HIGHWAY BRIDGES IN ONTARIO, CANADA: AVOIDING STRUCTURAL COLLAPSES THROUGH ASSESSMENT, MANAGEMENT, AND RESEARCH

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Abstract: The main contribution to the corrosion of bridges in Canada is heavy use of de-icing salts, which can lead to severe rebar corrosion and girder soffit spalling in reinforced concrete (RC) highway bridge structures. Herein, recent challenges faced by authorities in dealing with corrosion problems in Canadian highway bridges are first discussed, and typical approaches for the assessment and management of these structures are described. A methodology to evaluate the residual strength of spalled RC bridges is then proposed. The basis for the proposed methodology is a modified area concept, which can be used to consider the effects of exposed reinforcement at various locations along the girder length. A multipoint analysis program is developed that employs this concept, along with graphical spalling surveys and structural drawings, to evaluate RC bridge girders. The program is then adapted for a full bridge analysis and to consider other effects of corrosion, such as bar section loss and bond deterioration. A case study bridge is evaluated to show that the developed program offers a viable tool for the rapid assessment of spalled bridge girders to facilitate the prioritization of rehabilitation projects. Lastly, the methodology is tested in a pilot laboratory study.

Keywords: highway bridge, aging infrastructure, corrosion of the reinforcing, de-icing salts.

1. Introduction

The assessment and management of existing highway bridges in Ontario is currently an area of major concern. Aging infrastructure, exposed in the Canadian winter to the action of de-icing salts, frequently requires rehabilitation or even rebuilding (Council of the Federation [1]). Bridge maintenance interventions must be prioritized and queued due to high demand and limited budgets. The primary deterioration mechanisms are corrosion of the reinforcing steel and subsequent degradation of the concrete steel interface, leading to considerable spalling of the girder and deck soffits, sufficient to cause portions of the reinforcing steel to become exposed (Figure 1). Although highly undesirable, in many cases, bridges in this condition have to remain in service due to planning or funding restrictions and may in fact still provide adequate levels of safety, due to redundancy of the structure and conservative assumptions made in the original designs. In the following sections we describe the practical aspects of bridge management in Ontario, as specified by the Ministry of Transport Ontario (MTO).

Research on the strength assessment of corroded concrete bridges and related experimental work on members with exposed reinforcement (modelling corroded) is then discussed.

Fig. 1. Highway bridge girder soffit spalling

2. Management practices for highway bridges in Ontario

The approaches used for management of bridges outlined herein are applicable to Ontario bridges, which are under the jurisdiction of the Ministry of Transportation of Ontario (MTO). The discussion covers several established procedures aimed at ensuring the safety of existing structures.

2.1. Biennial visual inspection

Every bridge is inspected biennially according to the Ontario Structural Inspection Manual (OSIM) [2]. All major structural components are rated for their respective percentage in four conditions, namely excellent, good, fair, and poor. The inspection data are then input into the Ontario Bridge Management System where a bridge condition index (BCI) is generated for every bridge to represent its weighted average condition in terms of a percentage of the perfect condition. In other words, a new bridge will start with a BCI of 100%, which would gradually depreciate over time. The BCI is then input into the Asset Management System where some minor adjustments to the index are made for the type of structure and the nature of the deterioration. The outcome is a priority index used to prioritize bridges within the rehabilitation program and to project the funding requirements for the immediate 1 to 5 years, as well as the 5 to 10 year horizon. Critical components that require immediate work to ensure safety are flagged in the OSIM report and the structural engineer takes appropriate action.

2.2. Detailed condition survey and rehabilitation strategy

For bridges that fall into the 1 to 5 years program for rehabilitation, a detailed condition survey according to the MTO Structures Rehabilitation Manual [3] would usually be conducted to obtain more accurate information about the state and types of deterioration of all the major components in order to determine the most appropriate rehabilitation strategy. Several issues need to be checked during the detailed condition survey, which include: obtaining cores and sawn samples for decks, a corrosion potential survey according to ASTM C876 [4], a delamination survey, a cover meter survey, a chloride profile, compressive strength and air void tests of cores, ground penetrating radar GPR may be used as a pre-screening tool for selected decks, and an AC resistance test for epoxy coated rebar.

The chloride profile and cover to rebar are usually the most important factors affecting the choice of rehabilitation treatment. Due to the high cost of traffic management and contract administration, the minimum life expectancy required for rehabilitated bridges is usually 25 to 30 years, unless the deterioration is in such an advanced stage that replacement would be a better option. When the preferred option is not so obvious, a life-cycle financial analysis according to the MTO Structures Financial Analysis Manual [5] is carried out for comparison of different treatments.

The proactive rehabilitation techniques that can be used to prevent further deterioration due to chloride contamination of RC elements are: elimination of expansion joint over piers using link slabs, conversion of expansion joints to semi-integral abutments, application of thermal sprayed zinc galvanic anode to chloride contaminated substructures and deck soffit, and application of a cathodic protection system to post-tensioned decks where excessive removal of chloride contaminated concrete would be undesirable.

2.3. Preservation management

The priority index mentioned in Section 2.1 can only capture the current needs of the inventory based on visual inspection. It cannot predict the long term deterioration of the bridge components due to hidden defects and chloride contamination. As an example, a component containing epoxy coated rebar within the splash zone of de-icing salt may not exhibit any corrosion damage if the chloride at rebar level has not exceeded threshold. However, given time and if no additional protection is provided, corrosion will initiate and eventually lead to damage. Preservation management is taking pro-active measures to protect those components that do not show deterioration right now and are not in the 5 year program based on their BCI. Preservation management has a high rate of return of investment since the cost of implementtation is usually a very small fraction of the rehabilitation cost, and if this management strategy is implemented in a timely manner, the large expenditure associated with rehabilitation can be deferred for a very long time. Unfortunately, due to fiscal constraints, preservation management has not been pursued very diligently over the past 20 years. However, the situation has changed since the Ontario government has committed to increase the rehabilitation budget substantially for at least the next 4 years, creating an opportunity to devote part of the available funding to preserve bridge components that are still in good condition instead of dealing only with those in poor condition.

