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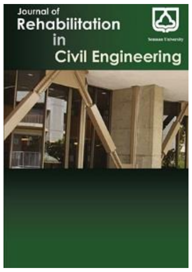

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Time-Dependent Structural Behavior of Repaired Corroded RC Columns Located in a Marine Site

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ABSTRACT

The chloride corrosion of reinforcing steel in reinforced concrete (RC) structures is a significant reason for premature deterioration and failure of RC structures in aggressive environments such as the Persian Gulf region. This is one of the major sources of engineering and economic problems in developed countries. On that account, modeling chloride permeation and investigating different methods for the repair and maintenance of RC structures exposed to corrosive environments are considered to be essential for optimizing the service life and life cycle cost of these structures. In this research, a finite element model is applied to assess the time-dependent capacity of corroded RC structures applying nonlinear analysis; this includes the impact of corrosion on inelastic buckling and low-cycle fatigue degradation of reinforcements. In this analysis, the influence of shotcrete repair after the initial cracking of concrete cover as a rehabilitation method on the performance of a corroded square RC column due to chloride-induced corrosion is investigated.

1. Introduction

The durability of reinforced concrete (RC) structures are significantly affected by the deterioration of their structural members. The deterioration caused by the corrosion of RC members is usually considered to be one of the main sources of structural degradation which may eventually result in the serviceability failure of RC structures under service or extreme loading conditions. An accurate estimation of the extent of degradation during the structure's lifecycle provides both engineers and decision-makers with valuable information which helps to ensure the safety of RC structures while reducing the associated costs. To this end, the current article focuses on the corrosion process caused by the attack of chloride ions and evaluates effects of a repair and

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rehabilitation method on the time-dependent performance of RC structures.

1.1. Literature Review

A number of repair or rehabilitation methods have been developed for damaged RC structures, including concrete overlay, epoxy injection of cracks, shotcrete repair, external posttensioning, externally bonded steel or composite plates, externally bonded concrete, penetrant sealers, wrapped carbonfibers, and the addition of supplemental or replacement members, among others [1].

Chloride induced corrosion includes two phases, initiation, and propagation, for RC structures in marine environments [2-4].

The initiation phase occurs when chloride penetrates into the concrete from its surface until the chloride concentration on the reinforced surface reaches a critical value of chloride concentration (Ccr), and until sufficient oxygen and moisture are present for corrosion time (Tcorr) to begin. The Ccr depends on several factors, including exposure conditions and the chloride ion value of the sea water [3].

The south of Iran has an aggressive marine environment along with a high evaporation rate, high temperature, and a high concentration of salt in water, which enhances chloride-induced reinforcement corrosion [5,6]. Significant studies [4-7] have been performed on chloride-induced corrosion as a deterioration mechanism in RC structures such as bridge columns and jetty structures in such aggressive environments. Predicting chloride ingress into the concrete at the initiation phase based on Fick's second law of diffusion is the main approach for durability design and service life determination of RC structures [8].

In the propagation phase, following the corrosion initiation time, the chloride concentration increases on the reinforcement surface, the production of the corrosion reaction and iron rust begins, and the percentage of corrosion increases from zero after Tcorr.

Some researchers have developed empirical models for predicting the chloride diffusion coefficient [7,9-11] and surface chloride concentration [12-14] in concrete in the initiation phase of corrosion, and empirical models for predicting the corrosion rate using concrete resistance [15-18] in the propagation phase of corrosion.

As a result, in existing RC structures, repair and maintenance to decrease the chloride diffusion into the concrete and reduce the corrosion rate of the reinforcement seems to be significant.

Aplying shotcrete on the surface of the concrete after the initiation of crack time due to rust expansion will be effective at delaying the failure time of RC structures and increasing their service life.

Cho et al. [19] inspected the influence of increasing the concrete cover thickness on the corrosion of the reinforcement as a result of chloride penetration into the concrete. They found that increasing the increment of concrete cover thickness reduces the corrosion rate. However, corrosion potential, electronic resistance, and current intensity are not dependent on the concrete cover thickness. Further, Enright et al. [20] applied shotcrete repair in their study. Before using this repair method, the damage to the corroded reinforcement was removed by sand blasting, and the corrosion process was stopped. Next, a layer of shotcrete was applied. The exterior surface of the shotcrete was exposed to the chloride ions, and the corrosion initiation time for the repaired beam was presumed to be the same as that for the non-damaged beam. The shotcrete repair showed very little effect on degradation in probabilistic analysis.

