# Seismic Assessment of Reinforced Concrete Frames: Influence of Shear-Flexure Interaction and Rebar Corrosion

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**Abstract.** The stock of existing buildings across most of the European earthquake-prone countries has been built before the enforcement of modern seismic design codes. In order to assure uniform levels of safety and reduce the social and economic impact of medium to high earthquakes costly seismic intervention plans have been proposed. But their application, in order to define which building should primarily be retrofitted, requires adequate vulnerability assessment methodologies, able to model the effective non-linear response and to identify the relevant failure modes of the structure. In the case of reinforced concrete (RC) buildings, due to the lack of application of capacity design principles and the aging effects due to exposition to an aggressive environment, existing structures can exhibit premature failures with a reduction of available strength and ductility. In the last couple of decades some state-of-the-art simplified models aiming at capturing the complex interaction between shear and flexural damage mechanisms as well as behavior of rebar corrosion have been proposed in specialized literature and, in some cases, implemented in regulatory building codes and guidelines. The present paper presents how those phenomena that have a significant impact in reducing the element capacity in term of strength and energy dissipation can be implemented in the assessment of the structures.

**Keywords:** Earthquake engineering · Seismic assessment · Reinforced concrete · Shear-flexure collapse · Rebar corrosion

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## 1 Introduction

The seismic vulnerability of the existing building stock in Italy and in most of the others earthquake-prone European countries is a topic of serious economic and social concern [5, 32, 35]. The need for the adoption of a systematic retrofitting or rebuilding intervention scheme grows as time progresses and existing structures become older and degrade further [9–11].

Existing buildings have been conceived when many sites were not classified as seismically prone and therefore neglecting in the design process the seismic action at all; even if the site was recognized as subject to earthquakes, the level of seismic demand was much lower than the one currently adopted [27, 28, 33, 34].

Furthermore the design standards enforced at the time of their construction relied on the admissible stress method, and, therefore, the resulting structures where designed to respond in the elastic range, leaving a false sense of security that the actual resistances (well beyond the nominal ones considered in the design process) would be assured by the adopted safety coefficients and by the inelastic behavior of materials and structures. On the contrary, the lack of a hierarchy of strengths that would be lately assured by the implementation, in modern codes, of capacity design principles, would not prevent the occurrence of non-ductile failure modes reducing the extent of the inelastic response.

Finally, many existing buildings are affected by significant structural degradation which leads to a decrease in their performance, especially in terms of safety requirements. This is either due to the neglecting during their conception of durability features or the premature natural decay of mechanical characteristics of materials and elements. One of the most dominant deterioration mechanisms of reinforced concrete structures is corrosion of reinforcement.

The situation of Italy is emblematic: less than 23% of buildings have been built after 1980 (when a major revision of the seismic code was implemented as a consequence of the Irpinia earthquake), but this case is absolutely not isolated across Europe as shown in a recent survey of building seismic exposure [29].

## 2 Objectives and Methods

The present study aims at contributing to the modeling capability of reinforced concrete (RC) frame elements under seismic loading. To pursue this objective, a comprehensive models has been developed, considering the most relevant damage modes affecting the response of existing RC frame elements.

After an examination of the general characteristics of the Italian building stock, the research has focused on the older RC structures designed and constructed between 1950–1960. The scope is modeling the seismic response of significant elements and sub-assemblages, considering the presence of corroded reinforcements.

During hydration of cement, an alkaline pore solution (pH up to 13.5) is obtained and in such environment a protective oxide film is formed spontaneously around the steel rebars embedded in RC elements. Passive film prevents damage of steel surface from corrosion phenomena, however, carbon dioxide or the presence of chloride ions

on the steel surface could destroy the protective oxide film so that corrosion could initiate.

Carbon dioxide produces the carbonation of cover concrete and it generally takes place on whole surface of steel resulting in a uniform corrosion, instead when the aggressive ions exceeds a critical threshold value, the protective passive layer on the rebar surface breaks down initiating the corrosion process giving as a result a localize corrosion, indicated as pitting corrosion. The later phenomenon is more relevant than the first one.

Those ions are generally present in the environment and are capable to reach the interior of the concrete element thanks to different penetration mechanisms, such as diffusion, capillary suction, permeation, migration or a combination of those mechanisms. Hence, the capability of a concrete to prevent or make negligible corrosion phenomena is due to concrete cover which represents a sound protection for the reinforcement acting like a physical barrier that prevents the chemicals (chloride ions, carbon dioxide, etc.) from reaching the reinforcement, but also, as already said, thanks to the natural alkalinity of the concrete that helps forming a passive film on the steel surface, thus chemically protecting the embedded bar against corrosion.

