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NUMERICAL INVESTIGATION OF R/C COLUMNS BEFORE AND AFTER FRP STRENGTHENING UNDER SEISMIC TYPE LOADINGS UTILIZING A NOVEL ANCHORING SYSTEM

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Abstract

The behavior of R/C columns before and after strengthening using FRP sheets is discussed here. The study focuses on the bending failure of columns. In particular, three numerical models of RC columns were developed representing a structural subassembly of a RC frame structure. One unstrengthened column and two strengthened with 1 FRP sheet and different bond strength. The numerical models employed all the mechanical properties of the materials explicitly, representing the concrete volume as solid finite elements, the steel reinforcement as wire elements and the FRP sheets as shell elements together with nonlinear interfaces capable to simulate the debonding mode of failure. The seismic type loads are applied as a monotonic horizontal load at the floor's slab level. Furthermore, as the structural upgrade is limited due to the delamination of the strengthening coating, a mechanical anchoring system is investigated and two more models are developed, with the FRP layers ideally anchored. The numerical predictions exhibited a significant increase of bearing capacity, over 50%, when FRP sheets are mechanically anchored. Finally, the novel anchoring system is fully simulated with all its details.

Keywords: RC columns; FRP strengthening; Seismic type loadings; Numerical simulation; Nonlinear interfaces; FRP anchorage

1. INTRODUCTION

The use of fibre reinforced polymer (FRP) sheets attached to R/C structural members has become a widely common retrofitting technique, offering high strength jacketing and ease at construction, while the weight and the stiffness of the strengthened members are not changed. Extensive research has been carried out for the use of FRP jacketing both experimentally and numerically focusing on either flexural or shear reinforcement of RC members. Manos et al. investigated the use of U-shaped FRP sheets wrapped and attached to rectangular or T-shaped RC beams as shear reinforcement [1,2,3]. Both experimental measurements and numerical predictions exhibited the effectiveness of this strengthening scheme provided that the delamination is prohibited by mechanical anchoring. Katakalos et al [4] investigated cantilever T-shaped RC beams strengthened with steel fibre reinforced polymer concluding that the anchoring system has a significantly more pronounced effect on the performance of the beams and the mode of failure than the type or spacing of the strips. The investigation of a 3D RC

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joint under seismic loads is investigated by Melidis et al [5, 6]. To ensure the effectiveness of the FRP flexural strengthened beams and columns an anchoring system using steel sections near the joint core region was employed. The same conclusion has arisen by other researchers investigating different RC structural sub-assemblies retrofitted with FRP jackets [7,8]. In order to prevent the debonding of FRP-concrete interface, which is the typical mode of failure of an FRP-strengthened concrete structure, a proper mechanical fixation should be used. Herein, the behaviour of RC columns before and after FRP strengthening are numerical investigated. The numerical models discussed here utilize solid finite elements to represent the concrete volume and 1D wire elements for the reinforcing rebars, longitudinal bars and stirrups, assigned with nonlinear constitutive laws derived by material testing. Shell elements were used to represent the FRP sheets bonded to the concrete surface using cohesive interfaces. The FRP mechanical properties were derived by material testing [9], while the interface behavior was calibrated according to available information over the literature [2,6]. The validation of the numerical methodology used here is verified by a number of other publications [2,5,6,10]. Thus, the numerical predictions discussed below can be considered realistic up to a point. The discussed here numerical methodology aims to evaluate FRP strengthening, with the proposed mechanical anchoring system, of RC structural members under seismic type loads.

2. METHODOLOGY

In what follows a brief description of the simulated RC columns is presented. The developed model refers to structural subassembly of a part of lower column and a part of upper column, which are hinged support at the location that the column's bending moment due to seismic loads equal to zero. The horizontal load is applied at the level of the floor slab, representing the inertia load induced by the shaking of the floor diaphragm as depicted in figure 1. The reinforcing details of the column's cross section with dimension 25x25cm2 is shown in figure 1. The longitudinal reinforcement of the column consists of 4 diameter 12mm steel rebars placed at each column's corner. The shear reinforcement of the model differ in the lower part and the upper part of the column. The first model, which is the before strengthening model, called here C0 has closed steel stirrups with diameter 8mm spaced at 75mm. The mechanical properties of the constitutive law for the steel reinforcing bars are given in table 1. The concrete volume is considered to be C16/20.