Preservation management work includes the following components: timely replacement of waterproofing on decks, application of sealer and migrating corrosion inhibitor on exposed components, replacement of damaged expansion joint seal, overcoating and zone painting of steel structures, and fatigue retrofit of vulnerable details.

2.4. Holding strategy

Holding strategies are used to retrofit existing, badly-damaged structures to extend their service life. The reasons such strategies are implemented are: the cost of a major rehabilitation is so high that a holding strategy to extend its life for up to 10 years followed by replacement would be the best option from a life cycle perspective, functional improvement is required but there is uncertainty in timing and planning, the structure presents safety concerns to the public, e.g. pieces of concrete falling onto roadway below, but the planning to rehabilitate or replace the structure would take a long time, the structure is deemed deficient, but there is no funding within the next 5 years to carry out a major rehabilitation or replacement, and environmental assessment for a new structure prevents it from being acceptable.

Treatments that can be used in a holding strategy include: externally-bonded FRP wrapping or strengthening to girders and columns, arc sprayed zinc galvanic anode to treat exposed reinforcing, polymer modified low permeability asphalt when deck top surface is not suitable for direct application of standard waterproofing membrane, polyester concrete overlay without waterproofing membrane and asphalt on deck, and application of sealer and migrating corrosion inhibitor on exposed components.

2.5. Full-Scale load testing

MTO often conducts full-scale load testing to verify that a deteriorated structure can remain in service safely without load restrictions, or to come up with the appropriate load limits for posting. Load testing is useful because the simplified method of analysis according to the bridge code is inevitably conservative. A structure that is deemed to be deficient based on evaluation may in fact have reserved capacity due to redundancy and better material strength than specified. Also, a deteriorated bridge component could sometimes be difficult to evaluate analytically. As an example, a concrete girder with extensive spalling and debonding of the main reinforcement at the mid-span would result in strain incompatibility between concrete and reinforcement. The amount of deterioration and section losses can sometimes be difficult to assess visually or through non-destructive testing.

Load testing is therefore an integral part of the MTO's bridge management tools to ensure safety for the travelling public. It also helps the Ministry to make sound decisions on selection of treatments for a deficient bridge, ultimately achieving the most cost- effective solution for maintaining the service of the infrastructure.

3. Analytical strength assessment method

Research on the assessment of the residual strength of corroded bridges was done at the University of Waterloo through the development of a computerized method that considers concrete-reinforcement bond and corrosion [6]. The need for such a tool is driven by the fact that although severe spalling in reinforced concrete bridges is undesirable, bridges in this condition may still provide adequate, safe levels of strength. The structural assessment of the residual strength of these bridges allows the rational queuing of rehabilitation projects.

The method described herein is based on the so-called modified area concept for considering the effects of spalling on the development of reinforcing bars and lap splices in reinforced concrete flexural members. The concept is the basis for the BEST (Bridge Evaluation Strength Tool) computer program developed as part of the described work.

3.1. Modified area concept

A reinforcing bar's cross sectional area is modified to model the reduced strength if its development length is compromised as illustrated in Figure 2. The modified area concept is founded on the following assumptions:

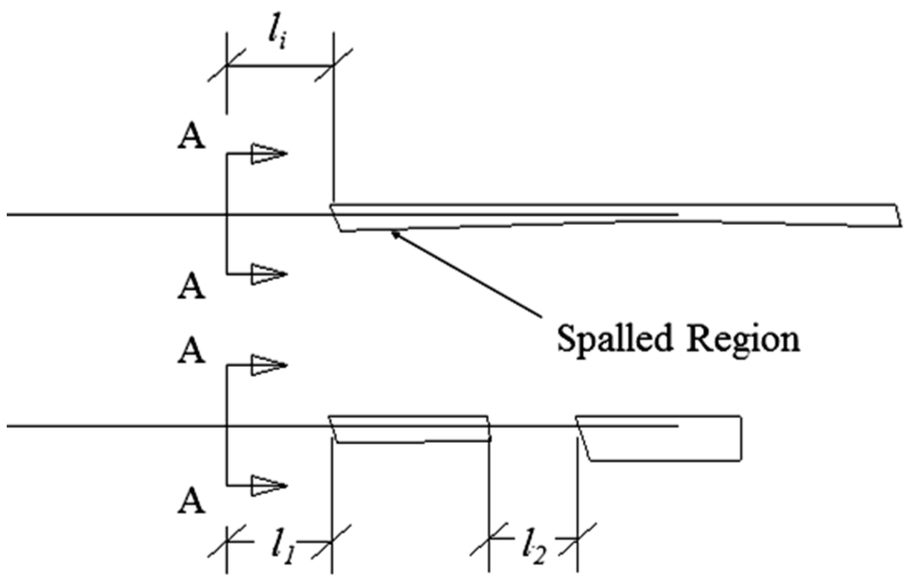
– The spalled areas are fully de-bonded with no remaining force transfer;
– The full concrete and bond strength can be achieved in the unspalled sections;
– The bond strength is linearly proportional to the available development length.

The cross sectional analysis of the beam (bridge girder) is done at discrete, predetermined locations. For example, in 100 locations in the example presented herein. All calculations done in this version of the program are based on equations from relevant the Canadian codes (CAN/CSA A23.3-04 [7], CAN/CSA S6-06 [8]). It should be, however, noted that strength calculations could adopt other code equations or mechanical principles for the calculations. The presented research was aimed at evaluation of bridges in Ontario, Canada and thus these formulas were adopted.

At a given cross section of a girder, the reinforcement is assessed first. Consider, a discrete location, 'A', at a distance greater than the required development length, l_d, from the end of the reinforcing bar. At Section 'A' the intact development length is l_i. If multiple spalled regions are present, then the intact length is computed as the sum of each intact section. If the intact length is less than the required development length (i.e. $l_i < l_d$), then in the proposed concept the bar's cross sectional area is proportionally modified as follows:

$$A_b' = A_b \cdot \left(\frac{l_i}{l_d} \right) \tag{1}$$

where: l_i = intact development length, l_d = required development length, A_b = actual (unspalled) bar cross sectional area, and A_b' = modified bar cross sectional area.
If the intact length is greater than the required development length (i.e. $l_i > l_d$), then the bar area is not modified.



