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EXPERIMENTAL ASSESSMENT OF SHEAR-CRITICAL EXISTING REINFORCED CONCRETE COLUMNS STRENGTHENED WITH PRE-STRESSED STEEL STRIPS

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Abstract

Existing Reinforced Concrete (RC) buildings generally are very vulnerable to seismic loading worldwide. Pre-code or low-code RC buildings, in particular, often exhibit shear failures in case of earthquake, especially in high-seismicity areas, like Italy. According to obsolete technical or seismic codes in force in Italy until '80s, beams and, above all, columns often have a minimum amount of transverse reinforcement, resulting very vulnerable to shear failures and, thus, requiring seismic improvements. Different strengthening techniques are nowadays spread in common practice to solve such critical failure typology, by using FRP wrapping, RC or steel jacketing.

This paper experimentally analyses the effectiveness of pre-stressed stainless steel strips externally applied on shear-critical RC columns to increase their shear strength without changing lateral stiffness.

Pseudo-static tests are performed under cyclically increasing top displacement on three nominally identical cantilever columns, comparing the as-built condition with two different retrofitting layout. Experimental results are described in terms of damage evolution and global response, to quantify the benefit of the investigated technique. Local strain measurements on stirrups and external strips are also investigated as a support to assess and improve code-based prescriptions about the design of this retrofitting technique.

Keywords: pre-code/low-code buildings, reinforced concrete, shear failures, experimental tests, pre-stressed steel strips.

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1 INTRODUCTION

Reinforced Concrete (RC) buildings worldwide often exhibit significant vulnerability to seismic forces. Structures designed with outdated or without seismic codes frequently experience shear failures in beams, columns, and beam-column joints during earthquakes. This issue is especially evident in moment-resisting frame RC buildings constructed in Italy before the 1970s [1, 2], as structural design at that time primarily accounted for gravity loads only (GLD buildings). Fig. 1a shows all the Italian municipalities classified as not seismic prone area before 1970: they are approximately 6700 municipalities spanning various seismic hazard zones (SZ), from the most dangerous SZ1 to SZ4. Over the past fifty years, the Italian government has incurred substantial annual costs due to direct and indirect earthquake-related damages [3], highlighting the urgent need for a large-scale seismic retrofitting strategy.

