



Article

Bond Performance of Geopolymer Concrete with Steel and FRP Reinforcements

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Abstract: The increasing demand for sustainable construction materials has driven the exploration of alternatives to traditional cement-based concrete. In this context, this study investigates a cement-less material, specifically an alkali-activated or geopolymer concrete (GPC), which presents potential environmental benefits. The material has been characterized with respect to both its fresh and hardened properties, providing groundwork for future structural applications. A key focus of the research is the bond behavior between GPC and reinforcing bars, including both steel and non-metallic fiber-reinforced polymer (FRP) bars. The use of non-metallic bars is particularly relevant as they offer the potential to enhance the durability of structures by mitigating issues such as corrosion. Current research lacks comprehensive studies on factors affecting stress transfer at the GPC-reinforcing bar interface, such as bar diameter, bond length, and surface finish. This study aims to expand knowledge on the bond between GPC and steel/FRP rebars through experimental and analytical approaches. The tests, which included different bar types and bond lengths, showed that GPC exhibited similar bond behavior with steel and ribbed glass FRP bars in terms of bond strength and stress-slip curves. The results indicate that GPC exhibits comparable bond strength and stress-slip behavior when reinforced with either steel or ribbed glass FRP bars.

Keywords: bond behavior; geopolymer concrete; steel bar; FRP bars



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1. Introduction

In recent years, geopolymer concrete (GPC) has emerged as a promising alternative to traditional ordinary Portland cement concrete (OPCC), as a consequence of the need for more sustainable construction materials. Geopolymer concrete is synthesized through the alkaline activation of aluminosilicate sources, such as fly ash, slag, or metakaolin, which results in a binder with significantly lower carbon emissions than Portland cement [1]. In particular, according to numerous studies, slag-based geopolymer mortars and concretes offer superior performance compared to Portland cement in several areas. They exhibit faster early strength development, reach higher ultimate strength, and demonstrate enhanced durability. Additionally, they provide better resistance to chlorides, sulfates, and acidic environments, as well as greater heat resistance [2–5]. However, it exhibited increased drying shrinkage [6]. These advantages align with growing global efforts to reduce the environmental impact of the construction industry, since slag-based concrete reduces CO₂ emissions by 55–75% compared to OPC [7].

Recently, polymer concretes (PC), which are based on organic polymer resins rather than cementitious binders, have also been investigated as alternatives to OPC. Compared

to geopolymer concrete, polymer concrete does not require the use of alkaline activators and, more importantly, does not emit carbon dioxide during its synthesis, making it a low-emission material [8]. However, geopolymer concrete offers several advantages over polymer concretes in structural applications. These include superior fire resistance, higher thermal stability, and better long-term durability due to its inorganic matrix [9,10]. Furthermore, geopolymer concrete is generally derived from industrial by-products, such as slag or fly ash, enhancing its sustainability. Despite this, GPC is characterized by relatively poor workability due to its viscous silicate binder, which can complicate casting and compaction processes [11]. Therefore, the choice between geopolymer and polymer concretes depends on the specific application requirements, with GPC being particularly suitable for structural elements demanding high mechanical performance and fire resistance.

One of the critical factors in the structural performance of reinforced concrete elements is the bond between the reinforcement and the concrete matrix. In OPC concrete, the bond behavior between steel reinforcement and the concrete matrix has been extensively studied, and empirical models have been developed to predict bond strength under different conditions [12]. However, the bond behavior of GPC with reinforcement continues to be a subject of research, particularly when considering the use of alternative reinforcement materials, such as fiber-reinforced polymer (FRP) bars. The inclusion of non-metallic FRP bars is of particular significance, as these materials offer substantial advantages in improving the long-term durability of concrete structures. In aggressive environments such as those containing chlorides or sulfates, non-metallic FRP bars offer superior corrosion resistance compared to steel reinforcement. Despite the high alkalinity of GPC, the absence of free calcium hydroxide and the lower permeability can make the use of FRP bars a viable long-term solution for enhancing durability [13]. This is particularly important in applications where the use of conventional steel reinforcement is prone to degradation due to environmental factors, such as exposure to chlorides or other corrosive agents [14–17].

In this context, research has been conducted on the bond behavior of GPC with both steel and FRP reinforcement [18]. In his work [19], Sarker reported that GPC exhibited higher bond strength compared to OPCC. In this case, the authors attributed this result to its greater splitting tensile strength and the presence of a dense interfacial transition zone between the aggregate and geopolymer paste, as observed also in [20]. Similarly, ref. [21] reported that the bond strength of steel bars in fly ash-based GPC was comparable in both beam-end and direct pull-out tests, with normalized bond strength increasing as rebar diameter decreased. Several studies also indicated that the higher bond performance of GPC often led to steel bar yielding, followed by splitting failure in the concrete, which in some cases was sudden and explosive [22–24]. As happens in OPCC structures, steel bar corrosion also affects the bond with the surrounding GPC paste. In fact, an excessive degree of corrosion of approximately 4% reduces the bond strength, and the negative effect of corrosion resulted to be more pronounced in rebars with smaller diameters [25].

While most research has focused on steel reinforcement, fewer studies have explored the bond-slip behavior between GPC and FRP. Notably, test results indicate that the bond strength of glass FRP (GFRP)—reinforced GPC is similar to that of OPCC [26]. However, it was also found that sand-coated GFRP bars demonstrated a lower bonding capacity compared to deformed steel bars, due to the absence of the mechanical interlocking contribution [27]. Furthermore, an experimental investigation of GFRP bar bonding in geopolymer concrete using hinged beam tests demonstrated that the bond-slip performance between GFRP bars and GPC can be sensitive to the surface treatment of the bars. The hinged beam tests showed that the bond strength of GFRP bars is comparable but lower than deformed steel bars in some configurations, due to the difference in mechanical interlocking [28]. The surface treatment plays a crucial role also under elevated temperature, as observed in [29].

In fact, in their work, they observed that ribbed FRP bars exhibit greater bond strength retention compared to sand-coated FRP bars after exposure to elevated temperatures. Recent work has also investigated the performance of FRP bars in geopolymer matrices exposed to marine environments, confirming their potential durability under specific conditions [30].

The combination of geopolymer concrete and FRP reinforcement represents a promising solution for durable and sustainable structural elements, combining the environmental benefits of GPC with the corrosion resistance of FRP bars. Despite their individual advantages, their combined use remains insufficiently explored, especially in terms of bond behavior. Notably, this study presents the first comparative experimental investigation of the bond performance between ribbed GFRP bars and deformed steel bars embedded in slag-based geopolymer concrete, under direct pull-out tests. In addition to comparing bar materials, the study also considers the influence of surface treatments and bond lengths, providing a systematic assessment of key parameters affecting the GPC-reinforcement interface. These findings have direct engineering implications for the design of durable, corrosion-resistant structural elements made with low-carbon concrete alternatives and contribute valuable data to support the future standardization of GPC-based reinforced members.

This study aims to address the gap in knowledge regarding the bond performance of geopolymer concrete with both steel and FRP reinforcing bars. Specifically, it will investigate the factors that influence bond strength, such as the reinforcement material (i.e., steel, glass, or carbon), bar surface treatment, diameter, and bond length. To this aim, this experimental work involves direct pull-out tests on slag-based geopolymer concrete specimens reinforced with steel and various types of FRP bars. In particular, glass and carbon FRP bars are used in this study. The CFRP bars have a sand-coated finish, while the GFRP bars feature two types of surface finishes: sand-coated and ribbed. Several bond lengths have been considered in the specimen setup, aiming to determine the transfer length of each bar type. The comparison between ribbed and sand-coated bars highlights the significant role of ribs in enhancing the bond with the concrete matrix through mechanical interlocking. Additionally, the bond behavior of ribbed GFRP bars in GPC is found to be comparable to that of deformed steel bars.

2. Materials and Methods

2.1. Design of GPC Mixes

The experimental campaign described herein has been performed on two geopolymer cementless concrete mixes, named GPC_1 and GPC_2 (Table 1).

Table 1. Mix formulations [31].