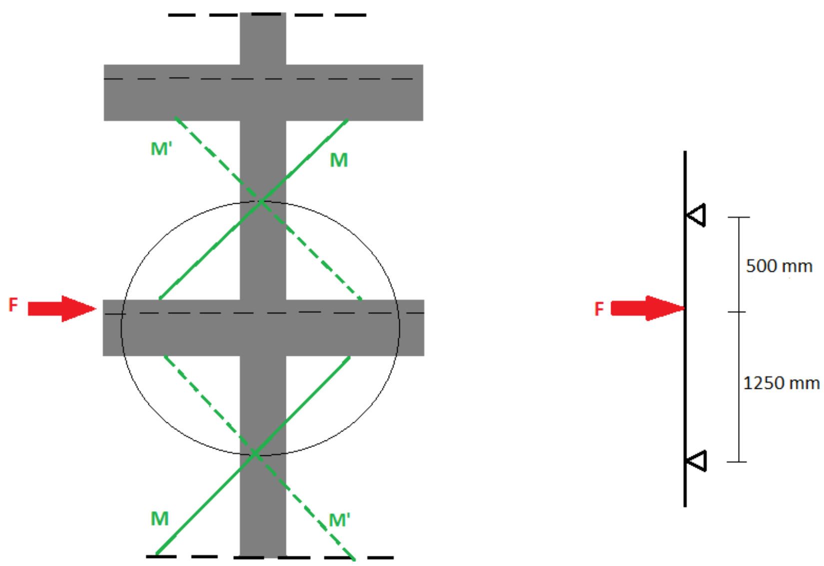

Figure 1 Continuous R/C column under seismic stress and moment diagram

Table 1: Mechanical properties of steel reinforcing bars

Steel reinforcing bars	Yield stress fy (MPa)	Yield strain εy	Ultimate stress fu (MPa)	Ultimate strain εu
Diameter Φ12mm (longitudinal)	549	0.0027	672	0.10
Diameter Φ8mm (stirrups)	549	0.0027	672	0.10

The presented modelling approach utilizes all the geometrical and mechanical properties described above. A detailed modeling is used. Concrete volume is represented with 3D finite elements assigned with a Concrete Damaged Plasticity constitutive law with compressive strength 20MPa. For this compressive strength, the detailed stress-strain relation was derived by the proposed methodology described in Eurocode 1992-1-1 (2004). For the plasticity parameters of the CDP law, the default values were chosen with a dilation angle equal to 40o. Longitudinal and transverse rebars are assigned with an elastic-plastic constitutive law according to table 1, as this simplification does not affect the overall response. Perfect bond is assumed between concrete and reinforcing rebars, as the bond-slippage of steel rebars is not the focal point of this investigation. The effect of inadequate reinforcement anchoring in the bending behavior of RC columns was investigated here [10]. The simulated columns are subjected to monotonic 3-point bending, as depicted in figure 2, in an effort to reproduce the envelope curve of horizontal load versus the horizontal deflection at the level of the slab and to determine the bending moment bearing capacity. To validate the numerical predicted maximum bending moments, these values are compared with the bending moments calculated by cross section analysis using an appropriate software (Anysection [11]).

