Survey of Costs, Economic Analysis, and Design Guidelines for Corrosion Protection Methods for Post-Tensioned Concrete Bridges

by

Kimberly Ann Grau, B.S.

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Survey of Costs, Economic Analysis, and Design Guidelines for Corrosion Protection Methods for Post-Tensioned Concrete Bridges

APPROVED BY
SUPERVISING COMMITTEE:

John E. Breen, Supervisor

Sharon L. Wood

To my family,

for their continual love and support.

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Kimberly Ann Grau, M.S.E.
The University of Texas at Austin, 2005
SUPERVISOR: John E. Breen

At the design stage, engineers and owners have the most influence on the future durability of post-tensioned concrete bridges. Yet design guidelines are not clear on what corrosion protection methods are appropriate for a given exposure environment. Current specifications do not ignore corrosion protection. However, many simply instruct the designer to protect the anchorage and prestressing steel from corrosion, leaving the selection of the appropriate method to the engineer's experience. As well, many decision-makers do not have a good understanding of how post-tensioning works, how it can be protected from

corrosion, or how much it will cost to increase the durability of the post-tensioning system.

This thesis includes a description of post-tensioning systems and corrosion protection methods as a reference for those not intimately familiar with the mechanisms and protection approaches. It gives the results of an industry cost survey that shows the expected cost increases, as a percentage of total initial construction cost, from including corrosion protection in the design. As well, life cycle analyses for several methods show their potential to save the owner money over the bridge's expected life.

Additionally, this thesis examines several current or proposed design specifications for their corrosion protection guidance. A new combination of these guidelines is proposed to give a more logical and designer-friendly option.

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CHAPTER 1
Overview

1.1 BACKGROUND

The current practice in the United States is to design bridges for estimated seventy five to one hundred year serviceable lives. In order to make this design life expectation possible, engineers must concern themselves with the durability of their designs. In concrete construction, the most common attack mechanism that affects durability is corrosion. Traditionally reinforced concrete has shown substantial corrosion distress[1]. Although pretensioned concrete has rarely had any reported corrosion problems, post-tensioned concrete has shown some corrosion distress in field structures[2,3]. Most of the corrosion is from the deterioration of bridge decks and substructures in reinforced concrete bridges due to saltwater exposure or the use of deicing salts. Thus, concrete reinforcement corrosion models and protection products were developed primarily to address the durability of reinforced concrete structures, not prestressed concrete structures.

Post-tensioned concrete does not experience nearly the amount of corrosion seen in nonprestressed reinforced concrete structures[1,4]. Though corrosion has been observed in post-tensioning systems, in the US there has been no known collapse of a post-tensioned bridge due to corrosion. A number of corrosion protection methods for post-tensioning have been developed from research studies and from field experience. Designers are sometimes wary of using advanced corrosion protection methods due to the perception that using additional protection layers will dramatically increase cost. Also, many design guidelines do not directly specify what system to use based upon the given exposure. This is not to say that durability is not addressed in the specifications. Severe exposure conditions are discussed and often direct the designer to protect the tendon and anchorages from corrosion. Designers generally have to rely on experience to know what protection methods are appropriate in a given setting.

Therefore, it would be helpful to owners and designers to have a better understanding of the cost increases associated with improved corrosion protection. In order for structures to be economically protected, it would further aid owners to understand life cycle analysis, which provides an economic evaluation of the protection methods. As well, a more direct guide to the selection of post-tensioning protection methods can help owners and designers to evaluate what design options best suit their needs.

1.2 OBJECTIVES

This thesis has the following objectives:

- Determine the cost increase, as a percentage of total construction costs, of post-tensioning corrosion protection methods.
- Discuss the corrosion of post-tensioning systems and various corrosion protection methods as a basic guide for owners and designers who are not intimately familiar with the underlying mechanisms and protection approaches.
- Evaluate the effect of using post-tensioning corrosion protection methods on annualized total bridge cost by running life cycle analyses.

- Examine post-tensioning durability design guidelines and suggest a more logical and designer-friendly way to combine the currently available guidelines.

1.3 ORGANIZATION

Chapter 2 of this thesis gives a detailed background of post-tensioning, including a discussion of how the system works, corrosion in post-tensioning, past problems in field structures, and previous research on post-tensioning durability. A description of available corrosion protection methods is given in Chapter 3. The economic analysis used to select the most cost effective options is given in Chapter 4. Chapter 5 discusses current durability guidelines and suggests a new combination of these options. Chapter 6 gives the results from the cost finding initiative and life cycle analyses. Chapter 7 is a concise listing of the conclusions in this thesis.

CHAPTER 2

Detailed Background

2.1 General use of Post-Tensioning in bridges

2.1.1 Reinforcing Concrete

Concrete is weaker in tension than in compression; it has only 10% of its compressive strength as tensile strength. To increase tensile capacity, concrete is paired with another material, usually steel. Steel has been the material of choice for this pairing because steel provides a dependable high tensile strength and, from the deformations on ordinary reinforcing bars, has good bond with the concrete. This reinforcing works to control the cracking in a reinforced concrete member, though it does not eliminate cracking. For example, when a concrete beam is loaded from above, the loading puts the top of the beam in compression and the bottom in tension as illustrated in Figure 2.1. The concrete surrounding the steel on the tensile side carries the tensile stresses until the stress exceeds the concrete's tensile capacity. When the tensile stresses reach this point the concrete cracks. If the beam was not reinforced, only a single, very wide, crack would form and the beam would fail in an immediate collapse under the cracking load. With reinforcing, the steel's bond to the concrete essentially transfers the concrete tensile stresses to the steel bar and the steel holds the concrete in place even when cracks are present. Thus the tensile side of a reinforced section will have more cracks than an unreinforced section, but they will be smaller and be well distributed throughout the high tensile stress region. Beyond acting as connector between the pieces of concrete, the reinforcing steel is carrying the tensile stress in the beam. Thus a reinforced concrete beam can carry additional load versus the unreinforced beam.

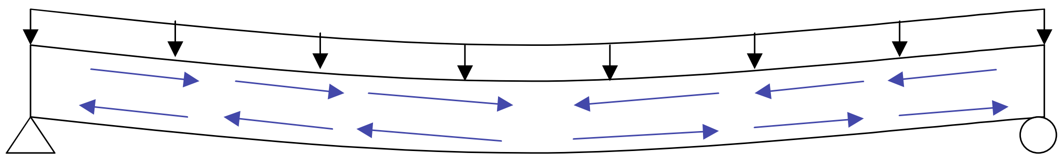

Figure 2.1: Internal Forces in a Simply Supported Beam With Uniform Load

2.1.2 Prestressing as Concrete Reinforcement

Prestressing methods have been developed in order to further utilize the compressive strength of concrete and take advantage of the development of high strength steel. These methods put the concrete into compression before the member enters service or carries other loads. Post-tensioning and pretensioning are the two methods of prestressing, with the prefixes referring to the time of stressing in relation to the development of the concrete strength. Stressing is a term that describes the stretching of the steel like a rubber band until it carries a given load.

In the case of pretensioning, the steel is stretched before the concrete is placed and then the concrete is cast around the stretched steel. When the concrete has achieved a minimum specified strength the steel is released. Like a rubber band that is let go after being stretched, the steel wants to return to its original state. The concrete, however, has bonded to the length of the strand and tries to hold the steel in its stressed position, as illustrated by Figure 2.2. Equilibrium is reached between the length change in the steel and the concrete. In this interaction between the steel cable and the concrete, the steel loses some of its initial tensile stress but maintains a substantial tensile stress while the concrete ends up in compression[5].

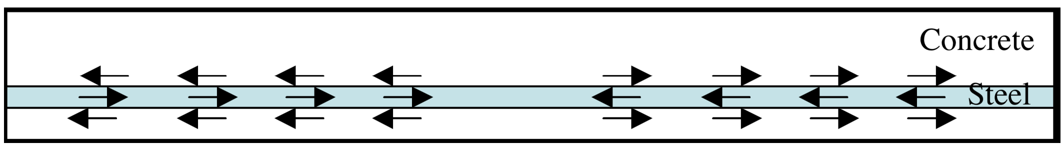

Figure 2.2: Concrete Restrains the Steel Retraction in Pretensioning

In post-tensioning the stretching of the steel is similar to the previous description only instead of being stressed before the concrete is placed the steel is stressed after the concrete is cured. This sequence is possible because there are ducts cast into the concrete when post-tensioning is desired. The steel strand is run through the ducts and then stressed after the concrete had reached a desired maturity. The restraint against the retraction of the steel is not from the bond between the concrete and steel. Instead the steel strand is secured at the far ends of the member by anchor heads[5] as shown in Figure 2.3. These anchor heads have conical holes for the strand to run through and for the seating of wedges that grip the strand and prevent the movement of the strand. The anchor heads bear on and compresses the concrete when the steel strands are stressed. Figure 2.4 illustrates the basic layout of an anchorage assembly.

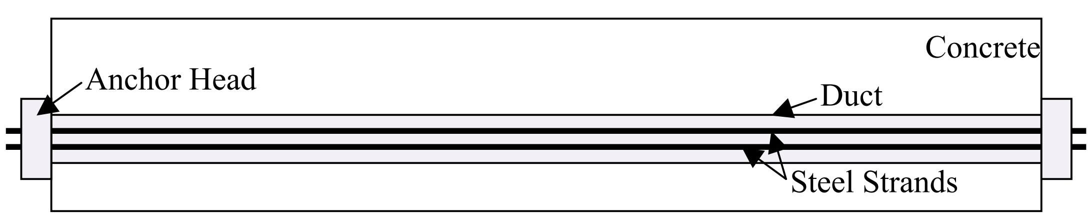

Figure 2.3: Basic Post-Tensioning Layout

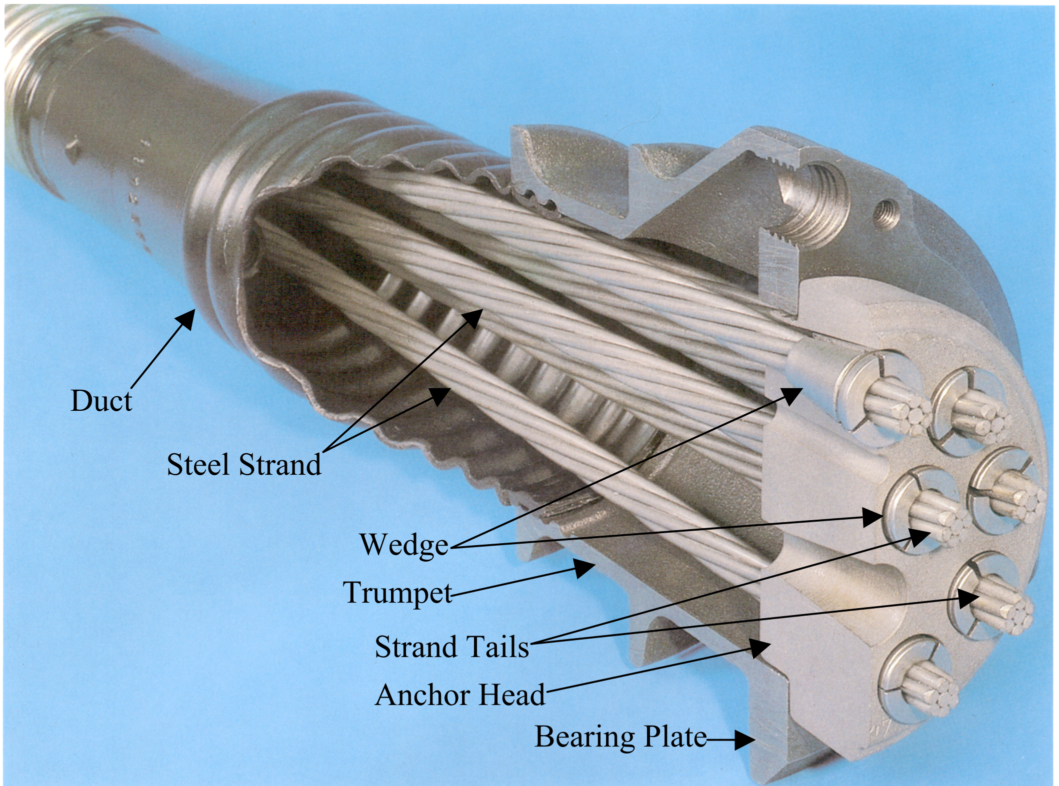

***Figure 2.4 Post-Tensioning Anchorage Assembly*[6]**

Post-tensioning has a number of different forms. The ducts can be either completely inside the concrete element, which is called internal post-tensioning, or outside of the concrete but inside the envelope of the section, which is called external post-tensioning. External post-tensioned members are often box-like, hollow sections where the ducts run inside the hollow portion though not entirely within the concrete. The interior of an externally post-tensioned box girder is shown in Figure 2.5. Both the internal and external post-tensioning can be designed as either a bonded or unbonded system. In bonded tendon applications, after the tendons have been stressed grout is injected into the remaining space in the ducts. This step provides bond between the steel strands and the duct and can act as a layer of corrosion protection. Especially in internal systems, this bonding helps to transfer further stresses developed in the strand from loading to the surrounding concrete throughout its length. In external post-tensioning applications the bonding could be more accurately called partially bonded as the duct is only in contact with the concrete section at a few discrete points along the tendon length. Post-tensioning methods generally use groups of steel strands rather than deformed steel bar although there are post-tensioning bars available, which can be used for straight sections[5].

Figure 2.5 Interior of an Externally Post-Tensioned Box Girder[7]

The induced compression of the concrete from prestressing helps to greatly reduce concrete cracking and deformations under service loads. These methods allow the concrete to carry a higher service load than conventionally reinforced concrete[5]. The stress felt by the concrete can be viewed as a summation of the stresses induced by each individually applied force. The prestressing induces a compressive force in a simply-supported beam, including on the bottom face. When a uniformly applied load on a simply-supported beam is considered, it induces compression at the top and tension on the bottom face. For prestressed concrete to reach cracking, enough load has to be applied so that the induced tension on the bottom face overcomes all of the initial compression in addition to the load that would have caused a conventionally reinforced section to crack. From this brief example, which is illustrated in Figure 2.6, it can be seen that a prestressed member can carry a significantly higher cracking load than traditional reinforced concrete, though how much more load is dependant on the level of prestressing.

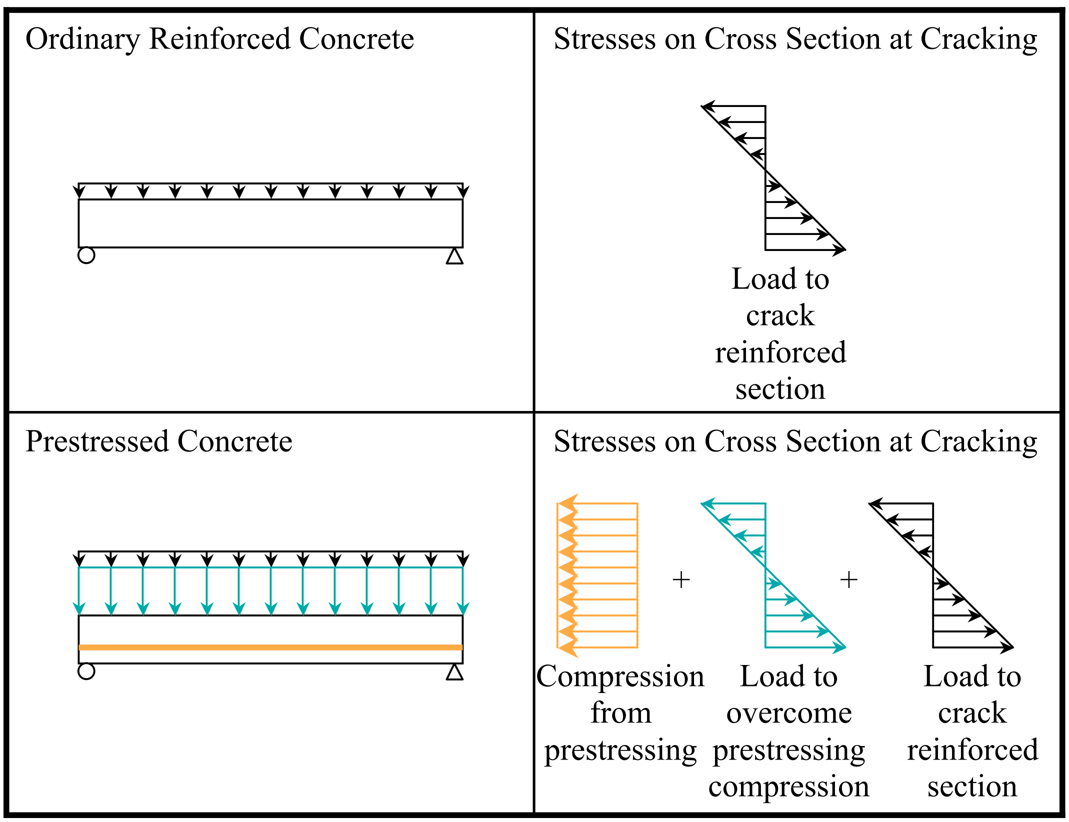

Figure 2.6: Comparison of Load Capacity

Post-tensioning methods are also used to take separate precast elements and hold them together so that they will act as a single member. These advantages allow the use of thinner profiles, longer spans on concrete bridges, and greater distances between support columns in a concrete framed structure. Precast segmental bridges can be constructed over existing roadways without disrupting traffic below due to the ease and erection speed possible through the use of post-tensioning[5]. In all of the uses of prestressing, it remains vital that the steel continue to carry load. As a result a large problem for the nation's bridges and the people who maintain them is that most steels can corrode or rust.

2.1.3 A Brief History of Post-Tensioning

Post-tensioning (PT) is still a relatively new concrete construction method. Eugene Freyssinet, a French engineer, first thought of post-tensioning in 1904 while giving lectures at the Ecole des Ponts et Chaussees[8]. The first successful use of PT in a major project came in 1935[8]. As early as 1907 Freyssinet was using post-tensioning to aid the construction of arch bridges such as the Bernard Arch (1908) and the bridge at Le Veurdre (1912). These projects made Freyssinet, and, as a result, much of the engineering world, aware of creep in concrete. The bridges had required extra stressing to counteract higher than expected deflections, which were caused by creep[8].

