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Paper 159

Ductility Demand on Steel Reinforcing Bars in Concrete Buildings

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Abstract

In the present paper, developed inside the framework of the European Research project *Rusteel*, the behaviour of steel reinforcing bars under the combined effects of low-cycle fatigue action and corrosion phenomena is studied. The project aims at the definition of the effective ductility capacity of reinforcements, to be compared with the ductility demand imposed by earthquakes, investigated through the execution of non-linear dynamic analyses on numerical models of representative modern reinforced concrete buildings. The comparison between demand and capacity will enable understanding of the effective relationship between the requirements of earthquakes and the capacity of rebars (strength and ductility), providing indications for the design and structural details of new building in seismic areas.

Keywords: low-cycle fatigue, corrosion, incremental dynamic analysis, bond-slip.

1 Introduction

Actual standards for constructions [1, 2] provide specific rules for the design of reinforced concrete buildings in seismic areas, based on the capacity design approach. According to these codes, buildings should be able to dissipate seismic energy through the development of deformations located in specific regions of the construction (generally known as "dissipative zones") in which the structural details (diameter and position of longitudinal and transversal reinforcements) are designed to let the structure able to reach a global collapse mechanism, consequently avoiding local and brittle situations such as soft storey. In particular, in reinforced concrete buildings the ductile behaviour of the entire structure is strictly related to the rotational capacity of single elements (beams) and consequently to the ductility of steel reinforcing bars (rebars) located at the ends of beams, in which the plasticization is expected.

A good and deep knowledge of the mechanical behaviour of steel rebars under seismic loading condition (low-cycle fatigue action, LCF) is then necessary for having a global understanding of the actual response of the structure.

Actually, at European level, Eurocode 8 [2] allows the use of steel rebars belonging to three different ductility classes, called "A", "B" and "C" in relation to the level of available A_{gt} (elongation to maximum load), respectively equal to 2.5%, 5.0% or 7.5% and to the value of hardening ratio, respectively ≥1.05, ≥1.08 and between 1.15 and 1.35 [3]. For buildings realized in high ductility class (HDC), the only use of class C is allowed for longitudinal steel reinforcements, while for buildings in low ductility class (LDC) both classes B and C are authorized. Italian standards for constructions [1], in addition to what herein presented, allows, only for stirrups, a low requirement of ductility (class A).

Nowadays, at a European level, no standards for the mechanical characterization of steel reinforcing bars under LCF are provided; only Spanish and Portuguese standards [4, 5, 6] present prescriptions for the execution of low-cycle fatigue tests, in which, nevertheless, the level of imposed deformation, the number of cycles and the frequency are not defined on the base of specific analyses taking into account the influence of real seismic events. In the current literature, many works are presented about the mechanical characterization of steel reinforcing bars under LCF action [7, 8, 9], but none of them provides a real correlation between the levels of ductility and strain rate used in the tests with the requirements imposed by earthquakes. Moreover, other works [10, 11] showed that the mechanical characteristics of steel reinforcing bars are deeply influenced by the effects of aggressive environmental conditions (corrosion phenomena): the spalling of the concrete cover leads to the premature buckling of steel rebars, while the cross section reduction causes the loss of both strength and ductility of the rebars, influencing the global dissipative behaviour of the reinforced concrete buildings. The investigation of the ductility capacity of rebars after corrosion is consequently necessary especially for those buildings that are located in specific aggressive areas, such as, for example, in proximity of the seaside (i.e. effects of chlorides). Even if the actual prescriptions for the sizing of the concrete cover [12] should prevent spalling, consequently protecting steel reinforcement, the knowledge of the mechanical behaviour of rebars after corrosion is necessary both for the monotonic and the cyclic loading conditions.

On the base of what herein presented and also taking into account the necessity of European standard's harmonization imposed by Mandate M115 [13] inside the revision of EN10080 [14], a detailed campaign of experimental low-cycle fatigue tests on uncorroded and corroded rebars was developed in the framework of a European research project funded by the Research Fund for Coal and Steel, called *Rusteel* (*Effects of Corrosion on Low-Cycle Fatigue (Seismic) Behaviour of High Strength Steel Reinforcing Bar,* 2012). The Rusteel experimental test campaign allowed the definition of the effective mechanical capacity of steel rebars with different production process (TempCore, Micro-Alloyed, Stretched and Cold-Worked) under low-cycle action, both for uncorroded and corroded condition. The real ductility *capacity* of steel rebars shall be compared with the effective ductility *demand* required by earthquakes, opportunely evaluated through the elaboration of

numerical models of representative reinforced concrete case studies and the execution of Incremental Dynamic Analyses (IDA).
In the present paper, the preliminary results of the experimental test campaign and of the numerical analyses are showed [2].

2 Methodology adopted in the project

The main aim of *Rusteel* project is the evaluation of the effective ductility demand imposed to steel reinforcing bars by earthquakes and, consequently, the individuation of the steel grade to use in relation to the seismicity of the area and to the structural criteria adopted in the design (in terms of ductility class, level of p.g.a. and structural details). A comparison between the effective ductility demand on steel rebars, opportunely determined through the execution of non-linear Incremental Dynamic Analyses (IDA) with accelerograms selected for maximizing the ductility requirements, and the effective experimental ductility capacity of rebars, determined through the execution of low-cycle fatigue tests, is necessary to reach the objectives proposed in the project. Moreover, the effects of corrosion phenomena on the mechanical characteristics of steel reinforcing bars, under both monotonic and cyclic loads shall be considered for the selection of the steel grade to use. Figure 1 presents a simple flowchart of the methodology adopted in *Rusteel* project.
In particular, the diagram evidences the tasks related to the determination of the *capacity* (on the left side) and the ones dealing with the individuation of the *demand* (on the right side). As regards the definition of the capacity, in the present paper some results of the mechanical characterization of steel reinforcing bars (both uncorroded and corroded) are presented; moreover, the numerical modelling procedure used for the analyses, with the definition of the constitutive laws selected for steel rebars, and some preliminary results are showed for what regards the determination of the seismic demand on reinforced concrete buildings.

