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Effect of Carbonation Induced Corrosion on Seismic Vulnerability of RC buildings

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Abstract
The properties of constituent materials influence the seismic performance characteristics of RC structures. The mechanical and geometrical properties might change as a result of exposure to adverse environmental conditions. Carbonation induced reinforcement corrosion is one of the major factors causing deterioration of reinforced concrete structure. This paper aims to develop the effect of carbonation induced corrosion of reinforcement on overall seismic performance of a RC building .The fragility curves are derived at the various time periods for different limit states. The results show an overall decrease in seismic capacity and increase in seismic vulnerability over time due to corrosion indicating the significant effect of deterioration due to corrosion effects on structural behavior.The capacity of the structure is reduced by 8% after 60 years and 29% after 100 years due to reinforcement corrosion. The corrosion of reinforcement degrades the mechanical properties of the materials in a reinforced concrete which decreases the stiffness of building and thus leads to decrease in capacity and increase in fragility of the building.

Keywords
Carbonation, Corrosion,Seismic vulnerability,Time dependent fragility curves

1. Introduction

The environment to which the structures are exposed have significant effects on the performance characteristics of the structure.Traditionally,seismic investigation of structures were done accepting that the structures are ideally kept up amid their lifetime and the effect of environmental exposure is not considered while analyzing the structures.The performance of the structure may degrade with age due to different environmental factors. Among different effects of exposure condition, reinforcement corrosion is one of the major cause of degradation of the structures. There are generally two types of reinforcement corrosion. First one is initiated by the ingress of chloride ions into the structures which is common in the areas close to the marine environmental conditions. Other type of corrosion is initiated by the ingress of atmospheric carbon dioxide into the structure called as carbonation induced corrosion. This type of corrosion is more pronounced within the areas exposed to high carbon dioxide content. Because the global carbon dioxide concentration is increasing everyday,structures are getting down experience pre-matured deterioration due to carbonation induced corrosion. The global trend of increasing carbon dioxide emissions, has serious consequences for the structures that are constructed within the urban environment. The depth of carbonation increases with increase in carbon dioxide gas concentration in the environment[1].So, the consequences of carbonation on seismic performance of structures have to be quantified and regarded during the analysis of structures. Corrosion is a time dependent complex process. So, to account for its effects in structural vulnerability, time dependent fragility analysis of structures should be conducted.This study aims to spotlight the consequences of carbonation induced corrosion on the response and vulnerability of structures subjected to seismic excitation and to derive fragility curves at different period throughout the lifespan of structure incorporating the direct and indirect effect of reinforcement corrosion on different material properties.

2. Stages of Reinforcement Corrosion

There are different factors that causes the corrosion of reinforcements. Some of them are associated with the design and construction phase like the water cement ratio used while preparing concrete mix, the type of cement used, the number of curing days provided while others are associated to the environment to which the structures are exposed like the relative humidity, the temperature of the surrounding, the effective rainfall etc. Reinforcement corrosion generally consists of two phases initiation and propagation phase.During initiation phase, the outer passive layer covering the reinforcement gets destabilized under the effect of carbon dioxides. At the end of this phase, the outer passive layer is completely destabilized marking the point of corrosion initiation. The reinforcement gets uniformly corroded due to the carbon dioxide ingress. At the propagation phase,the rust which is a corrosion product built up on the surface of the reinforcement and the volume of rust goes on increasing as the corrosion progresses.Due to this, tensile stresses are set up in the concrete surrounding the steel. Cracks start to develop on the concrete cover and after some time, the corrosion products might travel through these cracks and start to appear in the surface of the concrete.