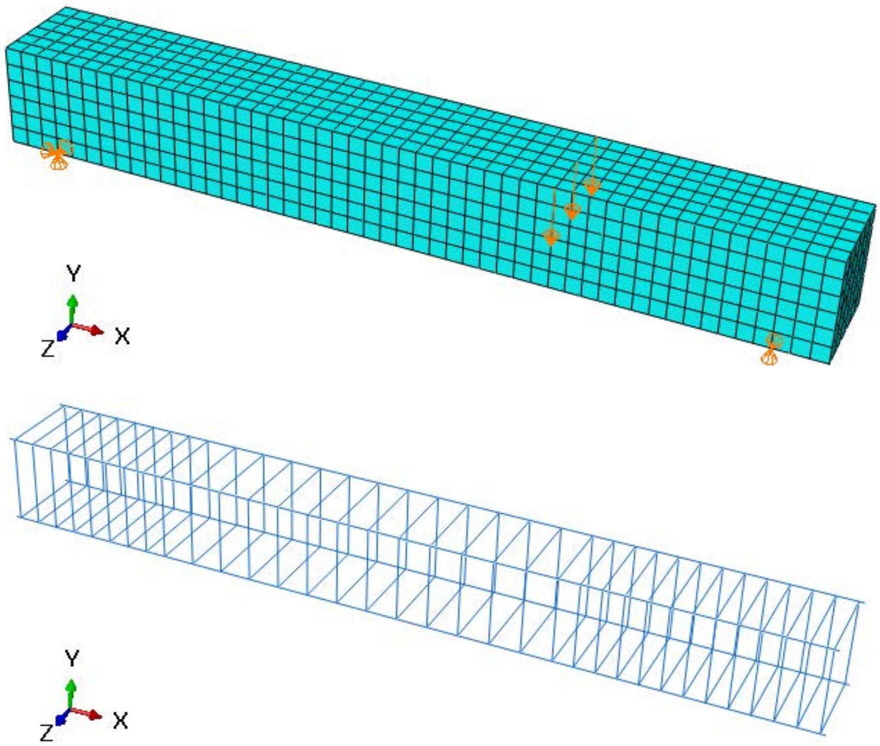

Figure 2: The concrete volume, the hinge supports and the applied load at the top and the steel reinforcing bar configuration at the bottom.

Following, 2 models are developed strengthened with FRP sheets only in the face under tension as shown in figure 3. The FRP sheets are bonded with a cohesive interface with bond strength 0.3MPa and 0.6MPa for the C-FRP-B03 and C-FRP-B06 models respectively. For the constitutive law, a brittle cracking model is utilized. This ideal elastic-brittle law is capable to replicate FRP's mechanical response with a rupture limit stress. The FRP sheets with thickness 0.13mm per sheet have a young's modulus 260GPa and a rupture stress equal to 2400MPa, according to experimental measurements available over the literature [9]. Again, the models are subjected to 3-point bending and the predicted behavior is compared with the virgin model (C0) in terms of load versus deflection curve, bending moment capacity and mode of failure.

Finally, two more models are examined. The first one, called here "C-FRP-A1" was developed with the same FRP jacketing, but instead of simply bonding FRP to the concrete surface, this time a steel U section tied at each end of FRP layers and embedded perfectly to the concrete is utilized to simulate a mechanical anchoring system at the end of the FRP sheets, as shown in figure 4. The later model was strengthened with the same methodology, but with 4 FRP sheets bonded and fixed. Again, the models are subjected to 3-point bending and the predicted behavior is compared with the previously mentioned models in terms of load versus deflection curve, bending moment capacity and mode of failure. The following table enlists all the examined models and their strengthening scheme.