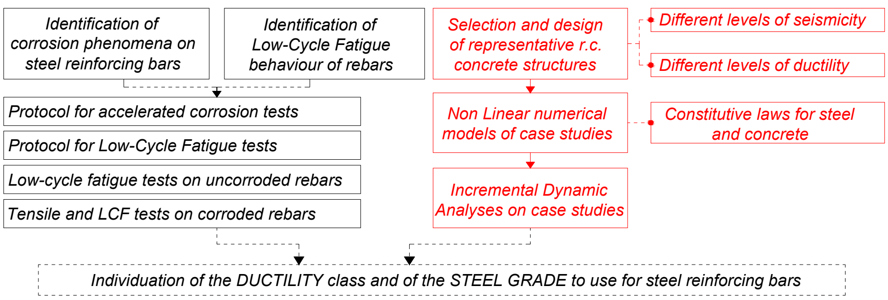


Figure 1: Flowchart of the project (demand and capacity).

3 Definition of the ductility capacity of steel bars

3.1 Mechanical characterization of rebars

In order to completely characterize the mechanical behaviour of steel reinforcing bars under both monotonic and cyclic loads, a representative set of steel rebars was

selected inside the research project *Rusteel*. The choice of the different bars to test was executed in relation to the necessity of including all the most diffused mechanical production processes - TempCore (TEMP), Micro-Alloyed (MA), Stretched (STR) and Cold-Worked (CW), and diameters (between 8 and 25 mm), to consider the behaviour of both stirrups and longitudinal reinforcements. Moreover, according to a preliminary accurate investigation on the actual European production standards for steel reinforcements, different steel grades (yielding strenght equal to 400, 450 or 500 MPa) and different ductility levels (A, B or C) were selected.
All the rebars to be tested were provided by the two steel producers involved in the project; in order to obtain widespread results, the variability due to production in different plants was taken into account testing rebars coming from three different plants (one in Italy, one in Germany and one in France, for B450C and B500B, diameter 16 mm). Table 1 presents the complete set of steel rebars selected for mechanical tests; the asterisk indicates the rebars selected for LCF tests.

Steel Grade	Diameter	Steel Process	Ribs	Furniture	More information
B500A	8*,12*	CW	ribbed	Prod.1	
B500A	8*	CW	indented	Prod.2	
B500B	8*,16*,20*,25	TEMP	ribbed	Prod.1	Same cast for all diameters
	16*			Prod.2	From 3 different plants
B500B	8*,12	STR	ribbed	Prod.2	German plants
B400C	8*,16*,20*,25	TEMP	ribbed	Prod.2	Spanish plants
B400C	16*,20*,25	MA	ribbed	Prod.1	Same cast for all diameters
B450C	16*,20*,25	TEMP	ribbed	Prod.1	Same cast for all diameters
	16			Prod.2	From 3 different plants
B450C	8*,12*	STR	ribbed	Prod. 1+2	

Table 1: Selected set of rebars for the mechanical characterization.

The preliminary experimental test campaign included tensile and hardness tests; three tensile tests for each steel grade, diameter and producer were executed.
For the of low-cycle fatigue experimental tests, a specific protocol was elaborated, since, nowadays, no specific prescriptions are provided about. According to actual literature, the main features to define for LCF tests are: the level of imposed deformation (ε) and the frequency of application (resulting in the strain rate), the number of cycles (N_f) and the free length of the specimen (L_0), strongly influencing the buckling of rebars. At European level, only Spain [4] and Portugal [5, 6] give some indications for the seismic requirements of steel rebars; according to Spanish standards, bars should be subjected to three cycles of deformation, equal to ±1.0%, ±2.5% or ±4.0% in relation to the diameter ($\phi \leq 16$ mm , $16 \leq \phi \leq 25$ mm and $\phi \geq 25$ mm), using a free length varying with the size of rebars. No specific information are given about the frequency to use. Portuguese standard on the other hand, prescribes the execution of 10 different cycles of deformation equal to ±2.5% with a frequency of 3.0 Hz and a free length of the specimen equal to 10 diameters.

Looking at the current literature, Kunnath et al. [7] executed low-cycle fatigue tests on steel rebars of medium-large diameter (between 19 and 25 mm), with a free length varying between 6 and 9 diameters and total strain amplitude between 1.5 and 3%, going on with the test until failure was reached. Mander et al. [14], considering a free length of 6ϕ, 8ϕ and 9ϕ (being ϕ the diameter of rebar), executed low-cycle fatigue tests with frequencies variable between 0.025 Hz and 0.15 Hz, resulting in an average strain rate of 0.005/s. Hawileh et al. (2009) [15] executed tests on steel reinforcing bars BS460B and BS500B with a frequency of 0.05 Hz and a level of deformation varying between 3.0 and 10.0%; the free length of the specimen was very small, neglecting possible buckling phenomena.
Rodriguez et al. [16] presented the results of low-cycle fatigue tests on bars of 16 mm with a gauge length of 30 mm and a free length function of the diameter (in order to be representative of the real spacing of stirrups); the frequency was equal to 0.005 Hz with two reversed cycles for each level of maximum strain ($\varepsilon_{max}/\varepsilon_{min}$=1.0 or $\varepsilon_{max}/\varepsilon_{min}$=2.3). Finally, Crespi [8] gave the results, both in terms of energy dissipated and number of cycles up to failure, of low-cycle fatigue tests on steel rebars of 14 and 20 mm of diameter (ribbed rebars), for imposed deformations ranging between 1.0% and 4.0%, frequency between 1.0 and 3.0 Hz and free length equal to 10ϕ. On the base of the presented data, a specific protocol for LCF tests was elaborated, in order to take into account all the significative factors herein listed. Tests presented in literature revealed buckling phenomena of steel rebars for a free length higher than 6ϕ; according to actual European and Italian standards [2, 1] the maximum spacing between stirrups cannot exceed 6ϕ or 8ϕ in relation to the ductility class adopted in the desing (6ϕ for HDC, 8ϕ for LDC). For the execution of *Rusteel* low-cycle fatigue test's campaign, two different free lengths were selected, in order to represent the effective situation of rebars in HDC and LDC buildings; moreover, two different levels of imposed deformation were adopted, respectively equal to ±2.5% and ±4.0%, the maximum number of cycles to execute was fixed at 20 and the frequency, after preliminary tests aiming at evaluating the effective influence of strain rate (Figure 2a), was established equal to 2.0 Hz (with possible reduction to 0.05 Hz in relation to mechanical requirements of instrumentation, especially for diameters higher than 16 mm). LCF tests were executed using a machine with 250 kN capacity in deformation control, imposing $\Delta l = \pm\varepsilon \cdot L_0$, function of the bar diameter. Table 2 summarizes the prescriptions of the protocol for LCF tests on bars of different diameters, while in Figure 2b an example of the experimental test's results is presented.
Preliminary analyses of experimental results showed buckling phenomena of steel reinforcements after one-two cycles in compression, both for small and large diameters and for a free length of 6 or 8 diameters; in particular, for HDC (L_0=6ϕ) and imposed deformation equal to ±2.5% rebars are, in general, able to support 20 cycles tension/compression without failure. 20 cycles were also obtained from rebars of small diameter for the same level of deformation in LDC (L_0=8ϕ).