3. Corrosion Modelling

Corrosion is a time dependent complex process. It is influenced by various factors. There are different models available that help in predicting the carbonation depth. Most of these models are based on Tutti [2] principle, that describes carbonation as a function of square root function of concrete exposure time to carbon-dioxide. The complex models have little usage because most of the input parameters are difficult to get. Also, its application demands an outsized number of tests to see material properties that in most of the cases require numerical solutions. So, during this research,among the models available, the model proposed bye,the model proposed by L. Guo, Y. Yingshu and G. Ou [3] is considered in which the parameters are easy to obtain that helps to estimate the carbonation in real conditions as the input parameters are easy to obtain.The model is expressed in equation (1).

$$X = 0.10292\frac{RH}{45}^{-0.4227}\frac{T}{10}^{0.7135}\frac{w/c}{0.35}^{1.3404}\sqrt{C*t} \quad (1)$$

Where, X is the carbonation depth in mm, RH is the relative humidity in the range between 45% and 95%; T is the environmental temperature between 10 degrees and 60 degrees centigrade; w/c is the water cement ratio in the range between 0.35 and 0.74; C is the carbon dioxide concentration; t is the carbonation time.The reinforcement bars start to corrode when the thickness of the uncarbonated concrete cover is less than 5mm as suggested by Yoon et al. [4]

4. Effect of Reinforcement Corrosion

There are different direct and indirect effects of reinforcement corrosion. Direct effect include the loss in reinforcement area. The area is lost as the intact reinforcement is converted into powdered form due to corrosion. This powdered rust occupied higher volume and exerts an expansive stress on the concrete cover. As a result, the compressive strength of concrete decreases. Not only the strength of cover, the compressive strength of the confined concrete also decreases due to reinforcement corrosion. The mechanical properties of reinforcement bars , typically yield and ultimate strengths decrease due to corrosion. All of these effects are secondary effects of reinforcement corrosion. It is not justifiable to decrease the strength of overall concrete section uniformly. So, different models are used to account for the degradation of cover concrete and the confined core.

4.1 Loss in reinforcement Area

The reinforcement is protected by a outer highly alkaline passive layer. Due to continuous ingress of atmospheric carbon dioxide, this layer weakens and ultimately breaks. The stage of weakening of the outer passive layer is the initiation phase of reinforcement corrosion and the stage at which the outer layer breaks is the phase of corrosion propagation. Once the outer passive layer breaks, the reinforcement bars start to corrode. The corrosion is uniform in case of carbonation. The rate of corrosion of reinforcement depends the environmental exposure condition and depends on the time of exposure [5]. But in this study, the rate of corrosion is taken as constant throughout the service life of the structure. This assumption has been made due to unavailability of reliable models to quantify the corrosion rate with respect to time. The rate of corrosion depends on the current density. The value of current density is taken as 3(μA/cm2) as suggested by Varjonen [6]. The time dependent reinforcement area is calculated by using model provided by Ghosh and Padgett [7] which can be expressed in equation (2):

where, r_{corr} denotes the rate of corrosion of reinforcement.

4.2 Degradation of Concrete Cover

After the onset of corrosion, the reinforcement as converted to powdered rust form. It occupies higher volume than the rebar. This causes expansive stress on the concrete and cracks start to appear in the cover. The compressive strength of the cover is reduced as the corrosion proceeds. This reduction has been obtained based on the following expression as suggested by Coronelli and Gambarova [8]

$$f_{cover}(t) = \frac{f_{cover}}{1 + k\frac{e^*}{e}} \tag{2}$$

Where f_{cover}(t) represents the compressive strength of concrete cover at any time t; f_{cover}=compressive strength of concrete cover initially; k is a constant equal to 0.1 for medium rebar, e is the concrete cover's compressive strain at peak strength and e* is the average tensile strain in the transverse direction. e* can be expressed as:

$$e^* = N_{bars} * \frac{w_{cr}}{D} \tag{3}$$

Where, Nbars represent number of corroded bars; D= lateral dimension of the section and wcr= crack width of the section. The crack width can be calculated as follows as suggested by Molina et al. [9]:

$$w_{cr} = 2\pi(v_{rs} - 1)X \tag{4}$$

Where, v_{rs} represents the ratio of volume of expansive products of rust particles to the volume of the intact steel reinforcement which is generally assumed to be 2; X= Carbonation depth. Thus, the concrete cover compressive strength at any time as the corrosion proceeds can be calculated by substituting values in equation (3) that leads us to equation (6).

$$f_{cover}(t) = \frac{f_{cover}}{1 + \frac{N_{bars}k2X\pi}{De}} \tag{5}$$

4.3 Degradation of confined concrete core

The degradation of confined concrete core has been modelled by using Mander's confined concrete model. The stress-strain model for corroded confined concrete has been obtained by modifying the parameters of Mander's uncorroded confined concrete model. N.S. Vu et. al[10].

