FLEXURAL STRENGTHENING OF RC COLUMNS WITH NEAR SURFACE MOUNTED FRP OR STAINLESS STEEL REINFORCEMENT: EXPERIMENTAL INVESTIGATION

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ABSTRACT:

The paper presents the results of a comprehensive experimental program aiming to provide a fundamental understanding of the behavior of reinforced concrete (RC) columns under simulated seismic loading, strengthened in flexure (of crucial importance in capacity design) with different types and configurations of near-surface mounted (NSM) reinforcing materials. The role of various parameters, such as carbon or glass fiber-reinforced polymers (FRP) versus stainless steel, configuration and amount of NSM reinforcement, confinement via local jacketing and type of bonding agent, is examined, by comparison of the lateral load versus displacement response characteristics. The results demonstrate that NSM FRP and stainless steel reinforcement is a viable solution towards enhancing the flexural resistance of reinforced concrete columns subjected to seismic loads. This is especially the case when the retrofitting scheme combines epoxy-bonded NSM bars with local confining jackets, provided in this study with textile-reinforced mortars (TRM).

KEYWORDS: columns; flexure; near surface mounted reinforcement; seismic retrofitting; strengthening; textile-reinforced mortar (TRM).

1. INTRODUCTION

Earthquakes worldwide have proven the vulnerability of existing reinforced concrete (RC) columns to seismic loading. Poorly detailed columns are the most critical structural elements, which may fail due to shear, compressive crushing of concrete, rebar buckling, bond at lap-splices and flexure. Seismic retrofitting of RC columns is a challenging task that may be addressed successfully today using externally bonded fiber reinforced polymers (FRP) for all the aforementioned failure mechanisms but the last one, that is flexure. FRP, in the form of jackets with the fibers typically in the columns' circumferential direction, are quite effective in carrying shear and in providing confinement, thus increasing the shear resistance and the deformation capacity of existing RC columns. However, effective strengthening of columns in flexure, often needed for instance to satisfy capacity design requirements (that is the elimination of weakness in strong beam – weak column situations) or when existing rebars have been affected by corrosion, calls for the continuation of longitudinal reinforcement beyond the end cross sections, where moments are typically maximum; hence, placement of externally bonded FRP reinforcement is not applicable. As a result, flexural strengthening of RC columns is typically achieved today by using RC jackets or some forms of steel jackets, namely steel "cages", also followed by shotcreting. RC jackets or steel cages covered by shotcrete require intensive labor and artful detailing, they increase the dimensions and weight of columns and result in substantial obstruction of occupancy. Therefore, the implementation of a low labor and minimal obstruction flexural strengthening technique for RC columns still remains a challenging task, which is addressed in this study through the use of near-surface mounted (NSM) reinforcement.

NSM reinforcement involves cutting grooves into the concrete cover and bonding rebars inside the grooves through the use of an appropriate filler (typically epoxy resin or cement-based mortar). The idea of NSM reinforcement was born in Europe for steel rebars in the late 1940s (Asplund 1949), but it was only recently when more hi-tech materials, such as FRPs and high quality epoxies, become available, that the technique was given substantial attention by the research community and practicioners. Research so far on NSM reinforcement for RC structures has focused on flexural strengthening of beams or slabs with an emphasis on bond aspects, on shear strengthening of RC beams and on flexural strengthening with prestressed NSM FRP bars; some of the most recent research results in these areas are reported in Triantafillou (2007).

The only study reported in the international literature on flexural strengthening of columns with NSM reinforcement is that of Barros et al. (2006), who tested medium scale RC columns (1 m long cantilever-type specimens) under cyclic flexure combined with axial load. In this study the authors reported a substantial increase in the strength of columns with NSM Carbon FRP (CFRP) strips compared to control (unstrengthened specimens), but no clear conclusions about the specimens' behaviour under cyclic loading were made, as the tests were terminated before failure was reached, at a tip displacement equal to 20 mm, corresponding to a drift ratio equal to 2%. This paper presents the first systematic study on NSM-based flexural strengthening of RC columns under simulated seismic loading. The investigation addresses column strengthening with NSM carbon or glass fibers, as well as stainless steel rebars. Another innovative aspect in this study is the combination of NSM reinforcement with local jacketing, which comprised the recently developed textile-reinforced mortar (TRM) confining system, described by Triantafillou et al. (2006) and Bournas et al. (2007). Details are provided below.

