THE EFFECTS OF CORROSION ON
REINFORCED CONCRETE WITH FIBER ADDITION

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THE EFFECTS OF CORROSION ON
REINFORCED CONCRETE WITH FIBER ADDITION

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Thesis

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ABSTRACT

Corrosion is known to negatively affect the structural integrity of bridges, buildings, and other structures made of concrete. This corrosion damage is both dangerous to the public and expensive to the owner of the structure. This thesis explores the use of polypropylene or basalt fibers to improve the structural performance of concrete that has been subjected to corrosion damage. The use of polypropylene fibers in the concrete mix improves moment capacity and ductility of the tested concrete beams. The result for the basalt fiber also showed an improvement in moment capacity.
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CHAPTER I
INTRODUCTION

The leading cause of deterioration of concrete structures in the modern world is due to the effects of corrosion of reinforcing steel. In 2002, the estimated annual cost of corrosion in the United States was $276 billion dollars, according to a NACE corrosion study (1). The same study estimated the cost for maintenance and capital cost of concrete bridges alone was $4 billion dollars. This value represents the direct cost of corrosion of these structures. Indirect costs, such as lost productivity, increased travel time, etc, were estimated to be ten times as much. Costs have only increased in the intervening years as infrastructure continues to decay over time.

In addition to the monetary costs, corrosion can cause catastrophic failure of bridges and other structures. In 1967, the collapse of the Silver Bridge connecting West Virginia, and Ohio was attributed to the corrosion of an eyebar. This catastrophic failure resulted in 46 fatalities and 9 injuries (2).

The two primary causes of corrosion of reinforcing steel in concrete are the ingress of carbon dioxide and chlorine. Both of these substances cause a breakdown in the passive layer of the reinforcing steel, allowing the active corrosion to progress. Once active corrosion of the steel begins, several detrimental effects occur.

The corrosion products occupy more space than the original iron atoms in the steel. As these corrosion products build up, the area around the reinforcement steel
tries to expand. Because of concrete’s relative weakness in tension, this expansion causes cracks to develop in the concrete. Eventually, these cracks can reach the surface, allowing a clear path from the external environment to the surface of the rebar.

In addition to the cracking of the concrete, the corrosion products also reduce the bond strength between the steel and concrete. This bond strength is essential for the reinforced concrete to act as a composite material. Degradation of the concrete through cracking can eventually lead to the concrete falling away from the structure. This more extensive form of cracking is known as spalling.

Cracking and spalling are signs of degradation that can be observed by the naked eye. These warning signs, however, are advanced phases of corrosion damage. Once concrete has begun to spall away from the structure, the structural integrity of the concrete member has already been compromised.
CHAPTER II

OBJECTIVE

The objective of this research is to explore the use of polypropylene and basalt fibers to mitigate the structural damage of concrete that is caused when the reinforcing steel corrodes. To evaluate the performance of polypropylene and basalt fibers, it is important to first understand how corrosion damage affects normal reinforced concrete, so that is also investigated.
CHAPTER III

LITERATURE REVIEW

Corrosion Process of Reinforced Concrete due to Chlorine Infiltration

Concrete has two main protection mechanisms that protect the steel reinforcement from corrosion. First, it provides a physical barrier that prevents the steel from coming in contact with the external environment. This prevents substances such as water, salt, or other damaging ions from reaching the iron atoms that make up the steel. Secondly, the concrete provides an extremely alkaline environment, normally with a pH between 12.5 and 13.5. Within this environment, the steel does not corrode actively, but rather the steel forms a protective, passive layer. Because of these two methods of protection, the steel is unable to corrode further, and a steady state is reached.

Step 1: Transport

After casting and curing, the concrete is exposed to some environment during its service life. There are as many different environments as there are uses for concrete. Focusing on environments that contain chlorine, some examples include: marine environments, or roads and bridges exposed to deicing salts. These chlorine ions are not harmful to the concrete; however they do cause corrosion in the reinforcing steel.
In order to affect the steel, these ions must first be transported through the concrete. During this stage, no actual corrosion takes place. The amount of time that this process takes depends on many variables: quality of the concrete, mix designs, presence of cracks, concrete pore size, aeration levels, ion concentration, temperature, and cover thickness, amongst others.

Step 2: Initial Corrosion

Once a certain threshold of chlorine ions has reached the surface of the steel, corrosion begins to occur. At this time the steel is no longer able to maintain the protective, passive film, and begins to corrode actively, losing iron atoms which form into various types of corrosion products. These corrosion products occupy a much larger volume than the original iron atom did. Because of this, the corrosion products begin to exert a pressure on the surrounding concrete. During this time, the buildup of corrosion products around the steel is relatively uniform. The buildup of pressure
causes the concrete surrounding the steel to be subjected to tensile forces. These forces continue to build as more and more corrosion products are formed.

Step 3: Internal Cracking

At some point, the concrete surrounding the steel will begin to crack. Multiple cracks will begin to form near the steel-concrete interface and begin to work their way through the concrete. During this time, corrosion products continue to form around the steel in a uniform manner. As corrosion products continue to accumulate, one crack will become dominant and will reach the surface of the concrete.

Step 4: Surface Cracking

Once the dominant crack reaches the surface, several things begin to occur. First, corrosion products are able to rapidly leave the interior of the concrete and will show up on the external surface as rust surrounding the crack. Second, it becomes a great deal easier for ions to transport through the concrete. The crack creates a faster path through the concrete than the tortuous path through the concrete’s pores. Because of this increase in availability of ions, the rate of corrosion can speed up dramatically. Third, this cracking begins to relieve the confining pressure surrounding the reinforcing steel.
Step 5: Spalling

As corrosion products continue to build up within the concrete, the additional pressures cause the concrete to lose local structural integrity. Eventually, pieces of concrete will begin to fall away, or spall, from the structure. In addition to the obvious dangers of falling concrete, the steel becomes directly exposed to the external environment, completely removing any protection from further corrosion damage to it.

Half Cell and Chemical Reactions

Corrosion is an electrochemical process much like that of a battery or fuel cell, only in reverse. There is both an anodic and cathodic reaction which involve the movement of electrons and result in the formation of corrosion products. In the anodic reaction, electrons are removed from the iron atoms in the steel, while in the cathodic reaction those electrons are consumed. The anodic reaction is as follows (4):

$$Fe \rightarrow Fe^{2+} + 2e^- \quad (3.1)$$

The cathodic reaction is as follows (4):

$$2e^- + H_2O + \frac{1}{2}O_2 \rightarrow 2OH^- \quad (3.2)$$

As can be seen from the cathodic reaction, this corrosion process requires the presence of oxygen and water. Without either or both of these, this type of corrosion
will not occur. Once these initial electrochemical reactions occur, the following reactions cause the formation of corrosion products (5):

\[
Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2 \quad (3.3)
\]

\[
4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \quad (3.4)
\]

\[
2Fe(OH)_3 \rightarrow Fe_2O_3 \cdot H_2O + 2H_2O \quad (3.5)
\]

As mentioned previously, these corrosion products occupy a great deal more space than the original iron atom.

**Effect of Corrosion on Bond Strength**

The bond between concrete and steel reinforcement is of paramount importance to the strength of reinforced concrete. Because concrete is so weak in tension, as compared to its strength in compression, steel is used as reinforcement. The steel takes the tensile forces in a RC beam after the concrete has cracked. However it is the bond between the steel and the concrete that makes this composite action possible. The corrosion of steel reinforcement affects this bond strength.

Bond strength between steel and concrete can be divided into two parts: adhesion, and mechanical (6). Adhesion itself comes from three different sources. First is the chemical adhesion between concrete and the steel. The second is the frictional
force between the concrete and steel. The last is the confining pressure exerted by the concrete on the reinforcing steel. The mechanical aspect comes from the ribs on the rebar. These ribs interlock with the surrounding concrete and resist any translational motion. In deformed bars, the mechanical action of the ribs is the dominant contributing factor. In smooth bars, there is very little mechanical action due to the absence of ribs. Therefore it is adhesion that provides the majority of bond strength.

The post-casting corrosion of steel rebar affects all of these different sources of bond strength. The chemical adhesion is destroyed because the steel and concrete become separated by a layer of corrosion products and are no longer bonded together. Frictional forces are lost because the corrosion products separating the steel and the concrete act as a lubricant.

At first, the confining pressure actually increases with the addition of corrosion products. The molecules of the corrosion products occupy more space than the original iron atoms. As the corrosion products build up, they apply a pressure force to both the concrete and the steel. This pressure continues to build up until the concrete cracks in tension. After this occurs, the confining pressure produced by the intact concrete is lost.

The mechanical bond between the steel and concrete is also affected by the buildup of corrosion products. The ribs on deformed bars have a much higher surface area to volume ratio than the rest of the rebar. Because the corrosion occurs only at the surface of the steel, it has a greater impact on the ribs, causing them to deteriorate
more rapidly. The effect of this is that the bearing area between the remaining rib and the surrounding concrete is reduced. This causes a deterioration of the bond strength.

When all of these different changes in bond strength are combined, the result is that the bond strength changes as a function of the amount of corrosion occurring. At first, the bond strength increases as the corrosion products begin to build up. The increase in confining pressure outweighs the decreases from the other sources. Eventually, the bond strength peaks, and then begins to decrease rapidly. Many bond strength vs. corrosion level curves in literature follow this trend of first increasing, then decreasing, and finally decreasing rapidly (7)(8)(9)(10)(11)(12)(13).
Effects of Corrosion on RC Beams

Corrosion has numerous effects on reinforced concrete beams. Initially, these effects can have little impact or can even be helpful, but as corrosion damage increases, the effects become detrimental to the health, capacity, and serviceability of the structure.

As part of the corrosion process, iron atoms react to form corrosion products. This removes those iron atoms from the reinforcing steel. One result of this is a loss of cross sectional area in the reinforcing steel. All post-cracking analysis of concrete members incorporates the cross sectional area of the reinforcement, and therefore all of these calculations are affected by corrosion damage. Mass loss is directly proportional to loss in radius squared. Assuming the volume of a length of rebar is cylindrical:

\[ \rho = \frac{m}{V} = \frac{m}{\pi r^2 L} \]  \hspace{1cm} (3.6)

\[ m = (\rho \pi L) r^2 \]

\[ m \propto r^2 \]  \hspace{1cm} (3.7)

Therefore, it takes about 2.7% corrosion damage to reduce the area of a #5 bar to a #4 bar. However, this assumes a uniform corrosion layer on a cylindrical surface. In reality, corrosion damage is almost never completely uniform. The effect of corrosion damage can be greater at some areas of reinforcement than others. In the example above, there could be some sections of reinforcement that retains a full cross sectional area, while other sections would be reduced below a #4 size. Additionally, there are ribs...
on any deformed rebar, so this complicates matters further as corrosion damage is based on surface area.

As discussed in section 3.3, bond strength is affected by corrosion damage. This can affect different derived properties of the concrete beam. Development length is based on the bond strength between the steel and the concrete. If that bond strength were to degrade, a longer development length would be needed to ensure full development of tensile forces in the reinforcing steel. Also because of this, the ultimate capacity of the beam is affected by the change in bond strength due to corrosion damage.

