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Earthquake Behavior of Reinforced Concrete Frames Subjected to Rebar Corrosion

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ABSTRACT

The aim of this paper is to investigate the earthquake behaviour of reinforced concrete frames subjected to rebar corrosion. A typical two-bay, four-story reinforced concrete (RC) frame is designed. Two different rebar corrosion scenarios and a design spectrum are selected. The deteriorated condition in these scenarios are included which are loss in diameter of rebar, changes of mechanical properties of reinforcement steel bars, bond strength and changes in damage limits of concrete sections. The RC frame is evaluated using a nonlinear static analysis method in sound condition as well as deteriorated conditions. The rebar corrosion effect on the global level is investigated by comparing the responses of each scenario with respect to the response of sound condition of the frame. The result shows that the progressive deterioration of frames over time can cause serious reductions on the load-bearing capacity. Hence the overall seismic behaviour of the frame is adversely affected.

Keywords. Assessment; corrosion; earthquake; rebar; reinforced concrete.

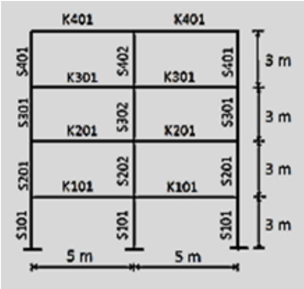
INTRODUCTION

Rebar corrosion on RC structures is a common problem. It results in considerable decreases on load-bearing capacity of structural systems. As the corrosion progresses, the corrosion products accumulate in interfacial transition zone (ITZ) and generate expansive pressure on the surrounding concrete. This internal pressure causes cracking in surrounding cover concrete and propagation of corrosion event. In addition to concrete cracking, chloride-induced rebar corrosion results in loss of the concrete–steel interface bond, and reduction of the cross-sectional area of reinforcement, thus reducing the load-carrying capacity of concrete structure. Rebar corrosion adversely affect the structural response such as decreases in load-bearing capacity and deformation of members. The degradation processes may be induced by diffusive attack of environmental aggressive agents. The most serious deterioration mechanisms are those leading to rebar corrosion and penetration of chloride ions. Such effects become of great concern for structures located in highly seismic zones, where the ductility properties and the actual collapse mechanisms are main issues in structural safety assessment. Rebar corrosion should be taken into account when RC structures are exposed to harmful attacks. The most important parameter responsible for degradation of concrete structures is the rebar corrosion. The main consequences of rebar corrosion are reduction in effective cross-section; reduction in concrete strength due to cracking and spalling; bond degradation; and ductility of reinforcement reduction (Rodriguez, 2002). Revathy, (2009) showed that increase in corrosion intensity is decreased the axial load bearing capacity and ductility of columns. Many other researchers (Almusallam, 1996; Mohammed, 2004) have also shown that the failure mode was changed ductile to fragile for structural elements or structural systems exposed to corrosion. Almusallam (2001) indicated a close relationship between the failure characteristics of steel bars and slabs with corroded reinforcement. A sudden failure of slabs was observed when the degree of rebar corrosion, expressed as per cent mass loss, exceeded 13%. Apostolopoulos (2008) concluded that an aged RC structure during its life span has accumulated damage in the load bearing elements from corrosion damage that suffered. This accumulated damage causes a degradation of the mechanical properties of the reinforcing steel bars. Ying, et al. (2012) show that higher corrosion levels and higher axial loads result in less stable hysteretic loops with more severe strength and stiffness degradations and worse ductility. One of the basic assumptions made in developing the RC theory is that perfect bond exists between concrete and steel bars. Bond strength is definitely a function of the concrete tensile strength. Insufficient bond can lead to a significant decrease in the load carrying capacity and stiffness of the structure (Shetty, 2011). Many researchers (Lee, 2002; Chung, 2008) proposed some equations relating corrosion level with bond strength. Chung, (2008) proposed a new bond strength equation from pull-out test results that correlates reinforcement mass loss with bond strength. They stated that the bond strength initially increases up to a maximum value, but eventually decreases for greater levels of corrosion. Auyeung, (2000) indicated experimentally that reduction in flexural capacity is occurred after 2% of diameter loss. Definition of an analysis method and tool is the first encountered difficulty in safety assessments of existing reinforced concrete structures (Berto, 2009). A generally accepted model that take into account all results of rebar corrosion could not been developed while there are some proposals considering bond losses due to rebar corrosion (Basheer, 1996). New models (Coronelli, 2004; Saetta, 2005) were developed in recent years which are considered deteriorations due to mechanical reasons as well as environmental agents. Nonlinear behaviour of the material is taken into account by spread plasticity approach in definite sections in these models. Degradation of material in spread plasticity models can be regarded as modifying primary equations a function of corrosion level in plastic hinges. The aim of this paper is investigation of the earthquake behaviour of RC frames subjected to rebar corrosion. The basic effects of rebar corrosion considered in this investigation are losses in diameter of steel bars, cracking of concrete, variations of mechanical properties of reinforcing steel bars, bond degradation, and force-deformation characteristics of the cross-section due to degradation.