1.2. Research Significance

In this research, a performance-based model of the life cycles of corrosion-affected RC structures is proposed with a focus on the effect of propagation of reinforcement corrosion on structural deterioration. Timely maintenance and repairs have the potential to increase the service life of corrosion-affected RC structures. The present study investigates the influence of using 15 mm shotcrete as a repair scenario in RC column with mixture design of PC concrete.

2. Analytical Studies

In this study, the nonlinear finite element modeling technique is employed through applying the fiberbeam-column element developed Afsar Dizaj et al. [21] to simulate the flexural response columns. The modeling technique has been calibrated and validated for a rectangular RC column of bridge piers, for both uncorroded and corroded columns [21].

A nonlinear fiber beam-column element is a line element in which the moment-curvature response at selected locations is determined by the fibersection assigned to that integration point. In RC columns, the most inelastic behavior occurs near the base of the column [22]. To model the strain penetration and the slippage of reinforcement anchored to the foundation, a zero-length section element is applied. Examples are modeled in OpenSees 2012 finite element software. A detailed discussion of the elements and zero-length section are available in reference [21].

By using the verified a finite element model of a square cross-section column, the time-dependent structural capacity of this column is investigated.

The section dimension, height of column, initial concrete cover, diameter of longitudinal reinforcement, number of longitudinal reinforcements in each section, diameter of horizontal reinforcements (ties) and tie spacing of circular column are equal to 250 mm, 1800 mm, 35 mm, 16 mm, 8, 8 mm and 80 mm, respectively.

The mechanical properties of vertical and horizontal tie reinforcement are presented by Afsar Dizaj et al. [21]. The concrete compressive strength is deliberated to be 30 MPa. In order to inspect the effect of corrosion on time-dependent displacement ductility and strength loss of corroded RC columns, different degrees of corrosion (i.e., mass loss ratios) are considered. Analyses are conducted for constant axial force ratio 0.1 ($Nu / \sigma c Ac$), where Nu is the axial force, σc is the compressive strength of concrete and Ac is the gross cross-section area of column cross-section. It is worth mentioning that corrosion of horizontal ties is also considered in these analyses.

At each time after corrosion initiation time, that annually added, we have calculated corrosion current density and crack initiation time, crack width and corrosion percentages of reinforcements by developed formulations in literature by MATLAB programming. Corrosion percentages of horizontal and longitudinal reinforcements are inputs of the finite element model proposed in OpenSees to calculate the corroded diameter of these reinforcements, the stress-strain behavior of concrete and steel after corrosion [21]. Finally, the time-dependent capacity of corroded RC square column in the finite element model is investigated by static nonlinear pushover analysis. Moreover, structural performance and failure modes of the repaired corroded square column are analyzed and the performance of the repair scenario is inquired.

3. Results and Discussion

3.1. Chloride Diffusion Coefficient Model

In the current paper, the diffusion coefficient model for PC concrete in the tidal zone of Qeshm Island in the south of Iran is used based on experimental data developed by Farahani [23].

$$D = 10^{(-12)} \times (156.8\, w/c - 43.482) \left(\frac{t_{ref}}{t}\right)^{1.0157\, w/c + 0.2571} \exp\left[\frac{U}{R}\left(\frac{1}{T_{ref}} - \frac{1}{T}\right)\right]\left[1 + \frac{(100-h)^4}{(100-h_c)^4}\right]^{-1} \quad (1)$$

where, t is the current time (in months), tref represents reference time (tref = 3 months) and n is the age factor, taking into account the time dependence of the chloride penetration into the concrete. w/c is the water-to-cement ratio $(0.35 \le w/c \le 0.50)$. For the temperature function, U (in J.mol-1) presents the activation energy of diffusion process, R is the gas constant (8.314 J. mol-1.K-1), T is the current temperature (in K), and Tref denotes reference temperature (Tref = 306.5 K). The value of U/R ratio for the Persian Gulf region has been predicted as 2948 K [24]. For the relative humidity function, h (in percent) presents the relative humidity of concrete specimen, and hc is the critical humidity level at which the diffusion coefficient drops halfway between its maximum and minimum values (≈75%).

