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Service Life Prediction of Protective Systems for Concrete Bridge Decks in Alberta

Prévision de la longévité des systèmes de protection des tabliers de ponts en béton, dans l'Alberta

Vorhersage der Lebensdauer von Schutzsystemen für Betonbrückenfahrbahnen in Alberta

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SUMMARY

This paper summarises certain aspects of deck reinforcement corrosion damage in a recently completed service life prediction study. It describes reinforced concrete deck damage as it relates to deck electrical potential levels. The paper also relates deck active half cell levels to time. Together these two relationships permit service life predictions for certain types of reinforced concrete deck systems subjected to corrosive chloride based de-cing salt environments.

RÉSUMÉ

Les auteurs résument certains aspects concernant l'endommagement des armatures des tabliers de ponts en béton. Ce rapport fait partie d'un récent examen sur les prévisions de la durée de vie, qui décrit les relations entre les dommages subis par les dalles en béton armé et les champs de différence de potentiel électrique qui y règnent. L'article donne le comportement temporel des valeurs électriques des demi-cellules actives de ces mêmes dalles. Ces observations permettent de prédire la longévité de certains types de tabliers de ponts en béton armé soumis à la corrosion du sel de dégel.

ZUSAMMENFASSUNG

Dieser Bericht ist eine Zusammenfassung bestimmter Aspekte der Beschädigung der Bewehrung in Betonfahrbahndecken. Er ist ein Teil einer kürzlich beendeten Untersuchung über Lebensdauervoraussagen und beschreibt die Beziehung, die zwischen der Beschädigung von bewehrten Betonfahrbahndecken und elektrischen Spannungsfeldern auf Fahrbahndecken besteht. Weiterhin ist beschrieben, wie sich aktive elektrische Halbzellenwerte auf Fahrbahndecken über die Zeit verhalten. Zusammen erlauben diese beiden Beobachtungen eine Lebensdauervoraussage für bestimmte Typen von Fahrbahndeckensystemen aus bewehrtem Beton, die der Korrosion durch Tausalz ausgesetzt sind.

1. INTRODUCTION

A significant number of bridge structures were introduced into the road networks of Canada and other countries influenced by freezing weather conditions during and prior to the time that roadway de-icing salts began to be used. The decks of these structures were designed and built prior to the recognition by the bridge engineering community that significant corrosion-induced damage would generally occur to the decks upon application of chloride based roadway de-icing salts. Over the last fifteen to twenty years the introduction of various new and existing technologies into deck construction, including epoxy coated reinforcement, membranes, sealers and significantly less permeable concretes, has curtailed this problem in new structures. However, the multitude of structures that pre-date these new developments have required and will continue to require maintenance and rehabilitation to offset the effects of de-icing salt induced corrosion. The study [1] that is summarized within this paper resulted in the development of models that are considered to be useful in characterizing corrosion induced damage as it relates to measurable electrical potential levels within older reinforced concrete deck systems. As well, predictive guidelines and models, that were originally developed in the study, for assessing service life of unprotected and protected deck systems used in Alberta, are presented herein. All of the relationships developed within the study were based to a large degree upon data collected by Alberta Transportation and Utilities (AT&U) pertaining to bridge structures located throughout the Province. Where data was lacking or incomplete "expert" opinion of experienced bridge engineers was employed in order to complete the development of the prediction models. Specific details about the data are presented within the original study [1].

The advent of bonded concrete overlays in the range of 50 mm to 100 mm thickness, sometimes used to enhance the service life of corrosion susceptible bridge decks, has led to overlay debonding as a second form of deck system failure, distinct from the corrosion induced mechanism. Debonding as a form of deck system failure was investigated and reported upon within the original study[1], but debonding is not reported upon within this paper.

2. BACKGROUND

2.1 Alberta's Climate and Its Impact on the Bridge System

Alberta is a landlocked province located in Western Canada (Figure 1), situated at great distance from major water bodies. Its climate is relatively dry with varying amounts of precipitation and cyclic air temperature dependent upon latitude and longitude within the province. Furthermore, the general weather patterns vary throughout the year as the climate fluctuates with four distinct seasons during the course of the year. Average daytime temperatures in the summer months are typically well above freezing, and during the winter months the average temperatures are below freezing. Spring and autumn are transitionary times of the year during which frequent freeze/thaw events occur.

