

## Article

# Axial Behaviour of Reinforced Concrete Columns Strengthened with Self-Compacting Geopolymer Concrete Jacketing and External FRP Confinement

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## Abstract

The structural performance of reinforced concrete (RC) columns can be substantially improved by using advanced confinement systems and sustainable cementitious materials. This study presents an integrated experimental, numerical, and analytical investigation of RC columns strengthened with self-compacting geopolymer concrete (SCGC) and carbon fiber-reinforced polymer (CFRP) under axial compression. Twelve column specimens were tested across four groups: conventional RC columns, unconfined SCGC columns, SCGC-jacketed columns (CONF. SCGC), and CFRP-wrapped columns (CONF. FRP), in three cross-sectional geometries: square (177 × 177 mm), rectangular (265 × 177 mm), and circular (Ø200 mm). Experimental results demonstrated that replacing conventional concrete with SCGC improved deformation capacity, increasing the ultimate axial displacement from 2.07 mm in the reference square RC column (RC C1) to 3.21 mm in the corresponding square SCGC specimen (SCGC C1). On a normalized stress basis, the unconfined SCGC specimens achieved ultimate axial stress values of 56–64 MPa compared to 41–49 MPa for the RC reference columns of the same geometry, representing material-level strength gains of 1.30–1.49×. The application of external confinement further enhanced column behaviour. The CFRP-wrapped specimens achieved the highest material-level strength efficiency, with normalized ultimate axial stress values of 98–108 MPa for the square and rectangular geometries, representing gains of 2.01–2.41× over the corresponding unconfined RC columns of identical cross-section. The SCGC-jacketed specimens achieved the highest absolute load capacities, with CONF. SCGC C2 reaching 8270 kN and a maximum stiffness of 5531 kN/mm and energy absorption of 19,740 kN·mm. However, on a normalized stress basis, the SCGC-jacketed square and rectangular specimens achieved 45–47 MPa, comparable to the RC reference columns, confirming that their absolute load gains are primarily attributable to section enlargement rather than intrinsic material strength enhancement. The circular SCGC-jacketed specimen achieved a normalized stress of 43 MPa, consistent with the same trend. A three-dimensional nonlinear finite element model developed in ABAQUS using the Concrete Damaged Plasticity model and cohesive zone interactions showed close agreement with experimental results, with mean prediction ratios of 1.03 for ultimate load and 0.96 for displacement. An analytical model provided conservative estimates of axial capacity. The findings demonstrate that CFRP wrapping offers superior material-level confinement efficiency, while SCGC jacketing provides the highest absolute load capacity through combined section enlargement and

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passive confinement, representing a potentially more environmentally friendly strengthening strategy for existing RC columns.

**Keywords:** RC columns; geopolymer concrete jacketing; CFRP wrapping; confinement; axial capacity; finite element analysis; sustainable strengthening

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## 1. Introduction

Reinforced concrete (RC) columns are critical primary load-resisting elements that control the strength, stiffness, and global stability of structural systems. Their performance under axial loading, particularly in terms of load-carrying capacity, ductility, and failure modes, is essential for ensuring structural safety and serviceability. Consequently, improving the behaviour of RC columns through advanced materials, innovative strengthening techniques, and optimised design strategies has remained a focal point of research in structural engineering [1,2]. Traditional approaches to column strengthening, including steel jacketing and section enlargement, have demonstrated effectiveness but are often associated with drawbacks such as increased self-weight, corrosion susceptibility, and construction complexity [2,3]. These limitations have driven the development and adoption of alternative confinement systems with enhanced efficiency and durability.

Among modern strengthening solutions, fiber-reinforced polymer (FRP) composites have gained widespread recognition for their high strength-to-weight ratio, excellent corrosion resistance, and ease of installation. In particular, carbon fiber-reinforced polymer (CFRP) has proven to be one of the most effective confinement materials for RC columns [4–6]. The confinement mechanism of CFRP relies on the passive lateral pressure exerted on the concrete core, which significantly improves compressive strength, ductility, and energy absorption capacity [5,6]. Wang et al. [5] demonstrated that partial CFRP confinement significantly enhanced the axial strength of RC columns, with the level of improvement governed by the CFRP strip spacing, number of CFRP layers, and wrapping arrangement. Wang et al. [6] reported that increasing the number of CFRP layers improved the confinement effectiveness of square RC columns, leading to higher axial strength and substantially greater axial strain capacity, although the increase in axial strength remained relatively limited. Ashteyat and Obaidat [7] demonstrated that CFRP rope rehabilitation effectively restored and, in some cases, enhanced the axial load capacity of heat-damaged RC slender columns beyond the levels of unheated control specimens. Extensive experimental and analytical studies have demonstrated that FRP-confinement alters the stress–strain response of concrete, leading to more ductile, strain-hardening behaviour than unconfined concrete [7–10]. Foundational design-oriented stress–strain models [9] remain the standard for predicting this behaviour, while recent data-driven frameworks and experimental campaigns continue to optimize FRP performance across various column configurations and damage states [4,11]. Parallel research has also explored the use of advanced cementitious composites, such as Strain-Hardening Cementitious Composites (SHCC) and High-Performance Fiber-Reinforced Concrete (HPFRC), demonstrating synergistic improvements in load capacity and ductility for RC columns [10,12].

Numerous experimental investigations have validated the effectiveness of CFRP systems in enhancing the axial performance of RC columns under various loading conditions, including monotonic, cyclic, and impact loading [1,7,13]. Furthermore, recent studies have explored hybrid strengthening techniques combining FRP with other advanced materials such as strain-hardening cementitious composites and high-performance fiber-reinforced concrete, demonstrating further improvements in load capacity and ductility [10,12]. The influence of confinement configuration, including full wrapping, partial wrapping, and

hybrid jacketing systems, has also been investigated, highlighting the importance of confinement effectiveness in determining overall column behaviour [3,4]. Simultaneously, the demand for sustainable construction materials has surged to mitigate the environmental footprint of ordinary Portland cement (OPC). Geopolymer concrete has emerged as a promising alternative to ordinary Portland cement-based concrete due to its significantly lower carbon footprint and comparable or superior mechanical properties [14,15]. Singh et al. [15] reported that geopolymer concrete can achieve compressive strengths of up to 80 MPa while providing an environmentally sustainable alternative to ordinary Portland cement by using alkali-activated aluminosilicate materials. Faridmehr et al. [16] demonstrated that self-compacting geopolymer concrete achieved 28-day compressive strengths ranging from approximately 47 to 70 MPa, while mixtures containing 40–60% fly ash attained slump flow values of 650–720 mm, confirming their suitability for densely reinforced structural elements. In structural applications, Fathi et al. [17] showed that reinforced SCGC beams exhibited shear behavior comparable to that of conventional reinforced concrete beams. Geopolymer binders are produced from aluminosilicate-rich industrial by-products, such as fly ash (FA) and ground granulated blast furnace slag, and are activated with alkaline solutions [18,19]. This process not only reduces greenhouse gas emissions but also promotes the utilization of industrial waste materials, contributing to potentially more sustainable development in the construction sector, subject to full life-cycle assessment accounting for total material consumption [15,20].

Extensive research has been conducted to evaluate the mechanical, durability, and microstructural properties of geopolymer concrete. Studies have reported favorable compressive strength, improved resistance to chemical attack, and enhanced long-term durability compared to conventional concrete [19,21]. In structural applications, geopolymer concrete has demonstrated promising performance in beams, columns, and other load-bearing elements [17,22]. The structural utility of geopolymers is further augmented by self-compacting properties, creating Self-Compacting Geopolymer Concrete (SCGC). The incorporation of self-compacting characteristics further enhances its applicability by improving workability, reducing segregation, and ensuring uniform material distribution, particularly in densely reinforced sections. Recent comprehensive reviews and experimental studies highlight SCGC's excellent fresh properties, homogeneous matrix distribution, and robust hardened mechanical characteristics without the need for mechanical vibration [16,23,24]. Furthermore, the sustainability and life cycle performance of these green composites make them ideal candidates for next-generation structural applications [25,26].

Despite the substantial body of research on both FRP-confined concrete and geopolymer concrete, studies that integrate these two approaches in structural retrofitting applications remain limited. In particular, there is a lack of comprehensive investigations that systematically compare the axial behaviour of conventional reinforced concrete (RC) columns and geopolymer concrete columns under identical confinement conditions. Important parameters such as cross-sectional geometry, material composition, and confinement configuration have not yet been fully clarified, even though previous studies have shown that column geometry, whether circular, square, or rectangular, significantly influences confinement efficiency because of differences in stress distribution and lateral pressure development [11,27]. However, these effects have rarely been examined within a unified experimental framework that simultaneously considers both conventional and sustainable concrete systems.

Furthermore, the successful use of advanced concrete jackets for section enlargement depends heavily on the bond integrity at the interface between the existing ordinary Portland cement (OPC) concrete core and the new jacket material. Recent studies on the interfacial shear and bond performance between OPC concrete and geopolymer-based composites have highlighted the complexity of this interaction [28–30]. Although several surface

preparation methods and engineered transition layers have been proposed to improve adhesion [14,31,32], the application of self-compacting geopolymer concrete (SCGC) as a passive confinement jacket for conventional RC columns has not yet been sufficiently validated through experimental research [22]. In parallel, finite element modeling has been widely employed to simulate the nonlinear response of confined concrete, including cracking, material degradation, and interfacial behaviour [4,33]. Nevertheless, additional experimental validation is required to improve the reliability of these numerical models and to support the development of robust design methodologies for emerging sustainable confinement systems.

Confinement efficiency is strongly influenced by cross-sectional geometry [34]. Circular columns generally exhibit more effective confinement because the lateral pressure generated by the jacket is distributed uniformly around the perimeter, whereas square and rectangular columns experience stress concentrations at the corners that reduce confinement effectiveness [6,35]. Although the influence of geometry has been investigated for conventional concrete and steel fiber-reinforced members [27], these geometric effects have not been comprehensively evaluated for self-compacting geopolymer concrete (SCGC) jacketing in direct comparison with external CFRP wrapping.

To address these research gaps, the present study provides a comprehensive experimental and numerical investigation of the axial compressive behaviour of concrete columns with different material systems, cross-sectional geometries, and confinement techniques. The experimental program includes both conventional reinforced concrete (RC) columns and SCGC columns, with strengthening achieved through SCGC section enlargement and external CFRP wrapping. Three cross-sectional shapes, square, rectangular, and circular, are examined to quantify the effect of geometry on structural performance under axial loading. In addition, a nonlinear finite element model incorporating surface-based cohesive interactions is developed to simulate material nonlinearity and interface behaviour between the original core and the confinement jacket. An analytical model based on confinement theory is also employed to estimate axial load capacity and to compare theoretical predictions with experimental observations.

By systematically comparing the performance of traditional and sustainable confinement systems, this study aims to establish a clearer understanding of the relative efficiency of CFRP and SCGC strengthening methods. The findings are expected to contribute to the development of efficient, durable, and environmentally sustainable retrofitting solutions, while also providing a foundation for future design guidelines for confined conventional and geopolymer concrete columns subjected to axial compression.

## 2. Experimental Program

### 2.1. Specimen Design and Grouping

A total of twelve column specimens were designed to examine the influence of material type, cross-sectional geometry, and confinement configuration on axial performance. The specimens were divided into four main groups: RC, SCGC, confined SCGC columns (CONF–SCGC), and CFRP-confined columns (CONF–FRP), as shown in Figure 1. The experimental configurations were selected to simulate the practical scenario of strengthening existing conventional RC columns, which represents the most common rehabilitation case in structural engineering practice. The OPC reinforced concrete core represents the existing in-service column, while the SCGC jacket and CFRP wrap represent two candidate strengthening systems applied to it. The inclusion of Group G2 (unconfined SCGC columns) was an integral component of the experimental design, directly linked to the primary study objective of decoupling the influence of material type from the effect of confinement. Without this group, any performance gain observed in the SCGC-jacketed specimens (Group G3) relative to the RC reference (Group G1) would reflect the combined

effect of material substitution and section enlargement simultaneously, with no means of isolating either contribution independently. Group G2 shares the same geopolymers material as Group G3 but without any confinement, and shares the same geometry and loading conditions as Group G1 but with a different concrete material. This two-reference experimental structure enables the study to quantify: (1) the material-level contribution of SCGC relative to OPC concrete (G2 vs. G1) and (2) the additional confinement contribution of the SCGC jacket beyond the material effect alone (G3 vs. G2). This approach is consistent with standard practice in experimental structural engineering for isolating the effects of individual variables.

Three cross-sectional geometries were considered for one-meter-height columns: square (C1) with a 177 mm side length, rectangular (C2) with 265 × 177 mm<sup>2</sup> cross-section dimensions, and circular (C3) with a 200 mm diameter. All specimens were designed with similar reinforcement ratios to ensure a consistent basis for comparison. Longitudinal reinforcement consisted of deformed steel bars Φ12 mm, while transverse reinforcement was provided using closed stirrups at specified spacing (5Φ8 mm/m'). The confined specimens included either increased cross-sectional dimensions (CONF-SCGC) or external wrapping using carbon fiber-reinforced polymer (CONF-FRP).

To ensure a consistent basis for comparison among specimens with different cross-sectional geometries, the longitudinal reinforcement was proportioned to achieve approximately equivalent reinforcement ratios across the core sections. The longitudinal reinforcement ratio ( $\rho_l$ ) was calculated according to Equation (1):

$$\rho_l = A_s/A_g \quad (1)$$

where  $A_s$  is the total area of longitudinal reinforcement and  $A_g$  is the gross cross-sectional area of the column core.

The selected reinforcement arrangements were designed such that the resulting longitudinal reinforcement ratios remained within a narrow range (approximately 1.44% to 1.45%) for all specimens, thereby minimizing the influence of reinforcement content on the measured structural response. This approach allowed the effects of cross-sectional geometry (square, rectangular, and circular) and strengthening configuration to be evaluated independently of significant variations in longitudinal steel ratio.

