

ASSESSMENT OF PROBABILISTIC CORROSION PROPAGATION MODEL FOR EXISTING REINFORCED CONCRETE BRIDGE DECK AND ITS RESIDUAL FLEXURAL CAPACITY

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ABSTRACT

This research presents an assessment approach for a probabilistic corrosion propagation model for embedded steel rebar in concrete subjected to chloride. The approach is applied to predict the level of corrosion propagation in the exterior layer of steel rebars of a highway bridge deck that was exposed to chlorides from various severe chloride concentrations. The probability of the corrosion propagation stage at different chloride concentrations was investigated using the performance function defined in different intervals of time (the time between the corrosion initiation and when the corrosive cracks appear, and the time after corrosive cracks appear). The probability of corrosion propagation was calculated using the Monte Carlo simulation method. This method accounts for uncertainties in the parameters governing the physical models of corrosion during the propagation stage, including concrete cover depth, the diameter of the steel rebar, and the compressive strength of concrete. The governing parameters are modeled as random variables with different levels of correlation. The proposed flexural capacity loss calculation was conducted due to corrosion in a reinforced concrete (RC) deck. A prediction equation model was developed to investigate the corrosion depth and the decrease in the corroded steel rebars as functions of time, from corrosion initiation to the appearance of the corrosive crack, and after its appearance, at different chloride concentrations. Finally, the percentage of reduction in the flexural capacity at various steel rebar diameter reductions compared to the intact rebar diameter utilized for the RC deck and T-Girder was assessed at various corrosion rates.

KEYWORDS: Probabilistic Corrosion Propagation Model; Chloride Concentrations; Corrosion Initiation; Corrosive Cracks; Flexural Capacity; Corrosion Rate.

1. INTRODUCTION

Corrosion of the steel rebars embedded inside the concrete of vital structures is a critical issue in reducing the life cycle of the structure [1]. However, these structures are subjected to various environmental and operational factors that may cause deterioration and reduce service life. One of the most common and severe forms of deterioration is chloride-induced corrosion of steel reinforcement, which affects the structural integrity and durability of RC bridges.

Tuutti [2] introduced a simplified model outlining the corrosion mechanism of reinforcing steel in concrete structures. Moreover, Tuutti [2] hypothesized it as a dual-stage process: an initiation stage, where corrosion commences, and a propagation stage, where visible damage becomes evident due to accumulated corrosion products. Various factors influence the duration of each stage, including environmental conditions, concrete properties, exposure time, and chloride diffusion rates [2, 3].

As detailed by several studies [4-8], the consequences of chloride-induced corrosion in RC structures include:

- i) a decrease in the rebar cross-sectional area and its load-carrying capacity,
- ii) a loss of steel ductility,
- iii) cracking and spalling of the concrete cover,
- iv) a loss of bond along the steel/concrete interface.
- v) Reduction in the total service life of the structure.

This research uses the Monte Carlo simulation (MCS) method as a computational method to develop a stochastic model for time-dependent reliability analysis. This method was chosen due to its accuracy



and suitability for long-term predictions. By increasing the number of samples in the MCS, the accuracy of forecasts improves, allowing for more precise predictions of the probability of corrosion propagation over extended periods. For each interval of time, the time-varying probability of corrosion propagation was predicted in this research using the MCS with 100,000 samples, assuming the random variables follow a normal distribution. Furthermore, in this study, all parameters are assumed to be independent and follow a normal distribution function.

Additionally, reliability is a vital indicator of the quality of specialized structures, such as RC bridges, that are exposed to extreme environmental conditions, including varying chloride concentrations from deicing salts. Reliability is defined as the probability of a performance function, denoted as $g(x)$, exceeding zero ($P\{g(x)>0\}$). Conversely, the probability of failure is defined as $P\{g(x)<0\}$. Typically, the complement of the probability of failure is calculated to determine reliability by assessing the durability of RC structures subjected to severe environmental conditions.

The main aims of this research are as follows:

- This research aims to investigate the time of corrosion initiation (T_i) and the time when corrosive cracks appear (T_{cr}) at different chloride concentrations applied on the concrete surface.
- The relationship between the projection of the corrosion depth and the decrease in the rebar diameter, respectively, over various times ($T_o < T < T_{cr}$) and ($T > T_{cr}$), will be investigated in this research based on R^2 at different chloride concentrations (3, 6, 10, and 18 kg/m³). Finally, a prediction equation was deduced to directly estimate the corrosion depth and the decrease in the corroded steel rebar diameter as a function of time, at the time between the corrosion initiation and the time when the corrosive crack appears, and the time after the appearance of the corrosive crack across the corresponding chloride concentrations.
- The probability of the corrosion-induced propagation stage at different chloride concentrations was investigated using the performance function defined in various intervals of time (i.e., the time between the corrosion initiation and the appearance of corrosive cracks, and the time after the appearance of corrosive cracks).

Finally, the residual flexural capacity of existing RC members, including the deck and T girder of the bridge after corrosion at different corrosion rates, would be investigated.

2. RESEARCH METHODOLOGY

2.1 Method of Calculation of the Probability of Corrosion Propagation due to Chloride Attack

The MCS method is a powerful numerical technique for accurately estimating the probability of carbonation or chloride-induced corrosion initiation [9-19]. The modeling was performed in this research using MATLAB. Additionally, the MCS method requires the generation of many random numbers [20] and involves three basic steps: (1) selecting the appropriate distribution type for each random variable; (2) generating random numbers based on the selected distribution; and (3) conducting simulations based on the generated random numbers [21]. The limit state function (LSF) is then evaluated for the generated random numbers, where each random number corresponds to a value of the LSF. Finally, the probability of corrosion is estimated by counting the number of LSF values less than zero and dividing that count by the total number of data points.

The performance function is generated as the difference between a term equivalent to resistance and another term equivalent to the load effect. The term resistance is used to refer to the critical corrosion depth. However, the term load effect is represented by the corrosion depth calculated over time caused by corrosion current density.

The probability of failure in the n-dimensional condition is based on the reliability theory, and it can be written using multi-integration on the joint probability density function as in equation (1) [21, 22].

$$P_f = P[G(x) \leq 0] = \int_{G(x) \leq 0} f(x) dx \quad (1)$$

where: x is the random vector that represents random variables, and $f(x)$ is the joint probability density function of the variables. The reliability index (β) is defined as the shortest distance between the space center of the problem and the limit state function in standard normal coordinates according to Hasofer and Lind [22].

In simple terms, when all the variables follow the normal distribution and are independent of each other, the failure probability is calculated as in equation (2).

$$P_f = \Phi(-\beta) \quad (2)$$

where: $\Phi(\beta)$ is the cumulative distribution function, and β is the reliability index.

2.2 Calculation of Time of Corrosion Initiation (T_i) due to Chloride Attack

The time of initiation due to chlorides depends on the chloride threshold (C_{th}), chloride concentration (C_s), chloride diffusion coefficient (D), and the concrete cover thickness (X) [23], as illustrated in equation (3) and Fig.1. The diffusion coefficient used in the analysis is equal to 1.27×10^{-12} m²/s according to Saassouh and Lounis [24].

$$T_i = \frac{X^2 \left[\operatorname{erf}^{-1} \left(1 - \frac{C_{th}}{C_s} \right) \right]^2}{4D} \quad (3)$$

Where: X : depth of concrete cover (m); D : chloride diffusion coefficient (m²/s); C_{th} : threshold level of chloride concentration (kg/m³); C_s : surface chloride concentration (kg/m³).

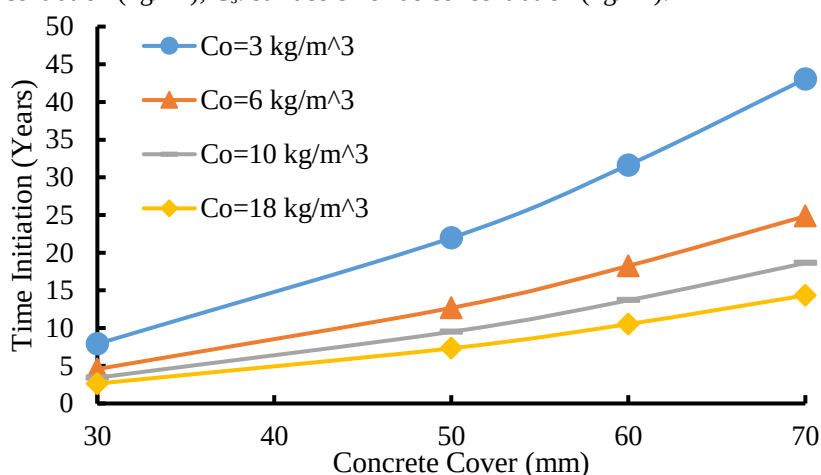


Fig.1. Projection of time of corrosion initiation (t_i) due to various chloride concentrations applied at different concrete covers.

2.3 Calculation of Time denoted by (T_{cr}) when corrosive cracks appear in the concrete.

The sign of flexural capacity deterioration appears when the reinforcing steel starts to rust, and the beginning of cracking of the concrete cover is the turning point, after which flexural capacity deterioration accelerates.

The time when corrosive cracks appear in the concrete depends mainly on two factors:

- The corrosion rate of reinforcing steel before cracking the concrete cover.
- The degree of steel corrosion. The main factors affecting the degree of steel corrosion at the time of cover cracking are the concrete cover thickness, concrete grade, concrete void content, and reinforcing steel diameter [25].