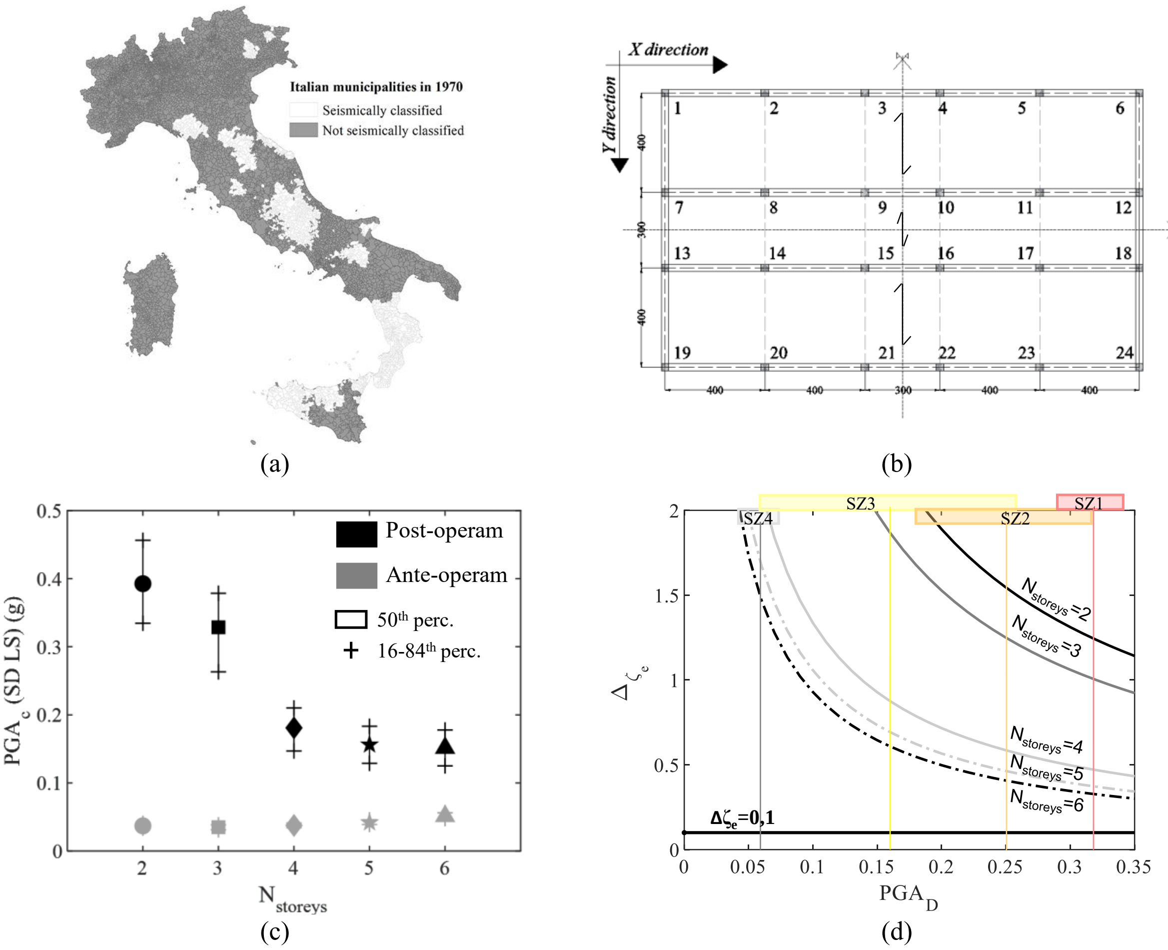


Figure 1. Seismic classification of Italian municipalities (a). Plan configuration of investigated buildings in [2] (b); PGA_C at SD LS, pre- and post-solving tensile joints and columns/beams shear failures as a function of number of stories ($N_{storeys}$) (c) and relevant ζ_e increment ($\Delta\zeta_e$) depending on PGA_D (d) - adapted by [2].

Previous research [2, 4] has demonstrated that addressing shear failures plays a crucial role in enhancing the seismic resilience of existing Italian RC structures. Specifically, De Risi et al. [2] applied a code-based approach to evaluate the benefits of mitigating tensile shear failures in beams, columns, and joints in pre-1970 GLD residential buildings, with a plan configuration as that shown in Fig. 1b. Their findings suggest that, even without altering lateral stiffness, the shortest buildings (with 2 or 3 stories) can achieve a capacity Peak Ground Acceleration capacity (PGA_C) at the Significant Damage (SD) Limit State (LS) equivalent to the PGA demand (PGA_D) — resulting in a safety index (ζ_e) of 1 — in approximately all the black-marked municipalities in Fig. 1a. Similarly, the tallest buildings (ranging from four to six stories) can also meet this condition, even if limiting to the areas where the PGA_D at SD LS does not exceed 0.15g. These findings underscore the necessity of implementing effective retrofitting techniques to address tensile shear failures in outdated RC buildings (see Fig. 1c-d).

Among the available strengthening methods, this study evaluates the efficiency of pre-stressed stainless steel strips (CAM® technology) applied to beams, columns, or joints. This approach enhances shear strength and displacement capacity while maintaining the existing lateral stiffness. This technology, originally developed for the seismic strengthening of unreinforced masonry buildings, is based on the use of externally-added prestressed stainless steel strips, and was successfully adopted also for existing RC buildings [5, 6].

In this work, experimental tests on shear-critical columns are presented, with and without the strengthening with pre-stressed steel strips. Specimens and experimental setup are described, first. Then main results are described in terms of global responses and observed damage, comparing the as-built and retrofitted outcomes to prove the effectiveness of the investigated technique.