3.2. Surface Chloride Concentration Model

The surface chloride concentration model for PC concrete in the tidal zone in the south of Iran is applied in consonance with experimental data developed by Khaghanpour et al. [12].

$$C_s = \mathrm{K}\, t^{\mathrm{d}} + \mathrm{C}_0 \quad (2)$$

where, Cs (in % weight of concrete) is the surface chloride concentration, K, d, and C0 are variable with water-to-cement ratio.

3.3. Corrosion Initiation Time

The corrosion initiation time (Tcorr in s) of longitudinal or horizontal reinforcement is calculated by Eq. (3). And, Eq. (3) is calculated by using the analytical solution of Fick's second law of diffusion for 1D presented by Luping [25].

$$T_{corr} = \left(\frac{x^2}{4D}\right) \times \left(erfinv\left(\frac{C_s - C_{cr}}{C_s - C_i}\right)\right)^{-2} \quad (3)$$

where, Ci (in % weight of concrete) is the initial chloride concentration experimentally equals to 0.015% weight of concrete for PC concrete [23,24] in Qeshm Island. D (in m2/s) is the proposed chloride diffusion coefficient model in Eq. (1). x (in m) is the required cover thickness for PC concrete in time t (in s) that the distance between the surface of concrete and surface of longitudinal bars, and between the surface of concrete and surface of horizontal bars are demonstrated by bar_depth and tie_depth, respectively.

The corrosion initiation time of PC concrete with water-to-cement ratios of 0.35, 0.4 and 0.5, when contemplating the critical value of chloride to be 0.07% of concrete weight in the Persian Gulf region [12], is equal to 2.26, 1.35 and 0.5 years. This result indicates that using PC concrete without pozzolanic materials and with low cover thickness is not suitable for corrosive marine environments such as the Persian Gulf. Many civil infrastructure projects, including bridges, wharves and jetty structures in the marine environments are degrading at an early age in the service life of structures as a result of chloride-induced corrosion. Therefore, preventing this destruction and increasing the service life of these structures is essential for corrosive marine environments.

3.4. The diameter of longitudinal and horizontal bars after corrosion initiation

The reduced diameter of longitudinal (dt) and horizontal (dt_tie) bars, are calculated by Eqs. (4) and (5), respectively.

$$dt = d_0 - \alpha P_{avg} \tag{4}$$

$$dt\_tie = d_{0\_tie} - \alpha P_{avg\_tie} \tag{5}$$

The value of α is 2 for uniform corrosion and varied from 4 to 8 for pitting corrosion [26]. d_0 and d_{0_tie} are the initial diameters of longitudinal and horizontal bars, respectively. The Pave(t) is average corrosion penetration depth based on the uniform volumetric mass loss at time t after corrosion initiation which can be calculated as follows:

$$P_{ave}(t) = \kappa \int_{T_{corr}}^{t} i_{corr}(t)\, dt \tag{6}$$

where, κ is the conversion factor from µA/cm2 to mm/year that equals to 0.0116, and $i_{corr}(t)$ is presented the corrosion current density of reinforcements at the time after the corrosion initiation time (Tcorr) calculated by Eqs. (7) and (8) [18]:

$$i_{corr0} = \frac{37.8(1-w/c)^{-1.64}}{x} \tag{7}$$

$$i_{corr}(t) = 0.85\, i_{corr0}(t - T_{corr})^{-0.29} \tag{8}$$

3.5. Crack initiation time

Liu and Weyers [27] suggested an empirical model for prediction of the crack initiation time

(t_{cr}) is calculated by Eq. (9).

$$t_{cr} = \frac{W_{crit}^{\ 2}}{2k_p} \tag{9}$$

$$k_p = 9.8\times10^{-5}\frac{\pi d_0 i_{corr}(t)}{\beta} \tag{10}$$

where, Wcrit is the total amount of critical rust products needed to fill the volumes of pores and rust expansion, kp is the rate of the production of corrosion products given by the empirical Eq. (10) suggested by Val [28]. where, β is the ratio between the molecular weight of steel and molecular weight of corrosion products, and d_0 is the initial diameter of the bar. Further details and derivation are available in [28].

The crack initiation time of the concrete cover of the no-repair scenario for PC concrete with 0.35, 0.40 and 0.50 water-to-cement ratios are calculated by Eq. (9) equal to 3.90, 3.53 and 2.69 years, respectively.

