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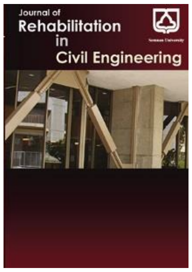

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Rehabilitation of Corroded Reinforced Concrete Elements by Rebar Replacement

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ABSTRACT

In this study, the replacement of corroded reinforcement with new reinforcement as a rehabilitation method is considered to reduce the impact of corrosion on the performance of reinforced concrete structural elements. Also, the effect of using high-performance concrete with the method of reducing the water-to-cement ratio, as a method for maintenance of reinforced concrete structures, has been analyzed. So, the influence of the above rehabilitation methods for maintenance of reinforced concrete structures on the corrosion initiation time of reinforcement, crack initiation time and crack width of the concrete cover thickness, the service life of a reinforced concrete structure due to corrosion, and corrosion percentage of reinforcement have been investigated. For this purpose, all equations and connection between them for the corrosion phenomenon modeling (including corrosion initiation phase, corrosion propagation phase and cracking) is integrated, and the corrosion parameters are calculated and compared for the marine environmental conditions. The results indicated that, the end time of service life of a reinforced concrete structure due to corrosion (t_f) increases 60.54% by applying the new reinforcement as a rehabilitation method. So, in concrete with a water-to-cement ratio of 0.35, the corrosion percentage of reinforcement in the new-reinforcement scenario has decreased by 15.60% compared to the no-repair scenario over 30 years.

1. Introduction

Contamination In recent years, early destruction of reinforced concrete (RC) structures and reduction of their service life due to the corrosion phenomenon and penetration of invasive fluids into concrete has been one of the most critical concerns of the civil engineering community in the field of concrete structures [1, 2]. High temperature and humidity are the main

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features of a corrosive environment. Considering that marine environments, especially in the Persian Gulf region, have both of the above components [3-6]. Therefore, the study of concrete status in such environments deserves special attention [7, 8].

Corrosion occurs in two phases: the initiation phase and the propagation phase [3, 9].

During the initiation phase, chloride ions penetrate the concrete cover and eventually reach the first layer of steel rebar. Chloride ions accumulate overtime on the surface of the steel rebar until the concentration of chloride ions on the surface of the rebar reaches a critical value. At this point, the chloride ions break the passive layer at the boundary between the surface of the steel rebar and the concrete, and the corrosion process begins. Oxygen and moisture are essential for accelerating the corrosion process. Chloride ions are not used in the corrosion process and intensify the corrosion process by removing the passive layer [10]. The initiation time of corrosion is a function of several parameters, including the concrete cover thickness, the surface chloride concentration, the quality of concrete and the type of steel rebar (steel rebar with epoxy coating, galvanized rebar, etc.) [11, 12].

When the passive layer around the rebar is broken, corrosion products or iron rust will be produced on the rebar surface. Iron rust production is caused by the dissolution of steel rebar in the water of cavities in the concrete.

The propagation phase starts when the iron rust around the rebar begins to increase and spread. Therefore, the stresses caused by the expansion of corrosion products lead to tensile stress in the concrete cover and lead to fractures (cracking, delamination and spalling) in the concrete cover. This phenomenon leads to a reduction in the final strength, a reduction in the connection between the concrete and the rebar, and ultimately the cross-sectional degradation [13].

The cracking of concrete cover due to corrosion is a known problem in concrete structures. In recent decades, many models have been proposed to predict the cracking of concrete cover due to corrosion.

The first model was introduced by Bezant in 1975 [14]. This model is based on the thick-walled cylindrical model to calculate the internal pressure created by corrosion. He set the time for the concrete cover to break when the first crack appeared in the inner radius. Some researchers used the finite element method to model cracking of concrete cover [15, 16] and others predicted the onset of the cracking and its spread in reinforced concrete tests using analytical relationships obtained from laboratory results [17-20]. Others used the theory of fracture mechanics to predict the time of cracking and the spread of cracks in concrete cover [21, 22]. In this study, he obtained the thick wall theory proposed by Liu and Weirs in 1998 [17] to predict the time of onset of cracking and its expansion in the concrete cover due to the accumulation of corrosion products.