2 TESTED SPECIMENS

Three identical shear-critical columns (Fig. 2) have been realized and tested: the first one (AB1) has been tested in its as-built condition; the other two specimens (R2 and R3) have been strengthened with a different number of pre-stressed steel strips, as described in section 3.

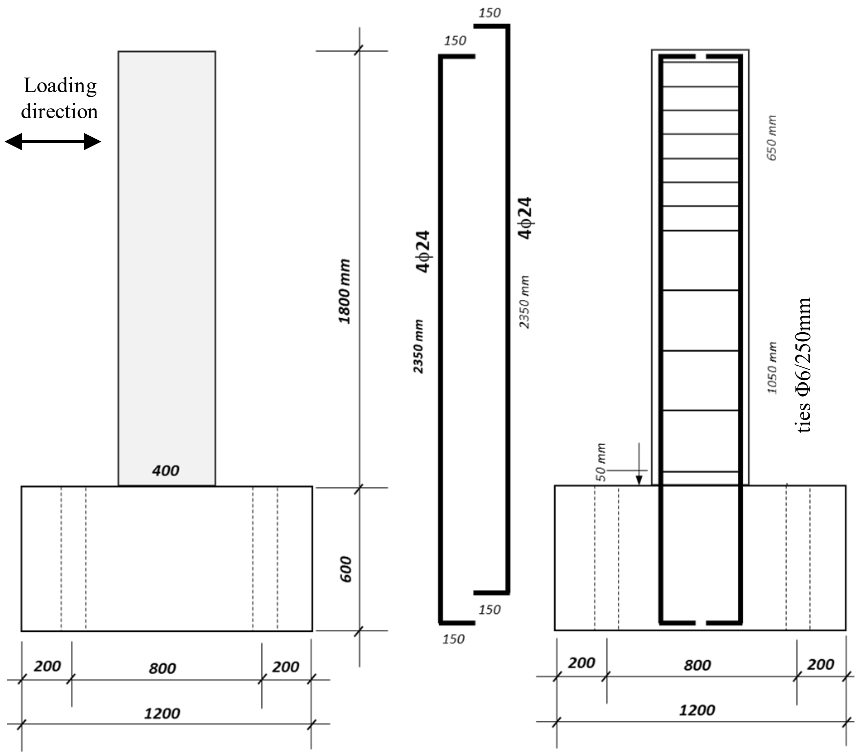


Figure 2. Geometric (dimensions in mm) and reinforcement details of column specimens.

Deformed bars with 24 mm and 6 mm of diameter have been used for longitudinal and transverse reinforcement respectively, to force the occurrence of shear failures, which are the main object of this study.

Mean yielding strength of 509 MPa and 604 MPa have been obtained from uniaxial tensile tests for longitudinal and transverse reinforcement, respectively. Mean concrete compressive strength (f_c) is equal to 11.5 MPa for columns and 21.5 MPa for the foundation block. Stirrups are 90°-hooked, as typical in obsolete RC buildings.

Based on mechanical properties of concrete and reinforcing steel, and due to the very low amount of transverse reinforcement with respect to the longitudinal reinforcement, a “pure shear” (S-) failure (without yielding) is expected to occur for specimen AB1: plastic shear load (about 230 kN) is significantly higher than the expected shear strength (112 kN, according to variable inclination (θ) truss model [7, 8].

3 STRENGTHENING DESIGN

Strengthened specimens R2 and R3 have been designed to observe a shear failure without or after column yielding, respectively. The retrofitting layout for both these specimens is shown in Fig. 3.

