Numerical simulation of the shear behaviour of reinforced concrete rectangular beam specimens with or without FRP-strip shear reinforcement

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To the memory of Professor Sir Gordon Higginson, Founder of the Dept. of Engineering Science Durham University, Vice Chancellor of Southampton University, U.K.

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ABSTRACT

The successful validation of a numerical model is presented that can realistically approximate the shear behaviour of reinforced concrete (R/C) rectangular beams strengthened against shear with externally applied open hoop fibre reinforcing polymer (FRP) strips. For this purpose, the measured load-deformation response of ten (10) full-scale R/C beam specimens is utilised. These specimens were loaded monotonically in a four-point bending arrangement up to failure. Open hoop FRP strip shear reinforcement was applied externally to upgrade the shear capacity of eight (8) R/C beam specimens. Four of these specimens had these FRP strips without anchorage, whereas for the other four the FRP strips were attached together with novel anchoring devices. This successful numerical simulation predicts with a very good degree of approximation the observed load-deformation behaviour and the ultimate shear capacity of all these specimens as well as the observed modes of failure including diagonal concrete cracking, debonding of the FRP strips in the case of no anchoring, or the plastification of parts of the anchoring devices plus the adjacent crushing of the concrete.

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1. Introduction

Reinforced Concrete (R/C) structures represent a substantial portion of the current infrastructure and building stock in most countries worldwide. A large percentage of these were built several decades ago and in some cases require retrofitting or structural upgrading in order to conform to more stringent design regulations, to accommodate additional live loads due to possible changes in the building’s function or to make up for the loss of strength brought about by environmental parameters that speed up structural deterioration. Moreover, the rapid evolution of seismic codes over the past two decades, which utilises the knowledge gained following devastating earthquakes, necessitates the structural upgrading of most concrete structures that were built according to older design regulations, following a different design philosophy, in which the role of ductility was not central to the design process. On the contrary, all current seismic design codes pay particular attention to enforce ductile behaviour on the various R/C structural elements, which is accomplished by prohibiting shear types of failure and favouring instead flexural types of failure with sufficient ductility [1,14,32]. This ductile behaviour is also desirable when one is faced with the task of designing a retrofitting scheme that aims at upgrading the seismic performance of R/C structures designed by older design codes [30]. This paper deals with the objective of upgrading the shear capacity of R/C beams, that are poorly reinforced against shear, as part of such a retrofitting scheme [25,26,31].

Among several upgrading techniques available, the utilisation of FRP sheets or plates that can be attached externally to existing structural components with the aid of resin has attracted considerable attention from both academics and practising engineers [2,15,6,28]. Various researchers have derived models for the prediction of the bond-slip response and required anchor lengths of FRP sheets attached to concrete [36,37,35,43,22,42,41]. Depending on the application, fibre reinforcing polymer (FRP) sheets can be employed to increase the flexural strength, the shear strength [38], the ductility and the serviceability performance of R/C structures, utilising these FRP sheets as additional external reinforcement.

Both carbon-CFRP or steel-SFRP sheets possess very high strength in the orientation of their fibres; however only a small part of the available strength can be exploited in practice, due to the onset of debonding at relatively small strains, which can initiate either at the ends of the FRP sheets, due to insufficient anchoring or within the mid-parts of the FRP sheets, when anchoring is provided at the FRP-strip ends, following the formation of cracks
in the concrete [5,7,10,11,39]. Debonding occurs either within the resin layer itself that attaches the FRP sheet to the concrete surface or, more frequently, below that resin layer within the adjacent resin rich concrete substrate, the strength of which depends on the concrete tensile strength and surface preparation. Hence, central to the successful application of FRPs on existing structures is the preparation of the concrete substrate, which often includes the partial removal of damaged concrete cover and sandblasting so that such FRP debonding is postponed. Alternative techniques aiming at mechanically strengthening the substrate have also been proposed [20]. The quality of the concrete substrate linked with surface preparation strongly depends on the working conditions and craftsmanship and hence there is an inherent high uncertainty in each application employing FRPs as a means of structural upgrading, which often leads to overly conservative design. Therefore, the development of innovative means for effective anchorage of the FRP sheets/plates in the concrete volume, which ensures the satisfactory transfer of forces from the FRP sheet to the concrete part of the structural member and allows for a more efficient utilisation of the FRP strength, is warranted [24–28].