Thus, the time, denoted by (T_{cr}), when corrosive cracks appear in the concrete, is calculated using equation (4). The water-to-cement ratio is assumed equal to 0.4 for the concrete mix used for the RC deck of the bridge, the concrete cover is equal to 70 mm, and the diameter of the steel reinforcing rebar is equal to 25 mm. Thus, the time when the corrosive cracks appear, denoted by (T_{cr}), is determined using equation (4) at different chloride concentrations ($C_s=3, 6, 10$, and 18 kg/m³) as shown in Fig.2.

$$0.373 \frac{(1 - W/C)^{-1.64}}{X} \int_{t_0}^{t_{cr}} t^{-0.29} dt = k_1 k_2 k_3 \left[0.15 \left(\frac{W}{C} \right)^{1.55} f_c^{0.34} \left(1 + \frac{2X}{d} \right)^{0.19} d^{-0.3} \right] \quad (4)$$

Where: W/C is the water-to-cement ratio; X is the concrete cover (mm); $k_1 = 1 - 0.07m_1 - 0.54m_2 - 2.47m_3$; m_1, m_2, m_3 represent the added quantities of fly ash, slag, siliceous dust (kg); k_2 is a correction coefficient for the steel reinforcing position, its value is equal to 1.0 at corners and 1.33 in the middle of the side; k_3 is a correction coefficient for the type of reinforcing rebar, its value is equal to 1.0 for ribbed rebars and 0.88 for plain rebars; f_c is the 28-day-compressive strength of concrete (MPa) and d is the diameter of steel reinforcement rebar (mm).

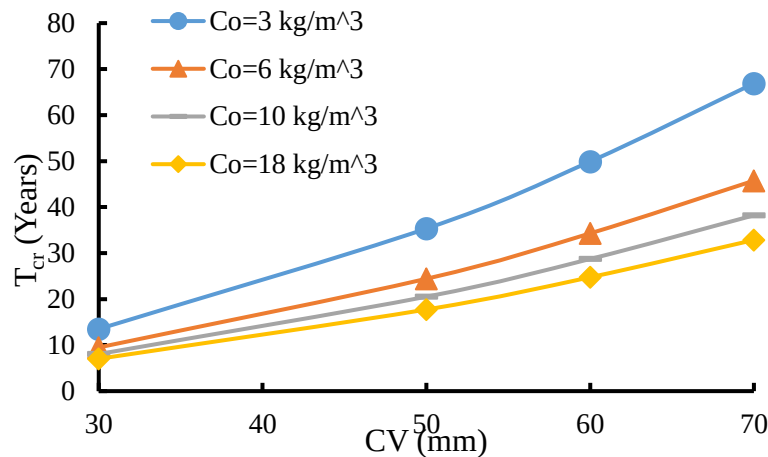


Fig.2. Projection of the time when the corrosive cracks appear (T_{cr}) due to various chloride concentrations applied at various concrete covers.

The second-degree polynomial functions were deduced between either the time of corrosion initiation or the time when the corrosive cracks appear, and the concrete cover for RC members based on the root mean square error ($R^2=1$), across various chloride concentrations applied on the RC members, as shown in Table 1.

Table 1. Prediction of the time of corrosion initiation and the time when corrosive cracks appear as a function of concrete cover across various chloride concentrations applied on the concrete surface.

Chloride concentration (C_s) (Kg/m ³)	Prediction of the time of corrosion initiation (T_i) (Years)	Prediction of the time when corrosive cracks appear (T_{cr}) (Years)
3	$T_i = (0.0088 \times CV^2) + (9 \times 10^{-8} \times CV) - 4 \times 10^{-5}$ ($R^2 = 1$)	$T_{cr} = (0.0122 \times CV^2) + (0.1192 \times CV) - 1.0141$ ($R^2 = 1$)
6	$T_i = (0.0051 \times CV^2) - (2 \times 10^{-6} \times CV) + 8 \times 10^{-5}$ ($R^2 = 1$)	$T_{cr} = (0.008 \times CV^2) + (0.1065 \times CV) - 0.8965$ ($R^2 = 1$)
10	$T_i = (0.0038 \times CV^2) - (1 \times 10^{-5} \times CV) + 0.0003$ ($R^2 = 1$)	$T_{cr} = (0.0065 \times CV^2) + (0.1009 \times CV) - 0.8443$ ($R^2 = 1$)
18	$T_i = (0.0029 \times CV^2) + (1 \times 10^{-6} \times CV) + 1 \times 10^{-5}$ ($R^2 = 1$)	$T_{cr} = (0.0055 \times CV^2) + (0.0964 \times CV) - 0.8026$ ($R^2 = 1$)

Where: T_i is the time of corrosion initiation (years); T_{cr} is the time when a corrosive crack appears (years); CV is the concrete cover, ranging from 30 mm to 70 mm.

2.4 Cracking state of the concrete cover due to corrosion

The cracking of the concrete cover caused by the corrosion of the reinforcing rebars is due to the corroded expansion force exceeding the tensile strength of the concrete. It is known by the Faraday corrosion law that the corrosion depth of the rebar is proportional to the corrosion current density. Additionally, the greater the corrosion current density reaches the steel reinforcing rebars, the deeper the corrosion depth of the steel rebars will be obtained. The corrosion current density can be regarded as the load random variable, and the impact of its cause is the corrosion depth [26, 27].

Thus, the relationship between the steel corrosion depth (mm) and time is illustrated in equation (5), which describes the expansion process of corrosion products on the surface of the steel rebar before corrosion that leads to the cracking of the concrete cover [26, 27].

$$\delta(t) = 0.0116 \int_{t_0}^t i_{corr}(t) dt$$

$$= 0.0116 \int_{t_0}^t 32.13 \frac{(1 - W/C)^{-1.64}}{X} t^{-0.29} dt$$

$$\delta(t) = 0.373 \frac{(1 - W/C)^{-1.64}}{X} \int_{t_0}^t t^{-0.29} dt \quad (t_0 < t < t_{cr})$$

$$\delta(t) = 0.52535 \frac{(1 - W/C)^{-1.64}}{X} (t^{0.71} - t_0^{0.71}) \quad (t_0 < t < t_{cr}) \quad (5)$$

where: t_0 is the time of corrosion initiation for reinforcing rebar (years); t is the time after corrosion of reinforcing rebar (years); T_{cr} is the time when the corrosive cracks appear (years); $i_{corr}(t)$ is the corrosion current density at the time (t) ($\mu A/cm^2$); X is the thickness of the concrete cover (mm).

Therefore, the cover cracks when the corrosion depth of the steel reinforcing rebar reaches the critical corrosion depth of the concrete cover. As a result, the critical corrosion depth (δ_{cr}) of the concrete cover that leads to the cracking can be regarded as structural resistance, and it is calculated using equation (6) [26, 27].

$$\delta_{cr} = k_1 k_2 k_3 \left[0.15 \left(\frac{m_w}{m_c} \right)^{1.55} f_c^{0.34} \left(1 + \frac{2X}{d} \right)^{0.19} (d)^{-0.3} \right] \quad (6)$$

According to equation (6), the decrease in the rebar diameter of a corroded reinforcing steel rebar at a time (t) is calculated using equation (7), assuming a uniform corrosion state.

$$\Delta D(t) = 1.051 \frac{(1 - W/C)^{-1.64}}{X} (t^{0.71} - t_0^{0.71}) \quad (t_0 < t < t_{cr}) \quad (7)$$

where: t_0 represent the time when steel reinforcing starts rusting (years), and T_{cr} represents the time when corrosive cracks of concrete cover appear (years).

2.5 Time-varying model of corrosion rate following cracking of the concrete cover.

Past research has shown little recognition of the rate of corrosion after cracking the concrete cover. Li [28] performed a great number of experiments on flexural members with load-induced cracks under a simulated marine environment. Yuan *et al.* [29] concluded that the corrosion rate increased after the cover had cracked. The empirical equation describing the relationship between corrosion current density and time is illustrated in equation (8).

$$i_{corr}(t) = 0.3683 \ln(t) + 1.1305 \quad (8)$$

The empirical equation (9) is based on the test results for the cracked specimens, which can be approximately applied to investigate the corrosion rates after corrosion-induced cracking of the concrete cover. According to equation (9), the decrease in diameter (mm) of a corroded reinforcing rebar at a time (t) is calculated using equation (10), assuming a uniform corrosion state.

$$\delta(t) = 0.0116 \int_{t_{cr}}^t [0.3683 \ln(t) + 1.1305] dt$$

$$= [4.272 \times 10^{-3} (t \ln(t) - t_{cr} \ln t_{cr})] + [0.884 \times 10^{-2} (t - t_{cr})] \quad (t_{cr} < t) \quad (9)$$

$$\Delta D(t) = 0.0232 \int_{t_{cr}}^t [0.3683 \ln(t) + 1.1305] dt$$

$$\Delta D(t) = [8.54 \times 10^{-3} (t \ln(t) - t_{cr} \ln t_{cr})] + [1.768 \times 10^{-2} (t - t_{cr})] \quad (t_{cr} < t) \quad (10)$$

2.6 Limit state function for the corrosive cracking failure state between the time of initiation and the time when the corrosive cracks appear.

The corrosion depth caused by corrosion current density is considered as a load effect, while the critical corrosion depth of the cover corrosive cracking is considered as structural resistance; the probability of failure is calculated using equation (12).

$$Z = R - S(t) = \alpha_{cr} \delta_{cr} - \alpha \delta(t) = 0 \quad (11)$$

$$P_f = P[R - S(t) < 0] = P[\alpha_{cr} \delta_{cr} - \alpha \delta(t) < 0] \quad (12)$$

where: α is the model uncertainty coefficient of equation (5) (see Table 2), and α_{cr} is the model uncertainty coefficient of equation (6) (see Table 2).

Table 2. Statistical parameters of main factors for the determination of the probability of corrosion propagation for corrosive cracking failure state [26, 27]

Random Variable	Coefficient of the mean value	Variable Coefficient	Distribution Type
X (concrete cover)	0.85	0.3	Normal
D (diameter of the reinforcing rebar)	1	0.015	
f_c (concrete compressive strength)	0.85	0.1	
α_{cr}	1	0.2	
α	1	0.2	

2.7 Maximum acceptable level of corrosive damage.

After the cracking of the concrete cover due to corrosion, the corrosion-induced crack provides an extremely convenient channel for the invasion of the erosive media, thus accelerating the corrosion speed of the reinforcing rebar. The structural damage caused by further corrosion of reinforcement is usually shown as an increase in crack width, peeling of concrete cover, and lower rigidity. All these damages affect normal service life, which is affected by the degree of corrosion (corrosion depth) and corrosion current density.

