APPENDIX D

HIGHWAY BRIDGES


Corroded bridge column


Cracking and spalling of piers in marine environment


Condition assessment


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Corroded rebar


Corroded concrete column


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Corrosion in the free length of tendon

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Epoxy-coated rebar construction on deck

Installation of titanium mesh impressed-current anode system on bridge deck

Sacrificial CP applied to pier columns in marine environment

HIGHWAY BRIDGES

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SUMMARY AND ANALYSIS OF RESULTS

Corrosion Control and Prevention

Cost of Corrosion

The dollar impact of corrosion on highway bridges is considerable. The annual direct cost of corrosion for highway bridges is estimated to be $6.43 billion to $10.15 billion, consisting of $3.79 billion to replace structurally deficient bridges over the next 10 years, $1.07 billion to $2.93 billion for maintenance and cost of capital for concrete bridge decks, $1.07 billion to $2.93 billion for maintenance and cost of capital for concrete substructures and superstructures (minus decks), and $0.50 billion for the maintenance painting cost for steel bridges. This gives an average annual cost of corrosion of $8.29 billion. Life-cycle analysis estimates indirect costs to the user due to traffic delays and lost productivity at more than 10 times the direct cost of corrosion. In addition, it was estimated that employing “best maintenance practices” versus “average practices” can save 46 percent of the annual corrosion cost of a black steel rebar bridge deck, or $2,000 per bridge per year.

While there is a downward trend in the percentage of structurally deficient bridges (a decrease from 18 percent to 15 percent between 1995 to 1999), the costs to replace aging bridges increased by 12 percent during the same period. In addition, there has been a significant increase in the required maintenance of the aging bridges. Although the vast majority of the approximately 108,000 prestressed concrete bridges have been built since 1960, many of these bridges will require maintenance in the next 10 to 30 years. Therefore, significant maintenance, repair, rehabilitation, and replacement activities for the nation’s highway bridge infrastructure are foreseen over the next few decades before current construction practices begin to reverse the trend.

Conventional Reinforced-Concrete Bridges

The primary cause of reinforced-concrete bridge deterioration is chloride-induced corrosion of the black steel reinforcement, resulting in expansion forces in the concrete that produce cracking and spalling of the concrete. The chloride comes from either marine exposure or the use of deicing salts for snow and ice removal. Because the use of deicing salts is likely to continue, if not increase, little can be done to prevent bridge structures from being exposed to corrosive chloride salts. Therefore, bridge designs and concrete mixes must be resistant to chloride-induced corrosion. This can be accomplished by: (1) preventing chlorides from getting to the steel surface (physical barriers at the concrete surface, coating the rebar, or low chloride-permeable concrete), (2) making the concrete less corrosive at specific chloride levels (inhibitors or admixtures), or (3) making the rebar resistant to corrosion (corrosion-resistant alloys, composites, or clad materials).

Over the past 20 years, there has been a trend in new construction toward utilizing higher quality concrete and more corrosion-resistant rebars. Longer bridge service life is currently achieved by using epoxy-coated rebars

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in the majority of new bridge construction, with the limited use of stainless steel-clad or solid rebars in more severe environments. The expected service life of a newly constructed bridge is typically 75 years and up to 120 years for stainless steel rebar construction. Admixtures to the concrete for the purpose of increased corrosion resistance have included corrosion-inhibiting admixtures and mineral admixtures such as silica fume. High-range water reducers permit the use of low water-cement ratio concretes that have lower permeability to corrosive agents and, thus, result in longer times to corrosion initiation of the rebar. Many of these methods are used in combination with each other to obtain a longer service life.

Many rehabilitation methodologies designed to extend the service life of bridges that have deteriorated due to corrosion of the reinforcing steel have been developed and put into practice within the past 25 years. These include cathodic protection, electrochemical chloride removal, overlays, and sealers. Although each of these methods have been shown to be successful, continuing developments are necessary to improve effectiveness and increase the life extension provided by these methods.

Prestressed Concrete Bridges

Whereas some of the methods discussed for conventional reinforced-concrete bridges are applicable to prestressed concrete components (e.g., high-performance concrete and corrosion inhibiting admixtures), special consideration for corrosion prevention of prestressed reinforced-concrete bridges is required.

Most of these bridges are relatively new and their numbers are relatively low; therefore, the overall economic impact is not as significant as for conventional reinforced-concrete bridges. However, failure of the high-strength prestressing steel can compromise the integrity of the prestressed concrete bridge (corrosion-related deterioration compromising the structural integrity of a conventional concrete structure is highly unlikely). This makes close attention to construction details and subsequent monitoring and inspection of the prestressed concrete bridges critical.

Corrosion prevention of pretensioned structures is primarily accomplished through the use of high-performance concretes or the addition of corrosion-inhibiting admixtures. Remedial measures such as cathodic protection are possible as long as care is taken to prevent overprotection that can lead to hydrogen-induced cracking of the high-strength steel. Other measures such as electrochemical chloride removal cannot be used for prestressed concrete structures because of the relatively large amounts of hydrogen produced at the steel surface during the removal process.

Recent failures of post-tensioned structures have underscored the importance of maintaining void-free grouting of the tendons, especially near the anchorage. Maintaining the integrity of the post-tensioned tendon starts with ensuring the integrity of the duct (typically polyethylene), followed by the application of a good-quality grout that is continuous around the strands. Placement of the grout is often more difficult when low water-cement ratio mixes and/or mineral admixtures are employed. Improved grouting practices are continuing to be developed. In addition, the use of corrosion-inhibiting admixtures can provide added protection against corrosion of the prestressing steel strands. Note that in August 2001, the American Segmental Bridge Institute conducted a 3-day training school for certifying grouting specialists. This training school will be held in the future once or twice a year.

Steel Bridges

The primary cause of corrosion of steel bridges is the exposure of the steel to atmospheric conditions. This corrosion is greatly enhanced due to marine (salt spray) exposures and industrial environments. The only corrosion prevention method for these structures is to provide a barrier coating (paint).

Changes in environmental protection regulations have brought about transformation of the approach to corrosion protection for steel bridges. Until the mid- to late-1970s, virtually all steel bridges were protected from corrosion by multiple thin coats of lead- and chromate-containing alkyd paints applied directly over mill scale on the formed steel. Maintenance painting for prevention of corrosion was rare and primarily was practiced on larger bridge structures. Since the majority of the steel bridges in the interstate highway system were constructed between 1950 and 1980, most of these structures were originally painted in this manner; therefore, a large percentage of the steel bridges in the interstate system are protected from corrosion by a coating system that is now beyond its useful service life.

Moreover, the paint system commonly used for steel bridge members contains chromium and lead and can no longer be used because of the effects it has on humans and the environment. The bridge engineers have a choice of either replacing the lead-based paints with a different coating or painting over the deteriorating areas. Removal of lead-based paint incurs high costs associated with the requirements to contain all the hazardous waste and debris.

Developments include: (1) improved and environmentally safe coating systems and (2) methodologies to optimize the use of these systems, such as “zone” painting (adjusting coating types and maintenance schedules based on the aggressiveness of the environment within different zones on the bridge). Overpainting techniques to eliminate the cost of expensive paint removal also have been developed.

Opportunities for Improvement and Barriers to Progress

A typical dilemma of bridge management is how to allocate the often insufficient funds for construction, rehabilitation, and maintenance. Compounding the problem is that funding typically comes from city, state, and federal sources with spending restrictions based on the funding source. This makes allocating the funds in order to optimize construction, rehabilitation, and maintenance decisions difficult. The cooperation of these different funding agencies is required to permit allocation of resources to achieve the best cost benefit.

An increased need for bridge inspection has placed additional drains on maintenance funds. In the case of prestressed concrete bridges, the issue of careful inspection becomes particularly acute because an individual failure of a tendon may have a significant impact on the structural integrity of the bridge. The importance of inspection was recently illustrated when tendon failures in two Florida bridges were identified through routine inspections before the safety of the bridges was compromised. The economic analysis performed in this study showed that monitoring of bridge condition and subsequent maintenance based on that information (information-based maintenance) was the most cost-effective maintenance strategy.

The economic analysis further indicated that capital funding for the higher quality materials of construction (e.g., epoxy-coated rebars) results in lower annualized costs due to postponement of repair/rehabilitation expenses incurred by the owner agency. The analysis further indicated that user costs (traffic delays during maintenance) are significant and can be 10 times greater than the direct costs to the owner/operator. This places a premium on the selection of materials of construction that minimize maintenance over the bridge service life. It also highlights the importance of careful planning for traffic control and alternative routes during bridge maintenance and rehabilitation activities.

The significant rise in costs for maintenance of steel bridges (environmental issues dealing with lead paint removal and handling of volatile organic compounds) has placed a significant strain on maintenance budgets. In fact, over the past few years, environmental regulations have become the single most influential force in the bridge painting industry. The focus for expenditures must shift to long-term effectiveness of dollars spent. This is a significant change in philosophy for a majority of the bridge painting industry. To date, bridge maintenance painting has been accomplished based on incremental budgets, rather than life-cycle considerations.

Additionally, the use of technological advances among bridge owners has not been uniform. This can, in part, be explained by the difference in funding and technical staffing between the agencies. Because of the perceived high costs of certain corrosion control methods, these methods go unused. With the general tendency to reduce the maintenance departments’ size and budget, corrosion control becomes one of the many responsibilities of personnel without the experience to understand the problems and without the knowledge of available solutions. There remains a significant need for life-cycle cost analysis to aid in the selection of repair-rehabilitation-replacement decisions.

Recommendations and Implementation Strategy

The technological advances, both in concrete (conventional and prestressed) and steel bridge corrosion control methodologies and construction materials, provide the opportunity that the newly constructed bridges will last considerably longer than the bridges that were constructed 20 to 30 years ago. However, newly developed materials of construction and corrosion control methodologies must be implemented properly over the entire bridge project (both design and construction phases).

These improvements, however, do not signify that the problems with corrosion on highway bridges will disappear soon. The percentage of deficient bridges, while declining, still remains high. At the same time, the costs of bridge repair and rehabilitation are steadily increasing, thereby offsetting any potential savings. Some of the bridges owned by state and city agencies simply cannot be replaced due to their historic value and/or the enormous strain on the traffic resulting from a bridge closure (e.g., the New York City East River bridges and the Oregon coastal U.S. Highway 101 bridges). These bridges are maintained and rehabilitated even at high costs.

There is an urgent need for allocation of greater monetary resources, so that the bridge engineers can properly maintain the structures based on timely inspections, thereby optimizing maintenance practices. At present, maintenance personnel are forced to make the choices based on inadequate funds, which will ultimately lead to a less-than-optimal cost benefit.

Despite appreciation of the corrosion-related issues in the bridge community, there is still a need for raising awareness and the transfer of the advanced methodologies for efficient corrosion protection to the end-users. The Federal Highway Administration (FHWA), which has amassed considerable research and field application data on corrosion protection methods for concrete and steel bridges, has served as an effective conduit for dissemination of such information through periodic demonstration programs and educational seminars. These demonstration programs have been successful and should be continued with increased staffing and funding levels.

There remains a considerable need for additional research in innovative construction materials such as corrosion-resistant alloy/clad rebars (metallic and non-metallic) and more durable concretes with inherent corrosion-resistant properties. In addition, research and development is needed in rehabilitation technologies that can mitigate corrosion with minimal maintenance requirements, such as sacrificial cathodic protection systems.

Summary of Issues

Increase consciousness of corrosion control costs and potential savings.	The bridge owners are typically aware of the severity of corrosion problems and the need to prevent corrosion through better construction and regular maintenance; however, the best intentions are often hampered by the shortage of funds and insufficient staffing. The agencies often face the necessity of spreading the funds over the large population of bridges, favoring the use of cheaper conventional materials of construction and methods of rehabilitation, despite higher life-cycle costs. The maintenance burden will probably increase and become more costly with time. There is a need for greater funding levels and better allocation of resources to encourage optimum life-cycle costing decisions. When the cost of a particular project is calculated, the indirect costs to society typically are not taken into consideration, although these can be considerably higher than the capital expenditures. At present, the decision-making process is controlled by the owner agency, which is primarily concerned with direct budget costs.
Change perception that nothing can be done about corrosion.	There is insufficient awareness of corrosion control in some of the agencies. Knowledge of advanced corrosion control methods is unevenly distributed among the bridge operators. Research, education seminars, and demonstration programs administered through FHWA should be given higher priority in the agency budget.

Advance design practices for better corrosion management.	The modern methods of corrosion protection are well documented in FHWA, NACE, and other industry publications. Limited use of some of the approaches to corrosion-resistant bridge construction is largely predicated on balancing the available capital funds with new construction and rehabilitation needs.
Change technical practices to realize corrosion cost-savings.	While lack of capital funding for higher cost, corrosion-resistant materials is certainly a concern, these higher costs may result in a lower annualized cost for the bridge. An example of this is the use of epoxy-coated rebars for concrete structures in non-marine applications. There is only a marginal increase in the overall construction costs (typically 1 percent); however, the extension of the bridge service life can be significant when compared to conventional black steel rebars. The use of epoxy-coated rebar is an example where practices have changed; the majority of new construction uses the new technology.
Change policies and management practices to realize corrosion cost-savings.	Diligent maintenance of steel and concrete bridges is imperative because it saves money in the long term. Some structures, such as post-tensioned bridges, require particular attention because they can suffer sudden catastrophic failures if not properly maintained, leading to significant losses (both direct and indirect). Often, optimum bridge management is hampered by funding mechanisms (there is an imbalance in maintenance, rehabilitation, and new construction funds); more flexible cooperation among funding agencies is required.
Advance life-prediction and performance assessment methods.	Many attempts have been made to develop life-prediction models for concrete bridge decks based on the materials of construction, repair materials, and exposure conditions. Although these models have become progressively more complex and require multiple data parameter inputs, they still fall short of the desired accuracy in predicting the remaining life of the structure. This failure is primarily because corrosion is dependent on a wide range of factors that are difficult to account for in the model. Further research is required in this direction, with an additional focus on making the models software-based and user-friendly to ensure the wider usage.
Advance technology (research, development, and implementation).	It is important to continue research efforts to further understanding of the impact of different corrosion control methodologies on bridge performance. There may be a potential benefit from establishing an industry-wide coordinating body to ensure that the efforts are not duplicated, and the findings become available to the community at large. Presently, research programs are sponsored by a variety of bodies, such as FHWA, state highway departments, National Cooperative Highway Research Program, the American Concrete Institute, the Precast/Prestressed Concrete Institute, or private institutions.
Improve education and training for corrosion control.	Despite the generally high level of awareness about the issue of corrosion in this sector, there is a disparity between the degree of awareness and the application of knowledge of modern corrosion control methodologies. Given the often insufficient staffing of the maintenance departments of the bridge owner agencies, education of the responsible personnel in corrosion control and monitoring methodology becomes particularly important. The use of the life-cycle cost analysis has been limited and should be aggressively promoted.

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SECTOR DESCRIPTION

Background

According to the National Bridge Inventory Database, the total number of bridges in the United States is approximately 600,000, of which half were built between 1950 and 1994.(1) The materials of construction for these bridges are concrete, steel, timber, masonry, timber/steel/concrete combinations, and aluminum. This sector is focused on reinforced-concrete and steel bridges, which make up the vast majority of these structures built since 1950 and which can undergo significant deterioration due to corrosion.

The elements of a typical bridge structure can be classified into two primary components, the substructure and the superstructure. The substructure refers to the elements of the bridge that transfer the loads from the bridge deck to the ground, such as abutments and piers. The superstructure refers to the elements of the bridge above the substructure, including the deck, floor system (beams or stringers), supporting members (beams, trusses, frames, girders, arches, or cables), and bracing. Other bridge elements, which are subject to corrosion, include guardrailings and culverts.

The maintenance burden of aging bridges has become significant. In a 1998 report by the American Society of Civil Engineers (ASCE), the condition of bridge structures was rated as “poor” and was recognized as being among the largest contributors to the U.S. infrastructure cost of corrosion.(2) The important issues related to corrosion causes and corrosion control with respect to steel reinforced-concrete bridges and steel bridges are discussed in detail below. The types of bridges refer to the superstructure from which the bridge is constructed.

Steel Reinforced-Concrete Bridges

Due to the specific concrete property of weak tensile strength as compared to its compressive strength, steel reinforcing is placed in the tension regions in concrete members, such as decks and pilings. The two primary forms of steel reinforcing in concrete bridges are “conventional” reinforcing bar (rebar) and prestressed tendons. The difference between conventional reinforcement and prestressed tendon reinforcement is that prestressed tendons are loaded in tension (prestressed) either prior to placing the concrete (pretensioned) or after placing and curing of the concrete (post-tensioned). In addition, prestressed-tendon steel typically has a higher tensile strength than conventional rebar steel.

The majority of the concrete deterioration leading to reduced service life and/or replacement is associated with conventional reinforced-steel bridge structures. This is, in part, due to these structures making up the majority of reinforced-concrete bridges and the longer in-service times experienced by these structures. Although conventional rebar and prestressed tendon bridge structures have specific design and construction corrosion-related concerns and consequences, the basic mechanism of corrosion is similar and many corrosion control methods are applicable to both (see below).

Conventional Reinforced Concrete

Reinforced-concrete bridges suffer from corrosion of the reinforcement and, consequently, concrete degradation due to the high tensile forces exerted by the corroding steel (corrosion products have a three to six times greater volume than the original steel). These high tensile forces cause cracking and spalling of the concrete at the reinforcement (see figures 1 and 2). Steel in high-pH concrete in the absence of chloride ions is normally passive and corrosion is negligible, which in theory should give reinforced-concrete structures an extremely long operating life. However, in practice, corrosion in concrete can be accelerated through two primary mechanisms:
(1) breakdown of the passive layer on the steel by chloride ions and, to a lesser degree, (2) carbonation due to carbon dioxide reactions with the cement phase of the concrete. For highway bridge structures with a relatively thick concrete layer over the reinforcing steel, the vast majority of problems are caused by chloride ion migration into the

concrete due to deicing salt application and marine exposure. Once the chloride ions reach the steel surface, the passive film becomes locally disrupted, creating conditions conducive to accelerated corrosion attack on the reinforcing steel.

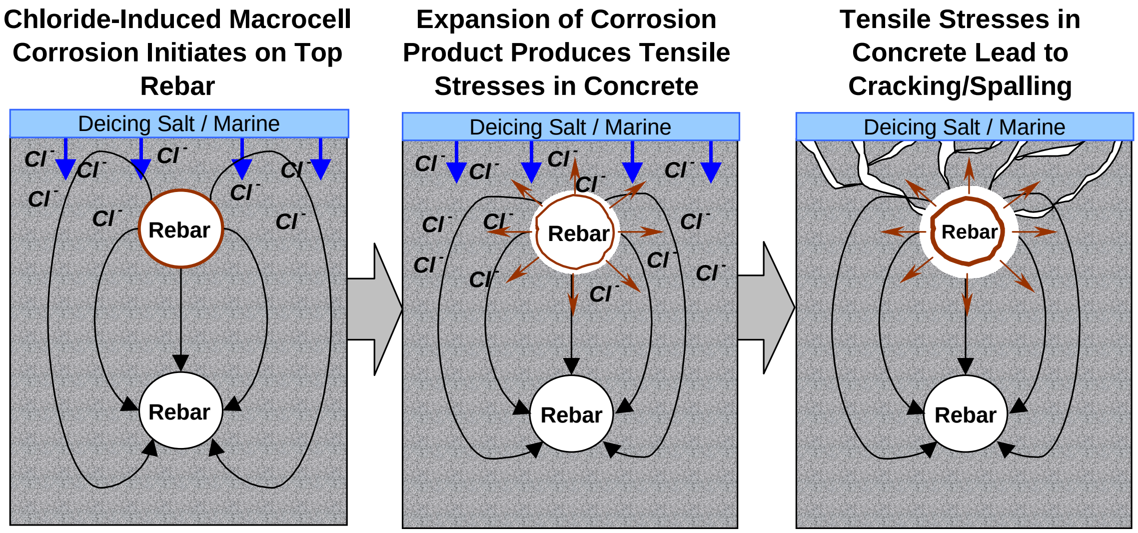

Figure 1. Schematic of corrosion damage to rebar.

Figure 2. Example of deteriorating bridge element.

In addition to the chloride ions necessary to disrupt the passive layer created by the high-pH concrete environment, oxygen is required for accelerated corrosion. Chemical, physical, and mechanical properties of concrete can have a significant effect on concrete deterioration by controlling: (1) the chloride and oxygen permeation in the concrete, (2) the sensitivity of the passive layer to chloride attack, (3) the rate of corrosion reactions at the steel surface following corrosion initiation, and (4) the rate of cracking and spalling of the concrete when exposed to the expansion forces of the corrosion products. Thompson et al. examined in detail the effect of concrete properties on the corrosion and concrete deterioration processes of bridge structures.(3-4) It was shown that concrete mix design has a significant effect on the corrosion of rebar.

The uneven distribution of chloride ions in the concrete and at the steel surface (high chloride concentration at the outside concrete surface and decreasing at distances into the concrete) also greatly affects corrosion. For example, the greater chloride concentration around the top layer of the reinforcing steel makes it anodic (accelerates corrosion) to the bottom (inside) reinforcement, which becomes the cathode (no or decreased corrosion). This type of accelerated corrosion due to chloride concentration difference is termed “macro-cell” corrosion.

Corrosion of steel in concrete is a very complex phenomenon. Although significant research on modeling the corrosion processes of steel in concrete has been performed, accurate life prediction for concrete structures is difficult.

Non-marine, corrosion-related reinforced-concrete bridge failures became a growing problem beginning in the 1960s in the “snowbelt” regions following the increased usage of deicing salts. In the worst cases, bridges began to require maintenance after a service life of as little as 5 to 10 years, with the average maintenance interval being around 15 years. In the 1970s and the 1980s, the quality of the concrete used for bridge construction generally improved. This, coupled with increased cover thickness and the use of epoxy-coated rebar, has lead to increased service lives. New bridge structures built and maintained with the use of the contemporary corrosion control methods (high-performance concrete, greater cover thickness, corrosion-resistant rebar, corrosion-inhibiting admixtures, overlays, sealants, and improved cathodic protection practices) are expected to have service lives between 75 and 120 years. However, in designing for a long-lived bridge structure, consideration must be given to the fact that changing load and capacity requirements may render such a structure functionally obsolete before it becomes structurally deficient. Therefore, emphasis should be placed on forecasting traffic loads and patterns and on designs to accommodate anticipated changes in the traffic volume.

Prestressed Concrete

Prestressed concrete bridges also face major corrosion-related issues. However, because most of these bridges are relatively recent and because their numbers are relatively low (18 percent of the bridges), the total economic impact of corrosion is not as great as that for conventional reinforced-concrete bridges (40 percent of the bridges). However, on an individual basis, failure of a prestressed concrete component may have a significant impact on the structural integrity of a bridge. Because prestressed concrete members rely on the tensile strength of the tendons to sustain load, the loss of even a few tendons may lead to the catastrophic failure of a bridge component.

The first prestressed concrete bridge in the United States was opened to traffic in 1950 and the majority of the 107,700 prestressed concrete bridges were built after 1960.(5) Corrosion problems associated with prestressed concrete structures have been recognized beginning in the 1990s. The FHWA report *Corrosion Protection: Concrete Bridges* summarized corrosion of prestressed concrete bridges, in addition to conventionally reinforced-concrete bridges.(6)

In the fall of 1992, the U.K. Ministry of Transportation imposed a temporary ban on the commissioning of grouted, bonded post-tensioned bridges. This ban resulted from the collapse of two footbridges in 1960, the collapse of a single-span, segmental post-tensioned bridge in Wales in 1985, and an examination of nine other segmental bridges. The United Kingdom is not the only place with the problem of voids in the grouted ducts resulting in insufficient coverage over prestressing steel strands. For example, in 1992, the post-tensioned Melle Bridge across the Scheldt River in Belgium, which was constructed in 1956, collapsed. This failure was traced to corrosion of the post-tensioned strands even though the bridge had been inspected, load tested, and rated satisfactorily. The U.K. moratorium was lifted in 1996 with the publication of the advisory report *Post-Tensioned Concrete Bridges: Planning Organization and Methods for Carrying Out Special Inspections* by the construction industry and owners of this type of bridge.

The underlying difficulty is that there are no reliable, cost-effective, and rapid nondestructive methods for providing assurance to the owners that the built structures have met construction specifications. One of the major inspection concerns is to determine whether the ducts in the post-tensioned bridge members have been completely filled with the grouts and whether there is uniform coverage over the prestressing steel. In many instances, it has been determined that there exists large void areas in the grouted ducts (i.e., partially filled ducts). In addition, it is very difficult to assess the condition of anchorage areas.

Voids in the grout can be a result of: (1) poor grouting application not completely filling the duct or (2) bleeding of the grout during curing in which a volume of the duct is filled with bleed water (typically at high points in the duct). In the case of bleed water formation, it has been proposed that the bleed water is sufficiently corrosive to initiate corrosion of the exposed strands. In other cases, chloride-bearing water can find its way through the anchorage area into the ducts and eventually initiate corrosion of the prestressing steel inside the duct. Water can also access the ducts through faulty and leaky joints. Over time, chloride ions can penetrate through the concrete cover and accelerate the corrosion of the prestressing steel in the ducts (either after the corrosion of the metallic ducts or through defective plastic ducts). In addition to causing pitting of the prestressed strands, corrosion reactions lead to the evolution of atomic hydrogen, which is subsequently absorbed into the steel, leading to hydrogen embrittlement of the steel strands and causing the strands to fail at lower than designed bridge loads. Since prestressed concrete bridge members rely on the tensile strength of the strands to resist loads, loss of even a few tendons can prove to be catastrophic. In addition, due to the high stresses to which the strands are subjected, corrosion can be accelerated.