The stress-strain model for uncorroded confined concrete proposed by Mander [11] can be expressed in equation (7) and equation (8).

$$f'_{cc} = f'_{co}\left(-1.254 + 2.254\sqrt{1 + \frac{7.94f'_1}{f'_{co}}} - \frac{2f'_1}{f'_{co}}\right) \tag{6}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \tag{7}$$

where f'_{cc} is the maximum confined strength and ε_{cc} is the corresponding axial strain which are modified later to account for the corrosion effects. f'_{co} is the compressive strength of unconfined concrete.

considering corrosion effects, the confined stress can be expressed as shown in equation (9).

$$f'_{cc} = (1 - \alpha X_{corr})\, f'_{co}\left(-1.254 + 2.254\sqrt{1 + \frac{(7.94f'_1)}{f'_{co}}} - \frac{2f'_1}{f'_{co}}\right) \tag{8}$$

The strain can be expressed as shown in equation(10)

$$\varepsilon_{cc} = (1 - \beta X_{corr})\varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{co}} - 1\right)\right] \tag{9}$$

Where, X_{corr} represents the corrosion level in terms of mass loss of transverse reinforcement; α is stress correction coefficient derived from the regression analysis of the test data and it equals 0.19 for confined section with single hoops configuration. β=0.49 for confined section with single hoops configuration.ε_{co} is the axial strain of unconfined concrete corresponding to maximum stress.

4.4 Degradation of reinforcement bars

The mechanical properties of steel bars, such as strength and ductility, may degrade significantly due to the corrosion effect. According to the experimental tests conducted by Du,Clark and Chan [12], the yield strength corroded reinforcement bars can be modelled using equation (11):

$$f_y(t) = (1 - 0.005)Q_{corr}(t))f_{yo} \tag{10}$$

And the ultimate strength can be modelled using equation (12).

$$f_u(t) = (1 - 0.005)Q_{corr}(t))f_{uo} \tag{11}$$

Where f_y(t) and f_u(t) represent the yield and ultimate strength of corroded reinforcement bars at any time t. f_{y0} and f_{u0} represent the initial yield and ultimate stress of uncorroded reinforcement bars. Q_{corr}(t) is the time dependent corrosion percentage of reinforcement bars in terms of loss of area which can be expressed as:

$$Q_{corr}(t) = \frac{A_0 - A_r(t)}{A_0} * 100 \tag{12}$$

Where A_0 is the initial reinforcement area and A_r(t) represent the time dependent residual cross section area of reinforcement due to corrosion.

