CALIBRATION OF BRIDGE DECK DETERIORATION MODELUSING NON-DESTRUCTIVE EVALUATION

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Abstract— Performance of highway bridge decks should be monitored with special care especially in the regions where deteriorating conditions weaken the deck capacity with time. Bridge deterioration model is a powerful tool to predict the performance of bridge structural elements based on the results of visual inspections and Non-Destructive Evaluation (NDE) e.g. Ground Penetrating Radar test (GPR). In this research, a system reliability based deterioration model for a simply-supported reinforced concrete bridge deck is developed and calibrated. The structural conditions of deck structural elements are applied in a non-linear Finite Element model while system level reliability indices are estimated accordingly. Primary deterioration model for the deck is first developed for the uniformly contaminated concrete deck. Afterwards, the developed primary degradation model is updated based on a GPR test localized defect map. This article demonstrates the procedure for development and calibration of system reliability based deterioration model of reinforced concrete bridge decks.

Index Terms—Deterioration Model, Ground Penetrating Radar test (GPR), Reliability, reinforced concrete bridge deck.

I. INTRODUCTION

In cold regions, bridge decks are excessively subjected to deteriorating de-icing salt materials; as the result, performance prediction becomes a critical task. Bridges in Canada were mostly built between 1950 and 1975. From about 60,000 bridges in Canada almost 30,000 have reinforced concrete decks subjected to deteriorating conditions [1].

Currently, performance degradation is mostly modeled based on visual inspections where the condition states are assigned to individual elements. Such visual evaluations are proven to be subjective and uncertain in the United States [2] and[3].

In the process of a visual evaluation, inspectors are not able to detect subsurface flaws as well as internal cracks, rebar corrosion, concrete delamination and voids. Non-destructive evaluation (NDE) could be a more effective technique to overcome such limitations. Several NDE techniques are available to monitor the condition of bridge decks. Ground Penetrating Radar (GPR) has been found to be an appropriate method to identify internal faults in a concrete deck. GPR results are mainly in the form of localized defect maps [4].

System reliability-based deterioration model is a rational approach to predict the deck performance over time. This articles shows the process to develop a system level reliability-based deterioration model which is calibrated based on the GPR test localized defect map. As a case study, a simply-supported reinforced concrete bridge deck designed based on the Canadian Highway Bridge Design Code (CHBDC-S6) [5]is assessed in this study. Deck element-level deteriorated conditions for different time intervals are applied in the non-linear Finite Element model developed for the whole structure and the system level reliability indices are estimated accordingly. The developed deterioration model is then calibrated based on the defect map obtained from a GPR test on a bridge with similar plan dimensions.

II. BACKGROUND

A. Ground Penetrating Radar Test (GPR)

Ground Penetrating Radar (GPR) is a rapid object-detection NDE method based on propagation behaviour of electromagnetic (EM) waves. When an EM beam is focused on a concrete element, in case internal crack, flaw or spall exist, part of energy reflects back and is recorded due to difference in dielectric characteristics [4]. GPR is an effective technique to detect delamination and steel corrosion in concrete bridge decks [6]. GPR antenna is moved on the deck surface and detects reflected energy from a delaminated surface or a corroded steel rebar in the form of grayscale image called GPR profile as illustrated in Fig. 1 [7]. When the antenna is dragged on the whole deck surface, a defect map could be generated showing the localized defect areas.

B. Degradation Process of Reinforced Concrete Deck in Cold Regions

In the cold regions where salt-based de-icing material is widely used, steel reinforcement is corroded due to presence of chloride ion. When the surface chloride content of $C_s$is induced into a concrete element with effective chloride diffusion coefficientD_c, gradually the quantity of chloride around the steel rebar becomes a threshold of C_{th} and the corrosion is initiated (onset of corrosion). Concrete cover of steel reinforcement can significantly delay the time to onset of corrosion. Steel corrosion rate λ depends on the electric current magnitude related to the corrosion process.