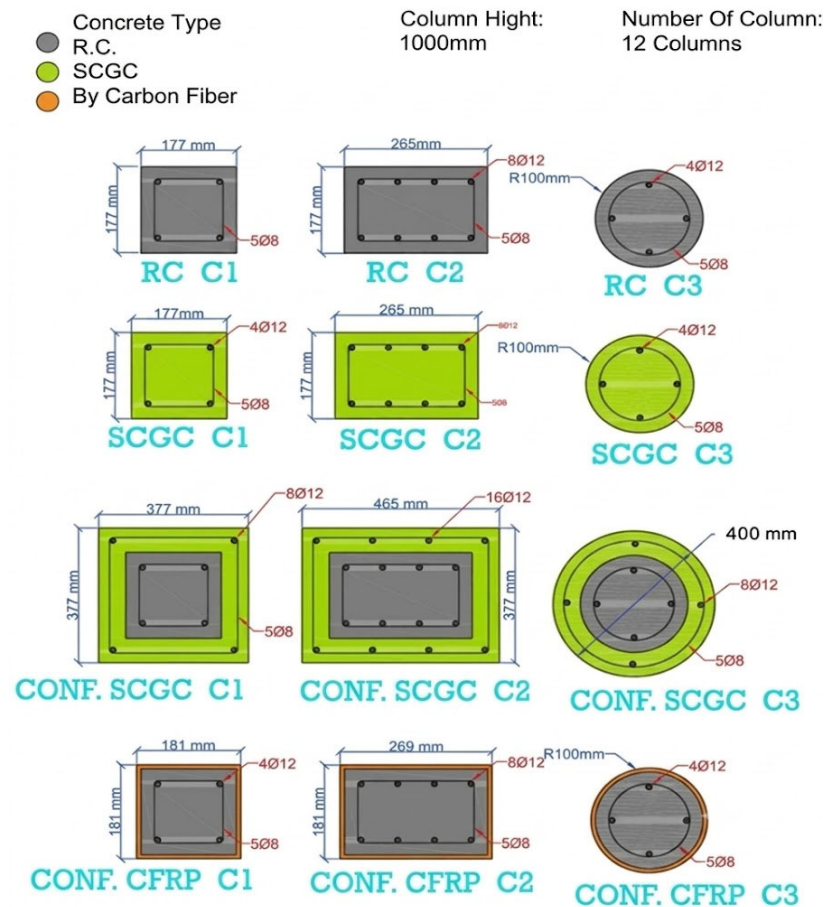
Table 1 summarizes the geometric properties, longitudinal reinforcement details, steel area ( $A_s$ ), gross sectional area ( $A_g$ ), and the corresponding reinforcement ratio ( $\rho_l$ ) for all tested specimens. The small differences in reinforcement ratio among specimens are attributed to practical constraints associated with bar arrangement and detailing requirements; however, these variations remained sufficiently limited to ensure meaningful comparisons between the tested columns.

**Table 1.** Geometric and longitudinal reinforcement details of the tested specimens.

| Specimen Group | Geometry    | Dimensions (mm) | Longitudinal Reinforcement | $A_s$ (mm <sup>2</sup> ) | $A_g$ (mm <sup>2</sup> ) | $\rho_l$ (%) |
|----------------|-------------|-----------------|----------------------------|--------------------------|--------------------------|--------------|
| Series C1      | Square      | 177 × 177       | 4Φ12                       | 452.4                    | 31,329                   | 1.44         |
| Series C2      | Rectangular | 265 × 177       | 6Φ12                       | 678.6                    | 46,905                   | 1.45         |
| Series C3      | Circular    | 200 (Dia.)      | 4Φ12                       | 452.4                    | 31,416                   | 1.44         |

Note: For strengthened specimens in Groups G3 and G4, the internal reinforcement details of the respective core geometries (C1, C2, or C3) were maintained to ensure consistency.

This study did not include repeated specimens for each test configuration. Therefore, the variation in the results cannot be measured statistically. For this reason, the effects of material type, geometry, and confinement method should be considered as general trends, not statistically confirmed conclusions. This limitation should be addressed in future research.



**Figure 1.** Cross-sectional geometries and reinforcement details of all tested column specimens.

## 2.2. Material Properties

The materials used in this study included conventional Portland cement concrete (PCC), self-compacting geopolymer concrete (SCGC), reinforcing steel, and CFRP. The PCC mix was designed to represent a standard structural concrete with a target compressive strength consistent with typical existing RC columns. The SCGC mix was adopted from the optimized composition reported in [21], which established the selected proportions as yielding adequate workability, compressive strength, and durability. Silica fume was incorporated into the SCGC mix to enhance matrix densification by filling microvoids, improve early-age strength, and reduce permeability [16,19,21]. The alkaline activator ratio (SS:SH = 2:1) and the total binder content of 900 kg/m<sup>3</sup> were adopted from the validated mix design reported in [21].

The physical, mechanical, and durability characteristics of these materials govern the behavior of the strengthened reinforced concrete columns under axial loading. In particular, the performance of the SCGC strengthening layer depends on the properties of its constituent materials and on its compatibility with the existing concrete substrate and external confinement system. The silica fume used in the SCGC mixture was an ultra-fine pozzolanic material composed of spherical particles with a typical mean diameter of approximately 0.15 µm, which enhanced matrix densification by filling micro voids within the binder structure. The fly ash was a Class F aluminosilicate by-product with low calcium content that contributed to long-term strength development through delayed pozzolanic reactions. Ground granulated blast furnace slag (GGBFS) exhibited latent hydraulic activity and improved early-age strength due to its high vitreous content and fine

particle size. Together, these supplementary cementitious materials formed a dense geopolymeric matrix with reduced permeability and improved durability compared with conventional cementitious systems. The reference SCGC specimens were cast using a geopolymer binder (SF, GGBFS, and FA ), fine glass sand, and coarse quartz aggregate. The detailed mixture proportions are presented in Table 2. The OPC mix was designed for standard structural concrete, while the SCGC mix was proportioned with an alkaline activator ratio of SS:SH = 2:1 and a total binder content of 900 kg/m<sup>3</sup>, as validated in [21].

**Table 2.** Mix Proportions (kg/m<sup>3</sup>).

| Constituent           | PCC  | SCGC |
|-----------------------|------|------|
| Cement (OPC)          | 450  | —    |
| Silica Fume (SF)      | —    | 180  |
| Fly Ash (FA)          | —    | 360  |
| GGBFS                 | —    | 360  |
| Fine Aggregate        | 600  | 615  |
| Coarse Aggregate      | 1120 | 410  |
| Water                 | 180  | 92.8 |
| Sodium Hydroxide (SH) | —    | 120  |
| Sodium Silicate (SS)  | —    | 240  |
| Superplasticizer      | 7    | 18   |

The CFRP sheets used for external confinement were SikaWrap-230 C, a unidirectional high-strength carbon fiber material. The sheets had an areal density of approximately 230 g/m<sup>2</sup> and a nominal thickness of 0.12 mm. According to the manufacturer’s technical properties, the carbon fibers possessed a tensile strength of approximately 4300 MPa, an elastic modulus of 238 GPa, and an ultimate elongation of 1.8%, making them suitable for confinement applications requiring high tensile resistance. The CFRP was externally wrapped around the strengthened columns to provide lateral confinement and improve axial load capacity. A two-component epoxy resin was used to bond the CFRP sheets to the concrete surface and ensure effective stress transfer between the concrete substrate and the composite wrap. The reinforcing steel used for the longitudinal bars and transverse ties provided the internal tensile resistance required for composite action within the columns. Therefore, the combined interaction among the concrete core, the SCGC strengthening layer, the reinforcing steel, and the CFRP confinement system determined the overall effectiveness and long-term structural performance of the proposed strengthening technique.

The 28-day compressive strength of (PC) was measured as 45 MPa, whereas the SCGC mixture exhibited a higher compressive strength of 102 MPa. The reinforcing steel used for longitudinal bars and stirrups showed yield strengths of 487.8 MPa and 324 MPa, respectively. These measured properties were considered in evaluating the influence of concrete type on the axial performance of the tested column specimens.

### 2.3. Confinement and Configuration Details

SCGC specimens were fully cast using geopolymer concrete without additional confinement. The CONF–SCGC (Section Enlargement) specimens were used to evaluate the influence of material replacement on axial behavior. In these specimens, confinement was achieved by increasing the cross-sectional area, providing additional lateral restraint and load-carrying capacity. The confinement layer thickness was maintained at approximately 100 mm, resulting in larger overall dimensions than those of the reference specimens. The jacket thickness of 100 mm was selected on the basis of two considerations: (1) practical constructability, ensuring sufficient space to accommodate the additional longitudinal

reinforcement and transverse ties within the jacket layer, and (2) consistency with jacket thicknesses reported in comparable RC column strengthening studies in the literature [2,3,22]. For the circular specimen C3, this thickness increases the section radius from 100 mm to 200 mm, while for the square and rectangular specimens C1 and C2, it adds 10 mm on each side.

The CONF. SCGC specimens were constructed by jacketing fully cured OPC reinforced concrete cores. The RC core specimens were first cast and cured for 28 days. Prior to jacketing, the core surfaces were mechanically roughened by wire brushing to enhance interfacial bond. The SCGC jacket was subsequently cast around the cured core, replicating a realistic structural retrofit scenario. This two-stage construction procedure provides the physical justification for the cohesive zone model adopted at the OPC–SCGC interface in the finite element simulations. In CONF–FRP (CFRP Confinement) specimens, CFRP confinement was applied externally to the column surface using carbon fiber sheets. The wrapping was designed to provide uniform lateral confinement, enhancing strength and ductility. Proper surface preparation and resin application were carried out to ensure adequate bonding between the CFRP and the concrete surface.

#### 2.4. Test Setup and Instrumentation

All specimens were tested under axial compression using a universal testing machine (Figure 2). The columns were positioned vertically, and the load was applied concentrically to minimize eccentricity. The loading rate was maintained at 0.5 mm/min throughout all displacement-controlled tests. Axial displacements were measured using Linear Variable Differential Transformers (LVDTs) attached directly to the specimen surface, not machine stroke, ensuring that the reported stiffness values ( $K$ ) reflect the true specimen deformation. The load–displacement curves are presented up to the point of ultimate failure, defined as a significant drop in load-carrying capacity or physical fracture of the specimen. The energy absorption capacity ( $E$ ) was calculated as the area under the experimental load–displacement curve from zero displacement to the ultimate displacement ( $\Delta u$ ), computed using the trapezoidal numerical integration method. Data acquisition systems continuously recorded load and deformation throughout the test. The specimens were subjected to monotonic axial compression under displacement-controlled loading. The loading rate was selected to ensure stable crack development and to accurately capture post-peak behaviour.

All column specimens were tested under concentric axial compression using a high-capacity universal testing machine. To prevent premature end crushing and minimize localized stress concentrations at the loading interfaces, the top and bottom 100 mm of each specimen were confined using steel collars, resulting in a clear unconfined test length of 800 mm within the central region of each column. The axial load was applied through hardened steel loading platens positioned at the specimen ends. The end collars were introduced solely to ensure uniform load transfer and to prevent localized failure at the loading zones, promoting failure within the central unconfined region. The observed failure modes confirmed that crack initiation, concrete crushing, and reinforcement buckling consistently occurred within the central unconfined zone rather than in the collar-confined end regions. Since identical end-restraint conditions were maintained across all specimens, the boundary conditions remained consistent throughout the experimental program, ensuring the validity of comparative evaluations of axial stiffness, ultimate load capacity, and energy absorption.



**Figure 2.** Test setup.

The principal experimental parameters were determined according to clearly defined criteria to ensure consistency and reproducibility of the reported results. Cracking was identified based on the first noticeable deviation from linearity in the load–displacement response, confirmed by continuous visual inspection throughout the loading history. The cracking load ( $P_{cr}$ ) and corresponding cracking displacement ( $\Delta_{cr}$ ) were recorded at this point. The ultimate load ( $P_u$ ) was taken as the peak axial load attained by the specimen, with the ultimate displacement ( $\Delta_u$ ) defined as the axial deformation at that instant. The initial axial stiffness ( $K$ ) was calculated as the secant stiffness between the origin and the cracking point, expressed as  $K = P_{cr}/\Delta_{cr}$ . The energy absorption capacity ( $E$ ) was evaluated as the area under the experimental load–displacement curve, integrated from zero displacement to the point at which the post-peak load dropped to 85% of the ultimate load. This integration limit was adopted to capture both the load-carrying capacity and the post-peak deformation behavior of each specimen while excluding the unstable final failure stage.

Strain gauges were installed at the mid-height region of the specimens to minimize the influence of local boundary effects near the loading zones. Axial strain gauges were oriented parallel to the loading direction to monitor longitudinal deformation, whereas transverse strain gauges were positioned perpendicular to the loading axis to capture lateral expansion of the concrete section. For the CFRP-confined specimens, additional gauges were attached along the hoop direction of the CFRP jacket to monitor circumferential strains associated with confinement activation and the onset of CFRP rupture.

The strain-gauge measurements were primarily employed to verify the stability of the loading process and to support the interpretation of the observed structural response. However, the main focus of the present study was the global behavior of the tested columns as characterized by load–displacement response, ultimate load capacity, stiffness, energy absorption, and failure mode. Therefore, detailed strain histories are not presented

herein, and this is acknowledged as a limitation of the current study to be addressed in future investigations.

