Chapter 1

INTRODUCTION

1.1 PROBLEM STATEMENT

A number of concrete bridges in Wisconsin and elsewhere have shown signs of deterioration due to aging, corrosion, and other detrimental factors. Although bridges are generally expected to yield a service life of 50 to 100 years, some bridges are exhibiting signs of distress at a much younger age. Increased traffic requirements, the use of deicing salts, and lack of adequate preventive maintenance programs contribute to deterioration of existing bridges. Considering the enormous cost and effort required to remedy bridge deficiencies, it is crucial that a concerted effort be made to identify practical, effective and economical methods for repair and rehabilitation of bridges. This research project addresses repair and rehabilitation techniques for reinforced and prestressed concrete bridges, focusing primarily on corrosion of prestressed concrete beam-ends.

1.2 BACKGROUND AND SIGNIFICANCE OF WORK

The most prevalent cause of deterioration in concrete bridges is corrosion. Diffusion of chloride ions through concrete can destroy the passivity of steel and initiate the corrosion process. In northern climates, such as Wisconsin, the primary source of chlorides is found in deicing salts used to melt snow in the winter. Bridges in northern climates are susceptible to corrosion in a manner different from bridges located in warm coastal climates. In such climates, girders are prone to corrosion mainly at the end regions (Figures 1and 2). This is the result of water leaking through faulty expansion joints and then reaching the girder ends. Improper drainage can also allow salt water to penetrate into other parts of the superstructure, including fascia girders. The resulting steel corrosion and the spalling of concrete, can cause irreversible damage to beam-ends. Rehabilitation of damaged beam-ends generally requires the complete removal of the damaged region, followed by reconstruction. Issues with this repair procedure include reoccurring spalls due to inadequate bond between the new and existing concrete. In addition, these types of repairs may not be very effective in the long term as contaminants in areas adjacent to the repair can, overtime, migrate to the repair region. This effect is more pronounced if drainage issues are not corrected. Since this type of damage is frequently encountered in northern states such as Wisconsin, the effectiveness of various traditional and state-of-the-art repair techniques are investigated.

Figure 1. Damaged Beam-Ends

Figure 2. Close-up of Beam End

1.3 OBJECTIVES

The primary objectives of this research were: (1) to collect and synthesize information on repair and rehabilitation methods for concrete bridges (2) to evaluate the effectiveness of preventive and corrective methods to address deterioration of prestressed bridge beam-ends and (3) to initiate development of an expert system software program to assist in the assessment, diagnosis, and repair of concrete bridges.

1.4 SCOPE OF WORK AND STUDY APPROACH

The scope of this research included: (1) a thorough literature review of concrete bridge rehabilitation techniques, (2) evaluation and testing of a number of preventive and repair regimes, (3) development of a basic form of an expert system software program and (4) preparation of a final report.

A thorough understanding of the state-of-the-art in the field of rehabilitation of concrete bridges, especially in northern climates, was considered crucial for the success of this effort. Therefore, a comprehensive review of available literature in relevant subject areas was performed. On-line sources of information, as well as conventional search databases were utilized.

An extensive literature database was developed using Microsoft® Access. Over 570 papers were catalogued. They include such searchable information as the title, publisher, author, and date. The database also includes the abstracts or summaries of many of the papers. The user can search the database by performing a keyword, title, or author query.

Based on the results of the literature review, a test plan and repair concept were submitted and approved by the Project Oversight Committee, appointed by the project's sponsor, the Wisconsin Department of Transportation (WisDOT). The work plan included performing laboratory tests on five new 8-foot long prestressed concrete bridge I-beams to address corrosion-damage and subsequent repair of beams ends due to chloride-laden water infiltrating through faulty expansion joints.

The beam-ends were subjected to wet/dry cycles of salt laden water to accelerate the corrosion process. In addition to the salt-water exposure, the beam-ends were subjected to an impressed electric current to assist in accelerated corrosion. Two “cathode” bars were placed in the beam and the entire reinforcement system (strands and bars) was made anodic. This creates a “reverse cathodic protection” system, thus accelerating corrosion. Some end regions were pretreated with a sealer, epoxy coating, polymer (resin), or fiber-reinforced polymer (FRP) composite wrap to assess their effectiveness in protecting the beam when subjected to an accelerated corrosive environment.

Several repair schemes were also implemented to evaluate their effectiveness in reducing corrosion and preventing further damage. As was done initially, sealer, epoxy coating, polymer (resin), and FRP wrap treatments were also applied after an initial exposure period of 6 months. In addition, one beam-end was patch repaired with no additional protection system to compare its performance with other systems. After the repairs were completed and the surface treatments applied, the beam-ends were again subjected to an accelerated corrosion regime. Finally, the protection systems were evaluated to determine which system(s) provided corrosion mitigation and the best corrosion protection.

An initial version of an expert system computer program, Concrete Bridge Assessment and Rehabilitation (ConBAR), was developed to assist in the diagnosis of concrete bridge deterioration problems and to identify repair, rehabilitation, or preventative maintenance options. This program includes a user-friendly interface that obtains relevant information on the subject bridge through a series of questions, and provides suggestions and recommendations to the user. The depth and variety of questions that ConBAR asks the user before making recommendations far exceed the scope of previous attempts at developing such expert system tools for concrete bridges. This necessitates a very large set of expert rules (based on combinations of possible answers) that must be incorporated into the program. This program currently includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program. It is important to emphasize that the tools developed are intended and expected to assist and facilitate the work of experienced maintenance personnel, and not to replace it.

Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

Deterioration of bridge superstructure and substructure elements is a common problem in the United States. A large number of bridges in the United States were built after the Second World War. Some of these bridges were not designed to withstand the current environmental and traffic requirements, and consequently are experiencing significant distress. Deterioration in bridges can take several forms and stem from various causes. Among the causes are corrosion, structural damage from vehicle impact, and deficiencies in the original design and construction. Methods of repair are numerous and they range from simple spot patching to more complex repair regimes. Since a significant number of bridges are considered deficient or obsolete, economical ways must be found to improve the infrastructure condition. Although complete rebuilding is sometimes deemed necessary, repair and rehabilitation can be far more economical when the methods are effective.

Corrosion of concrete bridge elements is a significant and costly concern due to the possibility of premature deterioration. "The annual direct cost of corrosion of highway bridges is estimated to be between $6.43 billion and $10.15 billion. Life-cycle analysis estimates the indirect costs to the user, due to traffic delays and lost productivity, at more than 10 times the direct cost of corrosion" [54]. Therefore, current rehabilitation methods must be evaluated to determine their performance and cost effectiveness. A number of studies have concluded that traditional repair schemes (i.e. concrete patching) lack longevity and are susceptible to continued deterioration. State-of-the-art materials and procedures (i.e. fiber reinforced composites) have shown in some studies to be an effective alternative for repairing corrosion-damaged concrete. This literature review briefly summarizes traditional and state-of-the-art procedures used to repair corrosion damaged bridge elements.

2.2 CORROSION MECHANISMS

Concrete is normally durable in moist, oxygen rich environments, but steel can be unstable under these conditions. Concrete provides a protective environment to embedded steel by supplying a physical barrier to the ingress of deleterious substances as well as a chemical protective shield. If the physical integrity of the concrete is altered, the protective capability of the concrete barrier is reduced. Protection is also provided to the reinforcing steel by the high alkalinity of the surrounding concrete. The high pH (12 to 13) of the pore water in concrete provides a natural passive chemical environment for reinforcing steel. As concrete ages, environmental exposure can lead to the breakdown of the passive layer. Corrosion would occur if the passive layer is destroyed and sufficient amounts of oxygen and moisture are present. Presence of chloride ions or carbonation can damage the passive layer and accelerate the corrosion process significantly.

The corrosion process is electrochemical in nature. It is driven by the appearance of cathodic and anodic regions on the metal surface (see Figure 3). This can be attributed to different chemical concentrations or the varying availability of oxygen or moisture at different locations along reinforcing bars. At the anode site iron is dissociated to form ferrous ions and electrons. The electrons migrate toward the cathodic site where the ferrous ions dissolve in the concrete pore solution. At the cathodic site, oxygen in the pore solution combines with the electrons to form hydroxyl ions. The ferrous and hydroxyl ions move in opposite directions through the pore solution, when they combine, ferrous hydroxide is precipitated. The precipitated corrosion products occupy a larger volume than the non-corroded steel. As the concentration of corrosion products increase, an increasing pressure is exerted on the concrete until it cracks and eventually spalls [26].

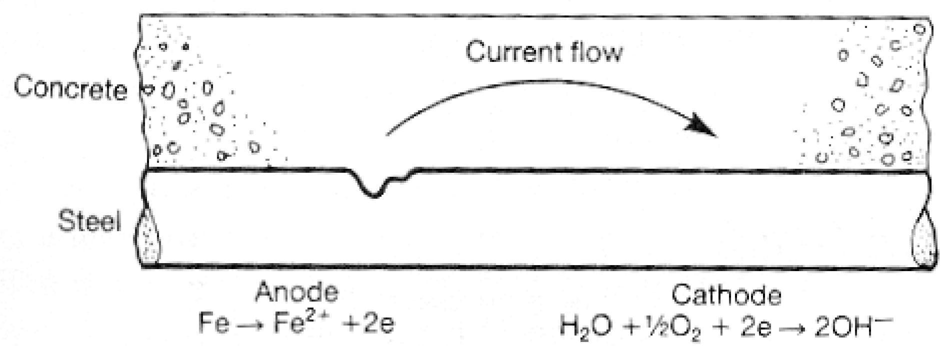

Figure 3. Schematic of Electrochemical Corrosion Process [31]

Pitting corrosion may result if the chloride concentration is highly localized. During pitting corrosion small pits, or holes, form on the steel surface. The volume of corrosion products may be insufficient to cause surface cracking, but it is possible that severe loss of steel cross-section may occur with very little prior warning from visible indications [7].

Stress corrosion can occur in prestressing steel. It is a highly localized corrosion that can lead to cracking of the prestressing steel due to the high stress levels present. The formation of a micro pit occurs in the tendon, and the tip of the pit is subjected to highly concentrated stress. The micro-pit is also undergoing dissolution as the active anode in a localized corrosion process. The combination of stress and rapid corrosion can initiate a crack that propagates rapidly leading to brittle fracture of the tendon [7].

2.3 DEICING SALTS & AND CORROSION DAMAGE

Corrosion can affect every element of a concrete bridge. In coastal regions, most bridge elements are affected more or less evenly by general exposure to chloride-laden air. Bridges over seawater are more affected on their undersides (e.g. deck bottom). Bridges in northern climates are affected differently. Bridge deck reinforcements are susceptible to corrosion because deicing salts are applied on the roadway surface in the winter. Girders experience corrosion mainly at their end regions due to salt-water leaking through failed expansion joints. Piers can be affected along much of their height when they are exposed to salt spray from vehicles, including snowplows, which travel under the bridge. Pier caps and pedestals are also affected by salt-water intrusion through leaky expansion joints.

The use of deicing salts in northern climates is not likely to be discontinued. The use of deicing salts has actually increased in the 1990's after a period of leveling off in the 1980's [54]. Some examples of road salt alternatives include calcium magnesium acetate (CMA) and potassium acetate (KA) [27]. While both CMA and KA appear to be viable road salt alternatives, the high cost of the material and equipment and facility modifications prevent widespread acceptance of these materials.

CMA acts more slowly and is less effective than salt in cold conditions. In general, nearly all studies of CMA rated the substance as an acceptable deicer but not as effective or consistent as salt when applied in equal amounts [27]. In 1991, the National Research Council (NRC) of Canada examined CMA as an alternative to road salt in deicing operations [27]. The study concluded that CMA is relatively harmless to plants and animals, noncorrosive to metals, and nondestructive to concrete and other highway materials. Because of its low density and small particle size, CMA may be dusty during handling and storage and may blow off roadways after application. In addition, when exposed to moisture, CMA can clog spreading equipment. The calculated ratio of CMA to salt for comparable ice melting is 1.7 to 1 [27]. A study conducted by the Wisconsin Department of Transportation in 1987 [27] reported application rates of CMA 1.2-1.6 times greater than salt. The average 1991 cost of salt was approximately $30/ton; whereas the cost of CMA was estimated to be between $500 and $700/ton [27]. Conversion to CMA would also incur additional costs associated with the modification of storage, handling, equipment and spreading operations.

Potassium acetate (KA) is often used as a base for commercial chloride-free liquid deicer formulations [27]. Its advantages include low corrosion, relatively high performance, and low environmental impact. Less research has been conducted on the application and effectiveness of KA. However, some studies have concluded that the substance has minimal impacts on human health and the groundwater supply [27]. The average 1991 cost of KA was $700-$800/ton [27].

The consequences of reinforcement corrosion include cracking and spalling of the concrete. Spalling concrete can be a safety issue for vehicles passing nearby as well as permitting or accelerating further deterioration. Spalled concrete also allows chloride-laden water to reach the reinforcement resulting in more corrosion. Also, as the reinforcement corrodes, the effective cross-sectional area of the steel is reduced, resulting in a decrease of structural strength. Therefore, the overall strength and stiffness of the bridge element is reduced.

2.4 CORROSION REPAIR METHODS

Traditional methods of repairing concrete bridges with corroded reinforcement fall into two general categories: (1) non-electrical (conventional) methods and (2) electrical methods. Conventional methods include patching, sealers and coatings, overlays, or combinations of these. Surface treatments, such as sealers, coatings and overlays, prevent the passage of potentially deleterious substances and subsequently may slow the deterioration process. The primarily employed electrical method is cathodic protection. Cathodic protection can reduce corrosion rates if the corroding element can be shifted to a cathodic condition through addition of a sacrificial anode with or without an externally applied potential (i.e. impressed current). An additional electrical method, akin to cathodic protection, is chloride extraction. The process involves the application of an external current (much higher than in cathodic protection), which causes the chloride ions to move away from the reinforcement.

2.4.1 CONVENTIONAL NON-ELECTRICAL METHODS

Conventional repair methods are classified into the following broad categories: (1) patches, (2) overlays, (3) sealers and coatings, and (4) crack injection. Each of these methods is employed to repair damaged concrete and to protect from further corrosion damage.

2.4.1.1 Patching

Patching involves removing the concrete area around the damaged region, typically with a chipping hammer, jackhammer or by water blasting. Any exposed reinforcement is cleaned and possibly treated with a corrosion inhibitor. The patch material is then placed inside formwork or by troweling [15]. Table 1 summarizes the expected life and costs associated with the two common patching options [54].

Table 1. Cost (adjusted to 1998) and Life Expectancy for Patching Options [46, 54]

Type of Maintenance	Average Cost ($\$/m^2$)	Range of Costs ($\$/m^2$)	Average Expected Life (years)	Range of Expected Life (years)
Bituminous Concrete Patch	$90	$39 to $141	1	1 to 3
Portland Cement Concrete Patch	$395	$322 to $469	7	4 to 10

Commonly used classes of patch materials include: (1) Portland-cement concrete, (2) hydraulic cement concrete [40], (3) polymer based (e.g. epoxy) patches, and (4) bituminous concrete patches. Portland-cement patches are the most commonly used, and construction workers are typically familiar with the installation techniques. Hydraulic (fast-setting) cement concrete materials are similar to regular concrete. They are generally self-leveling, do not require mechanical vibration, and are more stable at higher temperatures than cementitious materials. Polyurethanes and epoxies are relatively new patch materials. Proportioning and mixing are critical to material performance. Also, because of their relatively low viscosity, they are more difficult to place on vertical surfaces. Bituminous (or asphaltic) patches are commonly used to provide temporary riding surfaces on bridge decks in a rapid manner [17]. However, these patches have shown to have a service life of only 1 year, and should therefore be replaced with more durable patch material [36].

Patch repairs sometimes require partial or complete disruption of traffic because of the need for shoring the member under repair, partial removal of the contaminated concrete from damaged regions, cleaning of the corroded reinforcement, and placing the patch material. This is a labor-intensive process, yet some argue that patch repairs are not durable [39]. Patch treatments can mend spalls, but typically do not retard chloride-induced corrosion. In such cases, this type of repair will typically fail prematurely since no measures are taken to mitigate the primary source of deterioration. In addition, since the newly placed concrete consists of minimal to no concentration of chlorides, a reverse chloride gradient is created between the patch repair and the existing concrete [39]. The result is the failure of the patch and a need for subsequent repairs. This method, however, is generally successful when the source of damage is related to accidental or load-induced causes. The life of most patch repairs is limited to a maximum of 10 to 15 years.

A study conducted by Patel et al [40] as part of a 1990 Strategic Highway Research Program project (SHRP H-106, Innovative Materials Development and Testing) evaluated the performance of six rapid setting concretes, a polymer modified concrete, a polyurethane, an epoxy, an epoxy-urethane, and two bituminous cold mixes used for repairing partial depth spalls. In conjunction with the evaluation of the repair materials, five patching procedures varying in the methods of concrete removal and surface preparation were studied. These repair methods included saw and patch, mill and patch, waterblast and patch, jackhammer and patch, and adverse condition clean and patch. The "adverse conditions" involved installing the material when the temperature was below 40 °F and lightly spraying the concrete substrate with water.