The number of cycles that the steel bar is able to complete decreases with the increase of the deformation level required and of the diameter (Table 3). For example, for bars of 20 mm diameter and deformation of ±4.0% the maximum

number of complete cycles is equal to 7 (L_0=8ϕ); for bars of 8.0 mm diameter and a free length of 6ϕ, the specimens are able to complete at least 12 cycles.
In Table 3, the asterisk indicates that the maximum number of cycles was reached without the failure of the bar.

ϕ [mm]	L_0 [mm]		ε [%]	Δl (±) [mm]	N_f
20	6ϕ	120	± 2.5%	3.00	20
			± 4.0%	4.80	20
20	8ϕ	160	± 2.5%	4.00	20
			± 4.0%	6.40	20
16	6ϕ	96	± 2.5%	2.40	20
			± 4.0%	3.84	20
	8ϕ	128	± 2.5%	3.20	20
			± 4.0%	5.12	20

ϕ [mm]	L_0 [mm]		ε [%]	Δl (±) [mm]	N_f
12	6ϕ	72	± 2.5%	1.80	20
			± 4.0%	2.88	20
12	8ϕ	96	± 2.5%	2.40	20
			± 4.0%	3.84	20
8	6ϕ	48	± 2.5%	1.20	20
			± 4.0%	1.92	20
	8ϕ	64	± 2.5%	1.60	20
			± 4.0%	2.56	20

Table 2: LCF protocol for rebars of different diameter.

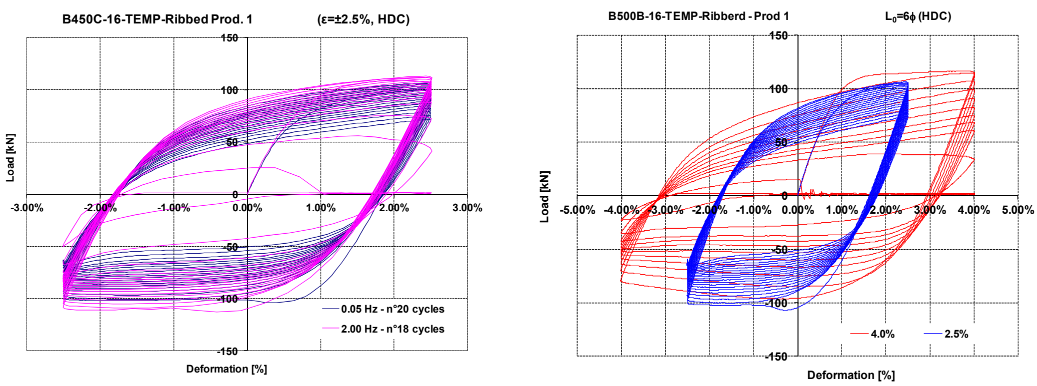


Figure 2: a) Influence of strain rate on the results of LCF tests on bars B450C, 16 mm; b) LCF tests for different levels of deformation on bars B500B, 16 (L_0=6ϕ).

Bar LABEL	L_0= 6ϕ		L_0= 8ϕ	
	± 2.5%	± 4.0%	± 2.5%	± 4.0%
B400C-16-TEMP-Rib.Prod. 2	19	8	15	10
B450C-16-TEMP-Rib. Prod. 2 (1)	20*	9	14	12
B450C-16-TEMP-Rib. Prod. 2 (2)	20*	18	17	8
B450C-16-TEMP-Rib. Prod. 2 (3)	20*	14	15	12
B500B-16-TEMP-Rib. Prod. 2 (1)	20*	9	15	11
B500B-16-TEMP-Rib. Prod. 2 (2)	20*	8	14	6
B500B-16-TEMP-Rib. Prod. 2 (3)	20*	10	15	9
B400C-16-MA-R-Rib.Prod. 1	20*	13	18	9
B450C-16-TEMP-Rib. Prod. 1	18	9	17	8
B500B-16-TEMP-Rib. Prod. 1	20*	12	14	7
B400C-8-TEMP-Rib. Prod. 2	20*	13	20*	12
B500A-8-CW-Ind. Prod. 2	20*	17	20*	12

B500B-8-STR-Rib. Prod. 2	20*	13	20*	9
B450C-8-STR-Rib. Prod. 2	20*	20*	20*	16
B500B-8-TEMP-Rib. Prod. 1	20*	12	18	9
B500A-8-CW-Rib. Prod. 1	20*	16	20*	12
B500A-12-CW-Rib. Prod. 1	20*	11	20*	9
B450C-12-STR-Rib. Prod. 1	20*	14	20*	13
B500B-20-TEMP-Rib. Prod. 1	20*	8	16	6
B450C-20-TEMP-Rib. Prod. 1	20*	6	19	6
B400C-20-MA-Rib. Prod. 1	19	8	18	8
B400C-20-TEMP-Rib. Prod. 2	20*	6	9	7

Table 3: Number of cycles tension/compression from experimental LCF tests.

3.2 Mechanical characterization of corroded steel bars

Recent studies in the current literature [10, 17] evidenced the progressive deterioration of the mechanical characteristics of steel reinforcing bars under aggressive environmental conditions. The individuation of the effective mechanical behaviour of rebars after corrosion phenomena is a quite recent problem, mainly developed in the last decades; the effects of corrosion on the mechanical properties of rebars were not usually taken into account since the presence of a correctly sized concrete cover, joined with ordinary external circumstances, is generally sufficient to guarantee the protection of reinforcing steel rebars, providing a thin passive layer that covers the reinforcement avoiding the generation of rust. If the pH falls to values below 11 (in the case of degraded concrete the Ph is close to 6.5), the passive layer starts to crack, becoming no more able to protect the spread of corrosion and leading to a decrease of the mechanical properties (strength and ductility) of rebars. The effects of corrosion on steel reinforcing bars can be summarized in three main aspects, that are: the reduction of the cross section of the bar (mass loss) with consequent decrease of the load carrying capacity [11, 18], phenomenon that increases with the duration of the exposure time, the cracking and spalling of concrete that leave reinforcements more exposed to buckling phenomena and, finally, a sensitive reduction of the ductility, expressed in terms of elongation to maximum load (A_{gt}).
In *Rusteel* project, a detailed investigation of the mechanical behaviour of corroded steel reinforcing bars was developed, in order to individuate the effects of aggressive environmental conditions on both the tensile and the low-cycle fatigue mechanical properties; this last condition, in fact, was not widely investigated and only some works are presented in the current literature [10, 18]. In order to correctly reproduce the effects of aggressive environmental conditions, a detailed preliminary research about the most common and convenient techniques of accelerated corrosion tests was executed, resulting in the selection of accelerated salt spray chamber on the base of the reduced time of execution and of the effectiveness of the corrosion process. A specific protocol for the execution of accelerated corrosion tests in salt-spray chamber (based on prescriptions presented in ISO 9227 [19]) was elaborated in collaboration with the other partners of the research project.