Components	GPC_1	GPC2
	Quantity [kg/m ³]	Quantity [kg/m ³]
GGBS	224	222
Silica Fume	48	–
Expanded Glass	–	56
Carbonate Filler	128	–
Super Fluidifying Additive	8	–
Sodium Hydroxide solution (activator)	–	26
Sodium Silicate solution (activator)	170	78
Natural Sand (0–4 mm)	1092	708
Coarse Aggregate (4–8 mm)	–	354
Coarse Aggregate (8–16 mm)	471	–
Magnetite (0–2 mm)	–	689
Water	140	122

The GPC_1 mix has already been deeply described in [31], and it includes natural aggregates (calcium carbonate), filler (natural calcium carbonate), GGBS, silica fume, and an alkaline activator (sodium silicate solution). Additionally, a polycarboxylate-based superplasticizer was used to improve the workability of the geopolymer concrete mix. Specifically, a high-range water-reducing admixture commercially available was added in accordance with the manufacturer's recommendations. The dosage was optimized to achieve the desired slump while maintaining the integrity of the geopolymer matrix. The natural calcium carbonate aggregates were applied in two size ranges: sand (0–4 mm) and coarse aggregate (8–16 mm). Both the aggregates and the filler conform to UNI EN 12,620 standards [32]. In line with UNI EN 206-1 [33], the filler, GGBS, and silica fume are classified as Supplementary Materials for cementitious binders, influencing the concrete's technical performance (e.g., workability, mechanical properties). Specifically, the carbonate filler is considered a type 1 addition (non-pozzolanic), while the GGBFS and silica fume are type 2 additions (pozzolanic).

According to the manufacturer, the alkaline solution contains 41.7–45.0% sodium silicate, with a $\text{SiO}_2/\text{Na}_2\text{O}$ mass ratio between 1.60 and 1.70. Sodium silicate is produced by melting high-purity sand with sodium carbonate (soda ash) in high-temperature furnaces, yielding a water-soluble silicate powder. When dissolved in water, this powder forms a solution known as waterglass or sodium silicate solution. The solution used in this formulation had a pH ranging from 12.6 to 13.8.

On the other hand, the GPC_2 mix consisted of natural aggregates (calcium carbonate), GGBS, expanded glass, an activator solution, and magnetite. Similarly to GPC_1, the natural aggregates in the GPC_2 mix were sand (0–4 mm) and coarse aggregate (4–8 mm), both characterized in accordance with UNI EN 1097-6 [34]. The magnetite aggregate used in this mix primarily contains 70% iron (Fe) and has a particle density approximately twice that of natural aggregates was used as a high-density aggregate in GPC_2 to examine the compatibility of alternative aggregate types with geopolymer matrices. This heavy aggregate functions as a “heat sink”, absorbing heat from its surroundings, storing it, and releasing it gradually. As a result, the heat generated during hydration is absorbed, reducing the maximum temperature reached within the concrete compared to normal aggregates. This helps mitigate the risk of thermal cracking, which is often a concern in thick sections typical of offshore structures. Although magnetite may slightly affect some mechanical properties, both mixes used in this study showed comparable compressive strength and curing behavior, allowing for a consistent evaluation of bond performance across different reinforcement configurations.

The GPC_2 mix used two types of alkaline activators: the first was the sodium silicate solution, identical to the one used in GPC_1, and the second was a 50% *w/w* sodium hydroxide (NaOH) solution.

All specimens were cured under standard laboratory conditions, at ambient temperature (approximately 20–22 °C) and relative humidity of about 60%, for the curing durations reported in the next sections.

2.2. Mechanical Properties of GPC

The complete physical and mechanical characterization performed on the GPC_1 formulation used in this experimental activity is extensively detailed in [31]. Based on the slump tests performed on five different castings, the GPC_1 mix herein defined resulted in having S5 workability class according to UNI 12350-8 [35]. The average density of all the GPC castings performed after 28 days of curing resulted to be 2108 kg/m³.

The compressive strength tests were performed on 150 mm cubic samples [36] at increasing curing ages. As shown in Table 2, at 28 days of curing, the average cubic

compressive strength resulted to be equal to 52.6 MPa, and a significant further increase up to 82.8 MPa at 106 days was recorded. As reported by [37], this compressive strength increase beyond the standard curing period of 28 days is probably due to the presence of GGBS, which contributes to the development of a denser microstructure in the binder. The elastic modulus of the GPC_1 mix was measured at 28 days of curing according to the “method B” reported in [38]. The average value of a total of eight cylindrical samples resulted to be 24.3 GPa, which is 48% lower than the value calculated according to Eurocode 2 [39] for an equivalent grade OPCC. The same observations have been made by [40], in which the elastic modulus of the fly ash-based GPC resulted in 50% lower values than the corresponding values obtained with an OPCC mix. This result can be attributed to two main factors: (i) in the analyzed geopolymer concrete, the hydration process leads to the formation of aluminum silicate, due to the presence of GGBS and Silica Fume, instead of the typical calcium silicate that forms in traditional concrete. As a result, the geopolymer concrete exhibits lower stiffness compared to conventional concrete; (ii) in geopolymer concrete, calcium hydroxide does not form in significant quantities, as GGBS lacks sufficient calcium oxides to produce substantial amounts of calcium hydroxide. Consequently, this also contributes to a less rigid structure in geopolymer concrete when compared to cement-based concrete. However, a detailed chemical study is necessary to validate the hypotheses outlined above.

Table 2. Cubic compressive strength results of both GPC_1 and GPC_2 mixes at different times of curing.

t [Days]	GPC_1		GPC_2	
	#	R _c [MPa]	#	R _c [MPa]
2	5	29.0 ± 2.1 ¹	-	-
3	-	-	2	16.4 (0.45) ²
7	5	52.7 ± 3.3 ¹	2	43.7 (1.83) ²
14	5	53.1 ± 3.6 ¹	2	40.7 (3.91) ²
28	17	52.6 ± 4.6 ¹	2	52.5 (6.07) ²
58			2	62.7 (4.24) ²
60	2	68.7 (1.71) ²	-	-
106	2	82.8 (7.00) ²	-	-

Number of specimens; ¹ Confidence intervals (CI) at 95%; ² Standard deviation.

Similarly to GPC_1, the compressive strength of GPC_2 has been determined at different times of curing. In particular, in Table 2 there are reported the results obtained from the first hardening stage, from three days up to 58 days of curing, are reported. Two cubes have been tested at each time of curing. Furthermore, 95% confidence intervals are reported in Table 2 for the results based on five or more specimens. For results derived from only two samples, the standard deviation is shown instead, in line with common practice for limited sample sets.

Additionally, displacement-controlled compressive tests were performed 28 days after the concrete was cast on GPC_1 mix to determine its complete constitutive behavior. As described in [31], the GPC_1 exhibited brittle behavior since the specimen failure was characterized by a quick load decay once the compressive strength peak was reached, as can be seen from Figure 1. From a total of five specimens, an average ultimate strain equal to 1.9‰ was set as the strain corresponding to a 10% stress decay from the compression strength, resulting in 46% lower than the standard design value suggested for OPCC by fib Model Code 2010 [12]. As can be seen in the graph of Figure 1, in the case of specimens GPC_C1 and GPC_C4, the failure was brittle and sudden, such that the LVDTs (Linear Variable Displacement Transducer) were not able to record the descending branch after

the peak. For this reason, these two samples have not been considered to determine the ultimate strain.

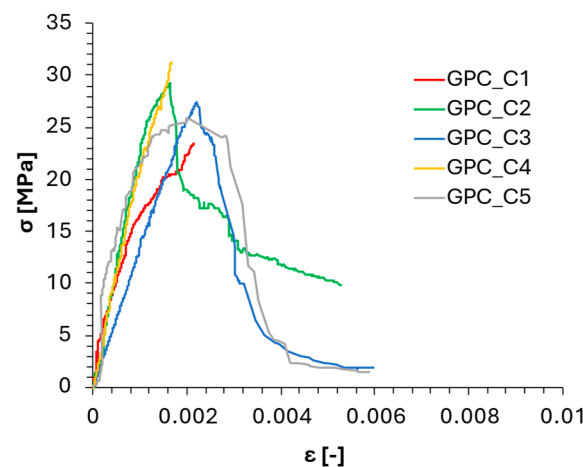


Figure 1. Constitutive law tests the results of the GPC mix [31].

Although a certain level of variability was observed among the specimens—particularly in the post-peak response—this is consistent with the brittle nature of geopolymer concrete and the localized cracking that characterizes its failure mechanisms.

2.3. Reinforcement Bars

Four kinds of reinforcing bars were considered in this activity: ribbed steel bars (Figure 2a), commonly used in the construction industry, with 12 and 16 mm diameter; sand coated GFRP bars (Figure 2b) with 12 mm diameter made of chemically resistant glass fiber and polyester thermosetting resin; ribbed GFRP bar (Figure 2c) with 10 mm diameter; sand coated CFRP bar (Figure 2d) with 8 mm diameter made of chemically resistant high tenacity carbon fiber and thermosetting resin.

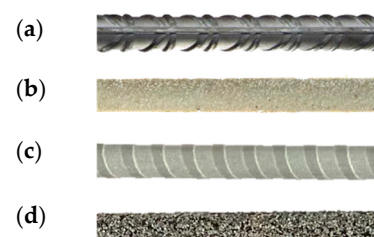


Figure 2. Reinforcement bars used include the following: (a) steel; (b) sanded GFRP; (c) ribbed GFRP; (d) sanded CFRP.

