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# SEISMIC PERFORMANCE ASSESSMENT OF RC STRUCTURES EXPOSED TO THE CORROSION AGGRESSIVE ENVIRONMENT OF THE PERSIAN GULF

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## ABSTRACT

Seismic capacity of RC structures can be affected by degradation phenomena such as corrosion in high seismic regions. The performance based design of RC structures should account for the time dependent variation of structural response due to degradation phenomena. Concrete deterioration due to different mechanisms such as the loss of the cross sectional area of the reinforcement bars, decrease of the capacity of corroded reinforcing bars, and stiffness degradation of concrete cover resulting from reinforcement progressive corrosion usually may lead to significant effect on structural capacity. Persian Gulf zone in Iran is one of the most vulnerable regions with high seismic risk. In this research the long term process for capacity estimation of typical RC structures in Persian Gulf zone as a function of time by using nonlinear dynamic and for seven ground motion records is estimated in different performance levels such as immediate occupancy, life safety and collapse prevention. The final results are sorted for maximum inter-story drift ratio. The outcomes are discussed in terms of seismic fragility curves and show that removing the concrete cover on columns in bottom of structure has a greater impact on the structural capacity of the RC structures than decreasing the rebar mechanical parameters.
*Keywords: Corrosion; Time-dependent Nonlinear Analyses; Seismic Response; Persian Gulf Zone; Performance Levels*

## 1. INTRODUCTION

Corrosion of reinforcement has the potential to affect all types of reinforced concrete structures, the RC structures in coastal regions are vulnerable to more damages because of seawater during their life cycle. In the past, performance of RC structures under a specific natural hazard such as an earthquake or wind or environmental stressor such as corrosion has been studied individually several times. Nevertheless, in real conditions, accurate assessment of RC structures vulnerability demands the effects of those phenomena to be studied simultaneously. Corrosion of the embedded bars in RC bridges may cause intensive problems within the life cycle of the structure under earthquake events. Recently, response evaluation of corroded RC structures such as bridges under earthquakes has been studied in some researches e.g. (Choe et al. 2008) and (Ghosh et al. 2010).
Concrete structures are increasingly being deteriorated in Persian Gulf region, mainly due to the chloride-induced corrosion of the embedded steel. Severity of this environment in which the average temperature exceeds more than 30 °C and the relative humidity is about 70-90 % has made Persian Gulf one of the most aggressive environments in the world (Ghoddousi et al. 1998). In high seismic regions, such as Persian Gulf zone in Iran, corrosion of reinforcement and concrete deterioration during their life cycle may weaken structures and make them more vulnerable to future earthquake hazards.
When the RC structures are located in coastal region, chloride ions existing in seawater penetrate the concrete cover, transport to the steel surface, destroy the protective passive layer, gradually increase and when the concentration on chloride ions at the surface of reinforcing bars reach a threshold value, corrosion is initiated. The effects of chloride-induced corrosion in RC structures and the effects of

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dynamic performance of structure are taken into account in this study (Kalantari et al. 2015).
By increase of exposure time of a RC structure to corrosive conditions, the degradation of reinforcement increases proportionally. This phenomenon results in spalling of the concrete and making the bars more slender. Structural weakening is calculated over the life of the RC structure and new properties of RC member are updated at any given time.
In this study, in contrast to the previous studies (Choe et al. 2008) and (Ghosh et al. 2010), three combined effects of corrosion (the loss of the cross sectional area of the reinforcement bars, decrease of the capacity of corroded reinforcing bars, and stiffness degradation of concrete cover resulting from reinforcement corrosion) were used in the time-dependent nonlinear analyses of a corroded typical RC frames while considering Persian Gulf environmental conditions.
To evaluate the structural capacity and seismic performance of deteriorated RC structures, a typical RC frame in Iran has been modeled. The design of these types of RC frames was done by Iranian earthquake CODE 2800 (Iranian earthquake code 2015).
When reinforcement bars get corroded, integrity and capacity are more likely to be reduced. In regions with seismic hazard, the strength reduction may become larger during an earthquake due to the loading demands. Therefore, recognizing time-variant risks shall help engineers and managers make more reasonable decisions considering optimization, inspection, maintenance and replacement of RC structures.
The results of this study are based on a numerical simulation with regard to valid results of previous researches. Bond strength reduction is not considered in this study and needs to be considered in another research. Also confinement effect in corrosion were disregarded.
In the present research, computational approach is used to predict corrosion initiation time while considering experimental results obtained from Persian Gulf environmental conditions which are used to determine the main parameters in corrosion process. It was assumed that the typical RC frame studied in this article is located in Persian Gulf zone. Eighteen ground motion were used to perform IDA analysis for this type of frame. Probabilistic analysis of RC frame performance was carried out and presented in fragility curves. So in this study the inter-relation of parameters in degradation of RC frames of Persian Gulf as an aggressive zone is considered in a model simultaneously

## 2. SERVICE LIFE OF CORRODED STRUCTURES

The service life of concrete structures exposed to chloride ions can be presented by the modified version of Tuutti's two-stage model (Tuutti 1982).