3.6. Corrosion Percentage

Finally, Eqs. (11) and (12) present the corrosion percent of longitudinal (ψ) and horizontal

($\gamma - tie$) bars, respectively.

$$\psi = 100\left(1-\left(\frac{dt}{d_0}\right)^2\right) \tag{11}$$

$$\psi\_tie = 100\left(1-\left(\frac{dt\_tie}{d_{0\_tie}}\right)^2\right) \tag{12}$$

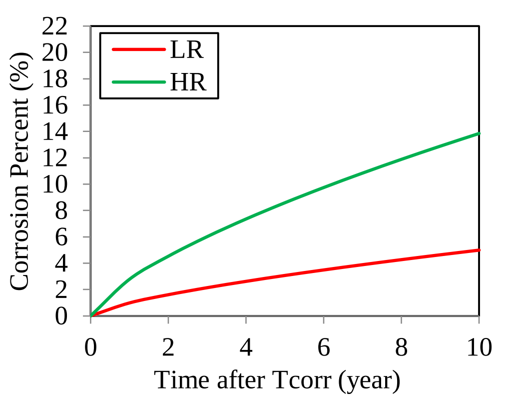
Figs. 1-3 demonstrate the corrosion percentage of longitudinal and horizontal bars over time (after the corrosion initiation time) of the no-repair scenario and applying the shotcrete as a repair scenario at the crack initiation time of PC concrete with different water-to-cement ratios 0.35, 0.40 and 0.50.

Fig. 1 (b) illustrates the corrosion percentage of longitudinal and horizontal bars decrease 32% and 37%, respectively by applying the shotcrete as a repair than no repair scenario, Fig. 1 (a).

Fig. 2 (b) portrayed the corrosion percentage of longitudinal and horizontal bars decrease 32% and 36.9%, respectively, by applying the shotcrete as a repair than no repair scenario, Fig. 2 (a).

Fig. 3 (b) depicted the corrosion percent of longitudinal and horizontal bars decrease 35.9% and 41.4%, respectively by applying the shotcrete as a repair than no repair scenario, Fig. 3 (a).

In order to investigate the influence of repair scenario on the structural performance of corroded columns, a series of time-dependent nonlinear pushover analyses have been conducted with various corrosion percentages.



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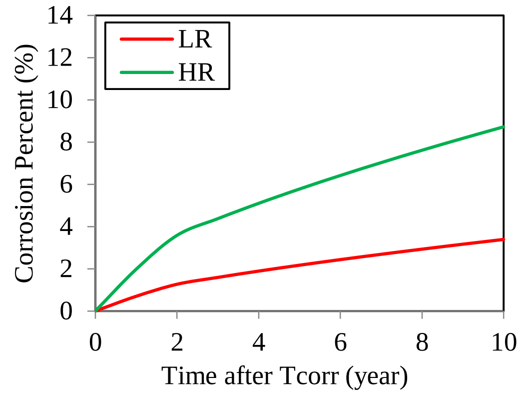
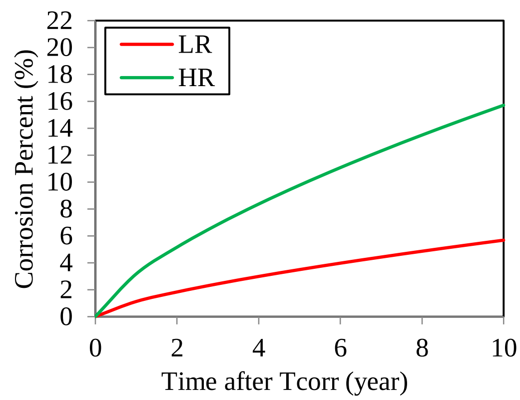


(b)

Fig. 1. Corrosion percentage of longitudinal and horizontal bars over time; (a) No Repair, (b) using shotcrete. (w/c=0.35).



