Earthquakes and Structures, *Vol. 5, No. 3 (2013) 321-341*
DOI: http://dx.doi.org/10.12989/eas.2013.5.3.321

Earthquake response of reinforced concrete frame structures subjected to rebar corrosion

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(Received March 6, 2013, Revised April 30, 2013, Accepted May 31, 2013)

Abstract. This paper investigates earthquake response of reinforced concrete regular frames subjected to rebar corrosion. A typical four-story reinforced concrete frame is designed according to Turkish Earthquake Code in order to examine earthquake response. Then different levels of rebar corrosion scenarios are applied to this frame structure. The deteriorated conditions as a result of these scenarios are included loss in cross sectional area of rebar, loss of mechanical properties of rebar, loss in bond strength and variations in damage limits of concrete sections. The frame is evaluated using a nonlinear static analysis in its sound as well as deteriorated conditions. The rebar corrosion effect on the structural response is investigated by comparing the response of the frame in each scenario with respect to the sound condition of the frame. The results shows that the progressive deterioration of the frame over time cause serious reductions on the base shear and top displacement capacity and also structural ductility of the corroded frames. The propagation time, intensity, and extensity of rebar corrosion on the frame are important parameters governing the effect of rebar corrosion on earthquake response of the frame.

Keywords: earthquake; frame; rebar corrosion; reinforced concrete, response

1. Introduction

Reinforced Concrete (RC) moment resisting frames are commonly used in Turkey as well as in some other countries as a structural system in low to medium rise buildings. Structural performance and serviceability assessment of RC frames should account for the time-dependent variation of structural response due to degradation phenomena. In high seismic regions, such as Turkey, rebar corrosion and concrete deterioration over time may weaken structures and make them more vulnerable to future earthquake hazards. Rebar corrosion adversely affect the structural response such as decreases in load bearing capacity of members due to loss in diameter of reinforcement steel, deterioration of bond between rebar and surrounding concrete and anchorage of reinforcement steel bars.The degradation processes may be induced by diffusive attack of environmental aggressive agents. The most serious deterioration mechanism is that penetration of chloride ions into concrete leading to rebar corrosion. Such effects become of great concern for structures located in highly seismic zones, where the ductility properties and the actual collapse

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http://www.techno-press.org/?journal=eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

mechanisms are main issues in structural safety assessment. Structural rehabilitation may be required in many times due to corrosion damages in buildings such as hospitals, public buildings, hotels, houses etc. before or after earthquakes. Also, the repairing and rehabilitation of structures needs considerable supplies (Stanish *et al.* 1999). 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 strength of concrete due to cracking and spalling; bond degradation; and ductility of reinforcement reduction (Rodriquez *et al.* 2002). Mechanical characteristics of concrete and reinforcing steel are important factors in static rehabilitation of existing buildings.

The aim of this paper is investigation of rebar corrosion effect on the earthquake response of RC frames. The reduction in cross-sectional area, cracking and spalling of cover concrete, variations of mechanical properties of reinforcing steel, bond degradation, and force-deformation characteristics of the cross section due to degradation are selected as basic variables. Sectional analysis are performed on corroded and sound state of potential plastic hinge sections as well as structural performance assessments are performed on corroded and sound state of the frame system. Different probable corrosion scenarios are included to represent corroded structural systems in practice. Structural performance of corroded systems is compared with respect to sound state structural behavior. Little work has been done about the effects of corrosion considering main consequences of rebar corrosion. This paper also includes sectional damage limit variations due to rebar corrosion in addition to basic variables mentioned above.

2. Rebar corrosion in reinforced concrete structures

Rebar corrosion is an electro-chemical process where the reinforcing bars are depassivated at first. Two stages of rebar corrosion could be considered. The first stage is called initiation phase involving transfer of aggressive agents. The second stage is called propagation phase and leads to damage in surrounding concrete as well as sectional reduction of rebar. When the concrete cover starts to spalling the corrosion is propagated. Durability of RC structures is widely affected from rebar corrosion. The propagated corrosion causes concrete cracks in longitudinal and tangential directions and these cracks may accelerate further corrosion, and subsequently structural performance will be deteriorated continuously. Concrete cover should prevent reinforcing bars in RC structural members. When cover concrete could not prevent reinforcing bars then the expected response from both of the structural system and its members would not be realized during an earthquake. Post-earthquake field observations in Turkey have shown that concrete strength was too low as compared with the needed strength. Such a concrete could not prevent itself and embedded reinforcement bars in it against aggressive environmental attacks. Structural members and systems generated from these members showed fragile behavior and failed easily.

Concrete is exposed to various aggressive environmental attacks and the environment is the leading factor to cause corrosion in RC structures. The concrete initially prevents corrosion by creating a passive thin oxide layer on the reinforcing bars due to the alkaline conditions. However, Corrosion may begin if this passive coating on the steel is lost by depassivation, due to carbonation or diffusion of chloride ions in pore water and reaching to the bars or both of them. The corrosion process transforms reinforcement bars into rust, leading to cross sectional area reduction and ductility change of the reinforcement bars, and volume expansion which is responsible from cracking and spalling the concrete cover. The ductility of a corroded bar depends on exposure environments, i.e. carbonation or chlorides (Hanjari 2010). Moreover, corrosion products weaken the bond which is very important for load transfer between rebar and the surrounding concrete. Tuutti's (1982) model is a widely accepted model for the deterioration of concrete structures. Deterioration could be distinguished into two stages as a function of time in this model. The first stage is called initiation phase involving transfer of aggressive agents. An important deterioration is not observed in this phase. The second stage is called propagation phase and leads to damage in surrounding concrete as well as sectional reduction of rebar. The propagated rebar corrosion causes concrete cracks in longitudinal and tangential directions of rebar and these cracks may accelerate further corrosion, and subsequently structural performance will be degenerated continuously. When cover concrete could not prevent reinforcing bars then the expected response from both of the structural system and its members would not be realized during an earthquake. Post-earthquake field observations in Turkey have shown that concrete strength was too low as compared with the needed strength. Such a concrete could not prevent itself and embedded reinforcement bars in it against aggressive environmental attacks. A structural system generated from these members shows fragile behavior and fails easily.