In the present paper, a novel anchorage device [24] for FRP sheets employed as external shear reinforcement, which was recently developed at the Laboratory of Strength of Materials and Structures of Aristotle University of Thessaloniki is studied numerically. Its effectiveness has been experimentally verified [21,25,26,31] The aim of the present study is to replicate numerically the experimentally obtained results and to identify the key parameters controlling the structural response of reinforced concrete beams strengthened against shear by external FRP reinforcement, that will allow the optimisation of the anchorage device and hence the more effective utilisation of the FRP sheets, thus leading to more efficient design.

2. Experimental Investigation

Several theoretical and experimental studies [9,44,4] have been carried out to analyze the phenomenon of the shear failure of reinforced concrete (RC) beams. Russo et al. [33] presents a comprehensive review of various proposed methods for predicting the shear behaviour of reinforced concrete beams without any transverse reinforcement. In the design of R/C beams, this is usually considered as an initial condition that is supplemented by the strength provided by the transverse reinforcement in order to predict the total shear capacity of such a structural element. Following this rationale a comprehensive experimental program has been designed and carried out at the Laboratory of Experimental Structural Mechanics at the Aristotle University of Thessaloniki aimed to assess the structural efficiency and effectiveness of shear strength provided to R/C beams by externally attached open hoop FRP transverse shear reinforcement. Initially, a number of R/C beam specimens with only longitudinal reinforcement were tested. Identical R/C beam specimens were then tested which were provided with open hoop FRP strips external shear reinforcement in order to study in this way the additional shear strength that can be attained by such a strengthening scheme. Many researchers examined the effectiveness of this type of shear reinforcement in the form of closed hoop FRP strips attached externally on rectangular R/C beam specimens [5,7,10,11]. However, although such closed hoop FRP strips are very effective they cannot be applied in practice as most R/C beams are accompanied by an R/C slab that makes such a closed hoop application almost impossible. Consequently, the full experimental program conducted at Aristotle University included either rectangular R/C beams or R/C T-beams being strengthened by open hoop FRP strips, as will be described in summary below. This open hoop FRP strip shear reinforcement was examined in two distinct forms. First, these open hoop FRP strips were simply attached on the external surface of the R/C specimens. This was done following the proper construction technique and using the proper resin recommended by current practice. In this case, as has been established by previous research [5,7,10,11], as well as by the current experimental sequence [25,26,28], the prevailing mode of failure is the debonding of the FRP strips which renders this type of shear reinforcement ineffective. This is because the FRP strips’ debonding occurs long before the tensile strength capacity of such FRP strips is reached. In order to prohibit such a premature debonding type of failure a novel anchorage device [24–27] is introduced together with the open hoop FRP strips that can be attached relatively easily at R/C T-beams, which is the primary practical application. In this Section, key experimental results dealing with the upgrading of the shear capacity of R/C rectangular beams are briefly presented. The observed performance from this experimental sequence will be utilised in the subsequent numerical simulation. More information on the experimental sequence and results is included in the work published so far [25–27,31].

2.1. Beam tests

A total of ten rectangular beam specimens were subjected to four point bending as part of the present study to assess the effectiveness of the proposed anchorage device. The mean dimensions and reinforcement details of these specimens are shown in Fig. 1, whilst the employed test arrangement and instrumentation is depicted in Fig. 2. Their cross-section was 120 mm wide and 360 mm deep (Fig. 1). Their overall span was 2700 mm whereas the shear span was equal to 900 mm (1/3 of the total length). These rectangular section beam specimens were simply supported and loaded up to failure.

Initially, two concrete beam specimens both without any FRPs, namely CRBs with 08/250 steel closed stirrups as shear reinforcement, and CRB without shear reinforcement, were tested to obtain the basic structural response. These two tests were utilised in the present numerical study to calibrate the material parameters of the concrete material model. Subsequently, four rectangular concrete beam specimens without any steel stirrups were reinforced against shear by externally attaching U-shape open-hoop FRP strips with the aid of resin (Fig. 3a); these specimens were tested to assess the effectiveness of such a conventional FRP strengthening technique without the application of additional anchorage. Two types of FRPs, namely CFRPs and SFRPs and two spacings (150 mm and 200 mm) were considered. Finally, another four specimens with the same open-hoop FRP strips arrangement as before but with additional anchorage were tested. This anchorage was provided by the novel anchorage device [24] depicted in some detail in Fig. 3a and b. In this anchoring device a steel rod is used to wrap around it the FRP strip; this steel rod is secured by a steel plate of equal length which is bolted by two anchor bolts as shown in Fig. 3b.