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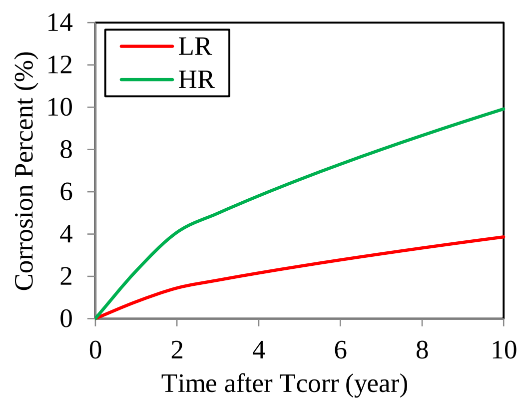
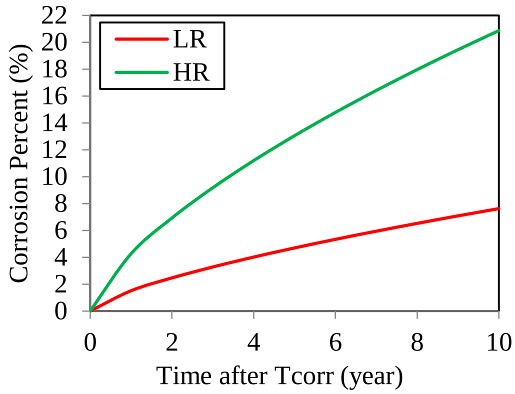


(b)

Fig. 2. Corrosion percentage of longitudinal and horizontal bars over time; (a) No Repair, (b) using shotcrete. (w/c=0.40).



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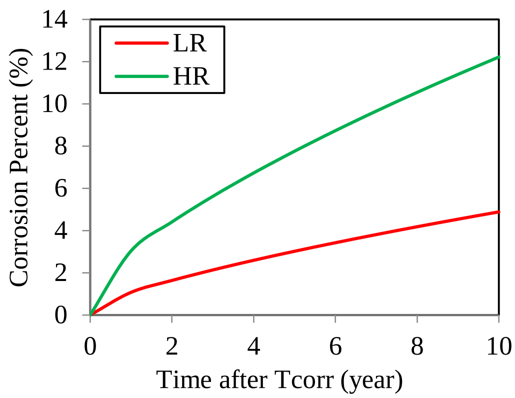
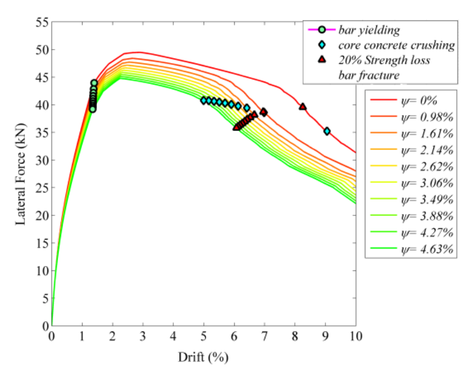


(b)

Fig. 3. Corrosion percentage of longitudinal and horizontal bars over time; (a) No Repair, (b) using shotcrete. (w/c=0.50).

3.7. Nonlinear behavior, failure modes, and time-dependent capacity

Figs. 4-6 reveal the results of monotonic pushover analyses. According to Figs. 4-6 (a), for the axial force ratio (ratio of axial force to section area and compressive strength) of 0.1, the failure mechanism of the square column, apart from the corrosion percentage equals to 0% mass loss; before 20% strength loss, it is core concrete crushing in compression. So, the distance between core concrete crushing in compression failure mode and 20% strength loss increases by increasing the water-to-cement ratio and the corrosion percentage. However, for any corrosion levels, bar fracture in tension failure mode not happen. As it is evident in Figs. 4-6 (b), by using the 15 mm shotcrete on the surface of concrete and using the same concrete mix design at the crack initiation time, results in a reduction in the corrosion percentage of a corroded square column compared to the previous no-repair scenario. Moreover, approximately up to 3.85%, 4.12% and 4.53% mass loss for 0.35, 0.40, and 0.50 water-to-cement ratio, respectively, the core concrete crushing failure mode occurs after 20% strength loss; just bar yielding is the dominant failure mode. Thus, the structural performance of the corroded square column increases by repairing with shotcrete. However, beyond 3.85%, 4.12% and 4.53% mass loss for 0.35, 0.40 and 0.50 water-to-cement ratio, respectively, core concrete crushing failure mode overtakes from 20% strength loss. Moreover, comparison between the shotcrete repair scenario and the no-repair scenario indicates that applying 15 mm shotcrete on the concrete surface causes the core concrete crushing in compression failure mode and 20% strength loss close to each other. So, using the 15 mm shotcrete causes the corrosion levels are less than no-repair. Moreover, applying 15 mm shotcrete increases structural capacity and performance of corroded RC square column.