The mechanical properties of all reinforcing bars considered in this study refer to the values reported in the manufacturers' technical datasheets. The sand-coated GFRP (sGFRP) bars have a guaranteed tensile strength of 500 MPa and a modulus of elasticity of 40 GPa. The ribbed GFRP (rGFRP) bars, with a diameter of 10 mm, have a guaranteed ultimate tensile strength of 1003 MPa and a mean modulus of elasticity of 60.3 GPa. The CFRP bars, with a diameter of 8 mm, have a nominal tensile strength of 1700 MPa and a modulus of elasticity of 130 GPa. The deformed steel bars exhibit a characteristic yield strength of 450 MPa and a modulus of elasticity of 210 GPa.

2.4. Test Methods and Experimental Campaign

The direct pull-out test setup and specimens were designed according to the RILEM Recommendations [41] as shown in Figure 3. Each GPC specimen was cubic with a side

equal to 10ϕ and a centered embedded bar in order to have a concrete cover equal to 4.5ϕ . Three different bond lengths equal to 2.5ϕ , 5ϕ , and 7.5ϕ were analyzed for both steel and FRP bars. These specific lengths were chosen to assess the variation in bond strength as a function of the bond length parameter. The 5ϕ length is considered consistent to assume a uniform distribution of the bond stress along the bonded length, thereby enabling a precise estimation of the bond stress versus slip relationship [18]. Moreover, the inclusion of smaller (2.5ϕ) and larger (7.5ϕ) bond lengths allows for a comprehensive evaluation of how bond strength evolves with changes in bond length. This approach is essential for determining the transfer length, which is defined as the bond length beyond which there is no significant increase in bond strength. The transfer length provides critical information for defining the anchorage length of the bars, facilitating the determination of the minimum length required to ensure effective load transfer.

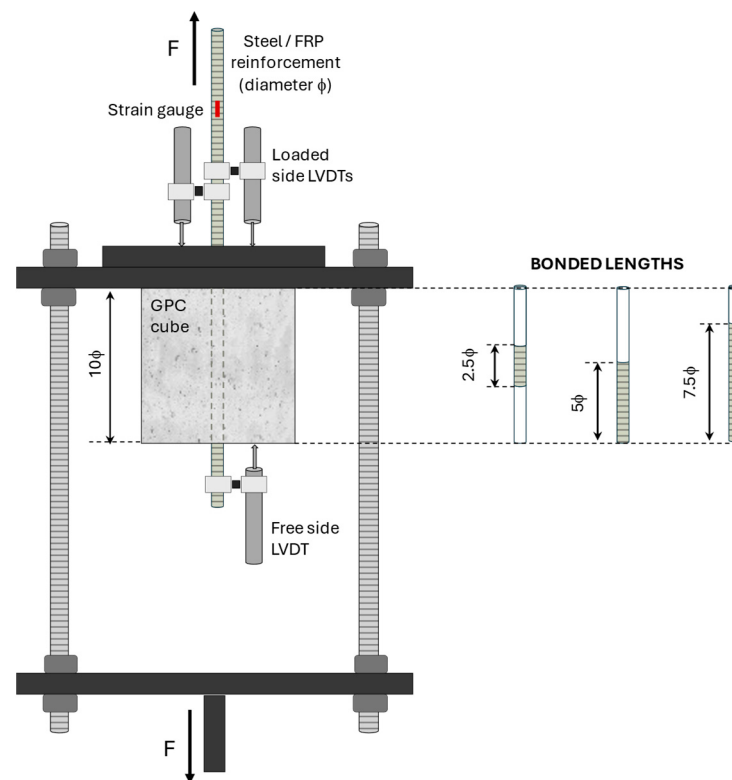


Figure 3. Pull-out test setup.

The unbonded zones were realized by applying a plastic sheet to the specific length of the bars.

Three LVDTs (Linear Variable Displacement Transducer) have been used as shown in Figure 3: one was anchored at the unloaded side of the bar, meanwhile the other two were positioned at the loaded side to record the corresponding slips. Moreover, a strain gauge was glued on the bar to control its deformation during the test. A tensile force was applied by means of a testing machine under displacement control at a rate of 0.2 mm/min . All tests were stopped when a free slip of 10 mm was reached, except when the specimens failed before (i.e., splitting failure).

The interface bond stress was evaluated in the hypothesis that the bond stress remains constant within the bond length, L_b , (Equation (1)):

$$\tau_b = F/(\pi \cdot \phi \cdot L_b), \quad (1)$$

where τ_b is the bond stress and F is the applied force.

The experimental program comprised 53 pull-out tests as reported in Table 3. The specimens were realized with the reinforcement bar previously detailed. The specimens are named using the following code: the first lowercase letter indicate the FRP bar finishing (s: sanded, r: ribbed); the capital letter the bar material (S: steel; G: GFRP; C: CFRP); the second number specifies the bar diameter (i.e., 8, 10, 12 or 16); the third symbol is related to the bond length L_b (2.5: 2.5 ϕ ; 5: 5 ϕ ; 7.5: 7.5 ϕ); the last number is finally added to specify the specimen in the test sequence.

Table 3. Direct pull-out tests experimental campaign.

Mix	Group ID	Age of Test [Days]	f_c [MPa]	Number of Specimens
GPC_1	S_12_5	60	68.7 (1.71) ²	4
	S_16_5	106	82.8 (7.00) ²	5
	sG_12_5	28	27.1 (11%) ¹	5
	S_12_2.5	117	19.5 (10%) ¹	5
	sG_12_2.5	117	19.5 (10%) ¹	5
GPC_2	sC_8_5	38	42.5 (1.86) ²	5
	sC_8_2.5	56	40.9 (1.65) ²	5
	rG_10_5	56	38.5 (2%) ¹	5
	rG_10_2.5	56	38.5 (2%) ¹	5
	rG_10_7.5	150	49.0 (3.21) ²	3
	S_12_7.5	150	49.0 (3.21) ²	3
	sG_12_7.5	150	49.0 (3.21) ²	3

¹ Coefficient of variation; ² Standard deviation.

All the results of pull-out tests obtained with the mix GPC_1 have already been shown and deeply discussed in a previous work [18]. In the following section, they are provided again only for comparative purposes.

3. Results

The results of the pull-out tests are presented and discussed below in terms of bond vs. slip curve, failure mode (P = reinforcement bar pulled out from the concrete cube; S = splitting tensile failure of the concrete cube), maximum (τ_b) and residual (τ_r) bond stress, slip at the free end corresponding to τ_b (s_b) and τ_r (s_r), and the corresponding average values ($\tau_{b,av}$, $\tau_{r,av}$, $s_{b,av}$, $s_{r,av}$), according to Figure 4. The model and the parameters considered represent a modification of the model by Bertero, Eligehausen, and Popov [42] originally developed to characterize the bond mechanism between deformed steel bars and concrete and later incorporated into the Model Code 2010 [12]. Specifically, in [43], the bond-slip relationship previously defined [42] was adapted to model the bond mechanism between FRP bars and concrete, for which no distinct horizontal (plateau) phase has been experimentally observed prior to the descending phase.

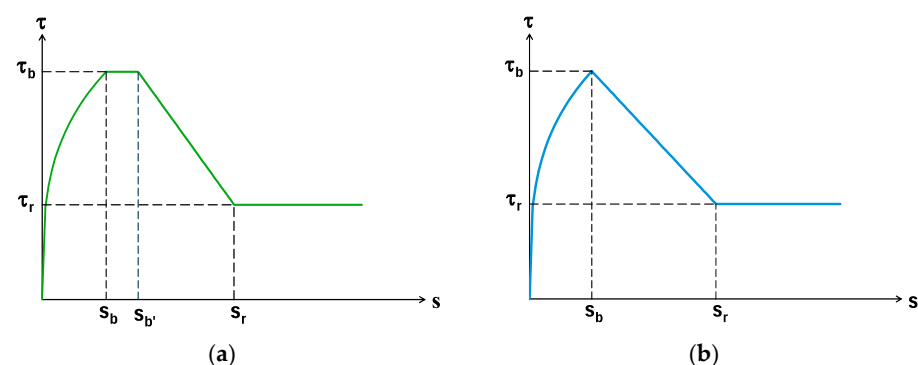


Figure 4. (a) BEP model [42]; (b) modified BEP model [43].