The study concluded that many of the patch materials were sensitive to water content and placement temperatures. Installation directions must be followed carefully, and appropriate product precautions, such as using ice water, placing at night, and storing the material in the shade, should be followed when extremely hot temperatures are encountered during placement. Care and understanding must be maintained to complete the repair. Proper surface preparation was crucial in all cases. In addition, the study found that a carbide tipped milling machine might be economical for removing deteriorated concrete when a large area requires repair. However, proper alignment of the milling head required considerable time and additional labor. This method might also pose a traffic hazard due to encroachment into the adjacent traffic lane. The study also found that the high-pressure (30,000 psi) water-blasting machine was not effective. Many equipment failures and an extremely slow concrete removal rate were observed in this particular study.

The patch materials were installed in 1991 from April to July at four test sites in four climatic regions in the United States. In addition, laboratory tests were conducted to identify correlations with field performance. The materials were evaluated on a periodic basis. The study reports findings 1 to 3 days after installation as well as one and three month's results. Figure 4 illustrates the repair performance after three months of evaluation. The percent of patches indicate the fraction of patches experiencing transverse cracking, wear, debonding, or failure. The author considered a patch repair failed when it could no longer carry traffic safely.

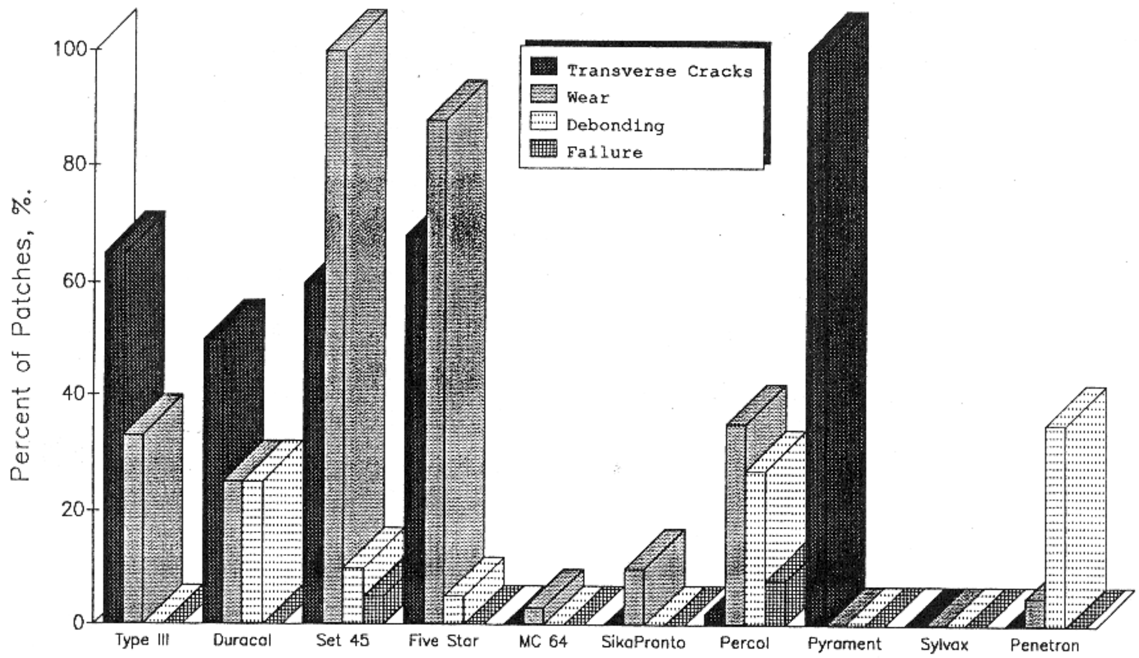

Figure 4. Distress Summary [1]

The report summarized the following observations about the repair materials:

- *Type III cement concrete* is a common patch material and hence the construction crews employed in that study were familiar with the placing, compacting, finishing, and curing techniques. The mix was found workable when the air temperatures were below 80° F, but the mix was found stiff and difficult to work at high temperatures.

- *Pyrament 505* (hydraulic cement concrete) is similar to install and finish as regular concrete and was found to be easy to mix, place, and finish under ambient temperatures. This product took more time for mixing than the other cementitious products evaluated and its workability under cold and wet conditions was more difficult.

- *Percol FL* (polyurethane) required proper equipment and a qualified technician. It was considered critical that clean, oven-dried aggregate be used with this product. Even a small amount of dust or moisture may cause poor bonding or bubbling. This product reportedly had low viscosity and therefore was difficult to place on slopes and grades.

- *MC 64* (epoxy) patches were relatively unfamiliar to the construction workers employed in the study. The proportioning and mixing was considered critical to material performance. Both the mix and finishing required a lot of personnel and time. This product also had low viscosity and was reportedly difficult to place on slopes and grades.

Research conducted for the Federal Highway Administration (FHWA) [52] included a laboratory study into the corrosion of prestressed concrete highway bridge elements and conventional repair methods used for these structures. Test specimens were precorroded through application of anodic current while exposed to chloride solutions. In order to study conventional concrete repairs, it was necessary to remove concrete from preselected areas and replace the chloride contaminated/deteriorated concrete with repair materials. Materials evaluated included conventional portland cement concrete, latex-modified fiber-reinforced patching mortar, and silica fume concrete containing either organic or inorganic corrosion inhibitors.

All specimens were exposed for approximately 200 weeks to a 15% solution of sodium chloride after repair. In many of the specimens, significant deterioration of the coatings applied to the prestressing strands and reinforcing steel had occurred over the four years of severe exposure. The distress was greater for the steel coated with liquid epoxy coating than the steel coated with a zinc-rich product. Typically, there was more disruption of coating and corrosion of base steel in regions where latex-modified mortar had been used as repair material than where conventional concrete or silica fume concretes were used. Corrosion was observed in repair areas where chloride contents were below commonly accepted threshold levels. When tendon bundles were cut and pulled apart, corrosion was observed on the interior surfaces of the individual strands, indicating that chloride ions had penetrated prior to the repairs. These ions then become available to foster additional corrosion over time after repairs have been applied. At repair area edges, steel coating failure and corrosion were, in general, greater than in the bulk of the repair, and testing demonstrated that chloride ions moved laterally into the concrete, raising the concentration at patch edges. Although only two specimens were available for comparison, an inorganic corrosion inhibitor appeared to be more effective in reducing the extent of corrosion than an organic-based product.

The study concluded that the patch repair systems did not offer long-term protection to rehabilitated prestressed concrete members. Even in low permeability patches (such as those based on silica fume concrete) chloride ions may penetrate vertically from the surface of the members and laterally from the adjoining un-repaired concrete. Field applied steel coatings lost their effectiveness overtime and deteriorated, exposing the underlying steel to corrosive agents. The study recommended that periodic repair and reapplication of protective systems might be necessary to maintain structures. Where periodic repairs are difficult to carry out, the study suggests complete replacement of distressed members may be a long-term cost-effective alternative.

A study conducted in 1993 by Sprinkel et al [45] for the Strategic Highway Research Program evaluated various rapid repair methods including the performance characteristics of some patch repair materials. The report states that patching methods can mend corrosion-induced spalls, but typically do not retard chloride-induced corrosion because not all concrete containing chlorides is removed. The research determined that the corrosion rates are high at the perimeter of the patch and is independent of the type of patch material used.

2.4.1.2 Overlays

Bridge deck overlays are primarily used to improve durability and service life of bridge decks. They can restore the quality of the deck surface and increase the effective cover for the reinforcement. An overlay must have a long-term stable bond with the repaired deck and sufficient resistance to environmental conditions such as vehicle traffic and chloride-laden water. Overlays are most effective when used in conjunction with a system that protects against further corrosion, such as corrosion inhibitors or cathodic protection. However, overlays do not address the presence of chlorides. Overlays typically extend the life of a deck 6 to 10 years [22].

A study conducted by the Michigan State University researched factors affecting the service life of corrosion damaged reinforced concrete bridge superstructure elements [36]. The study concluded that the amount and degree of contaminated concrete left in place influence the effectiveness of an overlay. Since the extent of surface damage primarily influences the decision to overlay a bridge deck, the amount of contaminated concrete left in place is similar for various environmental exposure conditions. Thus, if the original base concrete is not replaced or rehabilitated, the service life of overlays can be similar for all environmental exposure conditions.

Overlay permeability is an important material characteristic. High permeability allows moisture to penetrate through the overlay and into the concrete below. Permeability depends on the porosity of the overlay and the presence of cracks. Therefore, overlay cracking should be minimized, whether from shrinkage, thermal stresses, or fatigue, to prevent deterioration of the overlay. Considering the type and thickness of the material can lessen overlay cracking.

Table 2 summarizes the service life and costs associated with different overlay and patching options [54].

Table 2. Cost (adjusted to 1998) and Life Expectancy for Overlay Options [46, 54]

Type of Maintenance	Average Cost ($/m^2)	Range of Costs ($/m^2)	Average Time until Maintenance (years)	Average Expected Life (years)	Range of Expected Life (years)	Typical Thickness (in)
Portland Cement Concrete Overlay*	$170	$151 to $187	8.3	18.5	14 to 23	≥1.3-2.0
Bituminous Concrete with Membrane	$58	$30 to $86	5.1	10	4.5 to 15	≥1.6
Polymer Overlay**	$98	$14 to $182	6.4	10	6 to 25	≥0.3-0.5

*Includes latex-modified concrete (LMC).
**Polymer Overlays include: epoxy, epoxy urethane, methacrylate, polyester styrene, & polyurethane

Three common overlays include: latex-modified concrete (LMC), low-slump dense concrete (LSDC), and silica-fume concrete (SFC). Traditionally, LMC overlays constitute more than 90% of the overlays used for rehabilitation applications [54]. The LMC and SFC overlays are generally less permeable than dense concrete and are stronger, allowing a reduction in required thickness. However, increased thickness can be an advantage in protecting the underlying deck if the overlay cracks.

Detwiler et al [13] reported on the overlay of the IL 4 Bridge over Interstate 55 near Staunton, IL. In October 1986, the southbound lane was repaired using a standard LSDC overlay; and the northbound lane was repaired in March 1987 using a SFC overlay. The bridge provided an opportunity to compare the performance of the overlays using the same contractor to install the repairs and be exposed to the same environmental conditions. The overlay repairs were evaluated in July 1995. According to field survey and petrographic examinations, both the LSDC and the SFC overlay repairs were originally of high quality. Both performed well under the exposure conditions. The silica fume concrete appeared to provide better protection against the ingress of chloride ions. These results were consistent with the chloride ion profiles, which generally indicated that the chloride ion concentration of the silica fume concrete to be lower than for the low slump dense concrete at a given distance from the surface.

A study conducted by Sprinkel et al [45, 46] evaluated the performance of polymer overlays and concrete overlays. The study concluded that polymer overlays have a useful service life of 10 to 25 years when applied as a protection or rehabilitation treatment. Application of a multiple layer epoxy, a multiple-layer epoxy-urethane, a premixed polyester styrene with a methacrylate primer, or a methacrylate slurry were determined to be the "best-proven" overlay treatments. The research also concluded that high-early-strength hydraulic cement concrete overlays have tremendous potential, but considerable developmental work with the materials and equipment would be needed to overcome problems with installation time. High-early-strength portland cement concrete overlays such as those prepared with 7% silica fume or 15% latex and Type III cement reportedly had a potential service life of 25 years and could perform as well as conventional overlays with quicker installation and curing times.

A relatively new overlay material is conductive-concrete overlay. When connected to a power source, these overlays can generate enough heat through electrical resistance to prevent ice formation on bridge decks, or melt ice after it forms [53]. Conductive concrete is a cementitious mixture containing electrically conductive components that give it a stable and high electrical conductivity. The University of Nebraska and Nebraska Department of Roads have developed a conductive overlay that has "excellent workability and surface finishability" [53]. The studies indicated that conductive concrete containing 20% steel shavings and 1.5% steel fibers had the optimum resistivity and workability characteristics. Studies also indicated that anti-icing was more cost effective and energy efficient than deicing. The average energy cost per unit surface area is about \$0.074/ft^2 per storm (for Omaha, Nebraska). The material cost of conductive concrete is about \$270/m^3 compared to \$51.3/m^3 for conventional concrete. Although the cost of conductive concrete is higher, these overlays should be considered a heating element rather than repair materials [53]. In order to evaluate the durability of conductive concrete overlays under traffic loads, an overlay patch was placed on a bride deck of I-480 near the Nebraska/Iowa border in December of 1999 [53]. Visual inspections were conducted every 6 months, and indicated that there was no fiber exposure or any sign of corrosion, but some reflective cracking did develop.

2.4.1.3 Surface Treatments

The concrete is relatively porous and will absorb moisture. Absorbed moisture can lead to surface scaling and spalling when subjected to freeze-thaw cycles. If the water was contaminated with chlorides, steel corrosion would occur.

Blocking the ingress of water and other deleterious substances could reduce the natural permeability of concrete. In existing concrete, sealers and coatings could be used to form a seal that reduces the permeability of concrete. Two types of common surface treatments include coatings and penetrating sealers. The classification is based on the behavior of the treatment. The treatments provide a non-penetrating film, penetrate into concrete pores, or have intermediate behavior (Figure 5). As a preventative maintenance strategy, coatings and sealers offer significant long-term benefit when applied early on, especially in environments exposed to chlorides.

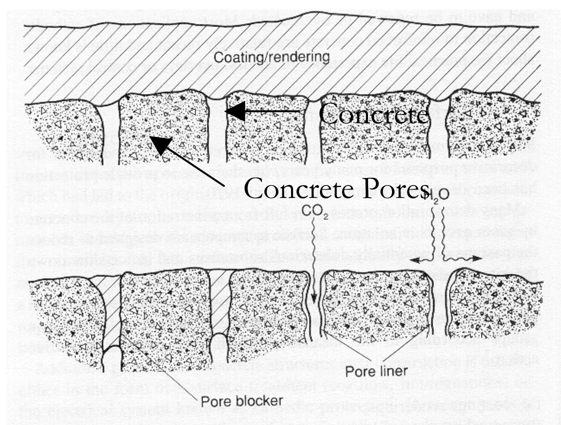

Figure 5. Coatings (top) versus Sealers (bottom) [26]

2.4.1.3a Coatings

Coatings form an impervious film that bridge over pores and provide an external physical barrier, which protects by slowing down the penetration of liquids and gases. They are designed to control water absorption, vapor transmission, and diffusion of aggressive liquids and gases through the concrete surfaces. Coatings are normally applied by brushing, rolling, or spraying the material onto the surface and are available in a variety of colors.

Two commonly employed coatings include cementitious and polymer coating systems. Cementitious coatings allow moisture to escape without debonding or blistering. However, since they do not possess elastomeric properties, they cannot bridge active cracks [8]. Polymer systems consist of epoxies, acrylics, or polyurethanes combined with filler, which provides bulk and thickness. Polymer coatings are hard and durable, but are impervious to vapor transmission. These coatings have blistered and peeled under high vapor pressure [8]. Mallett [36] reports that coatings can be expected to last 10 years and that some appear to be performing well after 30 years.

Bijen [31] summarizes the characteristics and performance of some common coatings. He reports the following:

- Epoxy coatings provide good adhesion to concrete, exhibit minor shrinkage, and are resistant to light chemical attack.

- Polyurethanes will adhere to dry concrete, are almost shrinkage free, are resistant to light chemical attack, but not to highly alkaline conditions.

- Acrylics display good adherence to concrete and good resistance to alkali, oxidation, and weathering.

Ibrahim et al [23] studied the effectiveness of concrete surface treatments including sealers and coatings. They evaluated several penetrating sealers (detailed in section 2.4.1.3b) and a two-component acrylic coating. The coating was found to be the most effective of the materials investigated in minimizing damage due to sulfate attack after 6 months of sulfate exposure. In addition it was determined to be effective in reducing the ingress of carbon dioxide. The coating also exhibited considerably lower chloride concentration in comparison to untreated specimens.

2.4.1.3b Sealers

Sealing existing concrete surfaces reduces the permeability of concrete, which can be improved up to one order of magnitude [9]. This is comparable to using silica fume or latex admixtures to reduce the permeability of new concrete [9]. Experiences in Canada indicate that, "if deterioration has not already progressed too far, maintaining the permeability of existing, exposed concrete at the levels obtainable with correctly applied sealers can reduce the rate of deterioration and result in a reasonably standard service life for exposed concrete" [9].

Penetrating sealers are low viscosity liquids that are capable of penetrating into concrete surface pores filling the cracks and voids. Two types of penetrating sealers include pore liners and pore blockers. Pore liners line the concrete pores and enable the concrete surface to become water repellant. Pore blockers penetrate into the pores and then react with concrete constituents. The resulting products are insoluble and hence, block the concrete pores [26].