When the corrosion process begins, the corrosion products first fill the porous area and cause the concrete around the rebar to expand. Since concrete is a weak material in tensile strength, with increasing corrosion, tensile stresses quickly reach the maximum tensile capacity of concrete and, as a result, cause cracking of the concrete cover [23].

In this study, to model the corrosion process of a RC element, equations tabulated in Table 1, are used in the calculations.

Table 1. Equations applied to model corrosion process of a RC element.

Equation Number	Equation	Definition
(1)	$C(x,t) = C_s - (C_s - C_i)erf(\frac{x}{2\sqrt{Dt}})$	$C(x,t)$ (% by weight of concrete): chloride concentration in depth x and time t [24]. D (in m^2/s): chloride diffusion coefficient. C_s (% weight of concrete): surface chloride concentration. C_i (in % weight of concrete): initial chloride. x (in m): distance between rebar surface and concrete surface. t (in s): exposure time. erf : error function.
(2)	$T_{corr} = \left(\frac{x^2}{4D}\right) \times \left(erfinv\left(\frac{C_s - C_{cr}}{C_s - C_i}\right)\right)^{-2}$	T_{corr} (in s): corrosion initiation time of longitudinal or horizontal reinforcement [3]. C_{cr} (in % weight of concrete): critical chloride concentration [3].
(3)	$D = 10^{(-12)} \times (156.8\,w/c - 43.482) \times \left(\frac{t_{ref}}{t}\right)^{(1.0157\,w/c + 0.2571)}$ $\times \exp\left[\frac{U}{R}(\frac{1}{T_{ref}} - \frac{1}{T})\right]$	D (in m^2/s): chloride diffusion coefficient [3]. t (in months): current time. t_{ref}: 3 months. n: age factor. w/c: water-to-cement ratio. T (in K): current temperature. T_{ref}: 306.5 K. U/R: 2948 K [3].
(4)	$C_s = \mathrm{K}\,t^{\mathrm{d}} + \mathrm{C}_0$	C_s (in % weight of concrete): surface chloride concentration [18]. t (in day): exposure time. d, K and C_0 : constants [25].
(5)	$t_{cr} = \frac{W_{crit}{}^2}{2k_p}$	t_{cr} (in year): time of crack initiation [26]. W_{crit}: critical rust total amount. k_p: production rate of corrosion products [27, 28].
(6)	$k_p = 9.8 \times 10^{-5} \frac{\pi d_0 \times \left\{0.85 \times \frac{37.8(1 - w/c)^{-1.64}}{x} \times (t - T_{corr})^{-0.29}\right\}}{\beta}$	β: steel molecular weight-to-corrosion products molecular weight-ratio. d_0 : initial diameter of the bar.
(7)	$w = K\left(\Delta A_s - \Delta A_{s0}\right)$	w (in mm): crack width of concrete cover due to corrosion [29]. ΔA_s (in mm^2): reduced cross-section of steel reinforcement. K : 0.0575. * To calculate $(\Delta A_s - \Delta A_{s0})$, for simplification, it is assumed that after cracking, the amount of corrosion rate does not change with time.
(8)	$\psi = 100\left(1 - \left(\frac{dt}{d_0}\right)^2\right)$	ψ (in %): corrosion percentage of rebar due to corrosion [30]. dt (in mm): reduced reinforcement diameter due to corrosion. d_0 (in mm): initial uncorroded reinforcement diameter.

2. Methodology

In this study, chloride corrosion models including the corrosion initiation phase, corrosion propagation phase and cracking of concrete cover suitable for the environmental characteristics of the Qeshm Island presented by the author and other researchers have been compiled. Using these models together, the corrosion percentages of rebars in different repair and rehabilitation scenarios are calculated programmatically in MATLAB software at different times. These are as inputs for modeling the nonlinear finite element reinforced concrete columns under the lateral and axial loading and corrosion phenomenon in OpenSees software to investigate the time-dependent performance of column.

