TIME DEPENDENT RELIABILITY OF RC BEAMS SUBJECTED TO CORROSION ATTACK

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ABSTRACT

Reinforcement corrosion can cause the strength and serviceability degradation of existing reinforced concrete (RC) structures. This paper presents the time-dependant reliability assessment of RC beams subjected to reinforcement corrosion considering the stochastic nature of load processes and uncertainties in the strength. For corrosion degraded RC beams, flexural and shear strength degradation modes are considered. By considering the variability associated with the parameters that could affect the strength of corrosion-degraded RC beams, evaluation of the following are presented: (i) time-dependant mean strength and (ii) time-dependant variability associated with the strength. It has been found that reliability is sensitive to both strength and loads.

INTRODUCTION

Decisions pertaining to continued service of corrosion-affected RC structures should be supported by quantitative evidence that the degraded strength of such structures is sufficient to withstand future extreme events within their intended residual service period with an acceptable level of reliability. For corrosion-affected RC structures time-dependent reliability evaluation considering the uncertainties in structural strength and strength degradation would help in the efficient allocation of limited resources for periodic inspection and maintenance of such structures. Safety evaluation and damage assessment of existing structures has been studied by many researchers owing to the socio-economic need of using existing infrastructures [1 – 5]. Mori and Ellingwood [2] developed a methodology that can be applicable for the assessment of time-dependant reliability of corrosion-affected RC structures. Their methodology takes into account both the randomness in strength and the stochastic nature of load processes which are modeled as Poisson processes.

This paper presents the time-dependant reliability estimation of RC beams subjected to corrosion attack. Corrosion will reduce the cross-sectional area of reinforcing bar, concrete section and the bond strength, further resulting in the reduced flexural and shear strengths of the RC beams. The reliability analysis is carried out for two limit states: (i) flexural failure and (ii) shear failure. The analysis also includes the variability in material properties, dimensions of beam, corrosion rates and loads. For illustrative purposes, reliabilities are evaluated for a deteriorating simply supported RC beam.

CORROSION PROPAGATION

For corrosion propagation in RC beams, it is assumed that: (i) the loss of flexural strength is mainly due to loss of cross-sectional area of steel, concrete section and bond between the concrete and the reinforcement, and (ii) the loss of shear strength is mainly due to the loss of cross-sectional area of steel and concrete section.

Loss of Cross-Sectional Area of Steel

Corrosion process is a dynamic process; growth of expansive corrosion products is given by Eq. 1 [6 – 7].

$$\frac{dW_r}{dt} = \frac{k_p}{W_r}\ ;\ k_p = 2.48614 \cdot \pi \cdot D_i \cdot i_{COR} \tag{1}$$

where W_r = mass of the expansive corrosion products (mg/mm); t = corrosion time (years); k_p = function of the rate of metal loss; D_i = initial diameter of reinforcement (mm); i_{COR} = annual mean corrosion rate (μA/cm^2). Various parameters associated with the loss of steel at time, t, since initiation of corrosion are evaluated by Eq. (2) [7].

$$W_s(t) = 2.42362 \cdot \sqrt{D_i \cdot i_{COR} \cdot t}\ ;\ D_r(t) = \sqrt{D_i^2 - 0.39245 \cdot \sqrt{D_i \cdot i_{COR} \cdot t}}\ ;\ A_{cor}(t) = 0.30835 \cdot \sqrt{D_i \cdot i_{COR} \cdot t} \tag{2}$$

where, $W_s(t)$ = mass of steel per unit length of the reinforcement (mg/mm) getting consumed by corrosion process; $D_r(t)$ = reduced bar diameter (mm); $A_{cor}(t)$ = loss of cross-sectional area of steel (mm^2).

Loss of Concrete Section

Reduction in concrete section occurs due to peeling of bottom, top and side covers to the reinforcements. Methodology for estimating the time required for peeling of cover concrete is explained in the reference [7].

Loss of Bond Strength between Concrete and Steel

The bond strength at time, t (years), from the initiation of corrosion is evaluated by using an empirical model of bond strength for corroded bars, given by Eq. (3) [7].

$$\tau_{bu}(t) = \tau_{bui} \text{ for } X_p(t) \le 1.5\ \%;\ \tau_{bu}(t) = 1.192 \cdot e^{-0.117 \cdot X_p(t)} \cdot \tau_{bui} \text{ for } X_p(t) > 1.5\% \quad (3)$$

where $\tau_{bu}(t)$ = bond strength at time, t (years), from the initiation of corrosion; τ_{bui} = initial bond strength for un-corroded reinforcing bar; $X_p(t)$ is defined as the loss of mass of reinforcing bar expressed as a percentage of original mass of the bar at time, t (years), from the initiation of corrosion; and $X_p(t) = [W_s(t)/W_{si}] \times 100$; W_{si} = initial mass of un-corroded reinforcing bar.