NUMERICAL ANALYSIS

Structural model

A four-storey, two-span RC frame is designed to represent low-rise regular structures in practice. The requirements of the TEC-2007 are followed in design with peak ground acceleration of 0.4g. A class Z3 soil defined in TEC-2007 that soil is similar to class C soil described in FEMA-356. Its spectrum characteristic periods are T_A=0.15 s, and T_B=0.60 s. The typical frame and section details are shown in Fig. 1. Reinforcement layouts were selected identical in beams at all story levels. Material properties of sound system are assumed to be 25 MPa for the concrete compressive strength and 420 MPa for the yield strength of both longitudinal and transverse reinforcements. Cover concrete is assumed 30 mm for all beam and column sections. 13.5 kN/m dead load and 10.5 kN/m live load are applied on all beams. Earthquake loading was combined with gravity loading G+0.3Q, where G denotes dead loads and Q denotes live loads.



400x400 (mm x mm) Column sections S101, S201, S301, S401			600x300 (mm x mm) Column sections S102, S202, S302, S402	250x500 (mm x mm) Beam sections K101, K201, K301, K401
Long. Reinf.	8Φ16		12Φ16	At ends 5Φ14 (top) + 3Φ14 (bottom)
				At span 5Φ14 (bottom) + 3Φ14 (top)
Hoop, (Φmm/mm)	At span	Φ8/150	Φ8/150	Φ8/150
	At ends	Φ8/100	Φ8/100	Φ8/100

Figure 1. Frame labeling, and section details.

Corrosion scenarios

Five scenarios including sound state of the frame for comparison are considered for numerical analysis (Table 1). The propagation time of corrosion is assumed as 10 years for four scenarios with a constant intensity as shown in Table 1. The diameter of corroded reinforcing bars after 10 years is estimated from Eqn. 1. The conversion factor, κ is used as 0.0116 in this equation. Table 2 shows revised characteristics of concrete and rebar for each scenario with initial characteristics. There are some empirical (Du, 2005) and experimental (Lee, 2009) formulas developed for such a revision. The formulas given in reference (Lee, 2009) are used in this paper (Eqns.2-5). They are developed as a function of corrosion percentage after experimental researches. The corrosion percentage is calculated from the loss of the mass before and after corrosion attack.

Table 1. Corrosion scenarios

Scenario	Place	Corrosion level
Ref.	At all elements	No corrosion
S1	At all elements	Moderate, (i_{corr}=1 $\mu A/cm^2$)
S2	At all elements	High, (i_{corr}=5 $\mu A/cm^2$)
S3	At only bottom story elements	Moderate, (i_{corr}=1 $\mu A/cm^2$)
S4	At only bottom story elements	High, (i_{corr}=5 $\mu A/cm^2$)