Revathy *et al.* (2009) studied corrosion effects on structural members and they showed that increase in corrosion intensity decreased the axial load capacity and ductility of columns. Many other researchers (Almusallam *et al.* 1996, Rodriquez *et al.* 1997, Mangat *et al.* 1999, Mohammed *et al.* 2004) have also shown that the failure mode was changed ductile to fragile for structural elements and systems exposed to types of corrosion. Pitting corrosion can lead to decrease in ductility of rebar. Palsson and Mirza (2002) have shown that corroded bars retrieved from an abandoned bridge demonstrated brittle failure in tension tests. Allam *et al.* (1994) and Almusallam (2001) evaluated the effect of corrosion on the mechanical properties of steel bars. Although Allam *et al.* (1994) showed that, rusting had an insignificant effect on the yield and ultimate tensile strength of the steel bars, 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 percent mass loss, exceeded 13%. Apostolopoulos and Papadakis (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. Important changes in seismic fragility along the service life of RC bridges are determined by Ghosh and Padgett (2012) due to corrosion deterioration. They concluded that other components of bridges are also shows rapid decreases in fragility.

Deterioration of concrete, loss of rebar cross-sectional area, and cracks in concrete results in considerable decreases on load bearing capacity of structural systems. The corrosion products with relatively lower density occupy much more volume than the original iron. 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 bond strength, and reduction of the cross-sectional area of reinforcement, thus reducing the load carrying capacity of concrete structure (Mehta 1997). Considerable decreases in the bond at the steel-concrete interface can be potentially the most dangerous for the safety of structures. In addition, the corrosion products developed at the reinforcing steel surface lead to a considerable decrease in the bond at the steel-concrete interface-a situation which can be potentially more dangerous to the safety of the structural element than the loss of the rebar cross-sectional area (Amleh and Mirza 1999).

One of the basic assumptions made in developing the RC theory is that perfect bond exists between concrete and steel bars. This assumption ensures compatibility of deformations of both concrete and reinforcement. If this perfect bond is break down by corrosion then the RC member globally acts like a tied arch instead of behaving a composite member. For a bar subjected to tension in a RC member, the basic requirement is that it should not be pulled out of concrete. The resistance to pull should be high enough to carry the force, which will cause the steel to yield. It is very important to understand the bond deterioration mechanism and to estimate the bond strength and the residual load carrying capacity of corroded RC beams (Wu 2012). Bond strength is definitely a function of the concrete tensile strength. Bond is influenced by many variables and its interaction with shear influences the behaviour significantly. The reduction of rebar cross sectional area and the simultaneous rust swelling induce a more or less significant decrease of the bonding between reinforcement steel and concrete. Insufficient bond can lead to a significant decrease in the load carrying capacity and stiffness of the structure (Shetty *et al.* 2011). Many researchers (Cabrera 1996, Lee *et al.* 2002, Chung *et al.* 2008) have investigations about the relation of corrosion level and bond behavior. They proposed some equations relating corrosion level with bond strength. Chung *et al.* (2008) proposed a new bond strength equation from pullout 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 *et al.* (2000) found loss of bond to be very critical. The experimental results indicated that after 2% of diameter loss there is a reduction in flexural capacity. Structural behaviour of elements such as columns or beams exposed to propagated corrosion is similar to plain concrete because of bond strength is almost wholly disappeared. Al-Sulaimani *et al.* (1990) showed that the effect of bond strength is relatively substantial when no concrete confinement is present. However, work performed by Ghandehari *et al.* (2000) shows that the effect of corrosion on bond strength is negligible when a high percentage of confining transverse steel is used. Some researchers such as Berra *et al.* (2003) and Lundgren (2005) have developed finite element modeling of the region around all of the rebar to investigate the bond mechanism for rebar corrosion effect. However, this type of three-dimensional modeling is not practical for the analysis of complete two or three dimensional frame structures.

There are very few models for flexural capacity of corroded RC beams or columns. Also they have own limitations and drawbacks that the results are highly dependent on the specific structures considered. Therefore it is very important to propose analytical models for bond strength and flexural strength calculation. Wu (2012) proposed an analytical model which is developed to predict the residual flexural capacity of corroded RC members. He concluded that the reduction of the bond between steel and concrete is the main factor of the mechanical degradation for flexural capacity.

The comprehension of giving precedence to durability more than strength is increasingly become crucial in design of RC structures. It is necessary to establish advanced design methodologies and design codes that can accommodate durability in addition to safety and serviceability concepts in the realistic analysis and design of concrete structures. Durability of a structural system is defined as the ability of resisting environmental attacks without decreasing structural performance under an acceptable limit. Concrete durability can be defined as three basic factors which are mix design, structural design, and effects of construction site, cure conditions, and environmental conditions. Degradation process of concrete over time is a consequence of the chemical, biological, physical and environmental attacks that the structure may suffer during its service life. Concrete cracks at the end of this process. The reason of these cracks is exceeding the tensional strain capacity of concrete. Therefore rebar corrosion is the most dangerous durability problem for RC structures. Therefore rebar corrosion could be accepted as the basic cause for durability problem in RC structures.

Definition of an analysis method and tool is the first encountered difficulty in safety assessments of existing reinforced concrete structures (Berto *et al.* 2009). Many finite element methods were developed in last decades for this purpose. The model used in the analysis should encounter deteriorations and change of stress distributions occurred in a time series on the structure. 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 *et al.* 1996). Nevertheless, new models (Coronelli and Gambarova 2004, Wang and Liu 2004, Seatta 2005) were developed in recent years by finite element methods 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. Two different methods can be used to define such a rule. The first method referred finite element analysis results by using effects of chemical and mechanical damages on nonlinear behaviour according as different corrosion levels. The second method is based on a macro level evaluation. Theoretically and experimentally developed special moment-curvature relations are used for each plastic hinge according to corrosion levels of rebar.