The protocol foresees the execution of wet/dry cycles of 90 minutes (90 minutes dry, 90 minutes wet, resulting in 8 cycles/day) with a pH of the salt spray chamber ranging between 5.5 and 6.2. Specimens of 500-600 mm length shall be opportunely prepared protecting them with a wax cover leaving free to corrode only a central part of about 20 mm (or the medium distance between subsequently ribs, Figure 3b-3c); the specimens shall be positioned in salt spray chamber with a slope of 60° respect to the vertical walls of the chamber in order to prevent salt generation (Figure 3a). After the end of the exposure period and before the execution of mechanical tests, steel corroded rebars shall be maintained at a temperature lower than -5°, in order to kept inside the Hydrogen volatile part eventually developed during the salt-spray tests, that can lead to premature brittle failures of rebars.
On the corroded samples, both monotonic and cyclic tests shall be executed; nowadays, cyclic tests are ongoing, while some preliminary results of monotonic tensile tests can be presented.
A reduced set of steel reinforcing bars was selected to be subjected to corrosion for periods of 45 or 90 days of corrosion (Table 4). Figure 4 shows the stress-strain diagrams obtained from tensile tests executed on different rebars of 16 mm diameter, steel grade B450C, after 45 and 90 days of exposure in salt-spray chamber.

ID	*Quality*	ϕ	*Producer*	*Surface*	*Process*	*Salt-Spray chamber (45 days)*		*Salt-Spray chamber (90 days)*	
-	-	-	-	-	-	*N° Tensile*	*N° LCF*	*N° Tensile*	*N° LCF*
1	B500B	16	2	Ribbed	TEMP	3	-	6	5*
2	B450C	16	2	Ribbed	TEMP	3	-	6	5*
3	B400C	16	2	Ribbed	TEMP	3	-	6	5*
4	B400C	16	1	Ribbed	MA	3	-	3*	5*
5	B500A	12	1	Ribbed	CW	3	-	3*	5*
6	B500B	25	1	Ribbed	TEMP	3	-	3*	5*
7	B500B	12	2	Ribbed	STR	-	-	3*	5*
8	B400C	25	1	Ribbed	MA	3	-	3*	5*
9	B450C	12	2	Ribbed	STR	-	-	3*	5*
10	B450C	25	1	Ribbed	TEMP	3	-	3*	5*

Table 4: Steel bars selected for salt-spray chamber tests and tests foreseen.

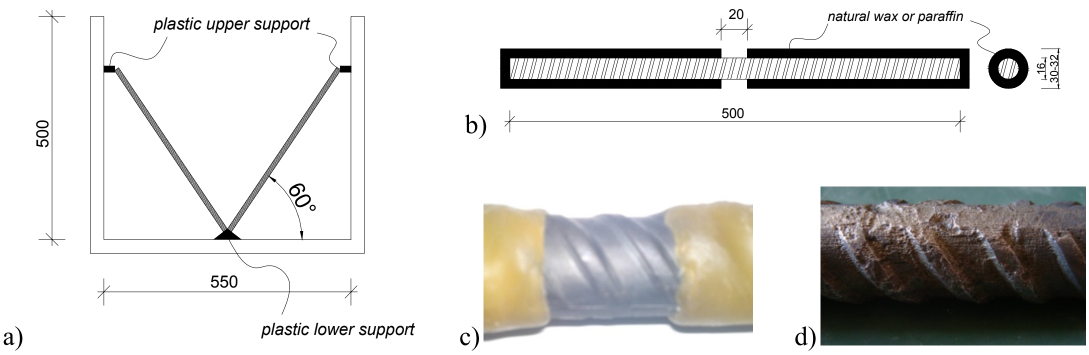
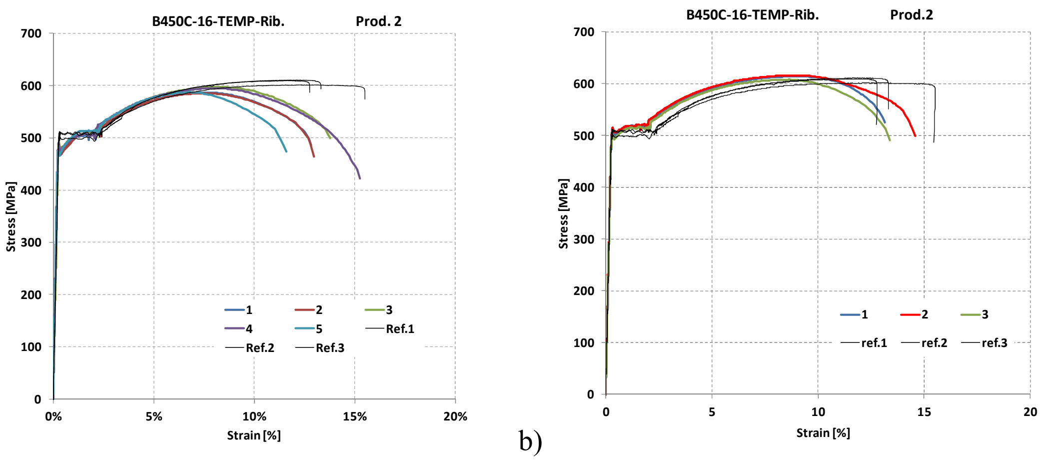


Figure 3: a) Disposition of rebars in salt-spray chamber, b-c) protection of rebar with paraffin or wax, d) one specimen after 90 days in salt-spray chamber.