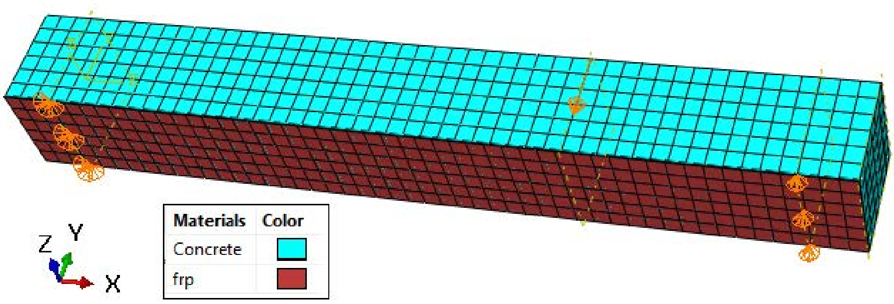

Figure 3 Model with FRP sheet placed at the face of column under tension

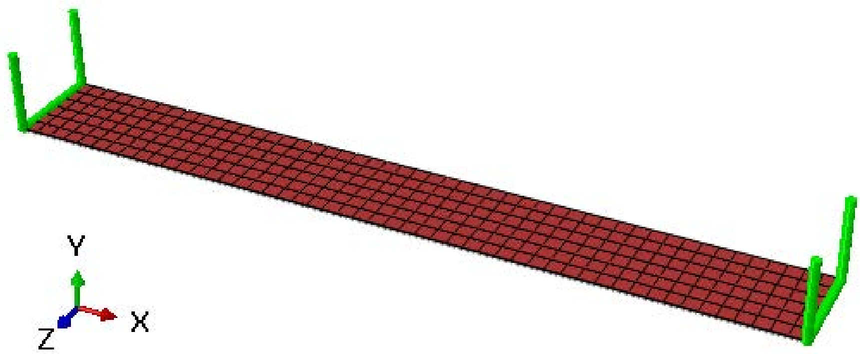

Figure 4: Model with mechanical anchored FRP sheets at each end of column with steel U-section (green lines)

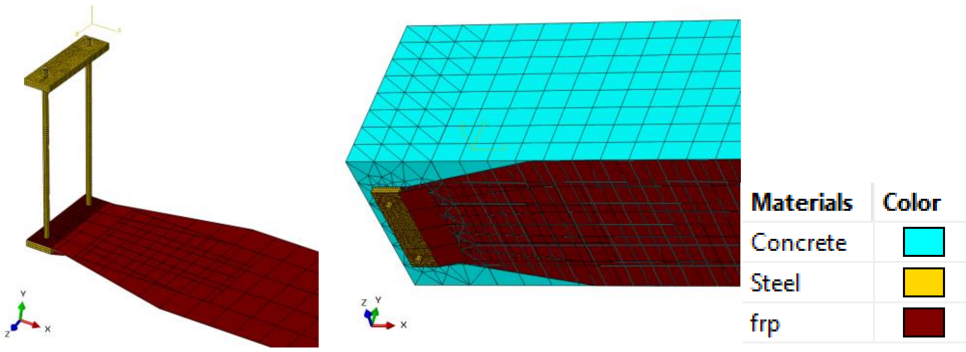

Figure 5: Model with detailed geometry of the anchoring system

Table 2: Description and naming of all the numerical models developed

Model name	Description of strengthening scheme
C0	Control model - Unstrengthened
C-FRP-B03	Strengthened with 1 FRP sheet bonded with bond strength 0.3MPa
C-FRP-B06	Strengthened with 1 FRP sheet bonded with bond strength 0.6MPa
C-FRP-A1	Strengthened with 1 FRP sheet bonded and anchored at each end
C-FRP-A4	Strengthened with 4 FRP sheet bonded and anchored at each end
C-FRP-AD	Strengthened with 1 FRP sheet bonded and anchored at each end with full detailing of the anchoring system

Results and discussion

Figure 6 depicts the response of the unstrengthened column in terms of horizontal load applied versus the corresponding deflection. The developed axial stresses of the reinforcing rebars are depicted at figure 5 as well. For an horizontal deflection almost 8mm, longitudinal rebars exceed the yield stress and the corresponding load equals to 74KN. For an horizontal deflection 40mm, the horizontal load equals 79KN and the longitudinal rebars. The developed bending moments equals to 26.60KNm and 28.50KNm respectively.