In order to evaluate loss of cross sectional area due to corrosion, it should be measured the depth of the corrosive attack penetration. When corrosion current density icorr (in A/cm^2) is measured then corrosion rate (mm/year) is determined. The reduced diameter of reinforcing bar after propagation time can be estimated with Eq. (1) for carbonation corrosion. Although corrosion current density (i_{corr}) is not constant in reality, it is assumed constant in Eq. (1).

$$\Phi(t) = \Phi_0 - 2P_x = \Phi_0 - 2i_{corr}\, k(t\text{-}t_{in}) \tag{1}$$

In Eq. (1); (t) (mm) shows the diameter at time t,$_0$ (mm) is the nominal diameter, t_{in} (years) is the time for corrosion initiation at the rebar surface, i_{corr} (µA/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 *et al.* 2008). The corrosion initiation time depends on the cover thickness and on the penetration rate of carbonation (Seatta 2005, Seatta *et al.* 1993, Seatta *et al.* 1999). Seatta and Vitalini (2004) reported that CO_2 concentration in the air is the most important effect on the carbonation rate. The other important factor for carbonation is the w/c ratio. As the w/c ratio increases the probability of carbonation increases. Dhir *et al.* (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 could be used for practical applications (Table 1).

3. Structural performance

Table 1 Probable initiation times for corrosion

Corrosion level	Current density, i_{corr} ($\mu A/cm^2$) Dhir *et al*. (1994)	BRITE/EURAM (1995)	Middleton and Hogg (1998)
Negligible	-	<0.1	-
Low	0.1	0.1-0.5	0.1-0.2
Moderate	1	0.5-1.0	0.2-1.0
High	10	>1.0	>1.0

Seismic performance of a structure is ability to sustain its main functions, such as its safety and serviceability, at and after a particular earthquake effect. According to basic concepts of the earthquake engineering, a building should survive rare, very severe earthquakes by sustaining significant damage but without globally collapsing. Also, it should remain operational for more frequent, but less severe earthquakes. The means of performance in earthquake engineering is providing predefined criteria that are related to displacements or other deformation terms. A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. Building performance levels are a combination of structural performance level and a nonstructural performance level. A properly engineered building does not necessarily have to be extremely strong or expensive. It has to be properly designed to withstand the seismic effects while sustaining an acceptable level of damage during service life.

The objective of performance based earthquake engineering is to achieve desired performance targets. The structure will show desired response during a particular earthquake in this manner (Xue and Chen 2003). The displacement demand of the structure is include post-elastic components. Therefore performance point of the structure must be determined by taking into consideration post-elastic behavior. Fundamental parameters of performance based analysis are capacity and demand concepts. In nonlinear static procedure, the basic demand and capacity parameter for the analysis is the lateral displacement of the building. The generation of a capacity curve defines the capacity of the building uniquely for an assumed force distribution and displacement pattern. It is independent of any specific seismic shaking. A point on the curve defines a specific damage state for the structure, since the deformation for all components can be related to the global displacement of the structure. By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found on the capacity curve that estimates the maximum displacement of the building the earthquake will cause. This defines the performance point or target displacement. The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met (Naeim 1996).

The seismic performance of a structure is dependent upon the performance characteristics of its critical components. The critical components are those that are necessary for vertical stability and those that comprise the seismic load path. Concrete frame construction has several potential failure modes that directly threaten the structure’s ability to sustain vertical loads and maintain stable lateral behavior. The largest concern is a brittle column failure mode caused by shear failure or compression crushing of the concrete. Also, localized concentrations of drift due to soft or weak story configurations are of serious concern (ATC-40, 1996).

4. Numerical example

In this section, assessment of seismic response evaluation methodology is applied to a four-storey, two-span RC moment resisting frame for different corrosion scenarios and in sound condition. It is considered as representing low-rise regular structures and in this respect it is selected as regular in elevation.

4.1 Structural model

The frame is designed according to the requirements of the Turkish Earthquake-resistant Code of buildings (TEC-2007) and Requirements for design and construction of reinforced concrete structures (TBC-500) with design peak ground acceleration of 0.4g. A class Z3 soil similar to class C soil of FEMA-356 (FEMA 2000) defined in TEC-2007 is used, which corresponds to a medium stiff sand-gravel site having an equivalent shear wave velocity of 200–400m/s and a site soil layer thickness is between 15m and 50m. Its spectrum characteristic periods are T_A=0.15s, and T_B=0.60s. The story height was 3m, and the width of each bay was 5m. The typical frame is shown in Fig. 1, while Fig. 2 provides the typical sections of structural elements with the reinforcement details. Reinforcement layouts were identical in beams at all stories. 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 30mm for all beam and column sections. The dead load on all of the beams is 13.5kN/m, while the live load is 10.5kN/m. Column dead loads are applied as nodal loads on each joints. Earthquake loading was combined with gravity loading G+0.3Q, where G denotes dead loads and Q denotes live loads. The structural model is fixed at the base nodal points, while other nodes are free with respect to rotation and translation. The first mode period is 0.61s, and mass participation factor is determined as 0.831.

4.2 Corrosion scenarios

Four different corrosion scenarios are provided for numerical analysis (Table 2). While scenarios S1 and S2 are applied on whole building, S3 and S4 are applied to only ground floor columns and beams. Also sound state of the frame (S0) is considered as for comparison of results. The elapsed time since the start of propagation of corrosion is assumed as 10 years for all scenarios with a constant corrosion intensity as shown in Table 2. The diameter of corroded reinforcing bars after 10 years is estimated from Eq. (1). The conversion factor, κ is used as 0.0116 in Eq. (1). Table 3 shows revised characteristics of concrete and rebar for each scenario with initial characteristics. There are some empirical (Du *et al.* 2005) and experimental (Lee and Cho 2009, Apostolopoulos *et al.* 2013) formulas developed for changes in yield strength, ductility, and ultimate strength of reinforcement steel. The formulas given by Lee and Cho (2009) as a function of corrosion percentage developed after experimental researches are used in this study and they are shown in Eqs. (2-5). The corrosion percentage is calculated from the loss of the mass before and after corrosion.

