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Reliability-based condition assessment of a deteriorated concrete bridge

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Abstract. In the existing bridge management systems, assessment of the structural behavior is based on the results of visual inspections in which corresponding condition states are assigned to individual elements. In this process, limited attention is given to the correlation between bridge elements from structural perspective. Also, the uncertainty of parameters which affect the structural capacity is ignored. A system reliability-based assessment model is potentially an appropriate replacement for the existing procedures. The aim of this research is to evaluate the system reliability of existing conventional Steel-Reinforced bridge decks over time. The developed method utilizes the reliability theory and evaluates the structural safety for such bridges based on their failure mechanisms. System reliability analysis has been applied to simply-supported concrete bridge superstructures designed according to the Canadian Highway Bridge Design Code (CHBDC-S6) and the deterioration pattern is achieved based on the reliability estimates. Finally, the bridge condition index of an old existing bridge in Montreal has been estimated using the developed deterioration pattern. The results obtained from the developed reliability-based deterioration model and from the evaluation done by bridge engineers have been found to be in accordance.

Keywords: bridge; reliability; deterioration; conventional steel-reinforced deck

1. Introduction

Civil infrastructures, specifically those in public transportation systems, have been subjected to deteriorating conditions such as aging, fatigue, corrosion, inadequate maintenance and special loading patterns (increasing load spectra). Therefore, they should be inspected and monitored regularly and should be rehabilitated whenever they fail to satisfy appropriate performance levels or required to conform to the latest code or standard (Huffman *et al.* 2006). Inspection, structural health monitoring, damage detection and condition assessment of a bridge constitute a fundamental and critical task in bridge management systems. There has been a significant progress in the areas of structural health monitoring, non-destructive evaluation and automated condition

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assessment techniques in the recent years (Yi *et al.* 2013, Li *et al.* 2014, Adhikari *et al.* 2014). Special care must be taken to accurately assess the bridge performance in order to make an appropriate decision on repair or strengthening. Among all civil public transportation infrastructure systems, highway bridges play a vital role in the transportation networks. Most of Bridges in Canada were constructed between 1950 and 1975, unfortunately, the required maintenance has not been done on these important public infrastructures. The cost of rehabilitation and replacement of deteriorated bridges has been estimated to be approximately $100 billion in the United States (McDaniel *et al.* 2010). The corrosion of steel reinforcement due to the application of salt-based de-icing materials is the main cause of deck degradation. Also, the global climate change effect may further aggravate the situation and reduce the resilience of highway bridges (Ikpong and Bagchi 2014).

Bridge Management Systems (BMS) adopt the Bridge Condition Index (BCI) derived based on the visual inspection results and the resulting element level condition indices. Deterministic load rating methods are also widely used to evaluate the safe live load carrying capacity of highway structures. In order to indicate the system level condition, instead of using the BCI, the system reliability-based condition index can be used in the BMS, or it can be added to the BMS as an additional parameter. A System reliability based deterioration prediction model helps decision makers to determine the best and most economic time for major interventions in a more rational manner (Ghodoosi *et al.* 2014 a,b).

The objective of the present research is to evaluate the system reliability of existing bridges at different time intervals using a rational and numerical technique in which uncertainty of structural parameters, correlation between structural elements, load redistribution, and redundancy of the structure can be considered. This article demonstrates the effectiveness in developing the degradation profile for the whole structure. The purpose of a reliability-based evaluation is to account for the uncertainties associated with loads and the resistance of the system using the probability of failure P_f , and the reliability index β as the safety criteria. The reliability index can be used as a benchmark to indicate whether the performance of a structural system is satisfactory or an intervention is necessary for each time interval. Estimating the reliability index for different time intervals, one is able to find the best fit deterioration function for a particular bridge structural system. The present research has been carried out by utilizing the reliability theory to establish a deterioration model based on the failure mechanisms of bridges. The developed method has been applied to simply-supported concrete bridge superstructures designed according to the Canadian Highway Bridge Design Code (CHBDC-S6). Non-linear Finite Element model of the bridge has been developed and the system reliability index has been determined for different time intervals. Finally the degradation profile of the bridge superstructures has been established and updated. As a case study example, the developed deterioration model has been implemented to an old bridge in Montreal and the condition of the bridge after 75 years has been predicted. The expected condition of such bridge based on the developed model is approximately in accordance with the results from the engineers' evaluation of the bridge.

