

**Report No. CDOT-2010-6
Final Report**



STRATEGIC EVALUATION OF DIFFERENT TOPICAL PROTECTION SYSTEMS FOR BRIDGE DECKS AND THE ASSOCIATED LIFE-CYCLE COST ANALYSIS

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August 2010

**COLORADO DEPARTMENT OF TRANSPORTATION
DTD APPLIED RESEARCH AND INNOVATION BRANCH**

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Technical Report Documentation Page

1. Report No. CDOT-2010-6		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle STRATEGIC EVALUATION OF DIFFERENT TOPICAL PROTECTION SYSTEMS FOR BRIDGE DECKS AND THE ASSOCIATED LIFE-CYCLE COST ANALYSIS				5. Report Date August 2010	
				6. Performing Organization Code	
7. Author(s) Yu-chang Liang, Weiping Zhang, and Yunping Xi				8. Performing Organization Report No. CDOT-2010-6	
9. Performing Organization Name and Address University of Colorado Dept. of Civil Engineering Boulder, Colorado 80309-0428				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. Final Report Sept. 1, 2005 to May 31, 2010	
12. Sponsoring Agency Name and Address Colorado Department of Transportation - Research 4201 E. Arkansas Ave. Denver, CO 80222				13. Type of Report and Period Covered	
				14. Sponsoring Agency Code 80.09	
15. Supplementary Notes Prepared in cooperation with the US Department of Transportation, Federal Highway Administration					
16. Abstract <p>Bridge decks deteriorate faster and require more maintenance and repair than any other structural components on highway bridges. Topical protection systems act as barriers to protect bridge decks from corrosion damage by preventing water, oxygen, and chloride ions from reaching the reinforcement. This study evaluated topical protection systems commonly used on highway bridge decks in Colorado, including low-permeability concrete overlays and waterproof membranes with asphalt overlays. Five bridges were selected for inspection in the project. Concrete cores were taken from the selected bridge decks, and chloride concentration profiles were obtained at various depths. Chloride concentration is an important indicator for the corrosion damage of rebars in concrete deck.</p> <p>The inspection results showed that all five bridge decks are currently in good shape, which means the protection systems used on the selected bridges are effective. In order to predict future performance of the protection systems, a model was developed to predict the corrosion initiation time for rebars based on the current inspection results. The main feature of the prediction model is that the effect of deterioration of the concrete on its chloride permeability can be taken into account by using the current and future inspection results, and the deterioration could be due to environmental and/or traffic loadings.</p> <p>Implementation The prediction model for corrosion initiation time will be a useful tool for evaluating the effectiveness of topical protection systems. To better predict the future performance of bridge decks and the effectiveness of the topical protection systems, more inspection data are required for more reliable information on the deterioration rates of the bridges.</p>					
17. Keywords LCCA, rebar corrosion, chloride concentration, concrete bridge decks, corrosion prediction models			18. Distribution Statement No restrictions.		
19. Security Classif. (of this report) None		20. Security Classif. (of this page) None		21. No. of Pages 71	22. Price

ACKNOWLEDGEMENTS

The University of Colorado at Boulder would like to acknowledge the financial support provided by the Colorado Department of Transportation for this study. The authors would like to thank the many CDOT personnel that assisted with this study. A special thanks to Aziz Khan of the DTD Applied Research and Development Branch; Ali Harajli, Mark Nord, and Golda Davydov of the Bridge Design and Management Branch; Patrick Kropp and Eric Prieve of the Materials and Geotechnical Branch; Glenn Frieler and Gary DeWitt of Region 4; and Mathew Greer of the Federal Highway Administration.

EXECUTIVE SUMMARY

Corrosion of reinforced concrete bridge decks has been a significant long-term problem for the nation's transportation infrastructure. There are several categories of methods that can be used to prevent corrosion damages to concrete bridge decks: coatings on concrete, concrete overlays, coatings on steel bars, and cathodic protection systems. This project focused on the first two categories of corrosion protection methods: coatings on concrete deck and concrete overlays. There are many different topical protection methods, such as low-permeability concrete overlays and waterproof membranes with asphalt overlays. Various types of sealers are also used on bridge decks and substructures in some states in an attempt to keep water and chloride ions from penetrating the concrete. Development, application, and effectiveness of some of the topical protection systems were reviewed and discussed in this report. Several overlay materials were reviewed in detail, including material proportions, construction methods, performance evaluations, and cost analyses. In addition to the literature review, on-site inspections were performed to examine the effectiveness of some topical protection systems used in Colorado.

Rebar corrosion in bridge decks is a commonly seen long-term durability problem of reinforced concrete bridge decks, and chloride ion in concrete is one of the necessary conditions for the corrosion damage to occur in the structure. Therefore, chloride concentration in concrete has been considered as an important indicator that can be used to evaluate long-term performance of the structure. There are three stages in the rebar corrosion process that can be related to chloride concentration in concrete:

- (1) The chloride concentration near rebars is lower than the critical value and the corrosion of rebars has not started.
- (2) The chloride concentration near rebars just reached the critical value and the corrosion of rebars just started.
- (3) The chloride concentration near rebars is much higher than the critical value and the corrosion of rebars already started sometimes ago.

In view of the damage process due to steel corrosion, a good topical protection system should be able to stop or slow down the chloride penetration process into the concrete, so the chloride concentration profiles in the concrete can be used to evaluate the effectiveness of the topical protection systems.

Five bridges were selected for inspection in this project:

- (1) C-16-DI – Located 6 miles west of Loveland, built in 1980, with 3 in. of asphalt wearing surface.
- (2) G-22-BG – I-70, MP362.2, east of Limon, built in 1975, with 2 in. of integrated concrete surface.
- (3) G-22-BH – I-70, MP362.2, east of Limon, built in 1975, with 3 in. of integrated concrete surface.
- (4) G-22-BJ – I-70, MP357.77, west of Limon, built in 1972, without overlay.
- (5) G-22-BL – I-70, MP361.74, east of Limon, built in 1975, without overlay.

Concrete cores were taken from the selected bridge decks, and chloride concentration profiles were obtained at various depths. A chloride profile obtained from a bridge can be used to identify the current stage of the corrosion process in the bridge, and a comparison of chloride profiles obtained from different bridges can be used to evaluate the effectiveness of different topical protection methods applied on the selected bridges, together with other information such as environmental and traffic conditions.

From the inspection results, we concluded that all bridge decks inspected are in stage (1), and that means there is no corrosion in the bridge decks. This conclusion was based on the facts that 1) no corroded rebars were found in the coring process, and 2) the chloride concentrations near rebars are much lower than the critical chloride concentration. Comparing the ages, environmental conditions, and traffic loadings of the bridges, we can conclude that the difference in the effectiveness of the topical protection systems used in the five selected bridges is not significant, and that the inspection results cannot be directly used to evaluate the effectiveness of the topical protection methods in stage (2) and stage (3) of the corrosion process.

In order to predict future performance of the bridge decks, a simple mathematical model was developed to predict the corrosion initiation time of the rebars (the end of stage (1)) based on the currently available chloride profiles. First, a chloride concentration profile obtained from a bridge is used to calibrate the chloride permeability for the concrete, and then the calibrated chloride permeability is used in the model to predict a chloride concentration profile in the future. When the concentration at rebar level reaches the critical chloride concentration, the corresponding time is the corrosion initiation time of the bridge deck. The prediction model was used to predict corrosion initiation times for all five selected bridges, and the results showed that the corrosion of the rebars in the selected bridges will take place in a very long time, in the range of hundreds of years, which is not reasonable. This is because the chloride permeability of the concrete were calibrated based on present chloride concentrations and the deterioration of the concretes due to future environmental and traffic loadings was not taken into account.

The model was then modified to have a time dependent concrete permeability, which reflects the rate of concrete deterioration. The corrosion initiation times of the five selected bridges were predicted using the modified method. The corrosion initiation time of Bridge C-16-DI is the longest, which means the current bridge deck condition for this bridge is the best in terms of corrosion damage of rebars. Another important reason for the long corrosion initiation time for this bridge is that the traffic loading on the bridge is much lower than the traffic loadings of other bridges (on I-70). All other bridges have shorter initiation times ranging from 6 to 16 years. Given the uncertainties in the prediction model, the conditions of the other four bridges may be considered as the same.

To better predict the future performance of bridge decks and thus the effectiveness of the topical protection systems, more inspection data are required to give us more reliable information on the deterioration rates of the bridges. Therefore, we strongly recommend that at least one more inspection be conducted on the five selected bridges.

There have been several life-cycle cost analysis (LCCA) approaches proposed for pavement and bridge structures. In a recent CDOT report (CDOT-DTD-R-2007-2 "Service Life and Cost

Comparisons for Four Types of Colorado DOT Bridges Decks”), costs related to rehabilitation of bridge decks were computed using the methods presented in NCHRP Report 483. In this report, the same methods were used for LCCA as in CDOT-DTD-R-2007-2. The performance evaluation of corrosion protection systems during the 75-year life span was calculated by the mathematical models. All cost data were obtained from CDOT databases. LCCA results were shown in a format that would be usable for upload to AASHTO Pontis Bridge Inspection Coding Guide.

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1. LITERATURE REVIEW ON PROTECTION METHODS FOR BRIDGE DECKS

1.1 Introduction

Among all structural components used for a highway bridge, bridge decks usually deteriorate at the fastest rate and require the most maintenance and repair actions. This is mainly because bridge decks are exposed to severe environmental conditions such as temperature fluctuation and moisture variation and direct traffic loadings. In addition to the temperature and moisture variations, deicing salts have been widely used in the winter months for anti-icing and deicing of bridge decks and pavements, which causes the so-called chloride-induced corrosion of embedded reinforcement in bridge decks. The corrosion of reinforcing bars has been a long-term problem for DOTs as well as the nation's transportation management agencies.

For chloride-induced reinforcement corrosion, chloride ions penetrate into concrete from the environment, and reinforcement in concrete decks begins to corrode once the chloride concentration in the vicinity of reinforcement reaches a threshold value. Using concrete with low permeability is a natural choice of protection for the embedded steels, which is the first line of defense to resist the penetration of chloride and moisture into concrete. However, cracks develop in concrete over time, and cracked concrete does not provide a good protection for the steel. Therefore, many other methods have been developed to prevent bridge decks from corrosion and prolong their service lives. These methods can be divided into four different categories:

- (1) **Topical protection systems** as barrier methods protect bridge decks from corrosion damage by preventing water, oxygen, and chloride ions from reaching the reinforcement and initiating corrosion.
- (2) **Corrosion inhibitors** added in concrete provide protection by raising the threshold chloride concentration level.
- (3) **Cathodic protection/prevention techniques** use impressed current and/or external anode to protect the reinforcement. This type of method is very effective even when the chloride ion concentration in the vicinity of reinforcement is above the threshold value.
- (4) **Alternative reinforcement steel bars** have been developed. They include materials that electrically isolate the steel from the concrete and create a barrier for chloride ions, materials that protect steel galvanically, and materials that have significantly higher corrosion thresholds than conventional reinforcing steel.

This part of the report focuses on topical protection systems commonly used on highway bridge decks, which provide a protective layer between the bridge deck and the environments. There are many different topical protection methods, such as low-permeability concrete overlays and waterproof membranes with asphalt overlays. Various types of sealers are also used on bridge decks and substructures in some states in an attempt to keep water and chloride ions from

penetrating the concrete. Development, application, and effectiveness of all these topical protection systems will be reviewed and discussed in this report.

1.2 Waterproof membranes

1.2.1 Types of membranes

Waterproof membranes on bridge decks can protect the embedded reinforcement in decks from corrosion, and protect deck slab from the deterioration induced by freeze-thaw cycles as well. Waterproof membrane systems have been used extensively on concrete bridge decks as a chloride barrier in the United States and Europe since the problem of reinforcement corrosion in bridges was first recognized in the 1960s. The rapid increase of the use of membranes as a preventive technique has led to the introduction of numerous membrane products. The membranes can be classified into three main categories: built-up membranes, preformed sheet membranes, and liquid membranes. Preformed sheet membranes are used very often in the U.S., while hot-applied rubberized asphalt, which is a liquid membrane, is used extensively in Canada.

Preformed sheet systems include various factory-manufactured rolls that are bonded to bridge decks to form a continuous membrane. According to a more detailed classification system in the United Kingdom (Price 1990), preformed sheets can be categorized into four categories: asphalt-impregnated (bituminized) fabric sheets, polymeric sheets, elastomer sheets, and asphalt (bituminous) laminated boards. Asphalt-impregnated fabric sheets consist of a central core of absorbent material impregnated and coated with asphalt cement. The materials for the core could be made of polyester fleece, glass cloth, or woven polypropylene. Polymeric sheets are extruded blends of various base polymers in which other polymers, binders, plasticizers, and inert fillers are included. Elastomer-based sheets are usually made of either vulcanized butyl or polyisoprene rubber. Asphalt laminated boards are a preformed lamination of asphalt-saturated felt sandwiching finely crushed aggregates.

Liquid membranes consist of one or two components of latex or chemical curing solutions that are applied to the concrete surface. Liquid membranes are divided into two categories: asphalt-based (bitumen-based) and resin-based liquid. Asphalt-based liquid are various bitumen solutions blended in hydrocarbon solvents, two-part polymer modified compositions, or refined natural or elastomer-modified mastic asphalts. The resin-based membranes are one of two-part moisture curing or two-part chemical curing based on urethane, epoxy, or acrylic resins. Urethane-based systems are all elastomers with carborundum or coal tar. Other polyurethane systems, known as pitch epoxies, are modified with coal tar. The acrylic systems are based on polymethacrylate resin. Some of the liquid systems are mineral filled.

The most common of the early built-up systems consisted of two or more layers of glass fabric mopped with coal-tar pitch emulsion (NCHRP 1970). These built-up membranes are not used very much today. The systems are generally too labor intensive and slow to construct to be cost effective. A different kind of built-up membrane was used in Oregon, which consists of a polypropylene fabric rolled into a hot-rubberized-asphalt membrane (Manning 1995).

Each type of membrane system has its own advantages and disadvantages. Preformed sheet membranes tend to perform better than liquid membranes in laboratory evaluations. The quality and thickness of preformed sheets can be controlled relatively well by the manufacturer, while the consistent quality and thickness of liquid systems is more difficult to control during field installation. However, preformed sheets can be labor intensive to install, require laps at the edge, and difficult to install on curved or rough decks. The liquid systems can be applied in one application by spray or squeegee without laps. The application does not depend on the geometry of the deck. Preformed systems are more vulnerable to poor workmanship at critical locations, such as curves, expansion joints, and drains. Any blister appearing after the installation has to be repaired by puncturing and patching the membrane. Liquid membranes are less vulnerable to poor workmanship, and blisters and pinholes are relatively easy to repair. Liquid systems are generally less expensive than preformed systems (Kilareski 1977). The advantages and disadvantages of many preformed and liquid systems were analyzed in a NCHRP study, in which 147 membrane systems were compared (Van Til et al. 1976).

1.2.2 Field application and performance

The use of waterproof membranes to protect bridge decks started in the early 1960s. Several states began using waterproof membranes in 1972, when the federal government introduced a policy that required bridge decks on the federal-aid system that were likely to be subjected to deicing salts to be protected against the corrosion of embedded reinforcement. However, data from surveys conducted between 1974 and 1994 showed a constant decline in the number of states that use waterproof membranes in new construction, likely because of increasing alternative protection systems.

In 1994, a NCHRP survey was conducted to inquire information about the use of various membranes in all 50 states in the United States and all 10 provinces in Canada. Forty-eight states and six provinces were responded to the survey (Manning 1995). Membranes have been used in the Northeast for decades, but are not used in most Midwestern states. Figure 1-1 shows the states that reported using membranes on new decks and for rehabilitation, respectively. The area of membrane was installed in the United States during 1992 on new bridge decks and on existing decks for rehabilitation were 620,000 ft² and 2,300,000 ft², respectively. It also indicated that 14 states were using preformed sheet membranes, and five states were using both preformed sheet and liquid membranes. There was one state using built-up membranes exclusively, one allowed built-up membranes to be used as an alternative to preformed membranes, and one allowed the use of any of the three systems. A survey by Chamberlin showed that 29 states in the United States use membranes as a standard deck protective system, and two states used membranes only as an experimental protection system. 20 other states have used membrane systems in the past but no longer consider them as standard. There is only one state that has never used any membrane systems.

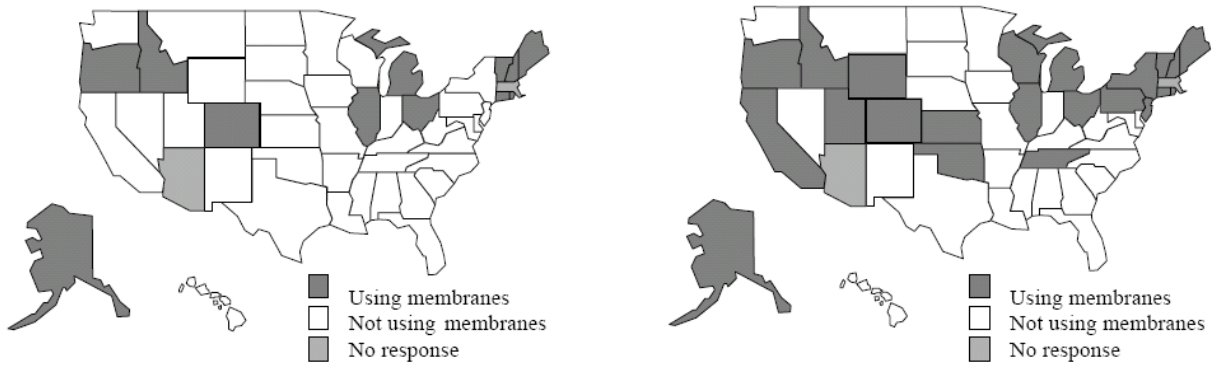


Figure 1-1 States using the waterproof membranes in new construction (right) and rehabilitation (left)

Some agencies have had good experience with membranes as corrosion protection systems, while others have not and no longer use them (Manning 1995). All of the bridges in Ontario are waterproofed using a hot rubberized asphalt (liquid) membrane. These membranes have been used in Ontario since the early 1970s and have worked extremely well. When used in Iowa, however, membranes did not perform well (Grove 1999).