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

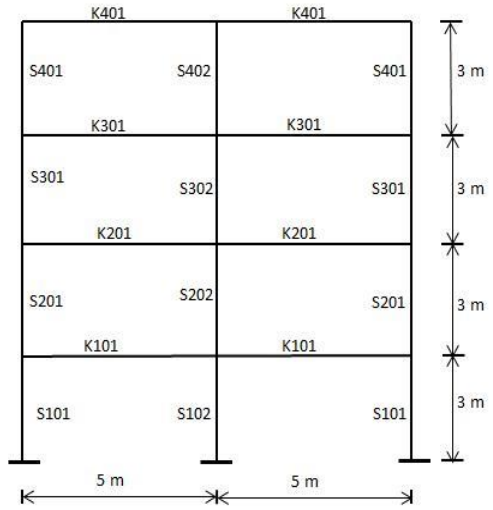


Fig. 1 Frame labeling and dimensions

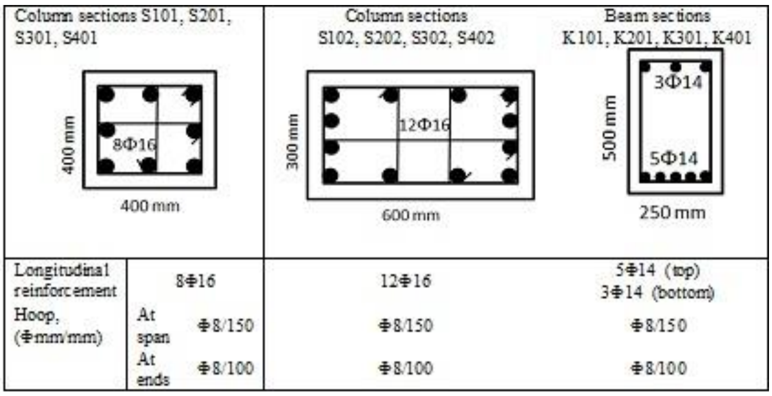


Fig. 2 Column and beam sections, and rebar layout

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

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

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

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.

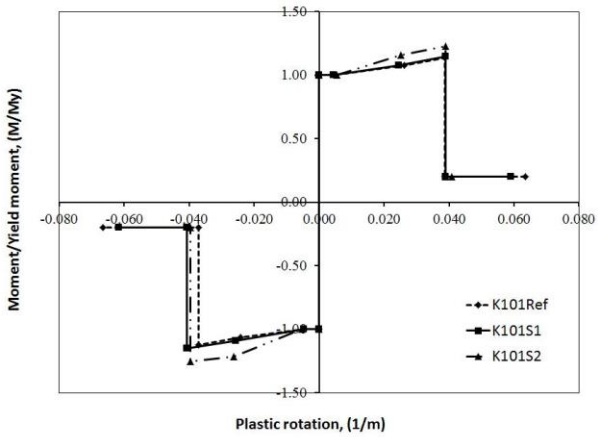
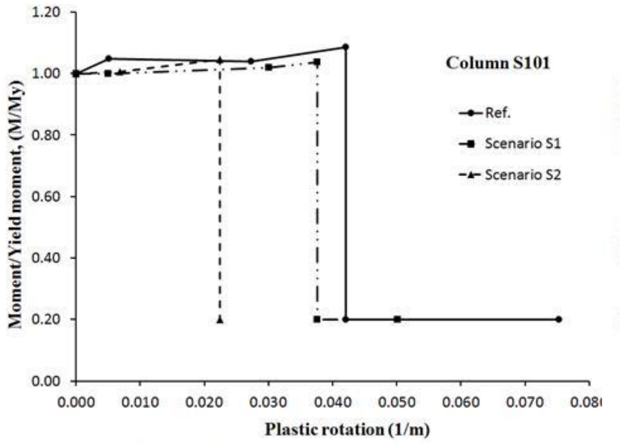
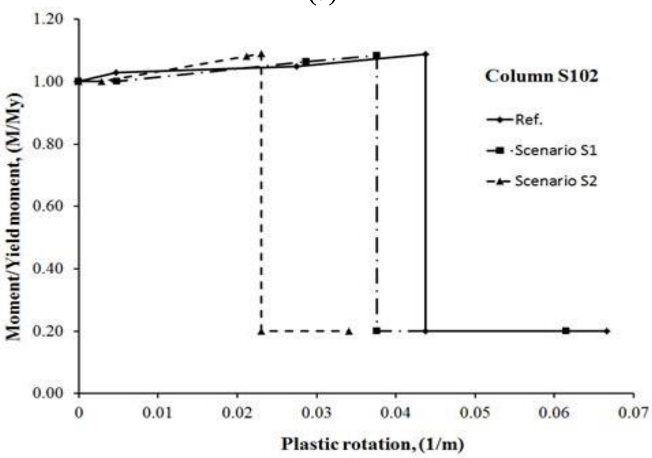


Fig. 3 Moment-plastic rotation curve for beam sections



(a)



(b)

Fig. 4 Moment-plastic rotation curves of column sections: (a) S101 and (b) S102

Table 3 shows transverse reinforcement ratios for scenarios.

It was well known that one of the main effects of the rebar corrosion is the degradation of the bond strength. It is difficult to develop a general and reliable model for predicting the influence of corrosion on bond-slip behavior (Berto *et al.* 2008). Moreover there is not any globally accepted model which introduced general-purpose structural analysis programs in order to take into consideration the bond strength. Therefore the probable bond strength loss is considered with 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. This concrete strength decreasing is also for concrete cracks developed in longitudinal and tangential directions of rebar in case of high level corrosion. Therefore, in order to introduce bond strength loss and probable corrosion cracks in concrete, the compressive strength of concrete is divided to 1.5 and 2.0 for moderate and high corrosion levels respectively.

