The Long Term Performance of Three Ontario Bridges Constructed with Galvanized Reinforcement

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ABSTRACT

This report summarizes the results of a research program undertaken by the Ontario Ministry of Transportation to assess the corrosion protection provided by galvanized reinforcement. Three Ontario structures were studied, built in 1975 and 1976 using galvanized reinforcement and construction practices typical of the time period. The investigation included periodic visual surveys of the structures as well as measurement of corrosion potentials, corrosion currents and degree of delamination of the concrete. Concrete quality and chloride content were also assessed. Findings of the evaluation indicate that the long-term (30-year) performance of galvanized reinforcement, while marginally better than that of conventional black reinforcement, showed evidence of corrosion and resulting delamination of the concrete when the chloride content of the concrete exceeded the threshold to initiate corrosion. On one structure approximately 10% of the deck had deteriorated and required rehabilitation before achieving even a twenty-year service life. Based on the structures surveyed, galvanized reinforcement in the Ontario highway environment did not provide the anticipated corrosion protection.

Key Words Corrosion, chloride ion, de-icing salts, highway bridges, reinforced concrete, delaminations, galvanized reinforcement.

INTRODUCTION

Uncontaminated Portland cement concrete provides excellent protection for steel reinforcement, both by acting as a barrier to corrosive ions and by providing a highly alkaline environment in which black steel passivates and corrodes at a negligible rate.

However, when a structure is exposed to an aggressive chloride environment, or if the design details or workmanship are inadequate, the concrete protection may break down and corrosion of the reinforcement will occur.

Galvanized reinforcing steel is black carbon steel that has been protected using a metallurgically bonded zinc coating. Typically, the thickness of the coating is 75 to 125 microns and it is applied using a conventional hot dip galvanizing process.

Galvanizing generally performs well in a neutral pH environment such as the atmosphere, however, in a high pH environment, such as concrete, the performance has been variable. Research conducted in 1988 by Andrade and Macius [(1)] explained that the corrosion rates of galvanized rebar may differ by an order of magnitude, depending on the alkali content of the cement. The lower the alkali content the better the corrosion performance of galvanized rebar. In the United States, the F.H.W.A.[(2)] has noted that due to environmental regulations, the alkali contents in cements are increasing which will decrease the service life of galvanized rebar.

Opinions regarding the long-term performance of galvanized reinforcement in concrete are generally broken down into two schools of thought. The galvanizing industry has promoted galvanized reinforcement as an effective and economical substitute for black reinforcement in those situations where black reinforcement will not have adequate durability. The industry states that this superior performance of hot dip galvanizing is due to the two-fold nature of the coating, since the coating is a barrier to chlorides and also provides sacrificial corrosion protection to the underlying black steel. Extended steel service life of up to 50 years with little or no maintenance has been reported by the galvanizing industry for bridges using galvanized reinforcement in Bermuda[(3)].

Many researchers and end users hold a less positive opinion on the performance of galvanized reinforcement. Research has generally shown that galvanizing is a less effective option than fusion-bonded epoxy coated reinforcement (ECR) in terms of resistance to chloride ions [(4)]. Galvanizing has one significant potential advantage over ECR in that it is expected to tolerate damage in handling, as the coating corrodes sacrificially and so defects are not as detrimental to performance as is coating damage to ECR.

Galvanizing has been used in many countries for a number of years and the reported long-term performance is mixed. As early as 1982, American Concrete Institute (ACI) Committee 345[(5)] stated “At best, galvanized steel reinforcement is no better than non-galvanized reinforcement; further it is probably not as good as far as corrosion is concerned.” An FHWA memorandum quoted in [(6)] suggests a 15-year life for galvanized rebar in good quality concrete. The FHWA conclusions were supported by Andrade et al [7] with respect to chloride attack. Researcher K. Clear [8] reported that galvanized rebar, when used in both mats in a bridge deck, will outperform black steel by only two years. McCrumb and Arnold [9] conclude that galvanized reinforcement will add five years to the service life of a bridge compared to black reinforcement.

Due in part to the variable reports of performance of galvanized reinforcement presented in the literature, the Ontario Ministry of Transportation's Bridge Durability Work Group initiated a project to assess the long-term performance of three Ontario structures constructed with galvanized reinforcement. The intent was that this evaluation would provide the ministry with verification of the corrosion performance that galvanized reinforcement offered in the Ontario environment and thus assist in determining its potential for future ministry use. Although MTO had never specified and does not currently specify the use of galvanized reinforcement for bridges, they were aware of other jurisdictions in the province that had used galvanized reinforcement on a trial basis in an attempt to improve the corrosion performance and durability of bridges.

PROTECTION MECHANISMS

The theory of how galvanizing works is well documented in the literature. Hot-dipped zinc (galvanized) coatings for reinforcing bars have been used since the early 1940's [10] and the specification requirements for these coated bars are contained in ASTM A767. After coating, the current ASTM A767 requires that the bars be chromate treated. This step is to preclude the adverse reaction of the zinc coated bars and fresh Portland cement paste (hydrogen evolution); however some research [2] indicates that this process is unnecessary. The life of the zinc coating is directly proportional to its thickness and the nature of the environment to which it is exposed.

Galvanizing produces a tough and adherent alloy layer coating, which is metallurgically bonded to the base steel. The coating material acts as a barrier to the steel by isolating it from corrosion inducing chemicals. Active metal coatings on steel such as zinc and aluminium provide both primary barrier protection as well as secondary cathodic protection where the coating is damaged and the substrate exposed.