### *2.1 Corrosion Initiation Phase*

In numerous researches, it has been expressed that the primary mechanism for chloride transport through the concrete pore system is diffusion, such as ACI 365 (2000), Choe (2008 and 2009) and Ghosh and Padgett (2010).
Most diffusion models are based on the solution of the one-dimensional version of Fick's second law in a semi-infinite solid; which is expressed as Eq. (1):

$$\frac{\partial C}{\partial t} = D_c \frac{\partial^2 C}{\partial x^2} \tag{1}$$

Where, C is chloride concentration at a distance x from the surface after the time t, and $D_c$ is diffusion coefficient. In a commonly employed solution under the assumption of a constant diffusion coefficient, and boundary conditions specified as $C = C_s$ and the initial conditions specified as $C = 0$ for $x>0$, $t = 0$, the chloride concentration C at depth x and time t is expressed as:

$$C_{(x,t)} = C_s \left[ 1 - erf\left( \frac{x}{2\sqrt{D_c t}} \right) \right] \tag{2}$$

Where, $C_{(x,t)}$ is the chloride concentration at depth x after time t, *erf* is the error function, $C_s$ is the

chloride concentration on the concrete surface, $D_c$ is diffusion coefficient (length$^2$/time), x is the distance from any point inside the concrete to the surface (length), and t is the time.
One of the major causes of chloride penetration is a condition to which concrete structure is subjected in marine environment (Ghods et al. 2005). Five exposure zones could be introduced including splash, tidal, submerged, soil and atmospheric zone according to the location of structural elements relative to seawater level. These five different exposure conditions are shown schematically in Figure 1.

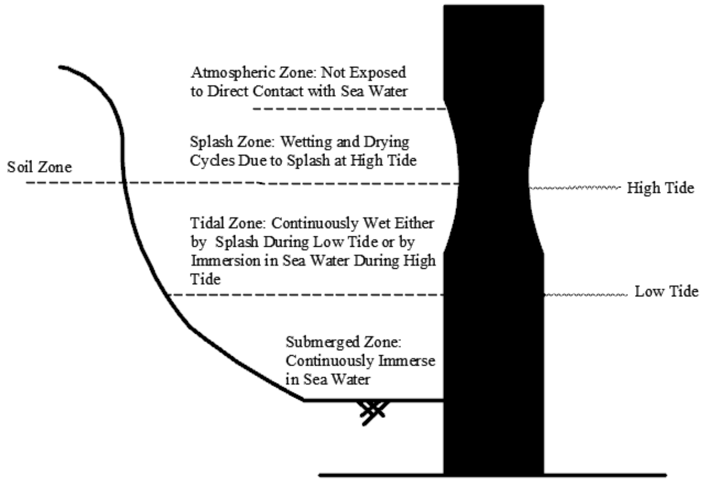



Figure 1. Schematically five different exposure conditions in Persian Gulf (Ghods et al. 2005)

Because the RC frame studied in this article is located in Persian Gulf zone, experiment results in this area are used to determine the main parameters in corrosion process. The diffusion coefficient and the chloride concentration on the concrete surface are determined based on researches of Ghods *et al.* (2005). They have calculated $D_c$ and $C_s$ by curve fitting of the chloride profiles to Fick's second law of diffusion in different mixture properties and exposure conditions.
Values given for $D_c$ and $C_s$ in Persian Gulf's environmental conditions for a three months period are given in Table 1 and 2.