Since the corrosion products formed at the steel/concrete interface have a mass density lower than steel, a volume expansion is obtained associated to tensile stresses and the cracking of the concrete cover, thus favoring further the ingress of the aggressive agent from outer environment [1].

The corrosion process is associated to the formation of different damage in structural elements, such as the loss of cross-sectional area of the reinforcing bars, the reduction of bond between bars and surrounding concrete and the crack propagation into the cover ultimately producing spalling and delamination of the concrete protective layer.

Particular emphasis in the development of the model has been given to the study of the shear-flexure interaction, that is deemed a pivotal issue for the assessment of existing buildings. A typical characteristic of existing buildings columns is the presence of low percentage of transversal reinforcement (poorly detailed and highly spaced stirrups).

According to the experimental evidence, structures with those characteristics, that would be considered substandard according the construction practice adopted today can show essentially two types of failure: a premature buckling of the steel rebars under compression or a premature shear failure, limiting the capacity to undergo inelastic deformation and therefore dissipate energy. Indeed the widening of flexural–shear cracks due to cyclic inelastic deformations, especially in the plastic-hinge region, reduce the ability of concrete to transfer the shear action through mechanisms relying on aggregate interlock. As a consequence there is a sectional shear capacity reduction, showing that under cyclic loading the shear strength of columns can be heavily dependent on the inelastic deformations and that shear strength degrades with ductility more quickly than flexural strength. Thus, it is important, when assessing the seismic response of existing structures to take into consideration in the numerical model the insurgence of those complex interaction phenomena affecting the overall response of the building structures. Instead, with regards to a premature shear failure, several researches and studies on shear strength have evidenced that, even in the case those

columns have been initially designed with nominal shear capacity exceeding the shear in equilibrium with flexural yielding, those piers could still fail early in shear due to the detrimental action of inelastic flexural deformations on the shear strength.

Furthermore, one of the most common failure mode of RC elements, observed in existing structures after an earthquake, is due to the buckling of longitudinal reinforcement [16]. Several researchers have investigated this problem analytically through either Euler theory of elastic buckling [2] or FEM modeling [6, 21] as well as experimentally [4, 12, 16, 25, 37].

Essentially, the instability of a longitudinal reinforcement under compression involves in a decrease of stress with an increase of strain (i.e. the post-yielding slope is negative), contrary to what happens in tension. An accurate knowledge of the stress-strain relationship of reinforcing steel is needed to provide the capability to model the inelastic buckling of rebars.

## 3 The Italian Building Stock

In Italy the general characteristics of the building stock at nationwide level can be obtained by the Census campaigns. The Census is conducted and elaborated by Italian National Institute of Statistics (ISTAT) every ten years to screen important information about the state of population and dwellings [13–15].

The Census data are collected and aggregated at different geographical levels. The basic unit for data collection is the single household and dwelling, but each dwelling is classified as being located within a building, of given characteristics as explained below.

In particular for each census tract is specified the number of buildings, the occupancy type (e.g. residential or not), the state of conservation, the number of stories, the construction year, the main structural typology (Reinforced Concrete or Masonry). A list of available information that are useful for an evaluation of the state of conservation and of the seismic vulnerability of the residential building stock is reported in Table 1. For sake of exemplification in Table 1 the variables have been taken from the 2001 census, but the data structure has been kept essentially the same throughout the subsequent campaigns.

In Table 2 an overview of the whole Italian stock is given: the number of buildings belonging to each age of construction and structural typology class is reported. As shown, many buildings belong to the "before 1919" class (more than 20%). In terms of typology, masonry buildings are the more numerous in absolute value (about 65%), but RC buildings (26% in total) become the prevalent typology in more recent years (starting from the 1981). Buildings belonging to "other" category are much less numerous. In order to protect privacy, the collected data are released by ISTAT only in aggregated format.

The publicly available database is organized for the entire nation adopting as minimum territorial extension the Census tract. A Census tract is a small, relatively permanent statistical geographical subdivision of a territory, designed to be relatively

homogeneous with respect to population characteristics, economic status and living conditions. In highly urbanized areas, a census tract generally has the dimensions of a building block. In Fig. 1 is mapped the percentage of structural type at municipality level.

