University Transportation Research Center - Region 2

Final Report

The Economy of Preventive Maintenance of Concrete Bridges

Performing Organization: Syracuse University

March 2016

Sponsor:
University Transportation Research Center - Region 2

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The Region 2 University Transportation Research Center (UTRC) is one of ten original University Transportation Centers established in 1987 by the U.S. Congress. These Centers were established with the recognition that transportation plays a key role in the nation's economy and the quality of life of its citizens. University faculty members provide a critical link in resolving our national and regional transportation problems while training the professionals who address our transportation systems and their customers on a daily basis.

The UTRC was established in order to support research, education and the transfer of technology in the field of transportation. The theme of the Center is "Planning and Managing Regional Transportation Systems in a Changing World." Presently, under the direction of Dr. Camille Kamga, the UTRC represents USDOT Region II, including New York, New Jersey, Puerto Rico and the U.S. Virgin Islands. Functioning as a consortium of twelve major Universities throughout the region, UTRC is located at the CUNY Institute for Transportation Systems at The City College of New York, the lead institution of the consortium. The Center, through its consortium, an Agency-Industry Council and its Director and Staff, supports research, education, and technology transfer under its theme. UTRC's three main goals are:

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Project No(s):
UTRC/RF Grant No: 49997-27-25

Project Date: March 2016

Project Title: The Economy of Preventive Maintenance of Concrete Bridges

Project's Website:
http://www.utrc2.org/research/projects/preventive-maintenance-concrete-bridges

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1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle The Economy of Preventive Maintenance of Concrete Bridges		5. Report Date March 2016
		6. Performing Organization Code
7. Author(s) Riyad Aboutaha and Haotian Zhang		8. Performing Organization Report No.
9. Performing Organization Name and Address Syracuse University Department of Civil and Environmental Engineering 151 Link Hall Syracuse, NY 13244		10. Work Unit No.
		11. Contract or Grant No. 49997-27-25
12. Sponsoring Agency Name and Address University Transportation Research Center (UTRC) Marshak Hall, Room 910 The City College of NY New York, NY 10031		13. Type of Report and Period Covered: Final Report
		14. Sponsoring Agency Code
15. Supplementary Notes Project funded in part with funds from the Syracuse University		

16. Abstract

The most economical approach to maintain existing concrete bridges is by adopting an active preventive maintenance approach. An in-depth investigation of the combined deterioration effects of various deterioration mechanisms is needed to establish sound thresholds for harmful chemicals in concrete bridge elements. Such established thresholds are critical for cost-effective maintenance of concrete bridges. This study investigated the economy of preventive maintenance for concrete bridges. This report presents an in-depth chemical evaluation and preventive maintenance of existing highway concrete bridges. In addition, it presents implementation of the proposed preventive maintenance approach that is based on quantitative assessment of the material chemical condition of the bridge. This research project involved the following activities: (1) Review of the current bridge inspection manuals for concrete bridges evaluation, (2) Identification of various deterioration mechanisms, (3) Extensive presentation of chemical NDTs, (4) Establishment of chloride diffusion coefficient, (4) Required preventive maintenance measures for different levels of harmful compounds found in concrete bridges, (5) cost of various activities presented in items 1-4, (6) Comparison of concrete bridge maintenance costs using the current practice and the proposed preventive maintenance approach. Based on the finding of this study, preventive maintenance may require slightly higher initial cost, however, it leads to long-term cost saving over the service life of bridges.

17. Key Words Concrete bridges, preventive maintenance, concrete deterioration, maintenance cost	18. Distribution Statement No restrictions		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 210	22. Price

Form DOT F 1700.7 (8-72)

Executive Summary

A cost-effective bridge is a bridge whose maintenance is based on its chemical condition over its entire service life. If deterioration mechanisms are prevented, a bridge would cost considerably less to maintain and safely serves its full design service life, if not longer. The practice of physical evaluation and delayed maintenance of deteriorated concrete bridges has resulted in large number of bridges in need of repair. The current inspection manuals are primarily focused on detecting physical damage in concrete bridge elements. If no physical damage is detected, very minimal maintenance actions are taken. This is the main reason why the number of US bridges that are classified as structurally deficient is on the rise. Given the need for future expansion of the US transportation network and increase in number of new bridges, there is a need for cost-effective maintenance process that prevents deterioration mechanism from starting, or at least stops it at a very early stage.

The most economical approach to maintain existing concrete bridges is by adopting an active preventive maintenance approach. An in-depth investigation of the combined deterioration effects of various deterioration mechanisms is needed to establish sound thresholds for harmful chemicals in concrete bridge elements. Such established thresholds are critical for cost-effective maintenance decision making, in a timely fashion, before any deterioration starts.

This study investigated the economy of preventive maintenance for concrete bridges. The report presents an in-depth chemical evaluation and preventive maintenance of existing highway concrete bridges. In addition, it presents implementation of the proposed preventive maintenance approach that is based on quantitative assessment of the material chemical condition of the bridge.

This research project involved the following activities: (1) Review of the current bridge inspection manuals for concrete bridges evaluation, (2) Identification of various deterioration mechanisms, (3) Extensive presentation of chemical NDTs, (4) Establishment of chloride diffusion coefficient, (4) Required preventive maintenance measures for different levels of harmful compounds found in concrete bridges, (5) cost of various activities presented in items 1-4, (6) Comparison of concrete bridge maintenance costs using the current practice and the proposed preventive maintenance approach.

ACKNOWELDGEMENT

The authors gratefully acknowledge the financial support provided by the University Transportation Research center (UTRC), which made this project possible. Many thanks to Mr. Thomas Siwula for his help throughout the project! Thanks are also extended to Mr. Mark Gardner for his contribution to the project.

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a = coefficient represents the isotherm of chloride binding capacity

A^* = coefficient of capillarity (kg/ $m^2\sqrt{s}$) ---Section 5.1.1.3 Equation 5.5

A = cross-sectional area of the sample (m^2)

b = coefficient represents the isotherm of chloride binding capacity

c = salt concentration (kg/ m^3)

c^* = CO_2 concentration in the atmosphere ---Section 3.3.2 Equation 3.6

C = chloride concentration (lb./yd^3; % by weight of the Concrete)

$C(x, t)$ = chloride concentration at depth x on time t (lb./yd^3; % by weight of the Concrete)

C_0 = initial chloride content in concrete (lb./yd^3; % by weight of the Concrete)

C_{bound}=Bound chloride (mg/g)

C_{cr} = estimated chloride threshold concentrations (lb./yd^3; % by weight of the Concrete)

C_{free}= Free Chloride (mole/L)

C_s = surface chloride concentration (lb./yd^3; % by weight of the Concrete)

C_{th} = chloride threshold value for corrosion initiation (lb./yd^3; % by weight of the Concrete)

D = diffusion coefficient (cm^2/s)

D_c = chloride diffusion coefficient (cm^2/s)

D_{PC} = chloride diffusion coefficient of Portland cement concrete (cm^2/s)

$D_{ref,294}$ =Chloride diffusion coefficient measured at reference time under temperature equals 20℃ (cm^2/s)

D_{ref} = reference chloride diffusion coefficient measured at reference time (cm^2/s)

D_{SF} = chloride diffusion coefficient of High performance concrete with silica fume (cm^2/s)

D_{28} = chloride diffusion coefficient measure at 28 days (cm^2/s)

$erf(*)$ =error function from mathematic equation chart

E_0 = dynamic modulus of elasticity of uncracked concrete (GPa)

E_d = dynamic modulus of elasticity of cracked concrete (GPa)

$E_d(t)$ is the dynamic modulus of elasticity at time t (GPa)

FA = portion of fly ash in cementious material (%)

$f_c(t)$ = the concrete compressive strength at time t (MPa)

g = acceleration due to gravity (m/s^2)

h = Current humidity (%)

h_c = Critical humidity level at which the diffusion coefficient drops halfway between the maximum and minimum value (%)

k = intrinsic permeability of materials (m^2)

k= amount of water soluble alkali in cement (mole/kg cement)

k_b = reduction factor of chloride binding capacity

k_c= reduction factor of curing conditions

$k_{cr,env}$ = environment factor for chloride threshold value

k_d = reduction factor of cracking

k_E = reduction factor of environmental factor

k_{E1} = the location index of environmental factor

k_{E2} = the concrete mix factor of environmental factor

k_{RH} = Effects of relative humidity

k_t = reduction factor of aging of the concrete

k_T = reduction factor of temperature

L = thickness of the sample (m)

m = empirical aging factor for chloride diffusion coefficient

Q^* = amount of chlorides taken (kg/ m^2) ---Section 5.1.1.3 Equation 5.5

Q = flow rate (m^3/s)

R = Gas constant (8.314 J mol^{-1} K^{-1})

SG = Portion of slag in cementious material (%)

t = time

T = temperature of the concrete (K)

T_{ref} = temperature used when the reference diffusion coefficient is measured (K)

U = Activation energy of diffusion process (J·mol^{-1})

v = apparent velocity of flow (m/s)

w/c =water cement ratio

x = depth measured from the surface of the concrete (cm)

α^* = concentration of the reactive compounds ----Section 3.3.2 Equation 3.6

α= Degree of hydration

ζ = constant accounts for the presence of silica fume in the concrete mix

η = dynamic viscosity of the fluid (kg/m·s)

ρ = density of the fluid (kg/ m^3)

Chapter 1 Background and Current Inspection Practices

1.1 Introduction

A cost-effective bridge is a bridge whose maintenance is based on its chemical condition over its entire service life. If deterioration mechanisms are prevented, a bridge would cost considerably less to maintain and safely serves its full design service life, if not longer. The practice of physical evaluation and delayed maintenance of deteriorated concrete bridges has resulted in large number of bridges in need of repair. The current inspection manuals are primarily focused on detecting physical damage in concrete bridge elements. If no physical damage is detected, very minimal maintenance actions are taken. This is the main reason why the number of US bridges that are classified as structurally deficient is on the rise. Given the need for future expansion of the US transportation network and increase in number of new bridges, there is a need for cost-effective maintenance process that prevents deterioration mechanism from starting, or at least stops it at a very early stage.

The most economical approach to maintain existing concrete bridges is by adopting an active preventive maintenance approach. An in-depth investigation of the combined deterioration effects of various deterioration mechanisms is needed to establish sound thresholds for harmful chemicals in concrete bridge elements. Such established thresholds are critical for cost-effective maintenance decision making, in a timely fashion, before any deterioration starts.

This study investigated the economy of preventive maintenance for concrete bridges. The report presents an in-depth chemical evaluation and preventive maintenance of existing highway concrete bridges. In addition, it presents implementation of the proposed preventive maintenance approach that is based on quantitative assessment of the material chemical condition of the bridge.

This research project involved the following activities:
(1) Review of the current bridge inspection and maintenance manuals for concrete bridges evaluation, which are presented in Chapters 1 & 2,

(2) Identification of various deterioration mechanisms, which is presented in Chapter 3,

(3) Extensive presentation of chemical NDTs, that are needed for proper chemical evaluation of bridge elements, which are presented in Chapter 4,

(4) Establishment of chloride diffusion coefficient, and thresholds for various harmful compounds, which are presented in Chapter 5,

(5) Required preventive maintenance measures for different levels of harmful compounds found in concrete bridges, which are presented in Chapter 6,

(6) cost of various activities presented in items 1-4, which is presented in Chapter 7, and

(7) Comparison of concrete bridge maintenance costs using the current practice and the proposed preventive maintenance approach, which is presented in Chapter 8, followed by Chapter 9, summary and conclusions.

After the tragic accident of the Silver Bridge collapse between Ohio and West Virginia on December 15, 1967, the Federal High way Administration (FHWA) issued the National Bridge Inspection Standards (NBIS) in order to better serve, protect and preserve the health, safety and welfare of the public.

The NBIS established a program for regular, comprehensive inspection of all federal highway system bridges. Minimum qualifications were set forth for bridge inspectors, specific types and frequencies for bridge inspection were established, and the reporting of certain standard information about each bridge was required. The code recommends the use of reference documents such as the AASHTO Guide Manual for Bridge Element Inspection, the AASHTO Manual for Bridge Evaluation (MBE), the FHWA Bridge Inspector's Reference Manual (BIRM), and the National Bridge Inventory (NBI) coding guidance document for the inspection and evaluation of the Nation's bridges (FHWA, 2012).

A national bridge inspection program has been in place ever since, and state and local agencies have performed bridge inspections in accordance with these guidelines. Incorporating the Federal Regulations, New York State has issued the Uniform Code of Bridge Inspection (UCBI) to establish a program of comprehensive bridge management and inspection within the New York State Department of Transportation (NYSDOT). The UCBI prescribes the standards for bridge inspection and evaluations; requirements for the establishment of a rating system; procedures for underwater inspections; requirements for the performance of bridge structural and foundation system evaluations in conjunction with an inspection and criteria for their need and frequency; and the qualifications of bridge design, construction, and inspection experience for licensed professional engineers who may perform or supervise bridge inspections and evaluations. The NYSDOT believe that success practice of this code would yield to a better understanding of the condition of bridges, prioritized and optimized ranking of bridges in need of major maintenance, rehabilitation and replacement, the development of a means to assess and project bridge performance and deterioration, and the development of data to quantify the effectiveness of preventive maintenance.

In the following section, the current inspection manuals published by NYSDOT, AASTHO, and FHWA are reviewed. The general inspection type, frequency, procedure, methods, and process are summarized. Discussion on the shortcomings of the current inspection manuals is performed and several recommendations are presented.

1.2 Current Inspection Requirements

1.2.1 Inspection Type

According to the Bridge inspection manual published by NYSDOT (2014), five types of bridge inspections exist and they are listed as follows:

1. General: General inspections are required for all highway bridges at a maximum interval of 24 months. These inspections encompass both biennial and interim inspections as defined in the Uniform Code of Bridge Inspection (UCBI). Interim inspections are performed during the calendar year between the required biennial inspections, and are required if one or more of the following conditions exist:
    - General recommendation (determined by inspector) of 3 or less.
    - Condition rating (weighted average of individual element ratings) of 3.0 or less.
    - Presence of an active or inactive Red Flag, or active Yellow Flag.
    - Posting for any load other than R-permit restriction.
2. Special in-lieu of: This category of inspections is used for granting and documenting special in-lieu of inspections for bridges that are scheduled for inspection due to the UCBI interim requirements. Interim inspections shall be performed at or near one year after each biennial inspection. Bridges open to highway traffic while undergoing repair, reconstruction or rehabilitation shall receive an interim inspection when due. For very large or unusual structures, a program of scheduled special inspections may be substituted for interim inspections. And in no case will a special in-lieu of inspection be granted in back to back inspection seasons.
3. Special Events: Information collected during these inspections does not directly impact the condition ratings of a structure. Use of these type inspections will not follow the normal Quality Control / Quality Assurance workflow that is utilized for general and diving inspections. This category includes flood watch inspection, post-flood inspection, post-seismic inspections, impact assessment, and found structural flag repair.
4. In-Depth Inspection: the in-depth inspections are usually performed as a follow-up inspection to an initial, routine, or event-driven inspection to better identify any deficiencies found. Underwater Inspections and Fracture-Critical Inspections are both types of in-depth inspection.
5. None Due to Construction (under contract): This category of inspections is used for bridges that are closed to all traffic due to reconstruction. Further that a general inspection must be performed within 60 days of reopening to traffic of the newly constructed bridge or any portion thereof. The inspection must occur after opening to traffic in order to ensure that the inspector can view the structural behavior of the bridge after having experienced live load conditions in all directions. Any portion of an existing bridge that is under contract and carries traffic remains on the inspection schedule.

In addition, four types of NYSDOT bridge diving inspections exist and they are as follows:

1. General Diving (commonly referred to as Diving): General Diving inspections are required at a maximum interval of 60 months.
2. Special Events: This category is reserved for events that are typically not scheduled, where, however, a diving inspection is performed.
3. In-Depth Inspection
4. None Due to Construction (under contract)

1.2.2 Inspection Frequency

The NBIS specifies frequency requirements for various inspection types. However, factors such as age, condition, and bridge design may dictate reduced frequencies.

Full routine inspection of bridges occurs at 24-month intervals at most state DOTs and for the majority of bridges. Two states, Minnesota and Ohio, require routine inspection at 12-month intervals. In New York State, Certain bridges require inspection at less than twenty-four-month intervals as motioned before. One the other hand, the inspection period could be extended to 48-month if the past inspection findings and analysis justifies the increased inspection interval with written FHWA approval.

The underwater inspections should be performed within a 60-month interval. Certain underwater structural should be inspected at less than 60-month interval if the established criteria are met considering such factors as construction material, environment, age, condition rating and known deficiencies.

An in-depth inspection is a comprehensive detailed inspection of an entire bridge, which frequently incorporates destructive, as well as, non-destructive inspection techniques. In-depth inspections are performed on an “as needed” basis to assist in making bridge rehabilitation versus replacement decisions, and to assist in the development of bridge rehabilitation designs.

And a special inspection is a unique inspection effort targeted at special situations or conditions and may be performed to study a unique or unusual bridge feature in greater detail than would have normally occurred during a general inspection. It is always performed on an “as needed” basis also.

1.2.3 Inspection Procedure

The general inspection procedure contains activities as follow:

1. Planning: In order to make the inspection orderly and systematic, the lead inspector should make plans in advance. Planning the inspection is necessary for an efficient, cost- effective effort, which will also result in a thorough and complete inspection.
2. Preparing: Preparation measures needed prior to the inspection include organizing the proper tools and equipment, reviewing the bridge structure files, and locating plans for the structure. The success of the on-site field inspection is largely dependent on the effort spent in preparing for the inspection.
3. Performing: This duty is the on-site work of accessing and examining bridge components and waterway, if present. Inspection procedures as presented in the NBIS should always be followed.

4. Preparing the report: Documentation is essential for any type of inspection. The inspector must get enough information to ensure a comprehensive and complete report for further use.
5. Identification of items for repairs and maintenance and follow-up for critical findings: The final basic duty is to identify items for repairs and maintenance. The inspector must identify such items to promote public safety and maximize longevity of the bridge.

1.2.3 Inspection Methods

Although significant advances have been made in the development of nondestructive evaluation technologies, visual inspection and physical inspection are still the predominant tools used to assess bridge conditions, especially for routine inspection. The advanced inspection is always treated as a complementary method that always performed on an as-needed basis.

1.2.3.1 Visual Inspection

Visual inspection techniques are the primary methods used to evaluate the condition of the majority of the nation's highway bridges. These subjective assessments may have a significant impact on the safety and maintenance of a bridge. Detection of cracks, spalls, wear, efflorescence and other defects is the primary task for a visual activity.

To achieve good quality visual inspection result, dirt and debris must be removed in advance. Careful visual inspection should be supplemented with appropriate special equipment and techniques. Use of mirrors will increase visual access to many bridge components. Sketches, photographs and video cameras should be used as required to record significant or unusual details.

Whenever practical, inspection should proceed from substructure to superstructure to deck since the cause of superstructure and deck deficiencies may be more apparent if the substructure was inspected initially. Thoroughness and completeness should be guaranteed for every inspection. If, for any reason, a specific component or member cannot be inspected, it must be noted in the bridge inspection report.

1.2.3.2 Physical Inspection

In some condition, the physical condition of a certain element should be inspected. Therefore, some proper inspection method should be selected to assess the element.

Physically inspections should examine areas of concrete or rebar deterioration that is identified visually by using an inspection hammer. This hands-on effort verifies the extent of the deficiency and its severity. A delaminated area will have a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound concrete will result in a solid "pinging" type sound. This test could also be conducted by using drag chain. The hollow sound may indicate delamination as well. For large horizontal surfaces such as bridge decks, a chain drag may be used. A chain drag is made of several sections of chain attached to a handle

Special attention should be given to the location, length and width of cracks found during the visual inspection and sounding methods. For typical reinforced concrete members, a crack comparator card can be used to measure the width of cracks. This type of crack width measuring device is a transparent card about the size of an identification card. The card has lines on it that represent crack widths. The line on the card that best matches the width of the crack lets the inspector know the measured width of the crack. For prestressed members, crack widths are usually narrower in width. For this reason, a crack gauge is recommended which is a more accurate crack width-measuring device.

Rebounding hammer is widely used to test the compressive strength of the concrete.

1.2.3.3 Advanced Inspection Techniques (FHWA, 2012)

Advanced inspection methods give inspectors the ability to further evaluate suspected deficiencies found during a visual inspection. They can also be used to perform inspections on members that are not accessible. There are two main classifications of advanced inspection methods. The first is labeled nondestructive testing or evaluation (NDT or NDE). This classification pertains to advanced inspection methods that do not impair the usefulness of the member being tested. Other testing, the second main classification, covers advanced inspection methods that may affect the structural integrity of the member being tested.

Generally, NDT is not practical for large-scale use on a bridge unless a defect has first been detected by visual means. NDT can be used to highlight or define the extent of the defect. The Iowa DOT initiated a program containing the purchasing of corrosion detection device and used it for the routine inspection as a means for determining concrete removal for repair (Marks, 1977). Since most types of NDT require special equipment, and detailed instructions to perform the various tests, and correctly interpret the results, it is essential to have the NDT performed and interpreted by qualified personnel.

New technology is making the use of these highly technical systems more economically feasible for bridge inspection. From this fact, advanced inspection methods are becoming more popular for supplementing visual inspection methods predominately used for routine inspection of bridge members. Current studies have been focusing directly on relating results from advanced inspection methods into Bridge Management Systems ratings.

1.3 Condition Rating Items

Bridge components are the major parts comprising a bridge including the deck, superstructure, substructure, channel and channel protection, and culverts. Bridge elements are individual members comprised of basic shapes and materials connected together to form bridge components. Accurate assignment of condition ratings would help guide further inspection and maintenance practice (FHWA, 2012).

1.3.1 Evaluating Elements

From the AASHTO manual for bridge element inspection, the inspector should evaluate each element of each component and assign to it a descriptive condition rating of “good,” “fair,” or “poor,” based on the physical deficiencies found on the individual element.

For example, reinforced concrete deck, element 12, includes all reinforced concrete bridge decks regardless of the wearing surface or protection systems used. The possible defects are delamination/spall/patched area, exposed rebar, efflorescence/Rust staining, cracking, abrasion/wear and damage. For each type of defects, the quantity criteria are given for condition rating. Speaking of cracking, the condition is governed by the width and the density of the cracks. The deck evaluation is three-dimensional in nature with the defects observed on the top surface, bottom surface, edge, or all. Deck top or bottom surfaces that are not visible for inspection shall be assessed based on the available visible surface. If both surfaces are not visible, the condition shall be assessed based on destructive and nondestructive testing or indicators in the materials covering the surfaces.

Environmental factors should also be included since the elements exposed to different environmental factors and service environments deteriorate differently. When inventorying and assessing the condition of the elements, an inspector should consider the environment in which the element is operating. The environment impact could be classified as benign, low, moderate and server. For concrete bridges, the predominant environmental factors affecting an element would be freeze-thaw cycles, tire chain wear and deck salting.

1.3.2 Evaluating Components

Numerical condition ratings should characterize the general condition of the entire component being rated. They should not attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition rating must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated. Condition ratings assigned to elements of a component must be combined to establish the overall component condition rating.

The current New York State inspection program requires recording condition information for all elements on a span-by-span basis. Elements are rated using the following scale:

1. Totally deteriorated, or in failed condition
2. Used to shade between ratings of 1 and 3
3. Serious deterioration, or not functioning as originally designed
4. Used to shade between ratings of 3 and 5
5. Minor deterioration, but functioning as originally designed
6. Used to shade between ratings of 5 and 7
7. New condition -- no deterioration
8. Not applicable
9. Condition and/or existence unknown

Based on the result of condition rating evaluation, items rated 3 or lower may require substantial rehabilitation and items rated 4 or higher may be corrected or improved with maintenance work.

1.4 Inspection Practice of Individual Bridge components

Each individual bridge component requires special inspection techniques. The following section summaries the current inspection methods used for different bridge elements; bridge deck, bridge pier, etc.

1.4.1 Bridge Deck

As they are subjected to direct contact with traffic and environmental loads, the useful life of a concrete bridge deck is one-half the life of the bridge (Bettigole, 1990). In severe environments, this could even be much shorter. Therefore, the deck slab needs special attention for damage assessment and to operate the rehab project during the early year.

The most common bridge deck material is concrete. The physical properties of concrete permit placing in various shapes and sizes, providing the designer and the constructor a variety of construction methods. The role of a concrete bridge deck is to provide a smooth riding surface for motorists, divert runoff water, distribute traffic and deck weight loads to the superstructure, and act compositely or non-compositely with the superstructure. There are four common types of concrete decks: conventionally reinforced cast-in-place, precast conventionally reinforced, precast prestressed, and prestressed deck panels with Cast-in-place topping.

Wearing surfaces are placed on top of the deck protects the deck and provides a smooth riding surface. The wearing surface materials most commonly used on concrete decks are generally special concrete mixes or bituminous concrete. Wearing surfaces are incorporated in many new deck designs and are also a common repair procedure for decks. In some cases, epoxy polymers overlays may be used on concrete decks to prevent the infusion of chloride ions and can help provide skid resistance and protected system for 15 to 30 years, depending on the volume of traffic.

Common concrete deck deficiencies may contain cracking, scaling, delamination, spalling, chloride contamination, freeze-thaw, surface breakdown, pore pressure, efflorescence, ASR, ettringite formation, honeycombs, pop-outs, wear, collision damage, abrasion, overload damage, reinforcing steel corrosion or prestressed concrete deterioration.

Both the top and the bottom surfaces of concrete decks shall be inspected and special attention will be given to locations as areas exposed to traffic or drainage, bearing and shear areas, bituminous overlays, stay-in-places forms.

The visual inspection of concrete decks should contain the inspection for cracks, spalls, and other deficiencies. The physical examination of a deck with a hammer can be a tedious operation since the geometry property of the bridge decks. In most cases, a chain drag is used, as shown in Figure 1-1. A chain drag is made of several sections of chain attached to a pipe that has a handle attached to it. It will be dragged across a deck and make a note of the resonating sounds. A chain drag can usually cover about a 3-feet wide section of deck at a time. According to the result of chain-drag testing, suspect area would be examined with a hammer to determine the exact dimension of the delaminated area, as shown in below.

Many of the problems associated with concrete bridge decks are caused by corrosion of the steel reinforcement. When the deficiency of a concrete deck progresses to the point of needing rehabilitation, an in-depth inspection of the deck is required to determine the extent, cause, and possible solution to the problem. Several techniques and methods are available.

Figure 1-1 Chain Drag and Hammer Testing (SHRP2, 2012)

1.4.2 Superstructure

Structural deck, conventionally reinforced concrete girders, precast and prestressed I-girders and Bulb Tees, and prestressed box beams are the most commonly used superstructure forms for a concrete bridge.

The common deficiencies that occur on concrete superstructure includes cracking, scaling, delamination, spalling, chloride contamination, freeze-thaw, efflorescence, ASR, ettringite formation, honeycombs, pop-outs, collision damage, abrasion, overload damage, reinforcing steel corrosion or carbonation.

For structural deck, the inspector are checked for any signs and extent of leakage, cracks, spalls, delaminated areas, rust stains on Stay-in-place forms, (FHWA, 2012). The inspection should include both a general view of the deck for the entire span and a close view with sounding where appropriate. The general view will show the extent of deterioration. Areas where heavy leakage and spalling have occurred, or concrete appears saturated, should be sounded with a masonry hammer and findings documented. Temporary removal of planking or netting installed to catch falling concrete may be necessary for access to inspect the deck. Areas of map cracking, wetness, efflorescence, and spalls should be expressed in percentages of the deck area.

For conventionally reinforced concrete girders, the inspection covers the following (FHWA, 2012):

- Deterioration at the end of the beam which can lead to loss of bearing area and local crushing of the remaining concrete, especially when there are low skews and short end of beam overhangs.
- Near bearing areas at the ends of slabs, girders, T-beams, channel beams, etc., for spalling and cracked concrete. Any diagonal cracking in spandrel columns or at the ends of beams, girders, etc., is serious.
- Areas near supports for diagonal (shear) cracks occurring on exposed vertical surfaces and projecting diagonally toward the top of the girder, beam, etc.
- Tension areas at mid-span of simple spans for flexural cracks extending transversely across the underside of the primary member. Longitudinal flexural

cracks in the deck when the primary rebars are transverse. Transverse flexural cracks in tops of beams (slab portions) at or near piers on continuous spans.

- Areas with efflorescence indicating contaminated concrete and with rust stains indicating rebar corrosion. Spalling, delamination, and pop-outs commonly associated with deterioration. In severe cases, rebars will be exposed; determine the section loss of any exposed rebars.
- Longitudinal cracks between adjacent channel and T-beams indicating possible broken shear keys, differential deflections under passage of live loads, leakage, etc.
- Shear or torsional cracks at open spandrel arch floor systems, bent cap interfaces, or in spandrel bent caps or columns. Cracks in tension areas of spandrel bent caps. Deterioration of closed spandrel arches and spandrel walls to include cracks, discoloration, spalling, exposed rebars, etc. Differential movement, change of alignment/profile or loss of fill. Shear cracks in rigid frame beams (beginning at the frame legs and propagating toward the adjacent span), in the frame legs (beginning at the top and propagating downward), and in the ends of frame beams at end spans.
- Flexural cracks in tension areas of rigid frames at the bottom of the frame beam at mid-span, inside faces of frame legs at mid-height, the base of each frame leg, and the outside corners of a simple-span slab frame. Areas at, near, or under drainage features such as scuppers, weeps, curb lines, etc., for the loss of fill or deterioration of concrete.
- Areas of previous repairs, impact damage, honeycombing, scaling, and any other conditions indicating potential deterioration of concrete or rebars.

For prestressed concrete girders, the physical condition and functional capability of the primary members are rated under this element. Alignment, profile, and impact damage with all primary members should be examined. The inspector should look for and document any cracks in the members. Most cracks in prestressed beams are potentially serious since tensile forces exist that might not have been accounted for in the design. Vertical or diagonal tension cracks in prestressed members are signs that the prestressing steel (tendon) has failed or is failing, and the loads are being carried by adjoining beams. This is a serious condition and steps should be taken to ensure the stability of the bridge as soon as possible. In addition to the visual inspection, the following activities, at a minimum, should be performed as regulated by the NYSDOT's inspection manual:

- Sound the beams at the support area and mid-span location and any other areas showing deterioration.
- Evaluate and estimate or, if possible, measure any loss to exposed tendons and note location.
- Quantify debonded tendons and fully or partially broken tendons. Note these locations. Investigate previously repaired areas.
- Check drain holes and clear if clogged.
- Document findings with notes, photographs and sketches including full crack and deterioration documentation.

The two most common causes of losing prestressing forces are impact and corrosion. Generally, deterioration occurring in prestressed concrete members is evident to the inspector, but in some cases, serious but latent corrosion of the prestressing strands may exist without many outward signs of problems. Therefore, the manual also requires the inspector to pay more attention to concrete delamination, hairline cracks, and efflorescence or rust stains at the level of the prestressing strands and longitudinal cracks in the beam. Recent research has suggested that once outward signs of prestressing steel corrosion are visible, deterioration occurs very rapidly. The inspector should pay particular attention to areas where the concrete is patched. In such cases, the Regional Structures office should try to get information on the condition of the prestressing steel from those who did the repairs.

All primary members should be examined for signs of overstressing, fire damage, impact damage, improper profile or alignment, or excessive movements with passage of live loads. Behavior of primary members can best be observed during the passage of heavy live loads. The cause of any unusual sounds or excessive movements with the passage of live loads should be checked. If it involves a deficiency in the primary members, it should be considered in the rating.

1.4.3 Substructure

The substructure is the component of a bridge that includes all elements supporting the superstructure. Its purpose is to transfer the loads from the superstructure to the foundation soil or rock.

Abutment is a substructure at the end of a single span or at the extreme end of a multi-span superstructure. An abutment provides support for the bridge and retains or supports the approach fill. A Pier is a substructure supporting spans of a multi-span structure at an intermediate location between abutments. It should support the superstructure with minimal obstruction to the flow of traffic or water. Besides its own weight, the pier must carry vertical and horizontal forces transmitted by the ends of two superstructure spans, and restrain any span movement.

There are two types of visual inspections that may be required of an inspector. The first, called a routine inspection, involves reviewing the previous inspection report and visually examining the members of the bridge. A routine inspection involves a visual assessment to identify obvious defects.

The second type of visual inspection is called an in-depth inspection. An in-depth inspection is an inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine methods. Hands-on inspection may be necessary at some locations. This type of visual inspection requires the inspector to visually assess every defective surface at a distance no further than an arm's length. Surfaces are given close visual attention to quantify and qualify any defects. (FHWA, 2012)

For concrete members, the common deficiencies are similar to the superstructure. However, it may also experience deterioration due to temperature changes, chemical attack, moisture absorption, differential foundation movement, design and construction deficiencies, and unintended objects in concrete.

Once the defects are identified visually, physical methods are used to verify the extent of the defect. Areas of concrete or rebar deterioration identified visually need to be examined physically using an inspection hammer. The location, length and width of cracks found during the visual inspection need to be measured and recorded.

1.5 Discussion on Current Inspection Manuals (FHWA, 2012, AASHTO, 2013)

Current inspection manuals mainly require routine inspection with a fixed interval. With this process, the routine visual and sounding inspections focus primarily on the bridge elements' physical condition. This inspection process does not pay enough attention to the components that physically seem to be in good condition, but upon further inspection, may not be. Therefore, this practice may not be the best solution for the problem that the US aging bridge system is facing. Without complete information, the physical evaluation-based inspection may lead to delayed maintenance of a deteriorated concrete bridge, which is fundamentally wrong. Problems exist because of a fixed inspection interval, reliance on visual inspection, and limited emphasis on chemical inspection.

1.5.1 Cyclic Inspection Interval

As stated previously, the current inspection manuals give recommendations for general inspection based on a fix inspection interval.

Certain bridges may qualify for an extended inspection interval if the previous inspection shows that the components perform at a high condition state and approved, written permission is issued by FHWA. On the other hand, bridges under serious condition (General recommendation/Condition rating equals 3 or less) require more frequent inspection with a shorter interval.

Without an understanding of the chemical deterioration mechanism and the actual deterioration rate of the bridge, the fixed interval inspection can only cause either excessive physical inspection, or delayed maintenance. If the deterioration of a certain bridge element is low, the inspection interval should be extended, and the duration of extension should be calculated scientifically based on the combined deterioration effects of various deterioration mechanisms. On the other hand, delayed maintenance is also a possible outcome for fixed interval inspection. Since the maintenance strategy is made based on the most recent inspection result. In some cases, the bridge component may deteriorate quickly from a higher condition state to a lower one, especially for condition 5 and 4. If the optimized treatment window closed between two inspections season, the available treatment may changes from low cost preventive maintenance or rehabilitation to high cost major repair or replacement.

1.5.2 Limitation of Visual Inspection

The current inspection manual mainly relied on visual inspection and physical conditions to assess the condition of a bridge element or component. It comes with several short comings, as it heavily relies on the physical assessment rather than the chemical assessment of the bridge condition.

First of all, visual inspection is not reliable since it can be affected by a myriad of factors. According to the result from (FHWA, 2001), actual routine inspections are completed with significant variability from the routine inspection tasks, as they depend on experience of individual inspectors. This variability is most prominent in the assignment of condition ratings, but is also present in inspection documentation. As an example, on average, four or five different condition ratings were assigned to each element. Based on the application of statistical models, it is predicted that only 68 percent of the condition ratings will vary within one rating point of the average. Similarly, it is predicted that 95 percent of the condition ratings from bridge inspections will be distributed over five contiguous condition ratings, centered about the average. Also, it was observed that condition ratings are generally not assigned through a systematic approach. Based on the distribution of the condition ratings and observations made during the study, the NBIS condition-rating definitions may not be refined enough to allow for reliable routine inspection results. Other factors that may affect the visual inspection results are fear of traffic, near visual acuity, color vision, formal bridge inspection training, light intensity, reported structure maintenance level, reported structure accessibility level, reported structure complexity level, inspector rushed level, and wind speed.

Furthermore, it is obvious that some components are hard to access for inspectors to visually inspect or develop a physical condition assessment due to location or geometry restraints. The bridge deck, for example, is a three-dimensional element that should be evaluated from the top and bottom surfaces and edges. However, in some conditions, due to the existence of a wearing surface, girders, or waterway, it is hard for inspectors to give an accurate assessment by visual inspection and a hammer. As motioned above, the accessibility may be a dominant factor for the accuracy of state rating of a bridge element. Figure 1-2 shows an inspector using a manlift to perform a hands-on inspection for the bottom of the deck. This equipment may not useful for bridges across waterways or bridges have a clearance height greater than 170 feet.

Figure 1-2 Manlift (FHWA, 2012)

For components like this, advanced non-destructive test methods should be adopted. Methods depending on sonic wave, such as Cross Sonic Logging, Sonic Echo/Impulse Response, and Impact Echo are commonly suggested. These methods are usually able to identify the details of problems, including location and type of defect, properties of concrete. Some other methods rely on visual sense, like Infrared Thermography, could quickly show the presence of crack and delamination, they are also recommended if the minimum traffic interruption is desired.

1.5.3 Insufficient Advanced Inspection

As mentioned above, an in-depth inspection by definition is a close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection. Hands-on inspection may be necessary at some locations. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, nondestructive or partially destructive field tests, also referred as advanced inspection methods, may need to be performed. An in-depth inspection, which includes both chemical and electrical NDTs and PDTs, can be scheduled independently of a routine inspection or as a follow-up for other inspection. Therefore, in-depth inspection is always requested on an as-need basis.

The criteria for different bridge components are different. The requirement for performing in-depth inspection using advanced inspection methods are discussed below.

For bridge deck, when the deficiency progresses to the point of needing rehabilitation, an in-depth inspection of the deck is required to determine the extent, cause, and possible solution to the problem. For instance, when any spalling or delamination is suspected or evident, the entire deck area should be surveyed to determine the extent of spalling and delamination before beginning repairs. The survey aspects need to be considered include the delamination survey, reinforcing cover survey, chloride content survey and corrosion potential survey. In order to complete the survey, ultrasonic delamination detector, magnetic field detector, samples of concrete powder produced by drilling holes and electrical resistivity measurements with a half-cell probe may be needed.

For bridge elements underwater, nondestructive testing, along with other testing methods, such as extracting samples for laboratory analysis and testing, boring and probing, is normally performed under a level III inspection. This type of inspection is needed if routine inspection yields to inconclusive results or the structures is critical whose loss would have significant impact on life or property. Other situations that may warrant an in-depth or a special underwater inspection include prior evidence of distress, unusual floods, vessel impact, unusual ice floes, prop wash from vessels, adverse environmental conditions, floating and build-up of debris and above water evidence of deterioration or movement.

For other bridge components, the advanced inspection technology are commonly used as a complementary method, while it is hard to access to perform routine inspection using visual and conventional inspection methods.

In general, it can be concluded that the in-depth inspection using advanced inspection testing methods are needed and required as follows:

1. Alternative methods to get inspection done while it is hard to get access for routine inspection including visual and conventional physical inspection methods.
2. Complimentary methods to get accurate condition assessment while routine inspection cannot gives conclusion about the deterioration condition about area, severity and progressing status.

3. Helping to make decisions regarding major rehabilitation, repair or replacement of deteriorated bridge components.

The potential benefits for using advanced NDT/PDT methods have been tremendously undermined. The advanced NDT/PDT methods should not be limited as in-depth inspection methods. It needs to be regulated into the routine inspection process in order to achieve more cost-effective bridge life-cycle management strategy.

1.6 Conclusions and Recommendations

1.6.1 Conclusions

A cost-effective bridge is a bridge whose maintenance is based on its chemical condition over its entire service life. The practice for physical evaluation and delayed maintenance of deteriorated concrete bridge is ineffective. The current inspection manuals are primarily depending on the result retrieved from visual inspection and physical inspections that are focused on detecting physical damage in concrete bridge elements. This inspection method may only lead to either over excessive inspection or delayed maintenance.

The fixed inspection interval, physical condition based test methods and the limited recommendation for advanced chemical NDT methods are the primary problems that prevent the realization of truly cost-effective preventive maintenance strategy.

In order to implement a scientific preventive maintenance program, the decision making process should rely on the results retrieved from chemical NDT methods which are recommended for every bridge element. Only by controlling the electrochemical deterioration mechanism in the concrete bridge, the deterioration of the bridge can be controlled with extended service life, enhanced safety and optimized life cycle cost.

1.6.2 Recommendations

Since the current inspection manuals have their limitations, several recommendations are given for the inspection practice.

1. A preventive maintenance approach responding to inspection result should be based on the chemical condition instead of physical condition of a bridge element. Instead of parameters such as crack width, delamination area, and reinforcing steel cross-section reduction, the chemical condition of the bridge element should be examined. Features such as chloride content, moisture content, carbonation depth, and porosity reactivity of aggregate should be examined. Threshold should be set for selection of proper maintenance based on the study of the theoretical and numerical deterioration mechanism.
2. Advanced inspection methods including chemical NDT methods should be included into routine inspection in order to provide scientific support for maintenance decision-making. Air and water permeability can be measured by drilling a small hole into the concrete, sealing the top with liquid rubber, and inserting a hypodermic needle; Air permeability can then be determined by filling the hole with water and measuring the flow in to the concrete at a pressure similar to that of rainfall; Moisture content can be determined using nuclear methods or

from concrete samples taken from the bridge and oven dried in a laboratory; Rapid chloride permeability test is one of the current standard test methods used to assess the resistance of concrete to penetration of chloride ions known as AASHTO 277-93; Accelerated mortar bar test, accepted by ASTM and AASTHO, can be used for ASR evaluation. These test along with the carbonation test, should be incorporated into routine inspection.

3. Bridge elements in good and fair condition should be examined as well as the elements in poor condition. The advanced inspection methods should not be only scheduled for emergency response or rehabilitation decision making assistant. It should be performed to every bridge element on a more frequent interval to achieve the most cost-effective preventive maintenance, in which the condition state is always required to be 6 or higher.
4. All the inspection data and preventive maintenance activity should be recorded for the optimization of the inspection scheduling. The advanced inspection methods including chemical NDTs gives the inspector the ability to evaluate the deterioration rate of a certain bridge element. Using a proper deterioration estimation model, the inspection intervals could vary according to the current element state and the deterioration rate.

Although the implementation of chemical NDTs seems wise, there are some issues that still need further investigation to make it practical. First of all, the mathematical deterioration model needs to be studied in order to provide a good preventive maintenance recommendation. The deterioration mechanism of concrete, as well as combined effects of various deterioration mechanisms should be researched. Secondly, thresholds for various harmful compounds, including combined effects should be established. Thirdly, the adoption of advanced inspection methods needs experienced and qualified inspectors; training and proper qualifications are needed for the inspectors to be able to select the test methods, perform the field or laboratory tests and interpret the tests results. At the same time, special equipment is commonly needed to perform a non-destructive test or a partial-destructive test. It might be a problem for State DOTs to make these purchase since the up-front payment may appear to be high, even though the expenses could be easily recovered due to savings from less expensive maintenance and longer service life of the bridge. The implementation of a new inspection strategy that could help better preventive maintenance practice needs the commitment from all the parties, the DOTs, inspectors and constructors.

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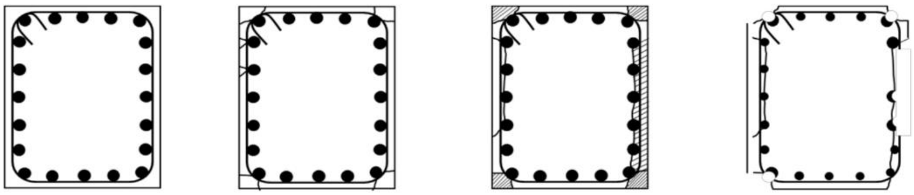
Chapter 2 Review of the Current Maintenance Methods

2.1 Introduction

Bridge maintenance can generally be classified into four stages: preventive, active, reactive, and delayed. The type of maintenance strategy an agency employs is largely related to the assets and capabilities of said agency. As described in detail in NCHRP 14-20 "Consequences of Delayed Maintenance," a balance must be reached between cost, desired service level, delay/catch-up periods, discount rates, and other factors. The typical outcome for different maintenance type is listed along with the concrete condition, structure safety and the cost for the retrofit measures as shown in Figure 2-1. It can be seen that if the small defects cannot be rehabilitated on time, it will become a threat for the overall safety of the structure and the maintenance cost will increase dramatically as the degradation goes on.



Maintenance Type	Active Preventive	Passive	Passive Delayed	Ignored
Condition	Sound	Cracked	Cracking & Delamination	Loss of section
Strength	High	Medium	Medium	Low
Safety	High	Medium	Low	Critical
Retrofit Measures Involves	Protective coatings /Sealers	Replace damaged concrete Corrosion protection system	May require bridge closure May require Bridge Lifting Chip concrete behind bars Corrosion protection system Replace damaged concrete	Close Bridge Lifting Bridge Superstructure Chip concrete behind bars Corrosion protection system Add steel rebars Replace damaged concrete
Cost	$	$$	$$$	$$$$

Figure 2-1 Qualitative cost of maintenance versus type of maintenance method

2.2 Current Maintenance Methods and Policies

2.2.1 Preventive Maintenance

Methods and policies relating to preventive maintenance activities are described in-depth elsewhere in this report. Preventive maintenance procedures generally consist of cyclically scheduled activities including:

- Washing/cleaning concrete structural decks and superstructure/substructure annually or biannually after the last deicing

- Ensuring that proper drainage is maintained by cleaning deck joints and drains of debris
- Sealing decks and other concrete elements on a scheduled basis
- Maintaining bearings (painting, lubricating if necessary, etc.)

2.2.2 Active Maintenance

Where preventive maintenance aims to prevent problems before they occur, active maintenance is the policy of actively treating and fixing these problems. Types of activities associated with active maintenance include the sealing of fine cracks and proactive repair of deck joints and bearings. Active maintenance may still be classified as preventive because these types of activities prevent or delay larger problem from occurring, like concrete spalling or bearing/joint failure.

Fine cracks can occur in concrete as a result of thermal stresses or drying shrinkage stresses, bending or flexure in the deck or element, foundation settlement, or many other issues. As described in AASHTO's maintenance manual, if these cracks are greater than .007 inches in width and reach the reinforcing steel, moisture and associated chlorides can be allowed to "initiate and support corrosion of the reinforcing steel," even with epoxy coated steel. To mitigate these risks, small cracks that exist over large areas can be effectively sealed by applying a liquid sealer to the entire surface. Additionally, large cracks that are not actively expanding can be sealed by injecting a sealant.

In some cases, an overlay may be applied to a bridge deck to increase service life. These overlays can be cement-based, asphaltic concrete with a waterproof membrane, or polymer-based.

2.2.3 Reactive Maintenance

Reactive maintenance is like active maintenance in that there is no action taken until a problem arises. For example, where an active maintenance strategy would take measures to seal a fine crack soon after it was noted, a reactive maintenance strategy would fix the issue after significant deterioration occurred (spalling, exposed reinforcing steel, etc.) by removing chloride-contaminated concrete and/or replacing corroded reinforcing steel, and patching the element.

Other examples of reactive maintenance activities are as follows:

- Replacing deck joints due to significant deterioration
- Replacing bearings or parts of bearings due to being frozen/corroded.

Because of the nature of reactive maintenance, it is typically performed on an as-needed basis with more urgent repairs (for instance, full depth holes in concrete decks) taking precedence. In fact, some state DOT manuals state that issues like potholes on concrete decks and spalling can simply be left alone until the structure is further compromised.

2.2.4 Delayed Maintenance

Delayed maintenance is simply the lack of maintenance of an element until it fails (and is then replaced). The use of this policy may be due to the lack of funding available for continual maintenance throughout the life of the bridge. However, delayed maintenance can lead to the necessity of rehabilitation or replacement sooner than if maintenance had been performed throughout the service life. Because of this, the higher cost of replacement/rehabilitation (and the traffic disruption that occurs because of it) occurs with higher frequency when the delayed maintenance strategy is used.

Additionally, delaying maintenance may lead to significant, unexpected failure of structures. For instance, in 2005 a highway ramp span partially collapsed in Albany, NY. The bearings that were the cause of the collapse were known to be out of alignment for more than 20 years. As a result, unforeseen corrosion occurred in the bearings and froze them. There are countless other examples of delayed maintenance leading to bridge closure or even collapse in recent years.

2.2.5 Different Maintenance Activities for various Bridge Elements

Due to the different function, geological location, and the exposure condition of the structure elements, the maintenance activities may vary for different bridge elements. The typical types of maintenance measures are summarized in Table 2-1.

Table 2-1 Typical types of Maintenance for Various Concrete Bridge Elements

Typical types of maintenance for various concrete bridge elements				
Bridge element	Preventive	Active	Reactive	Delayed
Concrete structural Deck	Annual or biennial washing; seal deck on a scheduled basis	Seal fine cracks as they appear	Remove concrete around spalled areas and patch; perform full-depth repairs; possible overlay entire deck if necessary	Do nothing until entire deck (or portions thereof) needs complete replacement.
Bearings	Clean and paint (where applicable) as needed	Realign bearings if necessary	Replace frozen or otherwise deteriorated bearings	Take no action until bearing failure.
Superstructural/substructural concrete	Annual or biennial washing; seal elements on a	Seal fine cracks as they appear	Patch spalled areas	Wait until serious deterioration has occurred where

	scheduled basis			significant levels of replacement or rehabilitation are necessary.
Deck joints	Clean and remove debris on annual basis (or as needed)			Wait until serious deterioration has occurred where significant levels of replacement or rehabilitation are necessary.
Deck drains	Clean and remove debris on annual basis (or as needed)			Wait until serious failure has occurred where significant levels of replacement or rehabilitation are necessary.

2.3 Current Practice for Preventive Bridge Maintenance

Preventive bridge maintenance aims to “preserve bridge components in their present (or intended) condition, forestalling development of a structural deficiency.” Preventive maintenance activities are generally either proactive or reactive. As stated in AASHTO’s Maintenance Manual for Roadways and Bridges (2007), typical proactive (scheduled) activities include:

- Cleaning decks, seats, caps, and salt splash zones;
- Cleaning bridge drainage systems;
- Cleaning and lubricating expansion-bearing assemblies; and
- Sealing concrete decks or substructure elements.

Cleaning and sealing activities is significant important to concrete bridges. Cleaning is an important part of preventive bridge maintenance because if helps facilitate drainage of water and the removal of contaminants from the deck and other elements. Sealing is an important activity in preventive maintenance in concrete bridge elements because sealing concrete helps reduce the intrusion of chloride and other ions into the concrete.

The high pH of concrete protects reinforcing steel because, in a highly alkaline environment, a microscopic oxide layer is formed around the steel, protecting it corrosion. However, it is widely thought that chloride ions penetrate this protective coating easier than other ions, leaving the unprotected steel vulnerable to corrosion (OHDOT, 2014). By keeping chloride ions from entering the concrete and reinforcing the steel, corrosion can be reduced and service life can be increased.

In addition to the concrete elements of bridges, other bridge components like bearings and joints also require preventive maintenance.

2.3.1 Concrete Structural Decks

In concrete bridges, the structural deck generally acts as the roof of the bridge, protecting supporting elements from most exposure to environmental factors like precipitation and exposure to deicing salt and other chemicals in the "snow-and-ice-belt." Additionally, concrete bridge decks may be exposed to marine saltwater in coastal areas. Left unchecked, chlorides from these sources gradually penetrate through the bridge deck and react with reinforcing steel.

Using high quality original construction materials like low water-to-cement ratio concrete, reinforcing steel with corrosion-resistant coating or by maintaining existing bridge decks with methods like sealing or the installation of cathodic protection systems can help counteract the negative effects of chloride penetration. (AASHTO, 2007)

2.3.1.1 Cleaning

Cleaning is perhaps the most fundamental form of preventive maintenance for bridge decks. By keeping the bridge deck clean and maintaining water runoff drains, water from precipitation (which contains chloride ions during winter deicing activities) is less likely to pond on the road surface and further the deterioration of the structural deck.

An example of a typical state transportation agency's policy regarding bridge deck cleaning is found in the Ohio Department of Transportation's online bridge maintenance manual: "Sweep (power broom) and flush with water (preferably power wash) at least once each year in the spring, after last use of deicer chemicals or grits for the season. This action is usually only necessary in the gutter areas (beyond the white edge lines)."

Additionally, the Pennsylvania Department of Transportation follows AASHTO recommendations and follows an annual cleaning cycle for most bridge decks. However, NYSDOT designates a biennial cleaning cycle for bridge decks. Other policy examples are listed below. The prescribed cleaning cycle by different is summarized in Table 2-2.

Table 2-2 Cleaning Cycle by DOTs

State Agency	Prescribed cycle period

New York State Department of Transportation	2 years
Ohio Department of Transportation	Annually after last deicing
Pennsylvania Department of Transportation	Annually after last deicing
Georgia Department of Transportation	Annually if exposed to snow/ice event that requires deicing; 2 years otherwise

2.3.1.2 Cracks and Repairs

Cracks in concrete decks can be classified as transverse cracks, longitudinal cracks and other types of cracks.