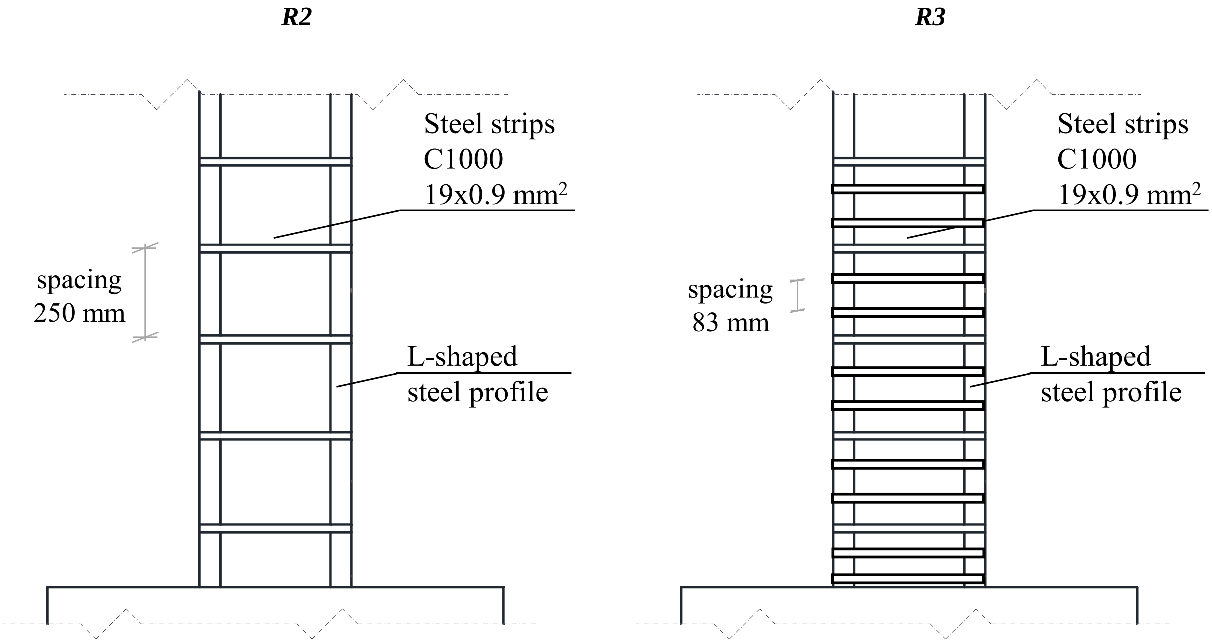
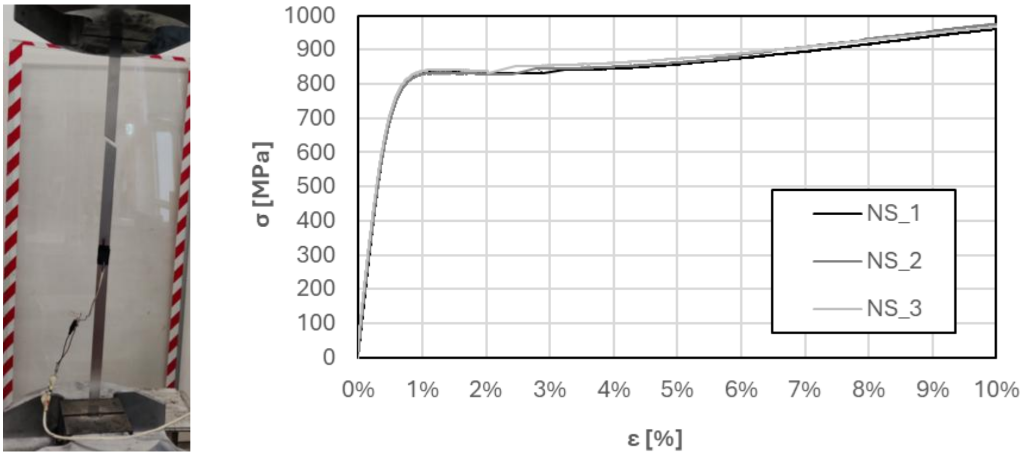


Figure 3. Retrofitting layout for specimen R2 and R3.