Table 1. Monthly average temperature (Ghods et al. 2005)

| Month | Temperature ºC) |
|---|---|
| May | 30.8 |
| June | 33.5 |
| July | 34.5 |

Table 2. Values of $D_C$ and $C_s$ after three months of exposure (Ghods et al. 2005)

| Zone | $D_c$(cm$^2$/yr) | $C_s$(% conc) |
|---|---|---|
| Atmosphere | 0.262 | 0.051 |
| Soil | 0.382 | 0.033 |
| Tidal | 1.555 | 0.542 |
| Submerged | 1.372 | 0.570 |
| Splash | 1.523 | 0.948 |

The time to onset of chloride induced corrosion ($T_i$) occurs when the chloride concentration at the depth of the reinforcement (cover depth) reaches the critical chloride concentration, $C_{cr}$ which depassivates the steel embedded in the concrete if sufficient moisture and oxygen are present. By setting $C_{(x,t)}$ equal to critical chloride threshold, $C_{cr}$ the time to onset of corrosion is determined as follows:

$$T_i = \frac{x^2}{4D_c}\left[erf^{-1}\left(\frac{C_s - C_{cr}}{C_s}\right)\right]^{-2} \tag{3}$$

where, x is depth of concrete cover, $D_c$ is chloride diffusion coefficient, $C_{cr}$ is threshold level of chloride concentration that causes dissolution of the protective passive film around the reinforcement and initiates corrosion, $C_s$ is the equilibrium chloride concentration at the concrete surface (Kashani et al. 2012), and *erf* is Gaussian error function.

### *2.2 Corrosion Propagation Phase*

During the propagation phase, the corrosion rate is a significant parameter to evaluate the damage of corrosion effects. Different models have been developed to determine the corrosion rate. In some models it is assumed that the corrosion rate is constant e.g. (Kashani et al. 2012), (Al-Sulaimani et al. 1990) and (Alonso et al. 1988) and in the other models assume that the corrosion rate is time variant (Stewart et al. 1998), (Martinez et al. 2009), (Yalcyn et al. 1996), and (Dura et al. 1998). The purpose of this study is to evaluate time-dependent seismic performance hence, the time-dependent corrosion rate developed by Vu and Stewart (2000) is used to compute the corrosion effects parameters over time. In this model, $O_2$ availability at the steel surface is assumed the governing factor. This model is suitable for regions with average relative humidity (RH) over 70% and temperature of 20$^o$C like Iranian coast of Persian Gulf. For this environmental condition, corrosion rate up to one year after the end of the corrosion initiation phase was expressed empirically by Eq. (4):

$$i_{corr(1)} = \frac{37.8\ (1\text{-}\ w/c)^{-1.64}}{d_c} \tag{4}$$

Where $i_{corr(1)}$ is the corrosion rate at the start of corrosion propagation (µA/cm$^2$), *w/c* represents the variable water-to-cement ratio and $d_c$ is cover depth (cm), which is the distance from the surface of steel bar to the surface of concrete structure. The corrosion rate at time $t_p$ during the propagation phase is expressed as Eq. (5):

$$i_{corr}(t_p) = i_{corr(1)} 0.85\ t_p^{-0.29} \tag{5}$$

Where $t_p$ is the time since corrosion initiation and $i_{corr(1)}$ is given by Eq. (4) for determining the corrosion rate at initiation of corrosion propagation phase (Vu et al. 2000).

## 3. STRUCTURAL DETERIORATION DUE TO CORROSION

To predict the strength of deteriorated steel reinforcement, the experimental results reported by Due et al. (2005a and 2005b) for estimating the residual strength of corroded bars were used in modeling of the deteriorated RC bridge. The empirical formula developed by Du *et al.* (2005a and 2005b) to evaluate residual strength of corroded reinforcing bars embedded in concrete is used to calculate time-dependent loss of yield strength in corroded reinforcing bars.

It is assumed that corrosion is generally uniform over the reinforcing steel surface. With this assumption, the diameter of the reinforcing bars will decrease with time and is a function of corrosion rate which is a time dependent phenomenon. Therefore, the reduced diameter $D(t)$ of a corroding reinforcing bar at time t after corrosion initiation can be estimated using Faraday's Law as:

$$D(t) = D_0 - k_{corr} \int_0^t i_{corr}(t)dt \tag{6}$$

Where $D(t)$ is the reduced diameter (length) of the reinforcing bar at some time, $D_0$ is the initial diameter of the reinforcing bar (length), $i_{corr}(t)$ is the corrosion rate (current/area$^2$), t is the time from corrosion initiation, and $k_{corr}$ is the corrosion rate conversion factor which is 0.023 to convert corrosion rate from μA/cm$^2$ to mm/year.

$$f_y(t) = [1.00 - 0.005m(t)]f_{y0} \tag{7}$$

where $f_y(t)$ is the yield strength of corroded reinforcement at each time step; $f_{y0}$ is the yield strength of non-corroded reinforcement; $m(t)$ is percentage of steel mass loss over time calculated from the consumed mass of steel per unit of length divided by original steel mass and t is time elapsed since the initiation of corrosion (years).