Corrosion protection methods adopted at present in the construction of prestressed concrete members included: (1) the application of highly impermeable concrete by using silica fume or fly ash additions and controlled curing of the concrete at the fabrication site, and (2) the use of corrosion-inhibiting admixtures. The use of epoxy-coated strands is not yet common in prestressed concrete members and additional research is needed.

Steel Bridges

Atmospheric corrosion of exposed steel is inevitable and can be seen everywhere, from steel buildings to automobiles to steel bridges. Painting of steel structures is the universal solution to corrosion due to exposure to environmental conditions. Paints themselves deteriorate due to moisture uptake, ultraviolet exposure, wear or mechanical damage, and exposure to chemicals. For example, the performance of the same coatings will vary significantly depending on exposure to industrial, urban, rural, or marine environments. Once a coating is compromised, corrosion can initiate and, often, is accelerated beneath a deteriorated coating more than in the absence of the coating. Therefore, selection of the proper coating for the right application is critical for a long service life. In addition, proper and timely maintenance of the structure can extend the overall life of the coating significantly.

There are approximately 200,000 steel bridge structures in the United States (see figure 3). Until the mid- to late-1970s, virtually all steel bridges were protected from corrosion by three to five thin coats of lead- and chromate-containing alkyd paints applied directly over mill scale on the formed steel. Maintenance painting for prevention of corrosion of the majority of these bridges has been rare and has been limited to larger bridge structures and toll bridges. Since the majority of the steel bridges in the interstate system were constructed between 1950 and 1980, most of these structures were originally painted in this manner. Therefore, a large percentage of the interstate steel bridges are protected from corrosion by an old coating system that is now beyond its useful life. Moreover, this coating system is considered to be hazardous to humans and the environment.

Figure 3. Steel bridge structure.

The current maintenance burden for corrosion protection of steel bridges presents a major challenge to bridge owners. The past decade has had significant increases in the costs associated with steel bridge maintenance painting. As recently as 10 years ago, bridge painting was a relatively simple operation with little emphasis on regulatory compliance, quality, or life-cycle performance of materials. Bridges were either painted over repeatedly in a low-tech, low-cost attempt to combat corrosion and deteriorating aesthetics, or they were cleaned by open abrasive blasting and were repainted. These approaches could be accomplished for \$11 to \$22 per m^2 of steel or less.(7) The increasing age of steel bridges has lead to the need for increased maintenance, resulting in higher maintenance costs. These costs have increased almost tenfold, largely facilitated by environmental regulations covering all aspects of bridge painting from construction to rehabilitation to routine maintenance. Over the past few years, environmental regulations have become the single most influential force in the bridge painting industry. Specifically, regulations having a significant impact are those regarding: (1) the volatile organic compound (VOC) content of protective coatings and (2) environmental and worker health and safety associated with the removal of lead-containing paint. Table 1 lists the most pertinent regulations and summarizes their effect on bridge painting operations.

Table 1. Effect of regulations on coating operations.

IMPACTING REGULATION	EFFECT ON COATING OPERATIONS
OSHA; CFR 29 1926.62, Lead in Construction, 1993	Establishes guidelines for protection and monitoring of workers removing lead paint from bridges. Requires lead training and monitoring for workers.
EPA; Resource Conservation and Recovery Act (RCRA), 1976	Regulates the handling, storage, and disposal of lead- (and other heavy metal) containing waste. Can increase the cost of disposal of waste from bridge paint removal by a factor of 10.
EPA; Title X, Residential Lead-Based Paint Reduction Act of 1992	Mandates training and supervision requirements for workers associated with lead-containing paint removal.
EPA; Comprehensive Environmental Response Compensation and Liability Act (CERCLA 1980 and Superfund 1986)	Assigns ownership of and responsibility for hazardous waste to the generator “into perpetuity.”
EPA; Clean Water Act, 1972	Regulates discharge of materials into waterways.
EPA; Clean Air Act Amendments, 1970	Mandates restrictions on allowable VOC content of paints and coatings. Regulates discharge of dust into air from bridge painting operations.

As maintenance budgets continue to shrink or remain static and the cost of bridge maintenance continues to rise, the focus for expenditures must shift to long-term effectiveness of dollars spent. This is a significant change in philosophy for a majority of the bridge painting industry. To date, bridge maintenance painting has been accomplished based on incremental budgets rather than life-cycle cost considerations.

Cable and Suspension Bridges

Although cable bridges comprise only a small percentage of the nation's bridges, they are typically highly visible. There are approximately 150 cable bridges, several of which are old (100 to 130 years). Stahl and Gagnon have reviewed cable bridge construction and corrosion control practices.(8) Concern over a few well-publicized cable failures and condition reports for other bridges has focused on the importance of thorough inspections and scheduled maintenance. Corrosion problems associated with these structures tend to be specific to the individual design, making general rules-of-thumb difficult to utilize. The corrosion problems are highly dependent on specific structural configurations, maintenance and operational practices, and local environmental conditions.

Corrosion concerns on cable-supported structures and corrosion control practices have been present from the early designs. For example, galvanized (zinc) coating of the wires was first used on the Brooklyn Bridge, which was completed in 1883. At that time, it was already standard practice to coat the wire with linseed oil, circumferentially wrap the assembled cable with soft galvanized wire laid into red lead paste, and to paint the finished cable. These corrosion control practices have been refined since then, but the basic principle of keeping the moist environment away from the steel surface remains unchanged.

Some of the oldest and best known bridges in the United States, such as the Golden Gate and Brooklyn bridges, are suspension bridges (see figures 4 and 5, respectively). Significant costs are incurred in maintaining these bridges, but because of historic reasons or strategic location, these bridges cannot be replaced or taken out of service for any length of time. Of specific concern with these bridges is the condition of the strands. The strands are susceptible to corrosion, stress-corrosion cracking, and hydrogen embrittlement, which can lead to premature failure of the strands.

Figure 4. Golden Gate Bridge (suspension).

Figure 5. Brooklyn Bridge (suspension).

A more recent design of cable bridges is the so-called cable-stayed bridge (see figure 6). Presently, there are only 30 cable-stayed bridges in the United States. However, because the integrity of the cables is critical to the structural integrity of the entire bridge structure, and inspection of the cables is very difficult, the cable-stayed bridges are built with special considerations for corrosion protection.(9)

Figure 6. SR509 bridge in Tacoma, WA (cable-stayed).

Although different levels of protection are used depending on the design and the environmental conditions, the following represents an example of current practice. The individual wires comprising the strand are epoxy coated or galvanized. In the monostrand construction, the interstices of the individual strands are filled with a corrosion-inhibiting grease and then each strand is sheathed with a high-density polyethylene (HDPE) sleeve. The stays consisting of multiple strands are encased in an HDPE tube and then injected with cement grout.

Despite precautions, failures of cables have occurred, but no catastrophic failures have been reported. To maintain this record, improved inspection procedures and maintenance programs need to be developed. Nondestructive techniques such as magnetic flux leakage (MFL) have been developed for the identification of corrosion in the free length of the cable, but it cannot be used in the anchorage areas, which are of significant concern. Refinement of the MFL method and development of new technologies need to be continued in order to provide more accurate and reliable methods for identifying cable problems at the earliest possible time.

AREAS OF MAJOR CORROSION IMPACT

The condition of the bridge inventory in the United States can be characterized by the significant portion of bridges that are listed as "structurally deficient" (bridge that can no longer sustain the loads for which it was designed). The nation's structurally deficient bridges as of the end of fiscal year 1999 and the preceding 7-year period are summarized in table 2.(5,10) The data include all materials of construction, including concrete, steel, wood, aluminum, and other material. The trend shows that, as older bridges are being replaced or rehabilitated, there is a decrease in both the number (118,757 to 88,184) and the percentage (20.7 to 15.0) of structurally deficient bridges. During the same period, the number of bridges in the inventory rose from 572,633 to 585,947.

Table 2. National Bridge Inventory data – structurally deficient bridges.(5,10)

	1992	**1993**	**1994**	**1995**	**1996**	**1997**	**1998**	**1999**
Bridges in Inventory	572,633	574,191	576,472	577,919	582,043	583,207	583,414	585,947
Number Deficient	118,757	111,543	107,512	103,686	101,544	98,521	93,119	88,184
Percent Deficient	20.7	19.4	18.6	17.9	17.4	16.9	16.0	15.0

The 1998 data are presented in more detail in table 3. This table focuses on those bridges constructed of materials that are subject to corrosion (conventional reinforced concrete, prestressed concrete, and steel). Of these three bridge types, steel has the highest percentage of structurally deficient structures, followed by conventionally reinforced concrete and prestressed concrete. Listings for each state (FHWA bridge inventory(5)) suggest that the states with colder and damper weather have a high percentage of reinforced-concrete deficient bridges. These states include New York, Alaska, Rhode Island, Pennsylvania, and Vermont. The most structurally deficient bridges for a single state are in New York, which also has a larger total bridge area for conventional reinforced-concrete and steel bridges than any other state.

Table 3. Structurally deficient bridges based on material of construction in 1998.(11)

	CONVENTIONAL REINFORCED CONCRETE	**PRESTRESSED CONCRETE**	**STEEL**	**OTHER**	**TOTAL**
Bridges in Inventory	235,151	107,666	200,202	40,395	583,414
Structurally Deficient	21,164	3,230	54,054	14,671	93,119
Percent Deficient	9	3	27	36	16

The estimated service life expectancy for each of the above bridge types is shown in table 4. Many of the steel and reinforced concrete bridges have reached or are approaching the end of their design service life, making bridge maintenance, rehabilitation, and replacement decisions a priority.

Table 4. Estimated service life for bridges with different materials of construction.(1)

MATERIAL OF CONSTRUCTION	**AVERAGE ESTIMATE (Years)**
Conventional Reinforced Concrete	72
Prestressed Concrete	73
Steel	58

The impact of corrosion on the highway bridge infrastructure has been estimated by several different sources using different approaches. Reconstruction of the nation's bridges was estimated to cost between $20 billion and $200 billion dollars.(6,11) An FHWA report on corrosion protection of concrete bridges estimates that the total cost to eliminate the backlog of deficient bridges (both structural and functional) is between $78 billion and $112 billion, depending on the time required to carry out the task.(12) In addition, the average annual cost through the year 2011 for just maintaining the overall bridge conditions (maintaining the total number and distribution of deficient bridges) is estimated to be $5.2 billion. While corrosion is not the sole cause of bridge deficiency, it is a major contributor to the costs given above.

An additional estimate of the total corrosion costs related to the replacement of structurally deficient bridges is possible using the National Bridge Inventory data for December 1999.(5) Unit costs for bridge replacement, calculated by taking the mean for all states, are given in table 5. The overall area of structurally deficient bridges (conventional reinforced concrete, prestressed concrete, and steel) is 34.2 million m^2 (368.5 million ft^2). Assuming that these structural deficiencies are largely attributable to corrosion (obsolete bridges were not included), and using the average unit cost data [$858 per m^2 ($80 per ft^2)], the total cost of replacing the structurally deficient bridges is estimated to be $29.3 billion (34.2 million m^2 x $858 per m^2).

Table 5. Highway Bridge Replacement and Rehabilitation Program unit costs.(5)

	1995	**1996**	**1997**	**1998**	**1999**
Unit costs,* $/m^2	768	771	836	855	858

*Average between federal aid and non-federal aid projects.

The overall magnitude of the corrosion-induced deterioration of concrete bridges has increased considerably in the last three decades due primarily to the increased use of deicing salts. Although the cost of bridge deck maintenance is high, the use of deicing salts is not likely to be discontinued. In fact, it has been reported that its use has actually increased in the first half of the 1990s after leveling off in the 1980s. Although some alternative means of deicing have been studied (namely, calcium magnesium and potassium magnesium acetates), the high price of the chemicals and lower efficiencies for melting ice prevents their widespread use.(13) Since the discontinued use of deicing salts is unlikely, understanding and utilizing other methods of corrosion control is important.

CORROSION CONTROL METHODS

Methods utilized for corrosion control on bridges are specific to the type of bridge construction and whether its intended use is for new construction or maintenance/rehabilitation of existing structures. In this section of the report, corrosion control practices are reviewed for the three types of bridge structures focused on in this sector (conventional reinforced concrete, prestressed concrete, and steel). For the purposes of discussion, conventional reinforced concrete and prestressed concrete corrosion control methods are combined. Although prestressed concrete bridges have very special concerns (e.g., anchorage in both post-tensioned and pretensioned structures and ducts for post-tensioned structures), the general corrosion control methods are applicable to both prestressed and conventional reinforced bridges.

Reinforced-Concrete Bridges

Conventional reinforced-concrete bridges refer to those with superstructure constructed with conventional reinforced concrete. Often, prestressed concrete and steel bridges will have conventional reinforced-concrete decks or substructures. Therefore, corrosion control practices for conventional reinforced concrete are applicable to components of many other bridge structures. Therefore, a significant amount of detail is provided for conventional reinforced-concrete corrosion control practices.

New Construction

Corrosion protection can be incorporated into new bridge structures by proper design and construction practices, including the use of high-performance concrete (e.g., silica fume additions), low-slump concrete, and an increase in concrete cover thickness. Each of these attempt to impede migration of chlorides and oxygen (or other corrosive agents) through the concrete to the steel rebar surface. However, eventually, these corrosive agents will penetrate through the concrete cover and cracks, making other corrosion control practices necessary. A widely used method of corrosion prevention is the use of coated carbon steel rebar and, to some degree, corrosion-resistant alloy/clad rebars. The typical organic rebar coating is fusion-bonded epoxy, while the metallic rebar coating is galvanizing (very limited use in bridge structures). Rebar cladding with a corrosion-resistant alloy (e.g., stainless steel) is relatively new. Solid rebars constructed of stainless steel alloys have been used on a limited basis. In addition, non-metallic composite materials have been used. Another corrosion control practice available to new construction is the addition of corrosion-inhibiting admixtures to the concrete.

Epoxy-Coated Rebars

A Technical Note prepared by the FHWA and summarized here reviews the use of epoxy-coated rebar in bridge decks.(14) Epoxy coatings (often referred to as powders or fusion-bonded coatings) are 100 percent solid, dry powders. These dry epoxy powders are electrostatically sprayed over cleaned, preheated rebar to provide a tough impermeable coating. The coatings achieve their toughness and adhesion to the substrate as a result of a chemical reaction initiated by heat. Since these epoxy powders are thermosetting materials, their physical properties, performance, and appearance do not change readily with changes in temperature. The epoxy coating becomes a physical barrier between aggressive chloride ions (permeating the concrete cover) and the steel rebar.

For many years, bridge deck deterioration, stemming from corrosion of reinforcing bars, has been the number one problem for bridges. Prior to 1970, it was thought that portland cement itself provided sufficient protection to the reinforcing steel against corrosion. In the early 1970s, it became evident that corrosion of the reinforcing steel was related to the increasing application of deicing salts. Unfortunately, this was not learned until after thousands of bridge decks containing black reinforcing steel showed signs of spalling about 7 to 10 years after construction. It was also observed that substructure members were also deteriorating because of the leakage of the deicing salts through joints or exposure to seawater. Although the deterioration of substructure components is less obvious than the deterioration of bridge decks, it is much more serious and costly to repair or rehabilitate substructures.

Epoxy-coated rebar was introduced in the mid-1970s as a means of extending the useful life of reinforced-concrete bridge components by minimizing concrete deterioration caused by corrosion of the reinforcing steel. The epoxy coatings are intended to prevent moisture and chlorides from reaching the surface of the reinforcing steel and reacting with the steel. Since the late 1970s, the highway industry has widely used epoxy coatings as the preferred protective system for bridge decks due to its excellent performance in resisting corrosion and significantly delaying subsequent deterioration of the concrete. As for all coating systems, the coating will degrade over time and corrosion of the rebar will proceed in the presence of sufficient chlorides in the concrete.

When used in substructures and exposed to a severely corrosive marine environment, the epoxy-coated rebars did not perform as well as in bridge deck applications. Such was the case with a number of concrete bridges located in the Florida Keys. Significant premature corrosion of the epoxy-coated rebar was observed in substructure members of these bridges after only 6 to 9 years. These members are subjected to salt spray in the splash zone where the usual wetting/drying cycles, and high water and air temperatures produce a very corrosive environment. The deterioration observed on the Florida Keys bridges and on some other bridges located in harsh environments raised questions concerning epoxy-coated rebar as a durable corrosion protection system.

After an evaluation of the performance of epoxy-coated rebar decks by several state departments of transportation agencies, the overall condition of the bridge decks was considered to be good. Deck cracking did not appear to be corrosion-related. Very few of the decks had any delamination or spalling associated with the epoxy-coated rebar. Any delamination or spalling associated with corrosion of epoxy-coated rebar was small and generally isolated. The epoxy-coated rebar did not appear to perform as well in cracked concrete as it did in uncracked concrete. Corrosion was observed on epoxy-coated rebar segments extracted from locations having heavy cracking, shallow concrete cover, high concrete permeability, and high chloride concentrations. Reduced adhesion and softening of the coating also occurred as a result of prolonged exposure to a moist environment. The number of defects in the epoxy coating had a strong influence on the adhesion and performance of epoxy-coated rebar. There was no evidence of significant premature concrete deterioration that could be attributed to corrosion of the epoxy-coated rebar. It was concluded that the use of sufficient good-quality concrete cover, adequate inspection, finishing, and curing of the concrete, and the use of epoxy-coated rebar has provided effective corrosion protection for bridge decks since 1975.

At present, epoxy-coated rebar is the most common corrosion protection system and is used by 48 state highway agencies. To date, there are approximately 20,000 bridge decks using fusion-bonded epoxy-coated rebar as the preferred protection system. This represents roughly 95 percent of new deck construction since the early 1980s.

The data from the Concrete Reinforcing Steel Institute (CRSI) shows that more than 3.6 billion kg (4 million tons) of epoxy-coated rebar (approximately 158 million m^2 of reinforced concrete) were used worldwide as of 1998, with 79 percent installed in the last 10 years.(15) A significant portion of this epoxy-coated rebar was used in bridge decks. Over the past 20 years, the formulation of the epoxy has been modified to achieve increased performance of the epoxy coating.(15)

To estimate the cost of different construction options, the cost of the baseline case for black steel rebar is first calculated. The following cost analysis is provided to compare epoxy-coated rebar to black steel rebar. The amount of rebar contained in a bridge deck depends on the design. A typical “traditional” bridge deck (e.g., with two mats – each mat contains one longitudinal and one transverse rebar at 15-cm (6-in) centers – one mat of No. 5 rebar and one mat of No. 4 rebar) contains 33.2 kg of steel per square meter of deck (6.8 lb per ft^2). Other designs (e.g., two mats – each mat contains one longitudinal and one transverse rebar at 20-cm (8-in) centers – both mats of No. 4 rebar) contain 19.6 kg of steel per square meter of deck (4 lb per ft^2). An average of these two scenarios gives 26.4 kg of steel per square meter of deck (5.4 lb per ft^2). The cost of black steel rebar is estimated at $0.44 per kg ($0.20 per lb).(16) Using 26.4 kg per m^2 (5.4 lb per ft^2) as the weight of rebar in a square meter of deck, the cost of rebar in a black steel deck is $11.60 per m^2 ($1.08 per ft^2). The cost of a deck installed using black steel rebar is assumed to be $484 per m^2 ($45 per ft^2).(6,16) It is estimated that black steel rebar provides an expected life of 10 years prior to required maintenance resulting from concrete deterioration due to corrosion of the rebar.(14)

Typically, the cost of epoxy-coated rebar adds $0.22 per kg ($0.10 per lb) to the cost of rebar, which is an increase in the cost of rebar of 50 percent.(16-17) This gives a cost of rebar for an epoxy-coated rebar deck of $17.40 per m^2 ($1.62 per ft^2) of deck or an increase in the cost of epoxy-coated rebar as compared to black steel of $5.80 per m^2 ($0.54 per ft^2) of deck. However, the rebar is a relatively small portion of the total deck construction costs. The added cost of epoxy-coated rebar depends on whether both mats of rebar are coated (many bridges have been constructed with only the top mat of rebar epoxy coated, although current practice typically uses both mats epoxy coated) and on the overall construction costs. Assuming the cost of new construction for a bridge deck is \$484 per m^2 (\$45 per ft^2) and both mats are epoxy coated, the increase to the total deck cost is 1.2 percent (\$5.80 / \$484 x 100). This value is consistent with other references discussed below. It is estimated that epoxy-coated rebar provides an expected bridge deck life of 20 to 40 years.(14,18) The service life depends, in part, on whether a single top mat of epoxy-coated rebar is used in conjunction with a bottom mat of black steel rebar versus both mats constructed of epoxy-coated rebar. With the current practice of coating both rebar mats and current coating formulations, a 40-year life is typically assumed. The costs for using only a single mat of epoxy-coated rebar would be estimated at 50 percent of that for both mats coated.

The Concrete Reinforcing Steel Institute (CRSI) estimates that the increase in the total cost of the structure due to coating both mats of rebar is typically between 1 and 3 percent.(17) An FHWA study provided data for three Illinois bridge decks (1994 construction data) and showed that the increase in the cost of the deck due to using epoxy-coated rebar on both mats was between 0.5 and 2.2 percent, with an average increase of 1.4 percent.(16)

The New York State Department of Transportation (DOT)(19) has been using epoxy-coated rebars in the top mat reinforcements for the past 20 years. A summary of the data is presented in table 6. For deck replacement, the increase in the cost of coating the top mat was approximately 0.1 percent and, for rehabilitation, the cost increase was approximately 0.25 percent. The New York State DOT estimates that this small increase in costs for epoxy-coated rebars gives at least a 10-year life extension for the bridge structures. One factor that explains the lower percent increase in the structure cost due to using epoxy-coated rebar is that, in New York, only the top mat of rebar was coated. In addition, the bridge construction costs are higher in New York, making the average percent increase due to using epoxy-coated rebar lower.

Table 6. New York State DOT data on epoxy-coated rebar costs for bridge deck replacement and rehabilitation.(19)

		1/1/90-1/1/97	**1/1/97-1/1/98**	**1/1/98-1/1/99**
Replace	Average area of deck, m^2	580	495	393
	Average cost per project, \$ (in millions)	0.93	1.11	1.04
	Cost increase due to use of epoxy-coated rebars, %	0.11	0.08	0.06
Rehabilitate	Average area of deck, m^2	3,645	573	1107
	Average cost per project, \$ (in millions)	1.66	0.32	0.89
	Cost increase due to use of epoxy-coated rebars, %	0.37	0.3	0.21

Metal-Coated/Clad Rebars and Solid Corrosion-Resistant Alloy Rebars

To provide a more corrosion-resistant rebar, a number of metallic coatings, metallic claddings, and rebar alloys have been tested. The most promising are galvanized (zinc-coated) rebars, stainless steel-clad rebars, and solid stainless steel rebars.(6) Titanium has also been discussed as a clad or solid rebar material, but its cost is significantly greater than that of stainless steel, and the increased corrosion resistance (relative to stainless steel) may not be required.

Galvanized Rebars

Hot-dipped galvanized coatings for reinforcing steel in concrete have been used since the 1940s. ASTM A767, "Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement," specifies the requirements for the galvanized coating. A Class I coating has a zinc coating weight of approximately 1,070 g per m^2 (3.5 oz per ft^2) and a Class II zinc coating has a coating weight of approximately 610 g per m^2 (2.0 oz per ft^2).

The effectiveness of galvanized rebars in extending the life of reinforced-concrete structures is questionable. In other applications, galvanized steel has been shown to extend the life of structures exposed to atmospheric conditions and low-chloride underground environments, but not high-chloride environments. An FHWA study by McDonald et al. reviewed the performance of galvanized rebar and is summarized here.(16) Several studies conducted in the 1980s and 1990s provided conflicting evaluations for the performance of galvanized steel in concrete. In general, the findings are in agreement with those for other exposure conditions, i.e., (1) zinc corrodes as fast (or faster) than steel in high-chloride environments and (2) zinc corrosion can be accelerated by macro-cell action when a large cathodic area is present. Accelerated macro-cell corrosion can occur when a galvanized upper mat of reinforcement is connected to a bare steel lower mat (in which the concrete surrounding the lower rebar mat has a lower chloride concentration than the concrete of the upper mat). Therefore, both mats of reinforcement should be galvanized. The general consensus is that galvanizing extends the life of the concrete structure due to a higher threshold for chloride-induced corrosion of the zinc-galvanized coating as compared to black steel.

Although galvanized rebar may provide a benefit in certain chloride-containing environments, the majority of the problems are associated with deicing salts and marine exposures where the chloride content of the concrete continuously increases to a point where any benefit of galvanization becomes marginal.

Stainless Steel Rebars

Research in stainless steel rebars has taken two directions, clad stainless steel over a carbon steel substrate and solid stainless steel rebar. If a stainless steel alloy is selected that possesses sufficient corrosion resistance for the service conditions, the primary concerns of cladding are: (1) adherence to rebar substrate, (2) defects formed after bending, (3) uniform cladding thickness [a typical cladding for stainless steel is 0.5 mm (0.020-in) thick], and (4) metallurgical changes due to the cladding process that may affect the corrosion resistance. It should be realized that the chloride threshold for pitting in a non-aqueous (non-homogeneous) environment such as concrete can be significantly less than for the same aqueous environment. Therefore, any research must utilize realistic concrete environments. For instance, the use of stainless steel piping in underground service, generally, has been discontinued due to pitting and subsequent perforation of the pipe in the non-homogeneous unsaturated soil environment with relatively low chloride contents. Pitting in conventional reinforced-concrete bridge components may not be as significant a concern as decreasing the average corrosion rate (overall metal weight loss).