Boiled linseed oil is one of the oldest materials used to seal concrete surfaces. It is low in cost, but in cases where it is exposed to traffic abrasion it must be frequently reapplied to maintain protection [8]. Silane sealers penetrate about ½ inch into the concrete and react chemically with concrete to form a layer that resists water and chloride penetration [8]. Siloxane is very similar to silane although not as effective in reducing water and chloride penetration [8]. Both silane and siloxane are sealers permeable to water vapor, which allows the concrete to dry out. In addition, both substances do not color the concrete.

Sodium silicate sealers penetrate concrete and react with calcium compounds to form insoluble calcium silicate, reducing the permeability. Penetrating epoxy sealers use a chemical reaction between the resin and hardener to create a protective film [8]. They permit some moisture vapor transmission, but normally allow less transmission than materials that do not develop a protective film [8].

A study conducted by Whiting et al [51] surveyed highway agencies in the United States and Canada regarding the use of penetrating sealers. The study did not specifically mention the type of sealer that was presented on the questionnaire. Of the agencies surveyed, 46 U.S. and 9 Canadian agencies employed sealers. The most widely used application of penetrating sealers was reported to be on concrete bridge decks. Only about 30% of the respondents were utilizing penetrating sealers in superstructure elements other than decks. The study stated that the respondents noted a variety of problems with the application of penetrating sealers to existing structures. Some of these concerns included the following: drift and evaporation in hot and windy conditions, difficulty in obtaining specified coverage, slippery surfaces, runoff during application, discoloration of concrete, and little or no apparent penetration. The respondents also stated that the performance of the sealers was less than desired. Some indicated that many penetrating sealers were ineffective (or at least not as effective as claimed) in reducing chloride ion infiltration. Other performance problems included: reduction of skid resistance, failure to improve freeze-thaw and scaling resistance, and failure to halt corrosion of reinforcing steel.

Sprinkel et al [45, 46] studied the performance characteristics of sealers as well as overlays and patch materials (section 2.4.1.2). The study did not specifically indicate the type of sealer that was evaluated. The investigation concluded that sealers could reduce the infiltration of chloride ions for 5 to 10 years and therefore extend the time until sufficient chloride ions reach the reinforcing steel to initiate corrosion. To ensure adequate skid resistance, sealers should be applied to decks with tined or grooved surfaces. The investigation found that protection provided by sealers varied with tests, exhibiting 0 to 50%, with an average of 32%, reduction in permeability. On the basis of life cycle cost analysis, the most cost effective protection system was determined to be the application of a penetrating sealer.

In contrast to coatings, penetrating sealers allow the concrete to breath since the pores are exposed to the atmosphere. This permits the concrete to dry-out, and with the moisture intake reduction the possibility of corrosion may be lessened. Since most sealers are clear in color, the color of concrete will generally not be affected when applied. In addition, since penetrating sealers are capable of infiltrating well into the surface, they are less affected by environmental weathering. This can lead to a longer service life when compared to coatings [26]. Bruner [8] compiled a table (reproduced in Table 3), which rates the performance of various sealers and coatings based on several criteria.

Table 4 summarizes the results of a study conducted by Ibrahim, Al-Gahtani, and Dakhil [23], which evaluated the effectiveness of sealers and coatings.

Table 3. Concrete Surface Treatment Selection Guide [8]

Material / Property	Boiled Linseed Oil	Silane	Siloxane	Sodium Silicate	Penetrating Epoxy	Cementitious Coating	Epoxy Coating
Ability to Penetrate	A	G	G	G	G	N/A	N/A
Ability to Bridge Cracks	N/A	N/A	N/A	N/A	N/A	P	VP
Ability to Bond to Concrete	N/A	N/A	N/A	N/A	N/A	G	G
Ability to Reduce Permeability	A	G	A	G	G	G	G
Allow Water Vapor Transmission	A	G	G	G	P	A	VP

VG – Very good performance
G – Good performance
A – Average performance
P – Poor performance
VP – Very poor performance
N/A – Not applicable

Table 4. Ranking of Surface Treatments [23, 36]

Sealer/Coating	**Environment**		
	Sulfate Attack	Carbonation	Chlorides
Control (no sealer)	7	7	7
Sodium Silicate	6	3	6
Silicone Resin Solution	5	5	5
Silane/Siloxane	4	4	4
Silane/Siloxane with an Acrylic Topcoat	1	1	1
Alkyl-Alkoxy Silane	3	6	3
Two Component Acrylic Coating	2	2	2

(Scale from 1 to 7, a rating of 1 implies the best performance)

This study assessed the performance of sodium silicate, a silicone resin solution, silane/siloxane, silane/siloxane with an acrylic topcoat, alkyl-alkoxy silane, and a two-component acrylic coating in preventing concrete deterioration due to sulfate attack, carbonation, and chloride-induced reinforcement corrosion. The study concluded that the penetrating sealers were not effective in reducing concrete deterioration due to sulfate attack. However, silane/siloxane and silane were partly effective in decreasing sulfate attack. Silane/siloxane with an acrylic topcoat was reported to be the most effective in reducing sulfate attack. The investigation also determined that none of the penetrating sealers were totally effective in preventing carbonation of concrete. In addition, the sealers did not perform as well as the coatings in reducing chloride diffusion. However, the coatings (both the silane/siloxane with acrylic topcoat and acrylic coating) were found to be the most effective in preventing carbonation, decreasing chloride diffusion, and reducing reinforcement corrosion. The performance of the surface treatments investigated in the study can be expressed in the following order: silane/siloxane with an acrylic topcoat > acrylic coating > silane > silane/siloxane > silicone resin solution > sodium silicate.

A study conducted for the Transportation Research Board in 1981 [41] researched the protection of concrete bridges against chloride penetration by various surface treatments (coatings & sealers) representative of all of the chemical types commonly used. Initially, 21 surface treatments including epoxies, methacrylate, urethanes, butadienes and a silane were subjected to preliminary screening tests. Based on the initial screening program, five products with low water absorption, low chloride ion uptake and good water vapor transmission characteristics were chosen for further testing. The five materials chosen were an epoxy, a methyl methacrylate, moisture cured urethane, a silane and polyisobutyl methacrylate. These materials were subjected to further testing to determine the effects of moisture condition of the substrate, coverage rate and different environmental conditions on the ability to protect against chloride ion intrusion. The five treatments reduced the chloride ion contents by 79 to 97% compared to the uncoated specimens [31]. The study concluded that the epoxy, methyl methacrylate and the silane were capable of providing added protection to concrete bridge surfaces to reduce intrusion of salt laden water.

Although in theory surface treatments can provide adequate protection against the initiation of corrosion, the reality of their effectiveness is quite different. These materials may inhibit the penetration of deleterious substances, but they do not mitigate the effects of the chlorides that are already present. The generally excepted service life of surface treatments is approximately 5 years [22].

A study conducted by the Federal Highway Administration [52] evaluated various corrosion repair techniques and protection systems for prestressed concrete elements. The prestressed concrete specimens were subjected to accelerated corrosive environments to induce corrosion in the steel. Penetrating sealers and coatings were applied to a set of specimens to study their effectiveness. This study concluded that the surface treatments were of limited effectiveness when applied to specimens subjected to active corrosion. In most cases, chlorides continued to penetrate into the concrete, though at a reduced rate. "While measurements indicated that corrosion activity was initially reduced after the application of the surface treatment, long-term trends suggest that over time corrosion activity may slowly increase back towards the initial levels [52]." The study also concluded that surface treatments applied to new structures would reduce, but not completely eliminate the ingress of deleterious substances. If low quality repair materials are used or incorrect construction procedures are employed, corrosion may still occur resulting in the cracking and spalling of the structure. However, in general, application of surface treatments in new construction significantly improves it long-term effectiveness, especially in chloride environments.

2.4.1.4 Crack Injection

Crack injection involves infusing cracks and other voids with a low viscosity adhesive material (resin). This process glues the concrete together and restores some of the original strength. A protective overlay or surface treatment can then be applied to the surface to prevent moisture penetration and continued deterioration [4]. It is essential to determine the cause of cracking and choose a resin with proper characteristics to ensure the effectiveness of the repair [26]. Crack injection is not applicable to cracks caused by reinforcement corrosion, or if a continuing process is responsible for their generation [26]. Cracks repaired by injection normally perform well if the cracks are dormant, but have not performed as well for active, moving cracks. The method is typically used and successful for hairline cracking and delaminations found on fewer than 30% of the deck area [2]. Sprinkel [46] reports that crack repair has an average service life of 10 to 20 years.

Resins normally consist of two components, an active ingredient and hardener. If large cracks or voids are to be treated, inert filler is also included. The components must be carefully and thoroughly mixed to obtain a final product with the desired properties [26]. The surface of the crack is cleaned and then sealed using polyester putty or other suitable material. One method of crack injection involves introducing the resin under pressure at the first port with the other ports open (see Figure 6).

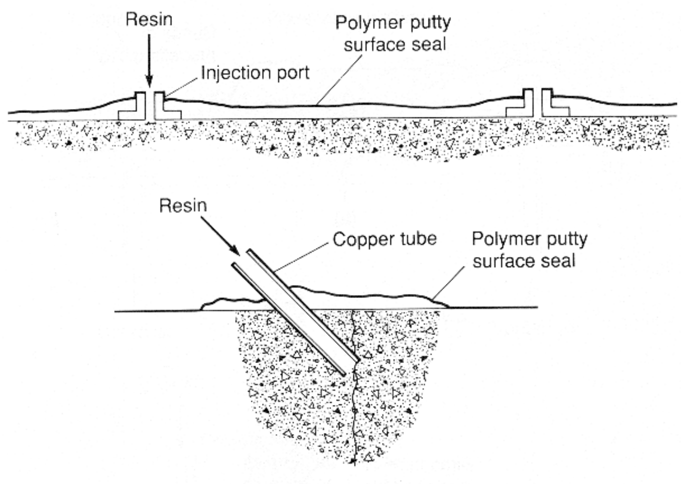

Figure 6. Crack Injection (Under Positive Pressure) [26]

Injection continues until the resin appears at the second port. The first port is sealed off and the process is repeated for subsequent ports. During injection, the region is monitored for signs of resin leaks from interconnected cracks or voids. When the process is complete, the ports are removed [26].

ACI Committee 244 [1] reports that epoxy injection has been successfully used in the repair of cracks in concrete structures. However, the document concludes that unless the cause of cracking has been corrected, the cracking will probably reoccur near the original crack. In addition, the report states that this technique is normally not applicable if the cracks are actively leaking and cannot be dried out. Wet cracks can be injected using moisture tolerant materials, but contaminants in the crack can reduce the effectiveness of the resin to structurally repair the crack.

In 1990, Calder [31] studied the protection afforded by crack injection to reinforced concrete slabs. The materials investigated included: epoxy resin, polyester resin, methyl methacrylate (MMA) resin and liquid silicate solution. Comparisons with unrepaired slabs after ponding with salt water for a period of three years led to the following penetration ranking: epoxy (best) > polyester > MMA > silicate (least good). These repairs prevented preferential access of chlorides into the cracks but did not prevent penetration of chloride ions from the surface. The investigation found that 80% of the corrosion was located at the cracks. The repairs reduced the total number of corrosion sites by half, but had little effect on the number of sites experiencing some section loss. There was little performance difference between the resins in improving the concrete durability, but the silicate solution was determined to be ineffective. However, each of the resins reduced the carbonation depth in comparison to the untreated specimens.

2.4.2 ELECTRICAL METHODS

Electrical repair methods can be classified into the following categories: (1) cathodic protection and (2) chloride extraction. Each of these methods is employed to arrest corrosion and to prevent further corrosion damage.

2.4.2.1 Cathodic Protection

Typical cathodic protection systems include the impressed current system and the sacrificial anode system. The impressed current system employs an external direct current supply. Corrosion is arrested by subjecting the reinforcement to a small direct current to prevent it from reaching an electrical potential that could cause corrosion. The sacrificial anode system uses an external anode, a metal higher in the electrochemical series (i.e. zinc), which corrodes in the process of providing protection. Sacrificial anode systems are simpler than impressed current systems. Sacrificial anode systems can use recycled materials, which can make them less expensive than impressed current systems. However, impressed current systems are usually employed because of its greater current range, ease of adjustment, and its longer service life [42].

Cathodic protection, if applied properly, can arrest steel corrosion in concrete. In 1982, a report by the United States Federal Highway Administration [31] stated, "The only rehabilitation technique that has proven to stop corrosion in salt-contaminated bridges regardless of chloride content is cathodic protection". Since corrosion is an electrochemical process, controlling the flow of an externally applied electrical current (impressed current method) can control corrosion. By applying an external potential, the corrosion rate is reduced by shifting the embedded steel to an artificially cathodic condition. The reinforcement is made cathodic relative to an anode located at or near the concrete surface. Cathodic protection eliminates electrolytic attack of steel and repels dissolved chlorides. If substantial corrosion exists, then cathodic protection could offer a more economical solution than extensive patching repairs [42]. If there is no loss of structural integrity, only repairs to spalled and delaminated concrete are required. There would be no need to remove large volumes of contaminated concrete. Cathodic protection has a high initial cost, but can extend the service life 20 to 30 years [22]. Presently cathodic protection remains an under-used technology for corrosion protection [54]. Table 5 summarizes the cost and life expectancy of several cathodic protection systems.

Table 5. Summary of Costs and Life Expectancy for Cathodic Protection Systems [54]

Type of Maintenance	Average Cost ($/m^2)	Range of Costs ($/m^2)	Average Expected Life (Years)	Range of Expected Life (years)
Impressed-Current (deck)	$114	$92 to $137	35	15 to 35
Impressed-Current (substructure)	$143	$76 to $211	20	5 to 35
Sacrificial Anode (substructure)	$118	$108 to $129	15	10 to 20

Three types of anodes used in impressed current systems include conductive mastic anode, conductive rubber anode, and titanium mesh anode. In the mid-1980's, Florida DOT began employing cathodic protection featuring conductive mastic anodes on its coastal bridges. Mastic paint was initially used, which included carbon to enhance its conductivity, on the regions requiring protection. A rectifier was installed at a central location on the bridge and wires were routed to the protected areas. The DOT experienced favorable results on the beams and decks, but encountered problems with the piles. Water from the high tides impaired the bond between the mastic and the piles and thus the current was poorly distributed to these regions [28].

The conductive rubber anode was developed to address this problem. The rubber anode can be in direct contact with water and continue to distribute current uniformly. The anode is a rubber mat that includes a large amount of carbon. The rubber mats are bonded to the concrete areas to be protected and then connected by wires to a rectifier. This system has performed well and has an expected service life in excess of 20 years [28].

When concrete is still in fairly good condition and only requires some patch repairs, the conductive mastic and rubber anode systems are practical and effective. However, if concrete has severe cracks and spalls and requires more extensive repairs, the titanium mesh system is more practical. Titanium mesh is fastened directly to the concrete element after all the loose and damaged concrete has been removed. The mesh is then embedded in a 2-inch thick gunite (shotcrete) coating. This system has experienced problems when the coating is in direct contact with water. If a member is determined to be structurally deficient, a reinforced concrete structural jacket can be used in conjunction with the titanium mesh. Forms are placed and the concrete is cast around the deteriorated member. The anode can be connected to both the existing reinforcement and any new reinforcement. The mesh-jacket system has proven to be effective in controlling corrosion [28].

The previous impressed current systems all used an external power source to provide the current to the system. The sacrificial anode system provides current by using a metal that is higher in electro-chemical potential than steel, with zinc being the most commonly employed. The anode can be applied either as a coating or sheets. The coating is sprayed on cleaned concrete and exposed reinforcement. The zinc coating typically has a service life on the order of 10 years, at which time in can be re-applied [26]. This system is recommended for applications not in direct contact with water since this accelerates the consumption rate of the anode, and significantly decreases the anode service life [28]. Another system employed uses zinc mesh sheets that are mechanically fastened to concrete. This system is typically used on bridge piles that are in direct contact with water [28].

There are several issues that must be considered when using cathodic protection to protect against corrosion. Selection of the proper anode for the application is critical, failure of the anode leads to failure of the system. In order for cathodic protection to be effective, the steel must be electrically continuous. Ensuring continuity after construction can be an expensive and difficult procedure [42]. In addition, the cathodic protection system itself, which includes an anode, power supply, and monitoring equipment, is costly in comparison to conventional repairs and requires constant monitoring. When an impressed current system is used there are difficulties in determining the correct applied potential and applying it uniformly to the system [3]. The principal concerns include the degradation of the steel/concrete bond, the hydrogen embrittlement of the steel and the alkali-aggregate reaction in the interfacial region [38]. In view of these issues, there is a need to monitor cathodic protection systems continuously to assure that they provide effective protection without detrimental side effects.

Degradation of the steel-concrete bond, associated with the softening of the cement matrix in contact with the metal has been reported in several studies that involved the application of high current densities for prolonged periods [38]. However, at the lower current densities normally required for cathodic protection, the bond strength is normally sufficient to minimize the concern for the structural integrity of the structure [14].