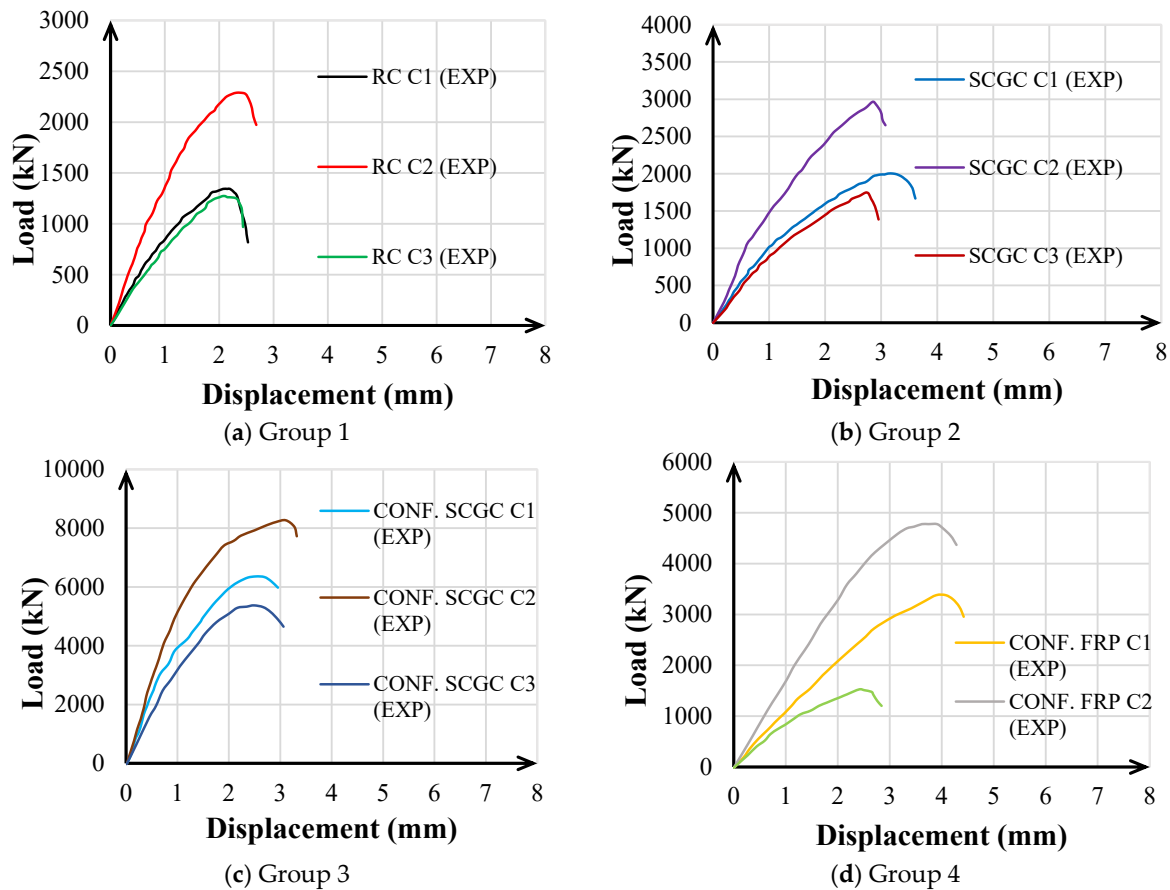
### 3. Experimental Results and Discussion

#### 3.1. Global Structural Response and Load Capacity

The experimental program showed a clear improvement in structural performance across the four specimen groups, with the improvement mainly influenced by the material makeup and the confinement method used. The reference RC specimens (Group G1) demonstrated steady load-carrying behaviour, with the control specimen RC C1 reaching an ultimate capacity ( $P_u$ ) of 1342 kN. When internal reinforcement was increased in RC C2, the ultimate capacity rose to 2288 kN, but the overall response still reflected typical reinforced concrete behaviour, see Figure 3a. The significant increase in capacity observed in specimen RC C2 compared to RC C1 (approximately 70%) suggests that the enhancement was not solely due to reinforcement ratio, but was strongly influenced by cross-sectional geometry and stress distribution. The rectangular configuration provides a larger effective load-bearing area and improved internal force redistribution, contributing to the observed strength gain.

The SCGC specimens (Group G2) exhibited improved deformation characteristics compared to conventional RC columns, as reflected by their higher ultimate displacements. However, the increase in axial strength was not consistent across all specimens. Although the SCGC C2 specimen reached a relatively high ultimate capacity (2965 kN), the SCGC C3 specimen (1746 kN) exhibited a capacity increase of 37% over its reference RC C3 specimen (1272 kN), consistent with the material-level improvement observed across the SCGC group. This indicates that the advantages of SCGC are not solely manifested in peak load capacity, but rather in enhanced ductility, improved cracking behavior, and better material homogeneity due to its self-compacting nature.

A substantial improvement in structural performance was observed in the confined specimens. The CFRP-confined columns (Group G4) demonstrated significant strength enhancement, with ultimate capacities ranging from 1527 kN to 4775 kN, along with notable increases in deformation capacity, see Figure 3d. This improvement is attributed to the passive confinement provided by the CFRP wrapping, which becomes effective at higher strain levels and delays the onset of failure. However, the most pronounced enhancement was observed in the geopolymer-confined specimens (Group G3). Specimen CONF. SCGC C2 achieved an ultimate load of 8270 kN, representing an increase of more than six times the capacity of the reference RC C1 specimen (Figure 3c). This exceptional performance cannot be attributed solely to increased stiffness of the confining material. Instead, it results from a combined mechanism involving section enlargement and triaxial confinement effects. The SCGC jacket increases the effective cross-sectional area, thereby directly enhancing the load-bearing capacity. Simultaneously, it restrains the lateral expansion of the core concrete under axial loading, inducing a triaxial compressive stress state that significantly increases the effective compressive strength of the confined core. This confinement mechanism delays crack propagation, suppresses brittle failure, and enhances both strength and ductility.



**Figure 3.** Experimental Load–displacement curves of the tested columns.

Although the specimens were designed to maintain approximately similar longitudinal reinforcement ratios, the observed variation in load-carrying capacity indicates that axial strength is not solely determined by reinforcement ratio. In particular, the rectangular specimens (C2) consistently exhibited higher capacities than their square and circular counterparts. This behavior can be attributed to differences in cross-sectional geometry, effective confinement, and stress distribution, which collectively influence the efficiency of load transfer and the mobilization of material strength. Overall, the results demonstrated a progressive improvement in structural performance from unconfined RC and SCGC specimens to CFRP- and SCGC-confined columns (Table 3). While SCGC material alone improved deformation behavior and cracking resistance, the introduction of confinement, particularly through SCGC jacketing, led to substantial gains in load capacity, stiffness, and energy absorption. The highest performance across all metrics was consistently observed in the CONF-SCGC group, confirming the effectiveness of combined section enlargement and passive confinement in improving axial structural response.

**Table 3.** Summary of the experimental results for the tested columns.

| Specimen | $\Delta cr$ (mm) | $P_{cr}$ (kN) | $\Delta u$ (mm) | $P_u$ (kN) | $K$ (kN/mm) | $E$ (kN·mm) |
|----------|------------------|---------------|-----------------|------------|-------------|-------------|
| RC C1    | 0.55             | 527           | 2.07            | 1342       | 958         | 2236        |
| RC C2    | 0.62             | 905           | 2.31            | 2288       | 1460        | 4085        |
| RC C3    | 0.67             | 502           | 2.06            | 1272       | 749         | 1975        |
| SCGC C1  | 0.77             | 803           | 3.21            | 2003       | 1043        | 4842        |
| SCGC C2  | 0.79             | 1286          | 2.85            | 2965       | 1628        | 5719        |
| SCGC C3  | 0.78             | 678           | 2.72            | 1746       | 870         | 3176        |

|               |      |      |      |      |      |        |
|---------------|------|------|------|------|------|--------|
| CONF. FRP C1  | 1.23 | 1321 | 3.9  | 3378 | 1074 | 9219   |
| CONF. FRP C2  | 1.16 | 1810 | 3.62 | 4775 | 1560 | 13,307 |
| CONF. FRP C3  | 0.7  | 602  | 2.41 | 1527 | 860  | 2769   |
| CONF. SCGC C1 | 0.53 | 2443 | 2.52 | 6358 | 4610 | 13,053 |
| CONF. SCGC C2 | 0.58 | 3208 | 3.11 | 8270 | 5531 | 19,740 |
| CONF. SCGC C3 | 0.7  | 2049 | 2.49 | 5372 | 2927 | 11,440 |

### 3.2. Load–Displacement and Deformation Behaviour

The load–displacement responses from experimental measurements provide critical insight into the columns’ deformation mechanisms. For the RC group, the curves exhibited a predominantly linear elastic response up to the cracking load ( $P_{cr}$ ), which occurred at 527 kN for RC C1 (Table 3). The displacement at ultimate load remained relatively small across all RC specimens, indicating high initial stiffness but limited deformation capacity, consistent with the brittle behaviour characteristic of unconfined RC columns. The relatively low cracking displacements of RC specimens further confirm their brittle response characteristics (Table 3). In contrast, the SCGC specimens showed larger ultimate displacements, with SCGC C1 reaching 3.21 mm, indicating improved deformation capacity and delayed crack propagation compared to conventional concrete. The corresponding increase in cracking displacement suggests enhanced resistance to crack initiation.

The geopolymer-confined specimens (Group G3) demonstrated a substantial increase in stable deformation, along with their high load capacity. Specimen CONF. SCGC C2 reached an ultimate displacement of 3.11 mm, while also exhibiting significantly higher cracking loads and ultimate loads, reflecting the strong confinement effect of the SCGC jacket. The CFRP-confined specimens (Group G4) exhibited the highest overall ductility. In particular, specimen CONF. FRP C1 reached an ultimate displacement of 3.90 mm accompanied by moderate increases in load capacity (Table 3). This behaviour highlights the effectiveness of CFRP confinement in enhancing deformation capacity, despite its relatively lower contribution to ultimate strength compared to SCGC confinement.

### 3.3. Elastic Stiffness and Energy Absorption

The structural efficiency of the tested columns was further evaluated in terms of elastic stiffness ( $K$ ) and energy absorption capacity ( $E$ ). The RC specimens exhibited stiffness values ranging from 749 kN/mm to 1460 kN/mm, indicating relatively high initial rigidity but limited energy dissipation capacity. The introduction of confinement mechanisms significantly enhanced both stiffness and toughness. CFRP-confined specimens demonstrated a noticeable improvement in energy absorption capacity (Table 3), attributable to the wrap’s ability to delay crack propagation and sustain deformation prior to rupture. This increase is primarily attributed to the FRP wrap’s ability to delay crack propagation and sustain deformation before rupture, thereby improving ductility.

In contrast, SCGC-confined specimens exhibited the highest overall structural efficiency. As shown in Table 3, specimen CONF. SCGC C2 achieved a peak stiffness of 5531 kN/mm, which is substantially higher than that of the unconfined RC specimens. In addition, the same specimen recorded a maximum toughness of 19,740 kN·mm, nearly 9 times that of the reference RC C1 specimen (2236 kN·mm). This significant enhancement is attributed to the combined effect of section enlargement and effective confinement, in which the SCGC jacket restrains the lateral expansion of the core concrete, enabling sustained load resistance and progressive energy dissipation prior to failure.

### 3.4. Failure Modes and Morphological Analysis

The observed failure patterns provide critical physical evidence of the influence of confinement type and material composition on the structural behaviour of the tested columns. The transition from brittle to more ductile response is strongly governed by the degree of lateral restraint introduced by the different strengthening systems. The RC specimens (Group G1) exhibited typical axial-compression failure, characterized by concrete crushing and longitudinal reinforcement buckling at mid-height, as shown in Figure 4. This mode reflects limited confinement and a relatively brittle post-peak response.



**Figure 4.** Observed failure modes of Groups G1 (RC columns) and G2 (SCGC columns).

In contrast, the unconfined SCGC specimens (Group G2) exhibited a distinct failure mechanism, characterized by diagonal shear cracking and localized spalling of the geopolymer cover (Figure 5). In particular, specimen SCGC C2, which reached a relatively high peak load, exhibited sudden, brittle collapse following crack propagation, indicating that, despite improved material homogeneity, the absence of confinement leads to unstable failure. A significant change in failure behaviour was observed in the SCGC-confined specimens (Group G3). As illustrated in Figure 5, the presence of the SCGC jacket acted as an external confining layer, effectively restraining lateral expansion of the core concrete. Failure in these specimens was characterized by the development of vertical cracks in the jacket, followed by gradual degradation of the confined core. This progressive failure mode explains the substantial increase in energy absorption capacity observed in these specimens, as the confinement delays crack localization and allows sustained load resistance.



**Figure 5.** Observed failure modes of Group G3 (SCGC-jacketed confined columns, CONF. SCGC) and Group G4 (CFRP-wrapped confined columns, CONF. FRP).

The CFRP-confined specimens (Group G4) exhibited a distinct failure mechanism governed by the tensile rupture of the FRP wrap (Figure 5). This occurred when the lateral dilation of the concrete core exceeded the tensile strain capacity of the fibers, generating high hoop stresses in the confinement layer. As a result, failure was sudden and brittle, despite the significant enhancement in deformation capacity prior to rupture, as evidenced by the high ultimate displacement recorded for specimen CONF. FRP C1.

### 3.5. Normalized Stress–Strain Evaluation

Although the load–displacement response provides a direct measure of each specimen’s structural performance, comparisons based solely on absolute load may be influenced by differences in cross-sectional dimensions. This is particularly important for the SCGC-jacketed specimens (Group G3), where the application of a 100 mm thick jacket substantially increased the effective cross-sectional area. Therefore, normalized stress and strain parameters were evaluated to distinguish the influence of section enlargement from the intrinsic efficiency of the strengthening system.

The axial stress was calculated as

$$\sigma = \frac{P}{A_g}$$

where P is the applied axial load and  $A_g$  is the gross cross-sectional area. For the SCGC-jacketed specimens (Group G3),  $A_g$  included both the original reinforced concrete core and the surrounding SCGC jacket (e.g., 175,305 mm<sup>2</sup> for specimen CONF. SCGC C2). For the CFRP-confined specimens (Group G4), the original cross-sectional area was adopted because the CFRP wrap does not significantly alter the member dimensions.

The axial strain was determined as

$$\varepsilon = \frac{\Delta}{L}$$

where  $\Delta$  is the measured axial displacement and L is the specimen height (1000 mm). The normalized response parameters, including cracking stress ( $\sigma_{cr}$ ), ultimate stress ( $\sigma_u$ ), cracking strain ( $\varepsilon_{cr}$ ), ultimate strain ( $\varepsilon_u$ ), normalized elastic modulus ( $E_c$ ), and modulus of toughness ( $u_t$ ), are summarized in Table 4.