2. Service life of steel-reinforced concrete decks exposed to chloride attack

The corrosion of steel reinforcement due to the application of salt-based de-icing materials is the main cause of deck deterioration. Corrosion-induced damage of any reinforced concrete bridge deck exposed to chlorides could be presented into four phases: early-age cracking of concrete, corrosion initiation of reinforcement, cracking of the concrete cover, and delamination or spall (Cusson *et al.* 2011). The following parameters affect the estimation of service life of a steel reinforced concrete deck: surface chloride content of concrete, C_s; effective chloride diffusion coefficient of concrete, D_c; chloride threshold of the reinforcement, C_{th}; corrosion rate of steel reinforcement, λ; and the concrete cover of steel reinforcement. All of the mentioned parameters are extremely variable, uncertain, and difficult to monitor. Therefore, the deterioration models should be updated and calibrated based on the visual inspections, non-destructive evaluations, and instrumental observations, if available. In this article, the field data are picked up from the literature for those locations which have similar environmental situation as compared to Canada with long and cold winter periods. The detailed explanation of the data collected from literature and the corresponding uncertainties could be found in Ghodoosi *et al.* (2014a). It is essential to mention that these data are adopted to build up a primary deterioration curve which must be updated for a specific bridge based on inspections and evaluations.

In order to predict the corrosion initiation, Crank's solution of Fick's second law of diffusion (Cusson *et al.* 2011) is here applied. The model developed by Liu and Weyers (1998) is adopted to predict the time to longitudinal cracking of concrete. The model suggested by Vu *et al.* (2005) is adopted here to estimate the time to spall. Based on this model the concrete delamination occur when the crack width reaches the limit of 1 mm.

3. Reliability assessment, realiability index

The statistical data on deck resistance *R* and bridge loading *Q* are used to define the limit state function *g(R, Q) =R - Q*. The reliability index, $\beta = -\phi^{-1}(P_f)$ is a safety margin indicator of the bridge deck, where ϕ^{-1} is the inverse of the standard normal distribution function and P_f is the probability of failure. Different techniques are available to estimate the reliability index. When the resistance *R* and the load *Q* are both normally distributed, the reliability index is calculated based on the Eq. (1), in this formulation μ is the mean value and σ represents the standard deviation of a normal variable. Several parameters should be considered in the estimation of concrete decks capacity; as a result, the Limit State function *g(R, Q)*, produces a complex expression, and reliability index calculation becomes challenging. The Rackwit-Fiessler iterative procedure (Nowak and Collins 2000) can be also adopted to estimate β. In the case where the load, *Q* and/or resistance, *R* are not normally distributed, they can be approximated as normal variables. Details of the Rackwits-Fiessler iterative method can be found in Nowak and Collins (2000).

