher, Korfiatis, and Ezeldin, 1998).

$$\frac{dq}{dt} = k \frac{hA}{L\mu}$$

(eq. 3.1)

where dq/dt is the flow rate, μ is the fluid viscosity, L is the length of the flow, and A is the flow area. Typical values of permeability for undamaged concrete members range from .010 to .300 μm/sec. The permeability coefficient can be drastically increased by the presence of cracks.

The permeability is important because the chemical reactions that cause corrosion of the steel require both water and oxygen. Although corrosion of the reinforcing steel will occur due to the presence of the initial water and oxygen in the cement paste, extensive corrosion requires water and oxygen that has permeated into the concrete to the location of the embedded steel. When concrete has cracked, the permeability coefficient greatly increases, allowing water and oxygen to penetrate into the

concrete much more quickly. If the water and oxygen penetrate to the location of the embedded steel, the steel will corrode.

The second main factor, other than cracks in the concrete, that affects permeability is the composition of the concrete. The critical aspects of composition, pertaining to permeability, are the porosity of the concrete, the type of aggregate used, the water to cement ratio, and the presence of pozzolanic materials. The permeability can be reduced by using an adequate amount of cement, using a low water cement ratio, ensuring proper compaction, and ensuring proper curing.

3.3 The Mechanisms of Embedded Steel Corrosion

The chemical process for steel corrosion is an electrochemical process, meaning that a cathode must receive electrons from an anode. These electrons are transferred between different parts of the embedded steel, one acting as the anode and the other as the cathode, through dissolved ions in the cement paste. The reaction proceeds as follows (Derucher, Korfiatis, and Ezeldin, 1998).

At anode: $Fe \rightarrow Fe(++) + 2e(-)$ (eq. 3.2)

At cathode: $2e(-) + H_2O + .5O_2 \rightarrow 2(OH(-))$ (eq. 3.3)

$$Fe(++) + 2OH \rightarrow Fe(OH)_2 \qquad (eq.\ 3.4)$$
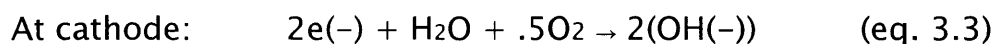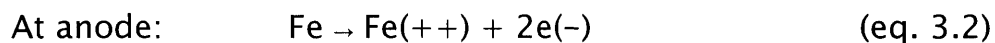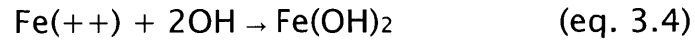
The iron in the steel, acting as a cathode, provides two electrons to the anode, where water and oxygen consume these two electrons in a reaction to produce two hydroxide ions. These hydroxide ions, products of the reaction at the cathode, react with the iron ion, a product of the reaction at the anode, to form iron hydroxide (rust). In reality, the iron ions can exhibit three different ionic states, but these states are neutralized by the iron ion combining with the hydroxide ion, meaning that rust can have the forms $FeOH$, $Fe(OH)_2$, and $Fe(OH)_3$.

This reaction not only results in a loss of material for the steel, the reaction results in a volumetric expansion of approximately 600 percent for the portion of the steel undergoing corrosion. This expansion causes tensile cracking in the concrete, and this cracking increases the permeability of the concrete, thus causing the corrosion process to accelerate.

3.4 Other Factors That Affect Corrosion

As discussed above, the high pH of concrete aids in protecting the steel from corrosion (MacGregor, 1997). This protection comes from the fact that the concrete, because of the high pH, is much more reactive than the

steel. As corrosive agents invade the concrete, the concrete neutralizes these agents before they react with the steel. When the pH of concrete is lowered to about 8 or 9, the concrete is no longer more reactive than the steel and therefore can no longer protect the embedded steel from corrosion.

As mentioned in Chapter 2, carbonation lowers the pH of concrete. Carbonation occurs as a result of exposure to carbon dioxide in the air or dissolved carbon dioxide in water. Carbonation occurs to some degree in all concrete members, and the effects must be limited. Typically, the pH reduces to a value between 11 and 12 due to carbonation, but if excessive carbonation occurs, the pH can be reduced to a value between 8 and 9. The lowering of the pH due to excessive carbonation causes the concrete to no longer be able to protect the steel from corrosion, and therefore, corrosion of the embedded steel occurs at a much faster rate.

Another factor that influences the rate of corrosion is the presence of chloride ions (Derucher, Korfiatis, and Ezeldin, 1998). Chloride ions dissolved in water, if the concentration is large enough, can create an electrolytic solution around the embedded steel. This electrolytic solution first destroys the protective layer of initial rust on the steel, and then acts as a catalyst, accelerating the corrosion reaction. A large concentration of chloride ions can induce corrosion at pH levels as high as 11.5. Chloride

ions are present in very small amounts in Portland cement and the water used in the hydration process. Certain types of aggregates, particularly aggregates from locations near seawater, and water-reducing admixtures can contain high amounts of chloride ions.

3.5 Steel Corrosion Summary

The degradation of the steel embedded in concrete is often the main factor in the overall capacity of the member to resist loadings. Corrosion of this steel results in rapid loss of strength for the steel and large volumetric expansions. These large expansions cause the concrete to crack, and these cracks increase the permeability of the concrete, thus accelerating the corrosion process. Eventually, if enough corrosion occurs, the load bearing capacity of the concrete member will be reduced to a value below the required load bearing capacity, and the member will fail.

The corrosion rate is reduced by the concrete, having a high pH and a low permeability coefficient, protecting the steel from invading corroding agents. The concrete loses its ability to protect the steel when the concrete permeability increases due to cracking, when the concrete pH is lowered due to carbonation, or when the concrete becomes electrolytic due to the presence of excessive chloride ions. These mechanisms of

inhibiting the ability of the concrete to protect the steel can occur simultaneously, and these mechanisms must be limited to ensure the performance of the concrete member throughout its service life.

In context of corrosion of embedded steel, the initiation phase consists of the destroying of the initial protective layer of rust on the steel and the concrete losing its ability to protect the embedded steel. The propagation phase of deterioration consists of the steel corroding, the resulting volumetric expansion, the resulting loss of strength, and the resulting acceleration of the corrosion process.

Chapter Four: Types of Physical Degradation

4.1 Introduction to Physical Degradation

Physical degradation of concrete is degradation that involves no chemical reactions. This type of degradation mainly involves altering the physical composition of the concrete member either by creating excessive pressures within the concrete member or by continually removing small particles from the concrete surface. These alterations to the composition result from continual wear of the concrete surface and exposure to varying temperatures. This chapter examines the various types of physical degradation and the effects of the degradation process on the capability of the concrete performance throughout its service life.