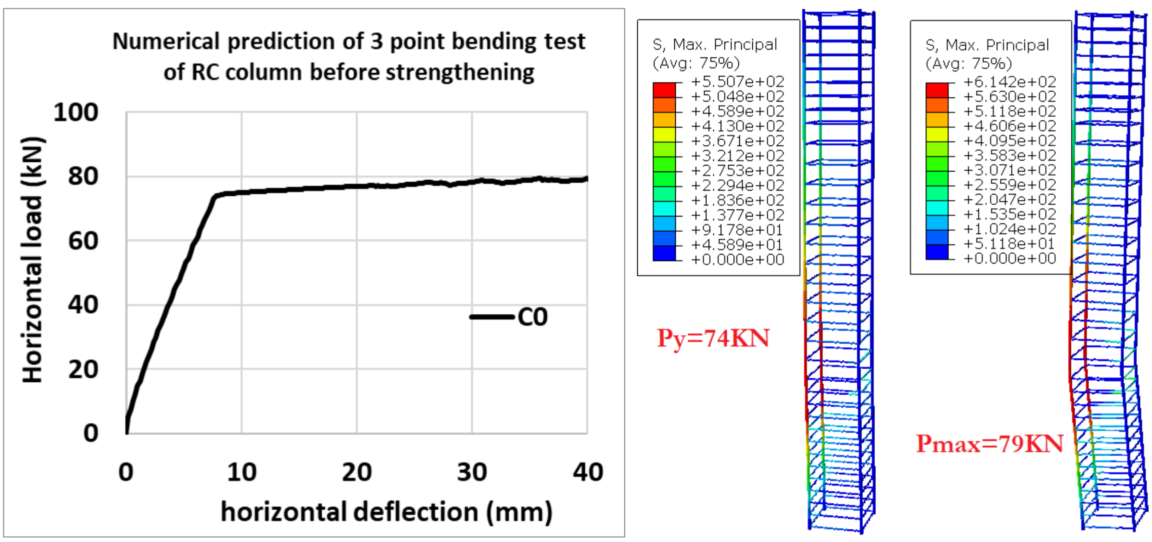

Figure 6 Response of unstrengthened model in terms of applied load versus deflection (left) and the developed axial stresses (MPa) of the reinforcing rebars at the yield load and the ultimate load (right).

The behavior of the retrofitted models with 1 FRP sheet bonded either with bond strength 0.3MPa or 0.6MPa are depicted in figure 7 in comparison with the control model C0. Additionally at the same figure the axial stresses developed at the reinforcing bars and the FRP sheet for the model with 0.6MPa bond strength are depicted at the maximum load and following the delamination of FRP that happens at the next step of the analysis that it is followed with a sudden loss of bearing capacity. For both models the longitudinal steel reinforcement yields for a deflection equal to 8.40mm, with a corresponding applied load 88.5KN and a developed bending moment 31.85KNm. The maximum load equals to 96.60KN and 110.40KN and the maximum bending moment to 34.75KNm and 39.75KNm respectively.

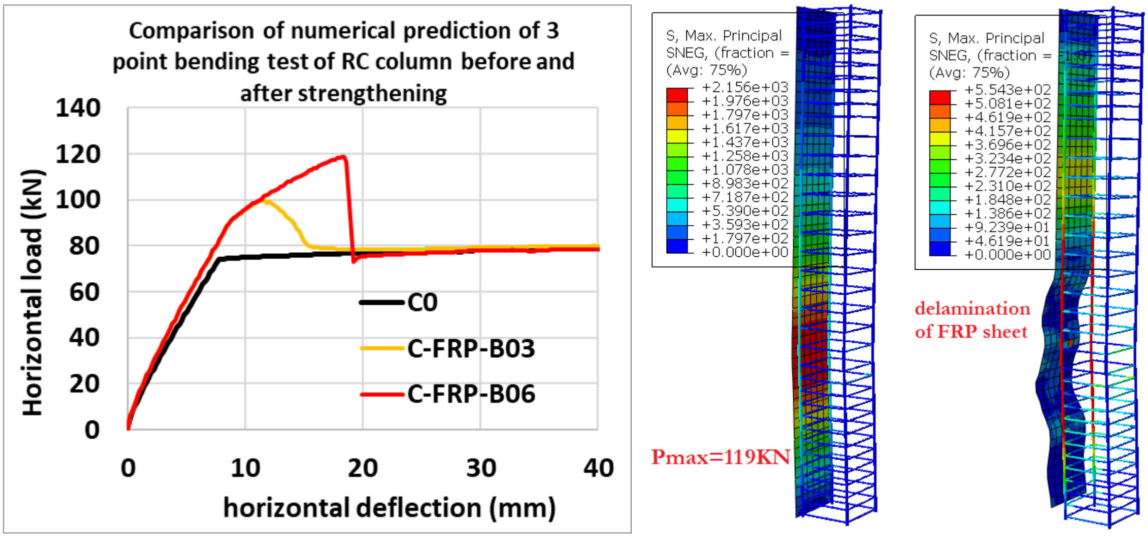

Figure 7: Response of strengthened models with 1 FRP sheet bonded in terms of applied load versus deflection and comparison with the unstrengthened model (left). The developed axial stresses (MPa) of the reinforcing rebars and FRP at the maximum load and the delamination of FRP sheet (right)for C-FRP-B06 model.