Fsg. 2 Modified area concept

3.2. Flexural capacity assessment

Flexural capacity is evaluated according to:

$$M_r = A_s' \cdot \phi_s \cdot f_y \cdot \left(d - \frac{a}{2} \right) \tag{2}$$

where: A'_s = total modified longitudinal steel in the cross-section, d is the depth to the centroid of the reinforcement, a is the depth of the assumed rectangular compression stress block in the concrete, and ϕ_s and f_y are the rebar resistance factor and yield strength. In order to illustrate this concept, a single bar in an RC flexural member is considered. Figure 3 show the assumed moment capacity of the member plotted along the bar length. If the spalled area is within l_d, the capacity plateaus and is reduced at locations outside l_d. This concept is then extended to include all of the bars in the cross section, which allows determining the total capacity at a given cross section along the member.

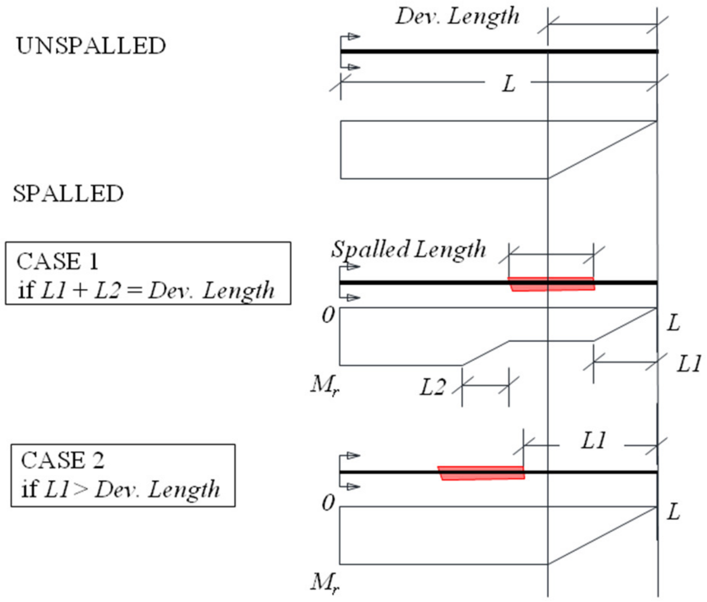


Fig.3 Moment resistance with unspalled and spalled bars

3.3. Shear capacity assessment

Shear design in the Canadian codes is based on the concept of concrete plus steel contributions to the total shear resistance of the member. There are two methods: general and simplified. Both require calculations of an average longitudinal strain ε_x at mid-depth. The CAN/CSA A23.3-04 simplified method assumes that the mid-depth strain is 0.00085, while the general method requires calculations involving moment, M_f, and shear, V_f, at the given cross section. The modified strain is calculated using the modified cross-sectional area, A'_s. For the general method:

$$\varepsilon_x' = \frac{\frac{M_f}{d_v} + V_f}{2 \cdot E_s \cdot A_s'} \tag{3}$$

where: d_v = the effective depth of the member; and E_s = elastic modulus of steel. For the simplified method:

$$\varepsilon_x' = \frac{A_s}{A_s'} \cdot \varepsilon_x \tag{4}$$

The deteriorated shear strength can then be computed for both shear design methods using the following formulas:

$$\beta' = \left(\frac{0.4}{1 + 1500 \cdot \varepsilon_x'}\right) \cdot \left(\frac{1300}{1000 \cdot S_{ze}}\right) \tag{5}$$

$$\theta' = 29 + 7000 \cdot \varepsilon_x' \tag{6}$$

$$V_c' = \phi_c \cdot \lambda \cdot \beta' \cdot \sqrt{f_c'} \cdot b_w \cdot d_v \tag{7}$$

$$V_c' = \frac{\phi_s \cdot A_v \cdot f_y \cdot dv \cdot \cot(\theta')}{s} \tag{8}$$

$$V_r' = V_c' + V_s' \tag{9}$$

where: V'_r, V'_c, and V'_s are total resistance, concrete contribution to resistance and steel contribution to resistance, respectively, S_{ze} is the crack spacing parameter, and ϕ_c and ϕ_s are concrete and steel material safety factors.

3.4 Effect of corrosion on reinforcement

Corrosion of the reinforcing steel (rebar) results in section loss (steel and concrete) and steel/concrete bond degradation. The residual capacity is affected by the residual strength of the reinforcement, strength of the steel-concrete bond, severity and distribution of spalling and the remaining composite behaviour of the member.

Uniform corrosion results in a reduced residual strength for a reinforcing bar directly proportional to the reduced area. The average remaining cross section can be determined at any exposed section of the bar. Conversely, for highly localized "pitting corrosion" an overall minimum cross section likely governs its strength and is difficult to locate along a length of reinforcing steel. Pitting tends to cause local stress concentrations, which can result in nonlinear behaviour beginning at lower load levels. Researchers have suggested that rebar yield strength reductions can be considered using empirical relationships in the following general form:

$$f_y = (1 - \alpha_y \cdot Q_{corr}) \cdot f_{y0} \tag{10}$$

where: f_y = yield strength of corroded bars at time T; f_{y0} = yield strength of the non-corroded bars; Q_{corr} = average section loss as a percentage of the original cross-section area; and α_y = empirical coefficient (= 0.012 according to [9]).

Different values for α_y have been proposed by various researchers [9–15]. Figure 4 summarizes the results of a number of studies on α_y. In the current research, the curve proposed in Cairns et al. 2005 [9] has been used.

3.5. Effect of corrosion on bond

The effect of corrosion on bond can be modeled as a two phase process. In the initial phase, the creation of corrosion products increases the bar diameter, increasing both radial stresses and the frictional component of bond. Bond strength increases as a result. In the second phase radial stresses exceed the threshold, bursting and splitting the concrete cover, resulting in longitudinal cracking at the surface and a reduction in bar confinement and bond strength.

The residual bond strength of corroded reinforcing bars embedded in concrete has been tested in several laboratories and reported in a number of references. Significant scatter is reported and attributed to the wide-range of bond specimens, bar types, and conditioning techniques employed [16].

