

Article

Shear–Flexure Integrated Strengthening of RC Beams with Near-Surface Mounted Carbon Fiber-Reinforced Polymer (CFRP) Ropes and Geopolymer Overlays

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Abstract

The strengthening of reinforced concrete (RC) beams requires repair systems that can enhance strength, stiffness, and energy dissipation without significantly increasing self-weight or compromising durability. This study explores the structural response of RC beams strengthened using an integrated shear–flexure system combining near-surface-mounted carbon fiber-reinforced polymer (NSM-CFRP) ropes and steel-reinforced geopolymer overlays in the compression zone. Monotonic three-point bending tests were performed on two RC beam specimens, one unstrengthened control and one strengthened beam, to obtain preliminary observations of load–deflection behavior, stiffness, ductility, and energy absorption. The strengthened specimen exhibited increases in ultimate load (28.6%), stiffness (13.6%), and energy absorption (7.65%) relative to the control beam, suggesting the potential for effective composite action between the CFRP ropes and geopolymer material. A three-dimensional nonlinear finite element model was developed using ATENA to support interpretation of the experimental response, incorporating detailed constitutive models for concrete, steel reinforcement, and CFRP ropes. The numerical predictions showed reasonable agreement with the experimental results. Within the limitations of the test matrix, the results indicate that the proposed dual strengthening system may offer a viable and sustainable approach for enhancing the shear–flexural performance of RC beams.

Keywords: RC beams; CFRP ropes; geopolymer; shear–flexure strengthening



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1. Introduction

Structural assessment and inspection reports frequently reveal deficiencies in existing buildings, particularly those constructed prior to the adoption of modern design codes and detailing requirements [1,2]. Although early structural design practices were largely governed by strength-based criteria, modern seismic engineering has increasingly emphasized ductility as a governing parameter for ensuring adequate deformation capacity and energy dissipation during earthquakes [3–5]. This shift in design philosophy is particularly relevant in seismically active regions such as Indonesia, where recurrent earthquake events over recent decades have caused widespread structural damage [6,7]. Consequently, the

development of strengthening and rehabilitation strategies that are both structurally effective and economically feasible has become an essential focus in earthquake-prone regions.

Continuous development and refinement of strengthening strategies are therefore essential to sustain or improve the structural integrity of existing buildings while ensuring acceptable performance levels throughout their service life [8–10]. Traditional rehabilitation techniques for deficient or damaged reinforced concrete (RC) members typically involve the application of steel or concrete jacketing systems [11–18]. While effective in increasing strength, such approaches inevitably increase structural self-weight and may introduce durability concerns, particularly corrosion-related issues when steel components are employed. These limitations motivate the exploration of alternative materials and systems that can improve structural performance without significantly altering mass or long-term durability.

Recent research has increasingly focused on the development and application of environmentally sustainable materials for the repair and strengthening of RC structures [19–24]. Within this context, geopolymer materials have gained significant attention as sustainable alternatives to traditional cement-based binders. Geopolymers are formed through an exothermic polymerization process involving silica (SiO) and alumina (Al₂O₃) activated in an alkaline environment, resulting in amorphous aluminosilicate networks characterized by high mechanical capacity. The presence of alkaline activators facilitates the formation of polymeric Si–O–Al bonds, which govern the material's mechanical and durability performance. Due to their favorable workability, chemical compatibility with concrete substrates, and thixotropic behavior, geopolymers have been widely identified as suitable candidates for structural rehabilitation applications [25].

Existing studies have demonstrated that geopolymer-based repair materials exhibit favorable bond behavior, which plays a critical role in enhancing the strength and long-term durability of rehabilitated RC elements. Pacheco-Torgal et al. [26] provided a comprehensive overview of geopolymer-based repair and strengthening techniques for concrete infrastructure, highlighting their potential across various rehabilitation scenarios. Effective RC repair, however, depends strongly on the compatibility between the repair material and the existing concrete substrate, especially with respect to interfacial bonding performance [27]. Zailani et al. [28] examined the bond behavior of a fly ash-based geopolymer applied to concrete surfaces and reported a denser microstructure with reduced porosity. Their findings further indicated that increasing the calcium content in the geopolymer formulation can significantly enhance both mechanical performance and bond strength.

Furthermore, fiber-reinforced polymer (FRP) materials have experienced rapid advancement and have become increasingly prominent in the field of structural strengthening and rehabilitation. FRP is composed of high-strength fibers, provides the primary contribution to strength and stiffness. These fibers are embedded within a polymer matrix that facilitates the transfer of stresses and strains between the reinforcement and the surrounding substrate through in-plane shear, while also protecting the fibers from environmental degradation. Due to their adaptability and high performance, FRP composites are now widely recognized as practical alternatives to strengthening techniques. These systems have been applied to a wide variety of structural materials, including concrete [29–37], steel [38,39], wood [40–42], and masonry [43–45], with the objective of improving load-carrying capacity and overall structural response.

FRP-based strengthening solutions also offer notable advantages over conventional rehabilitation methods. These advantages include reduced self-weight, high durability, and superior resistance to corrosion-related damage [46]. Moreover, the mechanical characteristics of FRP materials allow strengthening interventions to be implemented in a discrete and minimally invasive manner. This aspect is especially important for historical or archi-

tecturally valuable buildings, where extensive replacement or reconstruction of structural elements is generally undesirable or impractical. The ability to integrate FRP systems into existing structures without adversely affecting their architectural appearance enables an effective balance between structural enhancement and preservation requirements.

The combined application of FRP and geopolymer materials represents a promising approach for strengthening RC members by exploiting the complementary mechanical characteristics of each material. In such systems, FRP primarily contributes tensile resistance and crack-bridging capacity, while geopolymer materials enhance compressive performance and material compatibility within the compression zone. Previous studies have reported that hybrid strengthening strategies combining FRP with cementitious overlays or jackets can improve load-carrying capacity, ductility, and energy absorption, while delaying the formation of critical shear cracks in RC members subjected to combined flexural and shear demands [47]. Nevertheless, most existing hybrid FRP–overlay strengthening approaches employ FRP sheets, strips, or rigid rods in conjunction with cement-based overlays, where the shear and flexural contributions are designed and evaluated independently, rather than being mechanically integrated into a unified shear–flexural strengthening system, as proposed in the present study.

The novelty of the present study lies not in the general concept of hybrid strengthening, but in the specific configuration and integration of its components. The proposed system employs flexible near-surface-mounted (NSM) Carbon-FRP (CFRP) ropes arranged as side-installed, stirrup-like reinforcement to enhance shear resistance, combined with a steel-reinforced geopolymer overlay intentionally placed in the compression zone to function as an enhanced compression flange for flexural strengthening. This configuration enables the simultaneous activation of shear confinement and compressive load transfer within a single strengthening scheme, distinguishing it from previously reported NSM- or overlay-based retrofit approaches that primarily target either shear or flexure independently.

A further distinguishing feature of the proposed method is the integration mechanism itself, whereby the NSM-CFRP ropes enclose and confine the geopolymer overlay, promoting composite action between the existing concrete, the geopolymer layer, and the FRP reinforcement. This interaction mechanism has not been explicitly examined in prior studies on hybrid FRP–cementitious strengthening systems. Figure 1 illustrates the practical application of the proposed method in both a real structural configuration and the corresponding experimental specimen. In this study, the strengthening overlay consists exclusively of geopolymer concrete applied to an existing RC beam, while the underlying RC member represents the original structural substrate rather than a second composite material. Accordingly, no comparison between different overlay materials is intended, and the results presented herein pertain solely to the performance of the geopolymer concrete strengthening layer.

In addition to the experimental investigation, a numerical model was developed to predict the ultimate load capacity of the strengthened beams and to support the interpretation of the observed structural response. Given the limited number of studies on NSM-CFRP ropes for shear strengthening of RC beams, the present work aims to extend existing knowledge by clarifying the effectiveness and structural behavior of this strengthening strategy through combined experimental and numerical evidence.