**Table 1.** Residential building Census variables

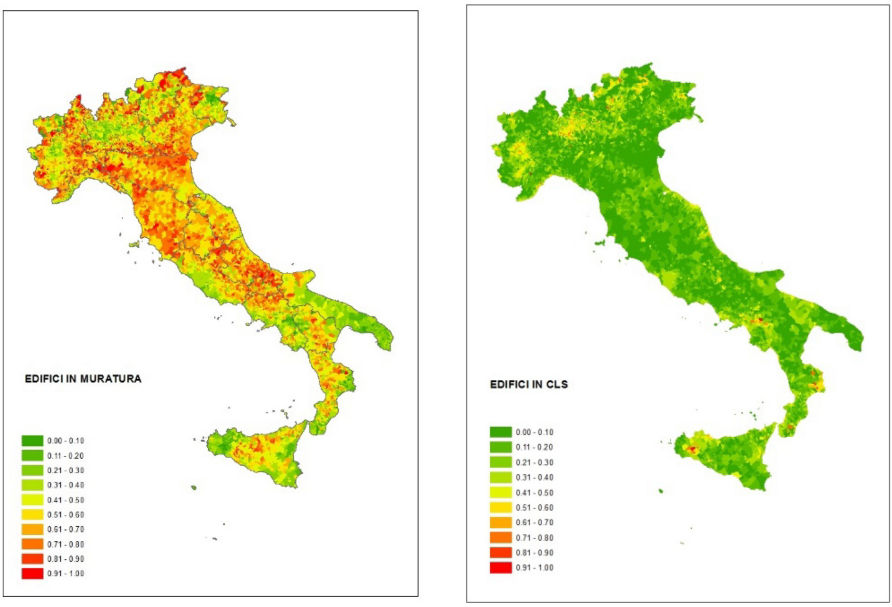
| Structural typology |
|---|
| Masonry (M) |
| Reinforced Concrete (RC) |
| Other |
| Number of stories |
| 1 |
| 2 |
| 3 |
| 4 or more |
| **Construction time** |
| Before 1919 |
| From 1919 to 1945 |
| From 1946 to 1961 |
| From 1962 to 1971 |
| From 1972 to 1981 |
| From 1982 to 1991 |
| From 1992 to 2001 |

**Table 2.** Number of residential buildings in Italy per construction time and structural typology.

| Construction time | Masonry | Reinforced concrete | Other |
|---|---|---|---|
| Before 1919 | 2'026'538 | – | 123'721 |
| From 1919 to 1945 | 1'183'869 | 83'413 | 116'533 |
| From 1946 to 1961 | 1'166.107 | 288'784 | 204'938 |
| From 1962 to 1971 | 1'056'383 | 591'702 | 319'872 |
| From 1972 to 1981 | 823'523 | 789'163 | 370'520 |
| From 1982 to 1991 | 418'914 | 620'698 | 250'890 |
| From 1992 to 2001 | 228'648 | 394'445 | 167'934 |
| **Total** | **6'903'982** | **2'768'205** | **1'554'408** |

As it is clear from the construction period break-down in Table 1, the Italian building stock has been built mostly after the II World War, during or right after the decade of the so-called 'Italian economic miracle' (from late 50' through early 60') and it is formed by buildings that often show low standards of quality.

The need of a strategic renovation of the built environment represents a crucial issue to assure the safety of population and the resilience of communities, to improve the quality of life and to foster the recovery of the construction sector.




(a) (b)

**Fig. 1.** Exposure: percentage of number of buildings for structural type aggregated at municipality level. (a) masonry buildings and (b) RC (reinforced concrete) buildings.

In the last years, the attention of public policies, thanks to the impulse of the European Union, has been driven mainly to the aspects of energy efficiency. However, a significant portion of Italian territory is earthquake-prone, as it has been unfortunately shown by recent events, so that, the possibility to combine energy and seismic retrofitting turns out to be a crucial opportunity.

The classification of the Italian territory in zones, recognized to be seismically prone and where special earthquake-resistant measures have to be adopted when constructing a building, started in 1909, after the devastating earthquake that in 1908 hit the cities of Messina and Reggio Calabria. The subsequent classification of the Italian territory, with the creation of new seismic areas, followed the major seismic events. Generally their occurrence, anticipated the creation of new seismic areas and the enforcement of new seismic regulations. Nowadays, after a revision of seismic zonation based on a site specific probabilistic seismic hazard analysis, the whole Italian country, with just the exception of Sardinia, is considered earthquake prone. Unfortunately, most of the building stock dates to periods when the application of seismic provisions was not mandatory in most of the country.