Several studies that examined the performance of solid stainless steel rebars were summarized by McDonald et al.(16) These studies showed that the austenitic stainless steel (Types 304 and 316) performed well, while the ferritic stainless steels (Types 405 and 430) developed pitting. In all cases, the stainless steel performance was greatly superior to carbon steel; with the stainless steel rebar generally performing with no (or negligible) corrosion. In a study summarized by Virmani and Clemena, Type 316 stainless steel-clad rebar greatly extended the estimated time to cracking of the concrete beyond that of conventional steel rebar (to 50 years), but not as much as solid Types 304 and 316 stainless steel (100 years).(6) In addition, McDonald et al. reported on two highway structures constructed with stainless steel rebar. Following a 10-year exposure, no corrosion was observed for solid Type 304 stainless steel rebar in a bridge deck in Michigan and for Type 304 stainless steel-clad rebar in a bridge deck in New Jersey.(16) However, at that time, the chloride levels in both bridge decks were below or at the threshold chloride level for corrosion initiation in black steel rebars.

The cost of solid stainless steel rebars is estimated to be $3.85 per kg ($1.75 per lb). Assuming similar weights of solid stainless steel rebar as used above for black steel rebar, the cost of solid stainless steel rebar is estimated at

\$101.64 per m^2 (9.44 per ft^2) (\$3.85 per kg x 26.4 kg per m^2 = \$101.64 per m^2). This is an increase in the cost of rebar of \$90 per m^2 (\$8.40 per ft^2) when comparing the cost of solid stainless steel to black steel (\$11.60 per m^2). Assuming that the cost of new construction for a black steel rebar bridge deck is \$484 per m^2 (\$45 per ft^2), the cost increase to construct the deck using solid stainless steel rebar is 18.6 percent (\$90 / \$484 x 100). This value is consistent with other references discussed below. It is estimated that the use of solid stainless steel rebar provides an expected life of 75 to 120 years.(6,16)

McDonald et al. estimated the costs, at three installation sites, of the use of rebar made from solid stainless steel.(20) The authors estimated that, compared to the cost of black steel rebar, the overall construction cost would have increased by 6 percent to 16 percent if solid stainless steel rebars were used.

Stainless steel rebars have been reported to be used in several projects in the United States, including Michigan and Oregon.(21) The Oregon DOT estimates that the cost of stainless steel rebar (Type 316LN, Nitronic 50) is approximately \$4 per kg (installed), with an overall cost increase of 10 to 15 percent when used in the deck and superstructure, and another 5 percent if used in the substructure. Although the cost of \$4 per kg is greater than that used in the analysis above, the percent increase in the cost of the structure is similar. The expected service life of the structure using stainless steel rebar was stated to be 120 years.

Fluctuation in the cost of raw materials used in the production of stainless steel has a significant effect on the economic viability of the use of stainless steel rebar in concrete decks. The rebar cost also is dependent on the grade of stainless steel used.

One means of minimizing the cost of the stainless steel rebar is to utilize stainless steel-clad rebar. It has been estimated that the cost of stainless steel-clad rebar is \$1.54 per kg (\$0.70 per lb), which gives a cost of \$40.66 per m^2 of deck (\$3.78 per ft^2) or an increase of \$29 per m^2 (\$2.70 per ft^2) over that of a black steel deck. Assuming the cost of new construction for a black steel rebar bridge deck is \$484 per m^2 (\$45 per ft^2), the cost increase to construct the deck using stainless steel-clad rebar is 6 percent (\$29 / \$484 x 100). It is estimated that the use of stainless steel-clad rebar provides an expected life of 50 years.(6)

The cost of stainless steel cladding can vary depending on the raw material market prices, just like solid stainless steel, but it is also dependent on the cladding manufacturer, the cladding thickness, and the chosen grade of stainless steel. The purity of the stainless steel is a consideration as well, since many cladding operations use recycled material to reduce costs. However, with proper quality control, stainless steel-clad rebar promises to be an effective means of control for bridge deterioration due to corrosion of the reinforcing steel.

Alternative Means of Protection

In addition to the use of coated or alloy rebar, other approaches to mitigate corrosion of the reinforcing steel in bridge structures include high-performance concrete, corrosion-inhibiting admixtures, or a combinations of these.

High-Performance Concrete

High-performance concretes were developed as a means of impeding ingress of chlorides to the rebar (by reducing concrete permeability). This is accomplished by using lower water-to-cement ratio concrete and adding mineral admixtures to the concrete mix. The most common admixtures are silica fume and fly ash (pozzolanic materials). Low water-to-cement ratios are achieved using high-range water reducers.

Although low chloride permeability is one of the main features of mineral admixtures, they impart other properties to the concrete (depending on the admixture selected), such as: (1) corrosion resistance (higher chloride threshold for corrosion and low corrosion rate following initiation), (2) greater cumulative corrosion prior to cracking, and (3) higher resistivity to minimize macro-cell corrosion. An FHWA study by Thompson and Lankard reviewed the effect on the corrosion of steel in concrete of several variables, including cement types, mineral admixtures, water-to-cement ratio, and aggregate type.(3-4) This study showed that silica fume was by far the most effective mineral admixture in mitigating corrosion of steel rebar. It also suggested that careful selection of the concrete mix components could greatly extend the life of a concrete bridge member.

The cost of a high-performance concrete depends on the admixtures used. Berke et al. estimated the addition of silica fume would increase the bridge cost by \$4.30 per m^2 (\$0.40 per ft^2).(22) Assuming that the cost of new construction for a black steel rebar bridge deck is \$484 per m^2 (\$45 per ft^2), the cost increase associated with the use of a high-performance concrete containing silica fume is 0.9 percent (\$4.30 / \$484 x 100). It is estimated that the use of silica fume admixture provides an increase in expected life of 10 years beyond that provided by black steel rebar in conventional concrete.(23)

Corrosion-Inhibiting Admixtures

In the past decade, the use of corrosion-inhibiting concrete admixtures has emerged as a promising method for delaying the onset of corrosion of prestressing and conventional reinforcing steel.(24) Inhibitors are usually employed with permeability-reducing pozzolanic additives such as fly ash or silica fume. As such, the concrete has low permeability and the corrosion inhibitor essentially increases the chloride concentration required for corrosion initiation. Inhibitor action may also reduce the rate of corrosion after initiation, resulting in less corrosion-induced concrete deterioration.

Inhibitors are compounds that are able to reduce corrosion rates when present at relatively small concentrations at or near the steel surface. Corrosion inhibitors are generally classified as organic or inorganic. Organic corrosion inhibitors generally work either by forming a protective film on the steel and/or by preventing the corrosive agents from reaching the steel. Inorganic corrosion inhibitors work by reducing either the oxidation or the reduction reactions at the steel surface.

Extensive technical literature exists on the inorganic calcium nitrite products. This product has been shown to provide passivity at relatively high chloride concentrations. Commercially available organic-based inhibitors are also available. The organic inhibitors are believed to be comprised of amides and esters. A recent National Cooperative Highway Research Program (NCHRP) project by Thompson et al. reviewed the performance of corrosion inhibitors used in concrete and performed a range of laboratory tests to assess the performance of the commercially available inhibitors.(25)

The cost of calcium nitrite (one of the most commonly used corrosion-inhibiting admixtures) with and without the addition of silica fume was discussed by Berke et al.(22) The cost of a calcium nitrite protection system was estimated to be \$5.40 per m^2 (\$0.50 per ft^2). Assuming that the cost of new construction for a black steel rebar bridge deck is \$484 per m^2 (\$45 per ft^2), the cost increase to construct the deck using calcium nitrite inhibitor is 1.1 percent (\$5.40 / \$484 x 100). It is estimated that the use of inhibitors may provide an increase in expected life of 20 to 25 years beyond that provided by black steel rebar and conventional concrete.(23)

Multiple Protection Systems

Corrosion inhibitors are increasingly used as a part of multiple corrosion protection systems in conjunction with epoxy-coated rebars and low-permeability concrete, especially for marine application. As yet, epoxy-coated seven-wire strands are not commonly used for prestressed concrete bridge members. In lieu of coated seven-wire strands, corrosion inhibitors have found their niche in the prestressed highway construction industry.(24)

Summary of Current Practice for New Construction

The following items summarize the current practice based on research, field performance, and emerging technologies.(6)

The preferred primary corrosion-protection system is fusion-bonded epoxy-coated rebars, which have been used in approximately 20,000 reinforced-concrete bridge decks and approximately 100,000 total structures. Epoxy-coated rebar has performed very well in alleviating the problem of corrosion-induced deterioration of concrete bridge decks. The only caution is its use in severe marine applications.

With continued updates in the American Association of State Highway and Transportation Officials (AASHTO) and American Society for Testing and Materials (ASTM) specifications for epoxy-coated rebar, this corrosion protection system will continue to improve. The specifications involve all aspects of the fabrication of epoxy-coated rebar, including the following: certification of coating plants; proper storage of coating powder at the plants; restrictions on surface imperfections on the bars; removal of dust and salt from the surface of the bars prior to coating; and better quality control of thickness, continuity, flexibility, adhesion, etc. In addition, requirements related to job-site storage and handling of the coated bars have also been established. All of these will result in improved performance of epoxy-coated rebar and more durable new concrete structures.

To provide even longer service life to the concrete decks (75 to 120 years) without any need to repair corrosion-induced concrete damage, a number of solid and clad corrosion-resistant alloy rebars are under development. Most notable are solid Type 304 or 316 stainless steel rebars and stainless steel-clad rebars, which have performed well in accelerated corrosion screening tests. Both of these two new alternative reinforcing bars have the potential to provide an excellent corrosion protection system, albeit at a higher initial cost. Although Type 316 stainless steel is a proven corrosion-resistant alloy, more research is needed for clad rebar and other alloys.

The combined use of epoxy-coated rebar and a corrosion-inhibiting admixture, such as calcium nitrite, could serve as a reliable corrosion protection system, especially for marine applications such as piles, etc. However, the long-term stability of this inhibitor is still under study. In addition, research efforts are underway to identify new inhibitors.

The combination of high temperature (38 °C) and an intermediate level of humidity or moisture (75 percent) have been identified as environmental conditions that lead to high corrosion rates for steel in concrete. Use of a low water-to-cement ratio concrete, incorporation of mineral admixture, and proper selection of cement type and aggregates contribute to production of a high-performance concrete with significant corrosion resistance.(4)

For the protection of high-strength, seven-wire strands encased in ducts, mix designs for corrosion-resistant grout for filling the ducts have been developed. In addition, an accelerated corrosion test method has been developed for evaluating new grout mixes.(26) These developments became the basis for a new grout specification recently published by the Post-Tensioning Institute (PTI) in 2001.

Prompted by the recent sudden collapse of two post-tensioned bridges in the United Kingdom and one in Belgium, the impact-echo nondestructive evaluation (NDE) technique was developed to detect voids in post-tensioned ducts. This equipment is now commercially available. A complementary magnetic-based nondestructive technique for assessing section loss in the high-strength steel strands in the ducts also has been developed. In combination, the impact-echo and the magnetic-based techniques allow inspection of post-tensioned systems, reducing the likelihood of any sudden collapse of post-tensioned bridges in the United States. Continued development of these techniques is required to increase reliability, accessibility around trumpet locations, resolution, and user confidence.

Summary of New Construction Cost Alternatives

Table 7 gives the costs of new construction alternatives for bridge structures. Also provided is the expected service life for each alternative. There are many choices for corrosion prevention, and careful life-cycle cost analysis and risk assessment are required to select the most appropriate one for any given application. In addition, the alternatives are not mutually exclusive (i.e., combinations of (1) corrosion-inhibiting admixture and silica fume or (2) epoxy-coated rebar and corrosion-inhibiting admixture have been used).

Table 7. Summary of costs and life expectancy for new construction alternative.

CORROSION CONTROL PRACTICE	**COST OF BAR**	**BAR WEIGHT PER DECK AREA**	**COST PER DECK AREA**	**INCREASE IN COMPARISON TO BASELINE**	**PERCENT INCREASE**	**ESTIMATED SERVICE LIFE**
	$/kg	kg/m^2	$/m^2	$/m^2	%	Year
Black steel (baseline)	$0.44	26.4	$11.60	NA	-	10
2-layer epoxy-coated rebar	$0.66	26.4	$17.40	$5.80	1.2%	40
2-layer solid SS rebar	$3.85	26.4	$101.64	$90.04	18.6%	75 - 120
2-layer SS-clad rebar	$1.54	26.4	$40.66	$29.00	6.0%	50
Calcium Nitrite CIA	-	-	-	$5.40	1.1%	30
Silica Fume	-	-	-	$4.30	0.9%	20

Rehabilitation

Salt-induced reinforcing steel corrosion in concrete bridges has become a considerable economic burden to many state and local transportation agencies that are generally tasked with maintenance and repair activities. Although the positive effect of adoption of corrosion protection measures can already be seen on individual structures, there are thousands of existing bridges constructed without the latest corrosion control methods. In addition, even the latest corrosion control methods are not likely to prevent all corrosion for the life of the bridge structure. Therefore, repair/rehabilitation of bridge structures and the mitigation of existing corrosion will be a major activity for bridge engineers for years to come.

There are several remedial methods available for rehabilitation of concrete structures that have deteriorated due to chloride-induced corrosion of the reinforcing steel. Because problems on concrete structures are typically found after significant deterioration has resulted in cracking and spalling of the concrete, the vast majority of the remedial methods are applied following removal and patching of the damaged concrete. The available methods are based on one of the following principles and have been summarized by Virmani and Clemena.(6)

- Provide a barrier on the surface of the concrete to prevent future ingress of chloride (overlays, membranes, etc.).
- Control the electrochemical reactions at the steel surface to mitigate the corrosion reactions by imposing the proper voltage field on the rebar (cathodic protection).
- Modify the concrete environment to make it less corrosive. One way of accomplishing this is to extract the chlorides from the concrete (electrochemical chloride removal).

Each state DOT has specifications and criteria for rehabilitation of deteriorated concrete bridge components. One example of a decision process for rehabilitation or deck replacement is as follows (Caltrans):(27)

- If spalling of the concrete is observed, the surface is checked for delamination by chain drag and core samples are taken to determine the chloride concentration.
- If the chloride concentration is greater than 1.8 kg per m^3 (3 lb per yd^3), the concrete is removed to depths where the concentration is less than 1.2 kg per m^3 (2 lb per yd^3).
- If less than 75 mm (3 in) is removed to reach an acceptable chloride level, the removed concrete is replaced by an overlay.
- If more than 75 mm (3 in) is removed, the entire deck must be replaced.

- Cathodic protection is applied only in the case of partial disbondment of the concrete and when there is no extensive spalling.

Surface Barriers

The application of an overlay of low-slump concrete, latex-modified concrete (LMC), high-density concrete, polymer concrete, or bituminous concrete with membrane on the existing concrete provides a barrier that impedes continued intrusion of chloride ions, moisture, and oxygen that are necessary for corrosion to continue. However, past experience indicates that when such barrier systems are employed without first decontaminating the existing concrete of the active corrosive agents, these corrosive agents become entrapped in the existing concrete and the effectiveness of the barrier may be neutralized. Traditionally, greater than 90 percent of the rehabilitation jobs used low water-to-cement ratio concrete or LMC overlay as the preferred method. FHWA Report FHWA-RD-98-088 indicated that state highway agencies estimate the life of these rehabilitation methods to be around 15 years.(6)

Various studies have reported performance and cost data for different overlay and patching systems. (See references 12, 28, 29, 30, and 31.) Table 8 summarizes the costs presented by Sprinkel et al.(29) These costs are based on both literature review and questionnaires sent to state DOTs. The cost data are for 1988. The numbers were discounted by 5 percent annual percentage rate (APR) to estimate 1998 costs. There is a wide range of cost and life expectancies provided, which probably corresponds to the range of application methods and detailed specifications used for these classifications.

Table 8. Cost (1998 adjusted) and life expectancy for overlay and patching options for concrete bridges.(29)

TYPE OF MAINTENANCE	**AVERAGE COST**	**RANGE OF COSTS**	**AVERAGE EXPECTED LIFE**	**RANGE OF EXPECTED LIFE**
	($/m^2)	**($/m^2)**	**(years)**	**(years)**
Portland Cement Concrete Overlay*	170	151 – 187	18.5	14 – 23
Bituminous Concrete with Membrane	58	30 – 86	10	4.5 – 15
Polymer Overlay/Sealer	98	14 – 182	10	6 – 25
Bituminous Concrete Patch	90	39 – 141	1	1 – 3
Portland Cement Concrete Patch	395	322 – 469	7	4 – 10

*Includes latex-modified concrete (LMC).

Cathodic Protection

Cathodic protection (CP) is a corrosion control method that imposes an external voltage on the steel surface in a manner that forces the steel to become cathodic (reduction reactions are favored and anodic reactions, which result in metal loss, are decreased), thereby mitigating corrosion. In simple terms, CP transfers the oxidation (anodic) reactions, which result in metal loss (and thereby corrosion) of the rebar, over to the anode of the CP system. Therefore, selection of the proper anode material for the application is critical, since anode failure results in CP system failure.

The primary strength of CP is that it can mitigate corrosion after it has been initiated. Although CP is often placed on pipelines, underground storage tanks, and other structures during construction, it is generally installed on bridge members only after corrosion has initiated and some amount of deterioration has occurred. The primary reason for not installing CP systems on bridge components during construction is that corrosion often does not initiate for 10 to 20 years following construction; therefore, the CP system maintenance and a large portion of the CP system design life would be used on a structure that is not corroding. Furthermore, the use of CP on newly constructed bridge components is limited since materials such as epoxy-coated rebar provide economic, long-term corrosion prevention for these structures. The exception to this is that CP is installed on newly constructed bridge pilings exposed to marine and brackish waters where corrosion is known to be a severe problem.

Although problems with early CP systems have cast a negative image with certain bridge engineers, current technology for bridge decks has proven to be quite reliable and improved technology for substructures is still being developed and tested. When properly applied and maintained, CP mitigates corrosion of the reinforcing steel and extends the performance life of a bridge. However, CP remains an under-utilized technology for steel-reinforced concrete structures.

CP systems are characterized by the source of the driving voltage that forces the rebar to become cathodic with respect to the anode. The two principal methods for applying CP are impressed-current CP and sacrificial (galvanic) anode CP. In an impressed-current CP system, an external power source is used to apply the proper driving voltage between the rebar and the anode. For impressed-current systems, the anode can be a wide range of materials since the driving voltage can be adjusted to suit the application and anode material selected. For a sacrificial anode CP system, the driving voltage is created by the electrochemical potential difference between the anode and the rebar. Therefore, selection of the anode material is more limited.

Impressed-Current CP

The basic characteristics of an impressed-current CP system are: (1) an external power source is required, (2) the driving voltage can be varied (variable power source), (3) the applied current can be varied, (4) the CP system can be designed for almost any current requirement, and (5) the CP system can be used in almost any level of resistivity. To date, more than 1.9 million m^2 (>20 million ft^2) of reinforced and prestressed concrete structures have been cathodically protected worldwide.

Anode selection and application have proven to be among the most difficult problems in designing CP systems for concrete structures with adequate life. The anode for a concrete bridge deck must have the following characteristics: (1) capability to withstand traffic loads, (2) resistance to environmental influences (moisture, temperature fluctuations, etc.), (3) sufficient durability to have a design life equal to or greater than that of the wearing surfaces, (4) sufficient conductive surface area to minimize or completely prevent premature deterioration of the surrounding concrete, and (5) it must be economical.

Over the past 30 years, several anode configurations have been utilized for concrete bridge decks and substructures, including those listed below.(6) A Strategic Highway Research Program (SHRP) study published in 1993 reviewed CP systems used for bridge structures, including performance, costs, and service life.(32) The cost data are for 1991; a discount rate of 5 percent was used to estimate the costs in 1998 dollars. The estimated costs and service lives are given, but it should be realized that specific problems have developed that have limited the actual service life achieved in some cases.

- Coke-asphalt anode system used high silicon iron anode material and required a wear surface. The application costs are estimated at \$92 per m^2 with a service life of 20 years.(32)
- Non-overlay slotted anode system used platinized-niobium-copper wire anode laid in regularly spaced slots designed to distribute CP current evenly to the rebar mat and was filled with a conductive polymer concrete. The application costs are estimated at \$92 per m^2 with a service life of 15 years.(32)
- Conductive polymer mound anode system used the platinized-niobium-copper wire anode with the conductive polymer mounded on the wire anode and a rigid concrete overlay on top. The application costs are estimated at \$137 per m^2 with a service life of 20 years.(32)

- Activated titanium mesh anode is secured to the concrete and covered with either a conventional concrete or a latex-modified concrete overlay. The application costs are estimated at $137 per m^2 with a service life of 35 years.(32)
- Activated titanium mesh anode is also applicable to substructures when overlaid with shotcrete. The application costs are estimated at $211 per m^2 with a service life of 35 years.(32)
- Other anode designs have been specially developed for use in impressed-current CP systems for substructures. These include sprayable conductive polymer coatings, metallized zinc coating, and conductive paints. Typical primary anode for the conductive polymer or paints is platinized-niobium wire attached to the concrete prior to application. The application costs are estimated at $76 per m^2 with a service life of 5 years.(32)
- The metallized zinc used either small stainless steel or copper plates epoxied to the concrete surface to make a connection back to the power source. The application costs are estimated at $137 per m^2 with a service life of 15 years.(32)

Problems with the CP systems have included: (1) debonding of the conductive coating that arises when the materials are used in environments where the concrete is constantly wet or when the materials are applied before the concrete is sufficiently dry; (2) degradation of conductive coating after extended current passage; and (3) increase in the electrical resistance between the anode and the steel due to insufficient moisture or accumulation of insulating byproducts at the anode/concrete interface.

Of the systems identified above, only the titanium mesh anode and metallized zinc are still actively used today. Furthermore, the use of titanium mesh on bridge decks is widely accepted in terms of providing long-term durability. The thermal-sprayed zinc is free of the debonding problem, but suffers from an increase in anode resistance over time. However, the Oregon DOT has had significant success with the thermal-sprayed zinc anode on substructure components (see Case Study 3). Recently, the use of thermal-sprayed titanium metal as a new anode has shown some promise when used on a trial basis on a bridge in Oregon.

Some DOTs, such as the Florida DOT, have undertaken experimental programs that investigate alternative energy sources for applied CP systems, such as solar power and long-life batteries.(33) The systems are intended for use on the substructure elements exposed to brackish waters.

In certain cases, CP offers the only acceptable service life extension as an alternative to replacement of a critical bridge component. For example, Oregon DOT has successfully implemented thermal-sprayed zinc CP systems on historic bridges (built in the 1930s) along Highway 101 (see Case Study 3).

Missouri DOT leads North America in the use of CP to extend the life of salt-contaminated and corroding concrete bridges. In Missouri, CP is primarily used for corrosion control of voided slab structures, although CP is also used on steel frame and stringer type structures. Conventional repair methods proved to be unsuccessful for limiting corrosion on bridges, many of which were built in the late 1950s and early 1960s. Since 1975, Missouri has installed CP systems on more than 140 bridges.

Many CP systems have been evaluated and used in Missouri. First introduced in 1986, the activated titanium mesh anode system with concrete overlay has become the exclusive CP system installed on Missouri DOT bridges. To date, this system has provided a high level of corrosion control to more than 30 bridge decks in the Kansas City and St. Louis areas.

Sacrificial Anode CP

The basic characteristics of a sacrificial anode CP system are: (1) no external power source is required, (2) the driving voltage is fixed, (3) the applied current is dependent on the driving voltage and the resistance between the rebar and the anode, (4) the CP system is limited to relatively low current requirements, and (5) the CP system is limited to relatively low-resistivity concrete environments.

Sacrificial anode CP systems have been used about as long as impressed-current anode systems for corrosion control of bridge decks. Two of the earliest field trials (1977) for sacrificial anode CP systems were: (1) perforated zinc sheets fastened on the deck with a bed of mortar, then covered with an concrete overlay, and (2) conventional zinc ribbons embedded in grooves cut into the concrete.(6) Both systems performed satisfactorily for 14 years prior to removal due to failure of the asphalt overlay and the necessity of widening the structure. Although the above field tests showed that sacrificial anode systems can be successfully applied to bridge decks, the majority of the CP systems on bridge decks are impressed-current systems.

Because of the relatively high resistivity of atmospherically exposed concrete substructures, most anodes utilize impressed current to achieve the necessary driving voltages to supply the current required for corrosion control. An exception to this is the use of sacrificial zinc anodes for CP of coastal bridges in Florida, which have a relatively low concrete resistance. However, studies continue to examine the use of sacrificial anodes primarily due to the benefit of very low maintenance compared to impressed-current CP systems. Two of these studies include the zinc-hydrogel anode system and the thermal-sprayed alloy anode system. The zinc-hydrogel anode system uses zinc sheet anodes (10-20 mm thick) attached to the concrete with ionically conductive hydrogel adhesive. Field trials have shown that this system is capable of supplying sufficient current for effective corrosion control. The thermal-sprayed alloy anode system utilizes a conventional metallization (flame- or arc-spraying) process to form metallized coating on the concrete surface. The two most promising anode materials were Al-Zn-In alloy and zinc.(6)

The cost of CP systems varies depending on the type of system used. Virginia DOT has issued a report entitled *Evaluation of Anodes for Galvanic Cathodic Prevention of Steel Corrosion in Prestressed Concrete Piles in Marine Environments in Virginia*.(34) This data and data published by Virmani(6) suggest that the sprayed Al-Zn-In alloy or the zinc-hydrogel alloy systems cost between \$108 and \$129 per m^2 (\$10 and \$12 per ft^2). The life of these systems is estimated to be 10 to 20 years.