Several researchers have suggested empirical formulations for predicting the bond strength of corroded rebar [16-21]. The comparison of the models is shown graphically in Figure 5. Input for the bond degradation model comparison is as follows: longitudinal bar diameter = 31.8 mm, concrete cover = 50.8 mm, development (l_d) = 36.2 mm, concrete strength, (f_c') = 20 MPa, stirrup strength, (f_y) = 230 MPa, stirrup spacing = 304.8 mm, stirrup diameter =12.65 mm. In the subsequent calculations the model proposed by Bhargava et al. 2007 [16] was used.

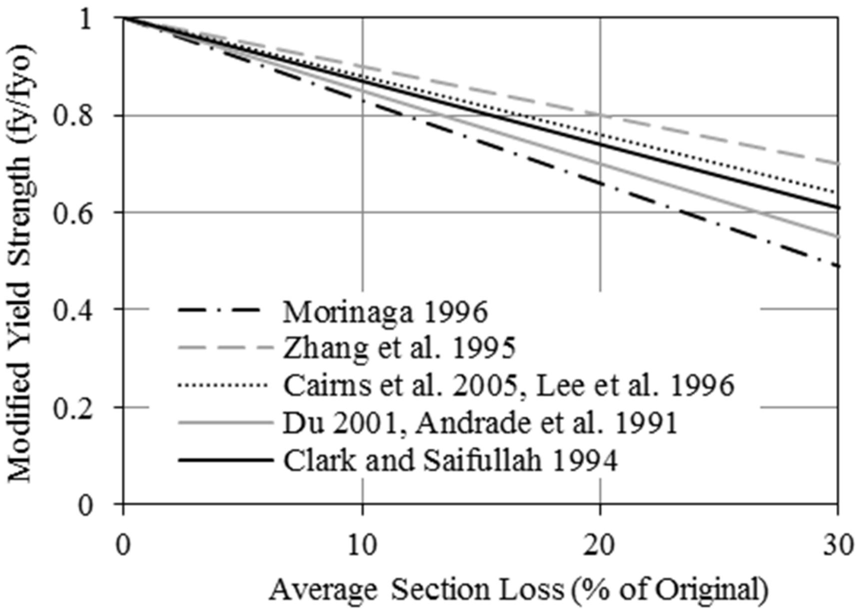


Fig. 4. Empirical models for residual yield strength of corroded rebar

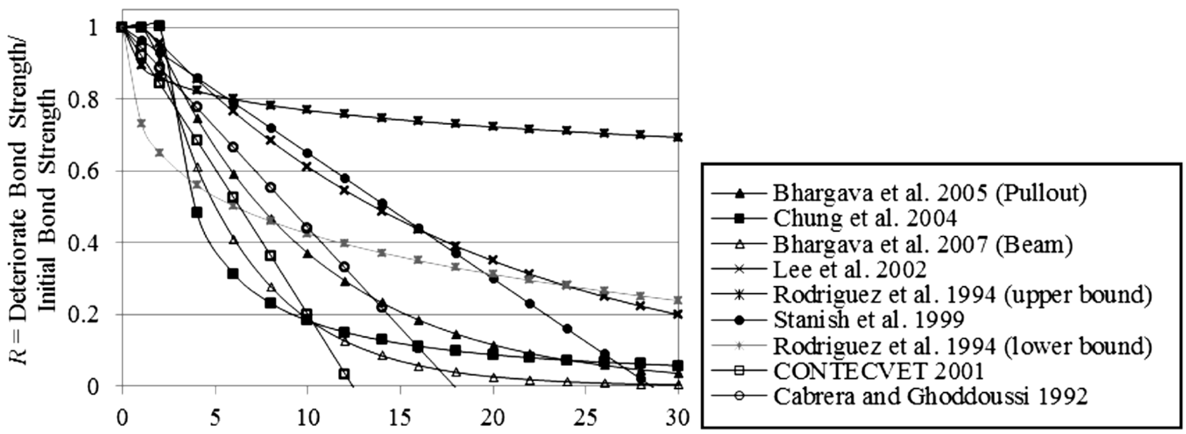


Fig. 5. Empirical models for steel-concrete bond deterioration

3.6. Evaluation of a multi-girder structure

For a full bridge analysis, the effects of redundancy must be considered. In this model, it is proposed that for each group of three girders, the two remaining girders are assumed to carry the tributary load acting on all three girders, once a single girder fails (assuming that the deck can span between the two good girders). The overall capacity of the structure is then estimated by evaluating groups of three girders in succession with the bridge capacity limited by the lowest capacity of any of the girder groups.

In the first step a sectional analysis of a single girder is performed at several predetermined cross-sectional locations (100 in this case). At each location, the flexural and shear capacities are calculated. The spalling surveys and reinforcing plans are used as the primary sources of BEST program input. Spalling surveys are created in Ontario to document deterioration by the Ministry of Transport Ontario (MTO), generally after field scaling procedures are carried out to remove loose concrete (see Figure 7). In Autocad [22], the reinforcement plan for the bottom lower reinforcement is then superimposed on the spalling survey for each girder. Points are added to all locations where this reinforcement intersects the spalled regions. Sample input for this step is shown in Figure 8. The data extraction function is then used to create point and line data text files that became the input for the Matlab [23] program.

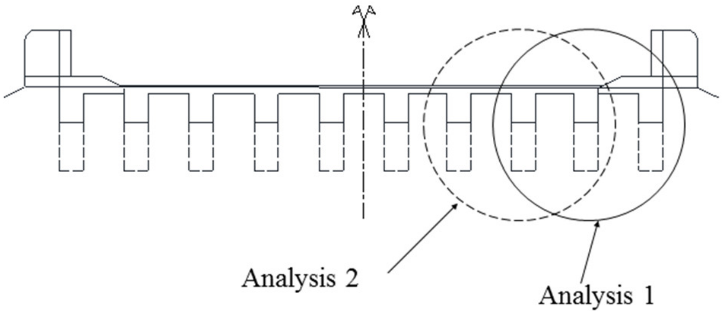


Fig. 6. Girder grouping for full structure analysis.