As a consequence, even the buildings with a structure supported by a RC frame, have been conceived without considering seismic provisions and are affected by important structural defects. In these buildings, designed mainly for gravity loads only, the resisting elements are arranged in just one direction, leaving the structure weak and flexible in the orthogonal direction. Furthermore, the structural codes neglected the basics of the capacity design and proper detailing, producing low ductile elements both at global and at local level, with for instance the potential formation of story collapse mechanisms.

Finally, the situation may be aggravated by the use of low quality or time degradation of materials. Indeed, together with the seismic risk, corrosion is one of the main factors causing deterioration of reinforced concrete buildings.

A periodic monitoring of structural condition would be required to assess the state of advancement of the corrosion phenomena and eventually repair their adverse effects. However, this task is judged too expensive to be conveniently undertaken over a large populations of buildings, distributed over large areas.

## 4 Numerical Model for the Seismic Assessment

The seismic response of a reinforced concrete frame element can be studied through a numerical model based on a componential approach, in which three main response mechanisms coexists: flexure, shear and bonding. As schematically depicted in Fig. 2, the lateral displacement of a frame column can, indeed, be theoretically seen as the sum of the displacements produced by those three components.

Flexure is by far the most relevant of those mechanisms and it is also the most investigated. The bonding is essentially responsible for the additional displacement due to the slippage of the longitudinal reinforcing bars in the anchoring concrete.

Finally, regarding shear deformation, it is admitted that in slender columns the contribution due to shear is relatively small, compared to flexure, so that in professional practice the shear influence has been generally neglected.

Despite this, shear deformations have a significant effect if the reinforced concrete element experiences damage in shear especially after diagonal cracking.

### 4.1 Flexural Behavior

The flexural behavior has been modeled with a distributed plasticity approach. In order to analyze the non-linear response of the element cross-section, it has been discretized in fibers, as depicted in Fig. 3.

A reinforced concrete section is essentially composed by:

- Unconfined concrete (in the cover)
- Confined concrete (in the core) and
- Steel reinforcing bar.

Three different kinds of constitutive relationship have been used to model the mechanical behavior of those materials and assigned to relevant fibers within the element sections.

The concrete has been modeled using the Popovics constitutive law [31]. The effect of stirrup lateral confinement on the core concrete has been considered adopting the Mander et al. model [20]. The longitudinal steel reinforcement has been modeled by the Menegotto and Pinto [24] constitutive law.

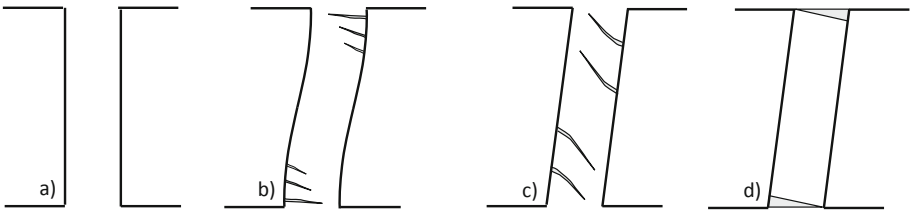



**Fig. 2.** Components of horizontal displacement. a) original configuration; b) bending deformation; c) shear deformation and d) bonding deformation.

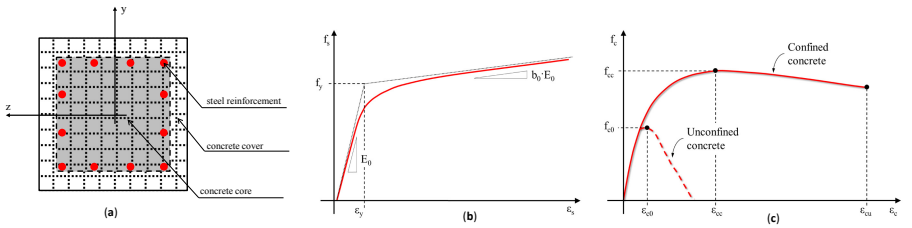



**Fig. 3.** Properties of a RC section: (a) fiber discretization of the section (y and z are the element local axis); (b) steel reinforcement and (c) cover (unconfined) and core (confined) concrete constitutive law.