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{1}$$

4. Description of the developed deterioration model

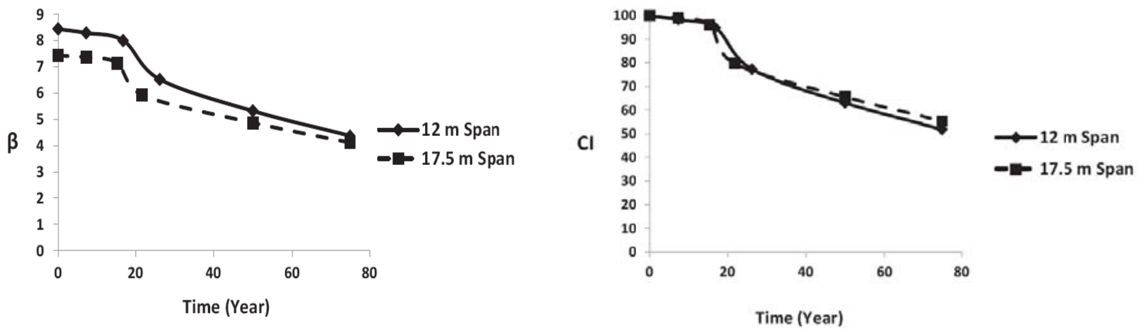
The structural specifications and the variation of the structural parameters obtained from the literature are incorporated into a structural analysis system as well as a Finite Element model and the best fit distribution of the system resistance is achieved based on the estimated set of resistance values. Using the distribution of the system resistance and the probabilistic load variation model, the reliability index β at the system level is calculated according to the procedure described earlier. The process of estimating the capacity of deteriorated structural system is repeated for different time intervals and based on the existing element-level deterioration models (Liu and Weyers1998, Cusson *et al.* 2011, Vu *et al.* 2005). As the structural condition deteriorates, the reliability index similarly decreases over time. The system level deterioration curve can be derived based on the reliability indices estimated for different time intervals. In this study, using random values generated from the statistical data and by Oracle Crystal Ball software (2008), the simulated random values of the deck capacity R have been estimated for different time intervals. The main structural parameters affecting the capacity of such system are identified as the followings: the yield strength of reinforcing steel reinforcement f_y, the modulus of elasticity of reinforcing steel E_s, the compressive strength of concrete $f_c^{/}$, the modulus of elasticity for concrete E_c, and the dimensions of reinforced concrete members. The details of the methodology for the proposed reliability-based deterioration model at the system level, variation of such structural parameters, and variation of bridge loads can be find in Ghodoosi *et al.* (2014a). Here, the live load model developed for calibration of AASHTO LRFD (2004) has been used for the reliability calculations (Czarnecki and Nowak 2007)

Here, it is assumed that the deicing salt contaminates the top surface of the deck through the permeable wearing surface and becomes airborne due to traffic beneath the bridge. Conservatively, it is assumed that deicing salt contamination occurs uniformly on all the soffits of the beam and the slab. Reduction of the steel bar diameter entails the capacity degradation of a structural element. Spall or delamination is the cause of decreased resistance due to concrete compression zone being reduced by the depth of spalled cover. Delamination of the top or bottom cover also leads to reduction of shear capacity due to loss of cross section by one or both covers.

The developed method has been applied to simply supported concrete bridge superstructures designed according to the simplified method and CL-625 loading as specified in the Canadian Highway Bridge Design Code (CHBDC-S6). The spans of the bridges are selected to be 17.5 m and 12 m (Centre-to-Centre of bearings). Four simply supported rectangular concrete beams (Roller support for one side and hinge support for the other), which are 2.3 m apart support the concrete slab with 0.2 m thickness. In order to meet the requirements of the code and for simplicity, a nominal concrete cover of 60 mm is assumed on all surfaces. Four concrete diaphragms are designed and located in the transverse direction at two ends and quarters of the span on each side. Using SAP2000 (2003), non-linear Finite Element models of the bridge decks has been developed and verified for the accuracy of results. The Finite Element modeling and verification process is explained in Ghodoosi *et al.* (2014a). The system reliability index has been determined for different time intervals. The best fit distributions for the ultimate capacity of the system and the resulting reliability indices estimated for the simply supported bridge with 17.5 m span are shown in Table1. The identical information regarding the 12 m span bridge is mentioned in Ghodoosi *et al.* (2014a). Fig.1a demonstrates the system reliability-based deterioration curves for 17.5 and 12 m span bridges. The generated reliability-based deterioration curves in Fig. 1(a) are normalized to yield the condition index (CI) of 100 at the time of opening the bridge to traffic for the first year. Fig. 1(b) shows the normalized condition index (CI)-based deterioration curves.

