

École polytechnique de Louvain

Bond-slip and Anchorage of Glass Fiber Reinforced Polymers Rebars for Concrete

Experimental Tests and Numerical Modelling

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Abstract

In light of the corrosion challenge faced by traditional steel rebars in reinforced concrete, the exploration of fiber-reinforced polymers (FRP) as an alternative has garnered significant interest. This thesis focuses on investigating the impact of various surface finishes on the bond stress-slip behaviour, as well as evaluating the influence of different embedment lengths ($10d_b$, $20d_b$, and $30d_b$). Additionally, a subset of specimens was instrumented with Distributed Optical Fibers Sensors (DFOS) to capture strain data along the embedment length. The experimental findings were then leveraged to refine the bond parameters of a numerical bond-slip model, capable of simulating anchorage behaviour. Notably, the results indicated that rebars with rough surfaces (such as Sand-Coated and Machining) outperformed smoother counterparts, exhibiting higher bond strengths. As for the embedment length effect, a discernible trend emerged: an increase in embedment length correlated with a decrease in bond strength due to a non-uniform distribution and the increasing presence of defects along the embedment length.

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List of Figures

2.1	Pultrusion process of GFRP (adapted from Saribiyik and Saribiyik, 2011)	8
2.2	Bond mechanisms: a)chemical adhesion, b)friction, and c)interlocking (adapted from Vint, 2012)	11
2.3	Failure mode: splitting (left) and pullout (right) failure (adapted from Sousa, 2015)	11
2.4	Failure mechanisms (adapted from J. Y. Lee et al., 2012)	12
2.5	Illustration of a pullout test principle	13
2.6	Typical pullout test setup (adapted from ISO, 2015)	13
2.7	Illustration of the average bond stress (adapted from Vint, 2012) . . .	14
2.8	BPE model (adapted from Lin and Zhang, 2014)	15
2.9	BPE modified model (adapted from Lin and Zhang, 2014)	17
2.10	Typical regions affected by strain penetrations (adapted from J. Zhao and Sritharan, 2007)	19
2.11	Behaviour of anchorage region with sufficient (left) and limited (right) embedment length (adapted from Sousa, 2015)	20
2.12	Fiber-based modeling of strain penetration effects with a zero-length element (adapted from J. Zhao and Sritharan, 2007)	20
2.13	Strain penetration modelling approaches: elongated element(left) and base spring (right) (adapted from Sousa et al., 2018)	21
2.14	Schematic representation of a Distributed Fiber Optical Sensor (adapted from Zhou et al., 2023)	22
2.15	Rayleigh Backscattering principle (adapted from Berrocal et al., 2020) .	23
3.1	Surface finishing	27
3.2	Standard cubes for concrete quality tests	29
3.3	Evolution of concrete compressive strength over time (adapted from “Properties of normal strength Portland cement concrete - CivilEngineeringBible.com”, n.d.)	31
3.4	Representation of a cubic sample	32
3.5	Spiral reinforcement	33
3.6	FRPs with their anti-adhesive region made with a PVC tube.	34
3.7	Aluminum tubes used for the anchoring device	34
3.8	Storage of the samples in a tank filled with tempered water.	35
3.9	3D representation of the cubic form	36