It has been postulated [10] that zinc corrosion products, usually zinc oxide, are less voluminous than iron corrosion products and thus develop lower expansive pressures in the concrete. They also tend to be friable in that they are loose and powdery minerals rather than bulky hard phases, and they migrate away from the reinforcement surface and fill cracks and voids in the concrete cover. Also, it has been reported that there is a delay in the initiation of corrosion since the zinc can tolerate chloride levels several times higher than that required to cause corrosion of black steel reinforcement[10].

The rate of corrosion is considered to be considerably below that of ferrous metals and depends on the environment. While a fresh zinc metal surface is quite reactive, the zinc metal forms a thin film of corrosion products when exposed to the atmosphere. The film of corrosion products transforms into a dense, transparent barrier layer that prevents strong attack on the zinc metal, is not water-soluble and erodes slowly over time.

The total life of a galvanized coating in concrete is made up of the time taken for the zinc to depassivate (which is longer than that for black steel), plus the time taken for the dissolution of the alloy layers in the zinc coating. Only after the coating has fully dissolved in a region of the bar will localized corrosion of the steel begin.

PROJECT SCOPE AND INSPECTION METHODS

The purpose of the project was to investigate the long-term performance of galvanized steel in selected concrete bridge decks subject to chloride application during their service life. There were concerns as to whether the galvanizing could provide long-term protection to the reinforcing bar when high levels of chloride ions accumulate at the depth of the reinforcement.

Investigative work on the bridge decks included the following techniques, which are discussed in more detail below:

1. Measurement of electrical potentials on the top mat of reinforcing steel (ASTM C876)
2. Measurement of acid soluble chloride ion contents of concrete at the depth of embedded steel reinforcement (internal MTO specification LS-417 "Method of Test for Determination of Total (Acid Soluble) Chloride Ion Content in Concrete".
3. Measurement of corrosion currents of top mat reinforcing bars using the linear polarization technique (non-standard).
4. Inspection of the structures to identify evidence of concrete deterioration in the form of visible cracking, spalling or other corrosion induced damage, cracking (ACI 224.IR).
5. Chain drag survey of exposed deck areas to detect subsurface delaminations (ASTM D4580).
6. Petographic examination and compressive strength testing of representative concrete core samples to determine concrete quality. (Compressive strength CSA A23.2-9C).
7. Measurement of clear concrete cover to the reinforcement, using a covermeter (non standard).
8. Concrete resistivity measurement using a Wenner 4 point probe (non standard)

<u>Potential Testing</u>

The potential of embedded reinforcing steel may be measured against a portable reference electrode with a high impedance voltmeter. The test results can provide useful information on the condition of the structure. According to ASTM C-876 if the steel reinforcement is passive the potential measured is small (0 to-200 mv) against a copper/copper sulphate cell. If the passive layer is failing and increasing amounts of steel are dissolving the potential moves towards –350mv. At more negative than -350mv the steel is usually corroding actively. The interpretation of the active/passive steel reinforcement in concrete is based on empirical observation of the probability of corrosion in structures containing black steel. However a means of interpreting half-cell data is not currently available in the literature for galvanized reinforcement in concrete. However, valuable information was provided in monitoring the half-cell potential of galvanized reinforcement for 30 years in both chloride free and chloride contaminated concrete.

Chloride Ion Analysis

It is generally accepted that the presence of chloride ions at the surface of embedded reinforcement can disrupt the normal passive behaviour of embedded reinforcement and initiate corrosion. Chlorides act as catalysts to corrosion when there is sufficient concentration at the steel surface to break down the passive layer. They are not consumed in the process but help to break down the passive layer of oxide on the steel and allow the corrosion to proceed quickly.

Total chloride tests are performed on acid digestions of the concrete sample. The results from these tests will include all chlorides present in the concrete. It is preferable to collect a series of drillings at different depths so that a chloride profile can be produced. The literature contains reports of a variety of different chloride concentrations associated with initiation of corrosion of steel in concrete. The ratio of Cl/OH, as well as oxygen availability influence the rate and the threshold for corrosion. Other variables such as cement type, mix design and the environment impact both the pH and the oxygen access and hence the precise corrosion threshold.

A conservative threshold value for corrosion initiation based on acid soluble chloride is about 0.025% by weight of concrete and is the value used in this report.

Corrosion Rate Measurement

Several electrochemical methods have been developed to provide information on the actual rate of corrosion of reinforcing steel. The measurement method used for this investigation was based on the linear polarization resistance method; also known as linear polarization, where the current and potential relationship close to the corrosion potential is determined. The technique is based on the fact that the d.c. current required to alter the natural electrical half-cell potential of the steel a few milivolts (10 mv.) is proportional to the natural corrosion rate of that steel. If a high current is required, the corrosion rate is high and vice versa.

Although this method has been largely used to access the corrosion rate of black reinforcement it can also be used to monitor the corrosion rate of other reinforcement such as stainless steel and galvanized reinforcement.

Visual Inspections and Sounding for Delaminations

Visual inspections included observation and documentation of the extent of concrete cracks, spalls and rust staining and any other evidence of corrosion activity. Any evidence of previous repairs was also noted.

As corrosion proceeds, the corrosion product formed takes up a larger volume than the steel consumed – building up stresses around the rebars and potentially causing a planar fracture at the level of the steel before the concrete spalls. Such fractures can be detected at the concrete surface using various means ranging from striking the surface with a chain or hammer and listening for a hollow sound, to sophisticated techniques using radar, infrared, sonic and ultrasonic equipment. In this investigation the chain drug technique was used following ASTM D4580.

