

Research paper

Experimental Investigation of Concrete Columns Strengthening with Perforated Internal Carbon Fiber Reinforced Polymer Tubes

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Abstract: Due to limited availability of traditional construction materials, fiber-reinforced polymer (FRP) composites are gaining attention for strengthening concrete structures. This study investigates a novel hybrid method of reinforcing self-compacting concrete (SCC) columns using internally placed perforated CFRP tubes and external CFRP confinement, also serves as lost formwork.

Axial compression tests were conducted to evaluate mechanical behavior, focusing on compressive strength, deformability, and secant stiffness. Four configurations were tested: unconfined specimens, internal-only reinforcement (I-series), external-only confinement (O-series), and combined internal-external reinforcement (hybrid O-1/I-1 and O-3/I-1). Holes (2.8% surface area) were made in the CFRP tubes to improve the bond between core concrete and the external jacket.

The hybrid system (O-3/I-1) achieved the highest performance, increasing compressive strength by 104% and deformability by 50% compared to unconfined specimens. External confinement alone resulted in an 82% strength increase and improved stiffness, while internal strengthening contributed to a 12-15% increase in strength and provided sustained load-bearing capacity even after outer concrete cover had failed.

Analytical comparisons confirmed compatibility of test results with theoretical models. The findings highlight effectiveness of this composite technique for structural applications such as bridge piers and high-rise columns. Further research is recommended on factors like perforation geometry and column slenderness.

Keywords: FRP composites, self-compacting concrete, perforated composite tubes, concrete columns

Nomenclature

A_c – area of concrete [mm²],
 A_{CFRPi}, A_{CFRPO} – cross-sectional area of carbon fiber [mm²],
 A_{ci}, A_{co} – cross-sectional area of the confined concrete [mm²],
 C_4 – coefficient for calculation of effective rigidity of an encased composite compression member [-],
 D_o – diameter of outer section [mm],
 D_i – diameter of inner section [mm],
 E_f – modulus of elasticity of carbon fiber [GPa],
 E_c – modulus of elasticity of concrete [GPa],
 El_{eff} – effective flexural stiffness for calculation of relative slenderness [kNm²],
 I_{ci}, I_{co} – moment of inertia of the concrete section about the elastic neutral axis of the composite section [mm⁴],
 I_{fi}, I_{fo} – moment of inertia of carbon fiber shape about the elastic neutral axis of the composite section [mm⁴],
 $L_c = KL$ – effective length of member [mm],
 $N_{pl,Rk}$ – characteristic value of the plastic resistance of the composite section to compressive normal force [N],
 N_{cr} – elastic critical normal force [N],
 P_{ACI} – compressive strength of composite structure according to ACI 318-11 [MPa],
 P_{AISC} – compressive strength of composite structure according to ANSI/AISC 360-16 [MPa],
 P_{EC4} – compressive strength of composite structure according to EN 1994-1-1 [MPa],
 P_e – elastic critical buckling load [N],
 $P_{Hybrid-CFFT}$ – compressive strength of hybrid concrete-filled fiber-reinforced plastic tubes confined by filament winding [MPa],
 P_{no} – available compressive strength of axially loaded doubly symmetric filled composite members [N],
 f_{cc} – design value of the cylinder compressive strength of concrete [MPa],
 f_{CFRP} – tensile strength of carbon fiber [MPa],
 f_{CC1} – nominal axial stresses at the beginning of the transitional zone [MPa],
 f_{CC2} – nominal axial stresses at the end of the transitional zone [MPa],
 f_{CCu} – ultimate nominal axial stresses [MPa],
 f_{ck} – characteristic compressive strength [MPa],
 t_i, t_o – thickness element [mm],
 α – coefficient related to the compression design curve; coefficient in general [-],
 ξ – confinement reduction factor [-],
 ε_{CC1} – nominal axial strains at the beginning of the transitional zone [-],
 ε_{CC2} – nominal axial strains at the end of the transitional zone [-],
 ε_{CCu} – ultimate nominal axial strains [-],
 η_a, η_c – factors related to the confinement concrete [-],
 γ_a, γ_c – partial factor [-],
 $\bar{\lambda}$ – Relative slenderness
 χ – reduction factor as a function of reduced slenderness ratio

1. Introduction

1.1. *Column reinforcement with closed sections profiles*

Fiber-reinforced polymer (FRP) materials, such as strips and mats, are becoming increasingly popular in civil engineering [1]. Initially, they were used for the repair or strengthening of existing structures made of wood [2,3] masonry [4,5], steel [6], or reinforced concrete [7]. Currently, their application extends to the construction of new buildings [8,9]. Designers can now take advantage of the benefits of composites, such as high tensile strength, a high Young's modulus, a favorable strength-to-weight ratio, high resistance to chemical corrosion, and ease of installation, from the early stages of the design process [10,11].

The concept of using closed-section profiles as reinforcement for reinforced concrete elements is not new. Initial pilot studies of this type were conducted by Wei et al. in the late 20th century [12]. In these studies, a steel outer tube, which served as lost formwork after concrete placement, was combined with a smaller-diameter inner tube. The connection was achieved by filling the space between the tubes with a concrete mix. These studies demonstrated key advantages of this solution, such as reduced deflection of steel tubes and confinement of the concrete. The strength of the composite in uniaxial compressive testing was 10-30 % higher compared to the sum of its individual components. In subsequent years, interest in this solution persisted, and the solution became known as Concrete-Filled Double Tube (CFDT).

Zhao and Grzebieta in their work [13] described compression and bending tests conducted on Concrete-Filled Double Steel Tube (CFDST) samples with a square cross-section. Based on the results obtained, they observed enhanced plasticity during compression and bending tests compared to empty steel sections. At the same time, a theoretical model was proposed to predict the load-carrying capacity of the manufactured elements. The presented model did not consider the influence of the external ring on the confinement of the concrete core. A detailed mathematical analysis, incorporating the effects of confinement was included in the work [14]. The model considered not only the constraints imposed by the tubes but also the reduction in concrete strength due to large displacements. Based on these studies, Tao et al. [15] conducted research for CFDT with circular cross-sections. Their objective was to develop a universal confinement reduction factor (ξ), which quantifies the influence of the external tube on the load-carrying capacity of the concrete section. Huang et al. [16] compiled previous CFDT research findings and developed numerical models based on them. The results of these analyses confirmed the utility of finite element modeling (FEM) effectiveness in estimating the strength of double-shell samples with steel tubes. Similarly, Yan and Zhao [17] conducted a verification of computational models using experimental data from CFDST samples.

External confinement of concrete columns is an effective method of enhancing their load-bearing capacity and ductility. One of the emerging approaches in this field involves the use of fiber-reinforced polymer (FRP) composites, which, due to their mechanical

properties and corrosion resistance, are increasingly being applied in structural engineering.

The literature includes numerous studies on concrete columns confined with FRP [18–20], columns reinforced with steel and additionally strengthened with FRP [21,22], as well as large-scale columns that more accurately reflect real structural conditions [23–25]. Han et al. [26] demonstrated the feasibility of combining CFRP wraps with internal steel profiles, confirming the effectiveness of such hybrid reinforcement systems under cyclic loading conditions, while also highlighting the sudden failure mode of the tested specimens.

However, FRP materials are not without drawbacks. Many of them lack the capacity for plastic deformation during tension, leading to a sudden failure without prior visible damage. This necessitates an alternative analytical approach, incorporating additional safety factors, which may reduce the computational efficiency of the structure [11]. To effectively model the axial behavior of such structures under load, numerous confinement models – known as analysis-oriented models – have been introduced. These models simulate axial stress distributions in concrete columns confined with FRP wraps [27–32], FRP strips [31,33], and hybrid systems combining steel reinforcement and FRP strengthening [34,35]. In recent years, there has also been growing interest in the application of artificial intelligence methods, including predictive models based on machine learning techniques, for analyzing and forecasting the behavior of FRP-confined columns [36,37].

Another disadvantage of FRP materials is their relatively low fire resistance compared to traditional steel reinforcement. The maximum operating temperature for FRP rods is typically within the range of 200–300 °C, depending on the manufacturer, which is less than half of the operating temperature for steel rods. The epoxy resin used to mount FRP mats and tapes reaches its glass transition temperature at 50–60 °C, rendering the structure susceptible to damage from sunlight exposure. Research presented in [26] illustrates the scale of the problem. Using heat radiators, the authors heated concrete and wood beams strengthened with CFRP and SRP strips to 80 °C. According to the resin's technical data sheet, it lost its properties at 58 °C. The results confirmed the destruction of all tested samples due to strips delamination. A possible solution to this issue might be to apply a protective layer in the form of insulation. Guidelines for the design of FRP reinforcement are provided in the Canadian Design Standard S806-02 [38]. The charts presented therein allow for determining the minimum cover value based on the critical temperature and fire resistance class, and are derived from the research conducted by Kodur and Baingo [39]. As detailed in [40], the location of FRP reinforcement has a considerable impact on structural fire endurance. Specifically, Near-Surface Mounted (NSM) systems, where FRP tapes are installed in specially cut grooves within the concrete, exhibit fire resistance 1.54 to 2.08 times greater than Externally Bonded (EB) systems.

Carbon fibers, widely recognized for their exceptional tensile strength, can also be formed into composite profiles, expanding their potential structural applications. In [41], samples reinforced with profiles composed of carbon fiber mats embedded in epoxy resin were investigated. These profiles, differing in diameter and layer configuration, were subjected to axial compression tests. The results indicated nearly a twofold increase in deformation capacity in samples with internal reinforcement compared to unreinforced concrete specimens.