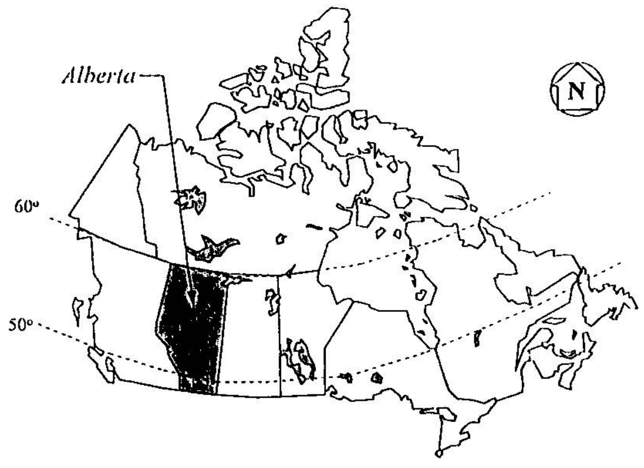

Figure 1 Map of Canada Showing Location of Alberta

One of the most negative impacts to the Alberta roadway bridge system has resulted from Alberta's freezing and thawing climate which has led to the widespread use of de-icing salts during the last 20 years, and for longer periods of time in certain portions of the Province. As it is now well known ([2] for instance), the application of chloride based de-icing salts to unprotected reinforced concrete elements generally results in reinforcement corrosion and associated concrete damage in the form of delamination and spalling. Unprotected, reinforced concrete decks, which exist on a significant number of the Alberta bridge structures, have consequently been affected by these circumstances.

2.2 Service Life and Failure

The *service life* of a bridge deck and/or its overlay, within the context of this study, is considered to be the period of time from deck and/or overlay installation to when the deck system fails to perform in an acceptable manner due to some predetermined level of damage occurring. Experience in Alberta suggests that the two major reasons for damage and ensuing failure are i) deck reinforcement corrosion resulting in delaminations and subsequent deck and/or overlay deterioration and ii) debonding of an overlay from the concrete deck. The corrosion-induced damage phenomenon is the focus of this paper.

Damage levels generally build up over time and it is considered that there is a threshold level of damage above which a deck system is considered completely failed. Complete failure is either due to the deck becoming unsafe structurally or due to it developing an unacceptably poor ridability resulting from damage levels that are too great to permit selective repairs. Levels of damage which preceed the threshold level sometimes manifest in the form of minor spalling and/or potholes and some limited amount of associated poor ridability. However, levels of damage that have proven economical to repair are often limited to internal delamination, cracking and debonding that are invisible, but which can be detected by non-destructive diagnostic methods such as chain drag delamination surveys and/or electrical potential corrosion surveys. Figure 2 depicts a generic life cycle for a bridge deck subjected to a corrosive environment and a repair application. It presents characteristic stages which are considered pertinent to this subject matter.