Compressive Strength and Air Void Analysis of Concrete Cores

The intent of the physical testing program is to obtain an assessment of the quality and durability of the concrete since it greatly influences corrosion activity. The concrete protects the embedded reinforcement physically and provides a high pH environment which passivates the embedded steel and ensures very low corrosion rates.

Compressive strength testing was carried out an cores according to CSA A23.2-9C. The compressive strength results were compared with the strengths specified on the original construction contracts. Wide variations in strength may indicate local areas of deterioration. Values of less than 20 MPa represent poor quantity concrete.

For current MTO work, concrete is normally considered to be properly air entrained if the air content of the hardened concrete exceeds 3% the spacing factor is less than 0.23 mm and the specific surface exceeds 25 mm^2/mm^3. The test method used was based on ASTM A457.

Measurement of Concrete Cover to the Reinforcement

Cover measurement is carried out on new structures to check that adequate cover has been provided to the steel according to specifications. The depth of concrete cover has a direct relationship with time-to-corrosion. Low cover will increase the corrosion rate both by allowing chlorides and moisture more rapid access to the steel.

The most common commercially available devices for measurement of cover, known as pachometers or covermeters use magnetic properties to determine the cover depth. There are several handheld devices on the market. Once activated the covermeters generate a magnetic field. When an external magnetic material is present such as a reinforcing bar, the magnetic field of the covermeter is distorted. The magnitude of the distortion is proportional to the size of the reinforcing bar and its distance from the probe.

Concrete Resistivity Measurements

Since corrosion is an electrochemical phenomenon, the electrical resistivity of the concrete has a bearing on the corrosion rate of the concrete, as an ionic current must pass from the anodes to the cathodes for corrosion to occur. The four-probe resitivity meter or Wenner probe was developed for measuring soil resistivity, and can be modified to measure concrete resistivity on site. In the four pin version current is applied between the two outer probes and the potential difference measured across the two inner probes. In one commercial device a wetting solution is applied to the sponges at the end of the pins and the pins are placed on the concrete surface. Resistivity is strongly affected by concrete quality (cement content, water-cement ratio, curing and admixtures used).

Interpretation of results is empirical. Previous work has shown that if resistivity measurements from the Wenner four-probe system are greater than 20 kΩ.cm a low corrosion rate is expected. If the resistivity readings are between 10-20 kΩ.cm low to moderate corrosion rates are expected. If the readings are below 10 kΩ.cm high corrosion rates are expected[6].

DESCRIPTION OF TEST SITES

The three bridges studied in this investigation were not constructed by MTO, but by three Ontario municipalities. While MTO has never specified the use of galvanized reinforcement for bridges, there are a very small number of such bridges on the municipal network.

1. Victoria Street Bridge – Wingham

The Victoria Street Bridge in Wingham, Ontario, was built by Huron County in 1975 and is located in a rural environment. The deck is a post-tensioned, 3 span continuous thick slab design. The concrete deck measures approximately 52 metres by 10 metres. All the reinforcing steel was galvanized but not chromate treated. The structure is shown in Figure 1.

2. Bridge Street Bridge – Dorchester

The Dorchester Bridge was originally built in the 1920's, carrying Bridge Street over the south branch of the Thames River. This rural bridge has four simply supported main spans. In 1976 the deck was replaced with supporting stringers and beams strengthened to accommodate a wider deck utilizing galvanized reinforcing steel. The deck surface is exposed concrete and is approximately 44 metres long and 7 metres wide. (Figure 2).

3. Bathurst Street Bridge Over Nordheimer Ravine - Toronto

The Bathurst Street Bridge is an four-span 143 metre long concrete reinforced deck on prestressed concrete girders, in an urban setting. The bridge is located in the city of Toronto and has an over all width of 26 metres. The bridge was built in 1975 using galvanized reinforcement in the deck, sidewalk, and barrier wall. The deck was waterproofed at the time of construction with a rubberized asphalt membrane overlayed with bituminous wearing surface. Figure 3.

RESULTS OF FIELD INVESTIGATION

Victoria Street Bridge - Wingham

A condition survey was conducted on the bridge deck in each of the following years: 1975, 1976, 1977, 1978, 1979, 1980, 1981, 1985, 1995 , 1998, 2001, 2002 and 2004. On the deck a grid was used to measure the galvanized reinforcement potentials and as an aid in evaluating concrete distress on the bridge deck.

A summary of the copper/copper sulphate half-cell readings for the years 1975 to 2004 are reported in Table 1 and graphically depicted in Figure 7.

Initially the average corrosion potentials measured were approximately – 1.0 volt. This potential slowly increased to – 0.27 volts and remained there for approximately ten years after construction. This value represents a passive corrosion potential for galvanized reinforcement embedded in concrete. After this passive potential was reached in 1985 the potential slowly decreased during the following 19 years indicating a loss of passivity.

The concrete cover to the reinforcing steel averaged 34 mm for the sidewalk and 59 mm for the deck.

Based on analysis of cores from the structure, it appears that the chloride ion corrosion threshold for regular steel was reached at the depth of the reinforcement in about 1995 (approximately 0.025% chloride based on mass of concrete after correction for background chlorides). During the 1998 investigation the first cracks and delaminations on the deck were detected (Figure 4). The average corrosion potential of the steel at that time was – 0.35 volts, with values as low as – 0.52 v.