Vermont has used sheet membrane systems as the standard corrosion protection method on its bridge decks since the early 1970s. An extensive evaluation of bridge deck membrane systems was conducted in Vermont. 33 membrane systems on 59 bridge decks over a period of 11 years (1971-1982), including 15 preformed systems, seven epoxies, five thermoplastic materials, four polyurethanes, and two tar emulsion systems were evaluated. The annual average daily traffic on the bridges ranged from 270 to 1990 vehicles. The average equivalent axle load was 3 to 18 kips. The average number of freeze-thaw cycles was between 80 to 115 cycles per year. The salt application during the investigation period was averaged 3 lb/ft² of deck surface. Membrane performance was assessed on the basis of the degree of chloride contamination. It was concluded that preformed membranes performed the best and could provide protection from chloride contamination for 50 years or longer. The tar emulsion systems exhibited the worst overall performance.

One of the earliest experimental users of membrane systems to prevent rebar corrosion was the state of Kansas. Membranes were used as a rehabilitation method on chloride-contaminated bridge decks. A long-term investigation was conducted over a period of 16 years. The bridges were located in area that experience about 60 freeze-thaw cycles annually and the ambient temperature varied from -40 to 49 °C. The bridges were exposed to 20 salt applications per year at a rate of 1300 lb per two-lane mile. Nine bridges were investigated with five different types of membranes. The bridges ranged in age from 12 to 15 year old at the time the membranes were installed. The membrane systems included in the investigation were one preformed, two liquid/preformed, and two liquid. Most of the investigated membranes in Kansas performed well. The study indicated that, the membranes retarded the downward movement of water and chlorides, which, in addition to the high evaporation, reduced the amount of imbibed water and

kept chloride and water near the surface. It was also pointed out that care in installation and proper timing of installation proved to be necessary during construction.

In another study, nine different membrane systems were installed on 35 bridge decks in Oklahoma between 1973 and 1975 (Ward 1978). Five of the systems were preformed sheets, three liquid membranes, and one built-up system. The asphalt surfacing was 1.5 in. thick, except for a few cases where 1.0 in. more sand asphalt was added to help protect the membrane. Three of the systems had to be removed within a few days of installation because of water under the membrane or damage during the paving operation. A fourth membrane lasted only 18 months before it broke up under traffic. Poor workmanship at the time of installation was reported.

Agencies in Ontario are fairly polarized on the issue of membranes, but those that have had success with membranes have generally used a thicker layer of asphalt over membranes (David Manning 1999). All bridges in Ontario are waterproofed using a hot rubberized asphalt membrane, and overlaid with 80 mm thick asphalt to ensure a smooth riding surface. The membrane comes in 50 lb cakes that are melted and spread onto the deck with a squeegee. Protection board is placed between the membrane and the asphalt overlay. These membranes have been used in Ontario since 1972 and have been working extremely well.

In the late 1990s, Vermont and New Hampshire began experimental use of other forms of waterproofing membranes, including torch applied sheet membranes and spray applied liquid membranes (Graham 2000). Although these systems are much more expensive than conventional sheet membrane systems, testing results show that both of these membranes have a stronger bond with the concrete deck (Roberts 2000). The torch applied systems consist of a primer and a 4.5mm thick sheet membrane with granular material to help increase bond with the deck. The sheets are laid on the deck and then heated with torches, either by hand or machine, until the underside is melted, causing the hot liquid bitumen to flow and form a bond to the concrete deck. The spray-applied membrane system consists of a primer, two layers of methylmethacrylate resin membrane, approximately 1.5mm thick, and a tack coat to promote a good bond with the asphalt overlay (Graham 2000).

Another case study used the performance of membrane systems in 20 interstate bridge decks over a period of seven years (LaCroix, J.E. 1985). The study concluded that membrane systems would have to perform well for at least 4.5 years in order to be an economical treatment. Also, it is suggested that repairing chloride-contaminated bridge decks by partial patching and installing membrane systems can be a cost-effective alternative to total deck replacement.

1.2.3 Summary

The average service life of conventional waterproof membranes applied to new decks is about 15 to 20 years, while the service life of rehabilitated decks is about 5 to 10 years, if minimal repairs are made, or 15 to 20 years if extensive repairs are made and the overlay thickness is at least 3 in. (Kepler et al. 2000). In many cases, the service life is determined by the asphalt overlay (Manning 1995). Torch-applied sheet membranes and spray-applied liquid membranes have

only been used in Vermont since 1997, but their expected service lives are more than 20 and 50 years, respectively (Roberts 2000).

According to NCHRP Synthesis of Highway Practice 220: Waterproofing Membranes for Concrete Bridge Decks (Manning 1995), waterproof membranes can be a cost-effective method for protecting reinforcing steel from corrosion in concrete bridge decks in the United States, if the following issues are properly considered:

- (1) Proper drainage should be provided so that water can drain quickly from decks. Seepage drains should be provided at low points to prevent water from staying on top of the membrane.
- (2) Protection board or a layer of sand asphalt should be placed over membranes that are susceptible to damage during installation and overlaying.
- (3) A minimum asphalt overlay of thickness 3.0 in. should be specified.
- (4) Softer and thicker membranes are vulnerable to rutting and shoving when used on grades exceeding 4% of where heavy trucks brake or turn. Different membranes or another protection method should be used in these situations.
- (5) Liquid membranes should be more than 0.08 in. thick, and preformed membranes should be more than 0.10 in. thick.
- (6) Ventilating layers are not recommended because they reduce the bond between the membrane and the deck.

1.3 Concrete sealers

Compared with other protection methods, concrete sealers are independent on the geometry of structure. They can be applied onto bridge decks, substructure members and deck undersides. There are many sealers available on the market. As illustrated in Figure 1-2, sealers can be film-forming coatings, or pore-blockers filling the capillary pores, or hydrophobic pore liner lining the surface of the pores. Most pore blockers are not appropriate for use on bridge decks because they do not offer good skid resistance and do not hold up under traffic wear (Sherman et al. 1993).

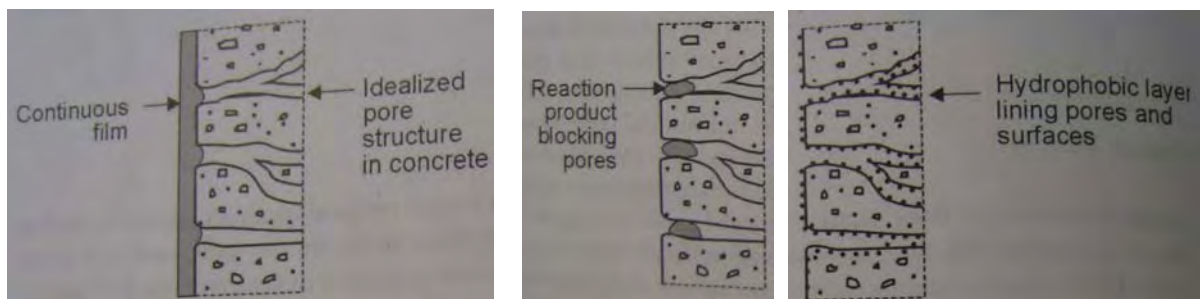


Figure 1-2 Different types of concrete sealers (Left: Film-forming coatings; Medium: Pore blockers; right: Hydrophobic pore liner)

Moisture within the underneath concrete needs to be able to pass through the sealer and escape to the environment to prevent high vapor pressures from building up in the concrete during drying

periods which could cause the sealer to blister and peel. Vapor transmission characteristics are considered to be an important property of a sealer.

1.3.1 Linseed oil

Linseed oil was the first sealer applied on concrete bridge decks in the United States. When it was recognized that deicing salts were causing corrosion problems on bridge decks, the use of linseed oil was expanded from a scaling reduction measure to a corrosion protection method.

Linseed oil needs to be reapplied every two to five years to maintain its performance, and many engineers think that linseed oil does not work well and is not cost effective (Meggers 1999), so its use has been discontinued in many states. For example, Kansas has not used linseed oil on its decks for at least the past 15 years (Meggers 1999). In some states, linseed oil is still used. In Missouri, a mixture of linseed oil and mineral spirits are applied to all new decks before they are opened to traffic, and then again one year later. It was reported by Missouri DOT that other sealers have been tried, but none of them have worked as well as the linseed/mineral spirits mixture (Wenzlick 1999). Texas also uses a linseed oil/mineral spirits mixture as a standard surface treatment on its bridge decks.

1.3.2 Epoxy

Epoxyes have been used as both penetrating sealers and as coatings. Epoxyes have good adhesion to concrete, aggregate and steel, high resistance to chemicals and weather, and quick curing without requiring high temperatures or pressures. Problems experienced with epoxy coatings include blistering and debonding from the deck, and occurrence of pinholes. Epoxy coatings were applied to a number of Kansas bridge decks in 1960s. When the coatings were evaluated, they were found to be full of pinholes and to have not sealed the decks (McCaskill et al. 1970).

1.3.3 Silane and Siloxane

Silane and siloxane sealers are silica-based materials that function as hydrophobic agents. Silanes are small molecules having one silicon atom, and siloxanes are short chains of a few silicon atoms. Unlike most oil-based sealers blocking the pores of the concrete, silanes and siloxanes react chemically with the concrete surface to form a hydrophobic layer. As a result, water molecules and chloride ions will be repelled while water vapor is allowed to pass through. Proper surface preparation is very important before the application of silanes or siloxanes. The deck must be cleaned free of oil, curing compounds and general road grime to make sure that the treatment material can actually reach the concrete substrate (Sherman et al. 1993). An NCHRP study found that the water absorption of concrete treated with silane was reduced to 30% of non-treated concrete when the silane was applied to clean concrete. Absorption increased to 47% when it was applied on top of linseed oil (Pfeifer and Scali 1981). The deck must also be dry to allow the silane or siloxane to penetrate into the concrete and bond with it chemically.

Silane or siloxane sealer systems require routine maintenance activities such as milling to insure proper performance. The milling process often damages the underneath membrane allowing the chloride ions from deicing salts to attack the concrete. When the silane or siloxane sealer system is used for bare decks without an asphalt overlay, it requires multiple applications every

one to two years in order to provide proper protection. This is because traffic wears off the silane or siloxane sealer especially in the areas of the wheel lines. These protection procedures add additional maintenance costs to highway users.

1.3.4 Methacrylate

Another sealer that has been used extensively in California, Virginia and Alberta is high molecular weight methacrylate. Methacrylates can be crack sealers or fine overlays, using sand and other fine aggregate for skid resistance. Methacrylates are generally applied as a three-component system consisting of a monomer, a promoter, and an activator, mixed together prior to application. They are usually low-viscosity materials, and are applied as a spray or with a broom or squeegee (Sherman et al. 1993).

It should be noted that cracks tend to reopen through the polymerized material over time, decreasing the effectiveness of methacrylates. Field application problems are encountered with this sealer mainly because most field crew is not familiar with application procedures of methacrylates, and thus are bothered by the smell. Another reason is that the set and hardening of the material are highly sensitive to the environment condition (Sherman et al. 1993).

1.4 Concrete overlays

1.4.1 Reinforced Portland cement concrete overlay

A reinforced Portland cement concrete (PCC) overlay consists of a monolithic layer of PCC with one layer of longitudinal and transverse reinforcing steel. This type of overlay protects decks because of its thickness and low permeability, and provides additional structural resistance because the existing concrete slabs and the overlay will behave in a composite manner after its completion. Therefore, reinforced PCC overlay might be used more on older bridges with thinner cross sections of decks than those being designed and constructed today.

1.4.2 Low-slump dense concrete overlays

It is well accepted that the natural capillary void system of a concrete can be reduced by increasing the cement content to over 800 lb/yd³ and decreasing the w/c ratio to about 0.30 to 0.32. Based on these criteria, low-slump dense concrete (LSDC) overlays have been developed to prevent chloride ions and moisture from penetrating into bridge decks.

The main problems of LSDC overlays are difficult placement, high cost, cracking on the surface and relatively lower skid resistance. Some of the construction problems can be overcome by using a high-range water-reducing admixture. Proper curing is important to prevent surface cracking.

LSDC overlay has been used by DOTs of Iowa, Kansas, Minnesota, and New York. LSDC overlays were first placed in the early 1960s in Iowa and Kansas, and are often referred to as "Iowa low-slump overlays". When bridge decks in Iowa needed rehabilitation, a water cement

grout was sprayed onto the dry deck with the intention of increasing bond, and then two inches LSDC overlay was immediately placed on top of the deck when the grout was still wet. The first LSDC overlay in Iowa lasted for 23 years before it had to be replaced. Some of LSDC overlays were in service for more than 25 years. During 1999, over 44,600 yd² bridge decks in Iowa were overlaid with LSDC overlays.

In 1970s and 1980s, a large number of bridge decks were overlaid with LSDC concrete in New York State. Fifty of those overlaid bridges between 1979 and 1981 were randomly selected and evaluated. At the time of inspection, damage that occurred in the interiors of the slabs was found in 20% of the bridge spans. All of the damage occurred in the form of delaminating in areas of the decks that had been scarified. The service lives of these overlays were estimated to be 25 years when about half of the surface of the deck was excavated to beneath the reinforcement before the overlay was placed.

1.4.3 High performance concrete overlay

Much effort has been made to reduce the permeability of concrete overlay on bridge decks so that the penetration of chloride ions into the concrete can be slowed down. High performance concrete (HPC) overlays with silica fume, micro silica, fly ash, slag, latex have been used by many DOTs. Where a bare concrete deck is desired, Region 6 of CDOT has been topping the deck with two inches of silica fume concrete.

To evaluate HPC bridge deck overlay, sixteen HPC overlays were placed on two 28-span bridges on Rte. 60 over Lynnhaven Inlet, Virginia Beach in 1996. Various additives were use including silica fume (SF), slag (S), fly ash (FA), latex modified concrete (LMC), corrosion-inhibiting admixture (CIA), shrinkage-reducing admixture (CQI), polyolefin fibers (PIF), steel fibers (STF), polypropylene fibers. Among the concrete mixes used, the HPC overlays with 5% SF, 15% FA, and 7% SF, respectively, cracked the most, while the overlay with LMC had the least crack (LMC is discussed in the next section). All of the HPCs showed low permeability and high bond strength. Because of the relatively higher costs of the ingredients, the overlays with steel fibers, polyolefin fibers, and latex were slightly more expensive, and the overlays with only silica fume and fly ash cost less.

HPC with SF overlays have been used in some areas in Iowa, but the traditional LSDC overlays have been performing well and are cheaper than HPI with SF, so the HPC with SF overlay was not determined to be cost effective for use in Iowa.

Micro-silica modified concrete (MMC) has begun to replace the conventional LMC in some states, due to the relative ease of placement. Both MMC and LMC generally perform well. However, distresses are sometimes reported shortly after construction. Early cracking was related to plastic and drying shrinkage. These cracks can propagate through the overlay and permit contaminated water to reach the deck

1.4.4 Latex-modified concrete overlay

Latex-modified concrete (LMC) is a Portland cement concrete in which an admixture of styrene butadiene latex particles suspended in water is used to replace a portion of the mixing water. This type of concrete has been used on highway bridge decks over the past 40 years and was the most commonly used deck overlay material in 1980s.

Compared to concrete without latex, LMC is reported to have higher resistance to the intrusion of chloride ions; to have higher tensile, compressive, and flexural strength; and to have greater freeze-thaw resistance. These features are helpful to mitigate the cracking of decks and to decrease the permeability of chloride in concrete overlays, and thus delaying the initiation of the rebar corrosion. The permeability of LMC overlays has been found to be only about 12% that of conventional concrete. The lower permeability of chloride can be attributed to the lower water/cement ratio and a plastic film produced by the latex particles within the concrete pore system. The higher strength can be attributed to the lower water/cement ratio and the stronger bond between the paste and aggregate produced by the plastic film. The LMC is less permeable to water and more flexible than regular concrete, enabling it to better withstand the expansion and contraction associated with frost action. LMC overlays consistently outperformed conventional and low slump dense concrete overlays in 16-years in Virginia. Because of high material costs and good performance, the thickness of LMC overlays is typically small, either 40 or 55 mm thick, thinner than other types of overlays.

To reduce the inconvenience to the public caused by traffic closure during overlay construction, the overlays are usually installed during off-peak traffic periods. Thus, a high early strength latex-modified concrete (LMC-HES) overlay has been developed and used successfully by Virginia DOT when a lane can only be closed for 1 to 2 days. Furthermore, in order to apply on many situations, such as the closure of a bridge lane can just last only for 8 hours or less (such as at night), a very-early-strength latex-modified concrete (LMC-VES) overlay has been developed and used in Virginia in the past decade. LMC-VES mixture proportions, installation equipment, and procedures are similar to those used for LMC overlays. However, when working with LMC-VES, the contractor must work faster because the concrete loses slump rapidly and the curing period is approximately 3 hours rather than 72 hours.

In 1993, silica fume latex modified concrete (SF-LMC) was used by Oregon DOT to overlay a deteriorated bridged deck. SF-LMC did reduce the permeability. However, a three-year field evaluation showed that the product could not be mixed and placed by a conventional mobile mixer because of pump seals. Modifications to the mixer would be costly. Thus Oregon DOT recommends against any further use of this product.

LMC overlays were found to have problems with plastic shrinkage cracks that deepen with age and with scaling in continuously saturated areas. It was reported in 2005 that the shrinkage of the LMC containing Type K (expansive) cement was much less than that of the traditional LMC containing Type I/II cement, while their compressive strength, permeability, bond strength and freeze- thaw resistance were similar. The use of Type K cement is estimated to increase the cost of the concrete approximately 2.6%, or about \$1/yd² for a 1.5 in. thick overlay. This is much less than the cost to seal the shrinkage cracks in an overlay: \$10/yd². Greater savings can also come

from the longer service life of a crack-free overlay. Therefore, Type K cement was recommended to be used in LMC and other type concrete overlay.