“C1000” strip typology is adopted. Mechanical properties of the adopted steel strips are obtained based on uniaxial tensile tests (see Fig. 4). It can be recognized that the constitutive law of this material does not exhibit a clear yielding branch. Therefore, as usual in these cases, a “conventional yielding stress” has been defined, as that producing 0.2% of residual strain after unloading ($f_{0.2}$). The mean value of $f_{0.2}$ in this case is equal to 773 MPa, evaluated among three tested specimens.



(a) (b)

Figure 4. Uniaxial tensile tests on steel strips adopted for strengthening (a); resulting constitutive law for three tested specimens (b).

The shear strength of retrofitted column has been calculated according to suggestions by Italian D.M. 2018 [7] for (more traditional) steel cages (Eq. 1), since specific prescriptions for pre-stressed steel strips are currently missing in codes. Nevertheless, the 50% reduction of the steel cage contribution ($V_{W,CAM}$) suggested by [7, 8] is here neglected.

$$V_R = \min\left(V_{R,s} + V_{W,CAM}; V_{R,C}\right)$$
$$V_{W,CAM} = 0.5\,\frac{2t_j}{s}\,b\,f_{yw}\,0.9d\,\cot\theta \qquad (1)$$

In Eq. (1), $V_{R,S}$ and $V_{R,C}$ are the shear strength contribution due to the transverse reinforcement, the prestressed steel strips (CAM) and the concrete strut. Additionally, t_j and s are strips thickness and spacing, respectively, b and d identify the width and effective depth of the column cross-section, while f_{yw} – yielding stress of the exterior steel cage - is herein assumed equal to $f_{0.2}$.

In summary, a single strip has been assumed for each layer. Based on the mean value of $f_{0.2}$ for pre-stressed steel strips used for this reinforcement, and on the Eq.s (1), designed strips spacing is 250 mm and 83 mm, for specimens R2 and R3, respectively.

4 EXPERIMENTAL SETUP AND INSTRUMENTATION LAYOUT

Each column has been tested by means of the setup scheme shown in Fig. 5. A cantilever scheme has been realized. The foundation block has been fixed at the laboratory strong floor by means of a system made up of steel plates/profiles and pre-stressed rods.

A pseudo-static displacement history has been applied 140 cm far away from the top of the column foundation (shear span, L_v=140 cm), thanks to a displacement control hydraulic actuator constrain at the top of the column.

Three push/pull cycles for each target displacement were performed. Target displacement were defined as 0.25, 0.50, 0.75, 1, 2, 3, 4, and 5 times the expected yielding displacement, starting from the negative (pull) side (see Fig. 6).

The applied displacement has been monitored by means of an external wire potentiometer, as in Fig. 5. Monitoring strain gauges have been also installed on the interior stirrups of each specimen and on the exterior steel strips to record their strain demand.