4.2 Abrasion and Erosion

According to Derucher, Korfiatis, and Ezeldin, abrasion is the result of continual dry friction on the concrete surface. Abrasion is often caused by continual vehicular or pedestrian traffic. Erosion is the result of continual friction on the concrete surface due to contact with fluids containing small solid particles. Erosion is common to concrete members frequently exposed to running water or strong winds.

The effects of erosion and abrasion depend mainly on the compressive strength of the concrete. As the compressive strength increases, abrasion and erosion resistance also increase. This dependence can be explained by the fact that the compressive strength can be correlated to the cohesion between particles in the concrete. As the compressive strength increases, this cohesion becomes stronger, and as a result, removing a particle from the concrete's surface requires more energy. This increase in required energy to remove a particle from the surface accounts for the increased abrasion and erosion resistance.

The effects of erosion and abrasion also depend on defects in the concrete, mainly segregation and bleeding. Segregation is the settling of different components of the concrete mix before hardening. The heavier components of the concrete, usually the coarse aggregates, settle to the bottom, and the lighter components, usually the water and Portland cement, remain at the top, resulting is a non-homogenous concrete member. Because of segregation, areas of the concrete member will contain too many coarse aggregate particles and not enough cement paste to provide cohesion between the coarse aggregate particles. The result of this lack of cohesion is a smaller compressive strength in areas of the concrete, and the areas with the smaller compressive strength are particularly prone to erosion and abrasion. Also, a note should be made

that concrete with a smooth surface is less prone to erosion and abrasion than concrete with a rough surface.

Bleeding is a result of using excessive amounts of water or improper distribution of the water during mixing. When improper distribution occurs, water pockets form in the concrete. When too much water is used, the water not consumed in hydration diffuses toward the surface and forms water pockets near the surface. These water pockets provide almost no cohesion, and therefore, the areas surrounding these water pockets are particularly prone to erosion and abrasion.

For abrasion and erosion, the initiation phase of degradation consists of the weakening of the bond between particles near the surface of the concrete exposed to traffic or fluids containing small solid particles, and the propagation phase consists of the removal of particles from the concrete surface. If enough concrete particles are removed from the concrete surface, the overall strength and aesthetics of the concrete will be compromised. The durability failure point is reached when the concrete's strength or aesthetics are compromised to an extent that the concrete can no longer fulfill its intended purpose.

4.3 Freeze/Thaw Degradation

The factors that influence the freeze/thaw degradation are the level of exposure to freeze/thaw cycles, the age of the concrete, the moisture content of the concrete, and the void content of the concrete. First, in order for freeze/thaw degradation to occur, the concrete must be exposed to freeze/thaw cycles. According to Swenson, depending on the moisture content of the concrete, significant damage can occur after a few cycles, for concretes with a high degree of saturation, or after many cycles, for concretes with a low degree of saturation. The degree of saturation is the ratio of the volume of voids filled with water to the total volume of voids.

The degree of saturation is a measure of the moisture content, and the moisture content is the critical factor affecting freeze/thaw degradation. The mechanisms involved in freeze/thaw damage involve freezing of excess moisture in the cement paste, thus causing expansive cracking. As the moisture content increases, the amount of cracking, resulting from alleviating the stresses induced by expansion of the moisture during the freezing process, increases. A factor that influences the moisture content at the time of exposure to freeze/thaw cycles is the age of the concrete (Swenson, 1969). During curing, the moisture not used in the hydration process gradually evaporates. Therefore, as the concrete ages,

the moisture content slowly decreases. This reduction of the moisture content causes concretes exposed to freeze/thaw cycles at an early age to be more susceptible to damage than older concretes.

The amount of freeze/thaw degradation also depends on the size and distribution of air in the concrete. According to Swenson, the air pockets in concrete aid in damage resistance in a number of ways. First, the air pockets provide insulation, causing less of the concrete to freeze. The air pockets also provide expansion areas when freezing occurs. When freezing occurs, the frozen moisture can expand into an air pocket, and as a result, no cracking will occur. Also, the air pockets prove a space for diffusing moisture to occupy, reducing the induced osmotic pressures. See Figure 7 below for an example of freeze/thaw damage.
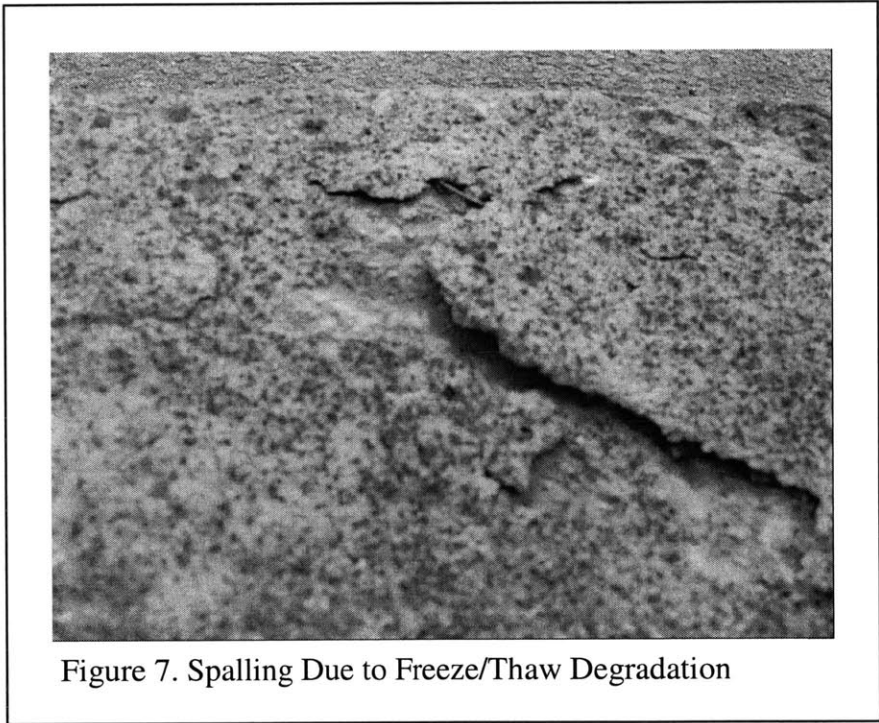
Figure 7. Spalling Due to Freeze/Thaw Degradation

4.4 Mechanisms of Freeze/Thaw Degradation

Freeze thaw degradation occurs because of two main mechanisms, the expansion pressures associated with freezing and the osmotic pressures induced by freezing. The freezing process begins in the larger voids in the cement paste. As discussed in Chapter 2, the water in the cement paste has dissolved alkalis resulting from the hydration process. The water in the larger voids freezes, but the alkalis do not. The result is a piece of ice surrounded by a solution with a very high concentration of alkalis in the void. The alkali solution is nearly incompressible, and as a result, the formation of this piece of ice results in a very high expansion pressure.