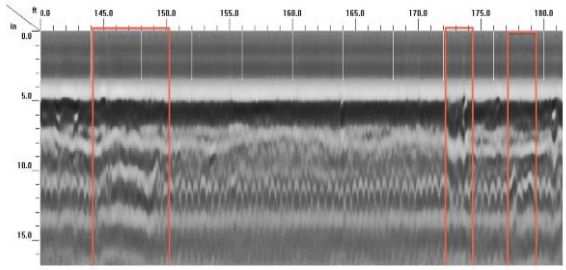


Fig.1. GPR scan and potential delamination [7]

The above mentioned parameters are very uncertain and variable in measuring and estimating [8].The developed degradation model based on such measured parameters should be updated and calibrated according to visual inspections and Non-destructive Evaluation (NDE), if available. In this study, the field data for the New York state are selected from Weyers (1998)where the values for C_s, D_c andC_{th} are reported as8.8 kg/m^3, 84 mm^2/year, 0.71 kg/m^3[9].Stewart and Rosowsky (1998) suggested the mean value ofthe corrosive electrical currentto bei_{corr}=1.5 $\mu A/cm^2$ for the uniform distribution. The corresponding corrosion rate related to a uniform reduction in the bar diameter could be stated as$\lambda \approx 0.0116 i_{corr}$ (mm/year)[10].
The time to the onset of corrosion is when the protective passive film around the reinforcement is dissolved and the chloride concentration reaches its critical threshold (C_{th}). Such timecould be estimated based on the Crank's solution of Fick's second law of diffusion [8].C_{th}depends on the concrete coverand the effective chloride diffusion constant, D_c. Longitudinal cracks appear as the result of the generated critical concrete tensile stress. Liu and Weyers (1998) proposed a model to predict the time to longitudinal cracking of concrete[11].The model suggested by Vu et al. (2005) is adopted in this research to estimate the time to spall[12].

C. Bridge deck Reliability Index

Reliability analysis is a rational technique to evaluate the structural safety. Reliability analysis is mainly based on the limit state function which is defined as $g=R-Q$, where R is the statistical variables representing the structural capacity and Q represents the loading on a structural system or element. The probability of failure is stated as the probability of (g<0). The safety margin of the structure is defined by the reliability index, $\beta = -\Phi^{-1}(P_F)$, where Φ^{-1} is the inverse of the standard normal distribution function and $P_F = P(g < 0)$ is the probability of failure. In this study the Rackwitz-Fiessler iterative method [13] isadopted for the reliability analysis purposes. In case where the load Q and the resistance R are normally distributed the reliability index β is estimated based on the equation (1). σ is thestandard deviation of a normal variable and μ is the corresponding mean value.In case wherethose variables are not normally distributed, Rackwitz-Fiessler iterative techniquecould be adopted to estimate the equivalent normal distribution of such non-normal variables.The details of such process could be found in [13] and [14].

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

Here, for bridge deck reliability analysis, the load model is presented in[15],[16] and [17].The maximum 75-year combination of Dead load, the Gross Vehicle Weight GVW (live load) and dynamic load are normally distributed variable, the details of such loadingis described in [18]. The resistance of the system R_{System} is the expected value of the capacity of the bridge deck based on the GVW's estimated for different transverse positions. It is important to note that the load combinations including wind, earthquake and collision forces require particular approach which is not in the realm of this study.

The best fit distribution for the bridge deck resistance is estimated based on the failure modes of the superstructure for different time intervals. For a steel-reinforced concrete bridge deck the failure modes are excessive deflection, the flexural collapse of the structural elements and the shear failure. Czarnecki and Nowak (2007) assumed that a bridge deck is unusable when the vertical deflection exceeds 0.0075 of the span length[17].

The simulations in this study prove that for a bridge superstructure designed based onCHBDC-S6 [5]flexural mode of failure always governs as opposed to the excessive deflection. It is found in this study that for a newly constructed bridge the flexural failure mode is governed by crushing of concrete in compression zone; however, for a severely degraded deck when the steel rebar is considerably corroded, the collapse is normally governed by the tension failure of the reinforcing steel bars. Shear failure of the concrete superstructure for different time intervals is not reported as the governing failure mode in this research.

Here, the deck resistance is defined as the maximum gross vehicle weight GVW of two side-by-side fully correlated trucks that causes the deck failure. The axle configuration of trucks are assumed to be similar to AASHTO standard truck [17] and [19].The magnitude of the load is increased gradually until one of the above mentioned deck failure modes occurs. The resistance of the system R_{System} is estimated as the expected value of the GVW's for seven transverse positions as recommended in [17].