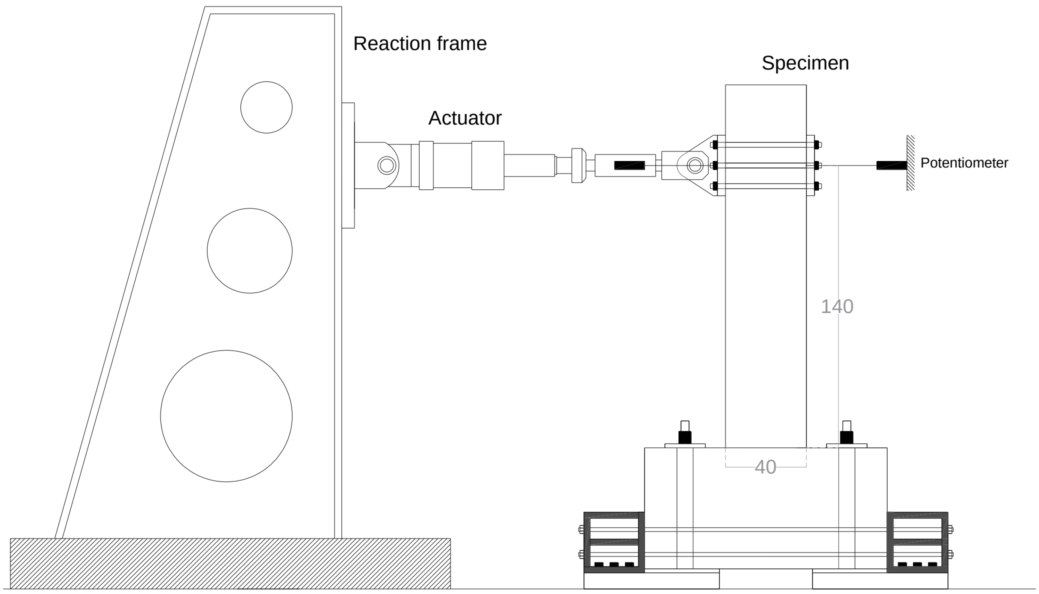


Figure 5. Experimental setup adopted for columns (dimensions in cm).

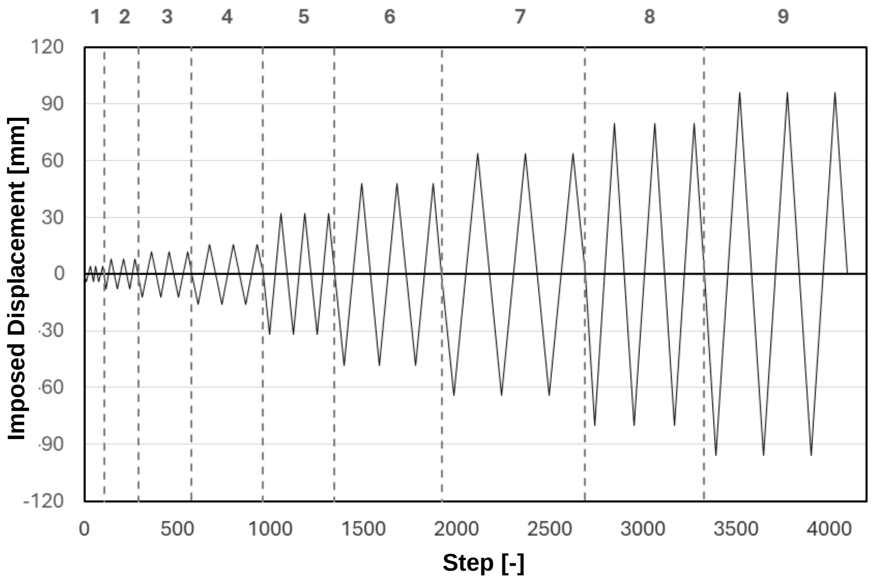


Figure 6. Cyclic loading displacement history expressed on imposed displacement. Indication of the evolution of cycles is also reported on the top of the plot.

5 MAIN EXPERIMENTAL RESULTS

The main experimental response is shown in this section in terms of column shear load (Force) versus top displacement envelopes (Fig. 7), comparing as-built and retrofitted specimens.

Final damage state for all the tests is also reported in Fig. 8.

First of all, a very important outcome is that related to the lateral stiffness of the tested specimens. Indeed, considering the first loading direction (negative, i.e., pull), it can be noted that the initial stiffness is not significantly influenced by presence of strengthening, thus allowing defining this strategy as a "local" strengthening technique.