Along with this expansion pressure, osmotic pressures are created (Swenson, 1969). The solution surrounding the ice has a very high concentration of alkalis. This high concentration causes nearby water to try to diffuse into the solution. If a path exists through the cement paste, the nearby water will diffuse into this solution to reduce the concentration of alkalis. Unfortunately, the decrease in alkali concentration allows more water to be frozen, and once the diffused moisture is frozen, the alkali concentration returns to its original maximum value. This freezing of the diffused water causes more expansion pressures and more water diffusing into the solution. If a path

does not exist for the water to diffuse into the solution, the water trying to diffuse causes an enormous osmotic pressure.

This diffusion process occurs continuously. Ice is formed in the larger voids, and the ice continues to grow due to the diffusing of nearby water. This diffusing of nearby water also induces osmotic pressures. As the process continues, both the expansion pressures and osmotic pressures increase. If these pressures are large enough, micro cracks will form around the void.

For freeze/thaw degradation, the initiation phase consists of the forming of the micro cracks around the voids, and the propagation phase consists of the micro cracks connecting to each other to form larger cracks. When these larger cracks become large enough to compromise the ability of the concrete to fulfill its purpose, structural or aesthetic, the durability failure point occurs.

4.5 Physical Degradation Summary

Physical degradation of concrete results from exposure to freezing temperatures and exposure to continual surface wear. The amount of degradation induced by surface wear decreases as the compressive strength of the concrete increases, and the surface wear degradation is

greatly increased by defects in the concrete and decreased by the exposed concrete surface being smooth. The amount of freeze/thaw degradation depends on the moisture content, affecting the amount of pressure induced by the expansion involved in freezing, and the size and distribution of the air in the concrete, affecting how the induced pressures are alleviated. Both physical degradation, freeze/thaw degradation and surface wear degradation, and chemical degradation often occur simultaneously, and the effects of these degradations must be limited in order to maintain durability throughout the service life of the concrete.

Chapter Five: Durability Design Approach

5.1 Overall Design Approach

For durable concrete structures, the durability failure point must occur at a later time than the than the prescribed service life. The fundamental goal of durability design is increasing the time required for the structure to reach its durability failure point. This time increase should be large enough to cause the durability failure point to occur at a time later than the end of the prescribed service life.

The time required to reach the durability failure point, for all types of degradation, depends on both the rate at which initiation occurs and the rate at which propagation occurs (Paeglitis, 2001). As a result, concrete durability design focuses on prolonging the initiation phase, meaning that measures are employed to maximize the time required for the initiation phase to be completed, and reducing the propagation rate, meaning that measures are employed so that the effects of degradations require more time to become severe enough to compromise the ability of the concrete to fulfill its purpose.

5.2 Prolonging the Initiation Phase

Prolonging the initiation phase involves employing measures to maximize the amount of time required for the initiation phase to be completed. In order to decide which types of measures to employ, a general approach is used. First, the concrete, in order for the initiation phase to occur, must be exposed to a degradation agent. This degradation agent can be chemical, causing harmful chemical reactions either between components of the concrete or between the concrete and harmful chemicals form the environment, or physical, causing the concrete to undergo harmful physical changes. The various types of degradation agents and the degrading mechanisms associated with each were discussed in previous chapters.

The fundamental concept in prolonging the initiation phase is limiting the concrete's exposure to the degradation agent. Typical methods of limiting the concrete's exposure to degradation agents are adjusting the mix proportions, manipulating the surface texture of the concrete member, and adjusting the structural detailing. Methods of limiting the concrete's exposure to degradation agents are discussed in further detail in the following chapters.

5.3 Reducing the Propagation Rate

Reducing the propagation rate involves employing measures to maximize the time required for the effects of degradation to become severe enough to compromise the ability of the concrete to fulfill its purpose. Once the concrete has been exposed to a degradation agent, deterioration occurs on a microscopic level during the initiation phase. These microscopic deteriorations join together, during the propagation phase, to form a macroscopic defect, and this defect continues to grow until the concrete can no longer adequately perform.

The fundamental concept in reducing the propagation rate is employing mechanisms to inhibit the microscopic degradations from joining together to create a macroscopic defect and inhibiting the macroscopic defect from growing. Typical methods for reducing the propagation rate include the use of fibrous reinforcement and increasing the amount of air in the concrete. Methods of reducing the propagation rate are discussed in further detail in the following chapters.

5.4 Durability Design Approach Summary

The overall approach in durability design consists is as follows. First, the types of potential degradation agents present must be identified. The

diagnosis of the potential degradation agents usually depends on the concrete composition, the chemical conditions of the environment, and the physical conditions of the environment. Once potential degradation agents are identified, the necessary measures must be employed to limit the concrete's exposure to these degradation agents. Limiting the exposure to these degradation elements prolongs the initiation phase. Ideally, the initiation phase would be longer than the required service life. Often times, however, the propagation phase begins before the service life ends. Once the propagation phase begins, the rate at which propagation occurs must be reduced so that the prescribed service life ends before the durability failure point occurs.

Durability design depends on three critical steps: analyzing the types of degradation agents present, employing measures to prolong the initiation phase, and employing measures to reduce the propagation rate. The previous chapters examined the analysis of the types and deteriorating mechanisms, and the following chapters will examine the various measures to prolong the initiation phase and reduce the propagation rate.

Chapter Six: Preventing Chemical Degradation

6.1 Preventing Alkali-Silica Reaction

As discussed in Chapter Two, the alkali-silica reaction is the reaction between alkalis in the cement paste, a bi-product of the hydration process, with silica compounds in the aggregates. This reaction causes the formation of an unstable gel layer surrounding the aggregate. The formation of this unstable gel layer causes a harmful volumetric expansion and creates osmotic pressures due to the water affinity of the gel layer. The volumetric expansion and the osmotic pressures cause micro cracking during the initiation phase, and the micro cracks can connect to for larger, more detrimental cracks during the propagation phase.