Transverse cracks are cracks that are perpendicular to the longitudinal axis of the bridge deck, and they are the main type of cracking found on reinforced concrete bridge decks. These cracks generally form at the surface of the bridge deck under which the transverse reinforcement is placed. Transverse cracks are also typically full depth and located 3-10 feet apart along the length of the concrete bridge deck. Transverse cracks always appear very early in the construction process; they typically appear soon after the casting of the concrete. The location and positioning of transverse cracks is critical to the service life and maintenance costs of reinforced concrete bridge decks. Since the transverse cracks generally develop above the transverse reinforcement, deteriorating chemical agents, like deicing chemicals, can easily find access to the reinforcing steel.

Longitudinal cracks occur primarily on solid and hollow slab bridges can generally are the result from differential movement along the beams. Skew of the beams, on the other hand, may be the cause of diagonal cracks formed on the bridge decks.

Cracks in concrete decks can be caused by: thermal stresses or drying shrinkage stresses; issues with production or placement like defective aggregates, too much water in the concrete mix, improper curing; bending and flexure stress in the deck; movement between beams and girders supporting the deck; foundation settlement (AASHTO, 2007). The general causes for crack formation are illustrated in Figure 2-2. Cracks may not appear simply because of lack of maintenance but still need to be remediated to avoid further deterioration.

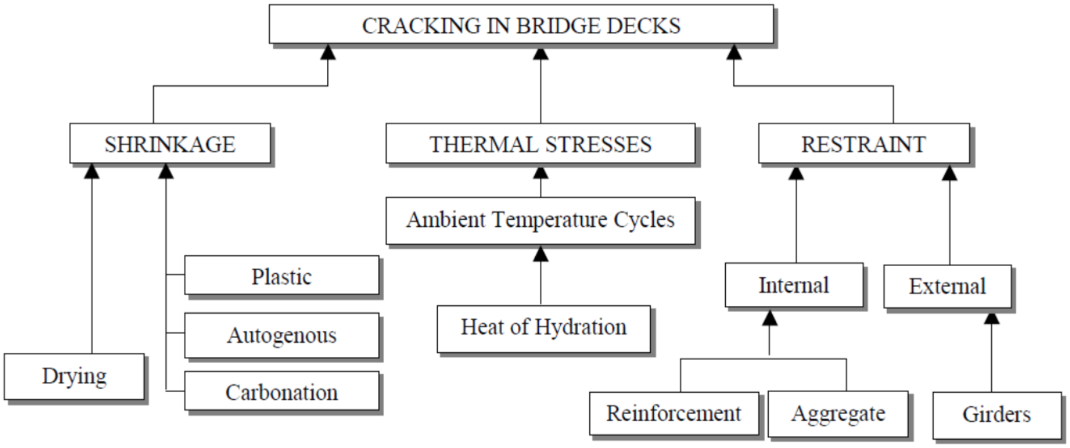


Figure 2-2 Cause of Cracking in Bridge Decks (S. Ganapuram, 2012)

It is well known that concrete has relatively low tensile strength, and this characteristic is one of the important causes of cracking. In its early age, concrete cracking occurs due to the restraint of the concrete. The volumetric movement of concrete can result from drying shrinkage, autogenously shrinkage, plastic shrinkage, and thermal loads (Kosmatka et al., 2002).

Since shrinkage of fresh concrete is the main cause of bridge deck cracking, control of the evaporation of water from the concrete surface is extremely important. Two construction practices that affect the water in concrete are fogging and curing. When designing the concrete mixture design used for a bridge deck, researchers recommend using a concrete with low early strength, low elastic modulus, low heat of hydration, high tensile strength, and high creep in order to mitigate shrinkage.

Based on the research, the transvers cracks density is affected by the following parameters.

1. Decks constructed with stay-in-place forms exhibited much less cracking than those built with removable forms.
2. The transverse crack intensity (total length of cracks per 100 ft2) increased as the span length increased.
3. Superstructure type had a significant effect on the amount of cracking observed. Steel bridges had more cracking than prestressed concrete bridges
4. Cracking is more prevalent on continuous spans than simple spans.
5. Construction practices were the single most influential variable in the extent of cracking observed in bridge decks
6. The use of retarder in the concrete mix is not an important factor.

The researchers determined that span type, concrete strength, and girder type were the most important design factors influencing transverse cracking. Material properties such as cement content, cement composition, early-age elastic modulus, creep, aggregate type, heat of hydration, and drying shrinkage also influenced deck cracking.

Based on the study done by various researchers, deck cracks that are less than 0.0079 in. (0.2 mm) in width typically do not need to be filled if subjected to moderate or slight aggressive environmental. Cracks that are less than 0.0059 in. (0.15mm) in width at a drying age of six months (< 0.004 in. or < 0.1 mm in width at a drying age of one month) have no significant problematic effect on either the deck. Therefore, there is no need to be filled. However, cracks that are greater than 0.0059 in. (0.15mm) in width at a drying age of six months (> 0.004 inch or >0.1mm in width at a drying age of one month) should be filled to prevent water and chlorides from penetrating the cracks.

While small cracks existing over large areas may be effectively sealed with the application of a liquid sealer over the entire deck, large, open cracks that are stable can be filled and sealed with an injected sealant. Crack sealants are typically epoxy or polyurethane-based. Perhaps the most common concrete deck crack sealer is High Molecular Weight Methacrylate (HMWM).

When examining the effectiveness of a concrete sealer, numerous criteria need to be examined including: absorption, chloride ion penetration resistance, scaling resistance, skid-resistance (if located on a wearing surface), and freeze-thaw durability. Additionally, the presence and size of cracks is important when choosing a sealing method.

For example, the Ohio Department of Transportation designates the following procedure in their online bridge maintenance manual:

- “Seal the entire deck surface with an ODOT- approved Saline or siloxane (once every 5 years).”
- “If the deck has obvious transverse cracks in the surface, the crack areas can be areas can be flooded with an ODOT- approved high molecular weight methacrylate (HMWM), or a gravity fed resin or a silicate solution.”

Additionally, the New York State Department of Transportation (NYSDOT, 2008), in its Fundamentals of Bridge Maintenance and Inspection manual, designates a four-year cycle for concrete bridge deck sealing with similar procedures for crack sealing.

Concrete sealers are tested per several criteria. A sample of acceptance criteria for state agencies as of is included below, but most state agencies followed similar acceptance criteria. In these states, sealers are tested per AASHTO, ASTM, and/or state-specific standards and then placed on that state’s approved materials list for future list. It is important to note that there is no mention of long-term performance criteria listed for the examined agencies.

A survey of state agencies in 1992 showed the usage of various testing methods for penetrating sealers (Whiting et al, 1992), as shown in Table 2-3:

Table 2-3 Test Procedure Reported by State Agencies (Whiting, 1992)

Test Procedure	Number of Agencies (As of 1992)
AASHTO T259	13

NCHRP 244[a]	9
ASTM C642	6
Absorption (Not ASTM C642)	6
Rely on Vendor Data	6
Penetration Depth[b]	5
Vapor Permeability	5
Other Tests	5
ASTM C672	3
AASHTO T277	2
Freeze-thaw Testing	2
Skid Resistance Testing	1

[a] Most agencies utilize the series II testing procedure described in NCHRP Report 244

[b] Test procedures developed by Oklahoma DOT (OHD L-40 "Method of Core Test for Determining Depth of Penetration of Penetrating Water Repellent Treatment Solution into Portland Cement Concrete")

The commonly used testing method standards and the descriptions are listed in Table 2-4.

Table 2-4 Testing Method for Sealers

Testing Method	Description
AASHTO T259	Resistance of Concrete to Chloride Ion Penetration
AASHTO T277	Electrical Indication of Concrete's Ability to Resist Chloride Iron Penetration
ASTM C642	Standard Test Method for Density, Absorption, and Voids in Hardened Concrete
ASTM C666	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
ASTM C672	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

NCHRP Report 244 Series II testing procedures	Test originally used to determine effect of concrete moisture content at the time of sealer application upon chloride intrusion into concrete; now used by many states and manufacturers as a standard method of testing effectiveness of sealers against chloride intrusion.

Based on the tests listed above, the states approved various sealers, as listed in the appendix A. From the list, it can be derived that saline is the most commonly used active ingredient for the various sealers. The concentration of the active ingredient varies from 20% to 100%, with the most of 40%, as shown in Figure 2-3. The solvent is generally either water or alcohol. The water weight gain reduction, absorbed chloride, moisture permeability and penetration depth are listed in order conduct a comparative analysis.

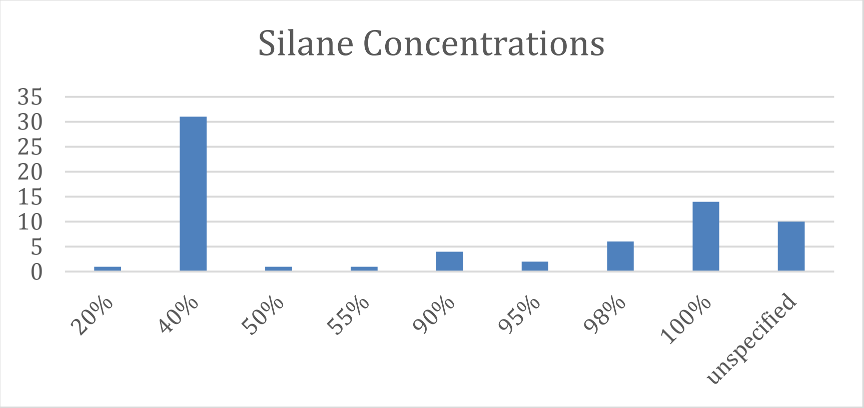


Figure 2-3 Saline Concentrations

The saline list shows that the amount of active ingredient is mostly grouped in the region of 40% or 85% or higher. For penetrating saline, the active ingredient concentration is always above 85%. And the target penetration depth is usually set at 0.4 inch, as shown in the Figure 2-4.

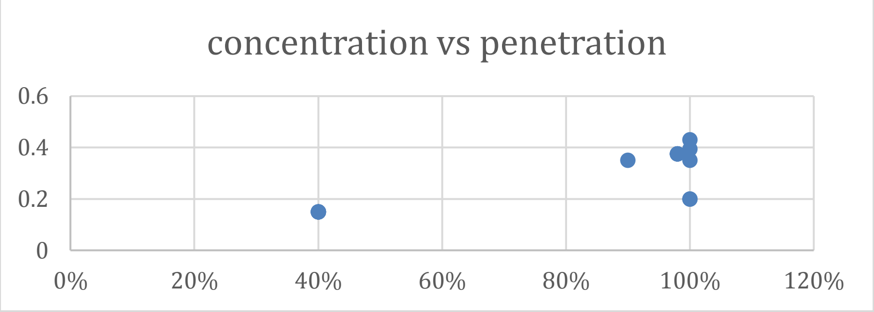


Figure 2-4 Saline Concentrations vs. Penetration

The effectiveness of the sealer is commonly tested by the water absorption resistance as well as the reduction in absorbed chloride after application of the sealers compared with the uncoated specimen. Figure 2-5 shows that most sealers will achieve a water absorption resistance higher than 80%, along with the reduction in absorbed chloride at the same level.

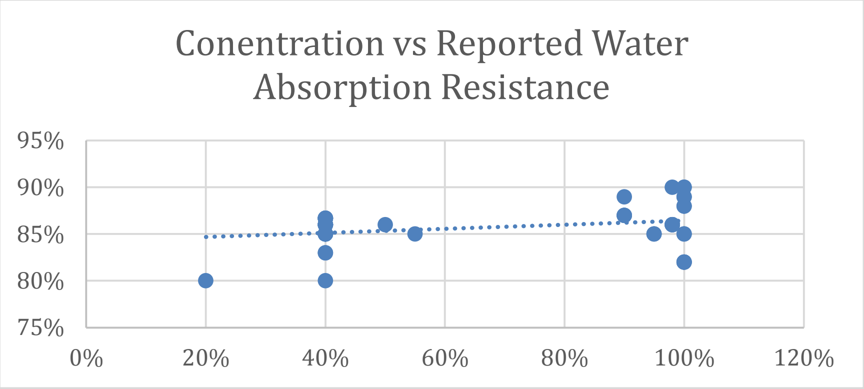


Figure 2-5 Saline Concentration vs. Water Absorption Resistances

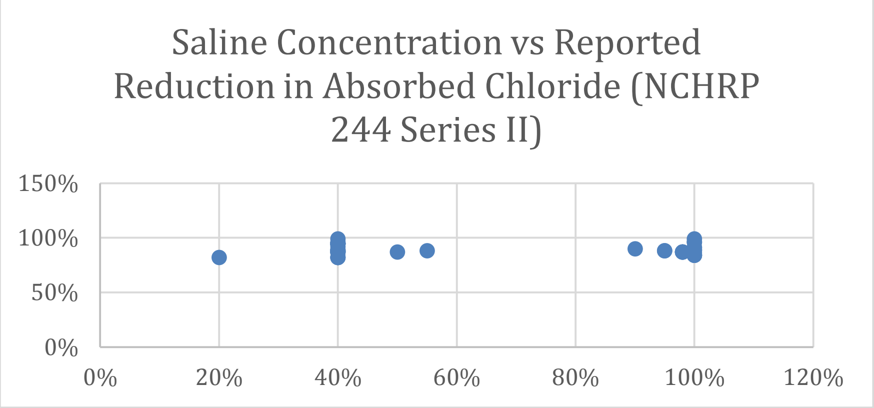


Figure 2-6 Saline Concentrations vs. Reduction in Absorbed Chloride

Among other parameters, the chloride diffusion process is mainly affected by the chloride concentration at the surface. Please see Figure 2-6. Therefore, the use of sealers will increase the time needed for corrosion initiation and ultimately elongation of the service life of the structure.

In general, several types of surface penetrating sealers and gravity-fill crack sealers have been applied successfully on concrete bridge decks. Based on their performance, Saline and siloxanes are considered the most popular penetrating sealers. In the gravity-fill group, HMWM and low viscosity epoxy are used favorably. HMWM can be used successfully to seal cracks wider than 1.0 mm (0.04 in.). However, results from the nationwide survey revealed HMWM is applied to cracks narrower than 1.6 mm (0.0625 in). For cracks narrower than 1.0 mm (0.04 in.) HMWM can be used following an application of Saline.

Saline may be applied first as surface treatment followed by HMWM as crack filler.

2.3.1.3 Expansion Joints

AASHTO recommends periodic cleaning of bridge decks, including the removal of debris. Problems that can arise when deck joints are not properly cleaned include water backing up and contributing to concrete corrosion for open joints with troughs. Additionally, when a filled butt joint is "not maintained to retain a watertight seal, the filler material below the seal will deteriorate and incompressible fine material can enter the joint, preventing it from relieving the stress of deck expansion."

Most state agencies examined, including those in New York, Ohio, and Georgia, specify the annual removal of debris from deck joints, typically during the deck cleaning or with brooms and shovels at a different time.

Table 2-5 State DOT's Policy for Expansion Joints Maintenance

State Agency	Prescribed cycle period
New York State Department of Transportation	Annual
Ohio Department of Transportation	Annual
Pennsylvania Department of Transportation	Annual
Georgia Department of Transportation	Annual

2.3.2 Substructure and Superstructure Concrete Elements

2.3.2.1 Cleaning

AASHTO's Maintenance Manual states "the presence of debris on the caps results in corrosion of the bearing systems since the debris tends to hold water (and associated deicing chemicals in freeze-thaw climates) for expanded periods of time. The penetration of deicing chemicals into concrete can corrode reinforcing steel, which in turn expands and breaks out (spalls) the covering concrete." To minimize the risks to structural concrete and bearings, most state departments of transportation designated annual or biannual bridge cleaning after the final application of deicing chemicals for the winter season with AASHTO recommending annual cleaning.

The Ohio Department of Transportation and Pennsylvania Department of Transportation follow AASHTO recommendations, and designate annual cleaning for superstructure/substructure elements, while NYSDOT designates biennial cleaning. The cleaning cycles set by DOTs for concrete substructure and superstructure elements are listed in Table 2-6.

Table 2-6 Cleaning Cycles by DOTs

State Agency	Prescribed cycle period

New York State Department of Transportation	Biennial
Ohio Department of Transportation	• Piers with unsealed joint above: Power wash annually • Backwalls: annually
Pennsylvania Department of Transportation	Annual
Georgia Department of Transportation	As needed
Florida Department of Transportation	Clean pier caps during inspection (not more than 24 month interval)

2.3.2.2 Sealing

As described in the section detailing concrete decks, sealing is an important part of preventive maintenance because reducing the intrusion of chloride ions into a concrete structure means extending its service life. Sealing of substructure and superstructure elements on concrete bridges can again be achieved with the use of penetrating type sealers, which are generally approved for use for both bridge deck and substructure/superstructure applications. Additionally, many state transportation agencies have provisions in their standard specifications for coating-type sealers for substructure/superstructure elements. As previously described, penetrating sealers react chemically with the concrete and fill pores while coating-type sealers simply block the pores.

An example of common practice can be found in the Ohio Department of Transportation's online bridge maintenance manual:

- For prestressed box beams/I-beams "Seal fascia beams with Saline/siloxane sealer every 5 years
- For concrete arches: "Seal all components of the arch within the splash zone with Saline/siloxane or epoxy/urethane."
- For piers: "If there is no unsealed deck joint over the pier, generally no maintenance required" or, "If there is an unsealed deck joint over the pier, power wash annually and seal with Saline/siloxane every 5 years or epoxy/urethane every 10 years"

Overall, current practice for state transportation agencies generally falls in line with these guidelines as shown in Table 2-7.

Table 2-7 State DOT's Policy for Superstructure Sealing

State Agency	Prescribed cycle period

New York State Department of Transportation	6 years
Ohio Department of Transportation	5 years
Pennsylvania Department of Transportation	Described above
Florida Department of Transportation	Unspecified

2.3.3 Bearings

Generally, all types of bearings used on concrete bridges have similar preventive maintenance requirements: cleaning the area of debris, painting the bearing assembly if necessary, and lubricating the bearing elements, depending on the type of bearing.

Given below in Table 2-8 are AASHTO recommendations for bearing types commonly used in concrete bridges and the associated recommended preventive maintenance activities.

Table 2-8 AASHTO Bearing Type and Recommended Preventive Maintenance Activities

Bearing type	**AASHTO Recommended preventive (cyclic) maintenance activities**
Sliding bearing	• Clean and remove debris from bearing area • Lubricate bearing if it has a grease fitting • Jacking bearing to remove the bearing surface for cleaning if it is corroded does not allow movement ○ clean, lubricate, reposition (if needed) • Consider installing a grease fitting if one isn't currently there • Lubricants can include waterproof grease, oil, or graphite among other options
Roller bearing	Since most roller assemblies are enclosed, simply keep bearing area cleaned and painted
Rocker bearing	• Keep assembly clean, lubricated, and painted. • Remove dirt and debris from under the rocker

	○ If problem persists, a cover should be designed and installed to keep contaminants away from the rocker. • Maintain pins to allow free movement (special ones can be made with grease fittings)
Elastomeric bearing	Maintenance rarely required unless it slips out of position; keep bearing area clean and free of debris
Pot bearing	Keep bearing area clean and free of debris

Current maintenance specified by the state agencies tends to follow these guidelines, as described below:

Table 2-9 Preventive Maintenance Activities for Bearings by State DOTs

State Agency	Bearing Type (if specified)	Prescribed Preventive Maintenance
New York State Department of Transportation	N/A	Clean and lubricate bearings at least once every four years • Develop a jacking plan if necessary • Flush with high pressure air or water to remove loose material • Remove rust and scale from bearings by scraping, wire brushing, or sandblasting and reflush • Jack structure pursuant to jacking plan and remove bearings, if necessary for proper service, depending on the type of bearing. • Clean and grind bearing surfaces to a smooth finish. • Grease or oil bearing wearing surfaces, as necessary. • Prime and paint bearing non-wearing surfaces, as necessary. • Reinstall bearing and lower bridge pursuant to jacking plan, if necessary.
Ohio Department of Transportation	Steel Sliding/	Power wash (especially at abutments) annually when washing decks. Make sure to concentrate efforts

	Steel Rocker	between rocker surfaces and base plates to remove dirt that could accumulate and impede movement.
Iowa Department of Transportation	N/A	“Although the best preventative maintenance to ensure proper function of bridge bearing components is to maintain watertight deck joint seals, maintaining joint integrity is an ongoing struggle. Because joints will inevitably fail, **periodic washing and either painting or sealing of bridge bearings will also be required**.” • Coat and lubricate steel bearing components with a polymer encapsulant such as Bridge-Mate or an approved equal.
Virginia Department of Transportation	N/A	Cleaning and lubricating bearing devices – every 4 years is the “preferred cycle”

2.3.4 Other Maintenance Systems

2.3.4.1 Crack Sealing

Although not entirely preventive, crack sealing plays an important part in cyclical maintenance of concrete bridges. Cracking in concrete decks can occur as a result of shrinkage and thermal changes. These cracks can increase vulnerability of concrete decks and other elements to water and chlorides entering the structure. Sealing out these contaminants helps mitigate the risk of further issues, like reinforcing steel corrosion. The New York State Department of Transportation, for example, specifies Joint and Crack sealants conforming to ASTM D 6690 (“Standard Specification for Joint and Crack Sealants, Hot Applied, for Concrete and Asphalt Pavements”) for use throughout the state on Portland cement concrete decks.

2.3.4.2 Underwater Protection

It is important to note that Saline and siloxane-penetrating sealers cannot be used where there is hydrostatic pressure present. As described by AASHTO, “cementitious and epoxy coatings have been applied to underwater surfaces to protect concrete against abrasion and to cover cracks and make small repairs.” Additionally, underwater cathodic protection systems have shown favorable results in the prevention and halting of corrosion in reinforcing steel.

2.3.4.3 Cathodic Protection Systems

Cathodic Protection Systems (CPS) can be installed in concrete bridge decks and underwater substructure elements. The CPS process is based on the creation of an electrical flow that reverses the ion exchange when chloride ion-containing water reaches the reinforcing steel. However, because this is an electrical process, it is very difficult to apply

CPS to elements that utilize epoxy coated reinforcing steel. It has been shown that CPS can halt reinforcing steel corrosion with proper CPS design.

2.4.4.4 Electrochemical Chloride Extraction

Electrochemical Chloride Extraction (ECE) is a similar application to CPS, but on a shorter-term scale. This process uses a sacrificial steel mesh anode surrounding concrete elements. An electrical charge is then induced to draw chloride ions toward the anode and out of the concrete. A case study conducted by the Michigan Department of Transportation estimated that the service life of a concrete bridge pier treated with ECE coupled with the application of a penetrating sealer was increased by ten years. This study saw significant improvements in the half-cell potential measurements and found that the reinforcing steel was repassified, reducing the ability of remaining chlorides to initiate corrosion.

While ECE is not necessarily a preventive maintenance activity in itself, this method can be coupled with preventive maintenance activities like sealing to significantly increase the service life of concrete bridge elements.

2.4 Conclusion

Most state agencies follow AASHTO’s recommendations for preventive maintenance. Also most DOTs stress the importance of annual cleaning of exposed bridge elements at the end of the salting season, as well as the importance of sealing concrete at intervals where economy and effectiveness would be maximized (within, but not much before, the effective lifespan of the sealers). However, cyclical maintenance is generally just that: cyclic. Little or no attention is paid to actual need. In state bridge maintenance manuals, as well as AASHTO’s maintenance manual, maintenance is assumed to be needed on a cyclic basis, but no attention is paid to effectiveness of these measures at individual sites or whether or not preventive maintenance measures should be performed more or less frequently in different environments and conditions. However, preventive maintenance actions based on the individual site condition needs to be applied based on its actual chemical condition. It needs not only the advanced inspection techniques or structure monitoring systems to detect the potential threats to the concrete bridge, but also an innovative mathematical model to anticipate the deterioration states of the structure that counts for different variables. The advanced non-destructive tests and the deterioration models for the concrete bridge systems are discussed in Chapter 3.

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Chapter 3 Concrete Deterioration Mechanisms

3.1 Introduction

The unique combination of steel and concrete has made concrete one of the most popular construction material systems in the world. However, the lack of understanding of the long-term performance of concrete and the severity of environmental impacts has caused serious problems. Existing concrete structures are experiencing higher deterioration rates and lasting for shorter service life due to different causes of deterioration of reinforced concrete structures such as corrosion, freezing and thawing action, carbonation and alkali-silica reaction. And durability issues are greatly ignored in design, construction, and maintenance during the service life of structures. These problems have reduced the service life of the structures or have forced extensive repairs, which both comes with huge economic costs. As a result, a basic understanding of deterioration mechanism of concrete subjected to chemical attacks by the surrounding environment is essential to conduct meaningful evaluation and selecting a successful maintenance strategy.

This Chapter presents the deterioration mechanism of concrete bridge components subjected to chemical attacks, surrounding environment as well as the impact of those degradations. The deterioration issues covered in this section are corrosion of reinforcing bars in concrete, freezing and thawing cycles, carbonation of concrete, and alkali-silica reaction.

3.2 Corrosion of Reinforcing Steel Bars in Concrete

In many cases, the root of a deterioration problem is caused by corrosion of steel reinforcement in concrete structures. Corrosion is a thermodynamically spontaneous and unavoidable reaction of metals that is adverse to the metallurgical process of the production of metals from raw ores. Steels, which are iron based, are especially vulnerable to corrosion. Depending on the environmental medium and steel properties, the corrosion rate of steel can change widely. The corrosion process could be delayed or slowed down by special measures but it can never be stopped in a natural environment without any protection.

For reinforced concrete bridges, because of the high alkalinity of the pore solution in the concrete, and the barrier provided by the cover concrete, the reinforcement has been believed to be safe from the outer environment. However, corrosion is often identified in those structures experiencing poor concrete quality, inadequate design or construction, and harsh environmental conditions.

Various causes are essential that govern the rate of corrosion. The most predominate ones are chloride content and carbonization. Chloride ions are introduced into the structure either during the initial mix of various components or by diffusion as a result of extensive use of deicing salts. Non-uniform concentrations of these ions produce a difference in the potentials in the steel. This potential difference along the length of the steel member is sufficient to initiate the corrosive reaction. Carbonization is a reaction involving concrete and carbon dioxide. The result of this reaction is a reduction in the alkalinity or pH of the concrete. At a reduced pH, the steel does no longer exist in a passive state. Other factors such as moisture content, oxygen content, inner cracks and temperatures may also influence the corrosion rate. The interaction among those factors needs to be extensively investigated and analyzed.

3.2.1 Corrosion Mechanism

Corrosion, in the context of steel reinforcement in concrete, is a self-generated electrochemical activity that requires the simultaneous presence of moisture and oxygen and resulting from differences in potentials that may result from non-uniformities of steel or in the chemical and physical environment of the concrete surrounding the steel.

An electrochemical cell is formed when an anode and a cathode are connected within a conductor. In the case of steel, the anodic reaction occurs as follow:

$$Fe \leftrightarrow Fe^{2+} + 2e^{-} \tag{3.1}$$

As this process occurs, a cathodic reaction is needed to consume the free electrons released form the anodic reaction, and this reaction is typically:

$$O_2 + 2H_2O + 4e^{-} \leftrightarrow 4OH^{-} \tag{3.2}$$

The concrete serves as the electrolyte for the corrosion of steel as it conducts the current by means of ionic diffusion. The process is illustrated in Figure 3-1.

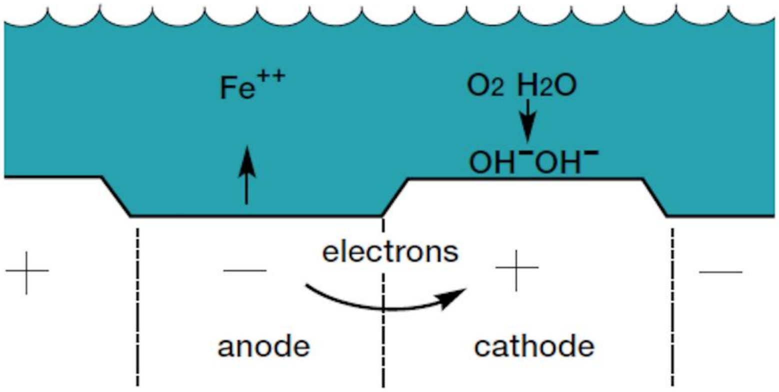

Figure 3-1 Electrochemical Corrosion Process

However, in some special case, the cathodic reaction may be in the form of hydrogen evolution. This might occur in two cases:

1. At a very negative potential or a very high cathodic current density;
2. In a carbonated concrete in which the pH value of the pore solution has become very low.

The intermediate corrosion product, Fe^{2+}, could be further transformed into $Fe^{3+}$and be accumulated at the surface of steel reinforcement. (Song, 1998)

Normally, the pore solution is rich in oxygen with a high pH value. So Fe^{2+} can stay in the form of $Fe(OH)_2$ or $Fe(OH)_3$, forming a thin passive film on the steel surface, which consequently retards the corrosion reaction. However, under some conditions the protective film may not be formed or the formed passive film would break down. This may be caused by the invasion of carbon dioxide ions, chloride ions, and other chemical elements. Once the alkalinity of the concrete drops down, the corrosion rate of the steel increases dramatically.

The corrosion types can be classified into the following types, mainly according to the corrosion mechanisms as well as partly to the damage forms:

- Uniform corrosion
- Galvanic corrosion
- Localized corrosion
- External current imposed corrosion
- Stress corrosion cracking and hydrogen induced embrittlement

3.2.2 Chloride-Induced Corrosion

Chloride ions are often cited as one of the predominate mechanisms causing corrosion today. The amount of chloride ions present in concrete greatly affects the passive film on the steel, regardless of the level of pH present in the concrete.

Chloride may be introduced into the concrete in its initial mixing state by the aggregates, water or the use of salt to thaw aggregate stockpiles. Another common source of chloride in concrete results from the use of admixtures containing large amount of chloride.

Aside from those chloride ions introduced into the concrete before construction, the chloride ion content continues to increase along with time. Diffusion, permeability and absorption are the most well-known chloride transport mechanism through concrete. Other phenomena such as chloride binding can also influence chloride ingress.

The permeability of concrete depends on its porosity as well as the size, distribution, shape, tortuosity and continuity of pores. Therefore it is affected by water cement ratio, type of cement, cement replacement materials and the progress of hydration. Chloride diffusion, on the other hand, is also controlled by factors as maturity, time, temperature, and source and concentration of salt solution. The chloride diffusion coefficient decreases with time due to several issues such as continued hydration and chloride binding. Along with other factors, the absorption is also influenced by cement content, aggregate type and gradation, compaction, and concrete grade. (Bioubakhsh, 2011)

The outcome of research and numerous investigations has led to the derivation of a threshold concentration of chloride ions. This threshold is defined as the minimum quantity of chloride required initiating corrosion of reinforcement in concrete. It has been demonstrated that levels of chloride content in the range of 0.1 to 0.4 percent of the mass of cement can be significant enough to initiate the corrosion process. For this reason, many individuals have adopted the value of 0.15 percent of the mass of cement as a limit to the chloride concentration.

After the chloride concentration hits the threshold value, corrosion may occur. As the rust build up at the surface of the reinforcing steel, the internal pressure will accumulate and final resulting in the cracking of the concrete cover, as shown in Figure 3-2.

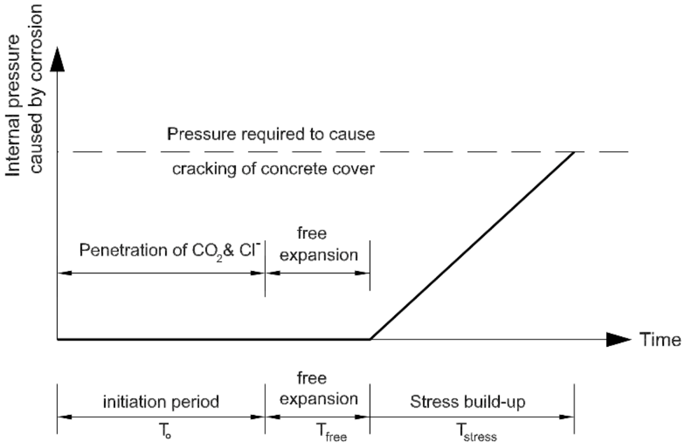

Figure 3-2 Prediction of Cracking Time Due to Chloride Ingress (Maaddawy, 2007)

3.2.3 Factors Affecting Chloride Induced Corrosion

3.2.3.1 Steel

Different types of steels have different microstructures and compositions, so different steels usually have different corrosion behaviors in concrete.

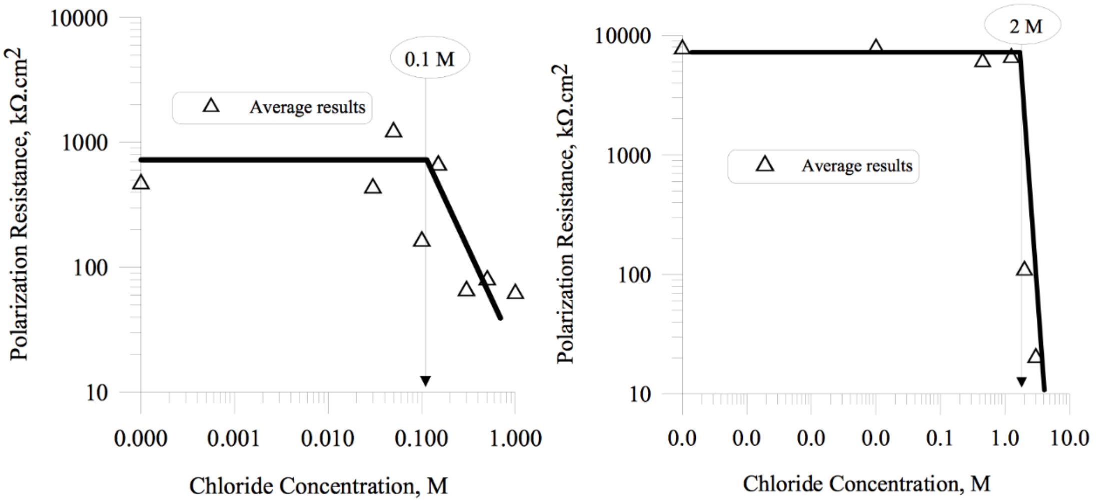

Figure 3-3 Polarization resistance of a) as-received rebar samples b) polished rebar samples (Ghods, 2004)

According to the experiment conducted by Ghods (2004), the polarization resistance is tested for as-received and polished surfaces as an indicator to the corrosion resistance. For both conditions, there is a threshold chloride concentration for the simulation pore solution above which the polarization resistance drops down dramatically, as shown in Figure 3-3. And it can be seen from the figure that the polished rebar can endure a much higher chloride concentration without the breakdown of the passive layer compared to the as-received rebars.

Furthermore, stainless steel has the much better resistant against corrosion than plain steel. It may remain passive and experience almost no corrosion even in relatively high chloride environment. MMFX2, another type of steel, also achieves a better performance compared to plain steel in corrosion environment by modify the microstructure of the steel. Galvanized steel uses a different approach to manage a longer service life by sacrificing the zinc coating as a corrosion delay mechanism. All these types of steel are more expensive than the plain steel. However, considering the savings in long term, these corrosion resistant steels should be recommended in future construction projects.

3.2.3.2 pH value of the Pore Solution

The pore solution in concrete is an electrolyte, which is physically absorbed in the pores of the concrete. It may contain various ions, such as sodium, potassium, calcium, hydroxyl, sulphate and sulfite, etc. The chemical composition of the pore solution has a great impact on the pH value of the concrete. However, the carbonation process, leaching or proceeding hydration can also affect the pH value of the pore solution at later stage. Once the pore solution is polluted by chloride ions or de-alkalized by carbon dioxide, the corrosion may occur.

The inhibiting effect of hydroxide ions against chloride induced corrosion, as a major factor influencing chloride threshold values for corrosion initiation was early recognized. Figure 3-4 shows the probability of corrosion versus the Cl^-/OH^-ratios. The suggestion to present the threshold values of Cl^-/OH^-ratios reflects this influence. (Angst, 2009)

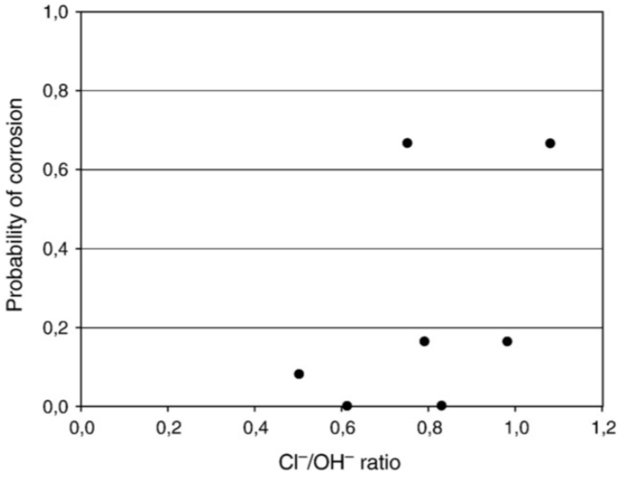

Figure 3-4 Probability of Corrosion vs. Cl^-/OH^-ratio (Angst, 2009)

3.2.3.3 Permeability

Higher porosity and larger pore sizes lead to more severe corrosion damage in the steel. If the concrete has low permeability, then the chlorides and carbon dioxide would be difficult to access the reinforcement and the possibility of corrosion would be low.

Low water cement ratio, good compaction, use of mineral admixtures and proper curing would lower the permeability of the concrete.

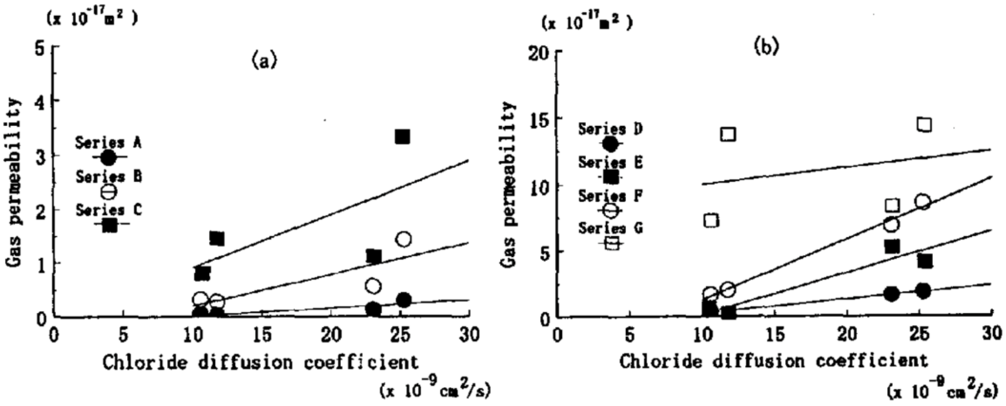

Figure 3-5 Gas Permeability vs. Chloride Diffusion Coefficients: (a) Air Dry for Gas Permeability (b) Oven Dry for Gas Permeability (Sugiyama, 1995)

Based on Sugiyama investigation, shown in Figure 3-5, the correlation between gas permeability and chloride diffusion coefficient is valid in particular when the water-cement ratio of concrete dominates the gas permeability. The correlation is based on the fact that the chloride diffusion coefficient is also a factor that significantly affected by the water-cement ratio. The correlation may become poor if the specimen experienced a longer period of drying for gas permeability due to the micro-cracks formed during the elongated drying period.

3.2.3.4 Moisture Content

If there were no water in the concrete, there would be no corrosion in the concrete. The moisture content has a complicated influence on the corrosion process since the resistivity increase as the moisture content gets higher and at the same time, the anodic and cathodic reaction rate is decreased due to the lack of oxygen.

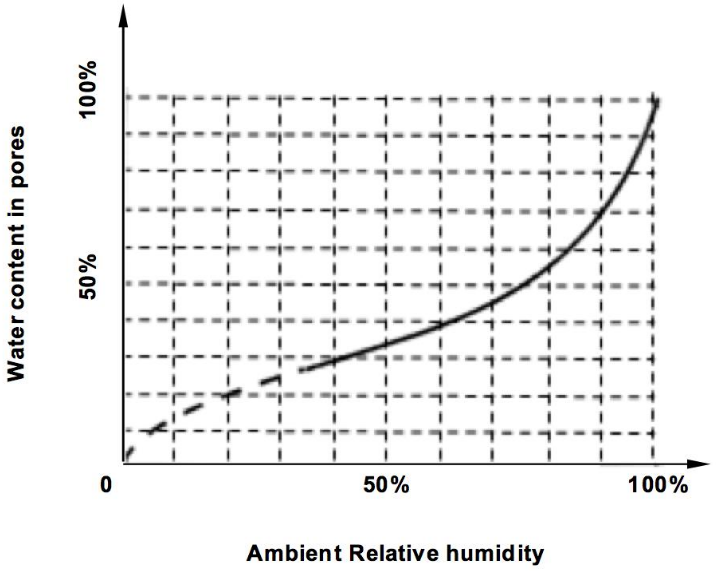

Figure 3-6 Water Content in Pores vs. Ambient Relative Humidity (CEB, 1992)

Pore waters evaporate from the concrete pores under the fluctuation of ambient relative humidity of the environment, as shown in Figure 3-6. However, research has shown that the water content in pores for specimen submerged in the water also decreases as the concrete cover depth increase. Therefore, in the real structure, the pore solution may not be available for the concrete in depth, which may cause a decrease of the chloride diffusion coefficient.

3.2.3.5 Components of Concrete

The components of concrete will significantly affect its ability to resist corrosive environment for the reinforcing steel. Low water cement ratio is a key factor for good quality concrete that can produce a more solid barrier between the steel and the corrosive particles. The aggregates, cement, admixtures and water should be chloride free. The effect of using admixtures should be studied thoroughly and the application should be carefully performed.

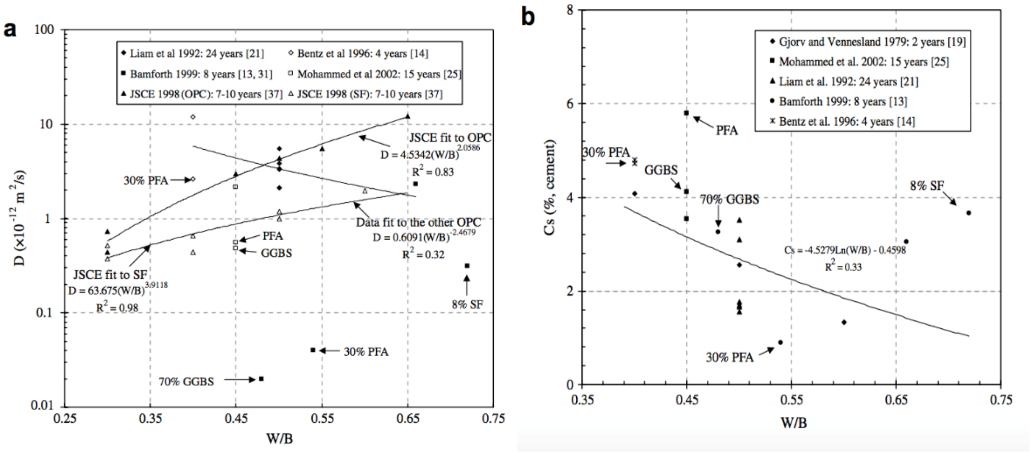

Figure 3-7 Surface Chloride Concentrations and Apparent Diffusion Coefficient for Different Concrete Mixes (Song, 2008)

From Figure 3-7, it is clearly can be seen that the diffusion coefficient has a positive relation with the water cement ratio. Also, the composition of the cement material has a great impact on the chloride transportation mechanism, mainly due to the change of microstructure of the cement paste and the binding capacity. The effect of components of concrete is a vital parameter to estimate the chloride diffusion process for its influence on not only the diffusion coefficient, but also the chloride threshold values for corrosion initiation. The incorporation of this factor will be discussed in detail in the Chapter 4.

3.2.3.6 Concrete Resistivity

The electrical resistivity of concrete is determined by the pore solution concentration, the microstructure of the concrete, the moisture and salt content as well as the temperature. It has a great impact on the ionic current flow and the corrosion activity. The concrete resistivity is also used as an indicator for the probability of corrosion. The relationship between corrosion rate and resistivity is shown in Figure3-8.

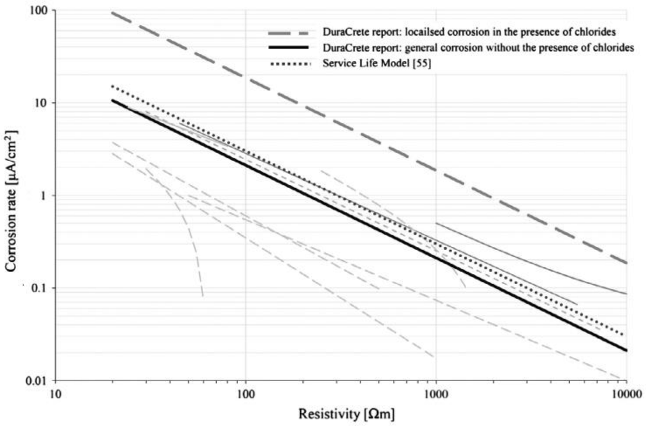

Figure 3-8 Corrosion Rate compared with Resistivity (Hornbostel, 2013)

3.2.3.7 Thickness of Cover Concrete

The thickness of concrete cover over the steel has an apparent effect on the corrosion initiation time. By increase the cover thickness, the corrosion process is delayed by elongated time that the chlorides needs to travel through. And the period of time is not a linear function due to the reactions between the chemicals and the cement past.

3.2.3.8 Temperature

Temperature can also influence the corrosion rate since all the processes involved in corrosion can be accelerated by higher temperature. Research shows that the higher temperature not only affects the chemical reaction rates, but also increases the chloride binding capacity of cements as well as the chloride concentration.

3.2.3 Effects and Impacts of Corrosion

3.2.3.1 Cracks

As corrosion occurs in a concrete member, the corrosion by-products, such as rust, begin to build up around the steel reinforcement. The rust occupies a volume up to 12 times greater than the volume of the original steel bar (Bennett, 1986). As the corrosion progresses, the pressure build up to such high level that cause crack initiation and propagation. These cracks are distributed along with the reinforcing steel and may even result in concrete cover pop out. The cracks, on the other hand, leave an easier path for the chemicals to get access to the reinforcing steel and accelerate the future corrosion rate.

3.2.3.2 Reduction of steel cross-section

Corrosion of the steel will reduce the steel cross-section as the chemical reaction consumes the iron during reaction. The iron by-product has no attribute to the strength of the structure. Therefore, the reduction of steel cross-section will in turn decrease the load carrying capacity of the structure.

3.2.4.3 Loss of Bond strength

Corrosion of steel reinforcement may also lead to debonding. When the bond between concrete and reinforcing bars is no longer present, the materials fail to act in a composite manner. The steel will tend to slip within the member, and tension forces in the concrete will not be transferred to the steel. This loss of bond will also decrease the load carrying capacity of the structure.

3.3 Carbonation of Concrete

One of the main reasons why reinforced concrete structures deteriorate and need repair is carbonation of the concrete. In many exposure conditions, the most important long-term risk of damage to reinforced concrete comes from the carbonation induced corrosion of the reinforcing steel bars. Although the effects of carbonation are often not as predominate as those involving chloride ions, carbonation of concrete is still considered an important corrosion mechanism.

The process of carbonation affects all concretes and cementitious materials exposed to carbon dioxide in the atmosphere or to carbon dioxide dissolved in solutions in contact with steel. Given certain conditions, even these small amounts of carbon dioxide can permeate through the surface of the concrete to some considerable depth. Calcium hydroxide ($Ca(OH)_2$) present in the cement reacts with the carbon dioxide to form water and insoluble carbonate ($CaOH_3$).

Carbonation is often not considered as a severe problem since the reaction rate stay low for good quality concrete with minimal cracking. However, factors such as large cracks, high permeability may greatly shorten the time for carbonation reach the surface of the reinforcement.

3.3.1 Carbonation Mechanism

From the chemical point of view, the carbonation of cementitious materials is a neutralization reaction of the basic compounds of hydrated cement (essentially $\mathrm{Ca(OH)_2}$and C– S– H) by carbonic acid. CO2, present in non-polluted air at 0.035% by volume, is dissolved in the aqueous pore solution and forms carbonic acid. (Houst, 2002)

The pore solution of hardened cementitious materials like mortar or concrete contains essentially sodium and potassium hydroxides. Indeed, the solubility of $\mathrm{Ca(OH)_2}$strongly decreases when the concentration of hydroxyl ions increases. Carbonation reactions can be written as follows:

$$\mathrm{2NaOH + \ CO_2 \rightarrow Na_2CO_3 + H_2O} \tag{3.3}$$

$$\mathrm{Na_2CO_3 + \ Ca(OH)_2 \rightarrow CaCO_3 + 2NaOH} \tag{3.4}$$

$$\mathrm{3Na_2CO_3 + 3CaO \cdot \ SiO_2 \cdot 4H_{O2} \rightarrow 3CaCO_3 + 2SiO_2 + 6NaOH + H_2O} \tag{3.5}$$

This process continues as long as $\mathrm{Ca(OH)_2}$and C – S – H are present and the pH of the pore solution keep dropping as a result.

3.3.2 Factors Influence Rate of Carbonation

Similar to the chloride diffusion, the carbonation depth is mainly modeled by the diffusion theory. By applying Fick's first law, the carbonation depth could be estimated by equation:

$$x = \sqrt{\frac{2Dc}{a}} \cdot \sqrt{t} \tag{3.6}$$

Where x is the carbonated depth at time t, D is the effective diffusivity of $\mathrm{CO_2}$, c is concentration in the atmosphere and a is the concentration of the reactive compounds.

Numerous factors influence the rate of carbonation. The following factors are among the most important ones:

- Water to cement ratio
- Curing conditions, i.e., the degree of hydration
- Cement content
- Type of cement
- $\mathrm{CO_2}$ concentration of the surrounding air

- Water content
- Temperature
- Alkali content
- Presence of damaged zones and cracks

3.3.3 Effects and Impacts of Corrosion

The carbonation process will decrease the pH value of the pore solution in the concrete, which may de-passivate the protective layer formed at the surface of the steel and initiates the corrosion. However, compared with chloride induced corrosion, carbonation induced corrosion will need a longer period to achieve its threshold. Therefore, the impact of carbonation effect will be included in the chloride-induced corrosion by introducing a factor that represents the pH value of the pore solution.

3.4 Deterioration of Concrete Due to Freeze-and-thaw effect

The deterioration of concrete structures due to freezing and thawing damage is a major problem in cold climate regions. There are two basic forms of deterioration, including internal cracking due to freezing and thawing cycles, and surface scaling due to freezing in the presence of deicer salts. Volume change from water to ice when freezing may cause deterioration either of the hardened paste or of the aggregate, or both.

Even though the freezing and thawing damage mechanisms are not well understood, the way to make concrete resistant to freezing and thawing is well recognized. In order to achieve adequate freeze-and-thaw resistance, low water cement ratio concrete and well-distributed air voids, high performance aggregates should be used. And adequate curing time and proper curing should be achieved before the first freezing cycle. Sealers could also be used to preventing water penetrating into the concrete and therefore prevents damage due to freeze-and-thaw cycle.

3.4.1 Mechanism of Freezing and Thawing Deterioration

3.4.1.1 Hydraulic Pressure Theory

Water expands 9% by volume during freezing. When the pores are critically saturated, extra water is expelled from the freezing site into the matrix. The drainage of viscous water along pore walls will induce hydraulic pressure, the magnitude of which depends on the rate of freezing, degree of saturation, pore structure, and the length of flow path to the nearest void for the water to escape (ACI Committee 201, 1992). When the pressure exceeds the tensile strength of concrete, cracking would occur. Freezing rate also influence the damage mechanism while hydraulic pressure is the main cause of freezing deterioration at a fast freezing rate.

3.4.1.2 Osmotic Pressure theory

During freezing, most of water was found to diffuse toward, not away from, the sites of freezing, and the expansions decreased with the increase of cooling rate. This is contrary to the hydraulic pressure theory. Powers attributed this phenomenon to osmotic pressure (Powers, 1975). The use of deicing salts would increase the salt concentration in the pore solution, resulting in larger osmotic pressure. This may be one of the reasons why the concrete with high chloride content shows low freezing and thawing resistance. The mechanisms of scaling during freezing in the presence of deicing salts can be partially explained by osmotic theory.

3.4.1.3 Thermodynamics and Surface Force Considerations

Water in the largest pores freezes first, and water in small pores cannot freeze or would freeze at very low temperature due to the high surface forces in small pores. These surface forces result in high chemical potential of water, and thus hinder ice crystal formation. The unfrozen water tends to reduce its potential and migrates to locations where it is able to freeze, such as the larger pores. As a result, drying shrinkage occurs in the gel structure while ice accumulates in large pores or cracks, leading to the formation of new cracks or extension of old ones. (Yang, 2004)

3.4.2 Effects of Deicing Salts on Freezing and Thawing Cycles Deterioration

Freezing and thawing cannot damage concrete if it does not become saturated with water. Deicing salts replenish the water available to enter concrete system and saturate it with melting snow. Constant replenishment and subsequent freezing will cause freezing damage within the concrete. The existence of deicing salt also contributes to the last two deterioration theories. The combined effects of temperature gradient and salt concentration gradient along the depth will probably keep the intermediate layer between the surface and the deep in the liquid state, and it would freeze upon intensive cooling. The dilative pressure resulted from this delayed freezing may cause the scaling of surface layer because it cannot be relieved through the frozen layers (Yamato, 1987).