In the following, even for the specific case related to tests with steel bars, the sliding value $s_{b'}$ will not be considered in the discussion

3.1. Steel Bar

The results obtained from tests performed on specimens with steel bars with diameters equal to 12 mm with bond lengths of 2.5ϕ and 5ϕ , [18] are still reported in Figure 5a–c and Table 4 as per comparison purposes, together with the results obtained with 12 mm bars and longer bond length equal to 7.5ϕ . The parameters discussed in the following section refer, for comparison purposes, to typical bond behavior parameters of FRP bars (i.e., the parameter $s_{b'}$ was not taken into account).

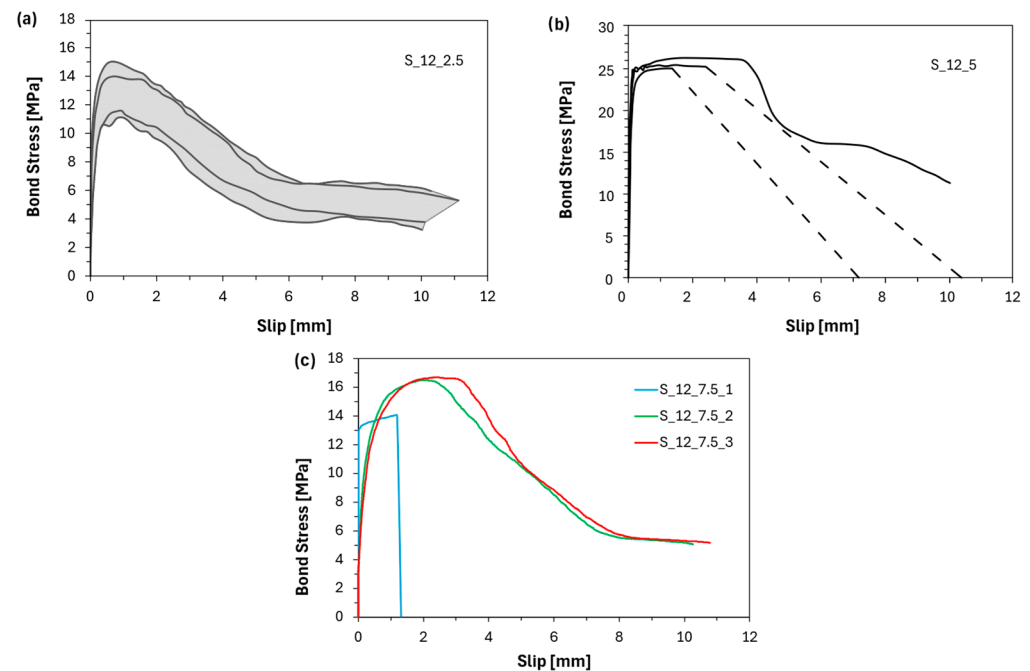


Figure 5. Pull-out test results with deformed steel bars: (a) diameter 12 mm with bond length 2.5ϕ [18]; (b) 12 mm with bond length 5ϕ [18]; (c) 12 mm with bond length 7.5ϕ .

Table 4. Pull-out test results between the GPC and steel bars.

	Failure	τ_b [MPa]	$\tau_{b,av}$ [MPa]	τ_r [MPa]	$\tau_{r,av}$ [MPa]	s_b [mm]	$s_{b,av}$ [mm]	s_r [mm]	$s_{r,av}$ [mm]
S_12_2.5_1	P	11.26	12.78	4.00	5.13	0.37	0.45	5.69	6.26
S_12_2.5_2	P	15.05		6.50		0.59		6.48	
S_12_2.5_3	P	11.70		4.40		0.35		6.23	
S_12_2.5_4	S	11.81	(13%) ¹	-	(22%) ¹	-	(25%) ¹	-	(7%) ¹
S_12_2.5_5	P	14.08		5.60		0.48		6.64	
S_12_5_1		-	25.48	-		-	0.19	-	
S_12_5_2	P	26.18		-		0.19		-	
S_12_5_3	S	24.88		-		-		-	
S_12_5_4	S	25.37		-		-		-	
S_12_7.5_1	S	14.03	15.73 (9%) ¹	-	5.14 (0.07) ²	-	1.25 (0.29) ²	-	7.94 (0.13) ²
S_12_7.5_2	P	16.48		5.09		1.04		7.85	
S_12_7.5_3	P	16.70		5.19		1.45		8.03	

¹ Coefficient of variation; ² Standard deviation.

Table 4 highlights the presence of missing data. Excluding the specimens that failed due to splitting—where the residual bond stress and the corresponding slip value were not recorded, as the τ - s curve exhibits the typical sudden drop after reaching the peak bond stress—some slip values at maximum load are also missing, as they were considered not

significant due to the yielding of the reinforcing bar. It is worth noting that the sudden splitting failures observed in several specimens are a direct consequence of the brittle nature of geopolymer concrete. Unlike OPCC, GPC does not form calcium hydroxide or C–S–H phases during hydration. Instead, the binder matrix mainly consists of N–A–S–H or C–A–S–H gels, which result in a denser but more rigid microstructure with limited energy dissipation capacity [44]. This intrinsic brittleness inhibits stress redistribution and favors the formation of abrupt tensile cracks along the bar interface, especially once the bond stress exceeds the concrete's splitting strength. As a result, specimens failing by splitting typically show an abrupt drop in bond stress after peak load, preventing the reliable measurement of residual bond or slip values.

The experimental data of the specimen with a bond length of 7.5ϕ confirmed the observations made at shorter bond lengths in [18]. In fact, the bond stress-slip curves exhibit an initial ascending branch, where the first portion is governed by chemical adhesion, followed by a segment dominated by mechanical interlocking. In the initial phase, where stress transfer between the steel and GPC is primarily achieved through chemical adhesion, relative slips are nearly zero. Once the chemical adhesion is overcome, micro-cracking occurs within the interfacial transition zone between the steel and concrete, leading to an increase in slip. In this second phase, the stress transfer mechanism shifts to mechanical interlocking.

From the data reported in the table, it can be observed that the bond stress remains constant (considering experimental variation) as the diameter of the steel bars increases. The recorded trend is in line with theoretical predictions, as no significant variation in the maximum bond stress is expected within the range of diameters analyzed. This is in accordance with the guidelines provided in national and European standards (NTC 2018, EC2, Model Code), which suggest a change in bond stress for diameters greater than 32 mm.

After reaching the peak bond stress, a reduction in stress is observed, followed by a horizontal plateau representative of the residual stress. This plateau probably corresponds to the frictional contribution at the interface between GPC and steel bars, which develops once the bond provided by chemical adhesion and mechanical interlocking has been lost. In fact, the average values recorded for the S_12_2.5 and S_12_7.5 samples, which stabilize at the same value, around 5 MPa (approximately 32% of the τ_b), suggesting that this parameter is dependent on the same physical mechanism at the interface.

The data recorded for the slip at the peak bond stress (s_b) do not allow for any conclusions to be drawn regarding this parameter. The values recorded in terms of s_r for the samples that exhibited pull-out failure are similar to each other even when accounting for the high experimental variability associated with this parameter, which is linked to the measurement instrumentation used (LVDT). This suggests that the slip corresponding to the onset of the friction-dominated phase is not significantly influenced by the bond length. However, measurement with high accuracy and an increased number of experimental data would be necessary to confirm this observation.

3.2. Sanded GFRP Bars

In Figure 6 and Table 5 the results of groups sG_12_2.5 and sG_12_5, already discussed in [18], are compared with the results of specimens with a bond length of 7.5ϕ . The shape of the bond stress-slip curves mirrors that observed for other bond lengths. In all cases, failure occurred by pull-out of the bar from the concrete cube. Since the sGFRP bars used in these tests were smooth, bond resistance was primarily due to chemical adhesion between the GPC matrix and the sand-coated bar surface. This is evident in the initial branch of the curves, which displays high stiffness and negligible slip, indicating an uncracked interface.