2. EXPERIMENTAL INVESTIGATION

2.1. Test specimens and experimental parameters

The experimental program aimed to study the flexural strengthening of old-type non-seismically detailed RC columns with NSM reinforcement and to compare the effectiveness of different flexural strengthening schemes. A total of eleven full-scale RC column specimens with the same geometry were constructed and tested under cyclic uniaxial flexure with constant axial load (Fig. 1a). The specimens were flexure-dominated cantilevers with a height to the point of application of the load (shear span) of 1.6 m (half a typical story height) and a cross section of 250x250 mm. To represent old-type columns, specimens were reinforced longitudinally with four 14 mm-diameter smooth bars (except for one specimen which had 12 mm bars) and 8 mm diameter smooth stirrups, closed with 90-degree hooks at both ends, at a spacing of 200 mm. The geometry of a typical cross section is shown in Fig. 1b.

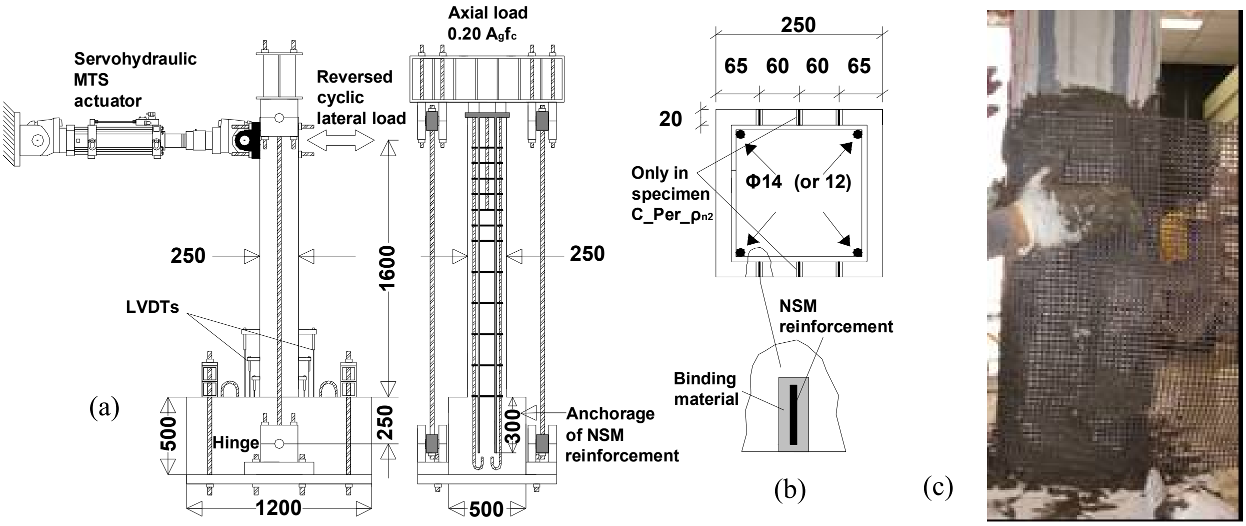


Figure 1 (a) Schematic of test setup. (b) Cross section of columns. (c) NSM reinforcement and application of TRM jacket.