Engineers had conceived of the post-tensioning method as early as 1886, when an engineer in California patented a method for stressing steel rods in floor slabs[9]. These early attempts were not successful because the creep and shrinkage negated all of the induced post-tensioning in low-strength steel rods. When concrete shrinks and creeps under load the member shortens and thus decreases the net stresses that were induced by post-tensioning. The effect is illustrated in Figure 2.7 from T.Y. Lin's textbook on Prestressed Concrete. Without the induced compressive force from post-tensioning, the concrete is unable to carry the higher loads for which it is designed.

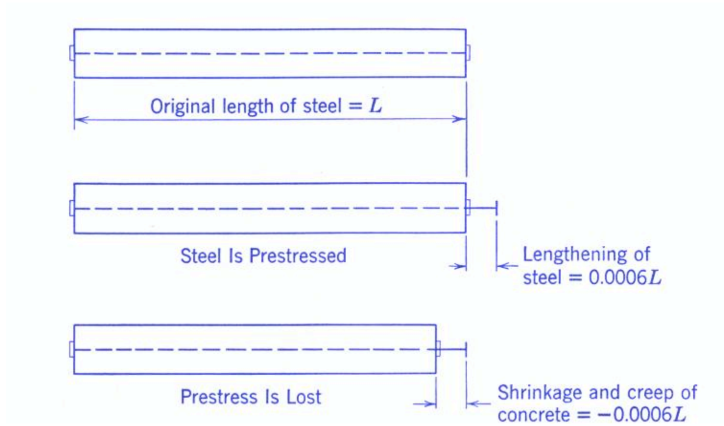

Figure 2.7 Prestressing with Ordinary Reinforcing Steel[9]

Freyssinet was able of overcome the loss of prestressing caused by the shortening of the concrete by using steel cables rather than ordinary steel bars. These cables could be stretched to 0.7% of their original length, which is much greater than the 0.15% that the bars could tolerate. An expected shortening of the concrete can be assumed at 0.1% of its original length. In the cable post-tensioned members there is six-sevenths of the original stress remaining, whereas in the post-tensioned ordinary bar it has lost two-thirds of its stress[5]. In Lin's textbook he used similar numbers to illustrate this point as seen in Figure 2.8[9]. These examples show that only by use of the high strength steel cables could post-tensioning succeed.

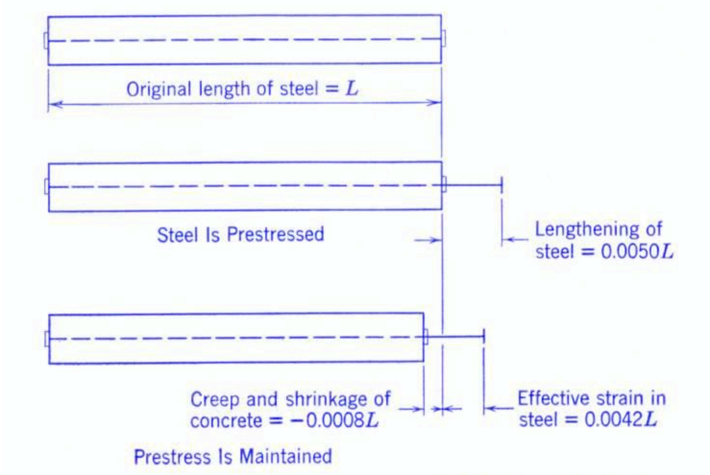

Figure 2.8 Prestressing with High Strength Steel[9]

As Freyssinet was French, it is no surprise that post-tensioning first came into use in Europe. The method gained acceptance for use in major projects after Freyssinet used it to arrest the sinking of the Martime Terminal at Le Havre in 1935[8]. Post-tensioning gained greater importance in 1945. The timing is possibly due to steel shortages during the reconstruction effort after World War II, as post-tensioning requires less area or volume of steel to reinforce a member than ordinary reinforcing[9]. The technology soon spread to the US where the method was first used for concrete storage tank construction[9]. The Walnut Lane Bridge in Philadelphia, featuring a main span of 160 feet, was the first major bridge in the US designed with post-tensioning. Although this bridge's construction began in 1949, a bridge in Madison County, Tennessee was the first post-tensioned bridge to open to traffic in 1950[9]. Other early examples of post-tensioning in the US include post-tensioning precast segments to form the pilings for the Lake Pontchartrain Causeway (1956 and 1969) and the post-tensioned girders supporting the roof of the Lyndon Baines Johnson Presidential Library (1966) in Austin, Texas.

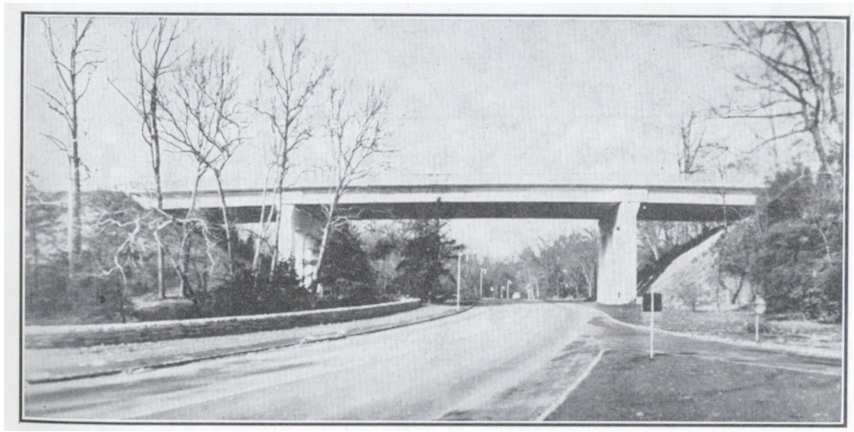

Figure 2.9 Walnut Lane Bridge[9]

Prestressed concrete is growing in popularity as a construction method. A 2001 study showed that prestressed bridges make up 18% of the total US bridge inventory. The majority of these bridges were constructed after 1960. As approximately 50% of all existing US bridges were constructed before 1950, the year the first US prestressed bridge went into service[1], prestressed bridges represent over one-third of all the bridges constructed since the method came into use in the US. This figure represents both pretensioned and post-tensioned bridges. For short to mid-length spans, pretensioned I-girders are more economical and thus the more popular prestressing system. As a result there are a greater number of new pretensioned bridges than post-tensioned bridges constructed. As of 1990 more than half of all new bridge construction used prestressing and approximately one-third of all the prestressing steel used in the US was consumed through post-tensioning construction. This value includes building construction, where post-tensioning is a popular method to reinforce slabs, including in residential construction. In all other developed countries, post-tensioning averages 66% of the total prestressing steel consumption[5]. Thus it can be seen that although the post-tensioning industry is relatively young it is a growing industry. The industry is constantly trying to improve quality of construction and durability of the system. Through research and examination of field performance, durability requirements for post-tensioned construction have improved. However, the importance of these improvements becomes more evident as the early post-tensioned bridges age. These bridges were built with little understanding of the long-term behavior of the systems and frequently were designed under the belief that using post-tensioning would eliminate all cracking in the concrete and make the bridges impervious to salt water and deicing salts. Thus there was not, initially, an emphasis on the design of corrosion resistant post-tensioning systems.

2.2 CORROSION IN CONCRETE

2.2.1 Basic Mechanism

Corrosion in steel, of which rust is the most common form, is the electrochemical reaction of the steel with water and oxygen that leads to the formation of corrosion product. The corrosion rate increases when in the presence of chloride ions. Calcium hydroxide, $Ca(OH)_2$, from concrete paste enters into an aqueous solution in the presence of moisture in concrete. The compound disassociates in water to form an alkaline pore solution. In the high pH environment that results, the pH averages between 12.5 and 13.8 and the embedded reinforcing steel forms a passive film that resists corrosion[10]. Over a very long time, if the concrete has access to oxygen and remains partially saturated, this passive film will slowly dissolve as shown in Equation 2.1[10]. The hydrated passive film is shown here as FeOOH:

$$FeOOH + H_2O \rightarrow Fe^{3+} + 3OH^- \qquad \textbf{Equation 2.1}$$

The passive film can have other chemical compositions such as $Fe(OH)_2$ or FeO_2[10]. Beyond dissolving, the film can react with water and oxygen to form corrosion product. Basic rust is formed through the following reaction series[11]:

$$2Fe + 2H_2O + O_2 \rightarrow 2Fe(OH)_2 \qquad \textbf{Equation 2.2}$$

$$2Fe(OH)_2 + H_2O + 1/2O_2 \rightarrow 2Fe(OH)_3 \qquad \textbf{Equation 2.3}$$

When chloride ions, Cl^-, are present at a sufficient concentration, they attack the steel's passive film and any exposed steel. The concentration required for chloride induced corrosion initiation is called the threshold concentration. The threshold value is dependent upon numerous variables including concrete permeability, carbonation, and age of the concrete. The chlorides form a salt with the passive film that can then dissolve and depassivate the steel as seen in equations 2.4 and 2.5[10]:

$$FeOOH + Cl^- \rightarrow FeOCl + OH^- \qquad \textbf{Equation 2.4}$$

$$FeOCl + H_2O \rightarrow Fe^{3+} + Cl^- + 2OH^- \qquad \textbf{Equation 2.5}$$

In this dissolution the chloride ion is released back into solution to continue its attack on the steel. When the unprotected steel encounters the chloride, it reacts to form a weak base and a strong acid, which has the net effect of decreasing the pore solution pH and thus increasing the corrosion rate[10]. The attack mechanism is shown in Equation 2.6[10]:

$$Fe^{2+} + 2H_2O + 2Cl^- \rightarrow Fe(OH)_2 + 2HCl \qquad \textbf{Equation 2.6}$$

Corrosion products have many chemical compositions, which are beyond the scope of this paper to examine. Even the passive film on the steel is comprised of corrosion product. The reason a particular product behaves as a passive film is that at a high pH the product formed on the steel surface does not undergo additional reactions to release it into solution. As a result the product is a stable covering on the steel that prevents additional corrosion reactions. Thus, when the pH decreases the passive film dissolves or reacts and the underlying steel is available for corrosion reactions. If the pH drop is minor, the disassociated OH^- from the corrosion product can help regain a high pH and the passive film regenerates. If the pH drop is great, especially when caused by the carbonation of concrete or coupled with the introduction of chlorides, the passive film does not regenerate and significant section loss can take place.

Another factor contributing to the corrosion of steel in concrete is carbonation. Over time, carbon dioxide reacts with the concrete pore solution to form calcium carbonate as shown in Equation 2.7:

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O \qquad \textit{Equation 2.7}$$

The resulting consumption of hydroxyl ions from the pore solution decreases the pH of the concrete to an average level of pH 8 to 9. This new level is below the passive region and increases the corrosion rate of the steel[10]. Carbonation can take considerable time however, as it is dependent on such variables as the relative humidity and permeability of concrete and thus is not as significant to corrosion in concrete as the presence of chlorides.

The two main problems with corrosion occurring are that corrosion reduces the steel's cross sectional area, and thus the steel's load capacity, and that the corrosion products are always greater in volume than the steel consumed in the reaction[10]. Thus, the corrosion process results in an increase of volume within the concrete. The initial products are able to migrate into the interstitial spaces of the concrete. After that, the growing product builds pressure within the concrete as it tries to expand but has nowhere to go. When the pressure exceeds the concrete's tensile strength, the concrete cracks[10]. It was observed in an earlier University of Texas at Austin study that the concrete developed cracks parallel to the steel corroding as a result of this volumetric expansion[11]. The cracks also allow water and chlorides to more easily reach the steel and thus increase the corrosion rate. The cracking can escalate to the spalling of the concrete cover. Eventually the steel section is so reduced that it can no longer carry its load and it breaks. At the point where the concrete is spalled and the steel is broken the member is in danger of collapse from insufficient remaining capacity.

2.2.2 Corrosion in Post-Tensioned Systems

In post-tensioned structures, the chlorides and water must get through not only the concrete but also through the duct and grout layers before they can initiate corrosion in the post-tensioning steel strand. Excess water or voids in the grout could induce corrosion of the steel strand. The anchorages are also vulnerable to corrosion. They can be compromised through corrosion of their components such as the gripping wedges, anchor head, and bearing plate and trumpet assembly. Due to the long use of steel duct in post-tensioning systems, and this duct type's poor performance in aggressive environments[11], there is no permanent barrier to prevent chloride ingress to the steel strand in many existing structures. Newer plastic ducts that, when defect free, prevent chloride ingress are increasingly used in new construction. In the case of steel ducts, a possible corrosion route would be the duct corroding through and then the chloride migration continuing through cracks or flaws in the grout until the threshold concentration is reached at the strand. Other weak points on steel duct systems include the splices between two duct sections, the anchorages, and grout vents.

Due to the nature of post-tensioning use, loss of steel cross section can have more significant results than in traditional reinforced concrete[1]. Post-tensioning allows a concrete member to carry a greater service load and to span greater distances than traditional reinforced concrete. Since the post-tensioning strand has a higher strength than reinforcing bar (270 ksi versus 60 ksi), the strand carries more force per unit of cross section than the bar. Thus, as a given amount of the cross section is lost, the post-tensioned member loses more structural capacity than the ordinary reinforced concrete member.

In precast segmental construction the only steel continuous through the joints is the post-tensioning. Using this method, precast elements are erected in series and held together by the post-tensioning tendons, rather like an internal rubber band holding together a series of blocks. Corroding post-tensioning steel can cause an imbalance of stresses within a structure, causing tendons to move and individual wires to fracture. Failure of the steel can lead to collapse of the structure. In response to this vulnerability, no tensile stresses are allowed in the bridge at full service load and extra capacity is routinely designed into the systems so that the structure can continue to support the design load even if some strands in a tendon have fractured. Thus, each member has multiple tendons that are each composed of a large number of steel strands, which are twisted bundles of steel wires, and there is redundancy in the system that can tolerate some breakages. The goal, however, is still to avoid all breakages if that is possible. To accomplish this goal, these post-tensioned systems need to be protected against corrosion.

2.3 OVERALL CORROSION PROBLEMS IN INFRASTRUCTURE

According to the corrosion report completed through Federal Highway Administration funding, corrosion of highway bridges alone costs the US an average of $8.29 billion a year[1]. This amount was generated from extensive research that looked at the number of bridges made obsolete or under-capacity from corrosion, the cost to maintain the bridges in their corroded state, and their replacement costs. The report considers all highway bridges, including ones that are constructed of masonry and heavy timber. However, by far the majority are either steel or concrete bridges. The concrete category is further subdivided into traditionally reinforced concrete and prestressed concrete. The prestressed bridges make up 18% of the total bridge inventory and thus do not contribute as much as the reinforced concrete bridges, which total 40% of the inventory, do to the cost of corrosion. In addition to having a smaller inventory, prestressed bridges have less corrosion than reinforced concrete bridges. The above values are for the entire prestressed inventory, which includes both pretensioning and post-tensioning. As there have been no known cases of corrosion in pretensioned concrete girders, this further reduces the inventory of prestressed concrete bridges that contribute to the national cost of corrosion. The cost or importance of the corrosion in post-tensioned concrete, however, is not insignificant[1].

In predicting the time to corrosion of a reinforced concrete member, there are many aids. For instance, Fick's second law of diffusion gives a formula for chloride concentration at a given depth in the concrete, x, at a given time, t:

$$\frac{c(x,t) - c_o}{c_s - c_o} = 1 - erf\left(\frac{x}{2 \cdot \sqrt{D \cdot t}}\right)$$

Equation 2.8[12]

In the preceeding equation, c(x,t) is the chloride concentration at depth, x, at time, t, c_0 is the initial chloride concentration in the concrete, c_s is the chloride concentration at the concrete surface, and D is the diffusion coeffecient. The time required for c(x,t) to equal the threshold for corrosion initiation at the steel depth is the initiation time. Once corrosion has initiated, one could apply a corrosion rate for steel to calculate a time to failure based on net section loss. Some computer models, such as Life 365, simply assume a propagation period until corrosion related maintenance is required[13]. These models have been developed to expedite the corrosion calculations without having to do laboratory testing to determine the diffusion coefficient for the particular concrete mixture and the exterior chloride concentration for the exposure conditions. When the diffused chlorides encounter a post-tensioning duct, there is no apparent method in this model to account for the time the chlorides require to work their way through the duct and grout to initiate corrosion of the strand.

The many components of a post-tensioning system make them more complex to model than an ordinary reinforced member. Instead of the chlorides simply needing to penetrate the concrete cover and reach a minimum threshold concentration, they have to breach several layers of obstacles to reach the post-tensioning strand. There are numerous ways the chlorides can attack the system simply because there are more components than in ordinary reinforced concrete, which only has reinforcing steel. Some of the routes can include breaching the post-tensioning duct. A galvanized corrugated steel duct was the standard used in post-tensioning for years until studies showed its propensity to corrode[3,11], which led to a decrease in its use in severe exposures. When galvanized steel duct is used, the chlorides are simply delayed on their trip to the steel strand. The galvanizing zinc coating is consumed, the duct is corroded and then chloride ions continue to diffuse into the grout. If there is a duct splice, which commonly consists of an oversized duct sleeve duct taped in place or a heat-shrink wrap, this location is generally a weak point in the duct. Studies have shown that the splices provide poor corrosion protection[11].

In other types of ducts such as integral plastic duct, there might not be a way for the chlorides to get through or they might be significantly delayed. Thus, the anchorages become the most likely location for chloride ingress. An anchorage, as shown in Figure 2.4, commonly consists of a bearing plate and trumpet assembly, anchor head and wedges, as well as the strand tails. These components work together to transfer the load carried by the steel strand to the surrounding concrete. The wedges grip the strand and bear against the anchor head. The anchor head bears on the bearing plate and the bearing plate bears on the concrete. Each of these pieces is larger than the item before so that the force is eventually distributed to a large area of concrete. The bearing plate, trumpet, and anchor head are all generally made out of very thick steel. Because of their thickness, they will not corrode to a point of failure before the strand and wedges, which are much smaller in section.

The wedges by far, are the smallest steel component of this system, and thus would take the least time to completely corrode. As well, they grip the strand and provide the essential mechanical connection that makes post-tensioning possible. If the teeth of the wedge corrode, then the load carried in the strand would be lost as the strand slides through the wedge. By the very nature of the wedge to strand interface, there are many small surfaces created that seem ideally suited to crevice corrosion. The space between the wedge pieces can also host crevice corrosion, as well as possibly acting as a straw for the capillary suction of moisture into the strand. Thus, the wedge to strand interface could be considered the critical component to protect in the anchorage. Unfortunately, the anchorages are often located at expansion joints, which eventually often leak, and thus extra effort must be made to prevent chloride bearing moisture from reaching the anchorage. Simple grout pourbacks can provide only a limited time of protection, as indicated though Fick's second law of diffusion. Two of the complexities that the traditional reinforced concrete corrosion model do not handle are the effect of wicking chloride bearing moisture into the tendon through the strand tails and the effect of grout voids on the corrosion initiation threshold.