Hydrogen embrittlement is a significant concern for cathodic protected prestressed steels. Hydrogen embrittlement occurs when steel is under high stress and a cathodic reaction is occurring simultaneously at its surface. The cathodic reaction evolves hydrogen atoms at the steel surface. The hydrogen atoms can diffuse and dissolve into the most highly stressed zones of the steel. The effect of hydrogen embrittlement in the stressed zone is to embrittle the steel, which can lead to the brittle fracture of the tendon [7]. The risk of embrittlement depends on a number of factors, and it appears to be low provided the potential is maintained at a level less negative than -900 mV [38].

Wagner [50] reports on research conducted regarding the use of cathodic protection of highly stressed steel tendons. Their research indicates that hydrogen penetrates steel and causes ductility reduction at potentials equal or more negative than those normally considered for the thermodynamic stability of iron. Their experimental work indicates that even short-duration exposure to cathodic potentials of significant magnitudes can produce hydrogen in the metal. The study also found that cathodic potentials more negative than the hydrogen evolution potential sustained for durations greater than 2 hours will result in a reduction in the dynamic load-carrying capacity of notched steel tendons. However, the results indicated that potential levels more negative then the hydrogen evolution potential would not result in a reduction in the static load-carrying capacity of unnotched prestressing tendons. The research continues to be conducted to determine the effectiveness of cathodic protection in known salt-contaminated full-sized prestressed concrete beams.

A limited number of laboratory studies have indicated a potential problem when cathodic protection is used on reinforced concrete structures constructed with alkali reactive aggregates [14]. If the cathodic current density is uniformly and consistently maintained at a low level, the risk of developing expansive alkali silica reaction (ASR) is reduced [38]. However, if the current distribution to the cathode is not reasonably uniform, the risk of locally induced ASR will be greatly increased. Hence, the European Draft Standard [38] recommends that the risk of ASR be considered.

2.4.2.2 Chloride Extraction

Chloride extraction involves the application of an external current, which causes the chloride ions to migrate away from the reinforcement and generates hydroxyl ions, which increases the alkalinity of the region. Research has shown that after application of this technique, the chloride concentrations are substantially reduced and a corresponding increase in the pH of concrete is observed [26]. Similar to cathodic protection, a distributed anode and overlay is applied to the surface. The overlay normally consists of sprayed cellulose fiber saturated with an alkaline solution [26]. The chloride ions migrate away from the steel and towards the anode and are removed with the overlay. Analogous to cathodic protection, there is no need to remove large regions of chloride contaminated concrete and then replacing it with new material before the application of this technique. However, any loose areas have to be repaired to ensure a continuous medium between the reinforcement and surface anode [26].

A typical chloride extraction system is illustrated in Figure 7. Electrical connections are established with the reinforcement. The temporary anode is installed on wooden battens using plastic plugs and bedded into the fiber layer. Anodes should be easy to bend and shape and are usually in the form of a mesh. Titanium meshes are inert and recyclable. Finally, an additional layer of cellulose fiber is sprayed over the mesh and connections are made to a power source [26].

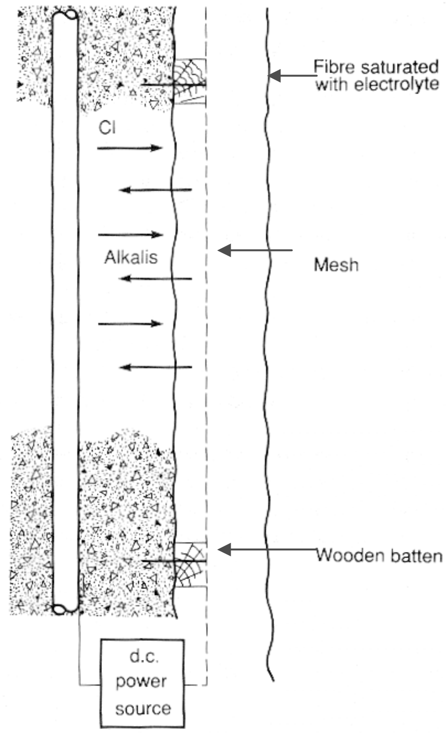

Figure 7. Chloride Extraction and Replenishment of Alkalis [26]

The replenishment of the alkalis is a quicker process than extraction of chlorides. Chloride extraction normally requires three to eight weeks to complete, whereas alkali replenishment requires three to six days to complete [31]. Extraction of chlorides is more difficult because some chloride ions may be bound with reaction products with the cement hydrates. The bound chlorides are in dynamic equilibrium with the chlorides dissolved in the pore water. As the process removes the chlorides in the solution, chlorides from the reaction products replace them. The rate at which chlorides can be extracted is controlled by the decomposition rate of the hydrates [26]. To overcome this effect, current has been applied intermittently to the system.

Bennett et al [6] conducted laboratory tests to determine the feasibility of chloride removal from reinforced concrete bridge components. The studies clearly show chloride extraction to be an effective technique for arresting chloride-inducted corrosion of reinforcing steel. After 3 ½ years no specimens showed a tendency to return to a corrosive condition. By contrast, the untreated control slab was badly delaminated and deteriorated. The treatments removed 20-50% of the chloride ions from the concrete, and relocated the remaining chloride ions away from the reinforcing steel. The percentage of chloride removal was dependent on the design of the reinforcement, with regards to spacing and bar placement, the degree of chloride ingression, and the chloride ion distribution.

The study also addressed several concerns, which may arise as a result of the passage of large amounts of current through concrete. The steel/concrete bond strength was measured over a wide range of current and charge. The application of very high current densities resulted in a reduction in bond strength when compared to control specimens. However, the use of lower current densities had no adverse effects. The compressive strength of the concrete was also reduced at high current densities as the specimens experienced a softening of the cement paste around the steel. The possibility of hydrogen embrittlement of the steel was also studied. Although a slight, temporary loss of ductility was observed, the researchers concluded that this loss was not structurally significant. The study also concluded that chloride extraction can cause an increase in the alkali cation concentration in the vicinity of the reinforcing steel and serious damage could occur if the chloride removal process was applied to concrete containing alkali-reactive aggregates. The use of a lithium borate buffer could be used to mitigate this problem.

Bennett et al [5] also reviewed field trials applying the chloride removal process to reinforced concrete bridge components. Four field validation trials were conducted between the fall of 1991 and fall of 1992. Chloride removal was conducted on an Ohio bridge deck, and bridge substructures in Florida, New York and Ontario. Active corrosion was occurring on a substantial portion of each selected structure, and chloride contamination was well above threshold levels. The treatment was applied until a total charge of 60 to 135 A-hr/ft^2 of concrete was accumulated. The pH of the electrolyte was maintained neutral or basic to prevent etching of the concrete surface and the evolution of chlorine gas. In summary, all four field trials were deemed successful and no detrimental side effects were observed. The report mentions that as of yet chloride extraction cannot be recommended for structures that contain prestressing steel or alkali reactive aggregates.

Manning and Pianca [32] report on the initial evaluation of electrochemical removal of chloride ions from a section of a concrete pier located on the Burlington Skyway. The evaluation included visual examination, corrosion potential, rate-of-corrosion measurements, and petrographic examination and measurement of chloride ion profiles from samples removed from the structure. After 13 months of treatment it was determined that the process was successful in moving chloride ions away from the reinforcing steel and in removing a substantial proportion from the concrete without apparent damage to the concrete. However, the process was unable to remove all of the chloride ions from behind the reinforcing steel. Therefore it is unknown the extent to which the chloride ions will initiate further corrosion in the pier.

A study conducted by the University of Minnesota [10] investigated methods for mitigating corrosion in reinforced concrete structures on the substructure of a bridge in Minneapolis, Minnesota. Several corrosion-damaged columns and pier caps were treated with electrochemical chloride extraction (ECE). Some structures were also wrapped with fiber-reinforced polymer (FRP) sheets or sealed with concrete sealers to prevent future chloride ingression. Embeddable corrosion monitoring equipment (resistivity probe) was installed to evaluate the effectiveness of the ECE treatment. The initial chloride concentration were reduced approximately 50% at each sample depth in each structure. The treatment was most effective near the concrete surface, and the overall effectiveness appeared to depend on the original chloride content (with locations containing high initial chloride concentrations being treated more effectively) and the proximity of the sample to the reinforcing steel. Several locations possessed chloride concentrations in excess of established corrosion thresholds. The study concluded, while the majority of the treated structures can be considered passive, corrosion can potentially reoccur once chloride ions remaining in the concrete migrate back to the reinforcing steel level.

2.5 FIBER REINFORCED POLYMER (FRP)

Due to widespread deterioration of concrete bridges, new materials and protection systems must be investigated to minimize costs and conserve resources. Past research and field studies have proven that patch repairs lack longevity and are susceptible to ongoing deterioration. The limitations imposed by conventional repair materials have led to the investigation of materials that have been widely used in the aerospace, sporting goods, and automotive industries. Fiber-reinforced polymer (FRP) composites are being increasingly considered for bridge applications due to their high strength-to-weight ratios, their corrosion and fatigue resistance, their ease of transport and handling, and their potential for tailorability. These materials have been implemented in a number of rehabilitation and demonstration projects. The application of FRP composites has been demonstrated to be a promising repair solution for many rehabilitation issues.

Composites are created through the combination of two material phases, one serving as the reinforcement and the other as the matrix. In generic terms, FRP composites are analogous to reinforced concrete. The fiber reinforcement can carry load in pre-designed directions and the resin behaves as a medium to transfer stresses and provide physical protection for the fibers. Common types of fiber used in structural applications include glass, aramid or carbon. Epoxy and polyester are the most common resins. The resulting composite behavior depends primarily on the fiber volume and direction, the mechanical properties of both constituents, and the fabrication procedure. Since composites have variable properties, a wide palette of material choices is available to the designer to fit the specific requirements of the situation.

Unlike metals, composites do not corrode, which makes them attractive in corrosive environments. Composites can be used in new construction as reinforcing bars and grids, or prestressing tendons to eliminate the development of corrosion. Composites have also been studied for their use as surface treatments to provide a barrier to corrosive elements [29, 39].

Several rehabilitation methods have been developed to repair and strengthen concrete structural members. These include the application of externally bonded FRP plates, the use of external or internal FRP prestressing strands, the use of composite wrap to repair corrosion-damaged elements, or even entire replacement bridge decks composed of composite sections.

These structural systems combine the mechanical characteristics of composite fibers, the compressive characteristics of concrete, and the ductility and deformation capacity of steel. In addition, these lightweight materials can reduce the quantities of steel and concrete in the structure, lower material transportation costs, enable quicker construction times, and lower labor costs. Some investigators believe that these cost savings can offset the higher material costs of FRP composites [37].

Recently, the use of fiber-reinforced composites (FRP) to repair damaged girders has been studied. Meier et al [33] studied the effectiveness of FRP plates to strengthen existing girders. Composites offer performance advantages not found in other materials (e.g. steel plates). These advantages include: corrosion resistance, easy to handle, available of endless bobbins therefore no joints are necessary, some do not debond when subjected to compressive stresses, and outstanding fatigue behavior [33].

Tedesco et al [48] performed a comprehensive finite element analysis of a deteriorated reinforced concrete bridge repaired with externally bonded FRP plate. The plates were unidirectional with the fibers oriented parallel to the longitudinal axis of the plate. The FRP plates were bonded to the concrete with readily available structural adhesive. Static and dynamic analyses of the bridge were conducted for conditions both before and after the FRP repairs, with loading by two identical test trucks of known weight and configuration. The results indicated that the bonding of the plates to the bridge girders reduced the average maximum mid-span girder deflections and reinforcing steel stresses by 9% an 11%, respectively. The results of the parametric study also indicated that increasing the FRP plate cross-sectional area can reduce the maximum girder deflections and reinforcing steel stresses by approximately 20% and 22%, respectively. Moreover, increasing the FRP plate modulus of elasticity was show to reduce both the maximum girder deflections and reinforcing steel stresses by 16%.

Fiber-reinforced polymer wrap has been researched considerably with regards to the repair and strengthening of corrosion-damaged columns. Bridge columns are especially vulnerable to corrosion-induced deterioration due to their frequent exposure to deicing salts. In Wisconsin, concrete columns can be exposed to deicing salts through failed deck joints or from salt spray from passing automobiles or snowplows. It is a relatively simple process to clean and repair the damaged columns followed with encasement in FRP composite wrap. The wrap slows down the corrosion rate by preventing the ingress of deleterious substances and also by confining the concrete core, thereby providing it with strength and ductility.

In a variety of studies, this application of composite material has been proven to increase the service life of columns. Research conducted at the University of Toronto [29,39] studied the effectiveness of composite wrap to rehabilitate corrosion-damaged columns. The results of the study indicated that the composite wraps, being strong and corrosion resistant, proved to be effective as a column jacketing material. "The repair option that performed best, with regard to the post-repair corrosion rate, strength recovery, and deformation capacity, was also the simplest and easiest to implement alternative, consisted of cleaning the damaged surface (without removal of contaminated concrete) and wrapping layers of FRP sheets directly to the column surface [29]."

A field study conducted in Quebec involved the repair of the Highway 10 overpass columns [37]. The columns required repair due to corrosion-damage, primarily caused by the close proximity of the highway lanes and the splashing of salt contaminated snow [37]. The project demonstrated that the relatively high costs of composite materials could be offset by a reduction in labor costs. The repair work required only three weeks time. The lighter weight of the material and the ease of application allowed a reduction in the number of workers as well as the number of labor hours. Other advantages observed were that formwork was not required for the column repair and the flow of traffic was not interrupted during the repair work.

FRP wraps have also demonstrated to be an effective alternative rehabilitation material for repairing and strengthening bridge piers. A bridge pier is exposed to a variety of loads, water currents, ice impacts, and corrosion attributed to deicing salts leaking through failed expansion joints or from the spray of salt-laden snow. Composites are often chosen to rehabilitate bridge piers due to their strength and durability.

A study conducted by Gergely et al [19, 20] involved the repair of a Highland Drive and Interstate 80 bridge pier in Salt Lake City. The thirty-five year old pier was severely corroded due to freeze-thaw action and the use of heavy deicing salts. The bridge had also experienced an increase in vehicle weight and traffic and lacked adequate seismic detailing. It was determined that the significant corrosion damage had reduced the initial capacity of the pier. Furthermore, it was concluded that the pier would experience severe damage in the event of a major earthquake. The rehabilitation of the bridge pier involved applying CFRP fabric on the columns, cap beam-column joint, and the cap beam haunches. Experiments were performed at the University of Utah and Utah State University to verify the repair design. If was found that the ductility of the column/pier was doubled and the shear strength of the wrapped joints were significantly increased. The construction cost 20% less than conventional repair methods and only required one week to complete. It was concluded that, when compared to traditional repair techniques, the advantages of the composite wrap repair method include that it is fast, non-intrusive, and does not increase the weight of the pier.

Corrosion-damage of concrete bridge beam-ends commonly occurs in northern climates. Corrosion of beam-end reinforcement often occurs due to the failure of the overhead deck expansion joint and improper deck drainage. The resulting steel corrosion and the spalling of concrete in the bearing zone can cause irreversible damage to the beam-ends. Conventional rehabilitation of damaged beam-ends generally requires the complete removal of the damaged region, followed by reconstruction. Common issues with this repair procedure include reoccurring spalls due to inadequate bond between the new and existing concrete and the high cost and time required completing the repair. In addition, if drainage issues were not addressed, the repair would likely not be effective.

Fiber-reinforced composites can be applied, with relative ease, around a concrete beam end. However, to our knowledge research has not been conducted regarding the effectiveness of FRP wrap to rehabilitate corrosion-damaged beam–ends. Since concrete beam-end corrosion damage is frequently encountered in Wisconsin, the effectiveness of various traditional and state-of-the-art repair techniques for addressing this problem should be examined.

2.6 SUMMARY OF CORROSION REPAIR METHODS

Surface treatments, while reasonably effective over the short-term, have demonstrated limited effectiveness over the long term, unless they are applied prior to chloride contamination.

Cathodic protection, while effective, is not commonly employed due to the high component and maintenance costs as well as the complexity of the method. In addition, due to the possibility of hydrogen embrittlement, cathodic protection of prestressed concrete beams is not recommended. Research studies have established the effectiveness of FRP composites to prevent and mitigate corrosion-damage in concrete columns. However, to date, no research has been conducted in regards to their effectiveness to prevent and mitigate corrosion-damage in prestressed concrete bridge beam-ends. Therefore, since corrosion damaged beam-ends are frequently encountered in Wisconsin, the need for experimental work studying the effectiveness of various surface treatments, including fiber-reinforced polymer wraps, to protect and mitigate corrosion damage in beam-ends is essential.

2.7 VEHICULAR IMPACT DAMAGE

Corrosion damage occurs over a relatively extended period of time. However, damage to bridges can occur instantly by an applied force from an over-height vehicle or water born vessels. Vehicle damage can have serious consequences and include both damage to concrete and damage to the reinforcing or prestressing steel. A 1992 report by the Texas Department of Transportation [18] has suggested that with increasing demands on infrastructure and new bridges being built, the occurrence of over-height vehicle impact will continue to rise.

Depending on when the damage occurs, full-scale repairs may not be able to be performed immediately. In this case, temporary measures should be instituted to protect the bridge. These measures include the removal of all loose concrete and installation of a barrier beneath the damaged member to catch concrete that may be inadvertently dislodged. Also, weight restrictions may be posted on the bridge to protect the most severely damaged members. The bridge should be monitored closely to prevent any further damage.