After the load effect and resistance model are given, the limit state equation and the probability of failure of the corrosion-induced damage reaching the maximum acceptable degree are expressed as illustrated in equations (13) and (14).

$$Z = R - S(t) = \alpha_d \delta_d - \alpha \delta(t) \quad (13)$$

$$P_f = P[R - S(t) < 0] = P[\alpha_d \delta_d - \alpha \delta(t) < 0] \quad (14)$$

where: α_d is the model uncertainty coefficient for calculating δ_d ; α is the model uncertainty coefficient.

$$\begin{aligned} \delta(t) &= \delta_1(T_{cr}) + 0.012 \int_{t_{cr}}^t [0.3683 \ln(t) + 1.131] dt \\ &= \delta_1(t_{cr}) + (4.272 * 10^{-3} (t \ln t - t_{cr} \ln t_{cr})) + (0.884 * 10^{-2} (t - t_{cr})) \quad t_{cr} < t \end{aligned} \quad (15)$$

where: $\delta_1(T_{cr})$ is the corrosion depth of the reinforcement at the time of cracking due to corrosion, which is calculated using equation (9).

Taking the critical corrosion depth (δ_d) of the reinforcement, when the surface of the concrete has the maximum acceptable appearance damage, as the structural resistance, δ_d can be calculated using equations (16) or (17) according to the type of steel rebar.

Circular steel rebars:

$$\delta_d = 0.255 + 0.012 \frac{x}{d} + 0.84 * 10^{-3} f_{cuk} \quad (16)$$

Ribbed steel rebars:

$$\delta_d = 0.273 + 0.008 \frac{x}{d} + 0.55 * 10^{-3} f_{cuk} \quad (17)$$

where: f_{cuk} is the standard value of compressive strength for a concrete cube.

2.8 Description of an existing RC bridge deck

The proposed flexural capacity loss calculation due to corrosion is illustrated on an RC highway bridge deck structure that is subjected to the application of deicing salts during winter. The reinforced bridge deck structure is made from a normal concrete deck with conventional carbon steel. Moreover, the reinforcement ratio is equal to 0.3% and is used for both top and bottom mats of reinforcement in the longitudinal and transverse directions, as illustrated in Fig.3. Table 3 illustrates the dimensions and the material properties for the RC deck of the bridge.

Table 3. Strength and dimension parameters for the RC deck of an existing bridge

Parameters	f_y (MPa)	f_c' (MPa)	Concrete cover (mm)	RC deck thickness (mm)	Diameter of steel rebar (mm)	Effective depth (d) (mm)
Deterministic Value	400	f_c' is assumed to be 35 MPa.	70	300	25	217.5

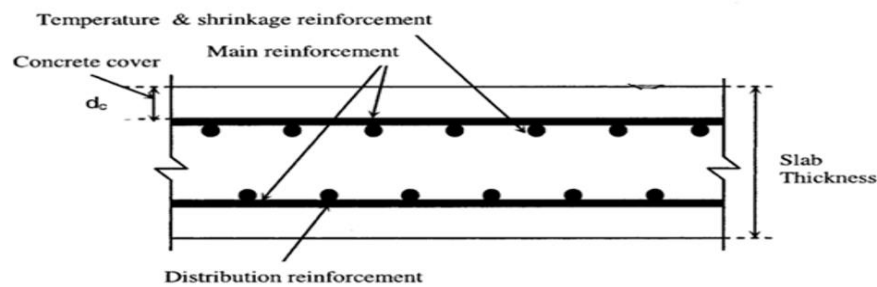


Fig.3. Details of steel rebars for an existing RC bridge deck.

2.9 Capacity loss of an existing RC deck/Girder of the bridge in flexural resistance after corrosion

The flexural strength of corroded RC members depends mainly on the total available area of rebars in the tension zone [30]. For uniform corrosion, as clarified in Fig.4, the total bending reinforcement area as a function of time (t) is calculated using equation (18) depending on the time of corrosion initiation.

$$A_s(t) = \begin{cases} \frac{n\pi D_b^2}{4} & t \leq T_i \\ \frac{n\pi [D_b - 2v * (t - T_i)]^2}{4} & t > T_i \end{cases} \quad (18)$$

where: D_b = diameter of steel reinforcement rebar; n = number of rebars; T_i = time of corrosion initiation; and v = rate of corrosion. Factor (2), as illustrated in equation (18), considers the uniform corrosion propagation process from all sides at the level of the rebar.

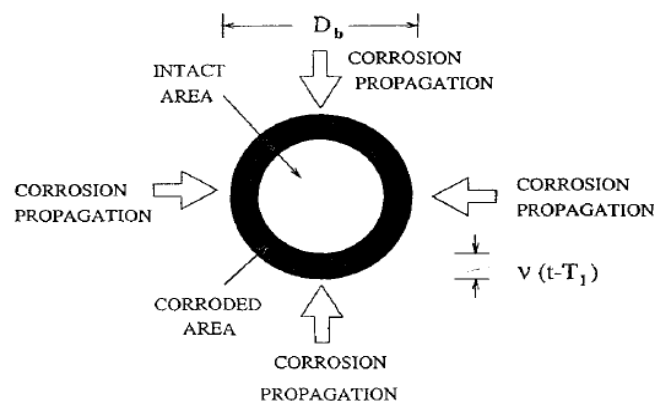


Fig. 4. Uniform corrosion in a reinforcing rebar adapted from [30].

The time of corrosion propagation (t_p) is the time required, after the onset of corrosion, to induce high tensile stresses in the concrete cover, which lead to its fracture through internal cracks, surface cracks, and spalling. It is given by the following equation (19).

$$t_p = \frac{\pi d (\Delta d)}{2 \times S \times j_r \times \left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \quad (19)$$

Where: Δd = increase in rebar diameter due to corrosion build-up; d = rebar diameter; S = rebar spacing; ρ_s = density of steel; ρ_r = density of corrosion product; α = molecular weight ratio of iron to the corrosion product; j_r = rate of rust per unit area ($\mu\text{A}/\text{cm}^2$). The values for ρ_s , ρ_r , and α are assumed to be equal to 7860 kg/m^3 , 3600 kg/m^3 , and 0.52 , respectively.

The average corrosion rate (v) for passive steel in concrete attacked by chlorides is about $100 \mu\text{m}/\text{yr}$ according to Frangopol *et al.* [30]. The corrosion rate changes with the environment, and no accurate data is available to predict the real corrosion rate. In this research article, the average corrosion rate (v) is assumed to be equal to $100 \mu\text{m}/\text{yr}$ in the calculation for the steel reinforcement rebars used in the RC deck subjected to corrosion.

As corrosion gradually progresses, the remaining capacities in bending moments decrease over time. The remaining flexural strength of the RC deck at the time (t), $M_r(t)$, may be computed as clarified in equation (20) and Fig.5 (according to the internal equilibrium condition).

$$M_r(t) = A_s(t) \Phi_s f_y (d - a_t/2) \quad (20)$$

Where: f_y = yield strength of the steel rebar; d = effective depth; and a_t = depth of the equivalent rectangular stress block over time of the corrosion is illustrated through equation (21).

$$a_t = (A_s(t) \Phi_s f_y) / (\alpha_1 \Phi_c F_c b) \quad (21)$$

where: Φ_c is a resistance factor for concrete (approximately 0.65 based on concrete compressive strength); Φ_s is a resistance factor for steel reinforcement (0.85) according to CSA-A 23.3-19.

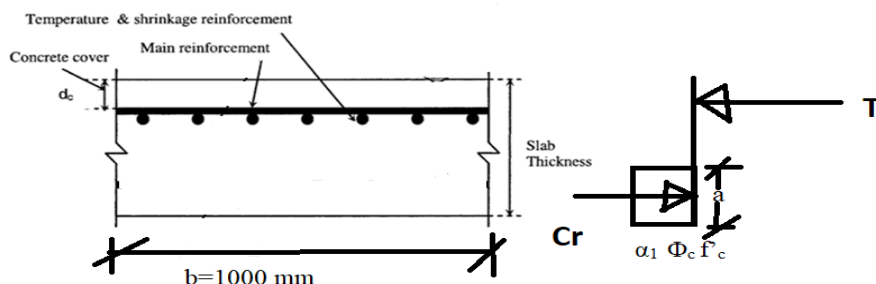


Fig.5. Analysis for equivalent rectangular stress block for an existing RC bridge deck.

In this research, the MCS method will be utilized to conduct the probability of chloride-induced corrosion propagation at different chloride concentrations, and the probabilistic model is validated compared with the results obtained by various researchers [10-19] for the probability of corrosion initiation due to chloride. Moreover, the current model coincides with the other results obtained by the different researchers mentioned. The current model shows the robustness and reliability of predicting the probability of corrosion propagation.

3. ANALYSIS OF RESULTS

3.1 Projection of the corrosion depths versus the time of corrosion initiation and the time when a corrosive crack appears at different chloride concentrations.

Fig.6 shows the relationship between the projected corrosion depth and the time between corrosion initiation and the time when a corrosive crack appears, which is calculated based on equation (5) for a concrete cover of 70 mm used for an existing RC bridge deck, and is a linear function (R^2 is approximately equal to 1) at different chloride concentrations, as shown in Fig.6.