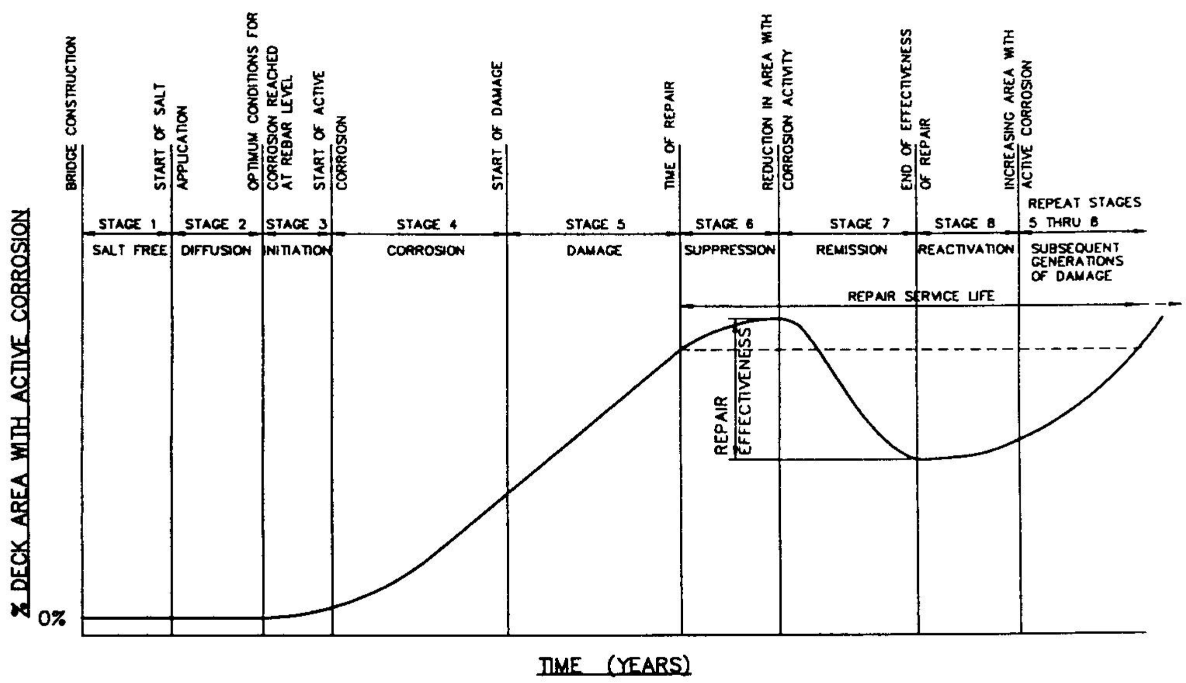

Figure 2 Bridge Deck Corrosion Life Cycle

NOTES TO FIGURE 2

Stage 1 - Salt Free - period of time from date of construction to start of de-icing salt application
Stage 2 - Diffusion - period of time from start of salting to the time that optimum conditions for reinforcement corrosion are reached
Stage 3 - Initiation - period of time to overcome protective barriers to corrosion and until active corrosion potentials can be measured
Stage 4 - Corrosion - period of time from the start of active reinforcement corrosion to when damage(spalling) is observed
Stage 5 - Damage - period of time from the start of observed corrosion induced damage to the time of repair
Stage 6 - Suppression - period of time from performing a repair or installing a protective system until reduction in deck areas of active corrosion potentials
Stage 7 - Remission - period of time during which the deck area with active corrosion potentials declines due to the effect of the repair
Stage 8 - Reactivation - the period of time during which the effectiveness of a repair begins to decline and measured deck areas with active corrosion potentials begin to increase
Stage 9 - Subsequent Generation Damage - the period of time during which deck area with active corrosion potentials is again increasing until some quantity of damage is again reached that results in another repair

3. CORROSION DAMAGE PREDICTION USING ELECTRICAL POTENTIAL SURVEYS

3.1 Relating Damage to Electrical Potentials

Based upon the premise that deck reinforcement corrosion activity can be predicted to exist by active ASTM copper/copper sulphate electrode (CSE) half cell potential readings [3], it is proposed that a mathematical relationship exists relating the area of corrosion induced damage within a bridge deck to the measured level of active CSE readings. This proposed relationship is as follows:

$$\text{Damage} = K \times CSE + A \qquad (1)$$

where Damage = % area of deck damaged due to rebar corrosion
K = a coefficient defining the rate of increase of Damage with time
A = a constant.
CSE = % total deck area with CSE readings more negative than some specified active level, such as -300 mV

It is likely that the relational coefficient "K" and the constant "A" vary depending upon both the specified level of CSE activity considered (eg. -300 mV vs. -350 mV vs. -400 mV etc.) as well as due to a series of physical and environmental characteristics that can vary between bridges, including such factors as deck reinforcement density, reinforcement cover, concrete strength/permeability, salt application rates, deck moisture levels, bridge girder type, influence of deck repairs and/or overlays.