Dhir (1994), Brite/Euram (1995), and Middleton and Hogg (1998) have classified corrosion rates according to corrosion current density values obtained from investigations on existing buildings and specimens produced in laboratory. This classification is used in this paper. The reduced diameter of reinforcing bar after propagation time is estimated with Eqn. 1. Although corrosion current density (i_{corr}) is not constant in reality, it is assumed constant in this equation.

$$\emptyset(t) = \emptyset_0 - 2P_x = \emptyset_0 - 2i_{corr}\kappa(t - t_{in}) \quad (1)$$

In Eqn. 1; $\phi(t)$ (mm) shows the diameter at time t, ϕ_0 (mm) is the nominal diameter, t_{in} (years) is the time for corrosion initiation at the rebar surface, i_{corr} ($\mu A/cm^2$) is corrosion current density, κ is a conversion factor of μA/cm^2 into mm/year for steel, and P_x (mm) is the average value of the attack penetration (Berto, 2008).

$$\sigma_{cy} = \left(1 - 1.98\left(\frac{\Delta_w}{100}\right)\right)\sigma_{sy} \tag{2}$$

$$\sigma_{ct} = \left(1 - 1.57\left(\frac{\Delta_w}{100}\right)\right)\sigma_{st} \tag{3}$$

$$E_{cs} = \left(1 - 1.15\left(\frac{\Delta_w}{100}\right)\right)E_{ss} \tag{4}$$

$$\delta_c = \left(1 - 2.59\left(\frac{\Delta_w}{100}\right)\right)\delta_s \tag{5}$$

Where σ_{cy} is the yield strength of steel after corrosion; Δ_w is corrosion percentage; σ_{sy} is the nominal yield strength of steel; σ_{ct} is the ultimate strength of steel after corrosion; σ_{st} is the nominal ultimate strength of steel; E_{cs} is elastic modulus of steel after corrosion; E_{ss} is the nominal elastic modulus of steel; δ_c is the elongation after corrosion; δ_s is the nominal elongation of steel. Bond strength is reduced by the cracking of concrete surrounding the reinforcement due to the swelling of the bars subsequent to the oxides formation. It is difficult to develop a general and reliable model for predicting the influence of corrosion on bond-slip behaviour (Berto, 2008).Therefore the probable bond strength loss is considered as a global decrease in concrete strength. Although different types of stress-strain relationship of concrete could be defined, the bond strength could not be directly defined in the software SAP2000. Since the strength of concrete is classified with respect to compressive strength the loss in bond strength could be introduce by decreasing compressive strength. Therefore, in order to introduce bond strength loss and cracks in concrete, the compressive strength of concrete is divided to 1.5 and 2.0 for moderate and high corrosion levels respectively.

Sectional analysis

The definition of the hinge properties requires moment-curvature (M–κ) analysis and the software XTRACT is used for this purpose. Constant axial forces due to G+0.3Q load combination applied on column sections in M–κ analyses. Axial forces are considered only for columns. Concrete strength, steel characteristic values, and loss in diameter of rebar are considered for each scenario as shown in Table 2. Then M–κ relations are transformed M/M_y-ϕ_p curves (Fig. 2). Therefore changes in strength and deformation capacity of structural elements with respect to rebar corrosion are used in pushover analysis as data.

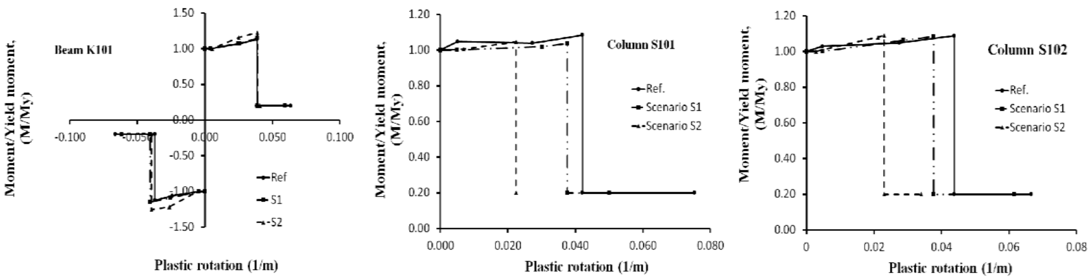


Figure 2. Moment-plastic rotation (ϕ_p) curves for sections; (a) K101; (b) S101; (c) S102