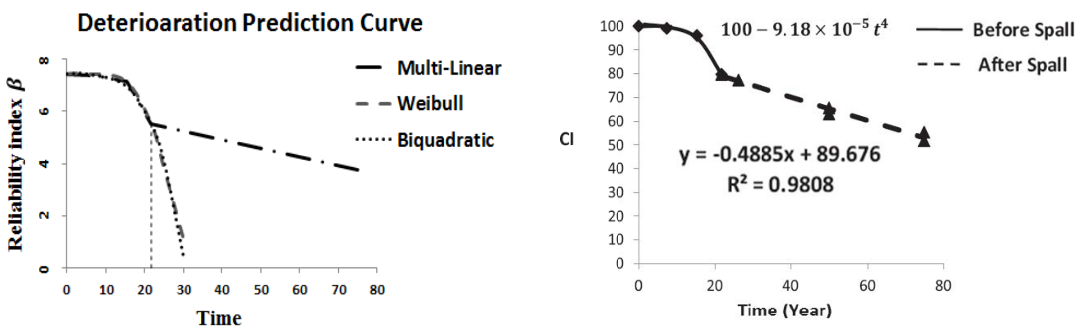
Based on the generated deterioration models, decision makers are able to predict the appropriate time for the major interventions. The acceptable level of reliability depends on the budget and the strategy of the owner of the asset. Structural safety could be also defined through Bridge Condition Index (CI) with the range from 0 to 100. The CI may be categorized into five groups: 0-19, 20-39, 40-59, 60-79, and 80-100 which represent dangerous, slightly dangerous, moderate, fairly safe and safe levels respectively (Miyamoto *et al.* 2001).

It is necessary to update the developed deterioration models based on the results of the inspections and evaluations during the life cycle of a specific bridge. As reported by Ghodoosi *et al.* (2014a), the degradation curves when calibrated up to the time of spall underestimate the resistance of the bridge superstructure after spall up to the 75th year (see Fig. 2(a)); As suggested, the deterioration curve should be divided in two different phases, before and after spall. Here; the best fit biquadratic convex curve proposed by Miyamoto *et al.* (2001) is fit to the CI values shown in Fig. 1(b) for the time period up to spall as given by the Eq. (2). After the time to spall, t_s, up to the 75th year, the best fit to the data obtained from two bridges is a linear function expressed in the Eq. (3). The best fit degradation curve for a newly constructed bridge is shown in Fig. 2(b). In this case the time to spall is estimated to be t_s = 21.69 years for the primary developed deterioration model.



(a) (b)

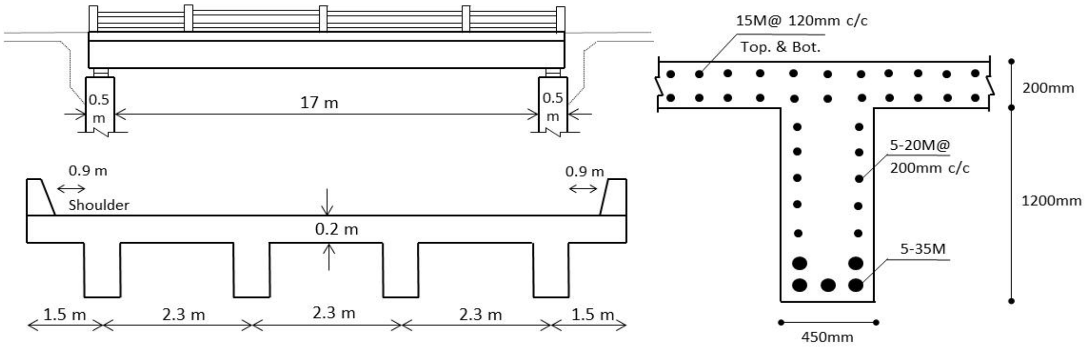
Fig. 1 (a) Deterioration Prediction Curves Based on Reliability Indices (b) Normalized Deterioration Prediction Curves Based on the Bridge Condition Index



(a) (b)

Fig. 2 (a) Reliability-based Deterioration Curves (b) The Best Fit Deterioration Curve for a Newly Constructed Conventional Bridge