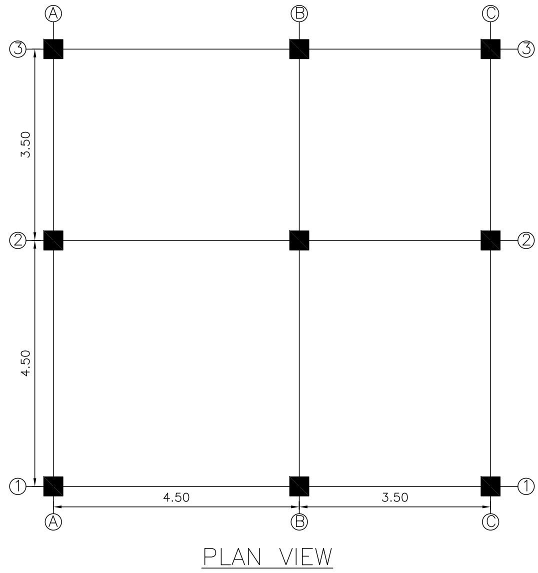

Figure 1: Plan View of building under consideration

5. Seismic Vulnerability Analysis

Seismic Vulnerability of structures represent the probability of damage of structure during course of any seismic event. The evaluation of seismic vulnerability of structure is prerequisite for effective planning, risk identification, management and mitigation of damages and losses of life and or property prior and after occurrence of an earthquake.Seismic Vulnerability of structures is generally expressed through a set of curves called fragility curves. Fragility curves are the plot of fragility functions which indicate the probability of exceedance of certain level of damage states for a particular input of ground motion.The fragility function in its simplest form can be expressed as the probability that the demand that the seismic excitation exerts in the structure exceeds the capacity of the structure.The overall time dependent fragility function of the buildings can be mathematically expressed as a two-parameter time-variant lognormal distribution [13]. It can be expressed as shown in equation (14).

$$P(\frac{DS}{IM}) = \phi(\frac{ln(IM) - ln(IM(t))}{\beta(t)} \tag{13}$$

Where, ϕ is the standard normal cumulative distribution function, IM is the intensity measure of the earthquake expressed in terms of PGA (in units of g) at the ground surface, IM (t) and β(t) are the median values (in units of g) and logarithmic standard deviations of the building fragilities at different points in time along its service life and DS is the damage state.

Traditional method of doing seismic vulnerability analysis of structures is based on the assumption that structures are kept optimally maintained during their life span and the effects of environmental exposure are not taken into consideration. But, the environment can have some serious degrading effect like corrosion that can effect the performance characteristics of structures during the course of seismic event. To account for such effects, a time dependant seismic fragility analysis of structures need to be performed.

6. Seismic Assessment of RC Building

In this study, a 2 floor residential building with 3.1m height each has been considered in the study. The model has been prepared on Etabs 19 [14] software. The plan and 3D model of the building is shown in Figure 1 and Figure 2.

The beams are of size 230mm x 350mm composed of 4 numbers of 12mm diameter HYSD 500 bars with 10mm diameter stirrups placed at spacing of 150mm centre to centre. The columns are of size 350mm x 350mm composed of 8 numbers of 12mm diameter HYSD 500 reinforcement bars with 10mm diameter HYSD 500 stirrups placed at spacing of 150mm centre to centre. The concrete used is M20. The building has been designed following the NBC 105:2020 [15] guidelines.

Corrosion has been applied to beams and columns. The input parameters used to obtain the carbonation

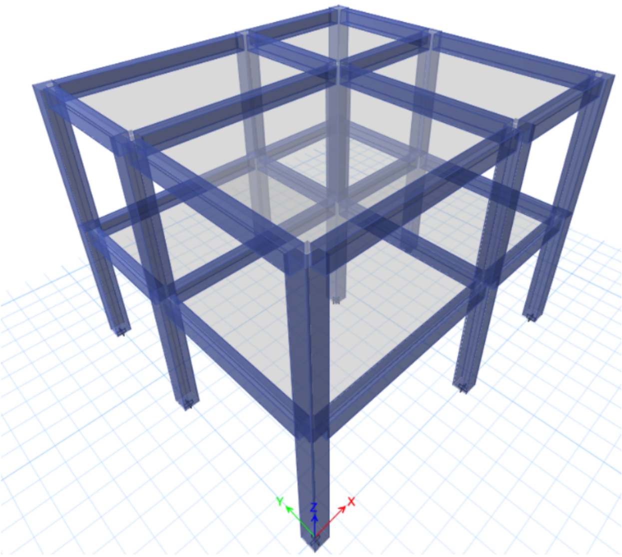

Figure 2: Three Dimensional View of building under consideration

Table 1: Parameters used in obtaining carbonation depth

Relative Humidity	74%
Water Cement ratio	0.45
Temperature	25°C
CO_2 concentration	500 ppm

depth are in consistent with the exposure condition of Kathmandu valley. These values are shown in Table 1. The degradation of concrete cover and the confined core modelled as per the methods described earlier. The plot of concrete variation in stress-strain plot of concrete as the corrosion proceeds is shown in Figure 3.