3.10	Schematic representation of the setup	37
3.11	Illustration of the setup	37
3.12	Schematic representation of a sample	39
3.13	Illustration of the samples	39
3.14	Schematic representation of the setup	40
3.15	Illustration of the setup	40
3.16	Representation of the cylindrical samples	41
3.17	3D representation of the cylindrical form	43
3.18	Schematic representation of the setup	44
3.19	Illustration of the setup	44
3.20	Bottom plate	45
3.21	Bottom LVDT	45
3.22	Top LVDT	45
3.23	Extensometer	46
4.1	Results of the shear strength of the silicon	48
4.2	representation of the upper bound, lower bound and mean behaviour of the silicon	48
4.3	Effect of the silicon on the on the pullout results	49
4.4	Surface finishing after test	50
4.5	Results from the pullout tests on the Calandering rebar	51
4.6	Results from the pullout tests on the Peel-ply/grid rebar	52
4.7	Results from the pullout tests on the Peel-ply rebar	53
4.8	Results from the pullout tests on the Machining rebar	54
4.9	Results from the pullout tests on the Sand-coated 16mm rebar	56
4.10	Results from the pullout tests on the Sand-coated 13mm rebar	57
4.11	Results of all valid tested GRRP	59
4.12	Influence of the embedment length on the relation between the free-end slip and the bond stress	61
4.13	Influence of the concrete compressive strength on the average bond strength	62
4.14	Evolution of the force versus the time	63
4.15	Evolution of the strain along the embedment length ($10d_b$) with time	64
4.16	Evolution of the strain along the embedment length ($20d_b$) with time	64
4.17	Evolution of the strain along the embedment length ($30d_b$) with time	65
4.18	Data recorded from the extensometer for the specimen Cal- $30d_b$ -1	66
4.19	Influence of the embedment length on the relation between the loaded- end slip and the bond stress	67
5.1	Representation of the IPs along the embedment length and the zero- length element (adapted from Sousa, 2015)	70
5.2	Simplified flowchart of the bond stress-slip numerical model (adapted from Sousa, 2015)	71
5.3	Evolution of the slip, the stresses, the force and the strains along the embedment length for steel rebars (adapted from Sousa, 2015)	71

5.4	BPE modified model (adapted from Lin et al., 2014)	73
5.5	Influence of the input parameter α	74
5.6	experimental results from Lee et al. (2012) for sand-coated and helically wrapped GFRP rebars	75
5.7	Comparison between the numerical and Lee's results for SC rebars . .	76
5.8	Comparison between the numerical and Lee's results for HW rebars .	76
5.9	Results for sand-coated rebars from Zhao et al.	77
5.10	Prediction of the numerical model for sand-coated rebars at various embedment lengths	77
5.11	Prediction of the bond-slip numerical model along the embedment length with the input parameters proposed by Lee et al.	78
5.12	Average bond stress-slip behaviour of the calendering GFRPs	79
5.13	Average bond stress-slip behaviour of the peel/ply grid GFRPs	80
5.14	Average bond stress-slip behaviour of the peel-ply GFRPs	81
5.15	Average bond stress-slip behaviour of the machining GFRPs	82
5.16	Average bond stress-slip behaviour of the sand-coated 16mm GFRPs .	83
5.17	Average bond stress-slip behaviour of the sand-coated 13mm GFRPs .	84
5.18	Numerical VS experimental results	85
5.19	Strain evolution along the embedment length ($10d_b$)	86
5.20	Strain evolution along the embedment length ($20d_b$)	87
5.21	Strain evolution along the embedment length ($30d_b$)	87

List of Tables

3.1	Summary of the tested GFRPs.	27
3.2	Summary of the samples notation	28
3.3	Required quantities for 15L of concrete	29
3.4	Results of the compressive strength for the concrete mixture with 400kg/m ³ (left) and 350kg/m ³ (right) of cement at a 14-day age . . .	30
3.5	Mean compressive strength of concrete on the cylinder at a 28-day age for the two mixtures.	31
3.6	Summary of the minimum length required for all the GFRP rebars .	33
3.7	Quantities required for 60L of the concrete	35
3.8	Results from the compressive test	36
3.9	Summary of the dimensions of the samples	38
3.10	Quantities required for 10L of the concrete mixture	39
3.11	Summary of the dimensions of the samples	41
3.12	Quantities required for 155L of the concrete mixtures	42
3.13	Results from the compressive test	42
4.1	Experimental results	49
4.2	Bond strength and slip data of ribbed GFRP rebars from the literature	55
4.3	Bond strength and slip data of Sand-coated GFRP rebars from the literature	58
4.4	Summary of all valid experimental results	60
4.5	Summary of all valid experimental results	68
5.1	Bond-slip input parameters proposed by J. Y. Lee et al., 2012	75
5.2	Summary of the bond parameters of the numerical model for GFRP rebars	88