Table 1 Best fit distributions for the ultimate capacity of the system



Deterioration State	Time based on nominal cover (year)	Best fit ultimate resistance (GVW) distribution	Mean resistance GVW (KN)	Standard deviation of resistance GVW (KN)	System reliability index β
Just after construction	0	Lognormal	1566.11	214	7.43
Onset of corrosion	7.34	Lognormal	1553.38	213	7.36
Onset of longitudinal cracking	15.27	Lognormal	1513.53	210	7.14
Onset of spall	21.69	Lognormal	1443.17	256	5.92
50^{th} year	50	Lognormal	1150.48	191	4.87
75th year	75	Lognormal	1051.62	192	4.11

$$CI = 100 - 9.81 \times 10^{-5} t^4, \qquad 0 < t < t_s \tag{2}$$

$$CI = -0.488t + 89.67, \qquad t_s < t < 75 \tag{3}$$

5. Adoption of the developed model to an existing super structure as a case study

As a case study, and to show the application of such developed deterioration curve, the developed model is adopted to an old superstructure in Montreal. On May 10, 2000 the city of Montreal asked SNC Lavalin Inc. for an emergency visual inspection of the Monk Bridge (Zaki 2000). The superstructure of the bridge was a conventional steel-reinforced slab on concrete beams. The results of the visual inspection revealed signs of severe deterioration on the concrete slab and beams. The destructive core test was also performed on the concrete slab. Cores were taken on site using core cutting machine and the laboratory results showed that cores were in severely deteriorated condition. The content of chloride ions passed the threshold of 240 ppm in about 50% of the concrete sample cores taken from the slab. Inspection of the concrete beams together with the laboratory results indicated that generally the concrete of the beams was in a much degraded condition. Spall and corrosion of steel reinforcements are evident in Figs. 3 and 4. The plan view and cross section of the bridge superstructure is shown in Fig. 5.

According to the drawings, the bridge was designed in 1925. Assuming that the construction of the bridge was finished in the same year, at the time of inspection (in 2000) the bridge super structure was about 75 years old. Lack of the historical data regarding any other inspection or interventions during the life cycle of the bridge makes it difficult to model its in-service degradation profile. Assuming that no major rehabilitation was implemented on the deck and concrete beams, the effort was made to compare the predicted condition of the bridge using the model developed in this study.

The results of inspections and load rating calculations according to chapter 14 of the Canadian code (CHBDC-S6) showed that the slab was no longer able to carry the current level of design loads. Likewise, the analysis of the deteriorated concrete beams showed that the beams could not carry the code design loads. The core samples showed that the beams were severely degraded through de-icing salt and corrosion was evident on the majority of beam surfaces. At the end, the bridge evaluators recommended that the bridge be posted for lower loads such that heavy vehicles (e.g. trucks) weighing more than 10 tons are not allowed on the bridge for the time being. Engineers suggested a comparison be made between the following two alternatives: major rehabilitation of the superstructure or demolishing and reconstruction (Zaki 2000).

As already mentioned in Section 4, a bridge is categorized as ‘dangerous’ and ‘slightly dangerous’ conditions when the condition index CI is in the range of 0-19 and 20-39, respectively. Dangerous condition is when the bridge should be removed from the service, the deck or any other severely deteriorated component should be demolished and replaced with a new system immediately, while slightly dangerous condition indicates immediate major repair (Miyamoto *et al.* 2001). Considering the engineer’s evaluation report for this bridge, the superstructure might be categorized as slightly dangerous. It is obvious that since such a bridge was designed long time ago, evaluating the bridge capacity with the current design loads, the value of the bridge condition index should be less than 100 even at the time when bridge was opened to traffic. Consulting the expert bridge engineers, the value of CI for such an old bridge could be in the range of 50-60 at the time of construction (Zaki 2000). The reason for such a drastic decrease in the condition index as compared to that of a newly designed and constructed bridge could be the poor construction and material quality controls as compared to the recent construction methods and technologies, and lower design loads at the time of such design as compared to the new design code loading. Further research is recommended to establish more rational criteria to prove this concept. The generated reliability-based deterioration curves shown in Fig.1a are normalized to yield the condition index of 60 at the time of opening the bridge to traffic for the first year, 1925. The deterioration curves in terms of the CI and the elapsed time (in years) are given by Eqs. (4) and (5) and are shown in Fig. 6 where the time to spall is estimated to be $t_s = 21.69$ years.