The effect of corrosion over the steel rebar has been considered through the reduction of the nominal diameter:

$$D' = D \cdot \sqrt{\left(1 - \frac{m_0 - m}{m_0}\right)} \tag{1}$$

where $D'$ and $D$ are the diameter of the corroded and uncorroded rebar, respectively, $m_0$ is the mass of the uncorroded rebar and $m$ is the mass of the corroded rebar. The ratio $(m_0 - m)/m_0$ measures the amount of corrosion in terms of mass.

Generally speaking, any mechanical property $X'i$ (i = stress/force/strain) of the corroded rebar in case of pitting corrosion may be calculated by:

$$X_i' = X_i \cdot \left(1 - \beta_i' \cdot \Psi\right) \tag{2}$$

where *Xi* (i = stress/force/strain) is the mechanical property of the uncorroded rebar and *β'i* is the coefficient which takes into account the pitting corrosion and can be evaluated as in Fig. 4.

A calibration of the coefficient *β'i* can be performed based on experimental data by Meda et al. [23] for i = $f_y$, $f_{max}$, $f_u$, $\varepsilon_{max}$ and b. The parameter *β'i* is obtained starting from Eq. 2 for given experimental corrosion percentage, uncorroded and corroded rebar properties i. These values are the red crosses in Fig. 4. Ψ is the corrosion percentage which can be evaluated by:

$$\Psi = 100 \cdot \left(\frac{m_0 - m}{m_0}\right) \tag{3}$$

The experimental effect of corrosion over the behavior of the rebar in tension and in compression is reported in Fig. 5.

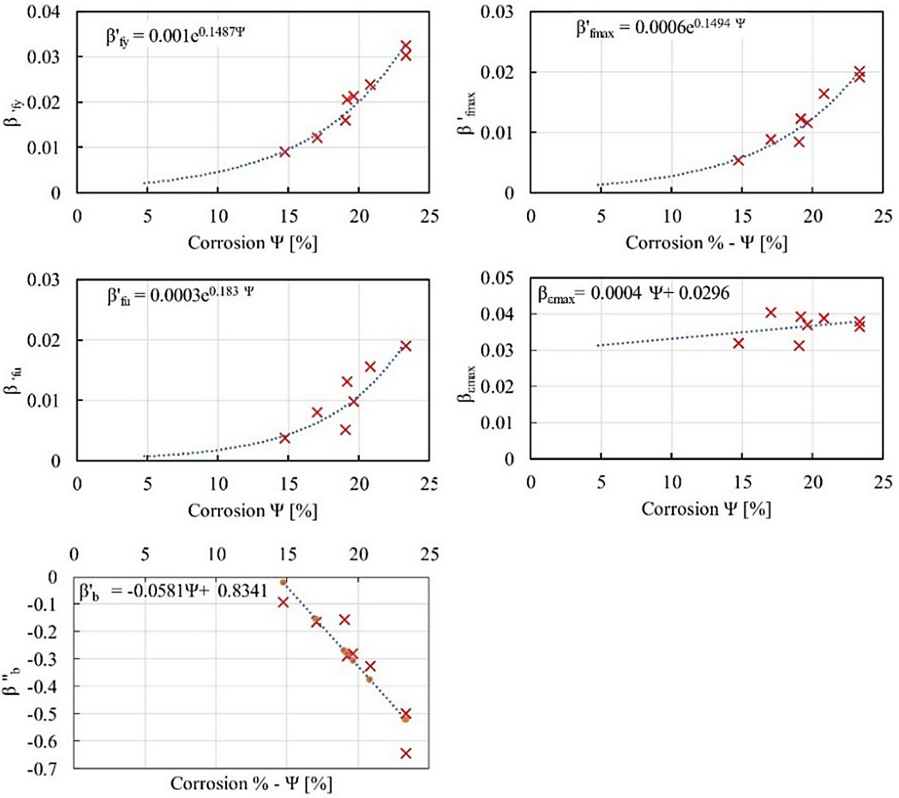



**Fig. 4.** Calibration of *β'i* for yield $f_y$, maximum $f_{max}$ and ultimate $f_u$ stresses; maximum strain $\varepsilon_{max}$; hardening ratio *b*. Red crosses represent the experimental data, whilst the black dot lines the analytical interpolation curves [17].