Concrete cover gets damaged because of reinforcement corrosion and can finally diminish after spalling off. When the stress at the boundary exceeds the tensile capacity of the concrete, the concrete cover will crack. Some attempts to develop a corrosion-cracking model have been made by various researchers. Some researchers have developed models to evaluate concrete cover degradation as a result of cracks which are in turn induced by reinforcement cracks, using the mentioned approach.

In this study, models developed by Li et al. (2006) and Zhong et al.(2010) were used to assess the stiffness degradation of the concrete cover resulting from cracked concrete caused by corrosion of reinforcement as a function of the corrosion rate.

Considering the environmental condition in this article, a life cycle equal to 50 years after Corrosion initiation time is assumed for the RC frame under study in Persian Gulf zone. Hence, the Capacity of structure under study will be evaluated every 10 years after the corrosion initiation time. The parameters employed for structure to compute the effects of corrosion on structural and dynamic characteristics are shown in Table 3.

Table 3. Values of the parameters for computing the effects of corrosion

| **Time (year)*** | | **0** | **10** | **20** | **30** | **40** | **50** |
|---|---|---|---|---|---|---|---|
| Diameter (mm) | 26 | 26.00 | 23.64 | 22.14 | 20.85 | 19.68 | 18.59 |
| | 22 | 22.00 | 19.64 | 18.14 | 16.85 | 15.68 | 14.59 |
| | 10 | 10.00 | 9.64 | 8.27 | 6.85 | 5.68 | 4.59 |
| Yield Strength | $(M_{loss}/M_0)\times100$ | 0.00 | 14.22 | 21.93 | 29.62 | 35.61 | 4096 |
| | $f_y/f_{y0}$ | 1.00 | 0.93 | 0.89 | 0.85 | 0.83 | 0.80 |
| Cover | $\alpha$ | 1.00 | 0.45 | 0.05 | 0 (N.C.)** | 0 | 0 |
| Corrosion Rate ($\mu A/cm^2$) | $i_{corr}$ | 24.00 | 10.27 | 8.40 | 7.47 | 6.87 | 6.44 |

*year: After corrosion initiation
**N.C.: No concrete cover

## 4. NUMERICAL STUDY

### *4.1 RC frame Model*

To evaluate the structural capacity and seismic performance of deteriorated frame, a typical RC frame in Iran has been modeled. The design of this type of frames was done by CODE No. 2800 (2015).

This model is a tow-dimensional, four-bay and seven-story frame with a rectangular column and beam

cross-section. An overview of the finite element model for the non-degraded frame is presented herein for completion prior to describing the influence of corrosion on the frame model. The present study focuses on the degradation of the RC columns of ground floor in a corrosive environment and the effects of corrosion are not expected in the beams.
The width of bays and height of stories are similar and equal to 4 and 3.3 meters respectively. A schematic view of the frame under study is illustrated in Figure 2.