4.3 Sectional analysis

Idealized moment-rotation diagram for SAP2000 (CSI 2000) input is obtained from moment-curvature analysis. The software XTRACT (Imbsen 2001) is used to develop a moment-curvature relationship for the sections where the plastic hinges will occur. The definition of the hinge properties requires moment–curvature (M–κ) analysis of each element. Mander's law (Mander *et al.* 1988) for concrete accounting for the confinement level and an elasto-plastic law shown in Appendix-7B of TEC-2007 for reinforcing steel are used in M–κ analysis. Constant axial forces due to G+0.3Q load combination applied on column sections in M–κ analyses. Axial forces are ignored for M–κ analysis of beam sections. Moment-curvature relations are transformed moment to yield moment versus plastic rotation curves for every section and scenario (see Figs. 3 and 4). Therefore changes in strength and deformation capacity of beams and column sections with respect to corrosion are considered in the analysis and these data are used in pushover analysis. Acceptance criteria are also shown in Figs. 3 and 4.

4.4 Pushover analysis

Displacement-controlled inelastic static pushover analysis is conducted considering gravitational and seismic loads defined according to TEC-2007 with the aid of the well-known SAP2000 computer code. 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. Infill walls effect is neglected in the pushover analysis. The reduced stiffness values introduced in the analysis EI_e, having taken into account the effect of axial loading on the degree of cracking. EI_e = 0.40EI_g for the beams and EI_e is in between 0.40EI_g and 0.80EI_g for the columns according to axial load level, where EI_g is the stiffness of the un-cracked section.

The moment-curvature relationship obtained from sectional analysis is converted to moment-rotation relationship because of the frame hinge property input is required for SAP2000 program. The plastic hinge length is assumed to be one half of the section depth.

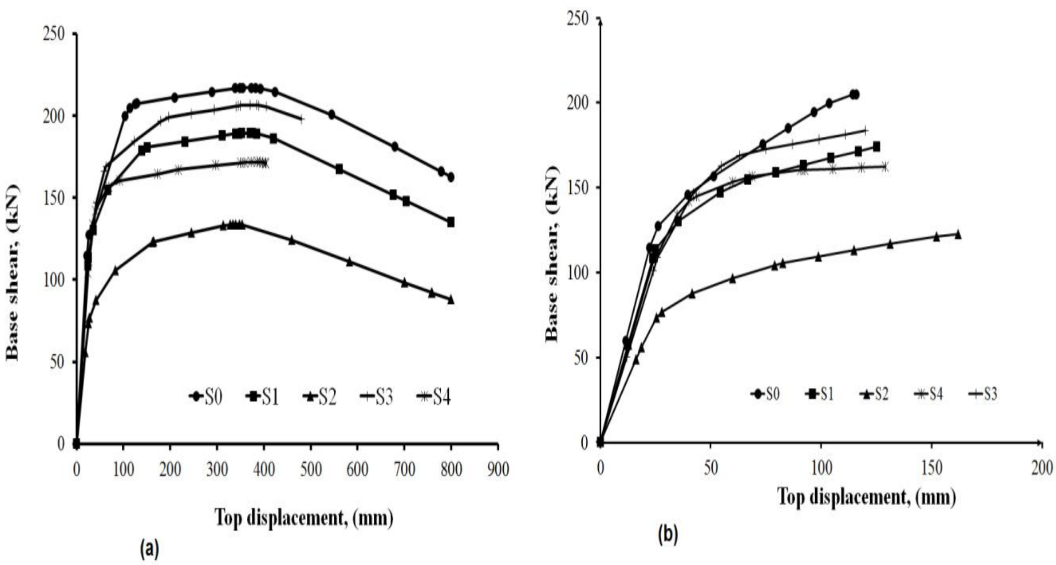
Table 2 Corrosion scenarios

Scenario	Place	Corrosion level
S0	At all elements	No corrosion
S1	At all elements	Moderate, (i_{corr}=1 μA/cm^2)
S2	At all elements	High, (i_{corr}=5 μA/cm^2)
S3	At only bottom story elements	Moderate, (i_{corr}=1 μA/cm^2)
S4	At only bottom story elements	High, (i_{corr}=5 μA/cm^2)

Table 3 Variables considered in all scenarios

	Variables										
	Concrete		Reinforcement steel						Transv. reinf. ratio (x10^{-4})		
	Elastic modulus E_c (MPa) (x10^4)	Conc. str., f_c (MPa)	Dia. (mm)	Area (mm^2)	Yield str. (MPa)	Ult. str. (MPa)	Elastic modulus, E_s (MPa) (x10^5)	Elong. (‰)	x	y	t
S0	3	25	8	50.26	420	550	2	1	53	23	76
			14	153.93	420	550	2	1			
			16	201.06	420	550	2	1			
S1	2.7	16	7.77	47.39	372.5	500.6	1.87	0.852	50	22	71
			13.77	148.87	392.7	521.6	1.92	0.915			
			15.77	195.27	396.1	525.1	1.93	0.925			
S2	2.5	12	6.84	36.74	196.3	317.7	1.38	0.303	39	17	55
			12.84	129.48	287.9	412.8	1.63	0.589			
			14.84	172.96	303.8	429.3	1.68	0.638			
S3	2.7	16	7.77	47.39	372.5	500.6	1.87	0.852	50	22	71
			13.77	148.87	392.7	521.6	1.92	0.915			
			15.77	195.27	396.1	525.1	1.93	0.925			
S4	2.5	12	6.84	36.74	196.3	317.7	1.38	0.303	39	17	55
			12.84	129.48	287.9	412.8	1.63	0.589			
			14.84	172.96	303.8	429.3	1.68	0.638			

The first step in pushover analysis is creating the computer model of the frame. An inverse triangle lateral load pattern is selected resembling probable distribution of earthquake loads determined according to TEC-2007. Definition of properties and acceptance criteria for the plastic hinges is also an important step. 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 which criteria are as follows:



(a) (b)

Fig. 5 Capacity curves of the frame system: (a) Up to failure and (b) Up to target displacement

$$\varepsilon_{cu} = 0.0035; \qquad \varepsilon_{cs} = 0.010 \qquad \text{(Minimum damage limits)} \tag{6}$$

$$\varepsilon_{cu} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135; \ \varepsilon_{cs} = 0.040 \qquad \text{(Life safety damage limits)} \tag{7}$$

$$\varepsilon_{cu} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018; \ \varepsilon_{cs} = 0.060 \qquad \text{(Collapse damage limits)} \tag{8}$$