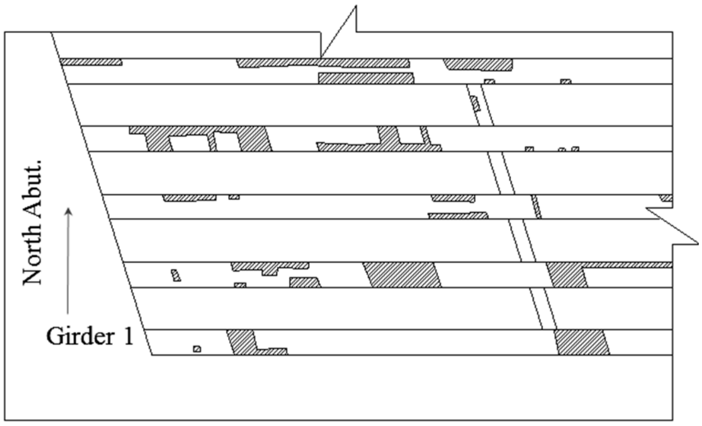


Fig. 7 Spalling survey sample

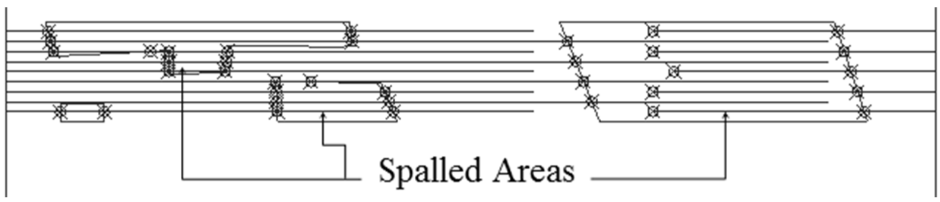


Fig. 8. Reinforcement plan-spalling survey superposition

The case study structure was built in 1960 s. It represents a common structural configuration on many major Canadian highways: a rigid frame bridge with a gently arched girder shape (Figure 9). Severe spalling and delamination of the girder soffits, shown in Figure 1, poses the greatest threat to the ultimate strength of the structure.

The developed program provides graphical output that compares the residual resistance (flexure or shear) to user-defined load effect envelopes. Figure 8 shows samples of the graphical output for the flexural and shear analysis of a single girder in the case study structure. The load effects are shown as dashed lines and indicate the envelope of the maximum moment due to the CL-625-ONT truck (CAN/CSA S6-06) [8] located at any position of along the bridge span. This envelope was obtained using a model of the bridge girder created with the structural analysis software SAP 2000 (2009) [25]. The residual resistances are shown as solid lines.

In order to show the effects of anchorage loss only, the analysis presented in Figure 8 (and 8(a)) considers only the effect of spalling and loss of anchorage on the capacity of the structure; no corrosion effects are considered in Fig. 8. In these figures, the x-axis origin corresponds with the north girder support and extends to the south support at an x-coordinate of 34.47 m.

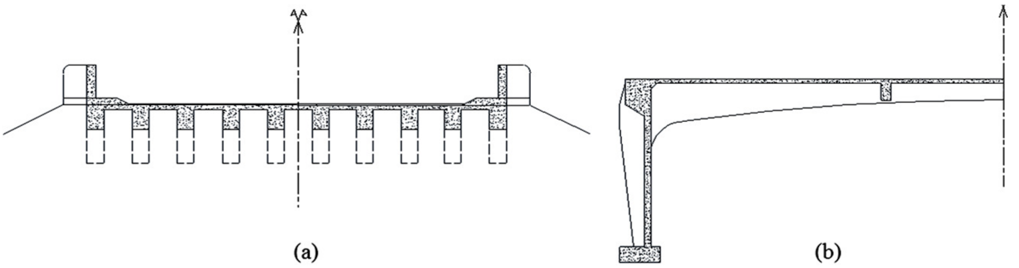


Fig 9. Bridge cross section and elevation

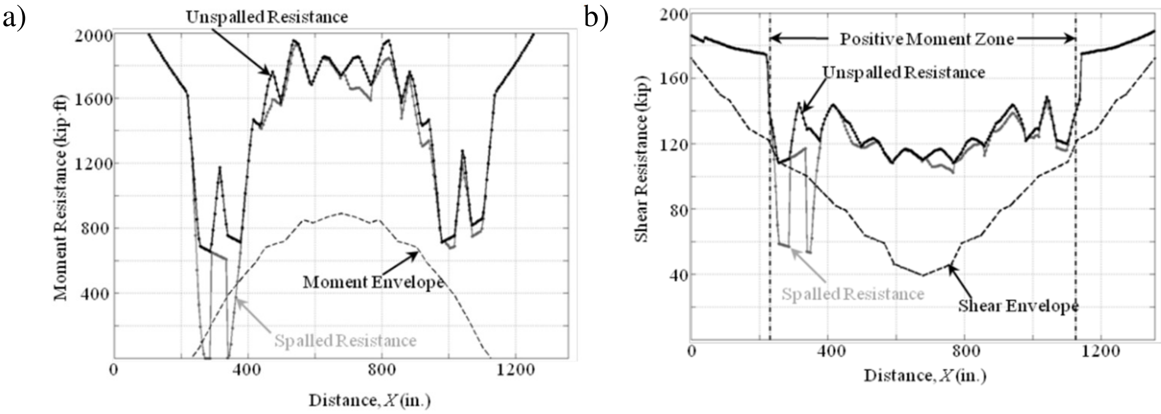


Fig 10. Strength evaluation of case study girder: (a) flexure (b) shear (general method)

In Figure 11(a) the results of the full bridge analysis are plotted for the flexural, simplified and general shear method verifications.