$$CI = 60 - 5.509 \times 10^{-5} t^4, \qquad 0 < t < t_s \tag{4}$$

$$CI = -0.293t + 53.81, \qquad t_s < t < 75 \tag{5}$$

The condition index at the end of the bridge life cycle (75th year) is estimated to be 31.8 and the bridge could still be categorized in a slightly dangerous condition. This finding is in accordance with the results obtained from the bridge engineer's evaluation. Since in the system reliability-based deterioration model the correlation between structural elements and load redistribution is included, the higher reliability and condition indices are obtained as compared to the single element level evaluations. There exist some drawbacks as follows:

There is a lack of information regarding the history of the visual inspections and probable interventions for this case study bridge. In case where enough information exists, the deterioration model should be updated to obtain more accurate results on CI.

Fig. 3 Spall and Deterioration on the Deteriorated Concrete Beam (Monk bridge Montreal), (Zaki 2000)

Fig. 4 Spall and Deterioration Under the Deteriorated Concrete Slab (Monk bridge Montreal), (Zaki 2000)

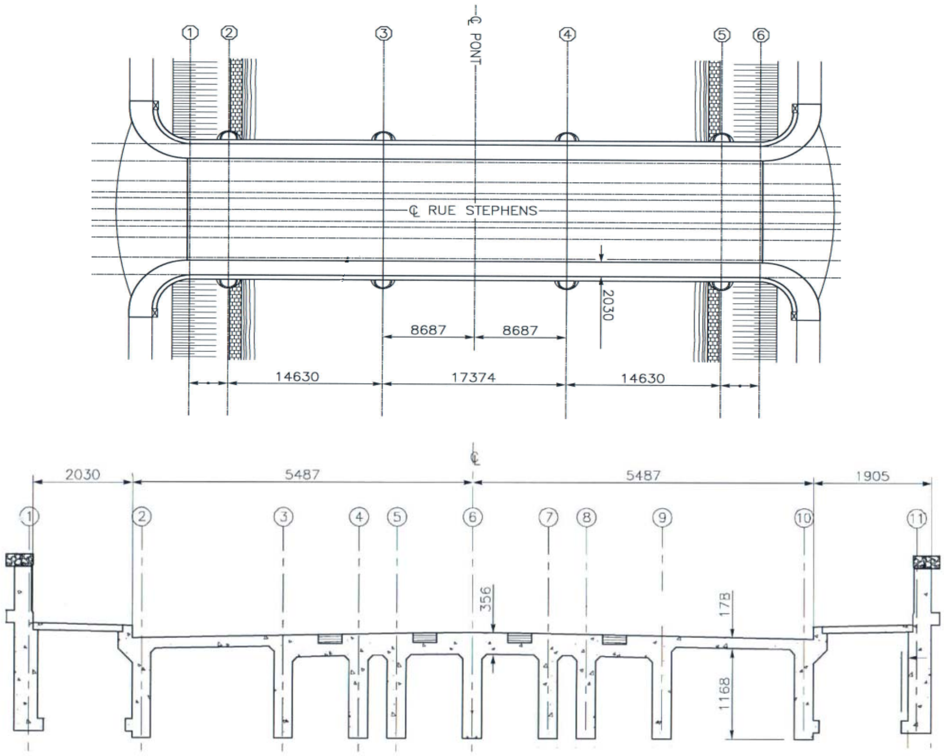


Fig. 5 Plan View and Cross Section of the Monk Bridge, Montreal (Zaki 2000, with permission).