The next 2 models include the mechanical anchoring of the FRP's ends to the concrete member. In this case, the delamination is prohibited and the effectiveness of the retrofitting methodology is greater. The response of these two models is shown in figure 8. The model strengthened with 1 FRP sheet exhibits nonlinear behavior for an horizontal load equal to 88.50KN, where the longitudinal steel bars yield. The ultimate load is about 118.80KN and at this point the FRP jacket fails. The corresponding developed bending moments at the steel yield point and FRP rupture point equals to 31.85KNm and 42.75KNm. The last model with 4 FRP sheets exhibits yielding of the reinforcing steel bars for an horizontal load equal to 118.50KN and a corresponding bending moment equal to 42.65KNm. The ultimate load, that is followed by the FRP rupture, is 255.75KN and the corresponding bending moment 92.05KNm.

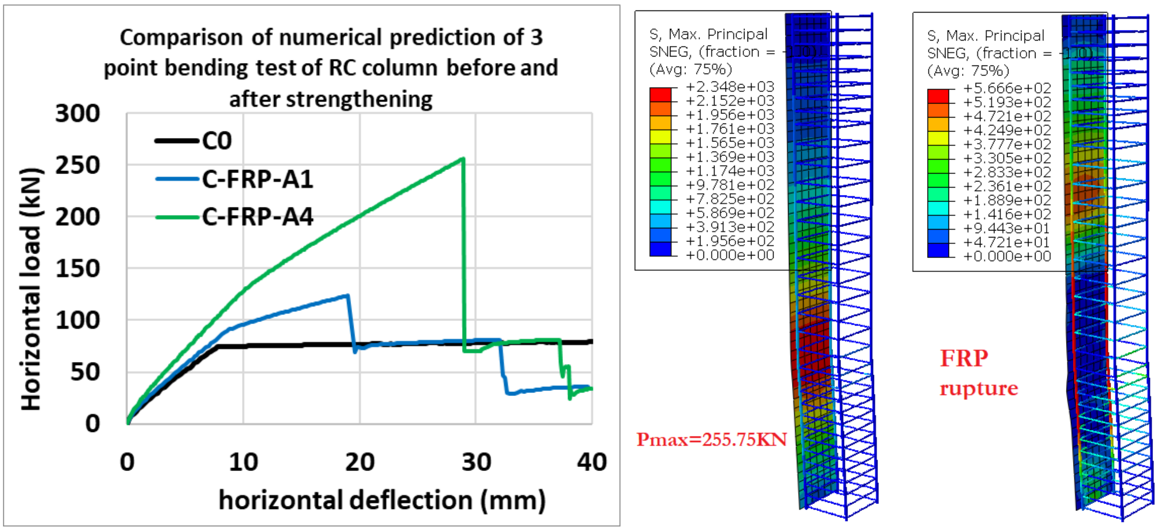

Figure 8 Response of strengthened models with 1 FRP sheet and 4 FRP sheets mechanical anchored in terms of applied load versus deflection and comparison with the unstrengthened model (left). The developed axial stresses (MPa) of the reinforcing rebars and FRP at the maximum load and the delamination of FRP sheet (right)for C-FRP-A4 model.

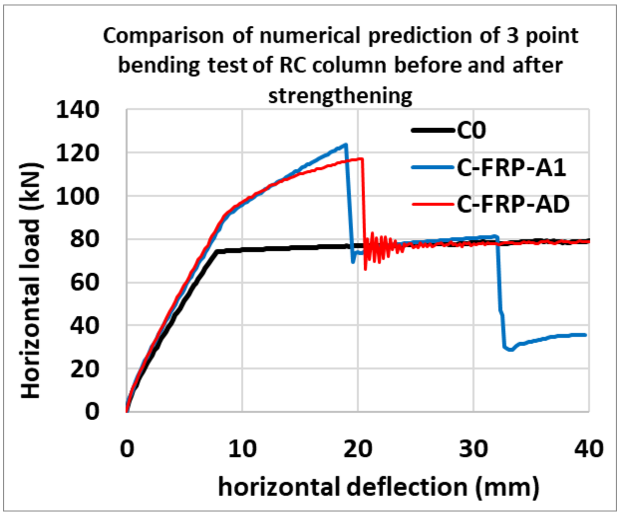

Figure 9 Comparison of numerical predicted response curves of unreinforced model and model with 1 FRP anchored ideally and with the detailed geometry in terms of horizontal applied load versus horizontal deflection