TIME-DEPENDENT FLEXURAL AND SHEAR STRENGTHS OF CORRODED RC BEAMS

Fig. 1 shows the typical cross-section of a simply supported un-corroded RC beam, which is subjected to flexure and shear under loads. Fig. 2 shows three representative schemes of deteriorated reinforced concrete sections for a corrosion affected RC beam [8]. Various notations with and without suffix 'i' shown in the same figures have their usual meanings. All the Cross-Sections in Fig. 2 indicate the reduced sections of main bars and shear stirrups at time, t, shown with the notations having their usual meaning. The reduced concrete section for a deteriorated corroded beam at time, t, is governed by the individual cover peeling time for top, bottom and side covers and in some cases it may be different than those are shown representatively in Fig. 2.

Time-Dependent Flexural Strength

For corroded RC beams time-dependent flexural strength is evaluated for two cases: (i) without considering the loss of bond between concrete and steel and (ii) considering the loss of bond between concrete and steel.

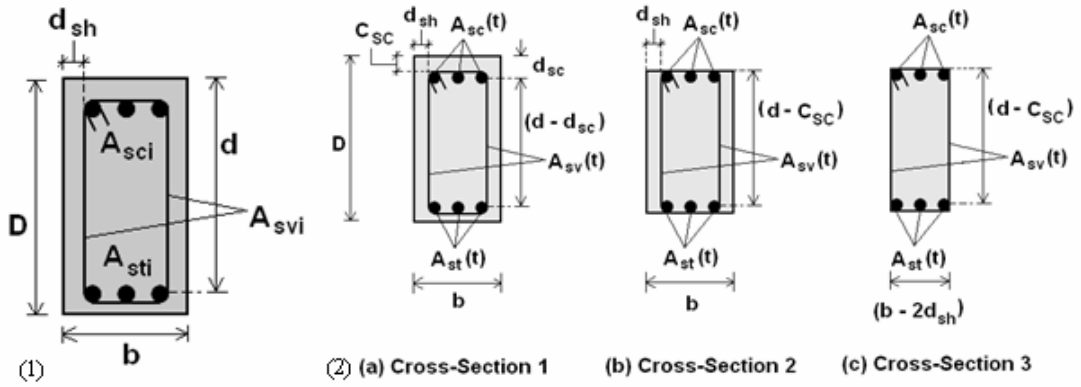

Fig. 1: Typical cross-section of an un-corroded RC beam; Fig. 2: Different schemes of deteriorated reinforced concrete cross-sections for corroded RC beams [8]

Time-Dependent Flexural Strength without Considering Loss of Bond

Reduction in flexural strength would be mainly due to loss of cross-sectional area of the steel and concrete section. For doubly reinforced sections, Eq. (4) – (5) can be obtained from equilibrium corresponding to time, t [9].

$$F_{cc}(t) = \kappa \cdot f_{ck} \cdot b(t) \cdot x_u(t) \cdot \left[\frac{3\varepsilon_{cc} - 0.002}{3\varepsilon_{cc}} \right];\ F_{sc}(t) = f_{sc}(t) \cdot A_{sc}(t) \quad (4)$$

$$Mu(t) = F_{cc}(t)\cdot[d(t) - Y_c(t)] + F_{sc}(t)\cdot[d(t) - d_{sc}]\ ;\ Y_c(t) = x_u(t) - x_u(t)\cdot\left[\frac{6\varepsilon_{cc} - \left(\frac{0.000004}{\varepsilon_{cc}}\right)}{12\varepsilon_{cc} - 0.008}\right] \quad (5)$$

where $\kappa = 0.446$; $\varepsilon_{cc} = 0.0035$; all other notations have their usual meanings. Eq. (2) will give $A_{st}(t)$ and $A_{sc}(t)$. $Mu(t)$ is evaluated by Eq. (5).

