Retrofit of corroded reinforced concrete buildings to improve their seismic capacity

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ABSTRACT: New materials and technologies are used to improve reinforced concrete buildings with new and fundamental structural and non-structural properties. The seismic capacity is one of the most important, both for safety and economic aspects. Nowadays materials and design experience allow constructing new buildings able to safely resist to severe earthquakes. However, degradation of structural members could take place as corrosion of reinforcement in concrete occurs and seismic capacity reduces as corrosion penetration increases. In order to understand the seismic behavior of corroded reinforced concrete structures, some parametric simulations have been performed by means of pushover analyses considering the variability of corrosion attack (one or more sides of structural elements) and the variability of cross section loss of bars (according to ageing period). Global seismic behavior of the structure has been evaluated after ageing as a crucial step to design structural interventions to retrofit corroded and aged structures. High performance concretes have been considered and their benefits have been remarked when used as retrofit of corroded members. In terms of strength and ductility the intervention is able to effectively improve the seismic capacity of deteriorated structures.

1 INTRODUCTION

The corrosion of reinforced concrete (RC) structures is a structural and safety issue as well as an economic cost. RC buildings are very widespread and most of them were built without any code referring to materials control and durability. Some recent statistics of Dept. of Trade and Industry, (2015), NACE (2009) and US Dept. of Transportation (2001), report that American economical investments in existing RC structures greatly exceed those aimed at the construction of new ones. Therefore, maintenance retrofit aimed at conservation of structures and elimination of the dangers seems to be a must for scientific and professional community to discover and use new non-destructive diagnostic methods and strategies, useful to have accurate, fast and affordable results, in order to design targeted retrofit and assess the residual seismic capacity of corroded RC structures. Considering existing structures under non-static conditions, the corrosion of bars and stirrups, yields to partial or total collapses of the building. Referring to new structures, the understanding of corrosion process can lead to design RC structures able to satisfy durability requirements and seismic capacity even when degradation occurs. The present paper is aimed to understand and evaluate the seismic behavior of a simple RC building, by means of push-over simulations. In order to assess the variation of seismic capacity, different levels of corrosion and different extensions of the phenomenon were considered. A retrofit design was simulated by using high performance concrete and push-over analyses were performed to evaluate the benefits of the retrofit in terms of seismic capacity.

2 BUILDING GEOMETRY AND PRELIMINARY ASSUMPTIONS

In order to perform Push-Over (PO) analyses, it was considered a RC building with two floors of dimensions (10.00x5.00) m^2 and a total height of 6.40 m. It is characterized by two frames with two spans of length 5.00 m along the X-direction, and three frames with two spans of length 5.00 m along the Y-direction. The hollow core concrete floors are oriented in the Y-direction. The concrete section of beams and columns is (30x30) cm^2, and both of them are reinforced by using 8 longitudinal bars of diameter 14 mm and stirrups of diameter 10 mm with a spacing of 20 cm, similar to laboratory specimens tested in both not degraded and degraded states. The experimental results obtained on the samples, were used in order to control and simulate the behavior of the whole structure. Beams were identified by using the label “T”, followed by three numbers. The first one indicates the floor level (1 or 2) while the other numbers indicate the columns on which it is positioned. Columns were identified by the label “P” followed by two numbers; the first one indicates the floor (1 or 2) and the other represents the identification number (Figure 1). The concrete has a cylindrical average strength, fcm=22.7 MPa, an elastic modulus, Ec=31 GPa, a Poisson's ratio, υ=0.2 and a specific weight equal to 25 kN/m^3, while the average value of the yield strength of steel is fym=407 MPa. The soil-structure interaction has not been modeled by assuming fixed constraints at the base of the columns at the first level. The slabs were considered infinitely rigid according to the Italian Code (NTC 2008) §7.2.6. Accidental eccentricity has been neglected and the "Master Node" to perform PO analyses, was placed in the geometric center of gravity of each slab, assumed coincident with the center of the masses. Two translational masses and a rotational mass have been assigned to each deck. The seismic action according to NTC2008 is evaluated starting from a "basic seismic hazard", which is the primary element to define seismic actions. Actions are measured in relation to the reference life, VR, which is obtained by multiplying the nominal life, VN, by a coefficient of use, CU, depending on the class of use of the building. In the present case, it was assumed a nominal life, VN, equal to 50 years, and CU=1, for a resulting VR of 50 years. The seismic actions can be defined by the spectrum of ground acceleration, Sa(T). The return period of the seismic action, T_R, was considered equal to 50 years considering the damage limit state and TR=475 years considering the ultimate limit state.