The delaminations covered approximately 0.2% of the deck area in 1998. The delaminations grew to approximately 0.5% of the deck area in 2001, and by 2004 approximately 1.2% of the deck area was delaminated. Figure 8 shows the progress of delaminations at the Victoria Street Bridge.

In 1995 a number of rust stains first became evident along cracks in both the sidewalk and the parapet walls.

During the 1995 condition survey corrosion currents were measured at 21 representative locations on the deck, using the linear polarization method. In 2004, 12 locations were tested using the same methodology and equipment. In 1995 the average corrosion current was calculated to be 1.07 $\mu a/cm^2$. In 2004 the average corrosion current was calculated to be 2.55 $\mu a/cm^{2.}$ (For conventional black reinforcement, a corrosion current in the range of 1.0 to 10 $\mu a/cm^2$ corrosion damage is expected in 2 to 10 years based on data from National Bureau of Standards 3LP equipment manufacture’s data).

Table 2 shows the chloride ion profile obtained on cores from the deck in the 1995 and 2004 investigations. In 1995 the chloride corrosion threshold was reached at the level of the reinforcement. In 2004 the chloride level was six times threshold at that level. Concrete resistivity measurements were taken on the bridge in 2001 at 14 representative samples. The readings averaged 25 kΩ.cm. Based on reference[(6)] if resistivity measurements from the Wenner four-probe system are greater than 20 kΩ.cm a low corrosion rate is expected.

Bridge Street Bridge- Dorchester

A condition survey of the Dorchester Bridge was conducted by the consulting firm M.M. Dillon Limited in July 1994 and reported in [(11)]. The surface of the deck was sounded to locate areas of delamination, and visible defects such as patches were also recorded. Eighteen years after construction, the exposed concrete deck was described by the firm as "in fair condition with numerous areas of delminations and a few spalls. Several areas of the deck had been previously patched with cementitious and asphaltic materials" (Figure 5).

Concrete cores were removed and analyzed to assess the acid soluble chloride ion. After adjusting for the background chloride, the chloride ion content at the level of the steel was calculated to be in excess of that required to initiate corrosion for conventional black reinforcement (Table 3).

The consultant found significant correlation between the deck areas that demonstrated concrete deterioration and areas where concrete cover was low and corrosion potentials were high (large negative potentials). The average concrete cover for steel in the deck was calculated to be 51 mm. However, the average concrete cover in delaminated areas was calculated to be 27 mm. The average corrosion potential in the deck was – 0.38v (relative to copper/copper sulphate). In delaminated areas the average potential was – 0.51v.

Condition surveys were carried out by the MTO in 1995 and 2003.

In July 1995 a condition survey was conducted by MTO personnel. The condition survey included the same tasks as the previous survey but also utilized corrosion current measurements of the top reinforcement in the deck by the linear polarization technique. In this survey approximately 10% of the deck area contained delaminations or patched areas. The average half-cell potential on the deck was – 0.45v. Ninety percent of the potential readings were more negative than – 0.35v, the remainder were between – 0.20 and – 0.35v. These values correlated well with the previous year consultant's condition survey when 82% of the readings were more negative than – 0.35v. Concrete cores removed from the deck and analyzed for the chloride ion confirmed previous results that the chloride ion corrosion threshold for black reinforcement was exceeded at the level of the reinforcement (Table 3). Linear polarization testing was carried out at twenty representative locations. The average corrosion current was calculated to be 2.25 $\mu a/cm^2$.

In 2003, MTO again conducted a condition on the bridge. The survey indicated that approximately 15% of the deck surface was delaminated or had already received a repaired treatment, an increase of 5% since the condition survey was conducted in 1995. The average half-cell potential on the deck was – 0.47v. Essentially all (99.6%) of the potential readings were more negative than – 0.35v. The potentials ranged from – 0.35 to – 0.69v.

Bathurst Street Bridge Over Nordheimer Ravine – Toronto

(A) Bridge Deck

A condition survey was conducted in 1995 by the consulting firm Davroc and Associates Ltd. , twenty years after construction of the bridge, and results are reported in [12]. The bituminous surfacing was reported to be in fair condition with numerous sealed transverse cracks at the east and west shoulders of the bridge and several sealed longitudinal cracks in the driving lanes. The waterproofing membrane was found to be generally well bonded to the concrete deck and to the asphalt.

The condition survey included removal of selective areas of the asphalt surfacing and underlying waterproofing membrane, to expose the concrete deck surface. Based on these sample areas the consultant indicated, "the deck slab was found to be in good condition".

Covermeter readings to the top reinforcement ranged from 20 to 70 mm and averaged 49 mm. The compressive strength of the concrete was found to be 63.0 MPa on average. (In 1975 the design-strength likely would have been specified as 4000 psi (27.5 MPa)).

The chloride content of the concrete deck was well below the threshold value required to cause corrosion. The reinforcement at the core locations was found to be in good condition with no evidence of rust formation.

Measurement of the concrete air void parameters indicated that the concrete was air-entrained, however, the specific surface and spacing factor were outside the currently accepted limits to provide freeze thaw durability.

In 2004 the condition of the Bathurst Street Bridge was surveyed by the consulting firm Morrison Hershfield and results reported in [13]. The concrete deck slab was "generally found in good condition".

Corrosion potentials indicated active corrosion over approximately 1% of the total deck surface. The majority of the deck area fell into the category (based on black steel reinforcement) of inactive corrosion activity of less than -.20v.