The specimens were designed such that the effect of a series of parameters on the flexural capacity of RC columns could be investigated. These parameters comprised: type of NSM reinforcement (CFRP strips, GFRP bars, stainless steel rebars); configuration of NSM reinforcement (CFRP strips placed with their large cross section side perpendicular or parallel to the column sides, depending on whether a proper concrete cover is available or not); amount - that is geometrical reinforcing ratio - of NSM or internal reinforcement; type of bonding agent for the NSM reinforcement (epoxy resin versus cement-based mortar); and NSM reinforcement with or without local jacketing at the member ends. A description of the specimens follows next. One specimen was tested without retrofitting, as *Control*. *C_Per* was strengthened with two CFRP strips symmetrically placed on each of two opposite sides of the column (those with highest tension/compression). The strips had a cross section of 16x2 mm and were placed inside 10x20 mm orthogonal grooves with the large cross section side *perpendicular* to the column side. This scheme is feasible only if the concrete cover is at least equal to 20 mm. $C\_Per\_\rho_{n2}$ was strengthened as C_Per, but with a higher geometrical reinforcing ratio for the NSM reinforcement (ρ_n = 0.3%) provided by placing three strips (instead of two, corresponding to ρ_n = 0.2%) on each column side. $C\_Per\_\rho_{s2}$ was strengthened as C_Per, but it was initially designed with a lower reinforcing ratio for the internal steel reinforcement. This specimen was reinforced with 12 mm-diameter bars (ρ_s = 0.72%), whereas all others had 14 mm-diameter bars (ρ_s = 0.98%). *C_Par* was strengthened with two CFRP strips (with dimensions as above) symmetrically placed on each of two opposite sides of the column, but with their large cross section side *parallel* to the column side; the strips were placed inside 20x5 mm orthogonal grooves. This scheme is expected to have less favourable bond characteristics compared to C_Per, but it may be easily applied if the concrete cover is small. *C_Par_J* had the same NSM reinforcement as C_Par and an additional confining jacket, which extended from the column base to a height of 600 mm. The aim of this jacket was mainly to protect the NSM reinforcement against premature failure due to buckling and/or debonding. Specimen *G* was strengthened with two 8 mm-diameter deformed GFRP bars symmetrically placed on each of two opposite sides of the column. The bars were placed in 20x20 mm square grooves. *S_R* was strengthened with two 12 mm-diameter deformed stainless steel rebars symmetrically placed on each of two opposite sides of the column, in 20x20 mm square grooves. As in all specimens above with NSM reinforcement, the bonding agent inside the grooves was epoxy resin. *S_M* had the same NSM reinforcement as S_R, but the bonding agent inside the grooves was a cement-based mortar. *S_R_J* had the same NSM reinforcement as S_R and an additional confining jacket, as used in C_Par_J. *S_M_J* had the same NSM reinforcement as S_M and an additional confining jacket, as used in S_R_J.

Of crucial importance in the selection of NSM reinforcement was the requirement of equal tensile strength for each of the reinforcing elements (CFRP strips, GFRP bars, stainless steel bars). Given that all these elements are commercial products, this requirement was satisfied by proper combinations of cross section geometries and material strength data. As a result of this choice, the axial stiffness (elastic modulus times cross section area) ratio of CFRP: GFRP: stainless steel was 1:0.7:4.9.

2.2. Strengthening procedures, test setup and materials

When their preparation was completed, grooves and holes were filled by injecting the bonding agent using a simple silicone gun, then the NSM reinforcement was placed into position and the bonding material in excess was removed. For the specimens receiving TRM jacketing a commercial textile with equal quantity of carbon rovings in two orthogonal directions was applied in four layers (Fig. 1c). Application of the mortar with this textile was made in approximately 2 mm thick layers. Of crucial importance in this method, as in the case of epoxy resins, was the application of each mortar layer while the previous one was still in a fresh state.

The columns were fixed into a heavily reinforced 0.5 m-deep base block, 1.2x0.5 m in plan, within which the longitudinal bars were anchored with 50 mm radius hooks at the bottom. The 14 mm-diameter longitudinal bars had a yield stress of 372 MPa , a tensile strength of 433 MPa and an ultimate strain equal to 17%; the respective values for the 12 mm-diameter bars were 330 MPa, 412 MPa and 23%. The corresponding values for the steel used for stirrups were 351 MPa, 444 MPa and 19.5%. In order to simulate field conditions the base blocks and the columns were cast with separate batches of ready-mix concrete. The mean compressive strength of concrete on the day of testing the columns, measured on 150x150 mm cubes, was equal to 26.9 MPa. CFRP strips had an elastic modulus equal to 145 GPa and a tensile strength equal to 2175 MPa, whereas the respective values for GFRP bars were 65 GPa and 1490 MPa. Stainless steel bars had a conventional yield strength equal to 670 MPa, a tensile strength of 760 MPa and an ultimate strain equal to 19%. The tensile force for each of the three NSM reinforcements (conventional yield force, in the case of stainless steel) was calculated as: 69.5 kN for the CFRP strips, 74.9 kN for the GFRP bars and 75.6 kN for the stainless steel bars.

For the specimens with resin adhesive for bonding of the NSM reinforcement, a commercial structural adhesive was used with a tensile strength of 30 MPa and an elastic modulus of 4.5 GPa. For the specimens with mortar as a binding material for bonding of the NSM reinforcement (stainless steel bars in specimens S_M and S_M_J), a commercial inorganic dry binder was used, consisting of cement and polymers at a ratio of about 8:1 by weight. The water:binder ratio in the mortar was 0.23:1 by weight, resulting in plastic consistency, good workability and high flowability. The average flexural and compressive strength values for this mortar were 6.31 MPa and 17.5 MPa, respectively.