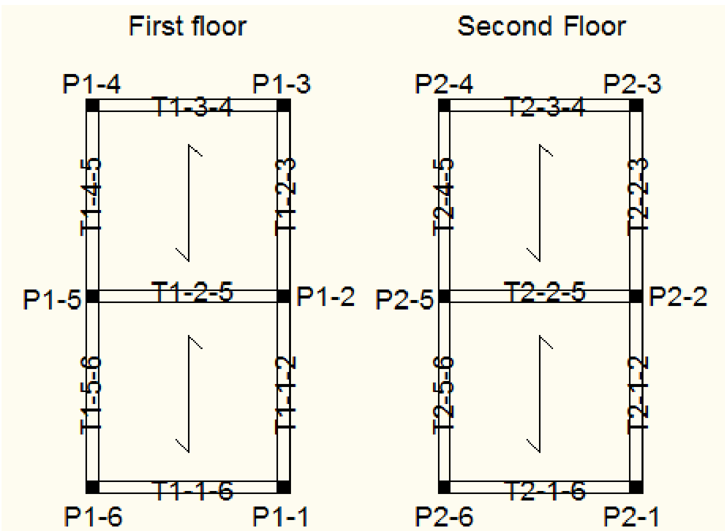

Figure 1. Plans of first and second floors with beams and columns identification label

According to NTC2008 the spectrum is defined by maximum horizontal acceleration to the site, ag, maximum value of the amplification factor of the spectrum of horizontal acceleration, F0, and the period of beginning of the constant speed portion of the spectrum in horizontal acceleration, T*c identified on the basis of the geographical reference grid and the seismic action return period, T_R.

The elastic response spectrum of the horizontal component is defined by the equations (3.2.4) of the §3.2.3.2.1 of NTC2008. The elastic spectra of the Damage Limit State (DLS) and the Lifesafe Limit State (LLS), were evaluated referring to a conventional damping of 5%. Table 1 shows the seismic masses, M, and the total weights, W.

Table 1. Seismic weight and seismic mass

FIRST AND SECOND FLOOR		
	W [kN]	M [t]
Slab	280.00	28.54
Walls	89.90	9.16
Columns	43.20	4.40
Beams	78.75	8.03
Total	491.85	50.14

3 MODELLING

Several numerical push-over analyses were performed, considering the non-corroded structure and then, considering three different configurations (sides of attack of aggressive agents) of corrosion and three different levels. The plastic hinge model is based on the model proposed by NTC2008. Figure 2 shows the bending moment-rotation graph characterizing the formation of the plastic hinges in case of non-corroded members and in the cases of the three corrosion levels considered. Equations (1), (2), (3) and (4) were used to calculate cracking rotation, θ_{cr}, yield rotation, θ_y: the rotation at Lifesafe Limit State, θ_{SLV} and the ultimate rotation θ_u (symbols are reported in NTC2008). Figure 3 shows the schematization of the non-linear relationship used for simulations. Assuming concentrated plasticity in two hinges, the non-linear relationship has been defined with reference to the rotations and not referring to the smeared curvature. Taking in account the calculation model, the rotation due to flexural deformation needs to be subtracted from the value of the elastic rotation. Therefore four hinges were associated to the columns (two base-hinges and two top-hinges per each direction) and two hinges were associated to the beams. The yield rotation was calculated according to Panagiotakos and Fardis (2001) and hinges are uniaxial in both principal directions.