One study investigated the relationship between the thickness of corrosion products and the ultimate beam capacity (14). The study used two sets of beams, one that had been cracked prior to accelerated corrosion, and one that was not. Both sets of beams experienced a similar reduction in strength.

The ductility of reinforcing steel is also affected by corrosion damage (15). A loss in ductility of steel is cause for concern, as steel ductility adds an additional warning sign that a concrete member is about to fail. If the ductility of steel reinforcement is reduced, there will also be reduced warning of an imminent failure.

It is also possible for the failure mode to change with corrosion damage. One paper found that the failure mode can change from bending failure to shear failure for the same beam design at different corrosion levels (16).
Effects of Sustained and Cyclical Loading on Corrosion in Beams

Much of the research on beam corrosion does not take into account the service conditions of a real structure. A beam in service almost always experiences some constant dead load as well as a varying live load. Some research has been conducted to try and take these conditions into account.

The results indicate that loading conditions have a significant effect on the rate and severity of corrosion in concrete beams. The specimens studied were 10 cm x 17 cm x 117 cm and used a single length of #6 rebar. In one set of tests, beams in different starting conditions and loading conditions were tested (17). One set of beams was preloaded, using four-point bending, and corroded in an unloaded condition. A greater amount of preloading cause a larger degree of corrosion to occur in the same amount of time under accelerated corrosion. For a beam preloaded to 45% of ultimate load, 3% corrosion was observed. A higher preloading condition of 75% of ultimate load showed 4% corrosion.

A second set of beams was subjected to sustained loading while the accelerated corrosion process was occurring. In this case, the amount of corrosion difference increased to the following levels: 45% of ultimate load produced 4% corrosion and 75% of ultimate load showed 6% corrosion. Finally, a third series of beams was tested under three different sustained loading conditions. The final results for this series was as follows: 0% sustained loading produced 1.5% corrosion, 20% sustained loading
produced 3.5% corrosion, and 60% sustained loading produced 5% corrosion over the same period of time.

It was also noted in the same study that larger amounts of deflection occurred in the samples subjected to larger amounts of sustained loading during corrosion. Also, the mode of failure changed from shear failure in lower corrosion specimens, to bond splitting failure in the higher corrosion specimens.

Another paper focused on the increase in deflection (18). This research also performed accelerated corrosion testing under sustained loading conditions. It was found that for a 23% of ultimate load, deflection increased by between 18% and 36%. For 34% of ultimate load, the deflection increased by between 50% and 83%. Much of the increase occurred early in the sustained loading process, at low levels of corrosion.

In summary, the effects of sustained loading while corrosion is occurring cause a magnification of the effects of corrosion on the capacity and serviceability of RC beams. Deflection, rate of corrosion, and rate of crack growth all increase when a beam subjected to sustained loading experiences corrosion, as compared to an unloaded beam. One additional paper showed conclusions similar to the above conclusions (19). Also, a critical review of the topic can be found in reference (20).

Cyclical loading has also been studied (21). The results indicate that bond strength decreased under cyclical loading for deformed and smooth reinforcing steel in both confined and unconfined conditions. Larger deflections also produced a greater loss of bond strength. When comparing deformed to smooth bars, the smooth bars
experienced a larger loss of bond strength than the deformed bars under cyclical loading.

Properties of Polypropylene Fiber

Polypropylene is an artificially made material that has a variety of uses. Uses include: carpeting, rope, textiles, and automobile interiors (22). It is a hydrocarbon that is produced under specific conditions of heat and pressure. The typical structure of polypropylene is a long chain of individual molecules (Figure 3.6.1).

\[
\text{--- CH}_2 \text{--- CH} \text{--- CH}_2 \text{--- CH} \text{--- CH}_2 \text{--- CH} \text{--- }
\]
\[
\text{ | } \text{ | } \text{ | }
\]
\[
\text{CH}_3 \quad \text{CH}_3 \quad \text{CH}_3
\]

Figure 3.3 - Typical polypropylene chain structure (23)

Polypropylene fibers are small hair-like fibers that are added to the concrete during mixing (see Figure 1). Some physical properties of polypropylene fibers are as follows (23):

- Tensile Strength – 65 – 110 ksi
- Young’s Modulus – 500 – 725 ksi (Varies w/ Diameter of Fiber)
- Thermal Conductivity – 0.95 Btu-in/ft²•hr•°F
- Coefficient of Linear Thermal Expansion – 4.0E-5 /°F
- Specific Gravity – 0.9
Melting Point – 165 °C

Additionally, polypropylene fibers have excellent chemical resistance, and specifically chemicals that will not attack concrete, will also not attack the polypropylene fibers (24). Polypropylene is also hydrophobic material. This property prevents adhesion to the concrete. Instead, polypropylene is anchored mechanically in the concrete and bond strength is achieved through a combination of mechanical interlocking and friction.

Figure 3.4 Bundles of polypropylene fibers. Each bundle of fibers breaks up during mixing into hair-like fibers. Individual fibers are 2-3” in length.
Polypropylene Concrete

Polypropylene can be added to reinforced concrete to affect various properties. It is used to increase impact resistance by increasing the toughness and energy absorption of the concrete (25). It can also be used to reduce the amount of shrinkage cracking and early age cracking of concrete (25)(26).

Polypropylene has this effect on concrete because it acts as a small tension element within the concrete. Once the concrete begins to crack, the polypropylene fiber crossing the crack will engage and absorb the tensile force. This mirrors how steel reinforces concrete in bending, only on a much smaller scale.

The addition of fiber to reinforced concrete has a variety of effects. The strength of the concrete increases in bending, tension, and compression (25)(27)(28). It should be noted that a volume fraction of 0.1% or greater is required to observe these increases (25). Bond strength is increased after cracking in the concrete has occurred (13) (29). Also, it has been demonstrated that both salt water and sea water have a beneficial effect to the bond strength between polypropylene and concrete (30). It has been shown that the addition of polypropylene fibers in concrete will increase the chloride permeability (31)(32). It is important to note that this increase is seemingly detrimental to the corrosion resistance of the concrete.
Properties of Basalt Fiber

Basalt fibers are a relatively new material, as compared to polypropylene fibers. Unlike polypropylene, which is an artificially created material, basalt fibers are made by melting basalt, then extruding it into fibers. These basalt fibers can then be formed into various shapes and forms in the same manner as carbon fiber, glass fiber, and other traditional fiber reinforcement materials. Basalt fibers are also less expensive than carbon and other fibers.

Additionally, basalt fibers are resistant to corrosion, radiation, UV light, acids, and changes in temperature. Some physical properties of basalt fibers are as follows (33):

- Tensile Strength – 360 ksi
- Modulus of Elasticity – 12900 ksi
- Specific Gravity – 2.6
- Rupture Strain – 3.15%

The basalt fibers used in this project are considerably thicker and more rigid than the hair-like polypropylene fibers. As such, it is possible to use a higher volume of basalt fibers than polypropylene fibers because they do not become tangled and clumped during the mixing process. Typical length of a single fiber is 1.5” to 2”
Transport Properties of Concrete

The first stage of the corrosion process of concrete is the transport of undesirable ions from the external environment to the steel reinforcement. It is important to have an understanding of how this process occurs.

Transport of Water

Water has been assumed to be transported through concrete by capillary action. However, because of the non-homogeneity of concrete, the standard equation for this process is modified empirically using the following equation (34):

\[
\frac{W}{A} = C \left[ 1 - \exp \left( -\frac{S t^2}{C} \right) \right] + S_g t^2 + S_o
\]  

(3.9)

Where;
W = Volume of Water Absorbed
A = Surface Area
C = Constant
S = Sorptivity Coefficient
S_g = Sorptivity of smaller pores or moisture diffusion – Note: S_g << S
S_o = Initial Condition – Over long time periods, negligible. This value is often assumed to be 0.
t = Time

The following table shows some empirically gathered values for C, S, and S_g for different materials and curing conditions (34):

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Curing Age (days)</th>
<th>C [cm]</th>
<th>S [cm/SQRT(d)]</th>
<th>S_g [cm/SQRT(d)]</th>
<th>S_o [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM Mortar Bench Dried</td>
<td>1</td>
<td>0.295</td>
<td>0.4</td>
<td>0.02</td>
<td>0</td>
</tr>
<tr>
<td>ASTM Mortar Oven Dried</td>
<td>1</td>
<td>1.2</td>
<td>1.2</td>
<td>0.015</td>
<td>0</td>
</tr>
<tr>
<td>HPM Oven Dried</td>
<td>1</td>
<td>0.295</td>
<td>0.65</td>
<td>0.221</td>
<td>0</td>
</tr>
<tr>
<td>Conventional Concrete</td>
<td>28</td>
<td>0.35</td>
<td>0.25</td>
<td>0.009</td>
<td>0</td>
</tr>
<tr>
<td>Bench Dried</td>
<td>28</td>
<td>0.165</td>
<td>0.12</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>

Applying these values for conventional concrete:

$$\frac{W}{A} = 0.35 \left[ 1 - \exp \left( \frac{-0.25t^2}{0.35} \right) \right] + 0.009t^2 = 0.35 \left[ 1 - \exp \left( -0.714t^2 \right) \right] + 0.009t^2$$

(3.10)
Fick’s Second Law

The transport of various ions through concrete is often described by Fick’s Second Law of Diffusion:

\[
\frac{\partial C}{\partial t} = D_{\text{eff}} \frac{\partial^2 C}{\partial x^2}
\]  (3.11)

Where;
C=Concentration of ions
x=Position
t=Time
\(D_{\text{eff}}\)=Effective Diffusion Coefficient.

Using the boundary conditions, \(C=C_0\) @ \(x=0\), \(t=0\) and \(C=0\) @ \(x=\text{infinity}, t>0\), the solution becomes:

\[
\frac{C(x,t)}{C_0} = 1 - \text{erfin} \left( \frac{x}{\sqrt{4D_{\text{eff}}t}} \right)
\]  (3.12)

Where;
\(C_0\)=Initial concentration

Each ion transported through concrete will have a different diffusion coefficient, \(D_{\text{eff}}\).

Transport of Chloride

Diffusion appears to be the primary factor for chloride penetration, rather than adsorption (35). The diffusion of chloride ions in concrete is normally described by
Fick’s law of diffusion. This law does not take into account the effects of chemical binding, ionic interaction, lagging motion of cations, and formation of electrical double layer on the solid surface (36).

Chloride binding affects the chloride diffusion coefficient. Various theoretical curves can be used to calculate the binding of chloride in concrete. The chloride diffusion coefficient can be modified to include chloride binding by using the following equations:

Langmuir isotherm:

\[ D^*_e = \frac{D_c}{1 + \frac{\alpha}{\omega_e (1 + \beta C_f) \alpha}} \]  \hspace{1cm} (3.13)

Freundlich isotherm:

\[ D^*_e = \frac{D_c}{1 + \frac{\alpha \beta C_f \omega_e}{\omega_e}} \]  \hspace{1cm} (3.14)

\( \alpha = 1.67 \) for pastes with w/c = 0.5
\( \beta = 4.08 \) for pastes with w/c = 0.5
\( C_f = \) Concentration of free chlorides
\( \omega_e = \) evaporable water content = \( (m^3 \text{ evaporable water}/m^3 \text{ concrete}) \)

The Freundlich isotherm provides a better fit at higher \( C_f \) values (37).
Transport of oxygen

Potentiostatic testing shows that oxygen diffusion is only the limiting factor when the steel in concrete during long term wetting. This occurs when the oxygen at the steel surface has been exhausted. Full immersion prevents oxygen from reaching the steel, so corrosion rate is negligible. Short term wetting can accelerate corrosion due to a change in the electrolytic configuration (38). Concrete cover thickness does not affect the corrosion rate because of a reduction in the oxygen diffusion rate for non-water saturated concrete. The concrete cover is needed for other reasons (39). One model is based on w/c ratio, concrete cover thickness, chloride concentration, moisture variation. Results indicate that oxygen is the limiting factor only if concrete is submerged, or in high humidity with a dense concrete cover and using low w/c ratio concrete (40).