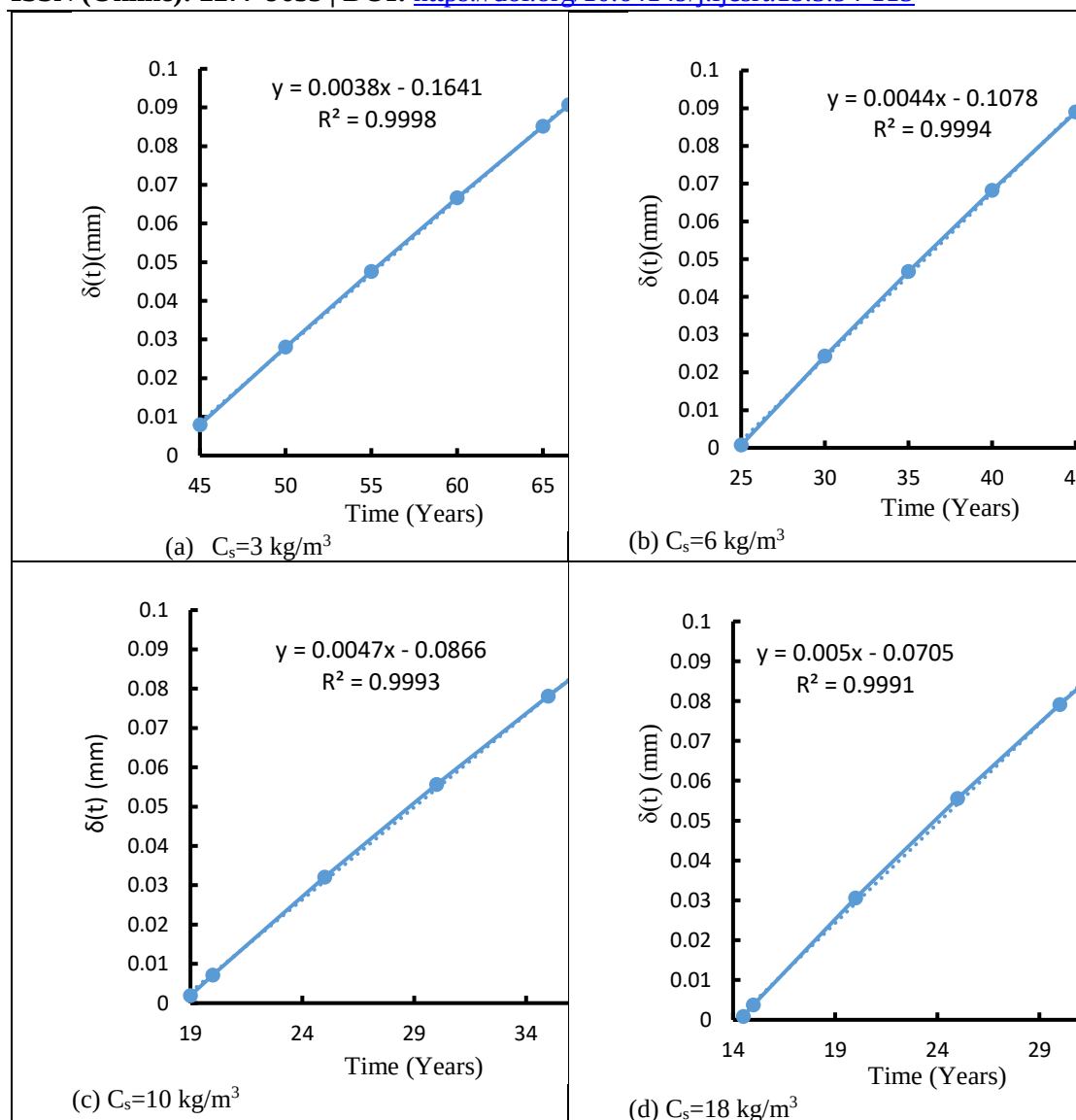


Fig.6. Projection of the corrosion depths at various chloride concentrations between the time of corrosion initiation and when the corrosive cracks appear for the concrete cover of 70 mm used for the RC bridge deck.

The decrease in the steel rebar diameter is calculated based on equation (7) (assuming a uniform corrosion state). Fig.7 illustrates the relationship between the projected decrease in the cross-sectional area of the steel rebar diameter of 25 mm due to the impact of various chloride concentrations, and the time between corrosion initiation and the time when a corrosive crack appears is a linear function at different chloride concentrations.

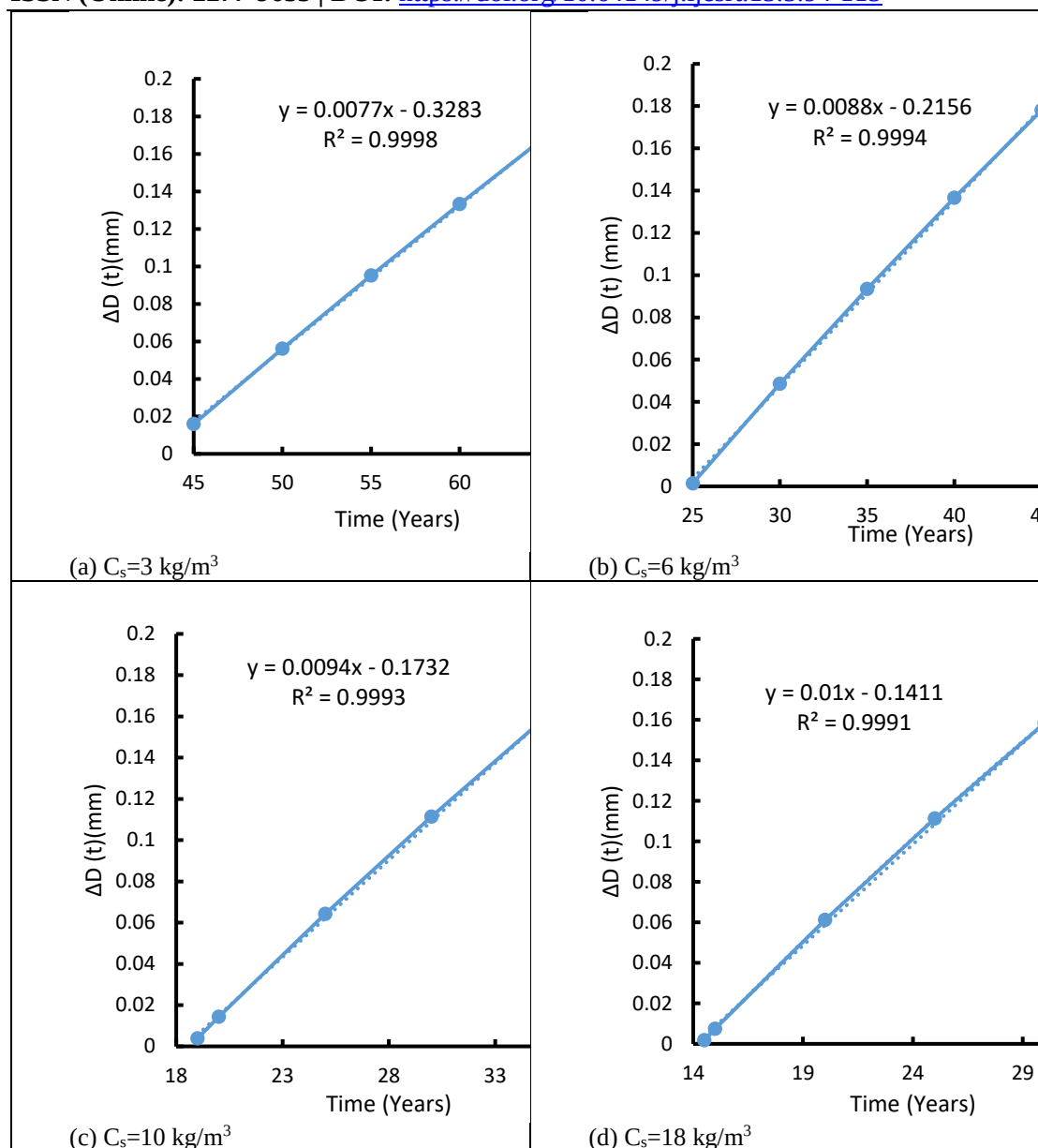


Fig.7. Projection of decrease in steel reinforcement rebar diameter at different chloride concentrations between the time of corrosion initiation and the time when the corrosive cracks appear for a concrete cover of 70 mm.

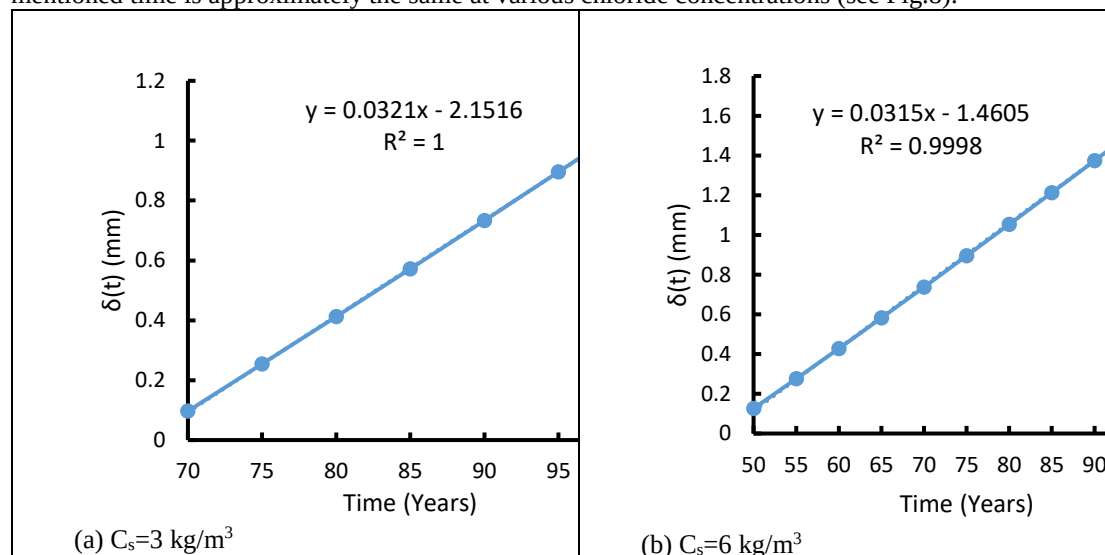
The concrete cover for an existing RC bridge used is equal to 70 mm, and the water-to-cement ratio is equal to 0.4 for an existing RC deck of a bridge that is subjected to different chloride concentrations. It was deduced from Fig.6 and Fig.7 that the relationship between the projection of the corrosion depth and the decrease in the rebar diameter, respectively, over time ($T_o < T < T_{cr}$), are linear relations according to R^2 , which corresponds to a value of 1 at different chloride concentrations (3, 6, 10 and 18 kg/m^3) as illustrated in Table 4. Furthermore, Table 4 shows a prediction equation for estimating the corrosion depth and the decrease in the corroded steel reinforcement rebar between the time of corrosion initiation and when the corrosive crack appears at different chloride concentrations.

Table 4. Linear functions for the prediction of the corrosion depths and decrease in corroded steel rebar diameter at various chloride concentrations for an RC deck of a bridge between the time of corrosion initiation and the time when the corrosive crack appears.