The survey included removal of 37 cores and sixteen sawn samples from the asphalt covered bridge deck. Chloride content was measured in cores C-18, C-20 and C-26. Core C-18 taken from an area of active corrosion, core C-20 from an area of uncertain corrosion area, and cores C-26 taken from an area that appeared to be inactive with respect to corrosion. The chloride corrosion threshold was exceeded at the level of the reinforcement in all three cores (Table 4).

Galvanized reinforcing steel bars were encountered in 36 out of 37 cores in the concrete deck slab. The galvanized coating layer was found to be consumed in 16 of the core samples leaving a blackened bar surface. Except for core C-25 where a concrete delamination was encountered at the 35 mm depth, there was no significant deterioration of the concrete deck in any of the cores and sawn asphalt samples. The amount of delamination on the deck was estimated to be small, less than 0.5% of the entire deck area.

The relatively good performance of the deck was generally attributed to the success of the waterproofing in reducing the ingress of the chloride ion from the majority of the deck (at least in the early years). It appears that the waterproofing is no longer effective, as chlorides have penetrated the deck to the point that the chloride corrosion threshold has been exceeded at the level of the reinforcement.

(B) Parapet Walls

In 1995 the east and west parapet walls were inspected by the consulting firm[12] and found to be in fair condition with several medium and wide vertical cracks present. Light rust stains were evident on the concrete surface corresponding to the location of the longitudinal reinforcement.

In 2004 the concrete parapet walls and aluminium railings were found by the consulting firm[13] to be in good condition. There were a few vertical narrow and medium cracks along the inside face of the concrete walls. No delaminations were detected by hammer sounding the walls.

(C) Sidewalks

In 1995 the east and west sidewalks were found to be in fair condition with numerous narrow to wide longitudinal cracks and several medium to wide transverse cracks present. Rust stains were evident along the longitudinal cracks at several locations corresponding to the position of the top transverse reinforcement (Figure 6).

Based on covermeter readings the cover to the top reinforcement of the east sidewalk varied from 30 to 100 mm with an average of 61 mm. Based on the core samples obtained from the sidewalk the concrete was found to be delaminated at the level of the transverse reinforcement. The cored reinforcement was found to be in a rusted to severely rusted condition. Two cores were removed to determine the chloride ion penetration. The chloride ion values at the level of the reinforcement were well above the threshold value and are shown in Table 4.

In 1995 the MTO sounded the sidewalks for delaminations. The west sidewalk was delaminated over approximately 2.6% of the entire surface area while the east sidewalk was delaminated over approximately 4.7% of the entire surface area.

In 2004, MTO personnel again sounded the west and east sidewalk area in order to update their performance. The west sidewalk had 10.3% delaminations and the east sidewalk had 12.3% delamination.

This is a significant increase over the extent of delaminations detected since the previous survey.

DISCUSSION OF RESULTS

Victoria Street Bridge - Wingham

In 1995 the chloride corrosion threshold for black steel reinforcement had been reached at the average level of the reinforcement and by 2004 the chloride ion level had increased to approximately six times threshold. The results of testing on the bridge are consistent with this increase in that they confirm that there was increasing corrosion activity between 1995 to 2004. From 1995 to 2004 the amount of delaminations increased from 0 to 1.2%. The half-cell potential readings became more negative and increased from an average of – 0.36V to – 0.44 volts. Corrosion currents as measured by the linear polarization technique increased from 1.07 $\mu a/cm^2$ to 2.55 $\mu a/cm^2$.

The service life extension provided by the galvanized reinforcement about three years, (1995 to 1998) appears to be similar to or marginally better than black steel reinforcement. Also, the progression of delaminations once initiated appears to be better (lower) than that expected from black steel reinforcement.

Bridge Street Bridge - Dorchester

Of the three bridges investigated the Dorchester Bridge demonstrated the highest level of corrosion activity and related deterioration. In 1995, 19 years after construction, approximately 10% of the deck had showed deterioration due to corrosion or had already received rehabilitation. The chloride ion content at the level of the reinforcement had reached approximately three times the threshold necessary to initiate corrosion. It is not clear when the chloride threshold reached the reinforcement so the service life extension provided by the galvanized reinforcement cannot be accurately established. During the period from 1995 to 2003 delaminations increased from 10% to 15% of the deck surface area. For comparison, a bridge deck under MTO jurisdiction would be a candidate for bridge deck rehabilitation once the amount of delamination exceeded 10% of the deck area, typically receiving a concrete overlay. The growth rate of the delaminations 1995-2003 appear to be less than what would be expected for black steel reinforcement.

Bathurst Street Bridge Over Nordheimer Ravine - Toronto

Of the three bridges investigated this bridge exhibited the best performance. Overall it appears that this was because the waterproofing membrane had been effective in reducing the ingress of the chloride ion. When tested in 1995 the chloride corrosion threshold had not been reached at the level of the reinforcement. By 2004 the waterproofing membrane had ceased to provide protection, as was evident as all three cores tested for chloride ion content had chloride levels exceeding the chloride corrosion threshold at the depth of the reinforcement. The first delamination was detected in the bridge deck in 2004, a considerably later age than for the other two structures.

However, on the sidewalks of this structure, which were not waterproofed and where the concrete surface was exposed, the extent of delamination increase from 4.7% to 12.3% on the east side and from 2.6% to 10.3% on the west side between 1995 to 2004. This is evidence of active corrosion of the galvanized reinforcement, causing destruction of the concrete, at a rate that would appear to be similar to what would be expected for black steel in a similarly exposed situation. This may be explained in that once the galvanizing is consumed the bar behaves like black steel reinforcement.