Utilizing the model for preventing degradation proposed in Chapter 5, the initiation phase must be prolonged and the propagation rate must be reduced. In regards to alkali-silica reaction, the initiation phase can be prolonged significantly beyond the service life so that the propagation phase does not occur within the service life. As a result, preventative measures often focus on prolonging the initiation phase rather than reducing the propagation rate. In order to prolong the initiation phase, the exposure of the concrete to the degradation agent must be limited.

First, the degradation agent is identified as the harmful reaction between alkalis in the cement paste and the silica in the aggregate.

The best way of limiting the concrete's exposure to this degradation agent is preventing this reaction from occurring. To prevent the alkali-silica reaction from occurring, the amount of reagents in the reaction, the silica in the aggregate and the alkalis in the cement, must be greatly reduced. In essence, the amount of the alkali-silica reaction that occurs can be significantly reduced by using aggregates with low silica contents and reducing the amount of alkalis present in the cement paste after hydration.

Aggregates with potentially high silica contents are typically volcanic, metamorphic, and sedimentary rocks (Austin and Coose, 1998). The alkali content and general reactivity of an aggregate can be tested by ASTM C-289 (Test Method for Potential Reactivity of Aggregates) and ASTM C-277 (Test Method for Potential Alkali Reactivity of Cement Aggregate Combinations). In general, high quality, high strength aggregates contain insignificant amounts of silica (Lane, 2001).

To limit the amount of alkalis present in the cement paste, pozzolanic admixtures can be utilized (Lane, 2001). Pozzolans are materials, usually added to concrete during mixing, that react with products of hydration to

form materials with cementious properties (Derucher, Korfiatis, and Ezeldin, 1998). In regards to alkalis, pozzolans react with calcium hydroxide, sodium hydroxide, and potassium oxide, to form cementious materials. Because of these reactions, the amount of alkalis in the cement paste is greatly reduced, thus reducing the amount of alkali–silica reaction that occurs.

Typical pozzolans include volcanic ashes, pulcimites, tuffs, certain shales, certain clays, ground blast furnace slag, and fly ash (Derucher, Korfiatis, and Ezeldin, 1998). The amount of pozzolanic admixtures needed to prevent alkali–silica reaction depends on the type of pozzolan used and the reactivity of the aggregate. Usually, adding 15 to 25% fly ash or 35 to 40% ground blast furnace slag will significantly reduce the alkali–silica reaction for highly reactive aggregates (Lane, 2001). The above percentages refer to percent by mass of the pozzolan to the total amount of cementious material used in the concrete mix.

In summary, the degradation due to alkali–silica reaction can be prevented by selecting aggregates with low silica contents, reducing the amount of silica available for the reaction, and by employing the use of pozzolans into the concrete mix, reducing the amount of alkalis available for the reaction. In many areas of the United States, aggregates with low silica contents are available at no increase in cost, and pozzolanic

materials are readily available with varying costs, depending on the type of pozzolan desired.

6.2 Preventing Alkali-Carbonate Reaction

As discusses in Chapter 2, the alkali-carbonate reaction is the reaction between carbonate compounds present in the aggregates with alkalis in the cement paste. The products of this reaction have a larger volume than the reagents, and the result is an overall volumetric expansion of the concrete paste surrounding the aggregate particles. These expansions cause micro cracking during the initiation phase, and the micro cracks can connect to for larger, more detrimental cracks during the propagation phase.

Using the degradation prevention model, the initiation phase must be prolonged and the propagation rate must be reduced. Similar to that of the alkali-silica reaction, the initiation phase of the alkali-carbonate reaction can be prolonged significantly beyond the service life so that the propagation phase does not occur within the service life. As a result, preventative measures often do not attempt to reduce the propagation rate. This section focuses on methods of prolonging the initiation phase. In order to prolong the initiation phase, the exposure of the concrete to the degradation agent must be limited. Similar to the alkali-silica

reaction, the degradation agent is identified as the harmful reaction between alkalis in the cement paste and the carbonate compounds in the aggregate.

Again, the best way of limiting the exposure to this degradation agent is preventing the reaction from occurring. Thus, the amount of reagents in the reaction, the carbonates in the aggregate and the alkalis in the cement, must be greatly reduced. The amount of the alkali-carbonate reaction that occurs can be significantly reduced by using aggregates with low carbonate contents and reducing the amount of alkalis present in the cement paste after hydration.

Aggregates with potentially high carbonate contents typically are composed of limestone and dolomitic compounds (Ausitn and Coose, 1998). In order to limit the amount of carbonate compounds present in concrete, aggregates of this type should not be used. The carbonate content and general reactivity of an aggregate can be tested by ASTM C-289 (Test Method for Potential Reactivity of Aggregates).

To limit the amount of alkalis present in the cement paste, pozzolanic admixtures can be utilized. In regards to alkalis, pozzolans reacts with calcium hydroxide, sodium hydroxide, and potassium oxide, to form cementious materials. Because of these reactions, the amount of alkalis in

the cement paste is greatly reduced, thus reducing the amount of alkali-carbonate reaction that occurs.

In summary, the degradation due to alkali-carbonate reaction can be prevented by selecting aggregates with low carbonate contents, reducing the amount of carbonate available for the reaction, and by employing the use of pozzolans into the concrete mix, reducing the amount of alkalis available for the reaction. In many areas of the United States, aggregates with low carbonate contents are available at no increase in cost, and pozzolanic materials are readily available with varying costs, depending on the type of pozzolan desired.

6.3 Preventing Sulfate Attack

As discussed in Chapter 2, sulfate attack is the reaction of sulfates, from soil or water that has been in contact with soil, with the concrete. In the first step, the sulfates react with calcium ions to form gypsum and a hydroxide of magnesium, sodium, or potassium. In the second step, the gypsum combines with hydrated calcium aluminate to form calcium sulfoaluminate. Both reactions involved are expansive, and the expansion causes cracking. The formation of these micro cracks constitutes the initiation phase, and these micro cracks connecting to each other to for a larger crack is the propagation phase.