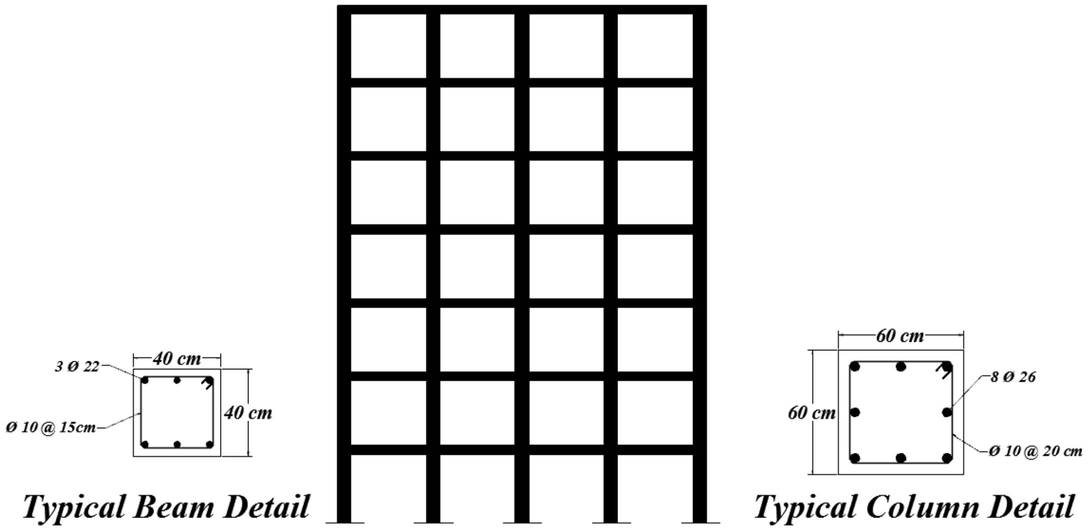



Figure 2. Schematic view of the RC frame and section details

The cover thickness in this research is 40 mm and represents the clear cover from the outer surface of the concrete to the edge of the transverse steel.
In this study, OpenSEEs software (OpenSEEs Manual, 2009) is used to analyze the structural response of the degraded structure after each ten-year interval. The OpenSEEs (Open System for Earthquake Engineering Simulation) software created by the University of California, Berkeley is used to build a finite element model of the RC frame to accomplish static and dynamic analyses.

### *4.2 Material Model*

In this study, the analysis of confined corroded column of the RC frame model was performed by using uniaxial Popovics concrete material object with degraded linear unloading/reloading stiffness according to the work of Karsan-Jirsa and tensile strength with exponential decay (Karsan et al. 1969).
The confined concrete (core) and unconfined concrete (cover) are defined using the Concrete04 material within OpenSees, which assumes that the concrete material has no tensile.
In addition to concrete, mechanical properties of reinforcing bars in the analyses must also be modeled. The longitudinal reinforcements are modeled with a bi-linear stress-strain relation that accounts for strain hardening in OpenSees. The steel reinforcement is defined using the Steel02.

## 5. INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis method (IDA) was first developed by Cornell in 2002, it was evaluated for a 20-storey building during a PhD thesis by Vamvatsikos under supervision of Cornell (2002). The results of this analysis with scaled records of the previous earthquake accelerograms are used such that the structure's response is covered from linear elastic phase to the collapse state. In current research, peak ground acceleration (PGA) has been considered as seismic intensity parameter in the incremental dynamic analysis. Furthermore, IDA method is used to develop seismic fragility curves as well as reliability assessment. To perform incremental Dynamic analysis, a series including seven earthquake ground motions is selected from FEMA p569 and normalized from 0.1 g to frame collapse. In this study, to provide the ground motion records, PEER earthquake database is used (PEER 2017). The list of ground motion records used in this study is presented in Appendix A. Finally, performance of the frame during 50-year service life after corrosion initiation is investigated by specifying the damage states and interpreting the obtained curves. Figure 3 demonstrates the incremental dynamic curves of frame drift with different age. Results derived from the IDA are employed in estimation of seismic fragility.