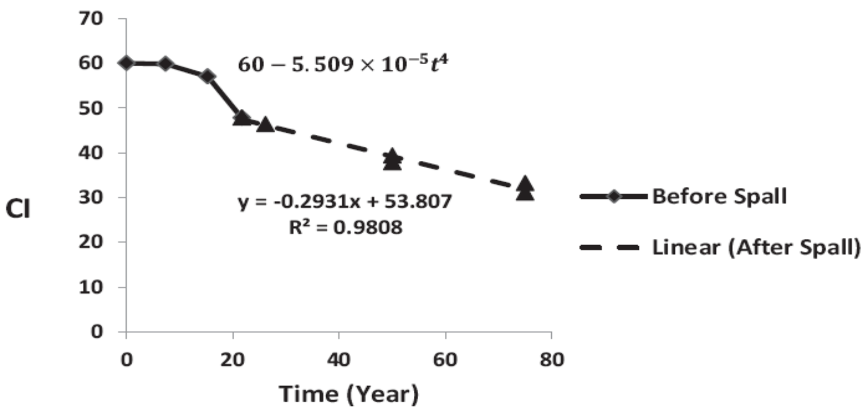


Fig. 6 The Best Fit Deterioration Curves for the Old Bridge

To develop the deterioration model, all the time periods are estimated based on the nominal concrete cover of 60 mm. However, for the Monk Bridge, the real cover depth is unknown. In case where the real concrete cover is documented in the specifications or it could be measured on site, a more accurate deterioration curve could be obtained. The developed deterioration curve is obtained based on the conservative assumption that deicing salt contaminates all the soffits of the beam and the slab in the case of an overpass bridge. However, the Monk bridge is built on a river, and as a result it may not get the salt splashing from underneath as is the case in overpass bridges; while the under-side of the bridge deck may be exposed to heavy moisture because of the river under it. In addition, the underside of the deck may be contaminated as the result of salty water overflow due to drain blockage.

The old existing bridges such as the Monk Bridge may not correspond to the loading specifications of the current design codes; therefore, a criterion needs to be established for a reasonable estimation of the condition index of such a bridge representing its undamaged condition which should be less than 100. An alternative to this approach is to start the deterioration curve with a CI of 100 since the bridge was deemed adequate at the time of construction based on the 1925 code. Then, over the years, the CI should be calculated based on the given code corresponding to the time at which the CI is calculated. However, this would require the vast knowledge of the changes in the bridge design codes and the detailed history of bridge inspection over time.

6. Conclusions

This article demonstrates how the proposed system reliability-based evaluation method can be applied for determining the structural condition of an existing bridge which represents an important step forward in bridge management systems. The condition index for a bridge at a given time is usually determined through the element level condition indices based on the bridge inspection data and often by the bridge inspectors. The condition index as determined above does not consider the interaction of the structural elements directly which in turn may not reflect the integrity of the structure. The system level condition index represented by the system reliability index considers the conditions of the individual elements in a bridge and the interaction of structural elements to determine the structural integrity of the whole system. The reliability index of a bridge can be calculated based on the condition information from visual inspection, non-destructive evaluation, past statistics and/or the structural health monitoring systems, if available.

This newly developed method can be easily integrated in the existing BMS by replacing the existing condition index by the reliability index or adding it as an additional regulatory parameter. The system reliability-based evaluation model presents important contributions in the field of bridge management because: A rational deterioration model for conventional bridges considering interaction between structural elements is developed by it, it helps the decision makers in predicting the appropriate time for major interventions from structural reliability point of view and It helps the bridge authorities in predicting and comparing the performance and life cycle cost of different structural systems over the bridge life span and selecting the most economic and appropriate one.

Here, the developed deterioration model is applied to an existing old bridge. Despite the lack of enough information about the history of the bridge, an attempt was made to compare the results obtained from the reliability-based prediction model and the deterministic evaluation techniques. It is observed that the obtained results are almost in accordance. The old existing bridges may not satisfy the loading specifications of the current design codes. In this research the expert judgment is used as a tool to estimate a condition indicator for old bridges just after construction. A more rational criterion needs to be established to objectively estimate the condition index of such bridges for the undamaged condition that could be rated less than 100.

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References

AASHTO. (2004), *LRFD bridge design specifications*, Washington, DC.

Adhikari, R.S., Bagchi, A. and Moselhi, O. (2013), "Automated condition assessment of concrete bridges with digital imaging", *Smart Struct. Syst*., **13**(6), 901-925.

Canadian Standards Association (CSA), (2006), *Canadian highway bridge design code (CHBDC)*, CAN/CSA-S6-06, Toronto.