In compression, buckling behavior can arise when the slenderness of the longitudinal rebar $\lambda$ is greater that the critical value of the slenderness $\lambda_{cr}$, where $\lambda$ is the slenderness of the longitudinal rebar defined as the ratio between the free length of the longitudinal rebar $L$ and its diameter $D$, according to the Monti and Nuti constitutive relationship [25, 26] depicted in Fig. 6.

Indeed, regarding the instability of compressed steel rebars, transverse reinforcements is required not only to prevent the shear failure or to provide the confinement of concrete, but should also provide the lateral support to prevent early buckling phenomena in reinforcing steel under compression [30].

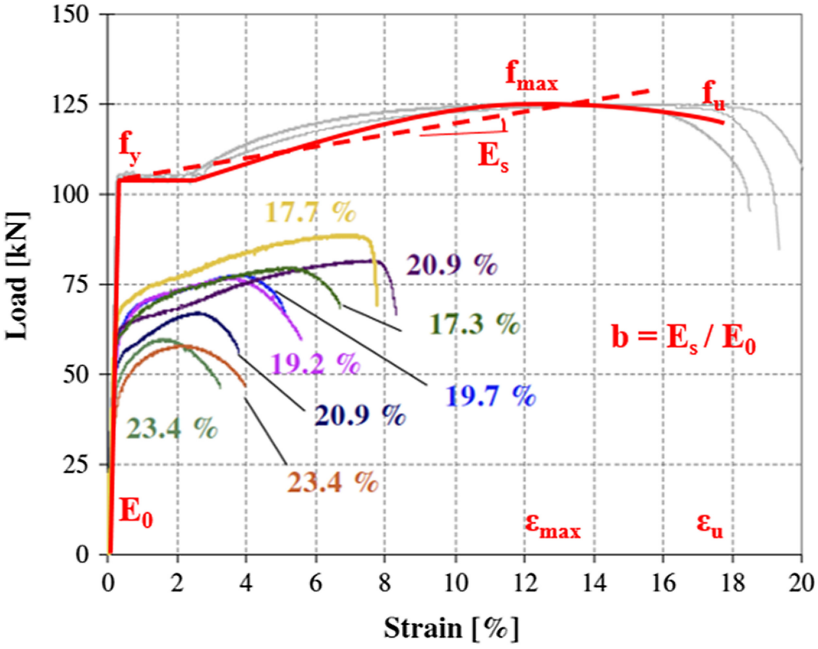



**Fig. 5.** Load-strain curves for corroded rebars in RC members for different corrosion percentages [18].

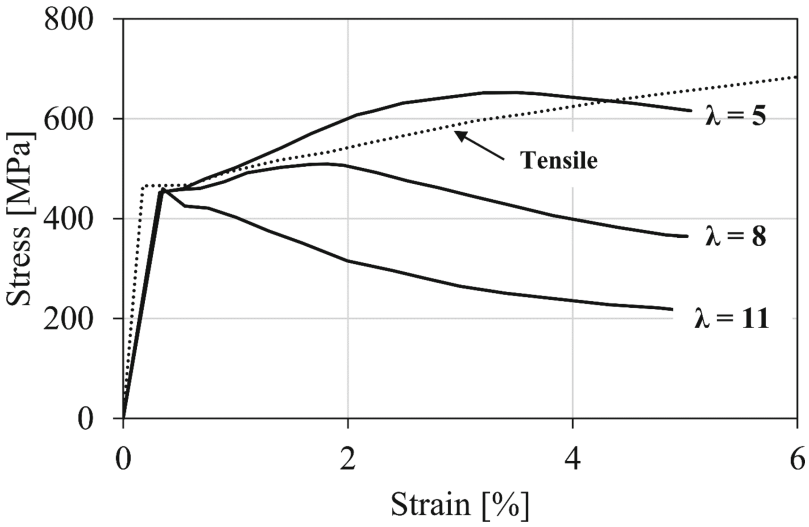



**Fig. 6.** Monotonic compressive stress-strain response with different slenderness ratio ($\lambda = L/D$ where $L$ is the free length of longitudinal rebar and $D$ is the diameter of longitudinal rebars) [26].

### 4.2 Slippage Behavior

The slippage of the reinforcing bars will cause rigid-body rotation of the column, that produces an additional source of the deformation, that can be significant, as depicted in Fig. 7 [3, 7, 36]. The slippage has been implemented in the numerical model through a couple of rotational slip springs at the top and bottom of the element with a linear constitutive relationship.