Table 2 Variables considered in all scenarios

Scenario	Variables								
	Concrete		Reinforcement steel					Transv. reinf. ratio	
	Mod. of El. E_s (MPa)	f_c (MPa)	Dia. (mm)	Yield str. (MPa)	Ult.str (MPa)	Mod. of El. E_s (MPa)	Elong. ($^0/_{00}$)	ρ_x (%)	ρ_y (%)
Ref	$3x10^5$	25	8	420	550	$2x10^5$	1	0.53	0.23
			14	420	550	$2x10^5$	1		
			16	420	550	$2x10^5$	1		
S1	$2.7x10^5$	16	7.77	373	501	$1.87x10^5$	0.852	0.50	0.22
			13.77	393	522	1.92x105	0.915		
			15.77	396	525	$1.93x10^5$	0.925		
S2	$2.5x10^5$	12	6.84	196	318	$1.38x10^5$	0.303	0.39	0.17
			12.84	288	413	$1.63x10^5$	0.589		
			14.84	304	429	$1.68x10^5$	0.638		

Pushover analysis

Displacement-controlled inelastic static pushover analysis is performed. The pushover procedure involves incremental application of a monotonic load until the control displacement is reached a pre-specified value or the frame collapses, whichever comes first. A concentrated plasticity approach is used with lumped hinges assigned at the ends of the beams and the columns. The analyses up to the formation of plastic hinges mechanisms is continued because of all brittle failure modes such as shear failures are prevented in sound state. The reduced stiffness values introduced in the analysis EI_e, having taken into account the effect of axial loading on the degree of cracking. The M–κ relationship is converted to moment-rotation (M-ϕ) relationship by assuming the plastic hinge length is one half of the section depth. An inverse triangle lateral load pattern is selected resembling probable distribution of earthquake loads determined according to TEC-2007. The deformation capacity limits in terms of strain values associated with different performance levels for beam and column sections defined in TEC-2007 is used. Because of these performance limits are given in terms of strain of concrete and steel its corresponding curvature values are read from the results of sectional analysis. Plastic hinges are assigned at top and bottom ends for every column, and assigned at left and right ends of every beam. Two load cases which are gravity loads (G+0.3Q) and lateral pushover loads cases run in turn. Lateral pushover load case started from the final conditions of gravity load case. Base shear versus top displacement curve of the frame is obtained. Target displacement is determined by Eqn. (6).

$$U_{XN1}^{(p)} = \Phi_{XN1}\Gamma_{X1}d_1^p \tag{6}$$

Where $U^{(p)}_{XN1}$ is target displacement; $\Phi_{XN1}$is modal amplitude at the roof level of frame for the first mode in X-direction; Γ_{x1} is participation factor in x-direction for the first mode shape; d_1^p is modal displacement demand for the first mode. After determination of target displacement than a second run is performed up to the target displacement is attained.

RESULTS AND DISCUSSION

Yield stress and deformation capacity are decreased as corrosion level increased and rebar diameter decreased (Table 3). Therefore a general decrease is observed in bearing capacity of RC members and structural systems generated from these members. Also, deformation capacity is limited in addition to decrease in load bearing capacity. An immediate failure as well as in S4 (Fig. 3) could be observed because of suddenly breakage of corroded rebar. Corrosion has adverse effects on material characteristics such as modulus of elasticity, elongation capacity of reinforcement steel. When these effects and loss in rebar diameter with bond losses is come together, then performance of the frame under earthquake loads is scaled down. Loss in bond strength affects the flexural stress transfer between the concrete and the reinforcing bars, which reduces the flexural strength of that member.