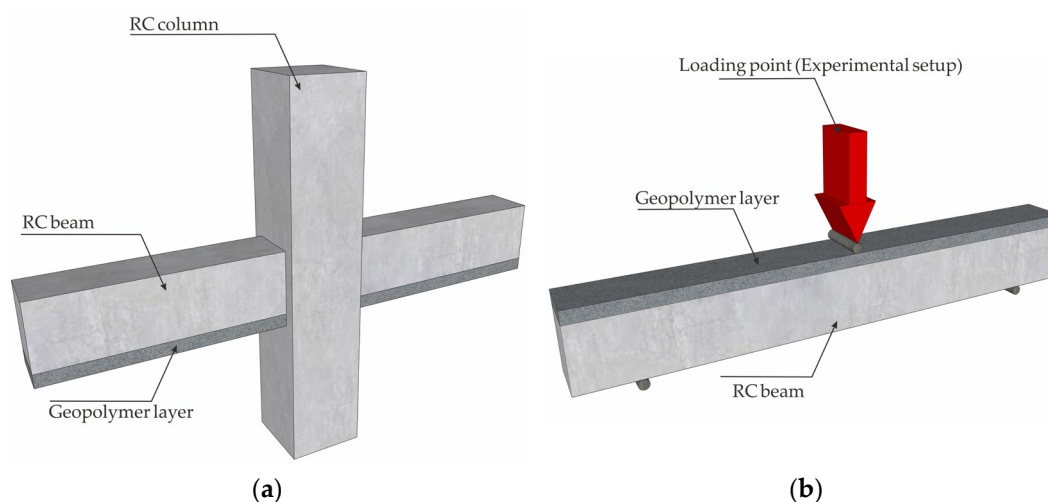


Figure 1. The application of the strengthening method: (a) Real structure; (b) Experimental test.

2. Experimental Program

2.1. Configuration of Test Specimens

Rectangular RC beam specimens were prepared and experimentally investigated in this study. The beams were constructed using the same batch of ready-mix normal concrete with a specified compressive strength of 25 Mpa to ensure consistency in material properties. Each beam measured 150 mm in width, 300 mm in depth, and 1000 mm in length. The longitudinal reinforcement layout comprised two deformed 13 mm diameter bars in the compression zone and three 19 mm diameter bars in the tensile zone, while shear reinforcement was provided by plain 6 mm diameter stirrups spaced at 250 mm intervals along the beam span.

One specimen was tested without any external strengthening and served as the reference beam (BN). The other specimen, denoted as BP, was strengthened using an integrated shear–flexural strengthening system, which incorporated NSM-CFRP ropes to enhance shear resistance and a steel-reinforced geopolymer concrete layer applied in the compressive zone to improve flexural performance. This strengthening configuration and its key components are illustrated in Figure 2.

The strengthening was applied after 28 days of concrete curing. Near-surface mounted (NSM) grooves were cut on both side faces of the beam within the strengthening region to install the CFRP ropes. Each groove had a rectangular cross-section with a depth of 10 mm from the concrete surface and a width of 10 mm, and was arranged at 75 mm center-to-center spacing along the beam length. The CFRP ropes were fully bonded using a two-part epoxy adhesive to ensure continuous force transfer along the embedded length. The NSM groove layout is illustrated in Figure 3.

A 100 mm thick steel-reinforced geopolymer overlay was then cast over the top (compressive) zone and reinforced with Ø13 mm longitudinal steel bars. The NSM-CFRP ropes were arranged to function as closed stirrups: the rope path ran up one side groove, crossed over the overlay at the top, and returned down the opposite side groove, thereby enclosing and confining the overlay and engaging the original RC section. Top and bottom anchorage was made explicit by embedding the rope legs and return portions within the side grooves and fully encapsulating them with epoxy, providing adequate development length to prevent slip and ensure a closed-loop force path. This detailing promotes composite action between the CFRP system, the geopolymer overlay, and the existing RC beam while enabling stirrup-like confinement and shear engagement.

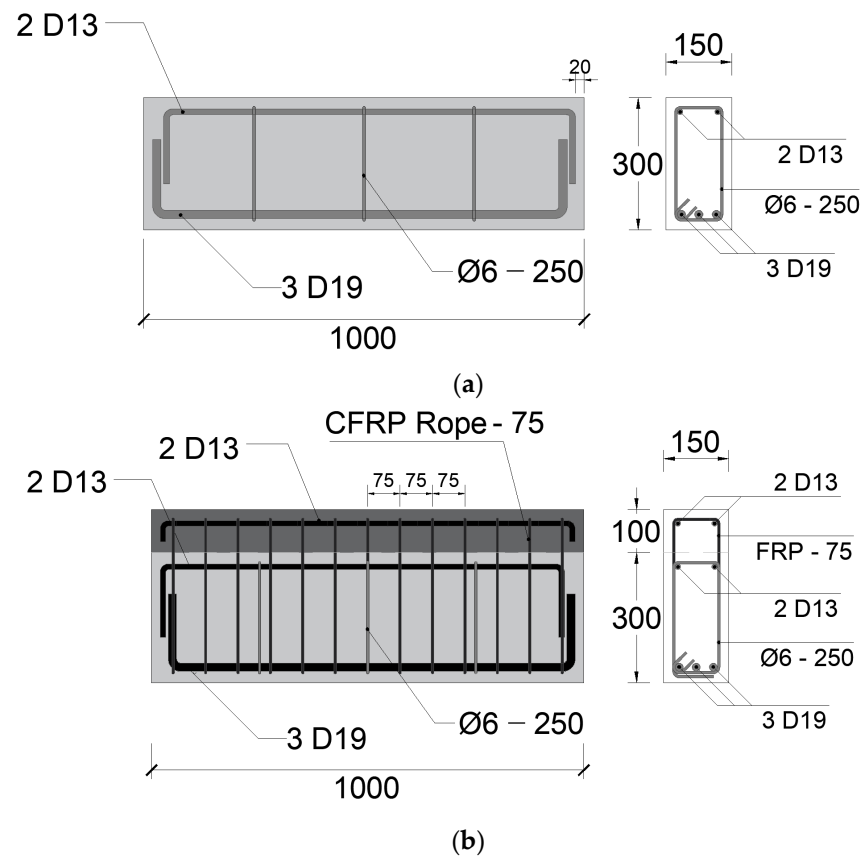


Figure 2. Geometry of the tested beams (Units: mm): (a) unstrengthened control specimen (BN); (b) strengthened specimen (BP).

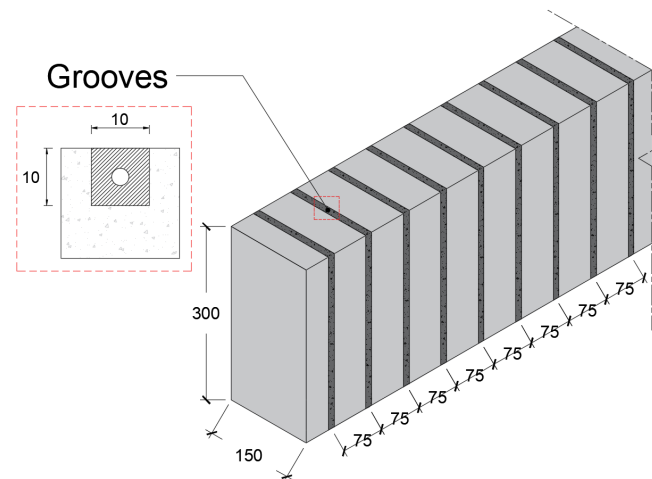


Figure 3. Layout and dimensions of NSM grooves for CFRP rope installation.

2.2. Materials

The geopolymer concrete employed in this study was prepared using coarse aggregate, river sand, alkaline activators (AA), and fly ash (FA) sourced as an industrial by-product. The binder system was based on a fly ash-activated aluminosilicate matrix with a limited addition of ordinary Portland cement (OPC), forming a hybrid geopolymer system rather than a purely alkali-activated material. The detailed mix proportions are summarized in Table 1.

Table 1. Detailed mix proportion of the geopolymer.