Time-Dependent Flexural Strength Considering the Loss of Bond

Reduction in flexural strength would be mainly due to loss of cross-sectional area of steel, concrete section and bond between concrete and steel. For doubly reinforced sections, Eq. (6) – (7) can be obtained from equilibrium corresponding to time, t [9].

$$F_{st}(t) = n_{st}\cdot\pi\cdot D_{str}(t)\cdot l_d\cdot\tau_{bu}(t)\ ;\ \varepsilon_{st}(t) = \frac{F_{st}(t)}{A_{st}(t)\cdot E_s}\ ;\ \varepsilon_{cc}(t) = \frac{\varepsilon_{st}(t)\cdot x_u(t)}{\left[d(t) - x_u(t)\right]}\ ;\ \varepsilon_{sc}(t) = \frac{\varepsilon_{st}(t)\cdot\left[x_u(t) - d_{sc}\right]}{\left[d(t) - x_u(t)\right]} \quad (6)$$

$$F_{cc}(t) = \kappa\cdot f_{ck}\cdot b(t)\cdot x_u(t)\cdot\left[500\cdot\varepsilon_{cc}(t) - \left(\frac{250000}{3}\right)\cdot\varepsilon_{cc}^2(t)\right]\ ;\ Y_c(t) = x_u(t) - x_u(t)\cdot\left[\frac{\left(\frac{1000}{3}\right) - 62500\cdot\varepsilon_{cc}(t)}{500 - \left(\frac{250000}{3}\right)\cdot\varepsilon_{cc}(t)}\right] \quad (7)$$

where n_{st} = number of bottom tensile steel bars; l_d = development length of the tensile bars. Eq. (7) is used for $\varepsilon_{cc}(t) \le 0.002$. For $0.002 < \varepsilon_{cc}(t) \le 0.0035$, $F_{cc}(t)$ and $Y_c(t)$ are evaluated using Eq. (4) – (5) by putting $\varepsilon_{cc} = \varepsilon_{cc}(t)$. Eq. (3) will give $\tau_{bu}(t)$; $Mu(t)$ is evaluated by Eq. (5).

Time-Dependent Shear Strength

Reduction in shear strength would be mainly due to loss of cross-sectional area of steel and concrete section. Shear strength corresponding to time, t is calculated by Eq. (8) [9].

$$\tau_c(t) = \frac{0.85\sqrt{0.8 f_{ck}}\left(\sqrt{1 + 5\beta(t)} - 1\right)}{6\beta(t)}\ ;\ \beta(t) = \frac{0.8\cdot f_{ck}\cdot b(t)\cdot d(t)}{689\cdot A_{st}(t)}\ ;\ Vu(t) = \tau_c(t)\cdot b(t)\cdot d(t) + \frac{0.87\cdot f_y\cdot A_{sv}(t)\cdot d(t)}{S_v} \quad (8)$$

where S_v = spacing of shear reinforcement; all other notations have their usual meanings.

TIME-DEPENDENT RELIABILITY ANALYSES

Stochastic Models of Structural Loads

Events resulting in significant loads on the structures occur randomly in time with random intensities. The duration of such significant load events is generally small and they occupy only a small fraction of the total life of the component or structure. Therefore, with these assumptions, a time varying structural load can be modeled as a sequence of randomly occurring pulses with random intensity, S_j, ($j = 1, 2, \ldots, n$) and a small duration, τ, as shown in Fig. 3. It is assumed that τ is sufficiently small, so that no degradation takes place during the application of the load. By assuming that the occurrence of the load events is described by Poisson point process, a simple stochastic load model can be obtained. The sequences S_j are assumed to be identically distributed and statistically independent variables described by distribution function $F_S(s)$. This simple stochastic load model describes the temporal load variation by its mean occurrence rate, λ, duration, τ, and the distribution function of the structural action, S, arising out of the load event.

Method of Time-Dependent Reliability Analyses for Single Component

Consider a structural member undergoing strength degradation and subjected to time variant load effects. The limit state probability or probability of failure in the time interval $(0, t_L)$ is given by Eq. (9) [2].

$$P_f(t_L) = 1 - \int_0^{\infty}\exp\left[-\lambda\left(t_L - \int_0^{t_L} F_S\left\{R(t)\right\}dt\right)\right]\cdot f_{R_o}(r)dr\ ;\ R(t) = R_0\cdot g(t) \quad (9)$$

where $R(t)$ = strength at time t; R_o = strength at un-degraded (original) state; $g(t)$ = time-dependent degradation function defined as the ratio of $R(t)$ and R_o at time t; $f_{Ro}(r)$ = probability density function (PDF) of R_0 and expressed in units that are dimensionally consistent with S. It is assumed that $g(t)$ is deterministic and independent of load history. If the member is subjected to two statistically independent load processes S_1 and S_2 and only S_1 is time varying, then by the law of total probability, the probability of failure is given by Eq. (10) [2].