When reinforced concrete begins to corrode, the steel reinforcement generally exhibits pitting corrosion from the local loss of its passive film. Steel's yielding capacity can be calculated by multiplying the cross sectional area by the yield strength of the steel. Thus, when the net cross section is reduced due to pitting or other corrosion mechanism, the steel's capacity is reduced. The same pitting corrosion attacks prestressing steels. The effects of these attacks can be quite different as the steels are of different strengths. For the same cross sectional area, a prestressing cable carries a higher load because the cable is made from a much higher strength steel than a bar. If a reinforcing bar and prestressing cable have the same cross sectional area and equal amounts of pitting, the prestressing cable loses more carrying capacity. Corroding the steel strands to fracture could result in serious cracking and even collapse of the structure, depending on the location and purpose of the post-tensioned element.

2.4 DOCUMENTED CORROSION PROBLEMS

In comparison with ordinary concrete construction, which engineers used to build some of the Roman Empire's enduring structures, a seventy-year-old construction method is still quite young. In the US, where post-tensioning was introduced later, the industry is even younger, with approximately only fifty years of experience. As mentioned earlier, when prestressing first came into use, there was much debate over whether cracking would be observed during the normal design life of a prestressed member. As many believed that the method would prevent cracking entirely, the design engineers were not as concerned about corrosion, as the uninterrupted concrete cover should have given significant protection. The prestressing should prevent cracking from flexural stresses as illustrated in Figure 2.6, which demonstrated the additional service load required for a prestressed member to reach its cracking load. However there are regions, such as the post-tensioned anchorage zones in most post-tensioned girders and deviators and blisters for anchoring external post-tensioning tendons that experience highly tensile stresses in the concrete, and usually develop cracks. These cracks are kept small by the presence of reinforcing bar inside the concrete. Prestressed elements could also have shrinkage cracks or shear-diagonal tension cracks. As there is a reasonable possibility that small cracks will develop in the concrete section, it would be conservative to assume during design that some cracking will occur. Due to the young age of the bridges constructed with post-tensioning, evidence of the long-term performance of the method is only recently becoming available. This performance shows the strengths and weaknesses of the method and indicates where improvements should be made. In the next section, a summary is given of some of the bridges and studies that have shown the industry that the standard post-tensioning system must be improved to increase durability.

2.4.1 Ynys-Y-Gwas and British problems

The United Kingdom (UK) began using post-tensioning methods before the US, and used some poor details that were avoided in early US bridges such as porous mortar joints between precast beams or segments[14]. Thus, it is no surprise that they experienced durability problems earlier as well. In the 1960s two UK footbridges, with very porous mortar joints, collapsed from corrosion of tendons[15]. From bridge inspections in the 1970s and 1980s a large number of grout voids were found in British post-tensioned bridges. These voids were observed to sometimes contain water and occasionally the grout at these locations was soft and damp. Extensive corrosion, ranging from mild pitting to strand facture, was also observed[16].

1985 brought the collapse of the Ynys-Y-Gwas bridge in Wales. This single-span, precast, segmental post-tensioned bridge had road salts penetrate the road slab[16]. When the chlorides reached the precast members, the buttering mortar at the joints proved unable to prevent further ingress[17]. The chlorides were then able to work through the joints to the tendons. The buttering mortar used was over an inch thick to accommodate the tolerances between the segments. Had the segments been match cast epoxy joints, as is the current common practice in US precast segmental construction, the joints would have been quite tight and the epoxy is less porous than the bulk concrete[3,11]. This last disaster, shown in Figure 2.10, prompted the UK DOT to declare a moratorium on the construction of post-tensioned bridges. After significant research and improved corrosion protection, the moratorium was partially lifted in 1996 to allow cast-in-place post-tensioned construction. In the UK, precast segmental construction is still disallowed pending further research[17].

Figure 2.10 Ynys-Y-Gwas at Failure, 1985[15]

2.4.2 Niles Channel

This bridge, built in 1981, which carries the only roadway to the Florida Keys, was discovered during routine inspection in the summer of 1999 to have lost one of its post-tensioning tendons. Resulting from the force imbalance after fracture, the tendon had moved 9 inches, as shown in Figure 2.11. There are six tendons supporting each span, so the loss of one was a significant reduction in available capacity for that span. As there are 39 spans in the 4557-FT bridge, this single tendon failure represents only 0.4% of the 234 total tendons present[18]. The fractured tendon had voids in the grout at the anchorage and the steel strands were actively corroding at this location. Salt bearing water had leaked through the joints, run down the joint diaphragm wall, and seeped into the anchorages located in these diaphragms. Autopsy of the failed tendon revealed that there had been cyclic recharge of the grout void with salt water, initiating the corrosion of the tendon. The span-by-span precast segmental construction of the Niles Channel Bridge was pioneered in Florida on the Long Key Bridge construction two years earlier. The Long Key Bridge has suffered from corrosion in the precast support piers, but not of the post-tensioned superstructure. The Niles Channel corrosion discovery lead to the replacement of the fractured tendon and the vacuum grouting of voids found in all other anchorages in the bridge. As well, Florida Department of Transportation then issued design recommendations suggested to improve grout quality that included the use of prepackaged grout, increasing the training of the grouting crew, requiring bottom up grouting to reduce grout void, and requiring the contractor to show grout vents on a plan of the post-tensioning system and submit a feasible plan for grouting[19].

Figure 2.11 Tendon Slip at Deviation Block[19]

2.4.3 Mid-Bay

Located near Destin, FL, this bridge was only seven years old when routine inspection in August 2000 found that two external tendons had failed. Span-by-span construction was used to erect the precast segmental box girders for this bridge. As six tendons support the typical span of the Mid-Bay Bridge, the loss of two throughout the bridge was an understandable cause for alarm resulting the bridge's temporary closure. The 19,265-FT bridge has a total of 846 tendons[1].

Therefore, the two failures represented only 0.2% of the total tendon count. The two failures were also located in different spans. After assessing the damage, the bridge was reopened under reduced load capacity while repairs were made[19]. Further investigation led to the replacement of eleven tendons in the bridge that had significant corrosion. These total of eleven tendons still only accounted for 1.3% of all the post-tensioning tendons in the bridge. One of the failed tendons had corroded in its free length where the high-density polyethylene duct had split, as shown in Figure 2.12. The other failure was a total fracture near the anchorage in a grout void. Chloride laden moisture was found to have penetrated through the anchorage, despite the presence of a bituminous protection coating. The investigation, which involved the inspection of every anchorage with a fiberscope, showed a number of anchorages that had undergone corrosion of the anchorage itself. A number of splits in the duct were also found due to a material deficiency in the plastic used during construction[17]. Grout voids were found at the anchorages and in the free length of the tendons. The anchorage voids in tendons not replaced were vacuumed grouted and anchor head protection was reapplied. The external tendons were wrapped to increase protection. As a result of the extensive corrosion problems it had experienced, the FDoT issued a design memorandum to improve durability in post-tensioning tendons. The new recommendations include requiring the use of pre-bagged grout, bottom up grouting, and inspection of the anchorages after grouting. As well, anchorages on the segment face were disallowed for balanced cantilever construction unless the contractor inspected and grouted the anchorage before erecting additional segments. Instead the anchors should be located in anchorage blisters[19]. As well, new AASHTO specification updates have disallowed the further use of dry joint segmental construction[20]. Without the now-required epoxy match casting, salt water was able to enter the box girder through the dry joints along the tendons length as well as through the expansion joints to reach the anchorage.

Figure 2.12 Corrosion in the Free Length[19]

2.4.4 Sunshine Skyway

The main spans of this Florida bridge are cable stayed, with the approach spans supported by post-tensioning. Constructed by balanced cantilever precast segmental methods in 1987, the bridge also has vertical post-tensioning used to join the precast segmental support columns. A routine inspection in September 2000 revealed that a vertical external tendon had fractured in one of the support columns[19]. Other vertical tendons showed signs of corrosion in grout voids, and a quarter of the external tendons had splits in the plastic duct. Many of these columns had leaks in their joints that resulted in large amounts of salt water filling the hollow columns. In addition, the grout in the external post-tensioning had numerous voids. The combination of pervasive salt water, breaks in the plastic ducts, and extensive voids resulted in corrosion of the vertical tendons. An example of this corrosion is shown in Figure 2.13. The longitudinal tendons of the superstructure appeared intact. Several tendons were replaced and the hollow pilings were filled with concrete to a point above the water line. The replacement tendons were double sheathed as a precaution against future chloride attack. As well, the Florida Department of Transportation amended its allowed design procedures to eliminate post-tensioning in submerged members with very limited exceptions. Other FDoT design recommendations issued, which were prompted by both the Mid-Bay Bridge and this incident, include requiring pre-bagged grout and bottom up grouting[21].

Figure 2.13 Corrosion of Vertical External Tendon[19]

2.4.5 Perspective on Reported Failures

Though the previously discussed tendon failures indicate that durability should always be carefully considered in bridge design and construction, these failures are not indicative of the behavior of most post-tensioned bridges. Throughout France's long history of post-tensioning only a few bridges built in the 1950s and 1960s have experienced any major corrosion problems[22]. In the 1990s they found that some tendons were not fully grouted, but only three significant corrosion problems[23]. These wire failures were a result of poor grout material and installation quality. Overall, France has seen good durability of its post-tensioned bridges[23]. Part of their success could be from their long-standing use of epoxy filled, match cast joints and waterproofing membranes on bridge decks.

In the US, the American Segmental Bridge Institute (ASBI) has twice surveyed the nation's segmental bridge inventory to evaluate their performance[4]. From the latest survey in 1999, 99% of the bridges had a superstructure rating of satisfactory or better. The satisfactory rating is described as the structural elements showing only minor deterioration. For comparison, the next lower rating, of fair, allows the structural elements to be sound, but exhibiting minor section loss, cracking, or spalling[4]. As nearly every bridge surveyed performed well, it shows that while corrosion problems can occur, as seen in the preceding discussion, they are not common. It should also be noted that the bridges surveyed are thirty years old or younger, so while the results are encouraging, design life performance is still an unknown. Therefore, it is important to continue to improve post-tensioning durability while striving to eliminate any corrosion related failures.

2.5 SUMMARY OF PREVIOUS RESEARCH

Previous research projects have evaluated the durability of post-tensioning tendons in concrete. Some of these projects are summarized below.

2.5.1 Jeff West

West was part of a group of graduate students at the University of Texas at Austin who all worked on aspects of durability of post-tensioned concrete bridges. Working together on a large TXDOT project, their main goals included examining post-tensioning uses in bridge substructures, developing experimental testing methods to evaluate the corrosion protection of post-tensioning variables, and developing preliminary corrosion protection guidelines. Prior to West's work, there were no US design guidelines for the durability of post-tensioning systems[3]. Among his individual contributions to the project West performed an extensive literature review on post-tensioning, concrete durability, and corrosion mechanisms. As well, he designed the large-scale long-term column and beam tests. Some of these large beams are still under exposure testing. He also continued exposure testing for the macrocell specimens and autopsied half the specimens after they had been exposed for 4.4 years. From his and his research partner's findings, West and Schokker co-authored preliminary design guidelines for the corrosion protection of post-tensioning tendons[3].

From West's work a correlation was seen between crack size and amount of corrosion in test specimens. As well, from examining prestressing level in the large-scale beams, he observed that overall corrosion damage decreased with increasing levels of post-tensioning. From his autopsy of half of the macrocell specimens, West saw superior corrosion performance of plastic duct specimens versus steel duct specimen. Two-thirds of the specimens with galvanized steel duct had locations where the duct had corroded through. As well, epoxy filled match cast joints showed far better corrosion protection then did the dry joint match cast joints. The combination of epoxy joints and plastic ducts had the best overall corrosion protection[3].

2.5.2 Andrea Schokker

In Schokker's dissertation, *Improving Corrosion Resistance of Post-Tensioned Substructures Emphasizing High Performance Grouts*[24], she studied post-tensioning grout mixtures with increased corrosion protection as well as detailing the construction of several long-term exposure tests with their preliminary results. As a result of her work in grout, Schokker found two grouts with good performance that are best suited to different types of post-tensioning. The grout with the best corrosion protection, showing more than a 40% increase in time to corrosion initiation versus plain cement grout in accelerated corrosion testing, was recommended for horizontal grouting applications. This grout has 30% fly ash and a 0.35 water-to-cementitious material ratio. For vertical grouting, an anti-bleed grout with 2% Sikament and a 0.33 water to cement ratio is recommended. This grout had similar corrosion protection to the plain grout, however it exhibited superior bleed prevention in horizontal grouting[24]. This work sparked the trend towards the use of commercial pre-packaged grouts, which are based upon her research, and improved training for the grouting personnel.

Schokker also constructed large-scale beam and column tests for long-term exposure. Variables under consideration included duct type, strand type, grout type, concrete type, level of prestressing, and anchorage end protection. Due to the time required for exposure testing, Schokker earned her degree before the final autopsy of the specimens[24]. Some of these specimens are still undergoing exposure[11].

2.5.3 Ruben Salas

Salas's work followed upon the work of West and Schokker. He continued exposure of all of the corrosion tests and autopsied the remaining macrocells, all of the columns, and half of the beams. From his results, Salas updated the corrosion protection design guidelines that were originally drafted by West and Schokker[11].

After autopsying three different specimen types, Salas developed overall conclusions on variable performance. In the aggressive testing environment plastic duct so consistently exhibited vastly superior performance over galvanized steel duct that he concluded it would be unwise to use galvanized steel ducts in aggressive environments[11]. As well in ducts, the quality of the splices affects the overall performance of the duct type and so he recommended further research on duct couplers. Both of the two steel duct splices, oversized duct sleeve and heat-shrink splice, showed poor corrosion protection[11]. From examining the effect of joint type, Salas recommended the use of epoxy filling on segmental joints be made mandatory. Gaskets should not be used with epoxy as they prevented complete filling of the joint and trapped moisture at the duct. The dry joints acted like big cracks, with increases in corrosion at the joint. Also with respect to cracking level, the autopsies showed that reducing crack width or density reduced the amount of corrosion in the specimen. This finding also correlates with the increasing levels of prestressing reducing corrosion as it also reduces crack openings. Using high performance concrete decreased the ingress of chlorides and higher covers decrease corrosion. From the grout autopsied, the then standard TXDOT grout had poor performance and while corrosion inhibitors and fly ash showed promise, complete grout information will not be available until the autopsy of the final beam specimens. From what was seen, grout voids had a detrimental effect on the duct, with corrosion frequently initiating in these regions. Salas recommended improving grouting methods[11].

2.6 PROTECTION STRATEGIES

In response to observed corrosion failures and research, numerous post-tensioning corrosion protection schemes have been developed. The strategies include improved techniques and innumerable different materials. The main material aspects include improving the grout, duct, and/or strand. For instance, using a pre-bagged grout for improved quality and a better trained labor force to reduce grout voids. Duct options include the traditional galvanized steel and an assortment of plastic ducts and accompanying couplers, many of which are part of proprietary systems. The strand can be treated with a variety of coatings, both metallic and nonmetallic, though the coatings are not commonly used in the US post-tensioning industry. Some of the aforementioned proprietary systems involve the entire system, such as trying to electrically isolate the post-tensioning tendons so that they cannot set up the corrosion cell. As well, the structure can be designed to minimize the post-tensioning tendon's exposure to aggressive environments. For instance, locating anchorages away from joints or providing a drip ledge to minimize moisture reaching the anchors. Using high performance concrete or protective coatings to reduce chloride ingress can also be employed.

The *fib* committee on the Durability of Post-Tensioning Tendons has considered a classification system to help the designer implement appropriate corrosion protection for the structure to reach its design life. The system, which is still under discussion, involves a matrix of options that allow consideration of the combined effects of structural protection and tendon level protection based upon the aggressiveness of the environment. Each of these three factors, environmental aggressiveness, structural protection, and the tendon protection level, has three levels. This proposal will be discussed further in Section 5.2.

2.7 LIFE CYCLE COSTING

To try to figure out which of the myriad options is the best one for a given project it is important to develop an evaluation method. The most logical approach is to determine what method, or combination of methods will yield the most benefit per the dollar spent on implementing them. As well, it is important to realize that all the money spent on a structure isn't necessarily in the initial construction costs. There are also costs associated with maintenance and repair through the design life of the structure. To examine the cost of a protection scheme one should look at the extra life gained by using this method as well as any delay in the onset of repair. These are cost savings that weigh against the expenditure at the beginning. The term life cycle analysis is growing more common as more people consider the structure's essential cost per year to help decide on the value of various construction or repair options.

The basic concept behind life cycle costing is to total the costs involved in the project, both initial and ongoing, and divide this value by the total expected life. An initially less expensive bridge may require more extensive and more frequent maintenance and have a shorter usable life than a bridge that cost more for construction. As a result, the initially less expensive option may cost more per year than the more “expensive” one.

CHAPTER 3
Protection Methods

3.1 ANCHORAGES

3.1.1 Black Steel

The standard anchorage commonly used in post-tensioning is a bearing plate and trumpet assembly. They can be as simple as two separate pieces of steel or a more complex cast iron arrangement. The cast iron varieties often have additional flanges along the trumpet to aid in the distribution of forces to the surrounding concrete. This arrangement also allows smaller bearing plates to be used and thus reduces the required spacing between the anchors. An anchorage with a flange on the trumpet is shown in Figure 3.1. Often spiral reinforcement is used in this region to increase the concrete's bearing capacity, which allows the transfer of load from the strand to the concrete. It is becoming more common to design built in grout vents in the anchorages to be used for post grouting inspection.