Material	Composition
Coarse aggregate:Fine aggregate	60%:40%
AA:FA	35%:65%
Aggregate:Binder (FA + AA)	70%:30%
Na ₂ SiO ₃ : NaOH (12 M)	2.5:1.0
Extra water	11.70% × Binder (FA + AA)
Extra cement	5.63% × Binder (FA + AA)
Superplasticizer	2.00% × FA

X-Ray Fluorescence (XRF) analysis of the fly ash indicated a high combined content of SiO₂, Al₂O₃, and Fe₂O₃ (91%), with a relatively low CaO content of approximately 5%, characteristic of low-calcium fly ash [48]. Although this calcium content contributes marginally to strength development, it is insufficient to ensure rapid setting and adequate early-age strength under ambient curing conditions, particularly for self-compacting geopolymer concrete used as a structural overlay. Therefore, a small amount of cement (5.63% of the total binder content) was intentionally incorporated to supplement calcium availability, accelerate early-age reaction kinetics, and improve setting stability and early strength development, without altering the fundamental geopolymerization mechanism.

The alkaline activator consisted of sodium silicate (Na₂SiO₃) and sodium hydroxide (NaOH) solutions, with the NaOH prepared at a molarity of 12 M and mixed with Na₂SiO₃ at a mass ratio of 2.5:1.0. In this system, NaOH facilitates the dissolution of aluminosilicate species from the fly ash, while Na₂SiO₃ promotes polymerization and binding of the reaction products [49]. An alkaline activator-to-fly ash (AA/FA) ratio of 0.54 was selected to ensure sufficient geopolymerization while maintaining self-compacting workability. The superplasticizer dosage was defined as a percentage of fly ash content rather than total binder content because fly ash constitutes the dominant reactive phase governing rheology, whereas the cement addition was minor and primarily served as an early-age strength accelerator rather than a workability-controlling component.

The geopolymer overlay applied to the strengthened beam was cured under ambient laboratory conditions without external heat curing and allowed to mature for 28 days prior to structural testing. The normal concrete used in the original RC beams was supplied by a local commercial mixing plant. The mechanical properties of both materials were evaluated using three standard cylindrical specimens (150 mm in diameter and 300 mm in height), with compressive strength determined in accordance with ASTM C39/C39M-14 [50] and splitting tensile strength measured following ASTM C496/C496M-11 [51]. In addition, the shear-bond strength at the interface between the geopolymer overlay and the existing concrete substrate was assessed using U- and T-shaped composite specimens based on the method proposed by Purwanto et al. [52]. The average mechanical properties measured at the test age are summarized in Table 2. Given that the geopolymer overlay was placed in the compression zone of the strengthened beam, its compressive behavior governed its primary structural contribution in the present study.

Table 2. Mechanical properties of normal concrete and geopolymer.

Parameter	Normal Concrete	Geopolymer
Compressive strength (MPa)	28.00	27.00
Splitting tensile strength (MPa)	2.70	2.40
Shear-bond strength (MPa)	1.90	2.00

The steel reinforcement cages for all RC beam specimens were fabricated using steel bars with nominal diameters of 6 mm, 13 mm, and 19 mm. Deformed steel bars with

diameters of 19 mm and 13 mm were used as the longitudinal reinforcement to provide the required flexural capacity, while plain steel bars with a diameter of 6 mm were employed as transverse stirrups for shear resistance and confinement. The yield strength and ultimate tensile strength of both longitudinal and transverse reinforcement, obtained from tensile tests, are summarized in Table 3.

Table 3. Mechanical properties of steel reinforcement.

Diameter of Steel Rebar (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)
19	473	633
13	334	461
6	328	439

One of the specimens was strengthened using flexible near-surface mounted CFRP ropes (SikaWrap® FX-50C) manufactured by SIKA (Baar, Switzerland), which were employed as side-installed, stirrup-like shear reinforcement. The CFRP ropes consist of bundles of unidirectional carbon fibers supplied in a dry, non-pre-impregnated (dry fiber) state and encased within a protective plastic envelope, making them suitable for NSM applications where impregnation occurs in situ through the bonding adhesive. According to the manufacturer's technical datasheet and composite testing conducted in accordance with ASTM D4018 [53], the CFRP rope has a nominal composite cross-sectional area of 64.52 mm², a tensile modulus of 2.3×10^5 MPa, a tensile strength of 2000 MPa, and an ultimate strain of at least 0.016. Based on these properties, the corresponding nominal tensile load capacity of a single rope is approximately 164 kN. The physical and mechanical properties of the CFRP ropes are summarized in Table 4.

Table 4. Physical and mechanical properties of CFRP rope (SikaWrap® FX-50C).

Property	Value
Physical properties of CFRP rope	
Weight	≥50 g/m (carbon fiber content)
Fiber cross-section	≥28 mm ² (based on carbon fiber)
Fiber density	1.82 g/cm ²
Dry fiber properties in the longitudinal direction of the fibers	
Tensile modulus, minimum	2.4×10^5 MPa
Tensile strength, minimum	4000 MPa
Strain at break	≥0.016
Composite properties according to ASTM D4018 [53]	
Tensile modulus	2.3×10^5 MPa
Tensile strength	2000 MPa
Composite cross-section	64.52 mm ²

The CFRP ropes were bonded using a two-component epoxy adhesive (Sikadur®-752, SIKA), which also served as the impregnation medium for the dry fibers. The epoxy adhesive exhibits a 28-day compressive strength of 54–64 MPa, a tensile adhesion strength of 2 MPa to concrete and 9 MPa to steel, and an elastic modulus in compression of approximately 1060 MPa, as reported in the manufacturer's datasheet and summarized in Table 5.

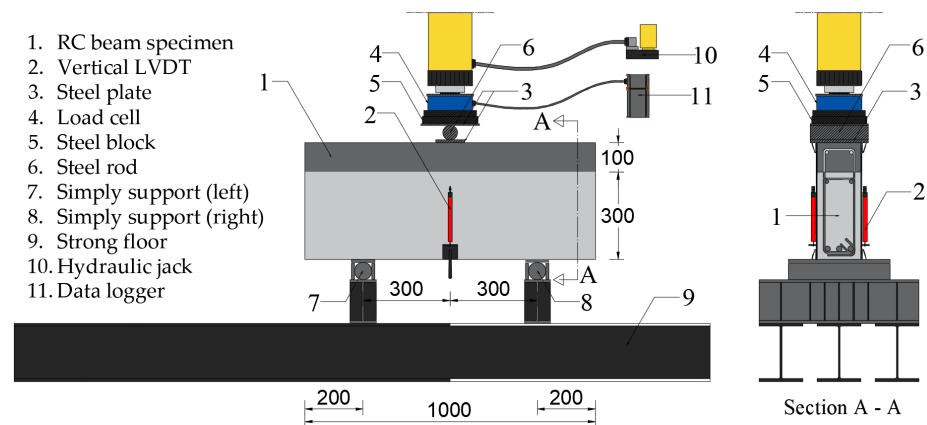
After installation, the strengthened beam was allowed to cure under ambient laboratory conditions for a minimum of 7 days before structural testing to ensure adequate epoxy hardening and bond development, recognizing that the bond behavior governs the effectiveness of NSM strengthening systems. The reported epoxy properties and curing protocol ensure reliable stress transfer between the CFRP ropes and the surrounding concrete substrate.

Table 5. Physical and mechanical properties of epoxy adhesive (Sikadur®-752).

Property	Specification
Physical properties	
Color	Yellowish
Packaging	5 kg set
Density	1.08 kg/L
Mechanical properties	
7 days compressive strength [54]	50–60 MPa
28 days compressive strength [54]	54–64 MPa
Modulus of elasticity in compression	1060 MPa
Tensile adhesion strength to concrete [54]	2 MPa
Tensile adhesion strength to steel [54]	9 MPa
Coefficient of thermal expansion	89×10^{-6} per °C

2.3. Test Setup

The experimental testing of the RC beam specimens was carried out under a three-point bending configuration with a clear span of 700 mm, selected to induce combined flexural and shear actions concentrated at the midspan. This setup ensured the development of a critical compression zone within the steel-reinforced geopolymer overlay of the strengthened beam, as illustrated in Figure 4. Vertical loading was applied using a hydraulic jack with a maximum capacity of 500 kN, and the applied force was transferred to the beam through a rigid steel block and steel rod assembly positioned at the loading point to promote stable and uniform load introduction.