In this research, the replacement of corroded reinforcement with new reinforcement as a rehabilitation method is considered to reduce the impact of corrosion on the performance of reinforced concrete structural elements. In this method, at crack initiation time (t_{cr}), the chloride ion-impregnated concrete cover thickness is removed, and the corroded rebar is removed. Then, the new uncorroded reinforcement, with the same properties as the initial uncorroded reinforcement, is replaced with the corroded reinforcement. After that, the element for concreting the new concrete cover thickness, with the same properties as the initial concrete cover, is molded.

So, another scenario is intended to compare and investigate the impact of the new-reinforcement scenario on the amount of corrosion of structural elements. In the no-repair scenario, no repairs were performed at the specified time.

3. Results and Discussion

3.1. Calculation of Corrosion Initiation Time of Reinforcement

The corrosion initiation time is calculated by Eq. (2). Table 3 indicates the values of the corrosion initiation time for concrete with different *w/c* for concrete elements located in the tidal zone, Qeshm Island, for the new-reinforcement scenario, before (t_{corr1}) and after applying repair (t_{corr2}), and for the no-repair scenario.

As shown in Table 3, the values of the corrosion initiation time in the no-repaired scenario decreases by 55% by increasing the *w/c*. Because, the pores in the concrete are more and the chloride ion reaches the rebar surface more quickly, and as a result, the time of corrosion initiation of rebar is reduced.

In the new-reinforcement scenario, the time of corrosion initiation of rebar before applying repair is the same after applying repair. Because, the new concrete cover thickness and new rebar have the properties of initial concrete cover thickness and initial rebar.

3.2. Calculation of Crack Initiation Time of Concrete Cover Thickness

The crack initiation time of the initial concrete cover thickness (t_{cr1}) before the repair, and the crack initiation time of the secondary concrete cover thickness (t_{cr2}) after the repair is given in Table 3.

It should be noted that the time required to initiate cracking, Eq.(5), in Table 1, is the time after the corrosion initiation time, and this time is not calculated from the start of the chloride permeation.

The crack initiation time of the concrete cover thickness for the no-repair scenario is indicated in Table 2.

Table 2. Crack initiation time of concrete cover.

w/c	0.35	0.40	0.45	0.50
t_{cr} (year)	2.54	2.36	2.10	1.82

As shown in Table 3, the value of the crack initiation time in the no-repaired scenario decreases by 28% by increasing *w/c*. As the ratio of water-to-cement increases, chloride ions are released more rapidly into the concrete. As a result, corrosion products are formed more rapidly, and the expansion of the corrosion products accelerates the crack initiation time of the concrete cover thickness.

In the new-reinforcement scenario, the crack initiation time before applying repair is the same after applying repair. Because, the new concrete cover thickness and new rebar have the properties of initial concrete cover thickness and initial rebar.

3.3. Calculation of Crack Width of Concrete Cover Thickness

Farahani et al. [31] found in their research that the end time of service life of a reinforced concrete structure due to corrosion (t_f) is when the crack width in the concrete cover, w_{lim}, reaches 0.50 mm.

In this research, the time required to the crack width in the concrete cover to reach 0.50 mm is defined as $t_{cr0.50}$.

Corrosion degradation times are also shown in Table 3 for the no-repair and the new-reinforcement scenarios and concrete with different *w/c*.

It should be noted that the calculated destruction time (t_f) from the start time of exposure or zero time, takes into account the sum of t_{corr1}, t_{cr1}, t_{corr2}, t_{cr2} and $t_{cr0.50}$.

Fig. 1 and Fig. 2 indicate the crack width of cover from the start time of exposure or zero time, for concrete with *w/c* 0.35, 0.40, 0.45 and 0.50 for the no-repair and the new-reinforcement scenarios. As can be seen, an increasing water-to-cement ratio of concrete mixture design, the crack width of concrete cover increases.