$$\theta_{cr} = \frac{\phi_{cr} \cdot L_v}{3} \quad (1)$$

$$\theta_y = \frac{\phi_y \cdot L_v}{3} + 0.0013 \cdot \left(1 - 1.5 \cdot \frac{h}{L_v}\right) + 0.13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \quad (2)$$

$$\theta_{SLV} = \frac{3}{4} \cdot \theta_u \quad (3)$$

$$\theta_u = 0{,}016(0{,}3)^{\nu} \left[\frac{max(0.01;\omega\prime)}{max(0.01;\omega)} \frac{f_c}{1}\right]^{0.225} \left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_v \cdot \frac{f_{yw}}{f_c}\right)} (1.25^{100\rho_d}) \quad (4)$$

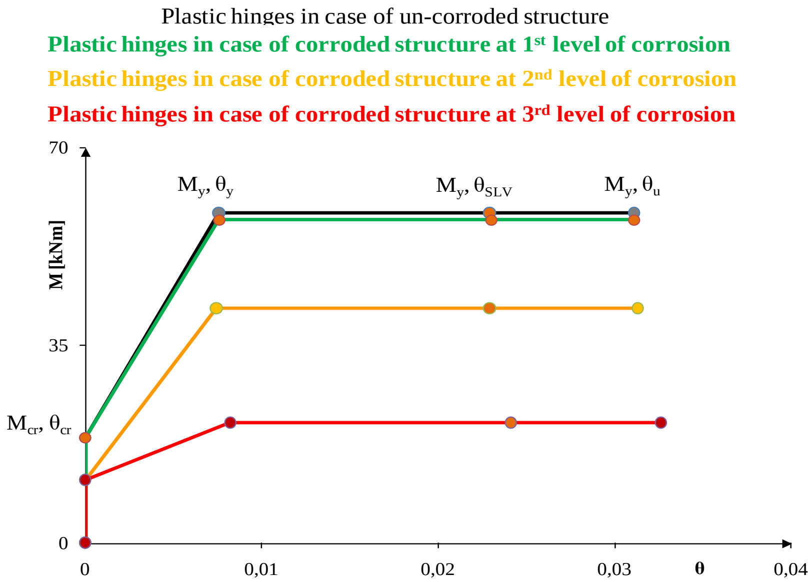

Figure 2. Plastic hinges

3.1 *Corrosion levels*

According to Bossio et al. (2013), the corrosion level, in terms of reduction of bar radius, x, was evaluated considering the relationship between concrete cover cracking and the volume of oxide inducing that crack. Assuming a crack opening value of 0.5 mm, leading to concrete cover loss, it was evaluated the reduction of bar radius, x, and, consequently, the loss of cross section of reinforcements. Three levels of generalized corrosion induced by carbonatation were defined. The first level provides a reduction of bar radius such as to induce the concrete cover cracking. The second level provides the spalling of the concrete cover and the third level considers a reduction of bar radius equal to 50% of the diameter. For the purposes to evaluate the time inducing the corrosion level, it was assumed that the structure was built in exposure class defined as "XC4" according to the UNI EN 206:2006 with a carbon dioxide diffusion coefficient k equal to 6 mm/year$^{0.5}$. The average corrosion rate was assumed equal to 70 µm/year, so 26 years are required to have the first level of corrosion (reduction of radius of stirrups equal to 1 mm and a low level of corrosion of longitudinal bars). The second level of corrosion occurs after 40 years and reduction of radius of stirrups is assumed equal to 2 mm while the reduction of diameter of longitudinal bars is equal to 2 mm. The third level of corrosion considers a reduction of the diameter of the longitudinal bars equal to 50% (reasonably reached after 75 years) and the residual diameter of stirrups equal to 1 mm. Table 2 shows the values of the diameter of the bars, Φbar, and stirrups, Φst, considered, for non-corroded structure and for the three levels of corrosion and the degradation level of the concrete cover, cc.