Table 1 lists all the particular aspects that characterise each one of these specimens. Instrumentation was provided to monitor the applied load as well as the deformation of these specimens. The cross-sections and conventional steel reinforcement details were identical for all specimens (Fig. 1).

2.2. Material tests

A series of standard tests have been carried out in order to establish the basic material properties for these beam specimens; e.g. basic material properties for the concrete, the reinforcing bars, the FRP strips as well as the steel plates and steel bolts employed in the anchoring device. These experimentally derived material properties were utilised in the subsequent numerical study. The measured mean concrete cylindrical strength for each specimen is
listed in column 3 of Table 1. The longitudinal and transverse reinforcement were grade S500, whilst the steel plates were grade S235. The bolts utilised in the anchoring device were manufactured by HILTI and are designated as HILTI HUS herein. The full stress–strain response of all steel parts was experimentally determined and utilised in the numerical studies. The experimentally determined Young’s modulus $E$, yield strength $f_y$ and ultimate strength $f_u$ are reported in Table 2.

Finally, unidirectional tension tests were conducted on resin rich CFRP and SFRP sheets up to failure to obtain the Young’s modulus ($E_1$) and strength ($f_{u1}$) in their fibre orientation and the Poisson’s ratio ($v_{12} = (E_1/E_2)v_{21}$). The remaining two material properties pertaining to the orthotropic nature of the material under the plane stress assumption, namely the transverse Young’s modulus $E_2$ and the shear modulus $G_{12}$, were derived by applying the conventional rule of mixture for composites [18], taking into account the manufacturer’s specifications for the resin (citation SIKA) and the FRP (citation SIKA) sheets and the experimentally determined volume fraction of resin and FRP sheets.

Both the experimentally and the analytically determined material properties are reported in Table 3. It should be noted that the analytical values obtained by applying the rule of mixture for $E_1$ and $v_{12}$ compare very well (discrepancy within 5%) with the experimental ones.

2.3. Summary of observed behaviour – modes of failure

The key findings of all beam tests are summarised in Table 1, where the ultimate shear force ($\text{exp} V_{\text{max}}$) resisted by these specimens is listed in column 7 of this table. This shear force value equals half of the totally applied external load. As can be seen, the shear strength of the control rectangular beam (CRB) without
any stirrups or CFRP/SFRP strips was recorded equal to 39.4 kN, whereas the shear capacity of specimen CRBs with stirrups Ø8/250 mm as transverse reinforcement was increased to 90.9 kN. From the results shown in Table 1, it is evident that the maximum recorded shear capacity for all strengthened rectangular beams with CFRP/SFRP strips was significantly greater than the shear capacity of either the CRB or the CRBs specimens. The observed mode of failure is listed in column 8 of Table 1. As can be seen, when FRP strips are without anchors the debonding mode of failure of the FRP strips prevails. This debonding mode of failure limits the observed shear capacity increase to 138% (nearly 2.4 times larger than the initial value of 39.4 kN), when the shear capacity of the strengthened specimens is compared to that of specimen CRB (without any shear reinforcement). When anchoring devices were utilised for prohibiting this debonding mode of failure they resulted in shear capacity increase of up to 210% (3.1 times larger than the initial value of 39.4 kN). When the shear capacity of the strengthened specimens is compared to that of the virgin specimen with nominal transverse reinforcement in the form of steel stirrups (CRBs), the resulting maximum shear capacity increase is equal to 7.6% when using CFRP strips without anchoring devices and 34.2% when using SFRP strips with anchors. The beneficial effect of anchoring the U-shape open-hoop FRP strips in order to inhibit the debonding mode of failure is thus demonstrated. The subsequent numerical study will focus on these important features of the observed behaviour, that is the prevailing mode of failure and the measured ultimate shear capacity ($V_{\text{max}}$).