In order to consider flexural mode of failure, rotation of the plastic hinges is controlled in a non-linear finite element model. The ultimate rotation of the plastic hinge is a function of the length of the plastic hinge (half of the section depth for a reinforced concrete

member), the curvature of the section at the start of yielding, and the maximum curvature of the section in the ultimate state.
Here, the trilinear Moment-Curvature diagram of a reinforced concrete element is plotted as explained in [18].The first cracking point of concrete in the tension zone is defined where the tensile stress in the outermost edge of the section reaches the $0.4\sqrt{f_c'}$ (f_c' is in MPa) for normal density concrete [5], the yielding point for a concrete element is when the tension steel reinforcement yields and the ultimate stage is when the concrete crushes in compression zone (0.0035 strain level according to [5]) or when the steel rebar ruptures in tension (10% strain for the steel bar according to [5]). The bilinear representation of the trilinear curve is obtained where both the diagrams represent an equal area under the curves representing the same amount of the absorbed energy[18].

Fig. 1 shows the methodology to estimate the reliability index for a bridge deck. This process is repeated for different time intervals during the bridge deck life span. While the load carrying capacity of the deck deteriorates over time, the reliability index β decrease accordingly.

In order to develop a primary deterioration model, it is conservatively assumed thatde-icing salt contaminates theslab top surface through the permeable wearingsurface and

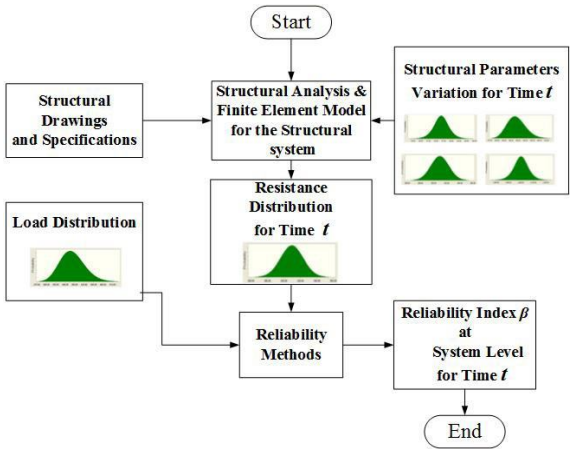


Fig. 1 The methodology to estimate the reliability index at the system level for each time interval

all soffitsof beams and slab when the salt-based material becomes airborne due to traffic beneath the bridge. Reduction of the steel rebar diameter due to corrosion results in the degradation of the capacity of beams and slab. Spall or delamination leads to reduction of concrete compression zone by the depth of spalled cover. Spall reduces the shear capacity due to the decreased depth of the cross section by one or both top and bottom covers.
Deterioration model for the concrete deck is developed by plotting the reliability indices for different time interval versus time. System reliability-based deterioration model can be developed for conventional and innovative structures based on this methodology[14] and [18].

As shown in Fig.1, in order to estimate the best fit distribution to the system resistance*R*,random values for the parameters governing the resistance of the deckare applied in the non-linear finite element model. To generate such random values from corresponding distributions the Oracle Crystal Ball software [20] has been used. The main parameters affect the capacity of a reinforced concrete deck are yield strength of the reinforcing steel (F_y), modulus of elasticity of reinforcing steel (E_s), compressive strength of concrete (f_c'), specific weight of cast in place concrete γ_c, slab thickness for cast in-situ concrete, thickness of beam web, depth of a beam, and cover for the steel reinforcement. The corresponding probability distributions for each parameter are mentioned in [18].

III. NON-LINEAR FINITE ELEMENT MODEL

To develop a 3D non-linear Finite Element model of the designed bridge deck SAP2000 [21] is adopted here. Simply supported superstructure of a bridge with 12m span has been designed in accordance withthe provisions of CHBDC-S6[5]. The geometry of the simply supported bridge superstructure and the cross section of the T-section main beams are illustrated in Table 1. The nominal concrete cover is assumed to be 60 mm.

The deck is modeled based on the grillage analogy as a planar grid of longitudinal (main T-section beams parallel to the roadway) and transverse members (perpendicular to the roadway).The calibration and verification process of the model is described in [18]. Two side by side fully correlated trucks are placed on the deck at the location in which the maximum moment is generated. Truck GVW is increased incrementally until the superstructure reaches the ultimate system capacity. The system resistance $R_{System}$is the expected value of the capacity of the bridge deck based on the GVW's estimated for different transverse positions [17]. In this process different set of random values for the above mentioned parameters are implemented in the Finite Element model. Finally, the best fit distribution to the bridge capacity is obtained using Oracle Crystal Ball software[20]. For each time interval, simulation process should be continued with different random values for variables until the estimated β stabilizes with no significant change in successive iterations.Deterioration model of the bridge superstructure is then obtainedby plotting the obtained reliability indices versus time.