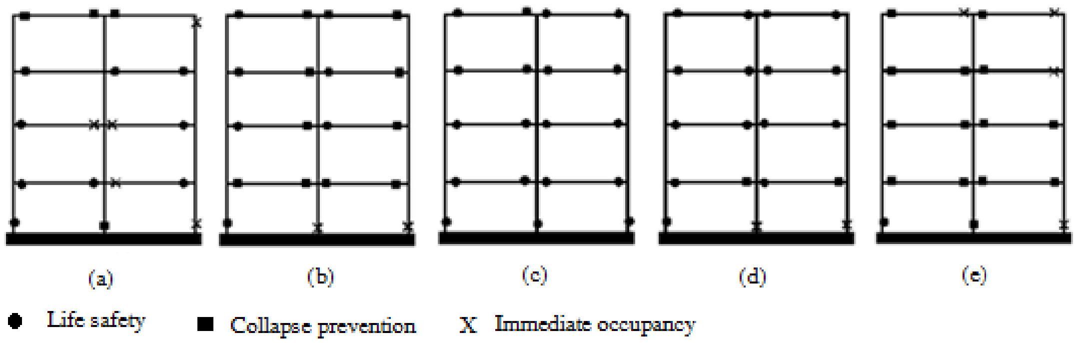


Figure 3. Plastic hinge formation when the first collapse observed on column base; (a) Reference, (b) S1; (c) S2; (d) S3; (e) S4.

Response limits fall into two categories which are global structural acceptability limits and element and/or component acceptability limits. Since accuracy of component acceptability limits plays a critical role it significantly affects the assessment result of structural performance. Element and component acceptability limit variations are derived from sectional analysis. Bending moment versus ϕ_p of plastic hinges is shown in Fig. 2. When corrosion deteriorations such as loss of rebar section, change of mechanical properties of reinforcement steel, crack in concrete is considered in sectional analysis M/M_y-ϕ_p curves shows different characteristics. It shows high ϕ_p capacity in the sound system. Also, after it loses moment capacity in high range (at the time M/M_y=0.20), it has a residual rotation capacity with nearly constant M/M_y ratio. However for the corroded systems, this typical behaviour changes according to corrosion level. For low level corrosion (S1), only ϕ_p capacity and residual rotation capacity are decreased, but the general shape of the curve is not changed. However for the high level corrosion (S2), ϕ_p capacity is decreased abundantly and residual capacity is not existed or is very small. This is an important result for global behaviour of the frame system. It was known that sectional deformation capacity of beams and column sections have a direct effect on global deformation capacity. The explained deformation capacity variations are observed especially in column sections that have axial load as distinct from beams. The main reason of decrease in ϕ_p capacity decreases is coming from the influence of local deterioration of corrosion. Loss of cover concrete and rebar cross sectional area, change of reinforcement steel characteristics, bond deterioration, and corrosion products that results in cracks which causes loss in strength are basic parameters of corrosion effect. Deformation limits of performance levels are also reduced depending on decreases in ϕ_p capacity of the corroded section. This means that performance level ranges are shortened. M-ϕ curves for scenarios reference and S1 is similar to Type 1 curve that is defined in FEMA-273 document which curves represent ductile behaviour. However M-ϕ curve of S2 corresponding high corrosion damage is similar to Type 2 curve representing another type of ductile behaviour. This curve is characterized by an elastic range and a plastic range, followed by a rapid and complete loss of strength.

Capacity curve of each scenario is compared with reference system as shown in Fig. 4. For the S1 including low corrosion rate, it can be observed that the difference of capacity curve in terms of base shear and top displacement is small as compared with the reference system. However there is large difference (38%) between S2 and reference system in terms of base shear capacity because of high corrosion rate and prevalence of corrosion on the frame. The S4 is also attracts attention with its poor behaviour. This scenario contains local corrosion that is corrosion present only in ground floor. The frame system shows low base shear and low top displacement. Both of S3 and S4 scenarios shows column mechanisms failure mode due to accumulation of destructive corrosion effects on the ground floor column and beams. On the other hand, failure mode of S1 and S2 scenarios that contains general corrosion overall the system are similar to failure mode of reference system which mode is beam mechanisms. As a result, failure mechanism is affected from corrosion effects where local corrosion (corrosion defects are accumulated in one story) is more dangerous than prevalent corrosion. Compatible results are available in literature. Berto et al. (2008) observed loss of ductility and tendency to the reduction of the load bearing capacity as the corrosion level increases. They concluded that the results are valid especially when the corrosive attack is concentrated at the basis of the columns. Analytical and experimental results given by Coronelli (2004) shows that corrosion affects both the strength and the ductility of a structure at ultimate. The bond conditions should be carefully assessed and modelled to predict the ductility of the structural element. The ductility is intimately associated with the failure mode and with the ultimate capacity.