1.4.5 Polymer concrete overlay

Polymer concrete has also been used for overlays. Portland cement is replaced by a polymer in the polymer concrete. Polymer concrete overlays are nearly or completely waterproof when they are crack free, so they can protect bridge decks from the ingress of water and chloride ions. Different types of polymers can be used for polymer concrete, such as epoxies with aggregates, polyesters with aggregates, methacrylates with aggregates, and polyurethanes with aggregates.

The use of polymer concrete overlays is generally limited to the rehabilitation of corroding bridge decks because of the high initial cost and difficult placement process. Missouri DOT began using epoxy-polymer overlays to rehabilitate bridge decks in 1990. These overlays prove to be cost-effective, and they protect the deck for 10-15 years by keeping water out. They are occasionally used as a preventative measure on sound decks in which corrosion is anticipated. These overlays are generally less than 0.5 in. thick and are seeded with sand or other fine aggregate to improve friction properties. Polymer concrete with larger aggregate is often used for repairs because of its high early strength and short set time (Sherman et al. 1993).

The performance of a polymer concrete is dependent on the bond between the overlay and concrete substrate, which depends on surface preparation, cleanness and field application techniques. Most failures of polymer concrete overlays are attributed to workmanship or improper handling of materials.

1.4.6 Fiber reinforced concrete overlay

To reduce early cracking and cracking width, various fiber reinforced concretes have been used by Oregon and South Dakota DOTs for concrete overlays. The use of polypropylene fibers in MMC was investigated in Oregon DOT since 1997. However, cracking resistance was found to be no better in the MMC overlay with fibers, compared to the MMC overlay without fibers. In Europe, Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) has been developed and tested in laboratories. It has very low permeability and very high tensile strength. High tensile strength helps to reduce cracking tendency of concrete. Its potential use for overlay of concrete decks was considered. However, the initial cost of UHPFRC is very high due to the use of large amount of fibers and fiber dispersion additives in UHPFRC.

1.5 Comparison analysis

In 1991, 152 bridge decks with LMC overlays and 153 with LSDC overlays were evaluated by a Strategic Highway Research Project. The results of the study indicated that the performance of the overlays was less dependent on the type of overlay than on the methods used to prepare the deck. Both overlay types performed very well when the concrete were removed to below the reinforcement and the exposed surface sandblasted to remove micro-cracks. When these

procedures are followed, it was estimated that potential service life of both overlay types can be 30 to 50 years, assuming that service life ends when 40% of the total deck area is damaged.

Evaluation of LMC-VES overlays constructed in Virginia indicated that the traffic could be placed on the overlay within 3 hours cure time rather than within 4 or 7 days required for the conventional LMC overlay; and the bond strength, resistance to chloride intrusion can be as good as those of LMC-HES and LMC. In addition, the life-cycle cost could be reduced by 25% in comparison with LMC.

2. DETAILED REVIEW OF SELECTED TECHNICAL REPORTS

Several overlay materials will be reviewed in more detail in this section, including material proportions, construction methods, performance evaluations, and cost analyses.

2.1 Very-early-strength latex-modified concrete (LMC-VES) overlay

The construction of overlays has become increasingly difficult because of increases in traffic volumes. Lanes in need of an overlay cannot be closed to traffic without significant inconvenience to the public, particularly on the Interstate Highway System, unless the overlay can be installed during off-peak traffic periods. Because of the slow strength development of conventional LMC mixtures, there is a pressing need to develop high early strength overlay materials for bridge decks. Two types of high early strength overlay materials were developed for this purpose. One of them is high early strength latex-modified concrete (LMC-HES) overlay. By using LMC-HES, a traffic lane can be closed only for 1 or 2 days, and thus, an overlay project can be done over a weekend (Sprinkel, 1988). The other one was developed for more critical situations where a bridge lane can be closed only for 8 hours or less. The special overlay material, called very-early-strength LMC (LMC-VES) overlay system was successfully developed and used to repair and overlay bridge decks since 1997 (Sprinkel, 1998).

2.1.1 Mixture proportions and construction procedures

Very early strength of LMC-VES overlays is achieved with a specially blended cement of high fineness and high Al_2O_3 and SO_3 contents, rather than the Type I/II cement used in the conventional LMC overlay. High early strength of LMC-HES overlays is achieved with Type III cement and the high cement content. The physical and chemical properties of the cements and mixture proportions for LMC-VES, LMC-HES and LMC are presented in Table 2-1 for comparison.

The procedure of installing a LMC-VES overlay is almost the same as for installing an LMC or LMC-HES overlay, although the time required for each step was less. The deck was scarified to remove the top 0.5 in. of the old concrete. Within 2 hours prior to the placement of the overlay, the exposed surfaces of the concrete were shot blasted, sprayed with water, and covered with a sheet of polyethylene. The overlay was quickly covered with burlap and polyethylene to provide a moist environment during the 3-hour curing period. The overlay was required to be at least 1.25 in thick.

A typical LMC-VES overlay construction sequence starts by closing a lane at 9 p.m.; the concrete surface and the patch area are removed by the midnight; the patching using LMC-VES is completed by 2 am; and the lane opens to the traffic at 5 am.

A typical LMC-HES overlay construction sequence starts by closing a lane at 9 p.m. Friday night; the concrete surface is removed by 6 a.m. of the next day, the overlay is completely placed by 11 am; the lane opens to traffic at 5 a.m. on Sunday or Monday depending on ambient temperature and job requirement.

Table 1-1 Comparison of LMC, LMC-HES and LMC-VES

Overlay type		LMC-VES		LMC-HES	LMC
Cement type		SB1	SB2	MT-III	MT-II
Chemical analysis (%)	SiO ₂	15.50	14.55	20.82	21.30
	Al ₂ O ₃	12.45	13.15	4.44	4.40
	Fe ₂ O ₃	1.41	1.25	2.12	4.30
	CaO	51.19	42.33	62.23	63.70
	MgO	0.99	2.14	3.24	3.00
	SO ₃	14.16	14.96	4.40	2.70
	Ignition loss	1.65	1.99	0.90	0.50
Physical analysis	Blaine fineness m ² /kg	642	775	504	365
Cubic compressive strength, MPa (psi)	3h	25.6(3720)	14.7(2130)	-	-
	6h	30.3(4400)	25.0(3630)	-	-
	1d	32.9(4780)	33.1(4810)	20.7(3010)	-
	3d	-	-	33.5(4860)	22.5(3270)
	7d	44.4(6440)	44.6(6470)	40.9(5930)	27.8(4040)
Mixture proportion kg/m ³ (lb/yd ³)	Cement	388(658)		481(815)	388(658)
	Fine aggregate	944(1600)		827(1402)	927(1571)
	Coarse aggregate	689(1168)		674(1142)	728(1234)
	Latex(includes 52% water)	121(205)		129(218)	121(205)
	Water	81(137)		97(164)	81(137)
Cost \$/m ² (\$/yd ²)	Treatment	78(65)		73(61)	73(61)
	Miscellaneous	28(23)		28(23)	28(23)
	Traffic control	9(8)		9(8)	55(46)
	Total	115(96)		110(92)	156(130)
	Life	30		30	30
	Life cycle cost % control	74		71	100

2.1.2 Performance evaluation

Though LMC-VES overlays have the similar mixture proportions as conventional LMC overlays, the specified minimum compressive strength of 2500 psi (ASTM C39) is achieved in 3hours for LMC-VES as shown in Figure 2-1, permitting Virginia DOT to open the lane more quickly to traffic. It was pointed out that curing temperature is a major factor in the development of compressive strength. Figure 2-2 shows the strength vs. age relationship for two batches of special blended cement cured in the lab at selected temperatures. Curing temperature below 10°C significantly retarded early strength development, and it was concluded that an overlay placed at these temperatures should not be subjected to traffic until adequate strength is obtained. VDOT's specification does not allow the placement of LMC at temperatures below 10°C.

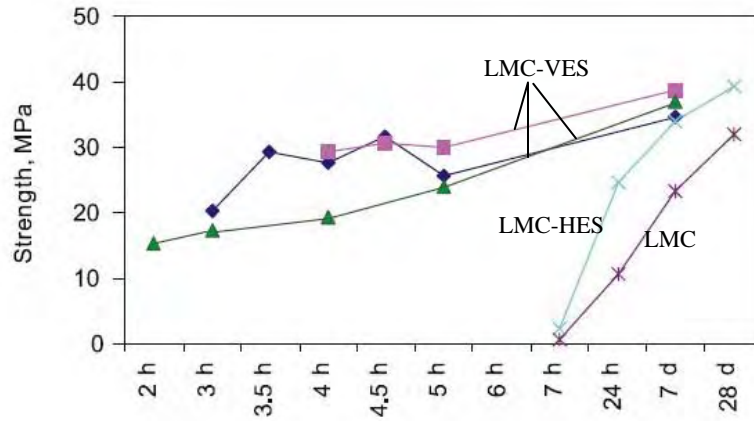


Figure 2-1 Strength vs. age for LMC-VES, LMC-HES and LMC

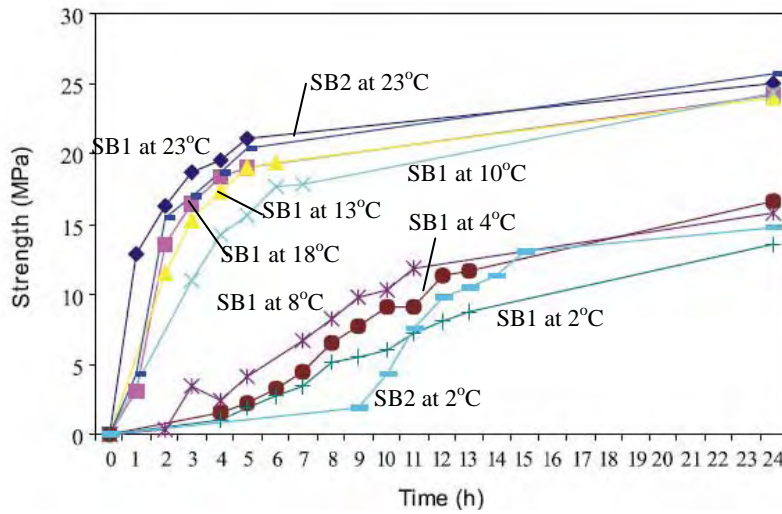


Figure 2-2 Strength vs. age for LMC-VES cured at selected temperatures

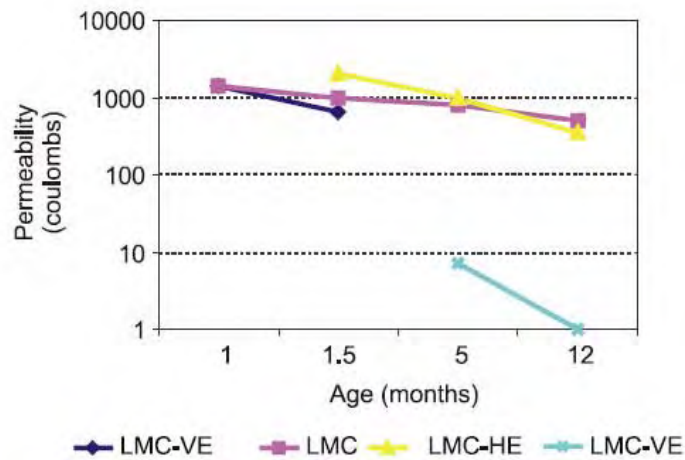


Figure 2-3 Permeability vs. age for LMC-VES, LMC-HES and LMC

LMC overlays usually extend the life of bridge decks by reducing the infiltration of chloride ions, Figure 2-3 shows the results of tests on cylinders prepared at the job sites, cured in cylinder molds for 24 hours, and cured in air thereafter until tested. The values for tests at 4 weeks, 6 weeks, 5 months, and 12 months of age are either about the same, or lower than conventional LMC.

A major factor controlling the life of an overlay is its bond strength with the old deck concrete. The results of tensile adhesion tests conducted on cores removed from the overlays showed that 82 ~90% of the failure area for tested cores was in the base concrete. The bond strength could be considered to be at least as high as the tensile strength of the base concrete. It indicated that LMC-VES overlays achieved high bond strengths with the appropriate selection and use of surface preparation equipment and procedures, mixture proportions, and placement and curing procedures.

High performance concrete overlays are susceptible to plastic shrinkage cracking. Plastic shrinkage cracking occurred in the two bridge overlays placed in 1997. However, LMC-VES overlays are less prone to cracking because they have less shrinkage than LMC and therefore can be more durable. The length change (ASTM C157) of LMC-VES specimens at 170 days is approximately 0.02%, compared with 0.06% for LMC specimens.

2.1.3 Cost analysis

Table 2-2 provides the comparison of four overlay systems. The special blended cement required for the LMC-VES costs 400 percent more and the Type III cement used in the LMC-HES costs 20 percent more than Type I/II cements used in the conventional LMC overlays. These cement increase the cubic meter cost of the concrete by approximately \$120 and \$9, respectively. These costs are more than offset by the large savings in the cost of traffic control. It was estimated that the cost for traffic control at five million per year on deck rehabilitation can be saved up to \$1.25 million per year by using LMC-VES overlays.

Table 2-2 Cost data of LMC, LMC-HES and LMC-VES

Overlay type		LMC-VES	Epoxy	LMC-HES	LMC
Cost \$/m ² (\$/yd ²)	Treatment	78(65)	29(24)	73(61)	73(61)
	Miscellaneous	28(23)	0(0)	28(23)	28(23)
	Traffic control	9(8)	9(8)	9(8)	55(46)
	Total	115(96)	38(32)	110(92)	156(130)
	Life	30	15	30	30
	Life cycle cost	115(96)	56(47)	110(92)	156(130)
	% control	74	36	71	100

2.2 Evaluation of various corrosion protection methods in Colorado

2.2.1 Effectiveness evaluation of corrosion protection methods

A CDOT research project reported comparisons of different corrosion protection methods used in Colorado (Xi et al. 2004). Nine bridges with different corrosion protection systems were inspected to study the effectiveness of different protection methods, including asphalt overlay with membrane (2 bridges), epoxy coated rebars (ECR) and corrosion inhibitor (2 bridges), impressed-current cathodic protection method (2 bridges), and sacrificial anode cathodic protection method with asphalt overlays (3 bridges). Base on the inspections, the following conclusions were drawn:

(1) Application of the cathodic protection method is quite effective in prolonging the life of the bridge decks that would otherwise need to be replaced. Considering the side effect of cathodic protection method, it was suggested that the performance of the cathodic protection system should be monitored continuously.

(2) Three cathodic protection systems were used in Colorado: sprayed zinc, impressed current, and sacrificial anode. The available results were not sufficient to conclude which system is more effective.

(3) The rebar protected by the concrete cover with corrosion inhibiting admixtures is more vulnerable to corrosion than that of ECR when significant cracks are present in the deck. The effectiveness of corrosion inhibiting admixture depends on the low ratio of chloride-to-nitrite ion, which cannot be maintained when the concentration of chloride ion continuously increases due to the presence of cracks.

(4) The effectiveness of a membrane heavily depends on service time, traffic load, and weather conditions. It was recommended that the effectiveness of waterproofing membranes should be studied based on bridge deck conditions.

2.2.2 Life-cycle cost analysis of corrosion protection methods

A CDOT research project reported life-cycle cost analysis of different corrosion protection methods used in Colorado (Hearn and Xi 2007). 82 decks were selected for the purpose of cost evaluation and comparison. Four different types of bridge decks were include:

(1) 19 decks built in 1980 that are protected with waterproofing membranes and asphalt overlay. These decks have uncoated steel reinforcement (Element 14 as shown in Table 2-3).

(2) 25 decks built between 1969 and 1975. These decks have uncoated reinforcing steel. 22 of these decks have been rehabilitated with rigid overlay. 11 were rehabilitated in 1999, 10 in 1995 and 1 in 1989 (Element 22 as shown in Table 2-3).

(3) 23 decks built in 1993 that are protected with penetrating sealers. These decks have epoxy-coated reinforcing steel (Element 23 as shown in Table 2-3).

(4) 15 decks built in 1991 that are protected with waterproofing membrane and asphalt overlay. These decks have epoxy-coated reinforcing steel (Element 26 as shown in Table 2-3).

The lengths of service life of bridge decks were extrapolated from trends observed in deck condition ratings based on CDOT bridge files. The service life is taken as the time in years for a bridge deck to deteriorate from new condition to condition rating “5.” The estimated service life for the four types of bridge decks are given in Table 2-3. Deck types in this study have median service life values that range from 31 years for decks having epoxy-coated reinforcing steel protected by sealers to 56 years for decks protected with waterproofing membrane and asphalt overlay.

Table 2-3 Estimated service lives of decks

Deck type	Service life estimate (years)		
	minimum	average	median
Element 14	31	78	56
Element 22 (No rehab)	19	34	31
Element 22 (rehab'd)	27	39	35
Element 23	15	29	31
Element 26	27	44	35

Deck costs are computed using methods presented in NCHRP Report 483 “Bridge Life Cost Analysis.” Costs for bridge decks are evaluated as initial costs, present values, discounted annualized costs, and annualized cost without discount factor. Considering the cost of materials only, decks with uncoated reinforcing steel and without protection against corrosion offer the least cost. Considering project costs, decks with uncoated reinforcing steel protected by waterproofing membrane offer the least cost.

2.3 Economic analysis of corrosion protection methods in other states

2.3.1 Cost analysis

(Kepler et al. (2000)) conducted a systematic study and compared the costs of various corrosion protection systems including topical protection systems. The costs of corrosion protection system for a concrete structure is considered to be a combination of the initial cost of the system, and maintenance, operation, and/or repair costs that occur within the service life of the structure.

For the purpose of economic evaluation, an economic life of 75 years is used to compare systems with useful lifetimes in excess of designed service lives (40 years). A typical 9 in. thick bridge deck with 2 in. of concrete cover over the top mat of steel is used, with the following exceptions: a thickness of 8 in. is used for bridges with no corrosion protection and black steel reinforcement, and a thickness of 8.5 in. is used for bridges with epoxy coated steel reinforcement as the only corrosion protection method. When overlays are included, the total deck thickness with the overlay remains 9 in.