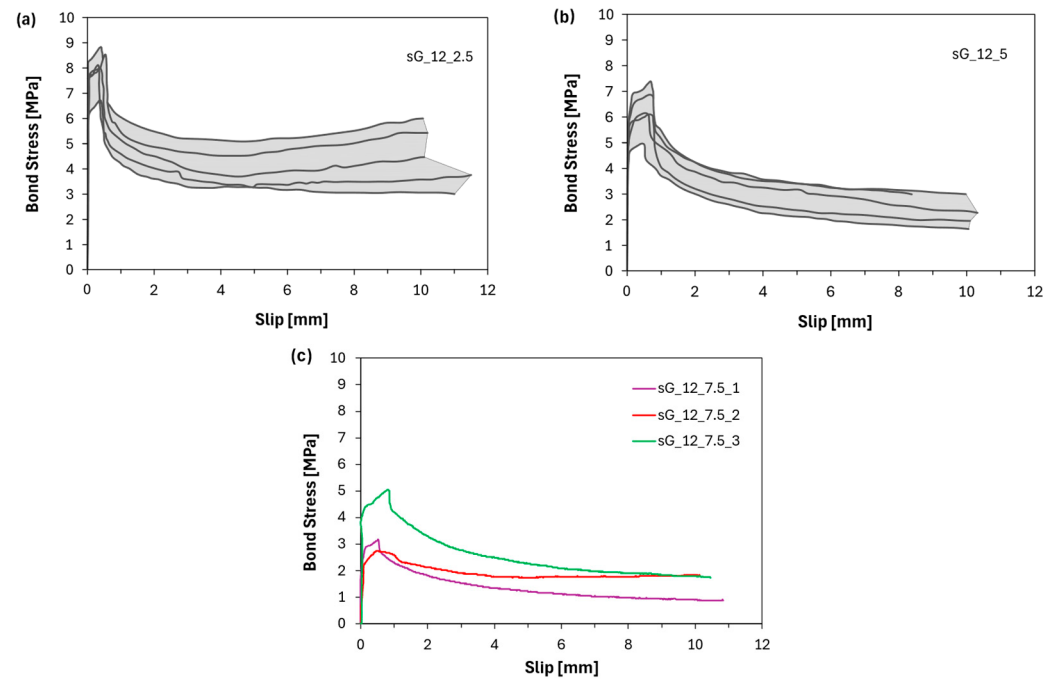


Figure 6. Pull-out test results with 12 mm sand-coated GFRP bars: (a) bond length 2.5 ϕ [18]; (b) bond length 5 ϕ [18]; (c) bond length 7.5 ϕ .

Table 5. Pull-out test results between GPC and sGFRP bars.

	Failure	τ_b [MPa]	$\tau_{b,av}$ [MPa]	τ_r [MPa]	τ_{rav} [MPa]	s_b [mm]	$s_{b,av}$ [mm]	s_r [mm]	$s_{r,av}$ [mm]
sG_12_2.5_1	P	8.02		3.80		0.08		0.51	
sG_12_2.5_2	P	8.84	8.06	5.30	4.06	0.02	0.04	0.49	0.51
sG_12_2.5_3	P	8.56		4.80		0.05		0.61	
sG_12_2.5_4	P	8.13		3.10		0.01		0.42	
sG_12_2.5_5	P	6.72	(10.13%) ¹	3.30	(23.53%) ¹	0.05	(66.07%) ¹	0.50	(13.45%) ¹
sG_12_5_1	P	6.66		3.10		0.26		0.79	
sG_12_5_2	P	4.82	6.10	1.90	2.58	0.08	0.18	0.55	0.73
sG_12_5_3	P	5.96		2.70		0.29		0.70	
sG_12_5_4	P	5.90		2.10		0.12		0.83	
sG_12_5_5	P	7.15	(14.47%) ¹	3.10	(21.65%) ¹	0.17	(48.70%) ¹	0.80	(15.49%) ¹
sG_12_7.5_1	P	3.18	3.66	0.96	1.55	0.17	0.17	0.68	0.94
sG_12_7.5_2	P	2.75		1.78		0.16		1.19	
sG_12_7.5_3	P	5.05	(33.41%) ¹	1.92	(33.39%) ¹	0.17	(3.46%) ¹	0.95	(27.14%) ¹

¹ Coefficient of variation.

When the first crack appears, the curve's slope decreases, marking a shift in bond mechanism to frictional resistance between the bars and concrete, maybe enhanced by the sand coating on the bars. A notable decline in bond stress follows the peak bond strength. In the final portion of the curve, the bond behavior is likely governed solely by friction between the bars and the concrete matrix, as indicated by the asymptotic trend. However, this last phase exhibits significant scatter between similar specimens, likely due to the presence of sandblasting on the bars, which detaches in a non-uniform manner after the peak, influencing the final stage of the curve.

The residual bond strength averages around 1.55 MPa within a slip range of 5–8 mm, representing approximately 42% of the peak bond strength. This value is consistent with those obtained for bond lengths of 2.5 ϕ and 5 ϕ , which showed residual bond strengths of 43% and 49%, respectively.

The high CoV value recorded for the parameter s_b prevents drawing any conclusions regarding its dependence on the selected experimental variables. The need for a more

accurate measurement technique and an increased amount of experimental data is reiterated in order to better analyze this influence.

From Table 5, it is observed that the residual slip, s_r , increases with increasing bond length. This can be primarily attributed to the distribution of bond stresses along L_b . In fact, as the bond length increases, the effect of stress non-uniformity in the considered section becomes more evident, and the bond stresses tend to be higher at the loaded end compared to the free end. Furthermore, since damage propagates progressively from one end to the other of the bar with increasing slip, for longer bond lengths, a greater slip will be required to damage the entire interface and reach the plateau, i.e., the residual bond stress.

3.3. Ribbed GFRP Bars

The results obtained with rGFRP bars are reported in Figure 7 and Table 6 for bond lengths of 2.5 ϕ , 5 ϕ , and 7.5 ϕ . All specimens exhibit pull-out failure except for rG_10_5_5 and rG_10_7.5_3, where concrete splitting failure occurred (Figure 7a,c).

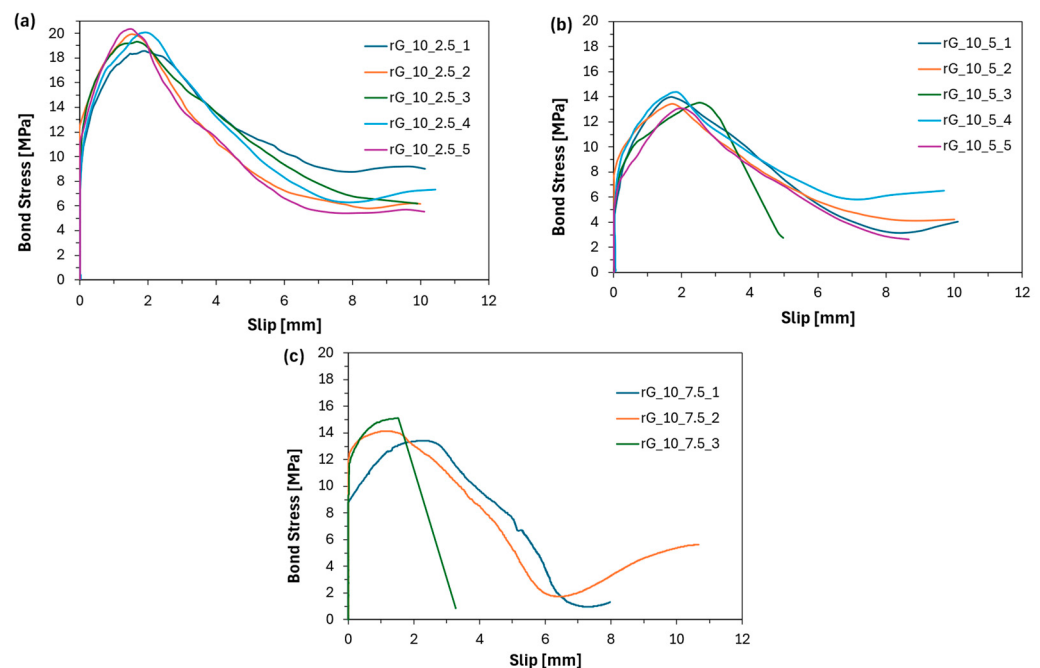


Figure 7. Pull-out test results with 10 mm ribbed GFRP bars: (a) bond length 2.5 ϕ ; (b) bond length 5 ϕ ; (c) bond length 7.5 ϕ .

Table 6. Pull-out test results between GPC and rGFRP bars.

	Failure	τ_b [MPa]	$\tau_{b,av}$ [MPa]	τ_r [MPa]	$\tau_{r,av}$ [MPa]	s_b [mm]	$s_{b,av}$ [mm]	s_r [mm]	$s_{r,av}$ [mm]
rG_10_2.5_1	P	18.59		8.90		1.86		7.48	
rG_10_2.5_2	P	19.95	19.66	6.00	6.70	1.49	1.62	7.92	7.70
rG_10_2.5_3	P	19.34		6.80		1.39		8.03	
rG_10_2.5_4	P	20.08		6.30		1.78		7.72	
rG_10_2.5_5	P	20.35	(4%) ¹	5.50	(20%) ¹	1.57	(12%) ¹	7.33	(4%) ¹
rG_10_5_1	P	10.10		3.50		1.53		7.56	
rG_10_5_2	P	13.46	12.91	4.20	4.20	1.81	1.77	8.07	7.47
rG_10_5_3	S	13.54		-				-	
rG_10_5_4	P	14.39		5.90		1.80		6.69	
rG_10_5_5	P	13.08	(13%) ¹	3.20	(29%) ¹	1.95	(10%) ¹	7.57	(8%) ¹
rG_10_7.5_1	P	13.43	14.22	1.39	1.63	1.50	1.04	6.37	6.21
rG_10_7.5_2	P	14.13		1.88		0.59		6.05	
rG_10_7.5_3	S	15.11	(6%) ¹	-	(0.35) ²	-	(0.64) ²	-	(0.23) ²

¹ Coefficient of variation; ² Standard deviation.