**Table 4.** Normalized response parameters of the columns.

| Specimen            | $\sigma_{cr}$ (MPa) | $\sigma_u$ (MPa) | $\varepsilon_{cr} (\times 10^{-3})$ | $\varepsilon_u (\times 10^{-3})$ | $E_c$ (MPa) | $u_t$ (MJ/m <sup>3</sup> ) |
|---------------------|---------------------|------------------|-------------------------------------|----------------------------------|-------------|----------------------------|
| Group 1 (RC)        |                     |                  |                                     |                                  |             |                            |
| RC C1               | 16.82               | 42.84            | 0.55                                | 2.07                             | 30,579      | 71.37                      |
| RC C2               | 19.29               | 48.78            | 0.62                                | 2.31                             | 31,127      | 87.09                      |
| RC C3               | 15.98               | 40.49            | 0.67                                | 2.06                             | 23,841      | 62.87                      |
| Group 2 (SCGC)      |                     |                  |                                     |                                  |             |                            |
| SCGC C1             | 25.63               | 63.94            | 0.77                                | 3.21                             | 33,292      | 154.55                     |
| SCGC C2             | 27.42               | 63.21            | 0.79                                | 2.85                             | 34,708      | 121.93                     |
| SCGC C3             | 21.58               | 55.58            | 0.78                                | 2.72                             | 27,693      | 101.09                     |
| Group 3 (CONF-SCGC) |                     |                  |                                     |                                  |             |                            |
| CONF. SCGC C1       | 17.19               | 44.73            | 0.53                                | 2.52                             | 32,435      | 91.84                      |
| CONF. SCGC C2       | 18.30               | 47.17            | 0.58                                | 3.11                             | 31,551      | 112.60                     |
| CONF. SCGC C3       | 16.31               | 42.75            | 0.70                                | 2.49                             | 23,292      | 91.04                      |
| Group 4 (CONF-FRP)  |                     |                  |                                     |                                  |             |                            |
| CONF. FRP C1        | 42.17               | 107.82           | 1.23                                | 3.90                             | 34,281      | 294.26                     |
| CONF. FRP C2        | 38.59               | 101.80           | 1.16                                | 3.62                             | 33,259      | 283.70                     |
| CONF. FRP C3        | 19.16               | 48.61            | 0.70                                | 2.41                             | 27,375      | 88.14                      |

The normalized results demonstrate that the strengthening systems enhance structural performance through fundamentally different mechanisms. Although the SCGC-jacketed specimens exhibited the highest absolute load capacities, their normalized ultimate stresses remained comparable to those of the reference RC specimens. This indicates that the observed increase in axial capacity primarily results from the enlarged load-bearing area provided by the SCGC jacket rather than a proportional increase in the compressive stress sustained by the material. Accordingly, the SCGC jacket functions as an additional structural component that increases the effective resisting area while simultaneously providing passive confinement to the concrete core.

In contrast, the CFRP-confined specimens achieved the highest normalized stresses, exceeding 100 MPa without increasing the cross-sectional dimensions. The increase in normalized stress confirms that CFRP confinement improves the mechanical efficiency of the existing concrete by restricting lateral dilation, delaying crack propagation, and maintaining a triaxial compressive stress state throughout loading. Consequently, the enhancement provided by CFRP originates from improved material utilization rather than section enlargement.

The normalized toughness values further distinguish the behavior of the two strengthening systems. The CFRP-confined specimens exhibited the highest toughness per unit volume, indicating superior energy absorption efficiency at the material level. Conversely, the SCGC-jacketed specimens achieved greater overall energy dissipation due to the larger volume of load-carrying material participating in the structural response. These findings emphasize that the normalized parameters complement the conventional load–displacement analysis by separating geometric effects from material behavior, thereby providing a more objective assessment of the efficiency of each strengthening technique.

### 3.6. Comparative Assessment Based on Cross-Sectional Geometry

In alignment with the revised presentation of the load–displacement curves in Figure 3, which are now grouped by geometry to facilitate direct comparison, the experimental results are additionally summarized according to cross-sectional geometry in Table 5. Unlike the original presentation, which categorized specimens by strengthening group, this geometry-based arrangement enables a direct evaluation of the influence of material type and confinement technique while maintaining an identical cross-sectional profile.

The specimens are classified into three geometric series: Square (C1), Rectangular (C2), and Circular (C3). Within each series, the structural responses of the conventional reinforced concrete (RC), self-compacting geopolymer concrete (SCGC), CFRP-confined (CONF. FRP), and SCGC-jacketed (CONF. SCGC) specimens are compared using common performance indicators, including cracking load ( $P_{cr}$ ), ultimate load ( $P_u$ ), stiffness ( $K$ ), and energy absorption ( $E$ ).

**Table 5.** Experimental performance comparison grouped by specimen geometry.

| Geometry Series            | Specimen ID   | $P_{cr}$ (kN) | $P_u$ (kN) | $\sigma_u$ (MPa) | $K$ (kN/mm) | $E$ (kN·mm) |
|----------------------------|---------------|---------------|------------|------------------|-------------|-------------|
| Series C1<br>(Square)      | RC C1         | 527           | 1342       | 42.8             | 958         | 2236        |
|                            | SCGC C1       | 803           | 2003       | 63.9             | 1043        | 4842        |
|                            | CONF. FRP C1  | 1321          | 3378       | 103.1            | 1074        | 9219        |
|                            | CONF. SCGC C1 | 2443          | 6358       | 44.73            | 4610        | 13,053      |
| Series C2<br>(Rectangular) | RC C2         | 905           | 2288       | 48.8             | 1460        | 4085        |
|                            | SCGC C2       | 1286          | 2965       | 63.2             | 1628        | 5719        |
|                            | CONF. FRP C2  | 1810          | 4775       | 98.1             | 1560        | 13,307      |
|                            | CONF. SCGC C2 | 3208          | 8270       | 47.17            | 5531        | 19,740      |
| Series C3<br>(Circular)    | RC C3         | 502           | 1272       | 40.5             | 749         | 1975        |
|                            | SCGC C3       | 678           | 1746       | 55.6             | 870         | 3176        |
|                            | CONF. FRP C3  | 602           | 1527       | 44.8             | 860         | 2769        |
|                            | CONF. SCGC C3 | 2049          | 5372       | 42.7             | 2927        | 11,440      |

This presentation provides a clearer assessment of the relative effectiveness of each strengthening technique for a given configuration. It allows for a direct evaluation of the improvements achieved by SCGC casting, external CFRP confinement, and SCGC jacketing. Furthermore, the geometry-based comparison highlights the influence of section shape on strengthening efficiency; specifically, it confirms that the rectangular specimens (C2 series) consistently exhibited higher absolute load-carrying capacities than their square and circular counterparts across all material systems, attributable primarily to their larger gross cross-sectional area. However, on a normalized stress basis, the square C1 specimens achieved comparable or higher  $\sigma_u$  values than C2, indicating that the superior absolute performance of C2 is largely an area effect rather than a shape effect.

The tabulated comparison demonstrates a consistent enhancement in structural performance from the reference RC specimens to the strengthened specimens across all geometries. In every series, the SCGC-jacketed specimens attained the highest axial capacities and stiffness values, primarily due to the combined effect of section enlargement and triaxial confinement. Conversely, while the CFRP-confined specimens provided lower ultimate loads compared to the jacketed specimens, they showed notable improvements in energy absorption (E) relative to their unconfined counterparts, particularly in the C1 and C2 series. Accordingly, this geometry-based presentation provides a more comprehensive interpretation of the experimental results and supports the comparative discussions regarding confinement efficiency and shape effects presented in the following sections. The exception is CONF. SCGC C3, where the normalized stress (42.7 MPa) remains comparable to the unconfined RC C3 (40.5 MPa), confirming that the fourfold area increase from the 100 mm radial jacket dominates the response for this geometry.

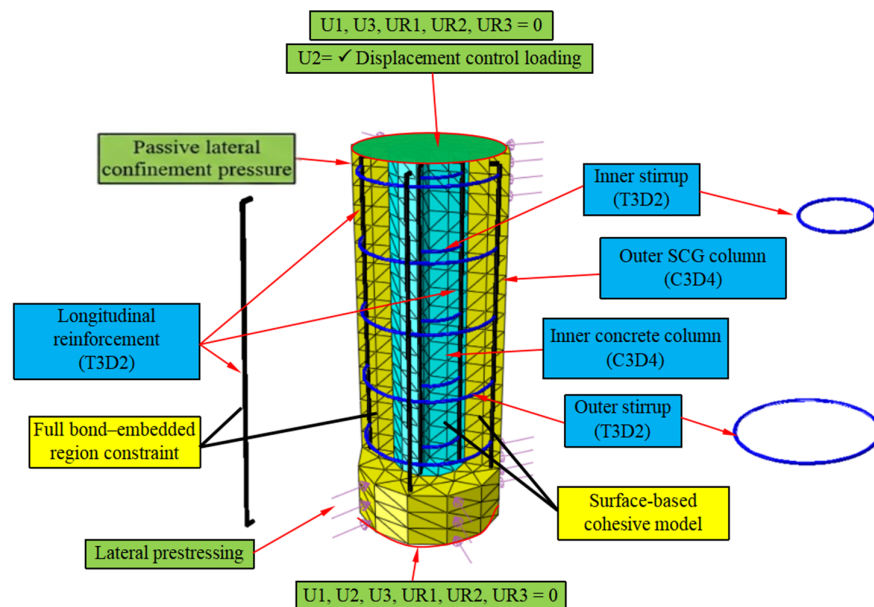
A notable observation arises when comparing the absolute load-carrying capacity of the circular CFRP-confined specimen (CONF. FRP C3, 1527 kN) with the unconfined rectangular control column (RC C2, 2288 kN). Although circular cross-sections theoretically optimize passive confinement pressure due to a uniform radial stress distribution, CONF. FRP C3 exhibits a lower absolute capacity than RC C2. This variation is primarily a function of geometric scale; the gross cross-sectional area of the rectangular C2 series is approximately 50% larger than that of the circular C3 series (46,905 mm<sup>2</sup> versus 31,416 mm<sup>2</sup>). When evaluating the normalized ultimate stress, CONF. FRP C3 achieved 48.61 MPa, which is virtually identical to the 48.78 MPa sustained by RC C2, confirming that the disparity in peak load is entirely area-dependent rather than a reflection of structural underperformance. However, CONF. FRP C3 exhibited a relatively modest strength enhancement of 20% compared to its direct unconfined control (RC C3, 1272 kN). This behavior points to a localized experimental sensitivity. Visual observations during testing and numerical strain data indicate that minor, unintended loading eccentricities exerted a more pronounced destabilizing effect on the smaller circular profile. Furthermore, a premature localized rupture of the CFRP jacket occurred near the mid-height region, triggered by high stress concentrations at the overlap zone before the full theoretical confinement pressure could be uniformly mobilized across the entire section. Consequently, while the CFRP wrap successfully enhanced member ductility, its contribution to the absolute peak capacity was partially constrained by these geometric and experimental boundary sensitivities.

## 4. Numerical and Analytical Modeling

### 4.1. Finite Element Model Development

A three-dimensional nonlinear finite element (FE) model was developed using ABAQUS/Standard to simulate the structural response of the tested column specimens. The modeling strategy was designed to replicate the experimental conditions and capture the interaction between concrete, reinforcement, and external confinement systems. A discrete modeling approach was adopted to explicitly represent each component of the

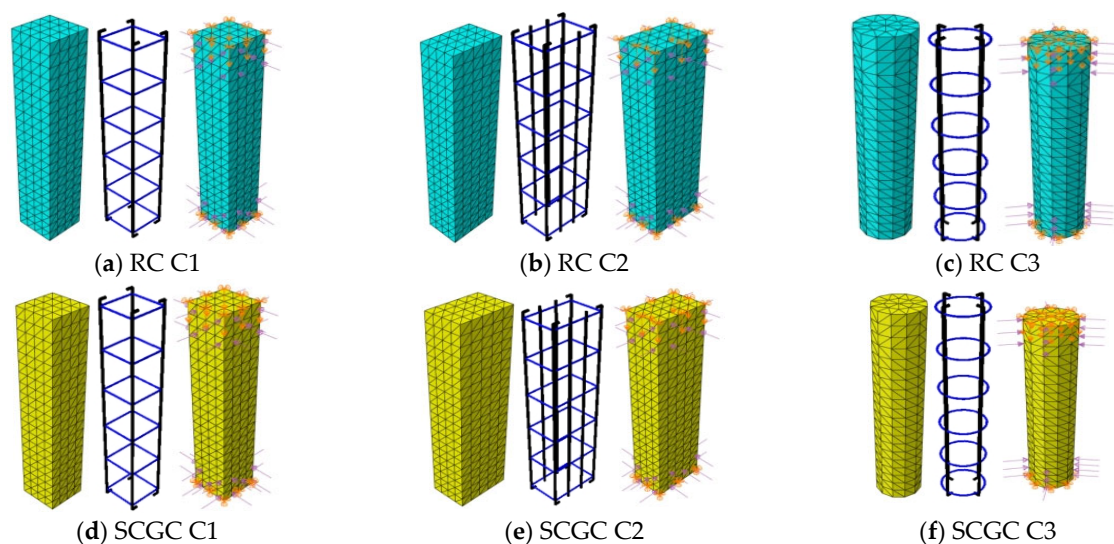
composite system. The concrete core, geopolymer concrete jacket (SCGC), and loading plates were modeled using four-node linear tetrahedral elements (C3D4), while the internal reinforcement cage was represented using embedded truss elements (T3D2). To illustrate the overall modeling strategy and mesh configuration, the developed finite element model is presented in Figure 6.