The costs used to determine the current costs for each corrosion protection system in the economic analysis are given in Tables 2-4 through 2-7. All figures are for the costs in 1999 U.S.

dollars. Times to repair and current costs for the various corrosion protection systems were obtained from literature and by talking to engineers at state and province transportation agencies. Repair costs include actual repair or material costs and incidental costs such as mobilization, traffic control, and repairs and improvements to other parts of the bridge (drains, barrier rails and approaches). Costs for other major repairs done at the same time, such as painting, expansion joints, or rockers, were not included in the total costs.

Table 2-4 Concrete Prices

Type of overlay	Cost \$/m ³
Ordinary concrete for bridge decks	425
Silica fume concrete	700

Table 2-5 Overlay Prices

Type of overlay	Thickness/mm	Cost \$/m ²
Low-slump concrete overlay	50	30
Silica fume concrete overlay	40	54
LMC overlay	45	66
Epoxy-polymer overlay	7	47
Bituminous concrete pavement	80	5.32

Table 2-6 Reinforcement Prices

Type of reinforcement	Cost \$/kg
Grade 420(black)	1.30
Grade 420 (epoxy-coated)	1.45
Stainless steel	5.51
Stainless steel-clad	1.76
Galvanized	1.63

Table 2-7 Repair Prices

Type of repair	Cost \$/m ²
Remove concrete (slab)	7
Remove bridge pavement	6-12
Machine preparation of area	21
Cold plane bituminous concrete	24
Full depth repair (about 6% of deck)	267
Partial depth repair (about 22% of deck)	190

Five years is added to the time-to-repair when epoxy-coated reinforcement is combined with silica fume concrete and silica fume, low slump, and polymer concrete overlays. No time is added to the time-to-repair of bridge decks using epoxy-coated reinforcement in combination with LMC overlays, waterproof membranes, or corrosion inhibitors over the times for decks with the same systems and black steel.

The present value for the costs of each corrosion protection method is calculated assuming discount rates of 2%, 4%, and 6%. The discount rate chosen for the comparison is important because, at a lower discount rate, the initial cost is relatively less important, maintenance costs are more important, and the present value for future cost is influenced less by time than at a higher discount rate. The total present worth of each corrosion protection method is calculated by adding the initial cost to the present values of the repair and replacement costs, the maintenance and operation costs, as shown in Table 2-8.

Table 2-8 Present value costs for corrosion protection options

No.	Corrosion protection method	New deck cost \$/m ²	Repair 1 cost \$/m ²	Time to repair 1 years	Repair 2 cost \$/m ²	Time to repair 2 years	Repair 3 cost \$/m ²	Time to repair 3 years	Repair 4 cost \$/m ²	Time to repair 4 years	Maint. & opp. cost \$/m ² /yr	Present value of costs at 2% \$/m ²	Present value of costs at 4% \$/m ²	Present value of costs at 6% \$/m ²
1	No protection	125	259	25	259	50						379	259	199
2	Epoxy coated steel (ECS)	135	259	30	259	55						365	245	191
3	Stainless steel reinforcement	277										277	277	277
4	Stainless steel-clad reinforcement	155										155	155	155
5	Galvanize reinforcement	150	259	27	259	52						394	274	216
6	Silica fume concrete	204	259	30	259	60						426	308	257
7	Low-slump overlay	150	242	30	242	55						365	253	202
8	Silica fume overlay	178	266	30	266	55						414	291	235
9	LMC overlay	188	278	25	278	50						461	331	268
10	Polymer concrete overlay	125	252	25	201	40	252	55				454	291	213
11	Preformed sheet membrane	164	169	20	169	40	169	60				406	292	238
12	Spray applied liquid membrane	204	27	20	27	35	214	50	27	70		322	255	228
13	Torch applied membrane	173	183	20	183	40	183	60				435	312	253
14	Hot rubberized asphalt membrane	160	27	20	170	40	27	60				263	210	186
15	Linseed oil	142	6	3	6	6	255	25	255	55		427	299	238
16	Silane	144	8	7	8	14	255	35	250	60		387	255	199
17	Titanium mesh cathodic protection	250	264	30	264	60					1	515	380	320
18	Slotted anode cathodic protection	277	291	30	291	60					1	565	418	353
19	Sprayed zinc cathodic protection	248	108	15	108	30	108	45	108	60	1	504	394	339
20	Electrochemical chloride extraction	125	355	25	355	55						461	299	222
21	Calcium nitrite	145	259	40	259	70						327	216	175
22	Organic corrosion inhibitor	145	259	37	259	67						338	224	180
23	ECS + silica fume concrete	185	259	35	259	70						379	267	223
24	ECS + low-slump overlay	154	242	35	242	60						349	238	193
25	ECS + silica fume overlay	182	266	35	266	60						396	275	225
26	ECS + LMC overlay	192	278	25	278	50						465	335	272
27	ECS + polymer concrete overlay	135	252	30	201	45	252	60				433	271	201
28	ECS + preformed sheet membrane	168	169	20	169	40	169	60				410	296	242
29	ECS + spray applied liquid membrane	208	27	20	27	35	214	50	27	50		326	259	232
30	ECS + torch applied membrane	177	183	20	183	40	183	60				439	316	257
31	ECS + hot rubberized asphalt membrane	164	27	20	170	40	27	60				267	214	190
32	ECS + calcium nitrite	149	259	40	259	70						331	220	179
33	ECS + organic corrosion inhibitor	149	259	37	259	67						342	228	184

2.3.3 Analysis results

(1) At a low discount rate (2%), solid stainless steel reinforcement is a cost-effective option compared to other options, but at higher discount rates (4%+), the present value cost of a deck with solid stainless steel is significantly higher than that of an unprotected deck for the 75 year economic life considered in this study.

(2) Stainless steel clad reinforcement is much less expensive than solid stainless steel reinforcement. Assuming that a bridge deck constructed using stainless steel-clad reinforcement does not require any repairs within its 75 year economic life. The present value of the cost of a bridge deck built with stainless steel-clad reinforcement is significantly lower than the present value for the cost of any other corrosion protection system.

(3) Low-slump, silica fume and LMC overlays have similar service lives, as long as they are constructed and cured properly. Among the three, low-slump overlays are the least expensive one.

(4) Epoxy-coated steel can be combined with membranes, overlays, corrosion inhibitors, or silica fume concrete, and is a low cost backup to these corrosion protection methods.

(5) Preformed sheet membranes with asphalt overlays have been used in Colorado. Hot rubberized asphalt membranes and spray-applied liquid membranes are less expensive options, Comparing with preformed sheet membranes.

3. TEST RESULTS OF CHLORIDE PROFILES IN CONCRETE CORES

3.1 The purpose of the inspection

Since chloride concentration in the concrete has been considered as an important indicator for the effectiveness of topical protection methods, we selected and inspected several bridge decks, took concrete cores, and obtained chloride concentrations at various depths. There are three stages in the rebar corrosion process.

1. The chloride concentration near rebars is lower than the critical value and the corrosion of rebars has not started.
2. The chloride concentration near rebars just reached the critical value and the corrosion of rebars just started.
3. The chloride concentration near rebars is much higher than the critical value and the corrosion of rebars already started sometimes ago.

The outcomes of the inspection will show which stage a bridge deck is in, and a comparison of the outcomes will confirm the effectiveness of the topical protection methods applied on the selected bridges.

Because of limited funding available for the project, only five bridges were selected for inspection in the project:

C-16-DI – Located 6 miles west of Loveland, built in 1980, with 3 in. of asphalt wearing surface.
G-22-BG – I-70, MP362.2, east of Limon, built in 1975, with 2 in. of integrated concrete surface.
G-22-BH – I-70, MP362.2, east of Limon, built in 1975, with 3 in. of integrated concrete surface.
G-22-BJ – I-70, MP357.77, west of Limon, built in 1972, without overlay.
G-22-BL – I-70, MP361.74, east of Limon, built in 1975, without overlay.

C-16-DI and G-22-BH were selected to compare the effectiveness of asphalt and concrete overlays (with the same 3.0 in. thickness), G-22-BG and G-22-BH were selected to compare the effect of thickness of concrete overlay (3.0 in. and 2.0 in. thickness), and G-22-BJ and G-22-BL were selected to compare with G-22-BG and G-22-BH on the effectiveness of concrete overlay.

Concrete cores were taken from the bridge decks, and chloride concentration profiles in the concrete cores were obtained in our laboratory. As shown in Section 3.3, all decks are in the first stage of the corrosion process, i.e., the corrosion of rebar has not started. Therefore, just comparing the current inspection results is not sufficient to evaluate the effectiveness of the topical protection methods. What we did in this project is to predict the future performance of the decks based on the current inspection results, and then the effectiveness of the protection methods is compared by using the predicted future performances. In order to predict the future performance of the protection methods, the measured concentration profiles will be used in the next chapter to determine chloride diffusivity of the concretes, the diffusivities determined in this way will be used in a simulation model to estimate future chloride profiles in the decks, and then the

effectiveness of the topical protection methods can be evaluated by comparing the predicted future chloride profiles.

3.2 Coring concrete samples

Bridge G-22-BJ and Bridge G-22-BL

Figure 3-1 shows the bridge G-22-BL. Figure 3-2 shows the coring site on the bridge G-22-BJ. For Bridge G-22-BJ, two concrete cores were taken. One core was taken from the traffic lane, 4 ft inside of the shoulder line. The core was broken into two portions during the drilling process. The top portion was numbered as J↑1L, and the bottom portion J↑2L. The other core was taken from the shoulder, 5 ft outside of the shoulder line. The core was numbered as J↑S. There was no steel bar found in the concrete cores.



Figure 3-1 Bridge G-22-BL



Figure 3-2 Coring concrete samples on Bridge G-22-BJ. It was broken in the traffic lane

The notations used for numbering the cores are as follows:

J – G-22-BJ

L (the first L) – G-22-BL

↑ - Points to the top surface.

1 and 2 – top and bottom portion.

L (the last L) – Traffic lane.

S – Shoulder.

For Bridge G-22-BL, two concrete cores were taken. One core was taken from the traffic lane, 4.5 ft inside of the shoulder line. The core was broken into two portions during the drilling process. The top portion was numbered as L↑1L, and the bottom portion was numbered as L↑2L. The other core was taken from the shoulder, 4.5 ft outside of the shoulder line. The core was numbered as L↑S. There was no steel bar found in the concrete cores. The locations of the drilled concrete cores are summarized in Table 3-1.

Table 3-1 Location of concrete cores

Concrete Core	Location	
J↑S	G-22-BJ West Bound	Shoulder
J↑1L		Traffic lane (Top)
J↑2L		Traffic lane (Bottom)
L↑S	G-22-BL East Bound	Shoulder
L↑1L		Traffic lane (Top)
L↑2L		Traffic lane (Bottom)

Bridge G-22-BG, G-22-BH

For Bridge G-22-BG, The core was taken 4.1ft inside of the shoulder line. The depth from surface to steel bars = 3.3 in. (The overlay thickness was designed to be 2.0 in.). Figure 3-3 shows G-22-BG.

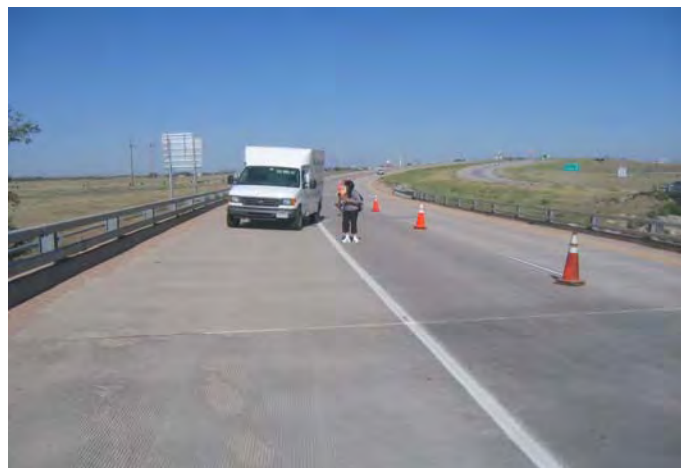


Figure 3-3 Bridge G-22-BG. CDOT traffic control team

For Bridge G-22-BH, the core was taken 3.5ft inside of the shoulder line. When taking core on the site, an interface was found between new overlay and old concrete on the depth of 3.6 in. (The overlay thickness was designed to be 3.0 in.). Figure 3-4 shows the coring samples on Bridge G-22-BH.



Figure 3-4 Coring samples on Bridge G-22-BH

Bridge C-16-DI

Figure 3-5 shows the traffic control and overview of Bridge C-16-DI. For Bridge C-16-DI, two concrete samples were taken. One core was taken from the traffic lane, 3.5 ft inside of the shoulder lane. The other core was taken from the shoulder, 5 ft outside of the shoulder line.



Figure 3-5 Bridge C-16-DI

When taking cores on the site, an interface was found between asphalt overlay and old concrete on the depth of 3 in. (The overlay thickness was designed to be 3.0 in.). We cut off the layer of asphalt overlay before doing the test for chloride concentration profile in our lab. Figure 3-6 shows the coring site on the Bridge C-16-DI.



Figure 3-6 Taking coring samples

3.3 Chloride profiles of concrete cores

Bridge G-22-BJ, G-22-BL

The chloride profiles of concrete cores were shown in Figures 3-7 through 3-10. In Figures 3-8 and 3-10, the profiles from two broken cores are shown as connected.

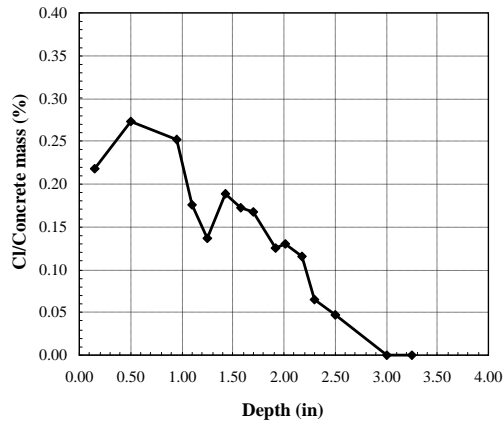


Figure 3-7 Chloride profile in the concrete core from G-22-BJ shoulder (J↑S)

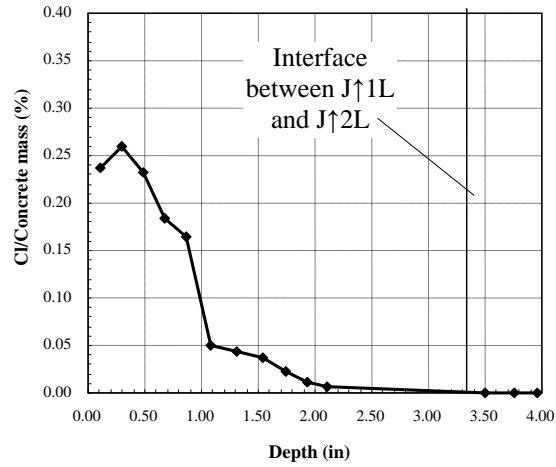


Figure 3-8 Chloride profiles in concrete cores from G-22-BJ traffic lane (J↑1L and J↑2L)

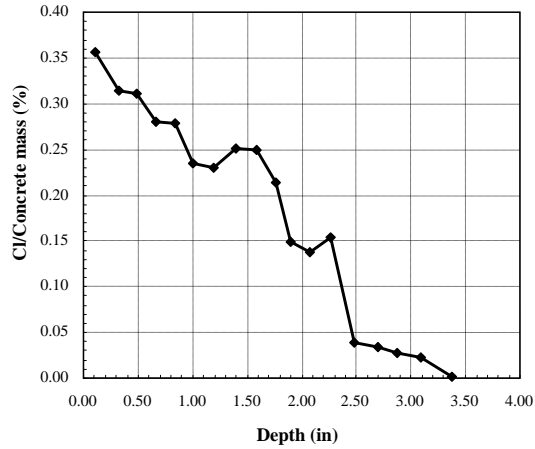


Figure 3-9 Chloride profile in the concrete core from G-22-BL shoulder (L↑S)

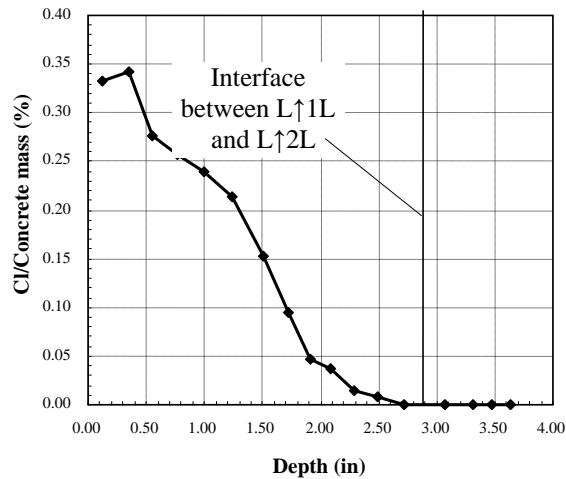


Figure 3-10 Chloride profiles in concrete cores from G-22-BL traffic lane (J↑1L and J↑2L)

Bridge G-22-BG, G-22-BH

The chloride profiles of concrete cores for Bridge G-22-BG and G-22-BH are shown in Figures 3-11 and 3-12.

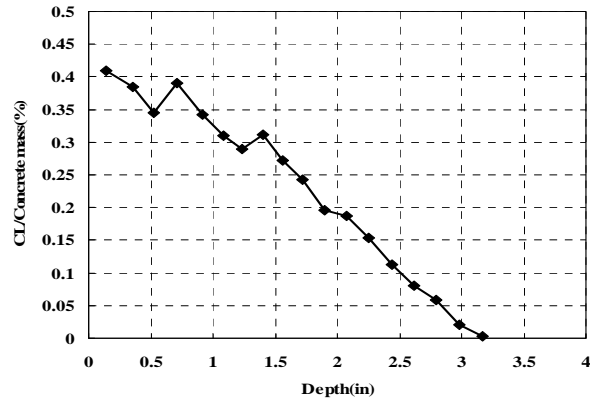


Figure 3-11 Chloride profile for Bridge G-22-BG

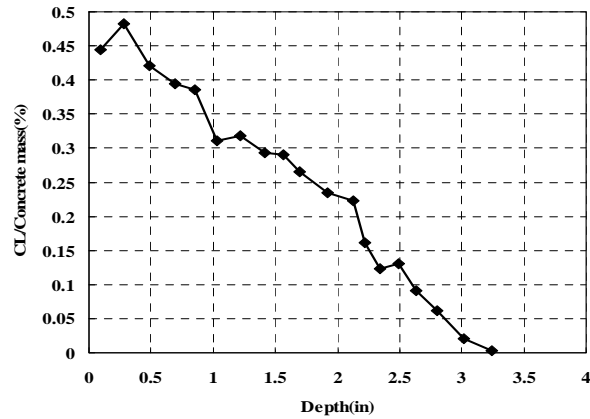


Figure 3-12 Chloride profile for Bridge G-22-BH

Bridge C-16-DI

The chloride profiles of concrete cores for the shoulder lane and traffic lane are shown in Figures 3-13 and 3-14.