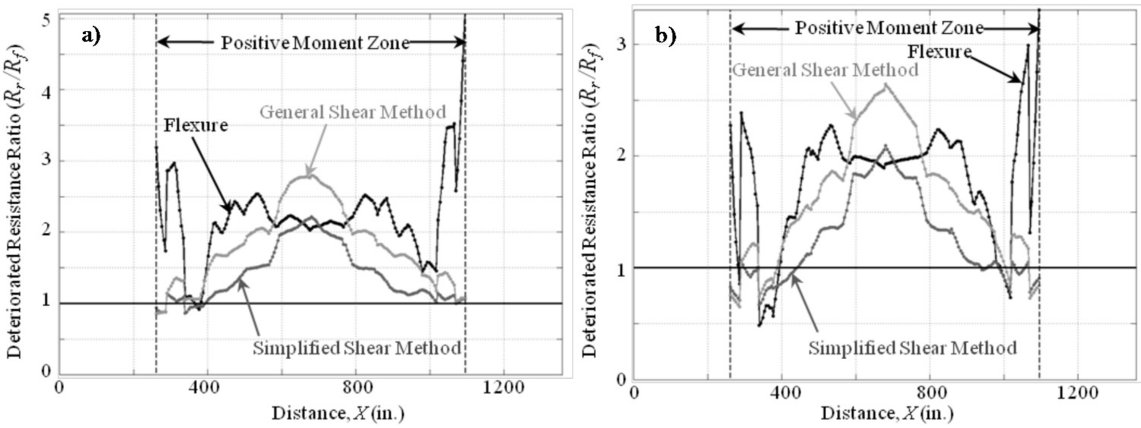


Fig. 11. Full structure analysis considering: (a) bond and development only, and (b) all deterioration effects

Rebar section loss and bond deterioration effects are not considered in this figure. Each curve represents the minimum factored resistance-to-load effect ratio at each point along the span for all groups of adjacent girders analyzed. Although these curves still drop below 1.0 at some locations along the span (indicating that the structure fails at these locations), the benefits of considering system behaviour are apparent. Assumptions of 15% corrosion within spalled sections and 6% corrosion in intact concrete were made for the purpose of demonstrating the effects of incorporating the rebar section loss and bond deterioration models. The impact of these assumptions can be seen by comparing Figs. 11(a) and (b). In Fig. 11(b), the resistance ratio at the worst section is now considerably less than 1.0.

4. Experimental Investigation on Beams with Exposed Reinforcements.

In order to better understand and predict the residual strength of the deteriorated bridges, a test program was designed, which involves mid-sized concrete beams with partially de-bonded flexural reinforcement. The goal of this experimental study was to determine the correlation between the spatial location and surface area of de-bonding with the strength of the beams [25].

The loss of bond was simulated by surrounding the longitudinal reinforcement with styrofoam prior to pouring the concrete (see Figures 12(a) and (b)). The beams were simply supported and loaded centrally by a point load. Stirrups were placed to protect against potential shear failures (Figure 12(c)) at a spacing of 190 mm. The results of testing of seven beams of dimensions of 2100×150×100 mm, with simple supports at 1900 mm distance, are presented herein.

The flexural reinforcing steel strength was 480 MPa. Three different types of specimens were designed: fully-bonded specimens, specimens with de-bonded reinforcement in the flexural zones, and specimens with de-bonded regions in the anchorage and flexural zones. The specimens are shown in Table 1. The "exposure percentage" is taken as the total de-bonded length divided by the total length of the reinforcement.

The beam was tested at a load rate of 0.03 kN/s. Loading continued until complete failure of the specimen. The beam deflection was recorded by linear variable differential transformer (LVDT) at 2 s intervals. Load-deflection curves for beam Samples 1–7 are shown in Figure 13.

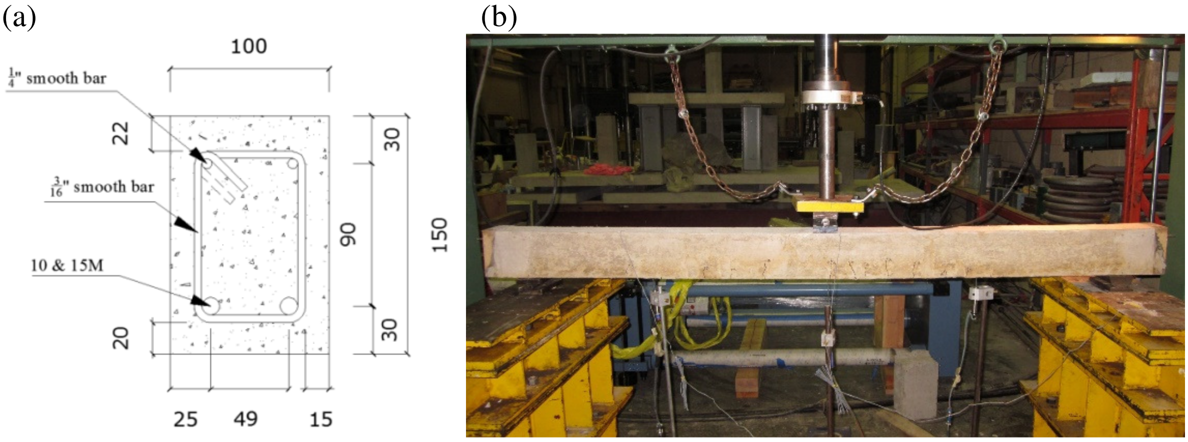


Fig. 12. Beam specimens: (a) cross-section, and (b) control beam during three-point bending test

Specimens 1, 2, 3, and 4 were made with exposed regions at the midspan with increasing exposure areas up to 85%, resulting in a 55% reduction in strength (see Figure 13 and 14). Specimens 1, 5, 6, and 7 have exposed regions near the supports (in the anchorage and flexural zones). Specimens 7 has the largest strength reduction with a strength that is only 36% that of the control specimen.

Figure 14(b) presents normalized outputs from the test showing a relationship between $Stiff_{exp}$ / $Stiff_{control}$ and L_{exp} / L_{total}, where: $Stiff_{exp}$ is the initial flexural stiffness of the sample with exposed reinforcing, $Stiff_{control}$ is the corresponding flexural stiffness for the control sample, and L_{exp} and L_{total} refer to the exposed and total beam length. In this figure, the stiffness was calculated using the load-displacement data up to 80% yield load. As shown in this figure, the stiffness of the beams was not affected significantly if the length of exposure is limited to less than 20% of the total length. Generally speaking, the data for Specimens 1–7 suggests that the stiffness reduces with an increase in exposure length.