### 4.3 Shear Behavior

The conceptual model accounting for the flexure-shear coupling follows the formulation codified in the ATC seismic design guidelines. In this model a shear-capacity curve degrades with displacement ductility.

In this study the phenomenological model illustrated in Fig. 8 and 9 has been adopted for modelling the shear spring, accounting for both strength and deformation components due to shear action.

As interaction model the one contained in OpenSEES (named Limited State Material) was used [8, 22].

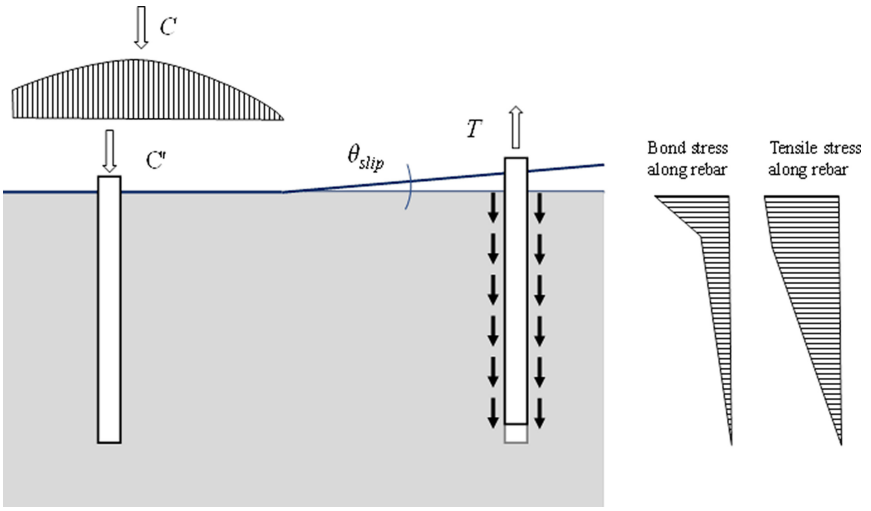



**Fig. 7.** Slippage model of the rebar. In figure C is the compression acting on the concrete and T is the tension on the steel bar.

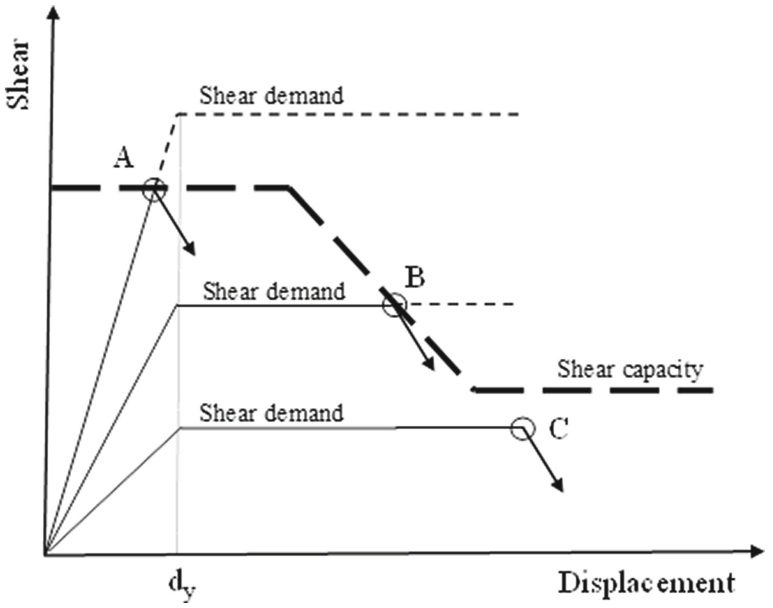



**Fig. 8.** Conceptual model for shear-strength degradation (dy: yielding displacement). Three cases are considered: (A) shear failure before flexural yielding (pure shear failure); (B) shear failure after flexural yielding (shear-flexural failure); (C) flexural failure.