Cathodic Protection for Prestressed Concrete Bridge Members

The primary concern for CP of prestressed concrete members is the possibility of hydrogen-induced cracking failure (hydrogen embrittlement) of the tendons at operating loads. Hydrogen production at the steel surface is a product of CP at potentials more negative than –0.90 V saturated calomel electrode. For this reason, CP for prestressed concrete has focused on the use of sacrificial anode systems and constant current or constant voltage rectifier impressed current systems. An additional concern is the application on bridge members that have an uneven electrical resistivity across the concrete surface. This will lead to the uneven distribution of the CP current and the possibility of overprotection in the low-resistivity regions. It is generally agreed that CP of prestressed concrete members can be accomplished safely and reliably if proper care is given to maintain minimum CP requirements and to prevent overprotection.

Electrochemical Chloride Extraction

Virmani and Clemena recently reviewed the use of electrochemical chloride removal.(6) When a direct current is conducted through concrete, the relatively mobile ions (such as chloride, hydroxide, sodium, potassium, calcium, etc.) in the concrete will migrate, with each ion moving toward the electrode with the charge opposite to it. The feasibility of removing the undesirable chloride ions from a contaminated concrete by such electrochemical means, instead of excavation of the contaminated concrete from a structure, was studied in the mid-1970s by Kansas DOT.

It was shown that chloride ions can be expelled from concrete by passing a direct current between the steel bars and anode, as in CP except at considerably greater current densities. However, unnecessarily high levels of direct current used in early investigations had some adverse effects on the concrete (e.g., decreased concrete-to-steel bond, increased porosity, and increased cracking in the concrete). The concern about these adverse effects on treated concrete delayed the use of electrochemical chloride extraction as a remedial method for the permanent rehabilitation of concrete bridges. Subsequent studies found that if the level of current applied is kept below 5 A per m^2 (0.5 A per ft^2), these adverse effects were not observed.

Because of the relatively high current densities (even at 5 A per m^2) and concerns about hydrogen-induced cracking, electrochemical extraction of chloride would not be used on prestressed concrete structures.

Full-scale pilot treatments have demonstrated that it is feasible and simple to conduct the treatment on full-sized reinforced-concrete bridge members, although it is comparatively more difficult to conduct the treatment on concrete piers. One difficulty encountered was predicting the necessary length of treatment required to reduce the chloride concentration at the steel rebar level to below the corrosion threshold or to some equilibrium concentration of chloride. Preliminary studies suggested that a total charge of 600 to 1,500 A-h per m^2 is sufficient in most cases, which means a total treatment time of 10 to 50 days is required.

While it is impossible to remove all the chlorides from the concrete by electrochemical means, chloride extraction depletes the amount of chloride immediately in contact with the steel and replenishes the passive layer (between 40 and 95 percent of the chloride ions are generally removed). Field data, so far, show that this is effective in stopping corrosion for at least 8 years. FHWA predicts that electrochemical removal technology will extend the life of bridges by as much as 20 years.(35) To date, there has been approximately 372,000 m^2 (4,000,000 ft^2) of concrete worldwide that has been treated.

The cost of electrochemical removal varies depending on the type and size of the structure. Treatment of bridge decks typically cost between \$53 and \$129 per m^2 (\$5 and 12 per ft^2), depending on the size and contract requirements. The cost of electrochemical removal on substructures (vertical and overhead applications) is between \$107 and \$215 per m^2 (\$10 and \$20 per ft^2). Very small substructures (i.e., one or two columns) may cost up to \$269 per m^2 (\$25 per ft^2) if done on a stand-alone basis.(36)

Summary of Current Practices for Rehabilitation

The following items summarize the current practices based on research, field performance, and emerging technologies.(6)

Overlays, such as latex-modified concrete, low-slump concrete, high-density concrete, and polymer concrete, are the most common method used for the rehabilitation of bridge decks. This procedure extends the life of the bridge deck by approximately 15 years.

Cooperative research with industry and states in the development of durable anodes, monitoring devices, installation techniques, etc. has led to application of impressed-current CP systems on bridge decks as a routine rehabilitation technique. Titanium mesh anode, used in conjunction with a concrete overlay to distribute protective current, is filling the need for a durable anode for use in impressed-current CP of reinforced-concrete bridge decks and is, in fact, now widely accepted by state and local transportation agencies.

For CP of substructure members, especially those in a marine environment, several promising sacrificial anode systems have been developed (i.e., thermal-sprayed zinc, thermal-sprayed aluminum-zinc-indium (Al-Zn-In) alloys, zinc hydrogel, and zinc mesh pile jacket systems). Initiatives in the industry and in some states, in cooperation with FHWA, have led to further developments and identification of anodes suitable for impressed-current CP of inland concrete substructures.

Through extensive fundamental research and evaluation of CP systems that have been installed, significant advances have been made in the technology for CP of prestressed concrete components. Concerns about a loss of bond between the prestressing steel and concrete and possible hydrogen embrittlement (from overprotection of the prestressing steel) have been alleviated by the establishment of criteria for qualification of prestressed concrete bridge components for CP.

Summary of Rehabilitation Cost Alternatives

Table 9 gives the costs of electrochemical rehabilitation alternatives for bridge structures. Also provided is the expected life for each alternative. Often, electrochemical methods are in competition with rehabilitation utilizing an overlay such as low-slump, high-performance, or latex-modified concrete (see table 8 for costs). The deck condition is often the controlling factor in the rehabilitation method selected. In some cases, a combination of these methods is selected, e.g., electrochemical removal followed by an overlay or an overlay in conjunction with CP to mitigate any further corrosion.

Table 9. Summary of costs and life expectancy for rehabilitation alternatives.

TYPE OF MAINTENANCE	AVERAGE COST	RANGE OF COSTS	AVERAGE EXPECTED LIFE	RANGE OF EXPECTED LIFE
	($/m^2)	($/m^2)	(years)	(years)
Impressed-Current CP (Deck)	114	92 - 137	35*	15 - 35
Impressed-Current CP (Substructure)	143	76 - 211	20	5 - 35
Sacrificial Anode CP (Substructure)	118	108 - 129	15	10 - 20
Electrochemical Removal (Deck)	91	53 - 129	15	10 - 20
Electrochemical Removal (Substructure)	161	107 - 215	15	10 - 20

*Current technology.

Deicing Alternatives

Calcium magnesium acetate (CMA) and potassium acetate (PA) have been identified as the most promising deicing alternatives. These compounds contain 76 percent and 61 percent of acetic acid, respectively, which represents approximately half of the formulations' costs. The annual usage of rock salt (sodium chloride) in the United States for deicing purposes is approximately 15.4 billion kg (17 million tons). A 1987 study showed that 910 kg (1 ton) of road salt, while costing $50, causes more than $1,450 in damages to vehicles, bridges, and the environment.(37) CMA's current price is approximately $1.10 per kg ($1,000 per ton) versus $0.04 per kg ($35 per ton) for rock salt.(38) This cost differential means that CMA usage will be limited to critical structures sensitive to corrosion unless some means of sharing costs based on the overall damage caused by the use of salt is devised.

In addition to the high price, CMA use is hampered by other limitations, e.g., CMA is slower acting than rock salt, if applied as a solid, and CMA exhibits marginal performance in light traffic, freezing rain, and dry and cold storm conditions. However, recent studies have shown that if the compound is applied as a concentrated solution or a pre-wetted solid, the rate of action is similar to that of a rock salt.(38) New York City DOT has implemented, on an experimental basis, a spray-on delivery of a liquid agent for anti-icing of certain sections of the Brooklyn Bridge deck (see Case Study 2).

Steel Bridges

In this section, various steel bridge coating installation and maintenance options are discussed, along with their costs and expected life. (7)

Coating Options

In addition to the traditional coating methodologies used on steel bridges, research to date has identified several technologies and maintenance methodologies that promise to provide cost-saving alternatives for bridge maintenance painting. Among these are: (1) the zone painting approach, (2) the use of overcoating or maintenance repair painting techniques, and (3) the selected use of metal spray coatings.

Traditional Coating System

A two- to three-coat system is traditionally applied over a clean, blasted surface. These coating systems include:(39)

- organic zinc primer, epoxy or polyurethane intermediate coat, and aliphatic polyurethane topcoat,
- inorganic zinc silicate primer, chemically curing epoxy or polyurethane intermediate coat, and aliphatic polyurethane topcoat,
- high-build, high solids, good-wetting epoxy primer with aliphatic polyurethane topcoat,
- three-coat waterborne acrylic, and
- three-coat, lead-free alkyd.

Zone Painting

Due to the increasing cost of the repainting of existing bridge structures, it has become economically advantageous to consider the use of zone painting approaches in lieu of wholesale removal and repainting of entire bridge structures. This concept is especially attractive for larger structures and, in fact, has been employed on structures such as the Golden Gate and Bay bridges in California and several of the bridges in the New York City area. These larger bridges have distinctly different exposure environments within the same structure simply because of their size and their location near saltwater. In addition, these bridges are maintained by bridge authorities, who collect tolls and generally have greater resources to focus on intermittent or periodic maintenance activities.

The vast majority of the bridges in this country are neither large nor maintained by toll authorities. Hence, the zone painting approach has not been applied on a widespread basis. This may change as the costs for full removal and repainting of even smaller structures have dramatically risen. The fact is that even on smaller structures, coating breakdown and corrosion is limited to areas where there are measurable levels of salt contamination and significant times of wetness. For bridges in marine or semi-marine environments, this is the entire structure; however, for bridges in non-marine environments (a majority of the bridges), these corrosive areas are generally limited to expansion joints, drainage, traffic splash, and tidal areas. If these areas can be isolated and maintained using a better corrosion protection system, large expenditures can be avoided on the remaining surface area of the bridge. This change in philosophy will require more informed engineering input during specification development and more oversight during repainting operations. In addition, improved inspection procedures and standards will be an essential input into the decision-making process.

Overcoating

Similarly, overcoating has become a more attractive option for state agencies as the cost of full removal and repainting has increased. This approach limits the amount of surface preparation to those areas that have failed paint and corrosion. These areas are spot primed and one or two full coats are applied over the entire structure for uniformity of color. This approach can be effective in less corrosive environments where the condition of the existing coating is relatively good. However, since this method of preservation will usually have a significantly lower initial cost than full repainting, the effect on life-cycle cost of this approach must be examined very carefully.

Metal Spray Coatings

Non-traditional bridge coating systems have been investigated for potential long-term performance benefits. While some of the candidates tested have not shown immediate usefulness (e.g., powder coatings), others, such as metallized coatings, appear to have the benefit of excellent long-term corrosion resistance. Although these systems are applied at a somewhat higher initial cost, the changing overall economics of bridge repainting operations has made their use more competitive in terms of life-cycle cost.

Coating Installation - Maintenance Costs

The coating system installation cost is not easy to define. Over the past several years, there have been significant changes in the methodology of bridge maintenance painting operations. The most significant changes have been in response to dramatic increases in environmental and worker protection regulations that impact these operations. The use of containment structures to capture hazardous waste and pollutants generated during removal of old coatings and the gradual institutionalization of worker health and safety practices associated with the removal of hazardous materials, have introduced significant cost impacts to bridge maintenance painting. This has caused a large diversity in operational practices and in the resultant cost of these operations.

The issue of applying protective coatings to the steel bridges to prevent corrosion is further complicated by the requirement to contain or remove the previously applied lead-based paint, as regulated by the Environmental Protection Agency. Congressional regulations (the Resource Conservation and Recovery Act and the Hazardous and Solid Waste Amendment) now require that all wastes be treated.

According to 1992 National Cooperative Highway Research Program (NCHRP) data,(40) approximately 80 percent of the steel highway bridges have been coated with lead-containing paints. The report estimated that $100 million to $130 million is spent annually on bridge painting. A total of 10 to 20 percent of the costs of bridge painting are incurred because of the requirement to contain paint, abrasive, and dust fallout. In addition, the costs of treatment can range from $0.33 to $0.55 per kg ($300 to $500 per ton) where lead paint removal activities generate an estimated 181 million kg (200,000 tons) of lead-contaminated abrasives.

The overall cost is comprised of the costs for surface preparation, the material itself, and application activities. The estimates for some of the above coating systems are given in table 10.(7) The service life of the coating systems is significantly affected by the service conditions. For example, a two-coat alkyd primer with the topcoat exposed to mild conditions (rural or residential area with no industrial fumes/fallout) would last only 3 years until the next maintenance. On the other end of the spectrum is the triple system consisting of a moist-cured urethane zinc-rich coat, a high-build acrylic urethane coat, and an acrylic urethane topcoat. The expected service life of this coating system in severe conditions (heavy industrial and chemical plant area with high levels of fumes and fallout) is 15 years.

Table 10. Cost for alkyd, epoxy, and epoxy/urethane systems in moderate industrial environment in the southeast United States.(7)

System	SSPC Surface preparation	DFT****	Cleaning Cost	Material Cost	Application Cost	Total Installed Cost	System Life (5-10% breakdown)	Cost/year
		(mm)	(\$/m^2)	(\$/m^2)	(\$/m^2)	(\$/m^2)	(years)	(\$/m^2)
Two-coat alkyd	2*	0.10	\$5.92	\$1.08	\$5.38	\$12.38	3	\$4.09
	6**		\$9.15			\$15.61	6	\$2.58
Two-coat epoxy	2	0.15	\$5.92	\$1.72	\$6.46	\$14.10	7.5	\$1.83
	6		\$9.15			\$17.33	10.5	\$1.61
	10***		\$10.76			\$18.94	12	\$1.61
Two-coat epoxy/ urethane	6	0.15	\$9.15	\$2.26	\$7.00	\$18.41	9	\$2.05
	10		\$10.76			\$20.02	10.5	\$1.94

*Hand-cleaned surface.
**Commercial blast.
***Near-white blast.
****Dried-film thickness.

Presently, the costs of total paint removal and repainting jobs can range from \$43.00 per m^2 (\$4.00 per ft^2) to as much as \$215.25 per m^2 (\$20.00 per ft^2).(41) This range can be partially explained by factors that make each bridge maintenance job unique, such as access for high structures or structures over water, the condition of bridge deterioration, and unusual traffic control. However, a significant portion of the cost range is attributable to uneven application of regulatory compliance measures for environmental and worker safety issues.

An alternative to paint removal is overcoating, which includes cleaning of the structure, priming rusty areas, and applying intermediate coats and topcoats either over repaired areas or over the full structure. The cost of overcoating for bridges was estimated to range from \$11 to \$54 per m^2 (\$1 to \$ 5 per ft^2), with some evidence that the tighter OSHA standards(42) push the cost up to \$86 per m^2 (\$8 per ft^2).

The present effort to implement bridge corrosion control maintenance practices, which achieve regulatory requirements and cost-efficiency, cannot be successful without the development of reliable task-based cost data for bridge painting jobs. These data are dependent on a variety of factors, which vary from local cost differences (e.g., labor) to structural differences (e.g., accessibility) to contractor costing rules (e.g., limits on certain items such as mobilization). Development of reliable data and an understanding of regional influences on these data will help to improve analysis of the cost data.

It is estimated that roughly 50 percent of the cost of an average maintenance painting job is now attributable to environmental protection and worker health measures. This increase in "other" job costs has raised the total cost of coating removal jobs from an average of \$54.36 per m^2 (\$5.05 per ft^2) in 1992 to an average of \$114.10 per m^2 (\$10.60 per ft^2) in 1995, while the cost for the actual work (surface preparation and coating materials) has stayed relatively constant. Note that the savings incurred by paying slightly less for a less durable coating material are minor as a percentage of the overall cost. This highlights the need for life-cycle cost analysis.

Estimated time to failure for several coating systems is presented in table 11. Table 12 presents the estimated costs for painting options used in the sample analysis. The costs presented in the table are composite figures based on information from several different sources (41,43) and are expected to vary across the United States. Table 11 data show that depending on the surface preparation (i.e., blasting versus overcoating) and the type of coating, the assumed service life (life to 10 percent of degradation) can vary considerably, from as few as 3 years to 30 years. Similarly, table 12 suggests that the longevity of a coating is closely related to the costs of surface preparation and coating application. For example, overcoating, lasting only a short time, is inexpensive at \$3.22 per m^2

(\$0.30 per ft^2), whereas, near-white metal blasting followed by metallizing, which is expected to serve for 30 years, costs 10 times as much.

Table 11. Coating system time-to-failure estimates in a marine environment.(41,43)

COATING SYSTEMS	ESTIMATED COATING SYSTEM LIFE*
Ethyl Silicate Inorganic Zinc/Epoxy Polyamide/Aliphatic Urethane over SP-10 Near-White Metal Blast	15 years
Epoxymastic/Aliphatic Urethane over SP-10 Near-White Metal Blast	10 years
Epoxymastic/Aliphatic Urethane Overcoat over Existing Paint and SP-3	4 years**
85% Zinc/15% Aluminum Metallizing over SP-10 Near-White Metal Blast	30 years***
Low-VOC Alkyd Three-Coat System Overcoat over Existing Paint and SP-3	3 years**

*Lifetime was defined as 10 percent degradation of the coatings.

**Estimates based on data from FHWA programs.

***Estimates based on the performance of metallized coatings in this program.

Table 12. Estimated costs for painting options.(41,43)

CATEGORY	TYPE	ESTIMATED COST, (\$/m^2)
Surface Preparation (labor + material)	SP-10 Near-White Metal Blast	\$13.45
	SP-3 Power-Tool Cleaning	\$ 6.46
Coating Application	Three-Coat Full Painting	\$13.45
	Overcoating	\$ 3.23
	Metallizing	\$26.91
Coating Material	IOZ/Epoxy/Urethane	\$ 5.27
	Epoxymastic/Urethane	\$ 4.52
	Metallizing	\$16.15
	Moisture-Cured Urethane	\$ 2.69
	Three-Coat Alkyd	\$ 2.05
Other Job Costs	Containment and Air Filtration Systems, SP-3 only	\$ 5.38
	Containment and Air Filtration Systems, SP-10 only	\$21.53
	Inspection, SP-3 only	\$ 5.38
	Inspection, SP-10 only	\$10.76
	Rigging	\$ 5.38
	Mobilization	\$ 5.38
	Hazardous Waste Storage and Disposal, SP-3 only	\$10.76
	Hazardous Waste Storage and Disposal, SP-10 only	\$26.91
	Worker Health and Safety, SP-3	\$10.76
	Worker Health and Safety, SP-10	\$21.53

Table 12 also contains information on extra costs such as containment and waste disposal-related costs, and worker health and safety costs. The numbers show that these types of costs are equal to or exceed the costs of surface preparation, coating material, and coating application.

A sample cost distribution, shown in figure 7 for a typical heavy-duty maintenance job on a steel bridge structure, indicates that only a small portion of the total job cost is attributed to paint and paint application.(41) More than half of the cost is taken by access, containment, and workers health costs. Not included are the lead abatement and waste treatment costs, which can result in as much as a sevenfold increase in cost.

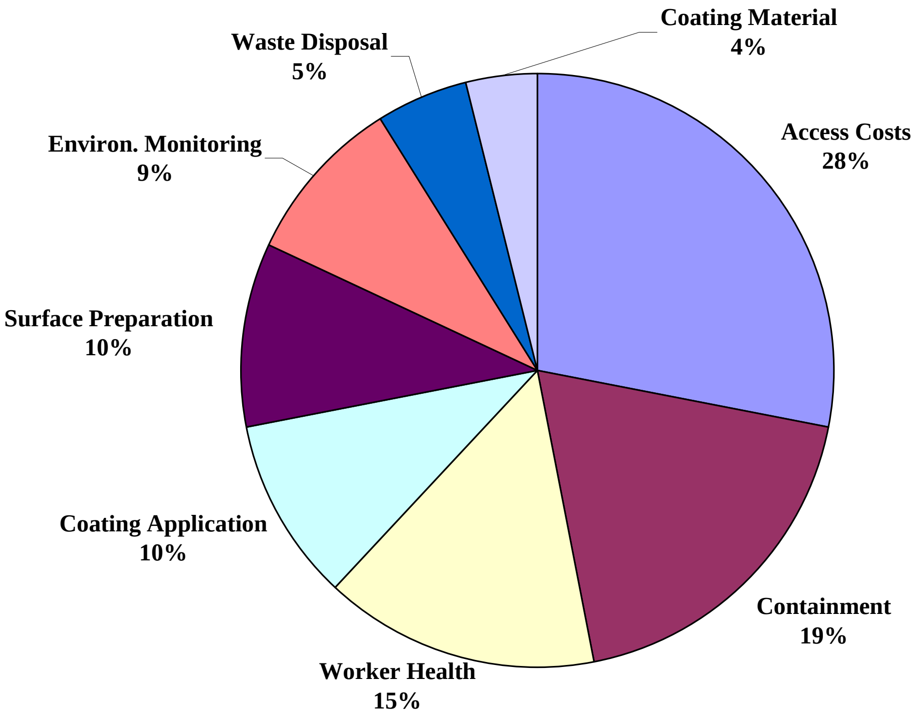

Figure 7. Cost distribution of coating application on steel highway bridge structure.(41)

Sample Cost Analysis for Coating Options

Sample life-cycle cost analysis data for different coating options, assuming a 60-year life span of a bridge, are presented in table 13.(7) The overcoating options offer the lowest initial cost; however, these are not always the lowest annual cost. In fact, the coating removal options show the lowest annual costs in a severe environment, but the choice was less clear as the environment was made less severe.

Table 13. Summary of sample analyses.

Approach	**Coating System**	**Coating Life**	**Surface Prep.**	**No. of Maint. Cycles**	**Cost Per Maint. Cycle**	**Total Cost in Present Day Dollars**	**Total Present Value**	**Annual Costs**
		(years)			**($/m^2)**	**($/m^2)**	**($/m^2)**	**($/m^2/year)**
Existing lead-based paint; repair and over-coat with three-coat alkyd	3-coat alkyd	3	SP-3	20	$56.94	$1,138.80	$477.92	$34.01
	Epoxymastic/ polyurethane	4	SP-3	15	$59.42	$891.30	$458.22	$32.61
Existing lead-based paint; full removal by blasting	85% Zn / 15% Al metallizing at 6 to 8 mils	30	SP-10	2	$158.77	$317.54	$227.44	$16.15
	IOZ/epoxy/ Polyurethane	15	SP-10	4	$123.68	$494.72	$300.64	$21.42
Existing lead-based paint; full removal and maintenance over approximately 20% of the surface area every 5 years after the initial 15-year service life.	IOZ/epoxy/ Polyurethane	15	SP-10	1	$123.68	$690.41	$351.01	$24.97 $0.00
	Maintenance	5	SP-3	9	$62.97			
Existing lead-based paint; remove and replace	Epoxymastic/ Polyurethane	10	SP-10	6	$120.23	$721.38	$413.33	$29.49
Existing lead-based paint; repair and overcoat	3-coat alkyd	10	SP-3	6	$56.94	$341.64	$156.72	$11.19
Existing lead-based paint; full removal	85% Zn / 15% A1 metallizing at 6 to 8 mils	60	SP-10	1	$158.77	$158.77	$158.77	$11.30

Cost of Corrosion for Bridges

The following analysis was used to provide an estimate of the annual direct cost of corrosion for highway bridges. The analysis is divided into: (1) cost to replace structurally deficient bridges, and (2) corrosion associated life-cycle cost for remaining (non-deficient) bridges, including the cost of construction, routine maintenance, patching, and rehabilitation.

The annual cost of structurally deficient bridges (see figure 8) is estimated as the cost to replace these bridges over a 10-year period; it is calculated using a $29.3 billion as a present value of the cost (see “Areas of Major Corrosion Impact” for calculation) at a 5 percent annual percentage rate (APR). Assuming annual payments for the replacement cost, the annual cost to replace structurally deficient bridges (both reinforced concrete and steel) over the next 10 years is $3.79 billion per year. Recall that this value is for the current number of deficient bridges and does not account for the additional ones added to this number each year. Therefore, this cost is potentially greater than that given here.

Figure 8. Examples of severe corrosion resulting in deficient bridges.

There are 543,019 concrete and steel bridges, of which 78,448 are structurally deficient (see table 3), leaving 464,571 bridges to be maintained. For the purposes of this estimate, it is assumed that all of these bridges have a conventionally reinforced concrete deck. The annualized life-cycle direct cost (no user cost) of original construction, routine maintenance, patching, and rehabilitation for a black steel rebar deck ranges in cost from $22,000 (experienced-based maintenance) to $18,000 (information-based maintenance with crack repair) for an "average" size bridge deck (see figure 19 at 5 percent interest). This annual life-cycle cost of $22,000 to $18,000 per bridge includes those costs associated with corrosion (see figure 9), as well as non-corrosion-related costs. To establish the corrosion-related costs requires the calculation of the life-cycle cost associated with a theoretical "corrosion-free" bridge deck (i.e., what if corrosion did not exist). The "corrosion-free" scenario used the same cost basis as the above bridge deck with corrosion, with the following assumptions for the life cycle: (1) cost of construction is the same as for the deck with corrosion, (2) annual routine maintenance is the same as for the deck with corrosion, (3) no patching is required, (4) an overlay is required for improved skid resistance at 50 and 85 years (an overlay life of 35 years), giving a bridge life of 120 years, and (5) deck is removed at 120 years. This scenario gave an annual cost for a "corrosion-free" bridge deck of $15,700 (see "Theoretical Corrosion-Free Bridge - Direct Cost Only"). Therefore, the cost of corrosion for an "average" bridge deck is estimated by the difference in the annual cost of a "deck with corrosion" and a "corrosion-free deck," or $6,300 ($22,000 - $15,700) to $2,300 ($18,000 - $15,700). The total estimated cost of corrosion for bridge decks is $2.93 billion ($6,300 per deck x 464,571 bridges) to $1.07 billion ($2,300 per deck x 464,571 bridges).

Figure 9. Examples of bridge deck corrosion.