The graphs in Figure 7 represent the bond vs. slip behavior obtained for rGFRP bars with GPC_2 mix. The curves resemble the theoretical one defined by the Model Code [8] for the bond of OPCC with steel bars. A first high-stiffness branch with slip close to zero can be observed until approximately 8 MPa and 12 MPa for all three bond lengths, respectively, governed by the chemical adhesion between the concrete matrix and the bar. After the chemical adhesion fails, a change in the slope of the curves occurs, leading to the bond strength peak dominated by mechanical interlocking due to the ribs on the bar. Finally, after the peak, the behavior transitions into a friction-governed horizontal plateau.

The bond strength results show that the average bond strength was 13.7 MPa (CoV = 4%) and 19.7 MPa (CoV = 4%) for bond lengths of 5 ϕ and 2.5 ϕ , respectively. For the 7.5 ϕ bond length, the maximum bond stress ranged from 13.43 MPa to 15.11 MPa, with specimen rG_10_7.5_3 showing the highest value. The bond strength corresponding to the shorter bond length (2.5 ϕ) was approximately 30% higher than that of the longer embedment length (5 ϕ), consistent with the trend observed in previous cases involving other types of FRP bars. Comparability between the results was ensured by casting all specimens with concrete of equivalent compressive strength. The residual bond stress was found to be 4.2 MPa (CoV = 29%) and 6.7 MPa (CoV = 20%) for bond lengths of 5 ϕ and 2.5 ϕ , respectively, representing 30% and 34% of the relative bond strength. For the 7.5 ϕ bond length, residual bond stress values were recorded for rG_10_7.5_1 and rG_10_7.5_2 at 1.39 MPa and 1.88 MPa. In this latter case, the residual stress corresponds to approximately 11% of the peak bond strength. However, the corresponding bond-slip curves do not clearly exhibit a constant plateau in the post-peak phase (Figure 6c), suggesting the need for an extended experimental campaign on this type of specimen to more accurately assess the final branch of the bond-slip response.

The previously observed trend relating to the slip value at the onset of the plateau, s_r , does not appear to be confirmed in the case of ribbed bars. This may be attributed to a different influence of the surface finish on the transfer mechanism associated with residual stresses. In fact, once the maximum bond stress, achieved through the action of the ribs, is exceeded, the residual resistance between the bars and the concrete may be limited, as the ribs have damaged (i.e., concrete crushing or cracking) the surrounding concrete.

3.4. Sanded CFRP Bars

As in the case of rGFRP, all the tests performed with sand-coated CFRP bars involved the GPC_2 mix.

The results of the bond behavior of GPC_2 with CFRP bars are reported in Figure 8 and Table 7. Regarding the stress transfer mechanism between GPC and sanded CFRP bars, the same observations made for sGFRP can be extended in this case (Figure 6). Specifically, the curves exhibit a high initial stiffness with almost zero slip, followed by a sharp decay in stress toward the asymptotic final trend. The predominant mechanisms governing these stages are chemical adhesion in the initial phase and friction between the concrete matrix and the bar in the final phase.

The bond strength results are summarized in Table 7. In particular, the average bond strength of GPC_2 with CFRP bars resulted to be 8.8 MPa (CoV = 31%) and 11.4 MPa (CoV = 40%) for bond lengths equal to 5 ϕ and 2.5 ϕ , respectively. In this case, an even greater scatter with respect to sGFRP bars has been observed. Considering the same surface finishing, the observed high variability of the results can be attributed to the non-uniform distribution of sand along the bonded length, also in this case. Moreover, the residual bond stress resulted to be 4.2 MPa (CoV = 29%) and 6.7 MPa (CoV = 20%) for bond lengths equal to 5 ϕ and 2.5 ϕ , respectively. These values resulted to be, respectively, 52% and 41% of the respective bond strength.

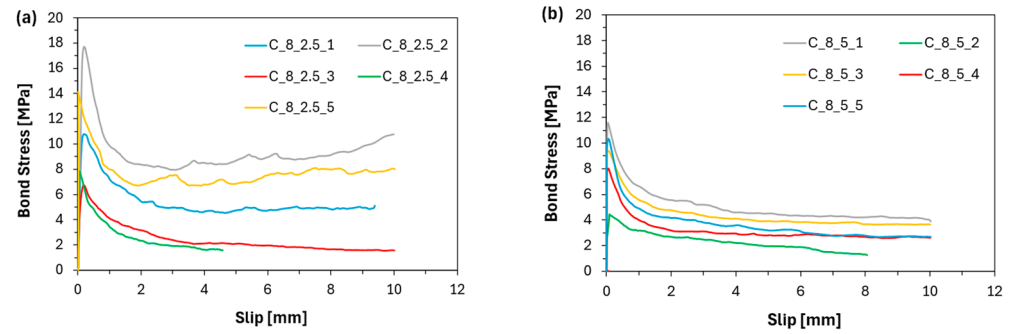


Figure 8. Pull-out test results with CFRP sand-coated bars: (a) diameter 8 mm with bond length 2.5 ϕ ; (b) diameter 8 mm with bond length 5 ϕ .

Table 7. Pull-out test results between GPC and CFRP bars.

	Failure	τ_b [MPa]	$\tau_{b,av}$ [MPa]	τ_{rf} [MPa]	$\tau_{r,av}$ [MPa]	s_b [mm]	$s_{b,av}$ [mm]	s_r [mm]	$s_{r,av}$ [mm]
sC_8_2.5_1	P	10.80	11.43	4.70	4.82	0.27	0.17	3.99	2.75
sC_8_2.5_2	P	17.70		8.30		0.24		2.10	
sC_8_2.5_3	P	6.73		2.10		0.17		3.65	
sC_8_2.5_4	P	7.83		2.30		0.04		1.97	
sC_8_2.5_5	P	14.11	(4%) ¹	6.70	(56%) ¹	0.15	(53%) ¹	2.03	(36%) ¹
sC_8_5_1	P	11.59	8.76	5.50	4.02	0.03	0.07	2.09	2.12
sC_8_5_2	P	4.44		2.60		0.12		2.09	
sC_8_5_3	P	9.40		4.70		0.02		2.16	
sC_8_5_4	P	8.03		3.10		0.08		2.10	
sC_8_5_5	P	10.34	(13%) ¹	4.20	(29%) ¹	0.09	(62%) ¹	2.15	(2%) ¹

¹ Coefficient of variation.

The findings previously reported for the slip values s_b and s_r of sand-coated GFRP bars may also apply to these bar types. Nevertheless, the high variability observed in this case necessitates further experimental testing for data validation.

4. Comparisons and Discussion

In the present paragraph, the obtained experimental results are compared, aiming to evaluate the effect of bond length and kind of reinforcing bars on the interface behavior. Concrete compressive strength is acknowledged as one of the parameters influencing bond performance. To remove its influence and allow for a consistent comparison of the results, the following correction factor has been applied to the experimental bond:

$$\left(\frac{f_c}{f_{c,REF}} \right)^{1/2}, \quad (2)$$

in which $f_{c,REF}$ has been assumed equal to the compressive strength obtained for a group of specimens chosen as reference, and f_c is the compressive strength of each other group of specimens.

This factor was derived from the expressions proposed in the fib Model Code 2010 [12] for the design bond strength, which is proportional to the square root of the concrete compressive strength. Specifically, it corresponds to the ratio between two bond stress formulations, $\tau_{b1} = 2.5 \cdot \sqrt{f_c}$ and $\tau_{b2} = 2.5 \cdot \sqrt{f_{c,REF}}$ leading to the simplified form of Equation (2). This normalization allows for a meaningful comparison between specimens cast with slightly different concrete strengths.

A more detailed comparison with analytical models, such as those proposed by the fib Model Code 2010 [12], was presented in a previous work [18]. It has been observed that the estimation of the ultimate bond strength between OPC concrete and steel bar proposed by the fib Model Code 2010 can also be safely used for GPC and steel bar. Furthermore,

that study also included a comparison with OPC-based specimens from Aiello et al. [45], tested under similar conditions. The bond strength obtained with GPC was higher than that obtained in the case of OPC, for both deformed steel and sand-coated GFRP bars. In particular, the GPC bond strength was 48% higher in the case of the steel bar and 55% higher in the case of the GFRP bar.