Besides, Salt is a hygroscopic material and exposed on a surface, it will contribute in keeping the surface wet. Applied to an ice or snow covered surface, it will provide additional water, causing increased degree of saturation.

3.4.3 Deterioration of Concrete Exposed to Freezing and Thawing

There are two types of Freeze-thaw damages: surface scaling or internal cracking. The former may occur on both horizontal and vertical surfaces, but mainly where water or snow can naturally deposit and the surface remain wet for periods. The susceptibility to surface scaling will increase significantly in the presence of de-icing chemicals. Internal cracking under field conditions is less commonly observed or recognized. Sampling for structural analyses by plane sections or thin sections will provide valuable information for the identification and elimination of other causes. The phenomenon may be observed on parts of structures in direct contact with free water and subjected to capillary suction, such as the lower parts of supporting walls and dam structures above the water surface. It is also believed that freeze-thaw cracking may combine with or start after deterioration initiated by other detrimental mechanisms, such as alkali aggregate reactions. (Ronning, 2001)

3.5 Deterioration of Concrete Due to Alkali-silica Reaction (ASR)

The ASR in concrete is a chemical reaction between reactive forms of silica present in the aggregates and the high alkaline pore solution. Two main mechanisms constitute the ASR.

Firstly, silica is dissolved from the aggregates, whereby a gel is formed and secondly, the swelling of the gel by imbibition of water, which results in the expansion and deterioration of the affected concrete. (Bangert, 2004)

The term alkali-silica reaction is somewhat misleading- the reaction itself is actually between the OH^- ions in the pore solution and certain siliceous components of the aggregates. The alkalis, specifically sodium and potassium, do not actually attack the reactive silica. The importance of the alkalis is that their presence in high concentrations in the pore solution results in an equally high concentration of OH^- ions (to maintain charge equilibrium). It is this high OH^- concentration, and thus high pH, that leads to the initial breakdown of reactive silica components in the aggregates. (FHWA, 2003)

3.5.1 Mechanism of ASR

The reaction begins with an attack of the alkaline hydroxides present in the cement on the reactive silica particles in the aggregate. When poorly crystalline hydrous silica is exposed to a highly alkaline solution, there is an acid-base reaction between the OH^-ions in solution and the$\mathrm{Si-OH}$ groups:

$$\equiv \mathrm{Si-OH} + OH^- \rightarrow \equiv \mathrm{Si-O}^- + H_2O \qquad (3.7)$$

As additional OH^- ions penetrate into the structure, some of the siloxane (Si-O-Si) linkages are also attacked, following the equations shown below (Dent-Glasser, 1981):

$$\equiv \mathrm{Si-OH-Si} \equiv +2OH^- \rightarrow \equiv 2(\mathrm{Si-O}^-) + H_2O \qquad (3.8)$$

To maintain charge equilibrium, positive ions (Na^+ and K^+) diffuse into the structure to balance the negative charges present on the terminal oxygen atoms. The disruption of the siloxane linkages ultimately weakens the structure. Provided that sufficient amounts of alkali-hydroxides are available, this process continues, producing an alkaline-silicate solution.

Due to the absorption of water, the gel may have a volume significantly larger than the silica particles originally attacked or consumed. In saturated concrete, the amount of water available in the reaction is abundant. Therefore, the rate of gel growth is dependent on the rate of the alkali-silica reaction. However, in the case of an unsaturated member, the rate of expansion or growth of the gel does not occur simultaneously with the reaction. It is prolonged over a longer period of time since the expansion rate is dependent on the ability of water vapor to diffuse through the surface of the concrete (Hobbs, 1988).

3.5.2 Factors Affecting ASR

For alkali-silica reaction to occur, three conditions must be present:

- Reactive forms of silica in the aggregate
- High-alkali (pH) pore solution
- Sufficient moisture

The amount of gel formed in the concrete depends on the amount and type of reactive silica, and the alkali hydroxide concentration in the concrete pore solution. Natural aggregates contain various forms of silica minerals. Internal sources of alkali (sodium and potassium) can come from the cement, aggregates, admixtures, and mix water. When the alkali and silica react, they form the gel reaction product. External alkalis can come from a number of sources, but the predominant source is anti-icing or deicing chemicals. Exact composition will vary, but the gel always contains alkali, calcium, silica, and water (Xu, 1987).

3.5.2.1 Reactive silica in the aggregate

Reactivity is a function of the type and form of constituents composing the aggregate. An aggregate that presents a large surface area for reaction—poorly crystalline, many lattice defects, amorphous, glassy, and micro-porous—is susceptible to reaction (Poole, 1992). Fine and coarse aggregate containing more than the following quantities of constituents are considered potentially reactive (Farny, 2007):

- Opal—more than 0.5% by mass
- Chert or chalcedony—more than 3.0%
- Tridymite or cristobalite—more than1.0%
- Optically strained or microcrystalline quartz—more than 5.0% (as found in granites, granite gneiss, greywacke, argillites, phyllites, silt- stones, and some natural sands and gravels)
- Natural volcanic glasses—more than 3.0%

3.5.2.2 High-alkali-content pore solution

As the pH, or alkalinity, of the pore solution increases, potential for the alkali-silica reaction increases. At higher concentrations of alkali hydroxides, even the more stable forms of silica are susceptible to attack (Xu, 1987). If the alkali concentration is great enough, the alkali hydroxides break stronger silicon bonds found in less reactive aggregates to form the gel reaction product.

3.5.2.3 Sufficient moisture

Moisture allows migration of alkali ions to reaction sites, and the resulting gel absorbs moisture, leading to expansion. For this reason, deleterious ASR does not occur in concretes that are dry in service. Research has shown that expansive ASR can occur in concrete having a relative humidity above 80% (Stark, 1991). Any reduction in permeability, by using a low water-cement ratio, supplementary cementitious materials (SCMs), or other means, reduces movement of moisture and alkalis into and within the concrete.

3.5.2.4 Wetting and drying

Dry exposures reduce potential for expansive cracking due to alkali-silica reactivity. It’s unknown whether continuous saturation or cycles of wetting and drying cause more damage (Palmer, 1992). But it is known that alkali migration can occur with alternate wetting and drying, concentrating alkalis near the drying zone and inducing reaction there (Perenchio, 1991). Therefore, minimizing the wet-dry cycle as well as controlling the available moisture by providing sufficient drainage is desirable in preventing ASR.

3.5.2.5 Temperature

Structures in warmer regions are more susceptible to ASR than those in colder regions, because the ASR rate always having a positive correlation with temperature. For the majority of aggregates, higher temperatures also mean larger ultimate expansions. However, there are certain aggregates may have a larger ultimate expansion in lower temperatures.

3.5.3 Effects of ASR reaction

The results of the alkali-silica reaction on a concrete member are independent of the type or source of both the alkali and silica particles. An alkali-silica reaction occurring in a concrete member may result in the following outcomes:

3.5.3.1 Expansion

Typical indicators of ASR presence are longitudinal cracks, map cracking, spalled concrete surfaces, or relative displacements of different portions of a structure. There are two theories proposed to describe the mechanism of expansion caused by ASR, the absorption theory and osmatic theory.

The main concerns about expansion due to ASR is that it may exacerbate other deterioration mechanism especially for those structures exposed to deicing salt, freeze-thaw attacks and sulfate exposures.

3.5.3.2 Cracking

A network of cracks is a strong indication that expansive ASR is the dominating role of the concrete deterioration. The cracking patterns are influenced by the geometry of the member, the presence of the reinforcing steels and the applied stresses on the member.

3.5.3.3 Popout

In addition to cracking and expansion, pop-outs may result from an alkali-silica reaction. A popout is generally a small cone-shaped cavity in a horizontal concrete surface left after a near surface aggregate particle has expanded and fractured. (NRMCA, 2007)

3.5.3.4 Color Change

Darkened or blotchy areas are often associated with ASR. Areas along cracks may be bleached, pinkish, or brownish in color in a radius of an inch from the crack.

3.5.3.5 Reduction in concrete Strength

In the numerous tests conducted on concrete members subject to expansion, the compression strength decreased as the expansion of the member increased. The same reduction in strength with increasing expansion was experienced when measuring the tensile capacity of the concrete member. Therefore, it is concluded that the reduction in strength, both compressive and tensile, is dependent on the level of expansion in the concrete member.

3.6 Combined Effect of Different Deterioration Mechanisms

As discussed above, the deterioration mechanism of concrete bridge components subjected to chemical attacks may include but not limited to: corrosion of reinforcing bars in concrete, freezing and thawing cycles, carbonation of concrete, and alkali-silica reaction.

In many cases, the root of a deterioration problem is caused by the corrosion of the concrete structures. The corrosion process is an electrochemical mechanism in which metal is reduced to a lesser state of energy, that being its natural ore. Various causes exist; the most predominate being chlorides, carbonization, and oxygen. Aside from the previously mentioned elements, chloride ions, oxygen, and carbon dioxide, there are other chemicals known to cause a reduction in the pH of concrete.

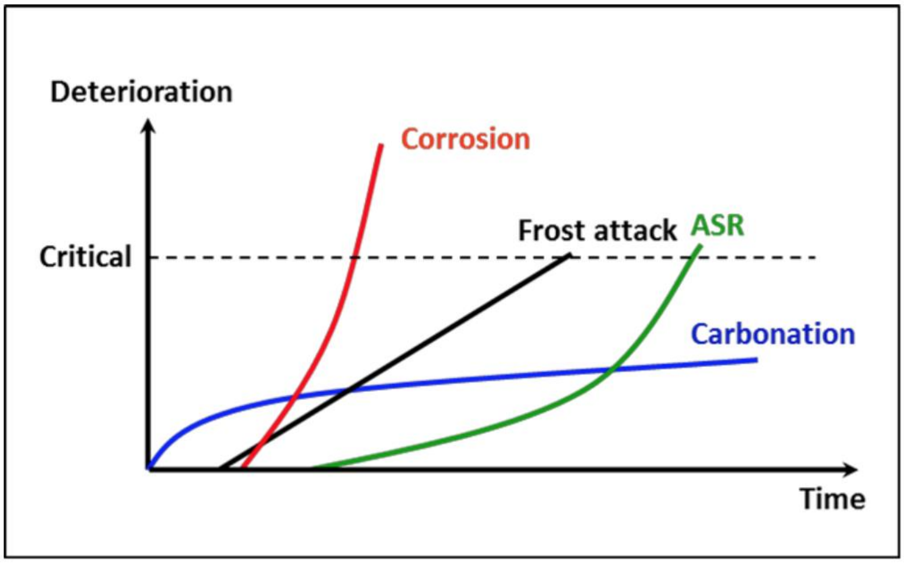

Figure 3-9 Simplifies deterioration mechanism, (Aboutaha, 2004)

Figure 3-9 shows simplified deterioration mechanism in severe environments. The combined effects of these mechanisms are more damaging, which may start at time zero and increasing with a greater slope. The carbonation process will affect the pH value of the pore solution in the concrete, which may leads to the depassivation of the steel. However, on the other hand, it may also change the binding capacity of the concrete as well as slow the chloride ingress velocity. So the overall impact may be controversial and depends highly on the engineering judgment and the experimental work. It is also true for the impact on corrosion caused by ASR and freeze-and-thaw effect. The fine cracks caused by these deterioration mechanism may dramatically affects the chloride diffusion coefficient, and the cracks near the surface is also involving other transportation mechanisms which will leads to a higher chloride concentration. The combined effects of different deterioration mechanism shall be discussed in the following chapter.

3.7 Summary

This chapter presented the deterioration mechanism of concrete structures with specific reference to concrete bridge components. The deterioration mechanisms of corrosion, freezing and thawing effect, carbonation of concrete and alkali-silica reactions were presented. The corrosion of reinforcing steel bars is the primary limit state in cold region due to the intentional use of deicing salt. Other deterioration factors play a secondary role that may accelerate the rate of corrosion of the reinforcing bars.

The following Chapter 4 presents the various NDTs, and Chapter 5 presents the deterioration models for different deterioration mechanisms based on various factors, e.g. chemical content concentration, concrete permeability, distribution of cracks, ages of the concrete, etc. The coupling effects of different mechanisms are also discussed.

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Chapter 4 Introduction of Chemical NDTs

Deterioration of concrete bridge components starts with chemical reaction between harmful compounds and the construction materials that make up the bridge element. Development of physical deteriorations are always superseded by chemical reactions. In order to prevent the physical deterioration of a bridge member, the chemical condition of the bridge should be assessed, and preventive measures should be taken. This chapter presents the various chemical non-destructive tests that could be used for chemical assessment.

4.1 Chemical Non-Destructive Testing

Current practice for bridge inspections relies heavily on visual inspection: the inspector makes note of visual deterioration and perhaps utilizes other relatively simple methods like sounding to check for delamination. Other, more advanced methods like ground-penetrating radar and infrared thermography can also be utilized for more in-depth inspections. These methods, however, only make note deterioration that results from corrosion due to chloride intrusion and carbonation, not the presence or progression of the mechanisms themselves.

The chemical condition of concrete is very closely associated with the potential for corrosion of reinforcing steel. The pH value of concrete is especially important because the high pH of concrete (around 12) results in the formation of a microscopic protective oxide layer around uncoated rebar. However, if the pH falls to around 9 to 10, this layer can be compromised and corrosion can begin to occur. The decreasing alkalinity of concrete (decreasing pH value) arises mainly from the penetration of CO_2. However, if the chloride content of the concrete reaches a certain threshold, corrosion will occur regardless of the state of the microscopic oxide layer.

Another concern for the durability of concrete structures is Alkali-Silica Reactivity (ASR), which can cause surface cracking and allow for the ingress of water, chloride ions, and other contaminants into the structure.

Tests for pH level, chloride content, and ASR all exist, in addition to tests the electrical state of the structure (half-cell potential, for example).

4.2 Current Practice

In AASHTO’s Maintenance Manual for Roadways and Bridges, several tests are described to provide further information than available from a strictly visual inspection. However, these tests are described as being used “to properly plan bridge repair,” rather than as a part of routine (annual or biennial) inspection. In the following sections, chemical-based tests are italicized.

4.2.1 Corrosion-related

Chemical NDTs used for corrosion-related testing include the following:

- **Chloride content measuring-** is performed using powdered sampled obtained from various depths with a concrete drill. By measuring the chloride content at these depths, the level of ingress can be determined. AASHTO gives the threshold value at 1 lb./ft^3.

Additionally, non-chemical NDTs can be used to measure corrosion potential and levels and include:

- **Delamination Survey-** for concrete decks. Delamination is "internal cracks caused by corrosion of reinforcing steel."
- **Corrosion potential survey-** measures the electrical potential of the reinforcing steel.
- **Corrosion Contour maps**- provide the location of delamination, chloride contamination, and active corrosion by typing corrosion tests to a 4 foot (or smaller) rectangular grid established on concrete decks.
- **Corrosion rate measurement**
- **Permeability tests-** shows the resistance of concrete against the intrusion chlorides.

4.2.2 Non-corrosion related

In addition to measuring parameters that are directly related to corrosion, other factors that lead to concrete deterioration can be measured using NDTs. These tests include the following:

- **Coring**- can be used to measure general state of concrete, depth of cover, and to measure factors like carbonation and chloride levels
- **ASR testing**
- **Carbonation testing**
- **Ultrasonic Pulse Velocity**
- **X-ray inspection**
- **Computer-assisted tomography**

4.3 Types of Chemical NDTs

Chemical NDTs require samples of concrete from various depths to analyze the chemical makeup of the element. Of particular importance to the state and service life of concrete bridges are: the state of Alkali-Silica Reactions, carbonation, and chloride content. All three currently have chemical test kits available for purchase in the construction market.

ASTM/AASHTO standards for chemical NDTs are listed in the Table 4-1. Additionally, SHRP 2 listed many of these methods, particularly those for bridge deck inspection, as being important for bridge evaluation.

Table 4-1 Standards and Descriptions for Current NDTs

	Non-Destructive Testing Method	Description	Standard

Physical	Delamination Survey (Chain Drag/Sounding)	Areas of delamination by a trained inspector dragging a chain/sounding on concrete decks	ASTM D4580 (Includes other methods, as well)
	Coring/Drilling	Used to obtain specimens for laboratory testing. Also useful to determine integrity and depth of concrete and cover.	ASTM C42 (coring)
	Petrographic Analysis	Visual analysis of the physical state of concrete samples; can be used to measure concerns like ASR	ASTM C856
Electrical/Radiological	Ultrasonic Pulse Velocity	Can be used to detect internal changes in concrete like deterioration, cracking, and changes due to freezing/thawing. (Malhotra)	ASTM C597
	X-ray inspection	Provides information about physical characteristics like composition, density, and structural integrity (Malhotra)	ASTM E1742
	Computer-assisted tomography	X-ray based, can be especially useful to determine internal crack patterns and bond state between concrete and rebar.	
	Corrosion Potential Survey	Electrical measurements are between the deck surface and (uncoated) reinforcing steel. The half-cell corrosion electrical potential is measured.	ASTM C876
Chemical	Chloride Content Measurement	Chloride content of powdered samples of concrete at several levels between the concrete surfaces and level of reinforcing steel.	ASTM C1152, AASHTO T260, AASHTO T332

	ASR Testing	Chemical test developed by Los Alamos Nat'l Laboratories that utilizes two chemicals (sodium cobaltinitrite and rhodamine B) to detect the presence of ASR gel in concrete samples. Other methods like petrographic analysis can be used (ASTM C856).	
	Carbonation Testing	Uses chemicals like phenolphthalein to determine pH of concrete samples, which	
	Permeability Test	Standardized test used to measure the resistance of a concrete sample to the ingress of chlorides.	AASHTO T277

4.4 Detailed introduction for Chemical Tests

4.4.1 Carbonation Depth

Carbonation is one of the major factors to cause structure deterioration. The steel reinforcement is physically and chemically protected by the surrounding concrete in reinforce concrete, since the pore solution constitutes a vary alkaline environment (pH close to 13.5). Due to the impact of carbonation, the pH value of the pore solution will decrease and it may cause depassivation of the reinforcement under chloride attacks. The carbonation process, which causes the variation of the pH value of the pore solution in the concrete, has a vital impact on the chloride threshold value for corrosion initiation.

The carbonation depth can be used for the following purposes:

- To evaluate the cause of corrosion when conducting corrosion survey
- To Estimate service life where penetration of the carbonation frontier is critical
- To monitor the effectiveness of applications for re-alkalization

4.4.1.1 Phenolphthalein Indicator Solution

A well-established and convenient method to monitor the carbonation depth is using phenolphthalein indicator solution. The solution is a colorless acid/base indicator, which turns purple when the pH is above 9, denoting the presence of $Ca(OH)_2$. It indicates the boundary at which the carbonated front meets the uncarbonated concrete, where concrete is alkaline, as shown in Figure 4-1. (Parrott, 1987)

Figure 4-1 Phenolphthalein Indicator Solution on Concrete Core (Parrott, 1987)

To conduct the test, a solution of 1% Phenolphthalein in 70% ethyl alcohol is used to determine the carbonation depth. In the field, the 1% phenolphthalein solution is made by dissolving 1gm of phenolphthalein in 90 cc of ethanol and then made up to 100 cc by adding distilled water. On freshly extracted cores the core is sprayed with phenolphthalein solution, the depth of the uncolored layer (the carbonated layer) from the external surface is measured to the nearest mm at 4 or 8 positions, and the average taken. If the test is to be done in a drilled hole, the dust is first removed from the hole using an airbrush and again the depth of the uncolored layer measured at 4 or 8 positions and the average taken. If the concrete still retains its alkaline characteristic the color of the concrete will change to purple. If carbonation has taken place the pH will have changed close to 7 (i.e. neutral condition) and there will be no color change.

A lot of field tests methods and instruments are based on phenolphthalein indicator solution, such as Carbo Detect SystemTM provided by James Instruments Company, CARBONTEST® provided by Tecnoindagini Srl Company, and Deep Purple and Rainbow Indicator provided by Germann Instruments Company, and NDIS 3419 proposed by the Japanese Society for Non-Destructive Inspection.

4.4.2 Chloride Content

The measurement of chloride content at the concrete cover could be used to estimate the risk of corrosion initiation, and therefore, to optimize repair and maintenance costs. There are many lab techniques and field measurements for measuring chloride content in reinforced concrete structures.

For the semi-destructive methods, the most popular techniques are potentiometric and Volhard methods. They measure free and total chlorides in concrete cores extracted from in service structures.

For the non-destructive methods, the most studied and developed general methods could be classified in (i) ion selective electrodes (ISE), (ii) electrical resistivity (ER), and (iii) optical fiber sensor (OFS). Other methods such as grounding penetrating radars (GPR), capacitive methods, or electrochemical impedance spectroscopy (EIS) have been used for corrosion assessment, for detection of chlorides, or for chloride diffusion coefficient estimation in laboratory concrete specimens (Luque et al., 2014).

The theories behind the methods are usually very simple and the effects of a number of parameters are not fully understood. Most of the test methods are used to make comparisons. Different concrete compositions or binders are compared but the relevance of such a comparison must of course be judged in the light of the test results. A “better” concrete in a test may not necessarily have a higher chloride resistance in practice. (Nilsson, 1996)

4.4.2.1 NordTest Method (Volhard Method)

Volhard titration is a reference method for determination of the total acid-soluble chloride content in concrete. The common test standard is called NT BUILD 208.

If the intention is to determine the average content of chlorides in the concrete, one or several test specimens should be selected so that the estimated amount of concrete exceeds about 1 kg. If the intention is to determine the content of chlorides in a certain place in the concrete, e.g. the content of chlorides in a concrete cover layer, then a reciprocating drilling machine should be used and so much of the drill cuttings collected from the actual place in the concrete that the amount of cement in it may be estimated to exceed about 2 g.

The drill cuttings do not need to be prepared for a test analysis. The selected test specimens should be crushed with a hammer or similar tool to a size that no material is lost. The crushed material is then ground in the mill or grinder until a particle size less than 0.1mm is obtained.

This technique is mostly semi-destructive, time-consuming and costly. Furthermore, their destructive nature could lead to additional indirect costs such as traffic delay, traffic management, road closures and lost productivity, which increase costs further.

4.4.2.2 Potentiometric Method (AASHTO, T260)

Potentiometric method is an electrochemical method in which the potential of an electrochemical cell is measured while little to no current is passed through the sample. I titrimetric methods, this measurement can be used to indicate the end point. When an ion selective electrode is used, the measured potential is related to the ion concentration in solution and a quantitative determination can directly be made.

The titration method can be refined by replacing the silver indicator electrode by a chloride selective electrode. In this case the measured difference in potential is a function of the chloride concentration in the solution. The titration will be less sensitive to other ions present in the solution.

This method has very good precision if the operator is skilled and calibration is performed carefully.

4.4.2.3 Ion Selective Electrode

The use of chloride selective electrodes to measure the chloride content of hardened concrete is a fast method. It is a fast test method. However, since it is very difficult to produce a good calibration curve for various concretes and chloride contents, the test hardly can reach an accurate result compared to Volhard titration.

A commercially available field test method using this technique is called Rapid Chloride Test. Figure 4-2 shows correlating values from chloride analysis using Volhard titration and rapid chloride test. The performance of the rapid chloride test performs well in this case.

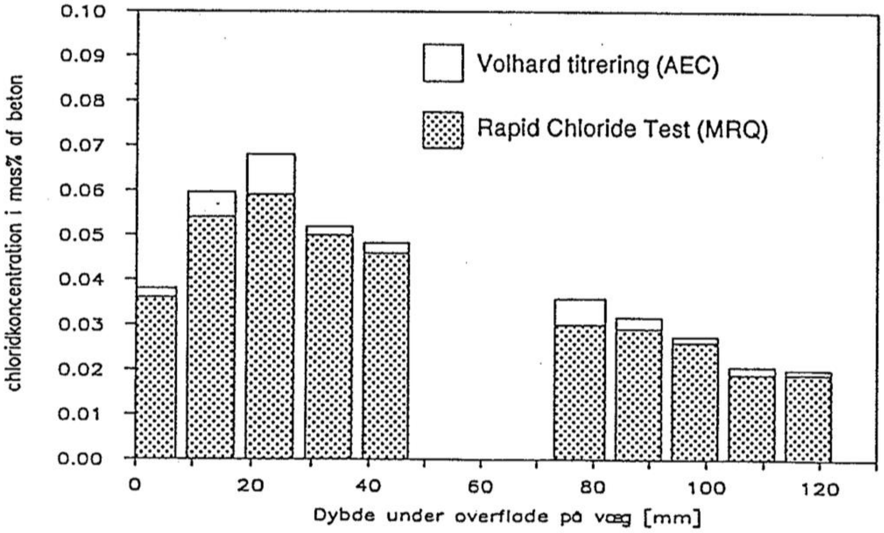

Figure 4-2 Comparison of Volhard Titration Test and RCT (Nilsson, 1996)

4.4.2.4 Quantab Chloride Titrators

Test using Quantab chloride titrators is a field test method that gives fast but not very accurate test results. The test materials are easy to use, disposable and inexpensive. It is a great way to obtain quick, quantitative answers in the field. The Quantab chloride titrators are often made in a form of Titration strip. When the strip is placed in and solution, fluid rises in the strip by capillary action. The reaction of silver dichromate with chloride will cause the color change in the capillary column. When the capillary column is completely saturated, the chloride concentration will be determined as silver chloride.

However, other than chlorides, other chemicals as bromides, iodides, sulphides, strong acids and strong bases can also react with the strips and may introduce error into the test results.

4.4.2.5 Colorimetric Spray Indicators

A colorimetric method is performed by spraying first a solution of 1g fluorescein in a 70% ethyl alcohol solution and then a 0.1 M aqueous silver nitrate solution on a fractured surface of the concrete sample. The surface changes into dark colored if free chlorides are present. It is an analytical method to show weather enough free chloride ions are present without giving a quantitative value.

4.4.3 Chloride Resistance Test

The chloride resistance of concrete is a factor that governed by the concrete properties and the minimum cover thickness. The concrete durability is usually determined by the maximum water-cement ratio traditionally. However, the use of chemical admixtures and supplementary cementitious materials has provided a great flexibility for the combination of concrete properties. Therefore, some advanced more detailed test should be performed in order to detect the chloride resistance of the concrete.

4.4.3.1 Rapid Chloride Permeability Test

The rapid chloride permeability test (RCPT) was developed in 1981 by David Whiting for the Federal Highway Administration, FHWA (Gergely, 2006). It was developed to provide an alternative to the salt ponding test, which is a long-term test.

The basic principle behind the RCPT is the applied voltage technique. This technique is based on the principle that a charged ion, such as chloride ions, will migrate in an electric field in the direction of the pole of the opposite charge.

Although the test shows good correlation for Portland cement concrete, it does not provide high correlations with concretes with admixtures or supplementary cementitious materials. The presence of silica fume and fly ash will change the pore fluid conductivity and the micro-structural characteristics of the concrete, and it will cause an increase in the amount of charge passed during the RCPT test, which may leads to an overestimated value for the chloride permeability.

Mineral admixtures are not the only concrete additives that disrupt the RCPT. Nitrite-based corrosion inhibitors also cause unduly high RCPT values. These inhibitors are used to fend off corrosion of rebar through chemical attack.

4.4.3.2 Salt Ponding Test

The salt ponding test is the most widely accepted test method for determining the chloride permeability of concrete. There are two versions of this test: AASHTO T259 and ASTM C1543 (AASHTO & ASTM, 2002). The AASHTO test consists of 42 days of preparation and 90 days of ponding. The ASTM method lasts for a subjective length of time determined by the concrete type. Both tests require a 3% salt solution to be ponded on concrete slabs measuring 12” square by 3” thick. This solution is changed every two months for the ASTM method, while it is not changed for AASHTO. The chloride concentration is determined for 0.5-inch slices of the slab. (Gergely, 2006)

There are some critics for the salt ponding test. The first one is that due to the nature of the test, the method gives an average chloride concentration over a 0.5” section instead of a real chloride profile. This difference will introduce error during the curve fitting process.

Another issue is that the salt ponding test allows chloride ingression by other chloride transpiration mechanism. This test allows for other transport mechanisms including sorption and wicking. The concrete should be dried for 28 days before the solution is added. When the solution is added, there will be suction of the chloride solution due to the wetting effect.

4.4.3.3 Rapid Migration Test

Tang and Nilsson proposed a variation on the conventional migration cell unique enough to be mentioned separately. A migration cell is set up with a specimen 50 mm thick and 100 mm in diameter, and an applied voltage of 30 V, as shown in Figure 4-3. (Stanish, 1997)

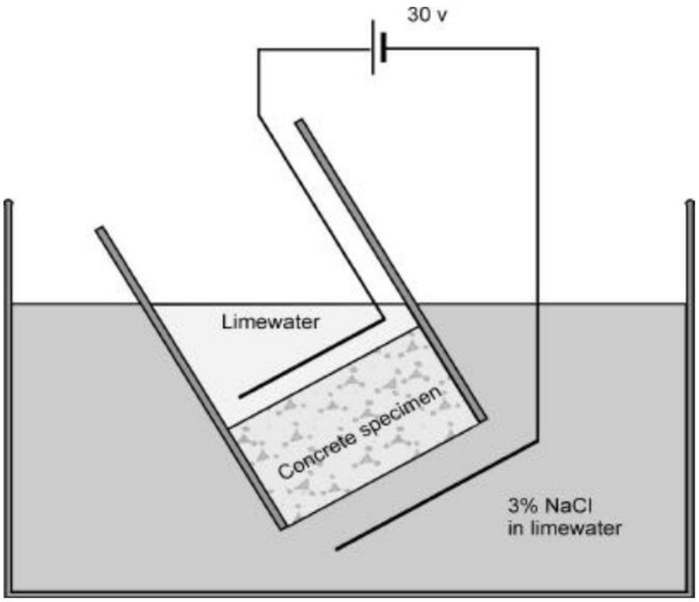

Figure 4-3 Tang and Nilsson migration cell (Stanish, 1997)

This test method is similar to the RCPT in that a 2-inch thick cylinder with a 4-inch diameter is subjected to an applied voltage for a period of time. The difference in this test is the length of time, typically 24 hours, and the voltage used, ranging from 10-60 VDC. This test method has been suggested to be a better option than the RCPT test for a wider variety of concrete mixes.

One of the major benefits of this test is that it allows for the calculation of a non-steady state diffusion coefficient. This diffusion coefficient is a function of the applied voltage, temperature of the solution, thickness of the specimen and the depth of chloride penetration. Also, the use of corrosion inhibiting admixtures did not affect the RMT results like it does with the RCPT results. This suggests that the RMT is capable of testing a wider range of concretes than the RCPT.

4.4.3.4 Bulk Diffusion Test

A bulk diffusion test has been developed to overcome some of the deficiencies of the salt ponding test to measure diffusion. The NordTest is the first formally standardized version of the bulk diffusion test. Compared with salt ponding test, the test specimen is saturated with limewater instead of dried for 28 days in order to eliminate the wetting effect. The only face left uncovered is the one exposed to a 2.8 M NaCl solution. And the specimen is left this way for a minimum of 35 days before evaluation. The typical set up for the test is shown in figure 4-4.

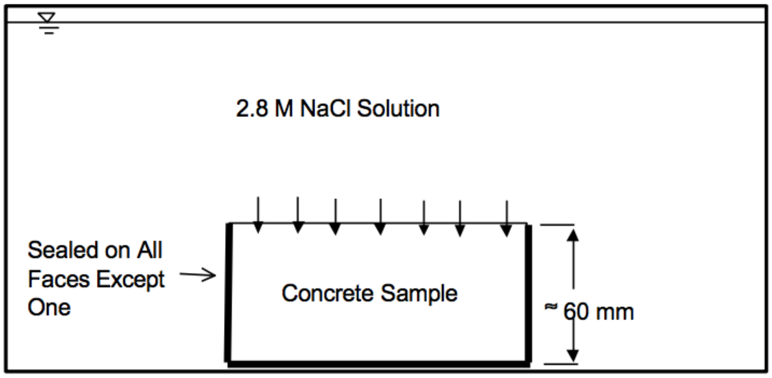

Figure 4-4 Bulk Diffusion Test (Stanish, 1997)

To evaluate the sample, the chloride profile of the concrete is determined by mounting the sample in either a mill or lathe with a diamond tipped bit. The chloride content of the powder is then determined according to AASHTO T260. The error function solution of Fick's Second Law is then fit to the curve and a diffusion value and surface chloride concentration is determined.

This test is still a long-term test. For low quality concretes, the minimum exposure period is 35 days. This period must be extended to 90 days or longer for higher quality concrete, which has duration similar to salt ponding test. (Gergely, 2006)

4.5 Survey Response

In order to get a better understanding of the current inspection and maintenance practice adopted by the State DOTs, a survey is conducted and passed to all the States DOTs in 2014. The survey questionnaire is attached in Appendix D. A total of 9 responses were received from the State DOTs. The survey is separated into three sections, general section regarding to the general states of the states bridges, inspection sections that related to the current inspection practice and the Maintenance section that involves maintenance strategies, application and costs.

8 out of 9 DOTs reported the current structurally deficiency and functional obsolete percentage of their concrete bridge inventory, as shown in Table 4-2. The majority of them have a lower percentage of functional obsolete and structurally deficient bridges compared with the national average. And that might become evidence that this survey is biased since only the DOTs that have confidence in their current bridge management system replied our survey.

Table 4-2 Survey Questionnaire: Question 1

Q1: What is the percentage of the concrete bridges that are structurally deficient (SD) and functional obsolete (FO) during last year? And what is the target percentage of the concrete bridges that are structurally deficient (SD) and functional obsolete (FO) that your agency predict in 2020 year?

	In the past year				In 2020			
	≤10%	10%~20%	20%~30%	≥30%	≤10%	10%~20%	20%~30 %	≥30%
SD	7	1			7			
FO	6	2			5	2		

For the percentage of work that assigned to bridge inspections, maintenance and minor repair actives, or major rehabilitation and reconstruction, except one of the DOT that states they assigned 40% of the work for inspection, the remaining States shows that they allocate most of their recourse for major rehabilitation and reconstruction and little of the work or the funding are assigned to tasks with bridge inspection and routine, preventive maintenance or minor rehabilitation.

Considering the nondestructive tests performed in different states, one of the states use only in house inspection specialist and two of them hire external staff only. The rest of the states use a mixed team of in house and external staff to conduct the NDT within their region.

However, regarding NDT data interpretation and analysis, the responses varied widely. One response indicated that inspection specialist was responsible for this task, while another State assigned the work to structural engineers. The rest of the states used a team consisted of either inspection specialist and structure engineer, or inspection specialist and material engineers to do the job.

Regarding the question of who is responsible for integrating the NDT data into the bridge management system, the answer also shows diversity. Inspection specialist and structure engineers are often required to do this task. Among all responses, two states indicated that this task is not done in their region. It means that NDT data is not used efficiently and well. The NDT data will have a great value if it is incorporated into the bridge management systems, since the parameters that used to determine the state of the bridge elements are not limited to physical parameters such as delamination, cracks, etc. Furthermore, chemical NDT data helps researcher to develop a more precise scientific model for concrete deterioration. Otherwise, the NDT data are just case sensitive data that used to make rehabilitation decisions for corrective maintenance.

Question 7 is a question about the inspection method for bridge component in different conditions. The response could be found in the following table.

Table 4-3 Survey Questionnaire: Question 7

Q7: What are the NDT tests conducted on the following bridges? Please identify the bridge element where the tests will be applied.

	Inspection Method			Bridge component		
Bridge Condition	Sounding Test	Advanced Test	Chemical NDT	Bridge deck	Superstructure	Substructure
Relatively new	3	2	☐	4	3	2
Moderately deteriorated	8	5	2	8	5	5

Severely deteriorated	7	6	4	8	5	7

It is clear that the sounding test is the mostly used technique, especially for bridge deck component. And as the bridge components deteriorate, the inspection is more thorough and more advanced test and chemical NDT are involved.

The Ground Penetrating Radar is the most popular advanced testing methods used according to the survey, followed by Infrared thermography (IR) and Ultrasonic testing. Magnetic Resonance testing and Impacted Echo is also used by some DOT. Only one out of 10 DOTs used half-cell potential test.

Based on the response from survey questionnaire as shown in Table 4-4, the chemical NDTs proposed at that time are ASR reaction test, Carbonation test, Chloride content test, concrete penetrability test, air/water permeability test and moisture content test. One state DOT pays more attention to chemical NDTs and utilizes ASR reaction test, carbonation test, concrete penetrability test and moisture content test often, with the assistance of application of chloride content test and permeability test rarely. The others reported rare or never used chemical nondestructive test methods. Most of the responses indicated that the purpose of conducting chemical NDTs is mainly for repair and rehabilitation instead of preventive maintenance.

Table 4-4 Survey Questionnaire: Question 9

Q9: What Chemical NDTs does your agency use? Please identify how often the tests are considered and the main purpose of performing the tests.									
	NDT Usage by type			Maximum Interval (years)			Main Purpose		
	Very often	Often	Rare or never	≤2	2~5	>5	Preventive Maintenance	Routine Maintenance	Repair and Rehabilitation
ASR reaction test	1	1	5	0	0	3	0	0	4
Carbonation test	0	1	5	0	0	2	0	0	2
Chloride content test	1	1	6	0	0	5	0	0	5
Concrete penetrability test	0	2	6	0	0	2	0	0	3
Air/Water permeability test	0	0	6	0	0	1	0	0	1
Moisture content test	0	1	5	0	0	1	0	0	1

According to the survey, it can be conclude that the chemical NDTs are not performed efficiently in the current States DOTs practice. The shortcomings of the current application of chemical NDTs are as follows:

- Chemical NDTs are not used as inspection tools that guide the preventive maintenance plans; instead they are being used to assist in the decision making for repair and rehabilitation.
- Chemical NDTs should be performed on not only heavily deteriorated bridge components, but also on bridge components that still in good or fair condition.
- Chemical NDTs should be used for a shorter inspection interval in order to monitor the concrete properties.

- The data of chemical NDTs should be integrated into the bridge management systems so that more precise scientific deterioration models for concrete bridge may be developed.
- More resources and funds might be needed to conduct more frequent chemical NDTs for more concrete bridges.

4.6 Needs for Chemical Non-destructive Testing

As previously described, the visual inspection and most of the physical inspection techniques can only detect physical deteriorations such as cracks, delamination or corrosion after they occur. Without the presence of chemical NDT, it is difficult to predict the occurrence of future deterioration. By examining the chemical condition of the concrete underneath the surface, parameters like chloride intrusion, carbonation, and corrosion can be detected prior to physical degradation. Based chemical NDT data, potential problems could be mitigated before expensive, extensive repair and rehabilitation are necessary.

Electronic tests like x-ray and ground-penetrating radar involve expensive equipment and can involve lengthy and expensive lane closures (especially when structural decks are being examined). On the other hand, chemical tests for measuring ASR, carbonation, and chloride content are all performed using cores and/or by drilling holes and utilizing comparatively low-cost tests. For example, a test kit for measuring carbonation would cost about $250.

Currently, bridges are typically visually inspected biennially. By incorporating non-destructive testing into the inspection process, inspectors and engineers can gain a better understanding of not just the physical state of the bridge, but the chemical state as well. Chemical tests for processes like ASR, carbonation, and chloride intrusion can indicate problems before they are necessarily visible and enable state DOTs to utilize preventive maintenance measures to mitigate the effects of these problems.

The test results from the carbonation test, chloride concentration test and the chloride diffusion test will help researcher to build up more accurate deterioration models for the concrete with different composition and properties under different service conditions. The Carbonation test is essential for determine the cause of corrosion initiation and to check the effectiveness for chemical re-alkalization application. The chloride content test is useful for determining whether the reinforcing steel is prone to corrosion and to estimate the remaining time for corrosion initiation. The chloride diffusion test is usually performed for new constructions in order to give a reference value for the chloride diffusion coefficient. Other tests, like half-cell potential test, resistively meter test, combined with the result from other tests can be used not only for detect corrosion, but also for calibration of the theoretical chloride threshold value.

While electrical/radiological testing methods like GPR do not necessarily require physical samples of concrete to be taken from structures, there are some drawbacks to their use. First, these methods only provide the equivalent of visual inspection at all depths of the structure instead of a chemical analysis. Second, they are mainly used in the inspection of concrete deck (and deterioration is still a major concern in the superstructure and substructure of concrete bridges). Finally, for inspecting bridge decks, lane (or entire bridge) closures are required while the lengthy procedures are performed. Comparatively, chemical NDTs, while requiring sampling, can be performed completely on-site in relatively little time with minimal equipment and personnel, minimizing the impact on traffic. Additionally, these tests can determine if further examination with more expensive, disruptive methods is necessary. However, the assistance of these electrical/ radiological testing is still helpful and should be performed more frequently in order to assess the minor damage caused by shrinkage, abrasion or other reasons. The presences of the fine cracks, which can be detected by these methods, are essential to estimate the chloride diffusion coefficient.

In general, the non-destructive tests should be performed on concrete bridge components more frequently, and particularly, should be performed regularly on the components that still in a good or fair condition state. The test results could be integrated into the deterioration model for the estimation of bridge deterioration process as well as the remaining service life of the bridge components. Only with the collection of the NDT test data and refined deterioration model, the scientific approach for optimization of the preventive maintenance methods for concrete bridge system can be achieved.

4.7 Conclusion

Chemical NDTs are essential for estimating the deterioration process and prediction of the service condition state for concrete bridge elements. The test should be applied on all elements, especially those elements that are still in good or fair condition. Carbonation tests, chloride content tests, chloride diffusion tests and ASR test should be performed in order to quantify the parameters for numerical deterioration equations. The following tests should be performed:

- Carbonation Test: The carbonation depth can be used to evaluate the cause of corrosion; to estimate service life where penetration of the carbonation frontier is critical; to monitor the effectiveness of applications for re-alkalization.
- Chloride Content Test: The chloride content test can be used to evaluate the cause of corrosion; estimate the apparent diffusion coefficient by curve fitting using the chloride diffusion equations; and monitor the effectiveness of preventive maintenance measures such as sealers and membranes by comparing the chloride concentration.
- Chloride Diffusion Test: The chloride diffusion test can be used to test the resistance of concrete for chloride ingression under exposure; estimate the diffusion coefficient and the remaining service life of the specimen; check and verify the effectiveness of preventive maintenance applications such as sealers and membranes.
- ASR Test: The ASR test can be used to determine if the concrete structure is prone to ASR deterioration mechanism.

Other than the tests listed above, the properties of the concrete should be tested and recorded. The compressive strength, porosity and permeability can be tested and used as a reference to get a value for the water cement ratio for the existing bridge if no historical data is available. The dynamic modulus of elasticity should also be tested for all new structure.

The NDT can be also used as a quality control tool. Combining the results of carbonation test, chloride diffusion test and chloride content tests with numerical deterioration models, the effectiveness of the preventive maintenance measures can be evaluated. Therefore, the NDTs are not only a method for inspection. They should also be adopted as part of a preventive maintenance plan.

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Chapter 5 Thresholds for Various Harmful Compounds

In order to prevent chemical and physical deterioration of concrete bridge components, it is important to identify the thresholds at which a bridge member would start deteriorating; e.g. chloride limits at which corrosion of steel bars in concrete starts. This chapter presents various harmful compounds and their limits.

5.1 Mathematical Models for Chloride Transportation Mechanism

5.1.1 Chloride Transport Mechanism

Chloride ions and other aggressive particles penetrate through concrete via different mechanism depending on the driving force involved. The most well-known chloride transport mechanisms are diffusion, permeability and absorption. Other phenomena such as chloride binding can also influence chloride ingress.

The moisture content of concrete and the surrounding environment determine the driving force and the mechanisms by which chloride penetrates into concrete. In saturated concrete, chloride transport occurs by diffusion through the pore solution. However, under unsaturated condition, which is a common state for concrete with exposed surfaces, the movement of chloride ions is largely controlled by absorption through the capillary pore system and diffusion of chlorides through pore solution.

In bridges, concrete experiences wet and dry circles due to rain or condensation. Liquid in the pores evaporates progressively from the surface. Under this circumstance, the chloride will most likely enter the concrete surface initially by absorption and then diffuse into inner area. A reservoir will be generated and topped up by periodic absorption events. If the concrete dries out to a greater depth, subsequent wettings carry the chlorides deeper into the concrete. Thus it would appear that absorption and diffusion are important transport mechanism associated with chloride ingress in bridges. (Hong, 1999)

Since the chloride diffusion is the most dominating mechanism for chloride ingression into the concrete and it is suitable for long-term modeling, the chloride diffusion is the only transportation mechanism that we adopt in our model.

5.1.1.1 Diffusion

When concrete is saturated such as in submerged conditions, diffusion is the dominant mechanism for chloride transportation. Also, it is suitable for long-term performance evaluation. For non-steady-state conditions, the concentration gradient changes time, the flux can be simply described according to Fick's second law:

$$D\frac{\partial^2 C}{\partial x^2} = \frac{\partial C}{\partial t} \tag{5.1}$$

Where,

C= Chloride concentration

D= Diffusion coefficient

t= Time

x= Depth

Assuming constant *D*, the equation can be solved by applying the error function solution:

$$\mathrm{C}_{(\mathrm{x,t})} = \mathrm{C}_0 + (\mathrm{C}_\mathrm{s} - \mathrm{C}_0)(1 - \mathrm{erf}\frac{\mathrm{x}}{2\sqrt{\mathrm{DT}}}) \quad (5.2)$$

Where,

C_0= Initial chloride content in concrete

C_s= Surface Chloride Concentration

$\mathrm{C}_{(\mathrm{x,t})}$= Chloride concentration at depth x on time t

$\mathrm{erf}(*)$= Error function from mathematic equation chart

5.1.1.2 Permeation

Permeability is the movement of a liquid under hydrostatic pressure. Permeability can be described by Darcy's law, which states that the steady-state rate of flow is directly proportional to hydraulic gradient. (Basheer et al, 2001)

$$v = \frac{Q}{A} = \frac{k\rho g}{\eta}\frac{\Delta h}{L} \quad (5.3)$$

Where,

v= Apparent velocity of flow (m/s)

Q= Flow rate (m^3/s)

A= Cross-sectional area of the sample (m^2)

k= Intrinsic permeability of materials (m^2)

ρ= Density of the fluid (kg/ m^3)

g= Acceleration due to gravity ($\mathrm{m/s}^2$)

η= Dynamic viscosity of the fluid (kg/m·s)

Δh= Drop in hydraulic head through the sample (m)

L= Thickness of the sample (m)

When the flow is of an unsteady state (i.e. the flux changes with time), the hydraulic head may not decrease linearly along the direction of flow and in which case the flow velocity is given by the following equation.

$$v = \frac{k\rho g}{\eta}\nabla h \quad (5.4)$$

The permeability of concrete depends on the porosity, water-cement ratio, type of cement, cement replacement materials and the progress of hydration.

In general, for concrete bridges, the most structure components are not in contact with water under pressure. Therefore, the chloride ingress caused by permeation is generally not considered for the service life estimation.

5.1.1.3 Absorption

The transport of liquids in unsaturated porous concrete due to capillary effect is called absorption. The amount of chlorides sucked into the concrete depends on the salt concentration, the duration of suction and the coefficient of capillarity.

$$Q = \frac{cQ}{\rho} A\sqrt{t} \tag{5.5}$$

Where,

Q= Amount of chlorides taken (kg/ m^2)

A= Coefficient of capillarity (kg/ $m^2\sqrt{s}$)

c = Salt concentration (kg/ m^3)

ρ = Density of the fluid (kg/ m^3)

t = Duration (s)

There is still a lack of experimental data, and the available data shows some inconsistencies. The chloride binding properties are decisive for the prediction of the effect of absorption. However, the nature for the binding capacity of chloride ions is still under investigation. Therefore, chloride ingress due to absorption is also excluded from the analysis of service life prediction of the structure. The chloride concentration should be taken under 0.5 in depth into the concrete in order to rule out the fluctuation of chloride concentration caused by these effects.

5.1.2 Chloride Diffusion Coefficient

In order to apply the chloride diffusion model based on Fick's law, the diffusion constant needs to be specified. The diffusion constant is a function of the permeability and condition of the concrete, environmental factors such as exposure and climate, care of construction, and subjection to wearing and use. The input diffusion coefficient for NCDOT chloride ingress model is 0.0783-0.147 in^2/year. Table 5-1 shows diffusion constant specified by several states' departments of transportation. (Ward-Waller, 2004) More diffusion coefficient published in the literature are listed in Appendix C.

Table 5-1 Mean Diffusion Constants for Various States (Ward-Waller, 2004)

	Mean
California	0.25
Delaware	0.05
Florida	0.33
Indiana	0.09
Iowa	0.05

Kansas	0.12
Minnesota	0.05
New York	0.13
West Virginia	0.07
Wisconsin	0.11

5.1.2.1 Theoretical Approach

Japan Society of Civil Engineers, JSCE (2002) proposed equation for estimation of chloride diffusion coefficient by:

$$log\mathrm{D}_c = -3.9\left(\frac{\mathrm{w}}{\mathrm{c}}\right)^2 + 7.2\left(\frac{w}{c}\right) - 2.5 \qquad For\ OPC \tag{5.6}$$

$$log\mathrm{D}_c = -3.0\left(\frac{\mathrm{w}}{\mathrm{c}}\right)^2 + 7.2\left(\frac{w}{c}\right) - 2.5 \qquad For\ concrete\ with\ slag\ or\ SF \tag{5.7}$$

Boulfiz et al. (2003) investigated mathematical models and numerical simulations for water movement and chloride ions ingress by diffusion and advection in cracked and uncracked concrete under saturated or unsaturated conditions and derived the following equation:

$$log\mathrm{D}_c = -3.0\left(\frac{\mathrm{w}}{\mathrm{c}}\right)^2 + 5.4\left(\frac{w}{c}\right) - 13.7 \qquad For\ concrete\ with\ slag\ or\ SF \tag{5.8}$$

$$log\mathrm{D}_c = -3.9\left(\frac{\mathrm{w}}{\mathrm{c}}\right)^2 + 7.2\left(\frac{w}{c}\right) - 14 \qquad For\ OPC\ or\ Concrete\ with\ FA \tag{5.9}$$

Ehlen’s Life-365$^{\mathrm{TM}}$ Life Prediction Models and computer program also come up with a solution for estimation of the diffusion coefficient. The base case concrete mixture is plain Portland cement concrete with no special corrosion protection strategy. For this case, the following values are assumed:

$$D_{28} = 1 \times 10^{(-12.06 + 2.40w/cm)} \tag{5.10}$$

The addition of silica fume is known to produce significant reductions in the permeability and diffusivity of concrete. The presence of the silica fume is affecting the diffusion coefficient as shown below.

$$D_{SF} = D_{PC} \cdot e^{-0.165 \cdot SF} \tag{5.11}$$

The diffusion coefficient rate is based on the water cement ratio and the type and proportion of cementitious materials. It also decays with time and governed by the following equation.

$$\mathrm{D(t)} = \mathrm{D}_{28}\left(\frac{\mathrm{t}_{28}}{\mathrm{t}}\right)^m \tag{5.12}$$

For ordinary plain Portland cement concrete, m equals 0.2.

Neither fly ash nor slag are assumed to affect the early-age diffusion coefficient, D_{28}, or the chloride threshold, C_t. However, both materials impact the rate of reduction in diffusivity and hence the value of m. The presence of the silica fume is affected by the diffusion coefficient decay factor as shown below. (Life-365, 2014)

$$m = 0.2 + 0.4^{(\%FA/50\,+\,\%SG/70)} < 0.6 \tag{5.13}$$

(Only valid up to replacement levels of 50% of fly ash or 70% of slag)

Ferreira (2010) provide a model suitable for concrete with high-performance Portland cement, silica fume, fly ash and slag.

$$\mathrm{D(t)} = \mathrm{D_{ref,294}Kf(t)f(T)} \tag{5.14}$$

$$\mathrm{f(T)} = \left({}^{T}\!/_{294}\right) \cdot \exp\left[{}^{U}\!/_{R} \cdot \left({}^{1}\!/_{294} - {}^{1}\!/_{T}\right)\right] \tag{5.15}$$

$$\mathrm{f(t)} = \left(\frac{\mathrm{t}_{ref}}{\mathrm{t}}\right)^{m} \tag{5.16}$$

Where,

U = Activation energy of diffusion process (J mol^{-1})
R = Gas constant (8.314 J mol^{-1} K^{-1})

In addition, a diffusion coefficient estimation model was presented by Saetta (1993) for silica fume concrete;

$$\mathrm{D} = \mathrm{D_{ref}f_1(T)f_2(t)f_3(h)} \tag{5.17}$$

$$\mathrm{f_1(T)} = \exp\left[{}^{U}\!/_{R} \cdot \left({}^{1}\!/_{T_{ref}} - {}^{1}\!/_{T}\right)\right] \tag{5.18}$$

$$\mathrm{f_2(t)} = \zeta + (1-\zeta)\left(\frac{28}{t}\right)^{0.5} \tag{5.19}$$

$$\mathrm{f_3(h)} = \left[1 + {}^{(1-h)^4}\!\Big/_{1-h_c{}^4}\right]^{-1} \tag{5.20}$$

Where,

ζ = Constant from 0 to 1

h = Current humidity (%)

h_c = Critical humidity level at which the diffusion coefficient drops halfway between the maximum and minimum value (%)

5.1.2.2 Influencing Factors for Diffusion Coefficient

Besides the parameters included in the scientific models introduced above, such as temperature, water cement ratio, binder type and hydration process, the chloride diffusion coefficient is also affected by other parameters.