SUMMARY AND CONCLUSION

The performance of galvanized reinforcing steel was evaluated at three different Ontario bridge decks located in corrosive environments due to deicing salt exposure. Results of the work revealed that galvanized reinforcing steel showed evidence of corrosion, and resulting damage to the concrete, when the chloride corrosion threshold for black steel was exceeded for any significant length of time. The extent of the corrosion damage was shown to be extensive.

Based on the findings of this investigation, the following conclusions have been drawn.

1. Corrosion of galvanized reinforcing bars was initiated soon after the chloride corrosion threshold (for black steel) was reached at the top level of reinforcement.
2. Corrosion of the galvanized reinforcement caused significant damage to the concrete, in the form of delamination and cracking. In the most severely damaged of the three structures, 10% of the surface area was delaminated or had already been repaired, within 20 years of construction.
3. Galvanized reinforcing bars are not recommended as the primary or sole means of corrosion protection for structures exposed in the Ontario highway environment. Based on the findings of this study, they do not provide effective long-term protection from corrosion.
4. The use of a waterproofing membrane was effective in providing protection from chloride ingress, although effectiveness decreased over the long-term. This finding supports the ministry's current policy of application of waterproofing to all new bridge decks, and maintenance of waterproofing decks.
5. Adequate concrete cover to the steel, and good quality concrete, also played a significant role in slowing the rate of chloride ingress, to prolong time to corrosion.

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Table 1

Victoria Street Bridge, Wingham

Half-cell Corrosion Potential*and Concrete Cover to Reinforcing Steel

Year	Average	Standard Deviation	Range	
Bridge Deck Corrosion Potential (V)				
1975	0.98	0.05	0.90	1.17
1976	0.57	0.04	0.47	0.66
1977	0.72	0.03	0.64	0.78
1978	0.27	0.03	0.17	0.35
1979	0.30	0.03	0.18	0.35
1980	0.31	0.03	0.20	0.36
1981	0.31	0.03	0.22	0.39
1985	0.27	0.05	0.17	0.38
1995	0.36	0.05	0.21	0.50
1998	0.35	0.07	0.16	0.52
2001	0.40	0.07	0.21	0.58
2002	0.41	0.07	0.24	0.64
2004	0.44	0.07	0.25	0.61
Sidewalk Corrosion Potential (V)				
1995	0.43	0.06	0.32	0.58
1998	0.41	0.06	0.24	0.54
2001	0.41	0.06	0.24	0.54
2002	0.45	0.06	0.32	0.55
2204	0.51	0.06	0.39	0.61
Concrete Cover to Reinforcing Steel (mm)				
Sidewalk	34	10.5	19	58
Deck	59	9.9	38	82

***Note negative signs are omitted for corrosion potentials**

Table 2

Chloride Ion Profiles Victoria Street Bridge, Wingham

Sample 1, 1995

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.4932	0.4685
10-20	0.3288	0.3041
20-30	0.1244	0.0997
30-40	0.1934	0.1687
40-50	0.0863	0.0616
50-60	0.0523	0.0276
60-70	0.0391	0.0144
70-80	0.0350	0.0103

Sample 2, 1995

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.3315	0.3068
10-20	0.2866	0.2619
20-30	0.1909	0.1662
30-40	0.1271	0.1024
40-50	0.0734	0.0487
50-60	0.0525	0.0278
60-70	0.0452	0.0205
70-80	0.0258	0.0011
80-90	0.0323	0.0076
90-100	0.0304	0.0057
100-110	0.0247	0

Sample 3, 2004

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.547	0.522
10-20	0.545	0.520
20-30	0.341	0.316
30-40	0.263	0.238
40-50	0.192	0.167
50-60	0.181	0.156

Table 3

Chloride Ion Profiles, Bridge Street Bridge, Dorchester 1995

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.4734	0.4491
10-20	0.5357	0.5114
20-30	0.3982	0.3739
30-40	0.2801	0.2558
40-50	0.1749	0.1506
50-60	0.1090	0.0847
60-70	0.0432	0.0189
70-80	0.0243	0

% chloride based on mass of concrete 1994

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.576	0.552
20-30	0.453	0.429
40-50	0.334	0.310
60-70	0.196	0.172

Table 4

Chloride Ion Profiles Bathurst Street Bridge, Toronto

Sample 1, 1995 Sidewalk

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.4575	0.3948
10-20	0.5059	0.4432
20-30	0.4001	0.3374
30-40	0.3030	0.2403
40-50	0.2760	0.2133
50-60	0.2052	0.1425

Sample 2, 1995 Sidewalk

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.4845	0.4218
10-20	0.4448	0.3821
20-30	0.3277	0.2650
30-40	0.2448	0.1821
40-50	0.1840	0.1213
50-60	0.1161	0.0534
60-70	0.0822	0.0195
70-80	0.0671	0.0044
80-90	0.0626	0
90-100	0.0627	0

Sample 3, 2004 Deck Core C20

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.156	0.126
20-30	0.112	0.092
40-50	0.081	0.052
60-70	0.052	0.023
80-90	0.029	0

Sample 4, 2004 Deck Core C26

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.146	0.117
20-30	0.127	0.098
40-50	0.089	0.060
60-70	0.075	0.046
80-90	0.040	0.011

