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Review

# Comparative durability analysis of CFRP-strengthened RC highway bridges

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The paper presents parametric analysis of durability factors of RC highway bridges strengthened with CFRP laminates during their service life. Durability factors considered are concrete cover and CFRP laminate thickness. Three deterioration factors were considered. First, growth of live load with time. Second, resistance reduction due to chloride-attack corrosion which causes reduction in steel properties. Corrosion losses are evaluated through a time–temperature dependent corrosion current. Two types of corrosion are considered; uniform and pitting corrosion. Third, deterioration due to aging of CFRP. The reliability analysis is controlled by three failure modes; concrete crushing, CFRP mid span debonding and CFRP rupture. Monte-Carlo simulation is used to develop time dependent statistical models for rebar steel area and live load extreme effect. Reliability is estimated in term of reliability index using FORM algorithm. For illustrative purpose, a RC bridge is assumed as an example. The reliability of interior beam of the bridge is evaluated under various traffic volumes and different corrosion environments. The bridge design options follow AASHTO-LRFD specifications. The present work also extends to calibrate CFRP resistance safety factor corresponds to three target reliability levels, $\beta = 3.5$, 3.85, and 4.2. The results of the analysis have shown that corrosion has the most significant effect on bridge life time followed by live load growth. Pitting corrosion type is more hazardous than uniform. Also, initial safety index is proved to be traffic dependent. AASHTO design equation (that corresponds $\beta_{target} = 3.5$) seems to be overestimated for strengthening purpose. Strengthening with ($\beta_{target} = 4.2$) provide better reliability than $\beta_{target}$ proposed by AASHTO provision with no significant differences in CFRP amounts required.



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## 1. Introduction

The performances of reinforced concrete bridges during their service life are strongly affected by many deterioration factors. The most important deterioration factor is corrosion of the reinforcement rebars. A considerable research effort has been done to evaluate corrosion effects. It has been found from these studies that corrosion causes reduction of steel properties – area, yield strength, and ultimate strain – [1,2], losses of bond between concrete and steel bars, cracking, and spalling of concrete cover [3,4]. Another factor of great importance in evaluating the reliability of highway bridges is the growth of live load – truck weight and number per day – with time. Additional resistance through a certain strengthening technique is required to compensate for losses in section resistance or to support new amounts of live loads. In the past, additional number of doweling bars embedded in a new concrete volume which is adjacently casted to the old concrete element. More recently, steel plates is attached to the concrete surface of tension zone using adhesive materials or bolts. Even more recently, carbon fiber reinforced plastic CFRP materials have been used for the purpose of strengthening; Laminates are externally bonded to extreme concrete tensile surface to produce additional bending strength. CFRP strengthening provides many advantages comparing to the two former techniques such as ease of installation, strength to weight ratio, and high resistivity to exposure conditions (e.g., [5]).

In the last two decades, the flexural behavior of CFRP strengthened RC beams has been well studied in a significant numbers of experimental and numerical publications. It had been proven that, strengthening using CFRP is capable to enhance the performance or RC concrete structural elements e.g. [6,7]. Different failure modes were observed, which can be classified into two types according to the location of occurrence. First, modes occur at position of maximum moment. Such modes are concrete crushing, CFRP mid span debonding (due to flexural crack or flexural-shear crack), and CFRP rupture. Second, failure modes occur at plate end. Such modes are interfacial shear stress induced CFRP end debonding, cover separation, and CFRP debonding (due to pure shear crack). The increase in section performance due to strengthening depends on actual deterioration level of concrete section. The ACI Committee 440 [5] reports an increase in nominal resistance ranges between 10% and 160% when strengthening RC elements with CFRP materials. However reduction in ductility requirements ranges between 5% and 40% may be observed after strengthening.

Reliability of non-strengthened concrete section damaged by pitting and uniform corrosion was extensively discussed in many previous publications i.e. [8–12]. In an early study proposed by Plevris et al. [13], the authors suggested a specific reduction factor ($\Psi$ = 0.8) for the CFRP contribution to strengthened element resistance. However the study was limited to FRP rupture failure mode only assuming full composite action. However, the authors neglected mid span CFRP debonding. A recent publication by Pham and Al-Mahaidi [14] discussed the reliability of strengthened beams considering multiple types of failure modes; CFRP rupture, intermediate crack debonding, and end debonding. Atadero and Karbhari [15] studied the reliability versus time of CFRP strengthened concrete beams considering uniform corrosion and FRP durability. The authors focused on the importance of the statistical properties of CFRP material on a CFRP safety factor, which had been calibrated for various targets reliability indices.