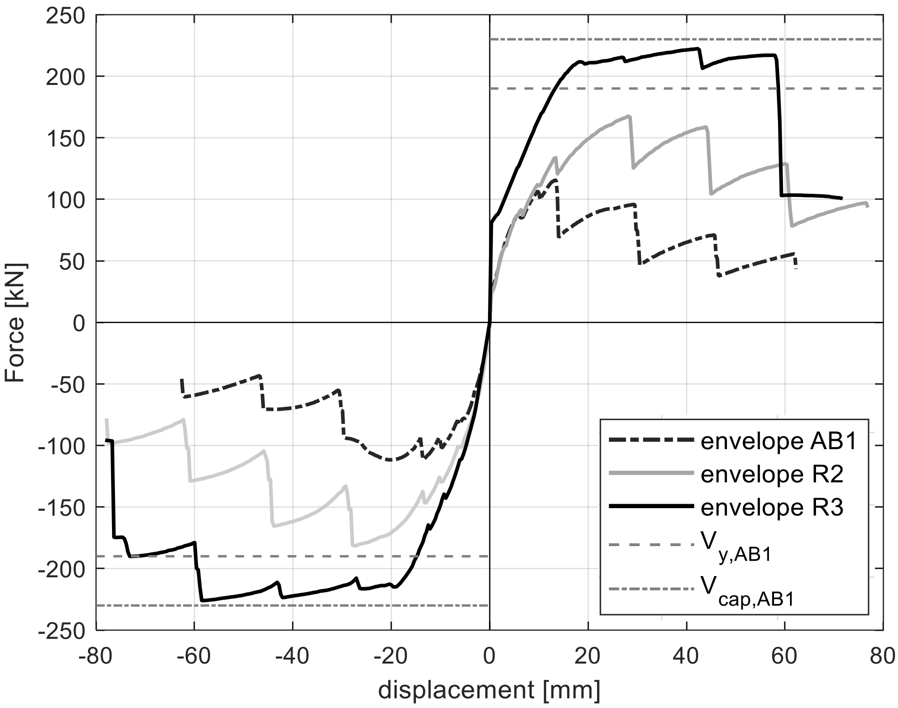


Figure 7. Comparison among the envelopes of global responses.

Figure 8. Damage state at the end of tests.

The specimen AB1, as expected, exhibited a pure shear failure, with a maximum force of 115 kN, namely significantly lower than the expected yielding load ($V_{y,AB1}$, equal to about 190 kN). Softening phase appears quite stiff due to the progressive evolution and widening of diagonal shear cracks throughout the column. At the end of the test, the column cover appeared totally ejected and some stirrups completely opened (Fig. 8).

Retrofitted specimens reached a higher lateral load, thanks to the presence of the exterior strips: maximum achieved loads was 181 kN and 226 kN for specimens R2 and R3, respectively.

R2 had a S-failure typology, as the AB1, whereas R3 overcame yielding before softening phase began (column capping shear load, V_{cap}, is very slightly higher than the maximum experimental force). For specimen R2, at the peak load, one of the prestressed steel strips opened due to the failure of a seal (where the effective transverse area of the strip is reduced). A similar failure occurred for specimen R3, involving the first five strips nearest to the foundation, but only in the softening phase.

In summary, a light shear strengthening (R2) lead to +57% shear strength increment; viceversa, for higher strengthening percentage (R3) the maximum achieved load was higher (+97% with respect to the as-built condition), along with the ductility capacity (about 3).

Lastly, the local analysis of recorded strains on interior stirrups and on external strips revealed very interesting information, as shown in Fig. 9 for the two retrofitted specimens. This figure looks at the peak load condition, and it shows the ratio between the actual stress (σ) at the peak load (calculated from the recorded strain and the constitutive law) and the yielding stress, for all the monitored stirrups (f_y) and strips ($f_{0.2}$). It can be observed that the maximum stress levels are achieved where there is a significant diagonal crack (generally close to the bottom portion of the column); in these zones, both the stirrups and strips are very close or have overcome the yielding condition. This also means that, at the peak load, interior stirrups and external strips contribute to the total shear strength with their yielding stress, and, thus, shear strength of strengthened columns should be calculated by neglecting the 50% of reduction suggested by Italian and European code (see Eq.1).