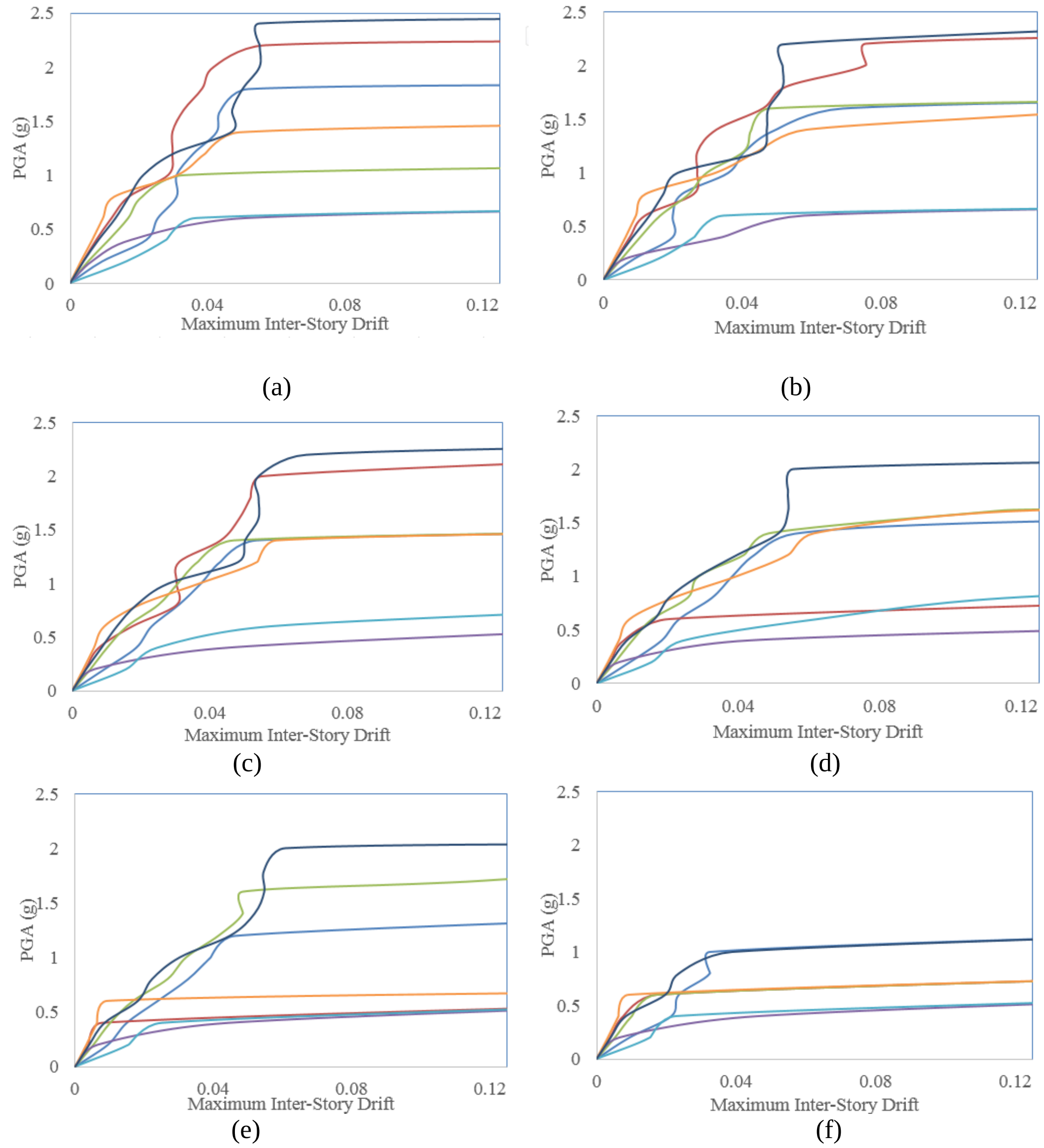



Figure 3. IDA curves for RC frame (a) pristine, (b) 10years, (c) 20 years, (d) 30 years, (e) 40 years, (b) 50 years.

## 6. DAMAGE STATES

Three performance levels were referred to assess structural performance. Maximum inter-story drift ratio of the frame was considered as performance index. FEMA 356 (2000) represented performance level for maximum structural inter-story drift ratio. These drift ratio are 1%, 2% and 4% for immediate occupancy (IO), life safety (LS), collapse prevention (CP) respectively. The recent research like Kashani et al. (2012) showed that the damage limit states that are used for seismic fragility analysis of corroded structures should be considered as time-variant parameters. The framework developed in this paper provides a computational platform for other researchers to be used in seismic fragility analysis of corroded structures. But these have not expressed as a specific limit state for different corroded structural component as a function of time.

## 7. SEISMIC FRAGILITY CURVE

Seismic fragility of a system describes the probability of the system to reach or exceed different degrees of damage, including possible collapse. The fragility function represents the probability of exceedance of a selected Engineering Demand Parameter (EDP) for a selected structural limit state (LS) subjected to a ground motion intensity measure (IM). Mathematically, seismic fragility function is illustrated by the following expression:

$$Fragility = P[LS \mid IM = y] \tag{8}$$

In this equation, LS and IM represent limit state or damage state of the frame and intensity measure, respectively. Fragility probability is calculated by:

$$p_f = P\left[\frac{D}{C} \geq 1\right] \tag{9}$$

Where D and C are drift demand and capacity of the structure, respectively. After estimation of the dispersion, which is a conditional amount on the earthquake intensity, the probable seismic demand can be obtained by using Eq. 10 [25]:

$$P(C \mid IM = x) = \Phi\left(\frac{\ln x - \mu}{\sigma}\right) \tag{10}$$

Where P(C | IM=x) is the probability that a ground motion with IM = x will cause the structure to collapse, Φ is the standard normal cumulative distribution function (CDF) and β is the standard deviation of the natural logarithm of (IM). Figure 4 shows the steps to develop fragility curve in current study.