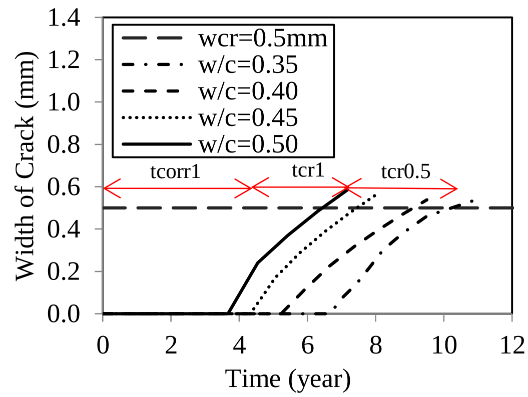

Fig. 1. Crack width of concrete cover in no-repair scenario.

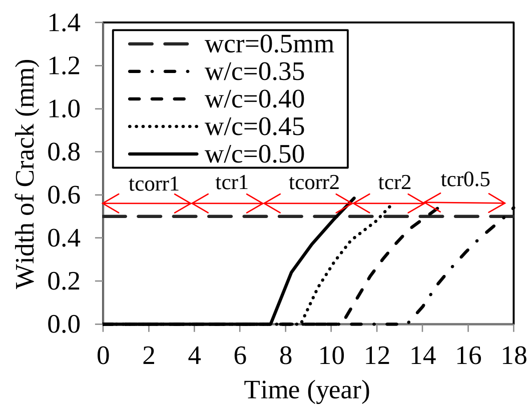

Fig. 2. Crack width of concrete cover in new-reinforcement scenario.

Table 3. Corrosion initiation time and crack initiation time, after and before applying repair.

Repair	*w/c*	t_{corr1} (year)	t_{cr1} (year)	t_{corr2} (year)	t_{cr2} (year)	$t_{cr0.50}$ (year)	t_f (year)
no-repair	0.35	4.12	2.54	0.00	0.00	4.34	11.00
	0.40	2.78	2.36	0.00	0.00	3.80	9.03
	0.45	2.23	2.10	0.00	0.00	3.30	7.63
	0.50	1.85	1.82	0.00	0.00	2.82	6.49
new-reinforcement	0.35	4.12	2.54	4.12	2.54	4.34	17.66
	0.40	2.78	2.36	2.78	2.36	3.80	14.26
	0.45	2.23	2.10	2.23	2.10	3.30	11.96
	0.50	1.85	1.82	1.85	1.82	2.82	10.16

According to Fig. 1 and Fig. 2, for example in concrete with water-to-cement ratio of 0.35, the time that the crack width of the concrete cover reaches 0.50 mm or t_f, in the no-repair and the new-reinforcement scenarios are 11.00 and 17.66 years, respectively.

Also, as shown in Fig. 2, the time that the concrete cover crack width reaches 0.50 mm or t_f, in the no-repair and the new-reinforcement scenarios, in concrete with a water-to-cement ratio of 0.50 compared to concrete with a water-to-cement ratio of 0.35, is reduced 41.0% and 42.5%, respectively.

As can be seen, when the *w/c* increases, the value of crack width of the concrete cover increases.

3.4. Calculation of Corrosion Percentage of Reinforcement

Fig. 3 and Fig. 4 indicate the corrosion percentage of reinforcement after corrosion initiation time before repair (t_{corr1}) for concrete with 0.35, 0.40, 0.45 and 0.50 *w/c* for two scenarios; the no-repair and the new-reinforcement. As can be seen, when the *w/c* increases, the percentage of corrosion of reinforcement increases.

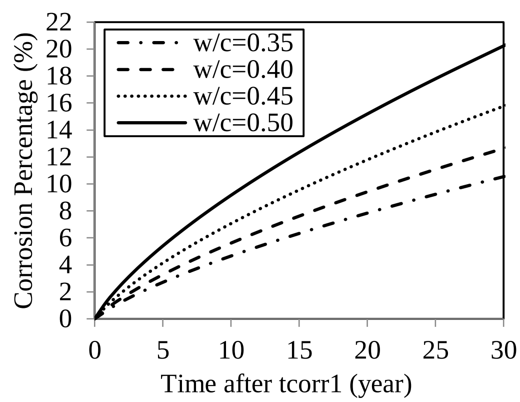

Fig. 3. Corrosion percentage of reinforcement in no-repair scenario.