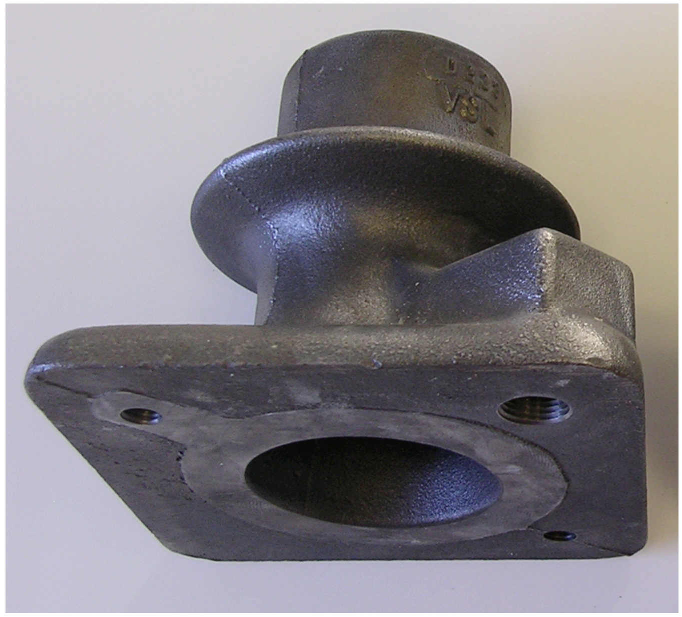

Figure 3.1 Black Steel Anchorage[25]

3.1.2 Galvanized Steel

For increased corrosion protection, the anchorages can also be galvanized. Some state codes are requiring this practice for all new bridge construction. The geometry and design of these anchorages are the same as for the basic black steel variety. In practice, they are the same anchorages that have simply been hot dipped in zinc to galvanize them. The zinc coating corrodes preferentially to the underlying steel, thus increasing time to corrosion initiation for the steel anchorage[10]. A galvanized version of the anchorage from Figure 3.1 is shown in Figure 3.2.

Figure 3.2 Galvanized Anchorage[25]

3.1.3 Insulation

For electrically isolated systems and the highest level of corrosion protection currently available for the anchorage, the steel anchorage can be insulated from the rest of the anchorage assembly. As shown in Figure 3.3, the steel bearing plate is isolated from the strand by extension of the plastic trumpet through the opening in the center of the plate and by provision of an insulation pad before the anchor head and the bearing plate. A plastic protective cap covers the entire end of the anchorage while the tendon is sheathed with plastic duct, trumpets and couplers. These anchorage assemblies are generally part of a proprietary system for electrically isolated post-tensioning tendons. As part of a system, they are usually designed to especially fit with other plastic parts by the same manufacturer.

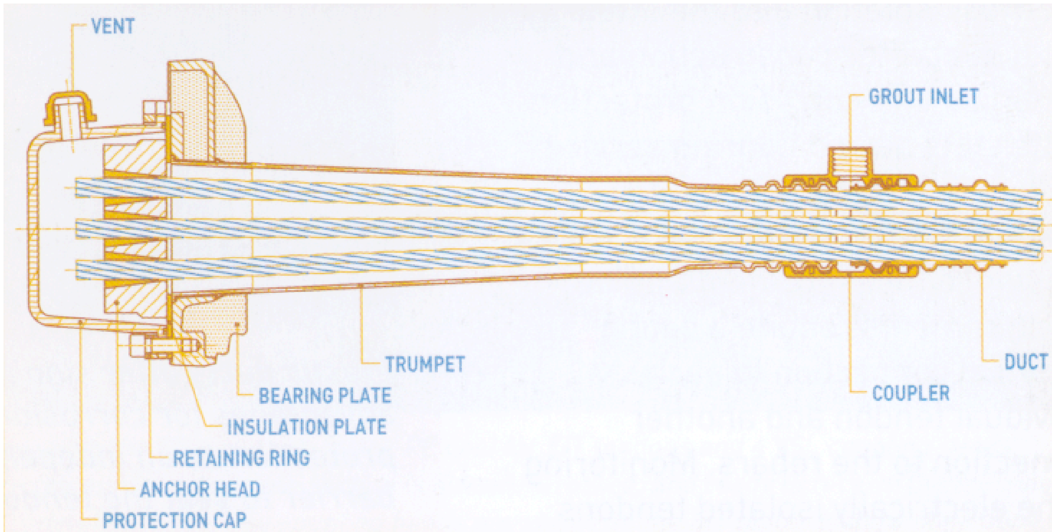

***Figure 3.3 Diagram of Electrically Isolated Anchorage*[26]**

3.1.4 Coatings

When installed, anchorages are frequently located at expansion joints. As these joints often leak, it is important to take additional steps to prevent the water that comes through the joints from reaching the anchors. In the current corrosion problems section there was often a moisture recharge of the system through the anchors located at expansion joints. That moisture was generally coming through an expansion joint and seeping through the mortar pourbacks installed to protect the anchors. Once the moisture has reached the anchor it is able to enter the tendon through the strand tails and around the bearing plate. After stressing is completed, coating the entire exposed pourback or anchorage assembly, which includes the exposed bearing plate, anchor head, wedges and strand tails, with an epoxy or other sealant provides an additional barrier to moisture ingress. Thus a coating should prevent or at least slow moisture entering the tendon and initiating corrosion of the anchor.

3.1.5 Grout Caps

It has been recent practice to use temporary grout caps over the anchorage during grouting to ensure grout encapsulation of the strand tails and anchor head. Once the grout is set these caps would be removed and reused in another grouting operation. The remaining grout block, however, did not provide the corrosion protection desired. Moisture was able to diffuse through the grout and reach the steel. For increased protection there are now permanent caps used to add an additional layer of protection to the anchor. Permanent caps can be either metal or plastic, as shown in Figures 3.4 and 3.5. With metal caps there is the possibility that they will eventually corrode themselves. However these pieces are generally thick metal castings that will take a significant time period to corrode through. Plastic caps do not corrode. However, plastic has been known to degrade from exposure to UV rays. Thus, the particular grout cap selection should be considered based upon the needs of the individual structure.

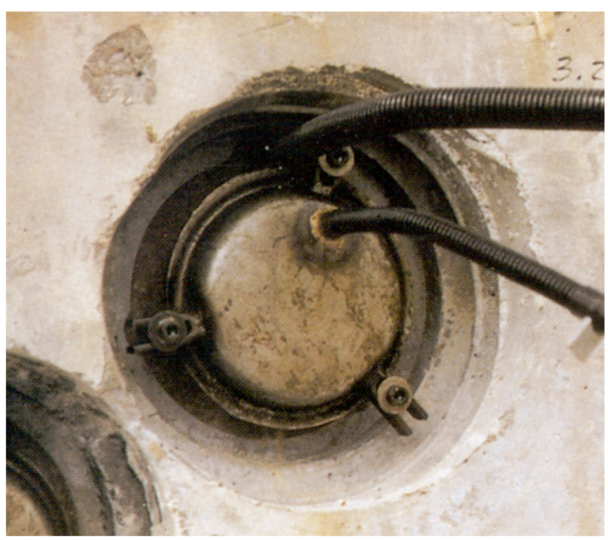

Figure 3.4 Metal Grout Cap[27]

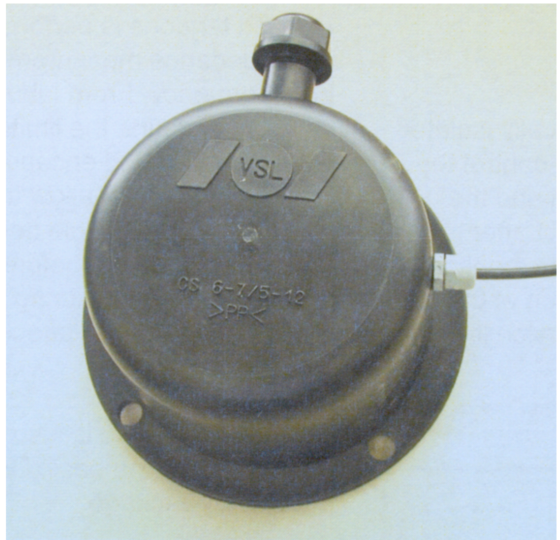

Figure 3.5 Plastic Grout Cap[28]

3.2 DUCTS

3.2.1 Steel

Galvanized thin walled steel ducts made out of spirally wound steel strips are the most commonly used duct type in current internal tendon post-tensioning construction. An example of this duct type is shown in Figure 3.6. These corrugated tubes are easily bent on site to fit the detailed tendon layout. However, they have not performed well in aggressive exposure studies[3,11]. In macrocell testing, some of the steel ducts corroded through in 4.4 years of continuous imposed electrical current with an alternating chloride solution exposure. This extremely aggressive testing environment is probably equivalent to several decades of actual exposure. Thus, this duct type offers very little corrosion protection to the enclosed tendon when compared with plastic ducts that behaved very well in the same study[3,11]. Splices between two lengths of steel duct is commonly accomplished through the use of a sleeve of oversized steel duct that is duct taped on either end to seal the opening. The length of the sleeve is proportional to the diameter. A heat activated shrink wrap has also been used. Both of these splices behaved poorly in these aggressive exposure studies[11].

Figure 3.6 Corrugated Galvanized Steel Duct[25]

Thick walled steel pipe can also be used for post-tensioning duct. While its use would substantially increase the time to corrosion penetration versus galvanized corrugated duct, the pipe must be bent into its desired shape at a fabricator before shipping and has very little tolerance for misalignment due to its rigidity. Most commonly, this pipe is used in external post-tensioning in relatively short lengths when the tendons pass through concrete at deviators or diaphragms. Experience with the short deviators has shown that the desired angles of bend are frequently misaligned. Its use for complete assemblies is not practical if tendons are to be curved as is usual in continuous tendons. However, due to their thickness, steel pipes can be welded or threaded to make a splice, which would be very beneficial along the ducts.

3.2.2 Plastic

Plastic corrugated duct is available for internal tendons in post-tensioned construction. The corrugations can be spiraled along the length, circumferential hoops, or circumferential hoops with additional longitudinal ribs. Examples of the two duct types with circumferential hoops are shown in Figure 3.7. In internal ducts, plastic ducts may also be sensitive to high temperatures that are generated by the curing of massive concrete sections. For external post-tensioning there are smooth walled plastic ducts available. In external post-tensioning these plastic ducts are the most commonly used variety of sheathing. For the corrugated ducts, splices between duct sections are generally proprietary preformed couplers that snap, lock, or slide into place. A few of these couplers are shown in Figure 3.8. Plastic ducts, both corrugated and smooth, can also be plastic welded to create a splice. In external post-tensioning, the smooth ducts are generally spliced to steel pipe when passing through concrete. The splice used is frequently a neoprene sleeve or boot. Frequently, external tendon plastic ducts have shown cuts, abrasions and penetrations form careless scraping by workers and even by unsealed nail holes used by inspectors checking for grout voids. These punctures greatly diminish the integrity of the duct's corrosion protection. Like the aforementioned plastic grout caps, plastic duct is sensitive to UV rays and must be sheltered onsite before installation.

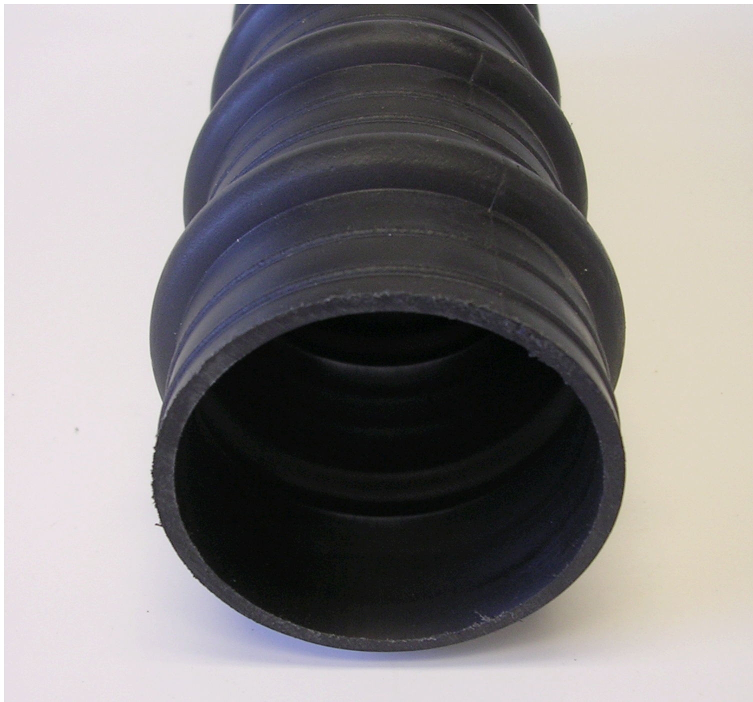

Figure 3.7 Corrugated Plastic Duct[25]

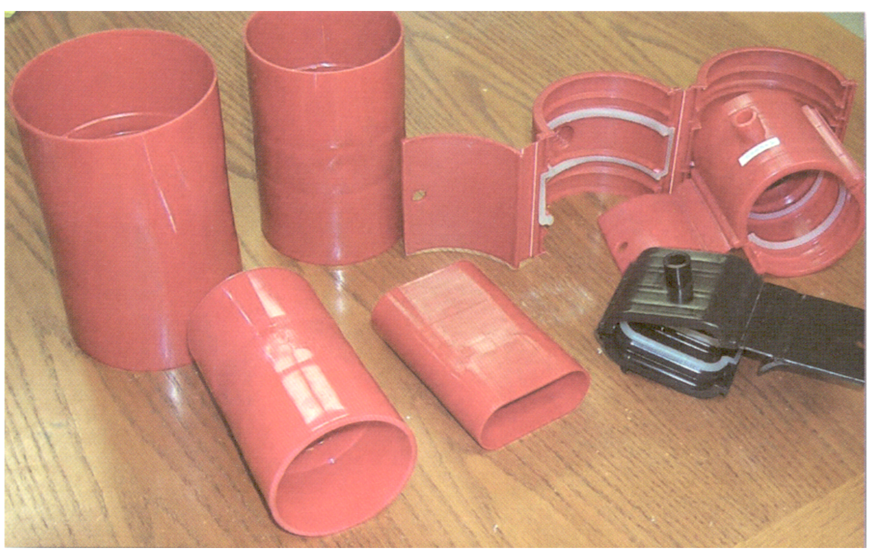

Figure 3.8 Preformed Plastic Couplers[29]

3.2.3 Other materials

In earlier years of post-tensioning's development there were some other materials used for duct. Paper and other biodegradable materials were often used. However, due to their temporary nature they provided poor corrosion protection. One of the early US post-tensioned bridges, the Pontchartrain Causeway, didn't use any ducts at all. To cast the pier segments a pressurized hose was used to create a cavity in the concrete. This hose was then removed and reused for additional casting and the post-tensioning steel was simply run through the resulting voids[30]. The first edition of the AASHTO LRFD Bridge Construction Specifications, published in 1998, listed provisions for cored ducts, which are cavities cored into the hardened concrete[31]. Cored ducts have no sheath material. Obviously the absent duct material cannot protect the prestressing steel and the protection must come from the concrete cover, grout, or strand coatings.

3.2.4 Segmental Construction

Part of the concern with using internal post-tensioning in precast segmental construction is that the duct cannot be a continuous piece between segments. Instead, it is a series of smaller pieces, cast within each segment. Thus, there would be a break in the protective duct at each joint. This continuity of the duct is replaced by the effectiveness of the epoxy filling of the joint. With epoxy-filled match cast joints, study has shown these joints to be less permeable than the adjacent concrete as long as the epoxy has been properly applied[3,11]. Match cast dry joints do not offer the same protection[3,11]. Due to concern over the proper filling of epoxy joints in practice, there have been a couple of methods developed to try to provide uninterrupted duct level corrosion protection at these joints. One solution is to cast a larger duct into the segments and then run a smaller and longer duct through these to provide better protection. As well, special couplers have been designed for use with match casting, such as the Freyssinet Liaseal shown in Figure 3.9, to seal the joint between the duct pieces.

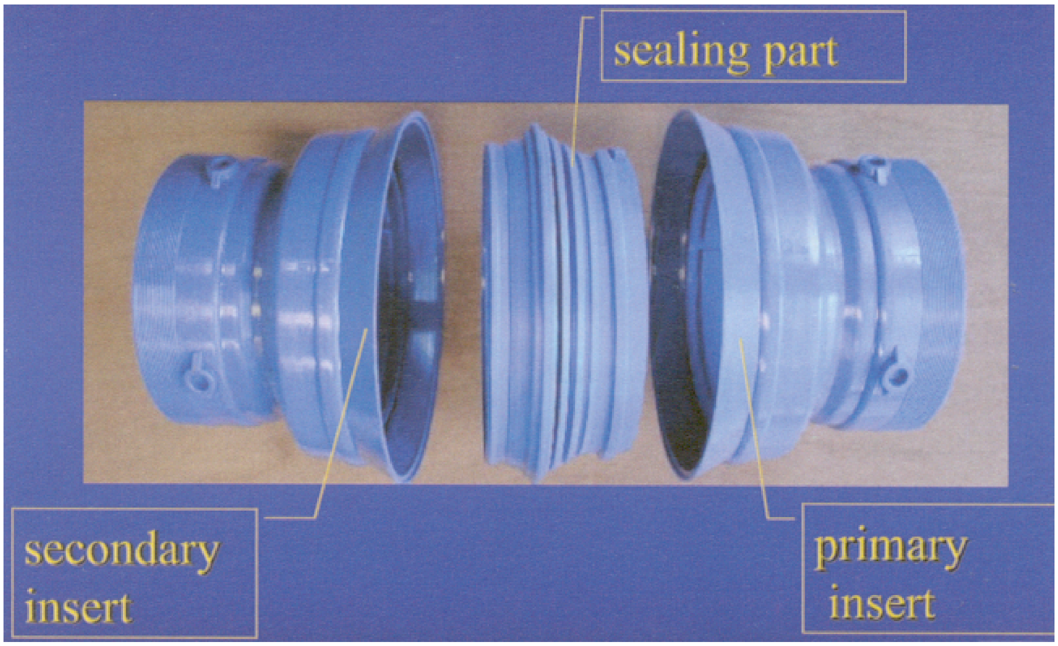

***Figure 3.9 Parts of a Freyssinet Liaseal*[32]**

3.3 STRAND

Strand level protection has been more popular in cable stays. However, there are numerous options currently available and under long-term corrosion protection study at the University of Texas at Austin[33]. These varieties include epoxy filled and coated, hot dipped galvanized, solid stainless steel, stainless steel clad, and copper clad steel strand[33]. While epoxy filled and coated strand has been available for considerable time, there have been reservations about the use of this strand type in post-tensioning applications since the wedge teeth have to cut through the epoxy to anchor the strand. The concern is that this negates the protective barrier at one of the most exposed areas of the system. While many new strand types and special wedges for epoxy coated strands have been proposed, further testing of these strands will be required to establish their material properties and corrosion resistance behavior. In current post-tensioning practice, seven wire prestressing steel is most commonly used with no additional strand level corrosion protection and is shown in Figure 3.10.