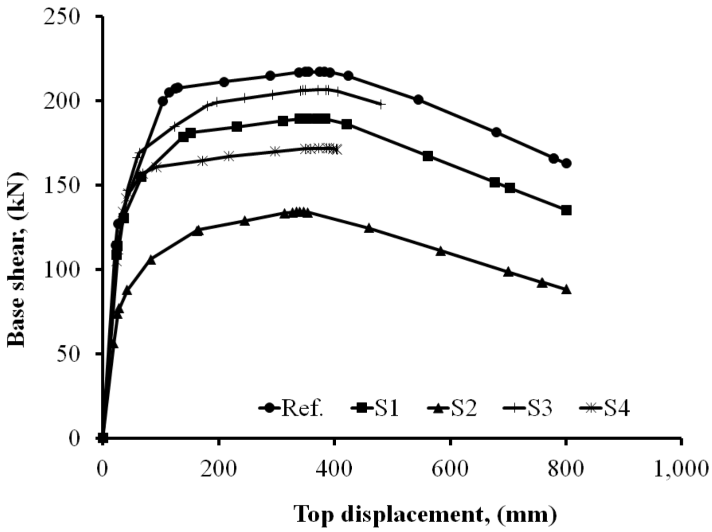


Figure 4. Capacity curves of the frame system

Column plastic hinges affect the system behaviour more than beam plastic hinges. Plastic hinges must be formed at beam ends at first in order to dissipate more energy with beam mechanisms. As can be seen from Fig. 3 the reference system showed the expected behaviour from earthquake-resistant frames. As corrosion damage is increased, variations in the order of plastic hinge formation are observed. When compared with the sound system, less plastic deformation is observed (Fig. 3.b) on beam plastic hinges till the collapse limit is attained on the base of column S101 in S1 scenario. The case of scenario S2 is shown in Fig.3.c has very different behaviour from the other scenarios. A simultaneous exceeding of collapse limit at the base of all columns in the ground floor is observed. This generation produces a sudden failure. The S4 also shows less ductile behaviour because of exceeding of collapse limit at column plastic hinge before the exceeding immediate occupancy limit at three beam plastic hinges. This means that there are unused deformation capacities at beam plastic hinges before the failure of the system (Fig.3.e). Fig.3.d also shows similar behaviour of S3 that includes moderate level corrosion at only the ground floor. However corrosion effect on the system behaviour is less as compared to S4. System ductility is decreased with corrosion similar to the behaviour of irregular frames in elevation.

Relative inter-storey drift corresponding target displacements are shown in Fig. 5. A general increase trend is observed in scenarios including corrosion damages with respect to reference system. The maximum inter-storey drift is appeared in the first floor in S4 when compared to other scenarios and the sound state. This is because of accumulation of corrosion damages in the first storey columns and beams. Hence inter-storey drift in this story is increased. The inter-storey drifts in S2, which corresponding prevalent high level corrosion damage on the frame is higher than that of sound state. Multifaceted influence of corrosion damages caused such a result. Because of loss of rebar cross-section, cracks in concrete, bond deterioration, and loss in mechanical properties of reinforcement steel results in deteriorations in component and on the whole structural system.