Beyond external reinforcement, composite FRP profiles have also been explored as multifunctional structural elements. One such application is their use as formwork, which initially serves as a mold during casting and, following concrete curing, functions as permanent external reinforcement. This dual-role strategy has been successfully implemented in various structural components, including ceiling beams [42], bridge decks [43], and column elements [44], with further design recommendations outlined in the guidelines provided by Abramski [8].

In the study [45], attention was drawn to the potential of placing composite reinforcement inside concrete columns. In the proposed solution, a glass fiber reinforced polymer (GFRP) tube serves as both longitudinal and transverse reinforcement, while its embedment within the concrete allows the concrete cover to act as a protective layer. Compared to traditional externally bonded systems and concrete-filled FRP tubes (CFFT), this approach significantly enhances fire resistance and performance under dynamic loads. Perforated tubes facilitate better integration between the concrete core and cover, increasing the interface strength and delaying cover spalling under extreme conditions. Although the presence of perforations reduces strength and deformation capacity, numerical analyses have shown that reducing hole diameter is an effective strategy for optimizing column performance. This type of internal FRP reinforcement may be particularly suitable for structures exposed to fire, impact loads, or aggressive environments where conventional steel reinforcement is prone to corrosion.

The primary objective of this research is to investigate the structural performance of concrete columns internally reinforced with perforated CFRP tubes. While previous studies have explored similar reinforcement systems using perforated GFRP tubes [45], the novelty of the present work lies in the use of carbon fiber composites. CFRP offers enhanced mechanical properties – particularly in stiffness and tensile strength – which may influence the failure mechanism and stress distribution in a different way compared to GFRP-based systems.

The perforations in the CFRP tubes are intended to improve the integration between the concrete core and the surrounding cover by allowing better flow of the fresh mix and promoting mechanical interlock. Although the actual effect of these perforations on axial behavior and load transfer remains to be validated experimentally, the configuration is expected to influence the composite action between materials. The internal placement of the reinforcement may also offer protective advantages in environments exposed to dynamic loads or chemical aggressiveness. In selected specimens, external CFRP tube

was additionally used as stay-in-place formwork. This element not only acts as casting formwork but may also provide additional confinement to the concrete core. The combined use of internal and external CFRP elements could offer practical benefits in terms of structural durability and construction processes, particularly in scenarios where corrosion resistance and load resilience are critical.

A comprehensive experimental investigation was carried out at the Cracow University of Technology to evaluate the axial performance of concrete columns reinforced internally with perforated CFRP tubes. The study involved uniaxial compression testing, with the aim of assessing the load-bearing behavior, stiffness characteristics, and failure modes of the composite specimens. In the final stage of the research, existing analytical expressions for predicting axial load capacity were reviewed and compared against the experimental results to evaluate their applicability to the proposed reinforcement configuration.

1.2. Perforated inner CFRP tubes - Scientometric analysis

A review of previous studies utilizing closed-section profiles as concrete reinforcement has led to the concept of perforating the inner CFRP tube. This approach can potentially enhance load-bearing capacity, ductility, and the protection of composite reinforcement against elevated temperatures. To justify these assumptions and highlight the innovation of the proposed solution, a bibliometric analysis was conducted. This analysis, based on an international bibliometric database, aims to identify the main thematic areas and areas requiring further research. Additionally, the analysis considers the frequency of keyword occurrences, the number of citations, and the publication dates, allowing for the presentation of the dynamics of development of the selected research area.

The most accessible form of bibliometric analysis is keyword analysis, which is based on the number of publications containing specific keywords or their combinations. Individual keywords may refer to a wide range of topics, but the use of keyword combinations allows for narrowing down the theme and reducing the number of publications. The absence of publications containing the proposed keyword combination confirms the innovative nature of the research.

During the preliminary study, a bibliometric investigation was carried out using the set of keywords presented in Table 1. There is a growing interest in the use of closed-section profiles as internal reinforcement for reinforced concrete column elements. However, the concept of perforating inner tubes remains underexplored in current research.

Table 1. Summary of keyword occurrences for each database (accessed on 20.11.2024).

Selected keywords	Number of phrases (Scopus)
Step no 1	

tubes	724,290
concrete	578,348
perforated	48,324
CFRP	33,239
Step no 2	
inner + tube	26,161
Concrete + tube	15,096
self-compacting concrete	7,789
perforated + tube	3,072
CFRP + tube	1,428
Step no 3	
Concrete + inner + tube	1,288
Concrete + perforated + tube	57
Step no 4	
Concrete + reinforcement + inner + tube	182
Concrete + inner + CFRP + tube	104
Concrete + reinforcement + perforated + tube	10
Concrete + perforated + CFRP + tube	0

Table 2 provides the number of publications and patents directly related to the use of closed-section profiles, identified with a designated abbreviation as outlined in the excerpt. Additionally, a study was conducted in the bibliometric databases LENS, Scopus, and Springer to ensure a comprehensive analysis. The LENS bibliometric database, in particular, facilitates the detailed analysis of both scientific publications and patents.

Table 2. Summary of publications and patents reporting selected methods of reinforcing concrete columns with closed-section profiles [46–48].

	Concrete Filled Tube (CFT)	Concrete Filled Steel Tube (CFST)	Concrete Filled Double Tube (CFDT)	Concrete Filled Double Steel Tube (CFDST)	Concrete Encased Filled Double Tube	Concrete Encased Filled Double Steel Tube
Patents (Lens)	78,895	41,917	26,475	14,427	2,294	1,841
Publications (Lens)	16,378	11,745	3,566	2,167	231	163
Publications (Scopus)	9,621	9,596	1,051	1,075	24	23
Publications (Springer)	12,912	7,331	5,823	3,609	418	360

Next, using the keyword set "Concrete + inner + tube" a thematic relationship map and keyword frequency were generated with the VOSviewer program (Figure 1). To ensure clarity in presenting the results, the minimum number of connections was set to 5, reducing the total number of keywords from 6,579 to 706. The program enables graphical visualization of keyword frequency using a color scale. As the frequency of a keyword increases, its color transitions from blue to yellow. A visual analysis reveals the highest interest in utilizing steel profiles as reinforcement for reinforced concrete columns. Furthermore, the program provides a color-coded representation of keyword occurrence

over time. In this scale, blue indicates the oldest keywords, while yellow represents the most recent ones. This visualization highlights the ongoing nature of research in this field.

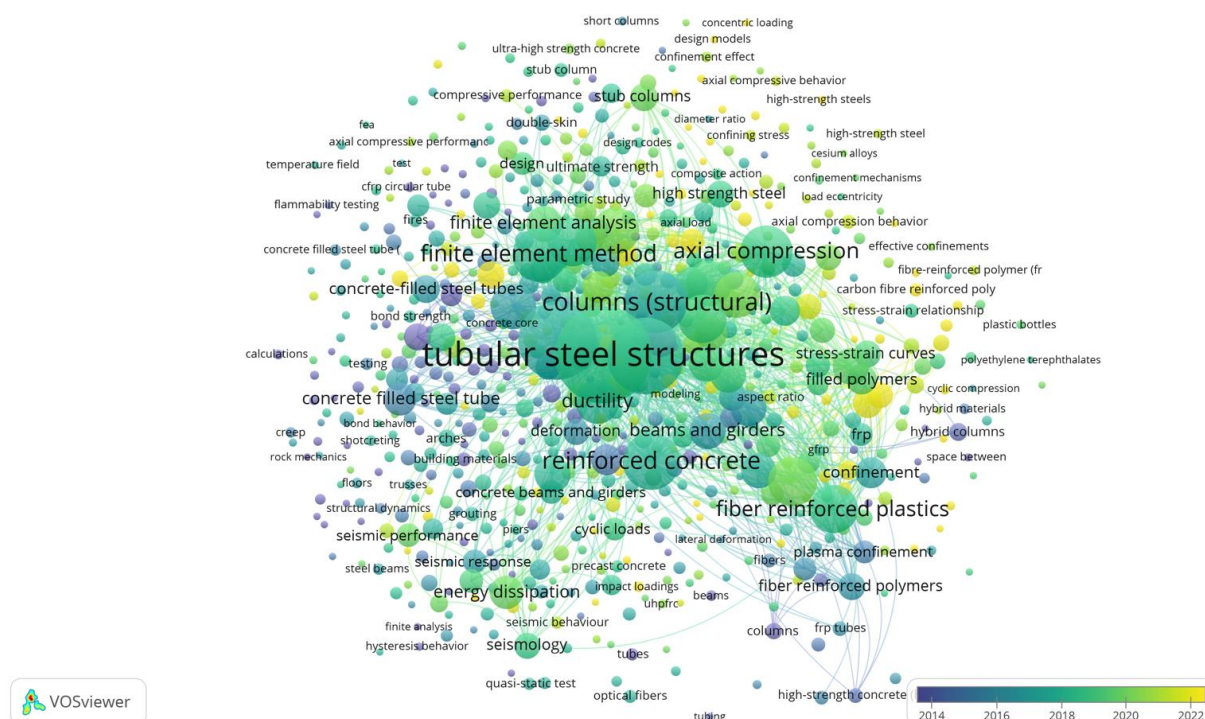


Figure 1. Visualize the incidence and relationship of individual keywords “Concrete + inner + tube” (accessed on 26.05.2025).