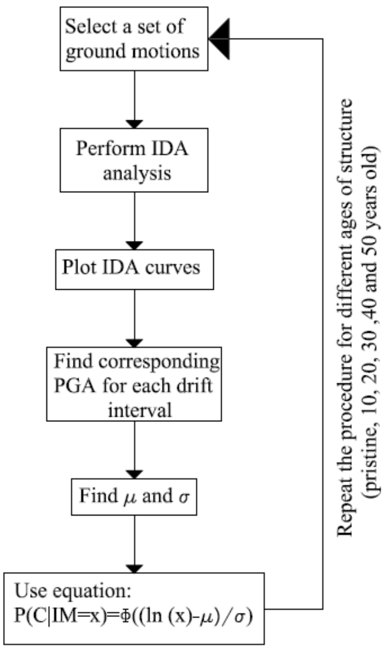
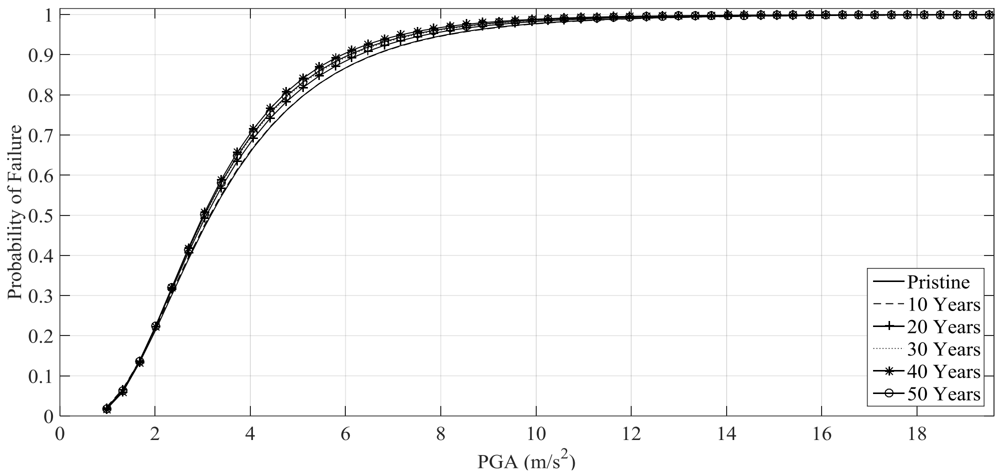



Figure 4. Fragility curve procedure in current study

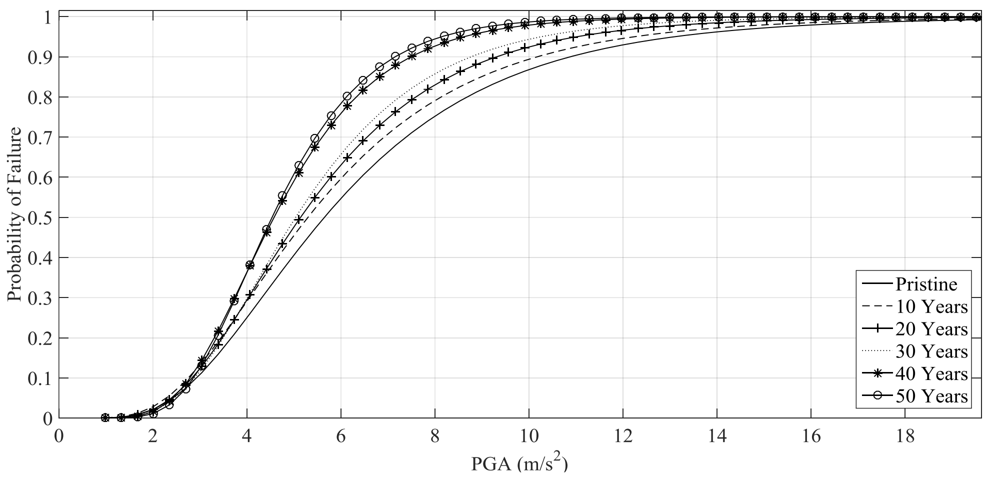
Mean and standard deviation of PGA are necessary to develop fragility curve. These parameters were calculated for every point, which are across the limit state intervals that has been mentioned.

Figure 5 presents all sets of fragility curves. Comparing the fragility curve of damage states within the set of seven ground motions in analyses of the RC frame, it is shown that when 1.0g ground motion, was exposed, the probability of reaching or exceeding I.O, L.S and C.P performance levels are 0.96, 0.85 and 0.5 respectively for pristine RC frame. This probability is approximately 0.98, 0.96 and 0.9 respectively for 50 years old structure (after the corrosion initiation time).