In Eqs. (6)–(8), ε_{cu} is crushing strain in the extreme fiber; ε_s is strain of reinforcement steel; (ρ_s/ρ_{sm}) is the ratio of existing confinement reinforcement at the section to the confinement required by TEC-2007. Because of these limit values 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. The pushover load cases are defined as fourth step. 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 in this way. Target displacement is determined by using the output of this analysis according to procedure given in TEC-2007. According to this procedure, modal displacement is determined from Eq. (9)

$$d_1^{(p)} = S_{di1} \tag{9}$$

$$S_{di1} = C_{R1} S_{de1} \tag{10}$$

$$S_{del} = \frac{S_{del}}{\left(W_1^{(1)}\right)^2} \tag{11}$$

In Eqs. (9)–(11), S_{di1} is nonlinear spectral displacement which is determined from linear elastic spectral displacement, W_1 is initial angular frequency; $C_{R1}$is spectral displacement ratio. If the first period of the frame, T_1, is greater than T_B then the spectral displacement ratio, C_{R1}, is equal to 1.0. Otherwise (in case of $T_1<T_B$), an iterative method described in TEC-2007 is followed. The target displacement is determined from Eq. (12),

$$u^{(p)}_{XN1} = \Phi_{XN1}\Gamma_{X1}d_1^{\,p} \tag{12}$$

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_{1p} is modal displacement demand for the first mode. After determination of target displacement for the frame than a second run is performed for pushover loads up to the target displacement is attained. All internal forces and deformations corresponds the target displacement are determined by this way.

5. Discussion of results

5.1 Sectional behavior

Sectional behavior is related with material and geometric properties of the corroded and reference sections. It can be determined by moment-curvature analysis. Material characteristics of rebar with respect to diameter and corrosion scenario are shown in Table 3. As corrosion level increased and rebar diameter decreased both of yielding stress and deformation capacity are decreased. Therefore a general decrease is observed in bearing capacity of RC members and

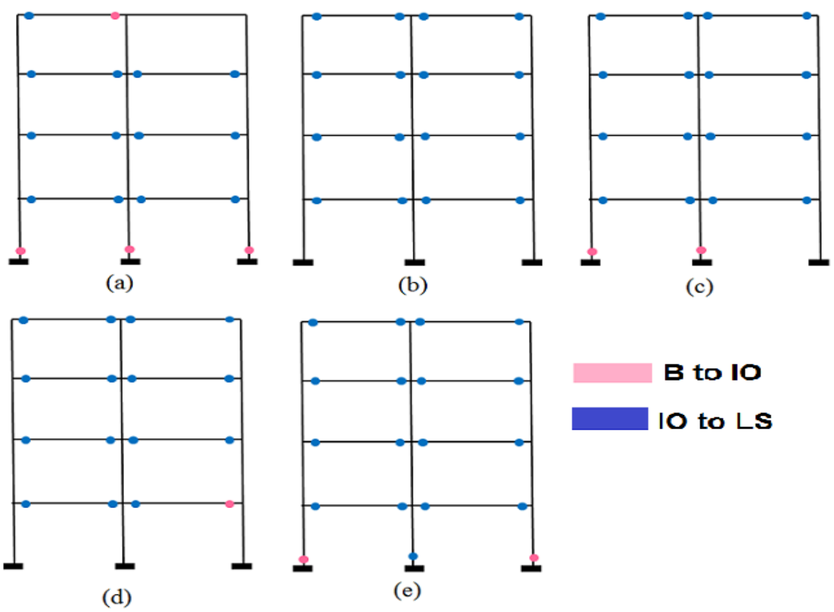


Fig. 6 Plastic hinge formation when target top displacement is attained: (a)S0, (b)S1, (c)S2, (d)S3 and (e)S4

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 scenario S4 (see Fig. 6) could be observed because of suddenly breakage of corroded rebar. Corrosion has adverse effects on material characteristics such as modulus of elasticity and elongation capacity of rebar. When these effects and loss in rebar diameter with bond losses is come together, then structural performance of the frame under earthquake loads is scaled down. Loss in bond strength affects the flexural stress transfer between the concrete and rebar, which reduces the flexural strength of that member and accordingly system behavior.

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 structural performance. Element and component acceptability limit variations are derived from sectional analysis. Bending moment versus plastic rotation of plastic hinges is shown in Figs. 3 and 4. When corrosion deterioration such as loss of rebar section, change of mechanical properties of reinforcement steel, crack in concrete is considered in sectional analysis dimensionless moment (M/M_y) versus plastic rotation curves shows different characteristics. It shows high plastic rotation capacity in the sound system (S0). 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 behavior changes according to corrosion level. For low level corrosion (scenario S1), only plastic rotation capacity and residual rotation capacity are decreased, but the general shape of the curve is not changed. However for the high level corrosion (scenario S2), plastic rotation capacity is decreased abundantly and residual capacity is not existed or is very small. This is an important result for global behavior 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 considerable plastic rotation 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 plastic rotation capacity of the corroded section. This means that performance level ranges are shortened. Moment-rotation curves for scenarios reference and S1 is similar to Type 1 curve that is defined in FEMA-273(FEMA 1996) document and these curves represent ductile behavior. However moment-rotation curve of S2 scenario corresponding high corrosion damage is similar to Type 2 curve which represent another type of ductile behavior. This curve is characterized by an elastic range and a plastic range, followed by a rapid and complete loss of strength.