Construction workers can significantly influence the final coulomb value for a concrete specimen. Poor consolidation and poor curing increase concrete permeability and increase the coulombs passing through the concrete. Using a 7-day moist cure instead of a 1-day moist cure can even decrease the chloride permeability by about 30%. Figure 5-1 shows the chloride diffusion coefficient determined by Rapid chloride permeability test versus the curing condition. It is clear that the chloride diffusion coefficient, represented by the charge passed through the concrete slice, is decreasing while the time of most curing days increasing.

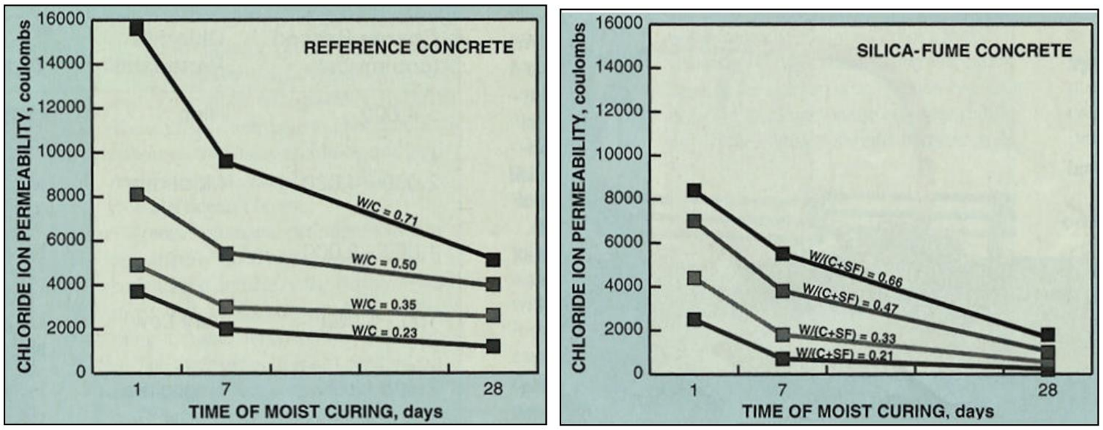


Figure 5-1 Time of moist curing vs. Chloride Ion Permeability (Suprenant, 1991)

The porosity and the permeability of the concrete will also affect the diffusion coefficient. Higher porosity and larger pore sizes lead to more severe corrosion damage in the steel. If the concrete has low permeability, then the chlorides and carbon dioxide would be difficult to access the reinforcement and the possibility of corrosion would be low.

Low water cement ratio, good compaction, use of mineral admixtures and proper curing would lower the permeability of the concrete.

Figure 5-2 shows the correlation between gas permeability and chloride diffusion coefficient based on Sugiyama (1996) investigation. This is valid in particular when the water-cement ratio of concrete dominates the gas permeability. The correlation is based on the fact that the chloride diffusion coefficient is also a factor that significantly affected by the water-cement ratio. The correlation may become poor if the specimen experienced a longer period of drying for gas permeability due to the micro-cracks formed during the elongated drying period.

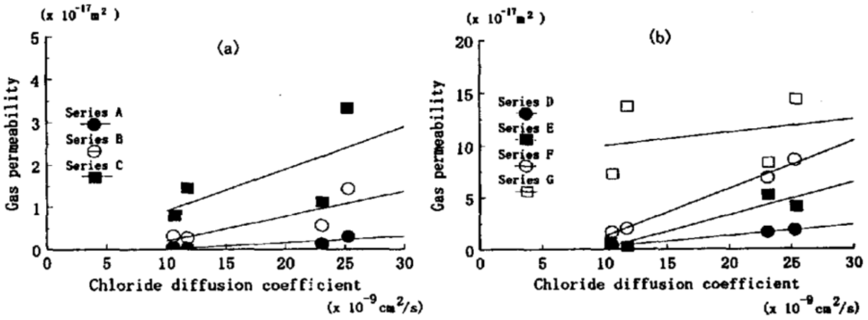


Figure 5-2 Gas Permeability vs. Chloride Diffusion Coefficients: (a) Air Dry for Gas Permeability (b) Oven Dry for Gas Permeability (Sugiyama, 1996)

Pore waters evaporate from the concrete pores under the fluctuation of ambient relative humidity of the environment as shown in Figure 5-3.

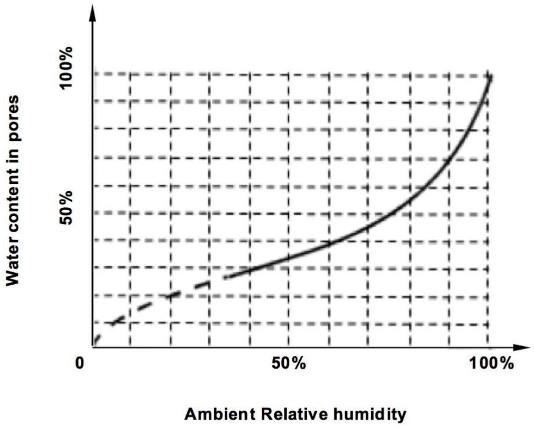


Figure 5-3 Water Content in Pores vs. Ambient Relative Humidity (CEB, 1992)

If there were no water in the concrete, there would be no corrosion of reinforcement in the concrete. The moisture content has a complicated influence on the corrosion process since the resistivity increase as the moisture content gets higher and at the same time, the anodic and cathodic reaction rate is decreased due to the lack of oxygen. Figure 5-4 shows the diffusion coefficient effected by relative humidity.

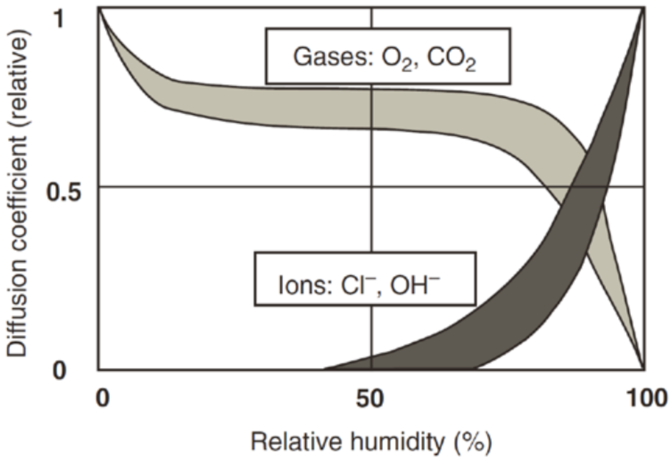


Figure 5-4 Relative Humidity vs. Diffusion Coefficients (Cement Concrete & Aggregates Australia, 2009)

However, research has also shown that the water content in pores for specimen submerged in the water also decreases as the concrete cover depth increase. Therefore, in the real structure, the pore solution may not be available for the concrete in depth, which may cause a decrease of the chloride diffusion coefficient.

Chloride binding capacity can also influence the chloride diffusion mechanism. It is not only associated with the pH value of the pore solution, but also with the associated cation as shown in Figure 5-5. More chlorides were found to be bound when the associated cation was Mg^{2+} or Ca^{2+} compared to Na^+.

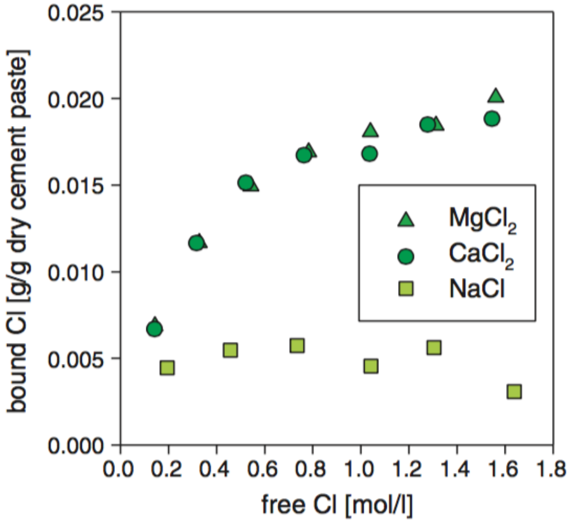


Figure 5-5 Chloride binding isotherms for pastes exposed to $MgCl_2$, $CaCl_2$, NaCl solutions (Weerdt, 2015)

The presence of cracks will also have a great impact on the chloride diffusion coefficient. Gerard (2000) states that for Ordinary Portland Cement with a water cement ratio of 0.45, the chloride diffusion coefficient jumped from $9.7 \times 10^{-9} cm^2/s$ for the virgin concrete to

$76.52 \times 10^{-9} cm^2/s$ after experiencing 95 freeze and thaw cycle. Relations regarding to the ratio between the chloride diffusion coefficients for undamaged and damaged concrete specimen has been correlated with the ratio of effective elastic modulus of concrete under these two different conditions.

Shekarchi (2009) performed a study on the long-term chloride diffusion in silica fume concrete in marine climates. Samples were taken at the ages of 3, 9 and 36 months and analyzed to determine the chloride diffusion coefficient. Also time-dependent chloride diffusion and compressive strength of concrete containing silica fume are investigated. Cubes of $150 \times 150 \times 150$ mm and prisms of $150 \times 150 \times 600$ mm in dimension were cast in steel mold and compacted on a vibrating table. The 150 mm cubes were used for the determination of compressive strength while the prisms were used to test chloride diffusion. The specimens were removed from the molds and were allowed to cure in water saturated with calcium hydroxide at 21 °C for 28 days. After the curing period the prism specimens were sealed on four sides using epoxy polyurethane coating to ensure one-dimensional diffusion as illustrated in Figure 5-6.

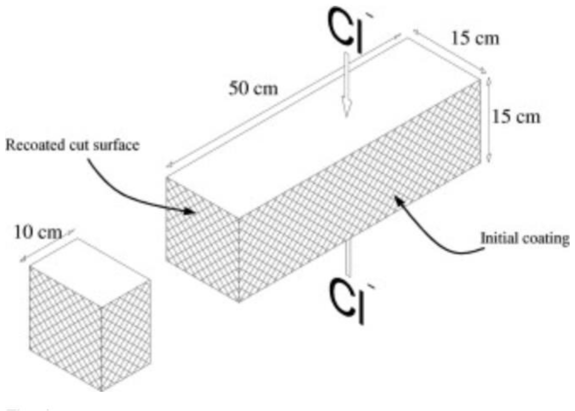


Figure 5-6 Concrete Specimen (Shekarchi, 2009)

The Evolutions of the diffusion coefficient for different SF content at sampling ages with a water-cement ratio is show in Figure 5-7.

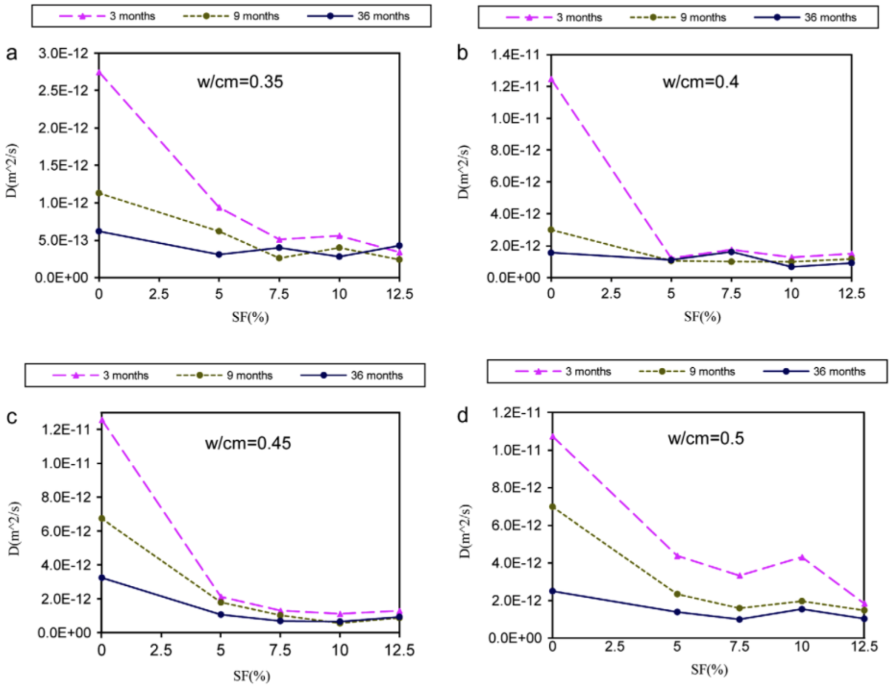


Figure 5-7 Diffusion Coefficients for Different Concrete with Different SF Content (Shekarchi, 2009)

5.1.2.3 Determination of the Chloride Diffusion Coefficient

Chloride diffusion coefficient is a vital parameter for estimation of the chloride-induced corrosion for reinforced concrete. However, the derivation of chloride diffusion coefficient is usually based on the curve fitting of the chloride profile using Fick's law. Therefore, no direct method is available to determine the chloride diffusion coefficient. It can be estimated either by the non-destructive tests like salt ponding test, rapid migration test, rapid chloride permeability (RCP) test, or by mathematical equations.

The chloride diffusion coefficients can be classified as:

- The real but unknown chloride diffusion coefficient.
- The achieved chloride diffusion coefficient obtained by curve fitting to a chloride profile.
- The potential chloride diffusion coefficient as measured by RCP test, etc.

The diffusion coefficient rate is based on the water cement ratio and the type and proportion of cementitious materials, like the equation developed by Ehlen. And like discussed above, relative humidity, cracks and freeze and thaw damage will affect the diffusion coefficient either. Therefore, terms to represent theses parameters should be added into the equation.

$$\mathrm{D(t)} = \mathrm{D_{ref}} \cdot f(t) \cdot f(RH) \cdot f(FT) \tag{5.21}$$

Where,

$\mathrm{D_{ref}}$ = The chloride diffusion coefficient measured by CTH or similar test, normally at a maturity age of 28 days.

$f(t)$= Effects of hydration of the concrete

$f(RH)$= Effects of relative humidity

$f(FT)$= Effects of freeze-thaw effect

The chloride diffusion coefficient measured by CTH or similar test, normally at a maturity age of 28 days. For concrete with different water cement ratio, the D_{28} can be estimated using the following equation.

$$D_{28} = 1 \times 10^{(-12.06+2.4^{W}/_{c})} \cdot e^{-0.165 \cdot SF} \tag{5.22}$$

Where, SF= the percentage of silica fume (%)

$$f(t) = \left(\frac{\mathrm{t}_{ref}}{\mathrm{t}}\right)^{m} \tag{5.23}$$

The presence of slag and fly ash materials will impact the rate of reduction in diffusivity and hence the value of m. The presence of the silica fume is affecting the diffusion coefficient decay factor as shown below. (Life-365, 2014)

$$m = 0.2 + 0.4^{(\%FA/50\ +\ \%SG/70)} < 0.6 \tag{5.24}$$

(Only valid up to replacement levels of 50% of fly ash or 70% of slag)

$$f(RH) = [1 + {}^{(1-RH)^4}\!/_{1-{RH_c}^4}]^{-1} \tag{5.25}$$

Where,

RH = Current humidity (%)

RH_c = Critical humidity level at which the diffusion coefficient drops halfway between the maximum and minimum value (%)

(Only valid up to replacement levels of 50% of fly ash or 70% of slag)

According to the research done by Zhao (2014), the diffusion coefficient increases corresponding to the degree of damage according to the damage mechanics theory. From the test result provided by Zhao, a linear function can be used to estimate the diffusion coefficient.

$$f(FT) = 1 + 1.75\mathrm{D} \tag{5.26}$$

D increases linearly when the freeze–thaw cycles increase from 25 to 75. Cement type has no significant influence on the damage degree of concrete.

$$D = \left[1 - {}^{E_n}\!/_{E_0}\right] \times 100\% \tag{5.27}$$

Where D is the degree of damage, %; E_n is the dynamic elastic modulus of concrete after a certain number of freeze–thaw cycle; E_0 is the initial dynamic elastic modulus of concrete. The E_0 should be tested and recorded for all new constructions.

The diffusion coefficient can be reduced by 30% if wet curing is used for at least 7 days. The effect of lightweight aggregate is not well understood yet. However, the lightweight aggregate usually has a higher permeability, therefore, the diffusion coefficient for concrete using lightweight aggregate should be increased by a certain factor. More research and data are needed in order to derive that factor.

If the achieved chloride diffusion coefficient (D_a) is used to estimate the chloride diffusion coefficient, it needs to be modified since the achieved chloride diffusion coefficient was significantly smaller than found from concrete specimens exposed and tested in the laboratory.

5.2 Thresholds for Chloride Content

Reinforcement corrosion in non-carbonated, alkaline concrete can only start once the chloride content at the steel surface has reached a certain threshold value. In the literature, this value is often referred to as critical chloride content or chloride threshold value, as shown in Figure 5-8. (Angst, 2009) The thresholds value for chloride concentration can be defined in two different ways: for scientific point of view, the critical chloride content can be defined as the chloride concentration required for corrosion initiation; where for a practical point of view, the chloride threshold value can be defined with visible or physical deterioration of the reinforced concrete structure.

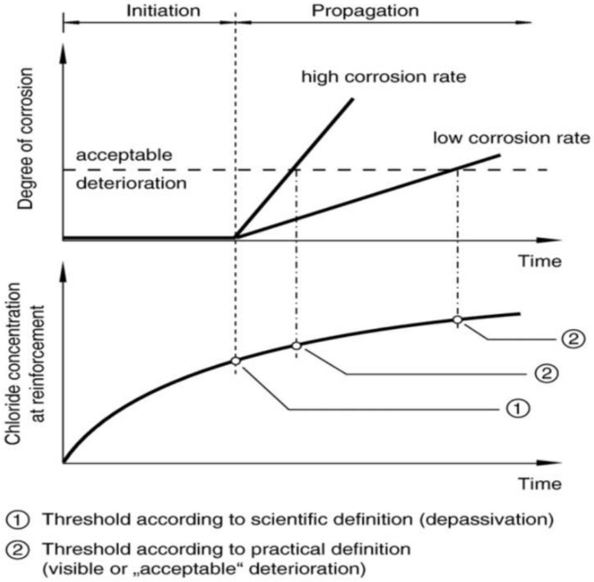


Figure 5-8 Definition of Chloride Threshold Value (Angst, 2009)

The critical chloride concentration is commonly expressed as total chloride, free chloride and $\mathrm{Cl^-/OH^-}$.

The main reason for using total chloride as the measurement of chloride threshold value is that the test is relatively simple and well documented in standards. The value is quantified by the weight ratio of chloride ions and the cement/binder. While the binder content is hard to detect, the total chloride concentration can be also expressed relative to the weight of the concrete.

By assuming the bound chloride has not contribution to the corrosion process, it is reasonable to remove all the bounded chloride from the total chloride concentration and using the free chloride as an indicator to the potential of corrosion initiation. This value could be related to either the weight of the cement or the weight of the concrete.

Also, critical thresholds values could be also expressed in the terms of $\mathrm{Cl^-/OH^-}$. Publications often cited in this regard are those by Hausmann (1967). A conservative value of $\mathrm{Cl^-/OH^-}$ is set as 0.6 as the critical value for corrosion initiation.

The critical values for chloride contents in the literature has been summarized and attached in Appendix B. The critical values are either estimated by the numerical model or detected from the experimental directly. Due to the numerous factors affecting on the corrosion process, the critical values shows a large variance under different circumstance.

The ranges of chloride threshold spans through the following limitations based on the literature reviewed:

- (% by weight of concrete): 0.05 to 0.1.(2-4 pound/yd3)
- Total chloride (% by weight of cement): 0.02 to 3.08. (Typically 0.4-0.6)
- Free chloride: 0.045 to 3.22 (mole/l) or 0.07 to 1.16 % by weight of cement.
- [Cl-]/[OH-] ratio: 0.01 to 20.

5.2.1 Influencing Parameters for Chloride Thresholds

5.2.1.1 Steel-Concrete Interface

The importance of entrapped air voids adjacent to the reinforcement steel surface needs to be emphasized since corrosion starts at the interface. Figure 5-9 shows the impact of interfacial voids between concrete and the reinforcing steel on the chloride threshold value. The presence of the air void may trigger the corrosion process without the chlorides are introduced internally or externally. The absence of the cement material would not be able to resist a local fall in the pH, which may leads to a premature corrosion initiation.

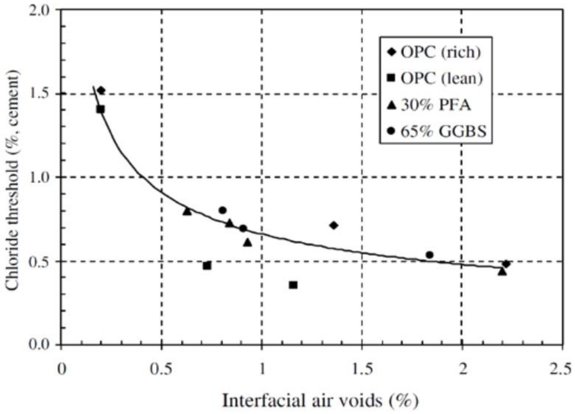


Figure 5-9 Chloride threshold as function of interfacial voids (Ann, 2007)

Although the physical condition of the steel–concrete interface in terms of the entrapped air void content has an effect on the critical value of chloride content, it is still hard to quantify the affects since there is only limited methods to detect air void content at the interface non-destructively and the method is only valid for large void and ribbed bar.

5.2.1.2 pH value of the pore solution

The pore solution in concrete is an electrolyte, which is physically absorbed in the pores of the concrete. It may contain various ions, such as sodium, potassium, calcium, hydroxyl, sulphate and sulfite, etc. The chemical composition of the pore solution has a great impact on the pH value of the concrete. However, the carbonation process, leaching or proceeding hydration can also affect the pH value of the pore solution at later stage. Once the pore solution is polluted by chloride ions or de-alkalized by carbon dioxide, the corrosion may occur.

The inhibiting effect of hydroxide ions against chloride induced corrosion, as a major factor influencing chloride threshold values for corrosion initiation was early recognized. The suggestion to present the threshold values of $\mathrm{Cl^-/OH^-}$ratios reflects this influence and it is shown in Figure 5-10. (Angst, 2009)

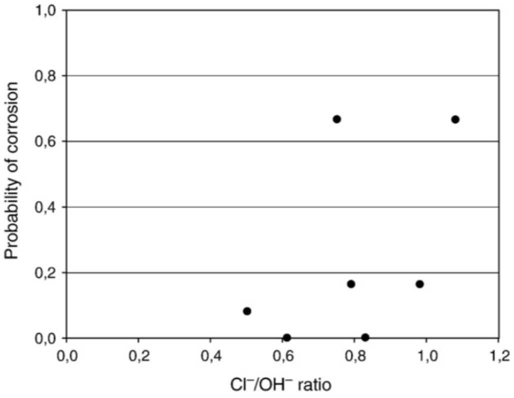


Figure 5-10 Probability of Corrosion Vs. Cl^-/OH^-ratio (Angst, 2009)

In addition, the pH value may also affect the binding capacity of the chlorides. Studies have shown that a reduction of the pH to a value below 12.5 results in the release of a considerable part of the bound chlorides, while yet another part is released if the pH value is reduced to a value around 12. (Poulsen, 2012)

Based on the experiments by Hausmann (1967), the chloride threshold value is most accurately expressed by using $\mathrm{Cl^-/OH^-}$ ratio. A value of 0.6 is suggested and verified by the following researchers.

5.2.1.3 Electrochemical Potential of Steel

The availability of oxygen and moister content at the steel surface are the two main factors determining the electrochemical potential of steel embedded in concrete. Figure 5-11 shows the potential compared with the chloride content. In order for pitting corrosion to occur, the equilibrium potential must be higher than the pitting potential. The pitting potential is dependent on the concentration of chloride and higher contents of chlorides can be tolerated if the steel has a greater negative potential.

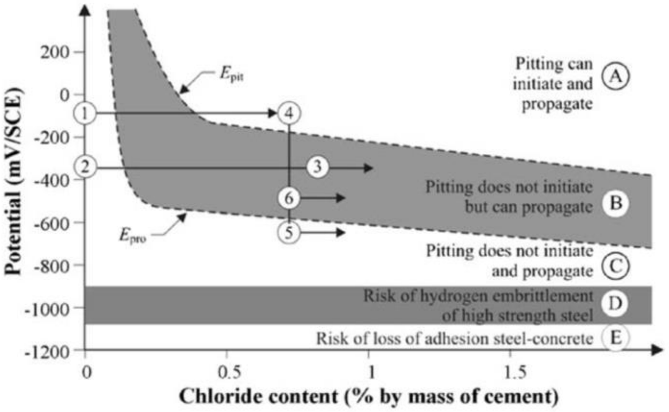


Figure 5-11 Steel potential versus chloride content in concrete (Bertolini, 2009)

Furthermore, compared with carbon steel, stainless steel has the much better resistant against corrosion than plain steel, as shown in Figure 5-12. It may remain passive and experience almost no corrosion even in relatively high chloride environment. MMFX2, another type of steel, also achieves a better performance compared to plain steel in corrosion environment by modify the microstructure of the steel. Galvanized steel using a different approach to manage a longer service life by sacrificing the zinc coating as a corrosion delay mechanism. All these types of steel are more expensive than the plain steel. However, considering the savings in long term, these corrosion resistant steels should be recommended in future construction projects.

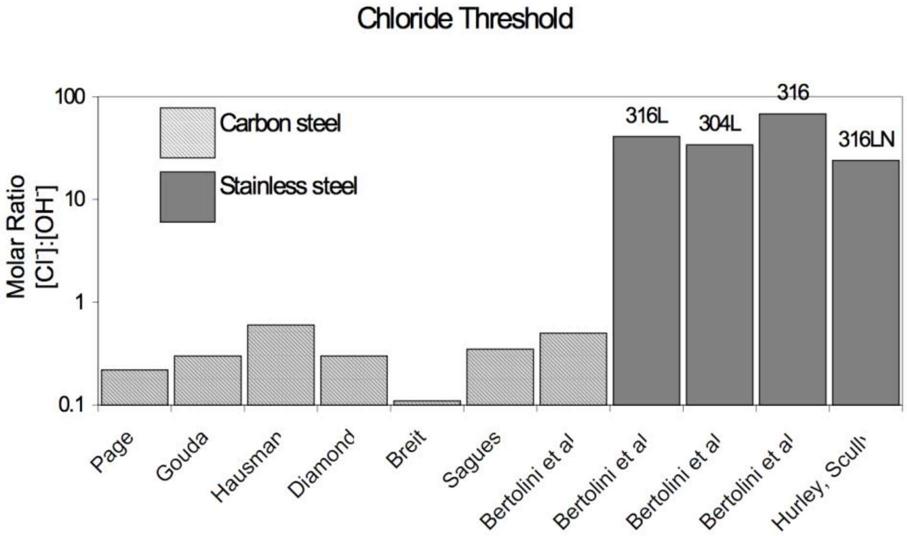


Figure 5-12 Chloride threshold value for Carbon steel and Stainless Steel (Hurley, 2008)

5.2.1.4 Binder Type

The binder type has great impact on the chloride threshold value. The effect of ground granulated blast-furnace slag (GGBS), fly ash (FA) and silica fume (SF) will change the critical chloride concentration in different ways. Figure 5-13 gives an example of the chloride threshold values for different binder type based on Poulsen's (2012) research.

GGBS has the effect of reducing the pH value of the pore solution, which promotes the initiation of pitting corrosion. However, it will increase the binding capacity of the concrete that may leads to a decrease of the free chloride content. Thus the overall effect of GGBS is hard to evaluate theoretically. Researchers give experimental reports that the GGBS either increase the chloride threshold value or decrease the value, and some other reports shows that the GGBS has no effect on the chloride threshold value.

Fly ash has the same impact on the pH value of the pore solution as well as the binding capacity of the binder. The fly ash may have no effects on the chloride threshold value. But due to its alternation of the pore structure in the concrete, the diffusion coefficient is affected by the presence of the fly ash and may slows the chloride ingress process and leads to a extended service life for the concrete component.

Unlike the other two substances, the Silica fume has a negative effect on the chloride threshold value. The pH value of pore solution is decreased and the binding capacity is decreased. The negative effect of silica fume on the chloride threshold value has been demonstrated in a number of studies. (Hansson, 1990; Petersson, 1993; Manera, 2008)

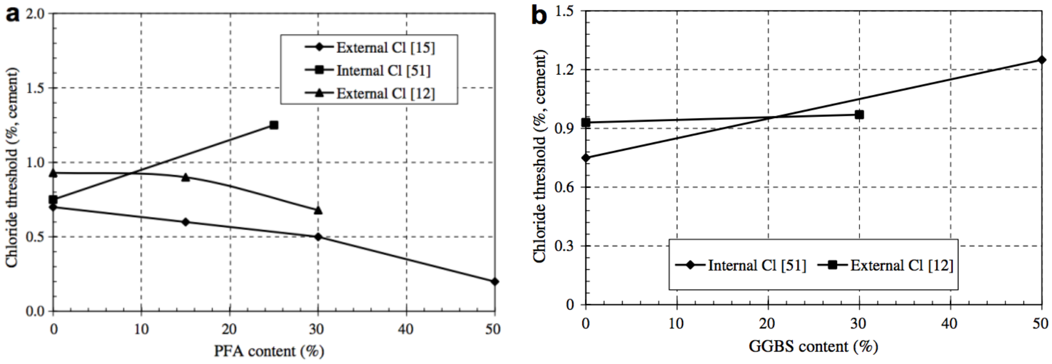


Figure 5-13 Chloride Threshold Values for Different Binder Type (Poulsen, 2012)

5.2.1.5 Relative Humidity

The effect of relative humidity on the chloride threshold level in laboratory-exposed mortars is shown in Figure 5-14, as presented by Pettersson (1996). As we could see that the threshold value for chloride ions increase when the available moister is controlled.

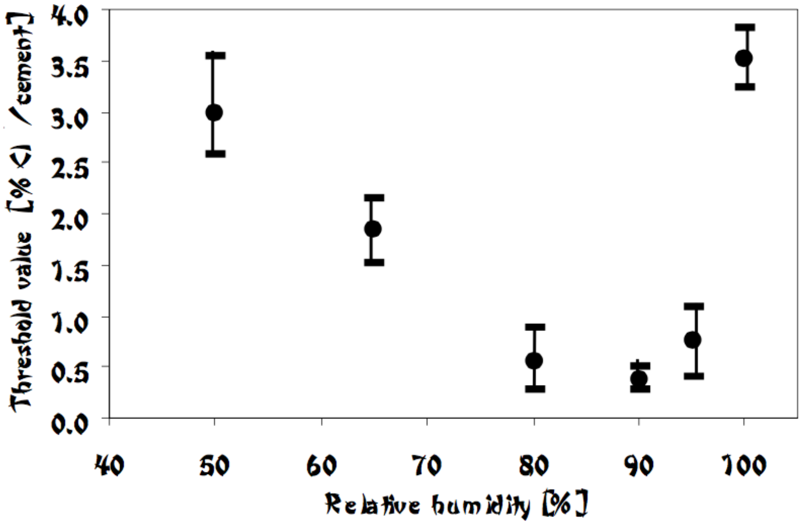


Figure 5-14 Relative humidity Vs. Chloride Threshold Value (Frederiksen, 2002)

5.2.1.6 Water-cement Ratio

Experimental data have indicated that concrete with lower water-cement ratio will have a higher chloride threshold value and vise versa, as shown in Figure 5-15. (Pettersson, 1992, 1994; Schiessel and Breit, 1995) This is mainly due to the following consequences of a low water-cement ratio (Nilsson et al., 1996):

- Reduced area available for pitting corrosion development at the interface between steel and concrete
- Higher resistivity of the concrete
- Lower chloride mobility
- Improved ability to maintaining a high alkalinity

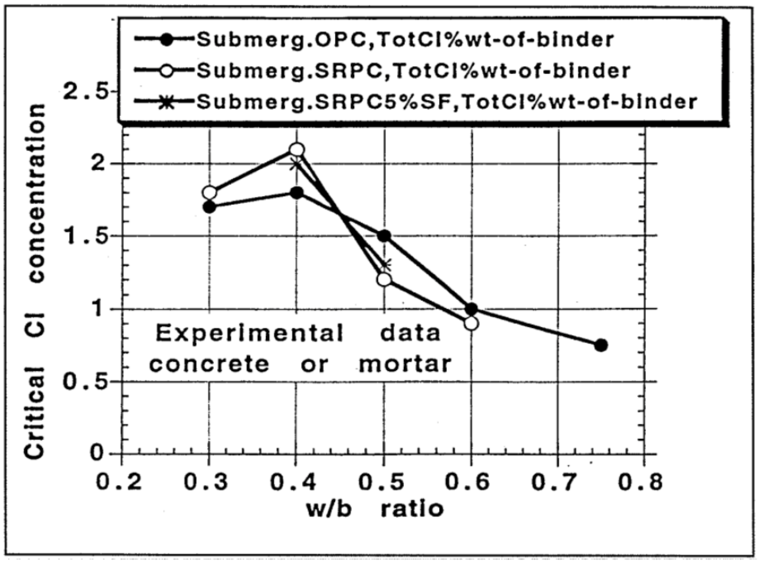


Figure 5-15 Water-cement Ratio Vs. Chloride threshold Value (Poulsen, 2012)

5.2.1.7 Degree of Hydration

The water-cement ratio as well as the degree of hydration has an effect on the porosity of the paste and by this the availability of moisture and oxygen at the reinforcement. The threshold concentration increases with increased concrete age. This is particularly evident when it is assumed that chloride and alkali is only dissolved in capillary water according to the research done by Fagerlund. (2011)

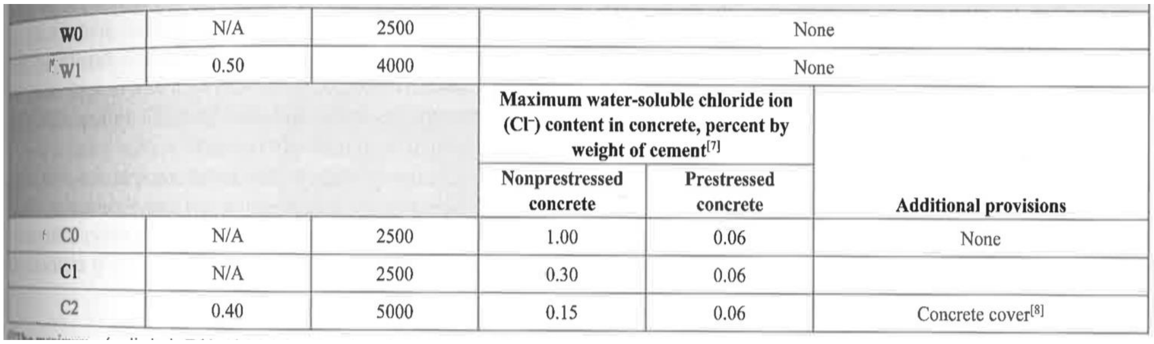
5.2.1.8 Inhibiting Substances

The presence of corrosion inhibitors can affect the threshold value dramatically. The $Ca(NO_2)_2$ inhibitor exhibits an inhibition effect only when the molar ratio between nitrite ions and chloride ions is less than 0.21. The ZnO and DMEA inhibitors can also effectively reduce the corrosion rate of steel in a saturated $Ca(OH)_2$ solution. All the inhibitors have a marginal effect on increasing the chloride threshold value for steel corrosion in a saturated $Ca(OH)_2$ solution. The reason may be due to the fact that the composition of the passive film on the steel surface does not change with the additions of the inhibitors. (Xu, 2013)

5.2.2 Models for Chloride Threshold

The ACI code has a limit for chloride for new construction. The acid-soluble and water-soluble chloride limits are listed in Table 5-2.

Table 5-2 Chloride Limits for New construction (ACI 318-14, 2014)



W0	N/A	2500	None		
W1	0.50	4000	None		
			Maximum water-soluble chloride ion (Cl^-) content in concrete, percent by weight of cement[7]		Additional provisions
			Nonprestressed concrete	Prestressed concrete	
C0	N/A	2500	1.00	0.06	None
C1	N/A	2500	0.30	0.06	
C2	0.40	5000	0.15	0.06	Concrete cover[8]

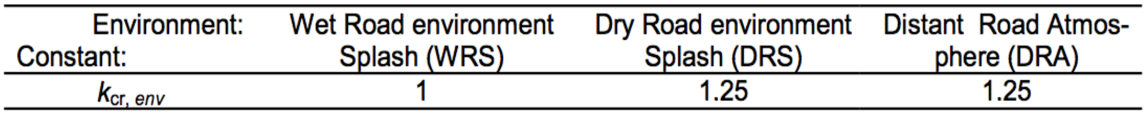
According to the AASHTO LRFD bridge construction specification (AASHTO, 2010), water used in mixing and curing of concrete shall be subject to approval and shall be reasonably clean and free of oil, salt, acid, alkali, sugar, vegetable, or other injurious substances. Mixing water for concrete in which steel is embedded shall not contain a chloride ion concentration in excess of 1,000 ppm or sulfates as SO4 in excess of 1,300 ppm. And admixtures containing chloride ion in excess of one percent by weight (mass) of the admixture shall not be used in reinforced concrete. Admixtures in excess of 0.1 percent shall not be used in prestressed concrete.

An approach similar to that made by Frederiksen (1997) is used in order to make semi-objective estimates for the threshold concentrations. The values could be obtained by:

$$C_{cr} = k_{cr,env} \times \exp(-1.5 \times eqv({}^{W}/{}_{C})_{cr})\ [\%mass\ binder] \qquad (5.28)$$

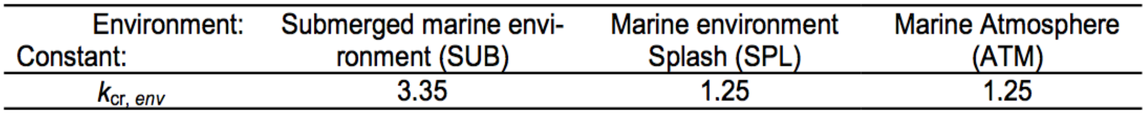
The $k_{cr,env}$ is arbitrary to represent the environment factor, as shown in Table 5-3 through 5-5. A suggested design values for threshold levels for black steel could be derived.

Table 5-3 The constant $k_{cr,env}$ for the road environment (Frederiksen, 1997)



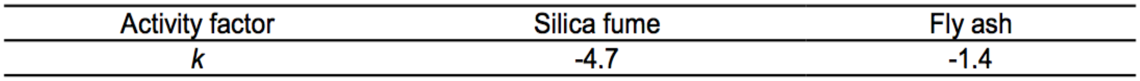
Environment: Constant:	Wet Road environment Splash (WRS)	Dry Road environment Splash (DRS)	Distant Road Atmosphere (DRA)
$k_{cr,\,env}$	1	1.25	1.25

Table 5-4 The constant $k_{cr,env}$ for the marine environment (Frederiksen, 1997)



Environment: Constant:	Submerged marine environment (SUB)	Marine environment Splash (SPL)	Marine Atmosphere (ATM)
$k_{cr,\,env}$	3.35	1.25	1.25

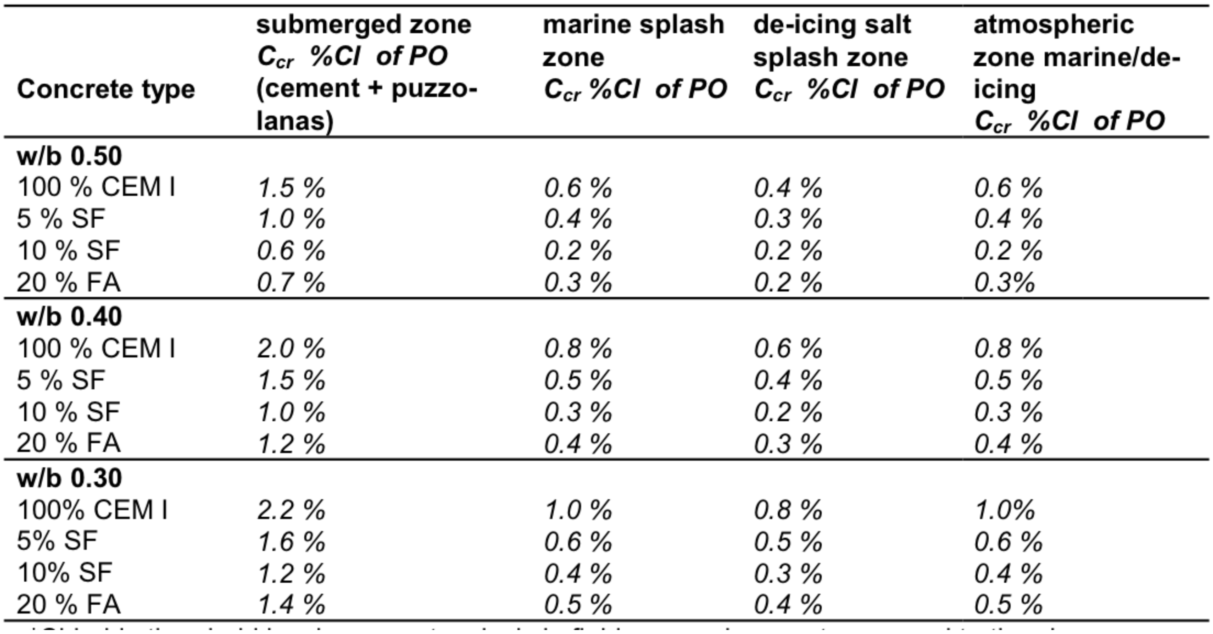
Table 5-5 The activity factors for corrosion initiation in the road environment to be used when calculating the $eqv({}^{W}/{}_{C})_{cr}$ (Frederiksen, 1997)



Activity factor	Silica fume	Fly ash
k	-4.7	-1.4

The Suggested design values for chloride threshold levels (black steel) in various Nordic exposure zones is expressed in Table 5-6. These values are only suitable for crack free concrete with a maximum crack width of 0.1 mm and a minimum cover of 25mm. (Frederiksen, 1997)

Table 5-6 Suggested design chloride threshold level (Frederiksen, 1997)



Concrete type	submerged zone C_{cr} %Cl of PO (cement + puzzolanas)	marine splash zone C_{cr} %Cl of PO	de-icing salt splash zone C_{cr} %Cl of PO	atmospheric zone marine/de-icing C_{cr} %Cl of PO
w/b 0.50				
100 % CEM I	*1.5 %*	*0.6 %*	*0.4 %*	*0.6 %*
5 % SF	*1.0 %*	*0.4 %*	*0.3 %*	*0.4 %*
10 % SF	*0.6 %*	*0.2 %*	*0.2 %*	*0.2 %*
20 % FA	*0.7 %*	*0.3 %*	*0.2 %*	*0.3%*
w/b 0.40				
100 % CEM I	*2.0 %*	*0.8 %*	*0.6 %*	*0.8 %*
5 % SF	*1.5 %*	*0.5 %*	*0.4 %*	*0.5 %*
10 % SF	*1.0 %*	*0.3 %*	*0.2 %*	*0.3 %*
20 % FA	*1.2 %*	*0.4 %*	*0.3 %*	*0.4 %*
w/b 0.30				
100% CEM I	*2.2 %*	*1.0 %*	*0.8 %*	*1.0%*
5% SF	*1.6 %*	*0.6 %*	*0.5 %*	*0.6 %*
10% SF	*1.2 %*	*0.4 %*	*0.3 %*	*0.4 %*
20 % FA	*1.4 %*	*0.5 %*	*0.4 %*	*0.5 %*

A more detailed model was proposed by Fagerlund (2011).

$$C_{cr} = 0.125 \cdot \alpha \cdot a \cdot \left(\frac{K \cdot k}{\frac{w_0}{c} - 0.19 \cdot \alpha} \right)^b + 3.55 \cdot K \cdot k \cdot \frac{\frac{w_0}{c} - 0.39 \cdot \alpha}{\frac{w_0}{c} - 0.19 \cdot \alpha} \tag{5.29}$$

The threshold value of total chloride as weight of binder is determined by four parameters, in which, a and b are the coefficients represents the isotherm of chloride binding capacity governed by the following equation:

$$C_{bound} = a \cdot {C_{free}}^b \tag{5.30}$$

Where,

a, b = coefficients

C_{bound}=Bound chloride (mg/g)

C_{free}= Free Chloride (mole/L)

α= Degree of hydration

k= amount of water soluble alkali in cement (mole/kg cement)

For the typical isotherm shown in Figure 5-16, a=13.5 and b=0.41.

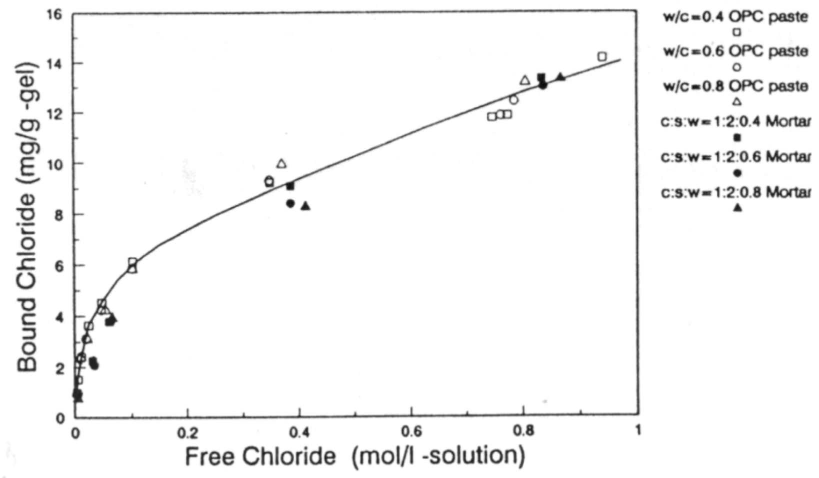


Figure 5-16 Relation between free and bound chloride in OPC (Tang, 1996)

5.3 Effectiveness of the Preventive Mainenance Masures

5.3.1 Sealers and Membrane

The deterioration model involves parameters other than diffusion coefficient and chloride threshold value. It also requires a maximum surface chloride concentration, *Cs*, and the time taken to reach that maximum, t_{max}, based on the type of structure, its geographic location, and exposure.

The surface concentration starts at 0 after construction and it increases along with the time due to the accumulation of chlorides. Once the maximum concentration is reached, the surface concentration remains relatively stable as shown in Figure 5-17.

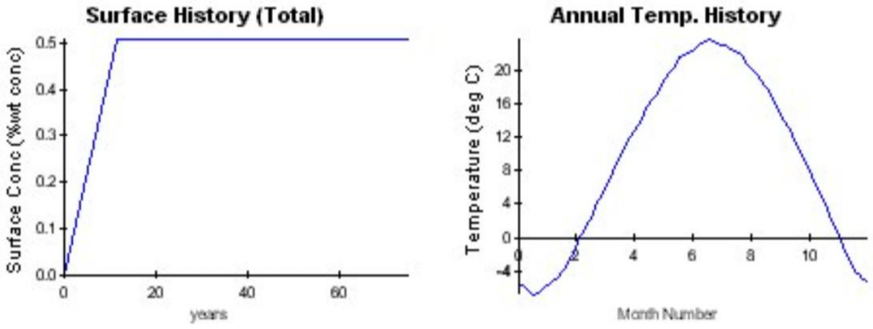


Figure 5-17 Example of Surface Concentration profile and Environmental Temperature (Life-365, 2014)

According to the research done by Ehlen, The membrane and the sealer are mainly affecting the surface concentration of the chloride ions. Figure 5-18 shows the effects of membranes and sealers on the chloride surface concentration. Membranes start with an efficiency of 100 percent, which deteriorates over the lifetime of the membrane. This means that the rate of build-up starts at zero and increases linearly to the same rate as that for an unprotected concrete at the end of the service life of the membranes. Sealers are dealt with in the same manner except the lifetime is considered shorter and it requires reapplication more frequently. The initial effectiveness of the sealer is usually 80%-90% based on Appendix A.

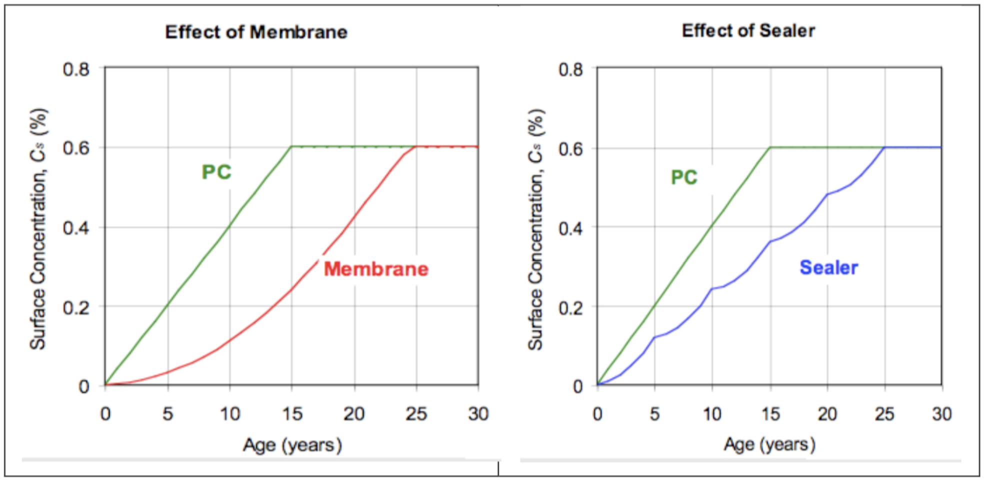


Figure 5-18 Effects of Membranes and Sealers (Life-365, 2014)

5.3.2 Overlays

The application of overlays can affect not only the diffusion coefficient, but also the concrete cover depth. The overlays can be used with or without the membrane. It is the most practical repair or rehabilitation method for an excessively spalled bridge deck. The presence of the overlay can be integrated into the model by adding a layer of material with a certain diffusion coefficient.

5.3.3 Cathodic Protection Systems

The cathodic protection system will affect the chloride threshold value since it applies current that prevents the corrosion process. The current distribution to the reinforcing bar is critical for evaluating the effectiveness of the system. The temperature, moisture in the concrete, concrete conductivity, and condition of the anode are the key factors that affect the performance of a cathodic protection system. The impact of a proper installed cathodic protection system on the numerical deterioration model needs further investigation.

5.3.4 Corrosion inhibitors

Corrosion inhibitors can be added into the concrete mix during the construction phase. The presence of the corrosion inhibitor will increase the chloride threshold dramatically. The effect of Calcium Nitrite Inhibitor on the chloride threshold varies with dose as shown in Table 5-7.

Table 5-7 Effects of CNI on Chloride Threshold

CNI Dose		Threshold, C_t (% wt. conc.)
litres/m^3	gal/cy	
0	0	0.05
10	2	0.15
15	3	0.24
20	4	0.32
25	5	0.37
30	6	0.40

5.4 Conclustion

The lack of knowledge about the long-term performance of concrete and the severity of environmental impacts has caused serious problems. The existing structures are experiencing higher deterioration rate and lasting for shorter service life due to different causes of deterioration of reinforced concrete structures such as corrosion, freezing and thawing action, carbonation and alkali-silica reaction, in which corrosion is the most dominating factor.

Chloride induced corrosion is the most common root cause for concrete bridge deterioration. Once the chloride content reach a certain threshold value, the passive layer formed on the surface of reinforcement rebar will dissolve and corrosion initiates. Due to the manner of chloride transportation mechanism, diffusion is used to model the long-term chloride ingression in our research.

The chloride diffusion process can be determined by knowing the value of diffusion coefficient. The equation is modified by introducing terms counts for the impact caused by the freeze and thaw damage based on the model proposed by Ehlen.

$$\mathrm{D(t)} = \mathrm{D_{ref}} \cdot f(t) \cdot f(RH) \cdot f(FT)$$

It is important to perform test and record the concrete properties in order to get a reference value for chloride diffusion coefficient and the damage index. The dynamic elastic modulus of concrete should also be tested.

If achieved chloride diffusion coefficient is used, the value should be multiplied by a factor since the achieved chloride diffusion coefficient is significantly smaller than found from laboratory test.