2. Experimental programme

2.1. The specimens

As part of the presented research, twenty-seven samples were prepared, including three reference samples (plain concrete) and twenty-four experimental samples (composites) with various configurations of external and internal composite reinforcement made of CFRP mats. The nomenclature of the samples is explained in Figure 2.

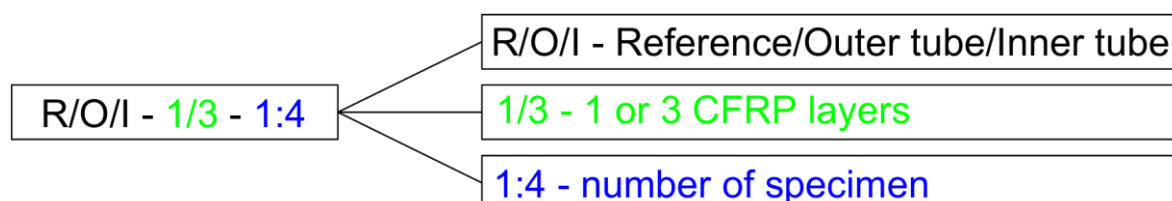


Figure 2. Determination of samples in the study.

In this study, seven types of cylindrical specimens were prepared. The reference specimens (labelled “R”) had a height of 300 mm and a diameter of 150 mm. The remaining specimens, which were subjected to various CFRP strengthening configurations, had dimensions of 160 mm in diameter and 320 mm in height. Specimens

with external CFRP strengthening (applied in either one or three layers) were marked with the prefix “O”. Those reinforced with internal perforated CFRP were labelled with prefix “I”, and specimens combining both internal and external reinforcement were marked as O-1/I-1 and O-3/I-1. The configuration and dimensions of all prepared specimens are shown in Figure 3.

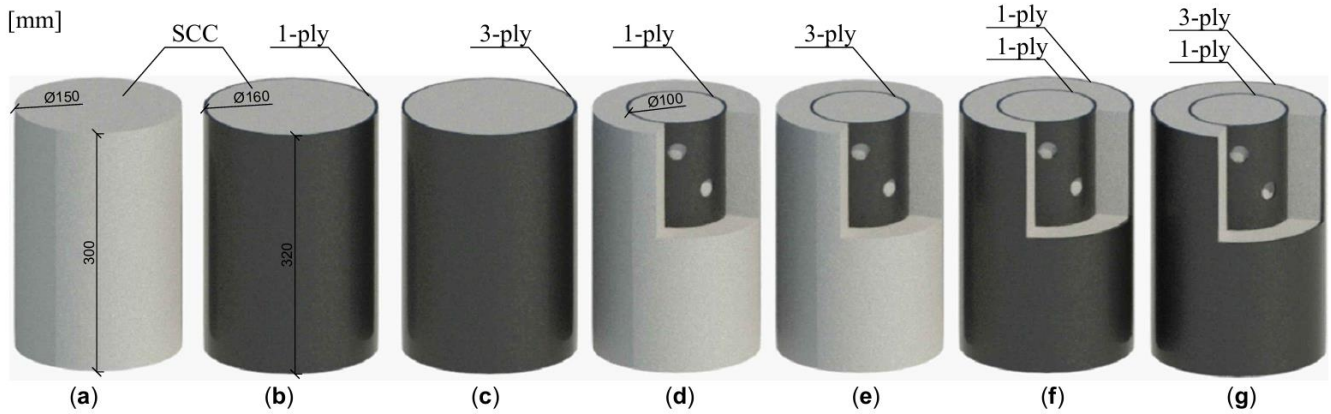


Figure 3. Configuration of tested cylindrical specimens: (a) Reference (R); (b) O-1; (c) O-3; (d) I-1; (e) I-3; (f) O-1/I-1; (g) O-3/I-1.

2.2. Material properties

Within the conducted research, materials used for preparing the concrete mix and materials for external and internal composite reinforcement can be distinguished. For the concrete mix, CEM II/B-M(V-LL) 32.5R cement was selected, meeting the requirements the European standard EN 197-1 [49]. The choice of cement was determined by its ability to minimize cracking in hardened concrete and facilitate the rapid early development of strength. Table 3 provides the basic parameters of the cement used.

Table 3. The parameters of the cement [50].

Cement type	Content	Compression strength (2 days) [MPa]	Compression strength (28 days) [MPa]	Begin of curing time [min]
CEM II/B-M(V-LL) 32.5R	Calcium Oxide (CaO) – 53.6 % Silicon Dioxide (SiO ₂) – 23.3 % Aluminum Oxide (Al ₂ O ₃) – 8.0 % Other – 15.1%	20.54	46.60	204

In the developed concrete mixture, basalt powder was used as a filler. A horticultural powder with the following chemical composition was used [63]: silicon oxide (SiO₂) – 50 %, aluminum oxide (Al₂O₃) – 20 %, iron oxide (Fe₂O₃) – 5–10 %, and secondary components constituting 25–20 % of the total mass. It is worth emphasizing that even a small content of this powder in the mixture can contribute to an increase in compressive strength [51] and an improvement in abrasion resistance.

Quartz sand with a grain size of 0–2 mm and diabase aggregate with a diameter of 2–5 mm were used as aggregates. The diabase aggregate was classified with a grain size of

G_C 90/10, an absorbability of WA₂₄₁ (water absorption lower than 1.0% of weight), and an abrasion resistance of M_{DE}25. The moisture content of both coarse and fine aggregates was taken into account when calculating the W/C ratio.

The matrix of the composite reinforcement was formulated using the epoxy resin Epidian 653 and hardener 635, from Ciech S.A. The epoxy resin exhibited a density of 1.13 g/cm³, a gel time of 40 minutes for a 100 g sample at 20 °C, a tensile strength of 30 MPa, and a bending strength of 70 MPa [52]. To enhance the viscosity of the resin, a thixotropic additive was incorporated into the mixture at 4 % of the resin's mass. The hardener-to-resin ratio was maintained at 33 g per 100 g of resin.

The composite's external and internal reinforcement consisted of unidirectional carbon fiber fabric. The SikaWrap-300C carbon fiber fabric, utilized in this study, had a density of 1.82 g/cm³, a thickness of 0.167 mm, a tensile strength of 4000 MPa, a modulus of elasticity of 230 GPa, and a longitudinal expansion at break of 1.7 % [53].

2.3. Preparation of specimens

The external and internal tubes, in cylindrical shape, were made using the wet layup method. Pre-greased and protected PVC pipes, with diameters of ø160 mm and ø100 mm, were wrapped with impregnated CFRP mats. Each pipe included a mat overlap covering 30 % of its circumference. The lamination procedure was carried out under dry conditions at a temperature of 20 °C. To eliminate air bubbles, each layer was finished by using a ribbed roller for air removal shown in Figure 4.



Figure 4. Application and venting of the soaked CFRP.

The prepared composite reinforcement was left at a temperature of $20\text{ }^{\circ}\text{C} \pm 2\text{ }^{\circ}\text{C}$ for 7 days to achieve full mechanical strength. Following the curing process, the PVC pipe molds were removed, and the pipes were halved to obtain samples with a height of approximately 30 cm. Consequently, the samples featured the cross-sectional geometries illustrated in Figure 5.

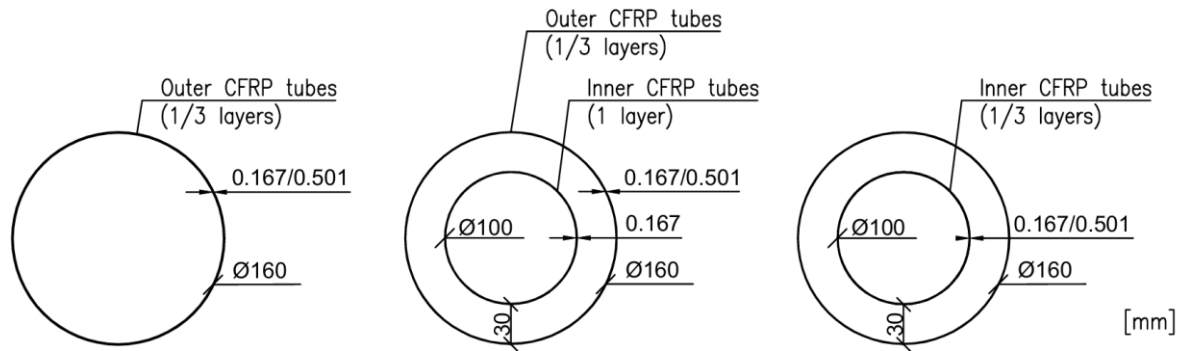


Figure 5. Scheme for laying CFRP pipes.

To ensure effective interaction between the external concrete jacket and the internal concrete core, apertures of $\varnothing 30$ mm in diameter were introduced in the inner CFRP tubes. The apertures were made at four different heights, evenly distributed along the length of the pipes. The total area of the apertures corresponds to 2.8 % of the total surface area of the internal reinforcement. A detailed scheme of the perforations is shown in Figure 6 and the finished perforations are shown in Figure 7.