Transport of carbon dioxide

This proposed equation for carbon depth is based on the measured strain in the tensile steel of a concrete beam. Data was obtained through strain gauges on two 15 year old beams stored in a loaded condition and protected from the environment (rain and chloride) (41).

\[
CD(\alpha_S) = CD_o \left[ 1 + \alpha \left( \frac{h^* \alpha_S}{d} \right)^3 \right]
\]

(3.15)
CD = Carbon Depth (mm)
\( \alpha = \) Emperical Constant = 4E-8 MPa\(^{-3}\) for \( \sigma_s \leq 250\) MPa

h = height of concrete beam (mm)
d = effective depth (mm)
\( \sigma_s = \) Stress in tensile steel (MPa)

One complex numerical method for determining carbonation depth based on temperature, moisture, and degree of carbonation is provided. The type of concrete, Portland cement or fly ash, is also taken into consideration (42).

Factors influencing transport: Cement Particle Size/Mixture Proportioning

Fine cements exhibit a larger early age strength, which in turn allows an accelerated construction process. The use of finely ground cement can lead to an increase in early age cracking. Because the particles are more finely ground, the rate of reaction increases. This increase in reaction speed also generates additional heat. The use of a more coarsely ground cement can help to prevent the problem of early age cracking (43).

Additionally, limestone filler can be used to replace a portion of the cement. The addition of this limestone filler has an impact on the autogenous shrinkage, and therefore the early age cracking of the concrete (44) (45).

The amounts of fine and coarse cements used in concrete can be engineered to allow some control over these properties. Properties such as isothermal heat release,
semi-adiabatic temperature rise, autogenous deformation, setting time, and compressive strength can be influenced by the blending of coarse and fine cement (46).

Shrinkage Reducing Admixtures (SRA)

These admixtures operate primarily by reducing the surface tension in the pore solution of the concrete. This has the potential to reduce autogenous shrinkage. This reduction comes as a result of reduced capillary pressures (47).

In addition to the reduction in surface tension, SRAs have other effects. One of these is an increase in the final viscosity of the pores of the concrete. This increase in viscosity helps to reduce the ingress of detrimental ions, such as chloride or sulfate (48).

High Volume Fly Ash

High volume fly ash concretes refer to those concrete mixes that replace 50% or greater of the Portland cement with fly ash. This fly ash is obtained as a byproduct of burning coal. Though it is normally considered a waste material, it can be effectively used in concrete as a binding agent, like Portland cement. The use of fly ash is one way that concrete can be considered more sustainable.
The main drawback to using fly ash is that it produces a significant delay in the setting time of the concrete. There has been some research into the possibility of mitigating this delay using powdered additions to the mixture (49).

VERDiCT/FLAIR

The acronym VERDiCT (Viscosity Enhancers Reducing Diffusion in Concrete Technology) is used to represent the idea that an increase in the viscosity of the pores of the concrete can decrease the amount of ion diffusion in the concrete. Specifically, the goal is to double the service life of concrete by doubling the viscosity of the pores (50).

Internal curing is used as a delivery mechanism. The acronym FLAIR (Fine Lightweight Aggregates as Internal Reservoirs) is used to describe this process. This has been shown to be more effective than the simple addition of viscosity enhancers directly to the concrete mixture (50).

Internal Curing

Internal curing can be used to reduce the autogenous shrinkage of early age concrete, and therefore reduce early age cracking (51). Different materials can be used to provide the internal curing. Most commonly used is some form of light weight
aggregate (LWA). The porosity of this aggregate is used to store water that can then be used to hydrate the concrete during curing. Other materials that can be used are super absorbent polymers (SAP) (51) (52), crushed returned concrete aggregates (53), or wood fibers. Additional studies have been performed with the goal of studying how internal curing affects the concrete over its lifetime (54).

Early Age Cracking

A good overview of the concept, causes, and mitigation strategies can be found in the 2008 paper: REACT-Reduce Early Age Cracking Today (55). The content of this paper is summarized in the following paragraphs. The three primary causes of early age cracking presented are: plastic shrinkage, autogenous shrinkage, and thermal deformation. Plastic shrinkage is caused by an excessive evaporation rate at the surface of curing concrete. Autogenous shrinkage occurs due to different factors. First, internal drying within the pores of the concrete causes internal stresses to develop. If these stresses are large enough, cracking can result. Second, the hydration reaction of the concrete causes a reduction of volume which in turn causes stress. The thermal deformation is caused by the variance of temperature during curing. If this variance is large, the concrete can crack.

Although many types of concrete can experience early age cracking, high performance concrete is particularly susceptible for the following reasons. Generally,
the strength of concrete can be influenced by the water-to-cement (w/c) ratio of the concrete. The lower this ratio is, the higher the strength of the concrete. There is a limit of about 0.3 beyond which the reaction of water with cement will no longer occur satisfactorily. With high performance concrete, one goal is to use as little water as possible. Because there is so little water in the mixture, autogenous shrinkage becomes much more of a problem.

Another method of increasing the strength of concrete is to grind the Portland cement very fine. This both increases the overall strength of the concrete, and accelerates the reaction that gives the concrete its strength. This acceleration makes the concrete stronger at an earlier time so that construction can continue on at a faster pace. Unfortunately, because the concrete cures more rapidly, the heat generated by the reaction is much higher. This causes a larger temperature variance which causes an increase in thermal deformation. This leads to an increase of early age cracking.

Three mitigation strategies are outlined in this paper. The first is using shrinkage-reducing admixtures (SRAs) as an additive to the concrete. This type of admixture reduces the surface tension of the water as well as increasing the viscosity of the pores. Additionally, they reduce the drying rates. The reduction in surface tension decreases the stresses experienced by the concrete.

Internal curing is the second mitigation strategy presented. The concept of internal curing is that an additional source of water is provided to prevent the pores of the concrete from drying out. This provides a more even reaction and helps to reduce the internal pore pressure, which in turn helps to prevent cracking. Different materials
can be used to introduce this internal water source. Light weight aggregates (LWA) can be saturated with water, and then added to the concrete mixture. Other materials such as wood or synthetic fibers can also be used.

Finally, mix proportioning is recommended. Several individual properties can be tweaked in addition to the inclusion of SRAs and internal curing. An increase in the w/c ratio will reduce early age cracking by providing additional water. Using a coarser cement will reduce the early age strength of the material, but will also reduce early age cracking. Coarse limestone powder can replace a portion of the cement. Also, fly ash can be used to slow the rate of hydration and the generation of heat.

Computer Models for Transport Properties

Some computer models have been designed for the purposes of simulating how corrosion affects concrete. Transport properties, environments, types of concrete, and different mathematical models have been incorporated into these different models.

TransChlor

A model called TransChlor was created to model the transportation of chloride ions through concrete. The input variables are the microclimate (function of the structural element’s exposure to solar radiation i.e. shade vs. direct sun), annual CO2
concentration, and weather conditions (Air temp, Humidity, precipitation], concrete cover permeability, and presence of de-icing salts (56).

DuCOM

One complex model developed is DuCOM. It takes into account various factors and has a framework as follows (57):
3D Numerical Chemo-Hygro-Thermo-Mechanical Model

This model applies the effects of the following processes – Transport of Capillary Water, Transport of Oxygen and Chloride, Immobilization of Chloride in Concrete,
Transport of OH- ions through electrolyte in concrete pores, Cathodic and Anodic Polarization, and Temperature. The final goal of this model is to simulate the corrosion of the reinforcement for any geometry and boundary conditions. The model presented transport of capillary water, oxygen, and chloride are influenced by the mechanical damage caused by cracking (58).

STADIUM

STADIUM is a commercially available software package that incorporates various models for the transportation of ions through concrete, as well as material properties of custom mix designs. This software calculates the time until corrosion initiation for a given situation. Local weather conditions, and salt spray history are taken into account, along with cover depth, mix design, and type of reinforcing steel.

This software package says nothing about what happens after corrosion initiation at this time. Once the chloride threshold for the given reinforcement steel has been exceeded, corrosion is said to have initiated. Cracking, and loading conditions are not taken into account.

Rapid chloride permeability test

This test was preceded by AASHTO T259 Test, or Ponding Test. The ponding test is conducted over a time period of 90 days and is preceded by a 28 day curing period for
the concrete under investigation. A 3% NaCl solution is allowed to pond on top of the concrete and after 90 days, the samples of the concrete are taken at different levels. These samples are tested for chlorine content.

Due to the amount of time required to complete this test, a less time consuming one was designed. Rather than 90 days, the rapid chloride permeability test (AASHTO T277 Test) can be completed in 6 hours.

![Figure 3.7 Rapid Chloride Permeability Test setup and output criteria.](image)

<table>
<thead>
<tr>
<th>Charge Passed (coulombs)</th>
<th>Chloride Ion Penetration</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;4000</td>
<td>High</td>
</tr>
<tr>
<td>2000-4000</td>
<td>Moderate</td>
</tr>
<tr>
<td>1000-2000</td>
<td>Low</td>
</tr>
<tr>
<td>100-1000</td>
<td>Very Low</td>
</tr>
<tr>
<td>&lt;100</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

During this test, the flow of ions is measured and reported. The main problem with this test is that it measures all ions, rather than just those of chloride (59)(60). Additionally, the results are compared to an imprecise scale. This test, however, is still used due to the ease of testing. Additionally, the voltage required to perform the test causes the concrete temperature to rise. This is especially a problem with low strength concretes as it can cause cracking that allows the flow of ions to accelerate (59).
Rapid migration test

The rapid migration test is similar to the rapid chloride permeability test in that it applies a voltage to cause chloride to migrate through concrete (59). Unlike the rapid chloride permeability test, it does not measure the flow of ions. Rather, once the sample has been tested for some length of time, it is removed and cut open. Then a silver nitrate compound is used to determine the chloride penetration depth. The test apparatus is pictured in Figure 3.8.

The silver nitrate reacts with the chloride to produce a white color, while in the absence of chloride it reacts to form a brown color. The depth of penetration is then used to determine the diffusion coefficient, $D$, according to the following equation:

$$D = \frac{RT}{zFE} \frac{x_f}{t}$$

(3.16)
PERMIT ion migration test

The PERMIT test was designed to be an in-situ testing of the chloride diffusion coefficient(61). The concrete is pre-saturated with a conditioning solution. After 24 hours, the PERMIT test is conducted. The testing apparatus (shown below) is applied to the prepared concrete. Chloride ions are allowed to flow through the concrete from one section of the apparatus to the other.