3. Numerical analysis

The general purpose FE software ABAQUS [19] was employed to generate FE models to simulate numerically the structural response of the previously described concrete beams strengthened with externally attached FRP sheets. The generated models were validated against all respective experimental results. The aim of this part of the study is to generate reliable FE models that can be utilised to enhance the understanding of the fundamental structural response of the FRP shear strengthening scheme of RC beams with and without anchorage devices and hence optimise the anchorage device design.

3.1. Modelling assumptions

The mean measured specimen geometries were utilised to simulate the test specimens. The full 3D geometry of the concrete volume and the steel plates employed for the anchorage of the FRP was modelled with 3-D brick-type F.E. elements, whilst the FRP sheets were idealised as planar F.E. elements and their actual...
thickness (0.75 mm) was utilised as a section property. The bolts were assumed cylindrical with a diameter equal to the equivalent diameter corresponding to the net bolt area (i.e. the threaded part of the bolts was not explicitly modelled). A further idealisation pertinent to the bolt assembly simulation is the assumption that the bolt hole diameter of the steel plate is equal to the assumed bolt diameter (i.e. the clearance has not been accounted for).

In order to reduce the computational cost, the symmetry of the structure with respect to the x–y and z–y mid-planes was exploited, by numerically simulating one quarter of the physical model and applying the appropriate boundary conditions at each of the two planes of symmetry as shown in Fig. 4. The load was applied incrementally as prescribed displacement on a steel plate (SP1) at the point of load application, which was applied iteratively utilising a smooth amplitude curve available in ABAQUS. Restraint of the structure in the direction of the prescribed displacement was provided by fixing the \( u_z \) at the supporting steel plate (SP2). All the restraints introduced to the model are shown in Fig. 4. The sum of the reaction forces in the y direction at SP2 yields reaction \( R_y \) that corresponds to half of the shear force that is exerted to the model structure (e.g. \( R_{SP1} \) Table 1).

The contact between the various parts of the model (steel bolts, steel plates and concrete volume) was explicitly modelled and a friction coefficient equal to 0.3 was assumed for tangential contact behaviour except for the contact between the concrete beam and the loading/reaction plates which was assumed frictionless. Variation of the friction coefficient applied to the bolt-concrete interface between the idealised frictionless and rough contact extreme cases was attempted, but no significant effect on the overall solution was observed. Moreover, in order to reduce computational cost, the actual detail of the cylindrical rod of the anchoring device through which load is transferred from the FRP strips to the concrete beam was somewhat simplified (see Figs. 3b and 5); towards this end, the displacements of the FRP's edge were constrained to be equal to the respective displacements of the steel plate's mid plane for the numerical representation of the anchoring device. The interface between the FRP and the concrete was simulated as a cohesive zone endowed with a suitable traction separation response, the properties of which are discussed below.

### 3.2. Material modelling

The auxiliary test results reported previously were utilised herein to define the material response. The SFRP/CFRP sheets were assumed orthotropic under plane stress conditions and the material properties reported in Table 3 were adopted. Moreover, due to some specimens failing by FRP rupture, damage and failure of the FRPs was explicitly modelled by adopting the Hashin damage model [17] incorporated in ABAQUS. Only tensile failure in the principal direction was considered and the ultimate stress reported in Table 3 was adopted, whilst other failure modes were considered irrelevant to the present study. A fracture energy equal to 0.01 was assumed to simulate damage evolution, which was rapid, due to the severe stress concentration following the onset of failure. Steel plates, longitudinal reinforcing bars, steel stirrups and steel bolts and plates were assumed elastic-isotropic hardening and the experimentally obtained stress–strain curves for the steel used for these parts were converted into the true stress-logarithmic plastic strain format according to Eqs. (1) and (2) and utilised to define the material response.

\[
\sigma_{true} = \sigma_{nom}(1 + \epsilon_{nom}) \\
\epsilon_{true} = \ln(1 + \epsilon_{nom}) - \frac{\sigma_{true}}{E}
\]