The differences in the two maintenance scenarios that resulted in this range of corrosion-related costs were the "experience-based maintenance" and "information-based maintenance with crack repair" (see "Life-Cycle Cost Analysis for Bridge Decks" for details). This difference represents the range of maintenance from minimal practice to best practice. The cost analysis estimated the cost of corrosion from $6,300 (minimal practice) to $2,300 per deck per year (best practice). These values show that a savings of 63 percent [($6,300 – $2,300) / $6,300] of the cost of corrosion is possible by improving the maintenance from minimal to best practice. However, the actual bridge maintenance practice is somewhere between the minimal and the best practice. If it is assumed that today's maintenance practice represents the “average” in the above range [($6,300 - $2,300) / 2 = $4,300], 46 percent savings [($4,300 – $2,300) / $4,300] or $2,000 per bridge per year can be achieved by improving maintenance practice.

These savings were calculated for black steel rebar decks for which improved maintenance can still provide savings. However, corrosion of many black steel rebar decks has progressed to the extent that improved maintenance will not make a significant difference. For those decks, other rehabilitation options must be considered (e.g., cathodic protection, overlays, or electrochemical chloride removal). If the savings of $2,000 per bridge per year is applied to the total number of bridges, the total savings would be $0.93 billion per year. As previously mentioned, this savings is not available today for all bridges, but the significance of “best engineering practice” for maintenance cannot go unnoticed.

The area of the substructure and superstructure (minus deck) was estimated to be similar to the deck surface area for an “average” bridge. The following was taken into consideration for estimating the cost of substructures and superstructures (minus deck): (1) repair and maintenance for the substructure/superstructure cost significantly more per surface area than the deck; (2) in non-marine applications, the percent of surface area deteriorated due to corrosion of the reinforcing steel is much less and often is limited to areas beneath expansion joints and drains, which are exposed to deicing salt runoff; and (3) conversely, corrosion problems are more prevalent on substructures than decks in severe marine environments. With these considerations, it was estimated that the cost of corrosion for substructures and superstructures (minus deck) is similar to the cost for bridge decks, i.e., $2.93 billion to $1.07 billion (see figure 10).

The cost for steel bridges has an additional cost for maintenance painting. The expenditure for painting steel bridges is estimated at $0.50 billion per year.(7)

The total annual direct cost of corrosion for bridges is estimated to be $10.15 billion to $6.43 billion, which is the sum of all costs itemized above ($3.79 billion to replace structurally deficient bridges over the next 10 years plus $2.93 billion to $1.07 billion for maintenance and cost of capital for concrete bridge decks plus $2.93 billion to $1.07 billion for maintenance and cost of capital for concrete substructures and superstructures (minus decks) plus $0.50 billion for the maintenance painting cost for steel bridges). This gives an average annual cost for corrosion of bridges of $8.29 billion. As seen in the case studies presented later, the cost of corrosion can be significantly greater than the above for individual bridges, especially those of historical significance or those that are critical to traffic flow. In addition, problems in post-tensioned bridges or cable and suspension bridges can be very costly to repair. Although the direct costs presented above are estimated by making broad assumptions, the calculated cost represents the relative cost of corrosion for the highway bridge industry sector. Life-cycle analysis estimates indirect costs to the user due to traffic delays and lost productivity at more than 10 times the direct cost of corrosion (see “Life-Cycle Cost Analysis for Bridge Decks”).

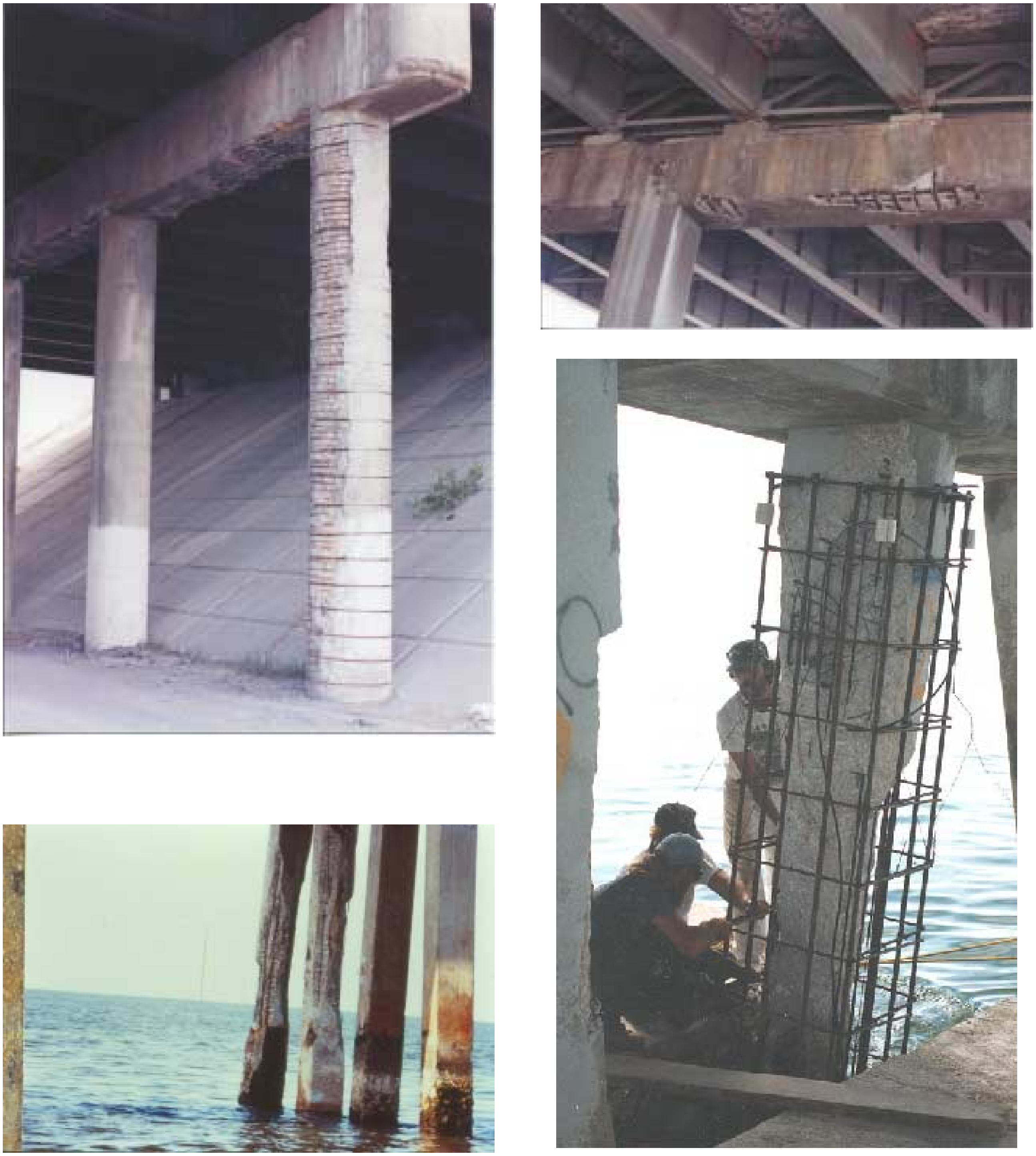

Figure 10. Examples of substructure corrosion.

CASE STUDIES

Case Study 1. Corrosion of Florida Post-Tensioned Bridges(44-45)

Recent inspections of two Florida post-tensioned bridges (Niles Channel and Mid-Bay bridges) have identified corrosion-induced strand and tendon failures. These recent failures highlight the need for better inspection techniques and tools to identify problems before bridge integrity is compromised.

Niles Channel Bridge

The Niles Channel Bridge is a 1,389-m- (4,557-ft-) long structure built in 1983 located in the Florida Keys. The superstructure is comprised of segmental, precast concrete post-tensioned box girders with a deck width of 11.7 m (38.5 ft). All of the conventional reinforcing bars are coated with epoxy. The box girders are post-tensioned by means of 19 strand tendon bundles situated along the interior web walls and anchored into the bulkheads at the ends of each 29.1-m (95.5-ft) span. The tendon bundles are encased in polyethylene ducts, which attach to the metal anchorage assemblies cast into the bulkheads. The polyethylene ducts join the anchorage externally with a banded rubber sleeve. Both the ducts and the anchorages are grouted with a portland cement-based material. The tensioning ends of the anchorage assemblies are protected by means of cast-in-place blocks (pour-backs) either 152 or 304 mm (6 or 12 in) thick, depending on the location.

During a 1999 routine inspection, it was noted that the pour-back concrete had spalled, exposing the wedge plate at the NW anchorage (middle tendon) at Expansion Joint No. 2 (Pier 9). Subsequent investigation revealed that each of the 19 strands making up that tendon had failed due to corrosion, thus rendering the tendon nonfunctional. The appearance of the failed strand is shown in figure 11. Following removal of the wedge plate and strands, the examination of the trumpet showed heavy corrosion on the upper-half of the trumpet with moderate pitting. The appearance indicated that the upper-half of the trumpet had never been in contact with grout, but instead that space had been occupied by bleed water. Although the heaviest corrosion on the strands was in the trumpet region, corrosion extended up approximately 1.5 m (5 ft) from the bulkhead. The likely cause of corrosion was determined to be due to bleed water being present at the time of construction, recharging of the environment within the void, and possibly aggravation by chlorides.


(a) (b)

Figure 11. (a) corrosion in the free length of tendon, (b) failed strands.

Further investigation revealed heavy corrosion on two strands of another tendon where the polyethylene duct attaches to the trumpet at the middle tendon, NE anchorage, at Expansion Joint No. 2 (Pier 9, but on the opposite web wall from the failed tendon). Both strands were broken at some point inside the trumpet and evidence of voids in the grout along with bleed-water staining was present. Although minor corrosion was observed on other tendons, no other significant corrosion was detected.

During further investigations, Florida DOT employed several nondestructive evaluation methods to determine the extent of the strand corrosion. These included the vibration method (frequency analysis of induced tendon vibration), electrical method (measurements of electrical resistance between the strands and the anchorage), and the magnetic flux method (detection of variations in the strength of the magnetic field applied to strands). The effectiveness of these methodologies in detecting corrosion or voids in the grout resulting from bleed water has not been fully established.

Mid-Bay Bridge

The Mid-Bay Bridge is 5,872 m (19,265 ft) in length and made up of 141 spans crossing Choctawhatchee Bay in Okaloosa County, Florida. The superstructure is comprised of segmental, precast post-tensioned box girders. During a routine inspection, a post-tensioning tendon in Span 28 was observed to be significantly distressed. The polyethylene duct was cracked, exposing the strands, of which several had failed. An immediate walk-through indicated complete failure of a post-tensioned tendon in Span 57, as evidenced by pull-out of the tendon from the expansion joint diaphragm.

The discovery of the failed strands and tendon led to a rigorous inspection and testing regimen consisting of the following:

- sounding post-tensioned tendons for voids,
- bore scope inspections of post-tensioned anchors,
- vibration testing,
- visual void inspections,
- magnetic flux testing, and
- grouting mock-up tests.

No single inspection or testing method is able to provide complete evaluation of the corrosion of external post-tensioned tendons. Techniques that provide information in the free length of the tendon do not give results in the anchorage zones. The approach taken for the Mid-Bay Bridge was to conduct a battery of tests chosen to develop an understanding of the tendon conditions. Based on these tests, corrosion was found at several locations, but no other tendon failures were discovered. The primary cause of corrosion was the presence of water in the tendons, probably due to bleeding of the grout, although other possible sources were not completely ruled out.

Based on the above inspections, 11 post-tensioned tendons were identified as needing replacement. In addition, significant repair was required at anchorages, including: (1) replacement of all pour-backs located at expansion joint piers – 89 required, (2) grouting of anchorage voids with strands visible – 274 required, (3) grouting of anchorage voids without strands visible – 316 required, (4) replacement of pour-back at interior piers – 307 required, and (5) coating of undamaged pour-backs with coal-tar epoxy – 408 required. Because of voids and cracks in the polyethylene duct, which serves as a protective barrier against the ingress of corrosive agents, wrapping of up to 35,000 linear m (115,000 linear ft) of duct was undertaken.

Summary of Case Study 1

Corrosion identified on post-tensioned tendons is a major concern to bridge engineers. Part of the concern is that it is extremely difficult to inspect for problems in post-tensioned designed structures. Therefore, only isolated problems are observed, and then only by visual observation during routine inspection. The concern is whether deterioration due to corrosion that affects the structural integrity of the bridge structure can be found during routine inspections prior to major failure.

The cause of the corrosion in both the Niles Channel and Mid-Bay bridges appears to be bleeding of the grout, resulting in water-filled voids in the tendon and subsequent recharging of the voids with external water/moisture. The bleed water is often sufficiently corrosive to permit corrosion of the high-strength steel strands. In both bridges, problems were identified during routine inspections, underscoring the importance of such inspections for all post-tensioned bridges. Several deficiencies in post-tensioned bridge construction and inspection practices are apparent from these two case studies:

- Bleeding of grouts remains a significant problem, even following much research and acknowledgment of the problem for many years. Improved grout placement methodology is necessary to minimize grout bleeding and void formation. Improved grout mix designs should be used (the Post-Tensioning Institute has recently issued new specifications for grouting of post-tensioned structures). Recently, pre-packaged grout mixtures have been made available, which minimizes problems associated with bleed water and formation of voids in the ducts.
- Overall post-tensioned bridge designs should be reviewed to permit improved inspection capability of the tendons and anchorages.
- Better methods are needed to evaluate construction practices to ensure void-free grout placement.
- Inspection technologies specifically designed or adapted to post-tensioned bridge designs need to be fully developed with respect to their effectiveness in detecting a certain type of flaw or performance criterion, resolution of detection, and reliability. One possible outcome would be that a nondestructive inspection report forms the basis for acceptance by the owner of a completed bridge.

Case Study 2. East River Bridges in New York City(46-48)

Introduction

The four East River bridges, the Brooklyn, the Williamsburg, the Manhattan, and the Queensboro, have permitted New York City to claim its place among the greatest centers of urban activity worldwide by connecting the boroughs of Manhattan, Brooklyn, and Queens. At their peak, the four bridges carried nearly 2 million passengers a day; today, they carry more than a million passengers and remain a vital link for the city and metropolitan area.

Their historical significance is as important as their practical significance. In 1883, the Brooklyn Bridge became the worlds longest suspension bridge, with a main span of 487 m and side spans of 284 m each. In 1903, the Williamsburg Bridge pushed the main span record to 488 m. The Manhattan Bridge opened in 1908 and was the first fully suspended bridge to be designed by large deflection theory (main span of 449 m and side spans of 222 m). The Queensboro Bridge is a five-span cantilever truss structure, built in 1912, with a longest span of 361 m and a total length of 1,136 m. The Brooklyn Bridge has become one of the most recognizable landmarks for New York City.

At nearly 100 years old or older, these bridges have undergone numerous repairs and are undergoing rehabilitation. The rehabilitation strategies for these bridges have evolved over the past two decades and will continue to evolve. Numerous innovative repair and rehabilitation designs are being tried or have been implemented on these bridges in an attempt to find new cost-effective and life-extending solutions to the bridge's aging problems.

Inspection Strategies

Bridge inspection and maintenance strategies, which have led to the current bridge condition, were ones of minimizing initial and current expenditures. Historically, the federal programs for investment and new construction led to the bridge management decisions to obtain maximum service at a minimum cost and then replace the structure at the end of the service life. Actions by FHWA and New York State DOT (NYSDOT) have gone a long way toward changing these policies. In 1971, the National Bridge Inspection Standards (NBIS) were established to promote a national bridge inspection and inventory program. NBIS set national policy regarding bridge inspection frequency, personnel qualifications, and reporting procedures. NYSDOT established a Uniform Code of Bridge Inspection, which prescribed bridge inspection standards and evaluation of all publicly owned bridges in the state. In 1988, NYSDOT was given the responsibility of inspecting all publicly owned bridges in the state.

The NYSDOT bridge inspection program identifies 30 to 40 items per span that are inspected and rated using a 1 to 7 numeric rating system: 1 indicates that the item is totally deteriorated or failed and 7 indicates that the item is in new condition or has no deterioration. The Brooklyn Bridge had a rating of 2.79 in 1989 and 2.88 in 1998 (a 3 rating is serious deterioration or not functioning as originally designed). The Manhattan Bridge had a rating of 3.23 (lower roadway) and 1.81 (upper roadway) in 1989 and 3.42 and 3.64, respectively, in 1998 (only slightly better than a 3 rating). Williamsburg Bridge had a rating of 1.88 in 1989 and 2.37 in 1998. The Queensboro Bridge had a rating of 2.65 (lower roadway) and 1.62 (upper roadway) in 1989 and 4.86 and 4.39, respectively, in 1998 (a 5 rating is minor deterioration and functioning as originally planned). Although, the ratings for the bridges have improved from 1989 to 1998, significant problems persist.

Rehabilitation

The East River bridges rehabilitation project began in 1980 and is currently scheduled to continue through 2008. Upon completion, the rehabilitation program costs are expected to be more than $2.5 billion. This program has been a cooperative effort between NYSDOT and the federal aid programs, which have funded a large part of these costs. Tables 14 through 17 present a summary of the major rehabilitation projects and their respective costs for each bridge.

A major focus of the Brooklyn Bridge rehabilitation program was an early 1980's inspection that revealed that the entire stay and suspension system was corroded and required replacement. The main suspension cables were found to be in good condition.

Table 14. Major rehabilitation projects for the Brooklyn Bridge.

REHABILITATION ACTIVITY	**COST ($ x million)**
Rehabilitate cables in anchorage and replace suspenders; rehabilitate balance of promenade and construct bikeway and new pedestrian ramp. (1988)	22.68
Rehabilitate and paint York, Main, William, and Prospect Streets structures and main bridge roadway deck overlay. (1988)	6.21
Replace suspenders, cable posts, stay cables, hand-rope necklace lights, main cable wrapping; paint suspended spans. (1991)	53.57

Table 14. Major rehabilitation projects for the Brooklyn Bridge (continued).

REHABILITATION ACTIVITY	**COST** **($ x million)**
Rehabilitate ramps D and H in Manhattan, permanent improvement of promenade at Manhattan approach. (1993)	17.92
Rehabilitate floor systems, stiffening trusses, roadways of suspended spans and Franklin Square trusses. (1994)	66.30
Rehabilitate ramp D and widen along FDR Drive. (1996)	11.39
Arch supports for Franklin Square truss structure.	7.50
Replacement of suspended span deck. (in progress)	33.80
Resurfacing of the main spans. (1998)	6.67
Rehabilitate and paint Brooklyn approach and ramps; rehabilitate and paint Manhattan approaches and ramps. (in progress)	115.00
TOTAL (All contracts, not summation of table)	**$351.26**

The Williamsburg Bridge was temporarily closed to all traffic (eight automobile lanes and two subway tracks) in 1988 following an inspection until an estimate of the structural safety could be completed. Although an original recommendation for replacement was made, an expert task force appointed by the mayor determined that rehabilitation was the best course of action. The technical decision for rehabilitation was predicated upon the determination that the four suspension cables could be saved with a complete re-wrapping, preceded by wedging, cleaning, oiling, re-splicing of broken wires, and re-anchoring of broken strands.

Table 15. Major rehabilitation projects for the Williamsburg Bridge.

REHABILITATION ACTIVITY	**COST** **($ x million)**
Replace main-span outer roadway. (1983)	11.20
Replace one-third of suspenders. (1984)	3.20
Component repairs of flag conditions on the north outer roadway and north inner roadway. (1994)	4.12
Rehabilitate main cables and new suspender system. (1996)	74.00
Demolish DOS and DOH buildings, replace entire south outer roadway approach structures, rehabilitate south outer roadway deck and south inner roadway deck of the main bridge, and replace south inner roadway substructure of the approaches. (1998)	155.00
Portion of Contract #6 BMT track structure work transferred to ongoing Contract #5 south approach roadway reconstruction work. (1998)	65.00
Paint main and intermediate towers. (in progress)	7.40
Reconstruct BMT subway structure; install new signals, tracks, and communications system. (in progress)	130.00
Miscellaneous rehabilitation work: tower rehabilitation, replace bearings travelers, architectural work, painting, suspender adjustment, tower jacking, construction of colonnades.	73.50
Replace north approach structures (Manhattan/Brooklyn), rehabilitate north half of bridge and paint the main bridge. (in progress)	202.80
TOTAL (All contracts, not summation of table)	**$748.51**

The Manhattan Bridge has had extensive and innovative work performed on the bridge anchorages in order to rehabilitate the cable eyebars, and dehumidification is planned for the anchorage chambers to reduce the risk of reoccurring corrosion problems.

Table 16. Major rehabilitation projects for the Manhattan Bridge.

REHABILITATION ACTIVITY	**COST**
	($ x million)
Repair floor beams. (1982)	0.70
Replace inspection platforms, subway stringers on approach spans. (1992)	6.30
Install anti-torsional fix (side spans) and rehabilitate upper roadway decks on approach spans on east side. (1989)	40.30
Eyebar rehabilitation – Manhattan anchorage chamber "C". (1992)	12.20
Replacement of maintenance platform in the suspended span. (1996)	4.27
Reconstruct maintenance inspection platforms, repairs to structural steel support system of lower roadway for future functioning of roadway as a detour during later construction contracts. (1997)	23.50
Install anti-torsional fix on west side (main and side spans); west upper roadway decks; walkway rehabilitation; rehabilitate cables in both anchorage chambers; dehumidify Brooklyn and Manhattan anchorages. (1993)	96.90
Removal of existing suspender ropes and sockets in the suspended spans; removal of existing main cable wrapping; cleaning of main cables; application of new protective paste on main cables; replacement of new main cable wrapping; reinforcement of truss verticals and gusset plates. (1987)	70.00
Interim steel rehabilitation and painting cable and saddle repairs on lower roadway; cable and suspender repairs, removal of parking deck, paint entire west side, all four cables. (1997)	124.10
Stiffening of main span; reconstruction of north subway framing; reconstruction of north upper roadway deck at suspended spans; rehabilitation of north approach spans; installation of Intelligent Vehicle Highway System for north and south upper roadways as well as for lower roadway. (in progress)	201.00
Rehabilitation of lower roadway. (in progress)	17.00
TOTAL (All contracts, not summation of table)	**$702.20**

Corrosion has severely reduced the floorbeam sections of the Queensboro Bridge. Although no longer functioning as designed, the bridge is still adequate for current loads. The roadways have been replaced with concrete-filled steel gratings and repainting with full lead removal has been completed in certain areas, while others are pending.

Table 17. Major rehabilitation projects for the Queensboro Bridge.

REHABILITATION ACTIVITY	**COST** **($ x million)**
Repair lower outer roadways reconstruct two ramps in lower Queens. (1984)	18.80
Reconstruct south upper roadway, replace inspection platforms, lighting. (1986)	31.50
Interim rehabilitation contracts A, B, & C (repairs to lower deck and main bridge approaches). (1985)	2.80
Interim rehabilitation, contract D (repairs to lower deck, main bridge, and new median barrier). (1988)	3.00
Reconstruct north upper roadway and Queens approaches A & B, rehabilitate bearings at Queens approach. (1989)	50.00
Reconstruct ramps C & D (Queensboro only, not Thompson Ave.). (1988)	10.40
Rehabilitate bridge bearings, pier tops, and truss lower chords. (1989)	18.00
Rehabilitate Queens approach trusses, lower inner roadways on the main span and approaches. (1996)	172.00
Rehabilitate lower outer roadways main span and approaches, (bikeway) cleaning and painting. (in progress)	161.40
Cleaning and painting main bridge upper trusses. (in progress)	48.50
TOTAL (All contracts, not summation of table)	**$516.40**

Preventive Maintenance

The East River bridges are part of the Preventive Maintenance Management System implemented by the New York City Department of Transportation in the early 1990s. The preventive maintenance system includes such action items as: (1) debris removal; (2) sweeping; (3) cleaning of drain system, pier and abutment tops, open gratings, and expansion joints; (4) washing of salt splash zones; (5) painting of the steel; (6) spot painting of the steel; (7) painting of salt splash zones; (8) patching of sidewalk; (9) sealing of cracks; (10) electrical maintenance; (11) oiling of mechanical parts; and (12) replacing of wearing surfaces. Painting and spot painting of the steel represents almost half of the overall cost of the program.

The elements directly related to corrosion control are washing of the deck and salt splash zones to remove deicing salts (once a year), painting of the steel (once every 8 years), spot painting of the steel, and painting of salt splash zones (once every 4 years). The painting cycle is shown in figure 12.

In addition to the rehabilitation costs described above, the preventive maintenance program expenditures for the Brooklyn Bridge for the 10-year period from 1999 to 2008 are estimated at approximately $6.2 million. The painting contracts for the 12-year period starting in 2000 are estimated at $48 million. The Brooklyn Bridge is also the site for an experimental anti-icing system, which uses calcium magnesium acetate (CMA) spray to prevent ice formation on the bridge deck. There are presently two manually operated systems. One system is designed to cover the three-lane width of the Brooklyn-bound roadway by two nozzle lines running on either side. The other system sprays the CMA from one side of the Manhattan-bound roadway. The costs of operating the two systems are estimated at approximately $300,000 per year for the 2000 to 2003 period.

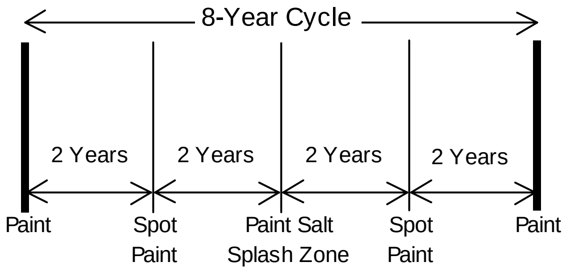

Figure 12. Preventive maintenance painting cycle.

Although some of the costs are contained in the rehabilitation contracts discussed above, it is estimated that $12 million is being spent each year for preventive maintenance programs on the four East River bridges.

Management

The magnitude of the rehabilitation and preventive maintenance investment continues to draw attention to the operation and management of bridges in the future. The management of the East River bridges has been alternatively under New York city, state, and, since 1989, once again, city management. Rehabilitation has been funded from federal, state, and city sources and contracts managed by both city and state, with federal participation. Maintenance has been traditionally funded from local taxes. The lack of dedicated funding is considered the primary cause for the decline of the bridge condition.