The typical values of diffusion coefficient for normal concrete were reported between 10^{-12} m^2/s and 10^{-11} m^2/s. If silica fume, fly ash or other alternative cementitious materials are used, the diffusion coefficient may reduce to 10^{-13} m^2/s.

Chloride diffusion coefficient is impacted by the freeze and thaw cycles dramatically. The value may increase to 3 times of the original chloride diffusion coefficient after 75 freeze and thaw cycles. Furthermore, once the freeze and thaw cycles exceeds 300 to 500, the concrete may experience weight loss lager than 5%. The average annual freeze and thaw cycle for New york city, Syracuse, and Buffalo are 39, 61 and 59 times respectively. Therefore, for unprotected bridge component, the structure are prone to freeze and thaw damage and the bridge may deteriorate more rapidly due to the combined effect of chloride induced corrosion and freeze and thaw.

The chloride threshold valued could be estimated by the equation proposed by Fagerlund. For simplification purpose, a value of 0.05% of the weight of the concrete can be used for ordinary Portland cement with black steel. The used of stainless steel will increase the chloride threshold value so high that the structure will be corrosion free under exposure condition for rural bridge in New York state for 100 years. However, due to the high initial cost, it is unlikely to use stainless steel widely as an alternative to black steel.

5.5 Reference

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Chapter 6 Selection of Preventive Maintenance Mearsures

6.1 Introduction

During the operation of concrete structures, myriad of options could be selected to prevent or reduce deterioration. The preventive maintenance measures could be classified into these four categories:

1. Creation of barriers to exposure
2. Prevention of the penetration of deleterious substances
3. Extraction of deleterious substances
4. Removal and replacement of deteriorated parts.

In detail, these options could be carried out by the detailed treatment described in Table 6-1. (Wang, 2010)

Table 6-1 Detailed classification of Preventive Maintenance Measures

1. Creation of exposure barriers:
• Moisture barriers, such as water proofing membranes. • Protective coatings for additional protection: e.g. epoxy (non-breathable moisture barrier), polyesters, acrylics (which allow water diffusion), polyurethane, bitumen, copolymer, and anti-carbonation coating (acrylic materials) • Surface preparation of concrete and reinforcement.
2. Preventing the penetration of deleterious substances:
• Polymer impregnation, such as percolating into concrete substance
3. Extraction of deleterious substances
• Cathodic protection that migrates chloride ions from the steel surface towards an anode. • Chloride extraction, by which chlorides are transported out of the concrete to an anode surface. • Re-alkalization by applying an external anode to the concrete surface (with the steel reinforcement inside the concrete acting as the cathode) and an electrolytic paste (comprising sprayed cellulosic fiber in a solution of potassium and sodium carbonate); the electrolyte moves into the concrete, increasing alkalinity.
4. Removal and replacement of deteriorated parts of structures
• Patch repair systems, such as the renewal and/or preservation of the passivity of steel reinforcement and the restoration of structural integrity by applying mortar or concrete to areas where deterioration occurs; materials for patch repair can be cementitious or epoxy-based, or comprise similar resinous materials (e.g. polymer concrete and polymer-modified concrete) • Concrete removal: e.g. the removal of damaged or deteriorated areas

The technology of bridge rehabilitation is constantly improving as researchers develop new rehabilitation methods that are more specific to particular bridge element; and new rehabilitation materials that are more efficient and having a longer life span. Since the use of preventive maintenance measures starts while the bridge components are still at relatively good conditions, no major rehabilitation works are needed. These preservation measures involve a range of specific actions and activities. The development of a preservation plan includes individual actions as well as grouping of actions that are required from economic perspective, and based on the condition of the concrete component and the exposure condition of that bridge element.

Other than annually washing of the concrete bridge to clean the accumulated residuals due to the use of deicing agents and to clean the bridge, different bridge maintenance and preservation actions can be used based on the bridge element type. For concrete bridge decks, sealers, waterproof membranes and overlays are the most commonly used methods to prevent or postpone the initiation of corrosion caused problems. For superstructure, crack sealing and concrete sealing are the most cost effective alternatives because of their performance on reducing and preventing chloride contamination. For substructure, due to the harsh environment with the splash zone, extreme protective system such as cathodic protection system might be useful. The recommended preventive maintenance actions and the result of the treatment are summarized in Table 6-2.

Table 6-2 Preventive Maintenance Actions

Action	Result of Treatment
Washing	Chloride Remove
Waterproofing sealant	Resist water and chloride penetration
Crack Sealing	Fill crack voids to resist water and chloride penetration
Overlays	Waterproofing, sacrificial wearing surface
Cathodic Protection System	Sacrificial elements for chloride resistance
Electrochemical Chloride Extraction	Chloride Remove
Re-alkalization	Restore the alkalinity of the concrete

6.1.1 Concrete Deck Sealers (Rahim, 2006)

6.1.1.1 Penetrating Sealants

The primary objective of a surface treatment sealer is to prevent capillary action at the surface, thus preventing the ingress of water and chloride ions into the concrete deck (Rahim, 2006). Application of surface sealer can be used for both new and older decks that have not been critically contaminated with chlorides. The effectiveness of a sealer is evaluated according to its ability to reduce ingress of chlorides into the concrete. The sealer must be able to penetrate the concrete to a depth sufficient to avoid corrosion under traffic, and last long enough so that the number of application is minimized.

Penetrants can be divided into two categories: water-repellent and pore blocking. Water-repellent refers to those materials that penetrate concrete pores to some degree and coat pore wall that it may contain from penetrating concrete pores, but allows gases and vapors to transmit through. Pore blockers are sealers of sufficiently low viscosity that allow the sealers to penetrate the concrete pores and seal them while leaving little or no measurable coating on the exterior surface of concrete (Cady, 1994).

The desirable penetration depth is about 6 mm with a minimum of about 3 mm. The quality of the concrete has a major effect on the penetration depth, where it may reach a higher value with poor quality concrete.

The service life of concrete sealer is governed by three major factors: sealer materials properties, service condition related to the durability, and the chloride diffusion related factors. It was reported that the best penetrating sealers appeared to provide protection for about three years while epoxy-based surface sealers appeared generally ineffective after one year. Other field investigation shows that the service live for solvent-based epoxy, water-based epoxy and Saline to be 10, 8 and 7 years, respectively. Based on the diffusion characteristics, chloride exposure condition, and sealer characteristics, the extended service life for decks treated with the aforementioned sealers under exposure conditions can be determined. Figure 6-1 shows the application of surface sealers on concrete bridge decks.

Figure 6-1 Application of Concrete Sealer (Johnson, 2009)

Sealing cracks in concrete decks is a routine part of bridge maintenance, but practices and products vary significantly. Research suggests that properly preparing the deck surface before the application of all types of sealant improves their effectiveness. A large scatter in the available data and the varying effectiveness of each of the considered sealants relative to a particular application prevent the identification of a single sealant that will work best in all situations. (Johnson, 2009) Some of the research team's significant conclusions and recommendations based on the overall trends were:

- Among the solvent-based sealers, saline products typically outperform siloxane products.
- Water-based products are not suitable for reapplication.
- Solvent-based products typically outperform water-based products.
- High solids content in the sealer typically improves performance. Researchers found that the sealant S40Si comes closest to fitting the above criteria.

For deck sealants, AASHTO T259 and ASTM C642 are commonly used to judge the acceptability of a given product. The NCHRP 244 Series II test is also widely used to quantify sealant performance.

6.1.1.2 Crack Sealers

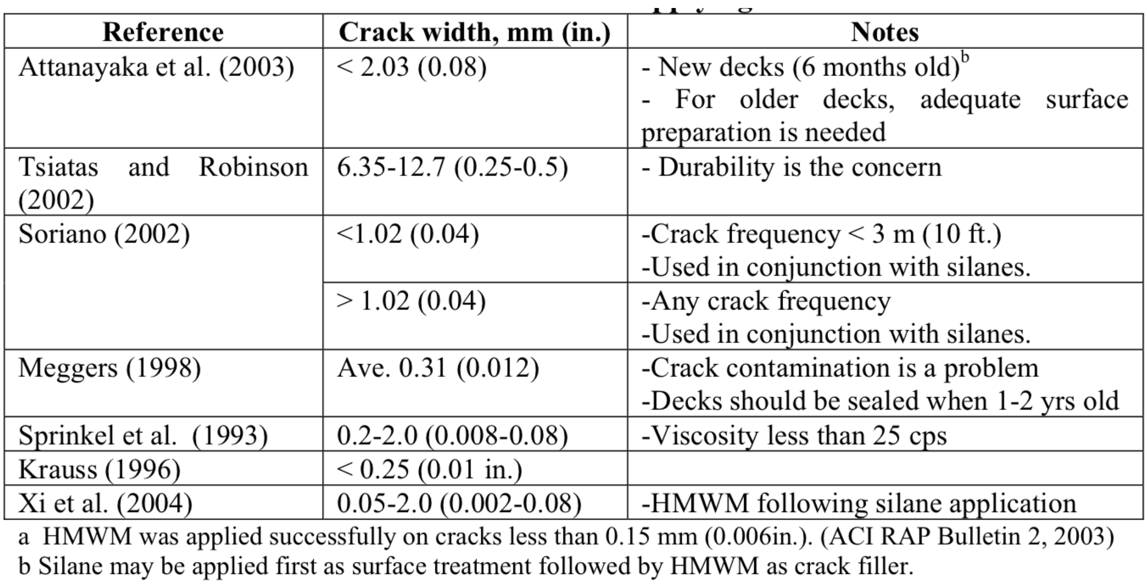
High Molecular Weight Methacrylate (HMWM), epoxy-based, and urethane-based are the most commonly used types of crack sealing materials. These materials are low viscosity materials that depend on gravity in filling the cracks. These Gravity-fill crack sealers consist of two or more low-viscosity liquid monomer or polymer components that can be mixed and poured directly over a cracked surface.

The crack sealer's ability to penetrate narrow cracks, seal large cracks effectively, and withstand freeze-thaw is important to the rehabilitation system. In this measurement, HMWM has a better performance compared with other materials. However, the polyurethane sealer has the advantages of fast curing, no-odor characteristic and ease of application makes it appropriate for hasty repairs and for small cracked areas where leaking may be a problem and sealing the underside of the crack is not practical.

The sealer property in the cracks is not affected by time, in general, however, surface abrasion and weathering may remove the resin from the surface after 3 to 4 years.

Different crack width criteria were found in the literature. These different criteria along with the references are listed in the Table 6-3.

Table 6-3 Crack Width When Applying HMWM (Rahim, 2006)



Reference	Crack width, mm (in.)	Notes
Attanayaka et al. (2003)	< 2.03 (0.08)	- New decks (6 months old)[b] - For older decks, adequate surface preparation is needed
Tsiatas and Robinson (2002)	6.35-12.7 (0.25-0.5)	- Durability is the concern
Soriano (2002)	<1.02 (0.04)	-Crack frequency < 3 m (10 ft.) -Used in conjunction with silanes.
	> 1.02 (0.04)	-Any crack frequency -Used in conjunction with silanes.
Meggers (1998)	Ave. 0.31 (0.012)	-Crack contamination is a problem -Decks should be sealed when 1-2 yrs old
Sprinkel et al. (1993)	0.2-2.0 (0.008-0.08)	-Viscosity less than 25 cps
Krauss (1996)	< 0.25 (0.01 in.)	
Xi et al. (2004)	0.05-2.0 (0.002-0.08)	-HMWM following silane application

a HMWM was applied successfully on cracks less than 0.15 mm (0.006in.). (ACI RAP Bulletin 2, 2003)
b Silane may be applied first as surface treatment followed by HMWM as crack filler.

The temperature also has a critical effect on the quality of the crack sealing activity. Some researchers recommend a temperature range from 4 to 38℃, while others suggest a minimum required temperature of 10℃ during the application of resin based sealers.

One of the most important steps in deck/crack sealing is the preparation of the surface or cracks to be sealed. The sealing will only be as good as the surface/crack preparation, regardless of the nature, sophistication, or type of the sealing material. Many surface and crack preparation methods have been included in the literature. These methods include; grinding, pressurized water, power broom, sand blasting, and forced air among others. The concrete must be at least 28 days old; surface must be clean, dry and free of curing compounds and pore blocking contaminants. For cleaning and preparing individual cracks, the ACI E-707 recommends beginning with wire brushes and wheels, followed by high pressure, oil-free compressed air to remove dust from the surface of the crack. If the crack surface is packed solid with dirt and/or debris, they must be removed by routing the crack surface and following up with compressed air to remove fines. Cracks may be air blasted or vacuumed to ensure they are free of water and dirt/dust.

6.1.1.3 Corrosion Inhibiting Impregnation Coating

Sika Company developed a corrosion inhibiting impregnation coating, Sika FerroGard 903 for hardened concrete surfaces. It is designed to penetrate the surface and then to diffuse in vapor or liquid form to the steel reinforcing bars embedded in the concrete. It forms a protective layer on the steel surface which inhibits corrosion caused by the presence of chlorides as well as by carbonation of concrete. It contains a combination of amino alcohols, and organic and inorganic inhibitors that protects both the anodic and cathodic parts of the corrosion cell and depositing a physical barrier in the form of a protective layer on the surface of the steel reinforcement. It can delay the onset of corrosion and reduce the rate of corrosion by 65% versus control specimen after 1 year, which is approved in both laboratory and field analysis. (Sika, 2011)

The recommended total application rate is 100 sq. ft./ gal with a unit cost approximately $1.3 per square feet. The estimated service life is about 5 years if it is applied along with water repellent penetrating sealer. The total application unit cost for the hybrid system is about $2.5 per square feet. In order to achieve the maximum effectiveness, it is highly recommended to apply the hybrid system prior to other preventive maintenance activities,.

6.1.2 Overlays (Cuelho, 2013)

6.1.2.1 Polyester Concrete Overlay

Polyester resins, as dense, impermeable polymers when cured, have been used by many DOTs as the binder material for bridge deck overlays. Polyesters are the products of chemical reactions between difunctional alcohols and anhydrides of dibasic organic acids. Initiators and promoters are often used at the time of application. Polyester resin overlay material can cure rapidly over a large range of temperatures (from 40°F to 100°F), so that traffic can be allowed over the surface within an hour after application. Polyester resin can be formulated to match the elasticity and thermal expansion of existing concrete. Polyester concrete, when used as thin overlay or thicker partial-depth overlay, requires no modifications to curbs, manholes or catch basins and can be applied on bridges without significantly increasing the dead load. Multiple coats can be applied, giving a final thickness of at least 6mm (1/4 inch). Well-constructed and maintained polymer concrete overlays can provide protection for the concrete slab against chloride intrusion for up to 25 years. Caltrans currently uses polyester concrete and Portland cement concrete (PCC) on bridge decks. Polyester concrete, as a major overlay material in both partial and complete overlays, has been used in California for the last 10 to 15 years.

6.1.2.2 PCC Overlay

PCC overlays exhibit all of the beneficial behaviors that lead to using a concrete deck in the first place, such as a strong, durable driving surface. However, Portland cement concrete needs to be applied in relatively thick layers (one to four inches), thus adding considerable dead load to the structure. This increase in thickness also causes alignment issues with the roadway if the top surface of the existing deck is not removed.

PCC overlays can be constructed using conventional PCC, latex-modified PCC, polyester-modified PCC, low-slump dense PCC, fast-setting PCC, with some variations involving steel fiber or silica fume, or high-range water-reducing mixtures. The primary function of PCC overlays is to replace deteriorated concrete or asphalt wearing surfaces with a durable and low- permeability material. Performance of PCC overlays has been found to vary considerably from one region to another, depending on design/construction factors and local climates.

6.1.2.3 Asphalt Concrete Overlay

Asphalt concrete overlays on bridge decks, or more broadly the asphalt-based overlays, include two general sub-categories: combined systems consisting of a waterproofing membrane overlaid with one or two courses of asphalt concrete (1.5 to 2 inches thick each), and asphalt concrete overlays without a waterproofing membrane. Both systems can add significant dead load to bridge structures. The total thickness of a combined system is usually between 2 and 4 inches. The economics of asphalt mixtures makes asphalt-based overlays a desirable option, which also provides good ride quality. There are many types of membranes including hot applied rubberized membranes, sheet membranes and liquid-applied polymer membranes. The membranes should be capable of bonding to concrete, bridging cracks, waterproofing, and bonding to AC overlays without being affected by temperatures as high as 150°C from hot repaving.

6.1.3 Electrochemical treatment

All electrochemical maintenance methods have principles and practical details in common. The main differences are the amount of current flowing through the concrete and the duration of the treatment.

A general set-up that is valid for all electrochemical methods is that by means of an external conductor, called the anode, a direct current is directed through the concrete to the reinforcement, which thereby is made to act as the cathode in an electrochemical cell. The final result of the current flow is to mitigate or stop the corrosion by re-passivation of the rebars due to polarization of the reinforcement to a more negative potential, or by removing the aggressive ions (chloride) from the pores of the concrete, or by reinstating the alkalinity of the pore solution. (REHABCON, 2000)

6.1.3.1 Electrochemical Chloride Extraction (ECE)

Electrochemical Chloride Extraction (ECE) is a similar application to CPS, but on a shorter-term scale. This process uses a sacrificial steel mesh anode surrounding concrete elements. An electrical charge is then induced to draw chloride ions toward the anode and out of the concrete. A case study conducted by the Michigan Department of Transportation estimated that the service life of a concrete bridge pier treated with ECE coupled with the application of a penetrating sealer was increased by ten years. This study saw significant improvements in the half-cell potential measurements and found that the reinforcing steel was re-passified, reducing the ability of remaining chlorides to initiate corrosion.

While ECE is not necessarily a preventive maintenance activity in itself, this method can be coupled with preventive maintenance activities like sealing to significantly increase the service life of concrete bridge elements.

Alkali-silica reaction (ASR) can result from an interaction between reactive aggregates and the alkali present in the concrete. Severe ASR results in slow expansion of the inner parts of a structure that induces cracking of the outer zones. During chloride extraction, hydroxyl ions are formed around the reinforcing steel, locally increasing the pH and sodium and potassium ions are enriched around the steel. These changes might stimulate ASR.

For practical application of Chloride Extraction on structures with potentially sensitive aggregate, it is recommended to evaluate the potential reactivity of the aggregate by considering its geological source and practical experience in structures containing the aggregate and study the concrete using microscopy for actual presence of reactive material

If deleterious expansion is found in representative tests, ECE should not be recommended.

6.1.3.2 Cathodic Protection Systems (CPS)

Cathodic Protection Systems (CPS) can be installed in concrete bridge decks and underwater substructure elements. The CPS process is based on the creation of an electrical flow that reverses the ion exchange when chloride ion-containing water reaches the reinforcing steel. However, because this is an electrical process, it is very difficult to apply CPS to elements that utilize epoxy coated reinforcing steel. It has been shown that CPS can halt reinforcing steel corrosion with proper CPS design. Figure 6-2 shows a typical setup for cathodic protection system.

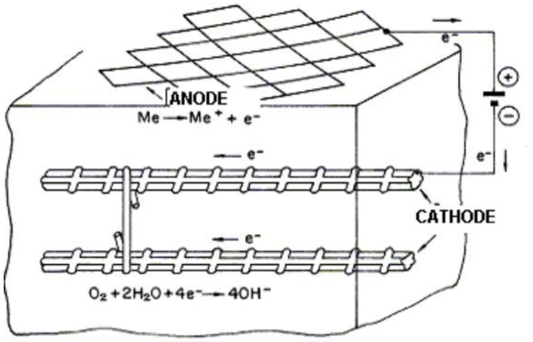


Figure 6-2 Cathodic Protection Process (REHABCON, 2000)

Cathodic protection of reinforcing steel is a technique that has been demonstrated to be successful in appropriate applications in providing cost effective long-term corrosion control for steel in concrete. The current circulation results in a reduction of the chloride content on the rebar surface or in the reduction of the chloride ingress into the concrete. Thus, this method is particularly suited when the corrosion is caused by chloride contamination since it will result in an increase in the OH/Cl ratio and restore the passivation phenomena.

6.1.3.3 Electrochemical Re-alkalization

Electrochemical re-alkalization is a method of preventing steel reinforcement corrosion induced by carbonation. The pH of the concrete around the steel increases, and the passivating properties of the concrete pore solution are restored. The technique involves passing a current through the concrete to the reinforcement by means of an externally applied anode mesh that is temporally attached to the concrete surface and embedded in an electrolyte reservoir, as shown in Figure 6-3. In the traditional application, a paste of sprayed cellulose with 1 molar solution of sodium carbonate is used as the electrolyte covering the concrete surface. The electrode outside the concrete (anode) and the reinforcement inside acting as cathode are connected to a direct current source. During treatment, the electrolyte is transported into the carbonated concrete. The dominant transport mechanism may vary, but electro-osmosis and migration of ions are the two main contributors. Simultaneously, electrolysis at the steel surface produces a very alkaline environment.

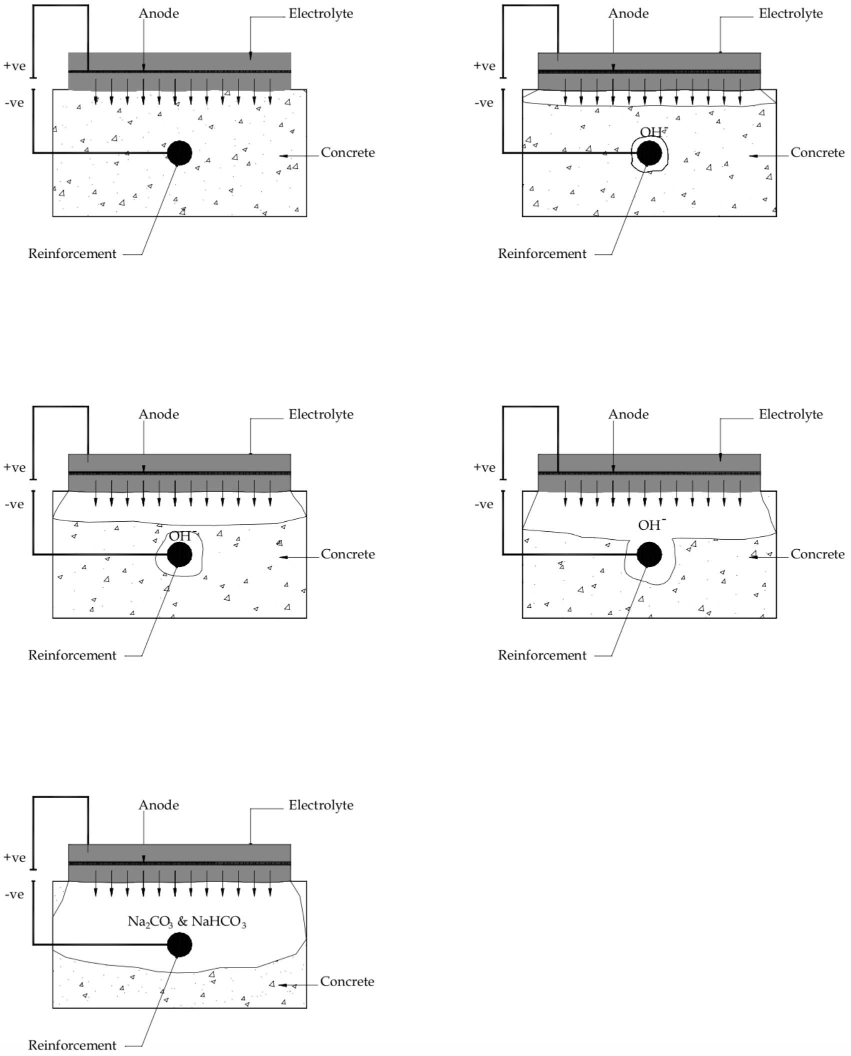


Figure 6-3 Re-alkalization Process (REHABCON, 2000)

Re-alkalization of concrete is a non-permanent electrochemical treatment, so, the treatment finishes after 1-2 weeks applying a current of around 0.8 to 2 A/m^2. After the treatment the anode must be removed, and the original concrete surface is left unchanged.

Re-alkalization is an electrochemical method that can be effective in stopping corrosion of reinforcement in carbonated concrete. It has a track record of over ten years. Its advantage is that carbonated concrete can be left in place; there is no need to break out and replace it. However, it has its limitation in two ways: the increased content of alkali ions and the production of hydroxyl ions might theoretically cause accelerated ASR for a concrete with aggregate; and the current that flows during electrochemical treatment increases the concentrations of alkali metal and hydroxyl ions in the pore water at the steel/concrete interface. This theoretically might change the pore structure and the bond strength.

6.1.4 Insulation of the Bridge deck for ice prevention

One method to reduce salt usage is to provide insulation against frost and ice formation. This concept was used to insulate the underside of a bridge deck and the subgrade of highway pavements and airfield runways. The main objectives were to reduce heat loss from the surface and prevent ice and frost formation, and to decrease the number of freeze-thaw cycles and salt usage.

Although the insulation system decreases the freeze-thaw cycles dramatically, the temperature-stabilizing effect of the urethane increased the potential for icing during most of the winter. In addition, evidence that the insulation had trapped water suggested that the concrete deck and structural steel could be adversely affected. The advantage of adopting this system is considered to be secondary, compared to the effect on the potential for icing. Therefore, it is not recommended without an internal heating system installed simultaneously.

6.1.5 Treatment for ASR

Besides the usage of sealers preventing the moisture ingression process, there are several other chemical treatments that could be used to deal with the ASR affected concrete.

Lithium nitrate can be applied either topically or by vacuum treatment, showed benefits in terms of reducing expansion or cracking when applied to bridge elements. However, it is not so effective due to the lack of penetration of the ions. Electrochemical methods were found to be effective in significantly increasing the depth of lithium penetration when applied to bridge columns. Lithium was driven all the way to the reinforcing steel (depth of 50 mm or 2 in.) in a concentration estimated to be sufficient to suppress ASR-induced expansion. However, the migration of other alkali ions (specifically sodium and potassium) leading to increased alkali concentration in the vicinity of the reinforcing steel (used as a cathode during treatment) was also observed (e.g., reinforced concrete columns, Houston field trial). This will be accompanied by an increase in hydroxyl ions (and pH) as a result of the cathodic reaction and to maintain electro-neutrality of the concrete pore solution. This phenomenon could potentially exacerbate ASR-induced expansion and cracking in this region.

Other than the methods used above, the ASR caused crack could be restrained by CFRP. The existence of transverse wrapping of CFRP could improve the compressive strength of the concrete and providing external compression that can help to prevent the growth of crack. CFRP could also serve as a barrier that insulates the inner concrete from the outside environment.

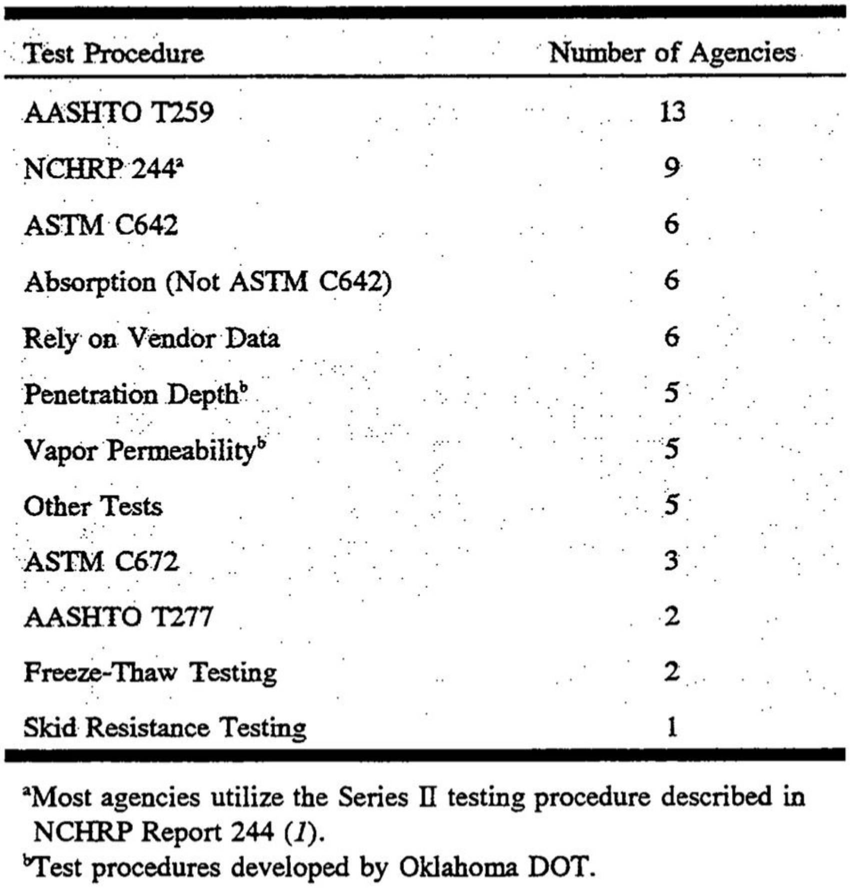
6.2 Evaluation of the Effectiveness of the Preventive Maintenance Measures

Since the chloride induced corrosion is the most significant cause for concrete deterioration for concrete bridge components, especially for concrete slabs, it is essential to determine if the concrete element has ongoing corrosion or to determine an estimated time-to-corrosion for a significant portion of the element. Factors that are considered include exposure conditions (surface chloride content), the diffusion coefficient, the concrete cover, and the type of reinforcing steel. The following discussion reviews methods to use during the field investigation for collecting the information required to support the time-to-corrosion determination.

6.2.1 Effectiveness of sealers

Several tests could be used for the evaluations of effectiveness of sealers. The commonly used tests are summarized in Table 6-4. As shown in the table, the most widely used method is AASHTO T259, "Resistance of Concrete to Chloride Ion Penetration". The second most widely used test is Series II of NCHRP 244. The absorption, voids in hardened concrete, the average penetration of sealers and vapor permeability are also selected to evaluate the effectiveness of the sealers as well. Tests for deicer scaling resistance (ASTM C672), freeze-thaw resistance (ASTM C666), rapid chloride permeability (AASHTO T277), and skid number (AASHTO T278) are used by a relatively small number of agencies (Whiting, 1992).

Table 6-4 Test Procedure for Evaluation of the Effectiveness of Sealers (Whiting, 1992)



Test Procedure	Number of Agencies
AASHTO T259	13
NCHRP 244[a]	9
ASTM C642	6
Absorption (Not ASTM C642)	6
Rely on Vendor Data	6
Penetration Depth[b]	5
Vapor Permeability[b]	5
Other Tests	5
ASTM C672	3
AASHTO T277	2
Freeze-Thaw Testing	2
Skid Resistance Testing	1

[a] Most agencies utilize the Series II testing procedure described in NCHRP Report 244 (*1*).

[b] Test procedures developed by Oklahoma DOT.

Field tests of penetrating sealers have used core or drill samples to determine the extent severity of chloride ion penetration. The effectiveness is evaluated based on its influence on the surface chloride concentration build up rate and the chloride diffusion coefficient of the concrete. Another approach can be selected by flooding the treated sections with water. If the concrete rapidly absorbs the water, it is considered that the sealer is ineffective; on the contrary, if the water bead up, then the sealer is judged to be effective. This test is qualitative and has significant subjective aspect.

6.2.2 Effectiveness of membranes

Although tests have indicated that membranes might be able to provide up to 50 years of service before corrosion of reinforcement becomes a problem, states responding to the 1994 NCHRP survey anticipated the service live of their membrane systems to be between 10 and 30 years. Most of the longest anticipated service lives came from states in New England, where membranes have been used for the longest time and contractors have many years of experience at membrane installation.

Some highway departments have experienced issues with debonding of membranes and stripping of asphalt overlays, requiring the removal and replacement of the membrane in ten years or less, depending on both the traffic and the environment. Other membranes deteriorate after about 15 years of service due to traffic stresses and age embrittlement. One of the major causes of debonding and stripping of the asphalt overlay is water that is trapped on top of the membrane. Freezing and thawing, along with pressure from traffic weaken the bottom part of the asphalt overlay and the bond between the overlay and the membrane.

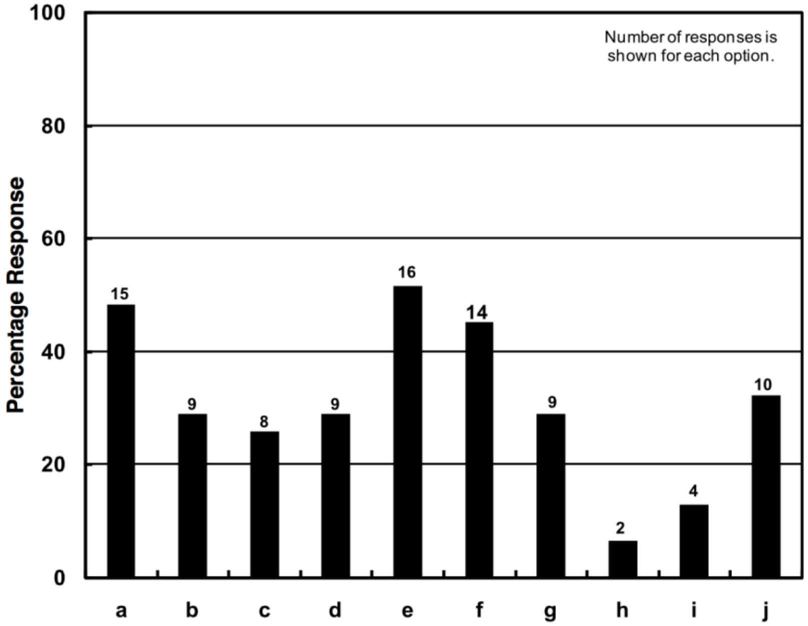


Figure 6-4 Selection Criteria (Russell, 2012)

Figure 6-4 shows the reasons for selecting a particular membrane system. In which: a. Cost; b. Speed of installation; c. Staged construction options; d. Surface preparation; e. Track record of previous installations; f. Desired service life; g. Availability; h. Coordination requirements; i. Product support; j. Others.

Various methods for evaluation of waterproofing systems are available in the field, including visual inspection, electrical methods, embedded devices, physical sampling, ultrasonic methods, and air permeability methods. (Russell, 2012)

Visual inspection of the asphalt surface may offer some indications of the condition of the membrane. Wide cracks, radial crack patterns, wet spots, and gaps at curbs or barriers may be signs of potential problems.

Virginia's standard specifications require that the water-proofing effectiveness of the membrane system be deter- mined in accordance with Virginia Test Method T 39. In this test method, the electrical resistance between the top surface of the asphalt and the top mat of reinforcement is determined using an ohmmeter. The specification requires a minimum resistance of 500,000 ohms. Areas having a lower resistance are to be repaired if determined by the engineer to be detrimental to the effectiveness of the system. If more than 30% of the deck area is determined to be detrimental to the effectiveness of the system, the membrane is to be replaced.

Ground penetrating radar, chain drag, and infrared thermography can also be used to examine the presence of voids, delamination and other defects.

Leak testing is also necessary. This test involves ponding the deck top surface with water and checking underneath for leaks. This method may not be feasible on some bridge decks owing to longitudinal or transverse slopes.

Bond testing is required for spray applied waterproofing membranes according to the special specification of NYSDOT. The testing of substrate is required after the primer has been applied, and after the membrane has been installed. A minimum adhesion of 145 psi is needed for Portland cement concrete.

6.2.3 Effectiveness of Cathodic Protection Systems (FHWA, 2001)

The standard test methods and generally accepted industry practices are used in evaluating the long-term performance of the CP systems. Some of the tests methods and practices may encounter problems for data interpretation under specific circumstances. The visual survey and delamination survey should be performed in order to detect any signs of corrosion-induced deterioration of the concrete component. Other than these common tests that can also serve for other preventive maintenance systems, the following tests should be performed to determine the effectiveness of the CP systems.

6.2.3.1 Electrical Continuity Testing

Direct Current method, alternating current measurement, and half-cell technique are the three test methods can be used to perform electrical continuity testing, among which the direct current method is the most commonly used one.

In the DC method, resistance and the voltage difference between two embedded metals are measured. The method requires measurement of resistance in both directions in order to overcome the impact of currents flowing between the embedded metals. When the DC technique is used directly on exposed reinforcement, as is the practice during construction of the CP system or condition evaluation of the structure, the maximum allowable resistance in each direction is 1 ohm (some in the industry use a criterion of 3 ohms). When the technique is used in an installed CP system, and the wires connected to the system grounds and the grounds of instruments such as the reference cells, current probes, null probes are used for the resistance measurement, the maximum allowable resistance in the each direction is dependent on the run of the wires.

The test result might be affected if the structure that are cathodically protected, are experiencing very active corrosion, have the presence of stray currents, or have some internal source of current.

6.2.3.2 The AC Resistance Measurements

The AC resistance measurements between anode and system ground were used to obtain circuit resistance of the system and to detect the presence of shorts between the anode and the embedded steel protected by the CP system.

The AC resistance measurements between reference cells and their respective grounds were used to identify malfunctioning reference cells. When the AC circuit resistance is very high, it may be indicative of the failure of certain types of reference cells. High circuit resistance in conjunction with no response by a reference cell to changes in CP current indicates a malfunctioning reference electrode. High resistance also makes the reference cell prone to noise pick-up and makes the measurement of accurate potentials somewhat difficult.

6.2.3.3 Chloride Ion Content Analysis

The chloride ion content analysis is also needed for evaluating the effectiveness of a CP system. The test should follow the procedure described by AASHTO T-260. The results of the chloride ion concentration at the steel depth are presented in order to give an estimation of the corrosively of the environment in which the CP system operating.

6.2.4 Other Tests

In addition to the tests used for evaluating the long-term performance of preventive maintenance measures, there are some other tests available to examine the effectiveness of short-term preservation actions. The chloride content test could be used to evaluate the performance of Electrochemical Chloride Extraction. In the same manner, the carbonation test could be used to test the effectiveness of the electrochemical re-alkalization applications.

6.2.4 Summary

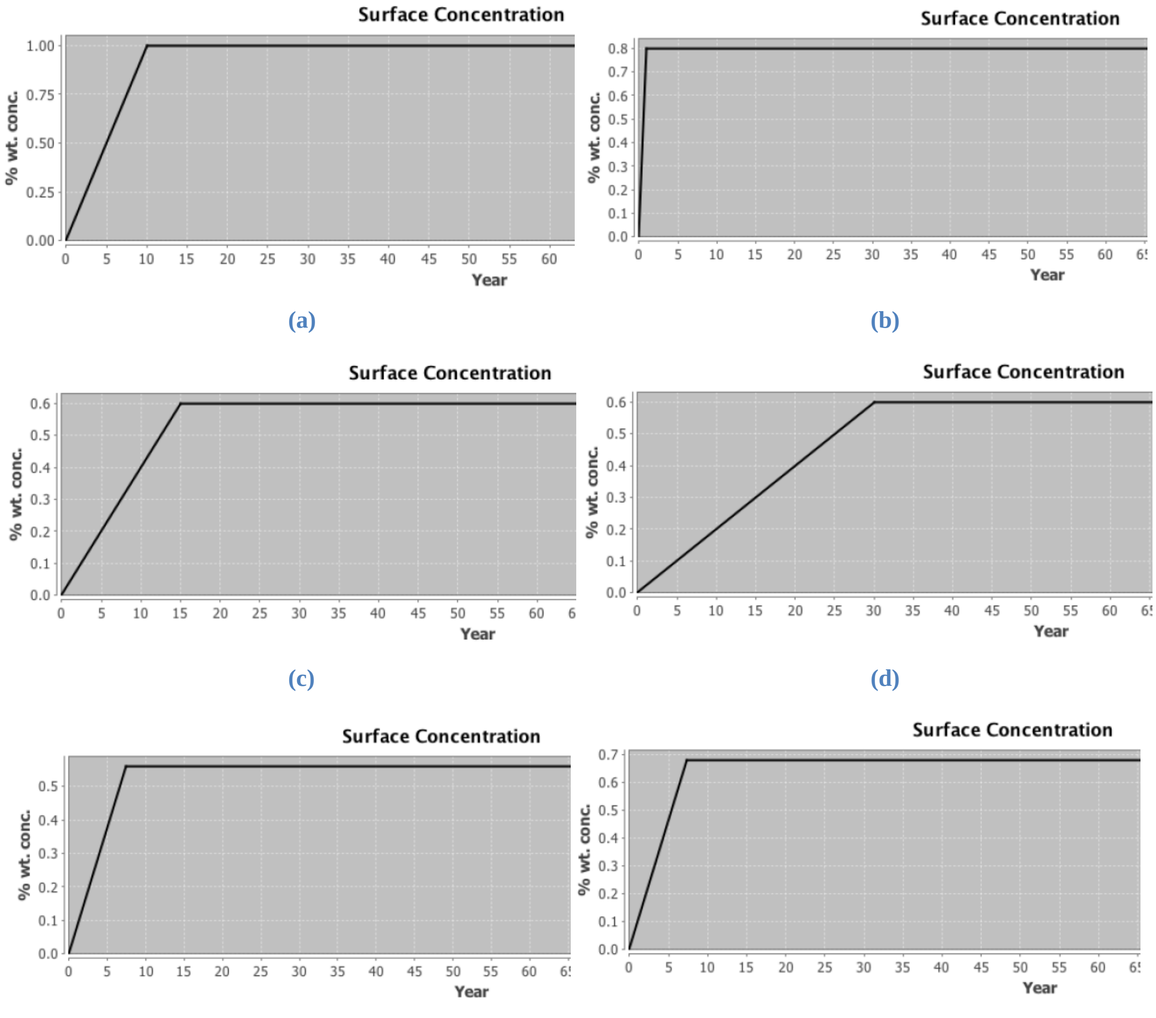
Evaluating the effectiveness of the preventive maintenance measure involves a set of nondestructive tests. The most commonly used tests methods are the chloride content tests and the tests that could give an indication for chloride diffusion coefficient. Since chloride induced corrosion is the most dominating cause for concrete bridge deterioration, the chloride profile and the chloride diffusion coefficient plays a significant role on the reliability of the preservation systems. The tests should be performed frequently enough in order to monitor the effectiveness of the preservation actions and the data should be integrated for future planning and estimation of the life span of these preventive maintenance systems.

6.3 Recommended Actions for Different Concrete Bridge Systems and Conditions

Among all the parameters that govern the chloride-induced corrosion for concrete bridge system, the most significant ones are the exposure conditions, concrete cover depth, chloride threshold values and the chloride diffusion coefficient. These parameters are discussed in detail in order to give a reasonable recommendation for preservation actions for a management strategy that gives a corrosion free service life for concrete bridges.

6.3.1 Exposure Conditions

Once the geological location of the bridge is selected, the exposure condition can be determined based on the historical data. The exposure condition is mainly affecting the maximum chloride concentration and the build-up rate of the chloride concentration, as shown in Figure 6-5.



(a)

(b)

(c)

(d)

(e)

(f)

Figure 6-5 Surface Concentration (a. Marine Spay Zone; b. Marine Tidal Zone; c. Within 800m of the Ocean; d. Within 1500m of the Ocean; e. Rural Highway Bridge; f. Urban Highway Bridge)

As discussed in the previous chapter, the presence of sealers and membranes has a significant role on affecting the surface concentration of the chloride ions as shown in Figure 6-6.

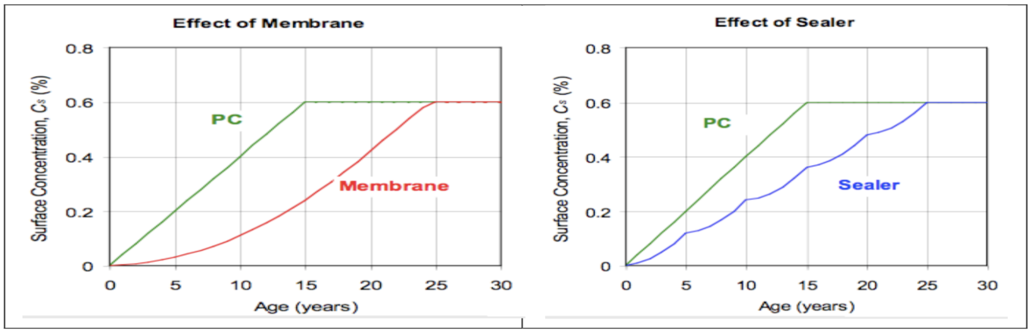


Figure 6-6 Effects of Membranes and Sealers (Life-365, 2014)

The application of sealer and membranes will elongate the service life of the structure. The magnitude of the service life elongation calculated by Life-365 gives reasonable outcomes that comply with the values reported by the NCHRP 14-23. According to the NCHRP 14-23 report, the life extension for bridge deck using sealers is approximately 2 years and the value is service life extension due to actions of patch and overlay is about 15 years. Table 6-5 shows the service life elongation by using sealers and membranes and it agrees with the conclusions given in the NCHRP 14-23 report.

Table 6-5 Service life report

Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
HPC		0.3	Silica Fume (5%);			Black Steel
OPC		0.42				Black Steel
HPC with Sealer		0.3	Silica Fume (5%);		Sealer	Black Steel
OPC with Sealer		0.42			Sealer	Black Steel
HPC with Membrane		0.3	Silica Fume (5%);		Membrane	Black Steel
OPC with Membrane		0.42			Membrane	Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
HPC	2.00E-12 m*m/sec	0.2	0.05 % wt. conc.	29.4 yrs	6 yrs	**35.4 yrs**
OPC	8.87E-12 m*m/sec	0.2	0.05 % wt. conc.	8.3 yrs	6 yrs	**14.3 yrs**
HPC with Sealer	2.00E-12 m*m/sec	0.2	0.05 % wt. conc.	34.2 yrs	6 yrs	**40.2 yrs**
OPC with Sealer	8.87E-12 m*m/sec	0.2	0.05 % wt. conc.	11.7 yrs	6 yrs	**17.7 yrs**
HPC with Membrane	2.00E-12 m*m/sec	0.2	0.05 % wt. conc.	39.3 yrs	6 yrs	**45.3 yrs**
OPC with Membrane	8.87E-12 m*m/sec	0.2	0.05 % wt. conc.	16.6 yrs	6 yrs	**22.6 yrs**

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

In this analysis, the concrete cover depth is set as a constant along with the chloride threshold value. Two set of chloride diffusion coefficient is selected for comparison. The variable is the usage of membrane and sealers that alters the surface chloride concentration. As we could see that service life is affected if the sealers and membranes are applied. The sealer increased the service life for HPC and OPC for 4.8 and 3.4 years, respectively. While the membranes elongate the service life for HPC and OPC for 9.9 and 8.3 years, respectively. Therefore, the sealer and the membrane are more effective for high performance concrete that has a lower diffusion coefficient.

In the real world, the accrual service life extension might be longer for structures treated with membranes. The main reason is membrane is always applied with a sacrificial overlay that can be removed after certain years. The Overlay can serve as an addition concrete cover that takes time for chloride to ingress into the concrete structure beneath them.

6.3.2 Concrete Cover Depth

The concrete cover is another vital parameter that dictates the service life of a concrete bridge element. The AASHTO LRFD Bridge Specifications require a minimum concrete cover of 100 mm (4 in.) for concrete that is in direct exposure to sea water, 75 mm (3 in.) for coastal concrete, and 65 mm (2.5 in.) for concrete that is exposed to deicing salts or on deck surfaces that are subject to tire stud or chain wear. The specification allows the minimum concrete cover over epoxy-coated reinforcement to be reduced to 40 mm (1.5 in.). (Kepler, 2000)

Based on the simulation of Life-365, if the high performance concrete is selected with a diffusion coefficient of $2 \times 10^{-12} m^2/s$ for a rural highway concrete bridge in New York State, the time for corrosion initiation varies with the concrete cover according to Table 6-6.

Table 6-6 Effect of Different Concrete Cover

Concrete Cover (in)	Time to corrosion initiation (year)
1.5	13.3
1.8	17.8
2	23.5
2.2	26.7
2.4	33.7
3	52.5

As could be seen in Table 6-6, along with the increase of concrete cover depth, the time to corrosion initiation is significantly deleyed. The impact of concrete cover is strongly associating with the chloride diffusion coefficient. If the chloride diffusion coefficient is relatively small, the impact of change in concrete cover will be more obvious.

6.3.3 Chloride Diffusion Coefficient

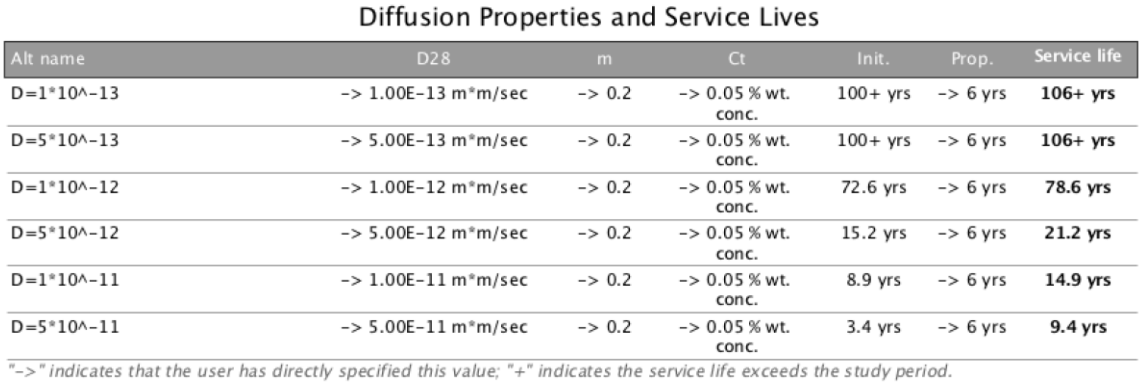
Chloride diffusion coefficient is one of the most important parameters when service life estimation of a concrete element is based on chloride-induced corrosion. According to previous works, the chloride diffusion coefficient varies in the magnitude between $1 \times$

$10^{-11} m^2/s$ to $1 \times 10^{-13} m^2/s$. The smallest value could be achieved by using high performance concrete containing silica fume, slag and fly ash or other additives, small water cement ratio, good curing and better quality control during construction. The larger value, on the contrary, may be the result of poor design of the concrete mix, poor curing conditions or the effect of extensive cracks due to other deterioration mechanisms.

In 1992, a 35 year-old bridge in Windsor, Connecticut was demolished to make way for a wider bridge. As longitudinally sawed sections of the bridge slab were observed during demolition, it became evident that the performance of the slab in terms of corrosion resistance had been excellent. No reinforcement corrosion was found in any of the demolition debris or in any of the cores taken from the concrete. The good performance of the bridge was attributed to a low w/c ratio, high cement content, well graded fine aggregate, a high dosage of water reducer/retarder, good consolidation, minimum cracking, and a bituminous wearing course that shielded the concrete from some chloride ions. The concrete cover over the reinforcement on the bridge ranged from 32 to 50 mm (1.25 to 2 in.), with most areas having very close to 38 mm (1.5 in.) of cover. (Kepler, 2000)

The estimated service life for a concrete slab on a rural highway concrete bridge in New York State with a clear cover of 2.5 in has been investigated and the results are summarized in Table 6-7.

Table 6-7 Service Life Estimation for Different Diffusion Coefficient



Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
D=1*10^-13	-> 1.00E-13 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	100+ yrs	-> 6 yrs	**106+ yrs**
D=5*10^-13	-> 5.00E-13 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	100+ yrs	-> 6 yrs	**106+ yrs**
D=1*10^-12	-> 1.00E-12 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	72.6 yrs	-> 6 yrs	**78.6 yrs**
D=5*10^-12	-> 5.00E-12 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	15.2 yrs	-> 6 yrs	**21.2 yrs**
D=1*10^-11	-> 1.00E-11 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	8.9 yrs	-> 6 yrs	**14.9 yrs**
D=5*10^-11	-> 5.00E-11 m*m/sec	-> 0.2	-> 0.05 % wt. conc.	3.4 yrs	-> 6 yrs	**9.4 yrs**

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

High performance concrete has very small diffusion coefficient value. When HPC is used, the structure may serve its full service life without corrosion. However, once the diffusion coefficient increased to a relatively high value due to deterioration, the remaining service life is shortened dramatically and the structure may not survive for a decade.

It is important to note here that the diffusion coefficient is not a constant over the period of time. The diffusion coefficient is not only affected by the decaying factor that represents the maturity of the concrete, it also influenced by freeze and thaw cycles and cracks over the surface of the structure, which eventually increase the diffusion coefficient with time. The typical diffusion coefficient in the cracks are $2 \times 10^{-9} m^2/s$ and $4 \times 10^{-9} m^2/s$ for a $50 \mu m$ wide crack and a $500 \mu m$ wide crack. The chloride ions may reach the reinforcement sooner due to the presence of these cracks.