The textile used in this study for the TRM jacketing was made with equal quantity of carbon fibers in two orthogonal directions. Each fiber roving was 3 mm wide and the clear spacing between rovings was 7 mm. The weight of carbon fibers in the textile was 348 g/m^2 and the nominal thickness of each layer (based on the equivalent smeared distribution of fibers in the circumferential direction) was 0.095 mm. The mean tensile strength and the elastic modulus of the carbon fibers (as well as of the textile, when the nominal thickness is used) were taken from data sheets equal to 3800 MPa and 225 GPa, respectively.

The columns were subjected to lateral cyclic loading through the use of a horizontally positioned MTS actuator, under a constant axial load corresponding to 20% of the members' compressive strength. Displacements and axial strains at the plastic hinge region were monitored using six rectilinear displacement transducers. The instrumentation also comprised a total of 12 strain gages for each column, which were mounted on one NSM reinforcing element per column side. Measurements from the strain gages on each NSM element were used to determine the local bond-slip relationship in the anchorage region as well as the NSM strain (equal to the fracture strain or the strain at debonding) in the section of maximum moment.

3. RESULTS AND DISCUSSION

3.1. Results

The response of all columns tested is given in Figure 2 in the form of load-drift ratio (obtained by dividing the tip deflection with the column's height) loops. The performance and failure mode of all tested specimens was controlled by flexure, as expected due to their design characteristics. This was an important requirement, as the main objective in this study was to evaluate the effectiveness of NSM reinforcement as a means of flexural strengthening of RC columns. The control specimen attained a peak load of about 33 kN and a drift ratio at failure of 6.25%. After yielding of the longitudinal reinforcement, the concrete cover and a part of the core over the lower 200 mm of the column disintegrated and bar buckling initiated after the concrete cover spalled off. With only one exception (column C_Par), all strengthened specimens displayed higher (up to about 100%) flexural resistance compared to the control specimen. Flexural cracking at the column base started at the early stages of loading and the number of cracks increased and propagated with increasing drift ratios, while inclined cracks propagated in the concrete surface at both sides of the grooves as a result of high pull out forces of the NSM reinforcement for most specimens. Contrary to the unstrengthened column, the failure of the strengthened specimens was never attributed to buckling of the internal steel, as a significant portion of the total force in the compression zone was carried by the NSM reinforcement. However, buckling of the longitudinal internal bars always occurred abruptly after failure of the NSM reinforcement. The behavior of each strengthened column is described in detail below.

The observed failure mode for specimens C_Per, C_Per_ρ_{n2} and C_Per_ρ_{s2} (with the strip large cross section side perpendicular to the strengthened column side) was due to tensile fracture of the CFRP strips at the cross section of maximum moment (column base) as shown in Fig. 3a. Compared with the control specimen, the peak force increased up to about 40% and the attained drift ratio (at peak force) was approximately the same, in the order of 3%. Fracture of the NSM strips resulted in a drop of the applied force, when the mean recorded strains of CFRP at the column base were equal to 0.95%, 0.93% and 0.85% for specimens C_Per, C_Per_ρ_{n2} and C_Per_ρ_{s2}, respectively. These values are nearly half the measured ultimate strain in uniaxial tests, indicating the detrimental effect of cycling on the tensile strength of CFRP strips. Partial debonding of the strips when subjected to high pull out forces in one direction of loading deprived their lateral restraint in the next loading cycle. As a consequence, the strips became vulnerable to high compressive stresses resulting in local buckling, which led to their tensile fracture at strains less than the ultimate uniaxial strain.

Specimen C_Par displayed rather poor flexural strengthening characteristics: It failed due to early debonding of the CFRP strips at a force marginally higher than the control specimen and a drift ratio of about 2%, with a mean recorded strain of the strips at peak force equal to 0.50%, that is well below their tensile capacity. Debonding of the NSM strips at such a low strain is attributed to their outward spalling due to buckling, as shown in Fig. 3b, rather than to their poor anchoring conditions and the strips' low resistance against pull out. This can be confirmed by examining the results in comparison with specimen C_Par_J, which was identical to C_Par but jacketed at the column end. In this specimen the TRM jacket provided lateral resistance to the strips against buckling, thus increasing the peak force substantially, by 45% and 25% in the push and pull direction, respectively, and the drift ratio at peak force to about 4% and 2.5% in the corresponding directions. The reduced activation of tensile strips in the pull direction as compared to the push is attributed to their debonding, a fact which is confirmed by the values of mean recorded strains at peak force equal to 1.6% and 0.85% in the push and pull direction, respectively. These values are in agreement with observations of strip tensile fracture in the push direction only.