The question of obtaining optimal bridge service for the funds spent has gained worldwide attention over the past two decades. During this period, the federal government has introduced bridge management programs and life-cycle cost analysis. A number of FHWA and Transportation Research Board (TRB) publications contain guidelines on bridge management in the United States.

One of the continuing management concerns is in maintenance and inspection staffing to accomplish the necessary work load. This has been a recommendation in various reports on the condition of the East River bridges. To that end, FHWA has made funding available to increase staff levels for maintenance personnel.

Summary of Case Study 2

The rehabilitation of the East River bridges is an important example of the bridge management challenges that face major metropolitan centers. It is a prime example of the cooperation required by federal, state, and city government agencies in addressing funding requirements and managing of critical infrastructure components. The result of the learned experiences for the East River bridges should be to eliminate the past error of designating maintenance as an “expense” item to be deferred until the “capital” funded replacement becomes inevitable. In addition, the East River bridges highlight the need for proper inspection prior to critical stage rehabilitation (resulting in temporary closure) or replacement as the only alternatives.

Case Study 3. Cathodic Protection of Historic Oregon Bridges(49-50)

Introduction

Oregon’s coastal highway includes more than 120 bridges, of which 12 are historic structures. The majority of these bridges are reinforced-concrete design and continued salt ingress from wind, fog, and spray have caused significant chloride-induced corrosion deterioration of the reinforced-concrete components. The major bridges were designed by Conde B. McCullough and built in the late 1920s and 1930s. The 1987 replacement of the Alsea Bay Bridge resulted in public protest at the loss of a landmark bridge.

The major issues of public funding and safety are similar to those faced by other DOTs. The issue of preservation of historic structures further complicates the decision-making process. Oregon DOT has selected CP to mitigate corrosion in an attempt to prevent further deterioration of the bridges.

Cathodic Protection Options

In 1985, Oregon DOT installed conductive carbon paint over platinum-niobium wire anodes on the north approach spans of the Yaquina Bay Bridge. This is the oldest carbon paint anode system still in service.

Since 1988, Oregon DOT has installed impressed-current thermal-sprayed zinc anode CP systems on five bridges: Cape Creek, Yaquina Bay, Depoe Bay, Big Creek, and Cape Perpetua. The total installed systems exceed 40,000 m^2 (430,000 ft^2), with an average cost of $151 per m^2 ($14 per ft^2) (in 1997 dollars). Approximately half of the cost is for the thermal-sprayed zinc and half for preparation of the concrete surface (including concrete repair). There are plans to install the thermal-sprayed zinc system on three additional bridges.

In addition to the thermal-sprayed zinc CP systems, demonstration projects are ongoing using non-consumable thermal-sprayed catalyzed titanium anodes on the Depoe Bay Bridge and thermal-sprayed Al-Zn-In and zinc hydrogel anodes on the Cape Perpetua Bridge. Although the thermal-sprayed zinc anode was originally designed as a sacrificial anode, Oregon DOT also utilizes this anode in conjunction with impressed-current CP systems. In many applications where the moisture content in the concrete is low, the concrete resistivity is too great for the application of sacrificial anode CP.

The zinc hydrogel anode is a 0.25-mm- (0.01-in-) thick zinc foil, backed with a conductive, pressure-sensitive adhesive. The adhesive is a 0.75-mm- (0.03-in-) thick hygroscopic acrylate polymer containing sulfonic acid. Application of the hydrogel is relatively simple and the concrete preparation is similar to that required for thermal spraying. The zinc hydrogel foil comes in rolls 250 mm (10 in) wide. The backing is removed, exposing the adhesive, and the foil is pressed on the concrete. The edges of the foil are sealed with silicon rubber. Painting of the foil backing is optional.

The Al-Zn-In and the titanium materials are thermal sprayed onto the concrete surface in the same way as the zinc thermal-sprayed anode. The Al-Zn-In anode is an aluminum alloy anode and has demonstrated in both field and laboratory testing its ability to provide effective levels of CP. The titanium is catalyzed following thermal spraying using a brush- or spray-applied cobalt nitrate amine complex in a pH 3.47 aqueous solution. A cost premium for installing the catalyzed titanium anode was 18 percent more than that for the thermal-sprayed zinc.

Moisture substantially reduces the anode resistance of the thermal-sprayed anodes by increasing conductivity of the electrolyte. The application of humectants, salts that attract water, is one way of increasing moisture content at the anode/concrete interface. Lithium bromide and lithium nitrate are two promising humectants that were tested in trials at the Yaquina Bay Bridge.

Cathodic Protection System Performance

CP system performance is based primarily on the current output of the anodes, circuit resistance of the anode rebar, and bond strength of the anode to the concrete. Based on the examination of current output for sacrificial anodes on the Cape Perpetua Half-Viaduct, the thermal-sprayed zinc performed best, followed by the zinc hydrogel and the thermal-sprayed Al-Zn-In alloy, with the Al-Zn-In alloy at or slightly below the desired current density for corrosion protection. It should be noted that development of the Al-Zn-In anode has continued with promising results. Humectant-treated anodes on the Yaquina Bay Bridge significantly decreased circuit resistance over the 90-day trial.

The early conductive carbon paint over platinum-niobium wire anode system continues to function well after 15 years of service, although it shows signs of aging. The thermal-sprayed zinc anodes have been in operation for 12 years. An estimated service life of 25 years is predicted for the thermal-sprayed zinc anodes at the operating current densities for these bridges.

Summary of Case Study 3

Several of the trial CP options appear promising, including the use of humectants to reduce anode resistance for the thermal-sprayed anode systems. In addition, the catalyzed titanium thermal-sprayed anode may be effective in maintaining a low circuit resistance and extending the life of the CP system.

Since 1985, the use of CP to mitigate corrosion and extend the life of critical bridge structures has been shown to be successful. Most of the structures have been protected utilizing thermally sprayed zinc anodes in conjunction with impressed-current CP. Without the use of CP to stop ongoing corrosion, many of these historical structures would have been (or would soon be) lost. With properly maintained CP systems, the service lives of these bridges can be significantly extended.

LIFE-CYCLE COST ANALYSIS FOR BRIDGE DECKS

When it comes to designing a reinforced-concrete bridge, bridge engineers have a variety of options to achieve the service requirements. There is a general understanding that comparing the options on the “initial cost” basis is not a good predictor of life-cycle costs, i.e., corrosion maintenance costs are also important. Past economic analyses have treated life-cycle costing in different ways, with only a few estimating indirect costs, such as user costs associated with disruption caused by deteriorating deck surfaces and maintenance, repair, and replacement. (See references 12, 18, 51, and 52.) For a bridge that carries a high volume of traffic, indirect (user) costs can be substantially larger than materials and labor costs for bridge repair/rehabilitation. This means that to capture the total economic impact of the project, the analysis must include these indirect costs. The best way to compare bridges with different rebar materials and different corrosion maintenance practices is on the basis of annualized value (AV), which represents discounted cash outflows related to both the construction/maintenance costs and user costs associated with these activities (see Appendix B, “Economic Analysis Methods”).

The following sections demonstrate this approach using direct and indirect cost calculations for several bridge deck designs [different rebar materials: black steel rebar, epoxy-coated rebar (see figure 13), and stainless steel rebar (see figure 14)]. The analysis focuses on decks rather than other bridge elements because the corrosion-related problems are most obvious on the deck, the most visible part of the bridge. In addition, only new construction alternatives are examined. Even for new construction, several alternatives such as inhibitors or high-performance silica fume concrete were not examined. Also not examined were several rehabilitation options, such as CP and electrochemical chloride removal. This does not mean that these options are not viable, certainly CP has proven to be success in mitigating ongoing corrosion on bridge decks and is an effective long-term rehabilitation alternative to replacement. The life-cycle costs given here are an example of how life-cycle costing can be accomplished.

The values used in this example, while being realistic estimates and originating from referenced sources, are meant to illustrate the relative magnitude of the components of the total economic impact of bridge construction and maintenance. The readers are encouraged to view this data as an example and are encouraged to input their own data based on their experience to evaluate life-cycle costs.

Figure 13. Epoxy-coated rebar deck construction.

Figure 14. Stainless steel-clad rebar deck construction.

Approach for Life-Cycle Cost Analysis

“Average” Bridge Scenario

This analysis uses an “average” reinforced-concrete bridge, based on the National Bridge Inventory (NBI) data (see table 18). The “average” bridge deck has a surface area of 583 m^2 (6,280 ft^2), two lanes in each direction, a length of 36.9 m (121 ft) and a width of 15.8 m (52 ft), and average daily traffic (ADT) of 24,000 vehicles.

Table 18. “Average” bridge deck parameters used in the analysis.

ITEM	VALUE	SOURCE
Average number of lanes	2.1	NBI
Average daily traffic (ADT)	24,000 vehicles	NBI
Average deck area	583 m^2 (6283 ft^2)	NBI
Average operating load rating	41.5 metric tons	NBI
Average bridge deck thickness	190.5 mm (7.5 in)	Estimated
Average concrete cover over top reinforcing bar mat	51 to 64 mm (2 to 2.5 in)	Estimated
Average bottom cover	25.4 mm (1 in)	Estimated

Design Options

The bridge is located in a moderate environment. The analysis focuses on three rebar design configurations:

1. Both rebar mats are black steel rebar.
2. Top rebar mat is epoxy-coated rebar – bottom rebar mat is black steel rebar.
3. Both rebar mats are solid stainless steel rebar.

The purpose of selecting these rebar configurations was to provide a range of possible conditions for the economic analysis. It is not proposed that these selections represent the most common or the only configurations worthy of consideration. The top-mat epoxy-coated rebar design was selected because there were deterioration models available for it from the literature. For all scenarios, the structural concrete quality is assumed to be the same. Also, it is assumed that, for all scenarios, the labor hours, the cost of labor, the cost of material, and the cost of equipment are the same.

Construction/Repair/Rehabilitation Options

For this analysis, four construction-repair-rehabilitation options were selected. For all the scenarios, the same type of maintenance sequence was applied.

Routine Maintenance

Annual routine maintenance costs are estimated at $1,000 per year. No user cost is associated with annual maintenance activities. These costs include any maintenance required on the bridge, including miscellaneous repair patching as the deck ages, but do not include scheduled maintenance for significant patching of deteriorating concrete deck.

Repair/Patch

Repair/patch is scheduled maintenance when the deck surface can no longer be maintained by "routine maintenance." Patching costs are estimated at \$90 per m^2 (\$8 per ft^2).

It is assumed that patching can be done on weekends, thus avoiding user costs. However, the worsening deck surface condition affects driving speed, which generates user cost and is accounted for in the analysis.

Rehabilitation

Latex-modified concrete (LMC) overlay is used for rehabilitation. The cost of this overlay is estimated to be \$170 per m^2 (\$16 per ft^2) with a service life of 18 years (see table 8). For the average bridge used in this study (583 m^2), the total cost of the overlay is \$99,100.

It is assumed that the rehabilitation takes 63 days, of which 45 days have user cost due to queuing.

Original Construction-Replacement

The original black steel rebar deck cost is assumed to be \$484 per m^2, which gives a total cost of \$282,200 for the 583-m^2 deck (table 21 gives the cost for each construction option). It was assumed that construction takes 135 days, of which 96 days have peak periods. User cost due to queuing for the 96 days of the construction is included.

After one rehabilitation cycle, the deck is replaced. User cost is estimated for the time period needed to remove the deck (90 days, 64 of which have user cost), but not for building another new deck.

Concrete Deterioration Model

The rate of deterioration with time is required to perform a life-cycle cost using the "information-based" maintenance practice described below. McDonald et al. published estimated times as a function of delamination for a black steel rebar deck and the top-mat epoxy-coated rebar deck (based on laboratory experiments), which is used as a foundation for the deterioration models (see figure 15).(18) The deterioration models also consider the impact of repairing cracks in the concrete. Repairs to the cracks tend to significantly extend the time to delamination in the early life of the deck, but after 10 percent delamination, the effect becomes much less.

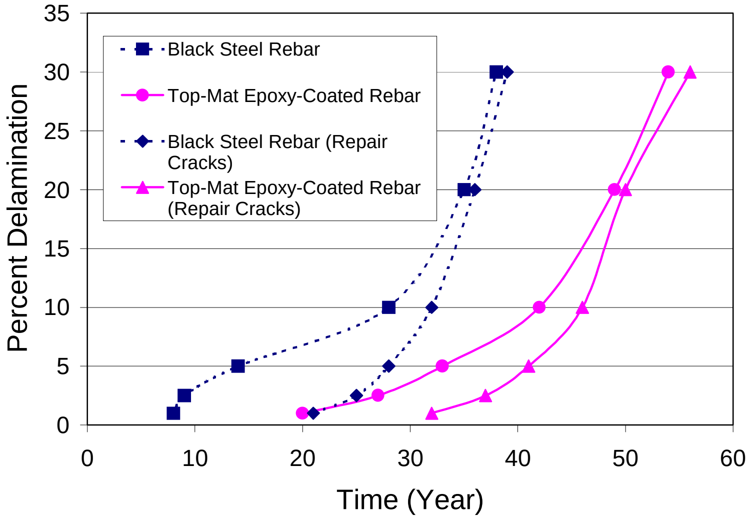

Figure 15. Deterioration models for the black steel rebar and top-mat epoxy-coated rebar deck.

Corrosion Management Alternatives

The approach employs two corrosion management scenarios:

1. Information-based practice.
2. Experience-based practice.

Corrosion management activities and their schedule determine the service life for each scenario (service life ends when the deck is replaced). In order to reduce the number of variables, it was assumed that for both scenarios, the same actions incurred the same costs (both agency and user costs).

Information-Based Practice

In this scenario, corrosion management decisions are based on the condition of the deck estimated through regular inspections. Because decisions are made based on known deck condition, maintenance is performed on a timely basis, thus ensuring optimum maintenance scheduling and maximum service life to the deck. In this scenario, the deterioration rate dictates the corrosion management activities, which are performed on the following schedule:

1. Repair/patching maintenance of the deck becomes a significant activity when 2.5 percent of the deck surface area exhibits delamination and spalling, which is assumed to affect the traffic flow. It is assumed that annual maintenance consisting of repair/patching of the deck continues until 10 percent of the deck surface has been affected. For the information-based maintenance, two maintenance scenarios are examined: (1) cracks in the deck are repaired as they appear and (2) no cracks in the concrete deck are repaired. Whether cracks are repaired or not defines the choice of the deterioration model (see figure 15).
2. It is assumed that when 10 percent of the deck surface area is delaminated, the deck is rehabilitated with a latex-modified concrete (LMC) overlay.
3. The life of this deck ends at the end of the service life of the rehabilitation overlay. For the purposes of this analysis, the service life is estimated at 18 years (table 8). At this time, the deck is replaced.

It is obvious that other milestones (percent delamination) could have been selected for determining repair/rehabilitation options. The selections made are provided as an example only and are not meant to apply to any specific bridge.

Experience-Based Practice

In this scenario, corrosion management is based on engineering experience and deck maintenance is scheduled based on the experience from similar bridges. This is often the case when there are several similarly designed bridges in a bridge inventory. For experience-based maintenance, activities are scheduled based on a specific time (deck life), not on the measured condition of the bridge deck, because regular monitoring of the bridge deck is not performed. As experience dictates different maintenance for black steel rebar decks versus epoxy-coated rebar decks, the maintenance follows the schedule(s) described below (recall that in the scenario used in this analysis only the top mat of rebar is epoxy-coated). It should also be noted that, other than for a few substructures of bridges in the Florida Keys exposed to severe marine environments, significant deterioration of a bridge deck made with epoxy-coated rebars has not been observed; therefore, the "experience-based" maintenance schedule for epoxy-coated rebar is "estimated" and not based on actual experience.

1. Repair/patching maintenance becomes a significant activity after 10 years for black steel rebar deck and 40 years for a top-mat epoxy-coated rebar deck.
2. Repair/patching continues on an annual basis until year 20 for black steel rebar deck and year 50 for a top-mat epoxy-coated rebar deck, at which time the deck is rehabilitated with an LMC overlay.
3. At the end of the 18-year service life of the rehabilitation overlay, the deck is replaced (year 38 for black steel rebar deck and year 68 for top-mat epoxy-coated rebar deck). For the purposes of this analysis, the rehabilitation overlay life is assumed to be the same as for the information-based scenario.

The schedule selected above is based on information discussed in the literature and by the industry experts; it is meant to be representative of "reasonable" values and users are encouraged to input their own experience and data.

Scheduling of Maintenance

Tables 19 and 20 summarize the maintenance scheduling for the scenarios involving black steel and epoxy-coated rebar. For the information-based maintenance, the year in which action is taken is governed by the deterioration model selected (see figure 15). The action is based on monitoring the deck, determining the extent of the damaged area, and acting after a specific area of the deck is affected by delamination/spalling. The year in which action is taken for the experience-based maintenance is based on the operator's historic operating experience and follows a preset schedule.

For the stainless steel rebar scenario, it is assumed that the deck does not show any corrosion damage. It is understood that the concrete surface wears down due to traffic abrasion, and worsened surface traction conditions require some treatment of the deck surface at around 50 years of age. An LMC overlay is applied at year 50 and 85 and is assumed to have a 35-year life (no corrosion of the rebar). This gives a service life of 120 years.

Table 19. Scenarios of corrosion management for decks with black steel rebar.

ACTIONS	**INFORMATION-BASED (NO REPAIR OF CRACKS)**		**INFORMATION-BASED (REPAIR CRACKS)**		**EXPERIENCE-BASED**
	% of deck damaged	Year for action	% of deck damaged	Year for action	Year for action
Patching starts	2.5	9	2.5	25	10
Rehabilitation overlay	10	28	10	32	20
Deck replacement	NA	46	NA	50	38

NA = not applicable

Table 20. Scenarios of corrosion management for decks with epoxy-coated rebar.

ACTIONS	**INFORMATION-BASED (NO CRACK REPAIR)**		**INFORMATION-BASED (CRACK REPAIR)**		**EXPERIENCE-BASED**
	% of deck damaged	Year for action	% of deck damaged	Year for action	Year for action
Patching starts	2.5	27	2.5	37	40
Rehabilitation overlay	10	42	10	46	50
Deck replacement	NA	60	NA	64	68

NA = not applicable

Cost Summary

Cost details are based on the previous discussion of corrosion control methods (tables 7 and 8). The initial cost of the black steel rebar deck is estimated to be \$484 per m^2 (\$45 per ft^2) or \$282,000 for the "average" deck. Cost for the top-mat epoxy-coated rebar deck and two-mat solid stainless steel rebar deck are assumed to be 0.6 percent and 18.6 percent greater, respectively, than the costs for a black steel rebar deck. (The 0.6 percent increase for the top-mat epoxy-coated rebar deck is assumed to be 50 percent of the increase given in table 7 for a two-mat epoxy-coated rebar deck.) Table 21 summarizes these costs and life expectancies for this analysis.

Table 21. Summary of costs used in the economic analysis.

COST OPTION	**COST PER DECK AREA**	**TOTAL COST**	**LIFE EXPECTANCY (EXPERIENCE)**
	\$/m^2	\$	Years
New Construction - Black steel (baseline)	484	282,200	10*
New Construction - Top layer epoxy-coated rebar	487	283,900	40*
New Construction – 2-layer solid SS rebar	574	334,600	120
Patching (Bituminous)	90	-	1
LMC Overlay	170	99,100	18**
Old Deck Removal***	240	139,900	NA****

*Time to scheduled patching.
**35-year life is projected for stainless steel rebar system with no corrosion (18-year life is for decks with ongoing corrosion).
***Assumed to be approximately 50% of new construction.
****NA = not applicable

Annualized Cost Analysis

General Procedure for Cost Analysis

The analysis consists of the following steps:

1. The variables in each scenario include the rebar material used in construction, the series of corrosion maintenance actions, and the schedule of each action.
2. The direct and user costs of each item are calculated, establishing the cash flow for each deck design/maintenance scenario combination. The cash flow for each scenario includes the direct cost of materials and labor, as well as the user costs associated with any corrosion management activities that interrupt traffic flow.
3. Using these cash flows, the present value (PV) is calculated. From the PV, the annualized value (AV) is calculated for the service life of the scenario.

Critical stages in the life cycle of a deck are summarized in table 22. The life-cycle costs are characterized by their AV, which serves as the basis for comparison. It is assumed that there is an existing bridge that is having its deck replaced

Table 22. Life cycle of scenarios.

ONE LIFE CYCLE FOR A SCENARIO		
YEAR	**DIRECT COST**	**USER COST**
Year 0	Total initial investment for constructing a new deck. The removal cost for the old deck is not included.	User cost associated with the construction of a new deck. User cost associated with the removal of an old deck is not included.
Service years	Maintenance, repair, rehabilitation.	User cost generated by worsening deck conditions and by lane closure required by any maintenance, repair, or rehabilitation action.
Last year	Cost of deck removal. Cost of new deck is not included.	User cost associated with removal of the deck. User cost generated by construction of new deck is not included.

Direct Costs

Direct costs of the corrosion management activities include material, labor, and equipment cost. The cost of traffic maintenance, if necessary, is added separately, unless otherwise noted. It was assumed that the costs for the same actions are the same for all studied scenarios.

In general, direct cost of one-time expenditures, such as the new construction, routine maintenance, rehabilitation overlays, and removal of old deck, were calculated the same way:

Direct cost = {unit cost of action} * {area where applied}

The cost of the annual maintenance was treated as a series of uniform annual payments:

Annual maintenance cost = {annual maintenance cost per area} * {deck surface area}

The corrosion management schedule determines the direct cost cash flow. The calculations of the present value (PV) of the cash outflows are presented in the following sections. (Initial investment happens in the "present," therefore no discounting is necessary.)

Annual maintenance (AM) is constant throughout the life cycle of the scenarios. This annual value is calculated back to the present by the following formula:

$$PDV\{AM\} = AM * [1 - (1 + i)^{-N}] / i$$

where

PDV = present discounted value, $

AM = cost of annual maintenance, $/year

N = length of the deck's service life, years

i = interest rate, %

The cost of patching grows annually at a constant rate (g); for the calculation of the PV of patching, a modified interest rate needs to be calculated by the following formula:

$$i_0 = (i - g) / (1 + g) \qquad \text{and} \qquad i > g$$

where

i_0 = is the modified interest rate, %

i = interest rate, %

g = constant annual growth rate, %

If the first payment (P_1) occurs in year 1, the present value of a cash flow that grows annually at a constant rate over n years can be calculated by the following formula:

$$PV\{P\} = [P_1 / (1 + g)] * [1 - (1 + i_0)^{-n}] / i_0$$

PV{P}, the present value of a cash flow series that starts at P1 in year 1 and grows at a constant rate g for n years when interest rate is i, is equivalent to the present value of an annuity of [P1 / (1 + g)] for n years when interest rates are i_0, where i_0 is given by the equation above.

However, the first payment for patching does not occur in year 1, but in year t. Therefore, the above formula calculated a value at year (t-1) that is equivalent to the cash flow series of patching through n years. This value needs to be discounted back to year 0 of the life cycle to determine the present discounted value of the patching:

$$PDV\{P\} = PV\{P\} * (1 + i)^{-(t-1)}$$

The PDV of one-time costs, such as the rehabilitation overlay (RH) and old deck removal (ODR), are calculated as follows:

$$PDV\{RH\} = RH * (1 + i)^{-tRH}$$
$$PDV\{ODR\} = ODR * (1 + i)^{-tODR}$$

where

RH = cost of rehabilitation overlay, $

ODR = cost of removing the old deck, $

t = year in which the cost is incurred

The PDV of the scenario is calculated as the sum of the PDVs of its cash flow:

$$PDV = I + PDV\{AM, P, RH, ODR\}$$

The annualized value (AV) of the scenarios is calculated from the PV using the following formula:

$$AV = PDV * I / [1 - (1 + i)^{-N}]$$

where

N = service life of the deck, years

Indirect (User) Cost

The only calculated social cost is that to the users. User cost is estimated as the value of time lost due to two causes: (1) the additional time it takes for drivers to reach their destination due to worsened driving conditions caused by corrosion of the bridge deck and (2) time lost due to the corrosion maintenance activities (repair, rehabilitation, and deck replacement) taking place on the deck and requiring closure of lane(s).

User cost is estimated as the product of additional travel time and the value of time. The value of time was assumed to be 50 percent of the average wage for 1998 ($8.50 per hour) for all scenarios. Traffic is characterized by the following parameters (see table 23):

Table 23. Traffic parameters.

PARAMETERS	VALUE RANGES
Average daily traffic (ADT)	20,000 – 32,000
Percent of daily traffic in peak, %	30 – 50%
Number of peak periods per direction per day	1
Length of peak period, minute	90 – 140
Discharge rate, throughput, cars / hr	Max: 2,400
Maximum waiting time before diversion, minutes	30

User Cost Due to Worsening Deck Conditions

Before the deck is repaired or rehabilitated, the condition of the deck can affect traffic flow, causing speed reduction and congestion, thus resulting in increased travel time and user cost. It was assumed that a worsening deck condition only slows the traffic flow, but does not cause congestion, which makes the user cost estimate very conservative. Other costs such as wear-and-tear on automobiles were not included in this analysis.

User Cost During Corrosion Management Activities

User cost can be incurred during corrosion maintenance activities, requiring the closure of a lane. This closure reduces the throughput (number of cars that can cross per unit time) of the bridge, causing slower traffic flow or congestion. The analysis of the user cost due to lane closure is based on a paper by Boardman and Lave,(53) which establishes the traffic speed-flow relationships for a four-lane (2 x 2) highway, and a general first come, first served queuing theory, which approximates total delay due to congestion. Two basic transportation system cases were assumed:

1. No Diversion, i.e., no alternative route is available for the drivers; thus, they must suffer the effect of the closure.