The amount of damage caused by vehicle impact can be classified as minor, moderate, or severe damage [18]. Minor damage consists of isolated cracks, nicks, shallow spalls, and scrapes. Moderate damage involves much larger cracks and spalls that expose undamaged reinforcing steel or prestressing tendons. Severe damage includes exposed, damaged steel and/or tendons and a significant loss of concrete cross section as well as girder distortion or lateral misalignment [18].

Repairs not only restore the structural integrity of the bridge, but also the appearance and durability of the damaged member. When the damage is classified as minor, the structural integrity of the bridge has not been compromised. The repairs are performed to restore the aesthetics and durability of the element. Typically, spot patching can fill cracks and spalls to protect the reinforcement from exposure [18].

Moderate damage is still considered non-structural. However, when reinforcing steel or prestressing tendons are exposed, all corrosion products should be removed and the steel should be treated with corrosion inhibitors before patching [18]. Splices of prestressing tendons, reinforcing steel and stirrups may be required if the members have lost significant amount of cross section due to corrosion. Any cracked, undamaged members should be epoxy injected to improve the durability of the element [18]. Cracks too fine to be injected should be treated with a silane sealer to prevent the ingress of deleterious substances [18]. It is recommended that the damaged element be loaded before the repair material is placed to ensure that recast concrete would regain prestress as originally intended.

Severe damage typically includes damage to the structural integrity of the member. A structural analysis of the bridge may be required to determine if the damaged member can be sufficiently repaired to return the bridge to its pre-damaged load-carrying capacity [18]. If prestressing tendons are severed, the tendons can be spliced by the following methods: external post-tensioning, internal splices, or metal sleeve splices. NCHRP Report 280 [43] provides a practice user manual for dealing with accidentally damaged prestressed concrete bridge members. The authors state in the reports that "they believe that sufficient research has been performed to document the effectiveness of the repair methods (listed above). No additional research is required prior to implementation of these methods in the field [43]."

External post-tensioning involves the use of high-strength rods or prestressing tendons jacketed against concrete corbels that have been recast against the girders. This method is suitable for splicing bundled strands or small tendons as well as a number of individual strands [43]. Figure 8 illustrates a standard external post-tensioning detail.

Internal splices incorporate a turnbuckle device to stress several strands. The device can be torqued to achieve the desired stress level in the strands. This method is inexpensive and easy to install [43]. Preloading must be employed in the spliced areas to restore compression in the concrete patch. Figure 9 illustrates a method for splicing a single ½ inch 270 K strand.

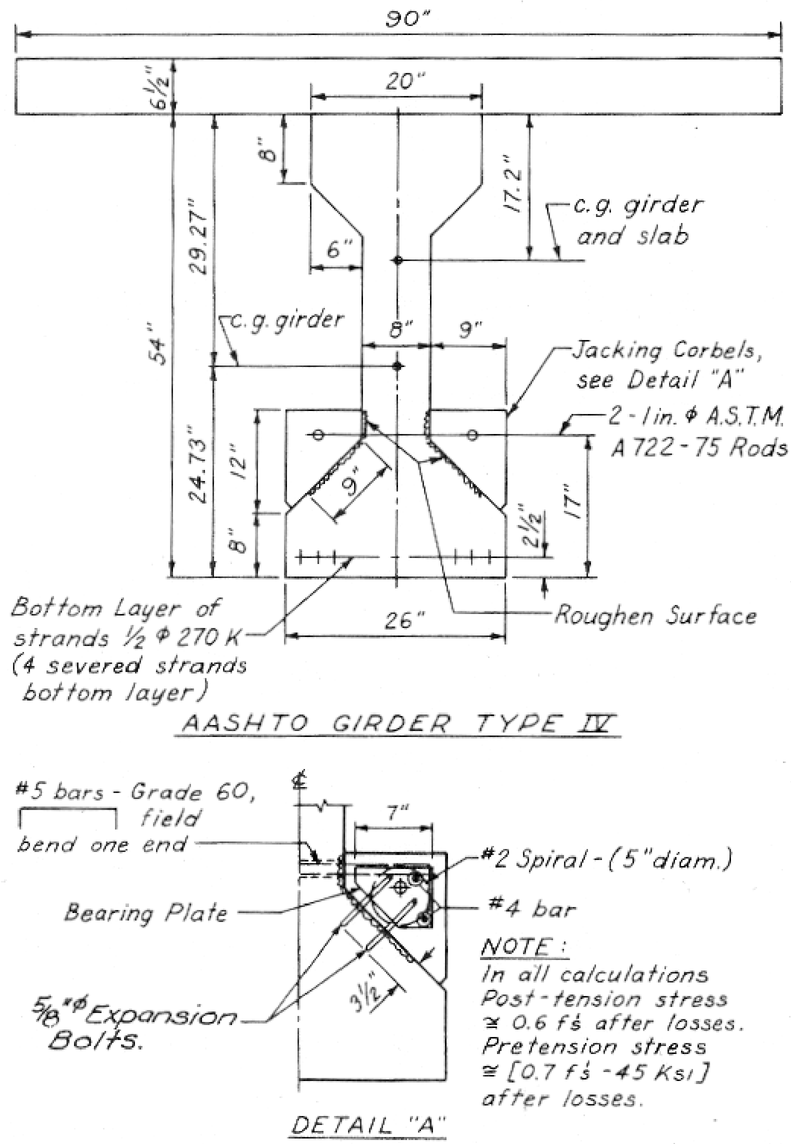

Figure 8. External Post-tensioning Detail [43]

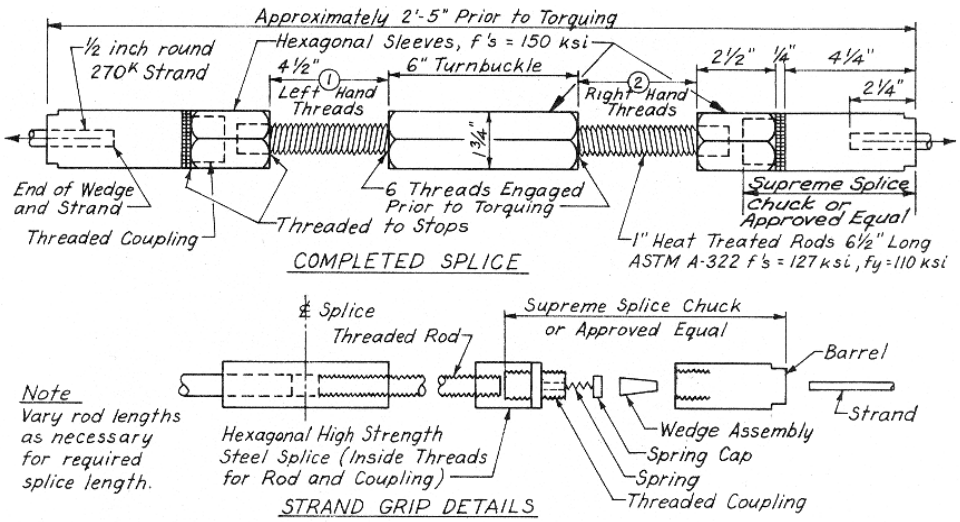

Figure 9. Single Strand Internal Splice [43]

Metal sleeve splices utilize metal plates bonded and/or bolted to the bottom and sides of a damaged girder. This method can be used to splice a large number of severed strands and when large volumes of loose or shattered concrete is present [43]. Figure 10 illustrates the use of a metal sleeve to splice ten severed ½ inch 270 K strands in an AASHTO Type IV beam.

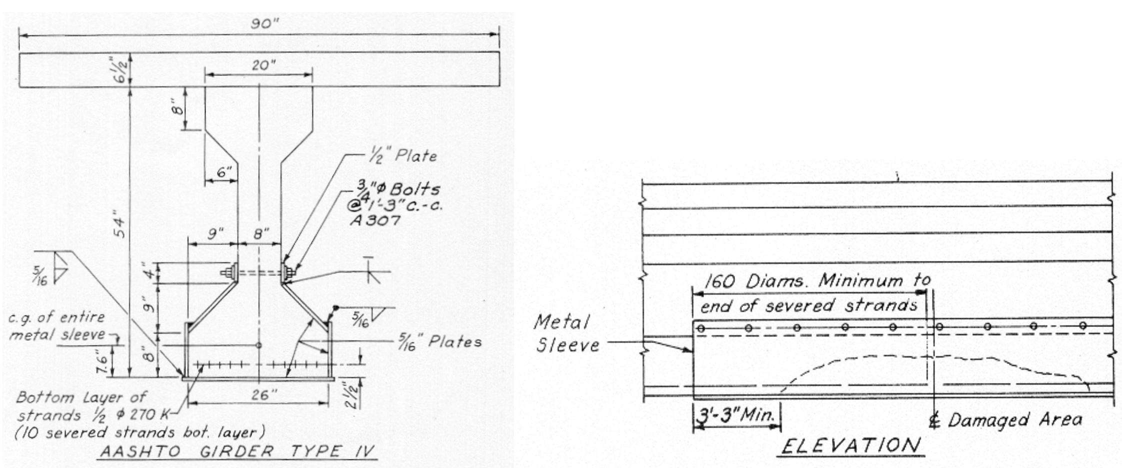

Figure 10. Metal Sleeve Splice [43]

Complete replacement of a member is normally the most expensive method of repair [43]. Replacing a member will require removing a portion of the roadway slab.

NCHRP Report 280 [43] recommends that the selection of a repair method should be based on an objective analysis. The selection of an appropriate repair method should be a based on the type and extent of damage. Table 6 was developed to compare the difference between methods used to repair severely damaged girders.

Table 6. Severe Vehicle Impact Damage Repair Method to Consider [43]

Damage Assessment factor	Repair Methods to Consider			
	Post-tensioning	Internal Splicing	Metal Sleeve Splice	Replacement
Service & Ultimate Load	Excellent	Excellent	Excellent	Excellent
Overload	Excellent	Excellent	Excellent	Excellent
Fatigue	Excellent	Limited	Excellent	Excellent
Adding Strength to Non-Damaged Girders	Excellent	N/A	Excellent	N/A
Combining Splicing Methods	Excellent	Excellent	Excellent	N/A
Splicing Tendons of Bundled Strands	Limited	N/A	Excellent	Excellent
Number of Strands Spliced	Limited	Limited	Large	Unlimited
Preload Required	Perhaps	Yes	Possibly	No
Restore Loss of Concrete	Excellent	Excellent	Excellent	Excellent
Speed of Repair	Good	Excellent	Good	Poor
Durability	Excellent	Excellent	Excellent	Excellent
Cost	Low	Very Low	Low	High
Aesthetics	Fair*	Excellent	Excellent	Excellent

*Can be improved by extending corbels on fascia girder
N/A – not applicable

The durability of the repaired girders should be, as nearly as possible, equal to the durability of the original construction. NCHRP Report 280 [43] recommends that the following guidelines be considered in repairing damaged prestressed concrete bridge girders to achieve acceptable durability:

1. All unsound concrete should be removed and surface preparation should be such that new material placed will be compatible with existing material. New material should have equal or better strength characteristics than original.

2. Epoxy bonding, epoxy grout, and epoxy injection materials and systems should be fully tested and approved, and should be applied by trained personnel. Particular requirements concerning ambient temperatures must be observed.

3. Additional reinforcement to bond new material to existing surfaces should be considered.

4. Preloading should be used (if necessary) to ensure that the repair section would not be subject to greater tensile stress under live load than the original section.

5. Additional prestress force as required ensuring repaired stress levels are no greater than original design stress levels.

6. To further increase durability, the repaired areas should be sealed with proven water retardant.

7. Where repair design dictates, commitment should be made to perform periodic preventative maintenance.

The repair cost of minor damage, such as nicks, spalls, scrapes, cracks, and exposed strands is relatively low. The cost of materials is relatively low, and agency personnel can normally perform the repair. The repair cost of minor and/or moderate damage (per girder) normally would not exceed 10% of the cost of replacing the girder [43].

The repair of severe damage, such as severed strands and major concrete loss, normally will require the services of a contractor. The cost of the repair depends on several factors, such as: the type of repair, traffic control measures, and the extent of damage. Because of these factors, there is not precise cost data available. It is estimated that the repair cost of severe damage (per girder) will vary from 15% to 50% of the cost of replacing the girder, depending primarily on the extent of damage [43].

Neale and Labossiere [37] described the application of composite materials for the rehabilitation of the Webster Parkade in Sherbrooke, Quebec. The composites were used to reinforce beams that did not conform to current standrards concerning bend and/or shear capacities. Following the composite rehabilitation, the strength capacity increased 15% of the initial bending strength of the beams, and 20% of the original shear strength.

Nanni and Gold [35] studied the repair of impact damaged concrete beams with CFRP plates. An over-height vehicle damaged four prestressed girders of the bridge overpass on highway Appia near Terracina, Rome. The conventional steel reinforcement was clearly visible after the loose concrete was removed. The concrete section was restored with non-shrink mortar. After surface preparation, CFRP plates were adhered to the girders. Combined with preloading, the bonding of the external plates restored the prestress that was lost upon vehicle impact. Furthermore, the author states that the strengthening approach was easy to perform and resulted in significant improvement in the ultimate load capacity and, to a lesser extent, the flexural stiffness.

Some issues must be addressed before the application of FRP plates to repair impact damaged prestressed concrete girders can be implemented with widespread acceptance. The strength and durability of the concrete-composite bond is critical to the success of the repair. It is necessary to avoid or at least limit the extent of FRP debonding in order to ensure the effectiveness of the strengthening repair and the ductility of the load-deflection response [50]. In addition, a high degree of quality control and quality assurance must be established during the installation of the repair. Other engineering issues that must be addressed are FRP materials low modulus of elasticity, low failure strain, and the fact that it cannot be bent after fabrication [47]. In addition, repairs employing FRP materials have a relatively high initial cost. FRP material can cost five times more than steel (by weight), but these numbers can be misleading since less FRP material can normally repair the same amount of concrete [47]. The extent of FRP composite applications will depend upon the resolution of these issues.

2.8 LITERATURE DATABASE

After completion of comprehensive literature review in the field of rehabilitation of concrete bridges, focusing especially on northern climates, was completed, an extensive literature database was developed using Microsoft Access. Over 570 papers and reports were cataloged, and include such searchable information as the title, publisher, author, and date. The database also includes the abstracts or summaries of many of the papers. The user can search the database by performing a keyword, title, or author query.

Chapter 3

EXPERT SYSTEM SOFTWARE

3.1 BACKGROUND

Expert systems have shown to be a useful tool to aid in the decision making process for a variety of applications in the construction industry. These systems have been applied in the fields of structural design, distress diagnosis, or repair schemes identification. However, according to Kaetzel and Clifton [24], the success rate in using expert system technology to develop practical applications in the construction industry is relatively low. They attribute this to user attitude, constraints in acquiring sufficient knowledge about a particular subject, and lack of easy-to-use development tools. Therefore if realistic expectations and sufficient knowledge base are in place, an expert system cannot replace the expert, but can assist in the decision-making processes. The complexity of bridge condition assessment and subsequent identification of repairs could be made more manageable by an expert system that could aid in the decision making process.

3.2 EXPERT SYSTEM TOOLS

Expert systems are also referred to as knowledge-based or decision support systems that emulate human expertise. They are normally designed to mimic the role of an expert. The user is prompted by a series of questions and statements, which will lead to a final conclusion or recommendation.

An example of an operational expert system in use today is Highway Concrete (HWYCON) Expert System. The program was designed to be used by inspectors and engineers, and is reportedly being used by some U.S. states, local governments, and city transportation departments [24]. It was developed to assist in the diagnosis, material selection, and general repair activities relating to concrete structures. To operate HWYCON the user answers questions about the structure and its environment. The program then provides the user with a hypothesis or recommendation. The knowledge base of the system includes digitized photographs, drawings, facts, rules of thumb, explanatory information, and tables. HWYCON has reportedly also been used to assist students at the University of Illinois in the diagnosis of distress in highway concrete structures, the selection construction and repair materials, and direction on the use of materials and procedures for repair. Kaetzel and Struble 1995 [25] report that HWYCON is useful for teaching the fundamental aspects of determining methods an materials for construction and rehabilitation of concrete highway structures.

Another construction related expert system, from Japan, is The Bridge Rating Expert System [24]. This system is designed to provide a serviceability rating for bridge structures in Japan. The system reportedly addresses the durability, load carrying capacity, and serviceability of bridges by incorporating knowledge from experts, probability theory, and a relational database component. The objective of the system is to rate the bridge condition in categories ranging from safe to dangerous. The Bridge Rating Expert System is reportedly in the developmental stage [24].

3.3 DEVLEOPMENT OF EXPERT SYSTEM (ConBAR)

The HWYCON program is significant because it is one of the first comprehensive efforts to apply expert system tools to highway condition assessments. However, an examination of the HWYCON program indicates that a number of areas of weakness can be identified, such as the following:

1. HWYCON program modules cover a very wide range of topics including structures, pavements, construction, materials, etc. However, perhaps because of its very wide breadth, its depth is somewhat limited and only handles problems of a very general nature. For example, only two or three questions are typically asked by the system before a problem is identified for a bridge structure.