$$P_f(t_L) = 1 - \int_0^\infty \int_0^\infty \exp\left[-\lambda\left(t_L - \int_0^{t_L} F_S\left\{R(t) - S_2\right\} dt\right)\right] \cdot f_{S_2}(S_2) dS_2 \cdot f_{R_0}(r) dr \; ; f_{Si}(S_i) = \text{PDF of } S_i \qquad (10)$$

ILLUSTRATION OF TIME-DEPENDENT RELIABILITY FOR A TYPICAL CORROSION-AFFECTED RC BEAM

The estimation of time-dependent failure probability is illustrated for a typical simply supported corrosion-affected RC beam as shown in Fig. 4. The beam is subjected to corrosion attack. The statistical parameters for the basic variables for material strengths, dimensions and annual mean corrosion rate appropriate for the RC beam are given in Table 1. They are similar to the ones suggested by other researchers [3 – 5]. Here it is noted that in subsequent presentation, time $t = 0$ shall indicate the initiation of corrosion unless mentioned otherwise.

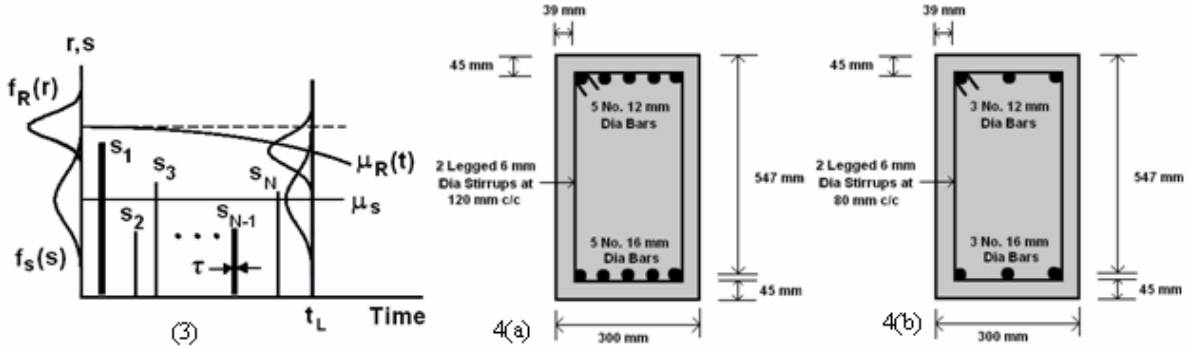

Fig. 3: Schematic representation of load process and resistance degradation [2]; Fig. 4: Typical RC beam cross-section: (a) at mid span, (b) at supports.

Table 1: Statistical parameters for basic variables for material strengths, dimensions and annual mean corrosion rate

Variables		Mean	c.o.v.	Distribution
Material strengths	f_{cm}	25.8 MPa	0.18	Normal
	τ_C	0.421 MPa	0.18	Normal
	θ	2.0	0.20	Normal
	f_y	466.88 MPa	0.11	Lognormal
	E_s	200000 MPa	0.051	Lognormal
Dimensions	b	310.3 mm	0.033	Normal
	D	614.4 mm	0.017	Normal
	C_B	46.6 mm	0.123	Normal
	C_T	48.2 mm	0.105	Normal
	C_S	40.6 mm	0.099	Normal
Annual mean corrosion rate	i_{COR}	5 μA/cm^2	0.1, 0.2 and 0.3	Normal
Note: All the notations have their usual meanings.				

Estimation of Time-Dependent Strength, g(t) and Coefficient of Variation Associated with Strength

The time-dependent mean strength and the coefficient of variation (c.o.v.) associated with the strength are evaluated using the "Latin Hypercube Technique (LHT)" [10] with 40 samples considering the variability in the variables given in Table 1. Figs. 5 – 7 illustrate the mean time-dependent strength of the RC beam for flexure without loss of bond, shear and flexure with loss of bond, respectively, evaluated using LHT with 40 samples. In the same figures, mR_{nb}, mR_{sh} and mR_{wb} stand for the sample mean flexural strength without loss of bond, shear strength

and flexural strength with loss of bond, respectively estimated using LHT; R^m_{nb}, R^m_{sh} and R^m_{wb} stand for the approximated mean flexural strength without loss of bond, shear strength and flexural strength with loss of bond, respectively, estimated considering all the variables to be at their mean value. V_I is c.o.v. for i_{COR} given in Table 1.