Sample 5, 2004 Deck Core C18

Depth (mm)	Measured Acid Soluble (% chloride)	Corrected for Background
0-10	0.113	0.085
20-30	0.099	0.070
40-50	0.077	0.048
60-70	0.054	0.025
80-90	0.033	0.004

Figure 1 General View Victoria Street Bridge

Figure 2 General View Bridge Street Bridge

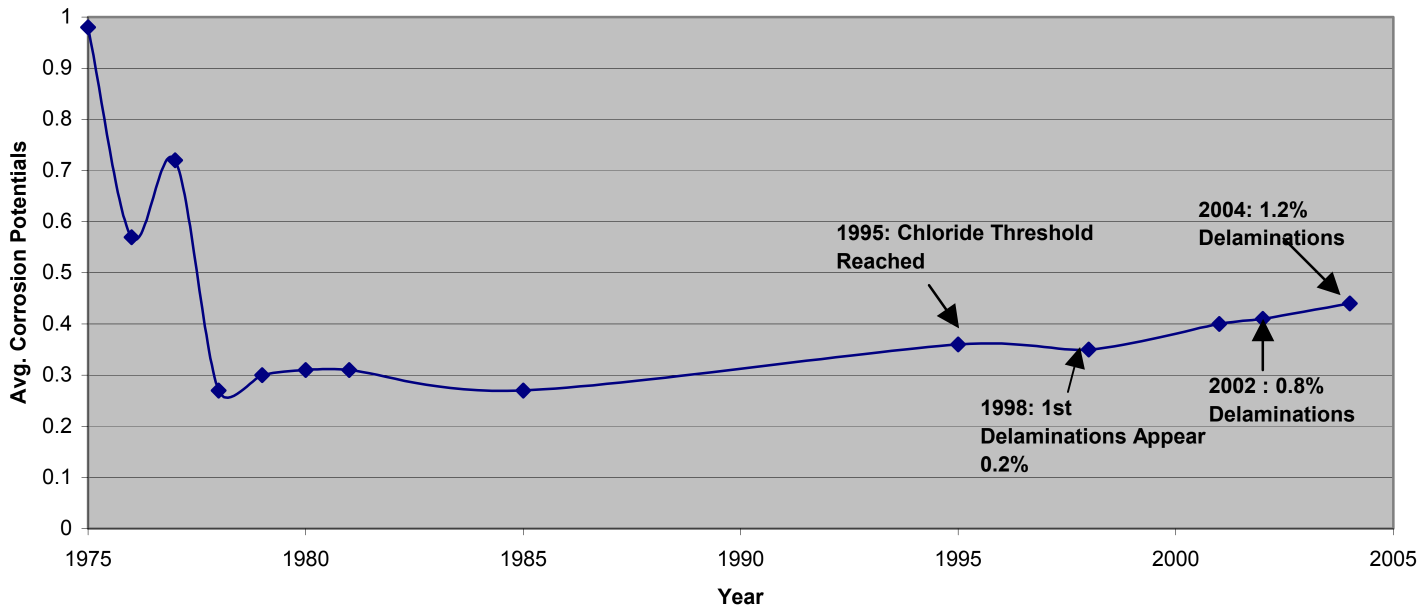
Figure 3 General View Bathurst Street Bridge

Figure 4 Victoria Street Bridge Showing Cracks in Concrete Deck (1998)