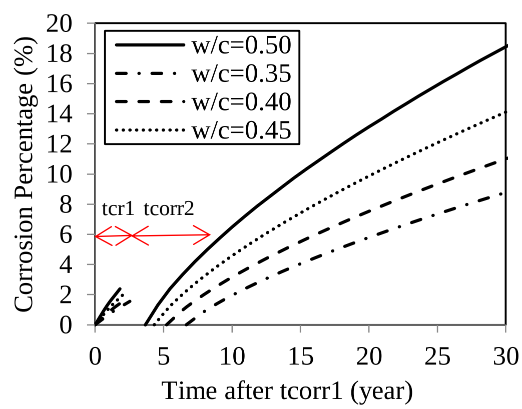

Fig. 4. Corrosion percentage of reinforcement in new-reinforcement scenario.

According to Fig. 4, for example, in concrete with *w/c* 0.35, the corrosion percentage of reinforcement in the new-reinforcement scenario has decreased by

15.60% compared to the no-repair scenario over 30 years.

Fig. 4 shows that the corrosion percentage of reinforcement increases with time after the time of corrosion initiation before repair (t_{corr1}) to crack initiation time of the initial concrete cover (t_{cr1}). Then, the repair scenario is applied, and the new reinforcement is replaced with the corroded reinforcement. Therefore, after t_{cr1}, the corrosion percentage of reinforcement is zero during t_{corr2}. It is worth noting that, as shown in Table 3 and Fig. 4, in the repair scenario of replacing the corroded rebar with new rebar, due to the replacement of the new rebar and the concrete cover similar to the initial amount of these, t_{corr2} is equal to t_{corr1}, and t_{cr2} is equal to t_{cr1}.

4. Conclusion

Most the reinforced concrete structures are damaged in corrosive environments during construction or in the early years of their service life due to the intensity of chloride corrosion. Therefore, the use of maintenance methods in the reinforced concrete structures located in corrosive and marine conditions to increase the durability and service life of these structures should be considered.

In this research, the replacement of corroded reinforcement with the new reinforcement and using high-performance concrete with the method of reducing the water-to-cement ratio, as the rehabilitation methods are considered to reduce the impact of corrosion on the performance of reinforced concrete structural elements.

So, the influence of these rehabilitation methods for maintenance of reinforced concrete structures on the corrosion initiation time of reinforcement, crack initiation time, crack width of the concrete cover thickness, service life of a reinforced concrete structure due to corrosion, and corrosion percentage of reinforcement have been investigated.

The results indicate that the time of corrosion initiation in the no-repair scenario decreases by 55% by increasing *w/c*. Because, when the *w/c* increases, the pores in the concrete are more and the chloride ion reaches the rebar surface more quickly. As a result, the corrosion initiation time of rebar is reduced.

In the new-reinforcement scenario, as a repair scenario, the corrosion initiation time before and after the repair is similar. This is the replacement of the initial concrete cover contaminated with chloride ion with a new concrete cover and new rebar with the corroded rebar with the same properties of the initial concrete cover and initial rebar, respectively.

The results show that in concrete with a water-to-cement ratio of 0.35, the time that the crack width of the concrete cover reaches 0.50 mm or t_f, in the no-repair and the new-reinforcement scenarios are 11.00 and 17.66 years, respectively.

Also, the time that the crack width of the concrete cover reaches 0.50 mm or t_f, in the no-repair and the new-reinforcement scenarios, in concrete with a water-to-cement ratio of 0.50 compared to concrete with a water-to-cement ratio of 0.35, is reduced 41.0% and 42.5%, respectively.

So, in concrete with a water-to-cement ratio of 0.35, the corrosion percentage of reinforcement in the new-reinforcement scenario has decreased by 15.60% compared to the no-repair scenario over 30 years.

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