Attempts to develop damage prediction equations that directly account for most of the many influencers were unsuccessful at this time. Comparisons of measured deck damage with measured CSE data from 66 different bridge structures repaired primarily during the 1980's, with the only variable considered being whether or not the bridge girder system was concrete or steel, resulted in the following relationships for predicting corrosion-induced damage in reinforced site cast concrete decks:

$$\text{Damage(concrete girder)} = 0.182 \times CSE_{300} - 3.90 \qquad R^2 = 0.84 \qquad (2)$$

$$\text{Damage(steel girder)} = 0.0894 \times CSE_{300} - 1.25 \qquad R^2 = 0.16 \qquad (3)$$

where CSE_{300} = the % total deck area with CSE potentials < -300 mV

The data was separated and relationships developed for the two categories of girder types because of the perception, based upon observations of experienced field personel, that the two types of bridges tend to perform differently, probably due to the inherent differences in flexibility and thermal sensitivity. Figure 3 shows equations (2) and (3) plotted, and it appears from this derivation that corrosion-induced damage begins to develop when CSE readings are more negative than -300 mV over about 10% to 25% of the deck area. Figure 3 also shows that *on average* less damage occurs in steel girder bridge decks than for concrete girder decks at a given level of CSE measured corrosion activity. It is hypothesized that the presence of the steel girder tends to increase the corrosion activity measured within the deck, but that this additional activity associated with the steel girder is not likely causing any additional damage to the deck.However, it is important to note that the variability of damage observed on steel girder decks, as represented by the reliability index, R^2, is significantly greater.

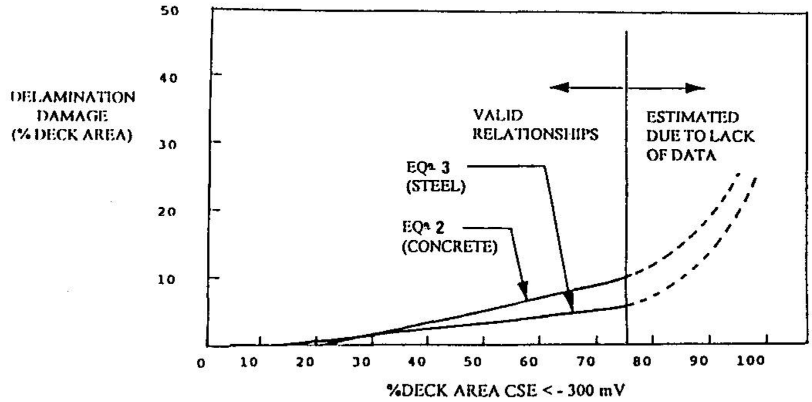

Figure 3 Damage vs.CSE , -300 mV

Equations relating Damage to average levels of CSE readings and to CSE levels more negative than -400 mV have been developed as well and are presented in the original study [1]. AT&U initiated its CSE half cell monitoring program in 1977, prior to the publication of ASTM C876. At that time, AT&U decided to select the -300 mV level to represent active corrosion. AT&U has used -300 mV consistently in its ongoing monitoring program having never adopted the corrosion activity level of -350 mV stipulated in ASTM C876.

3.2 Relating CSE Potentials to Time

A primary goal of this research was to develop service life prediction relationships that would permit the prediction of time to a defined level of deck damage, given an easily measurable deck characteristic like CSE potentials. A relationship complimentary to the Damage vs. CSE relationship (1) of the previous section is proposed to facilitate this prediction, and it relates the CSE levels in a deck system with time. The following relationship is considered to be appropriate for the majority of the corrosion life cycle during which corrosion rate of increase is considered to be linear (Stages 4 & 5 as per Figure 2):

$$CSE_{300} = C \times \text{time} + B \tag{4}$$

where CSE_{300} = % of total deck area with CSE potentials < -300 mV
C = a coefficient defining the rate of increase of CSE_{300} with time
B = a constant.