(a) (b)

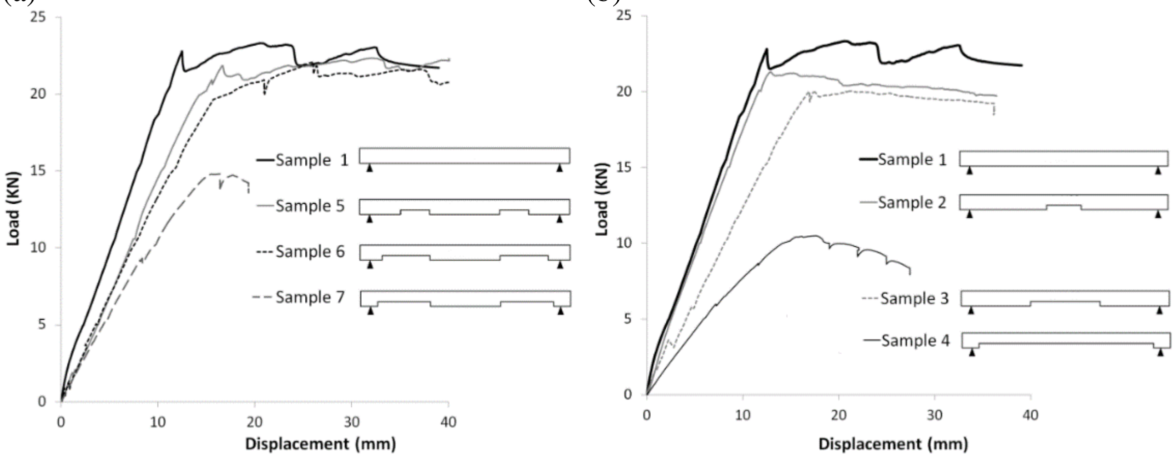


Figure 13: Load-Deflection Curves for the Tested Beam Specimens

(a) (b)

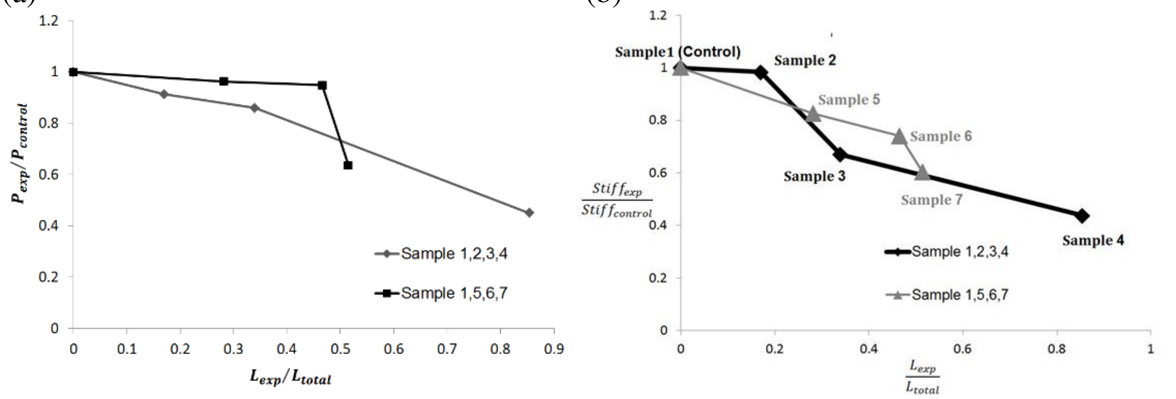


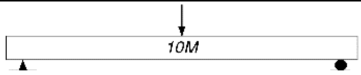
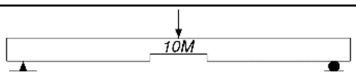
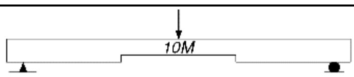
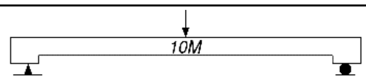
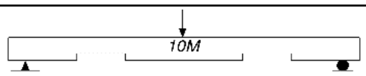
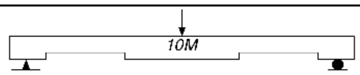
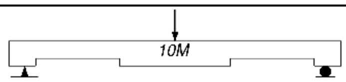
Figure 14: Effect of exposure length on a) beam strength, and b) beam stiffness

5. Conclusions

This paper summarizes recent experiences related to the assessment and management of bridges that have been exposed to the effects of corrosion. In addition, the paper describes theoretical and experimental research related to this topic. The theoretical work involved the development of a computer strength assessment method for concrete bridges with corroded and exposed reinforcements. It can be concluded that the remaining structural capacity is directly related to the spatial distribution of the spalling relative to bar cutoffs and splices. Consideration of corrosion in the form of rebar section loss and bond deterioration should be considered in the analysis and will result in the decreased residual strength of the existing bridge. System effects and continuity should also be included in the structural analyses, as they can lead to improvements in the estimated bridge condition, since the spalled regions will generally vary from one girder to the next. The developed analysis program BEST, employing the modified area method, offers a viable tool for the rapid assessment of spalled and deteriorated reinforced concrete bridge girders.

The experimental research involved testing of reinforced concrete beams, with exposed reinforcements. It was found that after losing a major portion of concrete cover in the bottom face of the concrete beams, the structure is still able to withhold a significant amount of flexural strength. The factors that affect the change in beam behaviour are exposure length, geometry of exposed area, percentage area of reinforcement in beam section, concrete strength, and type of loading (although tests for only one loading type are presented herein). When appropriate anchorage is provided, the debonded specimens maintained at least 80% of their strength when compared to the fully bonded specimen.