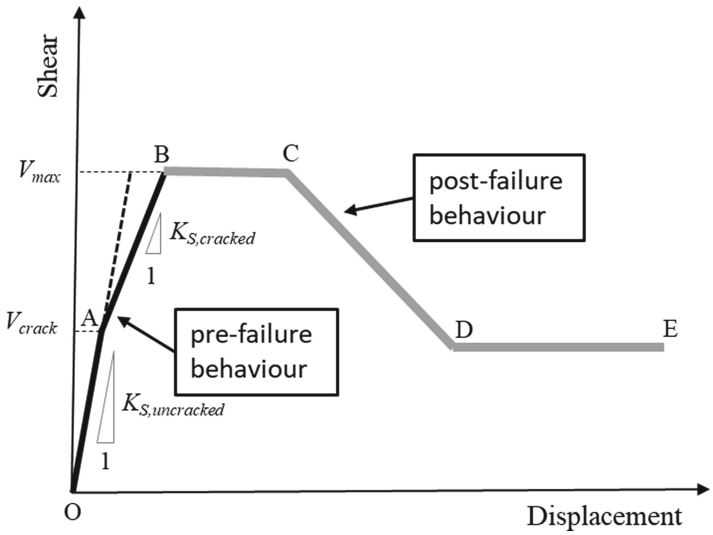



**Fig. 9.** Phenomenological model for the shear spring.

## 5 Numerical Validation

Using the model explained in previous chapter, the experimental response of a series of full-scale columns tested by Lynn 2001 [19] was simulated analytically by OpenSEES. The tested columns have a double cantilever configuration (they are fixed at top and bottom edges). The section as square shape of dimensions 457 × 457 mm$^2$, the longitudinal steel reinforcement was placed uniformly around the perimeter of the columns and they were #10 as nominal diameter while the transversal reinforcements were hoop with #3 and 457 mm respectively as nominal diameter and spacing (ρ" = 0.001), axial force was equal to P = 503 kN and concrete compressive strength was equal to fc = 26 MPa.

In Fig. 10 the experimental behaviour of the specimen marked as 3CLH18 is shown. The experimental response demonstrates a clear shear strength degradation after the formation of the flexural plastic hinge. The shear failure occurs immediately after flexural yielding. The numerical monotonic (black line) and cyclic (red line) response are investigated and the numerical curves approach well the experimental one, above all the monotonic response.

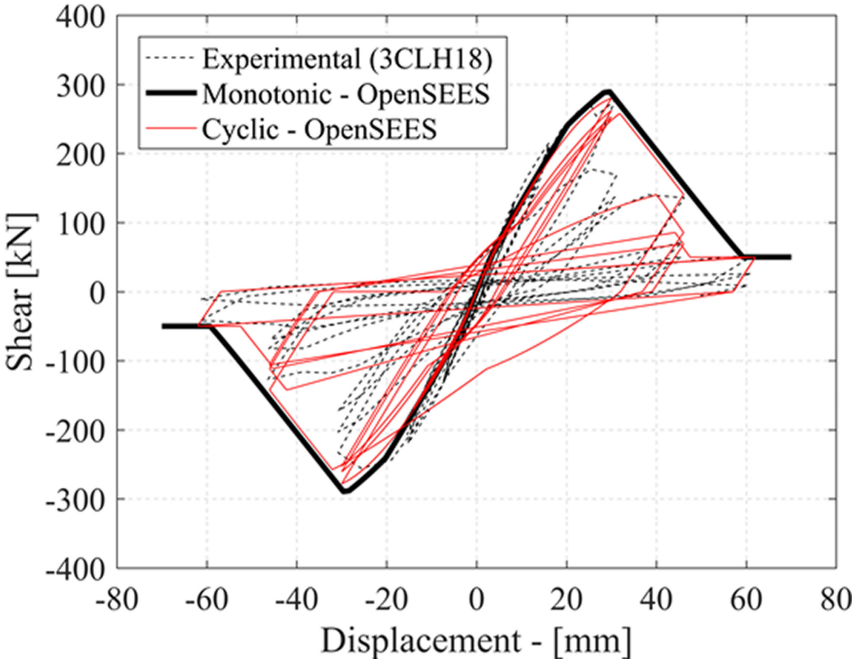



**Fig. 10.** Comparison of numerical cyclic response with experimental results after the calibration of some parameters.

## 6 Conclusions

A finite element modelling technique has been presented for the assessment of the seismic performance of ageing reinforced concrete structures, considering the deterioration of longitudinal and shear reinforcement due to corrosion. The effect of corrosion has been taken into account by the using of both corroded diameter and corroded mechanical proprieties, while the shear-flexural interaction phenomena has been introduced, in the FE model, through the incorporation of a zero-length shear spring in series with a flexural column element and a rotational slip spring.

The accuracy of the modelling has been tested on different case studies. Strengths and drift capacities of some column tested cyclically have been compared with the numerical OpenSEES model and a good agreement between the numerical prediction and experimental data can be observed.

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