To validate the numerical predictions, a cross section analysis software was used to calculated the yield and ultimate bending moments for the control model and the models with mechanical supported FRP sheets. All the calculated moments, together with the numerical predictions for all models and the corresponding increase of moments in comparison with the control model are enlisted in table 3.

Table 3: Summary results of numerical predictions and cross section analysis for all models in terms of yield bending moment (reinforcing steel yields) and the maximum bending moment.

Model name	Numerical predictions		Cross section analysis [11]	
	My (KNm)	Mmax (KNm)	My (KNm)	Mmax (KNm)
C0	26.60	28.50	25.50	28.70
C-FRP-B03	31.85 (+20%)	34.75 (+21%)	-	-
C-FRP-B06	31.85 (+20%)	39.75 (+39%)	-	-
C-FRP-A1	31.85 (+20%)	42.75 (+50%)	30.50 (+20%)	43.80 (+53%)
C-FRP-A4	42.65 (+60%)	92.05 (+220%)	45.75 (+79%)	81.00 (+180%)
C-FRP-AD	31.85 (+20%)	42.50 (+49%)	30.50 (+20%)	43.80 (+53%)

The use of 1 FRP sheet bonded to the concrete surface as flexural reinforcement leads to slight increase of yield and ultimate moment, about 20-40%. The increase increment is highly affected by the bond strength between the FRP jacket and the concrete. A conservative value of 0.3 MPa leads to an increase of ultimate moment about 21%, while assuming the bond strength 0.6MPa, which is a realistic value for application without any surface treatment, exhibits a maximum bending moment increase equal to 39%. In both cases, the mode of failure is the delamination of FRP sheet, before exploiting the FRP jackets tensile strength, which here was assumed 2450MPa. Following the 1 FRP sheet was fixed at its both ends with a U-shaped steel section. This strengthening scheme did not exhibit much higher bearing capacity, as for a slightly great moment (+50) the FRP developed stresses equal to its maximum stress and the rupture of the jacket occurred. Lastly, instead of 1 FRP sheet, 4 FRP sheets were used with mechanical support. This time the increase of the bearing capacity was remarkable, as the maximum bending moment increase almost 220%. The model that includes the actual geometry of the investigated anchoring system has almost identical response with the one with the ideally mechanical anchoring. Thus, the effectiveness of the novel system is validated.

Conclusions

The behavior of R/C columns before and after strengthening using FRP sheets is discussed here. The study focuses on the bending failure of columns. For this purpose, 6 numerical models were developed simulating explicitly all the different materials used. The first model is the unstrengthened model. Its response is used as a comparison for the effectiveness of the FRP strengthening schemes. Together with Finite Element Analysis, Cross Section Analysis were conducted in order to validate the numerical predictions. The main conclusions of the present investigation can be summarized as follows:

- The numerical models compared with the results of the cross-section analysis in terms of yield bending moment (the developed moment that the longitudinal rebars exceed yield limit) and the ultimate bending moment, can capture satisfactorily the bearing capacity of the examined RC column.
- In all cases the application of the FRP jackets to the RC column improved its overall response, exhibiting an increased bearing capacity ranged from 21% to 220%.
- The attachment of FRP sheets simply bonded to the concrete surface, is highly affected by the bond strength between concrete and FRP sheet and the predominant failure of such members is the delamination of the strengthening jacket. Thus, the high tensile strength of the FRP is not fully exploited.

- The mechanical anchoring system examined here for the FRP sheets effectively prohibits the delamination of the FRP and leads to a further increase of bearing capacity. The mode of failure of these models is the rupture of FRP sheets.
- Comparing the two models with the FRP layer perfectly anchored to the edges of the R/C column and the one with the actual geometry of the investigated anchoring system, the effectiveness of the employed anchoring system can be concluded.

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