Similar to the alkali-silica reaction and the alkali-carbonate reaction, the initiation phase of sulfate attack can be extended significantly beyond the service life. As a result, the propagation phase does not occur within the service life, causing preventative measures to not focus on reducing the propagation rate. This section focuses on methods of prolonging the initiation phase. In order to prolong the initiation phase, the exposure of the concrete to the degradation agent must be limited. Similar to the previous sections, the degradation agent is identified as the harmful reaction between sulfates and the concrete.

The best way to limit the exposure to the degradation agent is preventing the reaction from occurring. The first reaction, because calcium ions are inherently present in cement paste, must be prevented by limiting the amount of sulfates in the concrete. Limiting the amount of sulfates in the concrete can be accomplished by producing concrete with low permeability and high quality (Derucher, Korfiatis, and Ezeldin, 1998). The permeability of the concrete can be reasonably correlated to the water-cement ratio. By using a low water cement ratio, the amount of excess water in the cement paste after hydration is reduced. Much of the excess water after hydration, especially near exposed surfaces, evaporates, leaving small voids in the cement paste. These small voids increase the permeability, thus making the concrete more vulnerable to

sulfate attack. By limiting the amount of excess water in the cement paste by using a low water-cement ratio, the amount of voids caused by evaporation is reduced, and as a result, the permeability is reduced.

Along the same logic of reducing the voids in the concrete, good quality concrete should be used. By ensuring quality in the concrete, the amounts of excess voids, due to defects and unnecessary cracking, are limited. These larger excess voids often cause large increases in the permeability, making the concrete more susceptible to sulfate attack.

The second reaction, the reaction between gypsum and calcium aluminate, must be prevented by reducing the amount of calcium aluminate in the cement paste. There are special types of cement, Type II and Type V Portland cement, with limited amounts of calcium aluminate. For Type II cement, the calcium aluminate content must not exceed 8%, and for Type V cement, the calcium aluminate content must not exceed 5% (Roy, 2002). By using either Type II or Type V cement, the amount of calcium aluminate is greatly reduced, and the detrimental effects of the reaction between gypsum and calcium aluminate are significantly reduced.

Lastly, the use of pozzolans increases the sulfate resistance of concrete. The pozzolans react with calcium hydroxide to form cementious

materials, and as a result, the amount of calcium hydroxide in the cement past is reduced. Because calcium hydroxide is a reagent in the expansive reaction to form gypsum, the reaction in step one, the degradation effects of sulfate attack are reduced (Lane, 2001).

In summary, concrete's resistance to sulfate attack is significantly increased by preventing the detrimental reactions involved in sulfate attack from occurring. The reactions can be prevented by limiting the amounts of reagents available for the reaction. For the first reaction, the amount of calcium hydroxide ions can be reduced by using pozzolans, and the amount of sulfates can be reduced by ensuring low permeability in the concrete. For the second reaction, the amount of calcium aluminate can be reduced by using Type II or Type V Portland cement. By preventing these detrimental reactions from occurring, the initiation phase of sulfate attack is greatly prolonged, and thus the overall durability of the concrete is increased.

6.4 Preventing Acid Attack

As discussed in Chapter 2, acid attack occurs in concretes exposed to acids from soils, groundwater, industrial wastes, or acidic rainwater. The acid combines with calcium hydroxide in the cement paste to produce water and a hydrated calcium compound. The hydrated calcium

compound is water soluble, and these hydrated calcium compounds are removed from the concrete by exposing the concrete to water. This process results in wearing away of the concrete surface.

The initiation phase consists of the exposure of the concrete member to acids, and the resulting reactions and wearing away of the concrete surface constitutes the propagation. The degradation agent, therefore, is the acidic environment. Very little can be done to reduce the propagation rate because the propagation rate depends on the availability of water on the concrete surface. Therefore, to increase the ability of concrete to resist acid attack, the exposure to the acidic environment must be limited.

In severely acidic environments, such as certain industrial facilities, protective coatings must be used to protect the concrete (Derucher, Korfiatis, and Ezeldin, 1998). These coatings create a non-reactive barrier between the concrete and the environment, thus eliminating the concrete's exposure to the degradation agent. However, these coatings are often costly and therefore should only be used in highly acidic environments.

In most natural environments, the acidic conditions are less severe. To prevent acid attack corrosion, the permeability of the concrete must be

reduced. By reducing the permeability, the acid exposure occurs only on the surface of the concrete. The acid gradually wears away the surface, but this process, if the permeability is low enough, proceeds very slowly. With a reduced permeability, the time required for the acid attack degradation process to significantly damage the concrete exceeds the service life. Methods of decreasing the permeability of the concrete are using a low water-cement ratio and using high quality concrete, as discussed in the previous section. Also, a smooth surface will increase the concrete's durability to acid attack.

In summary, because the reaction between concrete and acid is not preventable, concrete's resistance to acid attack is increased by limiting the areas in which the reaction occurs to the surface of the concrete. By reducing the permeability of the concrete, the detrimental reactions occur only near the surface. Furthermore, with a reduced permeability, the damaged portion of the surface must be removed before the further damage occurs. By reducing the concrete permeability, the degradation process involved in acid attack proceeds very slowly, thus allowing little damage to occur during the concrete service life. For concrete with very a long service life or concrete in a severely acidic environment, protective coatings may be required to ensure the concrete's performance.

Chapter Seven: Preventing Physical Degradation

7.1 Preventing Abrasion and Erosion

As discussed in Chapter 4, abrasion is the result of continual dry friction on the concrete surface, mainly caused by constant vehicular or pedestrian traffic. Erosion is the result of friction on the concrete surface due to contact with fluids containing small solid particles, mainly caused by running water or strong winds. For both abrasion and erosion, the initiation phase of degradation consists of the weakening of the bond between particles near the exposed surface of the concrete, and the propagation phase consists of the removal of portions of the concrete surface.

In order to prolong the initiation phase, the effects to the degradation agent, the weakening of the bond between concrete particles due to abrasion or erosion, must be reduced. The weakening of the bond between particles is mainly due to fatigue induced by the repetition of frictional stresses. Therefore, if these frictional stresses are reduced, the effects of fatigue occur at a much later time and thus, the initiation phase will be prolonged. Because the friction stresses are induced by the environment, the reduction of the friction stresses only occurs when the stresses are distributed across the maximum area on which the force is

applied. This distribution of the friction force results in an even distribution of stresses amongst the components of the concrete, resulting in minimized stress applied to each individual component of the concrete. The best way to ensure an even distribution of friction force across the concrete surface is to use a smooth concrete surface. Uneven concrete surfaces cause stress concentrations, and these stress concentrations induce fatigue related weakening. By using a smooth concrete surface, the frictional stress distribution is optimized, and the harmful fatigue related weakening on the individual components of the concrete is minimized.