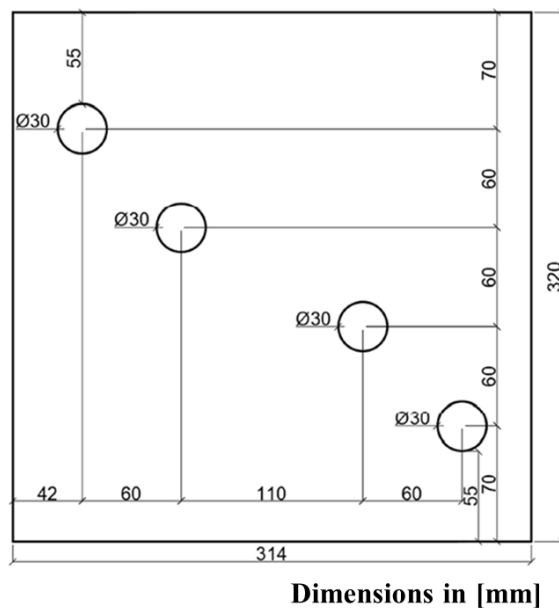


Figure 6. Scheme of perforations in the inner CFRP tube.



Figure 7. Example of perforation inner CFRP tube.

The next step was the preparation of a self-compacting concrete mix (SCC). For research purposes, a composition was developed with the following components:

Table 4. Summary of components of the designed (SCC) mix.

Material	Cement [kg/m ³]	Quartz sand [kg/m ³]	Diabase aggregate [kg/m ³]	Water [kg/m ³]	Basalt powder [kg/m ³]	Superplasticizer [kg/m ³]
SCC	475	850	1000	194	125	9.5

Prior to preparing the SCC mix, moisture tests on the aggregates were conducted to determine the final W/C ratio. The tests were performed using an OHAUS MB35 moisture analyzer, revealing a moisture content of 3.3 % for quartz sand and 1.4 % for diabase aggregate, resulting in a W/C ratio of 0.43.

The mixed components were mixed in a laboratory mixer. The designed mix aimed to produce self-compacting concrete (SCC). To confirm its properties, a flow/cone slump test was performed (Figure 8) in accordance with the EN 12350-8 standard [54]. The results classified the mix as SF2 (consistency class) and VS2 (viscosity class), as shown in Table 5.

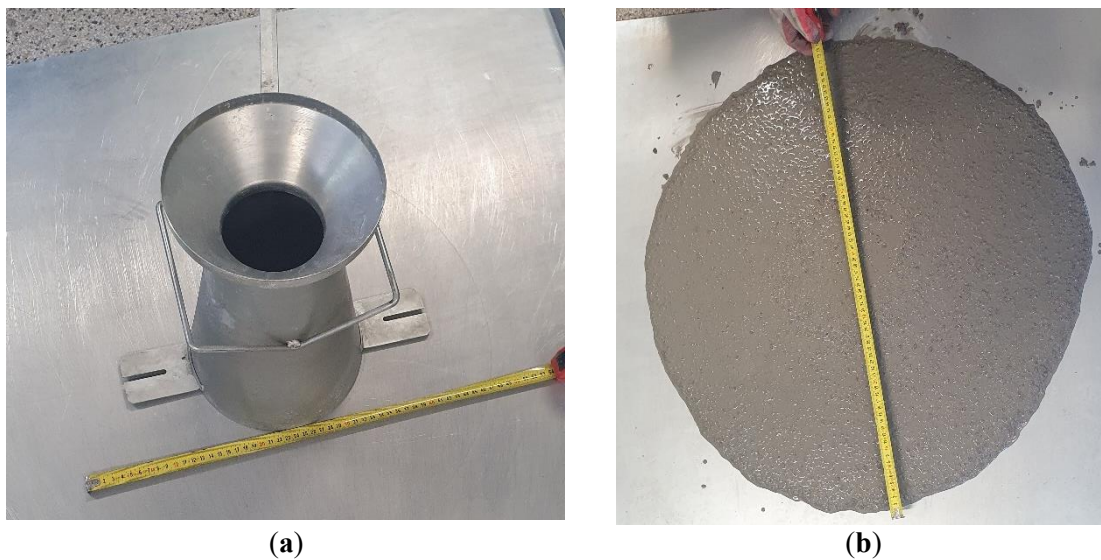


Figure 8. Test of consistency class and viscosity class of SCC (a) experimental stand; (b) slump flow test.

Table 5. Test results of concrete mixture by cone spreading method according to EN 12350-8 [54].

	Results	EN 12350-8		Class of prepared SCC according to EN 12350-8
		Class	Boundary values	
Viscosity Class	5.0 s	VS1	< 2.0 s	VS2
		VS2	≥ 2.0 s	
Consistency Class	730 mm	SF1	550-650 mm	SF2
		SF2	660-750 mm	
		SF3	760-850 mm	

After conducting the flow tests, the forms were filled with the prepared concrete mix and allowed to cure for 28 days. To minimize shrinkage, the samples were kept in the molds and covered with foil throughout the curing period. Before testing, contaminants and residual mix were carefully removed from the sample surfaces, and the compression surfaces were polished to ensure uniformity.

2.4. Instrumentation and loading conditions

A series of tests were conducted on the prepared samples, including the determination of compressive strength through destructive testing using a hydraulic press (Figure 9) conducted in accordance with the EN 12390-3:2019 standard [55]. As specified by the standard, the specimens were loaded continuously at a constant rate of 0.2 MPa/s to 1.0 MPa/s until failure.

The results obtained from the initial test were used for subsequent studies to determine the secant modulus of hardened concrete. These tests were carried out on a compression testing machine following Method B of the EN 12390-13:2014 standard [56]. The procedure involved three loading cycles at a rate ranging from 0.2 MPa/s to 0.6 MPa/s. For strain measurement, at least two sensors were symmetrically positioned concerning the axis of the sample (Figure 10). The initial stress value and the upper stress limit were determined based on the results of the first uniaxial compressive strength test, with the specific values detailed in Table 6. The upper stress limit σ_a and pre-stress values σ_p were determined as fractions of the compressive strength f_{cm} in accordance with EN 12390-13:2014 [56], where the upper stress was limited to $1/3$ of f_{cm} , and the compressive strength itself was estimated within the range $0.5 \leq \sigma_a \leq (0.1 \div 0.15) f_{cm}$, representing the upper and pre-stress levels, respectively.



Figure 9. Compressive strength test.



Figure 10. Testing the modulus of elasticity of samples.

Table 6. Upper stress limit and pre-stress value for testing the modulus of elasticity.

Specimen name	Compressive strength f_{cm} [MPa]	Upper stress limit σ_a [MPa]	Pre-stress value σ_p [MPa]
R - 1	51.1	$0.5 \div 7.67$	17.03
I-1 - 1	54.8	$0.5 \div 8.22$	18.27
I-3 - 1	51.9	$0.5 \div 7.79$	17.30
O-1 - 1	62.2	$0.5 \div 9.33$	20.73
O-3 - 1	99.0	$0.5 \div 14.85$	33.00
O-1/I-1 - 1	76.2	$0.5 \div 11.43$	25.40

O-3/I-1 - 1	103.0	0.5÷15.45	34.33
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According to the EN 12390-13:2014 standard [56], during the application of the load in successive test cycles, measurements of maximum deformations must be recorded. The comparison of deformation values obtained during the second and third cycles should show a difference of no more than 20 % for the initial reference values and no more than 10 % for subsequent measurements. If these thresholds are exceeded, the discrepancy must be documented in the test report.

3. Results and discussion

The analysis and discussion of the results obtained from the conducted research were structured according to the tested strength parameters. Section 3.1 provides a detailed account of the uniaxial compression test results, including the measured vertical and lateral deformations. Section 3.2 focuses on the outcomes of the modulus of elasticity test. Lastly, Section 3.3 offers an in-depth analysis of the structural behavior of the tested elements, addressing the concrete structure, damage models, and the impact of perforations on the location of damage initiation.

3.1. Stress-strain behavior

The results of the stress-strain behavior are summarized in a graph (Figure 11) and a Table 7. By analyzing the stress-strain diagrams for individual specimens (Figure 12), three critical points were identified, with corresponding stress and strain values recorded. These points are: the destruction of the outer jacket ($\varepsilon_{CC1}, f_{CC1}$), the point of stabilization of the sample after the destruction of the jacket ($\varepsilon_{CC2}, f_{CC2}$), and the complete destruction of the specimen ($\varepsilon_{CCu}, f_{CCu}$). Values marked with an asterisk ($\varepsilon_{CCu}, f_{CCu}$) correspond to the peak stress and strain at which failure of the CFRP-confined concrete specimens occurred. In these cases, no post-peak strain hardening was observed, and the specimen failed immediately after reaching the maximum stress. In contrast, the remaining specimens exhibited post-peak behavior characterized by a drop in stress followed by a subsequent increase due to strain hardening.

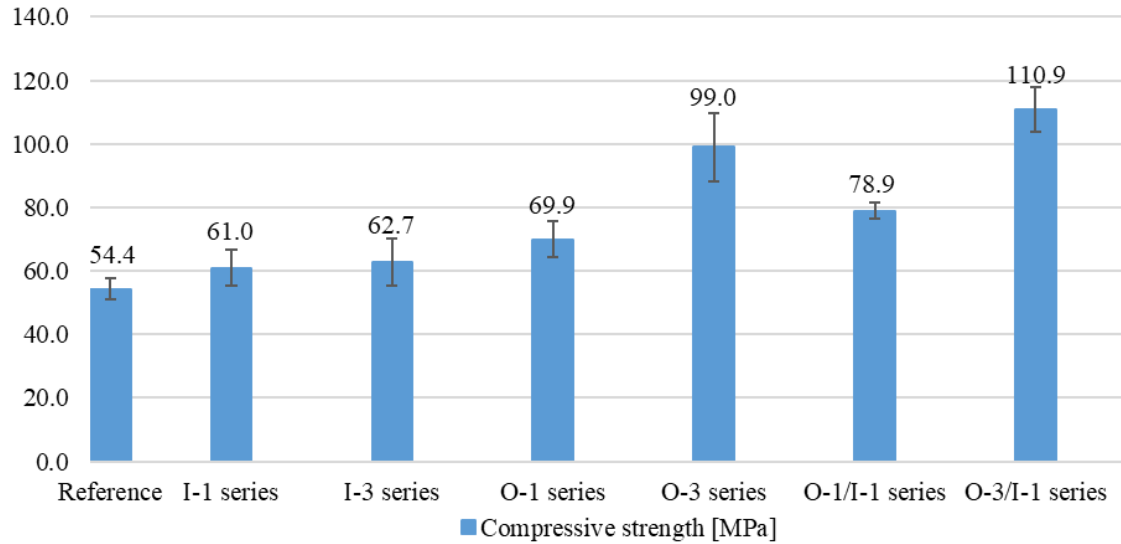


Figure 11. Resulting ultimate stress values for individual samples.