Specimen G, strengthened with 8 mm-diameter GFRP bars, displayed some distinct behaviour characteristics: At a drift ratio a little higher than 2% some of the GFRP bar ribs experienced shear fracture (Fig. 3c), resulting in slippage between the bars and the epoxy adhesive inside the grooves. The mean recorded strain in the bars when this phenomenon initiated was 0.45%, well below the bars' ultimate strain. In this specimen, failure in both directions was due to buckling of the GFRP bars, at drift ratios in the order of 5%, with a mean recorded strain of GFRP equal to 1.1%; the attained degree of strengthening was about 1.20-1.25.

Specimen S_R failed when the bars buckled suddenly (Fig. 3d) at a degree of strengthening more than 1.6 and a drift ratio of about 5%, with a buckled length approximately equal to 0.5 m. Its jacketed counterpart, that is specimen S_R_J, displayed an improved behaviour, comprising stable hysteresis loops until large drift ratios, in the order of 8%. This specimen attained the maximum flexural resistance, which was nearly double that of the control specimen. The confinement exerted by the TRM jacket at the base of this specimen prevented buckling of the NSM bars, which fractured in the pull direction, as shown in Fig. 3e, when the tensile strain was approximately equal to 10.1 %. This value is nearly half the measured ultimate strain in the uniaxial tests, indicating again, as in the case of specimens C_Per, C_Per_ρ_{n2} and C_Per_ρ_{s2}, the detrimental effect of cycling on the ultimate uniform elongation of stainless steel bars.

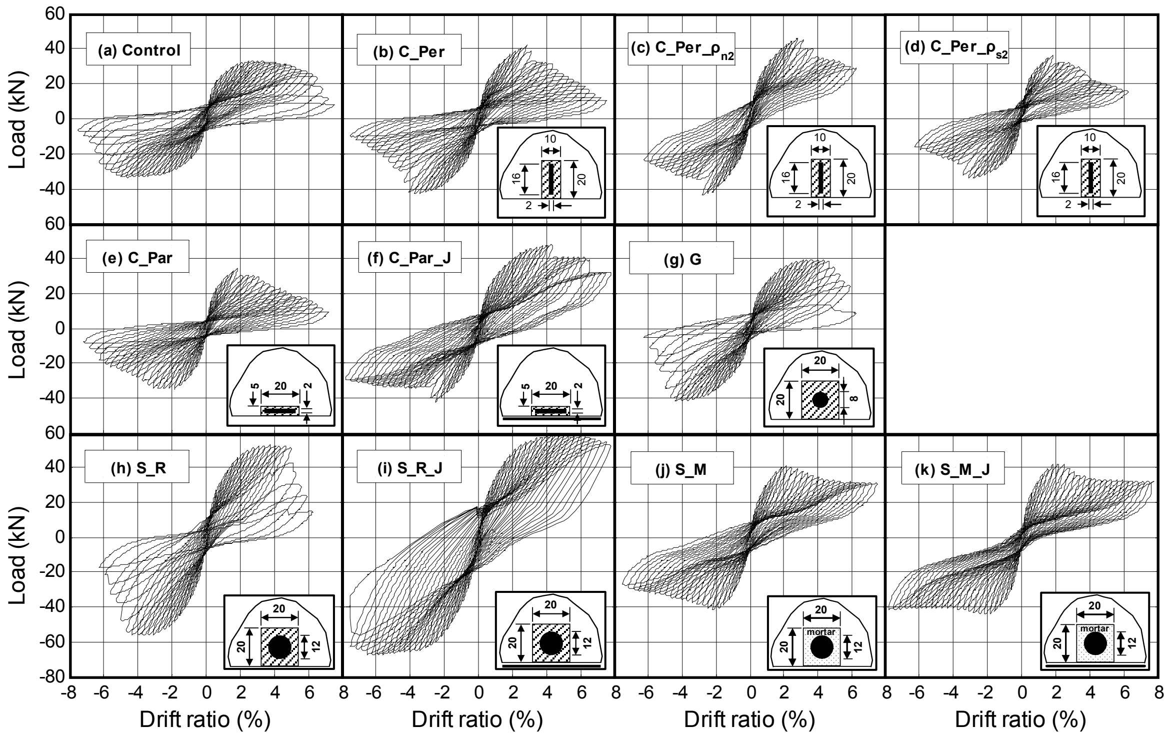
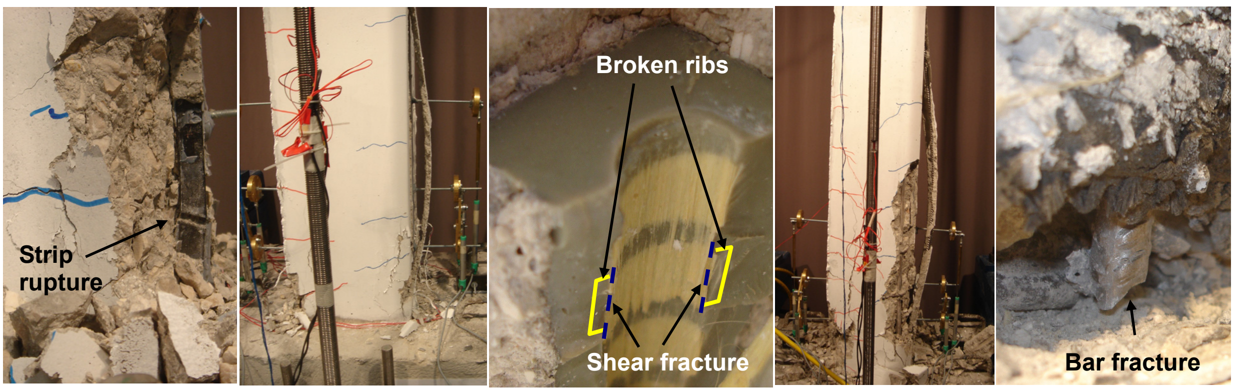