The primary deterioration model is obtained based on the assumption of uniform de-icing salt contamination and deterioration for the deck top surface and soffits of

the bridge. Figure 2 illustrates the developed Finite Element model of the case study bridge superstructure. The best fit distributions for the deck capacity and resulting reliability indices for different time intervals are demonstrated in Table 1. The primary system reliability based deterioration model is obtained by plotting the reliability index versus time as shown in Figure 3.The best fit equation to the linear part of the curve (after onset of spall) shown in Figure 3 is mentioned in equation (2).

$$\beta = 4.37 - 0.044(t - 75), \quad 26.15 < t < 75 \qquad (2)$$

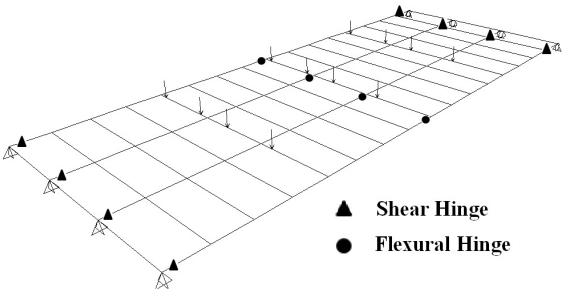


Fig 2. FE model using Grillage analogy, two side-by-side trucks are loaded on the bridge.

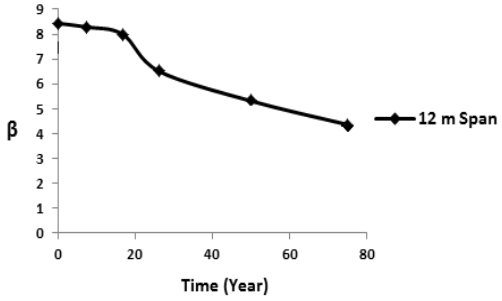
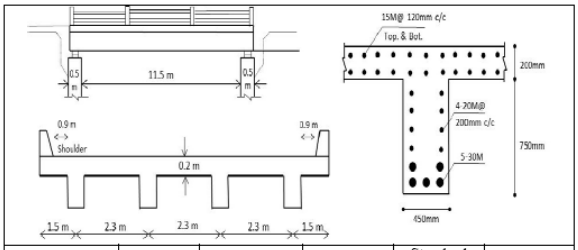


Fig 3. Primary Deterioration Prediction Pattern

Table 2 Reliability Indices and Best Fit Distributions for the Ultimate Capacity of the bridge superstructure



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	1694.56	205	8.44
Onset of corrosion	7.34	Lognormal	1677.6	206	8.22
Onset of longitudinal cracking	16.8	Lognormal	1626.77	203	8.00
Onset of spall	26.15	Lognormal	1524.02	236	6.52
50th year	50	Lognormal	1122.17	152	5.32
75th year	75	Lognormal	928.77	118.22	4.37

IV. CALIBRATION OF THE PRIMARY DETERIORATION MODEL BASED ON GPR RESULTS

Apparently, uniform de-icing salt contamination and degradation of the deck top surface and soffits may not be a realistic assumption. To overcome this problem, the reliability index of the case study bridge deck is calculated based on the results of a Ground Penetrating Radar (GPR) test implemented on a segment of a bridge with similar dimensions.

Figure 4 shows a spall defect map for the top surface of 77 square meter section of a concrete bridge deck located in Montreal, Canada obtained from GPR non-destructive test [7]. The age of the bridge deck was almost 45 years at the time of inspection. The bridge deck was demolished and replaced afterwards. Based on the similar dimensions of the GPR defect map and the case study bridge already discussed, it is assumed that such top surface spall defect map is applied tothe 12 m span case study bridge deck after 45 years. The GPR test for soffits of the superstructure is a very complicated process and such defect map for the deck underneath is not available; therefore, a uniform soffit salt contamination is conservatively assumed here. A uniform reduction in the bar diameter $\lambda \approx 0.0116 i_{corr}$ (mm/year) is considered here where the mean value of $i_{corr} = 1.5\ \mu A/cm^2$ as suggested in[10].