Chloride Concentrations (Kg/m ³)	Time Initiation (T _i) (Years)	Time of appearance of Corrosive Cracks (T _{cr}) (Years)	Equations for Prediction of the Corrosion Depths (Y) (mm)	Equations for Prediction of the decrease in the Rebar Diameter ($\Delta D(t)$) (mm) ($\Phi 25M$) (mm)
C _s =3 kg/m ³	43.1	66.9	Y=0.0038 (t)-0.1641 (R ² =0.9998)	$\Delta D(t)=0.0077(t)-0.3283$ (R ² =0.9998)
C _s =6 kg/m ³	24.9	45.8	Y=0.0044 (t)-0.1078 (R ² =0.9994)	$\Delta D(t)=0.0088(t)-0.2156$ (R ² =0.9994)
C _s =10 kg/m ³	18.6	38.2	Y=0.0047 (t)-0.0866 (R ² =0.9993)	$\Delta D(t)=0.0094(t)-0.1732$ (R ² =0.9993)
C _s =18 kg/m ³	14.3	32.9	Y=0.005 (t)-0.0705 (R ² =0.9991)	$\Delta D(t)=0.01(t)-0.1411$ (R ² =0.9991)

3.2 Projection of the corrosion depths versus the time after a corrosive crack appeared at different chloride concentrations.

Fig.8 shows the relationship between the projected corrosion depth and the time after the corrosive crack appears, which is calculated using equation (9) for the corresponding concrete cover of 70 mm for an existing RC bridge, and is a linear function (R-squared is approximately equal to 1) at different chloride concentrations. Furthermore, it was noticed that the time when the corrosive cracks appear is different at various chloride concentrations, while the slope between the corrosion depth versus the mentioned time is approximately the same at various chloride concentrations (see Fig.8).



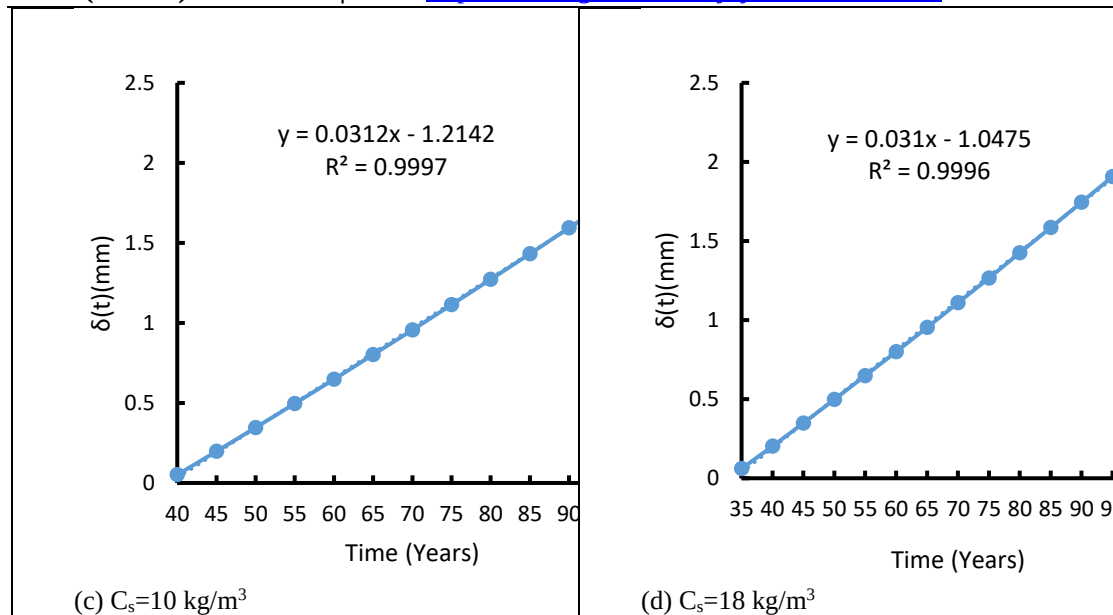
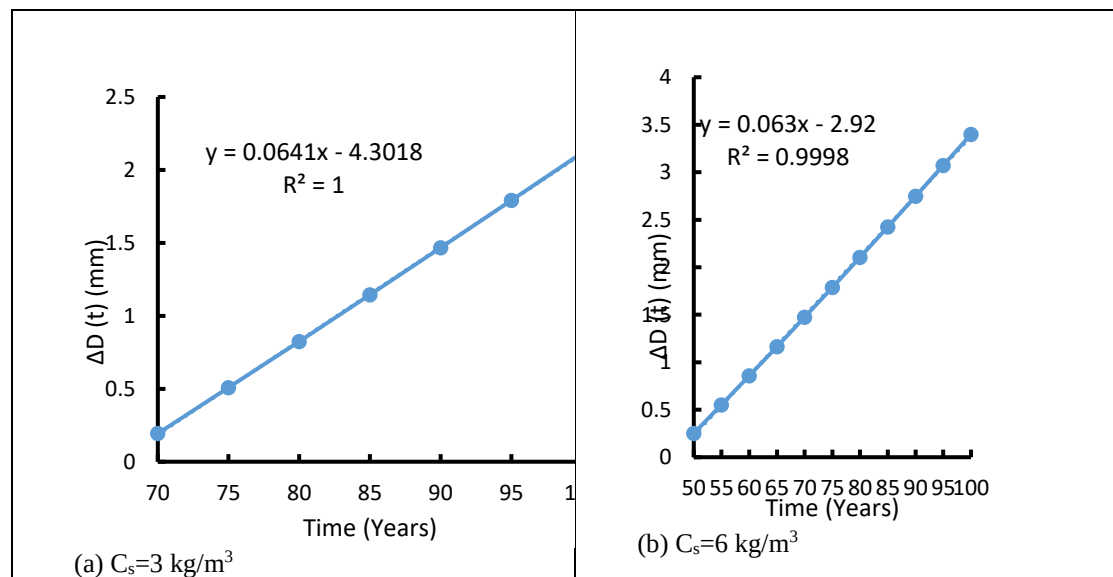


Fig.8. Projection of corrosion depths at different chloride concentrations after the time when the corrosive cracks appear.

Fig.9 shows that the relationship between the projected decrease in the corroded steel rebar diameter of 25 mm and the time after the corrosive crack appears is a linear function at different chloride concentrations. The decrease in the steel reinforcement rebar is calculated using equation (10) (assuming a uniform corrosion state). Moreover, the slope between the decrease in the rebar diameter versus the time after corrosive cracks appear is almost the same at different chloride concentrations, as shown in Fig.9.



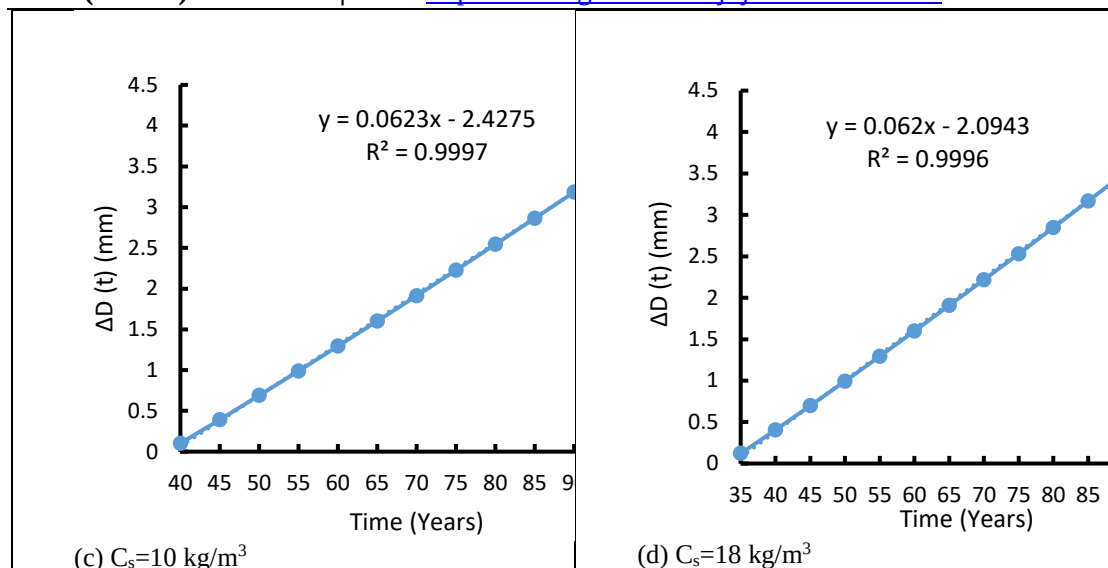


Fig. 9. Projection of the decrease in steel rebar diameter at different chloride concentrations after the onset of corrosive cracking.

It was deduced from Fig.8 and Fig.9 that the relationship between the projection of the corrosion depth and the decrease in the rebar diameter, respectively, over time (time after corrosive cracks appear) ($T_{cr} < T$), is a linear function according to the (R^2) values that are obtained at different chloride concentrations, as shown in Table 5. Moreover, Table 5 shows a prediction equation for estimating the corrosion depth and the decrease in the diameter of corroded steel reinforcement rebar at the time after the corrosive crack appears at different chloride concentrations.

Table 5. Linear equations for predicting corrosion depths and the decrease in the diameter of corroded steel rebar at different chloride concentrations for an RC bridge deck, between the time of corrosion initiation and the time when the corrosive crack appears.

Chloride Concentrations (Kg/m ³)	The time when Corrosive Cracks appear. T_{cr} (Years)	Equations for Prediction of the Corrosion Depths (Y) (mm)	Equations for Prediction of the decrease in the Rebar Diameter ($\Delta D(t)$) (mm) ($\Phi 25M$) (mm)
$C_s = 3 \text{ kg/m}^3$	66.9	$Y = 0.0321(t) - 2.1516$ ($R^2 = 1.0$)	$\Delta D(t) = 0.0641(t) - 4.3018$ ($R^2 = 1.0$)
$C_s = 6 \text{ kg/m}^3$	45.8	$Y = 0.0315(t) - 1.4605$ ($R^2 = 0.9998$)	$\Delta D(t) = 0.063(t) - 2.92$ ($R^2 = 0.9998$)
$C_s = 10 \text{ kg/m}^3$	38.2	$Y = 0.0312(t) - 1.2142$ ($R^2 = 0.9997$)	$\Delta D(t) = 0.0623(t) - 2.4275$ ($R^2 = 0.9997$)
$C_s = 18 \text{ kg/m}^3$	32.9	$Y = 0.031(t) - 1.0475$ ($R^2 = 0.9996$)	$\Delta D(t) = 0.062(t) - 2.0943$ ($R^2 = 0.9996$)

3.3 Probability of corrosion-induced damage versus the time between corrosion initiation and the time when a corrosive crack appears at different chloride concentrations.