2. The HWYCON program does not typically evaluate the extent or severity of a bridge problem.

3. HWYCON program does not generally suggest corrective actions for bridge problems.

These issues prompted the researchers to propose development of a bridge diagnosis program that focuses on concrete bridges, identifies the extent of the problem, makes recommendations, and incorporates the compiled rehabilitation literature database. The infrastructure and a basic form of the Concrete Bridge Assessment and Rehabilitation (ConBAR) software are therefore developed in this project. The objective in the creation of the ConBAR expert system was to provide an electronic guide that would help diagnose the problem(s), determine the extent of damage, and identify repair, rehabilitation, or preventative maintenance options for concrete bridges. This expert system will use data inputted by the user and a series of answers to questions prompted by the system. ConBAR provides a number of possible solutions along with their pros and cons, a suggestion, or a hypothesis. Recommendations for additional tests or sources of information are supplied to confirm or refute the hypothesis. The current state of development of ConBAR includes the complete infrastructure required as well as a limited number of expert rules, which must be expanded and enhanced in future developments of this program.

ConBAR expert system addresses cracking, surface defects (such as honeycombing and blistering), spalling, corrosion, vehicle impact damage, alkali-silica reactivity (ASR), and chemical exposure. The system also considers exposure conditions, previous repairs, bridge age, inspection information and other factors. The program knowledge base includes: (1) facts and rules of thumb, (2) visual information such as photographs and drawings, (3) indirect access to a rehabilitation literature database and (4) descriptive statements.

A session consists of a series of questions and supplemental information presented on a computer screen. A typical screen display consists of questions followed by a list of possible answers (see Figure 11). Pictures and detailed descriptions are also included for some questions to assist in the answering process. The user indicates the desired answer by clicking the button next to the answer with the mouse or choosing from a pull down list. An "enter" push button is provided to direct the program to the next step. When the questions have been completed, the system attempts to provide a solution or recommendation based on the responses to the questions. Three examples using the ConBAR program are presented in Appendix A.

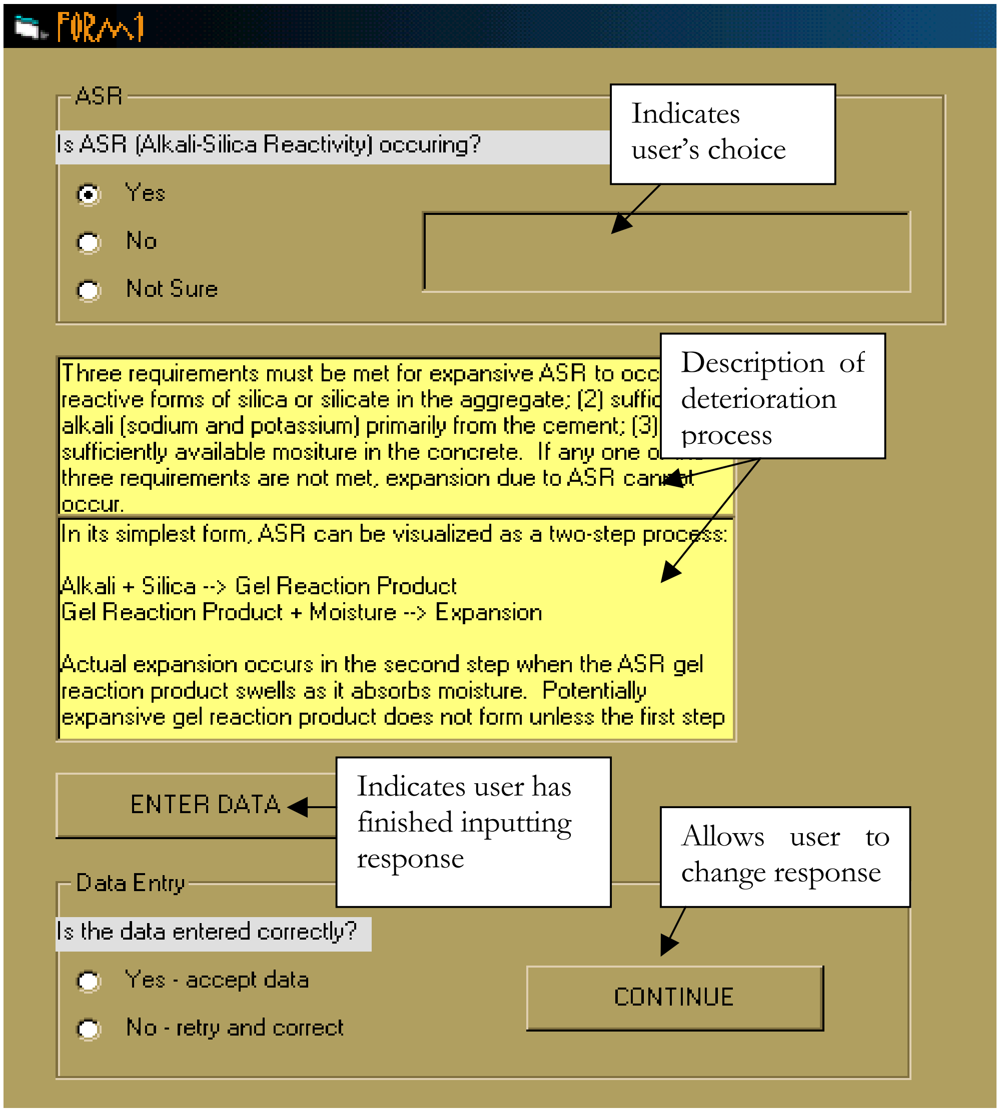

Figure 11. Sample of Expert System Screen

ConBAR expert system was programmed using both Microsoft Visual Basic 6.0 and CLIPS 6.20 (C Language Integrated Production System) programming languages. CLIPS is an expert system tool developed by the Software Technology Branch, NASA/Lyndon B. Johnson Space Center. CLIPS is designed to facilitate the development of software to model human knowledge or expertise in a great variety of applications. It is a tool for the construction of rule and/or object based expert systems. CLIPS provides a "facts list" that includes known information, a “knowledge base” that includes all the expert rules, and an “inference engine” that controls the execution of the rules (decides which rules are executed and when). Although CLIPS is a very powerful program, it is difficult for people who are unfamiliar with expert systems to run it. Therefore, a Visual Basic code was incorporated to more easily interface with the user to obtain the required information, and thus eliminate the need for the user to learn CLIPS. CLIP transforms the information collected into “fact lists” that are understandable by CLIPS. CLIPS then uses these “facts” to execute the expert rules previously written by the programmer. A CLIPS rule is similar to an IF/THEN statement in a procedural language like C or Pascal. Therefore if certain conditions are true then some rules “fire” and the selected actions are executed. CLIPS then returns the solution to Visual Basic where it is presented to the user and displayed.

C h a p t e r 4

EXPERIMENTAL PROGRAM

4.1 INTRODUCTION

Based on the results of the literature review, a test plan and repair concept were formulated to study the prevention and repair of corrosion damage to prestressed concrete beam ends due to chloride-laden water infiltrating through faulty bridge expansion joints. The objectives of the experimental program were to (1) determine the effectiveness of a sealer, epoxy coating, polymer (resin) coating, and FRP wrap in protecting against corrosion damage in new members and (2) to establish the effectiveness of these treatments and patch repairs in reducing/preventing continued corrosion in members that were already contaminated with chlorides.

The work plan included performing laboratory tests on five new 8-foot long prestressed concrete bridge I-beams. The beam-ends were subjected to wet/dry cycles of salt laden water (6% NaCl solution) to accelerate the corrosion process. In addition to the salt-water exposure, the beam-ends were subjected to galvanostatic accelerated corrosion methods to assist in quicker corrosion initiation time and to draw the chlorides into the concrete faster. Two cathodes (short length prestressing strands) were embedded in each beam end to facilitate reverse cathodic protection, thereby making the entire reinforcement system anodic. Selected end regions were pretreated with a sealer, coating, polymer coating, or FRP composite wrap to assess their effectiveness in protecting the beam when subjected to an accelerated corrosive environment. Some beam-ends were left untreated. After a time period of over six months, some of the previously untreated beam-ends were patch repaired or subjected to one of the prior stated surface treatments, and the accelerated corrosion process was continued for all specimens.

4.2 SPECIMENS

Pretensioned concrete beam specimens consisted of new 8-foot long AASHTO Type II sections as illustrated in Figure 12. The beams contained 18 - ½ inch diameter grade 270 low relaxation seven-wire prestressing strands. The magnitude of force on each strand prior to prestress transfer was 75% of the guaranteed ultimate tensile strength or approximately 30,980 lbs. All strands were straight and were cut flush with the end of the beams. The beams also contained stirrups and other conventional reinforcement as shown in Figure 13 (details provided in Appendix B). The conventional reinforcement was Grade 60 ASTM A614 steel with actual yield strength of 70 ksi and actual tensile strength of 109.2 ksi, per mill certification report. Two additional unstressed prestressing strands (2 feet long) were embedded in each beam end. These strands were electrically isolated from the main cage (strands and stirrups) to serve as cathode bars and facilitate the accelerated corrosion process. Please see Appendix B for the detailed shop drawings. Utilization of new beams in lieu of existing or in situ bridge beams had the following advantages: (1) enabled better control over the time schedule of the project, (2) all beams were subjected to the same controlled laboratory environment, (3) allowed uniform chloride exposure to all specimens, and (4) allowed incorporation of galvanostatic accelerated corrosion methods.

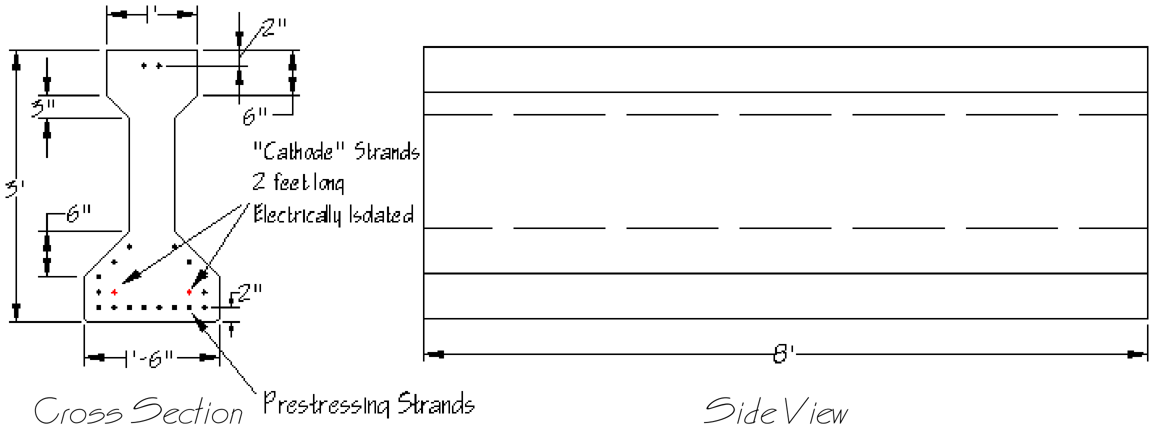

Figure 12. Design Details for Pretensioned Concrete Beam Specimens

Figure 13. I-Beam Steel Cage

All specimens were pretensioned and cast in January 2002 at Spancrete's production facility in Green Bay, WI. The steel was placed by the fabricators and verified by the investigators. Two cathodes were placed at each beam end. The investigators also verified the electrical continuity of the main steel cage and the electrical isolation of the cathodes prior to casting of concrete.

4.3 SPECIMEN EXPOSURE

After the beams were properly cured, they were delivered to the UW-Milwaukee Structures Laboratory. The indoor exposure regime was designed to simulate corrosion aging of prestressed concrete bridge beam-ends. The beams were positioned on neoprene pads on top of a constructed support system. Steel tube sections, with castors located at either end, supported the beams and a steel trough covered by roofing membrane (Figure 14). The support beams were built with castors to allow easy movement of beams in and out of their positions. The salt-water distribution system was constructed to subject the beams to controlled salt-water exposure. The system (illustrated in Figure 15) included the use of a water reservoir, located above the beams, which gravity fed the salt water to the beam-ends through a series of pipes, valves and hoses. The excess salt water was collected from each beam (trough system) and routed to a storage tank located in the building's basement. As needed, the water was then pumped back up to the reservoir. A photograph of the experimental set-up is shown in Figure 16. The beams were subjected to wet/dry cycles, which consisted of 4 days of exposure to salt water drip followed by 3 days dry. The salt-water exposure was designed to simulate the exposure commonly encountered in the field when the expansion joint fails and the bridge is subjected to deicing salt applications.

 

Figure 14. Beam Support System: front view (left) and side view (right)

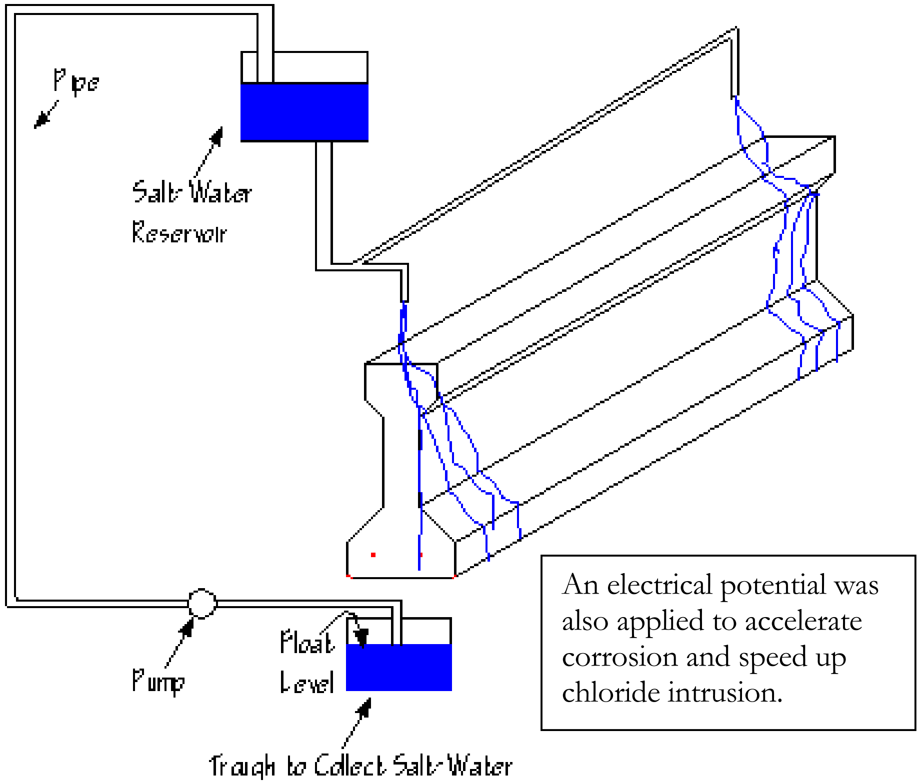

Figure 15. Specimen Exposure

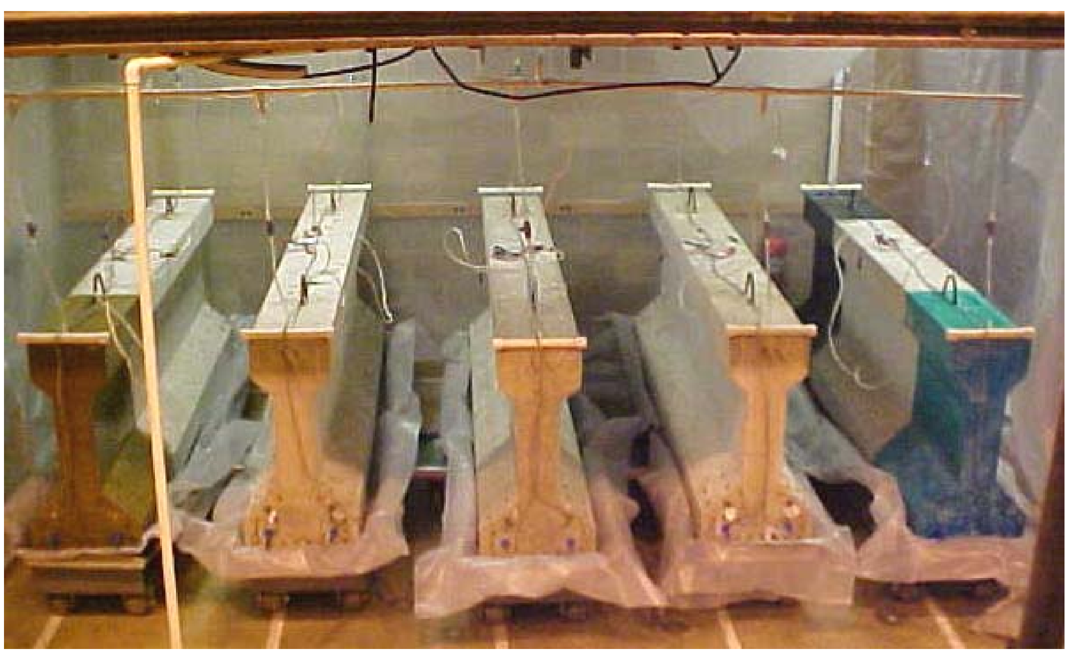

Figure 16. Initial Experimental Setup

After completion of the first accelerated corrosion cycle, the extent of corrosion damage of each beam was evaluated. Since the 6-month exposure did not result in the concrete spalling or significant tendon corrosion, the original exposure regime (Figure 16) was altered slightly to increase the likelihood of corrosion after the second cycle. Figure 17 illustrates the changes made to the system. Pipes (1 foot long) were added along the topsides of each beam end to allow salt water to flow along the side face of the beams. A larger pump was also added to facilitate the increased flow requirements.