Table 7. Experimental values obtained from testing the specimens.

Specimen	f_{cc1} [MPa]	f_{cc2} [MPa]	f_{ccu} [MPa]	ϵ_{cc1} [-]	ϵ_{cc2} [-]	ϵ_{ccu} [-]
R-1	-	-	51.1	-	-	4.71
R-2	-	-	57.9	-	-	4.70
R-3	-	-	54.1	-	-	4.78
I-1-1	54.8	29.1	23.7	3.99	5.10	10.08
I-1-2	68.7	37.5	37.4	4.30	6.23	8.14
I-1-3	59.3	33.6	31.9	4.21	6.13	7.31
I-1-4	61.2	36.7	34.7	4.60	6.47	8.36
I-3-1	51.9	30.2	42.6	4.20	5.03	12.01
I-3-2	67.1	40.1	44.1	4.92	6.63	11.87
I-3-3	67.3	42.7	44.9	5.12	6.86	10.39
I-3-4	64.6	41.5	40.8	4.32	5.73	11.26
O-1-1	62.2	59.2	59.2	7.91	-	10.16
O-1-2	75.8	71.6	71.6	6.80	-	10.92
O-1-3	70.1	68.3	68.3	5.81	-	10.22
O-1-4	71.5	52.9	52.9	6.28	-	9.11
O-3-1	-	-	99.0*	14.55	-	14.55*
O-3-2	-	-	113.0*	18.84	-	18.64*
O-3-3	96.9	93.0	99.2	17.90	10.65	17.83
O-3-4	87.0	74.3	83.8	9.05	8.38	17.40
O-1/I-1-1	-	-	76.2*	-	-	9.15*
O-1/I-1-2	-	-	78.9*	-	-	10.94*
O-1/I-1-3	-	-	78.3*	-	-	9.21*
O-1/I-1-4	-	-	82.2*	-	-	9.89*
O-3/I-1-1	99.4	94.1	103.0	12.81	15.81	22.30
O-3/I-1-2	119.9	113.2	116.3	17.07	18.96	21.50
O-3/I-1-3	-	-	108.9*	-	-	18.69*
O-3/I-1-4	108.6	-	111.6	14.50	15.41	19.83

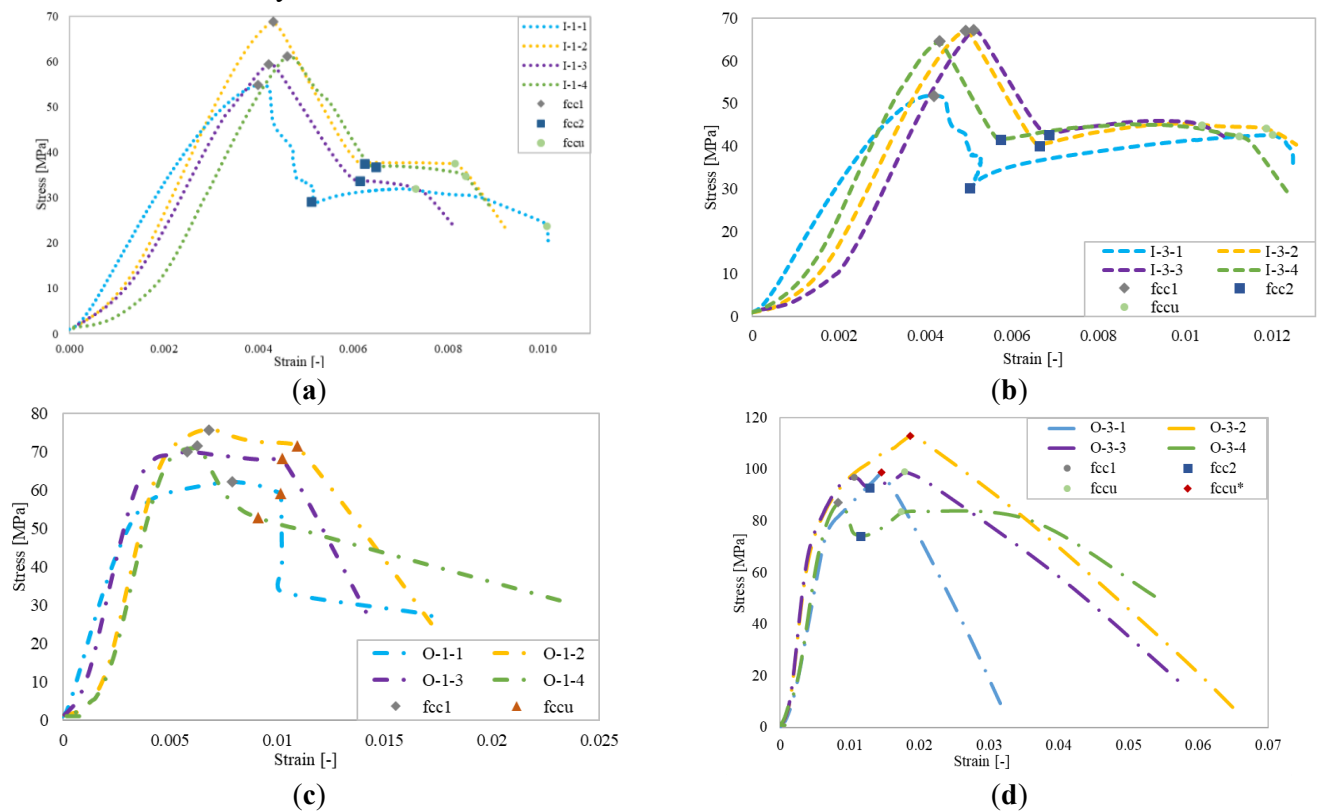
* Denotes specimens that failed immediately after reaching peak stress, with no post-peak hardening observed.

The highest increase in average compressive strength and ultimate strain was observed in the specimens with external strengthening (O-3 and O-3/I-1), confirming the

high effectiveness of lateral confinement of the concrete core when multi-layer external CFRP strengthening is applied. An improvement in mechanical performance was also recorded for the O-1/I-1 series, where, despite the use of only a single layer, an increase was noted in comparison to both the reference specimen and the I-series (by 45% and 13%, respectively). The use of a perforated CFRP tube in series I-1 and I-3 enabled a partial confinement effect from the inside, which also contributed to increased deformability of the specimens.

The combination of strengthening types (series O-1/I-1 and O-3/I-1) proved to be the most effective – specimens achieved the highest average values in terms of both stress and strain. This indicates a synergistic effect between internal and external strengthening, which enabled a more uniform stress distribution and delayed the onset of failure.

Failure in the O-1/I-1 series occurred immediately after reaching peak stress, without the occurrence of post-peak strain hardening. This behavior indicates a sudden loss of load-bearing capacity and reflects a brittle failure mechanism. Although these specimens exhibited relatively high compressive strength, the inability to undergo further deformation beyond the maximum load may suggest a local exceedance of the deformation capacity of the confinement system. Despite the high confinement level, the system could not sustain additional strain, which may have resulted from excessive stiffness or insufficient interaction at the interface between the internal and external reinforcement layers.



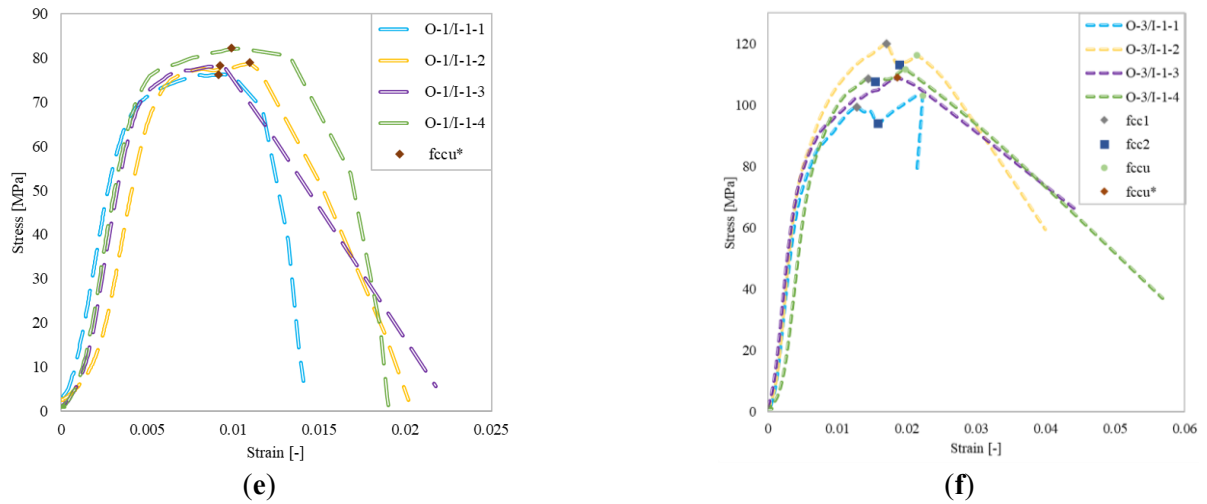


Figure 12. Stress-strain diagrams for specimens: (a) I-1; (b) I-3; (c) O-1; (d) O-3; (e) O-1/I-1; (f) O-3/I-1.