6.1 Pushover Analysis

Pushover analysis is a non linear static procedure that is used to evaluate the response of structures under seismic loads.It is a static analysis that investigates how far the structure can go into the inelastic range before it is on the verge of total or partial collapse.The reason for performing a nonlinear pushover analysis to the frame was to examine the failing point at different periods of time. The pushover curve also known as capacity curve is the plot of base shear versus displacement of the roof of the structure. The maximum displacement at the roof and base reaction of the structure during displacement controlled analysis can be obtained from the pushover curve. Figure 4 shows the outcome of pushover analysis.

Table 2: Accelerograms selected in the study

Earthquake	PGA
Chi-Chi	0.36g
Kobe	0.34g
Northridge	0.56g
Landers	0.78g
Hollister	0.19g
Imperial valley	0.32g
Loma Prieta	0.37g

Table 3: Damage states threshold

Damage State	Limit
Slight	$0.7D_y$
Moderate	D_y
Severe	$D_y+0.25(D_u-D_y)$
Complete	D_u

6.2 Time history Analysis

Time history analysis is dynamic analysis of structure. It is a step- by- step analysis of the dynamic response of a structure to the time varying load. The observation of the seismic behavior of the model with the time history analysis requires the use of accelerograms. The accelerograms used in the study are shown in Table 2.

6.3 Fragility Curves

Fragility curves describe the probability of reaching or exceeding structural damage states, provided the median estimates of structural displacement. The uncertainties associated with capacity curve properties, damage states and ground shaking are taken into account in these curves. FEMA [16] methodology has classified damage states into four different categories Slight, Moderate, Extensive and Complete. The results of time history analysis are used to derive time dependent fragility curves considering the corrosion effects.The fragility curves are expressed as the two- parameter time- variant log normal distribution function as described earlier.The damage state thresholds are defined as per Barbat and Pujades [17] and are shown in the Table 3. The fragility curves are plotted for uncorroded state at time t=0 years, at time t=60 years and at time t=100 years which are shown for different damage states in Figure 5 to Figure 8.

The values of yield and ultimate displacement; D_y and D_u are obtained from pushover curve using the method suggested by Elnashai and Di Sarno [18].