In Figure 4, continuous black lines represent the results of tensile tests on reference uncorroded rebars. As visible, corrosion phenomena leaded to some modifications both in strength and in ductility; the reduction of the yielding strength is evident especially after 90 days of exposure (Figure 4a); the shape of the stress-strain diagram at yielding is also modified. Different steel grades provided similar results. Table 5 summarizes preliminary results obtained from corrosion tests on B450C, B500B and B400C rebars (TempCore), diameter 16 mm; in the table, both modifications of strength and ductility and mass loss are shown.



a) b)

Figure 4: Stress-Strain diagrams for bars B450C-16-Tempcore after a) 90 days of exposure, b) 45days of exposure.

BAR LABEL	UNCORRODED			CORROSION 45 days				CORROSION 90 days			
	R_e	R_m	A_{gt}	Mass Loss	R_e	R_m	A_{gt}	Mass Loss	R_e	R_m	A_{gt}
	MPa	MPa	(%)	(%)	MPa	MPa	%	%	MPa	MPa	%
B400C-16-Temp-01	428	546	16.4%	9.87%	444	550	8.4%	13.6%	398	525	7.1%
B400C-16-Temp-02	438	541	15.6%	13.9%	449	548	7.5%	18.9%	401	521	5.8%
B400C-16-Temp-03	438	555	15.6%	15.5%	436	554	9.0%	12.2%	405	525	6.4%
B400C-16-Temp-04								15.9%	417	519	7.5%
B400C-16-Temp-05								16.0%	411	-	7.6%
B450C-16-Temp-01	507	611	13.8%	7.9%	509	614	6.9%	14.6%	481	600	4.3%
B450C-16-Temp-02	501	601	15.0%	7.5%	511	616	6.2%	6.1%	484	598	4.4%
B450C-16-Temp-03	510	686	12.0%	11.1%	504	608	5.7%	8.7%	500	611	5.1%
B450C-16-Temp-04								6.9%	497	608	5.7%
B450C-16-Temp-05								8.3%	481	600	4.1%
B500B-16-Temp-01	472	577	11.5%	20.8%	500	610	9.1%	24.3%	492	608	5.7%
B500B-16-Temp-02	503	687	13.9%	19.1%	491	604	6.3%	16.9%	477	596	4.6%
B500B-16-Temp-03	503	604	11.4%	25.6%	492	604	7.5%	44.8%	482	611	5.0%
B500B-16-Temp-04				-			-	16.9%	485	606	5.1%
B500B-16-Temp-05				-			-	27.7%	491	603	5.0%

Table 5: Results obtained from tensile tests on corroded steel bars (45 and 90 days).

4 Definition of the ductility seismic demand on steel bars

4.1 Selection and design of reinforced concrete case study

Several different reinforced concrete buildings were designed according to the prescriptions imposed by actual European and Italian standards [1, 2] for capacity design in seismic areas; in particular, four different functional destinations (commercial, residential, office and car park), corresponding to four different plans and elevations, were assumed.

The reinforced concrete structures were designed considering different levels of p.g.a., respectively equal to 0.25g and 0.15g for high or medium seismicity area, different levels of ductility (HDC or LDC) and, above all, different steel grades for reinforcements (B450C, B400C and B500B), in order to represent the effective European scenario of constructions. A preliminary detailed pre-sizing of the structures using static linear analysis, followed by the execution of Linear Modal Analysis with q factor, aimed at the optimization of the design in terms of ductility on steel reinforcements; this condition is necessary for the individuation of the maximum seismic demand on steel rebars through the execution of non-linear analyses. In the present paper, preliminary results of non-linear analyses executed on a residential building are presented. The selected case study was designed for a p.g.a. level equal to 0.25 g and considering a soil type of category B; concrete C25/30 and steel grade B450C were used respectively for concrete and longitudinal steel reinforcements and stirrups. The building was designed for High Ductility Class (HDC). Figure 5 shows the plan of the selected case study.

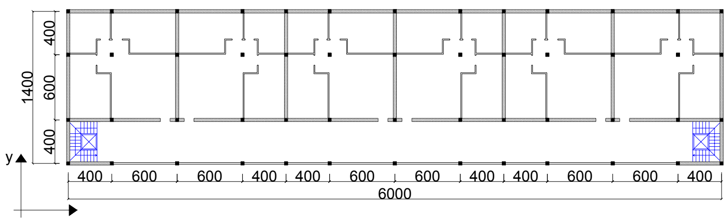


Figure 5: General plan of MRF residential case study building.

4.1 Elaboration of non linear models

For the determination of the seismic ductility demand on steel reinforcements, non linear bi-dimensional fibre models of case studies were realized using OpenSees software. Beams and columns were modeled as “beam with hinges” (BWH) elements: each element is divided into three different parts, two plastic hinges at the ends with defined length (L_p) and section, and an elastic central part, for which only the area section and the elastic modulus of material are required.

Elements’ sections in correspondence of plastic hinges were modelled as fibre sections; the constitutive laws of steel and concrete shall be able to represent both

the global and the local behaviour of the structure, the section and the rebars. In order to calibrate the constitutive non-linear models of materials, the experimental results of cyclic tests on simple structural elements [20, 21] were used.
For the modelling of concrete, the Braga-Gigliotti-Laterza (BGL) model [22] was used; this model, compared with traditional ones [23, 24, 25] allows to directly take into account the confinement contribution due to longitudinal steel reinforcements, layout and spacing of transversal stirrups, not needing complicate computational effort for the determination of the confinement coefficient. The BGL model was recently implemented in OpenSees [26]. As regards the constitutive law for steel reinforcing bars, the influence of slip phenomena between the reinforcements and the surrounding concrete shall be considered; even if for moderate loads the assumption of perfect bond between steel and concrete can be considered exact, the progressive increase of external actions leads to high relative displacement between concrete and bars, resulting in different strains in bars and concrete [27]. Traditional constitutive laws for steel that do not take into account the effects of relative slips between the reinforcement and the surrounding concrete, are consequently not able to represent the correct level of deformation on steel fibers of a reinforced concrete section. Refined models were presented in the past literature [27, 28] for the representation of slip phenomena between reinforcement and surrounding concrete; these models, elaborated for smooth rebars with hook anchorages, able to perfectly represent the experimental behaviour of structural elements both in monotonic and cyclic loading conditions, required a high computational effort: parametric analyses executed by Filippou et al. [27] evidenced the necessity of using, for a small structural rebar (length equal to 25 diameters), at least four integration points, resulting heavy to employ in non-linear analyses of ordinary buildings. Braga et al. [29] correctly reproduced the experimental cyclic behaviour of beam-column joints of existing reinforced concrete structures with smooth bars, using a simplified model taking into account the effects of relative slips. The bond-slip constitutive laws was developed for smooth rebars with anchorages, evaluating in particular the influence of the hook on the mechanical behaviour of the reinforcement.
The models herein presented provide stress-slip (σ-u) relationships; fibre models, nevertheless, require stress-strain (σ-ε) laws, involving an arbitrary "shift" to facilitate the implementation of the σ-u laws in the fibre section models [27].
In the present work, the tensile stress-slip (σ-u) model previously elaborated by D'Amato, Braga et al. [26, 29] was extended to the case of ribbed bars in new constructions, including some aspects (i.e. the real hardening behaviour of steel) not previously considered. The main assumptions at the base of the presented model are: 1) an elasto-plastic relation between bond stress and slip (τ-u) (Figure 6a), in agreement with the results obtained by Verderame et al. [30]; 2) the tensile stress-strain (σ-ε) law is represented as elasto-plastic with hardening (Figure 6b), and the slope of the hardening branch is defined in relation to the effective experimental tests executed on rebars; 3) the slip field is assumed bi-linear, with a first branch characterizing the behaviour before yielding and a second branch, with an increment of slope, that defines the behaviour in the hardening field (Figure 6c). The representation of the hook end as a linear elastic function, if the hook is present, is maintained, according to what proposed in past literature (Figure 6d).