The initiation phase can also be prolonged by using a low water-cement ratio. As the water-cement ratio decreases, the concrete strength, representing the cohesion between particles, increases. As this cohesion increases, more weakening, due to friction induced fatigue, must occur before a particle can be removed. With a reduced water cement ratio, the overall increase in cohesion prolongs the initiation phase in that more time is required to weaken the bond between particles to the extent that a particle can be removed.

To reduce the propagation rate, the rate at which the surface is removed from the concrete, the defects in the concrete must be minimized. Defects are areas where the concrete strength is significantly reduced due

to non-homogenous material distribution. After the bond between particles is weakened during initiation, defects in the concrete can cause large portions of the surface to be removed rather than only individual particles being removed. To avoid concrete defects near the concrete surface, proper mixing, transportation, pouring, and finishing must be insured so that the amount of bleeding and segregation is minimized (Bentz et al., 1999). Bleeding is a result of using excessive amounts of water or improper distribution of the water during mixing. Bleeding results in large water pockets in the concrete, and these water pockets provide almost no cohesion. As a result, the areas surrounding these water pockets are particularly prone to erosion and abrasion. Bleeding can be minimized by ensuring that no excess water is used for mixing, ensuring that mixing occurs thoroughly, ensuring the concrete is adequately mixed during transportation, and ensuring that no extra water is used for finishing (Bentz et al., 1999).

Segregation is the settling of different components of the concrete mix before hardening. The heavier components of the concrete, usually the coarse aggregates, settle to the bottom, and the lighter components, usually the water and Portland cement, remain at the top. Due to segregation, areas of the concrete contain too little cement paste and therefore lack the cohesion needed to resist abrasion and erosion degradation. Segregation can be minimized by ensuring proper mixing

techniques are used, ensuring that the concrete is adequately mixed during transportation, and that the concrete is properly placed and finished (Bentz et al., 1999).

In general, abrasion and erosion degradation can be minimized by ensuring that the concrete is of good quality. A low water cement ratio should be used to provide adequate strength, a smooth surface should be used to minimize fatigue stresses, and the defects in the concrete should be minimized by ensuring proper mixing, transportation, pouring, and finishing techniques are used.

7.2 Preventing Freeze/Thaw Degradation

As discussed in Chapter 4, freeze/thaw degradation is the result of water in the cement paste repeatedly freezing and thawing, causing cracking due to the associated expansions and contractions. For freeze/thaw degradation, the initiation phase consists of the forming of the micro cracks around the voids, and the propagation phase consists of the micro cracks connecting to each other to form larger cracks.

In order to prolong the initiation phase the expansion of the water in the cement paste due to freezing, must be reduced. These effects are reduced by minimizing the amount of excess water in the cement paste.

The amount of freeze/thaw damage increases as the amount of excess water in the cement paste increases. The best way to ensure an overall reduction in excess water in the cement paste is using a low water-cement ratio. The hydration process only consumes a portion of the water in the concrete mix, and the remainder of the water is required for adequate mixing and workability. The water not used in hydration remains in the cement paste, and this water is crucial for freeze/thaw damage. By using a low water-cement ratio, the excess water after hydration in the cement paste is minimized, and therefore, less freeze/thaw damage occurs per freeze/thaw cycle. The water-cement ratio should not exceed .45, and the slump should not exceed 3.5 inches (Swenson, 1969). With less damage occurring per cycle, the rate of the formation and growth of micro cracks is reduced, and thus, the initiation phase is prolonged.

Another way of reducing the amount of excess water in the cement paste is ensuring proper curing. As a general trend, during the curing process, concrete gradually experiences a reduction in its moisture content due to continued hydration and evaporation (Swenson, 1969). The continued hydration occurs throughout the concrete and provides an overall moisture content reduction. The evaporation occurs near the concrete surface, the area prone to freeze/thaw damage, and also results in a reduction of the excess moisture. Also, the age of the concrete when first

exposed to freeze/thaw cycles is crucial due to the evaporation of excess moisture. If the concrete has enough time for significant evaporation to occur before being exposed to its first freeze, the freeze/thaw resistance will increase.

Another way to minimize the amount of excess moisture in concrete is reducing the amount of time the concrete is exposed to water from the environment. This reduction can be accomplished by manipulating the shape of the concrete surface to avoid pooling of water on the concrete surface. Whenever possible, the concrete surface should be graded so that water flows away from the concrete surface rather than pooling. By reducing the time that the concrete is exposed to water, the amount of excess water in the cement paste decreases, and therefore, the freeze/thaw resistance increases.

The use of pozzolans also prolongs the initiation phase. As discussed in Chapter 4, as water in the cement paste freezes, the portion of water that does not freeze experiences a large increase in the concentration of alkalis. Upon freezing, this increase in concentration results in the diffusion of nearby water, and this diffusion results in either the ice crystal growing or enormous osmotic pressures. Both of these mechanisms cause micro cracks to form. As discussed in Chapter 6, pozzolans react with alkalis in the cement paste to produce cementious

materials, therefore decreasing the amount of alkalis in solution (Derucher, Korfiatis, and Ezeldin). This reduction in alkalis results in a reduction of the amount of water diffusing towards the ice crystals. The reduction in the amount of diffusing water causes both a reduction in the osmotic pressures and a reduction in the rate at which the ice crystals grow, and therefore, the micro cracking process proceeds at a slower rate, thus prolonging the initiation phase.

To reduce the propagation rate, the connection of the micro cracks to form a larger, detrimental cracks must be inhibited. As the ice excess water expands and creates micro cracks due to freezing, the mechanisms of the freezing cycle induce stress concentrations at the tip of the micro cracks and near areas of very high pressures. The use of air entrainment relieves these stress concentrations and allows the concrete to strain locally without inducing any larger cracks (Swenson, 1969). This phenomenon is caused by the ability of the air to easily compress and easily reshape itself due to the strain induced changes in the shape of the voids. In effect, the air pockets provide relief to the high pressures induced by expansions due to micro cracking and other stress concentrations. This pressure relief results in inhibiting the propagation of micro cracks to form a larger crack (Swenson, 1969), and thus, the propagation rate of degradation is reduced. Air entraining agents are organic compounds that, when added during mixing, create small air

bubbles that form the air pockets. These air pockets should be about .01 inches in diameter and should compose about 5 to 7% of the concrete volume (Swenson, 1969).