2. Diversion Available, i.e., alternative routes are available and thus drivers choose their route to minimize their travel time. It is assumed that the range of maximum tolerable delay by individual drivers is between 10 and 30 minutes.

User cost estimation for congested cases involves the following general steps:

- Estimation of the throughput of the bridge with and without lane closures to determine the maximum arrival and discharge rates for peak and off-peak periods.
- Selection of a value for the number of vehicles using the bridge at peak periods, the length of the peak period, and the discharge rate for the given average daily traffic.
- Approximation of a total delay time for cases with and without diversion using the general queuing theory.
- Estimation of the user cost per peak period as a product of total delay time and the value of time.

Lacking specific bridge data, the calculation makes some simplifying assumptions. It is assumed that the additional travel time due to congestion can be reasonably approximated by the total delay time due to queuing for the same number of vehicles. A general first come, first served queuing theory is used to estimate the total delay time for the number of cars affected by the lane closure. Discharge rate, arrival rate, and the peak traffic volume determine the total delay due to queuing per peak period.

For simplicity, in the analysis, the number of peak periods per day is fixed at two. The duration of each corrosion management activity determines how many peak periods are affected by the closure. (It was assumed that there were no peak periods during weekends.)

Then the cost of congestion to users (both with and without diversion) is estimated by multiplying the total time delay due to queuing per peak period by the number of peak periods in a day, the number of days of lane closure, and by the value of time (VoT). The PV and AV of the user costs were calculated by using the same formulas as presented for direct cost.

Results of Life-Cycle Cost Analysis

The majority of cost-benefit analyses are performed without considering the impact on the user (indirect costs). Typically, this is because the owner-operator is not willing to accept user costs as a part of the decision process (the owner-operator does not have to incur these costs). However, in examining the total impact of corrosion on the national economy, user costs often make up a portion greater than the costs incurred by the owner-operators. Therefore, the following analysis is divided into two parts, without and with user costs (direct and indirect costs, respectively). Example cost calculations are provided at the end of this section ("Sample Life-Cycle Cost Calculations").

In order to isolate the effect of rebar type and maintenance schedule, it was assumed that the same maintenance actions (annual maintenance, inspection, repair, and rehabilitation) were applied to all scenarios, except that the timing of the maintenance actions differed for the different maintenance scenarios. For example, repairing potholes on the decks takes the same amount of work-hours and the same level of traffic disruption regardless of the types of rebar and the scheduling of the activity.

The authors would again like to caution the reader that the results of the analysis presented here are meant to be an example of the economic impact of design parameters, maintenance scenarios, traffic options, and user costs. It is important to realize that the specific values were selected to be reasonable and are not typical of a particular bridge structure. The readers are encouraged to input their specific experience and data to evaluate life-cycle costs.

No User Costs

In the following analysis, no indirect (user) costs are included. This is typical of the majority of life-cycle costing performed. This analysis focuses on the effect of the rebar design and maintenance scenario.

Effect of Rebar Design (No User Cost)

Since it was assumed that the only design variable is the type of reinforcing bar used, the cost of rebar determines the initial price of the deck. Based on the initial construction cost ("sticker price"), black steel rebar design would always be the cheapest. However, black steel rebar has the shortest expected life and the highest cost of corrosion maintenance. Life-cycle cost analysis gives a more useful representation of the expenditures than does the initial cost. No user costs were included in the following analysis.

Cash flow associated with a bridge structure is characterized by high initial capital investment, the expected service life, annual maintenance costs, and repair and rehabilitation costs. Since the cash flow occurring in the future has to be discounted to the present time, a low interest rate used to discount the future cash outflows to their present value favors scenarios with low maintenance costs (e.g., stainless steel rebar deck design). A high interest rate heavily discounts future maintenance costs and tends to favor low initial cost (e.g., black steel rebar deck design). The comparisons of the deck designs/corrosion maintenance scenarios are based on the estimated annualized value (AV).

Figure 16 shows the "information-based with crack repair" maintenance scenario applied to the three rebar design cases. The first observation is that annualized costs increase with increasing interest rates. At low interest rates (below 3 percent), the stainless steel rebar design (high initial capital cost) has the lowest annualized cost; while at higher interest rates, epoxy-coated rebar has the lowest costs. Although the epoxy-coated rebar design has a lower cost than black steel rebar design at all interest rates, as the interest rate increases, black steel rebar design costs approach those for the top-mat epoxy-coated rebar design. It is interesting to note that black steel rebar and its increased maintenance has a lower annualized cost than stainless steel rebar design at interest rates greater than 5 percent.

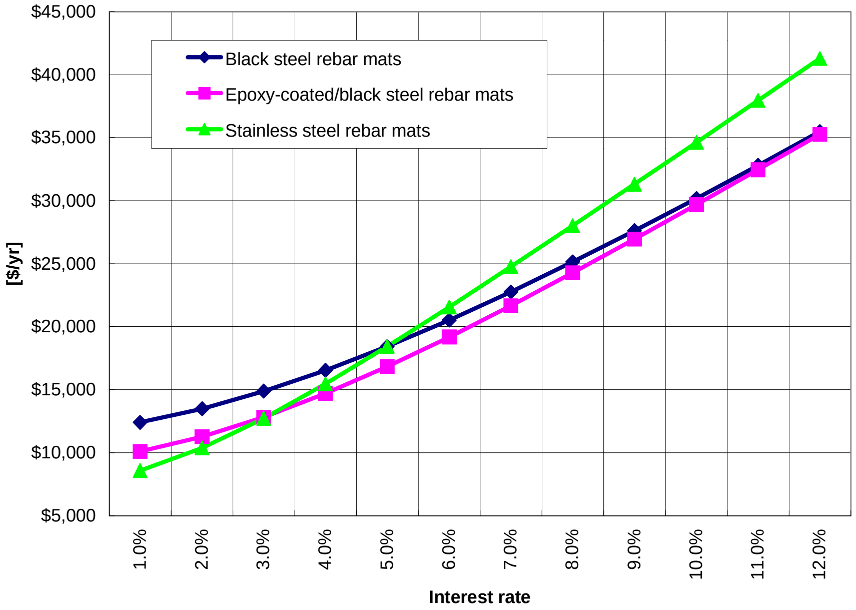

Figure 16. Effect of the interest rate on the annualized values for the "information-based with crack repair" scenario for the three bridge deck designs (no user cost).

Effect of Maintenance Approach (No User Cost)

Figures 16, 17, and 18 show the annualized costs for the three maintenance scenarios: (1) information-based with crack repair, (2) information-based with no crack repair, and (3) experienced-based, respectively. [Readers should note that the stainless steel rebar deck has only one maintenance scenario, therefore, its graph does not change in figures 16 through 18.] Figures 19 and 20 compare the three maintenance scenarios for the carbon steel and top-mat epoxy-coated rebar designs, respectively.

As maintenance is neglected (going from figure 16 to 17 to 18), more corrosion-resistant designs are preferred (the difference in the annualized cost for the black steel rebar design relative to the epoxy-coated rebar design increases). The cause of this effect can be seen by comparing figures 19 and 20. Figure 20 shows that the annualized cost for the corrosion-resistant epoxy-coated rebar design shows little sensitivity to the maintenance scenario used. In fact, table 20 shows minimal difference in the information-based (crack repair) and experienced-based scenarios.

Figure 19 shows that the more maintenance-intensive black steel rebar design was more sensitive to the maintenance scenario selected. The annualized cost decreased with the more aggressive maintenance schedule (the lowest annualized cost was achieved by the “information-based with crack repair” scenario and the “experience-based” scenario had the highest annualized cost). This observation suggests that regular inspection of the black steel rebar deck can lower cost through timely scheduling of the repairs.

In comparing the black steel rebar design to the stainless steel rebar design, the interest rate below which the stainless steel rebar design has a lower annualized cost increases with the less intensive maintenance schedule (5 percent for information-based with crack repair, 6 percent for information-based with no crack repair, and 8 percent for experience-based). Because the maintenance scenario had little effect on the epoxy-coated rebar design and only one maintenance scenario is used for the stainless steel rebar (no corrosion), the interest rate below which stainless steel has a lower annualized cost than the epoxy-coated rebar design remained constant at approximately 3 to 4 percent for the scenarios used. The annualized cost for epoxy-coated rebar design was lower than the black steel rebar design at all interest rates for each maintenance scenario.

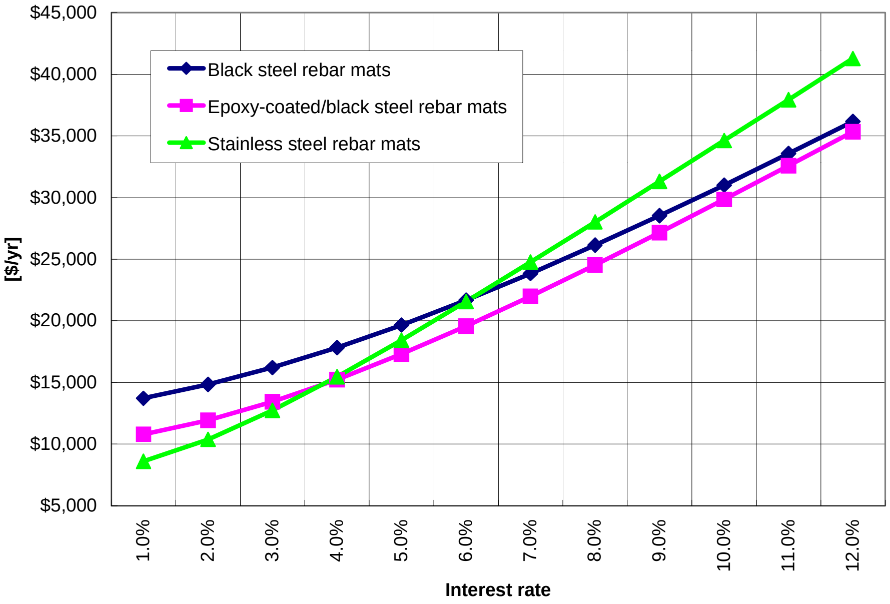

Figure 17. Effect of the interest rate on the annualized values for the “information-based, no crack repair” scenario for the three bridge deck designs (no user cost).

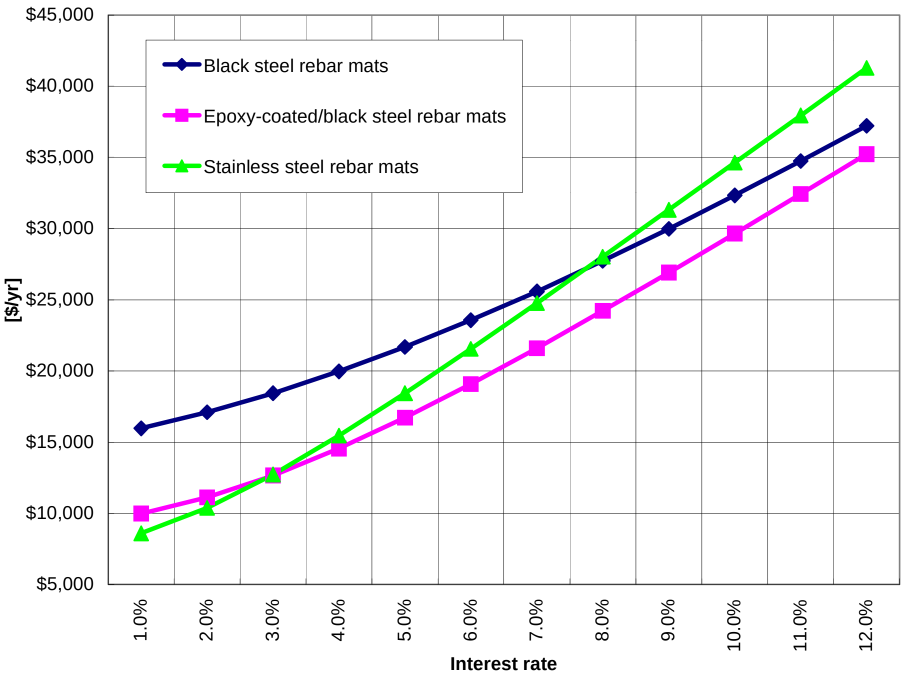

Figure 18. Effect of the interest rate on the annualized values for "experience-based" scenarios for the three bridge deck designs (no user cost).

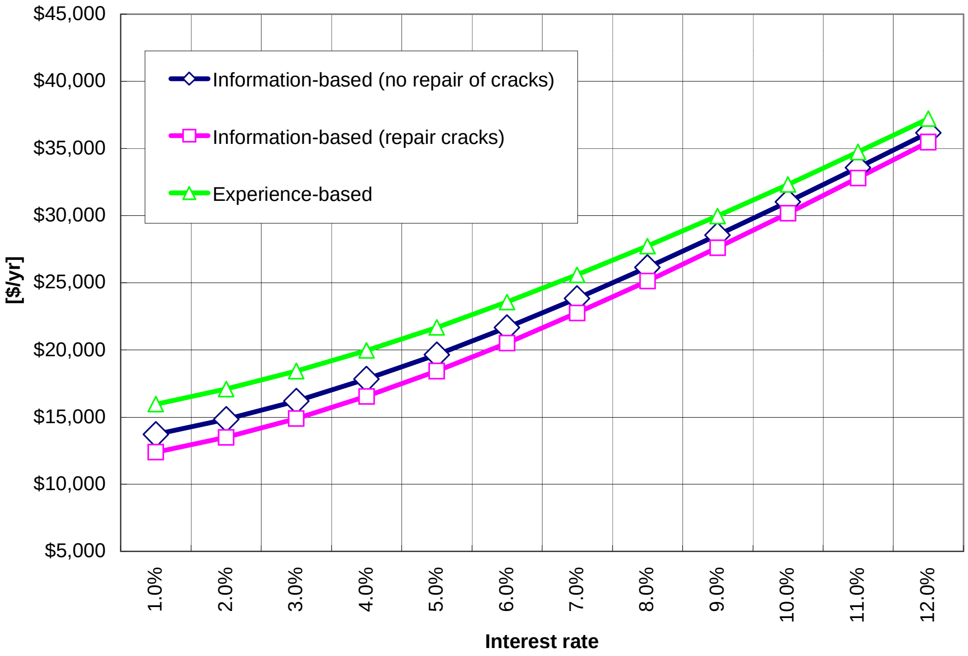

Figure 19. Effect of the interest rate on the annualized values for three corrosion maintenance scenarios and based on the black steel rebar design (no user cost).

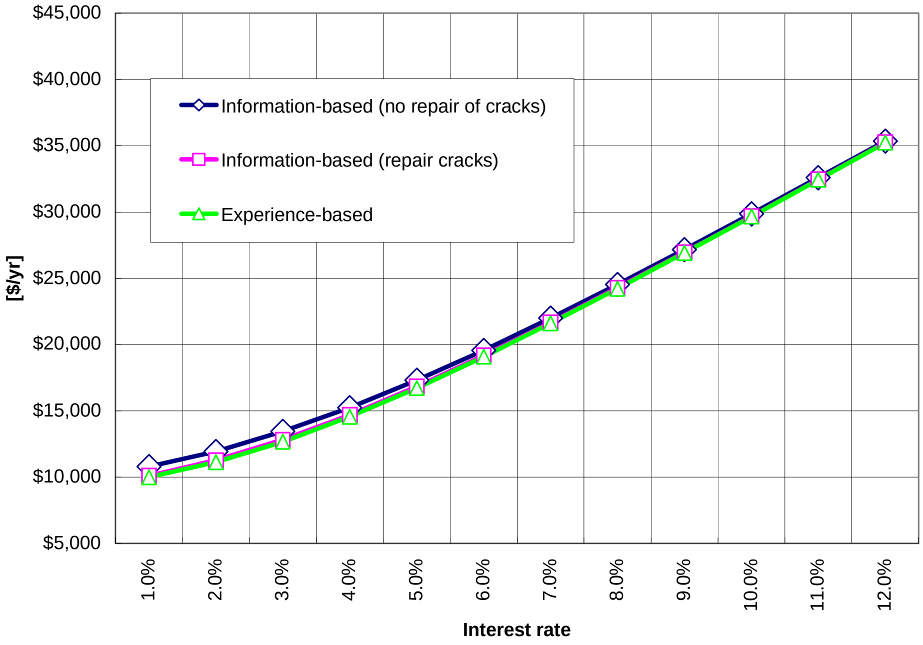

Figure 20. Effect of the interest rate on the annualized values for three corrosion maintenance scenarios and based on the top-mat epoxy-coated rebar design (no user cost).

User Costs Included

The effect of including user cost is to significantly increase the cost of both initial investment and later maintenance. The annual value of the scenarios is different for every level of user cost. The initial analyses are based on the traffic scenario that is characterized by the values given in table 24.

Table 24. Traffic scenario for user cost calculations.

Average daily traffic	24,000
% of ADT in peaks	40%
Length of peak, minutes	120
Discharge rate, cars / hr	1,700
User cost per day, $/day – no diversion	**$35,936**
Maximum waiting time, minutes	30
User cost per day, $/day – with diversion	**$29,291**

Effect of Rebar Design (User Cost Included)

Figure 21 shows that including the user cost in the analysis does not change the basic conclusions that lower interest rates favor high initial costs, while high interest rates favor high maintenance cost scenarios. For comparison, the cost for the black steel rebar design “with no user costs” is included in each figure. Including user costs increased the annualized cost of the bridge by a factor of 10 to 15.

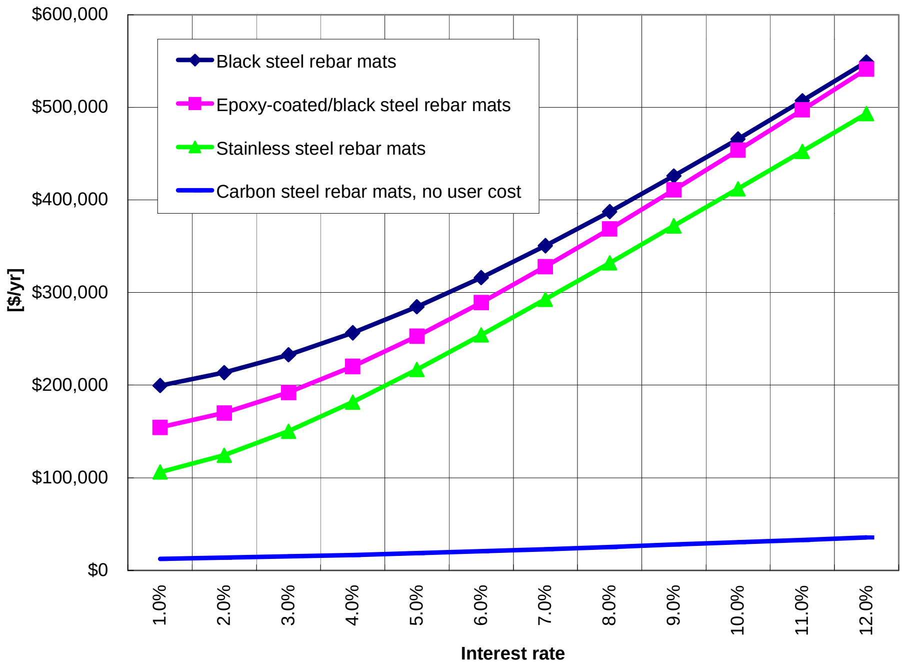

Figure 21. Effect of the interest rate on the annual values for "information-based with crack repair" maintenance scenario for the three deck designs (user cost included, no diversion).

The general relationship between the epoxy-coated rebar design and the black steel rebar design did not change from that discussed for "no user costs." The epoxy-coated rebar design had a lower annualized cost than the black steel rebar design at all interest rates. Including the user costs made the lower maintenance stainless steel rebar design also a lower cost design at all interest rates. This is because the lower maintenance stainless steel rebar design produced a significantly lower disruption of traffic than either the epoxy-coated rebar design or the black steel rebar design. Therefore, when comparing the stainless steel rebar design to the top-mat epoxy-coated rebar design, the life-cycle analysis including the user cost yields a different result (as to the most cost-effective approach) than the life-cycle analysis without the user cost.

As previously noted, bridge owner-operators do not incur user costs. Therefore, there is little incentive for the owner-operator to make decisions that include user costs in the life-cycle analysis. In addition, the capital budget of the owner-operator is a factor in deciding on the higher capital option of the stainless steel design. Both considerations influence the decision of the owner-operator in the selection of the bridge deck design.

It should be noted that in the epoxy-coated rebar design, only the top mat of reinforcing steel was coated, the bottom mat was black steel. It is generally accepted that epoxy coating on both rebar mats will extend the service life of the deck into the future and delay the time to the first required maintenance. Therefore, the two-mat epoxy-coated rebar design would become more favorable than the top-mat epoxy-coated design used in this analysis.

Effect of Maintenance Approach (User Cost Included)

Figures 21, 22, and 23 show the annualized costs, including the user cost, for the three maintenance scenarios: (1) information-based crack repair, (2) information-based no crack repair, and (3) experienced-based. The effect of the maintenance approach was similar to that described for the no user cost analysis, i.e., only the black steel rebar design was sensitive to the choice of maintenance scenario. The relationship between top-mat epoxy-coated rebar design and stainless steel rebar design remained the same for each maintenance scenario, i.e., the stainless steel rebar design is favored for all interest rates.

Including the user cost does not change the observation that the “information-based with crack repair” maintenance scenario has the lowest annualized cost (as compared to the other approaches to corrosion maintenance) for the black steel rebar design. This suggests that regular repairs to the black steel rebar deck can lower costs.

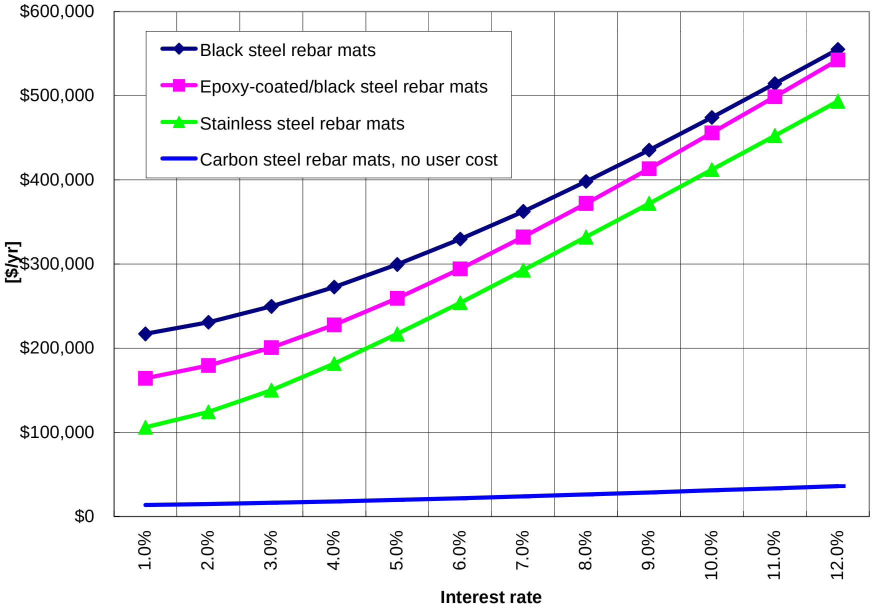

Figure 22. Effect of the interest rate on the annual values for “information-based with no crack repair” maintenance scenario for the three deck designs (user cost included, no diversion).

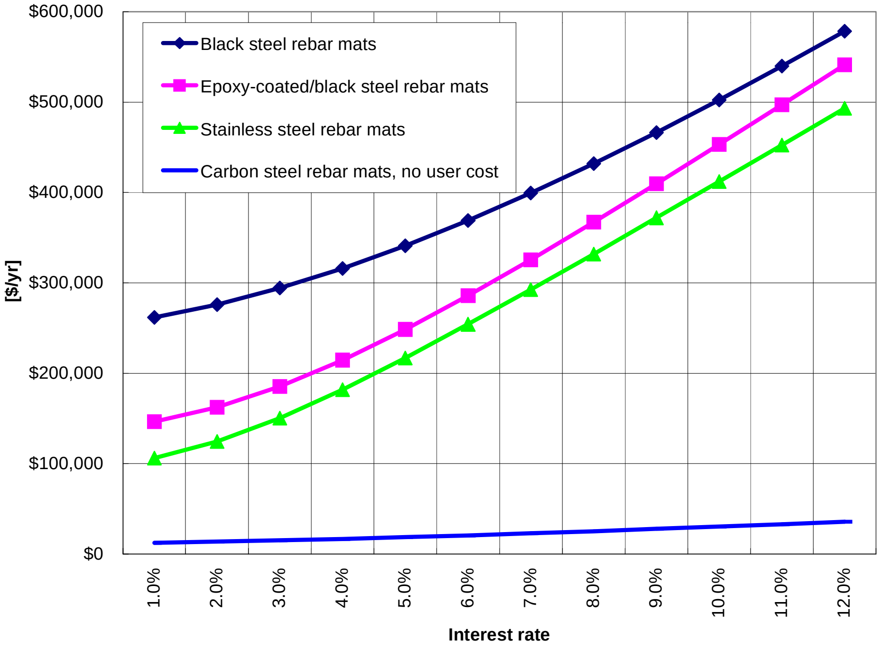

Figure 23. Effect of the interest rate on the annual values for “experience-based” maintenance scenario for the three deck designs (user cost included, no diversion).

Effect of Traffic Options

Two basic traffic options were analyzed: (1) no diversion of traffic and (2) traffic diversion available. “No diversion” means that no alternative route is available for the drivers; thus, they must suffer the effects of the closure. “Diversion available” means that alternative routes are available and, thus, drivers choose their route to minimize their travel time. It was assumed that the range of maximum tolerable delay by individual drivers is between 10 and 30 minutes. The user cost data presented above was for the “no diversion” case. No diversion is the worst case and results in the higher user cost.