The damaged plasticity model for concrete available in the ABAQUS material library was adopted to model concrete response, since it has been shown to perform satisfactorily in similar applications [23]. All material parameters were initially calibrated to accurately simulate the response of the two control beam specimens CRB and CRBs. The values adopted for the relevant material parameters, namely the angle of dilation \( \theta \), the eccentricity, the ratio of equibiaxial to uniaxial compressive stress \( f_{s1}/f_{so} \), the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield \( K_c \), and the viscosity parameter were 40, 0.1, 1.16, 0.666 and 0 respectively and were all within the range encountered in literature [23]. In accordance with similar studies [29], the stress strain response of concrete in compression was derived according to Saenz [34], whilst the tension stiffening response was defined in terms of tensile stress and axial deformation according to Cornelissen et al. [12], assuming a fracture energy equal to 0.06. The Young's modulus \( E \) and tensile strength \( f_{ct} \) were determined according to ACI [1] as a function of the compressive cylindrical strength, whilst Poisson's ratio was assumed equal to 0.2. The comparison between this assumed in the numerical simulation stress–strain concrete behaviour and the one measured by testing concrete cylindrical specimens taken during the construction of the beam specimens is depicted in Fig. 6. It must be borne in mind that these specimens were simply loaded in compression in such a way that the descending branch of the stress–strain curve could not be obtained.

The resin rich layer, within which the debonding of the FRP strip from the concrete volume occurs, was modelled as a cohesive zone [3] endowed with a traction–separation response. The initial elastic stiffness for all three modes of debonding was set equal to 106 N/mm², which is within the range of values proposed by Turon et al. [40]. The quadratic stress-based damage initiation criterion
available in ABAQUS was adopted, with the stress limit in all three principal directions being equal to the respective tensile concrete strength. Damage evolution was defined in terms of fracture energy with linear softening, whilst mixed mode behaviour was accounted for according to the model proposed by Benzeggagh and Kenane [8], with a power coefficient equal to 1.45, in accordance with the proposals of Obaidat et al. [29]. Best results were obtained for a fracture energy in both shearing and peeling modes equal to 0.25 (\(G_{II} = G_{III} = 0.25\) N/mm²), whilst the fracture energy \(G_I\) associated with the opening mode was set equal to 0.025. It should be noted that overall response is relatively insensitive to the assumed elastic stiffness and limiting stress adopted, but these properties have a significant effect on the numerical cohesive zone length [16,40], which defines the minimum required mesh size.

### 3.3. Non-linear analysis and discretisation

Linear 8-noded brick elements were adopted for the discretisation of the concrete beam, steel bolts, steel plates and loading plates, whilst the longitudinal reinforcing bars and the steel stirrups were discretised with linear truss elements embedded in the concrete region (i.e. no relative displacement between reinforcement and concrete was allowed for). Linear 4-noded shell elements were used to discretise the FRP sheets, the degrees of freedom (DOF) of which were tied to the respective DOFs of the underlying 8-noded 3D cohesive elements. The mesh size was dictated by the attempt to minimise computational time whilst maintaining accuracy and, upon extensive mesh convergence studies, a uniform mesh size of 15 mm was adopted for the concrete beam, cohesive zone and FRP sheets, whilst smaller mesh sizes were adopted for the steel plates, bolts and concrete regions surrounding the bolts as shown in [Fig. 5]. Due to severe convergence problems typically associated with strongly nonlinear response and with materials exhibiting non-monotonic stress–strain response [13], such as concrete and cohesive elements, the explicit dynamics solver ABAQUS/EXPLICIT was employed to perform the nonlinear analyses. Quasi-static response was achieved by specifying a slow displacement rate and checking that the kinetic energy was smaller than 2% of the internal energy for the greatest part of the analysis.

### 4. Results and discussion

The F.E. numerical predictions regarding the ultimate shear capacity (\(V_{\text{num}}\)) and failure modes with the ones observed during testing (see Table 1) are summarised in Table 4.

The ratio of numerically predicted over measured ultimate shear capacity is listed in column 3 of Table 4 with values very close to 1.0, which signifies a very good agreement of the