The top surface defect map shown in Figure 4 is then applied to the non-linear Finite Element model of the bridge superstructure as shown in Figure 5. Two types of flexural plastic hinges are considered here, spalled and non-spalled. Spalled plastic hinges are those which the top surface is delaminated and non-spalled are those which no spalling is reported on the top surface as shown in the defect map. In both types of plastic hinges the time to spall for the uniformly contaminated deck soffits is estimated according to the model suggested in[12]. For each main T-section beam parallel to the roadway, three flexural plastic hinges are introduced, one in the middle and two in each quarter of the beam. Two shear hinges are modeled at each corner similar to the previous case.
The resulting best fit distribution to the resistance of the bridge based on the Gross Vehicle Weight of two fully correlated trucks pass side by side over the deck is a lognormal distribution with the mean value of 1149.08 kN and the standard deviation of 143.59 kN.

The corresponding GPR based reliability index is estimated to be β=5.75. For the 45 years old uniformly defected deck the reliability index is β=5.69 obtained from equation (2).The comparison between the reliability indices for two cases shows that non-uniform spall of the deck surface causes slightly higher reliability index as compared to the uniform deck delamination.

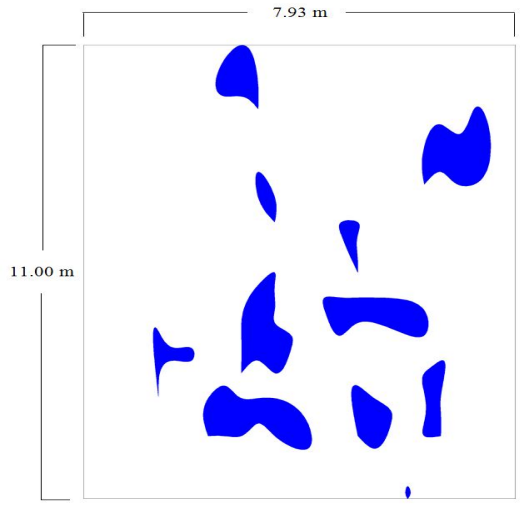


Fig 4. Map of defects based on GPR results[7]

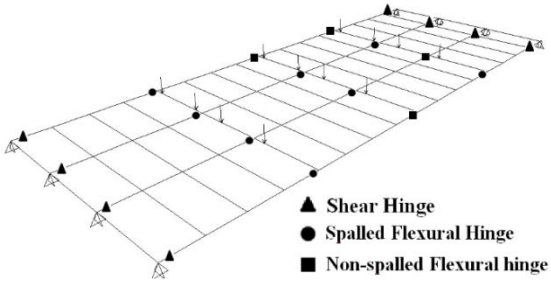


Fig 5. FE model based on the GPR test results

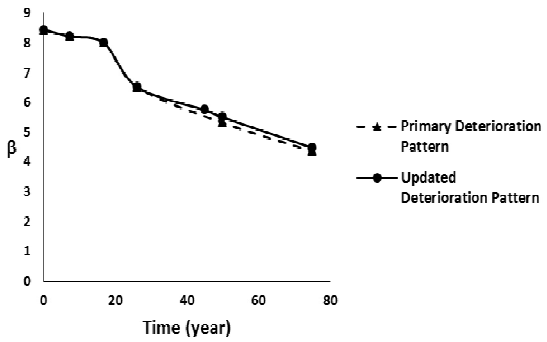


Fig 6. Primary and Updated Deterioration Prediction Patterns

Marginal one percent difference between two reliability indices indicates that assuming uniform delamination of the deck surface might be reasonable.It is recommended that such process be repeated for different scenarios of defect maps. Figure 6 shows the updated deteriorationpattern based on the result of GPR test as compared to the primary degradation model.

SUMMARY AND CONCLUSIONS

This articles demonstrates the effectiveness of reliability analysis to develop a system level deterioration model for bridge superstructure which could be updated and calibrated based on the results of Non-destructive evaluations (e.g. GPR). System reliability analysis is a powerful tool to assess the safety of structural systems.In this study the primary deterioration model of a bridge deck is first obtained based on the assumption of uniform degradation of the deck surface and soffits. As already mentioned measurement of parameters affect the deterioration process of concrete decks is very complicated and those parameters are very uncertain. Here,the field data for such parameters are adopted from New York State where the climate condition is very similar to Canada. One of the reasonable methods to achieve a reliable deterioration pattern is to calibrate and update the primary developed model by the results of nondestructive testing. In this research the primary deterioration curve is updated based on the map of spall defects obtained from non-destructive Ground Penetration Radar (GPR) test. This process could be repeated time by time to make sure that the degradation pattern of the deck is predicted in a reasonable and objective manner.

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