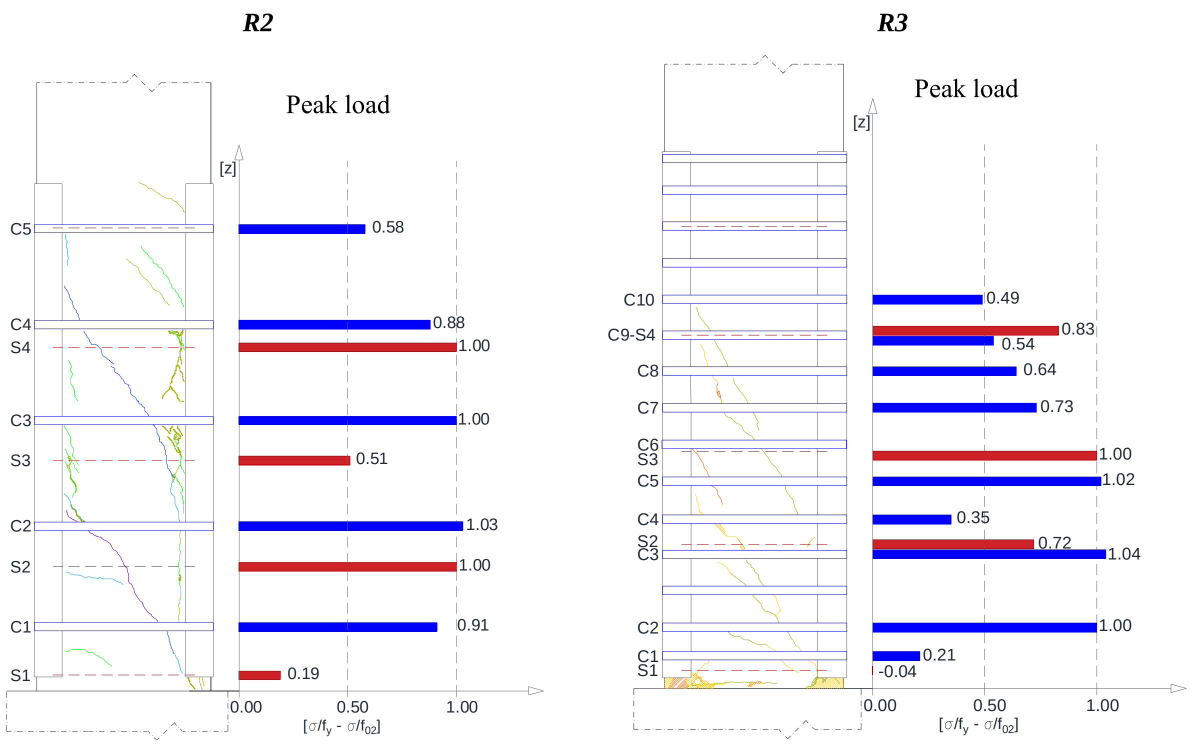


Figure 9. σ/ f_y for strips (in blue) and σ/ $f_{0.2}$ for stirrups (in red) for specimens R2 and R3 at the peak load achievement.

6 CONCLUSIONS

Solving shear failures is very important to prevent severe damage and collapse in obsolete RC buildings, especially when design to sustain gravity loads only. One of the existing shear strengthening techniques relies on the use of externa pre-stressed stainless steel strips (CAM ® technology). This technique has been investigated in this work by means of pseudo-static experimental tests under cyclic loading to prove and quantify its effectiveness.

Three shear critical columns have been tested, without and with pre-stressed stainless steel strips applied as an exterior reinforcement.

This technique has experimentally proved that shear strength of columns can be significantly increased, also favoring the reaching of their flexural capacity avoiding premature brittle failure. Furthermore, the technique is also able to increase the displacement capacity of strengthened columns and promote a softer degrading with respect to as-built condition.

The use of pre-stressed steel strips can, thus, be considered as a proper option for large retrofitting plans to increase resilience of existing RC building stock against earthquake actions.

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