Figure 3.9 PERMIT test apparatus (61)
CHAPTER IV

EXPERIMENTAL PROCEDURE

Formwork

Steel box molds of dimensions 6”x6”x24” were modified for use as formwork in this project. Two wooden 3”x3” ends were used to reduce the overall dimensions of the beam to 3”x6”x18” long. Holes in the wooden ends allowed a single length of rebar to be suspended at a central location in the concrete beam. For series 2, this hole was moved downwards such that the rebar would be positioned with a 1” concrete cover. For series 3 through 5, there was no hole in the end caps because it was determined that the rebar should not be exposed to external environments. For these series, the reinforcing steel was either hung from thread, or rebar chairs were used. Form oil was used to cover all surfaces of the formwork. Special care was taken to prevent form oil from contacting the reinforcing steel as this would be detrimental to the strength of the beam due to loss of bond strength that would be unrelated to corrosion damage. If any form oil was accidently applied to the reinforcing steel, that steel was removed and cleaned. 6 4”x8” plastic cylindrical molds were also used. Form oil was used with the cylindrical molds as well. Images of the molds can be found in figures 4.1.1.
Rebar Preparation

Grade 60 reinforcing steel was purchased with the stipulation that the rebar be as free from corrosion as was possible. From the purchased lengths, the cleanest surfaces were selected for use in testing. Any corrosion present was cleaned using a steel wire brush attachment to a Dremel rotary tool. Care was taken to minimize loss of metal mass from the rebar. Afterwards, the steel was cleaned using a CSM-2 Degreaser.

![Figure 4.1 Formwork for Beams – Empty and poured concrete.](image)

Mix Design

Series numbers 1, 2, and 3 were designed at a 0.40 water to cement (w/c) ratio. Series numbers 4 and 5 were designed at a 0.45 w/c ratio to allow polypropylene to be incorporated. If a 0.40 w/c ratio was used, the mixture would have been too dry to ensure an even distribution of polypropylene fibers. Some super plasticizer and water reducing admixtures were also used to improve the workability of the concrete.
Mix Procedure

A butter mix of 10% of the final mix was used. The first stage of the mix was the coarse aggregate, the fine aggregate, and 2/3 of the water. Any super plasticizer and water reducing admixtures were mixed with the water prior to addition to the mix. First stage was mixed for one minute. The second stage was the Portland cement and the remaining water. This stage was mixed for three minutes. The mixture was allowed to rest for an additional three minutes, and then mixed again for two minutes.

Pouring

After the formwork was prepared and the rebar in position, the concrete was poured in two layers. Vibration was used to ensure proper consolidation. In addition to the beams, 6 cylinders were cast for the purposes of compression strength testing. After pouring, the specimens were covered and allowed to cure.

Curing

Beams were allowed to set for one day before removal from the formwork. Afterwards, the beams were stored in a humid room for a minimum of 7 days. Exposed portions of steel were taped off during this time to prevent premature corrosion from taking place.
3.5% NaCl Solution

The sodium chloride solution used in experiments was made by mixing research grade sodium chloride solution and water. The proportions mixed were 20 L of water with 700g of NaCl. This produces a solution that is meant to simulate the amount of chlorine available in a marine environment. Values for the amount of sodium chloride vary throughout the literature from 3% to 5% (70).

Soaking in NaCl Solution

Each beam was soaked in NaCl solution to ensure saturation of chloride ions prior to the process of accelerated corrosion. It was determined that three days would be sufficient for this to occur.

Accelerated Corrosion

Because it would take years for noticeable corrosion to occur naturally, corrosion was accelerated for testing purposes. Faraday’s Law was used to determine the amount
of metal loss resulting from a constant current over a controlled time period. The derivation is as follows:

Faraday’s law states that:

\[ \Delta w = \frac{AIt}{ZF} \]  

(4.1)

The amount of metal lost can also be shown as:

\[ \Delta w = a\delta \gamma \]  

(4.2)

Setting (2) = (1) and solving for \( \delta/t \):

\[ \frac{\delta}{t} = \frac{AI}{ZF\gamma} \]

Substituting \( i \) for \( I/a \):

\[ \frac{\delta}{t} = \frac{Ai}{ZF\gamma} \]  

(4.3)

We have defined \( R \) in units of cm/year. Substituting all the known constants and conversion factors from seconds to year:

\[
R \left( \text{cm/year} \right) = \frac{\delta}{t} = \frac{56g \times i \times 1yr \times 365 \text{day} \times 24 \text{hr} \times 60 \text{min} \times 60 \text{s}}{2 \times 96500 \text{amp} \times s \times 7.86g} = 1164 \times i \]  

(4.4)

Percent of corrosion is defined as:

\[
\% = \frac{R \left( \text{cm/year} \right) \times T \left( \text{yr} \right)}{\delta \left( \text{cm} \right)} 
\]

(4.5)

Material loss (\( \delta \)) is defined such that:
\[ \delta = \frac{\Delta D}{2} \]  

(4.6)

(4.6) into (4.5):

\[ \% = \frac{2RT}{D} \]  

(4.7)

(4.4) into (4.6):

\[ \% = \frac{2 \times 1164 \times i \times T}{D} \]  

(4.8)

Where \( i \) is in amp/cm\(^2\), \( T \) is in years and \( D \) is in cm.

Using this equation, the \% of corrosion can be determined using a constant current density, and a given value for time. To apply this equation for purposes of experimentation, the values for \“\%\” and \“\( i \)\” are determined and the amount of time to run the experiment is solved for.

It has been verified that the calculated amount of corrosion using Faraday’s law is in good agreement with the actual amount of corrosion produced for mass losses from 4-7.27\% (71). This paper also goes on to say that the crack pattern does not change with higher values of current density.

Corrosion tanks were used for the process of accelerating the corrosion of the steel rebar. A Tekpower 3645A was used as a power supply. It was decided to use a current of 0.21 amps for these experiments. Using this value, the percentage of corrosion was approximately 1\% per day of current.
These corrosion tanks were set up using the schematic shown in Figure 4.2. The tank itself was made of plastic. The counter electrode used was stainless steel. 3.5% NaCl solution was used as the electrolyte for the corrosion cell. Insulated copper wire was used to connect the positive terminal of the power supply to the reinforcing steel, while a stainless steel wire was used to connect the negative terminal to the counter electrode. An image of the corrosion tank can be found in Figure 4.3.

![Figure 4.2– Schematic of Corrosion Tank](image)

![Figure 4.3 Image of Corrosion Tank](image)

**Power Supply Testing**

In order to use Faraday’s law as derived above, it was necessary to ensure that the current delivered by the power supplies was constant. Using the software provided
by the power supply manufacturer, both the voltage and current were tracked with time for the first set of samples. After verification that the power supplies could provide constant current to the samples, the experiments were continued.

Corrosion Cell

Four items are required for the formation of a corrosion cell: an anode, a cathode, an electrolyte, and an ionic path. In the accelerated corrosion process used in this set of experiments, the steel rebar is the anode and the stainless steel plate is the cathode. The NaCl solution acts as the electrolyte, allowing electrons to flow from the anode to the cathode. The ionic path flows through the power supply and the connecting wires. A complete electric circuit is required for electrons to flow. The rate of corrosion is accelerated by forcing more electrons to flow by the use of a power supply.

Naturally occurring corrosion is slightly different. In a naturally occurring RC member, different locations on the steel reinforcing bar acts as both the anode, and the cathode. These locations vary based on the amount of energy needed for an iron atom to reduce. The steel rebar also acts as the ionic path between the different anodic and cathodic locations. The electrolyte is the surrounding concrete.
Crack Width Measurement

Maximum crack widths were measured for series four through six. An optical measuring device was used to measure the crack widths at various locations along the bottom of the beam, post corrosion. The largest crack width found was recorded. These crack widths could then be compared between the three series.

Concrete Compressive Strength

Five to six, 4” x 8” cylinders were cast at the same time as the beams. The number depended on the amount of excess concrete available after the beams were cast. Each set of cylinders was tested for compressive strength at the same time as the beams were tested for bending strength.

A Forney MN-FX-500 was used to apply axial compressive load on the cylinders until failure. The load rate was kept at 36,000 ± 4,000 lb/min. This value prevented additional compressive strength from developing due to an excessively large loading rate, while allowing the compressive tests to progress in a timely manner.

Four Point Bending Test

After the accelerated corrosion process, the beams were allowed to dry for several days before the next portion of the testing process. The beams were next
subjected to a standard four point bending test. The purpose of this was to determine the maximum moment capacity of the beam. The moment capacity for initial cracking was also tracked during this testing process.

A 55 kip capacity MTS system was used to perform the testing. The amount of load was tracked, as well as the beam deflection. The span length used was 12.375” and the distance between the load points was 4”. Figure 4.4 shows this setup.

Normally, four point bending tests are conducted using strain gauges attached to the reinforcing steel. Because of how concrete cracks, it is impossible to attach the strain gauges to the concrete directly. Should a crack open at the strain gauge location, the gauge would not produce any meaningful data.

Such strain gauges would be impossible to use in these experiments because of the accelerated corrosion process. Two problems exist to prevent traditional gauges from working. First, corrosion products would destroy the bond between the gauge and the reinforcing steel, especially in the higher percentages of corrosion. Secondly, the
accelerated corrosion process involves current passing through the reinforcing steel. This current could damage any strain gauges attached to the steel.

![Contrasting Dots used for DIC](image)

Figure 4.5  Contrasting Dots used for DIC

For these reasons, it was decided to use Digital Image Correlation (DIC). This process uses a pair of imaging cameras to track the presence of high contrasting dots that are painted on the surface to be studied. How these dots move relative to one another is tracked and recorded. These movements are then processed by computer and a variety of data can be gathered.

Because of how the data is recorded, it is possible to go back and set up strain gauges at any location on the beam after the fact. This allows a greater freedom of location selection than traditional strain gauges provide. Other data can be tracked as well, including stresses, deflections, and crack widths.
Figure 4.6 Digital image correlation setup. MTS system (left), DIC cameras and lighting (middle on tripods), computer data recorder (right).

Figure 4.7 Sample photo of series one and two beams.
Series 1

The first series was used as a proof of concept series. Six beams were created and corroded at 0%, 1%, 2%, 3%, 4%, and 6%. These values were chosen to get a broad range of values, while still maintaining a reasonable time frame in which to conduct the tests. A 10% corrosion specimen, for example, would require an accelerated corrosion process of over 10 days. The mix design can be seen in table 4.1.

<table>
<thead>
<tr>
<th>Material</th>
<th>lb/yd$^3$</th>
<th>lb/batch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>675</td>
<td>50</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1424</td>
<td>105.5</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1424</td>
<td>105.5</td>
</tr>
<tr>
<td>Water</td>
<td>275</td>
<td>20.4</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td></td>
<td>40 ml</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>40 ml</td>
</tr>
</tbody>
</table>

Series 2

The second series was created with the reinforcing steel at one inch from the surface, rather than centered in the beam. This configuration would better simulate the concept of the beam representing a section of a one way slab. This series was abandoned after the discovery of the end effects seen in series 1. The mix design for this series can be seen in table 4.2.
Series 3

Series 3 used a single piece of rebar suspended using thread to avoid any of the rebar being exposed to external conditions. Because the entire rebar was embedded in the concrete, a copper wire was used to allow current to be passed through the rebar. The copper wire was stripped at one end and wrapped around the reinforcing steel, then twisted tightly to hold it in place. It proved quite difficult to position the rebar correctly because the rebar tended to shift up and down and from side to side while suspended by the thread. Normal concrete was used, with the mix design found in table 4.3.