Cusson, D., Lounis, Z. and Daigle, L. (2011), "Durability monitoring for improved service life predictions of concrete bridge decks in corrosive environments", *Comput.- Aided Civil Infrastruct. Eng.*, **26**(7), 524-541.

Czarnecki, A.A. and Nowak, A.S. (2007), "Reliability-based evaluation of steel girder bridges", *Procidings of the Institution of Civil Engineers, Bridge Engineering*, **160**(1), 9-15.

Ghodoosi, F., Bagchi, A. and Zayed, T. (2014a), "System-level deterioration model for reinforced concrete bridge decks", *ASCE J. Bridge Eng.*, 10.1061/(ASCE)BE.1943-5592.0000670 , 04014081.

Ghodoosi, F., Bagchi, A. and Zayed, T. (2014b), "Reliability-based condition assessment of an externally restrained bridge deck system considering uncertainties in key eesign parameters", *J. Perform. Constr. Fac.*, 10.1061/(ASCE)CF.1943-5509.0000699 , 04014189.

Huffman, S., Bagchi, A., Mufti, A., Neal, K., Sargent, D. and Rivera, E. (2006), "GFRP seismic strengthening and structural health monitoring of Portage Creek Bridge concrete columns", *Arabian J. Sci. Technol*., **31**(1), 25-42.

Ikpong, A. and Bagchi, A. (2014), "New method for climate change resilience rating of highway bridges", *J. Cold Reg. Eng.*, 10.1061/(ASCE)CR.1943-5495.0000079.

Li, H.N., Yi, T.H., Ren, L., Li, D.S. and Huo, L.S. (2014), "Reviews on innovations and applications in structural health monitoring for infrastructures", *Struct. Monit. Maint.*, **1**(1), 1-45.

Liu, Y. and Weyers, R.E. (1998), "Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures", *J. ACI Mater.*, **95**(6), 675-681.

McDaniel, M., Celaya, M. and Nazarian, M. (2010), "Concrete bridge deck quality mapping with seismic methods: case study in Texas", *J. Transport. Res. Record*, **2202**, 53-60.

Miyamoto, A., Kawamura, K. and Nakamura, H. (2001), "Development of a bridge management system for existing bridges", *J. Adv. Eng.Software*, **32**(10-11), 821-833.

Nowak, A. S. and Collins, K.R. (2000), *Reliability of Structures*, McGraw-Hill.

Oracle Crystal Ball 11.1.1 [Computer software]. Denver, Oracle.

SAP2000 15 [Computer software]. Berkeley, CA, Computers and Structures.

Stewart, M.G. and Rosowsky, D.V. (1998), "Structural safety and serviceability of concrete bridges subject to corrosion", *J. Infrastruct. Syst.*, **4**(4), 146-155.

Weyers, R.E. (1998), "Service life model for concrete structures in chloride laden environments", *J. ACI*

Mater., **95**(4), 445-453.
Vu, K., Stewart, M.G. and Mullard, J. (2005), “Corrosion-induced cracking experimental data and predictive models”, *ACI Struct. J.*, **102**(5), 719-726
Yi, T.H., Li, H.N. and Sun, H.M. (2013), “Multi-stage structural damage diagnosis method based on "energy-damage" theory”, *Smart Struct. Syst.*, 12(3-4), 345-361.
Zaki, A.R. (2000), *Structural condition of the Monk Bridge in Montreal*, Private Communication.

BMS	Bridge Management Systems
BCI	Bridge Condition Index
P_f	Probability of Failure
β	Reliability Index
C_s	Surface Chloride Content of Concrete
D_c	Effective Chloride Diffusion Coefficient of Concrete
C_{th}	Chloride Threshold of The Reinforcement
λ	Corrosion Rate of Steel Reinforcement
R	Deck Resistance
Q	Bridge Loading
CI	Condition Index
t_s	Time to Spall
f_y	Yield Strength of Steel reinforcement
E_s	Modulus of Elasticity of Reinforcing Steel
$f_c^{/}$	Compressive Strength of Concrete
E_c	Modulus of Elasticity for Concrete