According to the limit state function as mentioned in equation (11) for the corrosive cracking failure state between the time of corrosion initiation and the time when the corrosive cracks appear, the probability of the propagation stage due to chloride occurs when the corrosion depth exceeds the critical corrosion depth as mentioned in equation (12). The probability of corrosion propagation was calculated using MCS consisting of 100,000 samples generated for each random variable defined in equation (12). Fig.10 shows the probability of corrosion propagation for an RC deck of 70 mm concrete cover at different chloride concentrations. Furthermore, it was deduced that the probability of corrosion propagation increased sharply from the time of corrosion initiation till the time when the corrosive cracks appeared as follows: (43.1 to 66.9 years), (24.9 to 45.8 years), (18.6 to 38.2 years), and (14.3 to 32.9 year) for the following chloride concentrations (3, 6, 10, and 18 kg/m³) respectively as shown in Fig.10.

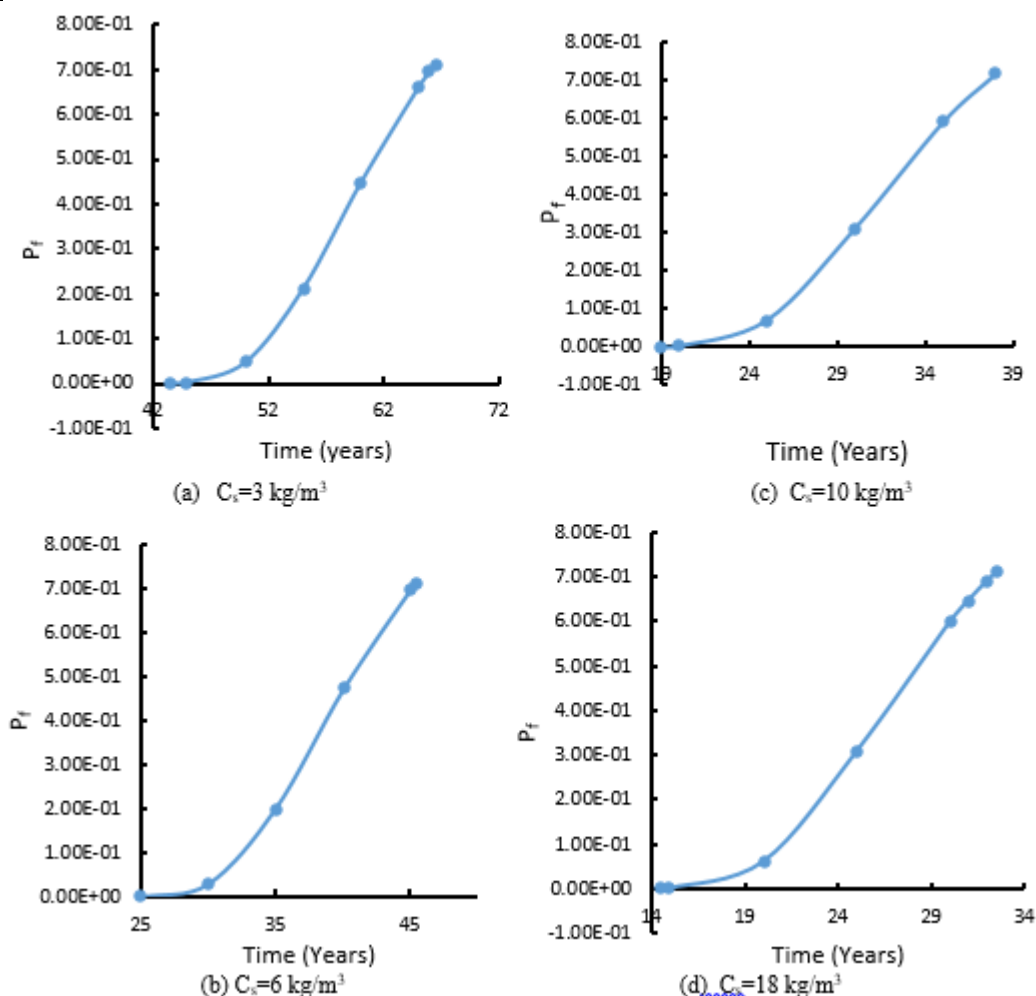


Fig.10. Probability of corrosion propagation for the steel rebars between the time of corrosion initiation (T_i) and the time when the corrosive cracks (T_{cr}) appear at different chloride concentrations.

3.4 Probability of corrosion-induced damage (acceptable level of damage) versus the time after the corrosive crack appears at different chloride concentrations.

The probabilities of corrosion-induced damage reaching the maximum acceptable degree and their corresponding reliability indexes were calculated using MCS of 100,000 samples after the time the corrosive cracks appeared ($t > T_{cr}$) at different chloride concentrations for an existing RC bridge having a concrete cover of 70 mm, as shown in Fig.11. The probability of corrosion damage reaching the maximum acceptable degree was deduced using equation (14). It was noticed that the probabilities of corrosion-induced damage increased sharply starting from year of the appearance of the corrosive cracks till a certain time as follows: (year 70 to year 80), (year 50 to year 60), (year 40 to year 50), and (year 35 to year 50) for chloride concentration of 3, 6, 10, and 18 kg/m^3 , respectively as illustrated in Fig.11. Finally, it was concluded that the probabilities of corrosion-induced damage are almost constant (approximately equal to 1) starting from the following years 85, 70, 60, 60, for the following chloride concentrations (3, 6, 10, and 18 kg/m^3), respectively as shown in Fig.11.

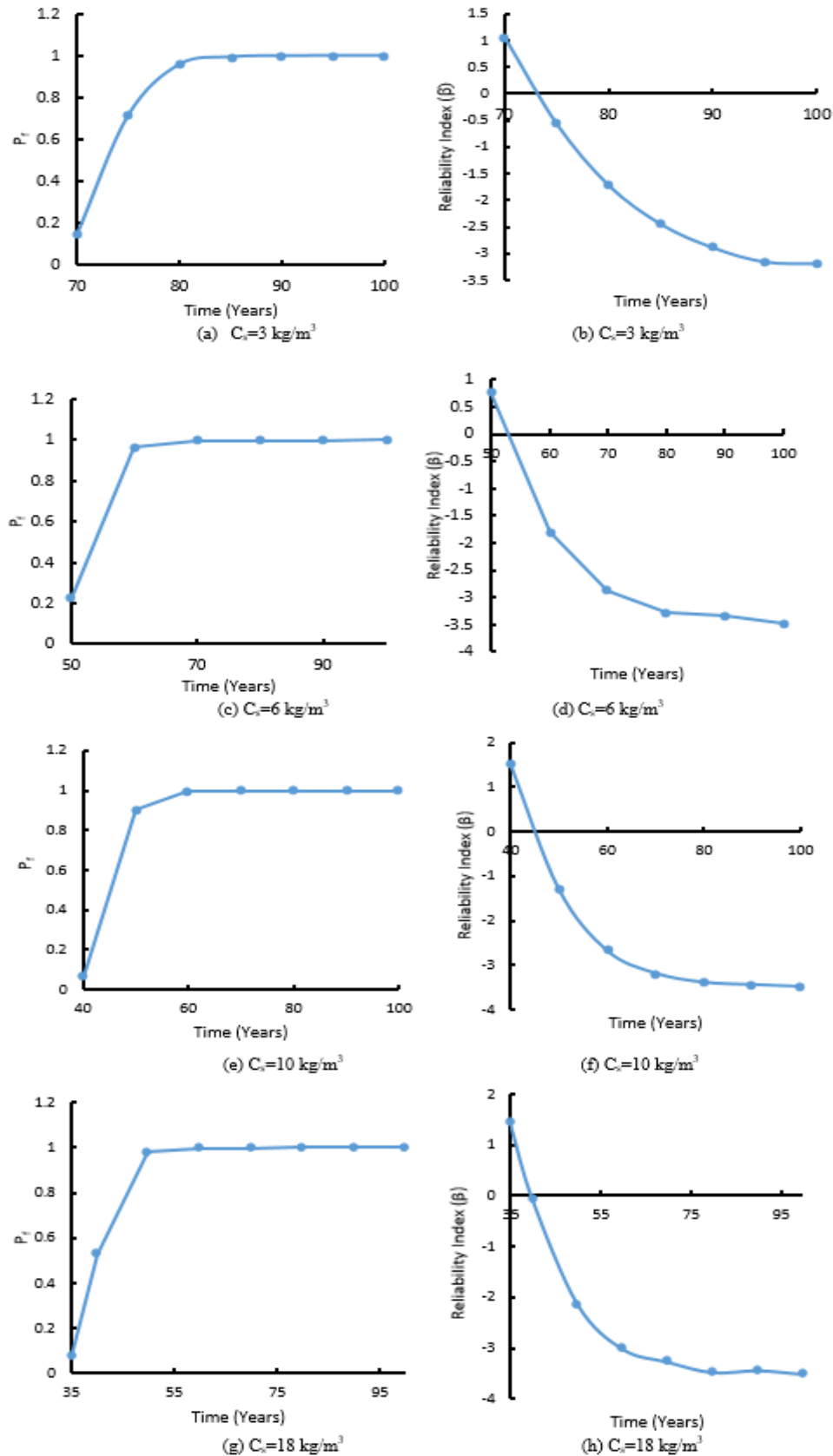


Fig.11. Probabilities of corrosion-induced damage (maximum acceptable level of damage) and reliability indexes at the time after the corrosive cracks appear ($t > T_{cr}$) at different chloride concentrations.