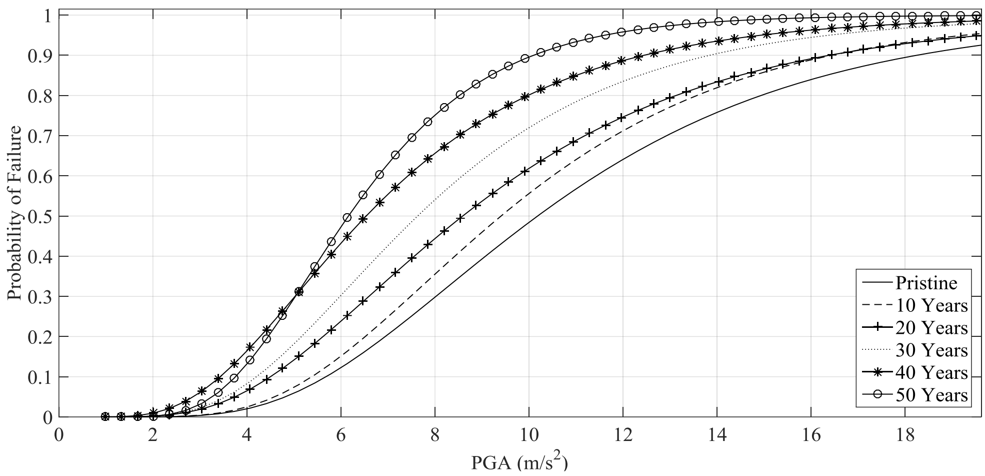
The increase in the probability of failure seen after the 20th year in all performance level, mentions the bigger effect of the removing concrete cover in comparison with degradation in the mechanical properties of the rebar in degradation resulted from corrosion. For example, in earthquakes with the intensity of 1.0 g and for collapse prevention performance level, variations probability of failure in the pristine, 10th, 20th, 30th, 40th and 50th year after corrosion initiation, are 0.5, 0.55, 0.61, 0.72, 0.80 and 0.89.




(a)




(b)




(c)

Figure 5. Fragility curves ((a) I.O, (b) L.S, (c) C.P performance intervals)

## 8. CONCLUSIONS

Persian Gulf is an area with an aggressive environment for RC structures in a high seismic zone. In this study, the long-term corrosion process of a deteriorated typical RC frame in Iran is analyzed as a function of time. Probabilistic analysis of frame performance was carried out and presented in fragility curves. The structural capacity and seismic performance level of the frame was predicted as a function of the corrosion rate. Three important corrosion effects considering experimental results obtained from Persian Gulf environmental conditions were used. The relationship between the corrosion rate and the structural effects caused by corrosion was explained. It is found that damage probability becomes greater as the structure becomes older, corrosion effects spread in the structure, and seismic hazard intensity increases. As a result, it is determined that the removing of concrete cover on bottom of columns has a greater impact on the structural capacity of the RC frame than decreasing the rebar mechanical parameters. Therefore, protecting concrete cover in such structures, which are exposed to corroding environments is the most important task in maintenance of these structures. Bond strength reduction is disregarded in this study; however, if it is proved to have a major effect, it should be taken into account. This study performed for typical RC frame in Iran that its details presented by CODE 2800. This type of frame is constructed in different part of Iran especially southern area with aggressive environment. So the results of research like this could help governors to present the vulnerability of these RC frame.

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## Appendix A

Appendix**.** List of ground motion records PEER (2017).

| Event | | Station | M | Angle (˚) | PGA (g) |
|---|---|---|---|---|---|
| EQ1 | Northridge | Beverly Hills | 6.7 | 9 | 0.443 |
| | | | | 279 | 0.488 |
| | | | | UP | 0.324 |
| EQ2 | Duzce, Turkey | Bolu | 7.1 | 0 | 0.739 |
| | | | | 90 | 0.805 |
| | | | | VER | 0.200 |
| EQ3 | Kobe, Japan | Nishi-Akashi | 6.9 | 0 | 0.483 |
| | | | | 90 | 0.464 |
| | | | | UP | 0.386 |
| EQ4 | Kocaeli, Turkey | Duzce | 7.5 | 180 | 0.311 |
| | | | | 270 | 0.364 |
| | | | | UP | 0.206 |
| EQ5 | Landers | Yermo Fire Station | 7.3 | 270 | 0.244 |
| | | | | 360 | 0.151 |
| | | | | UP | 0.135 |
| EQ6 | Manjil, Iran | Abbar | 7.4 | 0 | 0.514 |
| | | | | 90 | 0.499 |
| | | | | UP | 0.538 |
| EQ7 | Superstition Hills | Poe Road (temp) | 6.5 | 270 | 0.475 |
| | | | | 360 | 0.286 |
| | | | | UP | 0.286 |