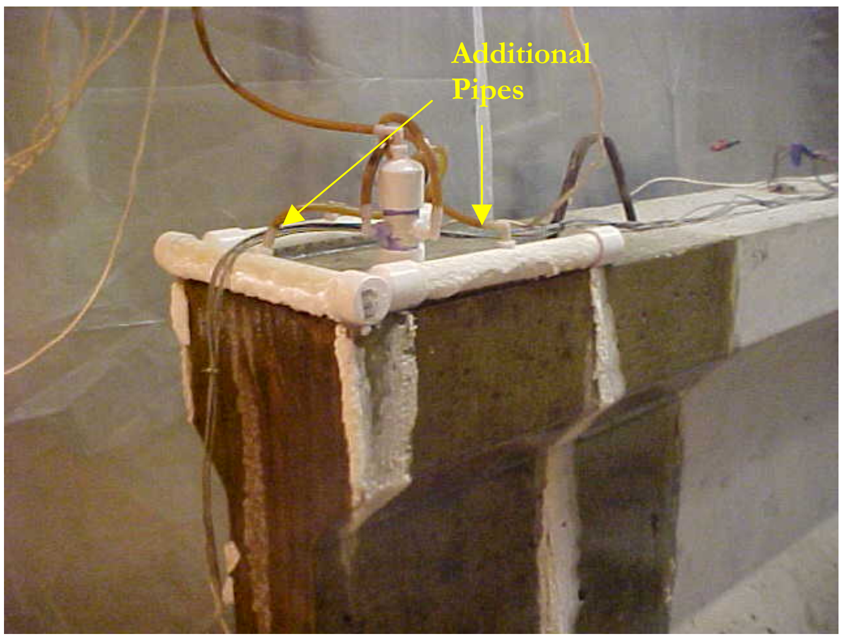

Figure 17. Final Experimental Setup

4.4 ACCLERATED CORRSION TESTING

Since the objective of the experimental program was to study the effectives of various protection systems to prevent or limit corrosion in prestressed concrete bridge beam-ends in a relatively short time period, it was necessary to rapidly induce corrosion in the specimens. All beams were subjected to the same accelerated corrosion regime. Accelerated corrosion was achieved by subjecting the specimens to cyclic wetting and drying, involving a 6% sodium chloride solution, and applying a constant voltage to the steel cage. The specimen ends were exposed to 4 days of salt-water drip, followed by 3 days of no water exposure. Past studies have shown [52] that chlorides can be forced to migrate into concrete at a faster rate under the influence of an applied electric field. The impressed voltage applied to the steel cage attracts the negatively charged chloride ions towards the steel at a higher rate than the chlorides normally diffuse into the concrete. A regulated voltage of 9V was applied across the anode (steel cage) and the two internal cathodes. The schematic drawing of the accelerated corrosion cell is depicted in Figure 18.

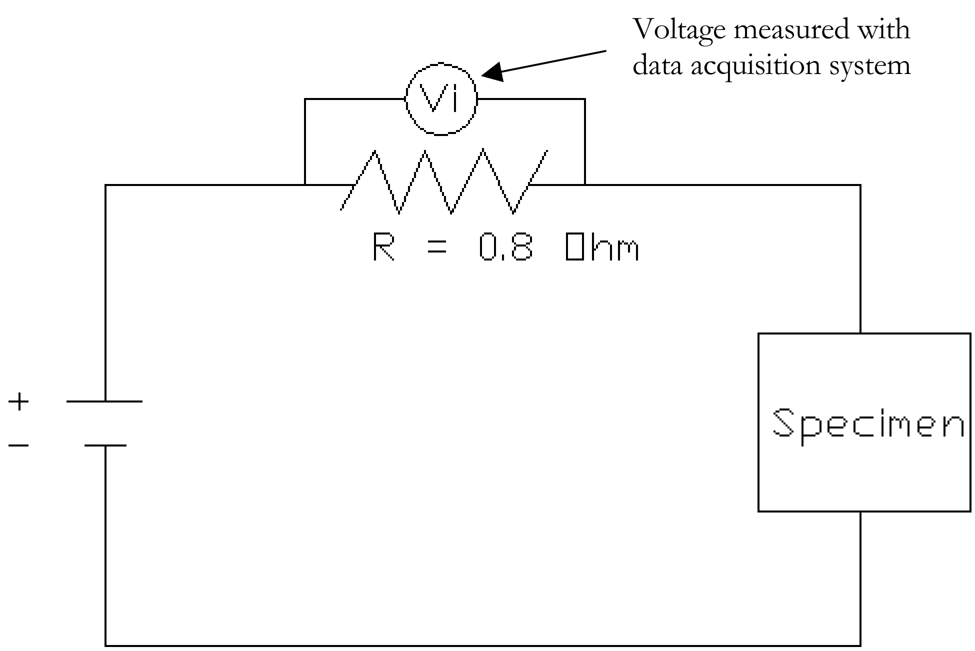

Figure 18. Corrosion Cell [23]

Lee [30] studied a similar accelerated corrosion regime at the University of Toronto. She subjected 12-inch diameter columns to an exposure regime that involved applying a 12V potential to the reinforcement cage of 12-inch diameter columns and subjecting them to 3% sodium chloride solution for wet/dry cycles of 1 day wet and 2 ½ days dry. In addition a study conducted for FHWA [52] implemented a corrosion system exposing the specimens to 15% sodium chloride spray along with applying a current density of 700 mA/m^2. The investigators concluded that the past research supported the effectiveness of the accelerated corrosion regime described above. The total accelerated exposure period was approximately 18 months.

4.5 MONITORING

The corrosion current was monitored continuously throughout the duration of the accelerated corrosion regime with a DATAQ data acquisition system. Figure 19, next page, depicts the circuit used to apply an electric potential to the beams and connect the data acquisition systems to the specimens.

The total steel loss, w_t, (grams) during the given corrosion timeframe can be determined by integrating the curve of corrosion current versus time and using the following equation [30]:

$$w_t = \frac{At_m}{zF} \sum \Delta t I_{ave} \qquad \text{[Eq. 4.5-1]}$$

where At_m is the atomic mass of the metal, z is its valency, F is Faraday's constant (96487 C/mol), Δt the time step, and I_{ave} is the average uniform current measured. For reinforcing steel, which is primarily iron, the atomic mass is 55.85 g/mol and the valency is 2.

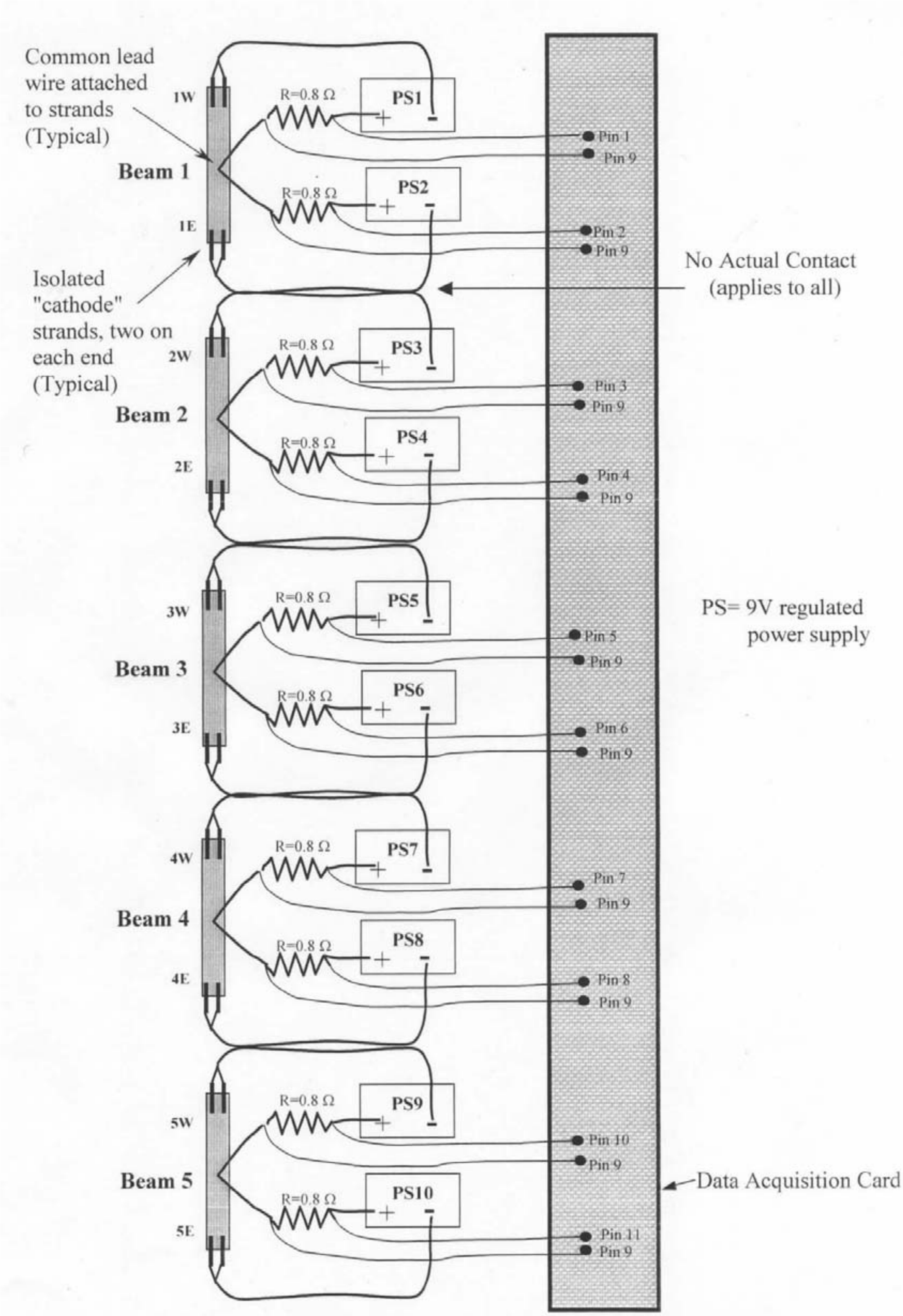

Figure 19. Wiring Diagram and Data Acquisition System

The chloride contents of the unexposed and exposed beams were determined by analyzing pulverized concrete samples at various depths. The initial chloride content measurements were taken at the center of a beam at ¼, ½, ¾, and 7/8 inch depths. The chloride contents after the first 6-month exposure cycle were measured on the bottom flange at 2 inches and 6 inches from the face of the beam at ¼, ½, ¾, and 1 inch depths. At the end of the 1 ½-year exposure period, chloride contents (various depths up to 1 ½ in.) were measured for all beam-ends on the sloping face of the bottom flange at a distance of 2 inches from the beam end.

Periodically, half-cell potential readings were taken. The potential difference between the surface of concrete and strands was detected by placing a copper-copper sulfate half-cell electrode on the concrete surface at different locations and measuring the potential difference between the steel cage and the concrete surface. The reference cell connected the concrete surface to a high-impendence voltmeter, which was also connected electrically to the steel cage. The voltmeter detected the potential difference at the test location. The half-cell reading would indicate the likelihood that corrosion was occurring. Half-cell readings were taken at twelve locations at each end of the beams and at one location in the center of each beam. The measurement points were spaced longitudinally at 6-inch increments and were located at center height of the surface being measured. The measurements were only taken on the non-treated beam-ends since surface treatments provide a non-conductive barrier that renders the half-cell measurements ineffective. A contour plot of the gathered data was developed for each region. Figure 20 illustrates the half-cell measurement locations.

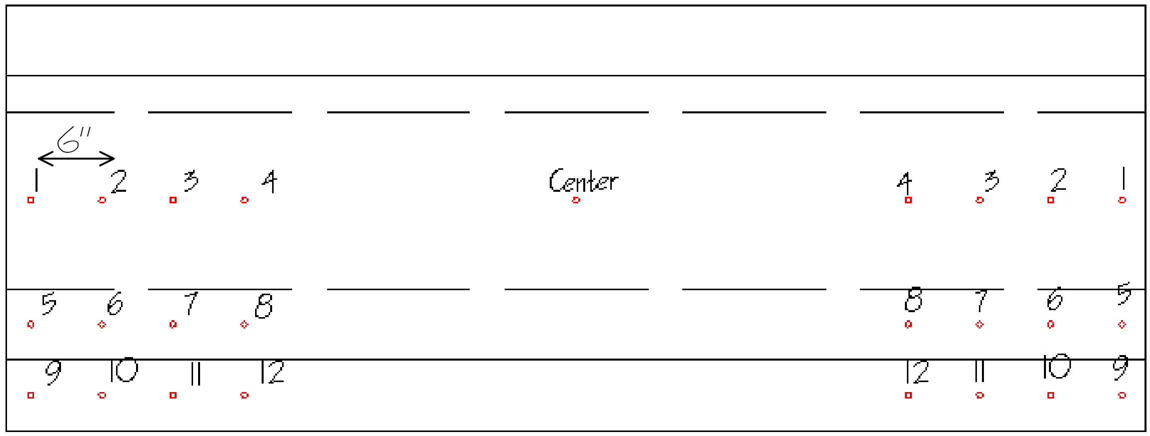

Figure 20. Half-Cell Measurement Point Locations

Expansion measurements were also periodically taken at each beam-end. Ten mechanical measurement points were attached to each side of the beam end at either 4-inch or 2-inch gage lengths. The measurement points were spaced longitudinally at 4-inch increments. A mechanical displacement-measuring device determined the expansion or contraction of the concrete to the nearest 10,000th of an inch. The expansion measurements were compared to readings taken from unexposed and untreated 4-inch and 6-inch cylinders, as well as a metal bar. Figure 21 illustrates the displacement measurement locations.

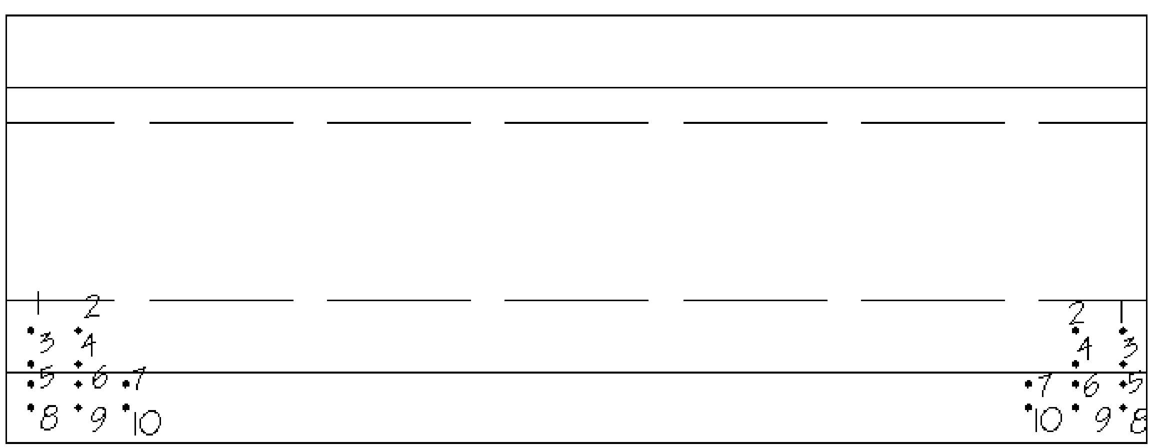

Figure 21. Displacement Measurement Locations

The specimens were visually monitored for cracking and spalling. Detailed crack maps were sketched at the end of each corrosion exposure cycle. The widths of the cracks were measured using a standard crack width comparator.

4.6 REPAIR MATERIALS USED IN THE EXPERIEMENTAL PROGRAM

Selected specimens were designated for pretreatment with one of four surface treatments, while others were left untreated in order to be repaired and/or treated after completion of the first exposure cycle (detailed in test plan discussion). In addition, all concrete beam surfaces were prepared in the same manner prior to the application of treatments, which is detailed in section 4.8.

4.6.1 Carbon Fiber Reinforced Polymer (CFRP)

CFRP sheets were applied to one beam-end prior to the accelerated corrosion regime and to a second beam end after the first exposure cycle of the testing. The system employed was REPLARK 30 manufactured by Mitsubishi Chemical Corporation. It consists of the carbon fiber fabric, primer, putty, and resin. Since the system is lightweight and flexible prior to curing, the sheets can be installed around circular and square surfaces, as well as around irregularly shaped surfaces. In these tests, two fabric/resin layers were installed on the beams, with fiber orientation in the two layers at 90° with respect to each other. Figure 22 illustrates the installation of the FRP system.