3.2. Secant modulus of elasticity

The secant modulus of elasticity was determined according to the procedure described in Section 2.4, following the EN 12390-13:2014 standard. It is important to note that this standard is intended for unreinforced concrete, and no equivalent standard currently exists for concrete confined or strengthened with composite materials such as CFRP. For this reason, the adopted procedure was applied to all series, including those with internal and external CFRP strengthening, which may introduce some methodological limitations.

The modulus values were calculated between stress levels corresponding to 5 % and 30 % of the average compressive strength of each series. The results presented in Table 8 show a significant increase in the average secant modulus for specimens with external strengthening (O-series), particularly in the O-1 and O-1/I-1 series. In contrast, samples with internal strengthening (I-series) exhibited slightly lower stiffness compared to the reference series.

These results are consistent with the confinement mechanism provided by external CFRP jackets, which effectively restrict lateral expansion of the concrete core and improve its initial stiffness under axial loading. Internal reinforcement, while beneficial in enhancing ductility and post-peak behavior, appears to be less effective in the initial loading phase, which explains the observed decrease in modulus values. A detailed breakdown of individual test results and calculations for each specimen series is provided in Annex A.

Table 8. Mean values of the secant modulus of elasticity for each specimen series.

Sample series	R	I-1	I-3	O-1	O-3	O-1/I-1	O-3/I-1
Secant modulus [GPa]	35.8	35.7	33.2	42.6	39.8	42.1	38.9
Standard deviation	1.20	2.78	2.60	3.92	1.67	2.40	1.05

3.3. Failure characteristics of the specimens

The reference specimens exhibited classical brittle behavior typical of unreinforced concrete. After a linear-elastic phase, failure occurred suddenly, without visible warning signs (Figure 13). Post-test examination confirmed proper preparation of the concrete mix, no pores or signs of segregation were observed. The specimens retained a characteristic conical shape, indicating a uniform distribution of compressive stresses (Figure 14(a)).

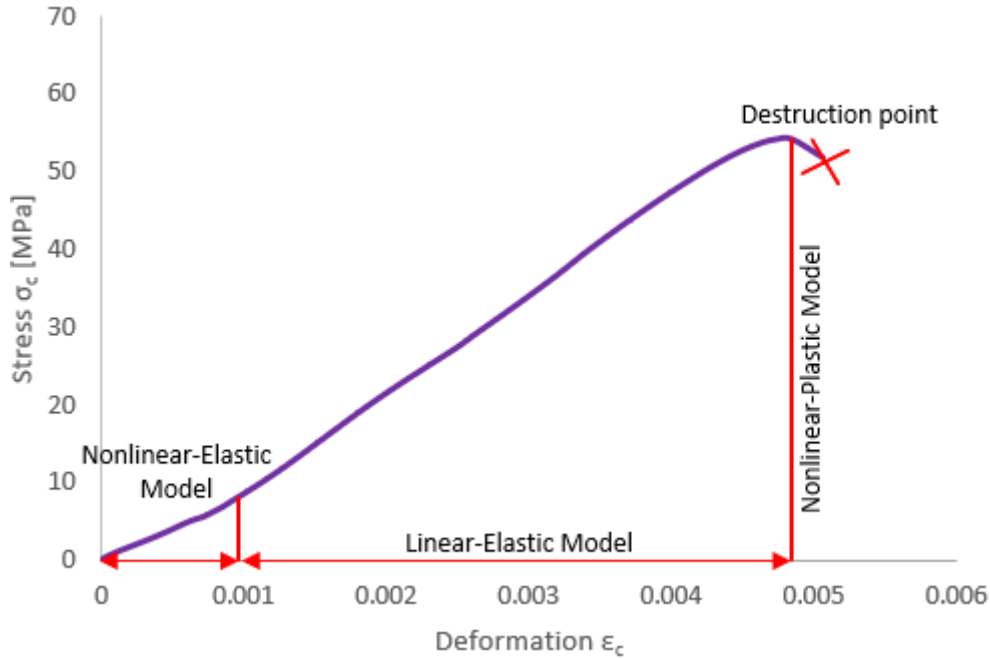


Figure 13. Stress-strain diagram of series R reference specimens.

In the externally strengthened specimens (O-1 and O-3 series), a noticeable increase in ultimate strain and a change in failure mechanism were observed. The stress-strain curves indicated partial material softening beyond the elastic limit, particularly in the O-1 series, where curve flattening was visible (Figure 15(a)). In the O-3 series, post-peak strengthening occurred, after an initial drop in load-bearing capacity, a secondary increase followed, demonstrating the active contribution of the external CFRP strengthening (Figure 15(b)). Failure was sudden but did not result in complete disintegration. A concentrated failure zone was visible in the middle of the specimens (Figure 14(b)).

The internally strengthened specimens (I-1 and I-3 series) initially exhibited behavior similar to the reference group, with a typical elastic response (Figure 15(c)). After the failure of the concrete shell, the CFRP-confined inner core enabled further load transfer, contributing to increased ultimate strain. The failure process was gradual, and post-test analysis confirmed proper concrete compaction. Localized damage was observed near the preformed $\varnothing 30$ mm holes (Figure 14(c)), which may have acted as stress concentrators. At the same time, surface observations indicated uneven contact between the concrete and internal CFRP tube, potentially limiting full material interaction.

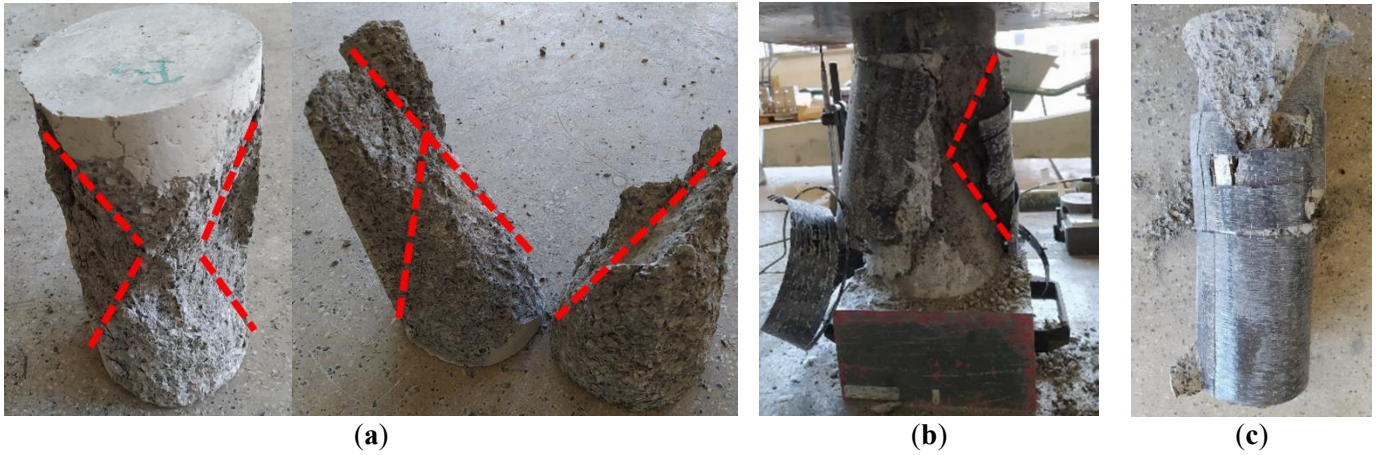


Figure 14. Representative destruction of samples: (a) Reference; (b) O-1-2; (c) I-3-4.

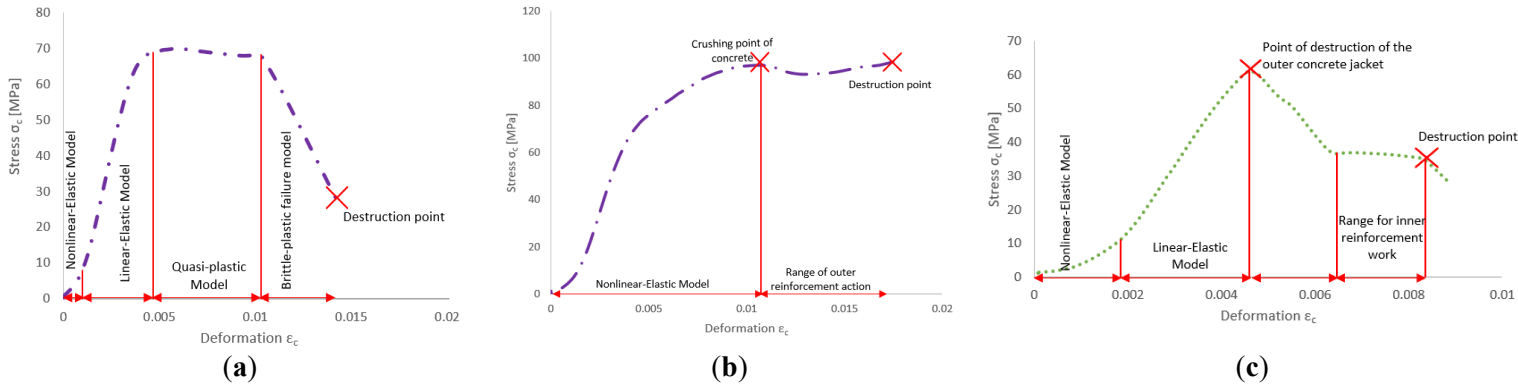


Figure 15. Stress-strain diagram of specimens: (a) O-1; (b) O-3; (c) I-1.