In summary, the initiation phase of freeze/thaw degradation can be prolonged by using a reduced water-cement ratio, ensuring proper curing of the concrete, limiting pooling on the concrete surface if possible, and the use of pozzolans. The propagation rate can be reduced by the use of air entrainment in the concrete. These methods require minor adjustments to the concrete and are inexpensive, and if employed properly, these methods will significantly increase the concrete durability in the context of freeze/thaw cycles.

Chapter Eight: Preventing Reinforcement Corrosion

8.1 Design Approach for Preventing Corrosion

As discussed in Chapter 3, the corrosion of the reinforcement steel embedded in concrete occurs due to the presence of water and oxygen that has penetrated into the concrete. This water and oxygen allows the electrochemical reaction of rusting to occur, and this reaction results in a very large volumetric expansion. This expansion results in severe damage to the concrete due to spalling, and the strength of the concrete member is rapidly lost as corrosion occurs due to the reduction in the load-bearing capacity of the reinforcing steel. The corrosion process is accelerated by the presence of cracks in the concrete, the presence of chloride ions, and the effects of excessive carbonation.

The corrosion of reinforcement steel in concrete is the most critical of degradations because steel corrosion, when compared to other types of degradation, results in the most rapid loss of strength and the most extensive physical damage to the concrete member (MacGregor, 1997). In context of corrosion of embedded steel, the initiation phase consists of the concrete losing its ability to protect the embedded steel. The propagation phase consists of the steel corroding, the resulting

volumetric expansion, the resulting loss of strength, and the resulting acceleration of the corrosion process.

Because the propagation phase occurs rapidly, the propagation rate would need to be reduced to a small percentage of its original value for any significant extension of the time required to reach the durability failure point. Because a reduction of the propagation rate of this magnitude is very difficult to achieve, the main focus of preventing the corrosion of reinforcing steel is prolonging the initiation phase.

The main approach in prolonging the initiation phase is extending the time required for the corroding agents, water and oxygen, to permeate through the concrete to reach the reinforcing steel. This extension can be achieved by reducing the permeability coefficient, increasing the cover, or providing protective coatings on either the concrete surface or the surface of the embedded steel.

8.2 Methods of Reducing the Permeability

The permeability coefficient can be reduced by using concrete with a low water-cement ratio between the concrete surface and the embedded steel (Derucher, Korfiatis, and Ezeldin, 1998). The water-cement ratio, as earlier mentioned, affects the amount of water remaining in the concrete

after hydration. Near concrete surfaces, the excess water often evaporates, leaving empty voids in the concrete. These voids significantly increase the permeability of the concrete, allowing the corrosion agents to reach the embedded steel faster. By using a low water-cement ratio for the concrete between the concrete surface and the embedded steel, the amount of excess water remaining in the concrete after hydration is reduced. This reduction in excess water causes a reduction in the permeability coefficient, and therefore, the initiation phase is prolonged because the corrosion agents require more time to reach the embedded steel.

Another way of reducing the permeability coefficient is by limiting the amount of cracking that occurs in the area between the embedded steel and the concrete surface. Due to concrete being weak in tension, cracks often occur in this area for reinforced concrete sections. As a result, the amount of cracking needs to be significantly reduced by altering the structural detailing process for concrete members susceptible to frequent wetting and drying cycles. If the structural detailing process cannot be altered so that tensile cracks are limited, fibrous reinforcement can be used to limit the propagation of the tensile cracks. Fibrous reinforcement employs the use of short, typically .1 to 1 cm long, fibers to resist crack propagation (Derucher, Korfiatis, and Ezeldin, 1998). Nylon is the most common material for fibrous reinforcement, but the use of any fiber

commonly used for fibrous concrete reinforcement will limit the amount of tensile cracking that occurs. By limiting the amount of tensile cracking, either by fibrous reinforcement or by altering structural details of the member cross section, the permeability coefficient can be significantly reduced. This reduction of the permeability coefficient prolongs the initiation phase.

A note should also be made that the high pH of concrete makes the concrete more reactive than the reinforcing steel. As corrosion agents permeate into the concrete, they often react with the concrete before they ever reach the embedded steel. The concrete, therefore, protects the steel from corrosion. If the permeability is reduced, the rate at which the corrosion process occurs is reduced by two mechanisms. First, the amount of time required for the corrosion agent to reach the embedded steel is increased as discussed in the previous paragraphs. Secondly, as the amount of time required for the corrosion agents to reach the embedded steel increases, the amount of reaction that occurs between the corrosion agents and the concrete also increases. This increase results in a significant reduction in the amount of corrosion agent, if any, that permeates into the concrete to the depth of the embedded steel. The presence of chloride ions, from certain admixtures, certain aggregates, or de-icing salts, and excessive carbonation reduce the pH of concrete, thus

reducing the reactivity of the concrete with the corrosion agents (Derucher, Korfiatis, and Ezeldin, 1998).

8.3 Effects of Increasing the Cover

Another approach to solving the problem of the corrosion agents permeating into the concrete to the depth of the reinforcing steel is increasing the distance, known as the "cover", between the exposed concrete surface and the embedded steel (MacGregor, 1997). By increasing this distance, the amount of time required for the corrosion agent to reach the embedded steel at a given permeation rate is significantly increased. Also, increasing this distance increases the amount of concrete to which the permeating corrosion agents are exposed. This increased exposure causes more reaction to occur between the corrosion agents and the concrete, and as a result, the amount of corroding agents that reach the embedded steel, if any, is significantly reduced. Also, by increasing the cover, the negative effects of carbonation and chloride ions on the concrete reactivity are reduced as the corrosion agents move away from the surface toward the embedded steel.

8.4 Protective Coverings

Another method of protecting the embedded steel from corrosion is the use of protective coatings. These coating are used either on the exposed surfaces of the concrete member or on the surfaces of the reinforcing steel. In either case, the coatings form a non-reactive barrier between the corrosion agents and the embedded steel, thus prolonging the time required to reach the durability failure point.

Coatings on the exposed surface of the concrete members are often used in areas where the concrete will be exposed to large amounts of carbonation and chloride ions, such as areas near seawater. These coatings are often made of polyurethanes or epoxies applied to the concrete surface. Both polyurethane and epoxy coatings, if applied correctly, can almost totally inhibit corrosion of the reinforcing steel (Derucher, Korfiatis, and Ezeldin, 1998).