List of Symbols

a	= Free length
α	= Parameter of the ascending branch of the BPE modified model
d_b	= Rebar's nominal diameter
e	= Free-end length
E	= Young's modulus
f_c	= concrete compressive strength
$f_{c,14}$	= Concrete compressive strength on cylinder at a 14-day age
$f_{c,14,cube}$	= Concrete compressive strength on cube at a 14-day age
$f_{cm,14,cube}$	= Concrete mean compressive strength on cube at a 14-day age.
$f_{c,cube}$	= Concrete compressive strength on cube at a 28-day age
f_{cm}	= Concrete mean compressive strength on cylinder at a 28-day age
$f_{cm,cube}$	= Concrete mean compressive strength on cube at a 28-day age
f_t	= Concrete tensile strength
f_y	= Yielding stress of steel reinforcement
L_{anch}	= Anchorage length
L_e	= Embedment length
L_d	= Development length
L_{SP}	= Strain penetration length
S	= Slip corresponding to τ bond stress
S_0	= Slip demand for the numerical model
S_1	= The free-end slip at the peak bond stress
S_3	= The free-end slip at the start of the residual stress
S_f	= Bar slip at free end
S_l	= Bar slip at loaded end
τ	= Average bond stress
τ_{max}	= Average bond strength
τ_f	= The residual or frictional bond stress
ϵ_{yd}	= Yielding strain of steel reinforcement

Contents

Abstract	I
Acknowledgements	II
List of Figures	III
List of Tables	VI
List of Symbols	VII
1 Introduction	1
1.1 Motivations	1
1.2 Objectives	2
1.3 Research novelties	2
1.4 Thesis organization	4
2 Literature review	7
2.1 Introduction to Glass-Fiber Reinforced Polymer (GFRP)	7
2.2 Manufacture process of GFRP rebars	8
2.3 Mechanical and bond properties	9
2.4 Bond mechanisms and failure mode	10
2.4.1 Pullout failure	11
2.4.2 Splitting failure	12
2.5 Pullout method for bond evaluation	12
2.6 Bond stress-slip relation	14
2.6.1 BPE model for steel rebars	15
2.6.2 Malvar's Model for FRP rebars	16
2.6.3 BPE-modified model for FRP rebars	17
2.7 Strain penetration effects	19
2.8 Distributed Fiber Optical Sensor (DFOS)	22
3 Experimental tests on GFRP rebars	25
3.1 Introduction	25
3.1.1 Characterization of the bond behaviour	25
3.1.2 Influence of the embedment length	26

3.2	Presentation of the tested rebars	26
3.3	Concrete study	28
3.4	Preparation of cubic samples	31
3.4.1	Presentation of the samples	31
3.4.2	Setup and instrumentation	37
3.5	Determination of the shear strength of the silicon	38
3.5.1	Presentation of the samples	38
3.5.2	Setup and instrumentation	40
3.6	Preparation of the cylindrical samples	40
3.6.1	Presentation of the samples	40
3.6.2	Setup and instrumentation	44
4	Analysis and discussion of the experimental results	47
4.1	Results from the silicon shear strength	47
4.2	Results from the pullout test with the cubic samples	50
4.2.1	Summary of the first set of tests	59
4.3	Results from the pullout tests with the cylindrical samples	61
4.3.1	Analysis of measurements taken by the LVDTs	61
4.3.2	Analysis of measurements taken by the DFOS	63
4.3.3	Summary of the second set of tests	68
5	Bond-slip numerical model	69
5.1	General description of the bond-slip model	69
5.2	Input parameters of the numerical model	72
5.2.1	Geometric properties	72
5.2.2	Material properties	72
5.2.3	Bond properties	73
5.3	Literature benchmark	74
5.4	Outputs of the numerical model	78
5.5	Adjustments and validation of the input parameters	79
5.6	Contrasting Empirical Findings with Model Projections	85
5.7	Summary	88
6	Conclusion and future developments	91
6.1	Conclusion	91
6.2	Future developments	92
	Bibliography	93

Chapter 1

Introduction

1.1 Motivations

Over the past few decades, numerous structures comprising reinforced concrete (RC) have been constructed, including bridges, buildings, roads, and various other civil infrastructures. Presently, these infrastructures demand regular maintenance to fulfill current requirements and extend their service life. One significant concern with these existing structures is the corrosion of steel reinforcement, attributed in part to the rise in atmospheric carbon dioxide concentration (J.-Y. Lee et al., 2017). Corrosion can lead to the deterioration of the mechanical properties of steel reinforcing bars (rebars) and eventually result in structural failure, necessitating expensive repairs.