6.3.4 Chloride Threshold Value

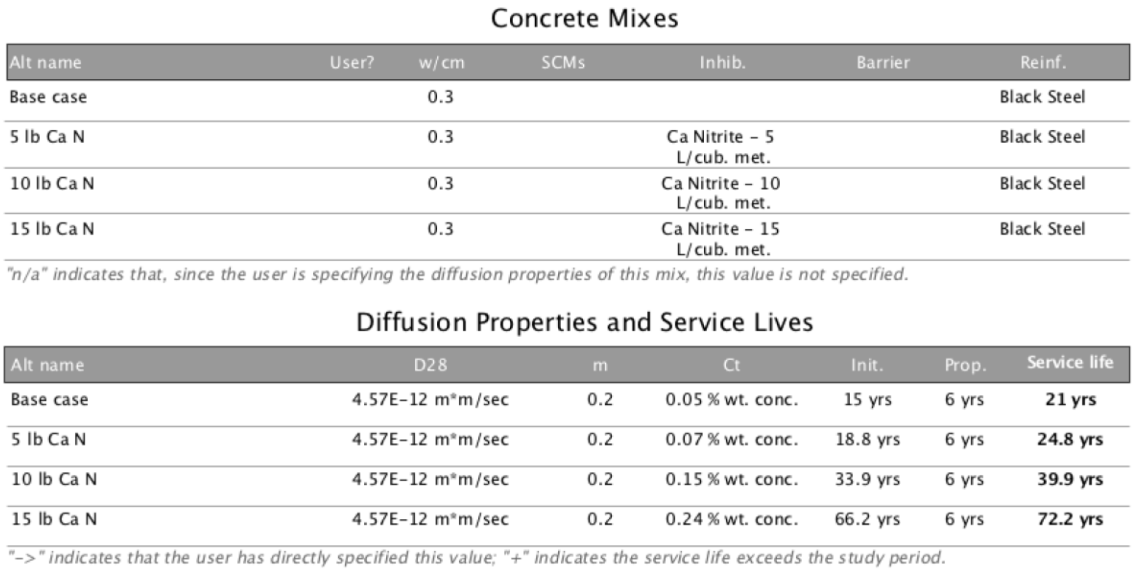
The chloride threshold value is mainly affected by the use of corrosion inhibitors in the construction phase. The use of Calcium Nitrite Inhibitor (CNI) is assumed to have no effects on the diffusion coefficient, or the diffusion decay coefficient. The effect of CNI on the chloride threshold varies with dose as shown in the Table 6-8.

Table 6-8 Effect of CNI

CNI Dose		**Threshold, C_t (% wt. conc.)**
litres/m^3	**gal/cy**	
0	0	0.05
10	2	0.15
15	3	0.24
20	4	0.32
25	5	0.37
30	6	0.40

The impact of the CNI on the service life of a bridge deck component in concrete slab on a rural highway concrete bridge in New York State with a clear cover of 60 mm was investigated and the results are listed in Table 6-9.

Table 6-9 Impact of Corrosion Inhibitors



Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Base case		0.3				Black Steel
5 lb Ca N		0.3		Ca Nitrite – 5 L/cub. met.		Black Steel
10 lb Ca N		0.3		Ca Nitrite – 10 L/cub. met.		Black Steel
15 lb Ca N		0.3		Ca Nitrite – 15 L/cub. met.		Black Steel

"n/a" indicates that, since the user is specifying the diffusion properties of this mix, this value is not specified.

Diffusion Properties and Service Lives

Alt name	D28	m	Ct	Init.	Prop.	Service life
Base case	4.57E-12 m*m/sec	0.2	0.05 % wt. conc.	15 yrs	6 yrs	**21 yrs**
5 lb Ca N	4.57E-12 m*m/sec	0.2	0.07 % wt. conc.	18.8 yrs	6 yrs	**24.8 yrs**
10 lb Ca N	4.57E-12 m*m/sec	0.2	0.15 % wt. conc.	33.9 yrs	6 yrs	**39.9 yrs**
15 lb Ca N	4.57E-12 m*m/sec	0.2	0.24 % wt. conc.	66.2 yrs	6 yrs	**72.2 yrs**

"->" indicates that the user has directly specified this value; "+" indicates the service life exceeds the study period.

As we could see that the usage of CNI has a significant impact on the service life of the bridge component and the extension of the service life is mainly affected by the dosage used for the concrete mix. One method of using CNI is integrating the inhibitors into the concrete mix. Since the corrosion inhibitor is added into the concrete mix, it is hardly considered as a preventive maintenance method. However, it should be recommended for all new constructions to use corrosion inhibitors in order to achieve a longer service life. Another application involves surface treatment using corrosion inhibitors. However, research has shown that the performance of surface treatment using corrosion inhibitors is not so well since the penetrating depth can hardly reach the surface of the reinforcement.

6.4 Selection of Preventive Maintenance Measures for Bridge Decks

For concrete bridge decks, the concrete cover depth is relatively small. However, bridge decks are subjected to sever exposure conditions due to the use of deicing salt and direct contact with the traffic loads. The decision procedure should comply with the following chart shown in Figure 6-7. This chart could be used as a field guide for preventive maintenance of concrete bridges.

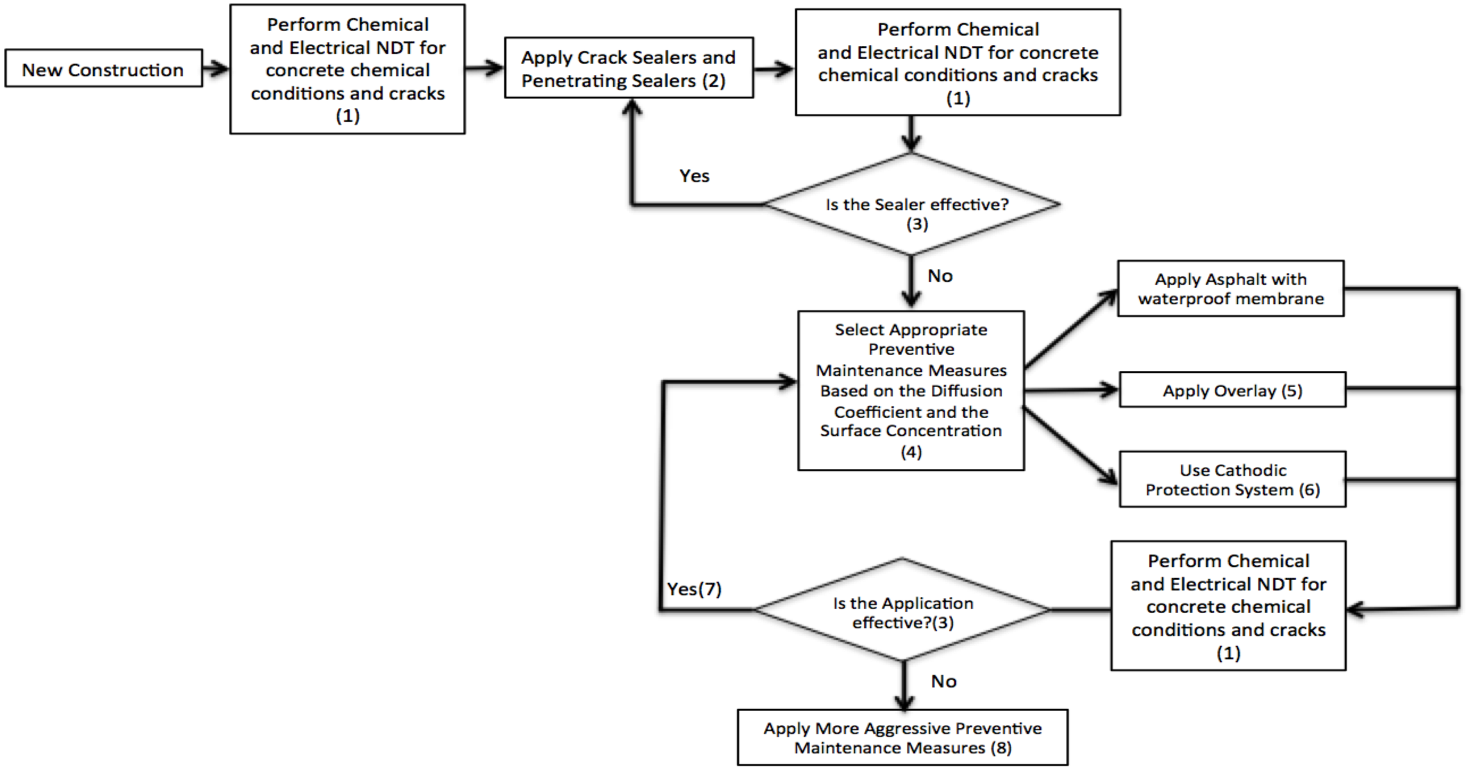


Figure 6-7 Flowchart for Preventive Maintenance Selection

(1). Chemical and Electrical NDTs should be performed after completion of a new bridge construction.
   a. Chemical NDT test for chloride content measurement, chloride resistance test, chloride diffusion coefficient measurement should be done in order to give a base value for the initial chloride content in the concrete and the reference chloride diffusion coefficient. Other parameters such as the compressive strength of the concrete, the air content, permeability and dynamic modulus of elasticity should also be recorded.
   b. Electrical NDTs are mainly used to detect and evaluate the presence and intensity of the cracks right after construction. In addition to visual inspection with naked eye or with the aid of crack detection microscope, technologies like acoustic emission test and ultrasonic testing could be used as a rapid detection method.

(2). Concrete sealers could be classified into two broad categories: surface sealers and crack sealers.
   a. Surface sealers contain silicates, siliconates, Saline, and siloxanes. The later three produce the same end product, a hydrophobic silica gel. The basic difference between these products is their molecule size. Since saline has the smaller molecule size compared to the other two, it gives saline the advantage of penetrating deeper into concrete than siloxanes and silicones. It has been reported that saline and siloxane categories are the most effective sealers. However, the effectiveness and service life of the sealer varies dramatically with different manufactures.
   b. Widely used crack sealers are high molecular weight methacrylate (HMWM), epoxy-based sealers and urethane-based sealers. Among which, HMWM and low viscosity epoxy are used favorably. The HMWM is approved nationwide that can be used for cracks narrower than 0.0625 inch. For cracks narrower than 0.04 inch at a drying age of one month, it can be used following an application of saline.

(3). Check the effectiveness of the preventive maintenance measures contains two main tasks. The first one is to check the deterioration of the concrete component and the second one is to check the effectiveness and soundness of the preventive maintenance methods.
   a. For concrete deterioration, the same non-destructive test mentioned in (1) should be applied every two years to keep track of the current concrete properties.
   b. For the effectiveness of sealers, chloride ion intrusion testing “AASHTO T259”, rapid permeability testing “ASTM C 1202”, depth of penetration “OHD L-40”, NCHRP 244 Series II tests should be performed. Ponding water on the surface of the treated concrete will also give a qualitative indication of the effectiveness of the sealers. In addition, tests for deicer scaling resistance (ASTM C672), freeze-thaw resistance (ASTM C666), rapid chloride permeability (AASHTO T277), and skid number (AASHTO T278) is also used to evaluate the effectiveness of the sealers by some agency.
   c. Various methods could be used to evaluate waterproofing, including visual inspection, electrical methods, embedded devices, physical sampling, ultrasonic methods, and air permeability methods. Ground penetrating radar, chain drag, and infrared thermography can also be used to examine the presence of voids, delamination and other defects. Leak testing is also necessary.

d. The purpose of concrete overlays is to create a low permeable concrete later over the conventional concrete on bridge decks. The effectiveness of the overlays could be monitored by adopting the same non-destructive tests used in (1). Also, delamination tests should be performed as well.

e. Applying electrical continuity testing, AC resistance measurements and chloride ion content analysis could monitor the effectiveness of cathodic protection systems.

(4). In the field, the concrete cover depth could be easily obtained by using cover meter testing. And for a given concrete mixture and steel type, the chloride concentration threshold for the corrosion initiation could be also estimated. The presence of ongoing corrosion could be also detected by NDT methods like half-cell potential testing. By applying the nondestructive test methods discussed in the previous chapters, the surface concentration and the diffusion coefficient could be derived. And based on these parameters, the recommendation for a preventive maintenance action selection chart is given in Table 6-10.

Table 6-10 Recommended Actions for Given Diffusion Coefficient and Surface Concentration

(%wt. cont.) Cs Dc (m^2/s)	<0.2	0.2-0.4	0.4-0.6	0.6-0.8	0.8-1
5*10^-13	Sealers	Sealers	Sealers	Sealers	Sealers
1*10^-12	Sealers	Sealers/ Asphalt with Waterproof Membrane	Asphalt with Waterproof Membrane	Asphalt with Waterproof Membrane	Asphalt with Waterproof Membrane
5*10^-12	Asphalt with Waterproof Membrane	Overlays	Overlays	Overlays	Overlays
1*10^-11	Overlays	Overlays	Overlays	Overlays	Overlays
5*10^-11	Overlays	Overlays	Cathodic Protection System	Cathodic Protection System	Cathodic Protection System

In which, the surface concentration could be estimated by performing the ASTM C1556, “Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion”. Concrete field samples do not directly meet the sample requirements in ASTM C1556, but can still be used to estimate diffusion and surface concentration by “fitting” the data to Fick’s second law of diffusion. Figure 6-8 illustrates how the ASTM method directly estimates the average surface chloride concentration as the intersection of the Fick’s second law-based grey line and the horizontal access, that is, the estimated chloride concentration at depth = 0 (the blue dot in the figure). (Life-365, 2014)

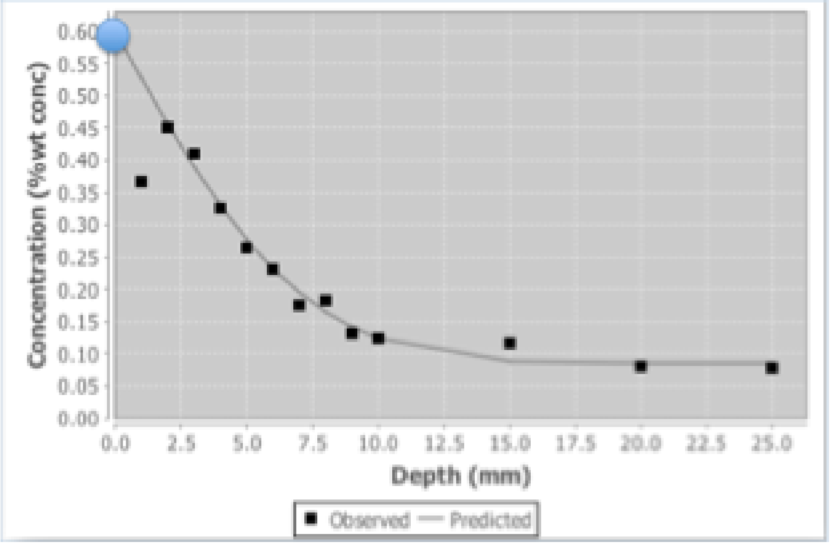


Figure 6-8 ASTM Estimate of Surface Chloride Concentration (Life-365, 2014)

The surface concentration can also be determined using the chloride content analysis. In that case, a model boundary has been set up beneath the actual surface of the concrete. The chloride concentration remains relatively stable at that level compare to the actual surface that experience wash and rain.

The diffusion coefficient could also be determined using NDT tests, e.g. AASHTO T259 “Salt Ponding Test”, bulk diffusion test, AASHTO T277 “rapid chloride permeability test”, rapid migration test, etc.

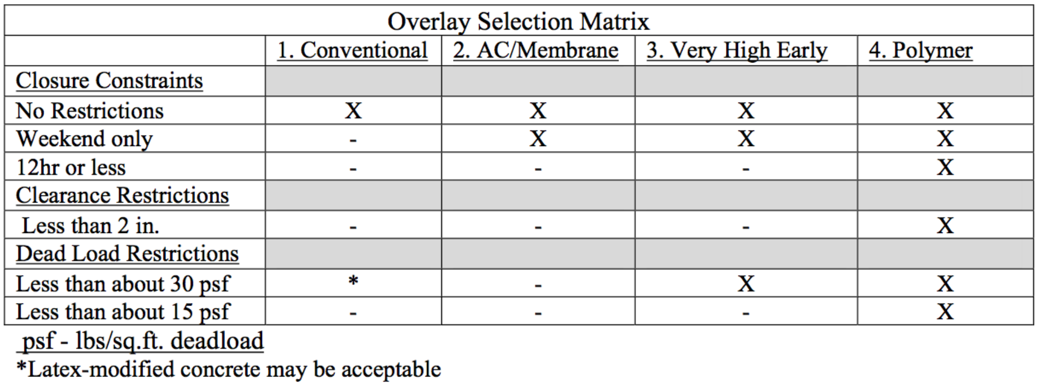
Some other indirect methods could also be used to determine the diffusion coefficient. Permeability of the concrete is related to the concrete resistance to chloride ion transportation. The field test for permeability measurement is fast and only takes 2-3 hours. Even though no direct theoretical relationship has been provided between the output from permeability test and rapid chloride permeability test, there is an empirical correlation between these two tests.

Once the diffusion coefficient and the surface concentration have been determined, the preservation actions could be selected from Table 6-10. After each application, the effectiveness of the preventive maintenance system itself should be monitored by apply proper nondestructive tests. The life span of the applied system is derived through series of test. The concrete properties should be tested for every inspection in order to update the real time transportation properties and to determine the impact of the applied preventive maintenance measures.

(5). Asphalt overlays with waterproof membrane are first considered since the cost is relatively low. However, if the effectiveness is not reaching the desirable state or the restriction for the thickness of the overlays are stated, polymer modified overlays should be used. The selection criteria are listed in Table 6-11 according to the research done by Krauss. (2009)

The most common type of overlay is low-slump dense concrete overlay, silica-fume concrete and latex-modified concrete overlay, thin epoxy overlays and polymer concrete overlays, in which, Polymer concrete overlays are generally used as a temporary repair method on damaged bridge decks.

Table 6-11 Overlay Selection Matrix (Krauss, 2009)



Overlay Selection Matrix				
	1. Conventional	2. AC/Membrane	3. Very High Early	4. Polymer
Closure Constraints				
No Restrictions	X	X	X	X
Weekend only	-	X	X	X
12hr or less	-	-	-	X
Clearance Restrictions				
Less than 2 in.	-	-	-	X
Dead Load Restrictions				
Less than about 30 psf	*	-	X	X
Less than about 15 psf	-	-	-	X

psf - lbs/sq.ft. deadload

*Latex-modified concrete may be acceptable

Some agencies list specific guidelines for selection of rehabilitation methods. While the guidelines are available, none are mandatory and they are not necessarily used to make decisions for every case. Kansas DOT has some general guidelines, it recommends the use of polymer overlay for concrete decks with 3-10% distress, and the use silica fume overlay for 10 - 50% distress, and for >50% distress further inspection of the deck is recommended. Virginia DOT uses very early strength overlays when lanes cannot be closed for long periods. Polymer overlays are used on decks in good condition, and gravity fill polymers are used to fill random shrinkage cracks. Wyoming DOT uses a rigid overlay of silica fume-modified concrete for decks having extensive spalling and cracking, patching if the extent of spalling and delamination is less than a couple hundred square feet, and a crack healer/sealer if the deck displays cracking but not delamination. If a deck needs increased friction over a sealed surface, a polymer thin-bonded overlay may be used. Ontario DOT patches, waterproofs and paves the deck if less than 10% of the deck requires removal, but if more than 10% of the deck requires removal, an overlay is applied, followed by waterproof and then paving with a wearing surface. (Krauss, 2009)

(6). Cathodic protection systems could be classified as impressed current and sacrificial anode based on the source of the current. Impressed current systems are used most often on bridge decks, but there are some impressed current anodes that can be used on bridge substructure members as well. Table 6-12 compares the characteristics of different cathodic protection systems.

Table 6-12 Comparison of Characteristic of Cathodic Protection Systems (J. Kepler, 2000)

Impressed-Current	**Sacrificial Anode**
External power required	Requires no external power
Driving voltage can be varied	Fixed driving voltage
Current can be varied	Limited current
Can be designed for almost any current requirement	Usually used where current requirements are small
Can be used in any level of resistivity	Usually used in low-resistivity electrolytes

(7). If the preventive maintenance methods are effective, the concrete properties will remain relatively stable and the concrete component will deteriorates at a slower rate. However, the remaining service life of the preventive maintenance application should be monitored. Actions need to be taken at the end of the service life of the previously applied preventive maintenance actions. The selection of the new actions should base on the present concrete properties detected from chemical and electrical NDTs. The service life reported from literature for the following systems falls in the range stated in Table 6-13.

Table 6-13 Expected Service Life for different Preventive Maintenance Systems (Krauss, 2009)

Preventive Maintenance Method	Expected Service Life Range (years) [Mean]
Asphalt Overlays with a Waterproof Membrane	3- 40 [12- 19]
Polymer Overlays	1- 35 [9- 18]
High Performance Concrete Overlays	10- 40 [16-29]
Low Slump Concrete Overlays	10- 45 [16-32]
Latex Modified Concrete Overlays	10-50 [14- 29]
ICCP Systems	25- 100
Galvanic Anode Systems	5- 20

(8). In some conditions, such as the exposure condition is extreme due to the extensive use of deicing agents or marine environment, or the concrete condition is relatively poor that the diffusion coefficient is high or the chloride concentration in the concrete is high, more aggressive preventive maintenance strategies should be applied. Cathodic Protection system is always an effective method to prevent corrosion initiation. However, in order to use cathodic protection system with a lower current density, electrical chloride extraction and re-alkalization may be needed. Combined systems such as an integration system involve overlays, sealers and cathodic protections system may be needed if desirable.

6.5 Selection of Preventive Maintenance Measures for Superstructure and Substructure Elements

The preservation strategies and approach to manage bridge superstructure and substructure elements are different from those used for bridge decks. Unlike the bridge decks, bridge abutments and piers are not directly subjected to the tire abrasion and deicing agent, therefore, the crack intensity due to external loading and chemical contamination due to deicing agent may not be an issue for the majority of these elements. The minimum concrete cover depth is also thicker than the requirement for bridge decks since the replacement of bridge superstructure and substructures are costly and time consuming. Thicker concrete cover delays the corrosion initiation by increase the time needed for chloride concentration to reach the threshold value at the surface of the rebar. These elements are also commonly over designed for the designing load in order to accommodate the development of the needs for transportation and leave a margin that tolerates deterioration in certain degree for the imperfection of future maintenance practice. The average deterioration rates of these elements are smaller than bridge decks, which leads to a longer design service life. If preventive maintenance and corrective maintenance are applied as needed on time for the superstructure and substructure elements, it may have an estimated service life of 100 years.

However, if the drainage system is poorly designed or maintained, the surface of bridge beams, piers and pier caps would be directly subjected to harmful compounds from the melting snow and other chemical residuals washed out by precipitation. In that case, the beam end as well as the top of the pier cap may experience accelerated deterioration if the joints are deteriorated without proper maintenance. Also, if the bridge piers are located next to roadways, it is highly possible that the bottom part of the column is covered with snow contaminated with deicing agents, which may leads to higher chloride concentrations compared to the other parts of the column and ultimately premature corrosion initiation. The splash zone of the marine structures also shows excessive corrosion if not enough attentions and actions are assigned to these elements. Furthermore, if the small defects are not treated properly or ignored, the deterioration rate will increase dramatically. For these elements suffering from poor design, delayed maintenance or extreme exposure conditions, extra care should be taken and more aggressive preventive maintenance actions should be adopted.

Similar to other concrete components, the common defects for concrete superstructures and substructures are cracking, spalling, rebar corrosion, delamination, scaling and concrete deterioration. And in order to address these issues, the following methods can be used: remove and replace all chloride-contaminated concrete; reduce the concentration and change the distribution of chloride ions by using electrochemical chloride extraction; stop or slow the ingress of future chloride ions by using a less permeable modified concrete overlay; stop or slow the ingress of future chloride ions by using sealers, membranes, and waterproofing materials; repair cracks to prevent chloride ion contamination; apply barrier coatings on the reinforcing steel in the repair areas; apply corrosion inhibitors in the repair or over the entire concrete element to either interfere with the corrosion process or modify the characteristics of the in-place concrete; and apply a cathodic protection system. Among all strategies and techniques, cathodic protection is the only technology that can directly stop further corrosion, even in the most corrosive environment, if designed, installed, and applied correctly. (Ainge, 2012)

Therefore, surface penetrating sealer with corrosion inhibiting impregnation coating is highly recommended as a preventive maintenance measure. The effectiveness of the sealer should be closely monitored for the purpose of identification of the proper time for reapplication. The sealers and the corrosion inhibiting impregnation coating should be reapplied no longer than every 5 years.

Once cracks are formed, the cause of the cracks should be identified and the maintenance actions should be performed immediately. Based on the form and the state of the crack, different crack sealers, such as cement grout, epoxy resin, polyurethane or acrylic resin, could be selected. In some cases, the surface treatment or overlays may be recommended if there is leakage.

Concrete patching is not recommended if the root cause of the deterioration is not addressed in advance. The patching becomes extremely unreliable and often shows a service life of 1 to 2 years if the cause of deterioration is not eliminated. Debonding is also a problem that results from a lack of adhesion between the patch and the existing members. In general, concrete pathing typically covers up the problem instead of preventing the deterioration from occurring. It can be used as solely a cosmetic repair measure that has to be applied with other preventive maintenance measures.

Among all the preventive maintenance measures, cathodic protection system is the only action that can stop the active corrosion. The application of sacrificial zinc anode system is considered to be effective in arresting corrosion. Its efficiency is depending on the chloride levels within the concrete members. It is the most effective method when placed in a relatively low chloride environment. If the existing chloride levels are too high for the sacrificial zinc anode to be effective, there are two options that could be adopted. The first option: the bridge is used without any major repair being made, and achieving the longest service life possible before the bridge needs to be completely replaced. This may be a more cost effective alternative depending on the age and condition of the bridge. The second option: use of an impressed current cathodic protection system. This system has a rather high initial cost, while it is the most comprehensive corrosion control choice for mitigating the corrosion development. It is a feasible choice if the overall bridge has a long remaining service life while other components are still in a good condition state. Thermal sprayed zinc system is also a good alternative among the cathodic protection systems. However, the application needs special care due to the toxicity that comes from zinc and its potential threats to the surrounding environment.

The least effective maintenance measure is doing nothing. Once the deterioration occurs, the deterioration rates will increase along with the degradation of the bridge element, which means that delayed maintenance is not acceptable for bridge superstructure and substructure.

In general, the recommendations for preserve bridge superstructure and substructure is as follow:

- Seal the surface of the elements using surface penetrating sealer with corrosion inhibiting impregnation coating with a maximum reapplication interval of 5 years.
- Identify the root cause of the deterioration and apply proper actions.
- Maintain a good condition rating for the drainage systems and joints.
- Seal the cracks with crack sealers immediately after they have appeared.
- Apply patching using concrete mix similar to the original concrete after the cause of deterioration has been treated for aesthetics purpose.

- Select the appropriate cathodic protection system for the bridge superstructure and substructure element depending on the chloride concentration and the exposure environment.
- If jackets or FRP wrapping are used for strengthening or repair of the element, cathodic protection system should be used in order to prevent potential corrosion activities in the concrete.

6.6 Recommends for selection of preventive maintenance measure

Recommendations for selecting preventive maintenance measure are as follow:

- For new constructions and renovated concrete bridges, perform thorough Non-destructive tests. The chloride diffusion coefficient of the concrete, chloride surface concentration, carbonation depth, concrete cover depth should be determined. Apply sealers as an initial preventive maintenance action.
- Perform NDT for every inspection interval in order to keep track of the real properties and chemical condition of the concrete, and then apply preventive maintenance systems. If the recommended action remains the same, reapply the preventive maintenance methods based on the tests outcomes for the sealers, membranes or overlays. If the concrete properties changed drastically, switch to a more aggressive preventive maintenance system.
- If the sealer is applied to the concrete, the effectiveness of the sealer should be monitored by performing non-destructive tests, such as AASHTO T259 testing, NCHRP 244 Series II Testing, surface absorption test and electrical resistance test.
- The application of sealer is the cheapest treatment among the preventive maintenance measures discussed above. However, the effectiveness of the sealer is limited and the expected service life of the sealer is only 2-5 years. Therefore, the sealer is only recommended as a preventive maintenance measure that should be applied as a routine maintenance method. The effectiveness of the sealer should also be monitored in order to determine the optimized reapplication rate.
- Asphalt overlays with waterproof membrane are first considered since the cost is relatively low. However, if the effectiveness is not reaching the desirable state or the restriction for the thickness of the overlays are stated, polymer modified overlays should be used. If this treatment is selected as the preventive maintenance method, visual inspection, electrical conductivity test, ground-penetrating radar, leak testing should be used in order to evaluated the soundness and effectiveness of the waterproof membrane system.
- If the overlays are selected, the diffusion coefficient of the overlay should be controlled to a relatively small value, and the chloride profile within the overlay needs to be closely monitored. Once the bottom part of the concrete overlay reaches a chloride concentration between 0.5-0.07% by weight of the concrete, remove and replace the overlay.
- Cathodic Protection system has an initial cost that is much high than the other preventive maintenance methods. The cathodic protection is not the most favorable solution for concrete slabs because of its initial cost. However, if the user cost is very high due to the heavy traffic or the geological location of the bridge, it might be a good solution especially when the exposure condition is extreme. It might also be economical compared to replacement of the whole element. After all, it is a solution that could successfully prevent corrosion initiation.

- The carbonation depth increases at a much lower rate compared to the chloride ingression. Most of the preventive maintenance measures also have an impact on retarding the carbonation ingress. Only if the carbonation front is close to the reinforcement and the chloride content in the concrete is low, the electrochemical re-alkalization should be used to restore the alkalinity of the concrete.
- If cracking due to ASR is present, the service life could be prolonged 2 to 5 year by apply High Molecular Weight Methacrylate treatment for bridge deck. Overlays may also prolong deck life of decks with ASR; however, close monitoring is suggested. (Krauss, 2009)

6.7 Summary

The remaining service life is mostly dependent on the exposure conditions, diffusion coefficient, concrete cover depth and the chloride threshold value. During the design phase, a concrete mix having a low water cement ratio, low permeability, and low diffusion coefficient is favorable. During the construction, good consolidation and moisture curing is helpful to have better quality concrete.

The effectiveness and the impact of the preventive maintenance measure were discussed. Each application of sealer may extend the service life of the concrete structure by 1-2 year. The use of overlays is highly recommended due to its better performance for long-term effectiveness. The cathodic protection is an effective preventive maintenance method, however, it is not recommended unless a cost analysis is conducted or in extreme conditions.

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Chapter 7 Cost Analysis

7.1 Introduction

The cost of any preventive maintenance activities for a concrete bridge system involves the initial cost of the systems, associating with the maintenance, operation, and repair cost within the service life of the structure.

Based on the survey response from the several States DOTs, the expected service life for a typical concrete bridge superstructure and substructure spans from 50 years to 100 years; in contrast, for bridge decks, a shorter service life is expected that only last for 35 to 85 years. For the purpose of economic evaluation, a service life of 100 years is used to compare different preventive maintenance systems and strategies with useful lifetimes in excess of the service life of the component.

Whenever a repair or rehabilitation work is necessary on a bridge, a significant portion of the cost of the activity comes from incidental costs, rather than the actual repair or material costs. Incidental costs include mobilization, traffic control, and repairs and improvements to other parts of the bridge, such as drains, barrier rails, and approaches. (Kepler, 2000)

There is another cost associated with preventive maintenance activities on bridges that is not considered in this economic analysis, which is the user's cost. The user costs are the costs incurred by the traveling public attributable to the application of the bridge preservation actions, which includes time lost due to delays or detours, accidents and other resources used. These costs are site specific and can make the total price considerable higher than the price used in this analysis. However, due to its nature of case sensitivity, this user costs are not included in this analysis.

7.2 Cost of Various Activities

7.2.1 Concrete Deck Sealers

7.2.1.1 Crack sealing

The crack sealing are mainly used to prevent the intrusion of moisture and other harmful compounds through existing cracks. These treatments typically require high quality materials and good preparation. Sealants commonly used by State DOTs are Reactive Methyl Methacrylate (MMA), High molecular weight methacrylate (HMWM) and epoxy-based sealant.

Based on the report done by Oman (2014), the crack sealants used by Minnesota DOT were examined. The cost for MMA resins from different manufactures ranges from $42 to $87 per gallon. While the material cost for Epoxy-based sealant covers a range from $42 to $81 per gallon.

However, since the crack sealant is not needed for the entire surface of the concrete structure, the actual cost of implementing crack sealant is significantly depending on the crack density and the unit price for the application is always measured in linear foot. Based on the NCHRP report 523, the cost is approximately $0.3-$1.5 per linear foot for crack filling and cracks sealing and the cost are slightly higher if it is for a small job.

7.2.1.2 Penetrating Sealers

Due to the effectiveness and the low cost of penetrating sealants, numerous products and systems are available in the market provided by different manufactures. The sealants could be installed with a common low pressure garden sprayer, as well as production field spraying equipment might also be used to improve installation time and application uniformity. Based on the rates reported by Soriano (2001), the price ranges of $0.16 to $0.40 per square feet for the product materials cost. This cost is the materials only, calculated from the unit price of the sealers and the application rate that recommended by the manufacture. According to the survey conducted by Krauss (2009), the cost for apply sealers is approximately $3-$5 per square feet, which includes the surface preparation, materials and application fees, as shown in Table 7-1.

7.2.2 Overlays

Based on the NCHRP report published by Krauss (2009), the cost for commonly used overlays are combined in Table 7-1. The cost and service life both experiencing a wide range and a large standard deviation. The cost and expected service life in Table 7-1 are the mean values presented in Krauss's report.

Table 7-1 Rehabilitation Method Summaries (Krauss, 2009)

Rehabilitation Method	**Expected Service Life Range (Years)**	**Cost Range ($/sq. ft.)**
Rigid Overlays		
High Performance Concrete Overlays	16-29	17-25
Low Slump Concrete Overlays	16-32	13-19
Latex Modified Concrete Overlays	14-29	18-39
Asphalt-Based Overlays		
Asphalt Overlays with a Membrane	12-19	3.1-7.6
Miscellaneous Asphalt Overlays	8-15	1-3
Others		
Polymer Overlays	9-18	10-17
Crack Repair	19-33	NA
Penetrating Sealers	4-10	3-5
Deck replacement	27-32	43-53

Other researchers also reported estimated values for different measures. According to the 2008 road report, asphalt overlays up to 2 inch thick cost $2.2 per square feet, and it goes up to $3.9 per square feet if the thickness reaches 2 to 4 inch. For polyester polymer concrete overlay, the application cost ranges from $8 to $17 per square feet with an average of $10.

The cost of overlays is strongly correlated with the oil price due to production and transportation costs. It is also affected by the scope of the project. The actual cost may vary based on the state and the current oil price.

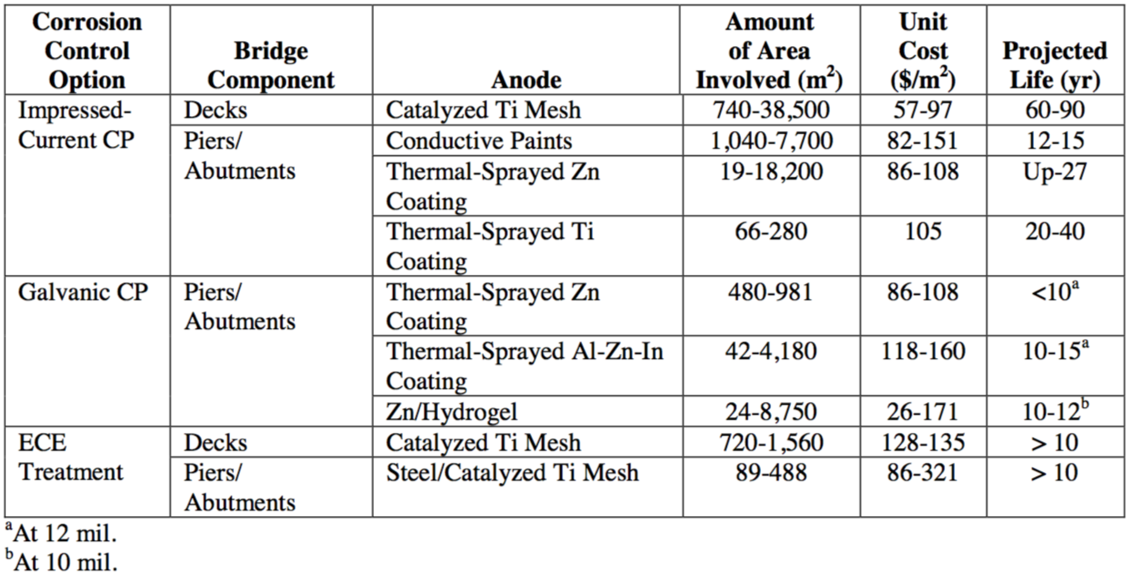
7.2.3 Electrochemical Treatment

The cost of electrochemical treatment depends on bridge specific factors, such as the accessibility. A large amount of concrete damage requires repair before the application of the electrochemical treatment and it may increase the total cost of the application. Spalled and delaminated concrete should be treated and contaminated steel should be cleaned before the installation of the electrochemical treatment.

The conceptual average cost for electrochemical re-alkalization is about $60 per square feet based on the Latah Bridge Rehabilitation Study (2012).

Table 7-2 shows the cost for cathodic protection systems and electrochemical chloride extraction summarized by Clemeña. (2000) The Cost for electrochemical chloride extraction ranges from $13 to $78 per square feet based on the SHRP-S-669 (1993). The cost includes single-use material, amortized materials, and labor, in which the labor cost contribute a major part for the increase of the cost. The cost of implementing ECE or CP for concrete bridges provided by Virginia DOT in 2000 is shown in the Table 7-2. Based on the values reported, the cost for ECE treatment ranges $11.9- $12.5 per square feet for bridge decks and $8-$29.8 for piers and abutments. Lee (2005) investigated the ECE applications in Iowa during the year of 2003, the average cost is reported $25 per square feet for the deck application.

Table 7-2 Cost for CP and ECE Application (Clemeña, 2000)

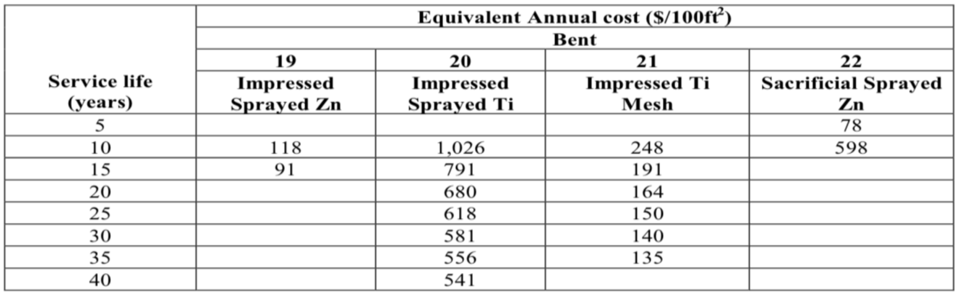


Corrosion Control Option	Bridge Component	Anode	Amount of Area Involved (m^2)	Unit Cost ($\$/m^2$)	Projected Life (yr)
Impressed-Current CP	Decks	Catalyzed Ti Mesh	740-38,500	57-97	60-90
	Piers/ Abutments	Conductive Paints	1,040-7,700	82-151	12-15
		Thermal-Sprayed Zn Coating	19-18,200	86-108	Up-27
		Thermal-Sprayed Ti Coating	66-280	105	20-40
Galvanic CP	Piers/ Abutments	Thermal-Sprayed Zn Coating	480-981	86-108	<10[a]
		Thermal-Sprayed Al-Zn-In Coating	42-4,180	118-160	10-15[a]
		Zn/Hydrogel	24-8,750	26-171	10-12[b]
ECE Treatment	Decks	Catalyzed Ti Mesh	720-1,560	128-135	> 10
	Piers/ Abutments	Steel/Catalyzed Ti Mesh	89-488	86-321	> 10

[a] At 12 mil.
[b] At 10 mil.

For cathodic protection system, the cost analysis is different due to various system types. Since the cathodic protection system is a long-term protection system, it is unfair to compare its huge capital investment with other protection systems. Therefore, the equivalent annual cost is used to compare CP for different anode system, as shown in Table 7-3. The equivalent annual cost is illustrated below. In general, the costs for CP falls in the range between \$10- \$30 per square feet of the surface treated.

Table 7-3 Equivalent Annual Costs for Anode System Per Unit Area (Etcheverry, 1998)

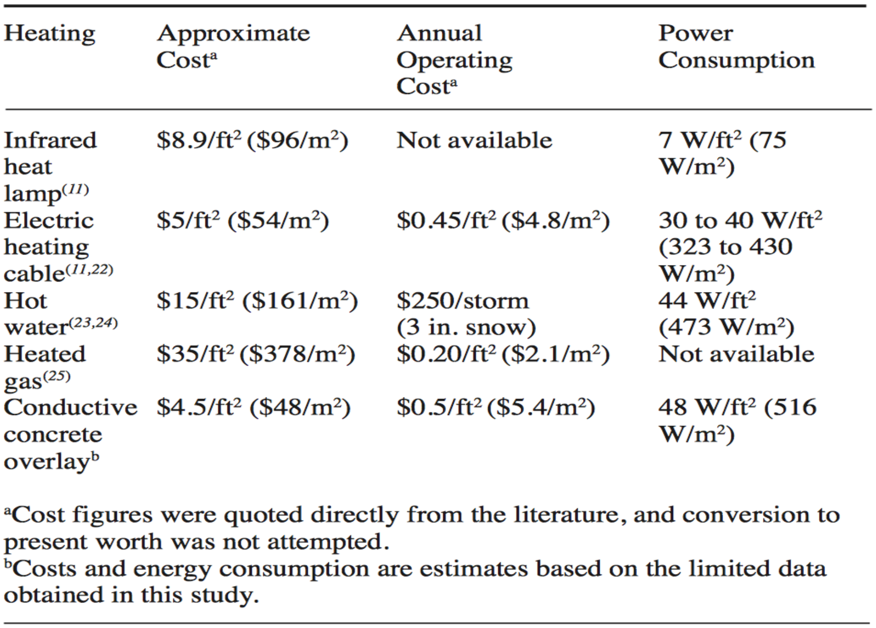


	Equivalent Annual cost (\$/100ft^2)			
	Bent			
	19	20	21	22
Service life (years)	Impressed Sprayed Zn	Impressed Sprayed Ti	Impressed Ti Mesh	Sacrificial Sprayed Zn
5				78
10	118	1,026	248	598
15	91	791	191	
20		680	164	
25		618	150	
30		581	140	
35		556	135	
40		541		

7.2.4 Other Activities

In order to migrate the freeze-thaw damage, the freeze and thaw cycles has to be controlled within a favorable range. Use of a heating system will not only decrease the freeze and thaw damage that might occur, but also will keep the bridge system free from usage of the deicing salt, which is the major source of external chloride ions. The approximate cost based on the heating system is generalized in Table 7-4.

Table 7-4 Comparison of Heating Systems (Yehia, 1998)



Heating	Approximate Cost[a]	Annual Operating Cost[a]	Power Consumption
Infrared heat lamp(11)	\$8.9/ft^2 (\$96/m^2)	Not available	7 W/ft^2 (75 W/m^2)
Electric heating cable(11,22)	\$5/ft^2 (\$54/m^2)	\$0.45/ft^2 (\$4.8/m^2)	30 to 40 W/ft^2 (323 to 430 W/m^2)
Hot water(23,24)	\$15/ft^2 (\$161/m^2)	\$250/storm (3 in. snow)	44 W/ft^2 (473 W/m^2)
Heated gas(25)	\$35/ft^2 (\$378/m^2)	\$0.20/ft^2 (\$2.1/m^2)	Not available
Conductive concrete overlay[b]	\$4.5/ft^2 (\$48/m^2)	\$0.5/ft^2 (\$5.4/m^2)	48 W/ft^2 (516 W/m^2)

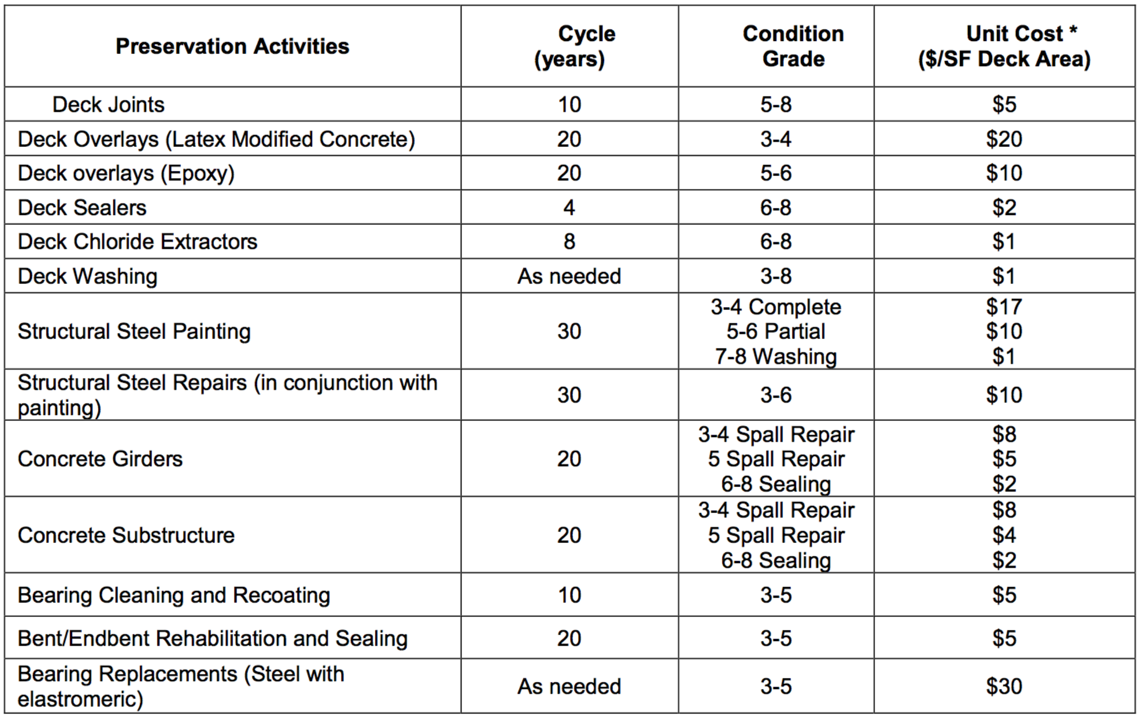
[a] Cost figures were quoted directly from the literature, and conversion to present worth was not attempted.
[b] Costs and energy consumption are estimates based on the limited data obtained in this study.

If the deck repair work were needed for a bridge deck with overlays, the replacement of the overlay would add an additional cost to the project. If the asphalt overlay with waterproof membrane is present on the current bridge deck, the replacement cost will depend on whether replacement of the asphalt layer or the asphalt layer and the membrane is needed.

7.3 Summary

The NCHRP 14-23 report gives the unit cost for various preservation activities. The unit cost for different preventive maintenance and repair activities is shown in Table 7-5.

Table 7-5 Cost Estimation for Preservation Actions (NCHRP 14-23, 2014)



Preservation Activities	Cycle (years)	Condition Grade	Unit Cost * ($/SF Deck Area)
Deck Joints	10	5-8	$5
Deck Overlays (Latex Modified Concrete)	20	3-4	$20
Deck overlays (Epoxy)	20	5-6	$10
Deck Sealers	4	6-8	$2
Deck Chloride Extractors	8	6-8	$1
Deck Washing	As needed	3-8	$1
Structural Steel Painting	30	3-4 Complete 5-6 Partial 7-8 Washing	$17 $10 $1
Structural Steel Repairs (in conjunction with painting)	30	3-6	$10
Concrete Girders	20	3-4 Spall Repair 5 Spall Repair 6-8 Sealing	$8 $5 $2
Concrete Substructure	20	3-4 Spall Repair 5 Spall Repair 6-8 Sealing	$8 $4 $2
Bearing Cleaning and Recoating	10	3-5	$5
Bent/Endbent Rehabilitation and Sealing	20	3-5	$5
Bearing Replacements (Steel with elastromeric)	As needed	3-5	$30

* to raise to acceptable LOS

Based on the rate given in Table 7-5 and the literature reviewed in the previous sections, a cost estimation table is constructed and given below. Table 7-6 presents the unit cost for various preventive measures for concrete bridges. The cost may vary based on the accessibility of the materials, the construction experiences, location of the job site and oil price, etc.

Table 7-6 Estimated Cost for Preservation Actions

Type of Activities	Cost ($/sq. ft.)
Sealer	
Crack Sealer	0.4-0.8
Penetrating Sealer	1-3
Overlays	
High Performance Concrete Overlays	17-25
Low Slump Concrete Overlays	13-19
Latex Modified Concrete Overlays	18-39
Asphalt Overlays with a Membrane	4-7

Others	
Cathodic Protection	10-30
Electrochemical Chloride Extraction	10-30
Electrochemical Re-alkalization	10-30
Deck Replacement	80-100
Asphalt Replacement	2-5
Remove Overlay	8-15

The costs listed in table 7-6 are case sensitive. It includes the costs for surface preparation, material costs and the installation costs. The actual cost for overlay replacement may be higher for the reapplication due to the fact that the existing surface need to be removed and that activity introduce additional costs to the total costs. In addition, the actual cost for implementing electrochemical treatment such as cathodic protection, chloride extraction and re-alkalization might also be higher since major rehabilitation work always needs to be done before the treatment is done.

The values are used in this report for the cost analysis and comparison between different preventive maintenance systems and application strategies. The actual costs for each application need to be identified for the purpose of optimize the preventive maintenance systems and the timing for implementing the actions.

7.4 References

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Chapter 8 Life-cycle Cost Analysis

8.1 Introduction

The bridge cost to an agency is never a one-time expenditure. A bridge represents a long-term, multiyear investment. Following its planning, design, and construction, a bridge requires periodic maintenance and possibly repair or rehabilitation actions to ensure its continued function and safety. (Hawk, 2003)

In this chapter, a 100-year design service life is considered for comparison of the cost for different maintenance strategies. The timing for actions are determined from deterioration curve given in literature, and the cost associated with different preventive maintenance measures are obtained from Chapter 7. The maintenance cost is compared by covering all the cost to the present value considering the time value. No user cost is considered in this analysis.

8.2 Condition Deterioration

Different condition deterioration scenarios are illustrated in Figure 8-1. The physical condition of a bridge element deteriorates over time, as shown by curve A-B without any treatment. If preventive maintenance actions are performed shortly after construction or periodically on the element, the deterioration rate would be decreased, and the service life might be elongated as illustrated by curve A-C. If preservation actions are performed after the bridge elements deteriorated, the effects of applying the perseveration actions will have two impacts on the bridge element: improvement in condition as shown in curve D-E and subsequent deterioration as shown in curve E-F.

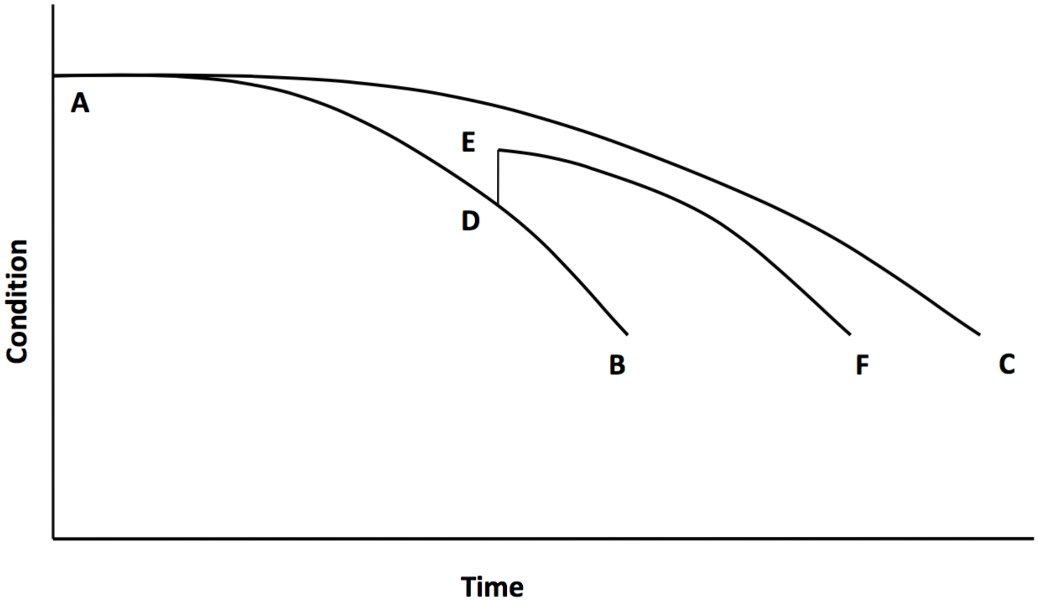


Figure 8-1 Effect of Preservation on Condition Rating Over Time (Johnston, 2014)

The deterioration models based on condition inspection data come with a different form, as shown in Figure 8-2. The conditions are integers; therefore the deterioration lines are represented by steps instead of a smooth line. If the number of possible condition states is large, the model is still stepped, but calibrated to match the physical model. The conditions are determined by the definition of the triggers for each condition state, such as delamination area, spalls, corrosion, and cracks. The time involved is the time needed for an element condition deteriorate from one condition to another.