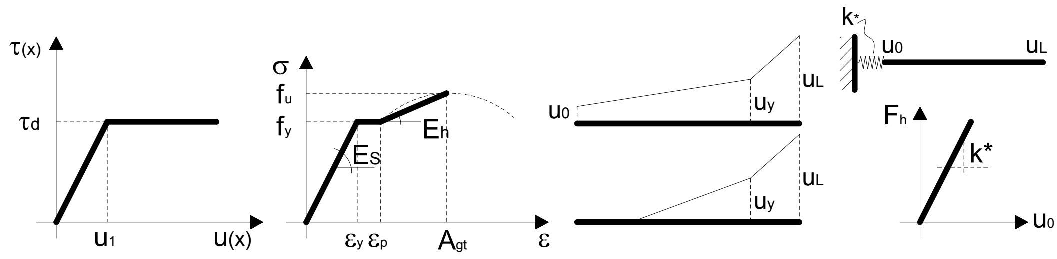


Figure 6: a) Bond stress-slip relation; b) stress-strain law for reinforcing bars; c) simplified slip field assumed; d) constitutive law assumed for hook.

Through the use of simple equations of equilibrium, compatibility and constitutive laws, a simple relation between axial stress on rebars and slip was obtained. For a steel reinforcing ribbed bar not characterized by the presence of hook in correspondence of one end (situation similar to the one in new constructions), the relative simplified slip field along the bar can be expressed as presented in equation (1), in which x in the general position along the length of the bar, L_y is the part of the rebar where the axial stress is higher than yielding stress (f_y), L_0 is the total anchorage length, u_y the value of the slip in correspondence of the free length when yielding is reached and u_L the free end slip in correspondence of the generic step of load.
The axial stress on steel reinforcing rebars can be expressed as presented in equation (2), in which the trend of bond stress is defined in relation to the value of slip in the generic point of the rebar.

$$u(x) = \begin{cases} \dfrac{x}{L_0} \cdot u_L & \textit{pre}-\textit{yielding} \\ & \textit{post}-\textit{yielding} \\ \dfrac{u_y}{L_0 - L_y} \cdot x & \textit{if } 0 \le x \le L_0 - L_y \\ u_y + \dfrac{u_L - u_y}{L_y} \cdot (x - L_0 + L_y) & \textit{if } L_0 - L_y < x \le L_0 \end{cases} \tag{1}$$

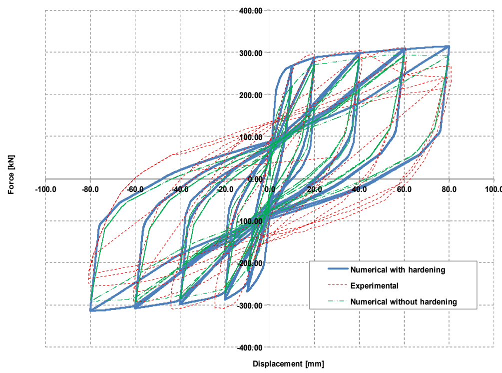
$$\sigma(x) = \frac{1}{A_b} \int \tau(x) \cdot \pi\phi \, dx \tag{2}$$

The length of the part of the rebar in which the yielding strenght is exceeded (L_y) can be evaluated considering the equilibrium of forces at the two ends of the bar interested by slips (3):

$$\begin{gathered} \int_0^{L_0 - L_y} \pi D \, \tau(x) dx = \frac{\pi D^2}{4} f_y \\ \int_{L_0 - L_y}^{L_0} \pi D \, \tau(x) dx = \frac{\pi D^2}{4} \left(\sigma(L_0) - f_y \right) \end{gathered} \tag{3}$$

Using the presented equations the axial stress-slip relationship is evaluated. For the shift from a stress-slip to a stress-strain relationship, a simplifying operation using as

parameter the length of plastic hinge L_p was used. For the definition of the plastic hinge length L_p to use in BWH elements, in the past literature many expressions [31], related to the geometrical and mechanical characteristics of structural elements, are provided. In the present paper the formulation given by Fardis [32] was used, considering the parameter a_{sl} equal to zero, since slippage phenomena were already taken into account in the constitutive model of material, as presented in the following.
The methodology herein presented was applied for the numerical representation of the experimental cyclic behaviour of a simple cantilever reinforced concrete column, with geometrical characteristics and section presented in [20]; the results, using both the preliminary model of Braga et al. [22] and the new one modified for introducing the effects of hardening, are presented in Figure 7. As visible, the presented model is able to lead to a good agreement between experimental and numerical results, both in terms of stiffness and strength, allowing the definition of a realistic stress-strain behaviour of steel reinforcing bars (Figure 7b).

a) 

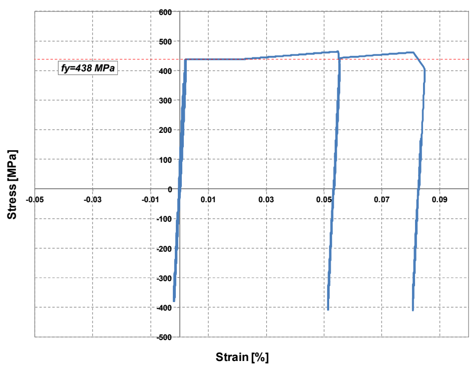
b) 

Figure 7: a) Comparison between experimental and numerical results [20]; b) Stress-strain diagram on steel fibre.