Table 2. Bars and stirrups diameters related to corrosion level

	ϕ_{bar}	ϕ_{st}	x_{bar}	x_{st}	ϕ_{bcorr}	ϕ_{scorr}	c_c
	[mm]	[mm]	[µm]	[µm]	[mm]	[mm]	[-]
1st level of corrosion	14	10	100	1072	13.7	8	cracked
2nd level of corrosion	14	10	1000	1972	12.0	6	spalled
3rd level of corrosion	14	10	3500	4472	7.0	1	spalled

4 CORROSION CONFIGURATIONS

Three different corrosion configurations were considered: (i) one façade of the building is corroded and its structural elements are corroded along the external side only both for longitudinal bars and external leg of the stirrups, (orange colored in figure 3), (ii) all façades of the building are corroded and all external sides of structural elements are corroded both for longitudinal bars and external leg of the stirrups and (iii) all structural elements of the building are corroded along all the sides both for longitudinal bars and the two legs of the stirrups. Figures 3 show the corrosion configurations. Considering the first configuration (figure 3a), the behavior of corroded columns is not axi-symmetric, particularly referring to the plastic hinges. Considering the second configuration (figure 3b), corner-positioned columns named “P1”, “P3”, “P4” and “P6”, are corroded along two sides. Considering the third configuration (figure 3c), the behavior of plastic hinges is symmetrical along the two main directions. The push-over analyses were performed both in the X-direction and Y-direction (both positive and negative). The first level of corrosion considers longitudinal bars and stirrups corroded on one side both for beams and columns; the concrete cover is cracked, but it continues to be considered part of the concrete section. The second level of corrosion considers the corner-positioned columns corroded on two sides, while central-positioned columns corroded on one side, as well as the beams, but the level of corrosion led to concrete cover spalling. The third level of corrosion considers beams and columns corroded on four sides and the concrete cover is spalled all around the perimeter. Corrosion levels lead to consider different types of beams and columns sections, both in terms of cross section of reinforcements and concrete; in particular, considering the second level of corrosion, it needs to be considered the eccentricity of the vertical load on the columns, due to the unsymmetrical spalling of the concrete cover. Moreover, the concrete geometry is different. The concrete cross section, at first level of corrosion, is (30x30) cm^2; considering the second level of corrosion, concrete cross section is equal to (27x30) cm^2 for central-positioned columns and (27x27) cm^2 for corner-positioned columns; when third level of corrosion occurs, concrete cross section is equal to (24x24) cm^2. Figure 4 shows the geometry of columns and beams considering corrosion levels and the correlation between design shear and bar reduction, x.