In the hybrid specimens (O-1/I-1 and O-3/I-1), the highest compressive strengths and ultimate strain values were recorded. The stress-strain diagrams (Figure 16) show characteristic flattening after the elastic stage and stabilization of axial force with increasing displacements. This behavior reflects the effective engagement of both internal and external reinforcement. The failure occurred suddenly, without prior visible indicators. The residual strength was lower than in the I-series specimens, which can be attributed to the tighter confinement effect of the external reinforcement. As a result, the concrete core failed, and the internal reinforcement was damaged before the external jacket was compromised, suggesting differences in stiffness and partial interaction between the confinement components (Figure 17).

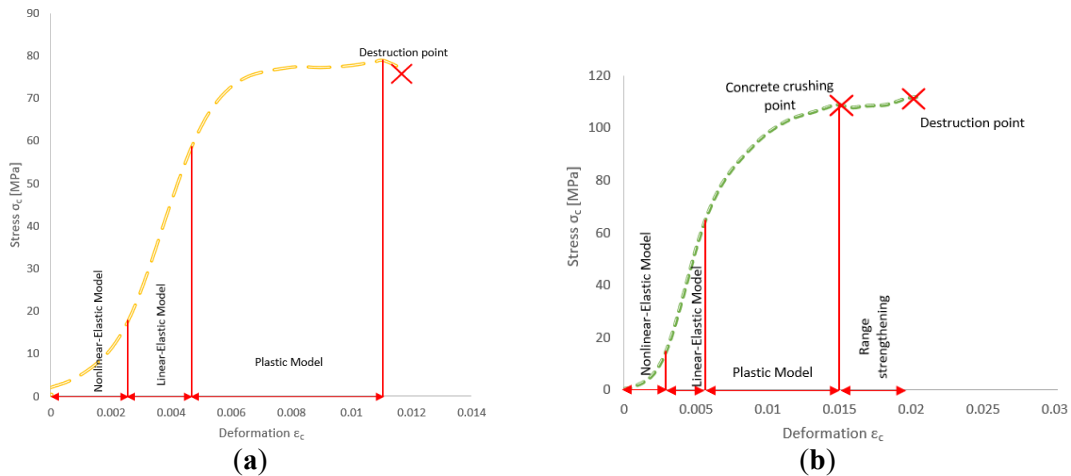


Figure 16. Stress-strain diagram of specimens with outer reinforcement (a) Series O-1/I-1; (b) Series O-3/I-1.

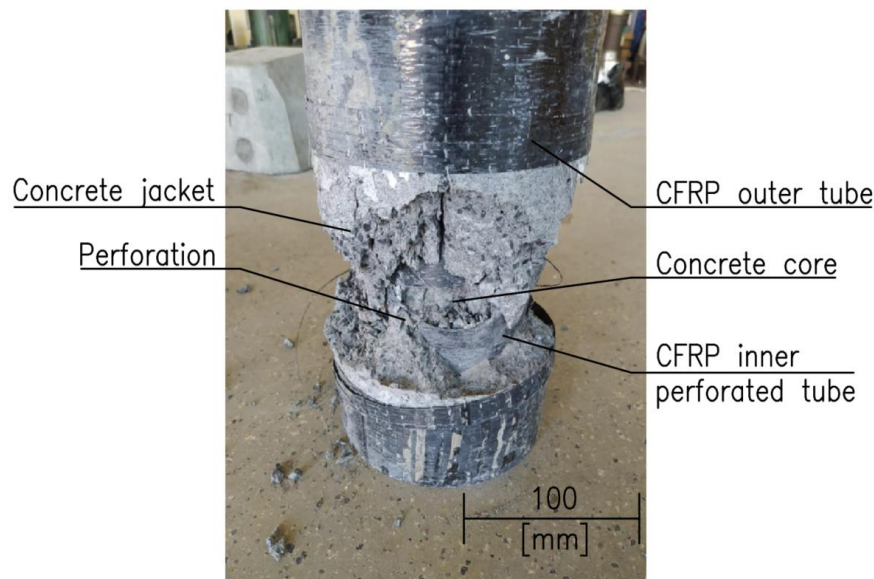


Figure 17. Destroyed sample O-1/I-1-3.

3.4. Comparison of results with main analytical models

Following experimental testing, a computational analysis was conducted to predict the strength of the tested elements. Although no unique formulas or standards exist for the tested composite elements, calculations were conducted for comparison purposes using the current standards: European [57], American [58,59], Brazilian [60] as well as formulas proposed by Kang I.-K. et. al. [61]. In the formulas, the nomenclature was adjusted for consistency, but the essence of the formulas remained unchanged.

Table 9 summarizes the formulas for predicting the axial load capacity in composite and hybrid structural columns, as outlined in various design standards and research publications. Eurocode 4 incorporates contribution of both steel and concrete components, accounting for parameters such as the relative slenderness ratio and the interaction between internal and external steel elements. The ACI 318-11 focuses on concrete structures, accounting for the reinforcement steel and concrete interaction coefficients to predict axial capacity. The ANSI/AISC 360-16 standard, specific to steel structures, includes parameters for effective length, slenderness ratio, and steel material properties. Similarly, the NBR 8800:2008 standard for steel structures addresses the influence of material properties and cross-sectional geometry, incorporating effective stiffness parameters to compute load capacity.

In contrast, research by Kang [61] evaluates the compressive strength of hybrid concrete-filled FRP tubes, considering the contributions of various material components. It incorporates two key terms: the compressive strength of concrete confined by filament winding fiber-reinforced polymer and the additional contribution from pultruded fiber-reinforced polymer.

It is noteworthy that while these standards were initially developed earlier, the listed dates reflect their most recent updates: Eurocode 4 in 2004, ACI 318-11 in 2011, ANSI/AISC 360-16 in 2016, and NBR 8800:2008 in 2008. Kang I.-K. et. al. research represents the latest advancement in this field, published in 2021. These updates highlight the continuous evolution of structural design methodologies to address contemporary engineering challenges and innovations.

A comparison of the experimental strength results with the values derived from the adopted formulas is summarized in Table 10.

Table 9. Formulas for strength prediction.

Source	Application of formula	Formula
EC 4 [57]	Composite structures (steel and concrete)	$P_{EC4} = \eta_a A_{CFRPo} f_{CFRP} + A_{co} f_{cc} \left(1 + \eta_c \frac{t_o}{D_o} \frac{f_{CFRP}}{f_{cc}} \right) + \eta_a A_{CFRPi} f_{CFRP} + A_{ci} f_{cc} \left(1 + \eta_c \frac{t_i}{D_i} \frac{f_{CFRP}}{f_{cc}} \right)$ $\eta_a = 0.25(3 + 2\bar{\lambda}) \quad (\eta_a \leq 1.0)$ $\eta_c = 4.9 - 18.5\bar{\lambda} + 17\bar{\lambda}^2 \quad (\eta_c > 0)$ where the relative slenderness ratio is $\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}}$
ACI 318-11 [58]	Concrete structures	$P_{ACI} = A_{CFRPo} f_{CFRP} + 0.85 A_{co} f_{cc} + A_{CFRPi} f_{CFRP} + 0.85 A_{ci} f_{cc}$
ANSI/AISC 360-16 [59]	Steel structures	$P_{AISC} = \begin{cases} P_{no} [0.658^{(P_{no}/P_e)}] & \text{for } P_e \geq 0.44 P_{no} \\ 0.877 P_e & \text{for } P_e < 0.44 P_{no} \end{cases}$ $P_{no} = A_{CFRPo} f_{CFRP} + 0.95 A_{co} f_{cc} + A_{CFRPi} f_{CFRP} + 0.95 A_{ci} f_{cc}$ $P_e = \frac{\pi^2}{(L_c)^2} EI_{eff}$ $EI_{eff} = E_f I_{fo} + E_f I_{fi} + C_4 E_c I_{co} + C_4 E_c I_{ci}$ $C_4 = 0.6 + 2 \left(\frac{A_{CFRPi}}{A_{CFRPi} + A_c} \right) \leq 0.9$
NBR 8800:2008 [60]	Steel structures	$N_{Rd} = \chi * \left(\frac{f_{CFRP} * A_{CFRP}}{\gamma_a} + \alpha \frac{f_{ck} * A_c}{\gamma_c} \right)$
Kang, I.-K.; Kim, S.-H. [61]	Hybrid concrete-filled fiber-reinforced plastic tubes confined by filament winding	$P_{Hybrid-CFFT} = f_{cc} A_{co} + 1.103 f_{CFRP} A_{CFRP}$

Subscript “i” refers to internal elements
Subscript “o” refers to external elements.

Table 10. Prediction results.