<table>
<thead>
<tr>
<th>Material</th>
<th>lb/yd$^3$</th>
<th>lb/batch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>675</td>
<td>55</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1424</td>
<td>116</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1424</td>
<td>116</td>
</tr>
<tr>
<td>Water</td>
<td>275</td>
<td>22.4</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td></td>
<td>40 ml</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>40 ml</td>
</tr>
</tbody>
</table>

Table 4.3 Series 3 Mix Design.
Series 4

Series 4 used rebar chairs to position the concrete. These plastic chairs were cut so that the center of the rebar would be one inch from the bottom surface of the concrete. Tape was used to hold the chairs in place during pouring. Like with series 3, copper wire was embedded in the concrete to provide electrical current during the accelerated corrosion process.

Polypropylene fiber was used in this series so that it could be compared to the normal concrete in series 5. The mix design can be found in table 4.4.

Table 4.4 Series 4 Mix Design.

<table>
<thead>
<tr>
<th>Material</th>
<th>lb/yd$^3$</th>
<th>lb/batch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>675</td>
<td>55.0</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1424</td>
<td>116.0</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1424</td>
<td>1160</td>
</tr>
<tr>
<td>Water</td>
<td>275</td>
<td>24.7</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td></td>
<td>40 ml</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>0 ml</td>
</tr>
<tr>
<td>Polypropylene Fiber</td>
<td>7.5</td>
<td>0.61 (277g)</td>
</tr>
</tbody>
</table>
Series 5

Series 5 was identical to series 4, except that no polypropylene fibers were used.

The mix design can be found in table 4.5.

Table 4.5 Series 5 Mix Design.

<table>
<thead>
<tr>
<th>Material</th>
<th>lb/yd$^3$</th>
<th>lb/batch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>675</td>
<td>55.0</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1424</td>
<td>116.0</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1424</td>
<td>116.0</td>
</tr>
<tr>
<td>Water</td>
<td>275</td>
<td>24.7</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td></td>
<td>40 ml</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>0 ml</td>
</tr>
</tbody>
</table>

Series 6

Series 6 was identical to series 4, except that basalt fibers were used instead of polypropylene fibers.

Table 4.6– Series 6 Mix Design.

<table>
<thead>
<tr>
<th>Material</th>
<th>lb/yd$^3$</th>
<th>lb/batch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>675</td>
<td>55.0</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1424</td>
<td>116.0</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1424</td>
<td>116.0</td>
</tr>
<tr>
<td>Water</td>
<td>275</td>
<td>24.7</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td></td>
<td>40 ml</td>
</tr>
<tr>
<td>Water Reducer</td>
<td></td>
<td>0 ml</td>
</tr>
<tr>
<td>Polypropylene Fiber</td>
<td></td>
<td>4.94 (2241 g)</td>
</tr>
</tbody>
</table>
CHAPTER V
EXPERIMENTAL RESULTS

Power Supply Testing

Verification that the power supplies could provide a constant current was needed in order to accurately use Faraday’s Law to control the amount of corrosion in each sample. The power supplies came with computer interface software that allowed the tracking of voltage, current, and wattage drawn by the power supply. Initially, the power supplies provided a non-constant current, as can be seen in figure 5.1. It was still possible to predict the amount of corrosion by solving for the area under the curve and using that in the calculations. Note that each unit of time is 5 seconds and represents an individual reading recorded by the power supply.
Figure 5.1 Sample output of current vs. time and voltage vs. time for Series 1 beams.

After contacting the manufacturer, the power supplies were adjusted and the current tracked again. This time, it was shown that the current was constant for the sample (figure 5.2). The abandoned series 2 beams were used to test the new power supply settings provided by the manufacturer. As can be seen from the series 1 beams, the current and the voltage were both somewhat constant. In the series 2 beams, the
current was held constant while the voltage varied significantly as the integrity of the concrete was compromised by expanding corrosion products.

![Current vs. Time Graph]

**Series 2 - Beam 4 - Current vs. Time**

![Voltage vs. Time Graph]

**Voltage vs. Time**

**Figure 5.2** Sample output of current vs. time and voltage vs. time for Series 2 beams.

Once it was determined that the power supplies would keep the current constant, the current was monitored, but not recorded as the voltage values were not needed for this project.
Series 1

After the accelerated corrosion process, the bottom surfaces of the beams were photographed to show the evolution and distribution of corrosion damage. As can be seen in Figure 5.3, discoloration and damage from corrosion becomes more severe as the amount of corrosion increases.
Figure 5.3 Series 1 Beams – Underside Photos
End Effects

Most of the corrosion in Series one occurred in the exposed portion of the reinforcing steel. Figures 5.4 and 5.5 show this phenomenon. This phenomenon existed in each of the series one beams, though 4%, and 6% showed more corrosion along the length of the rebar.

The reason for the end effects is the variance in resistance between the steel, the concrete, and the external environment. Though the reinforcing steel never came in contact with the NaCl solution, some of the water vapor condensed on the edge of the concrete and the exposed steel. This thin layer acted as both an electrolyte for a corrosion cell, and as a short circuit between the power supply and the stainless steel cathode. In this way, the current would enter the rebar, flow along its length, then flow through the condensed water vapor, into the solution, and to the cathode. This caused the current to bypass the concrete, and only corrode the ends of the rebar.

The samples with higher percentages of corrosion still showed corrosion along the entire length of the rebar. In one case, the corrosion at the end of the rebar was so severe that the end of the rebar broke off entirely (figure 5.6). It is due to these end effects that it was decided to fully embed the reinforcing steel in the concrete from series three onwards. Series two had been already cast and the corrosion process started. Because of the end effects, it was decided to abandon the series two specimens, and the tests were halted.
Figure 5.4  Exterior View

Figure 5.5  Post Corrosion End Effects of Rebar of End Effects

Figure 5.6 Steel exposed to environment corroded so severely that the end detached from the remaining rebar.
Maximum Moment Capacity

The maximum bending moment for each beam was calculated using the following equation:

\[ M = P^*a \]  

(5.1)

This value was calculated for each point that the load was recorded. Maximum moment capacity of the beam was the highest of these calculated totals and occurred just before the beam failed.

![Four Point Bending Diagram](image)

Figure 5.7 Four Point Bending Diagram

The maximum moment capacity first increased, and then decreased as the beams became more corroded. The maximum point on the moment vs. corrosion graph was at about 2% corrosion. At 3% corrosion, the maximum moment was still greater than the non-corroded specimen (0%), though was reasonably close. After 3%, the moment capacity of the beams dropped off rapidly until at 6%, less than half the original strength remained.
The load vs. deflection curves for each beam can also be reported in figure 5.9. The spikes in the early portions of the curve coincide with the appearance of cracks. Initial cracking and any secondary cracking can be easily seen.
One measure of toughness in beams is the area under the load vs. deflection curve. A best fit line was calculated for each load-deflection curve, and then integrated over the range of the deflection. These values for toughness can be found in Table 5.1. It can be seen in Figure 5.10, the toughness curve is similar to the moment capacity curve in shape.

Table 5.1 Series 1 Toughness.

<table>
<thead>
<tr>
<th>Corrosion Percentage</th>
<th>Area Under Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1560</td>
</tr>
<tr>
<td>1</td>
<td>2300</td>
</tr>
<tr>
<td>2</td>
<td>2080</td>
</tr>
<tr>
<td>3</td>
<td>1590</td>
</tr>
<tr>
<td>4</td>
<td>653</td>
</tr>
<tr>
<td>6</td>
<td>222</td>
</tr>
</tbody>
</table>
Figure 5.10  Toughness plotted vs. percentage of corrosion.

Post Breaking

After four point bending tests were completed, the remaining pieces of the concrete were analyzed. The profile of the corrosion damage was photographed, as were images of rebar.
Figure 5.11  Photographs of failed beams.
As can be seen, the majority of the corrosion damage occurred near the ends of the beams where the reinforcing steel was exposed to environment. This pattern disappeared at 6% corrosion, where the corrosion products were spread throughout the cross section. Minor corrosion damage at the ends of the 0% sample was caused by the
rebar being exposed to atmosphere. This represents another reason to ensure that the rebar is fully embedded in the concrete.

The severity of the corrosion damage progressed in a fairly steady manner from 0% to 4%, and then had a large jump at 6%. At this point, corrosion products had built up all along the interior of the beam, along a longitudinal crack.

Concrete Compressive Strength

Table 5.2 shows the individual and average compressive strength for the cylinders tested in series one.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lb)</th>
<th>Maximum Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>101170</td>
<td>8050</td>
</tr>
<tr>
<td>2</td>
<td>108930</td>
<td>8668</td>
</tr>
<tr>
<td>3</td>
<td>110020</td>
<td>8755</td>
</tr>
<tr>
<td>Average</td>
<td>106707</td>
<td>8491</td>
</tr>
</tbody>
</table>

Cracking Moment

The cracking moment of the series of beams was also tracked. The value for cracking moment was calculated by using the load at which the specimen first cracked. Recorded images from the DIC machine were used to locate the time at which the first crack occurred. Then, the amount of moment applied at that time was found using the data from the universal testing machine.
Using these values, the cracking moment can be compared for different values of corrosion. As can be seen in Figure 5.12, these values remain fairly constant until falling off rapidly at 6%. By this point, the concrete has been so damaged by the buildup of corrosion products that it cracks far more easily.

The theoretical cracking moment can be found using the following equation, where $f_c'$ is the average maximum stress from the average compressive stress of the compression cylinders:

$$M_{cr} = 7.5 \times \sqrt{f_c'}$$

(5.2)

Figure 5.12  Cracking Moment vs. % Corrosion
Series 2

Series 2 was abandoned after the end effects in series 1 were discovered and analyzed.

Series 3

After the accelerated corrosion process, the bottom surfaces of the beams were photographed to show the evolution and distribution of corrosion damage. Figure 5.13 shows these photographs. As can be noted, the end effects present in series one are not present in series three. Encasing the reinforcing steel entirely in concrete worked as intended. Note that the 4% beam seems to be less corroded than 3% beam.
Figure 5.13 Series 3 Beams – Underside Photos
Moment Capacity

Moment capacity for series three was calculated in the same way as series one. The moment capacity peaked somewhere between 1% and 2%. The value for 4% is clearly an outlier. This beam seemed considerably less corroded, both externally and internally after it was broken apart. As with the series 1 data, the moment capacity dropped below the un-corroded moment capacity at about 3%. By 6% corrosion, the moment capacity has been reduced by more than half.

![Maximum Moment vs. % Corrosion](image)

Figure 5.14 Maximum Moment vs. Corrosion Graph
The load vs. deflection curves for each beam can also be reported in Figure 5.15.

**Figure 5.15 Load vs. Deflection Curves for Series Three**

**Toughness**

Similar to series 1, the toughness was calculated by finding the area under the load-deflection curves for series 3. The results can be found in table 5.3. The toughness value for beam 3 (3% Corrosion) can be considered an outlier. In this case, there is the same pattern of increasing, then decreasing toughness.
Table 5.3  Series 3 Toughness, Series Three.