<table>
<thead>
<tr>
<th>Designation</th>
<th>Numerical ultimate shear capacity (V_{\text{num}}) (kN)</th>
<th>(V_{\text{num}}/V_{\text{exp}})</th>
<th>Numerical failure mode</th>
<th>Observed failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRB</td>
<td>39.44</td>
<td>1.00</td>
<td>Shear cracking</td>
<td>Shear cracking</td>
</tr>
<tr>
<td>CRBs</td>
<td>87.89</td>
<td>0.97</td>
<td>Yielding of stirrups</td>
<td>Yielding of stirrups</td>
</tr>
<tr>
<td>RB200C</td>
<td>97.99</td>
<td>1.00</td>
<td>Debonding</td>
<td>Debonding</td>
</tr>
<tr>
<td>RB200Ca</td>
<td>118.79</td>
<td>1.03</td>
<td>Anchorage failure</td>
<td>Anchorage failure</td>
</tr>
<tr>
<td>RB200S</td>
<td>87.54</td>
<td>0.93</td>
<td>Debonding</td>
<td>Debonding</td>
</tr>
<tr>
<td>RB200Sa</td>
<td>112.47</td>
<td>0.92</td>
<td>Anchorage failure</td>
<td>Anchorage failure</td>
</tr>
<tr>
<td>RB150C</td>
<td>99.65</td>
<td>0.99</td>
<td>Debonding</td>
<td>Debonding</td>
</tr>
<tr>
<td>RB150Ca</td>
<td>120.58</td>
<td>0.98</td>
<td>FRP rupture</td>
<td>FRP rupture</td>
</tr>
<tr>
<td>RB150S</td>
<td>96.95</td>
<td>0.94</td>
<td>Debonding</td>
<td>Debonding</td>
</tr>
<tr>
<td>RB150Sa</td>
<td>123.52</td>
<td>1.04</td>
<td>Anchorage failure</td>
<td>Anchorage failure</td>
</tr>
</tbody>
</table>

Mean value 0.98
Coef. of variation 0.05
numerically predicted ultimate shear capacity values with the respective experimental ones included in Table 1. Moreover, the numerically predicted modes of failure are also in very good agreement with the ones observed during testing, with the exception of specimen RB200Ca. This discrepancy can be attributed to the premature failure of the CFRP strip due to the non-ideal preparation of the concrete surface at the bottom fibre of this specimen prior to gluing this CFRP strip. Detailed discussion for each of the three types of rectangular beams considered in the present study follows.

4.1. Specimens of rectangular beams without FRP sheets

The observed behaviour of the two control beam specimens, which were initially tested to obtain the basic structural response of the reinforced concrete rectangular beam specimens without the added complexity of the attached FRP sheets, was, in general, well-predicted by the FE numerical models, as can be seen in Fig. 7, where the experimental and numerical load–displacement response is depicted. The load $V$ refers to the applied shear force and equals the half of the totally applied external load (Fig. 2), whilst the $d$ is the corresponding average displacement at the points of load application. In Fig. 8 the experimental failure mode, in the form of the dominant shear crack, and the numerical failure mode of specimen CRB are compared. Specimen CRBs displays a similar failure mode at a much higher load, with a major diagonal crack forming upon the yielding of the stirrups. It should be noted that in all these figures depicting numerical failure modes, the maximum principal plastic strain of concrete is depicted as a means to visualise concrete cracking since no discrete cracks form during the adopted numerical analysis.

4.2. FRP-strengthened beams without anchorage

The experimental and numerical load–displacement response of the concrete beam specimens strengthened with FRP sheets without the provision of additional anchoring devices is depicted in Fig. 9 for specimens RB200C and RB150S, respectively; as can be seen, an excellent agreement between the experimental and the numerical results is achieved. All these FE numerical simulations of the observed behaviour exhibited almost identical structural response, regardless of the employed FRP material (i.e. CFRP or SFRP) and the spacing considered. The beam specimens behaved elastically prior to the formation of the initial flexural cracks, whereupon the flexural reinforcement prevented the opening of the cracks. Subsequently, a diagonal crack started to form beneath the point of application of the load, which was interrupted by the presence of the FRP sheets which were stressed and prevented the opening of the diagonal crack. Finally, the debonding of the FRP sheets commenced and propagated rapidly, leading to the
formation of critical diagonal cracks and thus to failure. This observed behaviour was also accurately reproduced by the previously described numerical simulations.

The debonding occurred within a thin resin rich layer of concrete, the properties of which were assumed to be incorporated in the cohesive zone material properties. Typical numerical flexural cracks and the corresponding axial stresses in the reinforcing bars are depicted in Fig. 10, whilst the crack pattern and the FRP stresses in their principal direction at ultimate load are shown in Fig. 11. The numerical failure mode is compared with the corresponding experimental one for specimen RB200S in Fig. 12.