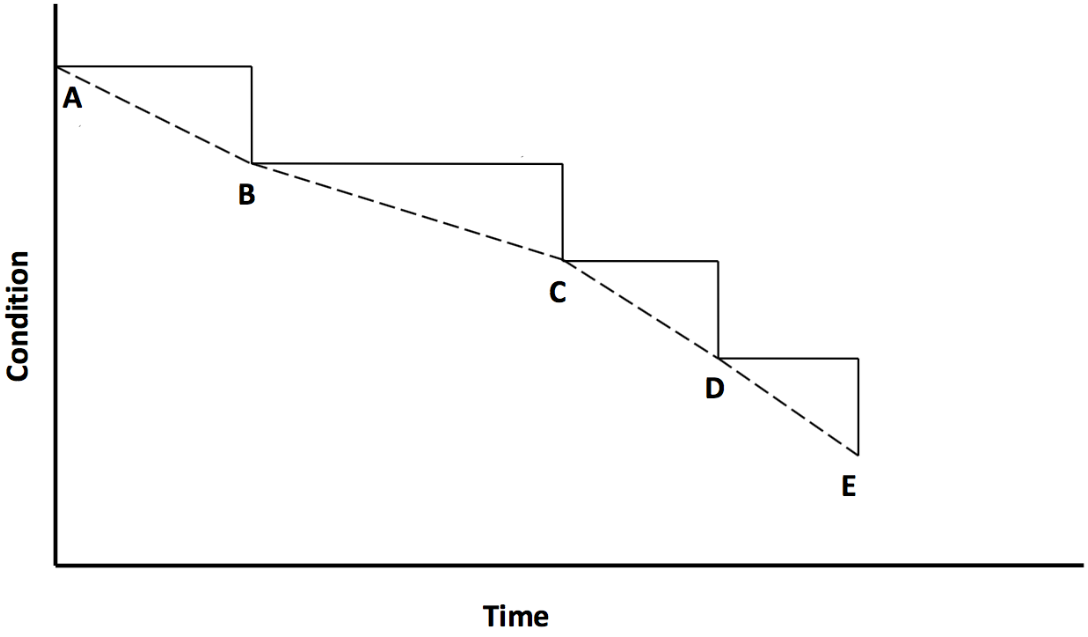
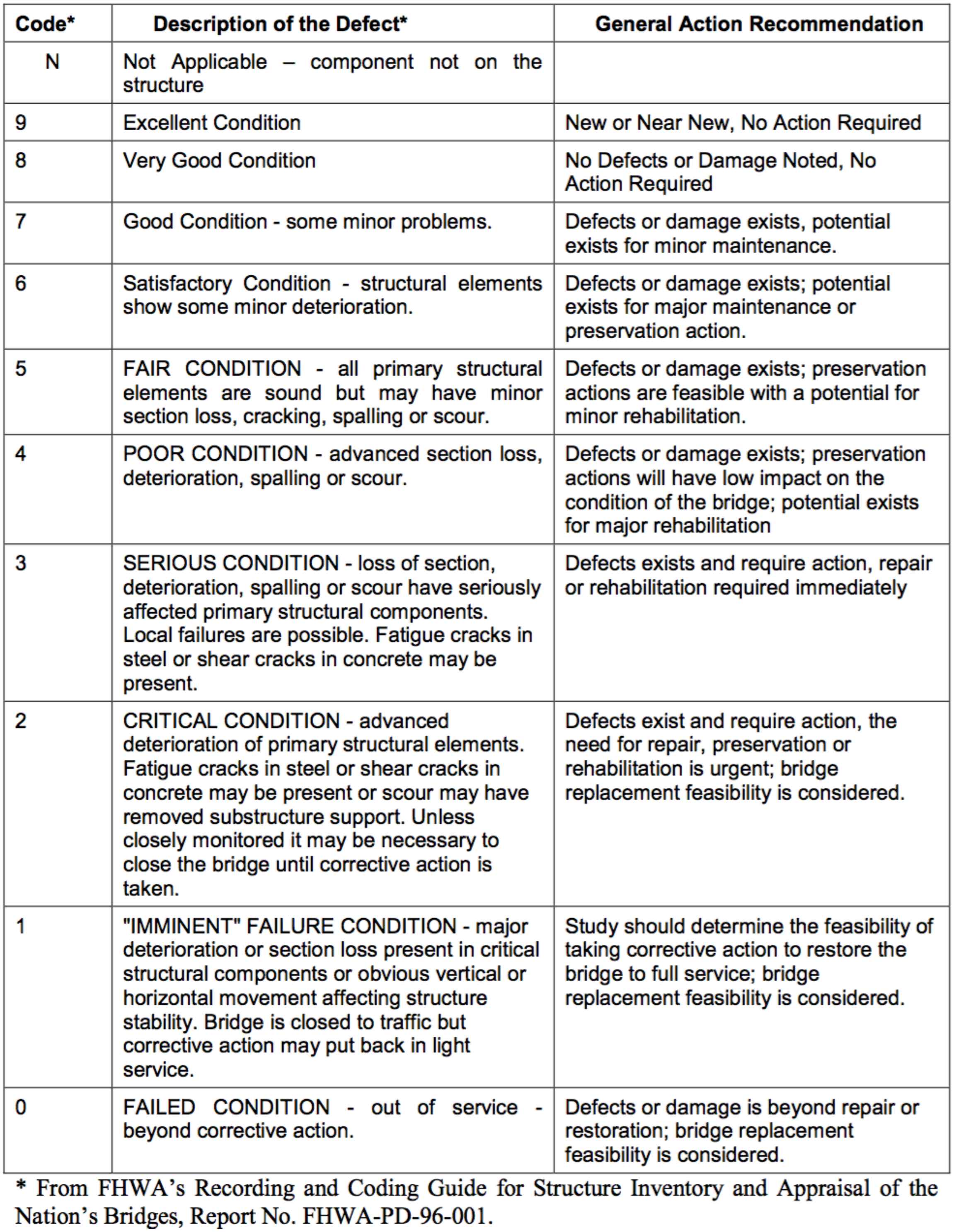


Figure 8-2 Example Condition State Deterioration Model (Johnston, 2014)

The condition state deterioration model is highly depended on the condition definition. The current national standard in the United States is based on the NBI coding guide published by FHWA. The condition rating scale is based on the safety of the bridge and has a high level description of the safety characteristics. These "condition" descriptions are tied to the four major components of the bridge: deck, superstructure, substructure and culvert. The condition is evaluated based on a 0 to 9 scale; where 0 is bridge closure and 9 in new condition. A condition of 3 or under is usually considered unacceptable condition that requires immediate action. Table 8-1 shows the condition descriptions and the general actions recommended. (Johnston, 2014)

Table 8-1 Description of NBI condition Ratings (Johnston, 2014)



Code*	Description of the Defect*	General Action Recommendation
N	Not Applicable – component not on the structure	
9	Excellent Condition	New or Near New, No Action Required
8	Very Good Condition	No Defects or Damage Noted, No Action Required
7	Good Condition - some minor problems.	Defects or damage exists, potential exists for minor maintenance.
6	Satisfactory Condition - structural elements show some minor deterioration.	Defects or damage exists; potential exists for major maintenance or preservation action.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.	Defects or damage exists; preservation actions are feasible with a potential for minor rehabilitation.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.	Defects or damage exists; preservation actions will have low impact on the condition of the bridge; potential exists for major rehabilitation
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.	Defects exists and require action, repair or rehabilitation required immediately
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.	Defects exist and require action, the need for repair, preservation or rehabilitation is urgent; bridge replacement feasibility is considered.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.	Study should determine the feasibility of taking corrective action to restore the bridge to full service; bridge replacement feasibility is considered.
0	FAILED CONDITION - out of service - beyond corrective action.	Defects or damage is beyond repair or restoration; bridge replacement feasibility is considered.

* From FHWA's Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges, Report No. FHWA-PD-96-001.

The new AASHTO Manual for bridge element inspection (2013) has a different system for element condition state identification. There are four condition states represented by a scale 1 to 4, where the element state is good, fair, poor, or severe, respectively. The condition is evaluated by the defects on the element, which are generally delamination, spall, cracks and exposed rebar for concrete components. An example of the condition description for a typical element is shown in Table 8-2.

Table 8-2 Description of Condition Rating based on the AASHTO Inspection Manual (Johnston, 2014)

Element	**Reinforced Concrete Deck** **Delamination/Spall/Patched Area**
Condition State 1	None.
Condition State 2	Delaminated. Spall 1 in. or less deep or 6 in. or less in diameter. Patched area that is sound.
Condition State 3	Spall greater than 1 in. deep or greater than 6 in. diameter. Patched area that is unsound or showing distress. Does not warrant structural review.
Condition State 4	Warrants structural review.

The current New York State inspection program (NYSDOT, 2014) requires recording condition information for all elements on a span-by-span basis. Elements are rated using the scale 1 to 9 as shown in Table 8-3, in which 1 is totally deteriorated and 7 in new conditions. The Codes 8 and 9 are denoted to "Not applicable" and "Condition unknown", respectively. Based on the result of condition rating evaluation, items rated 3 or lower may require substantial rehabilitation and items rated 4 or higher may be corrected or improved with maintenance work.

Table 8-3 Description of Condition Rating based on the NYSDOT Inspection Manual (NYSDOT, 2014)

Code	Description
9.	Condition and/or existence unknown
8.	Not applicable
7.	New condition -- no deterioration
6.	Used to shade between ratings of 5 and 7
5.	Minor deterioration, but functioning as originally designed
4.	Used to shade between ratings of 3 and 5
3.	Serious deterioration, or not functioning as originally designed
2.	Used to shade between ratings of 1 and 3
1.	Totally deteriorated, or in failed condition

8.3 Estimating Deterioration

Johnston (2014) used expert elicitation through a Delphi process to construct a temporary deterioration model based on the AASHTO Manual. An example is given in NCHRP report 14-23 (Johnston, 2014). The time from new or newly replaced to the weighted average condition state of an element is given in Table 8-4.

Table 8-4 Estimated Transition Time (Johnston, 2014)

Condition State	Age
1	0
1.5	15
2	23
2.5	30
3	35
3.5	40
4	50

A polynomial equation could be derived based on the transition time matrix, as shown in Figure 8-3. This polynomial equation is generally used as the deterioration curve that describes the condition deterioration process of the concrete bridge element.

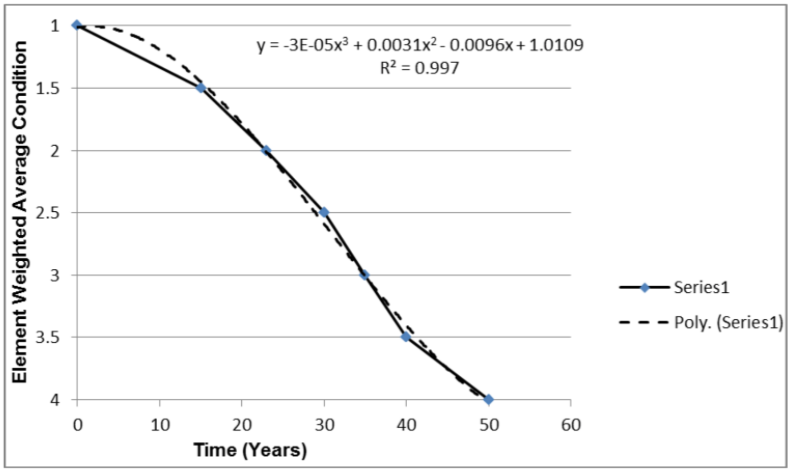


Figure 8-3 Deterioration Curves (Johnston, 2014)

As motioned previously, the deterioration curves are strongly connected with the condition rating system. Morcous (2011) developed the deterioration models for Nebraska bridges that are based on the NBI condition ratings of bridge components obtained from bridge inspections between 1998 and 2010.

For original bridge decks without overlays, the average transition period is listed in Table 8-5, based on the report done by Morcous (2011):

Table 8-5 Transition periods for Original Decks (Morcous, 2011)

Condition State	Transition Period (Years)
9-8	9.3
8-7	17.4
7-6	3.4
6-5	6.6
5-4	2.8

Based on the data above, the original deck deterioration equations could be derived as shown in Figure 8-4.

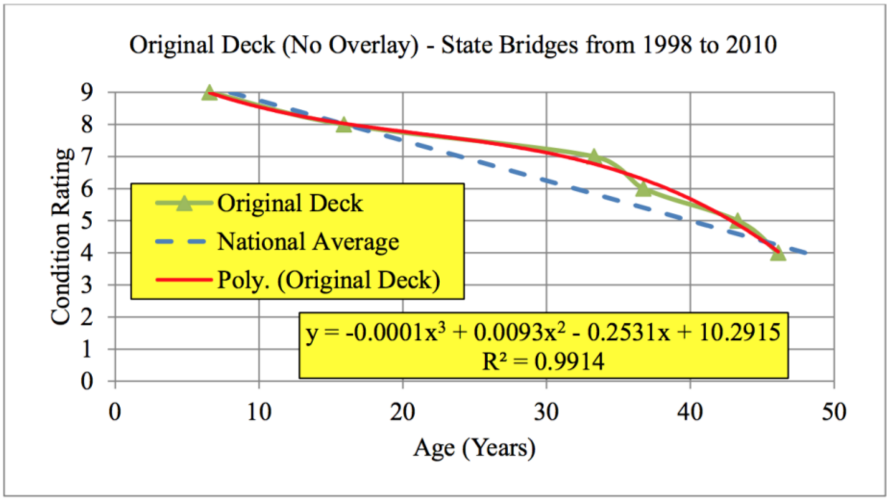


Figure 8-4 Original Deck Deterioration Curve (Morcous, 2011)

The green line represents the transition period derived from the field data and the red line is the polynomial function based on the field data. The blue dash line represents the national average, which assumes the condition of bridge decks drops down to the following condition step every 8 years.

Similarly, the average transition period for replacement decks and overlays is summarized in Table 8-6. The replacement decks are the decks that have year reconstructed and have a wearing surface of monolithic concrete. The condition "7 to 6" has a maximum transition period of 13 years for replacement decks. For overlays, the most commonly used overlays by Nebraska are silica fume overlay, latex modified concrete overlay and low slump concrete overlay. Since the use of silica fume and latex modified concrete overlay is a relatively new technique, limited data is available to determine the deterioration curve. However, for low slump concrete overlay, a deterioration curve is constructed. Results shows that the maximum transition period is approximately 7 years for overlays and it is for condition 9 to 8. The overlays deteriorates much faster than the original decks, however, it can be reapplied once the overlay reaches its service life. Therefore, the overlays reapplication should be scheduled carefully to minimize the cost as well as maintain a sufficient protection for the bridge deck.

Table 8-6 Transition periods for Replacement Decks and Overlays (Morcous, 2011)

Condition State	Transition Period (Years)	
	Replacement Decks	Overlays
9-8	6.1	7.5
8-7	5.1	3.8
7-6	13.2	3.3
6-5	3.6	3.4
5-4	2.7	1.3

Agrawal (2009) suggested the use of Weibull deterioration model instead of a pure Markovian model; and the deterioration rates is estimated using the NYSDOT bridge inspection data. The example deterioration equations are given in Table 8-7. The corresponding deterioration curves are shown in Figure 8-5 and Figure 8-6. In which, the condition of the concrete bridge elements are evaluated based on the NYSDOT inspection manual (2014) and the age is the time after construction completion in years.

Table 8-7 Deterioration Equations (Agrawal, 2009)

Element Type	Deterioration Curve Fitted Equation
Wearing Surface	
Integral or monolithic Portland cement	$CR = 7 - 0.1178904T + 0.0012462T^2 - 0.0000070T^3$
Concrete with membrane	$CR = 7 - 0.3488945T + 0.0211688T^2 - 0.0005196T^3$
Portland cement overlay	$CR = 7 - 0.1517338T + 0.0019529T^2 - 0.0000097T^3$
Structural Deck	
CIP concrete uncoated rebars	$CR = 7 - 0.0675608T + 0.0001411T^2 + 0.0000001T^3$
CIP concrete epoxy coated rebars	$CR = 7 - 0.0767927T + 0.0007988T^2 - 0.0000051T^3$
CIP concrete-other protection	$CR = 7 - 0.0793700T + 0.0005157T^2 - 0.0000023T^3$

** T= time in years*

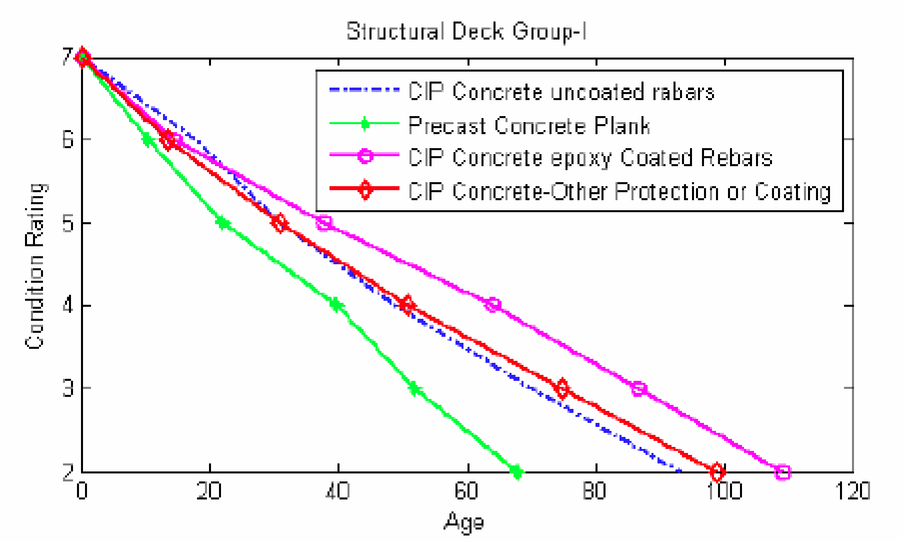


Figure 8-5 Deterioration curves for Structural Deck (Agrawal, 2009)

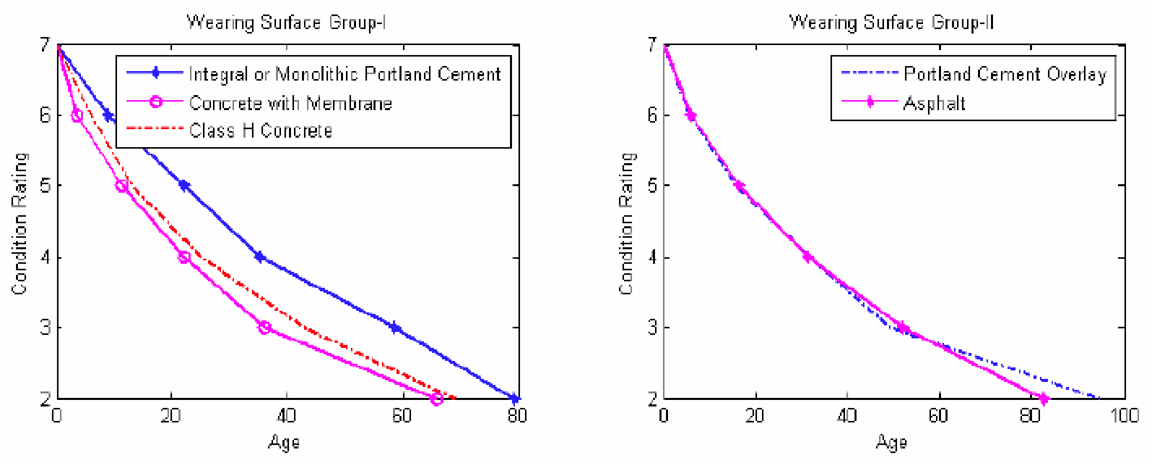


Figure 8-6 Deterioration curves for Wearing Surface (Agrawal, 2009)

8.4 Identifying Preservation Actions

The preservation actions are limited by the current condition state of the concrete bridge element. Some elements may have only one or two feasible actions especially when the bridge element is in excellent condition or totally deteriorated. In this analysis, doing nothing is always a feasible alternative for the preservation actions. Two main aspects of information need to be identified for preservation actions, the cost and the effects of the preservation actions. The typical cost matrix and the effectiveness illustration are shown in Figure 8-7. In which, the MACST(i, j) represents the unit cost of the maintenance activity that can uplift the condition state from i to j. Minor rehabilitation works will not change the condition states of the bridge elements, however, the elements will remain in the same condition state for a longer period, the unit cost is represented by MACST(i, i).

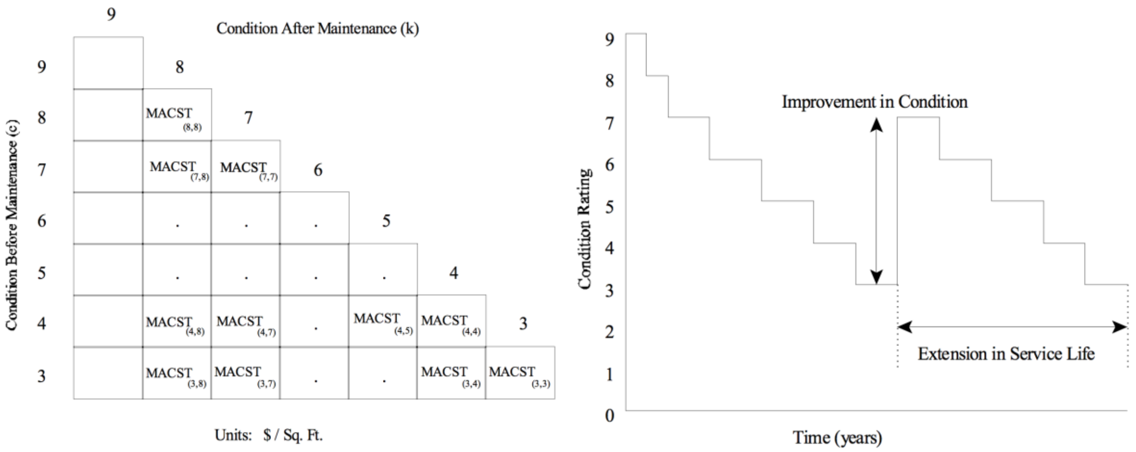


Figure 8-7 Preservation action unit cost matrix and effectiveness (Jebreen, 1995)

The unit cost of the preservation actions are influenced by the previous condition stated, target condition state after maintenance, the amount of work, accessibility and other effects. It is clearly that the cost is high if the condition state difference before and after maintenance is high. Major rehabilitation works, like bridge deck replacement, is costly but effective for restoring the condition state to excellent or near new condition. On the other hand, minor rehabilitation works, like washing and sealing, will not be an appropriate solution to enhance the condition state of a bridge element, but cost much less than the major ones.

The effects of the preservation actions can be classified into two categories: restoration of the condition state or extension of time in the same condition state. The typical treatment that will result in a restoration in condition state for concrete decks is application of overlays, which will eliminate spalls and delamination on the deck surface and set the condition of the bridge deck to a better state. Application of sealers is a typical action that will lead to the extension of time that a bridge element could stay in the same condition state. The presence of sealers will postpone the deterioration while the condition state is still in the state of good or fair.

8.5 Maintenance Actions for bridge decks

Comparing the deterioration equations derived from different condition rating systems, the service life and the transition period yields to a similar service life for concrete bridge component. Therefore, the following comparison of the maintenance strategy and scenarios were conducted based on the deterioration model assumed by comparison of the work done by Johnston (2014), Morcous (2011) and Agrawal (2009).

8.5.1 Original Bridge Deck

The deterioration rate within a certain condition state is assumed to be constant for each condition state, and the transition period is based on engineering judgment derived from the data in Johnston (2014), Morcous (2011) and Agrawal's (2009) reports.

The average service life for the original deck in Nebraska is assumed to be 40 years compare to the 50 year service life predicted by the bridge maintenance practitioners' opinion given in the NCHRP report 14-23 (Johnston, 2014). A general assumption of the transition time for concrete bridge decks is the condition drops down to the following one every 8 years.

However, the bridge element may experience accelerated deterioration once its condition state is poor. The transition time are given in Table 8-8.

Table 8-8 Assumed Transition Period for Concrete Decks

Condition State	Transition Period (Years)
9-8	9
8-7	17
7-6	4
6-5	7
5-4	3

The deterioration curve is shown in the Figure 8-8.

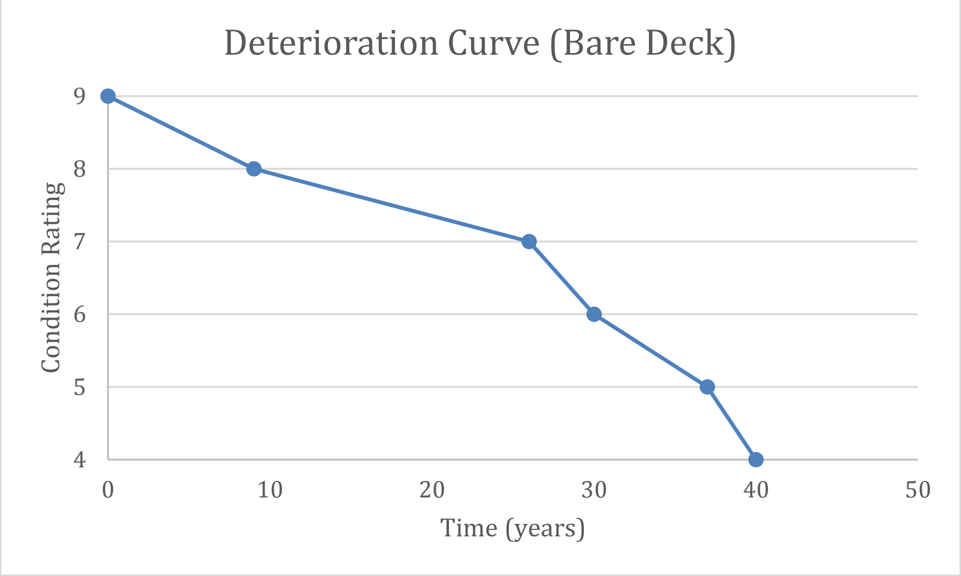


Figure 8-8 Deterioration Curve for Bare Concrete Bridge Deck

8.5.2 Bridge Deck with Sealer

The sealer should be applied to the concrete deck surface when the condition state of the bridge decks is still in good or fair condition. The application of sealer will not restore the condition to a higher state. However, the existence of the sealers will block the ingression of harmful compounds into the concrete, which may slow down the deterioration rate and eventually elongate the service life of bridge deck components. The service life elongate is assumed to be 1-3 years based on the effectiveness and the reapplication rate of the sealer. In this analysis, a two-year life extension is assumed for seal reapplied every 6 years. However, once the concrete has already deteriorated and has excessive cracking or other physical damage, the sealers will not perform well, nor extend the service life of the bridge deck. The assumed transition periods for concrete decks with the application of sealers are shown in Table 8-9. The sealer is assumed to be ineffective after the condition rating is 7 or lower. The service life of the concrete deck with the application of sealer every 6 years is assumed to be 50 years, 10 years longer than the bare concrete deck without any preservation actions.

Table 8-9 Assumed Transition Period for Concrete Decks

Condition State	Transition Period (Years)
9-8	11
8-7	25
7-6	4
6-5	7
5-4	3

The deterioration curves are shown in Figure 8-9. From the chart, it can be seen that the presence of sealer dramatically slows down the deterioration rate while the condition rating of the bridge deck is above 7. However, once the condition rating deteriorated to a state of 7 or lower, the deterioration rates are same for both scenarios with or without the application of sealer.

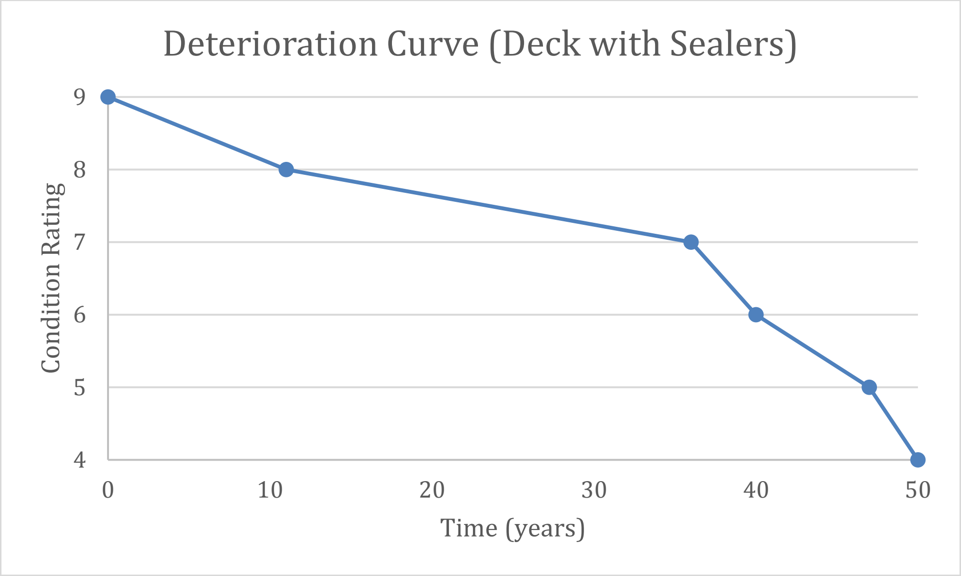


Figure 8-9 Deterioration Curves for Bridge deck

8.5.3 Periodically Bridge Deck Overlay

Bridge deck overlay is the most commonly used technique to restore the condition state of a concrete bridge deck component. It is assumed that the overlays are applied when the bridge deck's deterioration reaches the preset criteria, which might be the delaminated area, spalls area, corroded rebar and etc. Several researchers give recommendation for the use of overlay while 12% of the deck area is deteriorated. The transition period for overlays could be assumed based on the report done by Johnston (2014), Morcous (2011) and Agrawal (2009), as shown in Table 8-10.

Table 8-10 Assumed Transition Period for Concrete Decks

Condition State	Transition Period (Years)
9-8	7
8-7	5
7-6	3
6-5	4
5-4	1

The associating deterioration curve is shown in Figure 8-10.

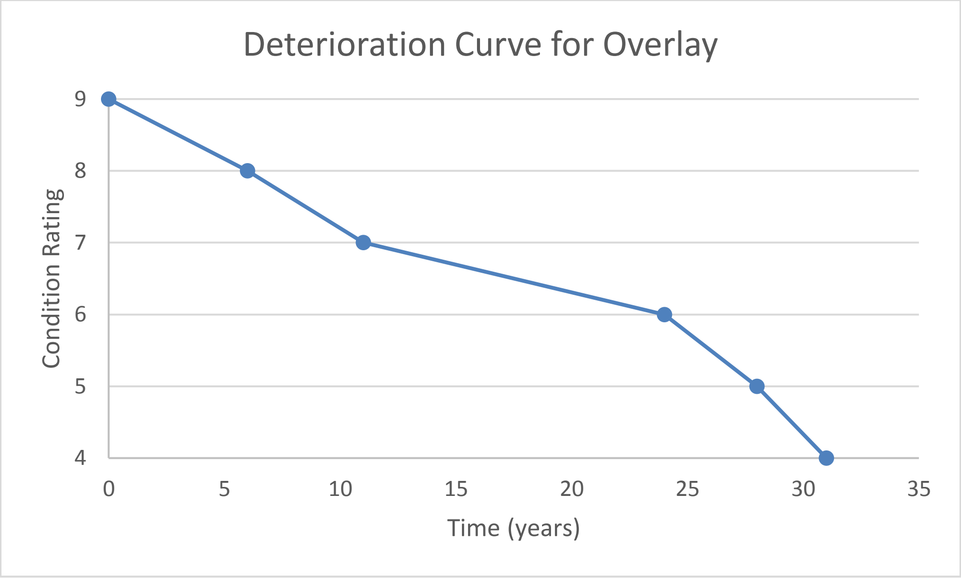


Figure 8-10 Deterioration Curves for Overlays

If the overlay is applied well and the effectiveness of the overlays remains 100% over their service life, the deck would be protected fully as long as the overlays existed. Furthermore, if the damaged portion of the deck is fully treated and the contaminated concrete is removed, the condition of the bridge deck will be restored to excellent or very good condition. That means the condition of the element will be brought back to a certain state that the condition rating is very close to 9 after every application of overlays in spite of the age of the concrete deck beneath it. However, it is unfortunately not true for any one of these scenarios in the real world. The effectiveness of the overlays cannot be perfect through its whole service life, which means the bridge deck will continue to deteriorate, but at a slower rate. Due to the aging and accumulation of deterioration of the concrete deck, the condition cannot always been brought back to a nearly new state. The accumulation of harmful compounds, the development of the cracks and other deterioration will have an impact on the condition rating of the bridge deck no matter how aggressive the preventive maintenance strategy is used.

If the first overlay is applied to the structure once the condition rating reaches 7; and reapplication is applied to the concrete deck once the condition rating of the overlay reaches 6, and perfect preparation/application is assumed, the deterioration curve is illustrated in Figure 8-11. The first part of the deterioration curve is dictated by the condition rating of the bridge deck and the following portion of the condition rating is evaluated from the performance of the overlays.

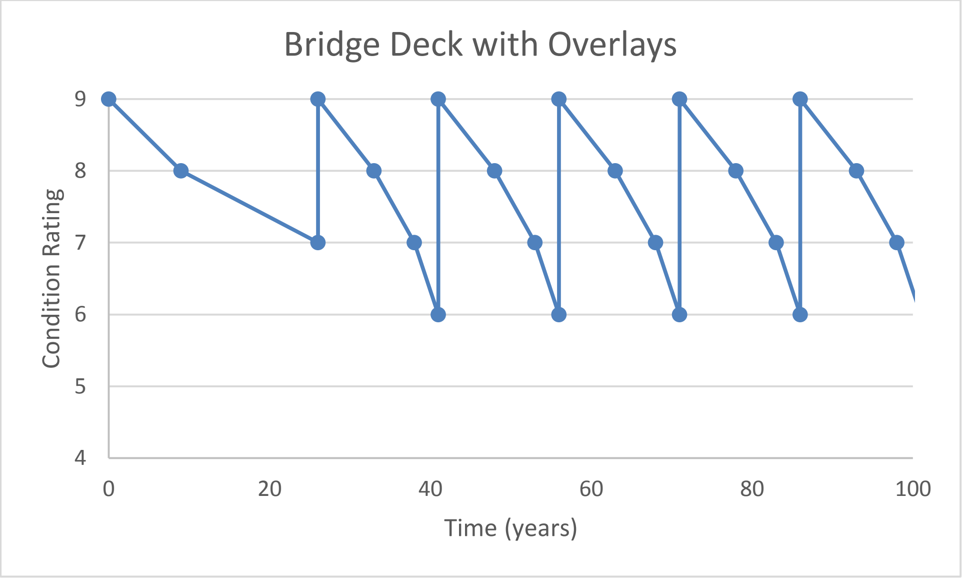


Figure 8-11 Condition Deterioration Curves for the Overlays

A bridge deck needs more effective protection systems after years of exposure to the harmful environment. Therefore, the frequency for reapplication of overlays needs to be increased. Assume the bridge deck condition can be restored to a maximum value of 8.8 after the first application of overlays and each reapplication of overlay will decrease the maximum achievable condition state by 0.2, and assume the deterioration rate is constant for the element in each condition state; a new deterioration curve could be developed as shown in Figure 8-12. Assume the bridge condition state deteriorates and the current condition state dictates the initial condition rating for the overlays, the blue line represents the deterioration curve for the overlays with the application years and the red line represents the condition deterioration curve for the bridge deck, respectively.

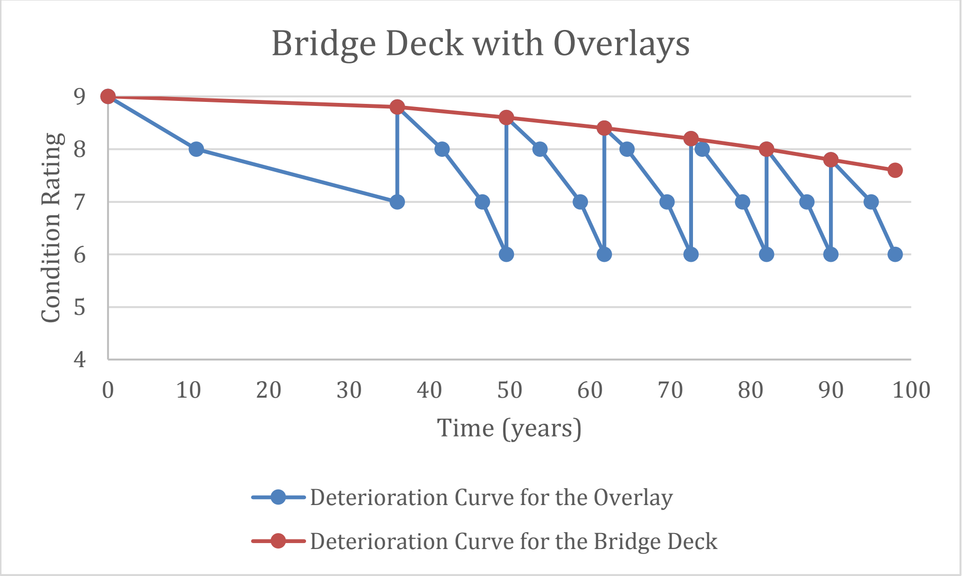


Figure 8-12 Condition Deterioration Curves with Overlays

8.5.4 Bride Deck with Asphalt Overlay with Waterproof Membrane

The use of asphaltic concrete (AC) overlay with membranes is familiar to bridge maintenance engineers. It may be assumed that all spalls are repaired and 50 % of the delaminated area existing at the time the treatment will be eliminated by the treatment if AC overlay is used. The expected service life for these systems varies between 12-19 years (Krauss, 2009), while is only 7 years according to the estimation given by Wisconsin DOT. (Adams, 2002)

A simplified deterioration model could be assumed by assuming the AC overlay with membrane will elongate the service life of the bridge deck by 8 years for the first application and the following reapplication will have an impact on the service life for 4 years.

The deterioration with the assumed effectiveness of the AC overlay with membrane on the concrete bridge deck is illustrated in Figure 8-13. The trigger for the application is assumed as 8.5 which represents the condition between very good and good, a condition that may need minor rehabilitation and correction work for the component to resume a better performance state.

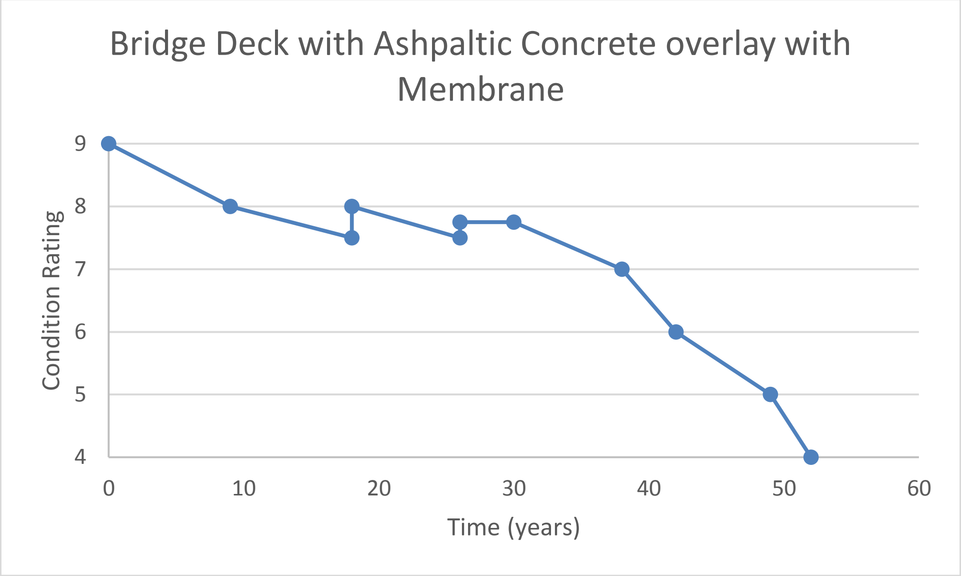


Figure 8-13 Deterioration Curve for Bridge Deck with AC overlay with Membrane

8.5.5 Replacement of the Deck

Bridge deck replacement is one of the most popular solutions for bridge decks that have experienced severe deterioration without proper treatment applied on time. Once the condition rating of the bridge element reaches the failure condition or fully deteriorated, there are not many choices other than the replacement of the element. The assumed transition period for the replacement decks is summarized in Table 8-11.

Table 8-11 Assumed Transition Period for Replacement Decks

Condition State	Transition Period (Years)
9-8	6
8-7	5
7-6	13

6-5	4
5-4	3

Figure 8-13 illustrates the bridge preservation strategy that deck replacement is the only valid alternative for bridge maintenance. Once the condition rating reaches 4, the deck replacement is applied.

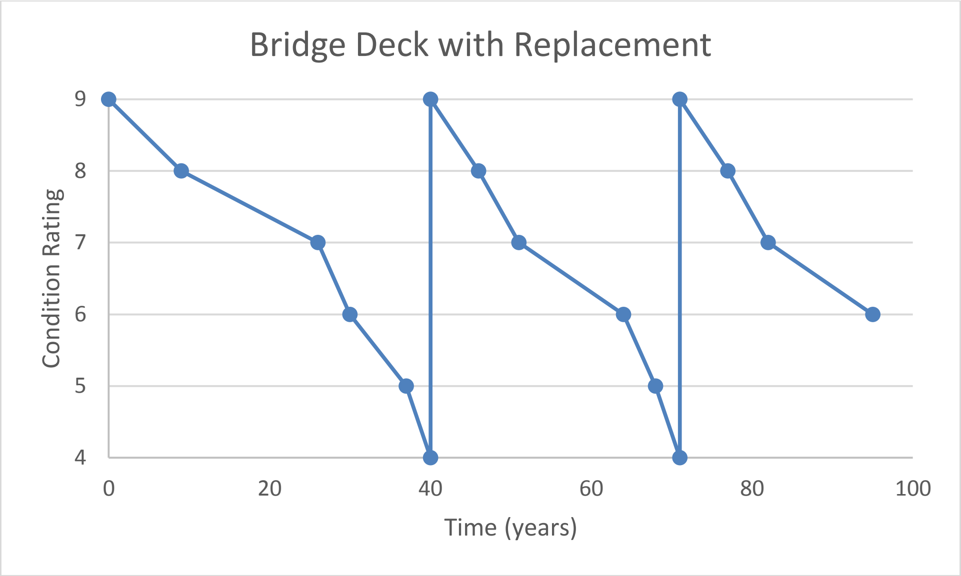


Figure 8-14 Deterioration Curves with Replacement

8.5.6 Cathodic Protection Systems

Cathodic protection systems are the most effective preventive maintenance systems for protecting the system from the damage due to ongoing and potential corrosion activities. If the sacrificial anode is selected, the anodes needs to be replaced approximately every 10 years based on the exposure conditions. If impressed current cathodic protection system is selected, then the expected service life could be 25 to 100 years. The concrete bridge element is considered corrosion free as long as the protection systems are functional. However, for concrete bridge deck systems, the expected service life for overlays is no longer than 30 years. The action of reapply the overlay may damage the cathodic protection systems, which means that the expected service life of the protection systems has a maximum value that is governed by the service life the overlays. In this case, the cathodic protection systems and the overlays are performed every 30 years after the first rehabilitation work. If the effectiveness of the overlays and cathodic protection (CP) systems are guaranteed, the deterioration rate of the bridge deck will be significantly lower than the average. Therefore, the condition state of the bridge deck is assumed to be 8.8 after first rehabilitation and the maximum condition state is decreased by 0.1 for each reapplication. The deterioration curve of the bridge deck is represented in Figure 8-15.

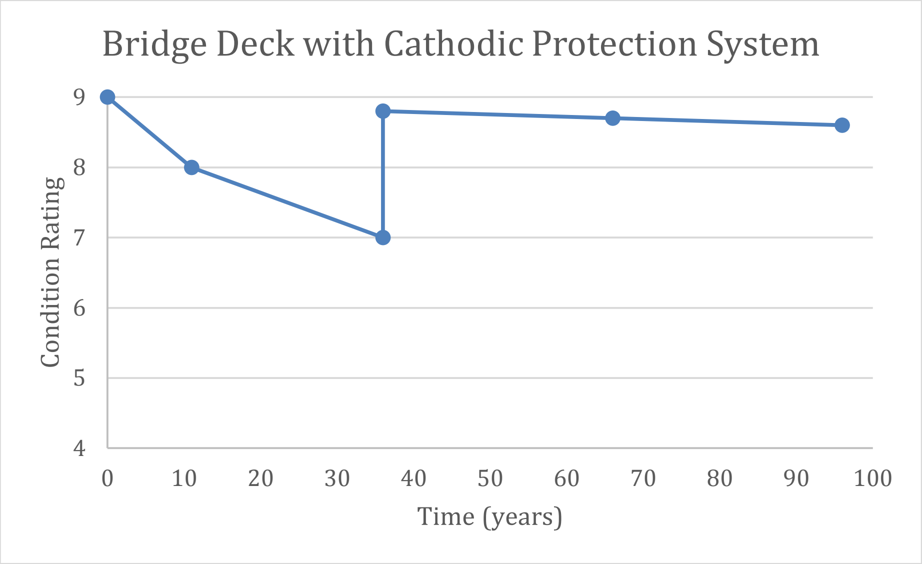


Figure 8-15 Deterioration Curves with Cathodic Protection System

8.6 Maintenance Scenarios and Associated Cost

For the purpose of life cycle cost analysis and comparison, different maintenance strategies and scenarios need to be analyzed. The following strategies for bridge deck preservation are considered:

1. New Construction; Deck Replacement
2. New Construction: Sealing; Deck Replacement
3. New Construction; Asphaltic Concrete overlay with waterproof membrane; Deck Replacement
4. New Construction; Sealing; Concrete Overlays
5. New Construction; Sealing; Concrete Overlays; Deck Replacement
6. New Construction; Sealing; Asphaltic Concrete overlay with waterproof membrane; Concrete Overlays
7. New Construction; Concrete Overlays with Cathodic Protection Systems

The cost and occurrence time for the treatment is also important for the calculation of the life cycle cost for each maintenance strategy. The associated cost is estimated from the chart summarized in Chapter 7. The most likely cost is assumed within the range governed by Table 7-6 and listed in Table 8-12.

Table 8-12 Assumed Cost for Each Activity

Type of Activities	Cost ($/sq. ft.)
Penetrating Sealer	2
Asphaltic Concrete Overlays with a Membrane	6
Concrete Overlays	17
Cathodic Protection	25
Asphalt Replacement	3

Remove Overlay	8
Deck Replacement	90

Scenario 1 is to replace the bridge once the condition state of the bridge drops to 4 and no other preventive maintenance measure are involved. It is used to as a base scenario to compare the effectiveness and the cost of different scenario. It also represents the bridge deterioration curves and maintenance cost under current maintenance strategy. Scenario 2 and 3 is the use of sealers and AC overlays with membranes, respectively, as the only preventive maintenance measure. Scenario 4 requires the use of sealers and concrete overlays as the major preventive maintenance measures. Scenario 5 is selected to investigate the possible minimum cost for 100 years of service life compared to scenario 6. Scenario 7 is the analysis of the concrete overlays along with cathodic protection systems, assuming both of have an estimated service life of 30 years.

The deterioration curve for Scenario 1 is illustrated in Figure 8-16, in which, deck replacement is the only option for bridge deck rehabilitation.

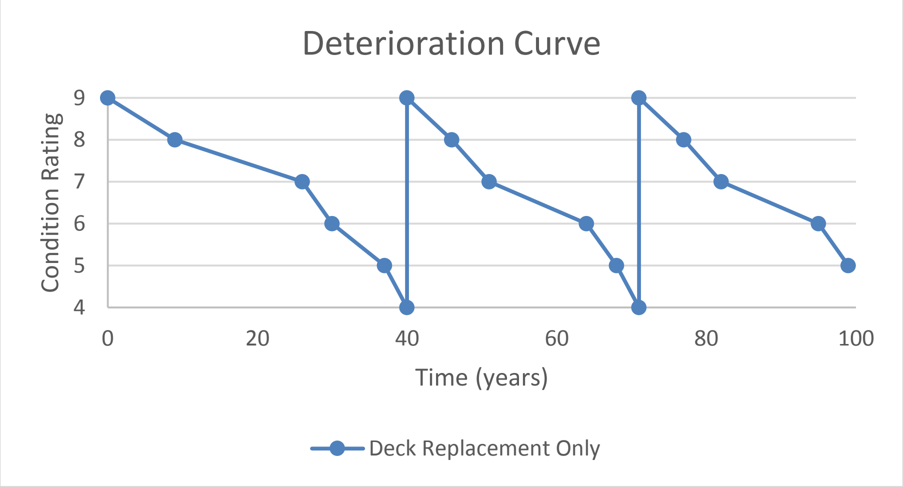


Figure 8-16 Deterioration Curve for Scenario 1

The cost of the treatment and the associated occurrence time for Scenario 1 is shown in Table 8.13.

Table 8-13 Cost for Scenario 1

Scenario 1		
Activity	Cost ($/sq. ft.)	Age (Year)
Deck Replacement	90	40
Deck Replacement	90	71

Figure 8-17 shows the comparison of deterioration cure for Scenario 1 and 2. Scenario 2 includes the use of sealers as preventive maintenance actions. However, based on the assumptions, the sealers are only affective while the condition rating of the bridge deck is above 7, the deterioration curves for the deterioration curves below 7 are identical for both scenarios. The sealer is assumed to be applied every 6 years and it will increase the service life of the bridge deck by 2 years for each application.

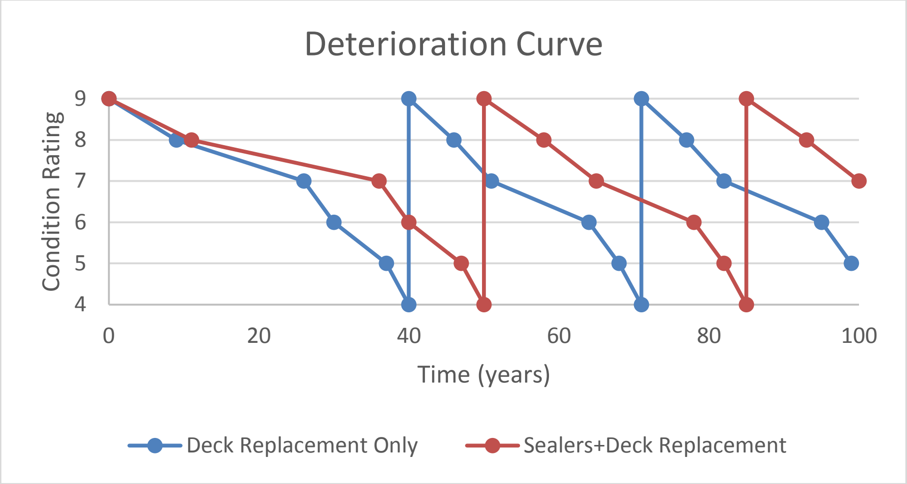


Figure 8-17 Deterioration Curve for Scenario 1 & 2

The cost of the treatment and the associated occurrence time for Scenario 2 is shown in Table 8.14.

Table 8-14 Cost for Scenario 2

Scenario 2		
Activity	Cost ($/sq. ft.)	Age (Year)
Apply Sealer	2	6
Apply Sealer	2	12
Apply Sealer	2	18
Apply Sealer	2	24
Apply Sealer	2	30
Deck Replacement	90	50
Apply Sealer	2	56
Apply Sealer	2	62
Deck Replacement	90	85
Apply Sealer	2	93
Apply Sealer	2	99

Scenario 3 uses asphaltic concrete overlay with membranes (AC with membrane) as the primary preventive maintenance actions. It is assumed that the first application asphaltic concrete overlays with membranes will increase the service life by 8 years and the second application will increase the service life by 4 years. Only two cycles are applied since the asphaltic concrete overlay with membrane cannot eliminate delamination and the bridge deck deterioration will continue developing. The deterioration curves are illustrated in Figure 8-18.

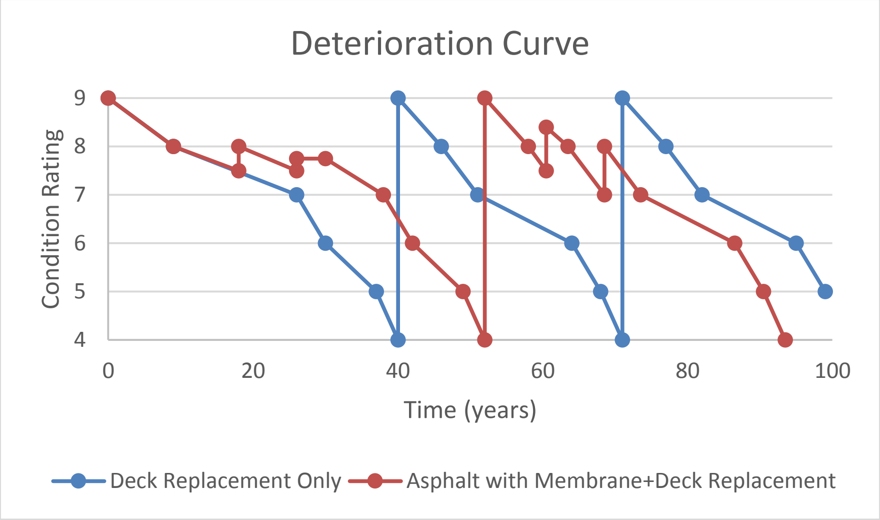


Figure 8-18 Deterioration Curve for Scenario 1 & 3

The cost of the treatment and the associated occurrence time for Scenario 2 is shown in Table 8.15.