Coatings on the surface of the reinforcing steel are often used in members where extensive cracking, either due to tensile stresses or shrinkage, is expected to occur. These coatings are almost exclusively made of epoxy. One drawback of using epoxy-coated reinforcement is that the epoxy makes bending the bars very difficult, resulting in high construction costs. If used properly, however, epoxy-coated

reinforcement can almost totally inhibit the corrosion of the reinforcing steel.

8.5 Steel Corrosion Summary

In summary, the corrosion of the reinforcing steel, because of its importance to the strength of the overall member, is an issue of particular importance. In order for the concrete member to meet its performance criteria throughout its service life, the corrosion of the reinforcing steel must be significantly reduced by employing the above discussed mechanisms. Many of the mechanisms, such as using a lower water-cement ratio or providing additional cover, are relatively inexpensive. Many other mechanisms, such as providing protective membranes on the surface of the concrete, are relatively expensive. The method employed to resist corrosion should be chosen depending on the contractor capabilities, the availability of materials, the level of exposure to the corroding agents, and the structural and architectural requirements of the concrete member. In any case, an initial investment in mechanisms to inhibit corrosion is advisable, because the costs associated with repairing concrete members damaged by corrosion of the embedded steel are the highest of all common concrete repair costs.

Chapter Nine: Summary and Conclusions

Recent trends in concrete durability design have favored the use of protective coatings. Although these coatings, if applied correctly, can totally inhibit degradation of the concrete member, these coatings are rather expensive. In some situations, the coatings are necessary, but in many cases, they are not. If the type of degradation agents to which the concrete is to be exposed during its service life can be accurately predicted, many times the durability design requires only minor, inexpensive changes to the concrete mix proportions, the mix ingredients, or the structural detailing. The following list summarizes the various types of degradation discussed in this document how to design for each type.

Type of Degradation: Alkali-Silica Reaction

Initiation Phase: micro-cracking due to expansions and osmotic pressures

Propagation Phase: formation of a larger, detrimental crack

Design Approach: prolong initiation

Prevention Mechanisms: limit amounts of silicas in aggregate, limit amount of alkalis in cement, use of pozzolans to reduce alkali content

Type of Degradation: Alkali–Carbonate Reaction

Initiation Phase: micro-cracking due to expansions

Propagation Phase: formation of a larger, detrimental crack

Design Approach: prolong initiation

Prevention Mechanisms: limit amounts of carbonates in aggregate, limit amount of alkalis in cement, use of pozzolans to reduce alkali content

Type of Degradation: Sulfate Attack

Initiation Phase: micro-cracking due to expansions

Propagation Phase: formation of a larger, detrimental crack

Design Approach: prolong initiation

Prevention Mechanisms: reduce concrete permeability by using a low water-cement ratio, limit defects in concrete, use of pozzolans to limit calcium aluminate

Type of Degradation: Acid Attack

Initiation Phase: exposure to acids

Propagation Phase: the resulting reactions and removal of the concrete surface

Design Approach: prolong initiation

Prevention Mechanisms: reduce concrete permeability by using a low water-cement ratio, use a smooth surface, use coatings if exposure is severe enough

Type of Degradation: Abrasion and Erosion

Initiation Phase: weakening the bond between particles

Propagation Phase: removal of portions of the concrete surface

Design Approach: prolong initiation and reduce propagation rate

Prevention Mechanisms: use a smooth surface, use a low water-cement ratio to increase cohesion between particles, limit concrete defects, eliminate segregation and bleeding

Type of Degradation: Freeze/Thaw Degradation

Initiation Phase: micro-cracking due to expansions and osmotic pressures

Propagation Phase: formation of a larger, detrimental crack

Design Approach: prolong initiation and reduce propagation rate

Prevention Mechanisms: limit excess water in concrete by using a low water-cement ratio, ensure proper curing, maximize the age at which the concrete experiences its first freeze/thaw cycle, manipulate surface shape to eliminate ponding, use pozzolans, use air entrainment

Type of Degradation: Corrosion of Reinforcing Steel

Initiation Phase: concrete losing ability to protect steel

Propagation Phase: corrosion of steel, expansions, and loss of strength

Design Approach: prolong initiation

Prevention Mechanisms: reduce concrete permeability by using a low water-cement ratio, limit the amount of tensile cracks, limit chloride and carbonation exposure, increase the cover, use protective coatings if necessary

In general, to design for concrete durability, the concrete should employ the use of a good quality, non-reactive aggregate, low alkali content cement with reduced amounts of calcium aluminate, pozzolanic materials, a low water cement ratio, proper mixing and curing procedures, proper transportation and casting procedures, proper finishing procedures and adequate quality control, and proper structural detailing. With careful attention given to these details, under moderate exposure and for a normal service life, the concrete should not reach its durability failure point before the end of its prescribed service life.

References

1. Derucher, Kenneth N., Korfiatis, George P., and Ezeldin, A. Samer. *Materials for Highway and Civil Engineers, 4th ed.* Prentice Hall, Upper Saddle River, New Jersey, 1998.

2. MacGregor, James G. *Reinforced Concrete Mechanics and Design, 3rd ed.* Prentice Hall, Upper Saddle River, New Jersey, 1997.

3. Kuennen, Tom. "TRB Reports Spotlight Advantages in Ready Mixed." *Concrete Products*, March 2001.

4. Lane, Stephen. "Alkali-Silica Reaction: Preventing Damage in Hydraulic Cement Concrete." *Transportation Research News*, December 2001.

5. Swenson, E.G. "Durability of Concrete Under Winter Conditions." Canadian Building Digest, August 1969.

6. Paeglitis, Ainars. "Durability Design Approach for Concrete Bridges." Inzenierbuve Ltd, Azenes st. 20. Riga, Latvia, 2001.

7. Bentz, Dale P., Clifton, James R., Ferraris, Chiara F., and Garboczi, Edward J. *Transport Properties and Durability of Concrete: Literature Review and Research Plan.* National Institute of Standards and Technology, September, 1999.

8. Austin, George S., and Coose, Kristen E. *Aggregate Reactivity and Concrete Durability in New Mexico.* New Mexico State Highway and Transportation Department, Abluquerque, New Mexico. September, 1998.

9. Roy, Della M. "Relationship of Portland Cement Characteristics to Concrete Durability". *National Cooperative Highway Research Program Research Results Digest.* Transportation Research Board, October, 2002.