This general relationship has been developed into a series of specific relationships applicable to various bridge deck conditions and systems through simple linear regression analysis of CSE data from several hundred bridge sites. In the data analysis process it was discovered that some bridge decks for a given protective system category performed better than other decks and so *Poor, Average* and *Good* performance sub-categories were established. Non-linear relationships have also been developed to represent Stages 3, 6, 7 and 8 of the corrosion cycle but they are not presented here (refer to [1]). Coefficients developed for use in equation (4) for three bridge deck protective systems are as follows:

PROTECTIVE SYSTEM	CORROSION STAGE	Slope C	Constant B	Reliability Index (R^2)
None - Exposed Concrete				
- Poor Performance	4&5	10.6	-104	0.83
- Average Performance	4&5	5.7	-104	0.86
- Good Performance	4&5	2.2	-57	0.12
Asphalt Covered Concrete				
- Poor Performance	4&5	11.8	-56.6	0.5
- Average Performance	4&5	3.75	-56.6	0.63
New LSDC Overlays on Concrete*				
- Poor Performance	4&5	4.60	0	.29
- Good Performance	4&5	0.80	-2.6	.61

** LSDC - Low slump dense concrete*

The sub-categories *Poor, Average* and *Good* are believed to result directly from environmental and physical characteristics unique to a bridge. Structure type, structure flexibility, traffic levels (which are likely often closely related to de-icing salt application rates) and freez/thaw cycles are all considered to influence the performance of a bridge deck. With the presently available data these factors cannot be incorporated into the prediction equations directly, and presently these important influencers are subject to some engineering judgement in the service life prediction process that is presented in the following section.

4. SERVICE LIFE PREDICTION OF DECK PROTECTION SYSTEMS

Application of the relationships presented in Sections 3.1 and 3.2, in conjunction with engineering assesment of influencing factors has resulted in the development of Table 1 Bridge Deck (Corrosion) Service Life Prediction, which lists the expected service life for site cast reinforced concrete bridge decks utilizing various protective systems presently in use in Alberta.

Two types of service life are listed within Table 1, the definitions of which have been established to be as follows:

Repairable Service Life is represented by 5% damage which corresponds to 40 to 65% of a deck having CSE_{300} levels.

Failure Service Life is represented by 60 to 80% damage which corresponds to 90 to 100% of a deck having CSE_{300} levels.

Some of the systems listed in the table have no supporting analysis at this time because sufficient data does not exist to undertake accurate analysis. It is anticipated that the data will soon be available, and the framework for analysis proposed herein and applied to the older deck systems will be appropriate to replace the listed service life predictions, now based upon expert opinion, with analytically supported predictions.

TABLE 1 BRIDGE DECK (CORROSION) SERVICE LIFE PREDICTION

PROTECTION SYSTEM	BASIC SERVICE AVERAGE LIFE		ADJUSTMENTS TO BASIC AVERAGE SERVICE LIFE DUE TO VARIOUS INFLUENCERS									
			GIRDER TYPE		TRAFFIC INTENSITY			FREEZE THAW CYCLES		DECK DRAINAGE		
	OPTIMUM REPAIR 5% DAMAGE	FAILURE >60% DAMAGE	CONCRETE GIRDERS	STEEL GIRDERS	ADT > 5000	ADT 5000> > 600	ADT < 600	> 115 PER YEAR	< 90 PER YEAR	BAD	GOOD	EPOXY MEMBRANE
AS CONSTRUCTED												
EXPOSED CONCRETE DECKS	20	30	1	-3	-5	0	10	-3	2	-2	2	16
ASPHALT COVERED DECKS	18	32	0	-3	-7	-1	7	-4	2	-3	3	n/a
NEVER REPAIRED LSDC OVERLAY	20	40	0	-5	-6	0	12	-3	3	-3	2	10
AS REPAIRED 1												
SILICA FUME CONCRETE OVERLAY	18	35	0	-5	-5	0	10	-3	3	-3	3	15
THIN LATEX MODIFIED CONCRETE OVERLAY	10	20	0	-6	-7	0	8	-3	3	-3	3	15
MEMBRANE AND ASPHALT OVERLAY	16	28	0	-3	-5	0	7	-3	3	-4	4	n/a
50 mm LSDC OVERLAY	15	27	0	-5	-6	0	10	-3	3	-3	3	15

NOTE 1 - All "AS REPAIRED" predictions listed in this Table are derived by expert opinion and are not based upon data analysis.

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