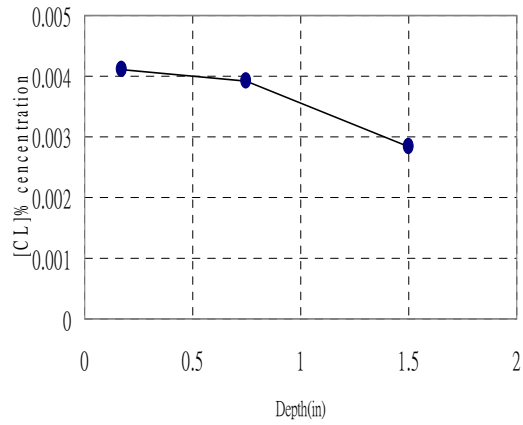


Figure 3-13 Chloride profile in shoulder lane

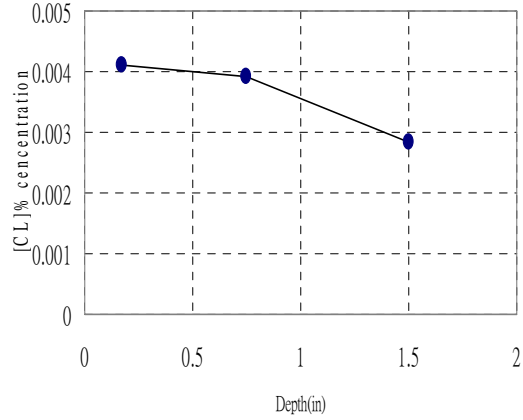


Figure 3-14 Chloride profile in traffic lane

4. PREDICTION OF ONSET OF CORROSION FOR THE BRIDGE DECKS INSPECTED

From the inspection results, we concluded that all bridge decks inspected are in good shape. No corrosion damage of rebars has occurred in the decks. This conclusion was based on the facts that 1) no corroded rebars were found in the coring process, and 2) the chloride concentration near rebars are much lower than the critical chloride concentration (see details later). All decks are in stage (1) as described in Chapter 3. In this case, the inspection results cannot be directly used to evaluate the effectiveness of the topical protection methods.

Although all decks are in stage (1), the inspection results can be used to predict corrosion initiation time in the future. The predicted results can be used to estimate the performance of the topical protection methods. To this end, we need to have an accurate model to predict future changes of chloride concentration profile for the bridge deck. The current inspection results are very important for the development of the prediction model, because they can be used to calibrate the model parameters.

In the next sections, we will first describe the diffusion model for characterize the chloride penetration process, then explain how to use the present inspection results to determine (calibrate) several important parameters of the diffusion model, and finally, use the calibrated model to predict the chloride concentration profiles in the future and thus predict corrosion initiation time of the bridge decks.

4.1 Chloride diffusion model

The penetration of chloride ions into concrete bridge decks can be described by diffusion equation (4.1). Using the so-called Boltzman transformation as shown in (4.2), equation (4.1) can be solved and the solution is shown in equation (4.3)

$$\frac{\partial C}{\partial t} = D_{cl} \frac{\partial^2 C}{\partial x^2} \quad (4.1)$$

$$\zeta = \frac{x}{\sqrt{4D_{cl}t}} \quad (4.2)$$

$$\frac{C - C_0}{C_{ini} - C_0} = erf\left(\frac{x}{\sqrt{4D_{cl}t}}\right) \quad (4.3)$$

In which

C = chloride concentration in concrete in % weight of concrete

C_0 = surface concentration of concrete in % weight of concrete

C_{ini} = initial concentration of chloride in the concrete in % weight of concrete

D_{cl} = chloride diffusion coefficient in cm^2/days . In the following calculations, we change it to in^2/days

x = depth in cm. In the following calculations, we change it to in.

t = time in days

erf is called error function. The equation (4.3) is called the error function solution of linear diffusion equation (4.1).

After the diffusion coefficient of concrete is determined and the critical chloride content is known, the error function solution can be used to predict the corrosion initiation time for the structure

$$t_{\text{initiation}} = \frac{X_{\text{cover}}^2}{4D_{cl}} \left[erf^{-1} \left(\frac{C_0 - C_{cri}}{C_0 - C_{ini}} \right) \right]^2 \quad (4.4)$$

C_{cri} = critical chloride concentration in concrete in % weight of concrete

X_{cover} = thickness of concrete cover in cm. In the following calculations, we change it to in.

4.2 Calibration of two model parameters and prediction of corrosion initiation times

There are two parameters, namely the diffusion coefficient, D_{cl} , and the surface chloride concentration, C_0 , in above equations that need to be determined by specific conditions of each deck. The diffusion coefficient of an existing concrete deck depends on many parameters related to initial conditions of the concrete and environmental conditions of the bridge deck, including concrete mix design, construction practice, traffic loading, and environmental loading. For different bridge decks, the diffusion coefficient of concrete is different. The surface chloride concentration depends on the amount of deicing salts applied on the deck. Since deicing salts are not applied on concrete decks all year long, the surface concentration is an average value. The best way to determine the two parameters is to calibrate the theoretical solution with the measured concentration profiles. Once these two parameters are calibrated, the chloride concentrations at any depth in bridge decks at any time can be calculated by equation (4.3), and the corrosion initiation time of rebars in the future can be predicted by equation (4.4). So, the next step is to use measured data to determine the diffusion coefficient and surface chloride concentration.

Table 4-1 Information on the five bridges

No	Bridge	Location	Year	Surface (Mat & Thick)	
1	G-22-BJ	I-70, MP357.77, west of Limon	1972	IC	3.3in
2	G-22-BL	I-70, MP361.74, east of Limon	1975	IC	2.9in
3	G-22-BG	I-70, MP362.2, east of Limon	1975	IC	2in
4	G-22-BH	I-70, MP362.2, east of Limon	1975	IC	3in
5	C-16-DI	6 MI W of Loveland	1980	Asphalt	3in

Table 4-1 summarizes useful information for the five inspected bridges. Table 4-2 lists several available critical chloride contents, C_{cri} , in the literature. C_{cri} is not a constant because the onset of corrosion of rebars does not depend only on C_{cri} . This is because the corrosion process of rebars is an electrochemical process. There are other influential parameters playing important roles in the electrochemical process, such as moisture content of concrete, pH value of pore solution near the rebar. In this study, we used two different values for C_{cri} . One is considered as the regular value $C_{cri} = 0.05\%$, which has been used widely in the research community; and the other is considered as the strictest critical value $C_{cri} = 0.021\%$.

Table 4-2 Codes

	Critical chloride content
Berke (1986)	0.039%-0.043%
Browne (1982)	0.055%
FHWA	0.0413%
ACI (1994)	0.021%
Cady and Weyers (1992)	0.025-0.05%

Calibration method

The calibration criteria for the two parameters D_{Cl} and C_0 is to find the optimum values of D_{Cl} and C_0 that lead to the best curve fitting of the concentration profiles between the theoretical prediction (using equation (4.3)) and the measured data. In order to do so, we tried many pairs of values for D_{Cl} and C_0 , compared model predictions with the measured data, and selected the optimum values for D_{Cl} and C_0 . Then, the two optimum values were used in equation (4.4) to predict the corrosion initiation times for the future. The following figures and tables show the calibration and prediction results for the five bridges in the shoulder and traffic lanes.

One can see that the predicted corrosion initiation times are all very large, which will be explained in detail in the next section.

Bridge G-22-BJ

For the chloride concentration in shoulder lane, the results are:

$$C_0 = 0.35\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The depth from surface to steel bars = 3.3 in.

The calibrating curves and the prediction on corrosion initiation times are shown in Figure 4-1 and Table 4-3. In Figure 4-1, the unit for D_{Cl} is cm^2/days , but we change it to in^2/days in Table 4-3.

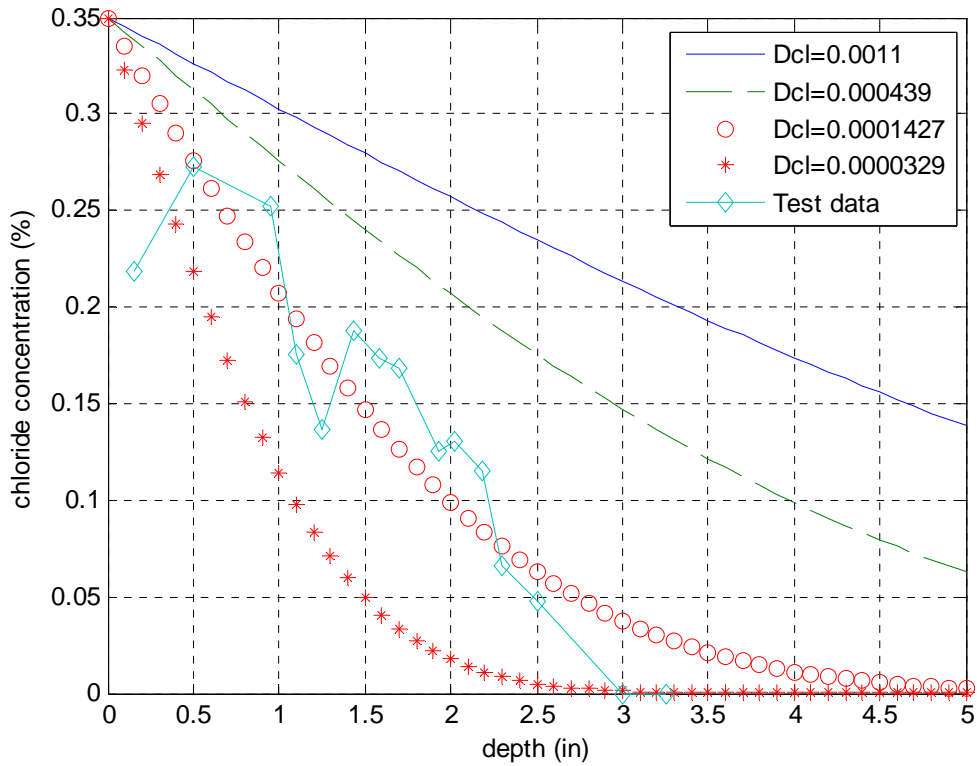


Figure 4-1 Chloride concentration profiles

Table 4-3 Prediction of corrosion initiation times for Bridge G-22-BJ (shoulder lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) (in ² /days)	Prediction time(years)
0.05	0.35	0.00002212	331
0.021	0.35	0.00002212	201

For the chloride concentration in traffic lane, the results are:

$$C_0 = 0.3\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The depth from surface to steel bars = 3.3 in.

The calibrating curves and the prediction on corrosion initiation times are shown in Figure 4-2 and Table 4-4. In Figure 4-2, the unit for D_{Cl} is $cm^2/days$, but we change it to $in^2/days$ in Table 4-4.

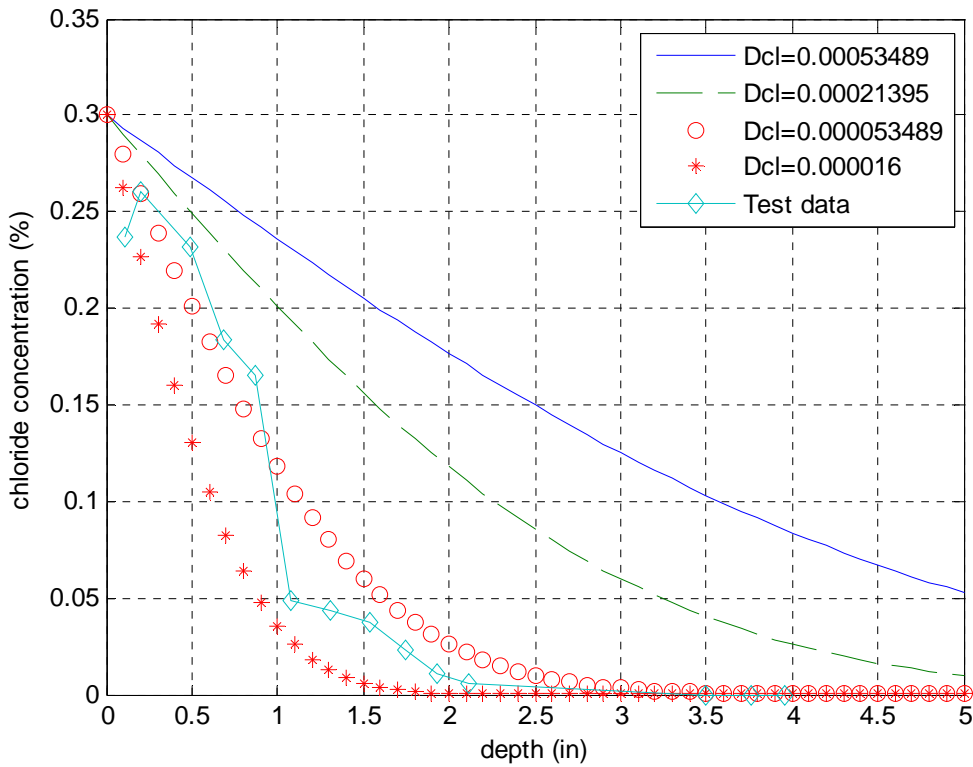


Figure 4-2 Chloride concentration profiles

Table 4-4 Prediction of corrosion initiation times for Bridge G-22-BJ (traffic lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) ($in^2/days$)	Prediction time(years)
0.05	0.3	0.0000083	941
0.021	0.3	0.0000083	548

Bridge G-22-BL

For the chloride concentration in shoulder lane, the results are:

$$C_0 = 0.45\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The depth from surface to steel bars = 2.9 in.

The calibrating curves and the prediction on corrosion initiation times are shown in Figure 4-3 and Table 4-5. In Figure 4-3, the unit for D_{Cl} is $cm^2/days$, but we change it to $in^2/days$ in Table 4-5.

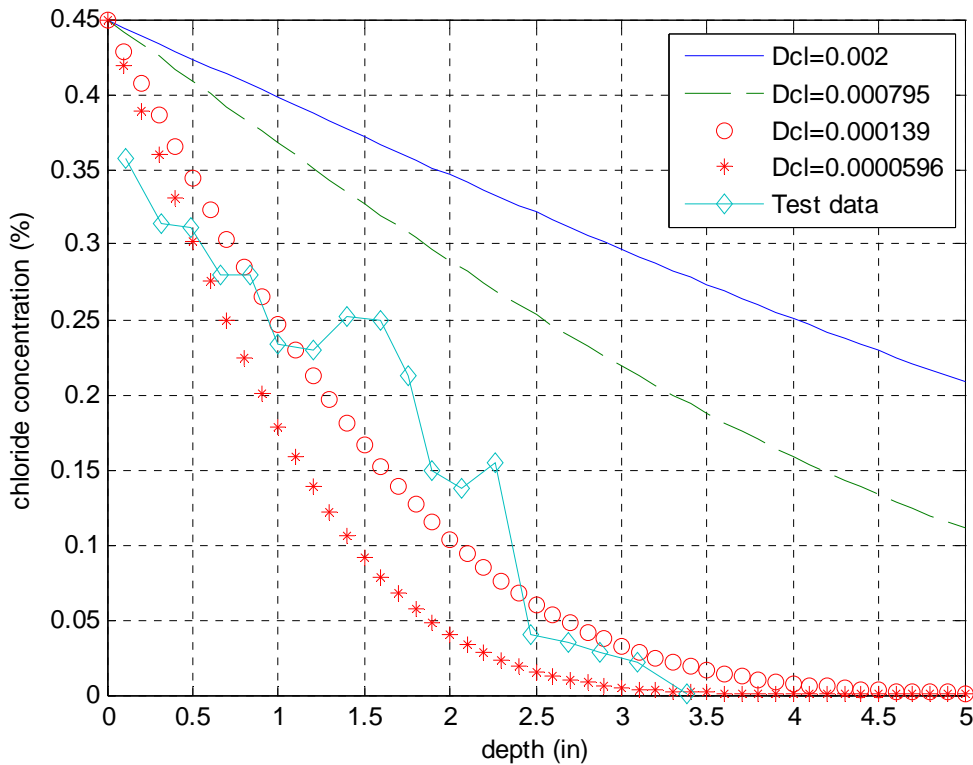


Figure 4-3 Chloride concentration curves

Table 4-5 Prediction of corrosion initiation times for Bridge G-22-BL (shoulder lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) ($in^2/days$)	Prediction time(years)
0.05	0.45	0.0000215	246
0.021	0.45	0.0000215	158

For the chloride concentration in traffic lane, the results are:

$$C_0 = 0.38\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The depth from surface to steel bars = 2.9 in.

The calibrating curves and the prediction on corrosion initiation times are shown in Figure 4-4 and Table 4-6. In Figure 4-4, the unit for D_{Cl} is $cm^2/days$, but we change it to $in^2/days$ in Table 4-6.