It is clear from Figs. 5 – 6 that R^m_{nb} agrees well with mR_{nb}, and R^m_{sh} agrees well with mR_{sh} except in the time interval of about 0 – 3 years where a slight difference between them is observed. This difference is mainly attributed to the randomness associated with the time of cover spalling at the bottom, top, and side resulting in the change of cross-section for the concrete. Once all the covers have spalled, the strength is mainly governed by $A_{cor}(t)$, which is caused by the i_{COR}. Therefore, after spalling of all the covers the strength becomes function of one variable i.e. i_{COR} resulting in good agreement between R^m_{nb} and mR_{nb}, and R^m_{sh} and mR_{sh}. Fig. 7 shows good agreement between R^m_{wb} and mR_{wb} except during the time interval of about 12 – 36 years; wherein large difference between them is observed. At point 'S' in Fig. 7 R^m_{wb} falls suddenly. This is attributed to the fact that from point 'S' onwards the force developed in the tensile reinforcement due to reduced bond strength between concrete and steel becomes less than the force due to yield strength of the tensile bars resulting in the reduced flexural strength as compared to the one shown in Fig. 5. The large difference between R^m_{wb} and mR_{wb} during the time interval of about 12 – 36 years is mainly attributed to the randomness associated with the time of occurrence of sudden fall. The occurrence time of such sudden fall is mainly governed by τ_{bui}, $X_p(t)$, E_s, and f_y. The slight difference between R^m_{wb} and mR_{wb} in the time interval of about 0 – 3 years is mainly attributed to the randomness associated with the time of cover spalling at bottom, top and side resulting in the change of cross-section for the concrete. In the time intervals of 3 – 12 years and 36 – 60 years, the flexural strength is mainly governed by the loss of cross-sectional area of tension reinforcement resulting in good agreement between R^m_{wb} and mR_{wb}.

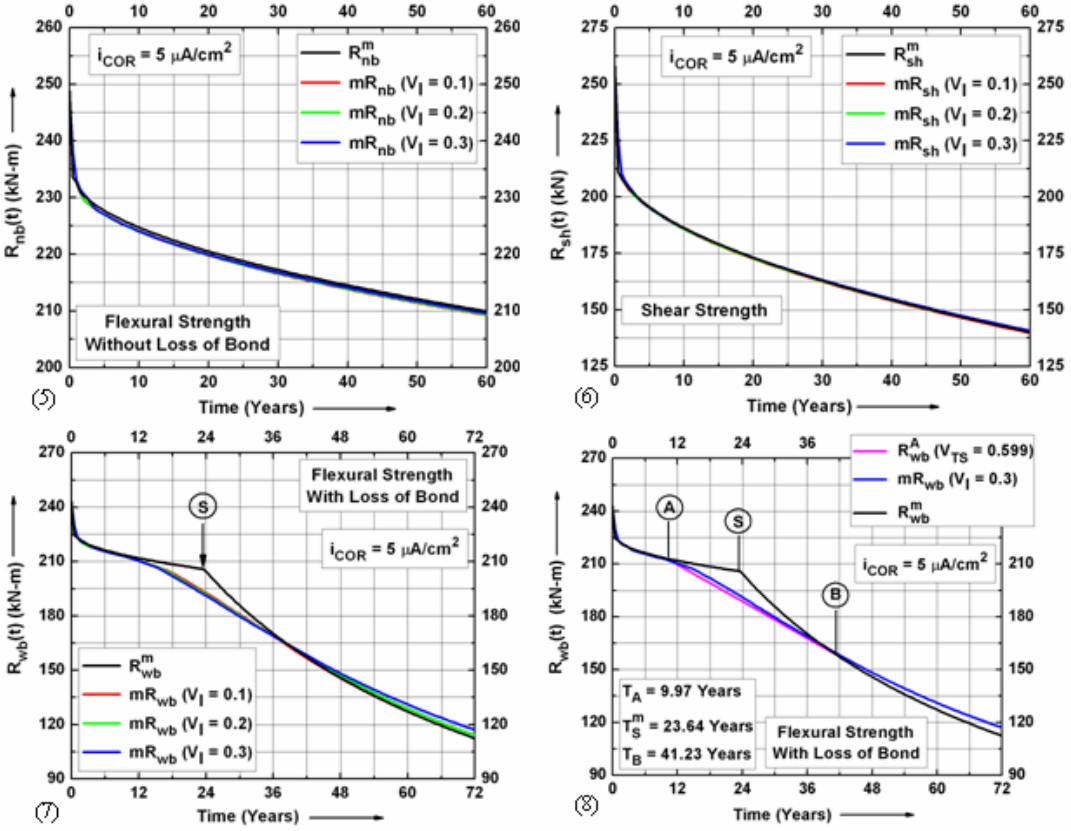

Fig. 5: Time-dependent flexural strength without loss of bond; Fig. 6: Time-dependent shear strength; Fig. 7: Time-dependent flexural strength with loss of bond; Fig. 8: Simple estimation of time-dependent flexural strength with loss of bond