Table 8-15 Cost for Scenario 3

Scenario 3		
Activity	Cost ($/sq. ft.)	Age (Year)
AC with membrane	6	18
Remove/Reapply AC with membrane	9	26
Deck Replacement	90	52
AC with membrane	6	60
Remove/Reapply AC with membrane	9	68

Scenario 4 represents the rehabilitation strategy with sealers and concrete overlays, as shown in Figure 8-19. The condition of the bridge will deteriorate with time and the deterioration rates are assumed as illustrated by the green line. The deterioration of the bridge deck will affect the service life of the concrete overlay. The service life of the concrete overlay is assumed to be 15 years for the first application and the minimum service life is assumed to be 8 years. The reapplication schedule could be found in the red curve which represents the condition deterioration of the concrete overlays. The cost associated with scenario 4 is listed in Table 8-16.

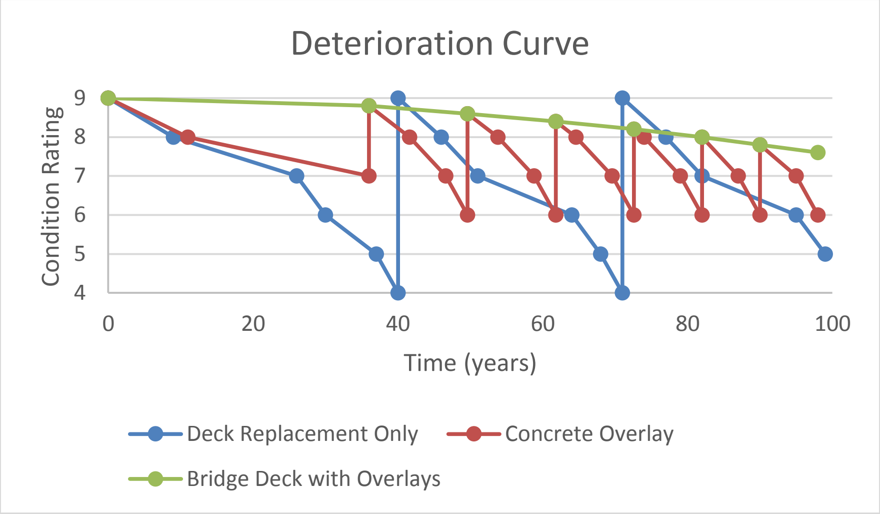


Figure 8-19 Deterioration Curve for Scenario 1 & 4

Table 8-16 Cost for Scenario 4

Scenario 4		
Activity	Cost ($/sq. ft.)	Age (Year)
Apply Sealer	2	6
Apply Sealer	2	12
Apply Sealer	2	18
Apply Sealer	2	24
Apply Sealer	2	30
Concrete Overlay	17	36
Reapply Concrete Overlay	25	50
Reapply Concrete Overlay	25	62
Reapply Concrete Overlay	25	73
Reapply Concrete Overlay	25	82
Reapply Concrete Overlay	25	90
Reapply Concrete Overlay	25	98

Scenario 5 assumes the use of sealers as the first preventive maintenance actions and then application of concrete overlay once the condition rating of the concrete deck reach 7 followed by deck replacement. The deterioration curve for concrete bridge deck with concrete overlay is derived by assuming the overlay will increase the service life of the bridge deck by 15 years and the deterioration rate remains constant within that time. The deterioration curve is illustrated in Figure 8-20.

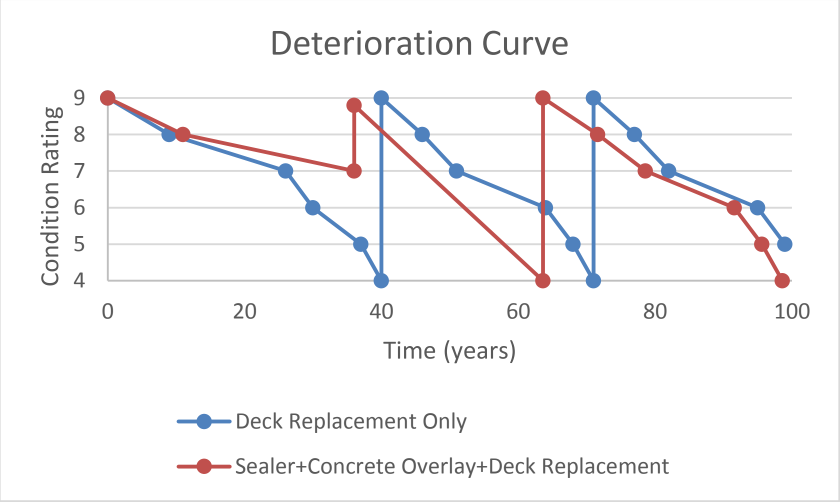


Figure 8-20 Figure 8-21 Deterioration Curve for Scenario 1 & 5

The cost of the treatment and the associated occurrence time for Scenario 5 is shown in Table 8-17.

Table 8-17 Cost for Scenario 5

Scenario 5		
Activity	Cost ($/sq. ft.)	Age (Year)
Apply Sealer	2	6
Apply Sealer	2	12
Apply Sealer	2	18
Apply Sealer	2	24
Apply Sealer	2	30
Concrete Overlay	17	36
Deck Replacement	90	64

Scenario 6 is used to compare with Scenario 5 in order to identify the better approach for preventive maintenance. In this scenario, sealer, asphaltic concrete overlay with waterproof membrane and concrete overlays are used to preserve the concrete bridge deck. Bridge deck replacement is not an option since the condition rating is restricted at a high level. The deterioration curve is illustrated as the red line shown in Figure 8-21.

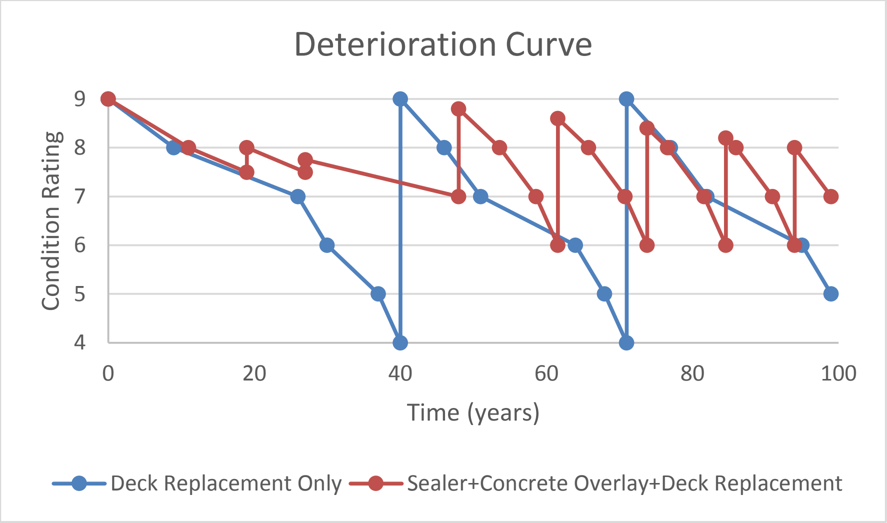


Figure 8-22 Deterioration Curve for Scenario 1 & 6

The cost of the treatment and the associated occurrence time for Scenario 6 is shown in Table 8-18.

Table 8-18 Cost for Scenario 6

Scenario 6		
Activity	Cost ($/sq. ft.)	Age (Year)
Apply Sealer	2	6
Apply Sealer	2	12
Apply Sealer	2	18
AC membrane	6	19
Remove/Reapply AC membrane	9	27
Concrete Overlay	17	48
Concrete Overlay	25	62
Concrete Overlay	25	74
Concrete Overlay	25	85
Concrete Overlay	25	94

Scenario 7 is the use of cathodic protection system. The service life of the cathodic protection system is assumed as 30 years since the conventional concrete overlay has a service life that does not exceed 30 years. Typically, cathodic protection systems need repair or reinstallation after the application of concrete overlays. The deterioration rate of the concrete deck is relatively low given the reliabliability and functional overlays and cathodic protection systems, as shown in Figure 8-23.

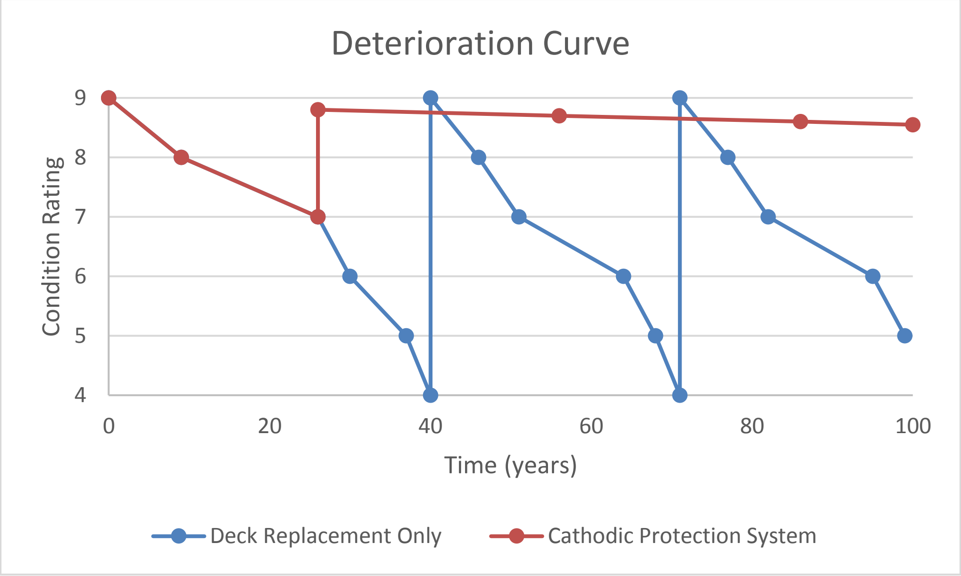


Figure 8-23 Deterioration Curve for Scenario 1 & 7

The cost of the treatment and the associated occurrence time for Scenario 7 is shown in Table 8-18.

Table 8-18 Cost for Scenario 7

Scenario 7		
Activity	Cost (\$/sq. ft.)	Age
Concrete Overlay +CP	42	26
Reapply Overlay +CP	50	56
Reapply Overlay +CP	50	86

8.7 Life-cycle Cost Comparison

In order to conduct this simplified life cycle cost analysis, a 100-year analysis period is selected. The 100-year period is determined by the average design service life reported from the questionnaire answers given by state DOTs. Also, the bridge may be functionally obsolete due to the increase of transportation demands.

Based on the literature reviewed, a 3% discount rate is assumed for the life-cycle analysis. For preventive maintenance actions conducted in year n, the equivalent present value can be derived from Equation 8-1:

$$(P/F,i,n) = F \times \frac{1}{(1+i)^n} \tag{8-1}$$

where, F is the future cost of the preventive maintenance, i is the discount rate and n is the year that the maintenance is applied.

All the strategies are compared based on the planning starts from the current year. Some single action's costs are not included in this analysis, such as rehabilitation of a small portion of an element in poor condition before applying treatment to the entire bridge component.

Given the data listed in Table 8-13 through 8-18, the life cycle cost for each maintenance strategy could be derived. The life cycle cost for different preventive maintenance strategies for a concrete bridge deck with a 100-year desirable service life are summarized in Table 8-19.

Table 8-19 Life Cycle Cost for Different Maintenance Strategies

Scenario No.	Description	Life Cycle Cost ($/sq. ft.)
1	New Construction; Deck Replacement	38.63
2	New Construction: sealing; Deck Replacement	34.82
3	New Construction; AC overlay with waterproof membrane; Deck Replacement	29.27
4	New Construction; Sealing; Concrete Overlays	29.86
5	New Construction; Sealing; Concrete Overlays; Deck Replacement	25.50
6	New Construction; Sealing; Asphaltic Concrete Overlay with Waterproof Membrane; Concrete Overlays; Deck Replacement	26.22
7	New Construction; Concrete Overlays with Cathodic Protection Systems	32.96

From Table 8-19, it is clear that all the preventive maintenance methods are cost-effective compared with the option of do nothing. The total cost for maintenance methods with deck replacement only is $38.63 per square feet, which is much higher than the other alternatives.

The high cost of scenario 2 comes from only applying penetrating sealers as the preventive maintenance measures. Since the effectiveness of the sealer is assumed to be 0 once the condition rating drops to 7 or lower, it is not an effective solution to prevent further deterioration. The sealer performs well on element with an excellent or very good condition states and needs to be applied with other preventive maintenance measures in order to be cost-effective.

Scenario 4 has a cost higher than scenario 5, which indicates bridge deck replacement might be a cost effective option for bridge deck maintenance. However, the condition state of the bridge deck for scenario 4 is higher, which means that the remaining service life is longer. On the other hand, if the user cost including the delay, detour and road close come from the deck replacement are incorporated, then scenario 4 would still probably be a favorable alternative.

Scenario 5 managed to achieve the lowest life cycle cost since it utilized the entire deterioration curve. The condition rating of the system reaches 4, which means the projected service life for the bridge deck under this maintenance strategy is 100 years. The preventive maintenance measures are scheduled perfectly for the assumed deterioration rates for each preventive maintenance systems.

The approach of using overlays and cathodic protection systems every 30 years gives and life cycle cost of $32.96 per square feet, which is a little higher than scenario 3 through 7. However, the overall deterioration rate for the concrete deck is smallest for this type of maintenance strategy and the structure may have a service life much longer than 100 years if it is desirable. The combination of overlay and cathodic protection systems will elongate the service life of the bridge deck without the need for deck replacement, which is favorable for bridges that have high daily traffic volume or the user cost is extremely high for deck replacement activities.

8.8 Summary

The cost analysis based on the presumed deterioration curves is conducted for the preventive maintenance strategies. However, the deterioration curve is case sensitive due to factors affecting the actual deterioration rate of the element such as the properties of the concrete, age, exposure conditions, and effectiveness of the current preventive maintenance systems and even history of the preservation actions applied on the concrete bridges. Also, the real cost for each actions may deviate from the values assumed in the cost analysis presented in this report. On a real project, the actual cost should be identified as well as the actual deterioration rate of the bridge element, given it specific surrounding environment. The deterioration rate could be estimated from the mathematical deterioration models or it can also be regressed from data analysis for a group of bridges in a similar condition.

For a concrete bridge deck, the most cost effective preventive maintenance is the combination of sealers, Asphalt with waterproof membrane, overlays and deck replacement in order to fully utilize the service life of the bridge deck and the preventive maintenance systems. The triggers for each preventive maintenance measure could be adjusted within a small range in order to accommodate this goal.

The cost of sealers is strongly depending on the service life of the sealer. The service life of the sealer ranges from 2-7 years. If the service life of the sealer is only 3 years compared to the 6 year assumed in the analysis, then the cost of sealer will double, which will have a great impact on the total cost.

The effectiveness of the overlay is also a crucial factor to the life cycle cost. The service life of the overlay is not only depending on the condition state of the overlay itself, it also affected by the condition state of the bridge deck beneath it. The service life of the overlay is shortened as the bridge deck deteriorating and ageing. Once the condition states of the bridge deck drops to a certain level, it might be economical to replace the bridge deck instead of reapplying the overlays.

The cost for hybrid systems of overlays and cathodic protection systems seems to be higher than other preventive maintenance strategies. However, the remaining service life after 100-year period is much long than the other alternatives. The actual deterioration rate for the concrete deck is the lowest since the deck will be corrosion free through its entire service life as long as the protection systems are functioning well.

The impact of the uncertainty for the deterioration rates, effectiveness, service life of the preventive maintenance measures, and associated costs are not discussed in this analysis. However, a future investigation will be helpful in order to determine the sensitivity of the life cycle cost affected by these factors.

The life cycle cost analysis may yield to different solution for components of superstructure and substructure. The difference of minimum required concrete cover, exposure condition will result in a different deterioration rates. Also, due to the complexity and time-consuming nature of replacing these element, cathodic protection systems is an essential preventive maintenance alternative for superstructure and substructure elements.

Compared to the long term cost due to the maintenance activities needed for maintaining the concrete bridge, the investment for using stainless steel or MMFX steel in the construction phase may be economical. Due to the high resistance of these types of steel, minimum maintenance is needed through a 100-year period. The difference in the capital cost will not be significant since the cost of reinforcement is only a small portion for the material cost, even though the unit price for stainless steel or MMFX steel is much higher than black steel. In addition, glass fiber reinforced polymer rebars could be a practical alternative to steel bars in bridge decks. It is also recommended to add corrosion inhibiters to the concrete mix, as it will prolong the service life of the concrete components.

8.9 References

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Chapter 9 Summary and Conlusion

A cost-effective bridge is a bridge whose maintenance is based on its chemical condition over its entire service life. If deterioration mechanisms are prevented, a bridge would cost considerably less to maintain and safely serves its full design service life, if not longer. The practice of physical evaluation and delayed maintenance of deteriorated concrete bridges has resulted in large number of bridges in need of repair. The current inspection manuals are primarily focused on detecting physical damage in concrete bridge elements. If no physical damage is detected, very minimal maintenance actions are taken. This is the main reason why the number of US bridges that are classified as structurally deficient is on the rise. Given the need for future expansion of the US transportation network and increase in number of new bridges, there is a need for cost-effective maintenance process that prevents deterioration mechanism from starting, or at least stops it at a very early stage.

The most economical approach to maintain existing concrete bridges is by adopting an active preventive maintenance approach. An in-depth investigation of the combined deterioration effects of various deterioration mechanisms is needed to establish sound thresholds for harmful chemicals in concrete bridge elements. Such established thresholds are critical for cost-effective maintenance decision making, in a timely fashion, before any deterioration starts.

This report presents the economy of preventive maintenance for concrete bridges. It presents an in-depth chemical evaluation and preventive maintenance of existing highway concrete bridges. In addition, it presents implementation of the proposed preventive maintenance approach that is based on quantitative assessment of the material chemical condition of the bridge.

Based on this investigation, the following recommendations are proposed:

1. Cost-effective maintenance of concrete bridges starts with the use of high quality concrete and durable materials.
2. As the corrosion of steel bars is the primary cause of deterioration of concrete bridges, it is recommended to reinforce new concrete bridge decks with GFRP bars, or stainless steel bars, or regular bars along with corrosion protection system.
3. Preventive maintenance of concrete bridges starts on day one; right after the construction has been completed.
4. Bridge inspection should be based on the chemical condition more than on the physical condition of the bridge. There should be a systematic bridge inspection and evaluation of the chemical condition of bridges. Such new approach would allow arresting deterioration mechanism (using preventive maintenance approach) before they start, long before deterioration of concrete and corrosion of steel bars have started.
5. Preventive maintenance approach may not eliminate the need for replacement of bridge decks during the service life of the bridge. Shrinkage, direct traffic wearing, fatigue stresses will continue to produce cracking in bridge decks. These cracks will reduce the service life of bridge decks, and require more aggressive maintenance measures.
6. There is no one preventive maintenance solution for all concrete bridges/bridge elements. The effectiveness of preventive maintenance is very much site dependent, as it is affected

by the quality of concrete, type of traffic, age of bridge, severity of surrounding environment, accurate measurement of the effectiveness of preventive maintenance measures, and the chemical condition of the bridge with and without maintenance measures.

7. Deterioration of concrete bridges is a two-step process; (1) harmful chemical reactions followed by (2) physical deterioration. The cost of freezing or delaying the harmful chemical reactions is much cheaper than the cost of fixing the physical damage.
8. The effective approach to freeze and/or delay the harmful chemical reactions starts with successful assessment of the chemical condition of the bridge through chemical non-destructive testing. In some cases complimented with other NDTs.
9. There is a need to conduct field tests to verify the effectiveness of maintenance actions. Lab test are insufficient to assess the effectiveness of these actions.

Appendix

Appendix A. Approved Sealers by State DOTs

State	Product Name	Manufacturer	Active Ingredient	Concentration	Solvent	Reported Test results			Penetration (in)
						Water weight gain reduction	Absorbed chloride (series II)	Moisture vapor permeability (series II)	
California	Sil-Act ATS 100-LV	Advanced Chemical Technologies	Saline	100%	None	82%	84%		
California	Xiameter OFS 6341*	Dow Corning	Saline	98%					
California	Protectosil Chem-Trete BSM 400-BA	Evonik Industries	Saline	100%	None	85%	86%	100%	
California	SL 100 Water Repellent	Prosoco, Inc	Saline						
California	Loxon 40% Saline Low VOC Water Repellent, A31T00840	Sherwin Williams	Saline	40%			95%		
Maine	Sikagard 7670W Clear	Sika Corporation	acrylic	100%					
Maine	Sealate T70 MX-30	Transpo Industries	HMWM						
Maine	Aquanil Plus 100	ChemMasters	Saline	95%	None	85%	88%	92%	
Maine	Aquanil Plus 40A	ChemMasters	Saline	40%	Acetone/Isopropyl	85%	88%	92%	

Maine	Weather worker 40% J29WB	Dayton Superior	Saline	40%		86%		100%	
Maine	Weather Worker S-100 J29A	Dayton Superior	Saline	90%	Alcohol	87%		100%	
Maine	Baracade Saline 100	Euclid Chemical	Saline	100%	None	89%	91%		0.20
Maine	Sikagard 705 L	Sika Corporation	Saline	100%	None		88%		0.39
Maine	Sikaguad 740 W	Sika Corporation	Saline	40%	Water	85%	96%		
Maine	Certivex Powerseal 40%	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
Maine	Sikagard 701W	Sika Corporation	Saline modified siloxane	20%		91%	90%	100%	
Maine	Protectosil AQUA-TRETE 20	Evonik Degussa	Saline/siloxane	20%	Water	80%	84%	93%	
Maine	Sil-Act ATS-100 LV	Advanced Chemical Technologies		100%	None	82%	84%		
Maine	Aridox 40 M	Anti Hydro							
Maine	Certi-Ven Penseal 244-40% AIM	Vexcon		40%		93.20%	94.60%	95%	0.15
Maine	Certi-vex Penseal 244 100	Vexcon				93.20%	94.60%	95%	0.15
Maine	Powerseal 20	Vexcon			Water	80%		90%	0.15
Massachusetts	SIL-ACT ATS-100LV	Advanced Chemical Technologies	Saline	100%	None	82%	84%		

Massachusetts	Enviroseal 40	BASF	Saline	40%	Water	85%	87%		
Massachusetts	Weather Worker J29A	Dayton Superior	Saline	90%	Alcohol	87%		100%	
Massachusetts	Baracade WB 244	Euclid Chemical	Saline		Water	85%	82%		0.38
Massachusetts	Dynasylan BH-N	Evonik Degussa	Saline	98%	None	86%	87%		0.38
Massachusetts	Protectosil Chem-Trete BSM 40 VOC	Evonik Degussa	Saline	40%	alcohol	86%	87%	100%	
Massachusetts	Powerseal 40	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
Massachusetts	Weatherguard P40 Sealer		Saline	40%					
Massachusetts	SLX100	PROSOCO		93%					
New Hampshire	Sil-Act ATS-100 LV	Advanced Chemical Technologies	Saline	100%	None	82%	84%		
New Hampshire	Powerseal 40	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
New Hampshire	Certi-Vex Penseal 244 100%	Vexcon				93.20%	94.60%	95%	0.15
New Hampshire	Certi-Vex Penseal 244 40% AIM	Vexcon				93.20%	94.60%	95%	0.15
New York	SIL-ACT ATS-100	Advanced Chemical Technologies	Saline	100%	None	88%	89%	100%	
New York	SIL-ACT ATS-100 LV	Advanced Chemical Technologies	Saline	100%	None	82%	84%		
New York	Enviroseal 40	BASF	Saline	40%	Water	85%	87%		
New York	Hydrozo 100	BASF	Saline	100%	None	90%	96%		0.35

New York	Aquanil Plus 100	ChemMasters	Saline	95%	None	85%	88%	92%	
New York	Aquanil Plus 40A	ChemMasters	Saline	40%	Acetone/Isopropyl	85%	88%	92%	
New York	Aquanil Plus 55 IPA	ChemMasters	Saline	55%		85%	88%	92%	
New York	Weather Worker S-100 (J29A)	Dayton Superior	Saline	90%	Alcohol	87%		100%	
New York	Baracade Saline 100	Euclid Chemical	Saline	100%	None	89%	91%		0.20
New York	Protectosil BH-N	Evonik Degussa	Saline	98%	None	86%	87%		0.38
New York	Iso-Flex 618-100	LymTal	Saline	90%	None	89%	90%		0.35
New York	KlereSeal 9100-S	Pecora	Saline	100%	None	85%	99%	102%	0.43
New York	PowerSeal 40	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
New York	Aridox 40	Anti Hydro		40%	Alcohol				
New York	Certi-Vex Penseal 244 BTS-100% (Fast Dry)	Vexcon		100%	None	84%			0.28
New York	Certi-Vex Penseal 244-100 AIM NY DOT	Vexcon		100%	None	93.20%	94.60%	95%	0.15
New York	Certi-Vex Penseal 244-400 AIM NY DOT	Vexcon		55%	Alcohol	93.20%	94.60%	95%	0.15
Ohio	Enviroseal 40	BASF	Saline	40%	Water	85%	87%		
Ohio	Masterprotect H 400	BASF	Saline	40%	Water	85%	87%		
Ohio	Aquanil Plus 40A	ChemMasters	Saline	40%	Acetone/Isopropyl	85%	88%	92%	

Ohio	Rainstopper 120	Textured Coatings of America	Saline	20%	Mineral spirits	80%	82%		
Ohio	Rainstopper 140	Textured Coatings of America	Saline	40%	Mineral spirits	83%	89%		
Ohio	Powerseal 40	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
Rhode Island	Crete Shield	Princeton Construction	Epoxy						
Rhode Island	Sil-Act ATS 42	Advanced Chemical Technologies	Saline	40%	Isopropanol/Methanol	86%	82%	100%	
Rhode Island	Enviroseal 40	BASF	Saline	40%	Water	85%	87%		
Rhode Island	Protectosil BH-N	Evonik Degussa	Saline	98%	None	86%	87%		0.38
Texas	Sil Act ATS 42	Advanced Chemical Technologies	Saline	40%	Isopropanol/Methanol	86%	82%	100%	
Texas	Burke Shield 40 TxDOT	Burke Co.	Saline						
Texas	SW-244-40	Chemical Products Industries, Inc.	Saline	40%	alcohol	80%	92%	100%	
Texas	Weather Worker 40% J29	Dayton Superior	Saline	40%		86%		100%	
Texas	Protectosil 40H	Evonik Degussa	Saline	40%	Alcohol			100%	
Texas	Hydrozo IPA 40	Hydrozo	Saline						
Texas	ICSC S40	Industrial Coatings	Saline						
Texas	Pentane 40	L&M Construction Chemicals	Saline						
Texas	Permaseal 44T	Pilgram Permacoat, Inc.	Saline						

Texas	Consolideck SL 40	PROSOCO	Saline	40%					
Texas	Weather Seal SL 40	PROSOCO	Saline						
Texas	SW-244-401	Sherwin Williams	Saline						
Texas	TPS-CST 40	Texas Polymer Systems	Saline						
Texas	Rainstopper 140	Textured Coatings of America	Saline	40%	Mineral spirits	83%	89%		
Texas	Powerseal 40%	Vexcon	Saline	40%	Water	87%	95%	95%	0.15
Texas	Burke Shield 244 TxDOT	Burke Co.	Siloxane						
Texas	HeyDi’ Siloxan	HeyDi' Americal Corp.	Siloxane						
Texas	WR-165	Marine Industrial Corp., Inc.	Siloxane						
Texas	Consolideck SX	PROSOCO	Siloxane						
Texas	SW 244	Sherwin Williams	Siloxane						
Vermont	SIL-ACT ATS-100	Advanced Chemical Technologies	Saline	100%	None	88%	89%	100%	
Vermont	SIL-ACT ATS-42	Advanced Chemical Technologies	Saline	40%	Isopropanol/Methanol	86%	82%	100%	
Vermont	Masterprotect H 1000	BASF	Saline	100%	None	90%	96%		
Vermont	Masterprotect H 440 HZ	BASF	Saline	40%	Alcohol	86%	99%	102%	
Vermont	Weather Worker J-29WB	Dayton Superior	Saline	40%		86%			
Vermont	Protecosil CIT	Evonik Degussa	Saline	100%	None				

Vermont	Protectosil BH-N	Evonik Degussa	Saline	98%	None	86%	87%		0.38
Vermont	Protectosil Chem-Trete 40 VOC	Evonik Degussa	Saline	50%	Denatured ethyl alcohol	86%	87%	100%	
Vermont	Sure Klean Weather Seal SL-100	PROSOCO	Saline	98%	Methyl Alcohol/Mineral Spirits	90%			
Vermont	Penseal 244 VOC Aim	Vexcon	Saline	40%	Stoddard solvent (petroleum-derived)				

Appendix B. Chloride Threshold Values in the Literature

Threshold Values or Ranges			w/c	(Binding capacity)	Binder type (Governing pH)	Age	Moisture Content	Steel potential (surface condition)	Environment	pH	Reference
Total Cl$^-$ (%wc)	Free Cl$^-$ (% wc)	Cl$^-$ /OH$^-$									
0.4			0.45		100% OPC		65% RH	smooth	laboratory		Richartz [1969]
3			0.6		100% OPC			smooth	laboratory, exposed to air		Gouda and Halaka [1970]
1			0.6		35% GGBS + 65% OPC			smooth	laboratory, exposed to air		Gouda and Halaka [1970]
2.4			-		100% OPC			smooth	laboratory, submerged, but aerated		Gouda and Halaka [1970]
1.2			-		35% GGBS + 65% OPC			smooth	Laboratory, submerged, but aerated		Gouda and Halaka [1970]
0.2 – 1.4			-		various			-	outdoor exposure, exposed to air		Stratfull et al. [1975]
0.4 – 0.8			0.4		100% OPC			cleaned, ribbed	laboratory, exposed to air		Locke and Siman [1980]
0.25 – 0.5			0.5		100% OPC		60% RH	sandblasted	laboratory, submerged		Elsener and Böhni [1986]
0.1 – 0.19			0.45		100% OPC			polished	laboratory, exposed to air		Hope and Ip [1987]
0.1 – 0.19			0.45		100% OPC			polished	outdoor exposure, exposed to air		Hope and Ip [1987]

0.4 – 1.37			0.4 – 0.6		OPC, FA, SRPC, SF, RHPC			cleaned, smooth	laboratory, submerged		Hansson and Sørensen [1990]
0.287			0.4		DK-OPC			cleaned, smooth			Hansson and Sørensen [1990]
0.258			0.45		DK-OPC			cleaned, smooth			Hansson and Sørensen [1990]
0.212			0.5		DK-OPC			Cleaned			Hansson and Sørensen [1990]
0.26			0.5		DK-OPC			As received			Hansson and Sørensen [1990]
0.391			0.5		DK-OPC			Rusted			Hansson and Sørensen [1990]
0.257			0.6		DK-OPC			cleaned, smooth			Hansson and Sørensen [1990]
0.212	71(% of total)		0.5		DK-SRPC(low alkali sulphate resistance portland cement)			cleaned, smooth		12.75	Hansson and Sørensen [1990]
0.237	30(% of total)		0.5		DK-RHPC(Rapid Hardening)			cleaned, smooth		12.38	Hansson and Sørensen [1990]
0.14	28(% of total)		0.5		DK-STD(standard flyash)			cleaned, smooth		12.62	Hansson and Sørensen [1990]
0.099	33(% of total)		0.5		S-SIO2(Swedish OPC+10% microsilica)			cleaned, smooth		12.56	Hansson and Sørensen [1990]
0.5			0.4 – 0.6					-	laboratory, exposed to air		Schiessel and Raupach [1990]
0.5 – 2.0			0.4 – 0.6					-	laboratory, exposed to air		Schiessel and Raupach [1990]

1.5 – 2.5		3 – 20	0.5		OPC, SRPC			cleaned, smooth	laboratory, exposed to air		Lambert et al. [1991]
0.125			-		70% OPC + 30% SF			-	laboratory, submerged		Takagi et al. [1991]
0.5 – 1.8	0.36 – 3.22 mole/l		0.4 – 0.6		OPC, SF, FA			cleaned	laboratory, exposed to air		Pettersson [1992]
0.5 – 1.0			0.5 – 0.7		100% OPC			ribbed	laboratory, exposed to air		Schiessl and Breit [1996]
1.0 – 1.5			0.5 – 0.7		OPC with GGBS or FA			ribbed	laboratory, exposed to air		Schiessl and Breit [1996]
0.7			0.32 – 0.68		OPC with Fly ash content = 0			ribbed	outdoor exposure, tidal zone (marine exposure)		Thomas et al. [1996]
0.65			0.32 – 0.68		OPC with Fly ash content = 15%				outdoor exposure, tidal zone (marine exposure)		Thomas et al. [1996]
0.5			0.32 – 0.68		OPC with Fly ash content = 30%				outdoor exposure, tidal zone (marine exposure)		Thomas et al. [1996]
0.2			0.32 – 0.68		OPC with Fly ash content = 50%				outdoor exposure, tidal zone (marine exposure)		Thomas et al. [1996]
	0.44 – 0.65 mole/l		0.75		100% OPC			not reported	laboratory, submerged		Elsener et al. [1997]
	0.056 mole/l	0.26	-		-			cleaned	laboratory, submerged		Breit [1998]

0.25 – 0.75			0.5 – 0.6		100% OPC, 100% SRPC and OPC with SF, FA or GGBS.			smooth	laboratory, submerged		Breit [1998]
0.4 – 1.5			0.3 – 0.75		100% SRPC or SRPC with FA, SF and GGBS			ribbed, as recieved	outdoor exposure (seawater)		Sandberg [1998]
1.24 – 3.08	0.39 – 1.16 % cem wt	1.17 – 3.98	0.5		100% OPC		100% RH	ribbed and smooth	laboratory		Alonso et al. [2000]
		0.7 – 1.7	-		-			sandblasted, cleaned	laboratory, oxygen supply		Zimmermann et al. [2000]
0.25 – 1.25	0.045 – 0.55 mole/l		0.6		100% OPC			sandblasted, cleaned	laboratory, exposed to air		Zimmermann et al. [2000]
0.2 – 0.4			-		100% OPC			-	outdoor exposure		Zimmermann [2000]
		0.01 – 2.5	-		-			as received, sandblasted, pre-rusted	laboratory, submerged		Li and Sagüés [2001]
0.73	0.50 % cem wt	1.76±0.3	0.5		OPC, SRPC, FA			ribbed, millscaled	laboratory submerged		Alonso et al. [2002]
0.23	0.36 mole/l	1.5	0.37		100% SRPC			ribbed	laboratory, exposed to air		Castellote et al. [2002]
0.15	0.33 mole/l	2	0.37		100% SRPC				laboratory, exposed to air		Castellote et al. [2002]
0.4			0.4 – 0.6		100% OPC (~7 and ~12% C_3A)		(23C, 50% RH)		laboratory, exposed to air		Whiting et al. [2002]
0.4			0.4 – 0.6		75% OPC + 25% FA (Class C and F)		(23C, 50% RH)		laboratory, exposed to air		Whiting et al. [2002]

0.02 – 0.24		0.05 – 0.62	0.5		100% OPC				laboratory, submerged		Trejo and Pillai [2003]
0.68 – 0.97	0.07 – 0.13 % cem wt	0.16 – 0.26	0.35 – 0.55		OPC with 15 to 30% FA or 30% GGBS				laboratory, exposed to air		Oh et al. [2003]
0.45	0.10 % cem wt	0.27	0.35 – 0.55		SRPC				laboratory, exposed to air		Oh et al. [2003]
0.52 – 0.75			0.45		100% OPC		exposed to air (22% RH and 30˚C for 60 days) then submerged		laboratory		Nygaard and Geiker [2005]
	0.4 – 0.8 % cem wt		0.5		100% OPC				laboratory, exposed to air		Mohammed and Hamada [2006]
1.1 – 2.0			0.6		100% OPC				laboratory, exposed to air		Manera et al. [2007]
0.6 – 1.2			0.6		90% OPC + 10% SF				laboratory, exposed to air		Manera et al. [2007]
1.1-2.0			0.6		OPC				laboratory, exposed to air		Manera et al. [2007]
1-1.5					High alkali OPC						Fagerlund et al. [2011]
0.7-0.9					Low alkali OPC						Fagerlund et al. [2011]
0.35			0.45	$C_b=7.2C/(1+4.3C)$	OPC with 2.43% of C_3A						Glass et al. [1997]
0.62			0.45	$C_b=7.2C/(1+3.2C)$	OPC with 7.59% of C_3A						Glass et al. [1997]

1			0.45	Cb=1.8C/(1+1.9C)	OPC with 14% of C3A						Glass et al. [1997]

Appendix C. Chloride Diffusion Coefficient Published in Literature

D_c ($*10^{-9}$ cm^2/s)	w/c	Binder type	Permeability coefficient ($*10^{-13}$ m/s)	Compressive Strength (MPA)	Air content	Density (kg/m3)	Oxygen Diffusion Coefficient ($*10^{-8}$ cm^2/s)	Test Performed	Test Duration	Reference
10.6	0.4	OPC		42.4	7	2298		ACPT	12-14 weeks	(Sugiyama, 1996)
23.1	0.4	OPC		25.7	6.5	1837		ACPT	12-14 weeks	(Sugiyama, 1996)
11.8	0.6	OPC		27	7	2314		ACPT	12-14 weeks	(Sugiyama, 1996)
25.3	0.6	OPC		24.7	6	1934		ACPT	12-14 weeks	(Sugiyama, 1996)
11.9-19.4	0.5	OPC		59.5	1.2			AASHTO T 277-83		(Zhang, 1994)
28.6-36.5	0.5	OPC Mortar		38	3.9			AASHTO T 277-83		(Zhang, 1994)
67	0.38	OPC				2010				(Liu, 2011)
60	0.38	OPC	0.9	71		2360				(Liu, 2011)
NA	0.54	OPC	14.7	49		2290				(Liu, 2011)
53	0.38	OPC	1.1	50		1900				(Liu, 2011)
59	0.38	OPC	1.9	47		1860				(Liu, 2011)
64	0.38	OPC	1.6	42		1740				(Liu, 2011)
NA	0.38	OPC	1.2	38		1610				(Liu, 2011)

90	0.38	OPC	4	34		1620				(Liu, 2011)
31.2	0.5	OPC						Spray Zone		(Costa, 1999)
53.2	0.5	OPC						Tidal Zone		(Costa, 1999)
1.21	0.5	OPC						Atmospheric Zone		(Costa, 1999)
30.4	0.5	OPC						Dockyard		(Costa, 1999)
16	0.3	OPC with superplasticiser						Spray Zone		(Costa, 1999)
å7.7	0.3	OPC with superplasticiser						Atmospheric Zone		(Costa, 1999)
13.1	0.3	OPC with superplasticiser						Dockyard		(Costa, 1999)
13.8	0.35							Spray Zone		(Costa, 1999)
6.7	0.35							Atmospheric Zone		(Costa, 1999)
9	0.35							Dockyard		(Costa, 1999)
39.5	0.4	OPC					9.3		10 weeks	(Ngala, 1995)
78	0.5	OPC					10.4		11 weeks	(Ngala, 1995)
126	0.6	OPC					13.64		12 weeks	(Ngala, 1995)

214.6	0.7	OPC					21.75		13 weeks	(Ngala, 1995)
3.9	0.4	OPC with 30%FA					5.79		14 weeks	(Ngala, 1995)
4.3	0.5	OPC with 30%FA					6.67		15 weeks	(Ngala, 1995)
9	0.6	OPC with 30%FA					7.51		16 weeks	(Ngala, 1995)
10.3	0.7	OPC with 30%FA					8.88		17 weeks	(Ngala, 1995)
32	0.3	OPC	1	100	3.5			NT BUILD 443	35 days	(Elahi, 2010)
9	0.3	7.5%SF	1.018	117.3	3			NT BUILD 444	36 days	(Elahi, 2010)
7.5	0.3	15%SF	0.979	120.8	2.5			NT BUILD 445	37 days	(Elahi, 2010)
9.5	0.3	50% blast-furnace slag	0.969	98.6	2.75			NT BUILD 446	38 days	(Elahi, 2010)
8	0.3	70% blast-furnace slag	1.038	74.3	2.5			NT BUILD 447	39 days	(Elahi, 2010)
12.5	0.3	20% Fly ash	0.733	79.5	3			NT BUILD 448	40 days	(Elahi, 2010)
16	0.3	40% Fly ash	1.16	58	2.5			NT BUILD 449	41 days	(Elahi, 2010)

7	0.3	20% Fly ash+SF	0.92	94.3	2.5			NT BUILD 450	42 days	(Elahi, 2010)
5	0.3	40% FA+SF	1.297	76	2.5			NT BUILD 451	43 days	(Elahi, 2010)
9.13	0.32	CPA-CEMI 52 5 PMES		72				NFP18305		(Truc, 2000)
28.53	0.32	CPA-CEMI 52 5R 425		92				NFP18306		(Truc, 2000)
1.3	0.55	CPA-CEMI 52 5 PMES		33.5				NFP18307		(Truc, 2000)
23.2	0.55	CPA-CEMI 52 5R 425		34.5				NFP18308		(Truc, 2000)
44.7	0.5	OPC								(Page, 1981)
14.7	0.5	OPC with 30%FA								(Page, 1981)
4.1	0.5	OPC with 30% BFS								(Page, 1981)
100	0.5	SRPC								(Page, 1981)
87	0.4	OPC							150 tidal cycle	(Mangat, 1987)
67.7	0.4	OPC with steel fiber							150 tidal cycle	(Mangat, 1987)
20	0.67	OPC		26				Fickian Law	775 days	(Vedalakshmi ,2009)

16	0.54	OPC		36				Fickian Law	775 days	(Vedalakshmi ,2009)
19	0.42	OPC		47				Fickian Law	775 days	(Vedalakshmi ,2009)
48.4	0.67	OPC		26				Warburg diffusion coefficient	775 days	(Vedalakshmi ,2009)
36.7	0.54	OPC		36				Warburg diffusion coefficient	775 days	(Vedalakshmi ,2009)
22.3	0.42	OPC		47				Warburg diffusion coefficient	775 days	(Vedalakshmi ,2009)
35.2-46.6		OPC						Tidal/ Splash Zone	16 years	(Funahashi, 1990)
21.3-33.9	0.5	OPC						Tidal/ Splash Zone	24 years	(Liam, 1992)
35		OPC						Tidal/ Splash Zone	20 years	(Kudoh, 1991)
44.1-4.91	0.5	OPC						Tidal/ Splash Zone	30 years	(Mustafa and Yusof, 1994)
120	0.4	OPC						Aerated	1 year	(Bentz, 1996)

13	0.4	OPC with 30%FA						submerged specimen	1 year	(Bamforth and Price, 1982)
65.3	0.66	OPC						submerged specimen	1 year	(Bamforth and Price, 1982)
8.9	0.54	OPC with 30%FA						Tidal Zone	3 years	(Bamforth and Price, 1982)
7.6	0.48	OPC with 70%FFBS						Tidal Zone	3 years	(Bamforth and Price, 1982)
39.8	0.72	OPC with 8% SF						Tidal Zone	3 years	(Bamforth and Price, 1982)
21.42	0.45	OPC						Submerged specimen	15 years	(Mohammed, 2002)
4.86	0.45	GGBS						Submerged specimen	15 years	(Mohammed, 2002)
5.52	0.45	PFA						Submerged specimen	15 years	(Mohammed, 2002)
0.42	0.44-0.6	OPC						Tidal/ Splash Zone	33 years	(Troconis de Rinco'n, 2004)
6.48	0.44-0.7	OPC						Tidal/ Splash Zone	38 years	(Troconis de Rinco'n, 2004)
0.27	0.44-0.8	OPC						Tidal/ Splash Zone	60 years	(Troconis de Rinco'n, 2004)

1.36	0.44-0.9	OPC						Tidal/ Splash Zone	64 years	(Troconis de Rinco'n, 2004)
9.7	0.45	OPC						Freeze and Thaw 0 Cycles		(Gérard,2000)
24.48	0.45	OPC						Freeze and Thaw 31 Cycles		(Gérard,2000)
41.64	0.45	OPC						Freeze and Thaw 61 Cycles		(Gérard,2000)
76.52	0.45	OPC						Freeze and Thaw 95 Cycles		(Gérard,2000)

Appendix D. Survey Questionnaire

General

1. What is the percentage of the concrete bridges that are structurally deficient (SD) and functional obsolete (FO) during last year? And what is the target percentage of the concrete bridges that are structurally deficient (SD) and functional obsolete (FO) that your agency predict in 2020 year?

	In the past year				In 2020			
	≤10%	10%~20%	20%~30%	≥30%	≤10%	10%~20%	20%~30%	≥30%
SD	☐	☐	☐	☐	☐	☐	☐	☐
FO	☐	☐	☐	☐	☐	☐	☐	☐

2. What percentage of work has been assigned to the following areas?
   Bridge Inspection: ____%
   Maintenance and Minor Repair Activities: ____%

3. What approximate percentage of total annual bridge funding (from all sources) goes toward the following areas?
   Bridge Inspection: ____% Routine/Preventive Maintenance: ____%
   Repair/Rehabilitation: ____% Bridge replacement: ____%

Inspection

4. Who does conduct the NDT in your state?
   ☐ In-house staff ☐ External staff ☐ In-house and external staff

5. Who does analysis and interpret the NDT data?
   ☐ Inspection Specialist ☐ Maintenance Engineer ☐ Structural Engineer ☐ Material engineer

6. Who does integrate the NDT data into the bridge management systems?
   ☐ Inspection Specialist ☐ Maintenance Engineer ☐ Structural Engineer ☐ Material engineer

7. What are the NDT tests conducted on the following bridges? Please identify the bridge element where the tests will be applied.

	Inspection Method			Bridge component		
Bridge Condition	Sounding Test	Advanced Test	Chemical NDT	Bridge deck	Superstructure	Substructure
Relatively new	☐	☐	☐	☐	☐	☐
Moderately deteriorated	☐	☐	☐	☐	☐	☐
Severely deteriorated	☐	☐	☐	☐	☐	☐

8. What advanced testing methods does your agency use?
   ☐ Ground penetrating radar (GPR) ☐ Infrared thermography (IR)
   ☐ Ultrasonic testing ☐ Impact echo (IE) testing
   ☐ Others: __________ ☐ Others: __________

9. What Chemical NDTs does your agency use? Please identify how often the tests are considered and the main purpose of performing the tests.

	NDT Usage by type			Maximum Interval (years)			Main Purpose		
	Very often	Often	Rare or never	≤2	2~5	>5	Preventive Maintenance	Routine Maintenance	Repair and Rehabilitation
ASR reaction test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Carbonation test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Chloride content test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Concrete penetrability test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Air/Water permeability test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Moisture content test	☐	☐	☐	☐	☐	☐	☐	☐	☐
Others: ______________ ______________	☐	☐	☐	☐	☐	☐	☐	☐	☐

Maintenance

10. Does your department adopt preventive maintenance policy for bridges?

☐ Yes ☐ No

If so, do you have preventive maintenance manual?

☐ Yes (please specify): ____

☐ No

11. How do you measure the effectiveness of the preventive maintenance measures?

12. How effective are the following preventive maintenance methods? What is the maximum interval for the activity? Are they cost-effective?

Activity	Effective	Moderate	Ineffective	Max. Interval	Cost-Effective
Cleaning of the structure	☐	☐	☐	________	Yes No
Lubricate expansion bearings	☐	☐	☐	________	Yes No
Correct approach panel settlement	☐	☐	☐	________	Yes No
Crack seal	☐	☐	☐	________	Yes No
Deck/Superstructure/ Substructure seal	☐	☐	☐	________	Yes No
Concrete coating	☐	☐	☐	________	Yes No
Flush winter residue	☐	☐	☐	________	Yes No

13. What is the percentage of bridges where the following measures have been applied?

Cathodic Protection: ____%   Electrochemical Chloride Extraction:____%

14. What are typical costs (labor and materials) associated with: (Please identify the unit cost)
    - ☐ Bridge cleaning (decks, joints, bearings, other elements): __________________________
    - ☐ Deck sealing: ____________________
    - ☐ Concrete superstructure/substructure sealing: ______________________
    - ☐ Any other common bridge maintenance activities like crack sealing or deck polymer overlays: _______________________

15. What is the typical service life of the following bridge components?

Bridge deck: _____   Superstructure: _____   Substructure: _____   Expansion Joints: _____

16. What are the major maintenance activities does your agency apply?

☐ Preventive Maintenance   ☐ Active Maintenance   ☐ Reactive Maintenance

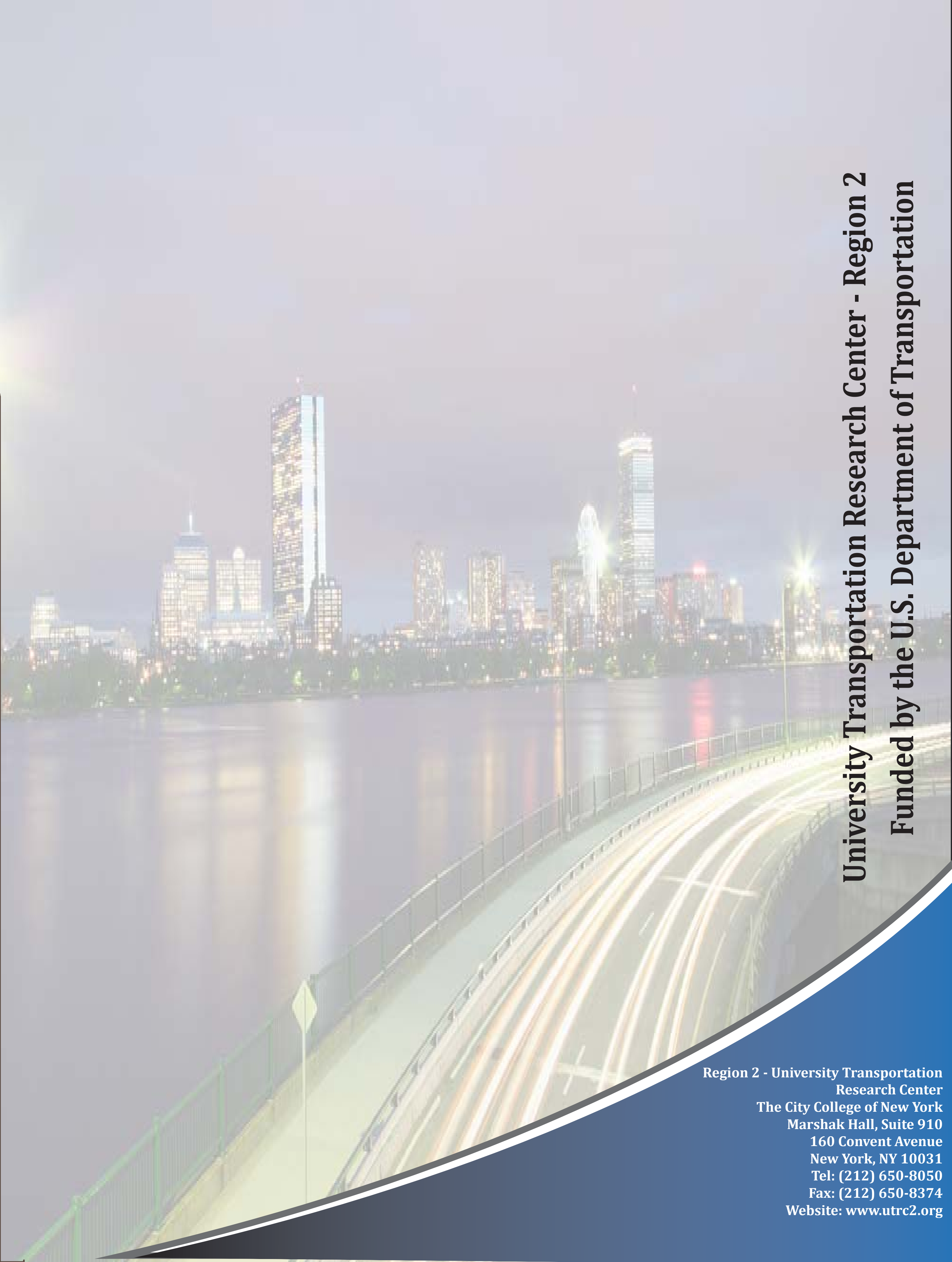
17. Does your state consider life-cycle cost analysis when comparing alternatives for the following types of bridge projects?

Bridge Replacements: Yes / No   Bridge Rehabilitation: Yes / No   Bridge Preservation: Yes / No

18. How do you consider the current maintenance practices?

☐Very effective   ☐Effective   ☐Not Effective

Comments:



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Funded by the U.S. Department of Transportation

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