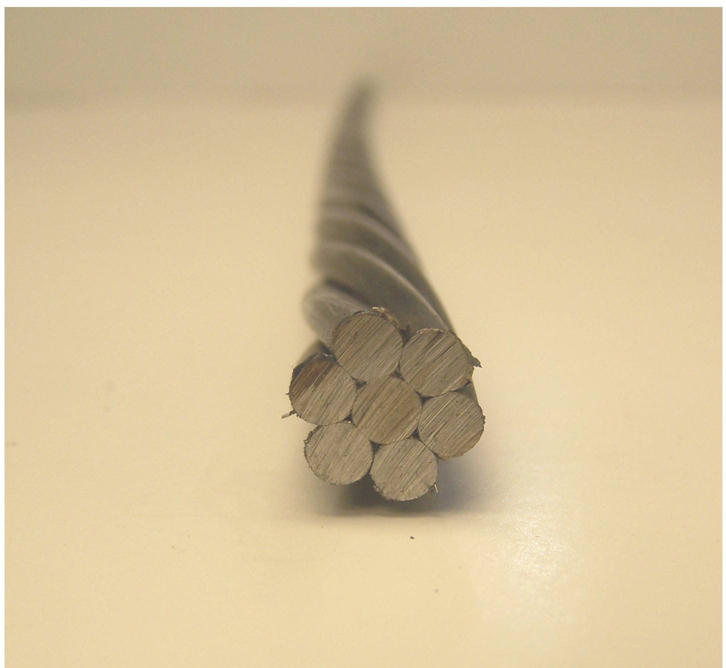

Figure 3.10 Seven Wire Prestressing Strand[25]

3.4 GROUTING

Almost since the introduction of post-tensioning, grouts have been used to fill the ducts after stressing to provide a level of corrosion protection as well as to bond the tendon so higher stresses could be developed under subsequent loading. Grouting is very effective for this latter purpose. However, since the grout is not prestressed it frequently cracks under live load. Cracked grout is not an effective corrosion barrier[34]. Nevertheless, complete filling of the duct prevents easy entrance for recharge water and is highly desirable.

Old practices were to use a plain cement and water grout after stressing the steel as a layer of corrosion protection. Tests and observation of field performance showed that the grouting practice often had numerous voids from trapped air, bleed water lenses, and a lack of grout fluidity. These voids led to increased corrosion of the adjacent steel duct, exposure of the underlying prestressing steel in places, and an effective lack of corrosion protection. The grout frequently had excessive bleeding, which caused large voids at the anchorages and lenses throughout inclined duct sections[24].

Schokker's research on the development of grouts stressed the reduction of bleed water to reduce the voids in the duct[24]. The current prepackaged grouts have shown superior performance in completely filling the ducts when used by a properly trained crew. Additional training of crews had also been proven essential to quality grouting. American Segmental Bridge Institute (ASBI) now offers Grouting Certification Training for grout technicians and members of the post-tensioning industry to help improve overall grouting quality[35]. Several state require all grouting to be done by ASBI certified grout technicians. Photographs from the 2005 ASBI training course grouting demonstration are shown in Figure 3.11 and 3.12. Figure 3.11 is an overview picture of the inclined and vertically grouted tendons. For each pair, the grout used in the left tendon is a plain cement and water grout and the grout in the right tendon is from a prepackaged mix.

Figure 3.12 is a close up of the bleed lens developing in the plain cement grout in the vertical tendon. Other grouting technique improvements include bottom up grouting, locating grout vents at high points in the duct profile, and mock-ups to ensure a particular grouting plan will work as planned[19].

Figure 3.11 Grouting Demonstration at ASBI training[36]

Figure 3.12 Closer View of Vertically Grouted Tendons[36]

3.5 Grout Vents

The most common grout vent is a smooth plastic hose. The hose is cut off at or an inch below the level of the concrete member once the grout is set. This arrangement does not provide corrosion protection as the interface between the plastic and the grout is a convenient route for the ingress of chloride bearing moisture. Newer grout vents are corrugated and have permanent caps or valves for added corrosion protection[37]. Figure 3.13 is an illustration of the smooth and corrugated grout vents and their common closure methods.

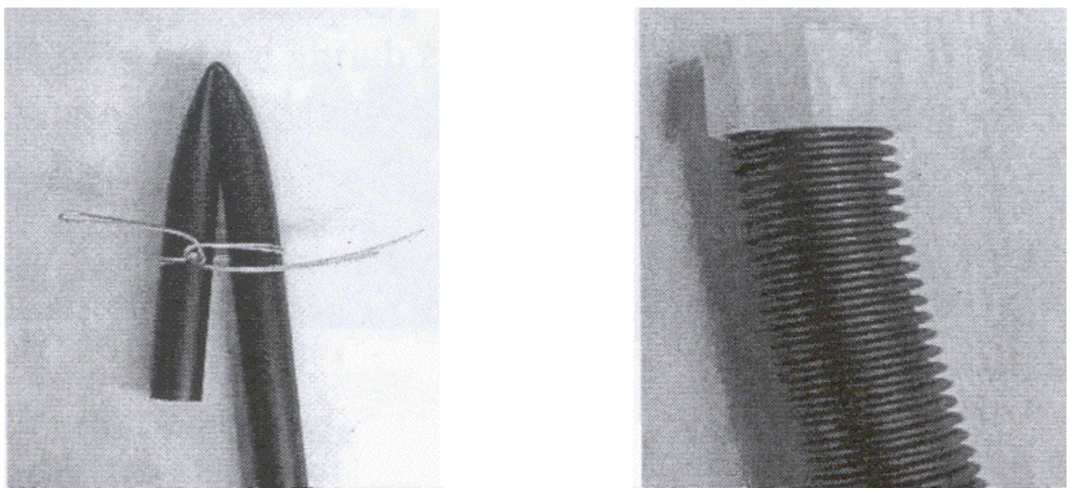

Figure 3.13 Smooth and Corrugated Grout Vents[37]

3.6 SEGMENTAL JOINTS

In precast segmental bridge construction, the different segments are tensioned together to act as single member. In some states, previous practice did not include any special treatment to these joints to provide corrosion protection, when using external post-tensioning. In testing, these dry joints leak and provide easy ingress for water borne chlorides[3,11]. Macrocell tests at the University of Texas at Austin have shown that epoxy coated joints provide superior protection in comparison to the dry joints[3,11]. The epoxy joint specimens initiated corrosion away from the joint, showing that the chloride diffused through the concrete cover before it could migrate through the epoxy. Specimens with incomplete epoxy coverage showed performance similar to dry joint specimens[11]. Therefore, high-quality construction practices are important to the effectiveness of this protection method. AASHTO now requires epoxy filled joints for all precast segmental construction[20].

3.7 PROTECTION LEVELS OF *FIB*

For the *fib*-IABSE Second Workshop on Durability of Post-tensioning Tendons, *fib* Task Group 5.4.2 proposed an option matrix for the selection of corrosion protection methods[37]. In this proposal, the tendon level protection and structural level protection are evaluated separately to rate their expected performance based on a predefined three level system. Combinations of the structural and tendon protection levels are used to address the needs of varying levels of environmental aggressiveness, which will be discussed more in Section 5.2. The *fib* proposal does not give precise definitions for the various protection levels, but instead describes them through examples[37].

3.7.1 Tendon Level Protection

3.7.1.1 Protection Level 1

This first protection level seems designed to specifically encompass the currently most common post-tensioning construction process, which does not provide significant corrosion protection. For example, galvanized steel corrugated duct, smooth grout vents, temporary grout caps, and black steel anchorages would all be a part of Protection Level 1 (PL1) [37]. Prepackaged grouts are now accepted to be part of standard practice as they allow full grouting of the tendons, which is in keeping with good construction practice. Thus, their use appears implied for this level. PL1 is appropriate for use in non-aggressive environments, or some marginally aggressive environments when used in combination with structural protection.

3.7.1.2 Protection Level 2

Protection Level 2 (PL2) offers greater protection than PL1, as could be assumed from the name. Examples of improvements needed to achieve the higher rating include the use of plastic ducts, permanent grout caps, corrugated grout vents with caps or valves left in place, and galvanization of the anchorage[37]. PL2 is suited for medium aggressive environments or highly aggressive environments when combined with structural protection.

3.7.1.3 Protection Level 3

The highest level of protection defined, protection level 3 (PL3), uses the best currently known protection methods. This level incorporates everything used for PL2 and adds electrical isolation as its major improvement. The importance of electrical isolation is that it prevents the tendon from setting up a galvanic cell within the tendon that causes corrosion. As well, enabling remote or electrical monitoring of the system allows the owner to quickly detect corrosion rate and initiation. With this information, the owner is better equipped to make informed decisions about maintenance and replacement, which can be viewed as an additional layer of protection[37].

3.7.2 Structural Level Protection

The design of the bridge itself can provide protection of the post-tensioning system by diverting moisture and chlorides away from the tendons and slowing chloride ingress. Again, due to *fib's* lack of precise definition it is currently uncertain what constitutes the three levels of structural protection. The level names are low, medium, and high protection, however only a single example is given for each. The proposal does list a number of possible ways to increase structural protection. Some basic examples that constitute improved structural corrosion protection include locating the anchorages away from joints, a drip ledge near the joint, and designing the post-tensioning system to be easily inspected. The example for the low structural protection level cites locating the anchorages at segment joints, no slab level surface protection, and a non-inspectable system. The medium protection level involves a surface coating on the concrete with normal concrete cover. Use of a waterproofing membrane, a high concrete cover and an easily inspectable system characterize the high level example[37].

CHAPTER 4
Economic Model

4.1 LIFE CYCLE COSTING

Life cycle costing, or life cycle analysis, is a method for considering all of the costs a structure is likely incur in its usable life. This method is much preferable as it selects a structure with the lowest overall costs rather than simply considering the initial construction costs. Though this idea may seem very logical, most projects are based on a lowest initial construction cost bid, ignoring the implications of the long-term costs. Alternative designs usually are developed to speed construction time or reduce first costs. They are generally not pitched as improving durability, unless that is a side benefit to a cost reducing measure. However, a bridge with superior durability could easily cost substantially less over its usable life due to reduced maintenance cost and maintenance frequency as well as lowering annual costs by simply having a longer usable life.

In order to compare two bridges by their effective cost per year, one first must total all the costs the bridge will likely incur. These costs include all initial construction costs as well as expected inspection, maintenance, and repair costs for the bridge's entire expected life. This total cost is then divided by the total expected life to give the bridge's annualized cost. The bridges can then be compared based on their effective yearly costs instead of simply by their initial costs[38]. Some other costs that can be considered in this calculation are indirect, or user costs. Examples of these costs would be the costs to both owner and user of closing a lane for inspection or maintenance work or the environmental impact of bridge construction. Though owners do not incur all of the indirect costs, they are real costs to society and can be up to ten times the value of the direct costs[1]. Many owners omit these costs completely from life cycle analysis. If the bridge is a toll-way, the cost of lost revenue due to a lane closure must also be considered. Thus if a method provides a reduction in maintenance frequency, it could save money by reducing the indirect costs and minimizing the loss of revenues. At the end of the bridge's expected life it may have a remaining salvage value. If there is an expectation that the bridge components may be sold at demolition, this estimated value subtracts from the bridge's total cost. The owner must prioritize what cost considerations are important in order to determine which costs to consider in a life cycle analysis[1].

There has been a resistance to use of life cycle analysis because so many subjective decisions or assumptions have to be made for proper implementation. Use of first costs as a basis for contract award avoids such subjectivity but clearly rewards defect prone long-term durability protection measures. Owners must move towards better consideration of durability in the their decisions and specifications. Life cycle analysis is becoming more widely used, especially in privatized and toll applications.

4.2 TIME VALUE OF MONEY

4.2.1 Description

Due to inflation, a dollar spent one year can never be directly compared with a dollar spent several years before or after that year. To compare costs effectively, all values must be converted to the currency of the same year. This practice is common in financial reports or other monetary comparisons. The comparison year can be any one chosen, although it is frequently a year near the date of the report so that the value of a dollar is similar to that with which the audience is familiar.

Whether one is calculating the future or past value of money, the equations used are similar. For a single present value, P, its future value, F, can be calculated as:

$$F = P * (1+i)^n \qquad \textbf{\textit{Equation 4.1}}^{39}$$

In the above formula, *i* is the interest or discount rate from inflation per period and n is the number of periods. Frequently the period considered is one year and the interest rate is the inflation rate. To find, instead, the present value, P, of a future amount, F, the following equation can be used:

$$P = F / (1+i)^n \qquad \textbf{\textit{Equation 4.2}}^{39}$$

The interest, or discount rate commonly used for inflation is 3% per year. This value does not assume that inflation will always be 3%, but that it has averaged that value for some time, and thus is a conservative value to assume in time value of money calculations[40].

4.2.2 Relationship to life cycle costing

If the owner wants the most accurate value for the life cycle analysis, the future maintenance costs, indirect costs, and salvage costs should all be adjusted for inflation[38]. This practice will adjust all future expenditures to their present value. Due to the uncertainty in estimating the values, it is not always worth the extra adjustments. Reasonable comparisons between effective yearly bridge costs can be made without adjusting for the time value of money. However, if there will be a substantial difference in future expenditures between different options, both in timing and amount, using time value of money will help give the best comparison between effective annual costs. This method is also useful for comparing costs between different years if one is trying to determine the actual costs of a method used in the past to one that is currently available. If the money used to pay for the bridge costs or money generated by tolls earns interest when not spent on the bridge, looking at time value of money can help determine the optimum timing for spending. Further discussion on interest bearing cash flow analysis can be found in Reference 39.

4.3 LIFE EXTENSION

The additional service life gained by the structure through the use of various corrosion protection methods would provide the designer or decision makers a clearer view of the methods' long-term benefits. These data prove particularly challenging to procure, partially because the methods are relatively new and also because testing had focused on comparative performance rather than quantitative values.

The most commonly encountered corrosion model for reinforced concrete involves breaking the corrosion and deterioration into several phases. The first phase is initiation. During the initiation period, chlorides diffuse through the concrete and gradually increase the chloride concentration at the level of the steel. When the concentration reaches an assumed threshold level, the steel is assumed to lose its passivity and initiate corrosion. The chloride diffusion is modeled using Fick's second law of diffusion, which was discussed in Section 2.3. The second phase is the propagation period. This second period is characterized by the active corrosion of the steel in concrete and concludes when the corrosion leads to cracking of the concrete. The third and final phase is the additional time to the limit state. What elements constitute the limit state are defined by the designer, such as amount of spalling or the concrete delaminating at the corroding steel. While the first phase can be modeled for steel in concrete, the second and third are simply assumed values[41]. The phases are often represented as a tri-linear curve shown in Figure 4.1. In the figure, T_i is the time to initiation, T_p is the propagation period, T_d is the time to the limit state, and T_f is the total expected usable life of the member[12]. As the damage during the initiation period is incurred very slowly versus the propagation period or time to limit state, the goal of corrosion protection methods is to increase the initiation time. Increasing this time also increases time to required maintenance, which is associated with the end of the propagation period, and results in decreased maintenance costs when they are averaged over the life of the structure.

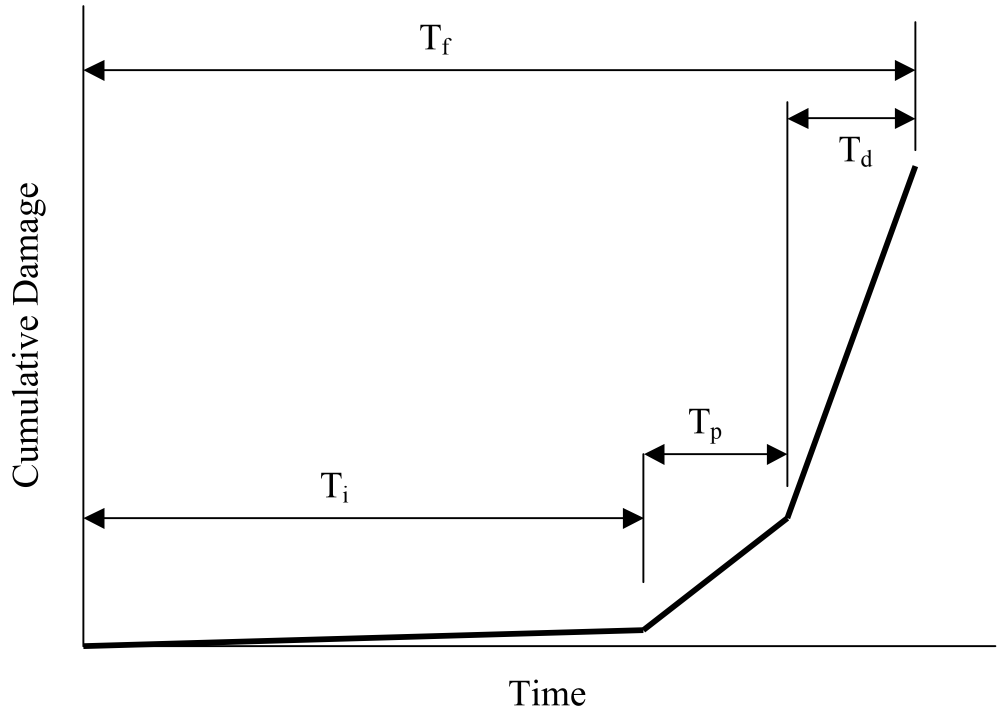

***Figure 4.1 Damage versus Corrosion Phases*[12]**

While the formulas used to find the initiation period for non-prestressed concrete structures cannot be directly applied to post-tensioned concrete, the basic idea of the three stages of corrosion is true for both types of reinforcement. Thus, the relative performance of post-tensioning corrosion protection methods that are based on damage can be seen as extending the time to initiation and thus the useable life of the protected concrete member.