5.2 System behavior

Capacity curve of each scenario is compared with the curve of reference system (S0), as can be seen in Fig. 5. Capacity curve of the sound system showed a strong nonlinear behavior. Negligible difference is observed in top displacement at failure state of scenario S1 as compared with the sound system. However 13% decrease is observed in base shear. On the other hand, 38% reduction of the load bearing capacity is observed in S2 as compared with the sound state of the system because of high corrosion rate and prevalence of corrosion on the frame. Low ductility is common property of scenarios S3 and S4. The scenario S4 is also attracts attention with its poor base shear capacity. Both of S3 and S4 scenarios shows column sway mechanism 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 has beam mechanism failure mode. As a general result, failure mechanism is affected from rebar corrosion effects where local corrosion (corrosion defects are accumulated in one story) is more dangerous than prevalent corrosion state. 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 and Gambarova (2004) shows that corrosion affects both the strength and the ductility of a structure at ultimate. The bond conditions should be carefully assessed and modeled to predict the ductility of the structural element. The ductility is intimately associated with the failure mode and with the ultimate capacity.

Capacity curves of frame system up to target displacement are shown in Fig. 5(b). Similar results mentioned above could be observed from this figure. Target top displacements in all scenarios are determined as higher than that of S0. This shows that corroded systems needs more top displacements with low base shear for the same earthquake effect. Especially the top displacement at the performance point is maxima at the scenario S2 which was the worst scenario. Sectional loss of concrete and rebar, changes in ductility and yield stress of reinforcement steel results in decrease in global stiffness of the structure. In case of ground floor corrosion (scenarios S3 and S4) there are relatively low difference in terms of base shear and target top displacement with the sound system. Target top displacement for design earthquake is attained before the plasticization of bottom sections of ground floor columns.

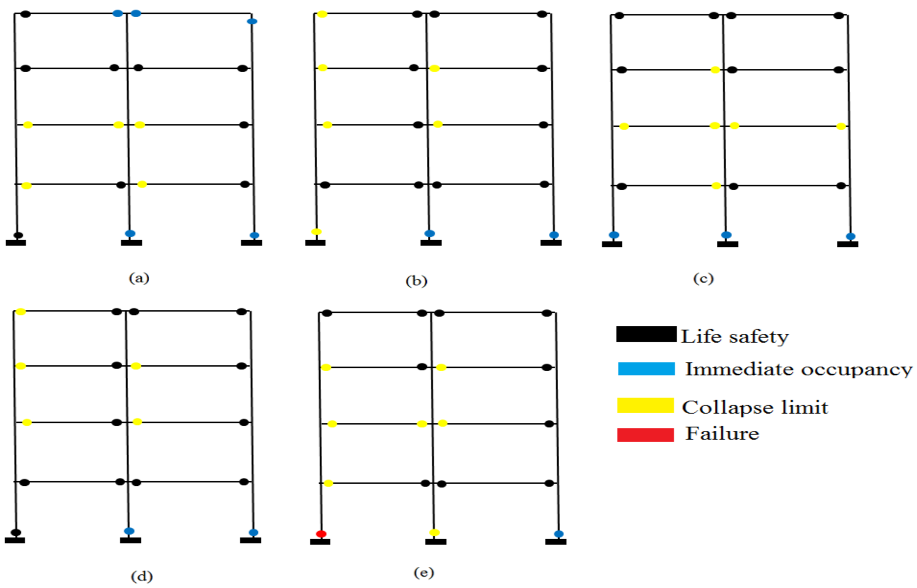


Fig. 7 Plastic hinge formations when the maximum base shear is attained: (a)S0, (b)S1, (c)S2, (d)S3, (e)S4

Plastic hinge formation sequence and plastic deformations on them are also analyzed. Fig. 6 shows plastic hinge formations when target top displacement is attained for all scenarios. Plastic hinges formed at column sections affects the system behavior more than beam plastic hinges. As could be seen on Fig. 6(a), beam plastic hinges are formed at first; and then column plastic hinges at column bottom ends at ground floor are formed in reference system (S0) as expected in earthquake-resistant structures. Energy dissipation is occurred with beam sway mechanism before failure. As corrosion damage is increased in scenarios, some variations at the order of plastic hinge formations are observed. When compared with the sound state of the system, less plastic deformations are observed on beam plastic hinges in S1 till the collapse limit is attained at the bottom end of the column S101 (see Fig. 6(b)). Earthquake behavior of the frame system is similar with S1 with respect to plastic hinge formation sequence but it has more plastic deformations in scenario S2 (see Fig. 6(c)). The plasticization sequence in scenario S4 is typically different than other scenarios. Only it has a column plastic hinge at ground floor for the target top displacement that plastic hinge exceeds life safety limit (see Fig. 6(e)).

Fig. 6(d) shows plastic hinge formations in S3 that includes moderate level corrosion at only the ground floor columns and beams. The corrosion effect on the system behavior is less as compared to scenario S4 that contains high level corrosion at ground floor. Table 4 summarizes how deformations are changed according to corrosion scenarios in all potential plastic hinges. The worst case is in scenario S4 where the number of structural elements exceeds IO level is maxima. The scenario S2 corresponding prevalent corrosion shows also more damages as compared with S1 or S0. This table shows that damages at member level are increased as corrosion level increases.

Fig. 7 shows plastic hinge formations when the maximum base shear is attained. The failure mechanism in S0 (sound state) is similar to ductile systems. The plastic hinge distribution on the structure evidences an unwanted earthquake behavior of the frame. The most column damage is occurred in S4 before collapse of the frame system in which “failure state” is formed at bottom end of a ground floor column. Also, “collapse limit” is exceeded in one column bottom end at the same floor. Two of three columns are practically failed at ground floor where corrosion damages are accumulated. As a result, a change of collapse mechanism occurs moving from a typical ‘beam sway’ to a ‘column sway’ mechanism in scenario S4 (see Fig. 7(e)). Although it has more damage as compared with the sound system, S2 did not show such a change of collapse mechanisms while it has extensive corrosion damages. As column sections are still “IO” performance level, beam sections are in “LS” or “C” performance level. This is because of a wholly strength and stiffness degradation is occurred in S2 (see Fig. 7(c)).