<table>
<thead>
<tr>
<th>Corrosion Percentage</th>
<th>Area Under Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>824</td>
</tr>
<tr>
<td>1</td>
<td>1450</td>
</tr>
<tr>
<td>2</td>
<td>2070</td>
</tr>
<tr>
<td>3</td>
<td>500</td>
</tr>
<tr>
<td>4</td>
<td>1690</td>
</tr>
<tr>
<td>5</td>
<td>207</td>
</tr>
<tr>
<td>6</td>
<td>180</td>
</tr>
</tbody>
</table>

Figure 5.16  Toughness plotted vs. percentage of corrosion.

Post Breaking

After four point bending tests were completed, the remaining pieces of the concrete were analyzed. The profile of the corrosion damage was photographed, as were images of rebar.
Figure 5.17  Post Breaking Images and Cross Sections
Figure 5.17 (cont) Post Breaking Images and Cross Sections
Concrete Compressive Strength – Series Three

Table 5.4 shows the individual and average compressive strength for the cylinders tested in series three. Figure 5.17 shows the failure mode of the cylinders.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lb)</th>
<th>Maximum Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>97790</td>
<td>7781</td>
</tr>
<tr>
<td>2</td>
<td>85820</td>
<td>6829</td>
</tr>
<tr>
<td>3</td>
<td>97080</td>
<td>7725</td>
</tr>
<tr>
<td>Average</td>
<td>93563</td>
<td>7445</td>
</tr>
</tbody>
</table>

Figure 5.18  Failed Compression Cylinders

Cracking Moment

The cracking moment for each beam, and the theoretical cracking moment were determined as with the beams in series one. Using these values, the cracking moment can be compared for different values of corrosion.
Figure 5.20 shows the photographs of the bottom side of the beams, after the accelerated corrosion process. The progress of the corrosion damage of the polypropylene concrete was similar to that of the normal concrete until the 6% sample.
Figure 5.20  Series Four Beams – Underside Photos
For comparison to set five, it is important to present the side view of the 6% corrosion beam. This image can be found in Figure 5.21.

Figure 5.21 Image of 6% corrosion beam, side view. Note that in the side view, the bottom of the beam is situated at the top of the photo.

Maximum Crack Width

The maximum crack widths for series four can be found in table 5.5. Figure 5.22 shows the progress in crack width as the amount of corrosion changes.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Crack Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No Crack</td>
</tr>
<tr>
<td>1</td>
<td>No Crack</td>
</tr>
<tr>
<td>2</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>0.3</td>
</tr>
<tr>
<td>5</td>
<td>0.35</td>
</tr>
<tr>
<td>6</td>
<td>0.5</td>
</tr>
</tbody>
</table>
Moment Capacity

Moment capacity for series four was calculated in the same way as series 1. With the polypropylene fibers added to the concrete mix design, the moment capacity was more consistent than with the normal concrete design. Though there was a slight decrease in maximum moment capacity as corrosion damage became more severe, the beams only lose about 25% capacity from maximum to minimum.
Load vs. Deflection

As with the maximum moment capacity for series four, the load vs. deflection curves are quite different than those in previous series. Because of the addition of the polypropylene fibers, the beams were able to deflect considerably more than the normal concrete beams. Rather than a sudden failure, the failure was more gradual. Even after failure, the beams were more intact in this series.

Figure 5.24 Load vs. Deflection Curves

Toughness

Similar to series 1, the toughness was calculated by finding the area under the load-deflection curves for series 4. The resulting toughness can be found in table 5.6.
For the polypropylene samples, it can be seen that values for toughness are considerably higher than that normal concrete presented in series 1, 3, and 5.

Table 5.6  Toughness, Series Four.

<table>
<thead>
<tr>
<th>Corrosion Percentage</th>
<th>Area Under Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3440</td>
</tr>
<tr>
<td>1</td>
<td>5060</td>
</tr>
<tr>
<td>2</td>
<td>3400</td>
</tr>
<tr>
<td>3</td>
<td>3020</td>
</tr>
<tr>
<td>4</td>
<td>2260</td>
</tr>
<tr>
<td>5</td>
<td>1930</td>
</tr>
<tr>
<td>6</td>
<td>3020</td>
</tr>
</tbody>
</table>

Figure 5.25  Toughness plotted vs. percentage of corrosion.

Post Breaking

After four point bending tests were completed, the remaining pieces of the concrete were photographed. It was not possible to get cross sectional profiles of the
cracked corrosion damage because the samples remained intact even after failure. As can be seen, the failure mode was consistent from beam to beam. Even as the percentage of corrosion increased, the beam maintained integrity, unlike with normal concrete beams.
Figure 5.26  Post Breaking Images.
Concrete Compressive Strength

Table 5.7 shows the individual and average compressive strength for the cylinders tested in series four. Figure 5.27 shows the failure mode of the cylinders. Note that unlike the samples without polypropylene fibers, these samples did not completely fall apart after failure, even though the failure mode was similar.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lb)</th>
<th>Maximum Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100230</td>
<td>7976</td>
</tr>
<tr>
<td>2</td>
<td>110810</td>
<td>8817</td>
</tr>
<tr>
<td>3</td>
<td>109580</td>
<td>8720</td>
</tr>
<tr>
<td>4</td>
<td>96580</td>
<td>7685</td>
</tr>
<tr>
<td>Average</td>
<td>104300</td>
<td>8300</td>
</tr>
</tbody>
</table>

Figure 5.27 Failed Compression Cylinders.
Cracking Moment

The cracking moment for each beam, and the theoretical cracking moment were determined as with the beams in series one. Using these values, the cracking moment can be compared for different values of corrosion.

![Graph Cracking Moment vs. % Corrosion](image)

**Figure 5.28** Graph Cracking Moment vs. % Corrosion.

**Series 5**

After the accelerated corrosion process, the bottom surfaces of the beams were photographed to show the evolution and distribution of corrosion damage. Note that the slight discoloration in the upper left quadrant of the 0% corrosion sample is not actual corrosion.
Figure 5.29  Series Five Beams – Underside Photos.
One other feature was observed in the 6% corrosion beam. This beam cracked along not only the bottom surface, but the top as well. Figure 5.30 shows this both from the side, and from the top. It is important to note this development because of how it compares to the 6% sample from series 4.

![Image of 6% corrosion beam, top view and side view. Note that in the side view, the bottom of the beam is situated at the top of the photo.]

Maximum Crack Width

The maximum crack widths for series five can be found in Table 5.8. Figure 5.31 shows the progress in crack width as the amount of corrosion changes.
Table 5.8  Maximum Crack Width, Series Five.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Crack Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No Crack</td>
</tr>
<tr>
<td>1</td>
<td>0.04</td>
</tr>
<tr>
<td>2</td>
<td>0.04</td>
</tr>
<tr>
<td>3</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>0.15</td>
</tr>
<tr>
<td>5</td>
<td>0.3</td>
</tr>
<tr>
<td>6</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Figure 5.31  Graph of change in crack width vs. corrosion percentage.

Moment Capacity

Moment capacity for series four was calculated in the same way as series 1. Without the polypropylene fibers as with series four, there was a large decrease in strength as the corrosion percentage increased. At 4% corrosion, the beam has lost about 45% of moment capacity. By 6% corrosion, the beam has lost almost all moment capacity.
The normal concrete beams used in series five had significantly smaller deflections at failure than the polypropylene concrete beams in series four. Additionally, the failures were sudden in nature.

Figure 5.32  Maximum Moment vs. Corrosion Graph.

Figure 5.33  Load vs. Deflection Curves for Series Five.
Similar to series 1, the toughness was calculated by finding the area under the load-deflection curves for series 5. The resulting toughness can be found in table 5.9. As can be seen in Figure 5.35, the values for toughness drop off rapidly after 2% corrosion.

<table>
<thead>
<tr>
<th>Corrosion Percentage</th>
<th>Area Under Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1580</td>
</tr>
<tr>
<td>1</td>
<td>1390</td>
</tr>
<tr>
<td>2</td>
<td>1850</td>
</tr>
<tr>
<td>3</td>
<td>527</td>
</tr>
<tr>
<td>4</td>
<td>396</td>
</tr>
<tr>
<td>5</td>
<td>408</td>
</tr>
<tr>
<td>6</td>
<td>48.1</td>
</tr>
</tbody>
</table>

Figure 5.34 Toughness plotted vs. percentage of corrosion.
As with series 1 and series 3, the normal concrete samples did not maintain integrity after failure.

Figure 5.35 Post Breaking Images and Cross Sections.
Figure 5.35 (cont) Post Breaking Images and Cross Sections.
Concrete Compressive Strength

Table 5.10 shows the individual and average compressive strength for the cylinders tested in series five. Figure 5.36 shows the failure mode of the cylinders.

Table 5.10  Compressive Strength, Series Five.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lb)</th>
<th>Maximum Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>77590</td>
<td>6174</td>
</tr>
<tr>
<td>2</td>
<td>75560</td>
<td>6012</td>
</tr>
<tr>
<td>3</td>
<td>74770</td>
<td>5950</td>
</tr>
<tr>
<td>4</td>
<td>79200</td>
<td>6302</td>
</tr>
<tr>
<td>Average</td>
<td>76780</td>
<td>6110</td>
</tr>
</tbody>
</table>

Figure 5.36  Failed Compression Cylinders.

Cracking Moment

The cracking moment for each beam, and the theoretical cracking moment were determined as with the beams in series one. Using these values, the cracking moment can be compared for different values of corrosion.

92
After the accelerated corrosion process, the bottom surfaces of the beams were photographed to show the evolution and distribution of corrosion damage. As can be seen, the extent of the corrosion damage appears to be less thorough in this series, than in series 4 or 5.

Figure 5.37 Graph Cracking Moment vs. % Corrosion.
Figure 5.38  Series Five Beams – Underside Photos.
As can be seen in Figure 5.39, there was no cracking along the top of the 6% beam. Additionally, only a hairline crack can be seen along the side of the beam. This contrasts with the normal concrete which cracked all the way through, and the polypropylene concrete, which cracked more than halfway through.

Figure 5.39 Image of 6% corrosion beam, top view and side view. Note that in the side view, the bottom of the beam is situated at the top of the photo.

Maximum Crack Width

The maximum crack widths for series five can be found in table 5.11. Figure 5.41 shows the progress in crack width as the amount of corrosion changes.
Table 5.11  Maximum Crack Width, Series Six.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Crack Width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No Crack</td>
</tr>
<tr>
<td>1</td>
<td>No Crack</td>
</tr>
<tr>
<td>2</td>
<td>No Crack</td>
</tr>
<tr>
<td>3</td>
<td>0.15</td>
</tr>
<tr>
<td>4</td>
<td>0.2</td>
</tr>
<tr>
<td>5</td>
<td>0.15</td>
</tr>
<tr>
<td>6</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 5.40  Graph of change in crack width vs. corrosion percentage.

Moment Capacity

Moment capacity for series six was calculated in the same way as series one. With the addition of basalt fiber to the concrete mix, there was very little loss of strength due to corrosion.
Figure 5.41 Maximum Moment vs. Corrosion Graph.