Figure 24 shows the case for the “information-based with crack repair” maintenance scenario for the “diversion available” traffic option. Figure 24 also includes the no user costs for each of the rebar designs. Comparison of figures 21 and 24 provide a measure of the difference in costs between the “no diversion” and “diversion available after 30-minute delay” traffic options. The annualized costs are less when diversion is possible, but none of the trends change.

Table 25 gives the annualized costs for the “information-based with crack repair” maintenance scenario at 5 percent interest. This provides a relative comparison of the costs for: (1) no user costs, (2) user costs with no diversion, and (3) user costs with diversion available. The largest magnitude increase in cost occurs when user costs are included. However, the traffic options also have a significant effect. An annualized cost decrease of approximately $35,000 occurred when going from a “no diversion” scenario to a “diversion available” (30-minute delay) scenario. Table 25 is an example in which stainless steel rebar design went from the least desirable to the most desirable upon including user cost.

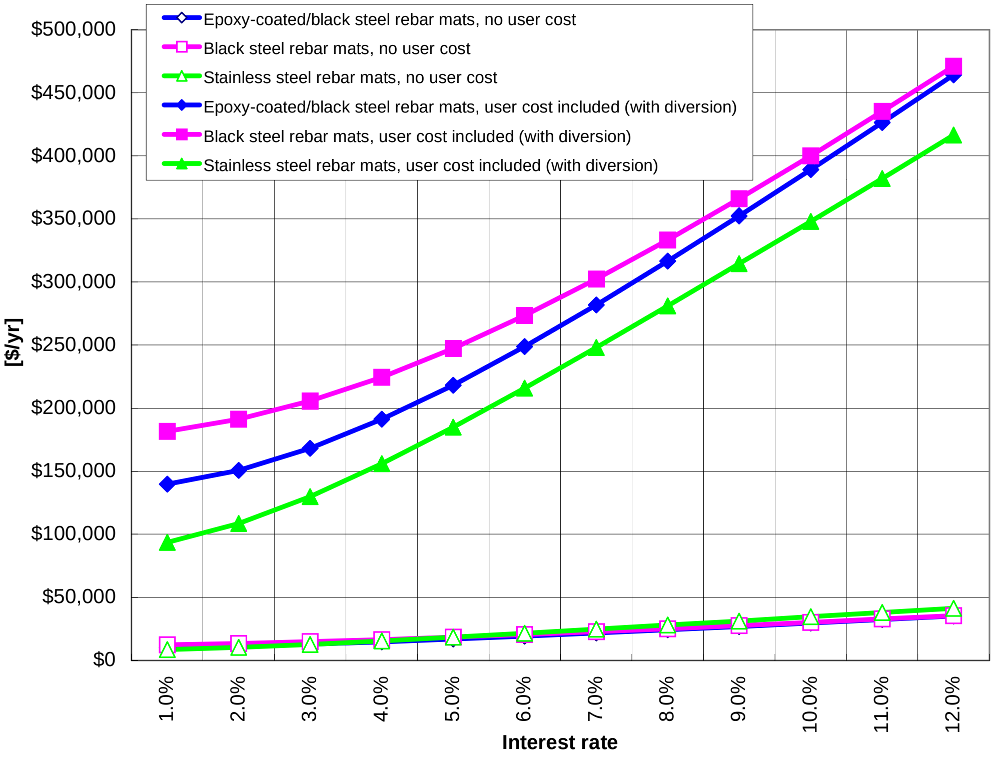

Figure 24. Comparison of the effect of the interest rate on the annual values for the “information-based with crack repair” maintenance scenario for the three deck designs with and without user costs (with diversion after 30 minutes).

Table 25. Annualized cost comparison for “information-based with crack repair” at 5 percent interest rate.

REBAR DESIGN	ANNUALIZED COST WITH NO USER COST	ANNUALIZED COST WITH USER COST, NO DIVERSION	ANNUALIZED COST WITH USER COST, DIVERSION AVAILABLE
Black Steel	$18,000	$284,500	$247,300
Top-Mat Epoxy-Coated	$16,800	$253,000	$218,300
Stainless Steel	$18,400	$216,800	$185.000

Effect of Traffic Flow

The sensitivity of user cost to traffic flow patterns is analyzed below. The traffic variables were chosen by trial to give the three desired levels of user cost (low, medium, and high). Table 26 shows the traffic variables and the user cost per day for these three levels. The daily user cost can vary significantly depending on the traffic flow assumed. The traffic flow assumed in the above analysis (see table 24) is most similar to the “low” level presented below (see table 26).

Table 26. Traffic scenarios corresponding to the user cost levels.

	“LOW” LEVEL OF USER COST	“MEDIUM” LEVEL OF USER COST	“HIGH” LEVEL OF USER COST
Average daily traffic	24,000	28,000	32,000
% of ADT in peaks	40%	40%	50%
Length of peak, minutes	140	120	140
Discharge rate, [cars/h]	1600	1700	2000
User cost per day, $/day – no diversion	**$28,784**	**$68,609**	**$124,025**
Maximum waiting time, minutes	30	30	30
User cost per day, $/day – with diversion	**$26,400**	**$39,502**	**$58,830**

Figure 25 shows the top-mat epoxy-coated rebar design annualized cost for the “information-based with crack repair” maintenance scenario with three different levels of user costs (with diversion) compared to that without user cost. A significant increase in annualized bridge cost occurs when the user costs increase. For example, at a 5 percent interest rate, the range of user cost presented in table 26 results in an increase in annualized cost of the bridge deck from $16,800 with no user cost to $205,000 at low user cost to $400,000 at high user cost.

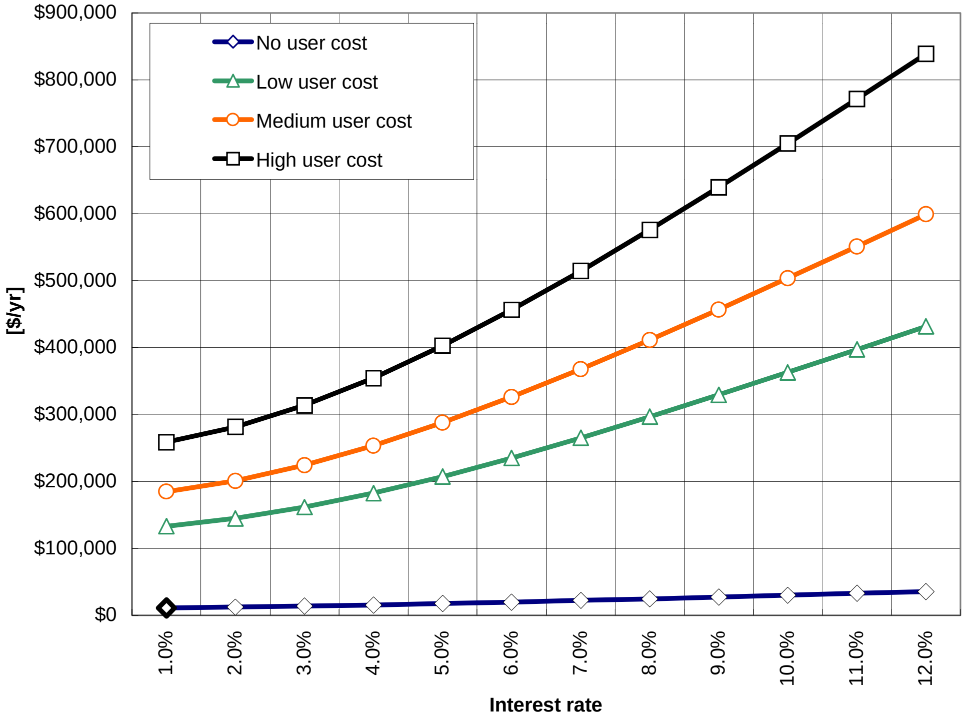

Figure 25. Effect of the interest rate on the annual values for the "information-based with crack repair" maintenance scenario for the epoxy-coated rebar mat design with three levels of user costs (with diversion).

Conclusions of the Life-Cycle Cost Analysis for Bridge Decks

1. Although direct costs are the primary driver for owner-operator cost option decisions, indirect (user) costs can be significantly more than the direct costs. Indirect costs were greater than the direct costs by a factor of 10 or more.

2. The discount rate had a significant effect on the deck design options considered. Therefore, incorporating the rate-of-return (discount rate) is strongly recommended for evaluating construction/maintenance options. For example, when considering direct costs only, the top-mat epoxy-coated rebar design was a lower cost option than the stainless steel rebar design at interest rates above 3 to 4 percent; below 3 to 4 percent, stainless steel rebar was the lowest cost option.

3. Including user costs in the life-cycle analysis favors corrosion-resistant rebar designs that eliminate corrosion-related deterioration repair and maintenance. For example, stainless steel rebar design provided a significantly smaller annualized cost option than top-mat epoxy-coated rebar design when user costs were included.

4. The maintenance approach can have a considerable effect on the annualized cost for black steel rebar design, but it is much less significant for more corrosion-resistant rebar designs. Early repair of cracks and selection of information-based maintenance to optimize repair scheduling resulted in a longer life for the black steel rebar deck and significantly reduced annualized costs.

5. Given the impact of the user costs on the life-cycle bridge deck costs, a significant emphasis should be placed on traffic planning options and on developing construction/repair/rehabilitation strategies that reduce lane/bridge closures and traffic interruptions.

Sample Life-Cycle Cost Calculations

Black Steel Rebar Bridge With Corrosion – Direct Cost Only

The example given is for the black steel rebar deck with the “information-based no crack repair” maintenance scenario at an interest rate of 5 percent. The following values apply:

Bridge deck surface area:	583 m^2
Unit construction cost:	\$484 per m^2
Total construction cost:	\$282,200
Annual routine maintenance cost:	\$1,000
Patching cost:	\$90 per m^2
Rehabilitation cost:	\$99,100
Old deck removal cost:	\$139,900
Patching starts in year:	9 (2.5% of deck damaged)
Patching ends in year:	28 (10% of deck damaged)
Rehabilitation in year:	28
Deck life ends in year:	46

Annual Maintenance

Annual maintenance (AM) is constant throughout the life cycle of the scenarios. This annual value is calculated back to the present by the following formula:

$$\text{PDV}\{\text{AM}\} = \text{AM} * [1 - (1 + i)^{-N}] / i$$

where

PDV = present discounted value, \$
AM = cost of annual maintenance, \$/year : 1000
N = length of the deck’s service life, years: 46
i = interest rate: 0.05

$$\begin{aligned}\text{PDV}\{\text{AM}\} &= \$1000 * [1 - (1 + 0.05)^{-46}] / 0.05 \\ &= \$17{,}900\end{aligned}$$

Patching

The cost of patching grows annually at a constant rate (g); for the calculation of the PV of patching, a modified interest rate needs to be calculated by the following formula:

$$i_0 = (i - g) / (1 + g) \qquad \text{and} \qquad i > g$$

where

i_0 = is the modified interest rate
i = interest rate: 0.05
g = constant annual growth rate: (increase in percent patching) / (years of patching to be performed)

$g = (0.10-0.025) / (28-9) = 0.00395$
$i_0 = (0.05 - 0.00395) / (1 + 0.00395) = 0.0459$

If the first payment (P_1) occurs in year 1, the present value of a cash flow that grows annually at a constant rate over n years can be calculated by the following formula:

$$PV\{P\} = [P_1 \ / \ (1 + g)] \ * \ [1 – (1 + i_0)^{-n}] \ / \ i_0$$

where

P_1 = cost of patching (\$90 per m^2) times the amount of deck patched (2.5% of surface area)

$$PV\{P\} = [\$90 * 583 * 0.025 \ / \ (1 + 0.00395)] \ * \ [1 – (1 + 0.0459)^{-(28-9)}] \ / \ 0.0459$$
$$= \$16{,}300$$

PV{P}, the present value of a cash flow series that starts at P1 in year 1 and grows at a constant rate g for n years when the interest rate is i, is equivalent to the present value of an annuity of [P1 / (1 + g)] for n years when interest rates are i_0, where i_0 is given by the equation above.

However, the first payment for patching does not occur in year 1, but in year t. Therefore, the above formula calculated a value at year (t-1) that is the equivalent of the cash flow series of patching through n years. This value needs to be discounted back to year 0 of the life cycle to determine the present discounted value of the patching:

$$PDV\{P\} = PV\{P\} \ * \ (1 + i)^{-(t-1)}$$

$$PDV\{P\} = \$16{,}300 \ * \ (1 + 0.05)^{-(9-1)}$$
$$= \$11{,}000$$

Rehabilitation

The PDV of one-time costs, such as the rehabilitation overlay (RH) is calculated as follows:

$$PDV\{RH\} = RH \ * \ (1 + i)^{-tRH}$$

where

RH = cost of rehabilitation overlay, \$
t = year in which the cost is incurred: 28

$$PDV\{RH\} = \$99{,}100 \ * \ (1 + 0.05)^{-28}$$
$$= \$25{,}300$$

Old Deck Removal

The PDV of one-time costs, such as the old deck removal (ODR) is calculated as follows:

$$PDV\{ODR\} = ODR \ * \ (1 + i)^{-tODR}$$

where

ODR = cost of removing the old deck, \$
t = year in which the cost is incurred: 46

$$PDV\{ODR\} = \$139{,}900 \ * \ (1 + 0.05)^{-46}$$
$$= \$14{,}800$$

Present Discounted Value

The PDV of the scenario is calculated as the sum of the PDVs of its cash flow:

$$PDV = I + PDV\{AM, P, RH, ODR\}$$

where

I = initial cost of the deck.

$$PDV = \$282{,}200 + \$17{,}900 + \$11{,}000 + \$25{,}300 + \$14{,}800$$
$$= \$351{,}200$$

Annualized Value

The annualized value (AV) of the scenarios is calculated from the PDV using the following formula:

$$AV = PDV * i / [1 - (1 + i)^{-N}]$$

where

N = service life of the deck, years: 46

$$AV = \$351{,}200 * 0.05 / [1 - (1 + 0.05)^{-46}]$$
$$= \$19{,}600$$

The annualized value for this sample calculation is $19,600 and corresponds to the value for black steel rebar at 5 percent interest given in figure 12.

Indirect Cost (User Cost)

The example given above without user costs is repeated below, but now includes user costs associated with deck construction/removal and maintenance activities. The illustrated traffic scenario is “no diversion.” The user costs included in the calculation include those due to the reduced throughput during the lane closure and those due to congestion. (See table 22 for assumptions concerning bridge construction.) In addition to the parametric values listed above, additional values apply:

Time to cross the bridge	0.03 h
Increase in travel time	150%
Days to build new deck	135
Days with user cost	96
Days to rehabilitate deck	62
Days with user cost	44
Days to remove the deck	90
Days with user cost	64
Value of time	$8.50/h
Traffic affected (only one lane closed)	12,000 cars
User costs due to traffic congestion (no diversion)*	$35,936

*All traffic congestion costs are calculated by a model developed by Boardman and Lave; the values of which are presented in table 22 and table 26 for the conditions discussed in this section.

Present discounted values and annualized values are calculated similarly to the direct cost example. (Recall that it was assumed there was no user cost associated with the patching activities.)

Bridge Construction

User cost due to lane closure during original bridge deck construction $P_{uc}\{BC\}$ is calculated as:

$P_{uc}\{BC\}$ = Time to cross * Increase in travel time * Days with user cost * Value of time * Traffic affected
$P_{uc}\{BC\} = 0.03 * (1+1.50) * 96 * \$8.50 * 12{,}000$
$= \$737{,}700$

Present value of this user cost $PDV_{uc}\{BC\}$ is equal to $P_{uc}\{BC\}$ as the costs occur at present time (year 0), i.e., $PDV_{uc}\{BC\} = \$737{,}700$

User cost due to traffic congestion during original bridge deck construction $P_{uc}\{BC\text{-}TC\}$ (and present discounted value $PDV_{uc}\{BC\text{-}TC\}$) is calculated using the appropriate cost from table 24.

$P_{uc}\{BC\text{-}TC\}$ = Days with user cost * User cost due to traffic congestion
$P_{uc}\{BC\text{-}TC\} = PDV_{uc}\{BC\text{-}TC\} = 96 * \$35{,}936$
$= \$3{,}449{,}900$

Rehabilitation

User cost due to lane closure during bridge rehabilitation $P_{uc}\{RH\}$ is calculated as:

$P_{uc}\{RH\}$ = Time to cross * Increase in travel time * Days with user cost * Value of time * Traffic affected
$P_{uc}\{RH\} = 0.03 * (1+1.50) * 44 * \$8.50 * 12{,}000$
$= \$343{,}300$

Present discounted value of user costs due to lane closure because of the rehabilitation activities ($PDV_{uc}\{RH\}$) is calculated as:

$PDV_{uc}\{RH\} = P_{uc}\{RH\} * (1 + i)^{-tRH}$
$PDV_{uc}\{RH\} = \$343{,}300 * (1+0.05)^{-28}$
$= \$87{,}600$

User cost due to traffic congestion during rehabilitation $P_{uc}\{RH\text{-}TC\}$ is calculated using the cost provided in table 24.

$P_{uc}\{RH\text{-}TC\}$ = Days with user cost * User cost due to traffic congestion
$P_{uc}\{RH\text{-}TC\} = 44 * \$35{,}936$
$= \$1{,}581{,}200$

Present discounted value ($PDV_{uc}\{RH\text{-}TC\}$) is calculated by discounting the $P_{uc}\{RH\text{-}TC\}$ value back to present time:

$PDV_{uc}\{RH\text{-}TC\} = P_{uc}\{RH\text{-}TC\} * (1 + i)^{-tRH}$
$PDV_{uc}\{RH\text{-}TC\} = 1{,}581{,}200 * (1+0.05)^{-28}$
$= \$403{,}400$

Old Deck Removal

User cost associated with the lane closures due to old deck removal $P_{uc}\{ODR\}$ is calculated as:

$P_{uc}\{ODR\}$ = Time to cross * Increase in travel time * Days with user cost * Value of time * Traffic affected
$P_{uc}\{ODR\} = 0.03 * (1+1.50) * 64 * \$8.50 * 12{,}000$
$= \$491{,}800$

Present discounted value for old deck removal ($PDV_{uc}\{ODR\}$) is calculated by discounting the $P_{uc}\{ODR\}$ value back to present time:

$PDV_{uc}\{ODR\} = P_{uc}\{ODR\} * (1 + i)^{-tODR}$
$PDV_{uc}\{ODR\} = \$491{,}800 * (1+0.05)^{-46}$
$= \$52{,}100$

User cost due to traffic congestion for old deck removal $P_{uc}\{ODR\text{-}TC\}$ is calculated using the costs provided in table 24.

$P_{uc}\{ODR\text{-}TC\}$ = Days with user cost * User cost due to traffic congestion
$P_{uc}\{ODR\text{-}TC\} = 64 * \$35{,}936$
$= \$2{,}300{,}000$

Present discounted value ($PDV_{uc}\{ODR\text{-}TC\}$) is calculated by discounting the $P_{uc}\{ODR\text{-}TC\}$ value back to present time:

$PDV_{uc}\{ODR\text{-}TC\} = P_{uc}\{ODR\text{-}TC\} * (1 + i)^{-tODR}$
$PDV_{uc}\{ODR\text{-}TC\} = \$2{,}300{,}000 * (1+0.05)^{-46}$
$= \$243{,}800$

Deteriorating Quality of Riding Surface Condition

The user cost calculations also include the costs due to the deteriorating quality of the riding surface (during the period covered by the patching activities). This cost is calculated on the basis of the data developed by Boardman and Lave(56) and presented in table 27.

Table 27. User cost data (costs due to deteriorating riding surface condition).

PERCENT DECK AREA AFFECTED BY PATCHING	YEAR IN WHICH PATCHING OCCURS	USER COST	PRESENT DISCOUNTED VALUE OF USER COST
2.50%	9	$5.08	$3.27
2.89%	10	$9.13	$5.60
3.29%	11	$15.22	$8.90
3.68%	12	$23.96	$13.34
4.08%	13	$35.99	$19.09
4.47%	14	$52.08	$26.30
4.87%	15	$73.04	$35.13
5.26%	16	$99.77	$45.71
5.66%	17	$133.24	$58.13

Table 27. User cost data (costs due to deteriorating riding surface condition) (continued).

PERCENT DECK AREA AFFECTED BY PATCHING	YEAR IN WHICH PATCHING OCCURS	USER COST	PRESENT DISCOUNTED VALUE OF USER COST
6.05%	18	$174.50	$72.51
6.45%	19	$224.67	$88.91
6.84%	20	$284.96	$107.40
7.24%	21	$356.63	$128.01
7.63%	22	$441.04	$150.77
8.03%	23	$539.62	$175.69
8.42%	24	$653.87	$202.74
8.82%	25	$785.36	$231.92
9.21%	26	$935.76	$263.17
9.61%	27	$1,106.78	$296.45
10.00%	28	$1,300.24	$331.68
	TOTAL	**$7,250**	**$2,300**

Total present discounted value user cost due to lower quality of the riding surface condition ($PDV_{uc}\{RSC\}$) is fairly minor:

$PDV_{uc}\{RSC\} = \$2,300$

Present Discounted Value of User Costs

The PDV_{uc} of the user costs is calculated as the sum of the individual PDV_{uc}'s due to lane closure and traffic congestion:

$PDV_{uc} = PDV_{uc}\{BC, RH, ODR, RSC\} + PDV_{uc}\{BC\text{-}TC, RH\text{-}TC, ODR\text{-}TC\}$
$PDV_{uc} = \$737,700 + 87,600 + 52,100 + 2,300 + 3,449,900 + 403,400 + 243,800$
$= \$4,976,800$

Annualized Value of User Costs

The annualized value (AV_{uc}) of the user costs is calculated from the PDV_{uc} using the following formula:

$$AV_{uc} = PDV_{uc} * i / [1 - (1 + i)^{-N}]$$

where
N = service life of the deck, years: 46

$AV_{uc} = \$4,976,800 * 0.05 / [1 - (1 + 0.05)^{-46}]$
$278,300

Total Costs (Annualized Value of Total Costs)

Combining the direct and user costs, we arrive at:

$AV_{total} = \$19,600 + \$278,300 = \$297,900$

The annualized value for this sample calculation is $297,900 and corresponds to the value for black steel rebar deck design at 5 percent interest given in figure 22.

Theoretical "Corrosion-Free" Bridge – Direct Cost Only

The life-cycle calculation given here is for the non-existent "corrosion-free" bridge (i.e., what if corrosion did not exist). A cost estimate for a "corrosion-free" bridge is necessary to compare to the cost of a bridge with corrosion; the difference is the "cost of corrosion." The example given below estimates the life-cycle direct cost for a black steel rebar deck that is "corrosion-free" and at an interest rate of 5 percent (same as given above for the corrosion example). The following values apply:

Bridge deck surface area:	583 m^2
Unit construction cost:	\$484/m^2
Total construction cost:	\$282,200
Annual routine maintenance cost:	\$1,000
Patching cost:	no cost (no corrosion-induced deterioration)
Rehabilitation (wear only) cost:	\$99,100
Old deck removal cost:	\$139,900
Patching starts in year:	never starts
Rehabilitation in year:	50 (for wear only, lasts 35 years)
Second rehabilitation in year:	85 (for wear only, lasts 35 years)
Deck life ends in year:	120

Annual Maintenance

Annual maintenance (AM) is constant throughout the life cycle of the scenarios. This annual value is calculated back to the present by the following formula:

$$PDV\{AM\} = AM * [1 - (1 + i)^{-N}] / i$$

where

PDV = present discounted value, \$
AM = cost of annual maintenance, \$/year: \$1000
N = length of the deck's service life, years: 120
i = interest rate: 0.05

$PDV\{AM\} = \$1000 * [1 - (1 + 0.05)^{-120}] / 0.05$
$PDV\{AM\} = \$19,900$

Patching

There is no corrosion, therefore no patching is required.

$PDV\{P\} = 0$

Rehabilitation at Year 50

The PDV of one-time costs, such as the rehabilitation overlay (RH), is calculated as follows:

$$PDV\{RH\} = RH * (1 + i)^{-tRH}$$

where

RH = cost of rehabilitation overlay: \$99,100

t = year in which the cost is incurred: 50

$PDV\{RH\} = \$99,100 * (1 + 0.05)^{-50}$

$= \$8,600$

Rehabilitation at Year 85

The PDV of one-time costs, such as the rehabilitation overlay (RH), is calculated as follows:

$$PDV\{RH\} = RH * (1 + i)^{-tRH}$$

where

RH = cost of rehabilitation overlay, \$99,100

t = year in which the cost is incurred: 85

$PDV\{RH\} = \$99,100 * (1 + 0.05)^{-85}$

$= \$1,600$

Old Deck Removal

The PDV of one-time costs, such as the old deck removal (ODR), is calculated as follows:

$$PDV\{ODR\} = ODR * (1 + i)^{-tODR}$$

where

ODR = cost of removing the old deck, \$139,900

t = year in which the cost is incurred: 120

$PDV\{ODR\} = \$139,900 * (1 + 0.05)^{-120}$

$= \$400$

Present Discounted Value

The PDV of the scenario is calculated as the sum of the PDVs of its cash flow:

$$PDV = I + PDV\{AM, P, RH[50], RH[85], ODR\}$$

where

I = initial cost of the deck

$PDV = \$282,200 + 19,900 + 0 + 8,600 + 1,600 + 400$

$= \$312,700$

Annualized Value

The annualized value (AV) of the scenarios is calculated from the PDV using the following formula:

$$AV = PDV * i / [1 - (1 + i)^{-N}]$$

where

N = service life of the deck, years: 120

$$AV = \$312{,}700 * 0.05 / [1 - (1 + 0.05)^{-120}]$$
$$= \$15{,}700$$

The annualized life-cycle cost for this "corrosion-free" scenario is $15,700.

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