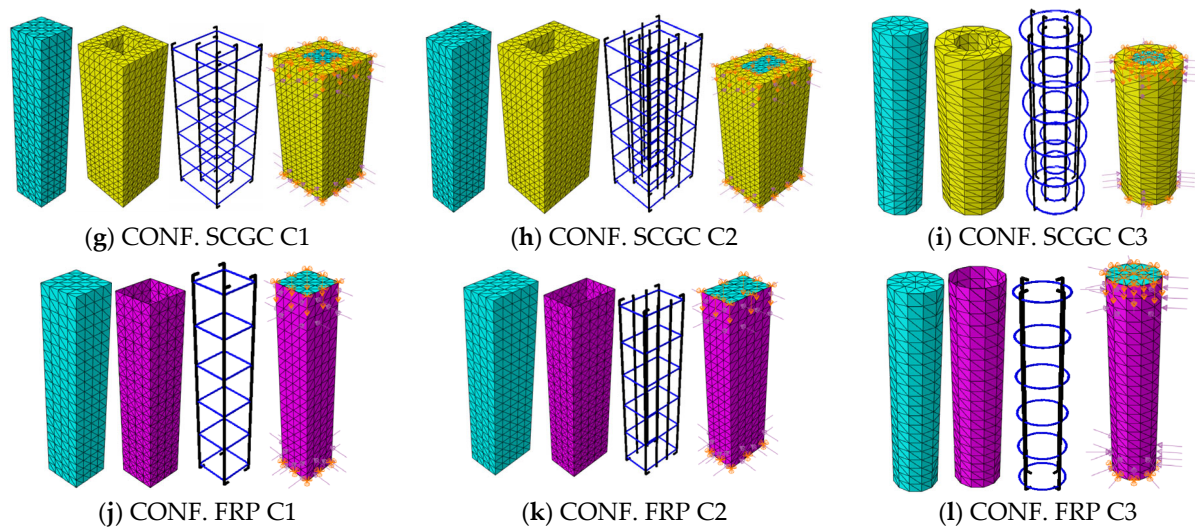


**Figure 6.** Three-dimensional finite element mesh and model components for the representative specimen CONF. SCGC C3.

#### 4.2. Discretization and Model Assembly

The numerical models were constructed to accurately reproduce the geometry and strengthening configurations of all experimental specimens. This includes control RC columns (G1), SCGC specimens (G2), SCGC-confined columns (G3), and CFRP-confined columns (G4). The discretized models for each group are presented in Figure 7, demonstrating the variation in cross-sectional geometry and confinement systems. This classification ensures consistency between the experimental and numerical programs, enabling direct comparison of structural behaviour across all specimen groups.





**Figure 7.** Finite element model configurations for all twelve specimens across four groups.

#### 4.3. Material Modeling and Constitutive Laws

The nonlinear behaviour of concrete and geopolymer materials was simulated using the Concrete Damaged Plasticity (CDP) model, which captures both compressive crushing and tensile cracking through damage mechanics. The model incorporates stiffness degradation and distinguishes between tensile and compressive responses (Figure 8). The reinforcing steel was modeled using an elastic–perfectly plastic constitutive law, while the CFRP confinement system was defined as an orthotropic linear-elastic material with failure governed by a maximum tensile strain criterion. The interfacial behavior between the concrete core and the SCGC jacket was simulated using a cohesive zone model (CZM) defined by a bilinear traction–separation law. This formulation enables the representation of progressive debonding under increasing load.

To ensure full reproducibility of the finite element simulations, the constitutive parameters adopted for all material models are summarized in Tables 6–9. The numerical inputs include the Concrete Damaged Plasticity (CDP) parameters, reinforcement properties, CFRP material characteristics, and cohesive-zone interface parameters.

Whenever available, the material parameters were derived directly from the experimentally measured properties of the tested materials. Manufacturer-provided data were adopted for the CFRP system, whereas selected parameters that could not be determined experimentally (such as CDP plastic flow parameters and cohesive fracture energies) were obtained from established literature and calibrated against the experimental response. The concrete core and SCGC jacket were modeled using the CDP formulation. The compressive and tensile constitutive relationships were based on the experimental peak values, with the post-peak softening curves and damage evolution parameters ( $d_c$ ,  $d_t$ ) determined through established analytical models for confined and unconfined concrete.

Reinforcing steel was represented using an elastic–perfectly plastic constitutive model, while the CFRP confinement system was modeled as an orthotropic linear-elastic material governed by a maximum tensile strain failure criterion. The interface between the existing concrete core and the SCGC jacket was simulated using a bilinear cohesive-zone model. The adopted interface stiffnesses, damage initiation parameters, and fracture energies listed in Table 8 were selected to reproduce the experimentally observed interaction and debonding behavior between the substrate and the strengthening layer. The use of these standardized parameters ensures that the triaxial confinement effects and the transition from the linear-elastic phase to failure are captured with numerical consistency across all specimen geometries.

**Table 6.** Plasticity parameters for the Concrete Damaged Plasticity (CDP) model.

| Parameter       | Description                                                  | Value   | Source                |
|-----------------|--------------------------------------------------------------|---------|-----------------------|
| $\psi$          | Dilation angle                                               | 36°–40° | Calibrated/Literature |
| $\epsilon$      | Eccentricity                                                 | 0.1     | Literature            |
| $f_{b0}/f_{c0}$ | Initial equibiaxial/uniaxial compressive yield stress ratio  | 1.16    | Literature            |
| $K_c$           | Ratio of the second stress invariant on the tensile meridian | 0.667   | Literature            |
| $\mu$           | Viscosity parameter                                          | 0.0005  | Numerical Stability   |

**Table 7.** Material properties for reinforcing steel and CFRP.

| Material | Property          | Symbol          | Value    | Source                |
|----------|-------------------|-----------------|----------|-----------------------|
| Steel    | Elastic Modulus   | $E_s$           | 200 GPa  | Assumption/Literature |
|          | Yield Strength    | $f_y$           | 400 MPa  | Assumption/Literature |
|          | Poisson's ratio   | $\nu$           | 0.3      | Literature            |
| CFRP     | Elastic Modulus   | $E_f$           | 238 GPa  | Manufacturer          |
|          | Tensile Strength  | $f_{tu}$        | 4300 MPa | Manufacturer          |
|          | Ultimate Strain   | $\epsilon_{fu}$ | 1.8%     | Manufacturer          |
|          | Nominal thickness | $t_f$           | 0.12 mm  | Manufacturer          |

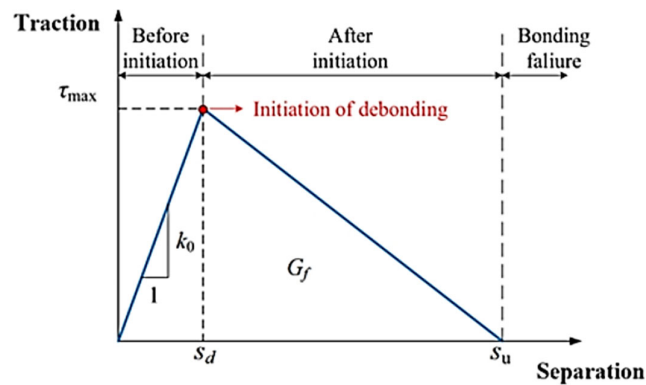
**Table 8.** Cohesive Zone Model (CZM) parameters for the Core–Jacket interface.

| Parameter        | Symbol           | Value | Unit   | Source     |
|------------------|------------------|-------|--------|------------|
| Normal Stiffness | $K_{nn}$         | 105   | MPa/mm | Calibrated |
| Shear Stiffness  | $K_{ss}, K_{tt}$ | 104   | MPa/mm | Calibrated |
| Normal Strength  | $t_{n0}$         | 3.5   | MPa    | Literature |
| Shear Strength   | $t_{s0}, t_{t0}$ | 5.0   | MPa    | Literature |
| Fracture Energy  | $G_{fc}$         | 0.1   | N/mm   | Calibrated |

**Table 9.** Constitutive and damage parameters for concrete compressive and tensile behavior.

| Loading State | Yield Stress (MPa) | Inelastic/Cracking Strain | Damage Parameter ( $d/d_i$ ) | Source                      |
|---------------|--------------------|---------------------------|------------------------------|-----------------------------|
| Compression   | 17.50              | 0.0000                    | 0.00                         | Derived from Exp. $f'_c$    |
|               | 35.00              | 0.0005                    | 0.00                         | Analytical Model            |
|               | 42.84              | 0.0015                    | 0.00                         | Experimental Peak [Table 3] |
|               | 32.10              | 0.0035                    | 0.25                         | Softening Curve             |
|               | 15.50              | 0.0100                    | 0.65                         | Softening Curve             |
| Tension       | 3.50               | 0.0000                    | 0.00                         | 0.1× $f'_c$ Assumption      |
|               | 2.10               | 0.0001                    | 0.40                         | Tension Stiffening          |
|               | 0.35               | 0.0005                    | 0.90                         | Fracture Energy $G_f$       |

The normal and shear stiffness values were calibrated against the experimental load–displacement response of the CONF. SCGC specimens (Table 8). The interface strength values were adopted from published experimental investigations on OPC–geopolymer concrete interfaces [28–30]. The numerical results confirmed that partial interfacial slip initiated near the column ends at approximately 70–80% of the ultimate load, consistent with the experimentally observed crack patterns. Full debonding did not occur prior to concrete crushing, confirming that the CZM was active throughout the loading history and that the interface remained partially bonded up to failure.



**Figure 8.** Bilinear traction–separation constitutive law adopted for the cohesive zone model (CZM) at the OPC concrete core–SCGC jacket interface.

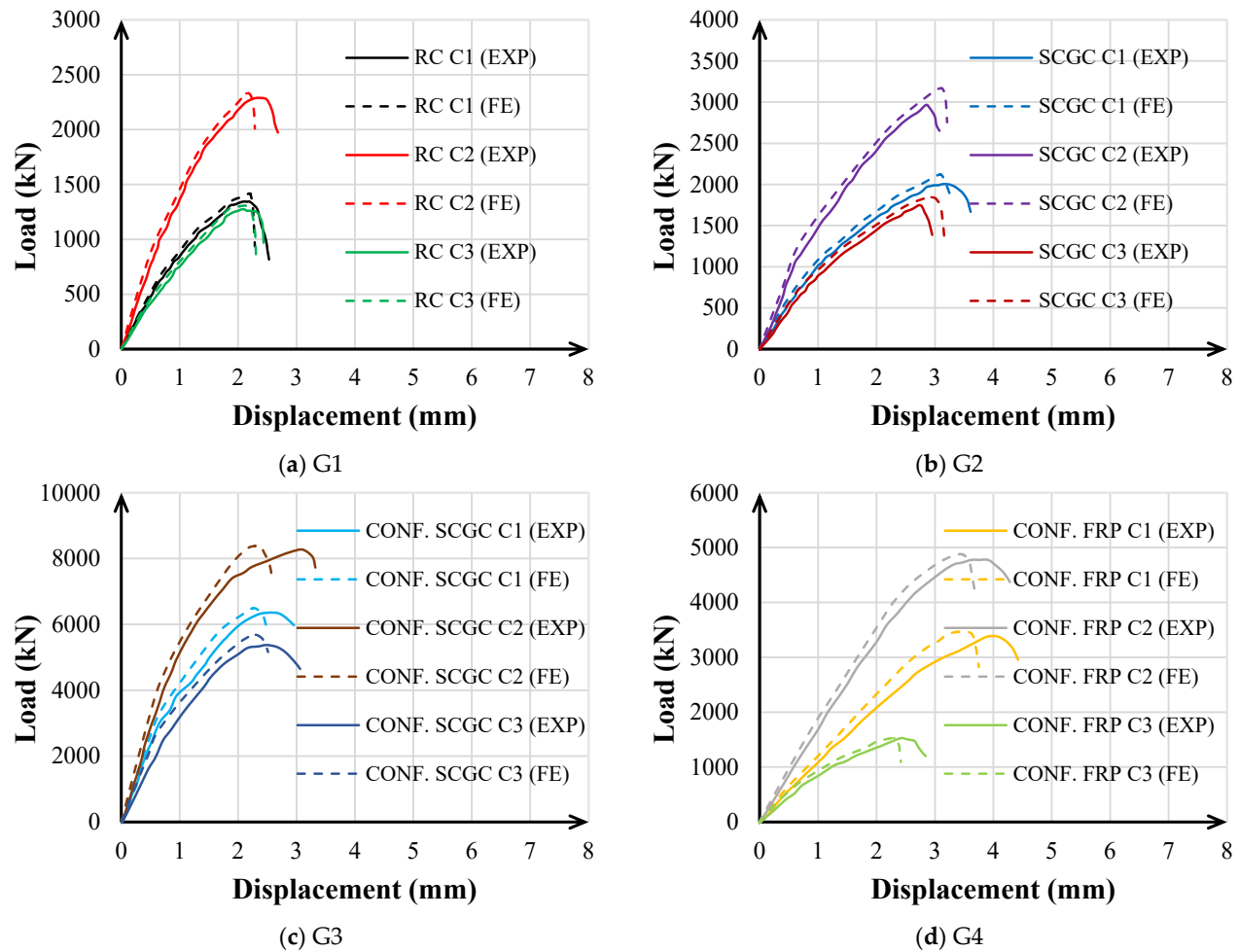
#### 4.4. Interface and Boundary Conditions

To simulate the physical constraints of the experimental setup, the model’s base was assigned a fully fixed boundary condition. The loading was applied at the top surface via a reference point coupled to the loading plate, utilizing displacement control to ensure numerical convergence during the post-peak softening phase. For the interface mechanics, a “surface-to-surface” contact algorithm was employed between the RC core and the SCGC jacket. This incorporated a cohesive zone model (CZM) to simulate the potential for interfacial debonding, governed by a bilinear traction–separation response.

#### 4.5. Interface Modeling, Boundary Conditions, and Model Validation

Boundary conditions were defined to replicate the experimental setup, with the base fully fixed and axial loading applied through displacement control at the top surface. This approach ensures numerical stability and enables accurate capture of post-peak behaviour. A mesh sensitivity analysis was conducted to ensure that the selected element size provides an optimal balance between computational efficiency and accuracy. The results indicated that further mesh refinement produced negligible changes in the load–displacement response. The validity of the FE model was assessed by comparing it with experimental results. As shown in Figure 9, the numerical predictions agree well with the experimental load–displacement curves across all specimen groups. The developed finite element model successfully captures the key behavioral characteristics observed experimentally, including the initial stiffness of the specimens, the ultimate load-carrying capacity, and the post-peak softening response. Furthermore, the model effectively reproduces the sudden strength degradation exhibited by CFRP-confined specimens, reflecting the brittle rupture behavior of the FRP confinement system.

The bottom loading plate was fully restrained against all translational and rotational degrees of freedom, while axial loading was applied to the top loading plate through a reference point using displacement control. Lateral boundary conditions were applied symmetrically within the top and bottom 100 mm end zones of each specimen, while the central 800 mm remained free to deform laterally. This boundary condition scheme was adopted consistently across all finite element models.



**Figure 9.** Comparison of numerically predicted (FE, dashed lines) and experimentally measured (EXP, solid lines) axial load–displacement curves for all specimen groups.