**Figure 4.** Experimental setup of the tested beam.

To reproduce simply supported boundary conditions, the beam was tested using a pin-roller support configuration, in which vertical translation was restrained at both supports while horizontal movement was permitted at one end to prevent unintended axial restraint. Steel bearing plates with plan dimensions of 150 mm × 100 mm and a thickness of at least 20 mm were placed between the beam and the supports to ensure uniform stress transfer and to mitigate local crushing at the support regions. The load was applied at midspan through a steel loading plate of identical plan dimensions, distributing the contact pressure over the full beam width and minimizing local concrete damage beneath the loading point. The test was conducted under quasi-static force control at a constant loading rate of 120 N/s, which is sufficiently low to avoid rate-dependent effects and dynamic response [55].

Vertical deflections were measured using two linear variable differential transformers (LVDTs) with an accuracy of ± 0.01 mm, installed at the midspan on opposite sides of the beam to capture global flexural deformation and reduce potential measurement bias.

Crack initiation and propagation were visually monitored, and crack locations and patterns were marked directly on the beam surfaces throughout the loading process until failure. No dedicated instrumentation was used to directly measure slip at the interface between the geopolymer overlay and the existing concrete, and interfacial behavior was therefore assessed qualitatively based on surface observations; this is acknowledged as a limitation of the experimental program. Load and displacement data were continuously recorded using a high-precision data acquisition system to ensure reliable measurement of the structural response.

3. Test Result and Discussion

3.1. Load–Deflection Behavior and Strengthening Performance

Figure 5 presents the load–deflection responses of the control beam (BN) and the beam strengthened with near-surface mounted CFRP ropes combined with steel-reinforced geopolymer in the compression zone (BP). The strengthened specimen exhibited a clear improvement in structural response compared to the control beam, characterized by higher load-carrying capacity, an extended linear elastic range, and a larger deflection at peak load. The BN specimen showed a distinct post-peak behavior, with an approximate 20% reduction in load following the peak, whereas the BP specimen reached its maximum load without a pronounced strength drop prior to failure, indicating a more stable response up to peak capacity.

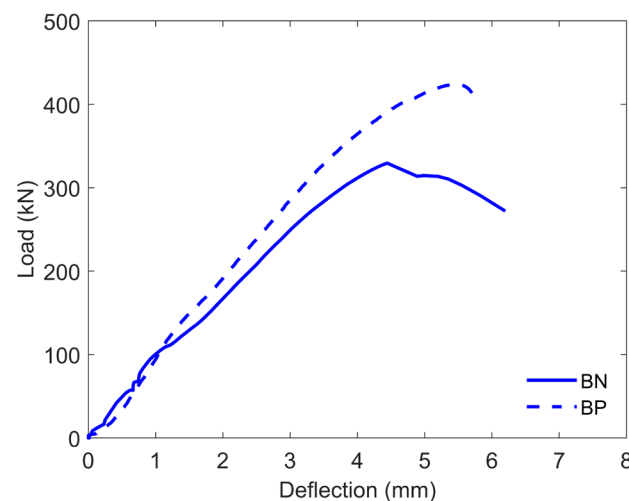


Figure 5. Load–deflection curves of tested beams.

The load capacities ($P_{\max}$) of BN and BP were recorded as 329.29 kN and 423.44 kN, respectively, corresponding to an increase of approximately 28.6% for the strengthened beam. This direct comparison demonstrates that the proposed strengthening approach effectively enhances strength and stiffness within the tested configuration, in agreement with trends reported for hybrid FRP–geopolymer strengthening systems [56,57]. However, for the short beam geometry considered in this study, the strengthening system did not result in a pronounced improvement in post-peak deformation capacity, contrasting with the more gradual post-peak degradation observed in longer beam specimens reported in earlier investigations [47].

The corresponding load–deflection parameters, including yield load (P_y) and yield deflection (δ_y), peak load ($P_{\max}$) with its corresponding deflection ($\delta_{P_{\max}}$), and failure deflection (δ_f) associated with the post-peak response, are summarized in Table 6, providing a quantitative basis for evaluating the strengthening effect relative to the unstrengthened control beam.

Table 6. Summary of characteristic load and deflection values and derived performance indices.

Specimen	Load (kN)		Deflection (mm)			Ductility	Stiffness kN/mm	Energy kN·mm
	Yield (P_y)	Peak (P_{max})	Yield (δ_y)	At Peak Load (δ_{Pmax})	Failure (δ_f)			
BN	309.99	329.49	3.97	4.44	6.20	1.56	83.07	13,490.58
BP	392.95	423.44	4.49	5.45	5.74	1.28	94.34	14,522.78

The yield point for each specimen was identified from the load–deflection curves using an equivalent elastoplastic (bilinear) approximation method [58]. The yield deflection of the control beam BN was determined to be 3.97 mm, while the strengthened beam BP exhibited a slightly higher yield deflection of 4.49 mm. Ductility, defined as the ratio between the deflection at failure (δ_f) and the deflection at yield (δ_y) according to Pam et al. [59], was calculated for both specimens. The ductility index of BN was 1.56, whereas BP exhibited a lower value of 1.28, representing a reduction of approximately 17.9%. This reduction is attributed to the different failure characteristics, as BN failed after a clear post-peak degradation phase, while BP reached failure without undergoing a substantial load reduction.

Stiffness was evaluated by calculating the slope of the line connecting the theoretical yield point (δ_y) to the corresponding load capacity (P_{max}). The stiffness values were 83.07 kN/mm for BN and 94.34 kN/mm for BP, indicating an increase of approximately 13.6% for the strengthened beam. The higher stiffness of BP contributed to increased resistance against crack initiation and early-stage deformation. In addition, the energy absorption capacity was assessed by calculating the area under the load–deflection curve [60]. The strengthened beam demonstrated a 7.65% increase in energy absorption compared to the control specimen. This improvement reflects the effective composite action between the NSM-CFRP ropes and the steel-reinforced geopolymer, which enhanced the toughness and overall structural resilience of the strengthened system.

3.2. Failure Mode

Figure 6 presents the failure modes and crack patterns observed in the control beam (BN) and the strengthened beam (BP); quantitative crack width measurements were not recorded and are acknowledged as a limitation. Clear differences in cracking development and failure characteristics were observed due to the integrated shear–flexural strengthening system. In the control specimen BN, initial cracking initiated near the midspan and rapidly developed into inclined diagonal cracks within the shear span as the load increased. These cracks propagated toward the loading point and support, forming a dominant shear–tension failure mechanism. Limited stress redistribution and the absence of additional shear reinforcement led to rapid crack widening and concrete crushing in the compression zone, governing the ultimate failure of the beam.

The strengthened specimen BP exhibited delayed crack initiation at higher load levels, indicating increased stiffness and improved crack resistance compared to the control beam. Diagonal cracks formed within the shear span; however, their propagation and opening were effectively restrained by the NSM-CFRP ropes, which bridged the cracks and limited their development. With continued loading, damage became increasingly concentrated in the compression region beneath the loading plate, where localized crushing was observed primarily within the geopolymer overlay. Based on visual evidence, this crushing was confined to the overlay layer in the immediate loading zone, while the underlying original concrete did not show extensive crushing away from this region. Secondary flexural cracks developed near the midspan at later stages but did not govern the failure process. Accordingly, the failure of BP is characterized by a combined shear–compression mechanism influenced by local bearing-induced crushing in the geopolymer overlay, rather than

a purely flexural failure of the original beam. No visible interfacial debonding or splitting cracks were observed between the geopolymer overlay and the existing concrete, indicating effective composite action of the strengthening system up to failure.