Fig. 10. Flexural cracks for specimen RB200S at d = 1.1 mm.

Fig. 11. Crack pattern and FRP stresses for specimen RB200S at ultimate load.

Fig. 12. Experimental and numerical failure modes for specimen RB200C.

Fig. 13. Experimental and numerical load-displacement curves for specimen RB200Sa and detail of localised deformation at bolt hole.

Fig. 14. Experimental and numerical cracking patterns for specimen RB200Ca.
4.3. FRP-strengthened beams with anchorage device

As described in Sections 4.1 and 4.2 all FRP-strengthened specimens to which no additional anchorage was provided failed abruptly, due to debonding of the FRP sheets. The provision of additional anchorage by means of the device shown in Fig. 3b allows for additional forces to be carried by the FRPs once debonding has occurred; these forces are transferred by the anchoring device finally through the bolts into the concrete thus resulting in an increased ultimate shear capacity and thus in a more efficient utilisation of the FRP material. The anchorage zone is stressed from the onset of loading but deformations and stresses remain small as long as the resin remains effective. Upon debonding, the anchorage zone experiences high localised stresses. The failure is no longer due to FRP debonding but rather localised above the anchorage zone, due to the high stresses transmitted by the bolts in the vicinity of the bolt holes. The failure mode is complex and involves a combination of local concrete cracking and crushing near the bolt holes, flexure of the compressive reinforcement due to the forces exerted by the bolts and possibly pullout of the anchoring bolts. In some cases, FRP rupture may occur prior to the failure of the anchorage device or anchorage zone. The generated FE models were in general able to capture both the FRP rupture and localised cracking above the anchorage zone. Fig. 13 depicts the experimental and numerical V-d response for specimen RB200Sa and the corresponding numerical failure mode which is principally extensive cracking and crushing of the concrete in the vicinity of the bolt holes. This was also observed during testing that led to the pullout of the anchor bolts. Fig. 14 displays the experimental and numerical failure modes for RB200Ca, which involves localised cracking in the vicinity of the anchorage zone which evolves into critical diagonal cracks.

5. Conclusions

1. The successful numerical simulation of the application of a novel anchoring device together with open hoop FRP strips, which enhances the efficiency of these FRP sheets applied as shear reinforcement for reinforced concrete rectangular beams, was demonstrated by this study.

2. The validation of the numerical simulation is based on the measured behaviour of two (2) full-scale reinforced concrete rectangular beams with or without nominal shear reinforcement as well as with eight (8) full-scale reinforced concrete rectangular beams that employed open hoop FRP strips, with or without novel anchoring devices, as a means of upgrading their shear capacity.

3. The success of the deployed numerical simulation is fully demonstrated by comparing first the observed and numerically predicted ultimate shear capacity for all these specimens. The achieved accuracy of the predicted ultimate shear capacity is excellent.

4. The success of the deployed numerical simulation is fully demonstrated by then comparing the observed and numerically predicted mode of failure for each one of the ten (10) tested specimens. It is shown that the deployed numerical simulations have been articulated with all the significant features, to be able to portray in a very realistic manner complex non-linear mechanisms such as the cracking and crushing of concrete, the debonding and fracture of the FRP strips and the yielding and fracture of the steel parts of the novel anchoring devices.

5. The success of the deployed numerical simulation also extends to the shape of the load–deflection curves when they are compared with the ones obtained during testing. Within the scope of the present paper, the studied beam specimens were subjected to static monotonic loads.

6. From the above, it must be seen that the accurate numerical prediction of a relatively complex behaviour is the result of all these intelligent features that were built into the numerical model together with the accurate representation of all the individual material features which were established by careful testing prior to the numerical simulation. The successful FE models, accurately capturing the basic structural response in terms of ultimate load, failure modes and overall load–displacement response, have been developed and discussed.

7. Based on the validated FE numerical models presented here, further research is underway to identify the key features affecting overall structural response and thus to optimise the structural arrangement of the proposed novel anchoring device in terms of strength and ductility.

8. A further goal is to derive a generic strengthening technique and associated design guidelines, which allow for a more efficient utilisation of the FRPs’ strength and significantly enhance the structural performance of existing structures strengthened with FRP sheets for both static and seismic type loads.

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