4.3.1 Test Data

The macrocell tests performed at the University of Texas at Austin were not designed to give actual life extension data for field performance as explicitly stated in Salas' dissertation on the project[11]. Results from these specimens are intended to give only comparative values. The large-scale beam tests that might have given additional life extension comparisons were largely concerned only with prestressing level. Those beams that addressed other corrosion protection methods are still under exposure and have not yet been autopsied. Like the macrocell specimens, the beams are also subjected to severe exposure conditions in order to determine the relative performance of protection methods in a reduced time period. It would be impractical to test in real time, as actual structures would reach failure before better protection methods could be tested. Although the beams do not have an imposed electrical current like the macrocells, they were pre-cracked and subject to cyclic ponding of three percent salt solution[3,11,24].

Half-cell potential readings essentially monitor the systems' resistance to electrical current. The resistivity has been shown to change with the initiation of corrosion. Although half-cell potential readings are still taken quarterly on the remaining beams, these readings have been shown to report corrosion activity when any component of the beam begins to corrode, including the basic reinforcing cage. As black steel was used for the reinforcing for these beams, which were then cracked and placed in a highly aggressive exposure setting, there is little doubt that the reinforcing is corroding. This belief is supported by earlier autopsy results, which showed reinforcing steel corrosion[11]. Thus, the time to corrosion initiation data found for the beam specimens could be from any component of the specimen and not necessarily the post-tensioning system.

Additional large-scale, long-term corrosion exposure beams are under construction at the University of Texas at Austin. The data from the performance of new corrosion protection materials in these specimens will not be available for a number of years. These new specimens feature epoxy coated reinforcing bar and plastic chairs in an effort to eliminate or at least minimize half-cell potential readings that indicate corrosion of non-post-tensioning elements[33]. The effectiveness of these precautions will not be known until the specimens' eventual autopsy. As well, until further research is able to correlate the accelerated corrosion test data and highly aggressive environmental large-scale test data to actual field performance, life extension data for post-tensioning protection appears impossible to extract. Currently, the test data only provide comparative performance data, not a quantitative difference.

Until more specific data are collected, the comparative performance data from the macrocell exposure can be used to give approximate life extension values. Vignos constructed these specimens[3]. They had an impressed electrical current and underwent ponded saltwater exposure on a four-week cycle. For two weeks they had ponded water and for the other two weeks they were dry[3]. West autopsied half of the specimens after 4.4 years of exposure and the remaining specimens, which were duplicates of those autopsied, continued exposure[3]. After a total of 8 years of exposure, Salas autopsied the remaining specimens[11]. Corrosion ratings were assigned to the specimens during autopsy based upon careful measurement and visual inspection[3,11]. Specifically, the duct, strand, and bottom reinforcing bars of the specimen were examined and each assigned a corrosion damage rating[3,11]. The total corrosion ratings were examined. These are the sums of quantitative deterioration indices based on actual measurements of variables such as degree of pitting and penetration of ducts. Thus, they give a rating for the duct, strand, and bar in order to examine the protection methods influence on the total system. Specimen variables include duct type, grout type, joint type, and level of prestressing. To find the comparative performance of a certain variable, the corrosion ratings for specimens with all variables the same except for the variable in question were compared. The percent differences in corrosion ratings found between the comparison specimens were averaged with all other specimen pairs that had the same variable in order to get the percent improvement in corrosion protection for that method. This process was repeated for each variable in the study except level of precompression. The results of this comparison are summarized in Table 6.1 and individual comparisons are shown in Appendix B. Since these data closely relate to annual repairs required rather than the actual time to corrosion initiation, the comparisons were used as reductions in maintenance costs for the life cycle analyses. As previously stated, this comparison is intended to only give a rough estimate of field performance and should not be taken as exact amounts of damage reduction.

For example, by comparing plastic ducts versus galvanized steel ducts in the macrocells, there was a 90% reduction in measured damage. This value is an average of the percent difference in total damage observed at 4.4 years by West and at 8 years by Salas. Other damage reductions are found in Table 6.1. As indicated above, the 90% reduction was applied to the yearly maintenance costs in the life cycle analysis. The maintenance costs, which include inspection, maintenance, and annualized rehabilitation costs, assumed for the evaluation were a percentage of the initial construction cost as recommended by Menn[38]. Thus, the base case had a yearly annualize maintenance cost of 1.2% of its initial construction cost of 100, or a yearly cost of 1.2. With a 90% reduction, the plastic duct improvement had a yearly cost of 0.12% of its initial construction cost. The initial construction cost was adjusted for the cost increase associated with that protection method. For plastic duct, the increase was 0.54% of the total initial construction cost. All of the percent increases found are listed in Table 6.2. Returning to the example, with an increased initial cost of 100.54 versus the base case's 100, the plastic duct had yearly costs of 0.12% * 100.54 = 0.12. These yearly costs are significantly less than the base case. To complete the life cycle analysis, the total lifetime costs of the bridge must be totaled. For these analyses an expected life of 75 years was assumed. Thus, the costs for the base case were the sum of the initial construction cost, 100, and the total yearly costs, 1.2*75 years. The total costs for the baseline were thus 190. Annualized over 75 years, the base case cost 2.53 per year. The plastic duct costs were also totaled and annualized: 100.5 + 0.12*75 = 109.5 total and 1.46 per year. Thus for the initial 0.54% increase in construction costs, the owner would save 42% on the bridge's lifetime costs. This savings represents over an 7800% return on investment.

In comparing life cycle analyses for protection levels 1 and 2, it was assumed that the combined effect of individual protection methods used in protection level 2 would result in a significant increase in durability. Therefore, protection level 2 was assumed to attain an increased design life and have decreased annual costs versus protection level 1. These analyses are discussed in Section 6.3.

4.3.2 Modeling

Life prediction models for corrosion in concrete have focused on traditional reinforced concrete. Post-tensioning systems are more complex systems than embedded reinforcing bars and thus are unable to directly use the available models. In most reinforced concrete models the corrosion initiation period is the time until enough chlorides have diffused through the concrete for their concentration at the reinforcing bar to reach an initiation threshold. While chlorides diffuse through concrete without regard to the type of reinforcing, this is not the only information needed to predict post-tensioning corrosion. In internal post-tensioning, the chlorides diffusing through bulk concrete would first have to breach the duct and then the grout. The available models do not cover the chlorides ingress through the duct and grout. In external post-tensioning, the majority of the duct is not in the concrete and thus the diffusion calculation is utterly ineffective in determining time to corrosion initiation in the strand's free length.

Besides corroding along the free length, several examples of corrosion in field structures were at the anchorage. Voids at the anchorage coupled with moisture recharge and salt presence can lead to this corrosion. Moisture recharge can be from moisture seeping behind the bearing plate or through the wedges. The concrete diffusion calculations cannot model the way moisture travels through the anchorages. The different chloride ingress routes described show the complexity of the post-tensioning system in comparison to a non-prestressed structure. This complexity prevents the diffusion calculation from being used as a stand-alone model for corrosion of the reinforcing steel in post-tensioned structures.

To make an estimate of time to corrosion initiation in a post-tensioned concrete bridge, one would have to consider all the various routes for chloride ingress and attack. Next, one would have to calculate the time to initiation for each route and the probability it will occur. Then take a weighted average, based on probability, of the various times to initiation. In order to determine all of these values, a significant amount of data would first need to be collected, especially to determine the probability of the initiation route. Even with this average value to corrosion initiation, one only has a baseline case. These calculations must be repeated for every protection method under consideration in order to get comparative life data.

Many of the factors that would impact the route to initiation and their probability to occur are dependent upon the geometry of the structure. For instance, the probability that chlorides would enter through the pourback depends on the location of the anchorage in the bridge system, the system geometry, exposure conditions, and the preparation and treatment of the pourback. If the anchorage is shielded by the structure, the life extension data for the effectiveness of the anchorage protection would be diminished. To assume a worst case scenario to determine the maximum effectiveness for a protection method would lead to an over estimation of its return on investment calculation if the anchorage is shielded in the final design.

The above example is intended to simply point out that calculating life extension data is very structure specific. Collecting data from actual structures would obviously give the most accurate life extension data. However, many of the systems now showing corrosion problems were constructed using now out-of-date construction practices or materials, especially the grout. It is not preferable to simply wait another twenty or more years to collect the field performance data on the various protection schemes currently available. Thus, artificially severe exposure testing is performed to get comparative performance data. Due to the nature of corrosion protection testing, where a specimen is exposed for years, this comparative data are accumulated slowly. Developing a model sophisticated enough to handle input of the structure's geometry, expected exposure conditions, probability of chloride attack path, and the many other variables affecting corrosion in post-tensioning systems is beyond the scope of this thesis. Therefore the available comparative macrocell performance data is used here for life cycle analyses.

The importance of the tendon and structural protection interaction, which makes corrosion initiation so daunting to predict, is addressed in the *fib* protection matrix.

4.4 METHOD FOR DETERMINING COSTS

4.4.1 Questionnaire

A questionnaire to discover cost trends in post-tensioning was developed by the author and her supervising professor as suggested at the conclusion of the *fib*-IABSE Second Workshop on Durability of Post-tensioning Tendons, held 11-12 October 2004. Although the questionnaire and several reminders were sent to approximately seventy leaders in the post-tensioning industry, only one fully completed questionnaire was returned. A few other responses provided useful cost information in another format. From the limited information gathered the average cost of the total post-tensioning system could be compared to the total cost of the bridge. This percentage was compared against the similar value calculated by Menn in his book on Prestressed Concrete Bridges[38]. Menn's value was based on a survey of nineteen post-tensioned bridges' cost data. As these two percentages were in general agreement, the cost data collected is assumed to be representative of the post-tensioning industry even though the data set was much smaller than desired. See Appendix A for a copy of the questionnaire.

4.4.2 Other inputs

As reported above, some cost data were provided in alternate formats. Data were received for the corrosion protection improvements of some Florida bridges. These data came with extensive descriptions of the post-tensioning system that allowed calculation or estimation of the remaining post-tensioning costs. Though it would have been desirable to compare the costs of an electrically isolated system with the other levels of protection provided, such information was unavailable.

4.5 PROTECTION METHODS CHOSEN

The various protection methods examined for cost increases or relative performance were limited by availability of information. For instance no information was available for the increase in cost to make a system electrically isolated or fully monitorable so these protection methods could not be quantified. Results for the cost increases for various protection devices are found in Section 6.2. This cost increase data is used in conjunction with the comparative life extension data derived from the macrocell specimens to give comparative annualized costs. The costs used in the life cycle analysis are expressed as a percentage of total construction cost. Menn's cost percentages[38] are used for bridge maintenance and inspection. The results from a life cycle analysis run using costs as a percentage of the total construction cost are given in Section 6.3. In addition to the annualized costs, a simple return on investment calculation was run for those variables examined with a life cycle analysis in order to show the lifetime cost savings derived from the initial investment in corrosion protection.

CHAPTER 5

Corrosion Protection Design

5.1 CURRENT DESIGN GUIDELINES

As of the second edition of the *AASHTO LRFD Bridge Design Specifications*[42] and the first edition of the *AASHTO LRFD Bridge Construction Specifications*[31], both published in 1998, very little guidance was provided for post-tensioned concrete corrosion protection. Largely, the designer and contractor were simply instructed to protect against corrosion of the prestressing steels. The only specific guidance in the Construction Specifications was in instructions to use a corrosion inhibitor to protect the steel in shipping and storage prior to use and in limiting the time tendons could be post-tensioned without grouting[31]. The Design Specifications indicated that designing the structure to protect the post-tensioning system would help with corrosion protection. As well, the commontary seemed to encourage life cycle analyses by warning that decreased initial construction costs may not yield decreased lifetime costs[42]. Implementation of these suggestions is still left to the experience of the design engineer.

Since each state DoT sets its own requirements for bridge design and construction, there are a number of different guidelines that govern post-tensioning corrosion protection. Largely, the differences are based on the differing experiences of the different state DoTs. Florida, which had numerous corrosion problems in post-tensioned bridges, has perhaps the most advanced specifications for promoting the corrosion protection of post-tensioning systems.

Due to Florida's geography, many of the state's bridges are in a marine exposure environment. As a result of the expected exposure and past field performance, thin walled steel corrugated ducts are no longer allowed in post-tensioning[43]. Plastic ducts are the standard. Internal post-tensioning uses corrugated plastic duct. External post-tensioning uses smooth plastic ducts along the tendon free length with thick-walled, smooth steel pipe in deviator blocks and diaphragms. As well, all precast segmental bridges are now required to use epoxy joints. Prepackaged grouts and bottom up grouting are used for improved grouting. Grout crews must now take additional training such as the ASBI grouting course for certification. Anchorages are required to be galvanized and use permanent grout caps[43].

These many improvements in the corrosion protection for post-tensioning are not required by all states, and all are not necessary for non-aggressive environments. However, wherever there is de-icing salt use or marine exposure, they are important and desireable recommendations. A universal design guideline should recognize the exposure demand and relate this demand to the available corrosion protection in the system. There have been some recommendations on how to achieve this balance.

5.2 *FIB* TASK GROUP 5.4.2 PROPOSAL

5.2.1 General Description

The *fib* Task Group 5.4.2 proposed a system to guide the designer in selecting an appropriate combination of tendon and structural protection for an expected environmental aggressiveness. This system was presented in the *Durability Specifics for Prestressed Concrete Structures: Durability of Post-*

tensioning Tendons[37], which was a draft proposal discussed at the *fib* and IABSE Second Workshop on Durability of Post-tensioning Tendons held October 11-12, 2004[44]. The main guide presented for corrosion protection method selection was an option matrix, which is reproduced in Figure 5.1.

		Structural Protection Layers		
		High	Medium	Low
Aggresivity/Exposure	High			PL 3
	Medium		PL 2	
	Low	PL 1		

***Figure 5.1* fib *Selection Matrix*[37]**

Following this design guide, a post-tensioning system under design is examined for the setting's exposure rating and the structure's innate corrosion protection. On the option matrix the tendon protection level indicated at the intersection of the exposure level and structural protection level is recommended for durable post-tensioned concrete. Three levels to each category are shown, though the proposal lacks clear definitions as to what qualities constitute each level. The tendon protection levels (PLs) and structural protection levels were previously discussed in Section 3.7.

5.2.2 A critique and revised matrix

Dr. M. Raiss presented a challenge to the proposed matrix at the Second Workshop on Durability of Post-tensioning Tendons[44]. His main concerns centered upon the lack of clear definition for the protection levels, the lack of options presented by the matrix, and that some current bridge designs with improved protection do not appear to fit within the example definitions of the levels[44]. In his critique, Raiss proposed an expanded option matrix, which is shown in Figure 5.2.

		Structural Protection Layers		
		High	Medium	Low
Aggresivity/Exposure	High	PL 2	PL 3	
	Medium	PL 1	PL 2	PL 3
	Low	PL 1	PL 1	PL 2

Figure 5.2 Revised Option Matrix[44]

Raiss' proposed new matrix allows more combinations of tendon protection levels and structural protection for a certain environment. This flexibility reflects the many design possibilities made possible through the use of post-tensioning corrosion protection methods. Raiss felt that the design of post-tensioning should not become prescriptive, but instead allow the designer room to creatively adapt the structure and tendon systems to best suit the exposure environment[44]. This belief, backed by several case studies he presented of bridges in the UK that did not fit easily into a protection level, served as the basis for his revised option matrix[44].

5.2.3 Future Development of *fib* Proposal

At the conclusion of the Second Workshop on Durability of Post-tensioning Tendons, J-P Fuzier, as chair of *fib* Task Group 5.4.2, stated that the Task Group would continue development of the proposal. In particular, the Task Group would work to better define the exposure levels and the structural protection layers. This work is ongoing.

5.3 HAMILTON CORROSION PROTECTION PROPOSAL

Trey Hamilton, in his PhD research at the University of Texas at Austin examined the corrosion protection of cable stay tendons[34]. As a part of his dissertation, *Investigation of Corrosion Protection Systems for Bridge Stay Cables*[34], he proposed a new design specification for determining the corrosion protection level demanded by the bridge's exposure environment. His recommendation implements the use of a corrosion protection demand factor (CPD) and a corrosion protection effectiveness factor (CPE) to guide the design of the cable stays' corrosion protection logically. The demand factor is derived from the structure's possible exposure to salts, both deicing and marine, the importance of the bridge, and the redundancy of the system. Tables are provided that give the demand factor values for each of these categories depending on the individual bridge[34]. These tables are reproduced in Tables 5.1 and 5.2. The sum of the individual category factors is the demand factor, as shown in Equation 5.1.

$$CPD = I + R + E \qquad \textit{Equation 5.1}^{34}$$

The corrosion protection effectiveness factor is similarly drawn from a table of possibilities, which is reproduced in Table 5.3. Depending upon the type and number of the structure's corrosion protection layers, it accumulates points towards its protection factor, which is a sum of the individual protection layers as calculated by Equation 5.2.

$$CPE = EB + IB_1 + IB_2 + BA \qquad \textit{Equation 5.2}^{34}$$

If the structure's corrosion protection effectiveness factor is greater than the corrosion protection demand factor, the structure is expected to last for its design life. There is no limit in this guide to the amount of corrosion protection methods that can be used. Thus, if local experience or owner preferences require a higher level than indicated by the demand factor it can be used without violating the recommendation.

Table 5.1 Importance Factor[34]

Type of Road	ADTT	Truck Loading	Importance (I)
Freeways, Expressways, Major Highways and Streets (or Rail Bridges)	2,500 or more	2,000,000	10
Freeways, Expressways, Major Highways and Streets	less than 2,500	500,000	8
Other highways and streets not included in the above categories	n/a	100,000	5

* Average Daily Truck Traffic

Table 5.2 Redundancy and Environment Factors[34]

Redundancy (R)	Nonredundant stay design	10
	Redundant stay design	5
Environment (E)	Harsh	10
	Mild	3

Table 5.3 Factors for Calculating Protection[34]

Location	**Type of Protection**	**Factor**
External Barrier (EB)	PE sheath (w/ Tape)	10
	Steel sheath (painted)	15
Individual Barrier (IB)	None	0
	Epoxy-coated strand (not filled)	4
	Epoxy-coated wire or bar	10
	Epoxy-coated and filled strand	10
	Greased and sheathed strand	10
	Galvanized wire, bar, or strand	12
Blocking Agent (BA)	None	0
	PC Grout	5
	Polyurethane	8
	Petroleum Wax	9
	Grease	8

5.4 A New Combination

The *fib* Task Group 5.4.2 proposal[37] echoes the idea Hamilton proposed in 1995[34] by using a protection rating versus the exposure rating. One of the main differences is that the *fib* proposal simply gave examples and a basic level name whereas Hamilton listed various protection options and gave them a numerical rating. Having only conceptual examples rather than concise definitions is a major weakness of the *fib* proposal, although the conceptual matrix is useful for ease of understanding the design method.