Table 1. The specimens and their properties

Specimen ID	Specimen Layout	Development %	Span Spalled %
1		100	0
2		100	16.9
3		100	33.9
4		45	85.4
5		100	28.1
6		60	46.6
7		45	51.4

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References

1. Council of the Federation, 2005. Looking to the Future: A Plan for Investing in Canada's Transportation System [online]. Available from www.councilofthefederation.ca/ [cited April 27 2011].
2. Ministry of Transportation of Ontario. Ontario Structural Inspection Manual (OSIM). Downsview, ON.
3. Ministry of Transportation of Ontario. MTO Structures Rehabilitation Manual. Downsview, ON.
4. American Society for Testing and Materials. 2009. ASTM C876: Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete. West Conshohocken, PA.
5. Ministry of Transportation of Ontario. MTO Structures Financial Analysis Manual. Downsview, ON.
6. Luckai, J. 2011. Evaluating the Effects of Spalling on the Structural Capacity of Reinforced Concrete Bridge Girders. M.Sc. thesis, Department of Civil and Environmental Engineering, University of Waterloo, Waterloo, Ont.
7. Canadian Standards Association. 2004. CSA Standard A23.3-04: Design of Concrete Structures. Mississauga, O.N.
8. Canadian Standards Association. 2006. CAN/CSA S6-06: Canadian Highway Bridge Design Code. Mississauga, O.N.
9. Cairns J., Plizzari G., Du Y.G., Law D.W., Franzoni C. 2005. Mechanical properties of corrosion damaged reinforcement. *American Concrete Institute Structural Journal* 102 (2): 256-264.
10. Morinaga S. 1996. Remaining life of reinforced concrete structures after corrosion cracking. In C. Sjostrom (eds), *Durability of Building Materials and Components: 127-137.* London: E&FN Spon.
11. Zhang P.S., Lu M., Li X.Y. 1995. The mechanical behaviour of corroded bar. *Journal of Industrial Buildings* 25 (257): 41–44.

12. Lee H.S., Tomosawa F., Noguchi T. 1996. Effect of rebar corrosion on the structural performance of singly reinforced beams. In C. Sjostrom (eds), *Durability of Building Materials and Components: 571-580.* London: E&FN Spon.
13. Du Y. 2001. *Effect of Reinforcement Corrosion on Structural Concrete Ductility*. PhD thesis, Department of Civil Engineering, University of Birmingham, Birmingham, UK.
14. Andrade C., Alonso C., Garcia D., Rodriguez J. 1991. Remaining lifetime of reinforced concrete structures: effect of corrosion in the mechanical properties of the steel. In *Life Prediction of Corrodible Structures: Proc. intern. Conf., Cambridge, UK., 23-26 September 1991.* National Association of Corrosion Engineers.
15. Clark L.A., Saifullah M. 1994. Effect of corrosion rate on the bond strength of corroded reinforcement, In R. N. Swamy (eds), *Corrosion and Corrosion Protection of Steel in Concrete*: *591-602*, Sheffield: Sheffield Academic Press.
16. Bhargava K., Ghosh A.K., Mori Y., Ramanujam, S. 2007. Corrosion-induced bond strength degradation in reinforced concrete - analytical and empirical models. *Nuclear Engineering and Design* 237 (11): 1140–1157.
17. Chung L., Cho S.H., Kim J.H.J., Yi S.T. 2004. Correction factor suggestion for ACI development length provisions based on flexural testing of RC slabs with various levels of corroded reinforcing bars. *Engineering Structures* 26 (8): 1013–1026.
18. Lee H.S., Noguchi T., Tomosawa F. 2002. Evaluation of the bond properties between concrete and reinforcement as a function of the degree of reinforcement corrosion. *Cement and Concrete Research* 32 (8): 1313–1318.
19. Rodriguez J., Ortega L., Garcia A. 1994. Corrosion of reinforcing bars and service life of R/C structures: corrosion and bond deterioration. In *'Concrete Across Borders', Proceedings of an Int. Conf., Odense, Denmark, 22–25 June 1993*.
20. Stanish K., Hooton R., Pantazopoulou S. 1999. Corrosion effects on bond strength in reinforced concrete. *American Concrete Institute Structural Journal* 96(6): 915–921.
21. Cabrera J.G., Ghoddoussi P. 1992. The effect of reinforcement corrosion on the strength of the steel concrete bond. In *Bond in Concrete: Proc. intern. Conf., Riga, Latvia, 15-17 October 1992.*
22. Autodesk 2011. Autocad 2011 User's Guide, Chapter 38. pp. 1833.
23. MATLAB 2010. R2010b, The MathWorks Inc., Natick, MA.
24. SAP 2000 Version 12. 2009. Computers and Structures, Inc. Berkeley, California USA.
25. Masnavi A. 2013. Bond and Development Length in Concrete Beams with Exposed Reinforcement, M.Sc. thesis, Department of Civil and Environmental Engineering, University of Waterloo, Waterloo, Ont.

MOSTY AUTOSTRADOWE W KANADZIE (ONTARIO). UNIKANIE ZNISZCZENIA KONSTRUKCJI POPRZEZ OCENĘ, ZARZĄDZANIE I BADANIA NAUKOWE

Streszczenie: Stosowanie dużych ilości soli do usuwania oblodzenia jest głównym czynnikiem powodującym korozję mostów w Kanadzie. Może to prowadzić do silnej korozji prętów zbrojeniowych i kruszenia się otuliny dźwigarów w żelbetowych konstrukcjach mostów drogowych. Najnowsze wyzwania, z którymi muszą się uporać władze zajmujące się problemami korozji mostów drogowych w Kanadzie, zostały w tej pracy omówione łącznie z opisem typowego podejścia do oceny i eksploatacji tych konstrukcji. Zaproponowano metodę oceny wytrzymałości skorodowanych żelbetowych mostów. Podstawą zaproponowanej metodologii jest koncepcja zmodyfikowanego obszaru, która może być zastosowana do rozważenia efektu odkrytego zbrojenia w różnych miejscach wzdłuż długości dźwigara. Został opracowany wielopunktowy program analityczny oparty na tej koncepcji razem z graficznymi pomiarami wykruszonej otuliny i rysunkami technicznymi służącymi do oceny dźwigarów mostów żelbetowych. Program został następnie zaadaptowany do pełnej analizy mostu i do oceny innych efektów spowodowanych przez korozję, takich jak utrata przekrojów prętów i niszczenie przyczepności. Zanalizowano przykładowy most żeby wykazać, że opracowany program stanowi realne narzędzie do szybkiej oceny niszczących się dźwigarów mostowych i ułatwia ustalenie priorytetów procesów naprawczych. Metodologia jest testowana w pilotowym badaniu laboratoryjnym.

Słowa kluczowe: most autostradowy, zużycie mostu, korozja zbrojenia, sól odladzająca.