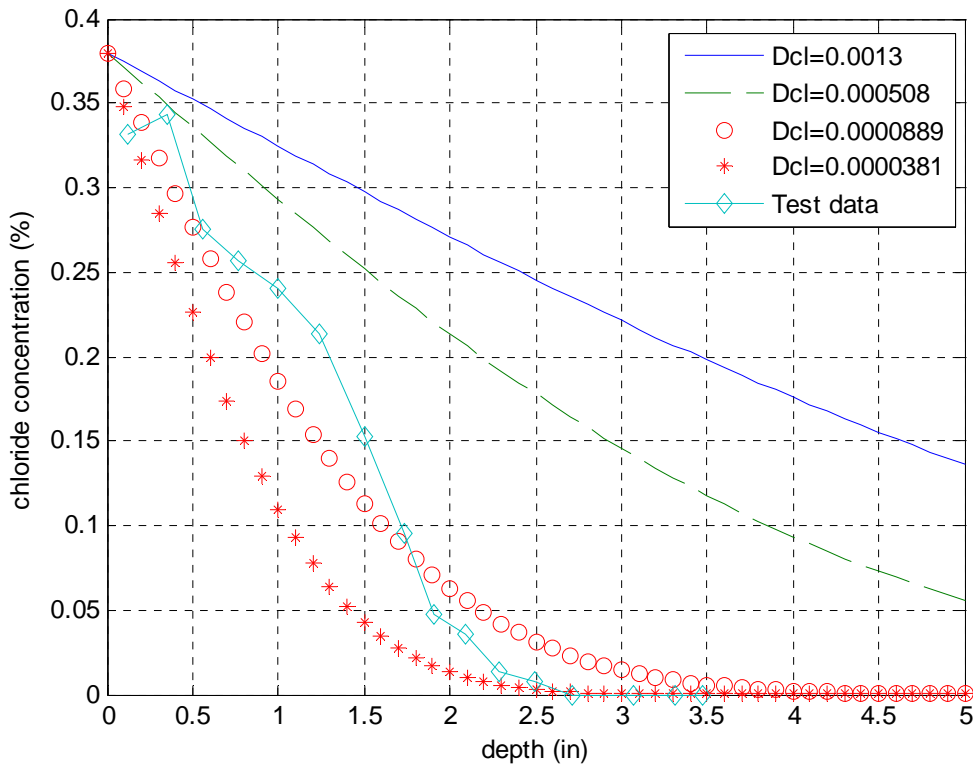


Figure 4-4 Chloride concentration curves

Table 4-6 Prediction of corrosion initiation times for Bridge G-22-BL (traffic lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) ($in^2/days$)	Prediction time(years)
0.05	0.38	0.0000138	368
0.021	0.38	0.0000138	158

Bridge G-22-BG

For the chloride concentrations, the results are:

$$C_0 = 0.6\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The depth from surface to steel bars = 3.3 in.

The calibrating curves and the prediction on corrosion initiation times are shown in Figure 4-5 and Table 4-7. In Figure 4-5, the unit for D_{Cl} is $cm^2/days$, but we change it to $in^2/days$ in Table 4-7.

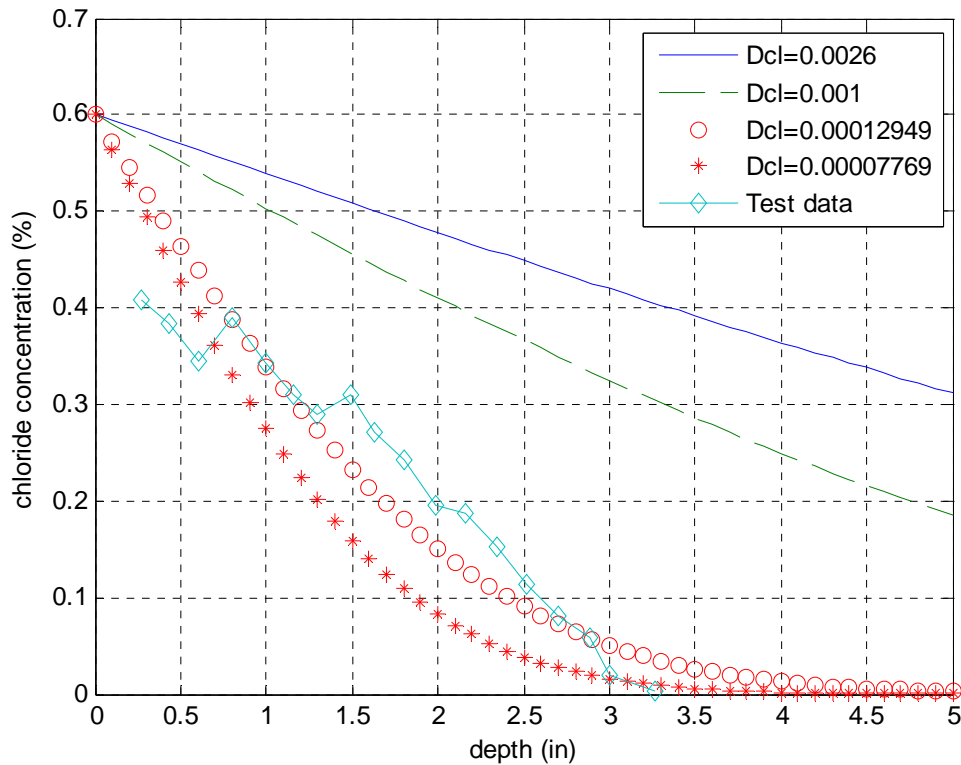


Figure 4-5 Chloride concentration curves

Table 4-7 Prediction of corrosion initiation times for Bridge G-22-BG

C_{cr} (%)	C_0 (%)	D_{cl} (current) ($in^2/days$)	Prediction time(years)
0.05	0.6	0.00002	253
0.021	0.6	0.00002	171

Bridge G-22-BH

For the chloride concentrations, the results are:

$$C_0 = 0.6\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

When taking cores on the site, an interface was found between new overlay and old concrete in the depth of 3.6 in. We considered the total depth from the surface to steel bar to be 6.6 in. The calibrating curves and the prediction of corrosion initiation times are shown in Figure 4-6 and Table 4-8. In Figure 4-6, the unit for D_{Cl} is cm^2/days , but we change it to in^2/days in Table 4-8.

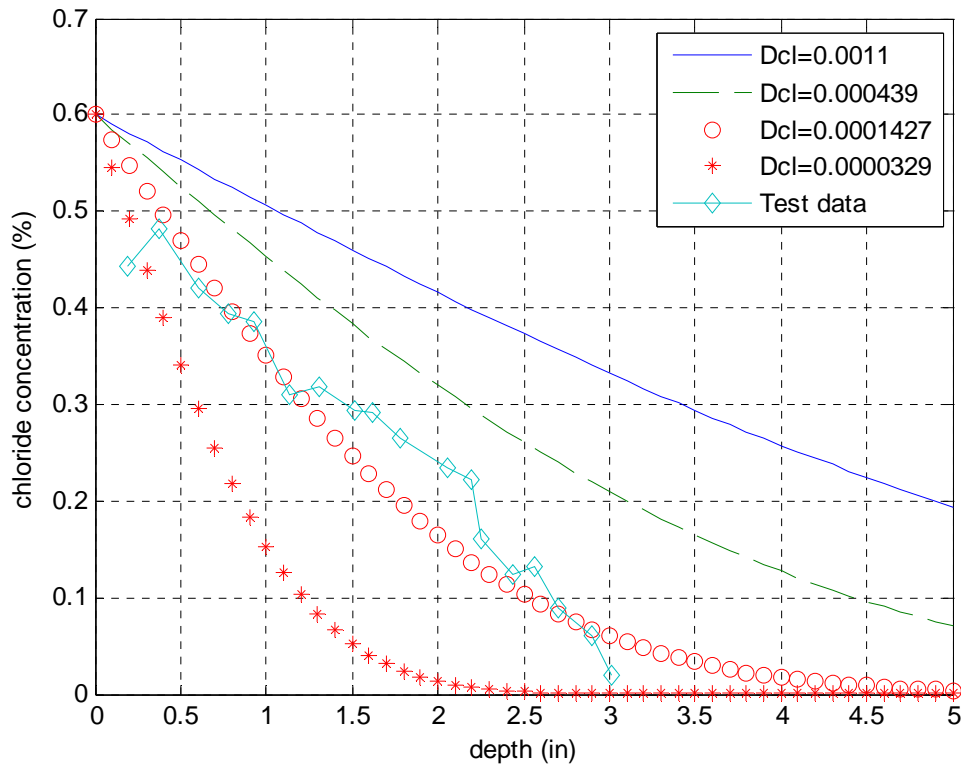


Figure 4-6 Chloride concentration curves

Table 4-8 Prediction of corrosion initiation times for Bridge G-22-BH

C_{cr} (%)	C_0 (%)	D_{cl} (current) (in^2/days)	Prediction time(years)
0.05	0.6	0.0000221	253
0.021	0.6	0.0000221	170

Bridge C-16-DI

For the chloride concentration in shoulder lane, the results are:

$$C_0 = 0.01\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

When taking cores on the site, an interface was found between asphalt overlay and old concrete in the depth of 3 in. We cut off the layer of asphalt overlay before conducting the chloride test in the lab. Therefore, we considered the total depth from the interface to steel bar to be 2.8 in. The calibrating curves and the prediction of corrosion initiation time are shown in Figure 4-7 and Table 4-9. In Figure 4-7, the unit for D_{Cl} is cm^2/days , but we change it to in^2/days in Table 4-9.

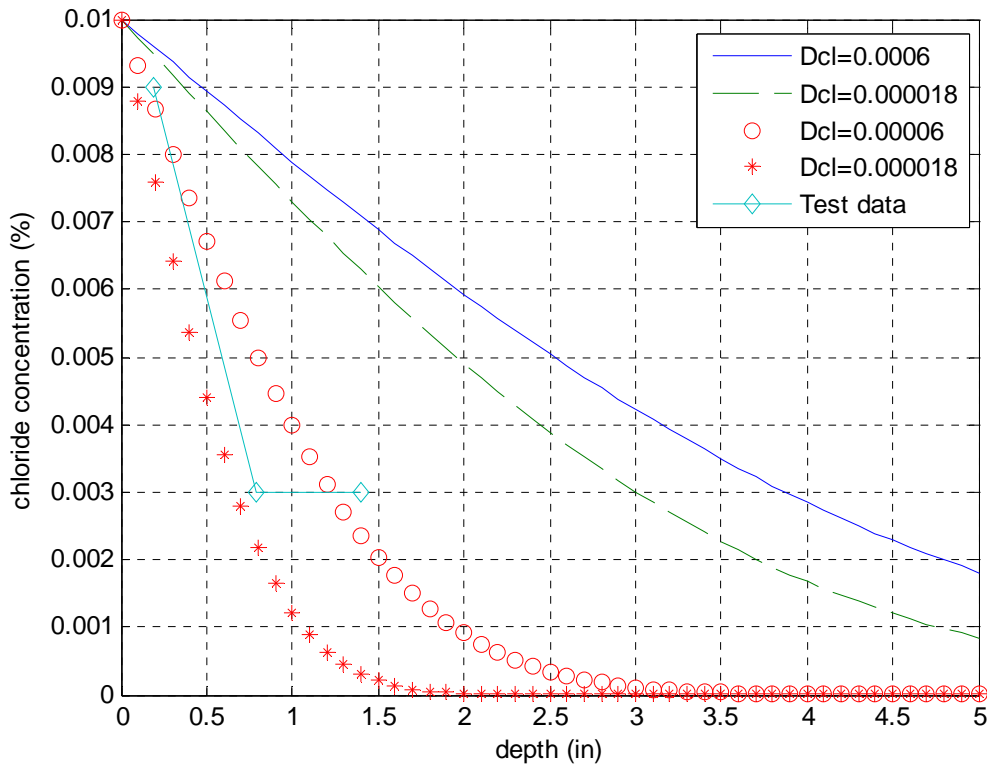


Figure 4-7 Chloride concentration curves

Table 4-9 Prediction of corrosion initiation times for Bridge C-16-DI (shoulder lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) (in^2/days)	Prediction time (years)
0.05	0.01	0.0000093	NaN
0.021	0.01	0.0000093	NaN

Where NaN means the calculated times are infinite and out of the range.

For the chloride concentration in traffic lane, the results are:

$$C_0 = 0.006\%$$

$$C_{cr} = 0.05\% , 0.21\%$$

$$C_{ini} = 0$$

The same situation happened as in the shoulder lane. We considered the total depth from the interface to steel bar to be 2.8 in. The calibrating curves and the prediction of corrosion initiation times are shown in Figure 4-8 and Table 4-10. In Figure 4-8, the unit for D_{Cl} is $cm^2/days$, but we change it to $in^2/days$ in Table 4-10.

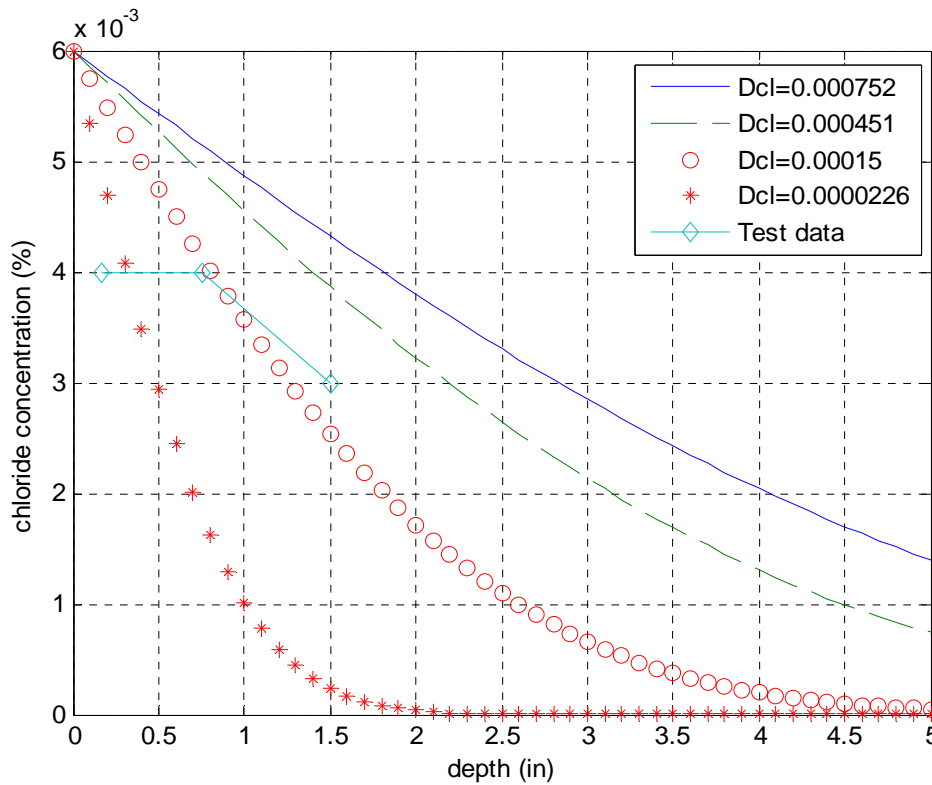


Figure 4-8 Chloride concentration curves

Table 4-10 Prediction of corrosion initiation times for Bridge C-16-DI (traffic lane)

C_{cr} (%)	C_0 (%)	D_{cl} (current) ($in^2/days$)	Prediction time(years)
0.05	0.006	0.0000233	NaN
0.021	0.006	0.0000233	NaN

Where NaN means the calculated times are infinite and out of range.

For Bridge C-16-DI, we measured three chloride concentrations in three different depths from both shoulder and traffic lanes. The measured concentrations from this bridge are much lower than other four bridges. The bridge decks had been covered by a thick layer of asphalt overlay (more than 3 in), which may be the reasons for the low chloride concentrations. In fact, even the surface chloride concentrations (0.01% and 0.006%) are much lower than the critical values (0.05% and 0.021%). This is why we only took readings at three different depths which are much less than the numbers of readings we did for other bridges.

4.3 Modification of the diffusion model

In the chloride diffusion model shown in equations (4.3) and (4.4), we considered chloride diffusion coefficient as a constant for each bridge (either from traffic or shoulder lane). The value obtained in the calibration is therefore the best estimated chloride diffusion coefficient for each concrete desk at the time of inspection. These estimated values for chloride diffusion coefficients can be considered as quite accurate for now. The concretes in the bridge decks are all in very good condition, therefore, the predicted corrosion initiation times are all very long, more than 100 or 200 years as shown in Tables 3 – 10. These unrealistic predictions are resulted from an unrealistic assumption used in the chloride diffusion model of equation (4.3), that is, the concrete will remain in the current good condition without any deterioration in the next 100 or 200 years. With various impacts of traffic loading, freezing-thawing, and drying-wetting cycles, the concretes will deteriorate, the diffusion coefficients will increase significantly, and the corrosion initiation times will be much shorter than those predicted in Tables 3 – 10.

In this section, the diffusion model of equations (4.3) and (4.4) will be modified to take into account the deterioration process of concrete. To this end, the diffusion coefficient in equation (4.1) will be considered as a time-dependent parameter (Xi et al. 2000; Mangat and Molloy 1994; Mangat and Limbachiya 1999)

$$D_{Cl} = D_i t^{-m} \quad (4.5)$$

D_i = a reference diffusion coefficient in cm^2/days . In the following calculations, we change it to in^2/days .

m = empirical constant representing the rate of deterioration of concrete, $m < 0$

Equation (4.5) shows that with increasing time, the diffusion coefficient D_{Cl} increases (since $m < 0$). The parameter m in equation (4.5) represents the deterioration process of concrete in bridge decks, which will be discussed later.