Figure 5 Bridge Street Showing Extensive Patches in Deck and Sidewalk

Figure 6 Bathurst Street Showing Cracks and Staining in East Sidewalks

Figure 7 Victoria Street Bridge - Wingham Average Corrosion Potentials



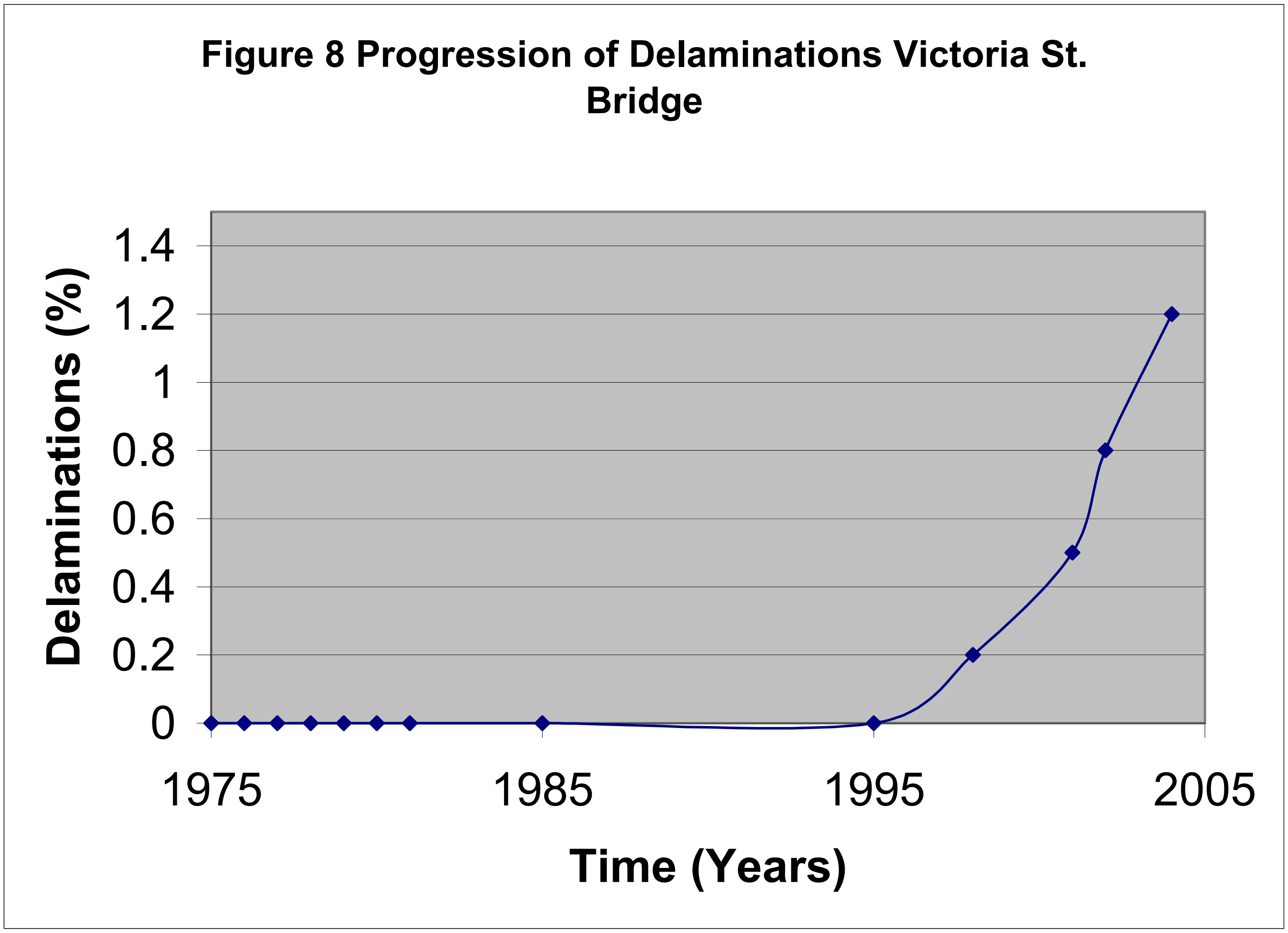


Figure 8 Progression of Delaminations Victoria St. Bridge
Delaminations (%)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1975
1985
1995
2005
Time (Years)

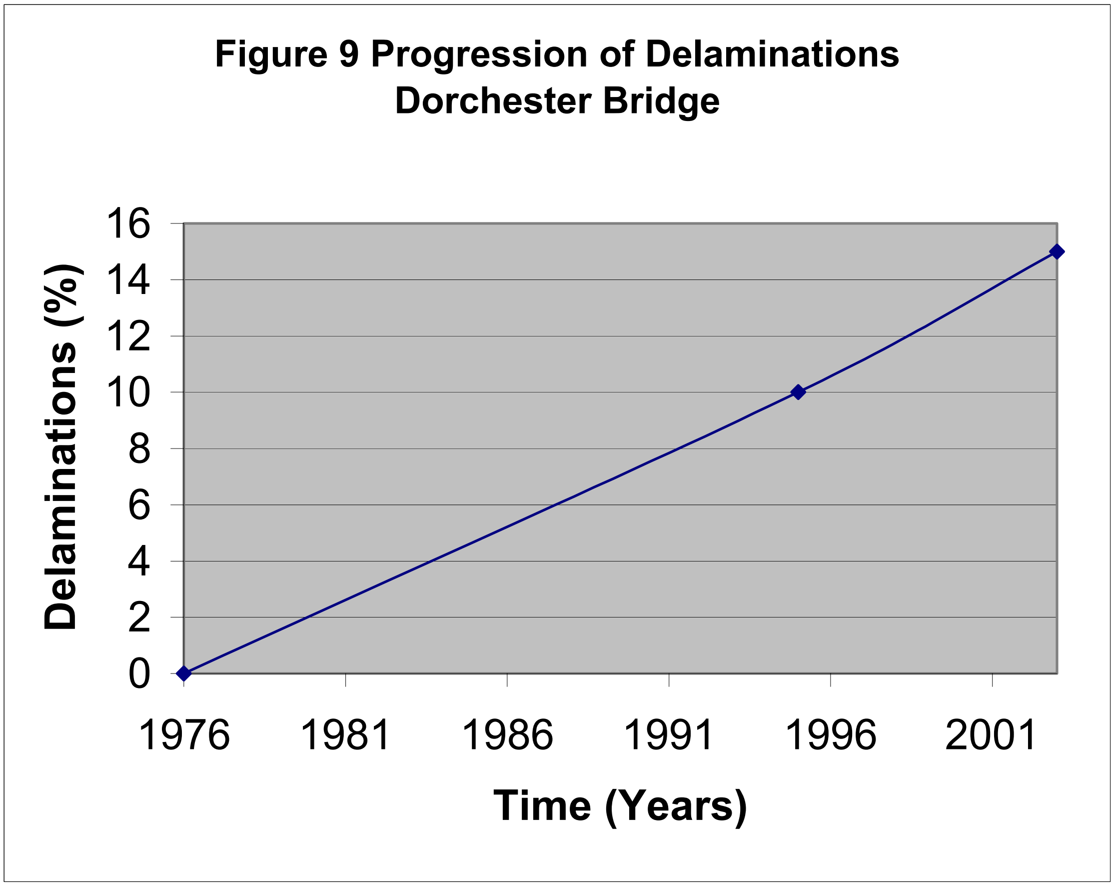


Figure 9 Progression of Delaminations
Dorchester Bridge
Delaminations (%)
0
2
4
6
8
10
12
14
16
1976
1981
1986
1991
1996
2001
Time (Years)