#### 4.6. Mesh Sensitivity and Convergence Analysis

To ensure the numerical stability and accuracy of the finite element results, a systematic mesh sensitivity study was conducted prior to the final simulations. This analysis aimed to identify an appropriate mesh density that balances computational efficiency with solution accuracy. Several mesh configurations with progressively refined element sizes were evaluated using the reference specimen RC C1 as a benchmark.

Table 10 summarizes the results of the mesh convergence analysis, comparing the performance of three distinct mesh densities: Coarse, Medium (Selected), and Fine. The evaluated parameters include the total number of elements, the total number of nodes, the ultimate load ( $P_u$ ), the displacement at ultimate load ( $\Delta u$ ), the initial stiffness ( $K$ ), the energy absorption capacity ( $E$ ), and the predicted failure mode.

**Table 10.** Mesh convergence results for the reference RC C1 specimen.

| Mesh Density | Element Size (mm) | No. of Elements | No. of Nodes | Peak Load $P_u$ (kN) | Disp. at Peak $\Delta u$ (mm) | Stiffness K (kN/mm) | Energy E (kN·mm) | Predicted Failure Mode |
|--------------|-------------------|-----------------|--------------|----------------------|-------------------------------|---------------------|------------------|------------------------|
| Coarse       | 40                | 1950            | 2840         | 1482                 | 2.35                          | 1025                | 2480             | Diffuse Crushing       |
| Medium *     | 25                | 8240            | 10,650       | 1416                 | 2.20                          | 965                 | 2315             | Mid-height Crushing    |
| Fine         | 15                | 37,120          | 44,800       | 1402                 | 2.18                          | 962                 | 2295             | Mid-height Crushing    |

\* Denotes the mesh configuration adopted for all numerical models in this study.

The results indicate that progressive mesh refinement produced only marginal variations in the predicted structural response once the element size reached 25 mm. Specifically, the difference in ultimate load and stiffness between the Medium and Fine mesh configurations was less than 1.5%, while the computational cost increased significantly for the Fine mesh. Furthermore, the failure mode, characterized by localized concrete crushing and reinforcement yielding at the mid-height, remained consistent across the refined models.

Consequently, the 25 mm mesh size was selected as the optimum compromise between numerical accuracy and computational efficiency. This discretization was considered sufficiently refined to accurately capture the nonlinear material behavior, including cracking patterns and triaxial confinement effects, as evidenced by the close agreement between the numerical predictions and experimental measurements reported in the following sections.

#### 4.7. Analytical Prediction of Axial Load Capacity

The axial load-carrying capacity ( $P_u$ ) of RC columns strengthened with SCGC jacketing and CFRP wrapping can be analytically estimated using a confinement-based superposition model. The model assumes concentric axial compression and accounts for the strength enhancement of the concrete core due to the triaxial stress state induced by transverse reinforcement, the SCGC overlay, and external CFRP wrapping. The total predicted axial capacity ( $P_{u,ana}$ ) is calculated as the summation of the contributions from the confined concrete core, the SCGC strengthening layer, the CFRP confinement (if applicable), and the longitudinal reinforcement:

$$P_{u,ana} = f'_{cc} A_c + \beta f_{cSCGC} A_{SCGC} + \gamma f_{l,CFRP} A_g + f_y A_{sl}$$

where

- $f'_{cc}$  is the confined compressive strength of the concrete core:  $f'_{cc} = f'_c + \alpha \cdot fl$ .
- $fl$  is the total effective lateral confinement stress:  $fl = fl_{stirrups} + fl_{SCGC} + fl_{CFRP}$ .
- $A_c$ ,  $A_{SCGC}$ ,  $A_{sl}$  represent the cross-sectional areas of the core, the SCGC jacket, and the longitudinal steel, respectively.
- $\alpha$ ,  $\beta$ ,  $\gamma$  are efficiency coefficients to account for the geometry-dependent confinement effectiveness of the SCGC jacket and CFRP wrap. The recommended ranges for these coefficients were established based on physical reasoning, observed experimental trends, and values reported in the literature for comparable confined column configurations. The following ranges were used:  $\alpha = 0.90$ – $1.00$  for the confined concrete core;  $\beta = 0.80$ – $0.95$  for the SCGC jacket; and  $\gamma = 0.95$ – $1.00$  for the longitudinal reinforcement.

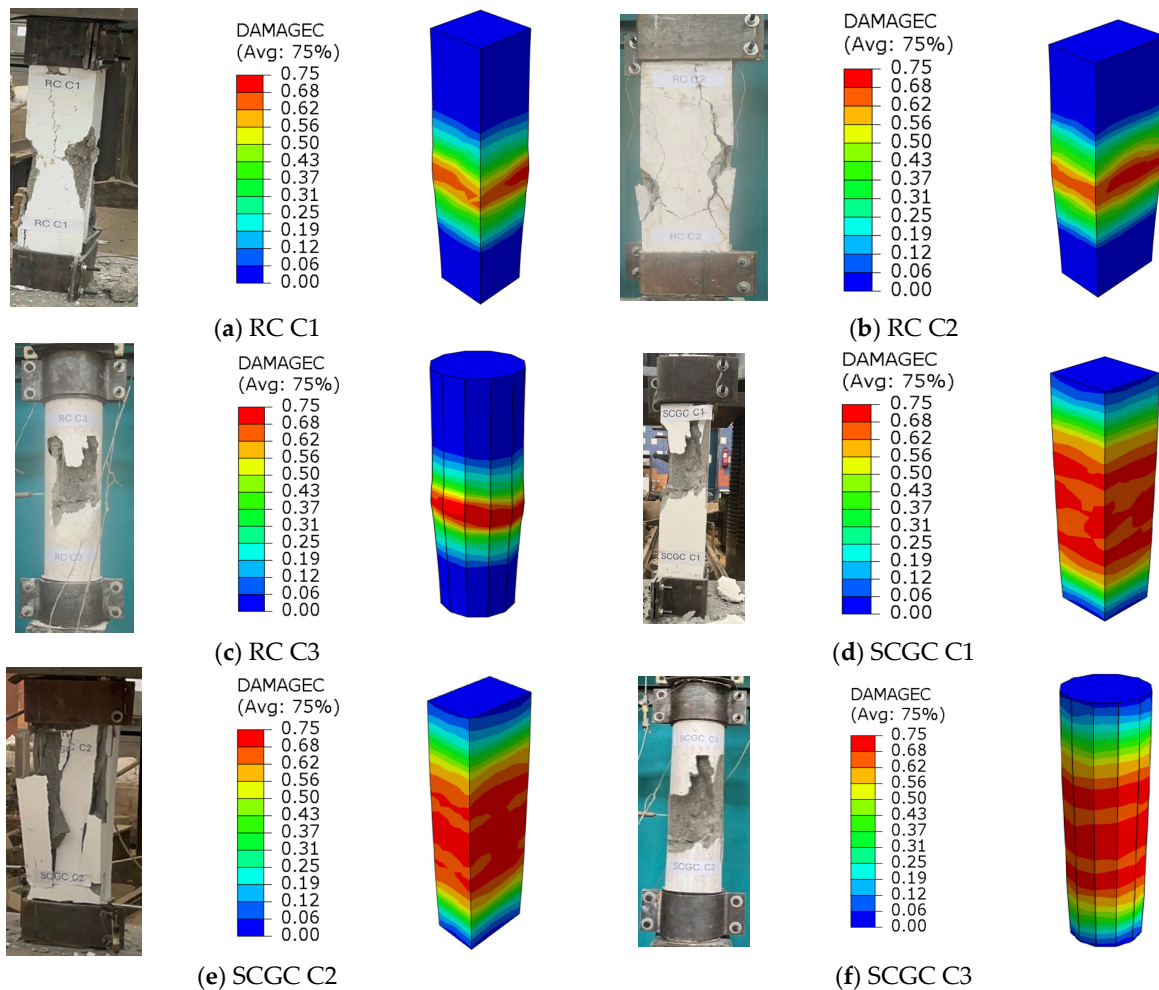
The model assumes a perfect bond at the core–jacket interface, neglecting slip or interfacial separation as supported by the cohesive zone modeling used in the numerical study.

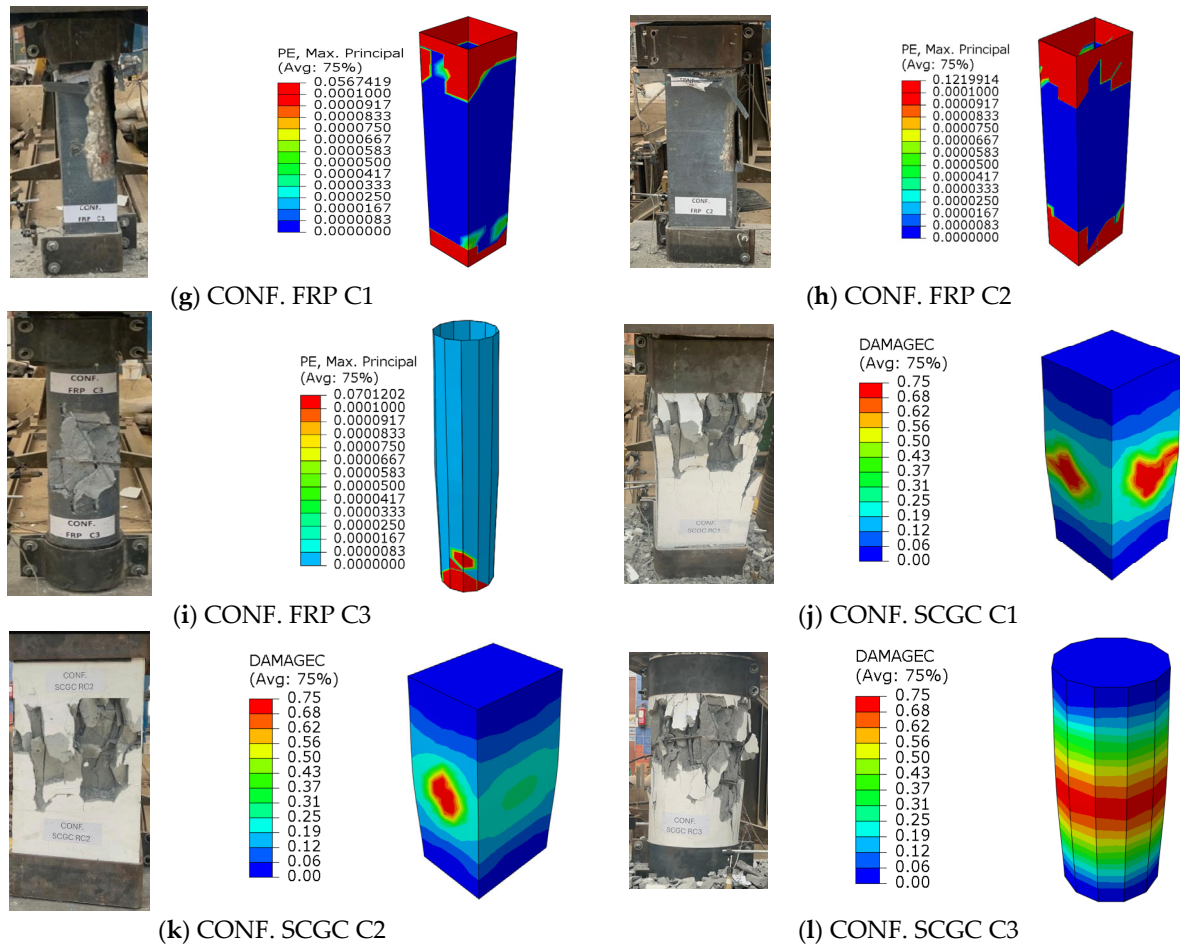
## 5. Comparative Assessment

This section presents a comprehensive evaluation of the experimental, numerical, and analytical results to assess the accuracy of the developed models and to establish the relative performance of the investigated strengthening techniques.

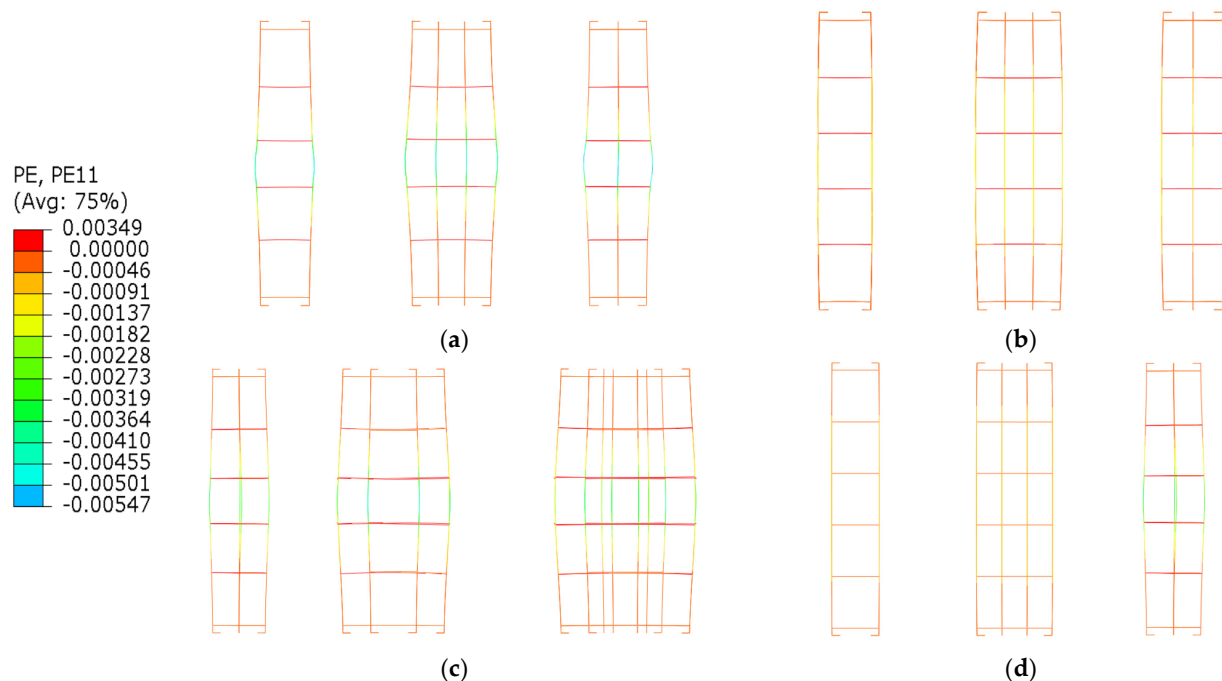
### 5.1. Comparison of Failure Modes

A qualitative comparison between the experimentally observed and numerically predicted failure patterns is presented in Figure 10. The finite element model demonstrates a strong capability to reproduce the dominant failure mechanisms across all specimen groups. For the RC and SCGC specimens, the numerical results accurately capture concrete crushing and localized damage zones, while for confined specimens, the model successfully simulates confinement-induced deformation and failure progression. For CFRP-confined columns, numerical predictions accurately capture the abrupt failure associated with FRP rupture. Further insight into the internal behavior is provided by the distribution of plastic strains in the reinforcement, as illustrated in Figure 11. The results confirm that yielding is primarily localized in regions of maximum stress concentration, consistent with the experimentally observed deformation patterns.