Combine equation (4.5) and equation (4.1) gives

$$\frac{\partial C}{\partial t} = (D_i t^{-m}) \frac{\partial^2 C}{\partial x^2} \quad (4.6)$$

Equation (4.6) is different from equation (4.1). The solution of equation (4.6) requires some further manipulation.

$$\frac{1}{(D_i t^{-m})} \frac{\partial C}{\partial t} = \frac{\partial^2 C}{\partial x^2} \quad (4.7)$$

Substituting $\partial T = (D_i t^{-m}) \partial t$ into Equation (4.7) gives

$$\frac{\partial C}{\partial T} = \frac{\partial^2 C}{\partial x^2} \quad (4.8)$$

This equation is similar to the linear diffusion equation (4.1), and thus the standard error function solution, equation (4.3), can be used here (with $D_{Cl} = 1$).

$$C = C_0 \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{T}} \right) \right] \quad (4.9)$$

Integrate $\partial T = (D_i t^{-m}) \partial t$ gives

$$T = \int_0^t D_i t^{-m} dt \quad (4.10)$$

And thus

$$T = \frac{D_i}{1-m} t^{1-m} \quad (4.11)$$

The new variable T can be considered as a new time scale, which is different from the real time scale, t . Substituting Equation (4.11) into Equation (4.9) yields

$$C = C_0 \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{\frac{D_i}{1-m} t^{1-m}}} \right) \right] \quad (4.12)$$

We can also rewrite equation (4.4) in the following form

$$t_{initiation}^{1-m} = \frac{(1-m)X_{cover}^2}{4D_i} \left[\operatorname{erf}^{-1} \left(\frac{C_0 - C}{C_0 - C_{ini}} \right) \right]^{-2} \quad (4.13)$$

The difference between the corrosion initiation times predicted by equation (4.4) and equation (4.13) is that equation (4.13) takes into account the change of diffusion coefficient due to the deterioration of concrete.

4.4 Corrosion initiation times predicted by the modified diffusion model

Now, the main question is how to determine the values of D_i and m in equation (4.5). Since D_i is the diffusion coefficient at a reference time, we can determine it using the current inspection results, i.e. considering the current time as the reference time. However, parameter m must be determined based on the rate of concrete deterioration by comparing the diffusion coefficients in at least two different times. Figure 4-9 shows the deterioration of the diffusion coefficient at different m values. As time increases, the m value has significant effect on the diffusion coefficient. So far, we only have the estimated diffusion coefficients based on the current inspection results. It will be the best if we can conduct another inspection sometime later on the same bridges and collect another set of data to determine the second point in Figure 4-9. Once we have two different diffusion coefficients at different times, the m value can be determined for each bridge. The m values for different bridges will be different, because each bridge has its own rate of deterioration, depending on its environmental condition and traffic loading. In this way, the corrosion initiation times for the bridges can be determined more accurately, and hence, the effectiveness of the topical protection systems can be evaluated more accurately.

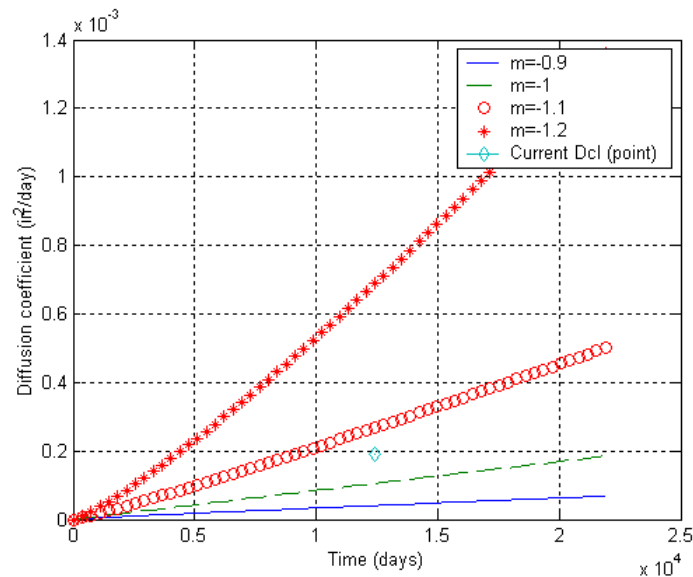


Figure 4-9 The effect of m values on D_{Cl}

In previous sections, the best curve fitting of the concentration profiles between the theoretical prediction and the measured data is based on the linear model (equation (4.3)). In this section, we use the modified nonlinear model (equation (4.5) and equation (4.12)) to perform the curve fitting using the same test data. For each bridge, we adjusted two parameters m and D_i in equation (4.12) in such a way that the predicted concentration profiles will best fit the test data. The corresponding parameters are the optimum values of m and D_i .

The parameter m is mainly affected by the traffic loading and environmental conditions. The value tends to be higher when the bridge is under heavy traffic and severe environmental conditions. The parameter D_i represents the quality of concrete in the mixture process. For example, lower water cement ratio may lead to a lower D_i value.

In equation (4.12), C_0 comes from the calibration results in Chapter 4.2 and the time used in the calibration for each bridge is the same as the year we took cores and got chloride concentration profiles. For example, Bridge G-22-BJ was built in 1972 and we took cores in 2007. The time in equation (4.12) is 35 years. The comparisons between these predicted profiles and real on-site test data are shown in Figures 4-10 through 4-14.

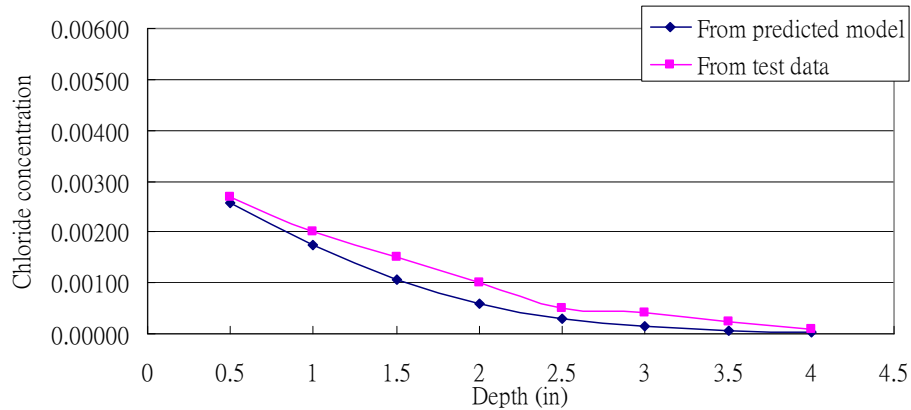


Figure 4-10 Comparison of Bridge G-22-BJ

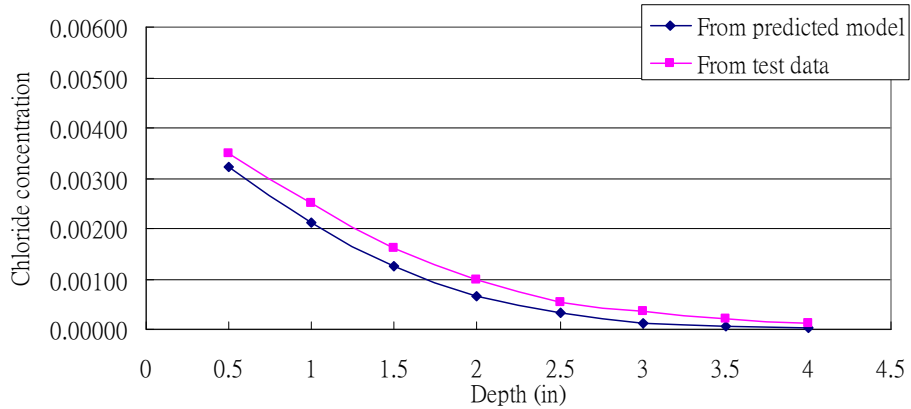


Figure 4-11 Comparison of Bridge G-22-BL

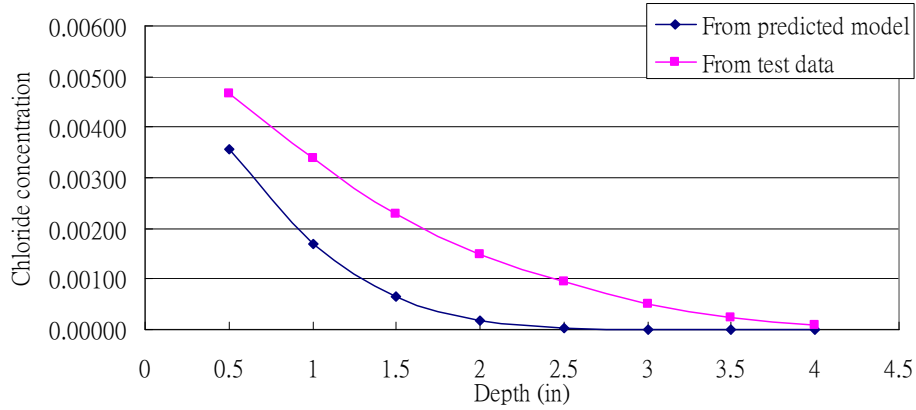


Figure 4-12 Comparison of Bridge G-22-BG

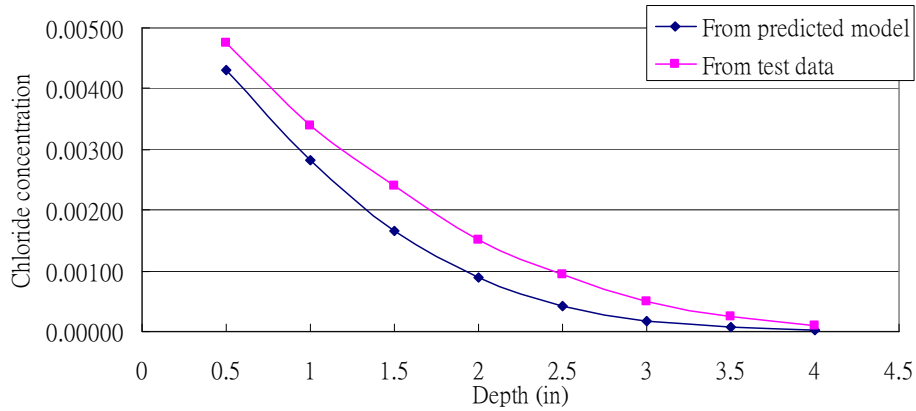


Figure 4-13 Comparison of Bridge G-22-BH

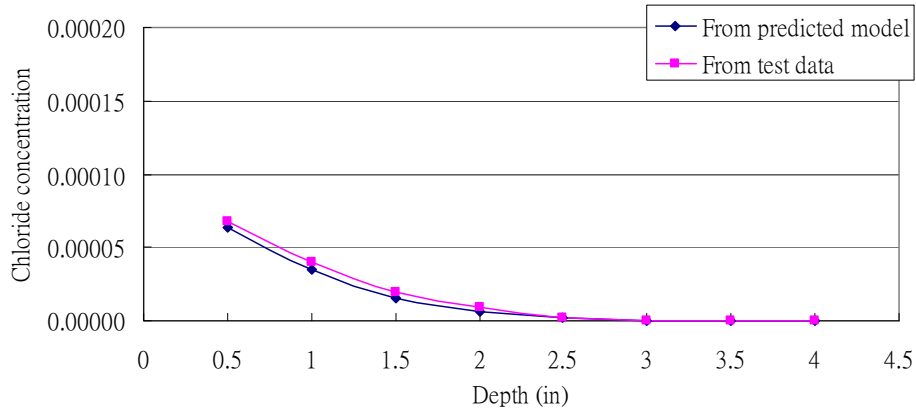


Figure 4-14 Comparison of Bridge C-16-DI

The calibrated m , D_i values and the predicted corrosion initiation time (using equation (4.13)) based on these optimum parameters are shown as Table 4-11.

Table 4-11 Estimations for the five bridges

Bridge	m	D_i (in ² /days)	D_{cl} (current) (in ² /days)	D_{cl} (in five years) (in ² /days)	Initiation time from now (years)
G-22-BJ	-1.063	0.000000008412	0.000189	0.000219	16
G-22-BL	-1.05	0.000000008412	0.000167	0.000193	9
G-22-BG	-1	0.000000006	0.000072	0.0000832	6
G-22-BH	-1.05	0.000000008412	0.000162	0.000188	6
C-16-DI	-0.99	0.000000008412	0.000078	0.000092	274

4.5 Observations on the measured data

1. The corrosion initiation time is reciprocally proportional to the diffusion coefficient, as shown in equation (4.13). This means that the corrosion initiation time will be reduced to 1/10 of the current value, if the diffusion coefficient increases 10 times. In the practice, 10 times increase in the diffusion coefficient is highly possible under heavy traffic and severe environmental conditions.
2. With the two different critical chloride concentration values, we can obtain a range of the corrosion initiation time. With the lower critical value (0.021%), we can determine a shorter corrosion initiation time, which can be considered as the lower bound of the initiation time. With the higher critical value (0.05%), we can determine a longer corrosion initiation time, which can be considered as the upper bound of the initiation time. In the practice, the actual corrosion initiation time of a bridge deck may be between the two bounds.
3. Compare the surface chloride concentrations among the five bridges, Bridge G-22-BH is the highest one (0.6%) and Bridge C-16-DI is the lowest one (0.01% or 0.006%). These values represent an average of the surface chloride concentration, which is an indication of the amount of deicers used on the bridge decks over the years.
4. At the same bridge, the surface chloride concentration values from shoulder lane and from traffic lane are different, but the difference is not very large.
5. For the predicted corrosion initiation time, Bridge C-16-DI is the longest, which means the current bridge deck condition for this bridge is the best in terms of corrosion damage of rebars. Another important reason for the much longer predicted corrosion initiation time for this bridge is that the traffic loading on the bridge is much lower than the traffic loadings of other bridges (on I-70). All other bridges have shorter initiation times ranging from 6 to 16 years. Given the uncertainties in the prediction model, the conditions of the other four bridges may be considered as the same.
6. In order to have a more accurate estimation of the rate of concrete deterioration, i.e. a better estimation for parameters m and D_i (see equation (4.13) and figure 4-9), we need to conduct another inspection of the same bridges in the future. Once we have two different diffusion coefficients corresponding to two different times, we can better define the curve in figure 4-9, obtain a better estimation on the value for m , and predict more accurately the corrosion initiation time of rebars for the bridge decks. Only then, the effectiveness of the topical protection systems used on the five bridges can be evaluated accurately.

5. LIFE-CYCLE COST ANALYSIS

5.1 Introduction

There have been several life-cycle cost analysis (LCCA) approaches proposed for pavement and bridge structures. In one of our neighboring states, the Utah DOT developed a LCCA program to estimate optimal strategies for repair and rehabilitation projects for highway bridges. In a recent CDOT report (CDOT-DTD-R-2007-2 “Service Life and Cost Comparisons for Four Types of Colorado DOT Bridges Decks”), costs related to rehabilitation of bridge decks were computed using the methods presented in NCHRP Report 483. In this task, we will use the same methods for LCCA as in the report CDOT-DTD-R-2007-2.

In this method, the performance evaluation of corrosion protection systems during the 75-year life span will be calculated by the mathematical models described in Task 4. All cost data will be obtained from CDOT databases. LCCA results will be shown in a format that would be usable for upload to AASHTO Pontis Bridge Inspection Coding Guide.

5.2 Bridge decks to be evaluated

A set of 82 bridge decks of four different types were studied for CDOT (Report No. CDOT-DTD-2007-2). The bridge decks included:

1. Twenty-five decks built between 1969 and 1975 having uncoated reinforcing steel. Twenty-two decks in this group were rehabilitated with rigid overlay between 1989 and 1999.
2. Twenty-three decks built in 1993 having epoxy-coated reinforcing steel and treated with surface applied concrete sealer.
3. Nineteen decks built in 1980 having uncoated reinforcing steel and protected with waterproofing membrane and bituminous overlay.
4. Fifteen decks built in 1991 having epoxy-coated reinforcing steel and protected with waterproofing membrane and bituminous overlay.

In this project, we performed a life cycle analysis for the five bridge decks we inspected. The chloride concentration profiles of the five bridge decks were shown in Chapter 4. The information for the bridge decks is summarized in Table 5-1.

Table 5-1 Bridge decks information

No	Bridge	Element	Built	Rehab	Concrete Protection	Surface (Mat & Thick)	
1	G-22-BJ	22 (No Rehab)	1972		None	IC	3.3in
2	G-22-BL	22 (Rehab)	1975	1999	Rigid Overlay	IC	2.9in
3	G-22-BG	22 (Rehab)	1975	1999	Rigid Overlay	IC	2in
4	G-22-BH	22 (Rehab)	1975	1999	Rigid Overlay	IC	3in
5	C-16-DI	14	1980		Membrane +Asphalt	Asphalt	3in

5.3 Deck designs and costs

For our purpose of cost evaluation and comparison, deck designs conform to the CDOT Bridge Design Manual [Bridge Design Manual, (2002)]. Costs for materials for deck construction are collected from CDOT published unit cost data for construction projects [2003 Cost Data (Construction)] and for maintenance projects [2003 Cost Data (Maintenance Projects)]. The cost data was collected from 2003. The basic design and cost data are shown in Table 5-2.

Table 5-2 Basic design and cost data

Bridge	Deck Span	Thickness	Steel Qty	Concrete Protection	Epoxy Coating	Material Cost	Project Cost
G-22-BJ Element 22 No rehab	8'-6"	8.25"	58.5 lbs/SY	None	None	\$ 104 / SY	\$ 624 / SY
G-22-BL G-22-BG G-22-BH Element 22 Rehab	8'-6"	8.25"	58.5 lbs/SY	Rigid Overlay	None	\$ 139 / SY	\$ 834 / SY
C-16-DI Element 14	7'-8"	8"	58.5 lbs/SY	Membrane + Asphalt	None	\$ 119 / SY	\$ 714 / SY

All project costs are adjusted to 2003 using the tables of the US Army Corps Civil Works Construction Cost Index system [Civil Works Construction Cost Index System, (2006)].

5.3.1 Cost comparisons based on deck materials

Costs of bridge decks are compared as unit costs for materials and maintenance over the service life of decks. These include the four types of deck plus the rehabilitated decks in the group of element 22 decks. All the items includes,

1. Initial unit material costs (IC)
2. Maintenance costs (MC): Periodic maintenance costs are incurred for decks with waterproofing membrane and decks with surface applied sealers.
3. Present value of materials replacement cost (PV1): The cost of future replacement of each deck type is computed as unit cost in 2003 dollars occurring at x years in the future, where x is the service life. The present value is the discounted value of the cost to construct the same type of deck at x years into the future.
4. Present value of deck material replacement using service life probability (PV2)
5. Annualized material costs at median service life (AC1): Simple annualized cost is deck unit cost divided by service life in years.
6. Annualized material costs at probable service life (AC2): Uncertainty in service life is recognized, and annualized unit cost is computed with population models of service life.
7. Annualized cost of deck materials using median service life and discount factor (AC3)
8. Annualized cost of deck materials using service life probability density and discount factor (AC4)

Average unit costs for the five bridge decks are computed as:

1. Initial material cost
2. Initial construction project cost
3. Present value costs discounted over the service life of the deck
4. Annualized costs over the service life, without a discount factor (conforming to GASB 34 alternative method [Primer GASB 34, (2000)])
5. Annualized costs over the service life, using a discount factor.