4.2. Preliminary Incremental Dynamic Analyses on a case study

In the present paper, the preliminary results of Incremental Dynamic Analyses executed on the case study are presented. IDA were executed using artificial accelerograms in agreement with the soil typology and response spectrum used for the design. The mean real mechanical characteristics of steel reinforcements, coming from the experimental tensile tests executed on 9 different specimens of steel grade B450C (TempCore process), diameter 16 mm were used: yielding strenght equal to 510 MPa, tensile strength 610 MPa and A_{gt} equal to 12.4%.
According to actual standards [1, 2], the capacity of reinforced concrete elements towards seismic action shall be evaluated through the definition of chord rotation and shear strenght, respectively for ductile (beams and column in flexure, with or without axial force) and brittle elements (shear in beams and columns).
The capacity θ_y of reinforced concrete structural members at Damage Limitation limit state (DL), expressed in terms of chord rotation at yielding θ_y, is evaluated using the expression A.10b presented in the Annex A of Eurocode 8 [2]:

$$\theta_y = \varphi_y \frac{L_V + a_v z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_V}\right) + 0.13\varphi_y \frac{d_{bL} f_y}{\sqrt{f_c}} \qquad (4)$$

in which φ_y is the yielding curvature of the element's section, $a_v z$ is the tension shift of the bending moment diagram, h is the height of the section, L_v the shear length, f_y and f_c respectively the strenght of steel reinforcement and concrete and d_{bL} the mean diameter of longitudinal rebars. The value of total chord rotation capacity (considering both the elastic and the inelastic part) at ultimate (Near Collapse, NC) limit state is evaluated with the following expression [2]:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016(0.3^{\nu})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225}\left(\min\left(9;\frac{L_v}{h}\right)\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}(1.25^{100\rho_d}) \qquad (5)$$

In which γ_{el} is equal to 1.5 or 1.0 respectively for primary and secondary elements, ν is the compression stress normalized to f_c, ω and ω' are the mechanical reinforcement ratio of the tension and compression longitudinal reinforcement, α is the confinement effectiveness factor, ρ_{sx} is the ratio of transverse steel parallel to the direction of loading, ρ_d is the steel ratio of diagonal reinforcement, f_{yw} and f_c are the strenght of stirrups and concrete respectively.
As regards the capacity of brittle mechanisms, the shear static strenght is evaluated according to the expressions presented in Eurocode 2 [3], both considering the static and the cyclic shear resistance, whose evaluation is necessary at Near Collapse (NC) limit state through the expression A.12 in Annex A of Eurocode 8 [2].
IDA were executed considering steps of p.g.a of 0.05g, until a maximum of 1.00 g; the design p.g.a. considered for the presented building was equal to 0.25g. Figure 8a shows the base shear-displacement curves obtained from non-linear static and dynamic analyses.
The evaluation of the structural behaviour of the selected building according to the expressions provided by Eurocode 8, is summarized in the Figure 9: with the filled square are evidenced those sections that reach their yielding capacity for p.g.a. equal to 0.35 g, with the filled triangle the ones reaching θ_y for p.g.a. equal to 0.40 g, the filled circle and the empty square represent those elements that reach the yielding respectively at 0.45 and 0.50 g and, finally the cross indicates the sections in which ultimate chord rotation occurs. For a p.g.a. level of 0.50 g a lot of structural elements are yielded (beams and columns of the first floor) but only for a very high level of p.g.a. (1.00 g) some elements reach the ultimate chord rotation limit (base section of 3rd floor columns and upper section of columns of the 4th floor). No shear mechanisms activate in beams or columns. In Figure 8b the interstorey drift profiles are shown; according to FEMA 356 [33], the interstorey drift limit for reinforced concrete structure should not exceed 1.0% or 4.0% (for permanent actions) respectively for Life Safety (LS) or Near Collapse (NC) limit state. As visible in Figure 8b, for p.g.a. equal to 0.40 g the interstorey drifts at 3rd and 4th floor are higher than 1%, and the situation get worst considering increasing seismic actions. The presented results shall be considered only preliminary results, further accurate investigations on the mechanical behaviour of rebars are still ongoing.

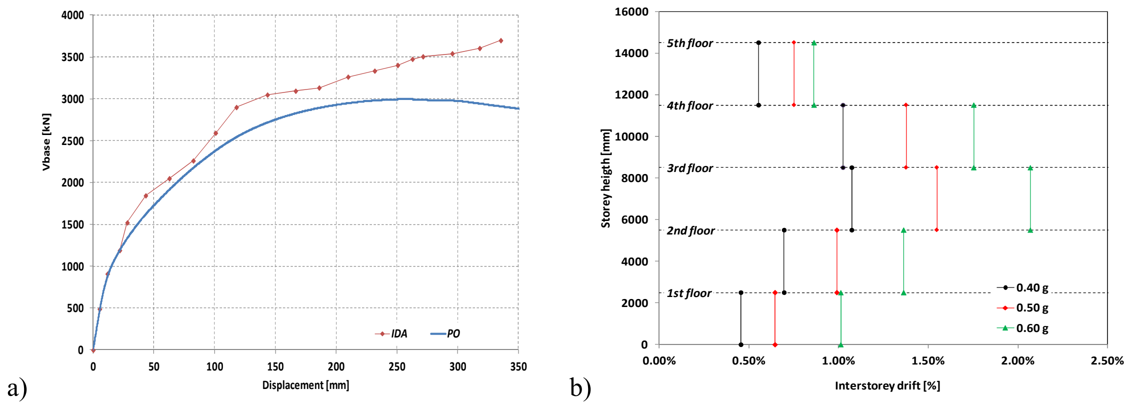


Figure 8: a) Base shear – displacement curves obtained from Pushover and IDA analyses; b) Interstorey drift profiles.