Load vs. Deflection

As with the polypropylene concrete in series four, series six showed more resistance to sudden failure. This can be seen in most of the load-deflection curves.

Figure 5.42 Load vs. Deflection Curves.
Toughness

Similar to series 1, the toughness was calculated by finding the area under the load-deflection curves for series 4. The resulting toughness can be found in table 5.12. As can be seen in Figure 5.43, the values for toughness in basalt fiber concrete were considerably more erratic than either the polypropylene or normal concrete. Some values were noticeably higher than the normal concrete, though others were lower.

Table 5.12  Toughness, Series Six.

<table>
<thead>
<tr>
<th>Corrosion Percentage</th>
<th>Area Under Curve</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3150</td>
</tr>
<tr>
<td>1</td>
<td>840</td>
</tr>
<tr>
<td>2</td>
<td>1780</td>
</tr>
<tr>
<td>3</td>
<td>3200</td>
</tr>
<tr>
<td>4</td>
<td>870</td>
</tr>
<tr>
<td>5</td>
<td>2410</td>
</tr>
<tr>
<td>6</td>
<td>1120</td>
</tr>
</tbody>
</table>

Figure 5.43  Toughness plotted vs. percentage of corrosion.
After four point bending tests were completed, the remaining pieces of the concrete were photographed. As with the polypropylene samples, the basalt fiber samples remained intact after failure. The polypropylene samples retained slightly more structural integrity than the basalt fiber samples.
Figure 5.44 Post Breaking Images.
Concrete Compressive Strength

Table 5.13 shows the individual and average compressive strength for the cylinders tested in series four. Note that like the polypropylene samples, these samples did not completely fall apart after failure, even though the failure mode was similar.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Maximum Load (lb)</th>
<th>Maximum Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>88400</td>
<td>7034</td>
</tr>
<tr>
<td>2</td>
<td>92010</td>
<td>7321</td>
</tr>
<tr>
<td>3</td>
<td>84160</td>
<td>6697</td>
</tr>
<tr>
<td>Average</td>
<td>88190</td>
<td>7017</td>
</tr>
</tbody>
</table>

Cracking Moment – Series Six

The cracking moment for each beam, and the theoretical cracking moment were determined as with the beams in series one. Using these values, the cracking moment can be compared for different values of corrosion.
Figure 5.45  Graph Cracking Moment vs. % Corrosion.
CHAPTER VI
DISCUSSION

Corrosion of Reinforced Concrete

The effects of corrosion on reinforced concrete members, and therefore the overall structure, are detrimental to both strength and serviceability. Exactly how detrimental is difficult to say because there are a great many variables associated with the materials in question, and the environment they are exposed to. Variables might include: Member geometry, porosity of concrete, placement and size of reinforcing steel, makeup of steel, amount of concrete cover, yearly precipitation, temperature, amount of deicing salts, presence or absence of Portland cement alternatives (such as blast furnace slag), and loading conditions.

The methodology of how corrosion damages concrete is important to understand. The first stage is the transport of ions through the concrete. The ion most frequently considered is chloride, though others may also be detrimental. Once the ions in question reach the reinforcing steel, actual corrosion can begin. Corrosion is an electrochemical process that, in this case, removes iron atoms from the steel and changes them into various corrosion products (Fe(OH)$_3$ for example). These corrosion products occupy a larger volume than the original iron atom. The buildup of corrosion
products inside the concrete causes an internal pressure that puts tensile forces on the surrounding concrete. Once these tensile forces increase beyond the concrete’s ability to resist them, the concrete begins to crack. This process continues until the crack reaches the surface. Once this primary crack has formed, the chloride can reach the steel much more rapidly because it no longer needs to follow a tortuous path through the concrete. Eventually, the concrete can spall away from the structure.

Corrosion and Moment Capacity

Though there is some initial increase in the strength of a concrete member, due to corrosion, it is unwise to incorporate this strength gain into any concrete design. For this reason, any design curve for available moment capacity needs to be truncated in the early percentages of corrosion.

For the beams tested in series 1 and 3, a maximum occurs at about 1%-2% corrosion. By 3% corrosion, the amount of remaining strength is approximately the same as a non-corroded beam. After this point, there is a rapid drop off of strength until, by 6% corrosion, there is only one third of the strength remaining. Figure 6.1 illustrates how a design curve based on the results from series 1 and 3 would be created.
The exact shape of the design curve would vary significantly according to different factors. The geometry of the beam, number of rebar, their spacing, cover depth, as well as the type of concrete would all necessitate a change in the curve.

With the addition of polypropylene fibers, or basalt fibers, the shape of this curve changes dramatically. Series 4, 5, and 6 are compared in figure 6.2. Several
important details stand out when comparing normal, polypropylene, and basalt fiber concrete.

First, is that the strength of the polypropylene and fiber concrete is noticeably higher than the normal concrete. This can be attributed to the additional reinforcing strength provided by the polypropylene fibers, or the basalt fiber.

Second, the polypropylene and basalt fiber concrete show less degradation in strength. If the first point in the normal curve is treated as an outlier so that it matches the series 1 and 3 curves, the design strength for 0% to 3% corrosion would be close to 1400 k*ft. By 6% corrosion, this strength has been reduced to 330 k*ft. This represents a 76% reduction in strength. Neither the polypropylene nor basalt fiber concrete shows this drastic reduction of strength at 6% corrosion. It should also be noted that if the design strength of the concrete is 1400 k*ft, both polypropylene and basalt fiber concrete maintain this strength from 0% to 6% corrosion with the lowest value being 1460 k*ft.

Third, though the normal concrete retains the curving shape of the previous two series (1 and 3), the polypropylene and basalt fiber concrete curves do not. There is no clear peak in strength at 2% corrosion, and obvious curved shape. This suggests that with the addition of these materials, the pattern of degradation changes significantly. It could be that the degradation is linear in nature, or that higher levels of corrosion are necessary to produce the rapid drop off in strength that was seen in the normal concrete.
Corrosion and Cracking Moment

Cracking moment is also affected by corrosion damage. It was observed that at higher levels of corrosion (5% or 6%), the beams cracked significantly sooner than the lower levels of corrosion. This suggests that the deterioration of the concrete that has been caused by corrosion damage reduces the cracking moment. Figure 6.3 shows the cracking moment for the normal concrete series.
When looking at the data from the fiber concrete series (figure 6.4), this drop in cracking moment is not observed. The cracking moment for these series is more consistent and also slightly higher than normal concrete. Figure 6.5 shows the cracking moments for all series.
Corrosion damage to reinforced concrete affects the strength of the reinforced concrete, as well as its serviceability. There are additional safety concerns, even if the reinforced concrete remains structurally sound. Advanced corrosion damage can cause spalling which can lead to additional property damage, or injury to persons. The addition of fiber to the mix design of reinforced concrete can help to mitigate the harmful effects of corrosion.

To understand how the addition of fiber can mitigate these effects, the method by which corrosion damages concrete should first be presented. After the initiation of corrosion at the surface of the steel reinforcement, corrosion products begin to form. These corrosion products occupy a larger volume than the original iron atoms. As these corrosion products continue to form, an internal pressure develops, as they attempt to

Figure 6.5 Cracking moment for all series.
expand away from the concrete. At first, they can fill up some of the pores or existing cracks within the concrete. Eventually, a critical pressure value is reached, and the surrounding concrete fails in tension. This causes one or more cracks to develop in the concrete at the steel-concrete interface. This process continues as the corrosion products continue to form, causing these cracks to propagate through the concrete. Eventually, one of these cracks will reach the exterior surface of the concrete. This damage to the concrete can become extensive throughout the reinforced concrete member or structure.

The addition of fiber to the concrete mix can help to limit this damage. Each fiber can act as a small tension element within the concrete, increasing its resistance to cracking. Once the concrete has begun to crack, the fiber engages and carries the tensile forces caused by the expansion of corrosion products. The process is similar to how steel is used in reinforced concrete.

Strength Increase

The addition of fibers to reinforced concrete produces an overall minor increase in the concrete strength. This is due to the fibers acting as small, individual tension members that engage whenever cracking occurs. Cracking is expected in reinforced concrete under normal operation, even without corrosion damage. Until the concrete cracks, the reinforcing tension steel and any fiber do not contribute to the strength of the beam. After cracking, the reinforcing steel engages and carries the tensile forces in the beam. Any fiber addition to the concrete will provide additional reinforcement.
When corrosion damage is considered, there is a more pronounced and significant effect with the addition of fibers. The strength of the beam decays at a much slower rate, because the fiber is able to mitigate some of the damage caused by corrosion. Figure 6.6 shows two important changes in how the moment capacity of the beams changes with corrosion damage. First is that the shape of the curve is different when comparing the normal concrete with either of the two types of fiber. With the normal concrete, the strength first increases slightly, then decreases rapidly. The polypropylene curve (in red) does not show a noticeable peak, nor is there any rapid fall off of moment strength in the higher corrosion percentages. The basalt curve (in orange) shows very little strength loss over the corrosion percentages studied. Should a service load of 1000 k*ft be applied to the beams, the fiber concrete beams would not fail, even at a 6% corrosion level. The normal concrete beam, on the other hand, can only carry approximately 500 k*ft in moment and would fail. It is important to note that the initial strength of all of these beams are well above 1000 k*ft at 0% corrosion.
Ductility Increase and Toughness

The ductility of the reinforced concrete beams also changes with corrosion. The damage caused by corrosion can cause the beam to become less ductile. This means that the beam can fail suddenly, and without warning when the beam’s capacity is exceeded. Compare the two graphs in Figure 6.7. Even though both beams fail at a similar load, the behavior of the two beams during failure is different. On the left, a significant amount of deflection is observed before failure of the beam. On the right, the beam fails suddenly without any large increase in deflection. With the inclusion of polypropylene fiber, this sudden failure is not observed. As can be seen in figure 6.8, even at 6% corrosion, significant deflection is seen before failure of the beam.
This observation can be better observed using the values for toughness. Figure 6.8 shows the toughness values vs corrosion percentages for the three normal concrete series. There is a general trend of increasing toughness from 0% to 2%, then a trend of decreasing from 2% to 6%. This trend matches how corrosion damage affects the ultimate moment capacity.
So not only does corrosion damage reduce the ultimate moment capacity of a beam, but the beam’s ductility and toughness are also reduced. This means that the failure of corroded members could occur under lower loading conditions than expected, and that the failures could happen with less warning when compared to uncorroded members.

![Corrosion vs. Toughness](image)

**Figure 6.9** Toughness vs. corrosion percentage for normal concrete.

With the addition of polypropylene fibers (see figure 6.10), the toughness and ductility of the corroded beams increase by a factor of 2. The same increase, then decrease pattern of toughness remains, though the peak appears at 1% rather than 2%. With the addition of basalt fiber, the results are inconclusive. As can be seen in figure 6.11, sometimes the basalt fiber is generally more effective than normal concrete, though this is not entirely consistent. Further testing is recommended in this area.
Figure 6.10  Toughness vs. corrosion percentage for normal and polypropylene concrete.

Figure 6.11  Toughness vs. corrosion percentage for normal, polypropylene, and basalt concrete.
Integrity of Concrete

Corrosion damage to reinforced concrete can become so severe that pieces of concrete can spall away from the structure. Extensive cracking caused by the expansion of corrosion products can cause this spalling to occur by weakening areas of the concrete near the reinforcing steel. This spalling concrete is not only dangerous to the integrity of the structure, but also poses a safety hazard to any persons or property below the structure.