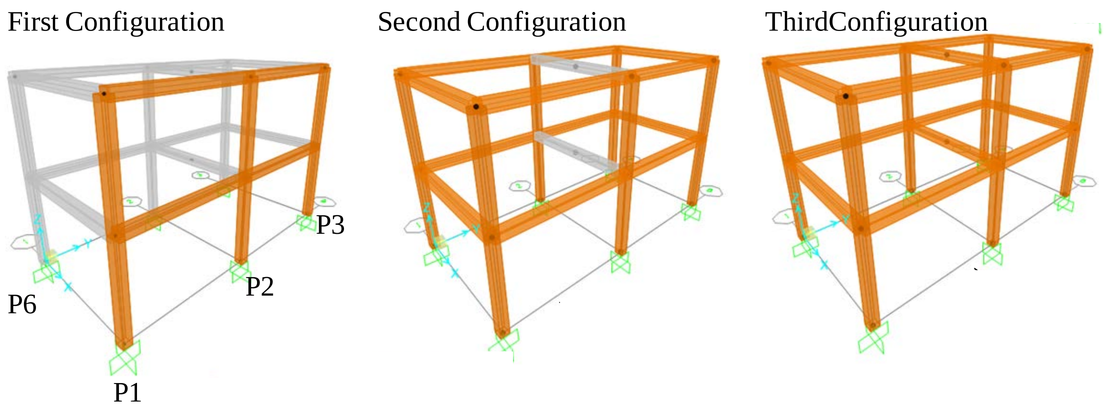

Figure 3. Corrosion Configurations

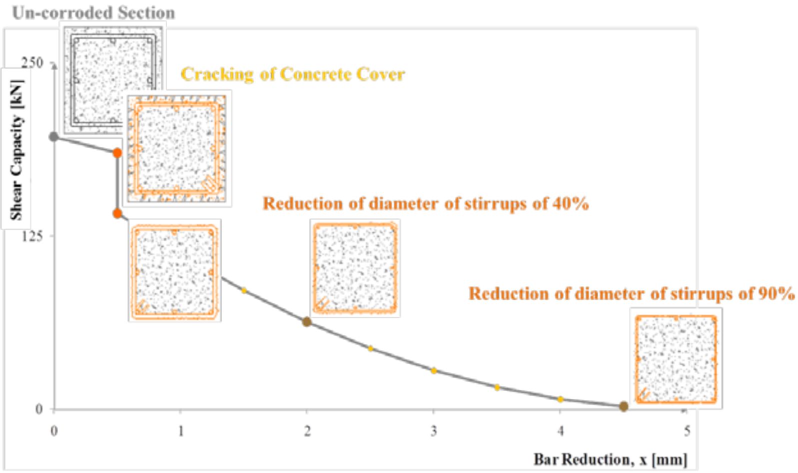

Figure 4. Corrosion Levels and correlation between shear capacity and bar reduction

5 PUSH-OVER ANALYSES RESULTS AND EVALUATION OF SEISMIC BEHAVIOR OF THE STRUCTURE

Shear failure was evaluated both according to Biskinis et al. (2004) and the Italian code NTC2008. Considering the first and the second level of corrosion, the structure shows a ductile failure because the Lifesafe Limit State occurs at the bottom of the columns of the central frame; with the exception of the third configuration that shows a shear failure occurring at beams loaded by floors under gravity loads, according to Biskinis et al. (2004), while according to NTC2008 the shear failure doesn't occur. Considering the second level of corrosion, according to Biskinis et al. (2004) the shear failure occurs to all perimetral-positioned beams, while according to NTC2008 model the shear failure occurs at beams loaded by floors. Considering the third level of corrosion and the second configuration, a brittle failure occurs in the X^+-direction. According to Biskinis et al. (2004) all elements present a shear failure, while considering the NTC2008, the shear failure occurs in all perimetral-positioned beams. Figure 5a shows the shear failure according to Biskinis et al. (2004) occurring for the loaded beams, when the force is applied in X^+-direction. Figure 5b shows the ADRS demand curve and the bi-linear curve for the un-corroded building and for corroded building at third level of corrosion and second configuration.

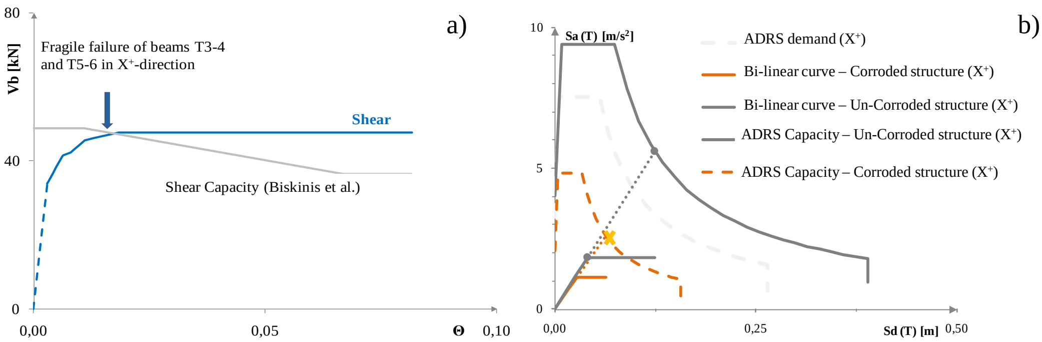

Figure 5. a) Shear failure according to Biskinis et al. (2004), b) ADRS representation of Push-over curves for un-corroded and corroded structure at third level of corrosion and second configuration X^+-direction.

6 RETROFIT

Considering the third configuration and the second level of corrosion (concrete section 24x24 cm^2), a retrofit was supposed by using a High Performance Concrete (HPC) and push-over analyses were performed. The HPC was used in order to restore a geometrical section of (30x30) cm^2. Figure 6 shows the bending moment-curvature graphs related to non corroded members, to corroded members and to retrofitted members. The bending capacity of corroded members is about 30% less than the non corroded members; ductility is also reduced, because of the concrete cover spalling and reduction of bars cross section. Considering the retrofitted section, the bending capacity is about 20% higher than the corroded one. Furthermore, the ductility of the retrofitted members is higher than the ductility of non corroded members, confirming the high performance of the HPC material both in terms of compressive strength and in terms of bending moment capacity. Figure 7 shows the return period of seismic action, T_R, related to corrosion configurations and considered corrosion levels, comparing the case of retrofitted structure, too. In the case of third configuration, due to failure under gravity loads, no return period can be identified for second and third levels of corrosion.