Due to the large difference between R_{wb}^{m} and mR_{wb} during the time interval of about 12 – 36 years, a simple estimation with less computational efforts is expected for the evaluation of mean time-dependent flexural strength with loss of bond. One of the simplest ideas is to locate the points A and B as shown in Fig. 8 on the strength curve for R_{wb}^{m} by evaluating the times t_A and t_B corresponding to them as 80% confidence limits based on mean, T_S^m, and c.o.v. of occurrence time of sudden fall, and then connect these points by a straight line. With this assumption times t_A and t_B are evaluated and the points A and B are located on the strength curve for R_{wb}^{m} (Fig. 8). Simple estimation of T_S^m, and c.o.v. of occurrence time of sudden fall are given in the reference [7]. The proposed strength curve for the mean time-dependent flexural strength with loss of bond in the time interval of (t_A – t_B) is evaluated by linearly interpolating the strength curve R_{wb}^{m} between the points A and B. In the time interval of (0 – t_A) and (t_B – 60), the strength curve is represented by R_{wb}^{m}.

Fig. 9 presents mean $g(t)$ as a function of time, t, for flexural strength with and without loss of bond and shear strength. The $g(t)$ has been evaluated based on the strength curves R_{nb}^{m}, R_{sh}^{m} and R_{wb}^{m} as obtained in Figs. 5 – 6 and 8. Figs. 10 – 12 present time-dependent c.o.v. for flexural strength without loss of bond, V_{RNB}, shear strength, V_{RSH}, and with loss of bond, V_{RWB} for the statistical variations of i_{COR}. The sudden increase in c.o.v. during the time interval of about 0 – 3 years is attributed to the randomness associated with the time of cover spalling at bottom, top and side resulting in the change of cross-section for the concrete. Once all the covers have spalled, the c.o.v. is influenced by the variability associated with i_{COR}.