The addition of fibers to the concrete mix can help to mitigate this hazard. The fibers can hold the concrete together when corrosion products attempt to expand and crack the concrete. Figure 6.12 shows photographs of three beams to illustrate this point. The normal concrete beam began to split lengthwise at the location of the reinforcing steel. The basalt fiber concrete began to split as well, but the damage is much less severe.
Figure 6.12 Integrity Comparison for Normal vs. Fiber. Normal concrete (bottom) experiences more extensive cracking than the basalt fiber concrete (top). Note that for photos on the left, the bottom of the beam is located at the top of the photograph. Photos on the right shows the top surface of the beam.

Fibers can also help keep the concrete intact even after failure. Even after testing to failure, beams with fiber addition did not break apart. Figure 6.13 shows compression cylinders tested to failure, as well as post-failure beam photos.

Figure 6.13 Integrity Comparison for Normal vs. Fiber, Post Failure. Top photos are normal concrete, while bottom are polypropylene fiber concrete. Left two photos are compression cylinders tested to failure. Right photos are 6% corrosion sample beams tested to failure.
CHAPTER VII

CONCLUSIONS

The following observations can be made:

- For normal concrete, the moment capacity of a corroded member first increases, then decreases rapidly as the amount of corrosion increases. Moment capacity begins to decrease at about 3% corrosion by weight. Beyond this value the moment capacity of the beam falls off sharply.

- With the addition of polypropylene fibers or basalt fibers, this rapid fall off of moment capacity is not observed. Though there is still some decrease in moment capacity, the decrease is more gradual and the total reduction is smaller.

- Cracking moment can also be affected by corrosion damage. At higher levels of corrosion (5%-6%), normal concrete beams cracked at a significantly lower moment. The addition of polypropylene fibers or basalt fibers prevented this drop in cracking moment.
• The addition of polypropylene or basalt fibers helps to keep the concrete intact both during the corrosion process, and after the failure of the beam. This property of fiber concrete could help with the problem of concrete spalling.

• Corrosion damage causes concrete beams to become less ductile. Failures can happen more suddenly and with less warning because of this decrease in ductility.

• With the addition of polypropylene fibers, corroded beams are shown to have a higher toughness value and maintain more of their ductility. The results in this area for basalt fiber were mixed with some beams showing increased ductility and others showing less ductility.

Further research is needed in the following areas:

• Testing is needed for larger specimens to determine the effect of geometry, and rebar placement has on corrosion damage.

• Some of the results for basalt fiber concrete were inconclusive and varied to a greater degree when compared to the polypropylene fiber concrete. As basalt fibers are relatively new, more research is needed for this material.
• Both polypropylene fiber concrete and basalt fiber concrete should be tested at higher corrosion values to determine if they continue to mitigate corrosion damage as the corrosion levels increase past those conducted in this study.
REFERENCES


47. —. Curing with Shrinkage-Reducing Admixtures. *Concrete International*. Oct 2005.


APPENDIX

METHODS OF MONITORING CORROSION DAMAGE IN RC STRUCTURES

There are two main types of monitoring methods for corrosion in reinforced concrete. The first involves systems that are built into the structure at either the time of construction or the time at which rehabilitation work is performed. These embedded sensors can be used to track various parameters related to the corrosion of the reinforcing steel, as well as the condition of the concrete. These sensors can provide data throughout the life of the structure, allowing the development of the corrosion to be tracked over time. This tracking allows the engineer or owner to observe the rate of corrosion in addition to the current state of the structure.

The other broad category is sensors that are brought in once investigation is deemed to be needed. These handheld sensors are able to be taken to different areas of the structure as they are not built into the structure itself. This method of observation offers only a “snapshot” into the current state of the structure. As with embedded sensors, many different parameters may be measured with these handheld sensors.
Embedded Sensors

Many types of embedded sensors exist and can be incorporated into the construction of a new structure. Also, some of these sensors are appropriate for use in a rehabilitation project so that further damage can be tracked. The actual mechanism by which these sensors operate varies from type to type. Some common mechanisms are listed as follows (62):

- Open Circuit Potential (OCP) Measurements
- Linear Polarization Resistance Measurements
- Concrete Resistivity
- Chloride Content
- Temperature

Open circuit potential measures the corrosion potential of the reinforcing steel against a reference electrode. Standard reference electrodes include saturated calomel electrode, copper/copper sulfate electrode, and silver/silver chloride electrodes (63). The half cell reaction of the reinforcing steel is measured as a way to detect the presence of corrosion. ASTM C876 is the standard by which the results of open circuit potential measurements are measured. The level of corrosion is graded according to the table 3.2 (64).
Table 3.2 Corrosion Severity Based on Half-Cell Potentials.

<table>
<thead>
<tr>
<th>Open Circuit Potential</th>
<th>Corrosion Severity</th>
</tr>
</thead>
<tbody>
<tr>
<td>mV vs. SCE</td>
<td>mV vs. CSE</td>
</tr>
<tr>
<td>&lt; -426</td>
<td>&lt; -500</td>
</tr>
<tr>
<td>&lt; -276</td>
<td>&lt; -350</td>
</tr>
<tr>
<td>-126 to -275</td>
<td>-350 to -200</td>
</tr>
<tr>
<td>&gt; -125</td>
<td>&gt; -200</td>
</tr>
</tbody>
</table>

As can be seen in the table 3.2, the resulting output is a risk of corrosion, rather than a level of corrosion. It is also important to note that open circuit potential measurements can determine nothing about the rate of corrosion.

Linear polarization measurements are performed by applying a small amount of current to the reinforcing steel and studying the resulting change in potential. The corrosion current density can then be calculated according to the following equations:

\[
R_p = \frac{\Delta E}{\Delta I} \quad (3.17)
\]

\[
l_{corr} = \frac{B}{R_p} \quad (3.18)
\]

\[
i_{corr} = \frac{l_{corr}}{A} \quad (3.19)
\]

Where \( R_p \) is the polarization resistance, \( \Delta E \) is the resulting change in potential, \( \Delta I \) is the applied current, \( B \) is the Stern-Geary constant, \( A \) is the surface area of the reinforcing steel, and \( i_{corr} \) is the corrosion current density. Once the corrosion current density is
calculated, the rate of corrosion can be determined. This value is only an instantaneous measurement, however. For a true understanding of the overall rate of corrosion measurements must be taken periodically. Mote that if $\Delta E$ must fall within the range of 10-30 mV for this method to be valid (64).

Concrete resistivity is a measurement of how well the surrounding concrete conducts electricity. Either DC or AC can be used for the purpose of this measurement. As with open circuit potential, the measurement of the concrete resistivity is compared to a chart and a qualitative measurement is produced. Table 3.3 shows the most commonly referenced set of data for interpretation of the resistivity of the concrete.

Table 3.3 Probable corrosion rate based on concrete resistivity (65).

<table>
<thead>
<tr>
<th>Resistivity (kΩ cm)</th>
<th>Probable Corrosion Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 20</td>
<td>Low</td>
</tr>
<tr>
<td>10-20</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>5-10</td>
<td>High</td>
</tr>
<tr>
<td>&lt; 5</td>
<td>Very High</td>
</tr>
</tbody>
</table>
The other two parameters typically measured in embedded sensors are chloride content and temperature. The amount of chloride present in the concrete is an important indicator of the possibility of corrosion. Temperature is measured because it can affect the rate of corrosion. Other sensors that can also be included in the embedded sensor package are pH, and relative humidity sensors (66).

It is important to note that no single sensor mechanism can indicate the state of corrosion in reinforced concrete. They indicate whether corrosion is likely to occur, or not and it takes judgment to interpret the results.

Embedded sensors also have some other disadvantages. Repair or replacement of the sensor units is often impossible because an area of the structure would have to be destroyed in order to provide access. Recalibration is also impossible for the same reasons. The life of the embedded sensor must also be taken into account. Some sensors only have a life of one to two decades (66). The expected life of the structure is usually much longer than this.

External Sensors/Handheld

Many of the techniques, such as linear polarization, open circuit potential and concrete resistivity are all used externally as well as internally. One of the primary differences is that if there is no pre-existing path to the rebar, one must be made. In many of these measurements, an electrical contact must be made with the existing
reinforcing steel. This is accomplished by drilling a small hole in the concrete after the reinforcing steel has been located.

![Guard Ring Linear Polarisation Diagram](image)

Figure 3.11 Schematic of external LPR sensor (63).

Similar to linear polarization, a technique known as surface potential measurements is sometimes used. This other technique does not require a direct connection to the reinforcing steel. Two electrodes are used, with one remaining in a fixed position and the other moved along the surface of the concrete. The measurements are interpreted relative to each other. Areas with the highest positive potentials are considered anodic have the highest chance of corrosion. These lowest areas are considered cathodic (63).

Another method that has been used in-situ is known as the galvanostatic pulse technique. Like external open circuit potential measurements, galvanostatic pulse requires that current be passed through the reinforcing steel. If no access is available, a small hole must be drilled into the concrete in the area of interest. Free corrosion potential, corrosion current and concrete resistivity all factor into this measurement (67).
Electrochemical impedance spectroscopy is a more complicated method of measurement that involves using alternating current. The results of this technique can tell the operator information about the resistivity and dielectric properties of the concrete, characteristics of the passive film, and mechanisms and kinetics of chemical reactions. Though in-situ testing of this nature is possible, it is often impractical because it is very time consuming to perform (63). This technique is more useful in a laboratory environment. It is often used in determining the effectiveness of corrosion inhibiting admixtures for concrete (68)(69).

Other infrequently used methods are (63):

- Tafel Extrapolation
- Harmonic Analysis
- Noise Analysis
- Ultrasonic Pulse Velocity Technique
- X-ray, Gamma Radiography Measurement
- Infrared Thermograph Electrochemical

One problem with external methods of corrosion detection is that unlike with embedded sensors, there is no history and past performance records for the engineer or operator to consult. Sensors tend to be brought in only after a problem has been discovered. At that point, the best that can be ascertained is the present condition of the structure.
Various sensor mechanisms have been discussed, along with how they can be used to detect key factors about corrosion. These methods were broken down into the two broad categories of embedded sensors and externally applied sensors.

- Internal, embedded sensors provide readings throughout their lifetime, allowing a comprehensive, chronological progression of corrosion damage. However, due to the nature of these sensors, they cannot be repaired, recalibrated, or replaced without great effort. These sensors may not last as long as the structure itself.

- External sensors and handheld devices offer the opportunity to make many of the same measurements that embedded sensors do, even on structures where no embedded sensors are located. However, this method of corrosion detection only involves a snapshot in time, providing information about what is happening now. It is often useful to know how the corrosion has progressed throughout time when making determinations about the structure’s health.

- The mechanisms presented only provide data which is used to make inferences about the state of the corrosion in a structure. Often, the readings given are only probabilities of encountering corrosion in the reinforcing steel. More accurate readings are difficult or impossible to get with current technology. A cheap, effective sensor that requires little
maintenance would be beneficial in reducing the overall cost of corrosion.

Further research is needed in this area.