The present work aims to compare the effect of concrete cover and CFRP laminates thickness on CFRP strengthened RC girders service life. The reliability analysis was performed considering growth of the live load with time, traffic volume effect, corrosion of steel bars, CFRP durability. We deal with failure modes of the first type as they are considered as principal failure modes, while the second types can be totally prevented using a certain system of anchorage. Live load model was proposed taking into consideration randomness in truck weight, position, multiple truck simultaneous presence (side-by side through bridge lanes), and amount of traffic volume (truck/day).

## 2. Corrosion of reinforcement rebar

### 2.1. Corrosion stages

The corrosion deterioration process can be divided into three stages; initiation, propagation before cover cracking, and after cover cracking. This classification corresponds to the observed variation of corrosion current parameters through each stage [8,13,16].

#### 2.1.1. Corrosion initiation

In many previous studies, chloride attack had been considered as a diffusion process of moisture through voids spread in concrete media, which is assumed also to be relatively moist. 1D Fick's second law is chosen to represent the diffusion process:

$$\frac{\partial C_{(x,t)}}{\partial t} = D_{cl}\frac{\partial^2 C_{(x,t)}}{\partial x^2} \tag{1}$$

where $C$ is the chloride ions concentration at a depth $x$ in the concrete in the diffusion direction, $t$ is the time, and $D_{cl}$ is the chloride diffusion coefficient in concrete which is taken as:

$$D_{cl} = D_{cl,ref} f_{cl1}(T) f_{cl2}(t) f_{cl3}(RH) \tag{2}$$

where $D_{cl,ref}$ is a value of $D_{cl}$ which corresponds to a reference temperature ($T_{ref}$ = 298 K), at a critical relative humidity ($RH_c$ = 0.75), and at a reference time ($t_{ref}$ = 28 days). The three functions in Eq. (2) were formulated by Val and Trapper [17] as:

$$f_{cl1}(T) = \exp[U_c(1/T_{ref} - 1/T)/R] \tag{3a}$$

$$f_{cl2}(t) = (t_{ref}/t)^{m_{age}} \tag{3b}$$

$$f_{cl3}(RH) = [1 + (1 - RH)^4/(1 - RH_C)^4]^{-1} \tag{3c}$$

where $T$ is the absolute temperature in (Kelvin), $U_c$ (=44.6 ± 4.46 kJ/mol) is the activation energy, $R$ is the universal gas constant, $m_{age}$ is the aging coefficient, and $RH$ is the relative humidity. $D_{cl,ref}$ is influenced by mix proportions, curing, compaction, and environment, and can be expressed as [9]:

$$D_{cl,ref} = 0.15 D_{H_2O}\frac{1 + \rho_c wc}{1 + \rho_c wc + \rho_c ac/\rho_a}\left(\frac{\rho_c wc - 0.85}{1 + \rho_c wc}\right)^3 \tag{4}$$

where $D_{H_2O}$ is the chloride diffusion coefficient in an infinite solution (=50491.08 mm$^2$/year for NaCl), $ac$ is the aggregate–cement ratio, $wc$ is the water–cement ratio. $\rho_c$ and $\rho_a$ are the mass densities of cement and aggregates respectively. Solution of Eq. (1) gives the

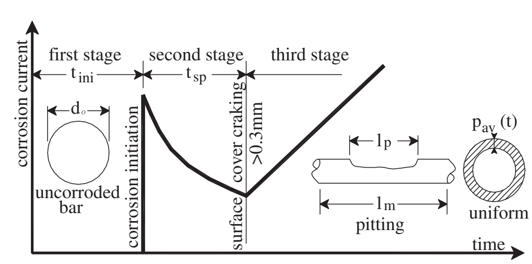


**Fig. 1.** Time-variant corrosion rate.