4.1. Effect of the Stiffness and Surface Finishing of the Rebars

To enable comparison among the different bar materials investigated, the experimental bond strength has been corrected using the coefficient defined in Equation (2). In this correction $f_{c,REF}$ refers to the compressive strength of specimens from group S_12_5. The comparisons have been made between bars with the same external finishing: ribbed or smooth with sand-coating layer, namely ribbed steel and GFRP bars, and sand-coated GFRP and CFRP bars.

Since in the case of bond length equal to 5ϕ , the common failure observed in the case of steel bar was splitting of concrete, the comparison with rGFRP bars has been made by taking into account the results obtained for bond length 2.5ϕ for both bars. The adjusted experimental values have been reported and compared in the graph in Figure 9 and in Table 8. From the graph and the values reported in Table 8 bond strength adjusted results are very similar. In fact, in the case of deformed steel bar, the bond strength resulted to be 24.6 (CoV = 11%), and in case of rGFRP it resulted in 26.4 (CoV = 4%), with a difference of about 7%, which is within the statistical variability of the experimental results.

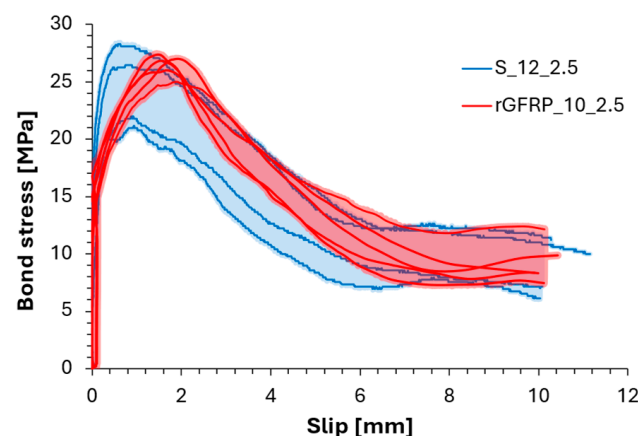


Figure 9. Comparisons between the results of pull-out tests with ribbed steel and rGFRP bars with a bond length of 2.5ϕ .

Table 8. Adjusted values of bond strength results obtained with ribbed steel and rGFRP bars with a bond length of 2.5ϕ .

Bar	Concrete	Sample	$\tau_{b,max,adj}$ [MPa]	$\tau_{b,max,adj,av}$ [MPa]	s_1 [mm]	$s_{1,av}$ [mm]
Steel	GPC_1 $f_c = 19.5$ MPa	S_12_2.5_1	23.8	24.6 (11%) ¹	0.369	0.42 (27%) ¹
		S_12_2.5_2	28.3		0.588	
		S_12_2.5_3	22.0		0.352	
		S_12_2.5_4	22.1		0.302	
		S_12_2.5_5	26.5		0.478	
rGFRP	GPC_2 $f_c = 38.5$ MPa	rG_10_2.5_1	25.0	26.4 (4%) ¹	1.86	1.62 (12%) ¹
		rG_10_2.5_2	26.8		1.49	
		rG_10_2.5_3	26.0		1.39	
		rG_10_2.5_4	27.0		1.78	
		rG_10_2.5_5	27.4		1.57	

¹ Coefficient of variation.

Despite the very similar bond strength, the graph clearly reveals a distinct difference in bond stiffness in the region between chemical adhesion loss (i.e., zone with no slip) and the maximum bond stress. To confirm this, Table 8 reports the s_1 values for each specimen. A clear difference is observed when comparing the average values $s_{1,av}$ of the two specimen groups. In particular, the slip at the bond strength was 0.42 mm (CoV = 27%) for deformed steel bar and 1.62 mm (CoV = 12%) for rGFRP, representing a 74% increase. As noted earlier, the segment of the bond–slip curve being analyzed reflects the contribution of mechanical interlocking to the bond mechanism. According to [46] bond stiffness of this branch is directly linked to the ribs' geometry, meaning that the ribs of the deformed steel bars provide a more efficient interlocking with the concrete matrix than the rGFRP.

In fact, as performed in [46], the bond index (or relative rib area) can be estimated using the following formula:

$$f_R = \frac{d_e^2 - d_i^2}{4ds}, \quad (3)$$

where d_e is the external diameter (rib crest), d_i is the internal diameter (rib root), d is the nominal bar diameter, and s is the rib spacing.

For the rGFRP bars, the experimentally measured values were 13.2 mm and 12.7 mm for d_e and d_i respectively, and measured s equal to 6 mm, so the bond index resulted to be 0.045. On the other hand, for the steel bar, it has been considered 12.8 mm and 12.0 mm for the external and internal diameter, respectively, and a rib spacing equal to 6 mm. The comparison between the two indicates approximately a 53% increase in the bond index for steel bars relative to rGFRP bars, which is consistent with the higher bond stiffness observed in the initial region of the bond–slip curve, where mechanical interlock governs the bond mechanism. In addition to this geometric effect, the large difference between the elastic modulus of steel and that of the resin matrix composing the rGFRP ribs further amplifies the bond stiffness gap between the two reinforcement types.

However, excluding the above-mentioned difference in mechanical interlocking contribution, the overall bond stress–slip behavior, in terms of maximum bond stress, the descending branch, and the residual stress plateau obtained with rGFRP bars, is nearly similar to that observed for steel bars, as clearly shown in Figure 9.

On the other hand, Figure 10 and Table 9 present the comparison between the results obtained with sGFRP and CFRP bars corrected using the coefficient defined in Equation (2). Also in this case, a good degree of overlap can be observed between the two types of bars with share the same surface finishing but differ in fiber material. The bond strength values of the two specimen groups are very close, even when considering the significant statistical dispersion recorded for specimens with rCFRP bars. However, the s_1 values reported in Table 9 indicate that sCFRP bars exhibit a higher stiffness compared to sGFRP bars. In fact, for a comparable bond strength level, the slip s_1 measured for the CFRP bar was 63% lower than observed for the sGFRP bar. This difference can be attributed to the higher elastic modulus of carbon fibers, which results in stiffer bars and, consequently, reduced slip under the same bond stress conditions.

4.2. Bond Length Influence

The correlation between the ultimate pull-out load F_b and the bond length to bar diameter ratio is presented in Figure 11.

Based on the experimental results, the ultimate load F_b for deformed steel bars shows a significant increase when the bond length increases from 2.5ϕ to 5ϕ , suggesting that the transfer length of steel bars—defined as the length beyond which no further increase in pull-out capacity is observed—may be reached at approximately 5ϕ . This is further

confirmed by the observation that F_b remains unchanged between bond lengths of 5ϕ and 7.5ϕ , indicating that the full bond capacity is already mobilized at 5ϕ .

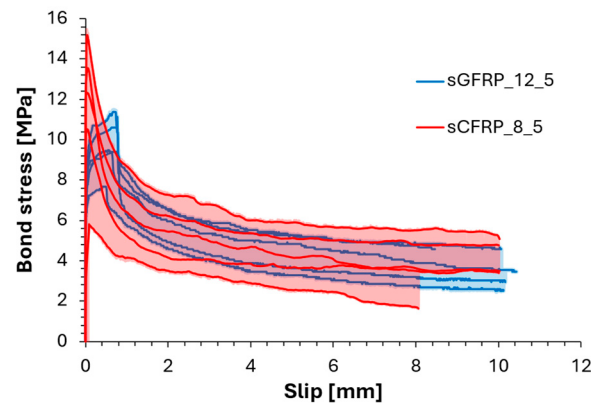


Figure 10. Comparisons between pull-out test results with sGFRP and CFRP bars with a bond length of 5ϕ .

Table 9. Adjusted values of bond strength obtained with sGFRP and CFRP bars with a bond length of 5ϕ .

Bar	Concrete	Sample	$\tau_{b,max,adj}$ [MPa]	$\tau_{b,max,adj,av}$ [MPa]	s_1 [mm]	$s_{1,av}$ [mm]
sGFRP	GPC_1 $f_c = 27.1$ MPa	sG_12_5_1	10.7	9.74 (15%) ¹	0.263	0.180 (48%) ¹
		sG_12_5_2	7.6		0.080	
		sG_12_5_3	9.6		0.285	
		sG_12_5_4	9.4		0.124	
		sG_12_5_5	11.5		0.168	
sCFRP	GPC_2 $f_c = 40.0$ MPa	C_8_5_1	15.3	11.54 (31%) ¹	0.028	0.067 (62%) ¹
		C_8_5_2	5.8		0.118	
		C_8_5_3	12.4		0.021	
		C_8_5_4	10.6		0.080	
		C_8_5_5	13.6		0.088	

¹ Coefficient of variation.