**Figure 10.** Comparison of numerical and experimental failure patterns of columns.



**Figure 11.** Numerically predicted plastic strain distribution in the longitudinal steel reinforcement at ultimate load for representative specimens from each group: (a) G1, (b) G2, (c) G3, and (d) G4.

### 5.2. Quantitative Comparison and Model Accuracy

The quantitative comparison between the experimental and numerical results is summarized in Table 11, covering both the cracking and ultimate stages. The ratios of numerical-to-experimental predictions indicate a high level of agreement. At the cracking stage, the mean ratios of displacement and load are approximately 0.93 and 1.05, respectively, indicating that the numerical model slightly underestimates deformation and marginally overestimates the cracking load. At the ultimate stage, the mean ratio ( $P_{u,FE}/P_{u,EXP}$ ) is 1.03, indicating high predictive accuracy across all specimen groups. However, the displacement predictions exhibited a noticeable divergence, particularly for the SCGC-jacketed specimens (Group G3). The model consistently underpredicted the ultimate displacement for all jacketed columns, with prediction ratios ranging from 0.74 to 0.92, as summarized in Table 11. The most significant discrepancy was observed in specimen CONF. SCGC C2, where the predicted displacement of 2.29 mm was 26% lower than the experimental value of 3.11 mm (ratio = 0.74). This systematic underprediction indicates that the finite element model is conservative in capturing the full mobilization of ductility and progressive load redistribution within the enlarged cross-section. These discrepancies are primarily attributed to three factors: (1) the idealized bilinear traction–separation law adopted for the cohesive zone model at the core–jacket interface, which may not fully capture the progressive slip and microcracking occurring at large deformations; (2) the assumption of perfectly concentric loading in the finite element environment, whereas minor accidental eccentricities in the experimental setup may have allowed greater sustained deformation; and (3) the post-peak material softening laws adopted for the CDP model, which may underestimate the residual ductility of confined geopolymer concrete. Consequently, while the model demonstrates robustness for load capacity estimation, it provides a conservative lower-bound estimate of deformation capacity for heavily jacketed configurations. Future modeling efforts should focus on improved calibration of the cohesive zone interface parameters and post-peak material laws to better capture the deformation behavior of SCGC-jacketed columns. The CoV values for load prediction remain low (below 5%), confirming the consistency and reliability of the numerical model for strength estimation across all specimen groups.

**Table 11.** Quantitative comparison between numerical (FE) and experimental (EXP) results.

| Specimen ID               | FE Results      | EXP Results     | Ratio (FE/EXP)    |
|---------------------------|-----------------|-----------------|-------------------|
| Group G3 (SCGC Jacketed)  | $\Delta_u$ (mm) | $\Delta_u$ (mm) | $\Delta_u$ ,ratio |
| CONF. SCGC C1             | 2.24            | 2.52            | 0.89              |
| CONF. SCGC C2             | 2.29            | 3.11            | 0.74              |
| CONF. SCGC C3             | 2.30            | 2.49            | 0.92              |
| Group G4 (CFRP Wrapped)   |                 |                 |                   |
| CONF. FRP C1              | 3.39            | 3.90            | 0.87              |
| CONF. FRP C2              | 3.48            | 3.62            | 0.96              |
| CONF. FRP C3              | 2.30            | 2.41            | 0.95              |
| Overall Statistics        |                 |                 |                   |
| Mean Ratio ( $\Delta_u$ ) |                 |                 | 0.96              |
| Mean Ratio ( $P_u$ )      |                 |                 | 1.03              |

### 5.3. Evaluation of Numerical Predictions

The comparison of ultimate load and displacement reveals that the FE model captures the overall trends in structural performance across all specimen groups. For control RC columns, the model provides close predictions with minor deviations, reflecting the relatively simple behaviour of unconfined concrete. For SCGC specimens, the numerical

model slightly overestimates the ultimate capacity in some cases, which may be attributed to idealized material properties and the assumption of uniform stress distribution. In contrast, for confined SCGC specimens, larger discrepancies are observed in some cases due to complex interactions between the concrete core and the external jacket, particularly in capturing interface behaviour and confinement efficiency. For CFRP-confined specimens, the model successfully predicts the enhanced load capacity and deformation behaviour; however, slight variations arise due to the simplified representation of FRP rupture and the assumption of perfect bonding. Part of the discrepancy between numerical and experimental results can be attributed to the FE model's assumption of perfectly concentric loading, whereas the experimental setup exhibited minor unintended eccentricities. This difference contributes to variations in stiffness and ultimate deformation predictions.

#### 5.4. Assessment of Strengthening Techniques

The combined experimental and numerical results enable a clear ranking of the effectiveness of the investigated strengthening techniques. The CONF-SCGC specimens exhibit the highest performance in terms of ultimate load capacity, stiffness, and energy absorption. This superior behaviour is attributed to the combined effect of section enlargement and strong passive confinement, which significantly enhances the triaxial stress state within the concrete core. The CONF-FRP specimens demonstrate substantial improvements in ductility and deformation capacity, although their ultimate strength enhancement is lower compared to SCGC-confined specimens. This behaviour reflects the passive confinement mechanism of CFRP, which is highly effective in delaying failure but is limited by its brittle rupture characteristics.

The SCGC specimens without external confinement show moderate improvement over control RC columns, primarily due to enhanced material properties rather than confinement effects. Finally, the control RC specimens exhibit the lowest performance, characterized by limited ductility and abrupt post-peak failure. The superior performance of CONF-SCGC specimens can be further explained using classical confinement models, where the confined compressive strength is a function of the effective lateral confining pressure. The SCGC jacket acts as a rigid confining medium, generating significant passive confinement as lateral dilation occurs. Both SCGC and FRP provide passive confinement; however, unlike the CFRP wrap, which requires larger lateral strains to generate effective confining pressure due to its flexibility, the SCGC jacket, owing to its higher stiffness, engages earlier in the loading history, resulting in higher initial stiffness and load capacity.

#### 5.5. CFRP Rupture and Confinement Activation

The CFRP confinement system was modeled using a maximum tensile strain failure criterion corresponding to an ultimate tensile strain of 1.8% (18,000  $\mu\epsilon$ ), as specified by the manufacturer. During axial compression, the lateral dilation of the concrete core progressively mobilized the confinement action of the CFRP wrap, resulting in the development of circumferential (hoop) tensile stresses within the fibers.

As loading increased, the hoop strains in the CFRP layer continued to rise due to the expansion of the confined concrete core. Numerical simulations and experimental observations confirmed that CFRP rupture was initiated when the local hoop strain approached the ultimate capacity. For instance, in specimen CONF. FRP C1, the failure was triggered as the recorded hoop strain reached approximately 14,800 to 15,500  $\mu\epsilon$ , representing about 82–86% of the manufacturer's ultimate strain capacity. This reduction from the theoretical 1.8% is consistent with established literature regarding the FRP efficiency factor, which accounts for stress concentrations and the triaxial state of the concrete core.

The numerical simulations confirmed that the highest hoop strain concentrations developed in the regions where CFRP rupture was experimentally observed (mid-height),

indicating that the confinement mechanism was fully activated prior to failure. These findings support the interpretation that the enhanced strength and deformation capacity of the CFRP-confined columns, reaching ultimate axial strains ( $\epsilon_u$ ) up to  $3.90 \times 10^{-3}$ , resulted from the progressive mobilization of lateral confinement until the tensile strain limit was reached.

### 5.6. Interpretation of Confinement Mechanisms

The improved axial performance of both strengthened systems stems from a common mechanism: restraint of the lateral dilation of the concrete core, which induces a triaxial stress state and enhances both strength and ductility [36,37]. The two systems mobilize this mechanism differently. The SCGC jacket, being stiffness-compatible with and directly bonded to the substrate, engages from early loading stages, so its contribution combines genuine confinement with an enlarged resisting section, the same enlargement-plus-confinement action documented for conventional RC jacketing [38]. The CFRP wrap, in contrast, is a passive, strain-activated system that develops hoop tension only once the core dilates appreciably near peak load [39]; this explains why CFRP-confined specimens show the largest normalized-stress gains despite unchanged cross-sectional area, with the entire gain attributable to confinement efficiency rather than section enlargement. The normalized ultimate stresses of the CFRP-confined specimens exceeded 100 MPa (up to 107.82 MPa for CONF. FRP C1), a gain of more than 250% over the unconfined reference, while rupture hoop strains (14,800–15,500  $\mu\epsilon$ ) reached 82–86% of the manufacturer's ultimate strain, consistent with the reduction in FRP hoop rupture strain below flat-coupon ultimate strain reported by Lam and Teng and attributed to jacket curvature, non-uniform concrete deformation, and overlap-zone effects [9].

Confinement efficiency was also strongly shape-dependent. Although circular sections are theoretically expected to sustain the most uniform confining pressure, CONF. FRP C3 showed the smallest strength gain (20%) among the CFRP group, well below the square and rectangular specimens. This is attributed to the combined effect of arching action between corners in non-circular sections, captured by the effectiveness coefficient  $k_e$ , and the well-documented stress concentration and premature FRP rupture at sharp corners of non-circular sections [40], compounded in the smaller circular section by loading eccentricity and localized rupture at the overlap zone [41]. Material stiffness compounded this behavior: the SCGC jacket's compatibility with the core produced gradual, distributed confinement and higher absolute energy absorption, whereas the stiffer CFRP activated later but confined more efficiently once engaged, yielding higher normalized toughness [39]. Confinement efficiency was therefore governed jointly by jacket stiffness/activation strain, cross-sectional geometry, and effective confined area, rather than by material type alone.

The observed strength gains (up to  $\sim 2.1\times$  for CFRP-confined rectangular columns, and larger absolute gains for SCGC jacketing) fall within the range typically reported for FRP-confined RC columns and are consistent with the well-established reduction in confinement effectiveness as cross-sectional aspect ratio and corner sharpness increase [42]. These results extend that established behavior to a sustainable-materials context, showing that SCGC jacketing follows the same enlargement-dominated trend reported for conventional RC jacketing [38], and that CFRP confinement of geopolymer-based sections follows the same strain-activation trend reported for OPC concrete, Lokuge and Karunasena [43] showed that FRP-confined normal-strength geopolymer concrete exhibits stress-strain behavior similar to FRP-confined high-strength OPC concrete, confirming that the underlying confinement mechanics are not altered by the use of geopolymer concrete.

### 5.7. Practical Implications and Engineering Significance

From a practical perspective, the results highlight that the selection of a strengthening technique should be guided by the desired performance objective. If the primary goal is to maximize load-carrying capacity and stiffness, geopolymer concrete jacketing provides the most effective solution. Conversely, if enhanced ductility and energy dissipation are required, particularly in seismic applications, CFRP wrapping offers significant advantages. The strong agreement between experimental, numerical, and analytical results confirms the reliability of the proposed modeling approaches. The developed finite element and analytical models can therefore be confidently used as predictive tools for the design and assessment of reinforced concrete columns strengthened with reinforcement. It should be noted that the extremely high strength gains observed in confined SCGC specimens necessitate careful consideration in practical applications, particularly with respect to scaling effects, constructability, and economic feasibility. Future studies should investigate the influence of jacket thickness, interface bonding conditions, and size effects to validate the scalability of the proposed strengthening technique.

## 6. Conclusions

This study presented an integrated experimental, numerical, and analytical investigation of reinforced concrete columns strengthened using SCGC and CFRP confinement systems. The results demonstrate that structural performance is governed by both material characteristics and confinement mechanisms. Unconfined SCGC specimens exhibited improved cracking behaviour and deformation capacity compared to conventional RC columns; however, their axial strength did not consistently exceed that of all RC counterparts. This indicates that SCGC primarily enhances ductility and material uniformity rather than peak load capacity alone. The introduction of confinement systems led to significant performance improvements. CFRP-confined specimens exhibited enhanced strength and ductility but failed in a brittle manner due to FRP rupture. In contrast, SCGC-confined specimens achieved the highest absolute load capacity, stiffness, and energy absorption. This superior absolute performance is attributed to the combined effects of section enlargement and passive triaxial confinement, where the SCGC jacket increases both the effective area and the confined compressive strength of the core concrete. Normalized stress comparisons revealed that the CFRP-confined specimens achieved the highest material-level strength enhancement, with ultimate axial stress values exceeding 100 MPa compared to 41–49 MPa for the reference RC columns, while the large absolute load gains of the SCGC-jacketed specimens were primarily attributable to section enlargement. The finite element model demonstrated strong agreement with experimental results, accurately capturing key behaviours including stiffness, peak load, and failure modes; however, the model provided conservative estimates of ultimate displacement for the SCGC-jacketed specimens, with the largest discrepancy of 26% observed in CONF. SCGC C2. The analytical model provided reliable, conservative predictions of axial capacity, confirming its applicability for design purposes. It is acknowledged that the experimental program comprises a single specimen per configuration, and the reported findings should therefore be interpreted as indicative trends rather than statistically confirmed results. The findings highlight the significant potential of geopolymer-based confinement systems as a potentially more environmentally friendly and high-performance alternative to conventional OPC-based strengthening methods. A formal life cycle assessment is recommended in future work to quantify the net environmental benefit per column. Future research should also focus on refining the modelling of interface behaviour, investigating scale effects, and exploring the long-term performance of SCGC-confined systems under varying environmental and loading conditions.