Figure 2 Load versus drift ratio curves for tested specimens (the inserts illustrate NSM reinforcement configurations).



(a) (b) (c) (d) (e)

Figure 3 Typical photographs of various failure modes: (a) Tensile fracture of NSM CFRP in columns C_Per, C_Per_ρ_{n2}, C_Per_ρ_{s2}; (b) Debonding and buckling of NSM CFRP in column C_Par; (c) Shear fracture of GFRP ribs in column G; (d) Buckling of NSM stainless steel bars in column S_R; and (e) tensile fracture of NSM stainless steel bars in column S_R_J.

For specimens S_M and S_M_J with mortar-filled grooves, the relatively low strength of the mortar in the base block resulted in gradual pull out of the bars and relative slip between bars and the surrounding mortar in the anchoring region, thus limiting the force transfer into the anchoring length and reducing the utilization of the NSM bars. The damage of the mortar inside the base block increased in a stable manner as the displacement increased up to the peak resistance of the specimens, which was marked at a drift ratio of about 2% for both directions of loading, corresponding to a strengthening degree in the order of 1.25-1.30. Apart from a slight reduction of the lateral load, the post peak response of both specimens was quite stable, displaying a marginal strength degradation to a load level defined by the residual friction between bar and mortar. This pull out resistance due to friction mechanisms resulted in a nearly rigid motion of the stainless steel bars into the anchoring region with practically the same slip along the bonded length, providing to columns S_M and S_M_J a pseudo-ductile behavior.

3.2. Discussion

All columns responded as designed and failed by flexural yielding of the internal steel, followed by failure of the NSM reinforcement. In terms of the various factors investigated in this experimental program, an examination of the results in terms of strength (average increase in the push and pull direction) but also in terms of overall response revealed the following information:

Type of NSM reinforcement (C_Per versus G versus S_R). Despite the roughly equal (monotonic) uniaxial strength of CFRP, GFRP and stainless steel bars, the latter were more effective, resulting in strength increase equal to 64%. The respective values for FRPs were lower (26% for CFRP and 22% for GFRP), due to failure of the FRP reinforcing elements at strains less than those corresponding to peak stress, as a result of cyclic loading. In terms of deformation capacity, quantified here by the drift ratio at conventional failure, stainless steel and GFRP bars outperformed CFRP strips by approximately 25%, due to the lower deformation capacity of carbon fibers in comparison with the other two materials.