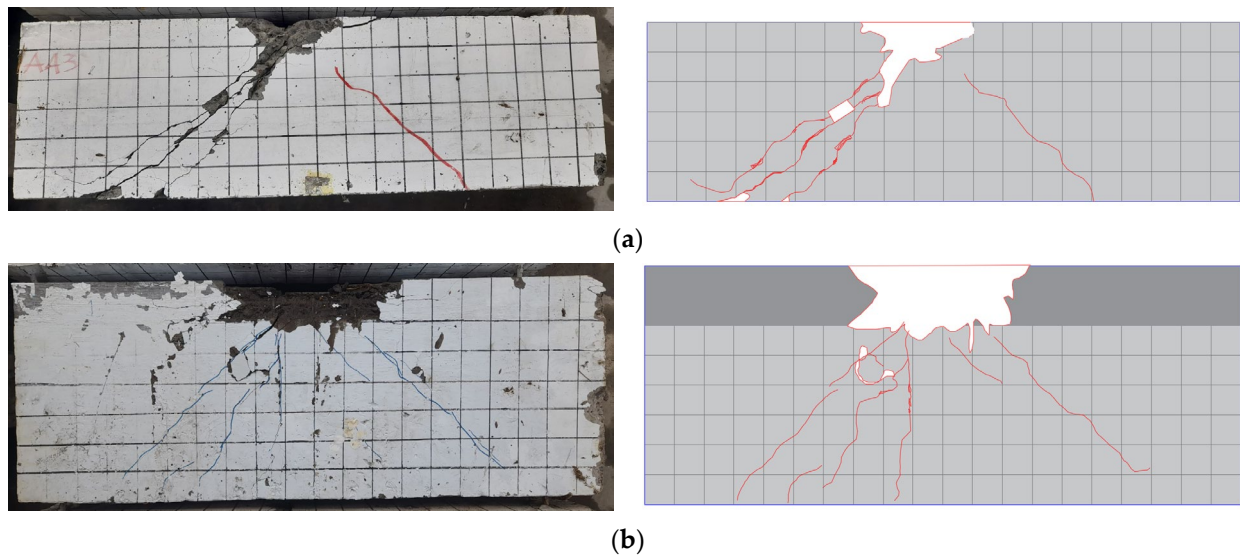


Figure 6. Failure modes of the tested beam specimens: (a) control beam (BN); (b) strengthened beam (BP).

4. Numerical Approach

4.1. Model Geometry and Boundary Condition

Three-dimensional finite element (FE) models were developed to simulate the behavior of the control beam (BN) and the strengthened beam (BP) tested in the experimental program. The numerical analyses were conducted using the ATENA software (ver. 2023.0.0.22492), a well-established finite element platform capable of simulating the nonlinear behavior of reinforced concrete (RC) elements, including cracking, crushing, and reinforcement interaction [61–63]. The FE models were constructed to closely replicate the experimental geometry, reinforcement configuration, material properties, loading scheme, and boundary conditions.

Due to the symmetry of the beam geometry, reinforcement layout, loading configuration, and boundary conditions, a symmetry-based modeling approach was adopted. Accordingly, only one-quarter of the beam was modeled to reduce computational demand while preserving the accuracy of the structural response. This quarter-model formulation improves computational efficiency without sacrificing predictive capability, as both geometric and loading symmetry are maintained. Figure 7 presents the FE representations of the control beam (BN) and the strengthened beam (BP), illustrating the modeled geometry, reinforcement detailing, and mesh discretization adopted in the analysis.

The numerical models included normal concrete, geopolymer concrete, longitudinal steel reinforcement (D13 and D19), CFRP ropes, and steel loading plates. The reinforcement arrangement and strengthening configuration in the FE models follow those of the experimental specimens. Normal concrete and geopolymer concrete were modeled as three-dimensional solid elements, while steel reinforcement bars and CFRP ropes were represented as embedded line elements to ensure full strain compatibility with the surrounding concrete. The steel plates at the loading and support locations were explicitly modeled as rigid components to ensure uniform load transfer and to prevent artificial stress concentration effects.

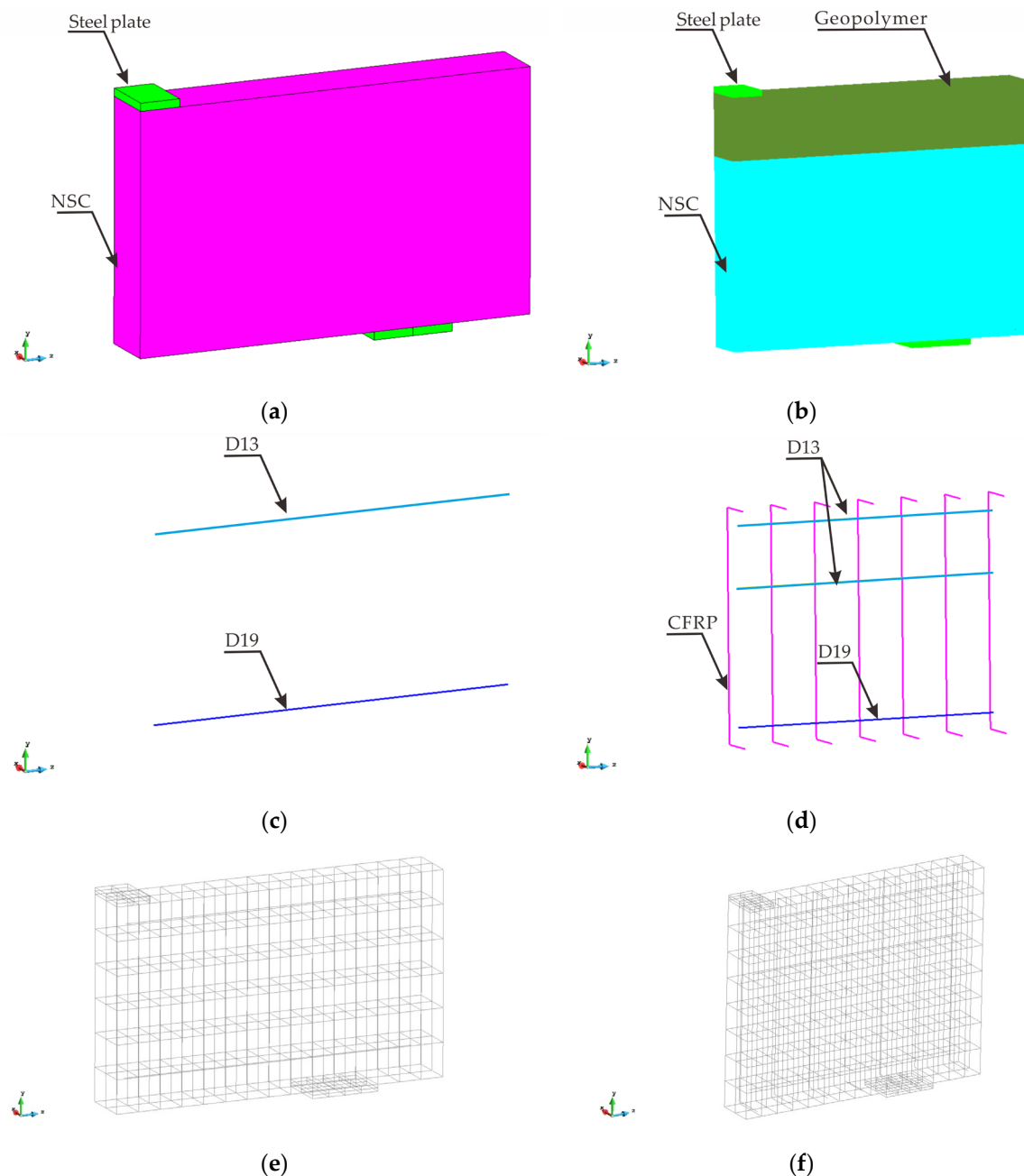


Figure 7. FE models of the RC beam specimens: (a) three-dimensional geometry of the control beam (BN); (b) three-dimensional geometry of the strengthened beam (BP); (c) steel reinforcement modeling in BN; (d) steel reinforcement and CFRP rope modeling in BP; (e) mesh distribution of BN; (f) mesh distribution of BP.

Model generation followed a systematic procedure beginning with the definition of nodal coordinates, which were subsequently connected to form line and surface entities before being converted into three-dimensional volumes. The control beam model incorporated two longitudinal reinforcement bars, whereas the strengthened beam model included three longitudinal reinforcement bars embedded within the concrete. Reinforcement elements were discretized as linear embedded elements, while normal concrete and geopolymer concrete were discretized using three-dimensional hexahedral solid elements. A perfect-bond assumption was adopted not only between the concrete matrix and the embedded reinforcement, but also at the interface between the existing normal concrete and the geopolymer overlay. This modeling assumption is justified by the experimental obser-

variations, which showed no visible interfacial debonding or slip up to failure, as well as by the measured interface shear–bond strength reported in Table 2. Accordingly, the numerical model focuses on capturing the dominant global load–deformation behavior governed by material nonlinearity and cracking, rather than local interface failure mechanisms.