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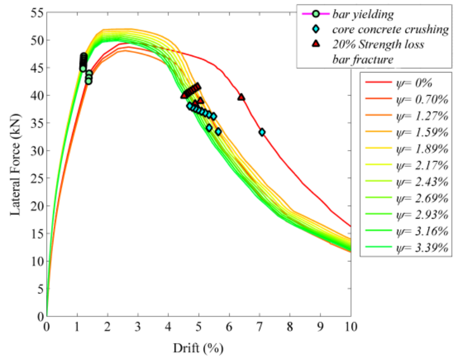
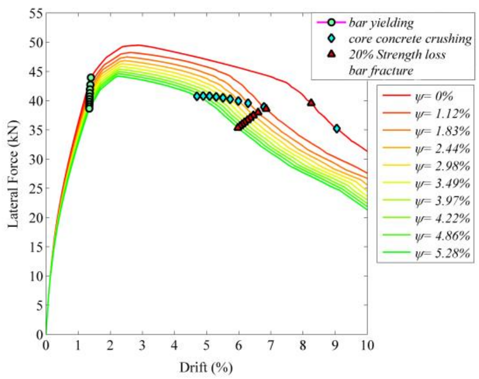


(b)

Fig. 4. Pushover curve; (a) No Repair, (b) using shotcrete. (w/c=0.35).



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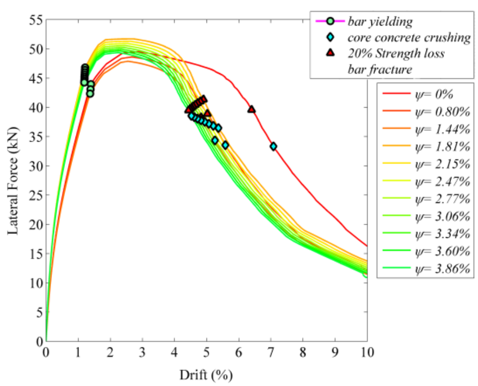
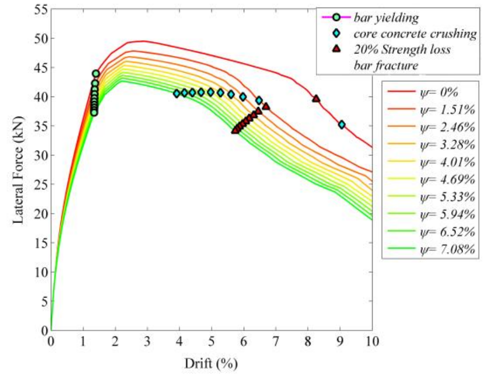


(b)

Fig. 5. Pushover curve; (a) No Repair, (b) using shotcrete. (w/c=0.40).



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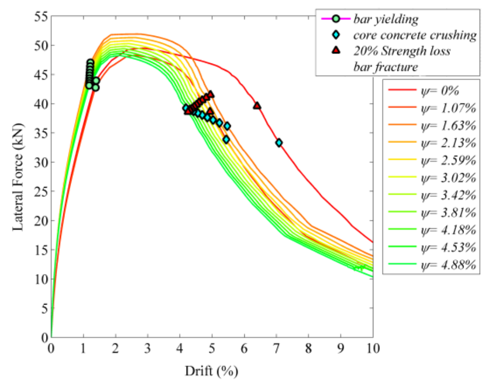

(b)

Fig. 6. Pushover curve; (a) No Repair, (b) using shotcrete. (w/c=0.50).

4. Conclusion

Inspection, repair, and maintenance of corroded RC structures exposed to marine environments such as the south of Iran are essential for improving the structural performance, durability and service life of these structures.

In this research, a finite element model is applied to assess the time-dependent capacity of corroded RC structures using nonlinear analysis. In this analysis, the influence of 15 mm shotcrete repair after the initial cracking of concrete cover as a repair or rehabilitation method on the performance of a corroded square RC column due to chloride-induced corrosion are inspected.

Results reveal that using 15 mm shotcrete causes the core concrete crushing occurs after 20% strength loss at each corrosion level. It is noteworthy to mention that increasing the water-to-cement ratio causes the core concrete crushing in compression failure mode to occur near to 20% strength loss in high corrosion levels.

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