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## References

1. Elsamak, G.; El-Zohairy, A.; Badawi, M.A.; Iskander, Y.; Bazuhair, R.W.; Ghalla, M. Investigation of the structural performance of reinforced concrete long slender columns strengthened with stainless steel, galvanized steel, and aluminum sheets. *Structures* **2025**, *81*, 110210. <https://doi.org/10.1016/j.istruc.2025.110210>.
2. Mukhopadhyay, T.; Naskar, S.; Dey, S.; Chakrabarti, A. Condition Assessment and Strengthening of Aged Structures: Perspectives Based on a Critical Case Study. *Pract. Period. Struct. Des. Constr.* **2019**, *24*, 05019003. [https://doi.org/10.1061/\(ASCE\)SC.1943-5576.0000431](https://doi.org/10.1061/(ASCE)SC.1943-5576.0000431).
3. Abdelraheem, H.; Ahmed, A.-R.; Ahmed, M.; Haridy, A.E.-K. Behavior of Rc Columns Strengthened with Steel Jacket Under Static Axial Load. *JES J. Eng. Sci.* **2022**, *51*, 93–108. <https://doi.org/10.21608/jesaun.2022.169609.1174>.
4. Ulloa, N.; Naranjo, E.; Arcos, P.; Mery Mendoza Castillo, M. Developing a Data-Powered Framework for the Capacity of Concrete Columns Wrapped with Fiber-Reinforced Polymers. *J. Sustain. Res.* **2026**, *8*, e260010. <https://doi.org/10.20900/jsr20260010>.
5. Wang, J.; Lu, L.; Lu, S.; Yang, J. Behavior and modeling of partially CFRP confined reinforced concrete columns under axial compression. *Eng. Struct.* **2024**, *303*, 117466. <https://doi.org/10.1016/j.engstruct.2024.117466>.
6. Wang, Z.; Wang, D.; Smith, S.T.; Lu, D. CFRP-Confined Square RC Columns. I: Experimental Investigation. *J. Compos. Constr.* **2012**, *16*, 150–160. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000245](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000245).
7. Ashteyat, A.M.; Obaidat, A.T. Experimental Investigation of Heat-Damaged RC Slender Spiral Columns Repaired with CFRP Rope. *Infrastructures* **2025**, *10*, 21. <https://doi.org/10.3390/infrastructures10010021>.
8. Gomaa, A.M.; Ahmed, M.A.; Khafaga, S.A.; Lotfy, E.M.; Hosny, S. Strengthening of corroded RC slab–column joints using thinly hybrid FRP under punching shear. *Sci. Rep.* **2026**, *16*, 6526. <https://doi.org/10.1038/s41598-026-36610-2>.
9. Lam, L.; Teng, J.G. Design-oriented stress–strain model for FRP-confined concrete. *Constr. Build. Mater.* **2003**, *17*, 471–489. [https://doi.org/10.1016/S0950-0618\(03\)00045-X](https://doi.org/10.1016/S0950-0618(03)00045-X).
10. Shahin, R.I.; Ghalla, M.; Elsamak, G.; Badawi, M.; Bazuhair, R.W.; El-Naqeeb, M.H. Comparative study on the use of SHCC, HPFRC, and GFRP for enhancing the axial performance of RC columns with reduced reinforcement capacity. *Structures* **2025**, *80*, 110103. <https://doi.org/10.1016/j.istruc.2025.110103>.
11. Alajarmeh, O.; Manalo, A.; Benmokrane, B.; Ferdous, W.; Mohammed, A.; Abousnina, R.; Elchalakani, M.; Edo, A. Behavior of circular concrete columns reinforced with hollow composite sections and GFRP bars. *Mar. Struct.* **2020**, *72*, 102785. <https://doi.org/10.1016/j.marstruc.2020.102785>.
12. Khalil, A.E.-H.A.; Elshafiey, T.F.; Baraghith, A.T.; Behiry, R.N.; Ali, H. Utilization of strain-hardening cementitious composites to restore the efficiency of square RC columns after fire exposure. *Eng. Struct.* **2025**, *322*, 119189. <https://doi.org/10.1016/j.eng-struct.2024.119189>.
13. Sun, J.-M.; Yi, W.-J.; Chen, H.; Peng, F.; Zhou, Y.; Zhang, W.-X. Dynamic Responses of RC Columns under Axial Load and Lateral Impact. *J. Struct. Eng.* **2023**, *149*, 04022210. <https://doi.org/10.1061/JSENDH.STENG-11612>.
14. Pacheco-Torgal, F.; Castro-Gomes, J.P.; Jalali, S. Adhesion characterization of tungsten mine waste geopolymeric binder. Influence of OPC concrete substrate surface treatment. *Constr. Build. Mater.* **2008**, *22*, 154–161. <https://doi.org/10.1016/j.conbuildmat.2006.10.005>.
15. Singh, B.; Ishwarya, G.; Gupta, M.; Bhattacharyya, S.K. Geopolymer concrete: A review of some recent developments. *Constr. Build. Mater.* **2015**, *85*, 78–90. <https://doi.org/10.1016/j.conbuildmat.2015.03.036>.

16. Faridmehr, I.; Nehdi, M.L.; Huseien, G.F.; Baghban, M.H.; Sam, A.R.M.; Algaifi, H.A. Experimental and Informational Modeling Study of Sustainable Self-Compacting Geopolymer Concrete. *Sustainability* **2021**, *13*, 7444. <https://doi.org/10.3390/su13137444>.
17. Fathi, M.E.; El-Zoughiby, M.E.; Mortagi, M.; Youssf, O.; Abdulazeez, M.; Tahwia, A.M. Shear Performance of Sustainable Self-Compacting Geopolymer RC Beams: Experimental and Numerical Study. *Infrastructures* **2026**, *11*, 84. <https://doi.org/10.3390/infrastructures11030084>.
18. Amran, M.; Al-Fakih, A.; Chu, S.H.; Fediuk, R.; Haruna, S.; Azevedo, A.; Vatin, N. Long-term durability properties of geopolymer concrete: An in-depth review. *Case Stud. Constr. Mater.* **2021**, *15*, e00661. <https://doi.org/10.1016/j.cscm.2021.e00661>.
19. Youssf, O.; Abbass, A.M.; Ahmed, A.K.; Tahwia, A.M. Sustainable High-Performance Geopolymer Concrete: The Role of Recycled Industrial Wastes in Strength, Durability, and Microstructure Enhancement. *Infrastructures* **2025**, *10*, 241. <https://doi.org/10.3390/infrastructures10090241>.
20. Zhu, Z. Research of Geopolymer Recycled Concrete. *Acad. J. Sci. Technol.* **2025**, *15*, 55–58. <https://doi.org/10.54097/636dfp90>.
21. Athobaiti, T.; Tahwia, A.M.; Abousnina, R.; Mortagi, M.; Youssf, O. Performance Evaluation and Microstructural Analysis of Eco-Friendly Self-Compacting Geopolymer Concrete. *Infrastructures* **2026**, *11*, 74. <https://doi.org/10.3390/infrastructures11030074>.
22. Reddy, K.R.K.; Rajkumar, D. Impact of Ferro-Geopolymer Encapsulation on the Structural Integrity of Concrete Columns Under Axial Load. *Rev. Compos. Matér. Avancés* **2025**, *35*, 273–280. <https://doi.org/10.18280/rcma.350209>.
23. Heshmati, M.; Sheikh, M.; Hadi, M. A comprehensive review of the fresh and hardened characteristics of self-compacting geopolymer concrete. *J. Sustain. Cem.-Based Mater.* **2025**, *14*, 816–835. <https://doi.org/10.1080/21650373.2025.2461182>.
24. Sherwani, A.F.H.; Younis, K.H.; Arndt, R.W. Fresh, Mechanical, and Durability Behavior of Fly Ash-Based Self Compacted Geopolymer Concrete: Effect of Slag Content and Various Curing Conditions. *Polymers* **2022**, *14*, 3209. <https://doi.org/10.3390/polym14153209>.
25. Kanagaraj, B.; Anand, N.; Alengaram, J.; Raj, S. Engineering properties, sustainability performance and life cycle assessment of high strength self-compacting geopolymer concrete composites. *Constr. Build. Mater.* **2023**, *388*, 131613. <https://doi.org/10.1016/j.conbuildmat.2023.131613>.
26. Zaid, O.; Sor, N.A.H.; Martínez-García, R.; De Prado-Gil, J.; Elhadi, K.M.; Yosri, A. Sustainability evaluation, engineering properties and challenges relevant to geopolymer concrete modified with different nanomaterials: A systematic review. *Ain Shams Eng. J.* **2024**, *15*, 102373. <https://doi.org/10.1016/j.asej.2023.102373>.
27. Kim, S.-W.; Park, I.-H.; Seok, S.; Choi, W.; Kim, J. Axial-Flexural Performance of Steel Fiber-Reinforced Concrete Columns: Effects of Axial Load Ratio and Steel Fiber Volume Fraction. *Materials* **2026**, *19*, 1014. <https://doi.org/10.3390/ma19051014>.
28. Ganeshan, M.; Venkataraman, S. Interface shear strength evaluation of self compacting geopolymer concrete using push-off test. *J. King Saud. Univ. Eng. Sci.* **2020**, *34*, 98–107. <https://doi.org/10.1016/j.jksues.2020.08.005>.
29. Mohsen, K.; Yousif, I.; Sadek, E.; Morsy, K. Eexperimental study of interface behaviour of geopolymer concrete. *Sci. Rep.* **2025**, *15*, 32696. <https://doi.org/10.1038/s41598-025-20036-3>.
30. Xu, F.; Chen, G.; Li, K.; Zhang, Z.; Luo, Z.; Li, S.; Yang, D.; Li, X.; Zhang, X. Interfacial bond behavior between normal OPC concrete and self-compacting geopolymer concrete enhanced by nano-SiO<sub>2</sub>. *Constr. Build. Mater.* **2024**, *411*, 134617. <https://doi.org/10.1016/j.conbuildmat.2023.134617>.
31. Kaur, M.; Chawla, H.; Kwatra, N. Effect of different surface preparation techniques on the bond behavior of normal and fiber-reinforced geopolymer concrete. *Constr. Build. Mater.* **2024**, *417*, 135252. <https://doi.org/10.1016/j.conbuildmat.2024.135252>.
32. Li, W.; Shumuye, E.D.; Shiyong, T.; Wang, Z.; Zerfu, K. Eco-friendly fibre reinforced geopolymer concrete: A critical review on the microstructure and long-term durability properties. *Case Stud. Constr. Mater.* **2022**, *16*, e00894. <https://doi.org/10.1016/j.cscm.2022.e00894>.
33. Dou, C.; Guo, Y.-L.; Zhao, S.-Y.; Pi, Y.-L. Experimental Investigation into Flexural-Torsional Ultimate Resistance of Steel Circular Arches. *J. Struct. Eng.* **2015**, *141*, 04015006. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001240](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001240).
34. Wu, Y.-F.; Wang, L.-M. Unified Strength Model for Square and Circular Concrete Columns Confined by External Jacket. *J. Struct. Eng.* **2009**, *135*, 253–261. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2009\)135:3\(253\)](https://doi.org/10.1061/(ASCE)0733-9445(2009)135:3(253)).
35. Rochette, P.; Labossière, P. Axial Testing of Rectangular Column Models Confined with Composites. *J. Compos. Constr.* **2000**, *4*, 129–136. [https://doi.org/10.1061/\(ASCE\)1090-0268\(2000\)4:3\(129\)](https://doi.org/10.1061/(ASCE)1090-0268(2000)4:3(129)).
36. Chen, P.; Cui, X.; Zheng, H.; Si, S. A Mesoscale Study on the Dilation of Actively Confined Concrete under Axial Compression. *Materials* **2022**, *15*, 6490. <https://doi.org/10.3390/ma15186490>.
37. Ibraheem, R.S.; Al-Zuhairi, A.H. A Comparative Study on Behavior of RC Columns Strengthened by CFRP and Steel Jacket. *E3S Web Conf.* **2021**, *318*, 03002. <https://doi.org/10.1051/e3sconf/202131803002>.

38. Ahmed, K.; Riaz, M. A Detailed Investigation of Reinforced Concrete Column Jacketing: Experiment, Theory and Numerical Analysis. 2016. Available online: <https://www.semanticscholar.org/paper/A-DETAILED-INVESTIGATION-OF-REINFORCED-CONCRETE-AND-Ahmed-Riaz/28003d585435d92ea81804f4660fa92f43fb6755> (accessed on 13 July 2026).
39. Lam, L.; Teng, J. A New Stress-Strain Model for Frp-Confined Concrete. 2001. Available online: <https://www.semanticscholar.org/paper/A-NEW-STRESS-STRAIN-MODEL-FOR-FRP-CONFINED-CONCRETE-Lam-Teng/4b26136beb552a21c0c3f316e89efd64ce2c6ad8> (accessed on 13 July 2026).
40. Wei, Y.; Xu, Y.; Wang, G.; Cheng, X.; Li, G. Influence of the Cross-Sectional Shape and Corner Radius on the Compressive Behaviour of Concrete Columns Confined by FRP and Stirrups. *Polymers* **2022**, *14*, 341. <https://doi.org/10.3390/polym14020341>.
41. Lam, L.; Teng, J.G. Hoop rupture strains of frp jackets in frp confined concrete. In *Fibre-Reinforced Polymer Reinforcement for Concrete Structures*; World Scientific Publishing Company: Singapore, 2003; pp. 601–612. [https://doi.org/10.1142/9789812704863\\_0056](https://doi.org/10.1142/9789812704863_0056).
42. Moussa, A.M.A.; Abdellatif, M.R.; Abdallah, M.S.E.; Farghal, O.A. Experimental study on the effect of aspect ratio on CFRP-confinement efficiency in rectangular RC columns. *Struct. Concr.* **2025**, *27*, 3081–3100. <https://doi.org/10.1002/suco.70228>.
43. Lokuge, W.; Karunasena, W. Ductility enhancement of geopolymer concrete columns using fibre-reinforced polymer confinement. *J. Compos. Mater.* **2016**, *50*, 1887–1896. <https://doi.org/10.1177/0021998315597553>.

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