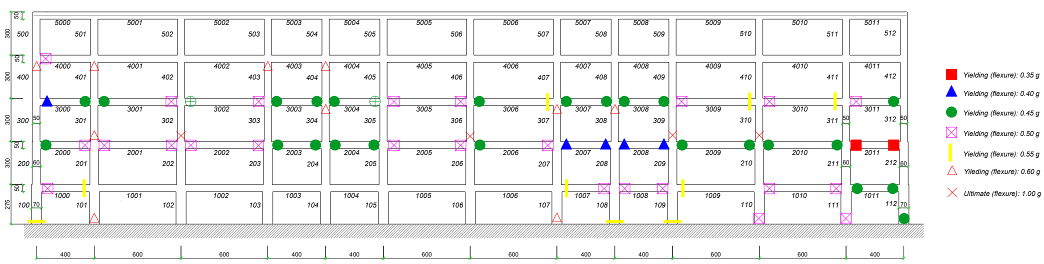


Figure 9: Activation of ductile mechanisms in beams and columns.

Figure 9 allows the individuation of the structural elements reaching the yielding for lower levels of p.g.a., according to the conventional expressions provided by Eurocode 8 for the determination of the chord rotation at yielding. Moreover, the elaboration of non-linear fibre models allows the individuation of the effective behaviour of steel reinforcing bars under seismic action, leading to the evaluation of the real maximum level of elongation imposed by the earthquake and, consequently, to the estimation of the ductility demand. Figure 10 shows some preliminary results of stress-strain diagrams on steel reinforcement fibers in beams 2007 and 2011 (between columns 107 -108 and 111 – 112, Figure 9), for a p.g.a. level of 0.25 g (equal to the one used in the desing), while Figure 11 refers to the behaviour of steel fiber in column n°107 for a p.g.a. level equal to 0.45 g. As visible, the maximum level of strain reached in steel reinforcing bars in the selected beam is equal to 6,5% and 2,7 % respectively for beam 2011 and 2007, while in the selected column, the required ductility appears slow, equal to 4.98% at 0.45g (at 0.25 g steel rebars are still in the elastic range).
The presented results show some inconsistencies between what predicted by standards and what really happens at fibre level: sometimes in fact, the levels of p.g.a. for which the elements reach the chord rotation at yielding (θ_y) according to Eurocode 8 are higher than the ones individuated through an accurate investigation of the steel fibre behaviour; this can be caused by the fact that the expressions presented in the standards are generally the results of interpolation of a large number

of experimental data, while a complete fibre model allows a complete and more realistic understanding of the effective structural behaviour.
Moreover, the results herein presented come from non-linear analyses executed using artificial accelerograms, that represent a very strong situation for buildings: further investigations, using real natural time histories, opportunely selected for maximizing the ductility demand on steel rebars, shall be executed and are still under elaboration.

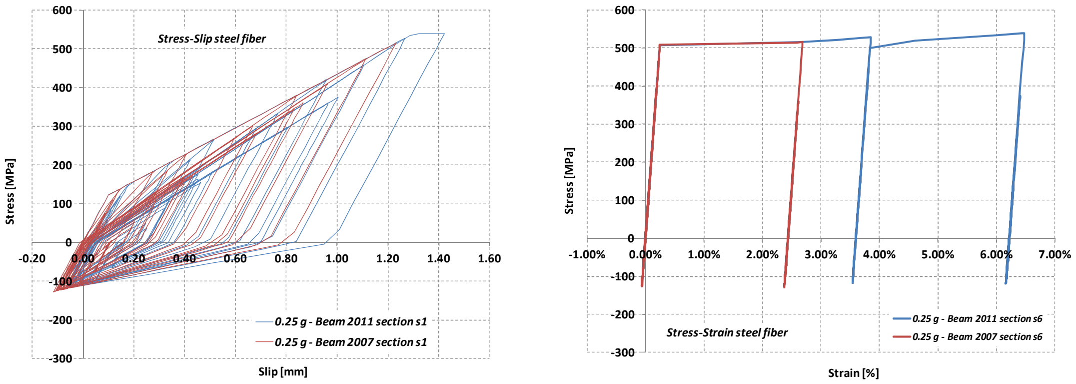


Figure 10: a) Stress-slip and b) stress-strain diagrams for bar B450C (16mm), beams 2007 and 2011, 0.25 g.

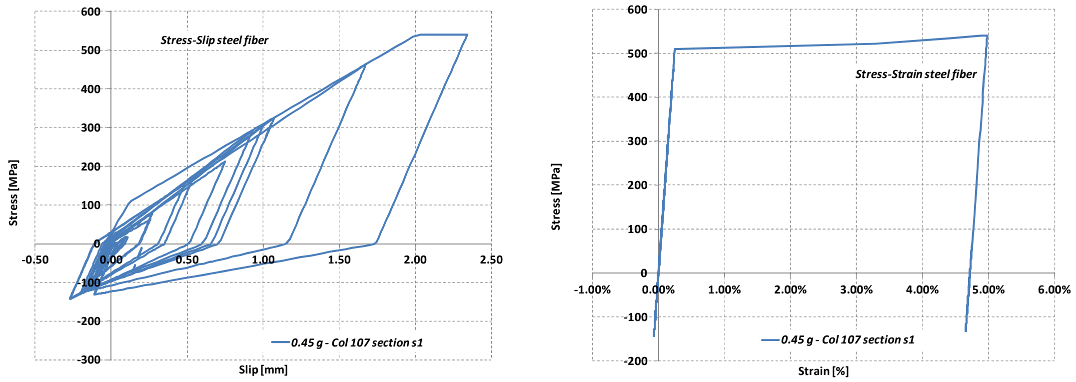


Figure 11: a) Stress-slip and b) stress-strain diagrams for bar B450C (16mm), column 107, 0.45 g.

5 Conclusions and remarks

In thispaper, the preliminary results of *Rusteel* research project are presented. In order to understand the influence of the combined effects of seismic action and aggressive environmental conditions on steel reinforcements, two different protocols, respectively for the execution of low-cycle fatigue and corrosion tests, were elaborated. The execution of experimental tests on corroded steel reinforcing bars showed the influence of corrosion phenomena on the reduction of mechanical properties, both in terms of strength and ductility (A_{gt}), allowing the individuation of the effective capacity of rebars, under monotonic and cyclic loading conditions. Moreover, the methodology adopted for the evaluation of the effective level of

ductility required by seismic action to an ordinary reinforced concrete building is showed.
A new constitutive law for steel reinforcements, able to take into account the effects of slip between bars and surrounding concrete, was elaborated on the base of the model proposed by D'Amato, Braga et al. [30, 33] and then modified to consider strain-hardening phenomena. Moreover, preliminary Incremental Dynamic Analyses executed using artificial accelerograms are presented, evidencing the level of strain imposed to rebars in reinforced concrete buildings. Further investigations and simulations about both ductility *demand* and *capacity* are still in execution.

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