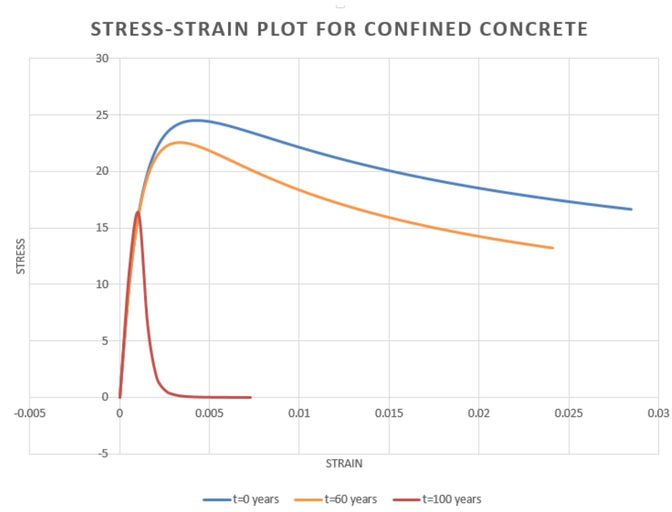

Figure 3: Stress-Strain plot for confined concrete at different time period

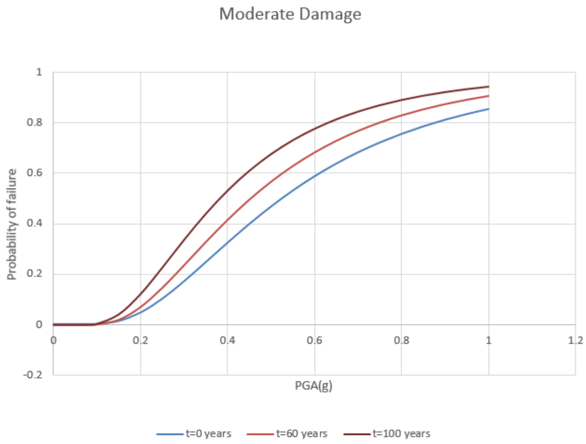

Figure 6: Fragility curve for Moderate Damage at different time period

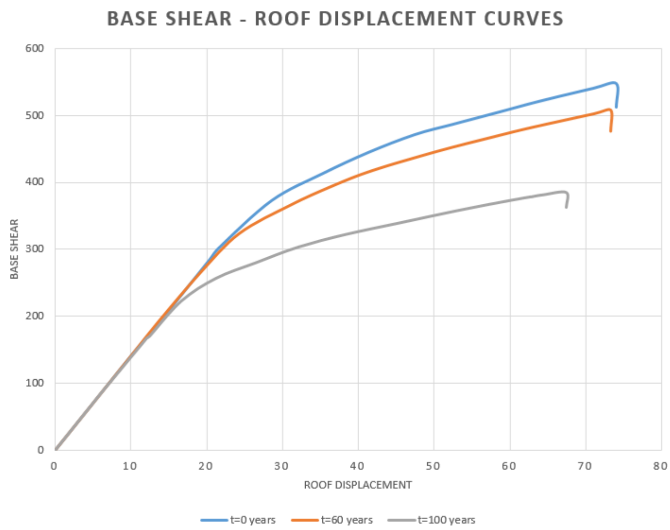

Figure 4: Pushover curves at different time period

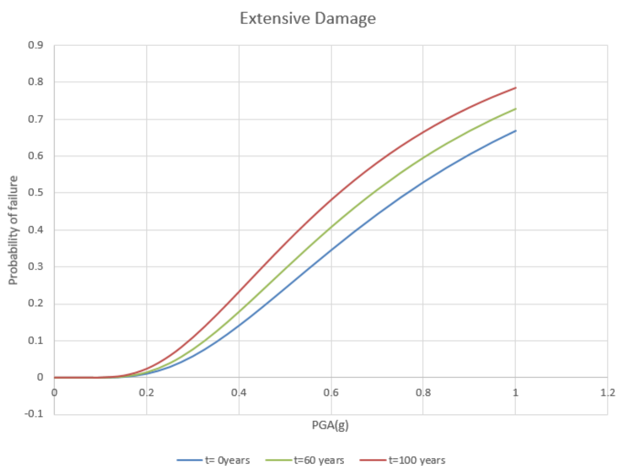

Figure 7: Fragility curve for Extensive Damage at different time period

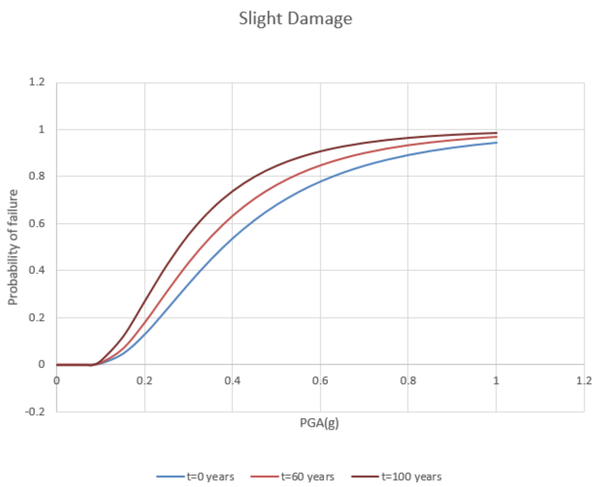

Figure 5: Fragility curve for Slight Damage at different time period

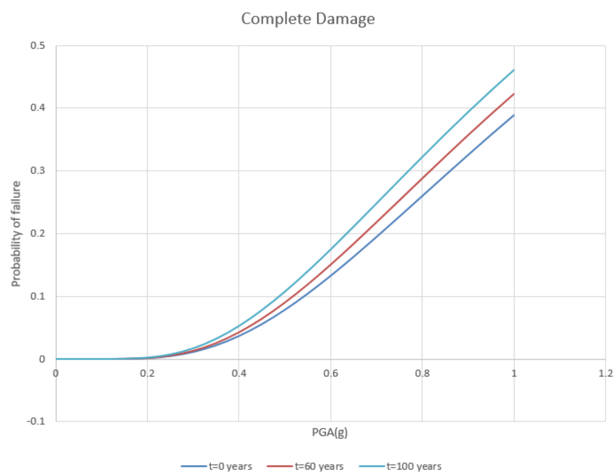

Figure 8: Fragility curve for Complete Damage at different time period

7. Result and Discussion

We can observe from the stress strain plot for confined concrete in Figure 3 that as the corrosion proceeds, the stress strain curve is becoming narrow indicating the transition from ductile failure to the brittle failure. Due to the reinforcement corrosion, it is reduced to rust. The rust occupies higher volume than the reinforcement itself so it exerts expansive stress on concrete that degrades the capacity of concrete. From the pushover curves in Figure 4, we can observe the effect of reinforcement corrosion on load carrying capacity of the building. The capacity of the structure is reduced by 8% after 60 years and by 29% after 100 years due to corrosion of reinforcements. The clear cover for beam reinforcements is less than for column reinforcements. So, the beam reinforcements start to corrode earlier than columns and initially, when the beam reinforcements corrode, there is not much difference in the seismic capacity of the building. But when column reinforcement start to corrode, the seismic capacity of the building starts to drop remarkably. This leads us to the understanding that the overall seismic capacity of the building is influenced much by the column than by the beam.

The fragility curves show the probability of exceedance of a certain damage state. From Figures 5 to 8, we can see that there is a growing trend in probability that the RC building exceed any damage states during the service life due to corrosion. The decrease in reinforcement area and the reduction in yield strength of the reinforcement bars reduces the moment capacity