For annualized costs and for present value costs, results are obtained both using median service life values and using service-life population models. The annual discount rate is 3.2%, as specified in circular A094 of the US Office of Management and Budget [Guidelines and Discount Rates for Benefit-Cost Analysis of Federal Programs, (2004)]. Comparisons for deck material cost and deck material cost ranks are shown in Tables 5-3 and 5-4.

Table 5-3 Comparisons for deck material cost

	G-22-BJ Element 22 No rehab	G-22-BL G-22-BG G-22-BH Element 22 Rehab	C-16-DI Element 14
IC	\$ 104 / SY	\$ 139 / SY	\$ 119 / SY
PV1	\$ 39.17 / SY	\$ 46.16 / SY	\$ 45.35 / SY
PV2	\$ 37.46 / SY	\$ 40.45 / SY	\$ 51.71 / SY
AC1	\$ 3.35 / SY	\$ 3.97 / SY	\$ 3.25 / SY
AC2	\$ 3.25 / SY	\$ 3.60 / SY	\$ 3.59 / SY
AC3	\$ 2.00 / SY	\$ 2.21 / SY	\$ 1.80 / SY
AC4	\$ 1.92 / SY	\$ 1.89 / SY	\$ 2.14 / SY

Table 5-4 Deck material cost ranks (1 = lowest cost)

	G-22-BJ Element 22 No rehab	G-22-BL G-22-BG G-22-BH Element 22 Rehab	C-16-DI Element 14
IC	1	3	2
PV1	1	3	2
PV2	1	2	3
AC1	2	3	1
AC2	1	3	2
AC3	2	3	1
AC4	2	1	3

5.3.2 Sensitivity to discount factor

Four cost evaluations (PV1, PV2, AC3 and AC4) employ a discount factor. The previous section computed deck costs using the OMB-specified annual discount factor equal to 3.2%. This section considers a range of discount factors, from 2% to 10%, to examine the sensitivity of deck costs to discount factor. In CDOT report (CDOT-DTD-R-2007-2), it was shown that the value of the

discount rate has significant effect on deck costs and some effect on rankings among decks. Element 22 decks (G-22-BJ, G22-BL, G22-BG and G-22-BH) are least costly by most evaluations and at most values of discount rates. Higher discount rates make rehabilitation of Element 22 decks more cost effective.

5.3.3 Cost comparisons based on deck projects

Comparisons for deck project cost and deck project cost ranks are shown in Tables 5-5 and 5-6.

Table 5-5 Comparisons for deck project cost

	G-22-BJ Element 22 No rehab	G-22-BL G-22-BG G-22-BH Element 22 Rehab	C-16-DI Element 14
IC	\$ 624 / SY	\$ 834 / SY	\$ 714 / SY
PV1	\$ 235 / SY	\$ 277 / SY	\$ 147 / SY
PV2	\$ 225 / SY	\$ 243 / SY	\$ 167 / SY
AC1	\$ 20.13 / SY	\$ 23.83 / SY	\$ 13.87 / SY
AC2	\$ 19.50 / SY	\$ 21.62 / SY	\$ 15.14 / SY
AC3	\$ 12.06 / SY	\$ 13.27 / SY	\$ 5.74 / SY
AC4	\$ 11.52 / SY	\$ 11.36 / SY	\$ 7.21 / SY

Table 5-6 Deck project cost ranks (1 = lowest cost)

	G-22-BJ Element 22 No rehab	G-22-BL G-22-BG G-22-BH Element 22 Rehab	C-16-DI Element 14
IC	1	3	2
PV1	2	3	1
PV2	2	3	1
AC1	2	3	1
AC2	2	3	1
AC3	2	3	1
AC4	3	2	1

5.3.4 Sensitivity to discount factor

The same as material cost analysis in Chapter 5.3.2; four project cost evaluations (PV1, PV2, AC3 and AC4) also employ a discount factor. The previous section computed deck project costs using the OMB-specified annual discount factor equal to 3.2%. This section considers a range of discount factors, from 2% to 10%, to examine the sensitivity of deck project costs to discount factor.

In CDOT report (CDOT-DTD-R-2007-2), it was shown that the value of the discount rate has significant effect on deck costs and some effect on rankings among decks. Element 14 decks (C-16-DI) are least expensive at all values of discount rate.

5.4 Deck service life

Service life is taken as the time required, in years, for a new bridge deck to reach NBI condition rating 5. The starting time is the initial construction of the bridge deck. The ending time is the first occurrence of condition rating 5 for decks without rehabilitation, or the re-occurrence of condition rating 5 after a deck rehabilitation.

In the report CDOT-DTD-R-2007-2, for rehabilitated element 22 decks (G22-BL, G22-BG and G-22-BH), two service life estimates are made: The first one is from initial construction to an occurrence of condition rating 5 before rehabilitation, and the other one is from initial construction to the second occurrence of condition rating 5. The second estimate includes the effect of the rehabilitation project to extend deck service life.

5.4.1 Length of service life

Estimates of service life for individual bridge deck in the CDOT study (CDOT-DTD-R-2007-2) were obtained by extrapolation of time domain trend lines for National Bridge Inventory (NBI) deck condition ratings. For this study, the time to reach NBI rating 5 was taken as the time to first rehabilitation of the deck, and also the initial service life of the deck.

Continued service after rehabilitation is examined for element 22 decks. Condition ratings are available from 1972 onwards. For most decks, initial condition ratings are reported one year after construction. For decks constructed in 1969, the initial deck ratings are reported three years after construction. Service life estimates are summarized in Table 5-7.

Table 5-7 Service life estimates

No	Bridge	Element	Service Life Estimate (years)		
			Minimum	Average	Median
1	G-22-BJ	22 (No Rehab)	19	34	31
2	G-22-BL	22 (Rehab)	27	39	35
3	G-22-BG	22 (Rehab)	27	39	35
4	G-22-BH	22 (Rehab)	27	39	35
5	C-16-DI	14	31	78	56

5.4.2 Population probability distributions of service life of decks

Estimates of service life for individual bridge decks were used to form models of service lives for populations of similar decks. These are called deck population models. Four probability distributions are considered for use. Selection of individual models is based on success in fitting available data. The four probability distributions are:

1. Rayleigh distribution;
2. Rayleigh distribution using a time-shifted origin; called an xo-Rayleigh distribution;
3. Exponential distribution; and
4. Exponential distribution using a time-shifted origin, called an xo-Exponential distribution.

For these bridge decks, a population model for deck service life is generated. Each population model indicates the probability, as a function of time in years, that a deck will reach NBI condition rating 5. Population models are used to include variability in service life during computation of deck costs. The times required to reach condition rating 5 for 10th, 50th, and 90th percentiles of deck populations are shown in Table 5-8.

Table 5-8 Years to service life percentile

No	Bridge	Element	Years to Service Life Percentile		
			10th	50th	90th
1	G-22-BJ	22 (No Rehab)	25	31	41
2	G-22-BL	22 (Rehab)	28	35	52
3	G-22-BG	22 (Rehab)	28	35	52
4	G-22-BH	22 (Rehab)	28	35	52
5	C-16-DI	14	33	56	110

5.4.3 Results for the service life of the five bridge decks

G-22-BJ (Element 22 No rehab)

Discrete estimates of mean service life and median service life are shown in Table 5-9. Population model parameters are listed in Table 5-10. Values of mean and median service life computed from population models are listed in Table 5-11.

Table 5-9 Discrete estimates of service life

G-22-BJ (Element 22 No rehab)	(years)
Mean service life	34
Median service life	31
Least service life	19

Table 5-10 Parameters of service life distributions

Distribution	s (years)	xo (years)	Err / N, %
Rayleigh	33	-	2.0
xo-Rayleigh	14	22	0.34
Exponential	54	-	4.3
xo-Exponential	15	27	0.46

Table 5-11 Mean and median service life using population models

Distribution	Mean (years)	Median (years)
Rayleigh	41	39
xo-Rayleigh	40	38
Exponential	54	37
xo-Exponential	42	37

G-22-BL, G-22-BG, G-22-BH (Element 22 Rehab)

Discrete estimates of mean service life and median service life are shown in Table 5-12. Population model parameters are listed in Table 5-13. Values of mean and median service life computed from population models are listed in Table 5-14.

Table 5-12 Discrete estimates of service life

G-22-BL, G-22-BG, G-22-BH (Element 22 Rehab)	(years)
Mean service life	39
Median service life	35
Least service life	27

Table 5-13 Parameters of service life distributions

Distribution	s (years)	xo (years)	Err / N, %
Rayleigh	34	-	1.8
xo-Rayleigh	14	22	0.45
Exponential	55	-	4.3
xo-Exponential	17	27	0.60

Table 5-14 Mean and median service life using population models

Distribution	Mean (years)	Median (years)
Rayleigh	43	40
xo-Rayleigh	40	38
Exponential	55	38
xo-Exponential	44	39

C-16-DI (Element 14)

Discrete estimates of deck service life are shown in Table 5-15. Population model parameters are listed in Table 5-16. Values of mean and median service life computed from population models are listed in Table 5-17.

Table 5-15 Discrete estimates of service life

C-16-DI (Element 14)	(years)
Mean service life	78
Median service life	56
Least service life	31

Table 5-16 Parameters of service life distributions

Distribution	s (years)	xo (years)	Err / N, %
Rayleigh	47	-	0.58
xo-Rayleigh	32	16	0.31
Exponential	81	-	1.80
xo-Exponential	35	29	0.28

Table 5-17 Mean and median service life using population models

Distribution	Mean (years)	Median (years)
Rayleigh	59	55
xo-Rayleigh	56	54
Exponential	81	56
xo-Exponential	64	53

5.4.4 Service life estimates using ACI Life-365

Service life estimates are also obtained from Life-365, a Windows-based software application prepared by ACI committee 365 that estimates time to initial corrosion, time to first repair and life-cycle costs for reinforced concrete structures [9].

Life-365 accepts input data on concrete properties, reinforcement type, protection system, geographic location and exposure. For the previous research, Life-365 was executed with default values for concrete properties. Geographic location was set to Denver, Colorado and exposure was urban highway. The reinforcement type (uncoated or epoxy-coated), concrete protection (bare, membrane, or sealer), deck thickness and clear concrete cover were inputted for each deck type. Outputs from Life-365 are shown in Table 5-18.

Table 5-18 Service life estimates using Life-365

No	Bridge	Element	Reinforcing Steel	Concrete Protection	Corrosion Initiation (years)	Time to Repair (years)
1	G-22-BJ	22(No Rehab)	uncoated	Bare	16.0	22.0
2	G-22-BL	22(Rehab)	uncoated	Bare	16.0	22.0
3	G-22-BG	22(Rehab)	uncoated	Bare	16.0	22.0
4	G-22-BH	22(Rehab)	uncoated	Bare	16.0	22.0
5	C-16-DI	14	uncoated	Membrane	17.9	23.9

Life-365 indicates that membranes (C-16-DI – Element 14) extend service life by 2 years from the decks with bare reinforcing steel (G-22-BJ, G-22-BL, G-22-BG, and G-22-BH – Element 22). Results from Life-365 largely depend on concrete cover and on deck thickness. We tried to use the software again with a certain cover thickness and deck thickness. We selected a single value of cover (3") and a single deck thickness (8.25"), then the outcomes are shown in Table 5-19.

Table 5-19 Comparison of Life-365 results for identical cover and thickness

No	Bridge	Element	Reinforcing Steel	Concrete Protection	Corrosion Initiation (years)	Time to Repair (years)
1	G-22-BJ	22(No Rehab)	bare	Bare	16.0	22.0
2	G-22-BL	22(Rehab)	bare	Bare	16.0	22.0
3	G-22-BG	22(Rehab)	bare	Bare	16.0	22.0
4	G-22-BH	22(Rehab)	bare	Bare	16.0	22.0
5	C-16-DI	14	bare	membrane	25.0	31.0

In report CDOT-DTD-R-2007-2, the results indicate that service life for decks with epoxy-coated reinforcements is longer by 15 years or more compared to decks with bare reinforcing steel. Waterproofing membranes increase service life by six or more years.

But in our cases, all five bridge decks belong to Element 22, Element22-Rehab, and Element14 are not epoxy-coated reinforcements. According to the results in Table 5-19, the time starts to repair in Element 14 is about 9 years longer than others (31 years vs. 22 years).

5.4.5 Comparisons of corrosion initiation times

In Chapter 4, the results of chloride concentration tests from the five bridges were shown. The results were then used in Section 4.2 to calibrate the two parameters in the chloride diffusion model (i.e. the diffusion coefficient D_{cl} and the surface chloride concentration C_0). In Section 4.4, the diffusion model was modified to include the effect of damage in concrete due to traffic and environmental loadings on the chloride diffusion coefficient. The corrosion initiation times of the five bridge decks were then predicted by using the modified model. The results are summarized in the following table.

Table 5-20 Prediction for corrosion initiation time

No	Bridge	Element	Corrosion initiation time (years)
1	G-22-BJ	22(No Rehab)	16
2	G-22-BL	22(Rehab)	9
3	G-22-BG	22(Rehab)	6
4	G-22-BH	22(Rehab)	6
5	C-16-DI	14	274

Because the predicted corrosion initiation times by our modified nonlinear diffusion model and the results by Life-365 have different starting point, we need to convert all results into the same starting point. Table 5-21 shows the comparisons of corrosion initiation times predicted by our modified diffusion model and Life-365 based on the real time scale.

Table 5-21 Comparisons of the corrosion initiation time

No	Bridge	Element	Life-365 Corrosion Initiation (years)	Modified nonlinear model Corrosion Initiation(years)
1	G-22-BJ	22(No Rehab)	2023	2026
2	G-22-BL	22(Rehab)	2023	2019
3	G-22-BG	22(Rehab)	2023	2016
4	G-22-BH	22(Rehab)	2023	2016
5	C-16-DI	14	2032	much longer

6. CONCLUSIONS AND RECOMMENDATIONS

1. Evaluation of LMC-VES overlays constructed in Virginia indicated that the traffic could be placed on the overlay within 3 hours cure time rather than within 4 or 7 days required for the conventional LMC overlay; and the bond strength, resistance to chloride intrusion can be as good as those of LMC-HES and LMC. In addition, the life-cycle cost could be reduced by 25% in comparison with LMC.
2. At a low discount rate (2%), solid stainless steel reinforcement is a cost-effective option compared to other options, but at higher discount rates (4%+), the present value cost of a deck with solid stainless steel is significantly higher than that of an unprotected deck for the 75 year economic life.
3. Stainless steel clad reinforcement is much less expensive than solid stainless steel reinforcement. Assuming that a bridge deck constructed using stainless steel-clad reinforcement does not require any repairs within its 75 year economic life. The present value of the cost of a bridge deck built with stainless steel-clad reinforcement is significantly lower than the present value for the cost of any other corrosion protection system.
4. Low-slump, silica fume and LMC overlays have similar service lives, as long as they are constructed and cured properly. Among the three, low-slump overlays are the least expensive.
5. Epoxy-coated steel can be combined with membranes, overlays, corrosion inhibitors, or silica fume concrete, and is a low cost backup to these corrosion protection methods.
6. Preformed sheet membranes with asphalt overlays have been used in Colorado. Hot rubberized asphalt membranes and spray-applied liquid membranes are less expensive options, comparing with preformed sheet membranes.
7. Five bridge decks currently in service were inspected. Chloride concentration profiles were obtained, and the concentrations at steel levels of the five bridges are all below the critical values for the onset of steel corrosion. So, the five bridge decks are in very good conditions.
8. The effectiveness of a topical protection system can be evaluated by corrosion initiation time of rebars embedded in concrete. A shorter corrosion initiation time is attributed to several parameters, including a less effective topical protection system. Therefore a prediction model for corrosion initiation time will be a useful tool for evaluating the effectiveness of topical protection systems. A mathematical model for chloride penetration in concrete (and for corrosion initiation time) was developed.
9. The chloride profiles of the five bridge decks were used to calibrate the two model parameters (i.e. the chloride diffusion coefficient and the average surface chloride concentration) in the conventional chloride diffusion model. The calibrated model was then used to predict corrosion initiation times for the five bridge decks. The predictions were not accurate due to

the fact that the effect of traffic and environmental loadings on the diffusion coefficients of the concretes was not taken into account in the model.

10. The conventional chloride diffusion model was modified by considering the chloride diffusion coefficient as a time dependent function. The modified diffusion model can predicted the corrosion initiation times more accurately. For the predicted corrosion initiation time by the modified model, Bridge C-16-DI is the longest, which means the current bridge deck condition for this bridge is the best in terms of corrosion damage of rebars. Another important reason for the much longer predicted corrosion initiation time for this bridge is that the traffic loading on the bridge is much lower than the traffic loadings of other bridges (on I-70). All other bridges have shorter initiation times ranging from 6 to 16 years. Given the uncertainties in the prediction model, the conditions of the other four bridges may be considered as the same
11. Better predictions can be obtained by the modified model if the five bridge decks could be inspected one more time in the near future so that both current and future chloride profiles can be used to calibrate the time dependent diffusion coefficient.
12. The five bridge decks allow comparisons between bare decks and decks with waterproofing membranes. From Life-365 analysis, the membranes (C-16-DI - Element14) extend service life by two years from the decks with bare reinforcing steel (G-22-BJ, G-22-BL, G-22-BG, and G-22-BH - Element22).
13. Histories of deck condition ratings were used to estimate deck service life and to generate population models of service life. Decks with waterproofing membrane (C-16-DI - Element14) have longer service life than bare decks (G-22-BJ, G-22-BL, G-22-BG, and G-22-BH - Element22).
14. Compare the outputs from Life-365 and from the present modified diffusion model, the predicted corrosion initiation times agreed quite well for the five bridge decks. The advantage of the present model is that it can incorporate available inspection data and better predict future performance of bridge decks.

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