A structured meshing strategy was adopted, with element sizes controlled through geometric segmentation to achieve an optimal balance between computational efficiency and solution accuracy. This meshing approach was informed by the findings of Zheng et al. [64], who reported that excessive mesh refinement results in minimal accuracy improvement while significantly increasing computational cost. The resulting mesh provided sufficient resolution to capture localized stress concentrations and crack propagation in critical regions.

To accurately reproduce the experimental setup within the quarter-span symmetry model, appropriate boundary conditions were applied. Vertical displacement along the central axis of the supporting steel plate was restrained using the “Constraint for Line” function. To enforce symmetry conditions, surface constraints were applied using the “Constraint for Surface” function, restricting displacement normal to the symmetry planes while allowing in-plane deformation. This ensured that the reduced model accurately reproduced the global flexural behavior of the full-span beam. Additional symmetry at the mid-span cross-section was maintained by constraining translational degrees of freedom in the x- and z-directions only, thereby avoiding artificial rotational restraint and preventing unintended stiffening of the numerical response. The implementation of boundary conditions in the FE model is shown in Figure 8a.

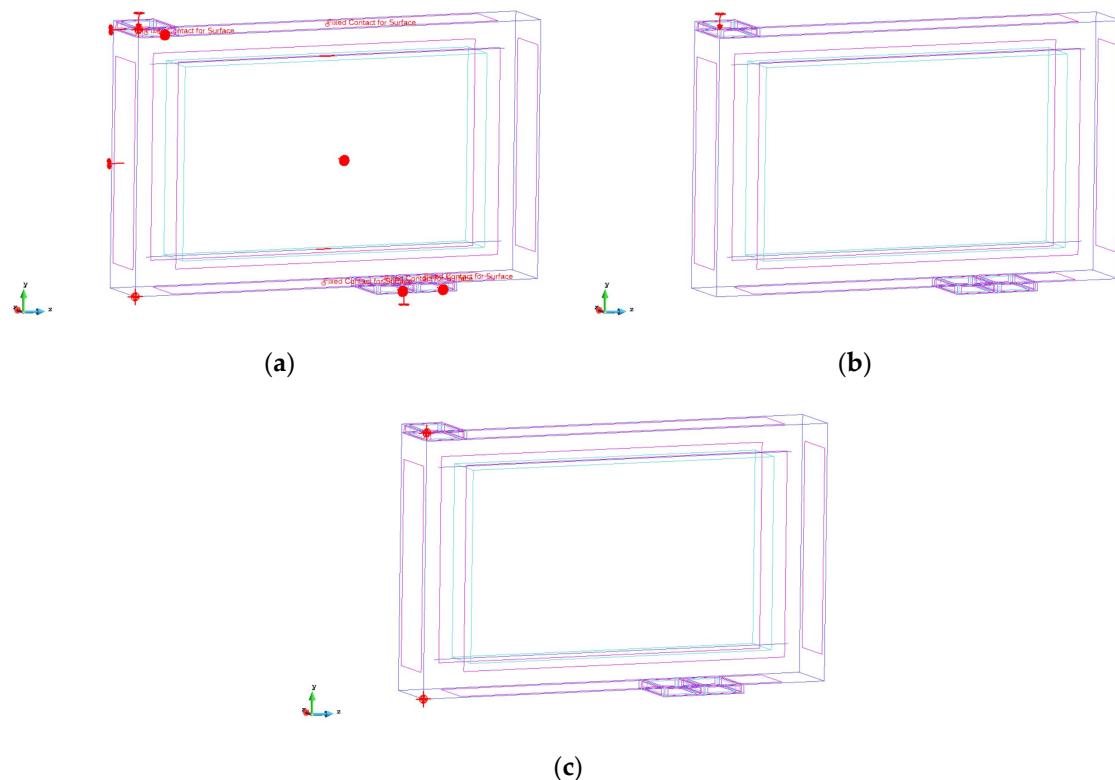


Figure 8. Implementation of boundary conditions: (a) boundary condition constraints applied to enforce symmetry and support conditions; (b) application of combined displacement- and load-controlled loading at the top rigid steel plate; (c) locations of monitored points used to record reaction forces and displacements. Red dots indicate the locations where boundary conditions/constraints were applied.

Loading was simulated using a combination of displacement- and load-controlled schemes by applying the “Displacement for Point” and “Load Force for Point” functions at the central node on the top rigid steel plate, as shown in Figure 8b. The structural response, including reaction forces and displacements, was recorded using monitored points defined within the FE model. Reaction forces were obtained from a monitored point assigned to the top steel plate, while vertical displacement was recorded at a monitored point located at the bottom of the steel plate. The locations of these monitored points are shown in Figure 8c.

4.2. Constitutive Material Modeling

4.2.1. Steel Reinforcement and CFRP Ropes

The constitutive behavior of the steel reinforcement was differentiated according to its structural function. The longitudinal reinforcement was described using a multi-linear stress–strain relationship, as presented in Figure 9a, enabling accurate representation of elastic behavior, yielding, and post-yield response. Meanwhile, the transverse reinforcement was modeled using a bilinear constitutive law with strain hardening, as illustrated in Figure 9b, to capture its contribution to shear resistance and confinement.

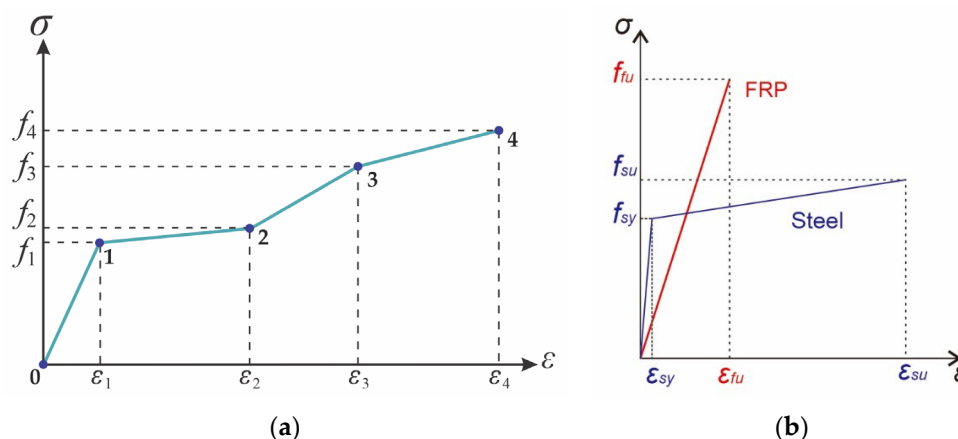


Figure 9. Constitutive models adopted for reinforcement and CFRP materials: (a) multi-linear stress–strain relationship for longitudinal steel reinforcement; (b) bilinear stress–strain relationships for transverse steel reinforcement and CFRP ropes.

Within the finite element framework, the longitudinal reinforcement was modeled using the discrete reinforcement formulation *CCReinforcement*. This approach explicitly represents each steel bar as a truss element embedded within the concrete continuum and assumes uniaxial stress conditions. Consistent stress–strain definitions were applied to maintain numerical compatibility and ensure reliable interaction between the reinforcement and the surrounding concrete matrix. Conversely, the transverse reinforcement was simulated using the smeared reinforcement model *CCSmearedReinf*. The mechanical properties of the steel reinforcement were determined from experimental tensile tests, and the resulting yield and ultimate strengths are reported in Table 3.

The CFRP ropes were explicitly modeled to represent the strengthening configuration adopted in the experimental program. The cross-sectional area of the CFRP ropes was adjusted to match the effective area used in the experimental specimens, with a value of 9.75 mm² assigned to each rope to ensure consistency between the numerical model and the physical strengthening system. The CFRP ropes were modeled using the *CCReinforcement* element type, which represents reinforcement as discrete, line-based elements embedded within the surrounding concrete continuum. The mechanical behavior of the CFRP ropes was defined using a linear elastic constitutive relationship up to failure, reflect-

ing the brittle tensile response characteristic of carbon fiber-reinforced polymer materials (Figure 9b). An elastic modulus of 230 GPa and an ultimate tensile strength of 2000 MPa were assigned based on the manufacturer's technical datasheet (Table 4), as direct experimental testing of CFRP ropes was not conducted in this study, and these values are consistent with those commonly reported in the literature. Failure of the CFRP ropes was governed by a maximum tensile stress criterion, beyond which the reinforcement was assumed to lose its load-carrying capacity.