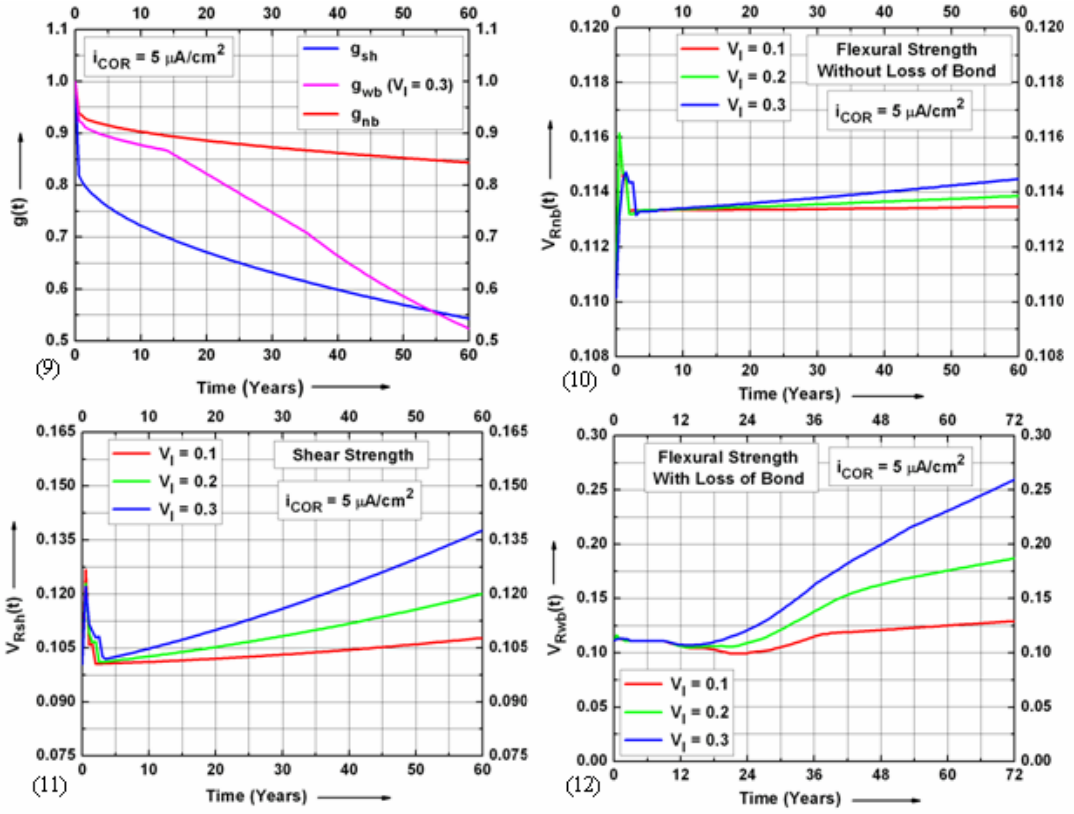

Fig. 9: g(t) as a function of time for flexural and shear modes of failure; Fig. 10. Time-dependent c.o.v. for flexural strength without loss of bond; Fig. 11: Time-dependent c.o.v. for shear strength; Fig. 12: Time-dependent c.o.v. for flexural strength with loss of bond.

Evaluation of Time-Dependent Failure Probability

For illustrative purposes, the corroded RC beam is subjected to the combination of time varying live load, *LL*, and the time invariant dead load, *DL*. The statistical properties of the load events and initial strengths are presented in Table 2. The time-dependent probability of failure, $P_f(t)$, for the corroded RC beam is evaluated by Eq. (10) for two cases: (i) ignoring the variability in time-dependent degraded strength, and (ii) considering the variability in time-dependent degraded strengths. For the case (i) mean degradation functions $g_{nb}(t)$, $g_{sh}(t)$ and $g_{wb}(t)$ as shown in Fig. (9) have been used for different failure modes. For the case (ii), $P_f(t)$ is evaluated for 40 random samples of strengths obtained through LHT. For illustrations, $P_f(t)$ are presented typically for a reference period of 60 years.

Table 2: Statistical properties for loads and initial strengths

Structural Loads					Initial Strength			
Load	λ	Mean	c.o.v.	PDF	Strength	Mean	c.o.v	PDF
Dead (DL)	-	$1.0 \cdot D_n$	0.07	Normal	Flexural Strength (NB)	248.82 kN-m	0.11	Lognormal
Live (LL)	0.5	$0.4 \cdot L_n$	0.5	Type I	Shear Strength	257.95 kN	0.10	Lognormal
D_n and L_n are nominal loads specified for design.					Flexural Strength (WB)	242.95 kN-m	0.11	Lognormal
D_n = 60 kN-m for flexure and 60 kN for shear L_n = 50 kN-m for flexure and 50 kN for shear					NB: without loss of bond; WB: with loss of bond			