3.5. Investigation of the residual flexural capacity for an existing RC bridge deck after the corrosion initiation stage

The residual flexural steel reinforcement cross-sectional area for the main rebars used for an existing RC bridge deck, due to the application of a chloride concentration of 6 kg/m^3 on the upper RC bridge deck, was calculated using equation (18) as a function of time after corrosion initiation according to Frangopol et al. [30], assuming a uniform corrosion state. The clear cover for the main rebars is assumed to be equal to 70 mm for an existing RC bridge deck. Fig.12 shows the profile for the residual total cross-sectional area for the steel rebars used in the flexural resistance for the RC deck at various corrosion rate scenarios, including (50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$), versus the reduction in the rebar diameter used for the RC bridge deck. Furthermore, the reduction in the bar diameter due to corrosion is investigated by subtracting the damaged diameter due to the corrosion rate from the intact diameter of the rebar. It was deduced that the residual cross-sectional area for the main steel bars used for the RC bridge deck decreased sharply from (6872.2 mm^2 to 3361.7 mm^2) and from (6872.2 mm^2 to 3177.6 mm^2) when the reduction in the bar diameter changed from 0 mm to 8 mm, at corrosion rates of 50 and 100 $\mu\text{m}/\text{year}$, respectively, as shown in Fig.12.

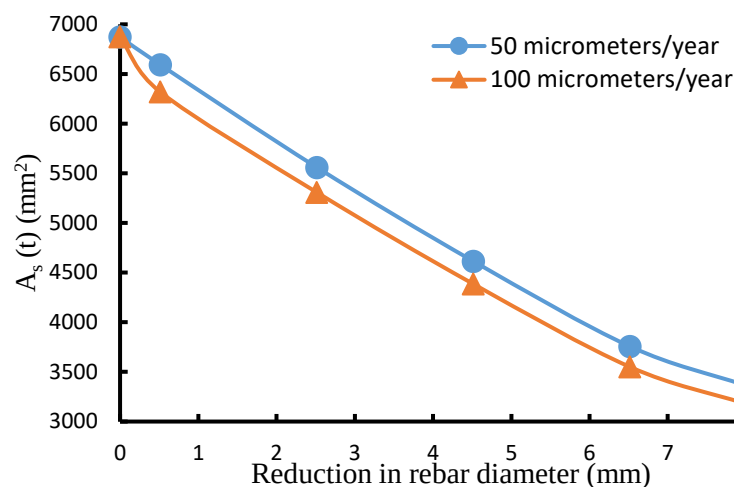


Fig.12. Total residual steel reinforcement cross-sectional area for the main steel rebars used for the RC bridge deck versus various losses of the steel rebar diameter.

The residual flexural strength capacity of the RC deck ($M_r(t)$) at the time (t) is computed using equation (20). Moreover, the residual flexural capacity of the RC deck at zero millimeters reduction in the rebar diameter is 364.52 kN.m for both 50 and 100 $\mu\text{m}/\text{year}$ as shown in Fig.13. It was observed that the residual flexural capacity for the RC bridge deck decreased sharply from (364.52 kN.m to 218.47 kN.m) and from (364.52 kN.m to 208.03 kN.m), when the reduction of the steel rebar diameter changed from 0 mm to 8 mm, at corrosion rates of 50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$, respectively, as shown in Fig.13.

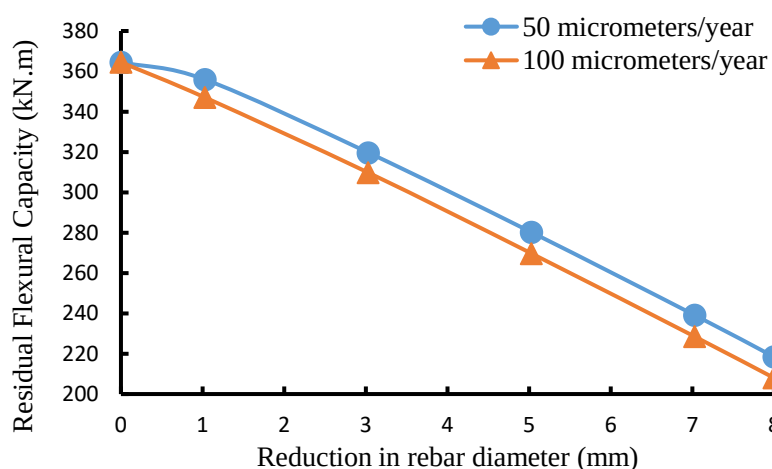


Fig.13. Residual flexural capacity resisted by the steel rebars for the RC deck of the bridge having a concrete cover of 70 mm.

The percentage reduction in the residual flexural capacity at different reductions of the steel rebar diameter compared to the intact rebar diameter used for the RC bridge deck was investigated. It was deduced that when the reduction of the steel rebar diameter changed from 1 mm to 8 mm, the percentage reduction in the residual flexural capacity increased sharply from 2.4% to 40.1%, and from 4.8% to 42.9%, at corrosion rates of 50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$, respectively, as shown in Fig.14. A linear relationship between the reduction in the rebar diameter and the percentage reduction in flexural capacity across various corrosion rates as shown in Fig.14.

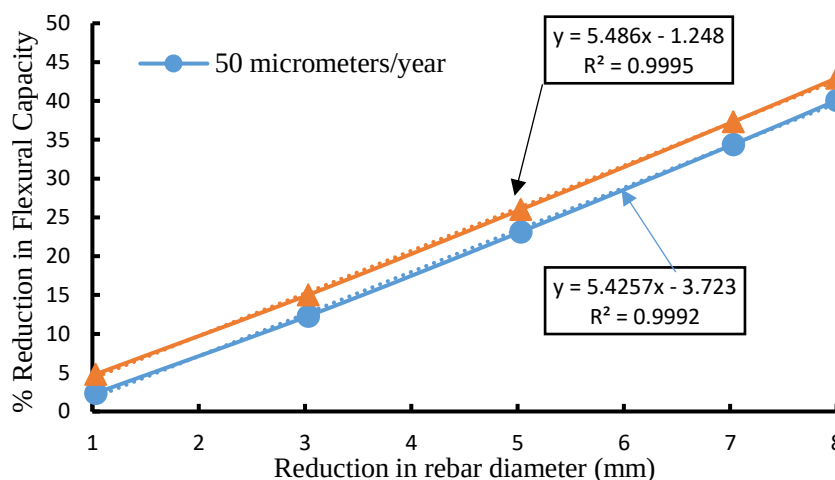


Fig.14. The percentage reduction in flexural capacity resisted by the main steel rebars used in the RC bridge deck due to various corrosion rate scenarios.

In the case of the RC bridge T girder, the following material properties are adapted from Frangopol et al. [30]. The compressive strength of the concrete used for the RC T-girder of the existing bridge is assumed equal to 35 MPa, and the yield strength for the carbon steel reinforcement used for the RC bridge is equal to 413.4 MPa according to Frangopol et al. [30]. Moreover, the clear concrete cover for the RC T girder is assumed to be 50 mm and the diameter of the steel rebar used in the RC T girder is assumed to be equal to 30 mm as shown in Fig.15. The RC T-girder is reinforced with 12-30 M as shown in Fig.15. In addition, the RC T girder is subjected to chloride concentration of 6 kg/m^3 .

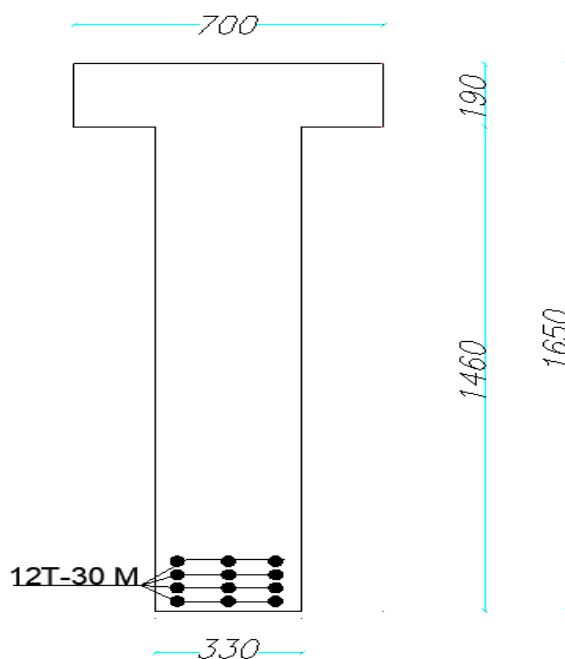


Fig.15. Cross-sectional dimension and steel reinforcement for an existing RC bridge T-girder.

Fig.16 shows the residual total cross-sectional area for the steel rebars used in the flexural resistance for the RC T-girder after the corrosion initiation versus the reduction of the rebar diameter used for the RC bridge deck at various corrosion rate scenarios, including (50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$). Furthermore, the reduction of the rebar diameter due to corrosion is investigated by subtracting the damaged diameter due to the corrosion rate from the intact diameter of the rebar used, which is 30 mm. In addition, it was deduced that the residual cross-sectional area for the main steel rebars used for the RC T-girder decreased sharply from (8482.3 mm^2 to 4263 mm^2) and from (8482.3 mm^2 to 3974.6 mm^2), when the reduction of the steel rebar diameter changed from 0 mm to 9 mm, at corrosion rates of 50 and 100 $\mu\text{m}/\text{year}$, respectively, as shown in Fig.16.

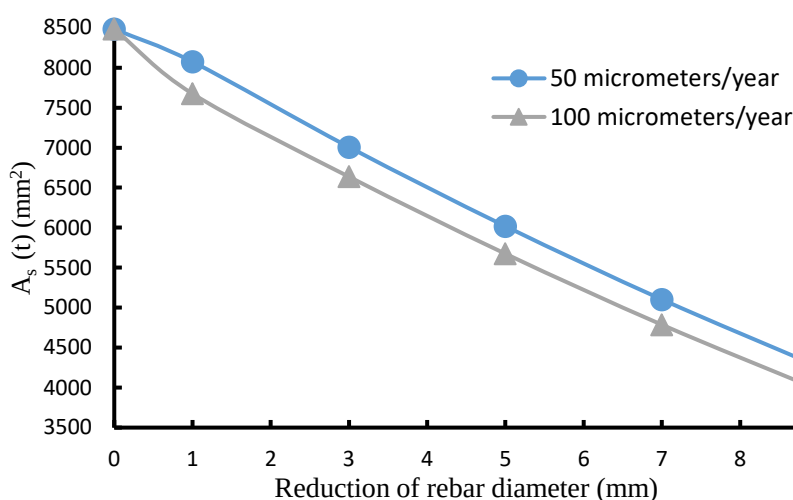


Fig.16. Total residual steel reinforcement cross-sectional area for the main steel rebars used for the RC bridge T-girder versus various reductions of the steel rebar diameter.