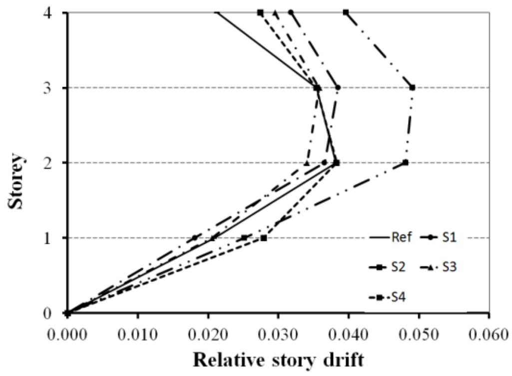


Figure 5. Relative drifts displacement

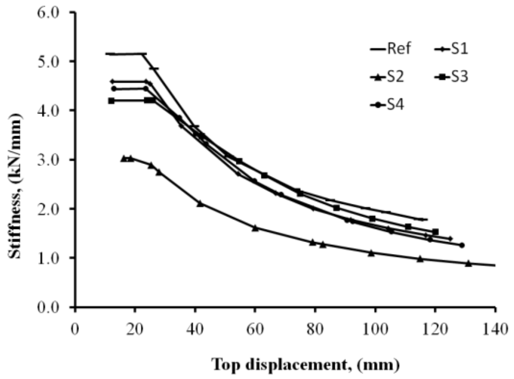


Figure 6. Stiffness degradation

The ratio of the lateral load to the top displacement is denoted as the secant stiffness. Fig. 6 shows the degradation of secant stiffness of frames versus the lateral top displacement. The stiffness of frame in S2 differs dramatically with respect to other scenarios. It has 32% less initial stiffness and the rate of degradation slows down. Zhong et al. (2010) states corrosion-induced cracks in RC structures degrade the stiffness of the concrete. It is mainly caused by the softening in the stress-strain relation in the cracked concrete. The reason of less initial stiffness in S2 is prevalent corrosion damages on the frame. The S4 having maximum inter-storey drifts does not differ in terms of stiffness degradation when compared to sound state of frame. Excessive plastic rotations at bottom ends of the ground floor columns in S4 with respect to other scenarios are observed for the target top displacement of the frame. Maximum ϕ_p at bottom end of column S101in S4 for the target top displacement is 3.64 times greater than that of in reference scenario. The same parameter is 5.83 for the column S102. No plastic rotations are observed at the same locations of columns S101 and S102 in the other scenarios S1, S2, and S3. Since the failure mode of structural systems is depending on the stiffness of its comprising members, stiffness degradation is important for structural systems. 26% damage level is realized in scenario S2 in terms of the cross-sectional area reduction. As a result major behaviour differences are observed between the S2 and the reference. Stiffness degradation, changes in M- ϕ_p relationships and capacity curves, strength deterioration, differences in plastic hinge formation sequence and some other indicators shows corrosion could change overall structural behaviour. This is compatible with findings of some researchers. Gonzales et al. (1996) observed that 25% damage in terms of the cross-sectional area reduction of reinforcement bars seems to be very significant in corrosion-damaged RC structures. Amey et al. (1998) predicts 30% of losses in rebar area as the failure criterion.

CONCLUSIONS

Rebar corrosion has considerable effects on global structural acceptability limits and component acceptability limits. Material and sectional properties, member and structural stiffness, component performance limits, bond strength, global deformation characteristics are basically affected parameters. Time, intensity, and extensity of corrosion on the frame affect structural behavior. Especially, high level and extensive corrosion can cause unfavorable results. Corrosion that propagates on the whole frame decreases structural performance in terms of strength and ductility and may change the failure mode of the structure. The effect is comparative with the corrosion rate and the time of propagation. The accumulated corrosion damages in one story such as in the ground floor is also effective on the structural behavior. The sequence of plastic hinge formation process and the rotational capacity utilization at plastic hinges are affected the degree and position of corrosion on the structure. The real level of structural health due to possible rebar corrosion of RC structures should be a taken into account in seismic evaluations. Further studies are required in order to represent corrosion deteriorations such as bond deterioration on RC members. Also, more research studies are necessary about slippage of reinforcement bars.

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