The author offers herein a proposed guideline that substantially improves the *fib* proposal. Using an easy to understand matrix is an important strength of this proposed guideline. By combining a numerical rating system with the option matrix, the modified guideline logically defines the *fib's* briefly described category levels. The option matrix is retained to show the conceptual relationship between the structural protection level, the tendon protection level, and the exposure environment.

Using demand and effectiveness factors similar to those proposed by Hamilton, the different exposure and protection levels are defined by ranges of the factors. Minimum threshold values are listed in Table 5.4. A modified version of the option matrix introduced by Raiss[44] is used and shown below, in Figures 5.3 and 5.4, both in conceptual form and with the numerical ratings. During the design process the engineer uses the conceptual matrix to gauge what combinations of structural and tendon protection levels are appropriate for the exposure environment. As well, the sum of the structural and tendon level protection factors are used to check the design for sufficient corrosion protection. The sum of the protection factors must meet or exceed the exposure's demand factor.

Table 5.4 Factor Threshold Values by Category

Aggresivity/Exposure	
Low	15
Medium	35
High	45
Structural Protection	
Low	10
Medium	20
High	30
Tendon Protection	
PL1	5
PL2	15
PL3	25

		Structural Protection Layers		
		High	Medium	Low
Aggresivity/Exposure	High	PL 2	PL 3	
	Medium	PL 1	PL 2	PL 3
	Low	PL 1	PL 1	PL 1

Figure 5.3 Modified Option Matrix [44]

		Structural Protection Layers		
		High: + 30	Medium: + 20	Low: + 10
Aggresivity/Exposure	High: 45	PL 2: + 15 = 45	PL 3: + 25 = 45	
	Medium: 35	PL 1: + 5 = 35	PL 2: + 15 = 35	PL 3: + 25 = 35
	Low: 15	PL 1: + 5 = 35	PL 1: + 5 = 25	PL 1: + 5 = 15

Figure 5.4 Option Matrix with Minimum Numerical Ratings

The actual numerical ratings for each corrosion protection method given here are somewhat arbitrary. With so little performance data available, it was not possible to establish a method to analytically derive the protection factors. Thus, the factors shown in Tables 5.5 and 5.6 are based upon the various methods' general reputation for corrosion protection. Where performance data were available from Salas' observations, the protection ratings are consistent with his findings. Table 5.7 gives descriptions of the exposure levels. With future research in post-tensioning durability and the accumulation of field data, an analytical method could refine these protection factors. As well, with additional data, a more extensive list of corrosion protection methods and their protection factors can be developed.

Table 5.5 Tendon Level Protection Factors

Tendon Protection Level	**Protection Factor**
Protection Level 1	*5 - 14*
Steel Duct	2
Black Steel Anchorage	1
Temporary Grout Caps	1
Ordinary Grout Vents	1
Sleeve and duct tape coupler	1
Total	*6*
Protection Level 2	*15 – 24*
Plastic Duct	9
Galvanized Anchorage	4
Permanent Metal Grout Cap	3
Valved or Capped Grout Vent	3
Preformed Couplers	4
Total	*23*
Protection Level 3	*25 +*
Plastic Duct	9
Galvanized Anchorage	4
Permanent Plastic Grout Cap	4
Valved or Capped Grout Vent	3
Preformed Couplers	4
Electrical Isolation	7
Total	*31*

Table 5.6 Structural Protection Factors

Structural Protection Level	**Protection Factor**
Low	*10 - 19*
Normal Cover	4
Normal Concrete	4
No Surface Protection	0
Anchorages at expansion joints	1
No Drip Ledge Present	0
Grout Pourbacks at anchorages	2
Total	*11*
Medium	*20 – 29*
Normal Cover	4
Normal Concrete	4
Surface Protection	4
Anchorages at expansion joints	1
Drip Ledge at expansion joint	5
Grout Pourbacks at anchorages	2
Total	*20*
High	*30 +*
High Cover	8
High Performance Concrete	8
Waterproofing Membrane on Deck	7
Anchorages away from joints	5
Drip Ledge at expansion joint	5
Epoxy Sealant at anchorages	4
Total	*37*

Table 5.7 Exposure Description

Exposure	
Low	No Salt Exposure
Medium	Mist Zone
High	Contact or Splash Zone

CHAPTER 6
Results

6.1 Comparative Corrosion Ratings

Listed below in Table 6.1, are the average performances of the test variables from the corrosion ratings assigned to the macrocell specimens at autopsy. The individual specimen comparisons are in Appendix B.

Table 6.1 Corrosion Rating Percent Decrease in Damage from Macrocell Specimens due to use of Corrosion Protection Method

Corrosion Rating Comparison - Averages			
	West[3]	Salas[11]	Average
Comparison	4.4 years	8 years	
Steel Duct v. Plastic Duct			
Average Plastic Duct Improvement	92%	88%	90%
Dry Joint v. Epoxy Joint			
Average Epoxy Improvement	88%	80%	84%
Dry Joint v. Epoxy & Gasket			
Average Epoxy & Gasket Improvement	54%	87%	71%
Normal Grout v. Corrosion Inhibitor			
Average Corrosion Inhibitor Improvement	-24%	2%	-11%
Normal Grout v. Silica Fume			
Average Silica Fume Improvement	41%	-169%	-64%

Prestressing level was an additional variable and is not shown as it was not a variable examined in the cost increase investigation. The grout with corrosion inhibitor had poor corrosion protection performance in the macrocell test. From this observation, Schokker[24] and Salas[11] concluded that additional research on corrosion inhibitors in grout is required before a recommendation regarding their use can be made. Therefore, the comparative performance data indicated from this variable is used in a life cycle analysis. The silica fume grout was only used in one specimen, so grout performance from Schokker was used to determine damage reduction from improved grouting. Schokker developed an optimized corrosion protection grout mix[24], which is approximately reproduced in current prepackaged grouts. Her dissertation, *Improving Corrosion Resistance of Post-Tensioned Substructures Emphasizing High Performance Grouts*[24], shows that this grout mix offers more than a 40% improvement in corrosion resistance versus a plain cement grout and nearly the same improvement over the old Texas DoT grout standard mix[24]. Therefore, a 40% reduction in damage is used in the life cycle analysis for improved grouting. This reduction in damage and those found from the macrocell data were used in the life cycle analysis as a reduction in annual costs. This analysis is further discussed in Section 6.3.

6.2 COSTS FOR INCREASED CORROSION PROTECTION

The costs for using various corrosion protection methods are shown in Tables 6.2 and 6.3. The data are from the survey and other cost information provided to the research team as described in Section 4.4. To protect the confidentiality of the cost data, all values are expressed as percentages and the bridges are labeled as A, B, and C in the tables. In alphabetical order, the cost data came from the Hacienda Avenue Bridge in Nevada, the Niles Channel

Bridge in Florida, and the Roosevelt Bridge in Florida. In the first table, Table 6.2, the costs are expressed as a percentage of the bridges' total initial construction costs. The second table, Table 6.3, has the protection costs expressed as a percentage of the total post-tensioning system. The average total post-tensioning cost calculated is similar to the average cost of 11.2% of the total initial construction cost found by Menn[38] and thus the protection cost data are assumed to represent the actual costs. A 28% markup was assumed on the projects that did not give this information based on a conservative average of percent markup used in the *2005 R.S. Means Building Cost Data* for post-tensioning in the field[45]. This value is in general agreement with the markup data provided. The totals for each category may not be a sum of the protection elements listed as the total used is from the actual project and some of the protection costs provided were for methods not used. As well, the average total for each category is an average of the total for each bridge, not a sum of the averages listed for each subheading. Additional cost data will allow inclusion of other protection methods in this table and refinement of the cost data present. This data gives a reasonable summary of actual corrosion protection costs.

By examining the data as both a percentage of total initial construction costs and as a percentage of the post-tensioning system, the costs can be seen from the perspectives of the designer and the post-tensioning supplier or sub-contractor. Though a protection method may have less than a one percent increase in overall project cost when looking at the total project, it can be ten times that cost increase when only considering the post-tensioning system. Due to the low bid culture in US construction, a six percent increase in costs in order to use plastic duct would likely cost a post-tensioning supplier or sub-contractor a potential job. A supplier or sub-contractor is not going to unilaterally increase the costs of a project, no matter the long-term benefit to the owner. Therefore, it becomes the owner's or the designer's, as the owner's agent, responsibility to specify corrosion protection methods for a project.

Table 6.2 Cost for Corrosion Protection Methods – Total Bridge

Individual Protection Costs					
Percentages are of Total Bridge Construction Cost				* = Not used in bridge	
		A	B	C	Average
Anchors					
	Additional cost for Galvanized	0.19%	0.23%	0.17%*	0.20%
	Epoxy Coated after Stressing	0.37%	0.32%		0.34%
	Permanent Grout Cap	0.26%	0.23%		0.24%
	Total Cost	0.82%	0.78%	0.40%	*0.67%*
Ducts					
	Steel	0.15%	0.17%	1.55%	0.62%
	Additional Cost for Plastic	0.17%*	0.38%	1.06%*	0.54%
	Improved Installation	0.41%	1.15%		0.78%
	Total Cost	0.56%	1.70%	1.55%	*1.27%*
Strand					
	Steel Cost	4.70%	6.16%	4.54%	5.13%
	Extra tendons for redundancy	0.77%			0.77%
	Total Cost	5.47%	6.16%	4.54%	*5.39%*
Stressing					
	Labor	0.35%	0.28%	0.10%	0.24%
	PT bars - for construction use		0.31%		0.31%
	Total	0.35%	0.59%	0.10%	*0.35%*
Grouting					
	Former standard	0.51%	0.23%		0.40%
	Additional Cost for Prepackaged	1.51%	0.70%		1.10%
	Improved Process	2.78%	2.21%		2.50%
	Total	4.80%	3.14%	0.63%	*2.86%*
Other					
	Epoxy Joints	0.39%	1.30%		0.84%
	Monitoring			0.03%	0.03%
Total Post-Tensioning Cost		12.39%	13.67%	7.22%	11.09%

Table 6.3 Cost for Corrosion Protection Methods – Post-Tensioning System

Individual Protection Costs					
Percentages are of Total Post-Tensioning Cost			* = Not used in bridge		
		A	B	C	Average
Anchors					
	Additional cost for Galvanized	1.57%	1.69%	2.38%*	1.88%
	Epoxy Coated after Stressing	3.04%	2.31%		2.68%
	Permanent Grout Cap	2.09%	1.69%		1.89%
	Total	*6.70%*	*5.69%*	*5.55%*	*5.98%*
Ducts					
	Steel	1.19%	1.20%	21.47%	7.95%
	Additional Cost for Plastic	1.39%*	2.83%	14.66%*	6.29%
	Improved Installation	3.26%	8.36%		5.81%
	Total	*4.45%*	*12.39%*	*21.47%*	*12.77%*
Strand					
	Steel Cost	37.99%	45.06%		41.53%
	Extra tendons for redundancy	6.19%			6.19%
	Total	*44.18%*	*45.06%*	*62.90%*	*50.71%*
Stressing					
	Labor	2.75%	2.04%		2.40%
	PT bars - for construction use		2.24%		2.24%
	Total	*2.75%*	*4.28%*	*1.14%*	*2.72%*
Grouting					
	Former standard	4.10%	1.72%		2.91%
	Additional Cost for Prepackaged	12.23%	5.13%		8.68%
	Improved Process	22.21%	16.20%		19.21%
	Total	*38.54%*	*23.05%*	*8.62%*	*23.40%*
Other					
	Epoxy Joints	3.38%	9.64%		6.51%
	Monitoring			0.33%	0.33%

From Table 6.2 and the description of the *fib* protection levels, Table 6.4 was developed to show expected costs for each level. The bridge's in Menn's survey are all in Switzerland and thus likely reflect a higher protection level than has been the standard in US construction. Thus, the average bridge cost of 11.2%

of total initial construction costs likely reflects costs required for higher protection levels averaged with protection level 1. One of the bridges used in this thesis followed the definition of protection level 1, and thus its costs values are similar to the cost found for this level. The other two bridges used several techniques that are included protection level 2. Where the average total cost for an element was listed in Table 6.2, that value is used for comparison. The cost for plastic duct was assumed as the base cost for steel plus the additional cost required to use plastic. The black steel anchorage was assumed to be the difference between the average total anchorage cost and the average cost of the galvanization improvement. The protection level 1 grouting is assumed to use standard grouting procedures plus prepackaged grout, and thus the total grouting costs from a bridge with this grouting combination was used as the cost. For protection level 2, the grouting was assumed to also have improved grouting processes, which include vacuum grouting, additional shop drawings, and additional grout vent locations. Therefore the protection level 2 costs were assumed to be the average total grouting value, which was heavily weighted by the two bridges with these increased processes. The steel strand costs also vary between the protection levels. For level 1 the average steel cost was assumed and for level 2 the average total steel cost was used, which included the cost of additional tendons for redundancy. No cost or performance information was available for electrical isolation, however this is the essential difference between protection levels 2 and 3. Therefore, an example life cycle analysis could not be made to compare protection level 3 performance.

Table 6.4 Protection Level Costs as a Percentage of Total Construction Cost

Protection Level 1	
Steel Duct	0.62%
Black Steel Anchorage	0.57%
Temporary Grout Caps	No Data
Grouting	0.63%
Stressing	0.27%
Steel Strand	5.13%
Total	*7.22%*

Protection Level 2	
Plastic Duct	1.16%
Improved Installation	0.78%
Galvanized Anchorage	0.67%
Epoxy after Stressing	0.34%
Permanent Grout Cap	0.24%
Grouting	2.86%
Stressing	0.27%
Steel Strand	5.39%
Total	*11.71%*

Protection Level 3
Same costs as Protection Level 2 except with the addition of electrical isolation, for which no cost data was available

6.3 Life Cycle Analysis

To illustrate the potential long term cost savings that inclusion of quality corrosion protection methods can provide, life cycle analyses were run with the available information. These analyses are based upon the relative corrosion ratings from the performance of macrocells in a highly aggressive environment.

As such, they serve as an illustration of the scale of potential savings but cannot be interpreted as giving actual savings amounts. The additional return on investment calculation further illustrates the dramatic lifetime savings possible from an increase in spending at the design and construction phase of a project.

A base case, using steel ducts and normal grout was assumed for comparison. An arbitrary initial construction cost of 100 was used and an annual cost, which includes inspection, maintenance, and annualized rehabilitation, was assumed to be 1.2% of initial construction costs. This value for annual cost was taken from Menn, who found that a range of 1 to 1.2% was a reasonable estimate of annual costs for bridges designed to minimize long term costs[38]. The high end of the range was used, which follows an example in the text that assigns the 1.2% cost to a bridge with higher maintenance costs[38]. As this analysis is for a comparative value, the annual costs only needed to be a reasonable and consistent cost. The percent reductions in damage observed from macrocell testing were used as reductions in annual costs for bridges with corrosion protection. For example, using plastic duct yielded an average 90% reduction in damage versus steel duct specimens. Therefore, the case using plastic duct is assumed to have 90% of the annual costs of the base case: 0.12% of initial construction cost instead of 1.2%. This assumption was made to represent the significant increase in durability expected from using plastic duct. The example analyses assumed a seventy-five year life expectancy. The cost increases assumed are from Table 6.2. The plastic duct case uses the average increase in cost for using plastic duct of 0.54% of total initial construction costs. Improved grouting assumes both the increase for better grout and improving the grouting process, a total of 3.60% (1.10% + 2.50% = 3.60%). The results of the life cycle analyses from using individual protection elements are shown in Tables 6.5-6.7.

Table 6.5 Base Case Life Cycle Analysis

Base Case	
Steel Duct and Normal Grout	
Total Initial Construction Cost	100
Annual Costs	1.2
Lifetime (years)	75
Total Annual Costs	90
Total Lifetime Costs	190
Annualized Cost	2.53

Table 6.6 Plastic Duct Life Cycle Analysis

Plastic Duct Improvement		
Construction Cost - Base	100	
Cost Increase	0.54%	
Total Initial Construction Cost	100.54	
Annual Costs - Base	1.21	
Cost Decrease	90%	
Resulting Annual Costs	0.12	
Lifetime (years)	75	
Total Annual Costs	9.05	
Total Lifetime Costs	109.59	
Annualized Cost	1.46	
Compared to Base Case		
Total Initial Construction Cost	0.54%	More
Total Annual Costs	90%	Less
Total Lifetime Costs	42%	Less
Annualized Costs	42%	Less
Return on Investment	7837%	

Table 6.7 Improved Grouting Life Cycle Analysis

Prepackaged Grout & Training		
Construction Cost - Base	100	
Cost Increase	3.60%	
Total Initial Construction Cost	103.6	
Annual Costs - Base	1.24	
Cost Decrease	40%	
Resulting Annual Costs	0.75	
Lifetime (years)	75	
Total Annual Costs	55.94	
Total Lifetime Costs	159.54	
Annualized Cost	2.13	
Compared to Base Case		
Total Initial Construction Cost	3.60%	More
Total Annual Costs	38%	Less
Total Lifetime Costs	16%	Less
Annualized Costs	16%	Less
Return on Investment	445%	

In addition to examining the effect of individual corrosion protection methods, the author was interested in the comparison of protection level 1 with protection level 2. Due to the expected significant increase in durability from using plastic duct with preformed couplers, galvanizing the anchorage, protecting the strand tails with permanent grout caps, and increasing the grouting quality, an increase in design life is assumed for protection level 2 versus protection level 1. Therefore, protection level 1 is assumed to attain a useful life of 75 years and protection level 2 is assumed to last for 100 years. These values are assumed for the purposes of comparison, based upon the author's opinion that using protection level 2 versus protection level 1 will substantially increase a bridge's useable life in aggressive environments. They are not guaranteed times for actual life. The cost increase between level 1 and level 2 of 4.49% (11.71% – 7.22% = 4.49%) of the total initial construction cost is assumed from the costs listed in Table 6.4. In addition to having a longer usable life, this increase in tendon level durability is also expected to decrease the annual costs of the structure. Thus, the annual costs for protection level 1 are assumed to be the same 1.2% of the total initial construction cost used for the base case in the preceding example. The reduction in maintenance costs for protection level 2 is assumed to be the same as the percentage reduction found for plastic duct, as the superior performance of the plastic duct is expected to have the greatest impact on this protection level's durability. For comparison, protection level 1 is taken as the base case with total initial construction costs of 100. The results of the protection level life cycle analyses are shown in Tables 6.8 and 6.9.