of the bars.The degradation of the properties of the constituent materials of structure decreases the stiffness characteristics of the building that leads to the increase in fragility and reduction in seismic capacity of the building. This underlines the importance of considering the effect of carbonation induced reinforcement corrosion on performing seismic fragility assessment.For the slight damage state, the exceedance probability at a PGA of 0.4g shows a increase of 18% after 60 years and 38% after 100 years, wheras the exceedance probability of moderate damage state exhibits a increase of 28% after 60 years and 66% after 100 years at PGA of 0.4g. For extensive damage state, the probability of exceedance at a PGA of 0.4g increases by 21% after 60 years and 64% after 100 years. For complete damage, the probability of exceedance at a PGA of 0.4g increases by 16% after 60 years and 45% after 100 years.

8. Conclusion

The effect of carbonation induced reinforcement corrosion on seismic capacity and vulnerability of RC building has been examined in this paper. A typical 2 storied residential building is selected to demonstrate the effect of corrosion. Based on the probabilistic seismic demand and seismic capacity estimates, the seismic fragility estimates of the structure is investigated. We can observe from the results that there is notable impact of carbonation-induced corrosion on the structural response when the seismic load is with a smaller PGA for slight and moderate damage states while there is significant difference on the structural response due to influence of corrosion when the seismic load is with a higher PGA in case of extensive and complete damage states.Thus indicating that the corrosion will have effect of increasing fragility of the structure in all ranges of seismic input. The pushover analysis have shown the decrease in seismic capacity by 29% at the end of 100 years due to corrosion.The fragility analysis of the structure has shown that the probability of exceeding a certain damage state increases due to corrosion which clearly highlights the importance of considering the effect of carbonation induced corrosion in performing seismic analysis of structures.

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