Table 4 Number of elements classified with performance levels for the target displacement

	A to B	B to IO		IO to LS	
	Beam&Column	Beam	Column	Beam	Column
S0	51	1	3	13	0
S1	52	0	0	16	0
S2	50	0	2	16	0
S3	52	1	0	15	0
S4	49	0	2	16	1

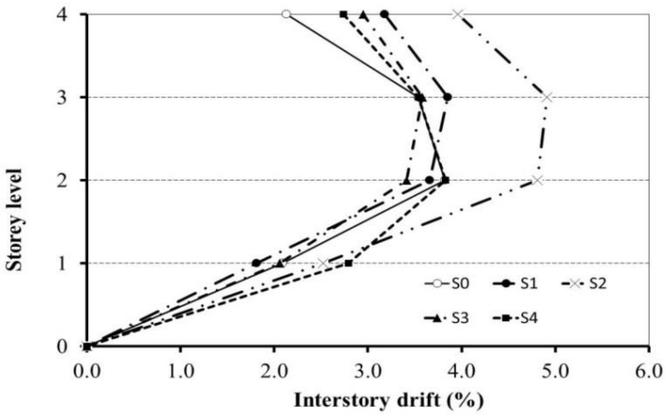


Fig. 8 Relative drifts corresponding target top displacement

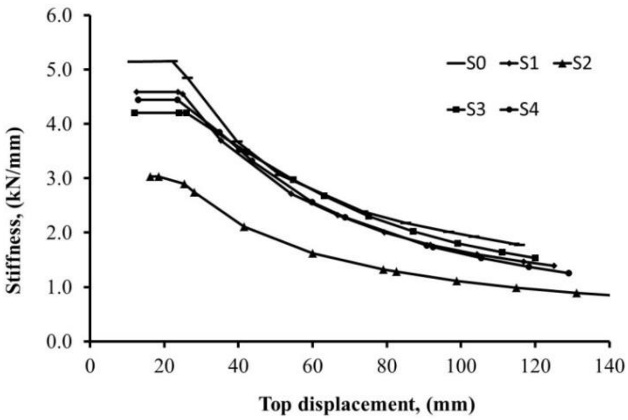


Fig. 9 Stiffness degradation

These two case results in a possibility of soft story behavior in case of local corrosion (corrosion at ground story).The other scenarios S1 and S3 have similar behavior of sound structure with more structural damages (Figs. 7(b) and 7(d)). Berto *et al.* (2009) have similar conclusions about collapse mechanism in case of corrosion of ground floor. They stated that an overall ductile behavior is occurred in sound system while a soft-storey failure mechanism is evident when corrosion affected the structure.

Since interstory drift ratio is directly related to level of structural damage it is important for a structural performance evaluation. Interstorey drift corresponding target top displacements are shown in Fig. 8. A general increase in interstorey drift values are observed in S1 and S2 scenarios that is because of decreases in stiffness with respect to S0. 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. The maximum interstorey drift is appeared in the first floor in scenario S4 when compared to other scenarios and the sound state. This is because of accumulation of corrosion damages in the first floor columns. The interstorey drifts in scenario S2, which corresponding prevalent high level corrosion damage on the frame is higher than that of sound state. When the lateral top displacement is considered S2 and S4 shows different profiles.

Stiffness is an important quantity used to measure structural integrity and it is necessary to examine the effect of corrosion on structural stiffness. The ratio of the lateral load to the top displacement of the frame is denoted as the secant stiffness. Fig. 9 shows the degradation of secant stiffness of frames versus the lateral top displacement. The stiffness of frame in scenario 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 scenario S2 is prevalent corrosion damages on the frame. The scenario S4 having maximum interstorey 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 scenario S4 with respect to other scenarios is observed for the target top displacement of the frame. Maximum plastic rotation at bottom end of column S101in scenario 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 behavior differences are observed between the scenario S2 and the reference. Stiffness degradation, changes in moment-plastic rotation relationships and capacity curves, strength deterioration, differences in plastic hinge formation sequence and some other indicators shows corrosion could change overall structural behavior.

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.

There is a probability of premature local concrete failure because of introducing the bond strength loss and probable cracks in concrete by reducing concrete strength in analyses. Modulus of elasticity of concrete is decreased 10% in low rate corrosion scenarios (S1 and S3) and 17% in high rate corrosion scenarios (S2 and S4) respectively. These decreases affect initial stiffness (EI) of potential plastic hinges. Decreases in concrete strength are another reason of local failures. Therefore the results should be approached with caution especially in scenarios S2 and S4 in this context.

6. Conclusions

In this study, comparative seismic evaluation of a regular multistory frame structure is presented that the structure is exposed different corrosion scenarios. From the results of this study, the following conclusions can be drawn.

- Earthquake behaviors of moment resisting frames are sensitive to rebar corrosion damages with varying corrosion levels.
- Rebar corrosion has considerable effects on global structural acceptability limits and component acceptability limits. Material properties, sectional properties, member and structural

stiffness, component performance limits, bond strength, global deformation characteristics are basically affected parameters from rebar corrosion.

- The time, intensity, and extensity of rebar corrosion affect structural behavior and hence structural performance. Especially, high level and extensive corrosion can cause unfavorable results such as change of collapse mechanism.
- 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. However local corrosion such as corrosion in only ground floor can cause brittle failure. The effect is comparative with the rate and propagation time of corrosion.
- The real level of structural health due to possible rebar corrosion of RC structures should be a taken into account in performance evaluations and in preparation of strengthening projects for existing buildings, especially in highly seismic areas.
- It should be kept in mind that some of local failures may be occurred due to concrete strength reductions which are executed for bond loss and concrete cracks.

Further studies are required in order to represent corrosion deteriorations accurately such as bond deterioration on critical sections of RC members. Also more research studies are necessary about slippage of reinforcement bars and moment-curvature relationship of the corroded sections on potential plastic hinges. Time history analyses are required to represent corrosion deteriorations in case of non-symmetric corrosive attacks on 3D structures.

Acknowledgments

This paper involves a part of the results of the research project (Grant No: 2010-45-06-01). The authors wish to express their gratitude and sincere appreciation to the Committee of Scientific Research Projects of the Bülent Ecevit University for their financial supports.

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