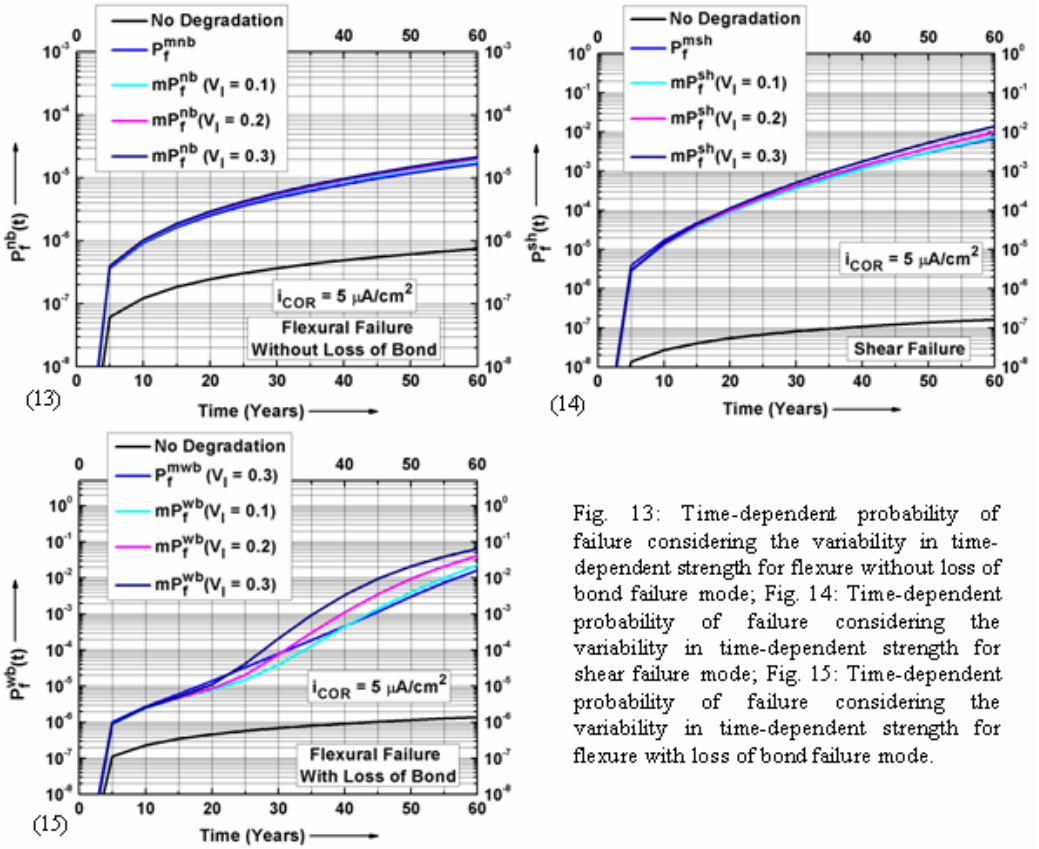

Fig. 13: Time-dependent probability of failure considering the variability in time-dependent strength for flexure without loss of bond failure mode; Fig. 14: Time-dependent probability of failure considering the variability in time-dependent strength for shear failure mode; Fig. 15: Time-dependent probability of failure considering the variability in time-dependent strength for flexure with loss of bond failure mode.

Results for $P_f(t)$, at time t (years) from the initiation of corrosion are presented in Figs. (13) – (15). In the figures $P_f^{nb}(t)$, $P_f^{sh}(t)$ and $P_f^{wb}(t)$ correspond to flexure without loss of bond failure mode, shear failure mode, and flexure with loss of bond failure mode. The same figures also present failure probabilities for the case of no degradation in flexural and shear strengths of the corroded RC beam. In the same figures, P_f^{mnb}, P_f^{msh} and P_f^{mwb} stand for the approximated mean probability of failure, wherein $P_f(t)$ is evaluated considering the variability associated with initial strengths only; mP_f^{nb}, mP_f^{sh} and mP_f^{wb} stand for the sample mean probability of failure,

wherein $P_f(t)$ is evaluated considering the variability associated with both initial and degraded strengths using LHT. These figures depict the following:

(i) In Fig. 13, good agreement is observed between P_f^{mnb} and mP_f^{nb} for all V_I values considered in the present study. It is noticed from Fig. 10 that for all the V_I values, the variability in degraded strength is about 4 % higher than that for the initial strength, which is a very low value, thus it may not affect $P_f(t)$ significantly, and thus resulting in good agreement between P_f^{mnb} and mP_f^{nb}.

(ii) In Fig. 14, good agreement is observed between P_f^{msh} and mP_f^{sh} for V_I = 0.1. It is noticed from Fig. 11 that at V_I = 0.1, variability in degraded strength is about 8 % higher than that for the initial strength, which is a low value, thus it may not affect $P_f(t)$ significantly, and thus resulting in good agreement between P_f^{msh} and mP_f^{sh}. It is also noticed from Fig. 11that at V_I = 0.2 and 0.3, variability in degraded strength is about 20 % and 38 % higher than that for the initial strengths, respectively, which are very high values, thus resulting in the noticeable difference between P_f^{msh} and mP_f^{sh} at these V_I values.

(iii) In Fig. 15 noticeable and/or large difference is observed between P_f^{mwb} and mP_f^{wb} for all V_I values considered in the present study. It is noticed from Fig. (12) that that at V_I = 0.1 – 0.3, variability in degraded strength is about 13 %, 58 % and 107 % higher than that for the initial strengths, respectively, which are high and/or very high values, thus resulting in the noticeable and/or large difference between P_f^{msh} and mP_f^{sh}.

CONCLUSIONS

Following conclusions are drawn based on the present study: (i) a methodology to evaluate time-dependent strength and associated c.o.v. is presented for corroded RC beams, (ii) it is proposed that, mean time-dependent flexural strength without loss of bond and shear strength can be evaluated by considering all the variable parameters to be at their mean values, (iii) simple estimation for mean time-dependent flexural strength with loss of bond is proposed, (iv) failure probability is sensitive to the mode of strength degradation in the corroded RC beams, and (v) if the difference between the variabilities in degraded and initial strengths is more than about 8 %, failure probability estimation should account for the variability in degraded strength.

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