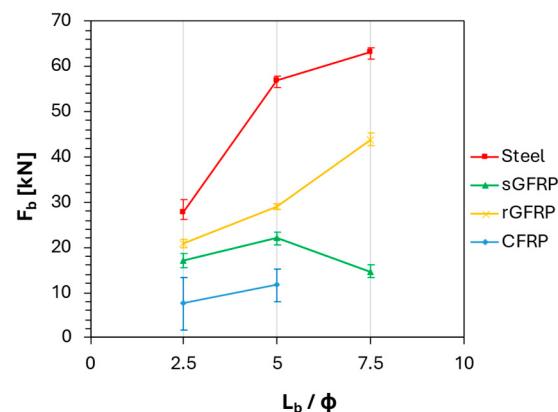


Figure 11. Average ultimate load vs. bond length on bar diameter ratio.

For sGFRP bars, the value of F_b remains relatively constant across all tested bond lengths (2.5ϕ , 5ϕ , and 7.5ϕ), suggesting that the ultimate bond capacity is not influenced by the embedded length within this range. This behavior may require further investigation to understand the underlying mechanisms responsible for the observed trend.

In contrast, rGFRP bars show a continuous increase in F_b up to a bond length of 7.5ϕ , suggesting that the transfer length for this type of bar exceeds the maximum bond length investigated and lies beyond 7.5ϕ .

4.3. Bond Stress vs. Slip Models

Four distinct models have been developed to describe the initial non-linear branch of the bond stress-slip relationship for GPC bonds, each corresponding to one of four specific bar types, and all based on experimental data. These models are based on the bond stress-slip relationship proposed by Eligehausen et al. [42]:

$$\tau = Cs^\alpha, \quad (4)$$

where τ represents the bond stress and s the slip. The parameter C is defined as $C = \tau_{max} / s_1^\alpha$, with α calibrated to fit experimental data.

Calibration results for steel bars and sGFRP bars have been previously discussed in [18] and are included here for comparison. For all specimen groups, a 95% confidence level was used, ensuring that the best-fit curve lies within the 95% confidence interval for all potential fitted lines based on the data.

Table 10 presents the calibration results, with τ_{max} values calculated as the average of experimental results within each group. Standard errors provide insight into the accuracy of the fitted values, while p -values (ranging from 0 to 1) reflect the statistical significance of the calibration. The χ^2 statistic is used to assess calibration effectiveness by quantifying the discrepancy between expected and observed results, considering the number of experimental data points. Additionally, $s_{1,cal}$ represents the maximum slip determined through the calibrated parameters, while $s_{1,exp}$ refers to the experimentally obtained value.

Table 10. Calibration results for bond stress vs. slip models.

Group ID	Bar	τ_{max} [MPa]	Parameters	Value	St. Error	p -Value	χ^2	$s_{1,cal}$ [mm]	$s_{1,exp}$ [mm]
S_12_2.5 [18]	Steel	13.1	C α	16.74 0.2556	0.275 0.008	<0.001 <0.001	3.090	0.38	0.42
sG_12_5 [18]	sGFRP	6.1	C α	8.02 0.1563	0.169 0.006	<0.001 <0.001	0.309	0.17	0.18
sC_8_5	sCFRP	8.8	C α	43.380 0.356	3.397 0.016	<0.001 <0.001	4.654	0.03	0.07
rG_10_5	rGFRP	13.7	C α	11.645 0.249	0.038 0.002	<0.001 <0.001	1.318	2.00	1.92

The low standard errors relative to the fitted values, along with p -values approaching zero, indicate a high level of statistical significance for these results. For rGFRP bars, the minimal difference of 4% between $s_{1,cal}$ and $s_{1,exp}$ further supports the validity of the model. In contrast, CFRP bars exhibited a 57% discrepancy between $s_{1,cal}$ and $s_{1,exp}$ long with a higher standard error, suggesting that this model is unsuitable for describing the bond stress-slip behavior of GPC with smooth sand-coated reinforcement bars.

The model demonstrates a good fit to the experimental data, with the calibrated slip values ($s_{1,cal}$) closely aligning with the experimentally observed values ($s_{1,exp}$) in most cases, supporting the validity of the model. However, for rCFRP bars, relatively high χ^2 values were recorded, along with a noticeable discrepancy between $s_{1,cal}$ and $s_{1,exp}$. The authors attribute this to the significant experimental dispersion observed for these samples, suggesting that further investigation is needed to better understand the results, also from a theoretical point of view. The graphs in Figure 12 illustrate the comparison between experimental data and the theoretical predictions for each bar type studied in this work.

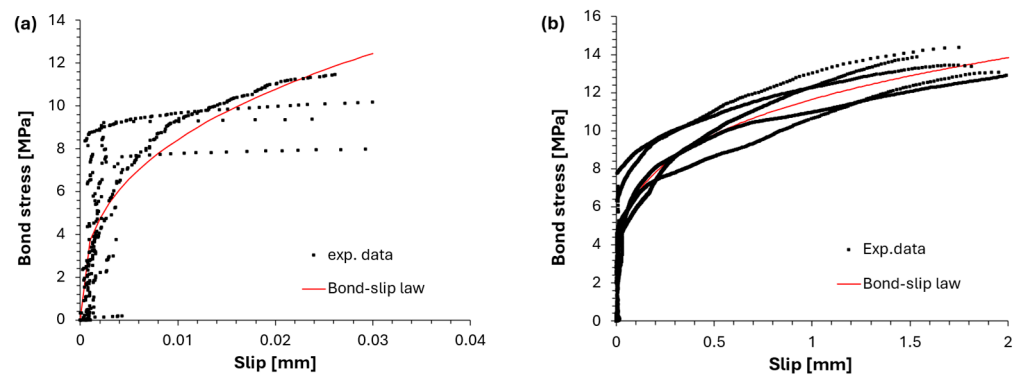


Figure 12. Experimental bond stress-slip law: (a) CFRP bar; (b) rGFRP bar.

5. Conclusions

In this study, an extensive experimental program was conducted to evaluate the bond performance between GPC and different reinforcing bars, including ribbed steel, sand-coated glass FRP, ribbed glass FRP, and sand-coated carbon FRP bars. The experimental campaign involved direct pull-out tests on two slag-based GPC mixes reinforced with these bars, aimed at assessing bond stress-slip behavior, failure modes, and residual bond stresses across a range of bond lengths and curing ages. This investigation is part of an ongoing effort to understand the suitability of GPC as an eco-friendly alternative to OPCC in reinforced structural applications.

Based on the results obtained, the following conclusion can be drawn:

- The bond strength of GPC with sand-coated FRP bars was about 60% lower than that of ribbed bars. This difference can be attributed to different bond transfer mechanisms at the interface. In the case of sGFRP and sCFRP bars, adhesion and friction were the predominant mechanisms, whereas mechanical interlocking governed the bond between GPC and both steel and rGFRP bars.
- The analysis of the bond behavior between GPC and rGFRP bars has yielded interesting results. In fact, despite the difference in the initial bond stiffening, the overall bond stress-slip curves, including the maximum bond stress, the subsequent descending branch, and the final residual stress plateau, exhibit a high degree of overlap between rGFRP and steel bars.
- Test results indicate that the ultimate load F_b for deformed steel bars increases up to a bond length of 5ϕ , suggesting that this length corresponds to the transfer length. For sand-coated GFRP bars, F_b remains constant across all analyzed bond lengths, implying no significant increase in load capacity. In contrast, ribbed GFRP bars exhibit a continuous increase in F_b , indicating that the transfer length extends beyond 7.5ϕ [42].

Further testing involving additional bar diameters could provide deeper insights into the bond behavior of GPC. The findings confirm that GPC, reinforced with both steel and ribbed GFRP bar, shows promising potential for sustainable structural applications, with bond behavior comparable to that of traditional reinforced concrete. Future research should build upon these results by conducting structural testing of GPC beam elements reinforced with FRP bars. Such investigations should focus on evaluating the bond behavior of GPC under more complex loading conditions, including flexural and shear stresses. In addition to the ongoing investigation on different reinforcement types and bond lengths in GPC, future work will include a direct comparison with OPC-based concrete. An experimental campaign is currently in progress to evaluate the bond behavior of FRP-reinforced OPCC specimens under the same conditions, allowing for a more comprehensive assessment of

the influence of the binder type on bond performance. Moreover, while this study focused specifically on bond performance, future research will also include a more comprehensive mechanical characterization of the geopolymer concrete to further support its structural use. This investigation will be crucial for advancing GPC applications in structural design, particularly for sustainable construction solutions.

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