4.2.2. Concrete

Concrete exhibits a pronounced nonlinear and brittle response under mechanical loading, which necessitates the use of advanced constitutive models in finite element (FE) simulations. Damage–plasticity-based formulations are widely adopted for this purpose, as they allow simultaneous representation of compressive crushing and tensile cracking by integrating plasticity theory with continuum damage mechanics. In this framework, compressive degradation is governed by the evolution of inelastic strains, while tensile cracking is simulated using a smeared crack concept, whereby element stiffness is gradually reduced as cracking progresses. Smeared crack models distribute cracking effects over a finite region by modifying element stiffness within the continuum, allowing cracks to evolve naturally in any direction and reducing mesh dependency. The orientation of cracking can be governed by assumptions, either a rotating or fixed crack approach.

The compressive constitutive response adopted in this study follows the CEB-FIP Model Code [65], which provides empirically calibrated formulations suitable for normal- and high-strength concrete. Accordingly, Figure 10a illustrates the uniaxial stress–strain relationship assumed in the numerical model, rather than experimental results obtained in this study. The post-peak response was represented using a linear softening branch to capture strain localization and energy dissipation associated with quasi-brittle compressive failure [66]. The uniaxial curve is divided into four characteristic phases corresponding to elastic behavior, nonlinear hardening, peak strength, and post-peak softening, which together govern damage initiation, propagation, and stiffness degradation in the FE analysis.

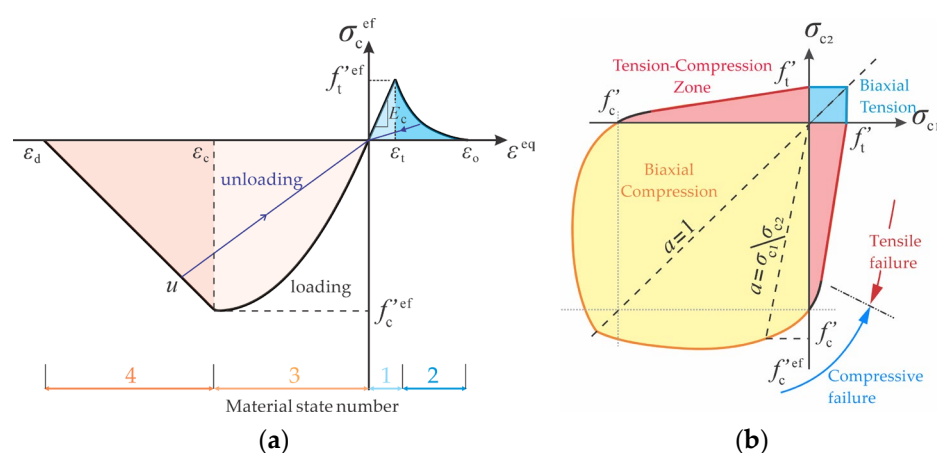


Figure 10. Constitutive relationships adopted for concrete in the numerical model, redrawn to document the material assumptions used in ATENA rather than to present experimental results: (a) uniaxial stress–strain response based on the CEB-FIP Model Code [65]; (b) biaxial strength envelope following the formulation proposed by Kupfer et al. [67].

The interaction between tensile and compressive stresses under biaxial loading was modeled using the experimental failure envelope proposed by Kupfer et al. [67], as illustrated in Figure 10b. This biaxial representation defines the strength domain adopted in the numerical model and is included to document the constitutive assumptions used in

ATENA, rather than to present new test data. The nonlinear response is expressed in terms of effective stress as σ_c^{ef} and equivalent strain ε^{eq} , where ε^{eq} is derived from the corresponding uniaxial stress σ_{ci} scaled by the elastic modulus E_{ci} in the principal direction, as defined in Equation (1). This formulation is inherently path-dependent and evolves with the loading history relative to the unloading reference point, allowing the model to capture stiffness degradation and damage accumulation under multiaxial stress states.

$$\varepsilon^{eq} = \frac{\sigma_{ci}}{E_{ci}} \quad (1)$$

A variable shear retention factor was incorporated into the model to simulate the progressive reduction in shear capacity [68]. This factor represents the reduction in shear stiffness with increasing crack width and modifies the initial shear modulus G_c to obtain a reduced shear modulus G , as expressed in Equations (2) and (3), where G_c is calculated from the elastic modulus E_c and Poisson's ratio ν .

$$G = r_g G_c \quad (2)$$

$$G_c = \frac{E_c}{2(1 + \nu)} \quad (3)$$

The shear stress along the crack interface, τ_{uv} , was computed using Equation (4), with γ denoting the applied shear strain. Fracture energy, G_F , which characterizes the energy dissipated during crack propagation, was defined following the formulation proposed by Vos [69] in Equation (5). Incorporation of crack propagation using the fracture energy model ensures a reliable advancement in tensile softening through the concrete matrix.

$$\tau_{uv} = G\gamma \quad (4)$$

$$G_F = 25f_t \cdot (\text{N/m}) \quad (5)$$

In the present study, both normal concrete and geopolymer concrete were simulated using the *CC3DnonLinCementitious2* material model implemented in ATENA, which is suitable for capturing nonlinear behavior, including cracking, crushing, and stiffness degradation. The compressive strength inputs were directly obtained from experimental testing at the structural test age, ensuring consistency between the experimental program and the numerical simulation. The remaining material parameters required by the constitutive model, including elastic modulus, tensile strength, fracture energy, Poisson's ratio, and critical compressive displacement, were determined using established empirical correlations recommended in ATENA documentation [70] and widely adopted in the literature. These correlations express elastic modulus, tensile strength, and fracture energy as functions of the experimentally measured compressive strength, allowing both materials to be represented in a mechanically consistent manner despite the limited number of direct material tests. This modeling strategy ensures that the numerical response of the geopolymer overlay, as well as that of normal concrete, is primarily governed by experimentally measured compressive strength, while maintaining compatibility with validated constitutive formulations commonly used for reinforced concrete simulation. The adopted material properties are summarized in Table 7.

For confined concrete regions incorporating transverse reinforcement, the *CCCombinedMaterial* formulation was employed. In this approach, the *CC3DNonLinCementitious2* concrete constitutive model was integrated with the *CCSmearedReinf* representation of stirrups to form a unified material description. This combined formulation allows the interaction between concrete cracking and smeared transverse reinforcement to be modeled

within a single continuum framework. As a result, the effects of shear retention and post-cracking stiffness contribution from the transverse reinforcement are captured in an averaged manner, providing a realistic representation of the composite behavior after cracking.

Table 7. Input properties for concrete and geopolymer used in the FE model.

Property	Formula	Value
General Properties		
Poissons ratio [70], ν	N/A	0.2
Shear retention factor	N/A	Variable
Critical compressive displacement [70], W_d (m)	N/A	−0.0005
Normal concrete		
Compressive strength, f'_c (MPa)	Test result	28
Elastic modulus [70], E_c (MPa)	$3320\sqrt{f'_c} + 6900$	24,457.78
Tensile strength [70], f'_t (MPa)	$0.5\sqrt{f'_c}$	2.64
Specific fracture energy [70], G_F (N/m)	$25f'_t$	66.10
Geopolymer		
Compressive strength, f'_c (MPa)	Test result	26.97
Elastic modulus [70], E_c (MPa)	$3320\sqrt{f'_c} + 6900$	24,141.63
Tensile strength [70], f'_t (MPa)	$0.5\sqrt{f'_c}$	2.59
Specific fracture energy [70], G_F (N/m)	$25f'_t$	64.90

5. Model Validation

Figure 11 and Table 8 present a comparison between the experimental results and the numerical predictions. The finite element analysis (FEA) demonstrated reliable accuracy in reproducing the load–displacement responses of the tested beams across the different loading stages. The ratios between the numerically predicted ultimate loads ($P_{\max, \text{num}}$) and the experimentally measured values ($P_{\max}$) were 0.90 and 0.91 for specimens BN and BP, respectively. Similarly, the ratios between the predicted maximum displacements ($\delta_{P_{\max, \text{num}}}$) and the experimental results ($\delta_{P_{\max}}$) indicated an overestimation for BN, with a ratio of 1.25, and a slight underestimation for BP, with a ratio of 0.98.