The residual flexural capacity of the existing RC T-girder of the bridge is analyzed theoretically due to various corrosion rate scenarios including (50 $\mu\text{m}/\text{yr}$ and 100 $\mu\text{m}/\text{yr}$) applied on the tension steel reinforcement rebars versus different reductions of the steel reinforcement diameter as shown in Fig.17. The residual flexural capacity for RC T girder is decreased sharply versus the increase in the loss of the steel rebar diameter as shown in Fig.17. Furthermore, it was observed that the residual flexural capacity is decreased significantly from 3948.1 kN.m to 2109.2 kN.m and from 3948.1 kN.m to 1972.1 kN.m, when the reduction of the steel rebar diameter changed from 0 mm 9 mm, for corrosion rates of 50 $\mu\text{m}/\text{yr}$ and 100 $\mu\text{m}/\text{yr}$, respectively as shown in Fig.17.

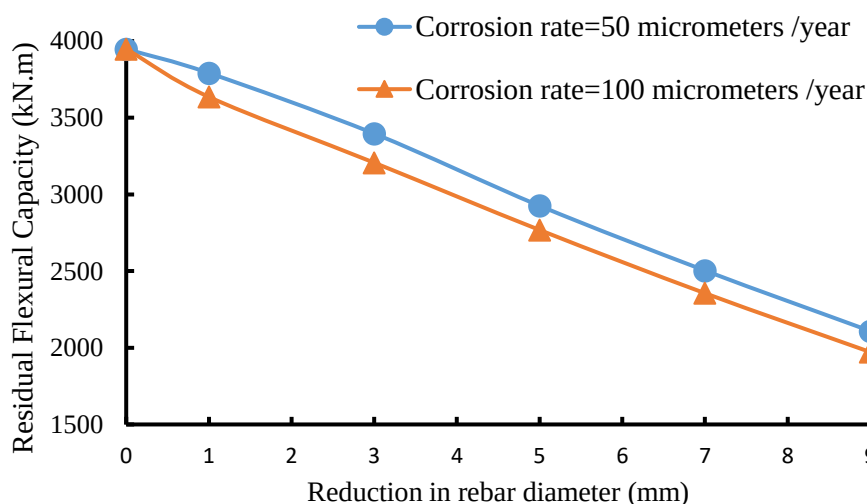


Fig.17. Residual flexural capacity resisted by the RC bridge T-Girder member at different corrosion rates.

The percentage reduction in the residual flexural capacity for the RC T girder of the bridge increased nonlinearly from 3.96% to 46.6% and from 7.9% to 50.1%, when the reduction of the steel rebar diameter

changed from 1 mm to 9 mm, at corrosion rates of 50 $\mu\text{m}/\text{yr}$ and 100 $\mu\text{m}/\text{yr}$, respectively, as shown in Fig.18. Moreover, Fig.18 shows a linear relationship between the reduction in the rebar diameter and the percentage reduction in flexural capacity across various corrosion rates.

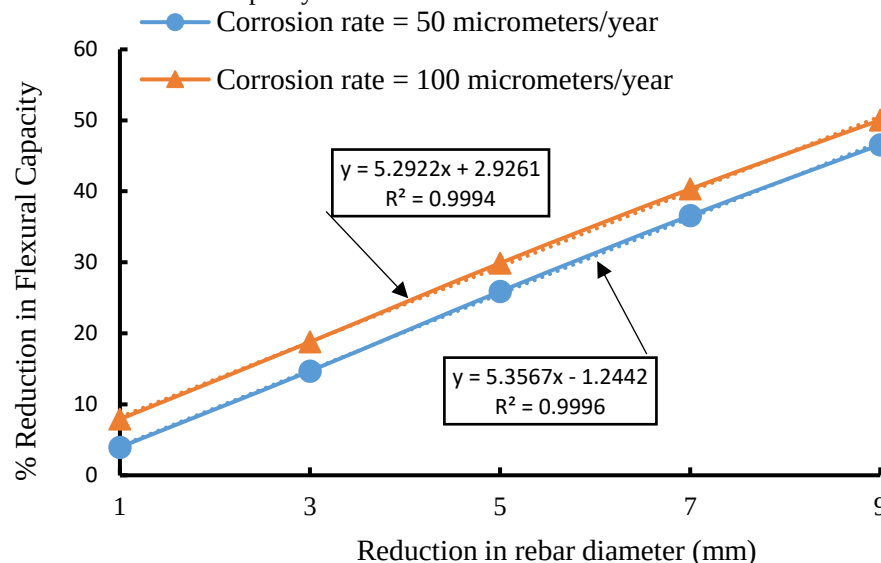


Fig.18. Percentage reduction in residual flexural capacity resisted by the RC bridge T-Girder versus various loss of steel rebar due to various corrosion rates.

4. CONCLUSIONS

- Second-degree polynomial functions were deduced for predicting whether the time of corrosion initiation or the time when the corrosive cracks appear, as a function of the concrete cover ranges for RC members, across different chloride concentrations applied on the top part of the RC members made of normal concrete.

- The relationship between the projected corrosion depth and the corresponding decrease in the rebar diameter, respectively, over various times ($T_0 < T < T_{cr}$) and ($T > T_{cr}$) is a linear relationship based on root mean square error, which corresponds to a value of 1 at different chloride concentrations (3, 6, 10, and 18 kg/m^3).

- A prediction equation was deduced to estimate the corrosion depth and the decrease in the corroded rebar diameter as a function of time between the corrosion initiation and the time when the corrosive crack appears, and the time after the appearance of the corrosive crack across various chloride concentrations applied on the top part of the RC deck.

The probability of corrosion propagation versus the time between the time of corrosion initiation and the time when the corrosive cracks appeared was conducted using an MCS method, with 100,000 simulations for an existing RC deck with 70 mm concrete cover subjected to different chloride concentrations. The following results were obtained:

- The probability of corrosion propagation increased sharply from (2.5×10^{-4} to 0.71), (8×10^{-5} to 0.72), (2.5×10^{-4} to 0.71), and (5.8×10^{-4} to 0.71) at the time of corrosion initiation till the time when the corrosive cracks appeared, as follows (43.1 to 66.9 years), (24.9 to 45.8 years), (18.6 to 38.2 years), and (14.3 to 32.9 year) across the chloride concentrations of 3, 6, 10, and 18 kg/m^3 , respectively which is applied on the top part of the RC decks. However, the probabilities of corrosion-induced damage after the time when the corrosive cracks appeared ($t > T_{cr}$) across different chloride concentrations applied on an existing RC deck are almost constant (approximately equal to 1) starting from the 85, 70, 60, and 60, years when the following chloride concentrations were applied on the upper part of the RC decks 3, 6, 10, and 18 kg/m^3 , respectively.

The residual flexural capacity for an existing RC deck, having a clear concrete cover of 70 mm, and the diameter of the main steel rebar used is equal to 25 mm. The following results were deduced:

- The residual flexural capacity for the RC deck decreased sharply from 364.52 kN.m to 218.47 KN.m and from 364.52 KN.m to 208.03 KN.m, when the reduction of the steel rebar diameter varied from 0 mm to 8 mm, at corrosion rates of 50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$, respectively.

- The percentage reduction in the residual flexural capacity of the steel rebar diameter for the RC deck exposed to corrosion compared to the intact rebar diameter was investigated. It was found that when the steel rebar diameter was reduced from 1 mm to 8 mm, the percentage reduction in residual flexural

capacity increased sharply from 2.4% to 40.1% and from 4.8% to 42.9% at corrosion rates of 50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$ applied on the top part of the RC deck, respectively.

The residual flexural capacity for an existing RC T-Girder, with a clear concrete cover of 50 mm for the tension steel rebars, and the diameter of the main steel rebar utilized is equal to 30 mm. The main points are deduced as follows:

- The residual flexural capacity for the RC bridge T-Girder decreased sharply from 3948.1 KN.m to 2109.2 KN.m and from 3948.1 KN.m to 1972.1 KN.m, when the reduction of the steel rebar diameter changed from 0 mm to 9 mm, at corrosion rates of 50 $\mu\text{m}/\text{year}$ and 100 $\mu\text{m}/\text{year}$, respectively.
- The percentage reduction in the residual flexural capacity for the RC bridge T girder increased sharply from 3.96% to 46.6% and from 7.9% to 50.1%, when the reduction of the steel rebar diameter changed from 1 mm to 9 mm, at corrosion rates of 50 $\mu\text{m}/\text{yr}$ and 100 $\mu\text{m}/\text{yr}$, respectively.

Limitations: The concrete cover used for an existing RC deck is 70 mm. In addition, the steel rebars utilized in the RC deck and RC girder are 25 mm and 30 mm, respectively. The concrete cover used in the RC deck varies from 30 mm to 70 mm. Moreover, the chloride diffusion coefficient used is equal to $1.27 \times 10^{-12} \text{ m}^2/\text{s}$ in the probabilistic corrosion model. Furthermore, the water-to-cement ratio is set at 0.4, and the cement content used in the concrete mix is 460 kg/m^3 . The corrosion rates for steel rebars are assumed to be 50 $\mu\text{m}/\text{yr}$ and 100 $\mu\text{m}/\text{yr}$. Different chloride concentrations were applied to the RC members, including 3 kg/m^3 , 6 kg/m^3 , 10 kg/m^3 , and 18 kg/m^3 . The compressive strength of concrete is assumed to be 35 MPa for different RC members.

For future studies, it is recommended to utilize the effect of severe climate change, supplementary cementitious materials, and eco-friendly materials, such as geopolymers concrete, etc., on the probability of corrosion propagation across different times for existing and future vital RC members.

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