Figure 22. Installation of CFRP System

Table 7 summarizes the properties of the carbon fiber sheet reported by the manufacturer. These properties are based on tests performed on laminate samples and are calculated using the net area method. The carbon fibers in the sheets are arranged parallel to one another and are held together with a thin weave of transverse glass fibers. The glass fibers do not contribute to the structural properties of the composite, but maintain the alignment of the carbon fibers during handling and installation. The sheets are also pre-impregnated in the factory with a small amount of resin to restrain the fibers [34].

Table 7. Carbon Fiber Sheet Properties

Properties	REPLARK 30
Fiber Areal Weight (lb/ft^2)	0.061
Thickness (inches)	0.0066
Tensile Strength (psi)	$555x10^3$
Tensile Modulus (psi)	$33.4x10^6$
Standard Width (inches)	13
Standard Length (feet)	328

Table 8 summarizes the properties of the primer, putty, and resin reported by the manufacturer. The primer penetrates into the concrete surface to increase the surface strength of concrete and to improve adhesion between the concrete and the carbon fiber sheet [34]. Primer PS401 is used for warm season applications with temperatures ranging from 68-95°F. The putty is used after the application of the primer to fill small holes, voids, honeycombs, pinholes, and other small surface irregularities to ensure a smooth final surface. The saturating resin is used to impregnate the reinforcing fibers, fix them in place, and provide a shear path to effectively transfer load between fibers and between the concrete substrate and fibers [34]. L700S-LS resin is used for warm season applications with temperatures ranging from 59-95°F.

Table 8. Primer, Putty, And Resin Properties

Property	Primer (PS 401)	Putty	Resin (L700S-LS)
Tensile Strength (psi)			>4200
Flexural Strength (psi)			>5500
Tensile Shear Strength (psi)			>1400
Adhesive Strength (psi)	>200	>200	>200
Compressive Strength (psi)		>7000	

The CFRP composite system is hand applied using a wet lay-up process. Dry, unidirectional, precut sheets of carbon fiber are impregnated with a saturating resin. The saturating resin, putty and primer bond the carbon fiber sheets to the concrete substrate. The laminate is formed using one layer of resin undercoat, one layer of carbon fiber sheet, and one layer of resin overcoat. The material properties of the REPLARK composite system as reported by the manufacturer are listed in Table 9. Section 4.8 details the surface preparation and application procedures implemented in this experiment.

Table 9. REPLARK 30 Composite Properties

Property	REPLARK 30
Thickness (inches)	0.0317
Tensile Strength (psi)	$115x10^3$
Tensile Modulus (psi)	$6.9x10^6$
Minimum Ultimate Breaking Load (lb/in)	3721
Guaranteed Ultimate Breaking Load (lb/in)	3675
Elongation (%)	1.7

4.6.2 Polymer (Resin) Coating

In order to assess the effectiveness of using only the polymer coating (P in FRP) of the composite system, two coats of the resin component of the RELPLARK 30 system (no fiber) were applied to one beam-end prior to the accelerated corrosion regime and to another beam end after the first exposure cycle of the testing. The properties of the resin coating are listed in Table 8 in section 4.6.1. The primer and putty were applied in the same manner as if the complete CFRP system was to be applied. Following the application of putty and primer, the first coat of resin was applied with a paint roller. After the first coat was tack free (3 to 4 hours) a second coat of the resin was applied.

4.6.3 Epoxy Coating

The coating used in this study was MASTERSEAL GP Epoxy Sealer. It is commonly employed to seal concrete surfaces to prevent deterioration such as spalling, scaling, cracking, and leaching. Test conducted by the manufacturer have reportedly shown that the coating could prevent over 94% of the chlorides in salt-laden water from entering concrete [11]. Table 10 summarizes the performance data of the coating as reported by the manufacturer.

Table 10. Coating Performance Data

Property	MASTERSEAL GP Epoxy Sealer
Reduction of water absorption into concrete (Test Procedure, NCHRP study, 12-19A)	91% minimum
Reduction of chloride content in concrete exposure test (Test Procedure, NCHRP study, 12-19A)	94% minimum
Solids (By weight) (By volume)	 50% minimum 58% minimum
Viscosity (mixed)	15 to 40 cps

MASTERSEAL GP could be applied with a squeegee, roller, or spray equipment to a clean, dry surface. A second coat was applied after the first coat became tack free (3 to 4 hours). Section 4.8 details the surface preparation and application procedures implemented in this experiment.

4.6.4 Sealer

The sealer used in this study was MASTERSEAL SL 40 VOC, a solvent based VOC-compliant silane penetrating sealer. The product creates a water repellent concrete surface, but still permits the concrete to breath. In addition, since it penetrates into the substrate, it generally does not alter the appearance of the concrete. Lastly, the manufacturer states that the sealer helps reduce efflorescence, atmospheric staining, and protects against damage caused by chloride intrusion [12]. Table 11 summarizes the performance data of the sealer, as reported by the manufacturer.

Table 11. Sealer Performance Data

Property	MASTERSEAL SL 40 VOC
Resistance to chloride (AASHTO T259 and T260)	Less than 0.22 lbs/yd^3 (criteria of 1.5) at ½" level Less than 0.00 lbs/yd^3 (criteria of 0.75) at 1" level
Average depth of penetration	0.22 inches (depending on substrate)
Water weight gain Absorbed Chloride (NCHRP 244 Series II Cube Test - 200ft^2/gal)	86% reduction – exceeds criteria 92% reduction – exceeds criteria
Moisture vapor transmission rate (OHD-L-35)	102%

The sealer was applied using a roller and paintbrush. Two coats were applied from the base of the beam up to ensure uniform distribution of the sealer. Section 4.8 details the surface preparation and application procedures implemented in this experiment.

4.6.5 Patching

In addition to surface treatments, the effectiveness of a patch repairs was also studied. Patch repairs involve removing portions of concrete and replacing it with some type of cement-based patching material. This type of repair is commonly used when large spalled or deteriorated regions need to be removed and repaired. Since spalling had not taken place at the time of patching, an area of the bottom flange in one previously untreated beam was removed to represent a spalled region. Section 4.8 details the surface preparation and application procedures implemented in this experiment. The patch material used in this study was "Vericoat Supreme", a one component, microsilica and latex modified, nonsag repair mortar produced by Euclid Chemical Company. This cement-based product is designed for trowel applied vertical and overhead repairs. Table 12 summarizes the properties of Vericoat Supreme as reported by the manufacturer [17].

Table 12. Vericoat Supreme Mechanical Properties

Property (28 day)	Vericoat Supreme
Compressive Strength (psi)	6200
Bond Strength (psi)	2100
Direct Tensile Bond Strength (psi)	310
Flexural Strength (psi)	650
Linear Shrinkage	-0.04%
Sulfate Resistance	+0.005%
Chloride Permeability (coulombs)	900
Working Time	30 minutes
Set Times (@ 70° F) Initial Set (hours) Final Set (hours)	 1 2 ½

Before application of the patch material a bond agent was applied to both the concrete and exposed steel surfaces. The bonding agent used in this study was "CORR-BOND"; a three part bonding agent composed of specialty water based epoxy and selected cementitious components produced by the Euclid Chemical Company. According to the manufacturer, this product facilitates a stronger bond between the existing and new concrete and provides protection against steel reinforcement corrosion. Table 13 lists the technical information of the bonding agent as reported by the manufacturer [16].

Table 13. CORR-BOND Technical Information

Property	CORR-BOND
Application Thickness (mils)	20
Slant Shear Bond to Concrete (psi) Open Time* 0 hours 12 hours	 2000 1950
Direct Tensile Bond to Concrete (psi) Open Time* 0 hours 12 hours	 400 350
7-Day Bond Strength (psi) (to wire brushed steel)	650

*Open Time: Time from the application of the COOR_BOND on 14-day old, hardened concrete until placement of the fresh concrete topping over CORR_BOND.

4.7 TEST PLAN

The test plan, detailed in Table 14 and illustrated in Figure 23 and 24, was employed to determine the effectiveness of various treatments to prevent prestressing steel corrosion. Two repair schemes were evaluated in this study. The first involved repairs where no concrete was removed and only a surface treatment was applied. Some specimens were treated with an epoxy coating, sealer, polymer coating, or CFRP composite wrap. The second repair scheme involved repairs where portions of concrete were removed and replaced with a patch material. Figure 25 illustrates the time period and repair method for each beam. End "A" indicates the west end of the beams and end "B" indicates the east end of the beams as they sat in the UWM Structural Laboratory.

Table 14. Laboratory Test Plan

Treatment \ Beam-End	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Prior to Exposure										
Pre-Coated (epoxy coating)										
Pre-Sealed (silane sealer)										
Pre-FRP Wrap										
Pre-Polymer Coating (resin)										
No initial Treatment										
After Exposure Cycle										
Coating (Epoxy coating)										
Sealer (silane)										
FRP Wrap										
Polymer Coating (resin)										
Patch Repair Only										
Do Nothing										

Phase I

Phase II

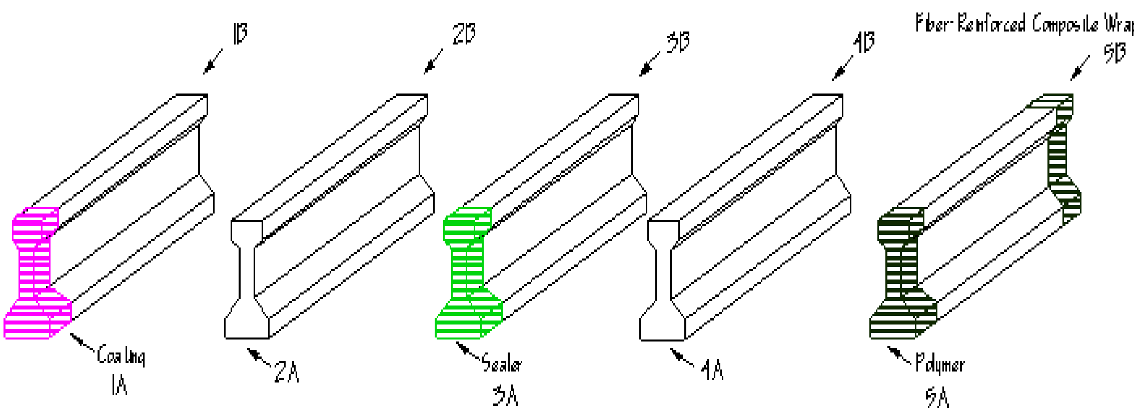

Figure 23. Laboratory Set-up Prior to Accelerated Corrosion

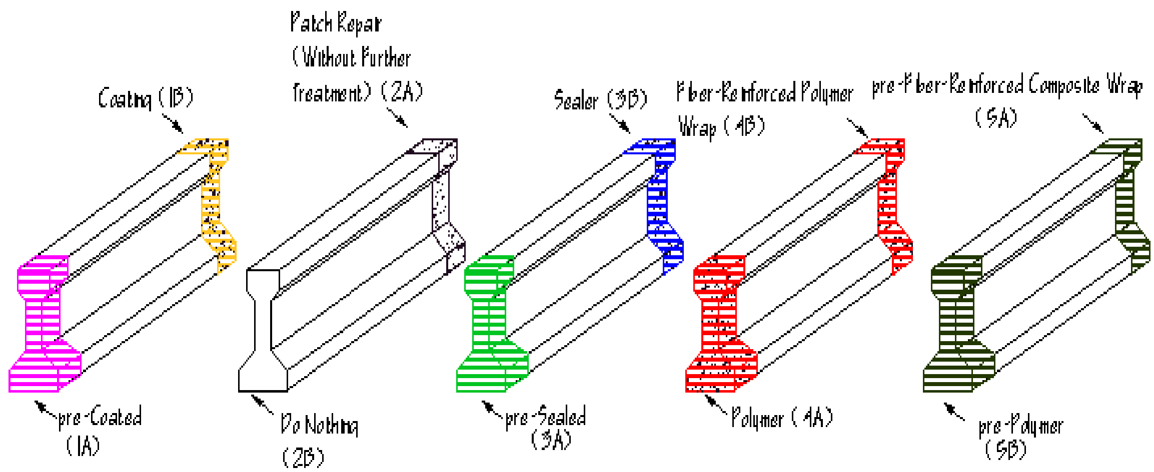

Figure 24. Laboratory Set-up After First Phase of Accelerated Corrosion

Beam End	→ 6 MONTHS →	→ 1 YEAR →		
1A	Epoxy Coating	No Additional Treatment	Dissection	
1B	No Treatment	Epoxy Coating	Dissection	
2A	No Treatment	Do Nothing	Dissection	← CONTROL
2B	No Treatment	Patch	Dissection	
3A	Sealer	No Additional Treatment	Dissection	
3B	No Treatment	Sealer	Dissection	
4A	No Treatment	Polymer Coating	Dissection	
4B	No Treatment	CFRP Wrap	Dissection	
5A	Polymer Coating	No Additional Treatment	Dissection	
5B	CFRP Wrap	No Additional Treatment	Dissection	

Figure 25. Repair Method & Time Period for Each Beam End

4.8 SURFACE PREPARATION & TREATMENT APPLICATIONS

Before exposure to the accelerated corrosive environment, four beam-ends (2-foot long sections in each beam end) were pre-treated with each type of surface treatment (i.e. silane sealer, epoxy coating, polymer resin coating, CFRP wrap). The surfaces were prepared by first grinding the surfaces of concrete, followed by thoroughly washing the surfaces to remove all accumulated dust and debris. After the surfaces were dry, an air hose was used to remove any remaining particles. The 2-foot long end sections for each beam end received their surface treatment. Manufacturer's instructions were followed in the application of the treatments. Table 15 summarizes the application rates and procedural notes of each material. The epoxy coating, resin coating, and silane sealer were applied with a paint roller.

Table 15. Surface Treatment Application Information

Surface Treatment	Notes
Epoxy Coating	Applied 2 coats
Silane Sealer	Applied 2 coats
Polymer Coating (resin)	Applied 2 coats after application of primer and putty
CFRP Wrap	Applied 2 layers (resin-sheet-resin-sheet-resin) after application of primer and putty

After exposure to the accelerated corrosive environment (over six months of exposure), the specimens subjected to surface treatments were allowed to completely dry. The same surface preparation and application procedure as stated previously was followed for the application of the various surface treatments.

Since the 6-month exposure did not result in spalling of concrete, it was determined that an 18-inch long concrete region was to be removed (Figure 26) for installation of the patch repair. A masonry saw was used to cut around the perimeter of the repair area to a depth of ½ inch at a 90° angle to the surface. A series of cuts were made inside the repair region to allow for removal of the concrete with a chipping hammer. The chipping hammer was used to chip out the concrete in the repair area allowing for a ¾ inch clearance behind the first layer of strands exposed. After all the concrete was removed from the repair region, the concrete and steel surfaces were cleaned with a wire brush followed by cleaning with an air hose to remove any loose particles or debris. The surfaces of both the steel and concrete were covered with two coats of a bonding agent (CORR-BOND). The patch material was installed by a trowel. The region was then moist cured under wet burlap and covered by plastic for 3 days.

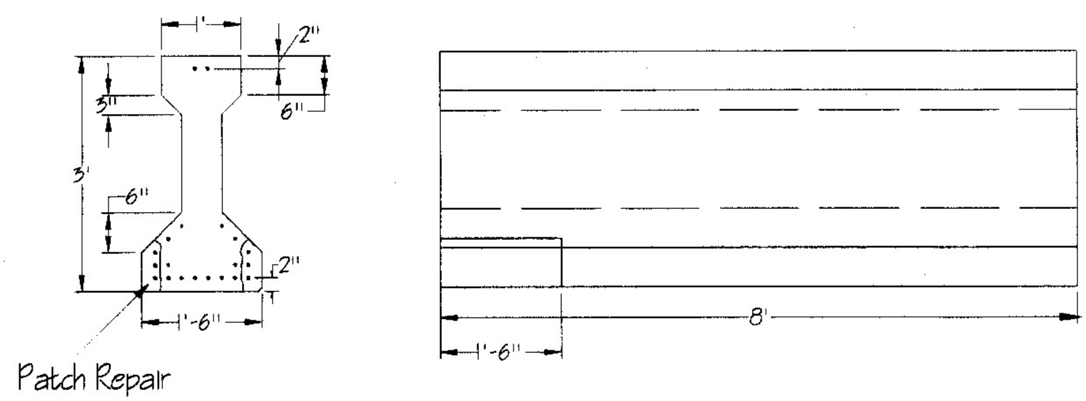

Figure 26. Beam Cross-Section with Patch Repair

All specimens were returned to the test area after the repairs were made and the surface treatments applied. The accelerated corrosion current and the salt-water exposure were re-initiated once the entire salt-water system had been cleaned and re-tested. The results of the monitoring program are presented in Chapter 5 of this report