Geometrical reinforcing ratio of NSM reinforcement (C_Per versus C_Per_ρ_{n2}). Increasing the NSM reinforcing ratio by 50% (three versus two strips in each side) resulted in a nearly proportional increase in strength, that is from 26% in specimen C_Per to 35% in specimen C_Per_ ρ_{n2}. Of course, this linearity may not apply in the case of large NSM reinforcing ratios.

Geometrical reinforcing ratio of internal steel reinforcement (C_Per versus C_Per_ρ_{s2}). Through the use of standard cross-section analysis based analytical modelling (Navier-Bernoulli hypothesis for plane cross sections) and the rectangular stress block approach for concrete in compression (without safety factors), a specimen similar to C_Per_ρ_{s2} but without NSM reinforcement has a predicted strength equal to 26.15 kN. Note that the same analysis predicted the experimentally obtained strength of the control column with an error of less than 5%, hence this model is considered reliable. By dividing the strength of specimen C_Per_ρ_{s2} (average value in the push and pull direction) with this value, the resulting degree of strengthening is approximately equal to 1.34. Therefore it is verified (and quantified) that the effectiveness of NSM reinforcement increases as the internal steel reinforcing ratio decreases: two NSM strips in each column side increased the strength by 34% for specimen with geometrical ratio of internal steel equal to ρ_s=0.724%, whereas the respective increase for the case of ρ_s=0.985% was only 26%.

Configuration of NSM strips (C_Per versus C_Par). In the absence of local jacketing, NSM strips placed with their large cross section side perpendicular to the column side were far more effective than those with their large cross section side parallel to the column side, due to the more favourable bond conditions. The strength increase in the former case was 26%, but only 4%, that is marginal, in the latter case.

NSM reinforcement with or without local jacketing (C_Par versus C_Par_J, S_R versus S_R_J, S_M versus S_M_J). Except for the case of mortar binder inside the grooves, which resulted in NSM debonding at the anchorage, local wrapping of the columns with TRM jackets resulted in dramatic improvements of the retrofitted specimens' response, by increasing both strength and deformation capacity. Jacketing with TRM improved the bond conditions and prevented buckling of the NSM reinforcement, thereby making the strength increase from 4% to 36% in the case of CFRP and from 64% to 90% in the case of stainless steel. In columns retrofitted with NSM bars placed inside mortar, jacketing offered a marginal increase in strength and a moderate increase in deformation capacity. Of all columns tested, the one retrofitted with the combination of stainless steel bars and TRM jacketing displayed the best response characteristics (Fig. 2i), with stable post peak behaviour and minimal strength degradation up to large drift ratios. On the basis of the results presented herein, it seems that the combination of NSM flexural strengthening and local jacketing is a viable means for increasing strength without compromising deformation capacity, which might be the case in unjacketed columns under low axial loads. In that respect it should be noted that higher axial loads would result in a lower drift ratio, as also confirmed here by a test of a column identical to the control specimen but with a normalized axial load equal to 0.3, in which case the drift ratio at failure was 3.75% (much lower than 6.25% recorded for the case of normalized axial load equal to 0.2). Hence, the improvements in deformation capacity are expected higher as axial loads increase.

Type of bonding agent (S_R versus S_M, S_R_J versus S_M_J). Epoxy resin was a much more effective bonding agent for NSM stainless steel. For the unjacketed specimens, when mortar was used (S_M) instead of resin (S_R), the increase in strength dropped from 64% to 24%; the corresponding values for jacketed specimens were 90% and 29%. Hence, the use of mortar instead of resin reduced the effectiveness of the strengthening scheme to about 1/3, due to pullout of the NSM bars.

4. CONCLUSIONS

A systematic study on NSM-based flexural strengthening of RC columns under simulated seismic loading was presented in this paper. The investigation addressed column strengthening with NSM CFRP or GFRP, as well as stainless steel. Another innovative aspect in this study was the combination of NSM reinforcement with local jacketing, which comprised the recently developed textile-reinforced mortar (TRM) confining system. The design of specimens allowed for an investigation of several variables, details of which are given above. This investigation proved that NSM FRP or stainless steel reinforcement is a viable solution towards enhancing the flexural resistance of reinforced concrete columns subjected to seismic loads. With proper design, which should combine NSM reinforcement with local jacketing at column ends, it seems that column strength enhancement does not develop at the expense of low deformation capacity.

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