Table 6.8 Life Cycle Analysis for Protection Level 1

Protection Level 1	
Total Initial Construction Cost	100
Annual Costs	1.2
Lifetime (years)	75
Total Annual Costs	90
Total Lifetime Costs	190
Annualized Cost	2.53

Table 6.9 Life Cycle Analysis for Protection Level 2

Protection Level 2		
Construction Cost - Base	100	
Cost Increase	4.49%	
Total Initial Construction Cost	104.49	
Annual Costs - Base	1.25	
Cost Decrease	90%	
Resulting Annual Costs	0.13	
Lifetime (years)	100	
Total Annual Costs	12.54	
Total Lifetime Costs	117.03	
Annualized Cost	1.17	
Compared to Protection Level 1		
Total Initial Construction Cost	4.49%	More
Total Annual Costs	90%	Less
Total Lifetime Costs	38%	Less
Annualized Costs	54%	Less
Return on Investment	855%	

The protection level comparison indicates that protection level 2 will yield over an 800% return on investment and this calculation does not even consider the cost savings afforded the owner by gaining an additional 25 years of useful life out of the bridge. Thus, the preceding analyses show the great economic advantage the owner gains from using corrosion protection. Owners are expending the money to both construct and repair the bridge, and thus improving the durability is to their economic advantage. As seen by the higher percent increases experienced by the post-tensioning contractor or supplier, it is not to their advantage to suggest a more expensive system. If an owner wants the sort of return on investment offered by using corrosion protection systems, approximately 400% to 7800% as shown in the examples, they must specify their use.

6.4 SUMMARY AND INDICATIONS

From the above analyses and the information available, a trend of substantial lifetime cost savings due to using corrosion protection is evident. If a bridge is built in a non-aggressive setting, such as in a region with no road-salt application or saltwater, then a high level of corrosion protection will not yield such significant savings. However, paying for quality materials and workmanship should still easily pay for themselves. Poor quality often results in early and increased maintenance and repair costs. Thus, where corrosion protection methods are appropriate, their use will save the owner in the bridge's lifetime costs. As well, simply from examination of the protection costs table, the low additional cost of corrosion protection is seen. Some of the main observations from the cost and performance data are listed below:

- The cost increase for adding corrosion protection was very low. Most individual protection methods cost less than 1% of the total initial construction cost.
- Specifying protection level 2 instead of protection level 1 increases the initial construction cost by approximately 4.5% while decreasing annualized costs by approximately 50%.
- Using corrosion protection can significantly reduce lifetime expenditures on the bridge as well as increase the bridge's usable life. By examining costs with life cycle analysis, it is seen the use of corrosion protection methods could save 15 to 40% of the total money spent on the bridge throughout its design life.
- Return-on-investment values from using corrosion protection methods in aggressive environments were very high.

Designers or other decision makers should refer to a design guideline to determine the protection level that is appropriate for a given exposure. If local experience indicates a higher level of protection is needed then the higher level should be used. A life cycle analysis developed for the particular structure and its environment will give the most accurate cost savings for each option. This process allows the designer to optimize the protection methods selected for minimum annualized bridge costs.

CHAPTER 7
Summary and Conclusions

7.1 SUMMARY OF RESEARCH

This thesis discussed the corrosion protection of post-tensioned bridges, focusing on the design decision-making process. At the design stage, the engineer and owner have the most influence over the future durability. Yet the current design guidelines are not clear on what protections methods are appropriate for a given exposure environment. As well, a low bid culture has encouraged owners to be reluctant to spend additional money on the initial construction costs in order to improve the quality of the bridge. To address the lack of corrosion protection design guidance the author had the following objectives:

- Determine the cost increase, as a percentage of total construction costs, of post-tensioning corrosion protection methods.
- Discuss the corrosion of post-tensioning systems and various corrosion protection methods as a basic guide for owners and designers who are not intimately familiar with the underlying mechanisms and protection approaches.
- Evaluate the effect of using post-tensioning corrosion protection methods on annualized total bridge cost by running life cycle analyses.
- Examine post-tensioning durability design guidelines and suggest a more logical and designer-friendly way to combine the currently available guidelines.

Chapter 2 is a detailed background of post-tensioning, including a discussion of how the system works and post-tensioning strand's susceptibility to corrosion. A brief summary of known corrosion problems in post-tensioned bridges as well as the research and methods that have been developed to address corrosion is also included. The basic concept behind the economic model that is discussed in Chapter 4 is also introduced here. This chapter is intended as a reference for those not familiar with post-tensioning.

Chapter 3 is a description of current post-tensioning protection methods. The *fib* Task Group 5.4.2 proposal is also introduced here, specifically focusing on its descriptions of different protection levels. This chapter is intended as a reference for those not familiar with corrosion protection methods for post-tensioned concrete.

Chapter 4 is a discussion on the economic model used to evaluate the benefits of corrosion protection. The method for collecting cost and performance data is discussed. The availability of information on the cost of all protection methods and their effectiveness limited the number of life cycle analyses calculated. This chapter also describes the basics of a probabilistic theoretical model that, if developed, would lead to a more sophisticated design process for corrosion protection.

Chapter 5 is a discussion of current and recently proposed design guidelines for post-tensioning corrosion protection. The author also proposed a combination of existing recommendations as a more logical and user-friendly design guideline.

Chapter 6 is a presentation of results from the cost survey and life cycle analyses. Though the information was limited and could only give relative values, the cost savings indicated from use of increased corrosion protection is significant. For the life cycle analyses considered, a return on investment calculation was also made to show the savings in relation to the initial expenditure.

7.2 CONCLUSIONS

The main observations from the economic research are listed below:

- The cost increase for adding corrosion protection was very low. Most individual protection methods cost less than 1% of the total initial construction cost.
- Specifying protection level 2 instead of protection level 1 increases the initial construction cost by approximately 4.5% while decreasing annualized costs by approximately 50%.
- Using corrosion protection can significantly reduce lifetime expenditures on the bridge as well as increase the bridge's usable life. By examining costs with life cycle analysis, it is seen that the use of corrosion protection methods could save 15 to 40% of the total money spent on the bridge throughout its design life.
- Return-on-investment values from using corrosion protection methods in aggressive environments were very high.

7.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The author experienced significant resistance from the post-tensioning industry when trying to collect cost data. This resistance came from the desire of various manufacturers and suppliers to protect cost data and perceived economic advantages. Thus any future attempts to collect cost information should try to request the cost in terms of a percentage of the total cost. This suggestion is no guarantee that there will then be cooperation. However, this route was not explored by the author. As all the information received was converted into percentages, this alternate reporting method would have caused no hardship to the research program.

Further research on the corrosion protection effectiveness of electrically isolated systems is recommended. This information would greatly aid the determination of an appropriate protection factor in a numerically based design guide and provide performance data to be used in a life cycle analysis. Several other protection variables do not have currently available data and are under exposure testing. Their final performance will not be known until final autopsy of the specimens.

With the collection of more performance and cost data the life cycle analyses should be updated, including the analysis of different variables. Correlation of field data to accelerated corrosion testing would greatly aid future life cycle analyses. A correlation method would give the relative performance data from testing equivalent actual life extension data or maintenance requirements.

APPENDIX A
Cost Survey

The cost survey developed by the author and her supervising professor and sent to post-tensioning industry representatives as described in Section 4.4.1 is included here as Figure A.1.

General Information *	
Bridge Name and Location:	
Bridge Type:	
Nature of Exposure:	
Contractor:	
PT Supplier:	
Date of Construction:	

Post-tensioning System Costs		Currency:		
	Material	Labor	Overhead & Profit	Total
a) Anchors				
b) Ducts				
c) Tendons				
d) Stressing				
e) Grouting				
f) Monitoring				

Overall Bridge Description	Indicate Units ft, m, ft^2, m^2
Span Lengths:	
Width:	
Total Deck Area:	
Total Cost:	

PT System Description		
Internal Tendons:	Number:	
	Size:	
	Duct Material:	
	Duct Couplers:	
External Tendons:	Number:	
	Size:	
	Duct Material:	
	Duct Couplers:	
Any special corrosion protection used:		

* If you determined costs for differing levels of protection, please submit a separate sheet for each alternate system.

Figure A.1 Cost Survey

APPENDIX B

Corrosion Rating Comparison

This appendix consists of the macrocell corrosion rating results from West's and Salas' autopsies and evaluation. The Table B.1 explains the specimen notation. Table B.2 and B.4 list the autopsied specimens and their ratings. Tables B.3 and B.5 are the specimen pairings used to find the average reduction in damage resulting from the use of corrosion protection methods.

Table B.1 Specimen Notation[3,11]

Joint Type	Duct Type	Joint Precompression	Grout Type
DJ: Dry Joint	**S:** Galvanized Steel	**L:** Low, 5 psi	**NG:** Normal Grout
SE: Standard Epoxy		**M:** Medium, 50 psi	**SF:** Silica Fume Added
	P: Plastic		
EG: Epoxy with Gasket		**H:** High, 190 psi ($3\sqrt{f'c}$)	**CI:** Corrosion Inhibitor
Example: DJ – S – L – NG			

West Results – 4.4 years[3]

Table B.2 West Corrosion Ratings

Corrosion Rating				
Information from West - 4.4 years				
Specimen	Strand	Bars	Duct	Total
DJ-S-L-NG-1	26	12	528	566
DJ-S-M-NG-1	43	12	325	380
DJ-S-H-NG-1	38	60	64	162
DJ-P-L-NG-1	6	17	0	23
DJ-P-M-NG-1	9	24	0	33
DJ-S-L-CI-1	114	4	42	160
DJ-S-M-CI-1	24	20	151	195
SE-S-L-NG-2	13	6	22	41
SE-S-M-NG-2	2	16	61	79
SE-S-H-NG-2	3	0	8	11
SE-P-L-NG-2	5	0	0	5
SE-P-M-NG-2	6	0	0	6
SE-S-L-CI-2	24	0	85	109
SE-S-M-CI-2	2	0	114	116
SE-S-H-CI-2	3	1	10	14
SE-S-L-SF-2	12	0	12	24
EG-S-L-NG-2	2	0	54	56
EG-S-M-NG-2	23	0	237	260
EG-S-H-NG-2	16	1	78	95

Table B.3 Comparison of West Corrosion Ratings

Comparisons	% Difference (- means first listed had lower rating)			
Steel Duct v. Plastic Duct	Strand	Bars	Duct	Total
DJ-S-L-NG-1 v. DJ-P-L-NG-1	77%	-42%	100%	96%
DJ-S-M-NG-1 v. DJ-P-M-NG-1	79%	-100%	100%	91%
SE-S-L-NG-2 v. SE-P-L-NG-2	62%	100%	100%	88%
SE-S-M-NG-2 v. SE-P-M-NG-2	-200%	100%	100%	92%
Avg Plastic Duct Improvement	4%	15%	100%	92%

Dry Joint v. Epoxy Joint	Strand	Bars	Duct	Total
DJ-S-L-NG-1 v. SE-S-L-NG-2	50%	50%	96%	93%
DJ-S-M-NG-1 v. SE-S-M-NG-2	95%	-33%	81%	79%
DJ-S-H-NG-1 v. SE-S-H-NG-2	92%	100%	88%	93%
DJ-P-L-NG-1 v. SE-P-L-NG-2	17%	100%	0%	78%
DJ-P-M-NG-1 v. SE-P-M-NG-2	33%	100%	0%	82%
DJ-S-L-CI-1 v. SE-S-L-CI-2	79%	100%	-102%	32%
DJ-S-M-CI-1 v. SE-S-M-CI-2	92%	100%	25%	41%
Avg Epoxy Improvement	65%	74%	27%	71%

Dry Joint v. Epoxy & Gasket	Strand	Bars	Duct	Total
DJ-S-L-NG-1 v. EG-S-L-NG-2	92%	100%	90%	90%
DJ-S-M-NG-1 v. EG-S-M-NG-2	47%	100%	27%	32%
DJ-S-H-NG-1 v. EG-S-H-NG-2	58%	98%	-22%	41%
Avg Epoxy & Gasket Improv.	66%	99%	32%	54%

Table B.2 continued Comparison of West Corrosion Ratings

Normal Grout v. Corr. Inhibitor	Strand	Bars	Duct	Total
DJ-S-L-NG-1 v. DJ-S-L-CI-1	-338%	67%	92%	72%
DJ-S-M-NG-1 v. DJ-S-M-CI-1	44%	-67%	54%	49%
SE-S-L-NG-2 v. SE-S-L-CI-2	-85%	100%	-286%	-166%
SE-S-M-NG-2 v. SE-S-M-CI-2	0%	100%	-87%	-47%
SE-S-H-NG-2 v. SE-S-H-CI-2	0%	0%	-25%	-27%
Avg Corrosion Inhibitor Improv.	-76%	40%	-51%	-24%

Normal Grout v. Silica Fume	Strand	Bars	Duct	Total
SE-S-L-NG-2 v. SE-S-L-SF-2	8%	100%	45%	41%
DJ-S-L-NG-1 v. SE-S-L-SF-2	54%	100%	98%	96%
Average Silica Fume Improv.	8%	100%	45%	41%
Average with Dry Joint Data	31%	100%	72%	69%

Salas Results – 8 years[11]

Table B.4 Salas Corrosion Ratings

Corrosion Rating				
Information from Salas - 8 years				
Specimen	Strand	Bars	Duct	Total
DJ-S-L-NG-2	612	54	15779	16445
DJ-S-M-NG-2	780	44	3054	3878
DJ-S-H-NG-2	137	606	361	1104
DJ-P-L-NG-2	116	201	0	317
DJ-P-M-NG-2	80	77	0	157
DJ-S-L-CI-2	86	22	674	782
DJ-S-M-CI-2	54	27	346	427
SE-S-L-NG-1	64	26	167	257
SE-S-M-NG-1	119	41	732	892
SE-S-H-NG-1	88	29	268	385
SE-P-L-NG-1	80	0	0	80
SE-P-M-NG-1	88	18	0	106
SE-S-L-CI-1	95	28	126	249
SE-S-M-CI-1	305	29	2445	2779
SE-S-H-CI-1	78	132	44	254
SE-S-L-SF-1	88	13	591	692
EG-S-L-NG-1	88	25	1096	1209
EG-S-M-NG-1	90	31	198	319
EG-S-H-NG-1	84	34	131	249

Table B.5 Comparison of Salas Corrosion Ratings

Comparisons	% Difference (- means first listed had lower rating)			
Steel Duct v. Plastic Duct	Strand	Bars	Duct	Total
DJ-S-L-NG-2 v. DJ-P-L-NG-2	81%	-272%	100%	98%
DJ-S-M-NG-2 v. DJ-P-M-NG-2	90%	-75%	100%	96%
SE-S-L-NG-1 v. SE-P-L-NG-1	-25%	100%	100%	69%
SE-S-M-NG-1 v. SE-P-M-NG-1	26%	56%	100%	88%
Avg Plastic Duct Improvement	43%	-48%	100%	88%

Dry Joint v. Epoxy Joint	Strand	Bars	Duct	Total
DJ-S-L-NG-2 v. SE-S-L-NG-1	90%	52%	99%	98%
DJ-S-M-NG-2 v. SE-S-M-NG-1	85%	7%	76%	77%
DJ-S-H-NG-2 v. SE-S-H-NG-1	36%	95%	26%	65%
DJ-P-L-NG-2 v. SE-P-L-NG-1	31%	100%	0%	75%
DJ-P-M-NG-2 v. SE-P-M-NG-1	-10%	77%	0%	32%
DJ-S-L-CI-2 v. SE-S-L-CI-1	-10%	-27%	81%	68%
DJ-S-M-CI-2 v. SE-S-M-CI-1	-465%	-7%	-607%	-551%
Avg Epoxy Improvement	-35%	42%	-46%	-19%

Dry Joint v. Epoxy & Gasket	Strand	Bars	Duct	Total
DJ-S-L-NG-2 v. EG-S-L-NG-1	86%	54%	93%	93%
DJ-S-M-NG-2 v. EG-S-M-NG-1	88%	30%	94%	92%
DJ-S-H-NG-2 v. EG-S-H-NG-1	39%	94%	64%	77%
Avg Epoxy & Gasket Improv.	71%	59%	83%	87%

Table B.6 continued Comparison of Salas Corrosion Ratings

Normal Grout v. Corr. Inhibitor	Strand	Bars	Duct	Total
DJ-S-L-NG-2 v. DJ-S-L-CI-2	86%	59%	96%	95%
DJ-S-M-NG-2 v. DJ-S-M-CI-2	93%	39%	89%	89%
SE-S-L-NG-1 v. SE-S-L-CI-1	-48%	-8%	25%	3%
SE-S-M-NG-1 v. SE-S-M-CI-1	-156%	29%	-234%	-212%
SE-S-H-NG-1 v. SE-S-H-CI-1	11%	-355%	84%	34%
Avg Corrosion Inhibitor Improv.	-3%	-47%	12%	2%

Normal Grout v. Silica Fume	Strand	Bars	Duct	Total
SE-S-L-NG-1 v. SE-S-L-SF-1	-38%	50%	-254%	-169%
DJ-S-L-NG-2 v. SE-S-L-SF-1	86%	76%	96%	96%
Average Silica Fume Improv.	-38%	50%	-254%	-169%
Average with Dry Joint Data	24%	63%	-79%	-37%

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VITA

Kimberly Ann Grau was born in Salisbury, North Carolina on January 16, 1980, the daughter of Robert W. and Cynthia L. Grau. After graduating from William G. Enloe High School, Raleigh, North Carolina, in 1998, she enrolled at North Carolina State University in Raleigh, North Carolina. During the summer of 2002 she studied abroad at Oxford University. She graduated as valedictorian of North Carolina State University with the degrees Bachelor of Science in Construction Engineering and Management and Bachelor of Science in History in December 2002. After graduation, Kimberly worked as a historian for the Department of Civil Engineering at North Carolina State University and studied abroad at the ROSE School at Istituto Universitario di Studi Superiori, the European School for Advanced Studies in Reduction of Seismic Risk, Pavia, Italy. She entered The Graduate School at The University of Texas at Austin in August 2003.

Permanent Address: 5200 Wood Valley Dr.
Raleigh, NC 27613

This thesis was typed by the author.