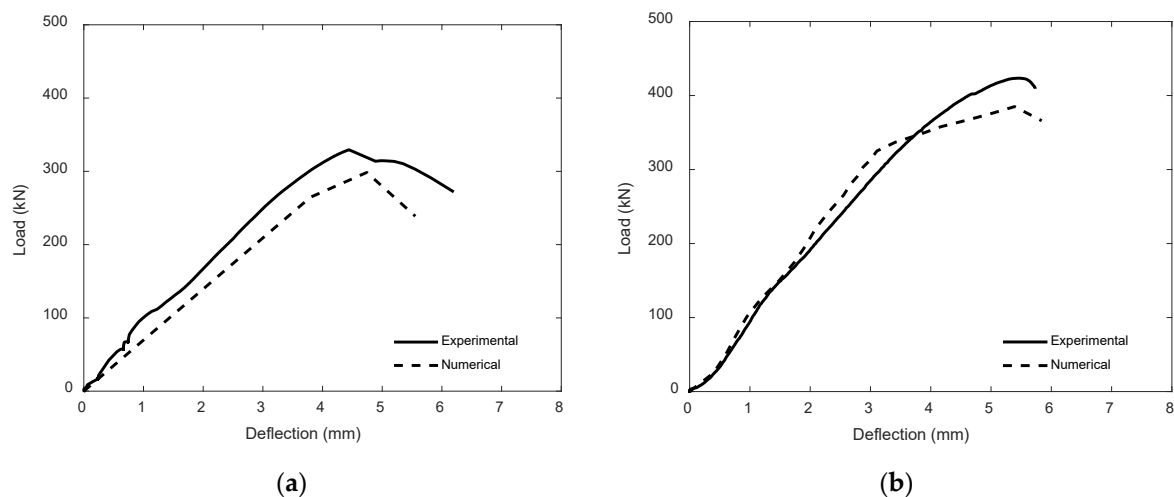


Figure 11. Comparison of experimental and numerical load–deflection responses at mid-span: (a) control beam (BN), (b) strengthened beam (BP).

Table 8. Comparison of load–deflection response parameters from experimental and numerical.

Specimen	Ultimate Load (kN)		$P_{\max,\text{num}}/P_{\max}$	Deflection at Peak Load (mm)		$\delta_{P_{\max,\text{num}}}/\delta_{P_{\max}}$	NMSE
	Num. ($P_{\max,\text{num}}$)	Exp. ($P_{\max}$)		Num. ($\delta_{P_{\max,\text{num}}}$)	Exp. ($\delta_{P_{\max}}$)		
BN	298.51	329.49	0.90	5.55	4.44	1.25	0.422
BP	385.00	423.44	0.91	5.39	5.45	0.98	0.009

The agreement between the numerical and experimental load–deflection responses was further quantified using the Normalized Mean Square Error (NMSE), calculated following Poli and Cirillo [71]. The NMSE was evaluated by sampling load values at uniform 2 mm intervals of mid-span deflection, enabling a point-by-point comparison over the entire loading history. The resulting NMSE values were 0.422 for the control beam and 0.009 for the strengthened beam, indicating a substantially improved predictive accuracy for the strengthened configuration. The relatively low NMSE values confirm that the developed FEA model is capable of reliably capturing the load–displacement behavior of RC beams strengthened in shear using NSM-CFRP ropes in combination with steel-reinforced geopolymer in the compression zone. Consequently, the validated numerical model can be regarded as a robust predictive tool to support future design development and experimental investigations related to RC beam strengthening.

Figure 12 provides a qualitative comparison of the crack patterns observed in the two beam specimens, highlighting the influence of CFRP rope strengthening combined with steel-reinforced geopolymer. It should be noted that, due to the adoption of a smeared-crack formulation, the numerical results are intended to represent the overall cracking behavior and dominant damage mechanisms rather than the exact geometry of individual cracks. For the control beam (BN), the numerical model indicates a more distributed cracking pattern, with pronounced cracking concentrated around the midspan region, which is characteristic of a flexure-dominated and relatively ductile response. In contrast, the strengthened specimen (BP) exhibits more localized cracking near the midspan, reflecting increased stiffness and a reduced deformation capacity.

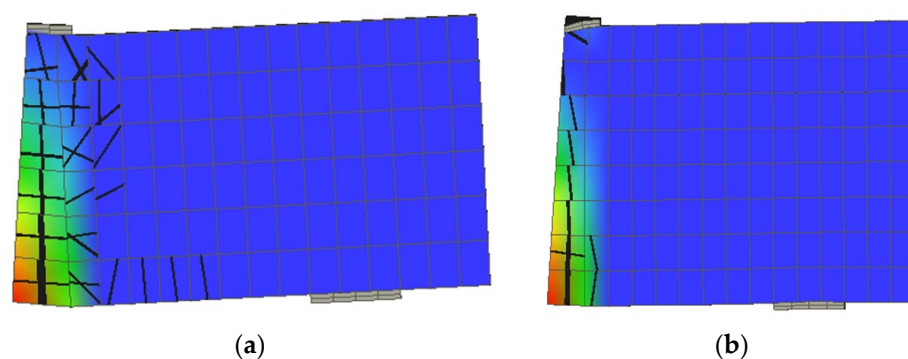


Figure 12. Numerical crack patterns of the beam specimens: (a) control beam (BN); (b) strengthened beam (BP). The color contour represents the simulated damage level (red indicates more severe damage, while blue indicates little to no damage). The black lines indicate the predicted crack paths.

6. Conclusions

In this study, the effectiveness of an integrated shear–flexural strengthening system for reinforced concrete (RC) beams, combining near-surface mounted CFRP ropes for shear enhancement and steel-reinforced geopolymer overlays in the compression zone, was eval-

uated through experimental testing and validated numerical modeling. Based on the results obtained, the following conclusions are drawn:

- The proposed integrated strengthening system significantly enhanced the structural performance of RC beams, demonstrating effective composite action between NSM-CFRP ropes and the geopolymer overlay.
- The strengthened beam achieved a 28.6% increase in ultimate load capacity and a 13.6% improvement in stiffness compared to the unstrengthened specimen, confirming the efficiency of the dual shear–flexural strengthening mechanism.
- Energy absorption capacity increased by 7.65%, indicating improved toughness and greater resistance to damage accumulation, which is beneficial for RC members subjected to seismic or other dynamic loading.
- Although the strengthened beam exhibited a reduced ductility index, failure occurred without abrupt strength loss, reflecting a stable response governed by increased stiffness rather than premature brittle behavior.
- The developed three-dimensional nonlinear FE model reproduced the experimental load–deflection response of the two tested specimens, yielding predicted-to-experimental ultimate load ratios of 0.90 for BN and 0.91 for BP, with low NMSE for the strengthened specimen and higher deviation observed for the control specimen.

The findings should be interpreted within the scope of this study. The experimental program was limited to two monotonic three-point bending tests, comprising one unstrengthened control beam and one strengthened beam. Although clear improvements in strength, stiffness, and energy absorption were observed, the limited specimen matrix restricts direct generalization. Cracking behavior was documented through visual observation and surface marking; quantitative crack width measurements and direct instrumentation of geopolymer–concrete interface slip were not performed and were therefore assessed qualitatively.

In the numerical analysis, compressive strengths were obtained from experimental testing, whereas several additional material parameters were derived from validated empirical correlations. While this approach is commonly adopted, it may influence prediction accuracy, particularly for the control specimen. Moreover, the experimental investigation was conducted under monotonic loading, and the performance of the proposed strengthening system under cyclic or seismic loading was not examined. Further studies incorporating expanded test matrices, enhanced instrumentation, and cyclic loading are recommended to generalize the findings.

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Abbreviations

The following abbreviations are used in this manuscript:

RC	Reinforced Concrete
NSM	Near-Surface Mounted
CFRP	Carbon Fiber-Reinforced Polymer
FRP	Fiber-Reinforced Polymer
ATENA	Advanced Tool for Engineering Nonlinear Analysis
NSM-CFRP	Near-Surface Mounted Carbon Fiber-Reinforced Polymer
AA	Alkaline Activator
FA	Fly Ash
NSC	Normal-Strength Concrete
LVDT	Linear Variable Differential Transformer
FE	Finite Element
FEA	Finite Element Analysis
NMSE	Normalized Mean Squared Error

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