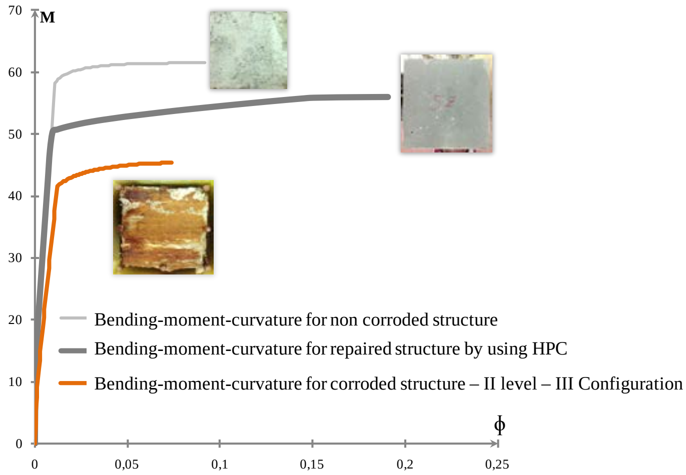

Figure 6. Bending moment-curvature graph in case of non corroded, corroded and retrofitted members

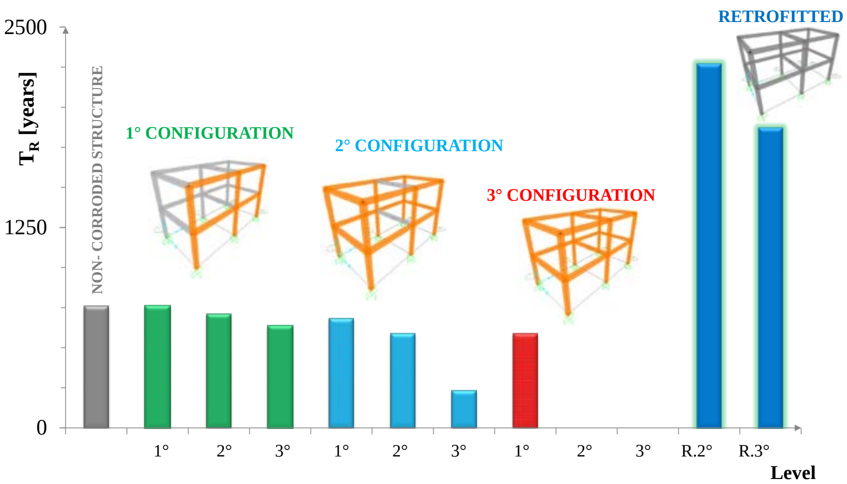

Figure 7. Corrosion levels and configurations related to Return period of seismic action yielding to lifesafe limit state for the structure

7 CONCLUSIONS

Carbonation and chloride ions attack lead to a progressive loss of mechanical and structural performance of RC structures. The present work, starting from a simple framed RC building, aims to highlight how the increase of the degradation, due to generalized corrosion, could significantly reduce the seismic capacity of a structure and in some cases it yields to brittle failures in the most stressed structural elements even under gravity loads. Performing push-over analyses showed that, as expected, the most critical case was the third level of corrosion of the third configuration. In this case, the cross section of stirrups decreased by 90% so a brittle shear failure occurs in the most stressed beams, causing a sudden structural collapse under gravity loads. Full of interest is the case related to the first level of corrosion for the second configuration. In fact, push-over analyses show that the structure does not take advantage of all its ductility, since shear failures, involving limited ductility, occur to external beams. The push-over analyses provided the capacity curves of the structure. Capacity curves have been bi-linearized and put in a ADRS domain: it was possible to define the demand curve and to assess the vulnerability of the structure according to the procedures adopted in the International Standards. The retrofit by using High Performance Concrete highlighted that structural elements can recover some of their bending capacity and can increase their ductility, reducing dramatically the seismic vulnerability of the structures.

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