Specimen	P_{exp} [MPa]	P_{EC4} [MPa]	$\frac{P_{EC4}}{P_{exp}}$ [-]	P_{ACI} [MPa]	$\frac{P_{ACI}}{P_{exp}}$ [-]	P_{AISC} [MPa]	$\frac{P_{AISC}}{P_{exp}}$ [-]	N_{Rd} [MPa]	$\frac{N_{Rd}}{P_{exp}}$ [-]	$P_{Hybrid-CFFT}$ [MPa]	$\frac{P_{H-CFFT}}{P_{exp}}$ [-]
I-1-1	1448.4	1290.6	0.89	979.1	0.68	1074.8	0.74	789.4	0.55	1144.2	0.79
I-1-2	1624.4	1264.9	0.78	956.7	0.59	1049.7	0.65	771.5	0.47	1117.8	0.69
I-1-3	1523.9	1290.6	0.85	979.1	0.64	1074.8	0.71	789.4	0.52	1144.2	0.75
I-1-4	1550.7	1252.2	0.81	945.6	0.61	1037.3	0.67	762.6	0.49	1104.8	0.71

I-3-1	1983.0	1793.6	0.90	1201.0	0.61	1295.5	0.65	988.0	0.50	1389.8	0.70
I-3-2	2001.3	1768.6	0.88	1178.4	0.59	1270.4	0.63	970.1	0.48	1363.2	0.68
I-3-3	2082.1	1781.1	0.86	1189.7	0.57	1282.5	0.62	978.8	0.47	1376.5	0.66
I-3-4	1942.4	1756.3	0.90	1167.3	0.60	1257.4	0.65	960.9	0.49	1350.1	0.70
O-1-1	1297.6	1554.1	1.20	1137.8	0.88	1242.3	0.96	922.2	0.71	1326.0	1.02
O-1-2	1504.9	1491.1	0.99	1086.7	0.72	1186.9	0.79	881.6	0.59	1266.3	0.84
O-1-3	1426.1	1522.5	1.07	1112.1	0.78	1215.0	0.85	902.1	0.63	1296.0	0.91
O-1-4	1473.1	1538.3	1.04	1124.9	0.76	1228.2	0.83	911.9	0.62	1310.9	0.89
O-3-1	2065.2	2313.0	1.12	1480.6	0.72	1581.7	0.77	1231.1	0.60	1704.1	0.83
O-3-2	2385.9	2332.5	0.98	1495.6	0.63	1598.7	0.67	1243.8	0.52	1721.6	0.72
O-3-3	2068.3	2313.0	1.12	1480.6	0.72	1581.5	0.76	1231.0	0.60	1704.1	0.82
O-3-4	1837.1	2332.5	1.27	1495.6	0.81	1597.7	0.87	1243.2	0.68	1721.6	0.94
O-1/I-1-1	1590.0	1698.0	1.07	1242.8	0.78	1347.1	0.85	1017.3	0.64	1441.8	0.91
O-1/I-1-2	1666.3	1714.5	1.03	1255.8	0.75	1361.7	0.82	1027.9	0.62	1457.0	0.87
O-1/I-1-3	1652.6	1714.5	1.04	1255.8	0.76	1361.2	0.82	1027.6	0.62	1457.0	0.88
O-1/I-1-4	1714.8	1698.0	0.99	1242.8	0.72	1347.2	0.79	1017.4	0.59	1441.8	0.84
O-3/I-1-1	2184.1	2291.8	1.05	1600.6	0.73	1702.1	0.78	1338.1	0.61	1837.4	0.84
O-3/I-1-2	2531.7	2291.8	0.91	1600.6	0.63	1702.7	0.67	1338.5	0.53	1837.4	0.73
O-3/I-1-3	2300.1	2291.8	1.00	1600.6	0.70	1702.2	0.74	1338.2	0.58	1837.4	0.80
O-3/I-1-4	2357.3	2291.8	0.97	1600.6	0.68	1702.4	0.72	1338.3	0.57	1837.4	0.78

The use of the EC4 standard leads to results on the safe side for specimens with internal reinforcement, while for other specimens the values obtained could potentially be underestimated. The results obtained using the ACI 318-11 [58] standard for all samples are on the safe side, showing a minimum reserve margin of at least 12 %. Similarly, using formulas from the AISC, NBR 8800 standards and $P_{\text{Hybrid-CFFT}}$, also leads to values on the safe side. The most similar strength predictions to experimental results were obtained using the formula proposed by Kang, I.-K. et. al [61].

4. Conclusions

The study was conducted to evaluate the effect of diverse types of composite reinforcement on the strength and behavior of concrete specimens in uniaxial compression. The results of the experiments indicate that the reinforcement solutions used have different effects on the mechanical properties of concrete, such as compressive strength, elastic modulus, and deformability. The main conclusions of the analysis are presented below:

- The O-3/I-1 hybrid-reinforced specimens showed the greatest increase in compressive strength, a 104 % increase over the reference specimens. Additionally, hybrid-reinforced specimens (O-1/I-1, O-3/I-1) had significantly higher deformability than specimens reinforced only internally. The deformability increased by 15 % and 50 %, respectively. The behavior of specimens with hybrid reinforcement is predominantly plastic. Furthermore, for specimens from the O-3/I-1 series, a strengthening phase is observed prior to failure.

- For O-3 specimens, the strength has increased by as much as 82 % compared to the reference specimens. What is more, the use of external reinforcement had a significant effect on the increase of secant modulus. External reinforcement exerts a greater confining effect, thereby dominating the internal reinforcement. This phenomenon is reflected in the behavior of samples from Series O, where the quasi-plastic phase appears prior to the failure of the sample.
- The use of CFRP inner perforated tube contributed to a 12-15 % increase in compressive strength. Significantly, the concrete core confined by the perforated composite shell retained its load-carrying capacity at high strains of 8.5 ‰ for specimens I-1 and 11.4 ‰ for specimens I-3. The experimental results from Series I demonstrate a distinct behavior characterized by elastic performance up to the destruction of the outer concrete jacket. After the external concrete layer fails, the samples demonstrate a quasi-plastic behavior, which can be interpreted as the work of the CFRP tube maintaining the integrity of the concrete core.
- All tested samples showed the correct failure mechanism in the uniaxial compression test. This demonstrates the correct design of the concrete mix and the high quality of the concrete manufacturing process.
- No adhesion between the concrete and the applied composite reinforcement was observed. In specimens with internal reinforcement, damage initiation was observed at the locations of holes made in the reinforcement.
- Based on the strength prediction formulas analyzed, the use of the formula developed by Kang and co-authors allows the results to be most like the empirical data obtained in the study.

The results of the study confirm the potential of using innovative reinforcement in improving the mechanical properties of concrete, while indicating the need for further research to optimize the design and technology of composite reinforcement.

5. Perspectives and futures

In the authors' view, the method of strengthening concrete elements presented in the article shows potential in the context of improving the load-bearing capacity of structures. The applied technical solutions can find wide application in engineering practice, especially in situations where it is necessary to optimize the design of new structures, such as bridge pillars and columns of tall buildings. The analysis of the results of the study suggests that the method can be a significant step forward in developing more efficient solutions in construction.

In further research steps, it is planned to expand the scope of the experiments by introducing additional variables, such as perforation spacing in the internal reinforcement and the slenderness of the reinforced elements. The inclusion of these parameters will provide a more comprehensive view of the effect of the reinforcement techniques used on the behavior of the structure. These variables can have a significant impact on the distribution of stresses and the overall performance of reinforced elements, so their

inclusion in future research will allow reinforcement methods to be better adapted to specific structural requirements.

In the next stages of research, the authors intend to conduct not only detailed analytical analyses, but also to use advanced numerical methods that will allow precise modeling of the behavior of concrete elements. Numerical analyses will make it possible to study in more detail the influence of various design parameters on the load-bearing capacity and durability of the structure and will allow validation of the experimental results obtained. Comparison of analytical and numerical results will allow evaluation of the compatibility of the two approaches, which in turn can contribute to further improvement of strengthening methodologies and development of more universal design guidelines.

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Annex A

Table A.1

Specimen name	Average value of applied pre-stress $\bar{\sigma}_p$ [MPa]	$\bar{\sigma}_p/\sigma_p$	Average value Upper stress limit σ_a [MPa]	$\bar{\sigma}_a/\sigma_a$
R-2	0.48	0.9690	17.0048	0.9983
R-3	0.49	0.9708	17.0049	0.9983
I-1-2	0.47	0.9391	17.8360	0.9764
I-1-3	0.49	0.9731	18.3056	1.0021
I-1-4	0.47	0.9344	17.6038	0.9637
I-3-2	0.47	0.9336	16.7431	0.9678
I-3-3	0.48	0.9604	17.1820	0.9932
I-3-4	0.47	0.9359	16.7414	0.9677
O-1-2	0.47	0.9315	19.7014	0.9502
O-1-3	0.48	0.9567	20.1971	0.9741
O-1-4	0.33	0.6659	20.6158	0.9943
O-3-2	0.33	0.6608	32.8046	0.9941
O-3-3	0.34	0.6865	33.2058	1.0062
O-3-4	0.35	0.6926	33.5955	1.0180
O-1/I-1-2	0.33	0.6635	25.9754	1.0227
O-1/I-1-3	0.33	0.6677	25.9573	1.0219
O-1/I-1-4	0.34	0.6802	25.8761	1.0187
O-3/I-1-2	0.33	0.6668	34.7703	1.0088
O-3/I-1-3	0.33	0.6692	34.7401	1.0079
O-3/I-1-4	0.33	0.6664	34.7358	1.0078