To combat corrosion, various methods are employed, including the use of stainless steel reinforcement, cathodic protection systems, concrete additives, and epoxy coatings, which are among the most frequently used techniques. However, it has been reported that the bond strength between coated bars and concrete is significantly reduced when using epoxy coating method (J.-Y. Lee et al., 2017). Regrettably, these corrosion inhibiting methods are often quite costly and only provide a gradual deceleration of corrosion effects (Vint, 2012).

Addressing the corrosion concern, composite materials comprising Fiber Reinforced Polymers (FRP) have been developed for implementation in the construction industry. This innovative material is being studied as a potential replacement for steel rebars embedded in concrete. Various aspects, such as bending and shear performance, material properties, and bond behaviour, have been extensively investigated over the last two decades. However, despite these studies, no standardized regulations have been established yet. Consequently, a wide range of commercially available FRP rebars exists, making it challenging to generalize their behaviour (Pecce et al., 2001).

For decades, the bond between steel and concrete has been studied, while there is a lack of data for Glass-fiber reinforced polymers (GFRP). Since the mechanical and physical properties are different from steel, the bond behaviour can not be assumed similar. Therefore, experimental data is needed to accept GFRP as a replacement for steel reinforcing bars and to develop reliable guidelines for design.

1.2 Objectives

The thesis is structured around three objectives, which encompass both experimental and numerical aspects.

- The initial phase of the experiments focuses on conducting pullout tests to characterize the bond-slip behavior of Glass-Fiber Reinforced Polymer (GFRP) bars featuring different surface finishes. In pursuit of this objective, meticulous adherence to the ISO 10406-1 standard was maintained, with the fabrication of cubic specimens by the prescribed methodology.
- Second part of the study focuses on investigating the impact of various embedment lengths on the bond stress-slip behaviour. These distinct embedment lengths will be $10d_b$, $20d_b$, and $30d_b$, with d_b representing the nominal diameter of the rebar. In contrast to the preceding tests, specimens will adopt a cylindrical shape for this phase of experimentation.
- Drawing upon the outcomes derived from the experimental campaign, the numerical facet of this study is geared towards calibrating the input parameters of the model. By supplying the model with appropriate inputs, it will be capable of faithfully simulating the behaviour exhibited by each reinforcing bar. Furthermore, the model is equipped to anticipate the progression of Slip (S), Force (F), Stress (σ), and Strains (ϵ) along the embedment length. This model's predictions must be validated through a comparison with experimental findings.

1.3 Research novelties

This thesis embarks on a comprehensive investigation into the behaviour of Glass-Fiber Reinforced Polymer (GFRP) rebars in concrete structures, aiming to address critical challenges associated with conventional steel rebars. The research unfolds novel contributions and innovations in the field of reinforced concrete, offering insights that can potentially revolutionize construction practices. Several key research novelties are presented below:

Advancing the Use of GFRP Rebars

While GFRP rebars have been gaining traction as a promising alternative to traditional steel reinforcement, this study endeavors to deepen our understanding of their performance under various conditions. By characterizing the bond behaviour and mechanical properties, this research contributes to the broader acceptance and implementation of GFRP rebars in civil engineering projects.

Investigation of Surface Conditions

The thesis explores the influence of different surface conditions on the bond-slip behaviour of GFRP rebars. Uniting experimental pullout tests and advanced numerical simulations, this examination sheds light on the effect of surface treatments on the interface between GFRP rebars and concrete. Such insights are critical for optimizing bonding mechanisms and enhancing the overall durability of GFRP-reinforced structures.

Novel Experimental Setup and Instrumentation

In the pursuit of accurate data acquisition and analysis, this research introduces a novel experimental setup using Distributed Fiber Optical Sensors (DFOS) to monitor the bond performance. This state-of-the-art instrumentation offers high-resolution measurements of strain evolution along the rebar, providing valuable real-time information during pullout tests.

Influence of Embedment Length

Through an extensive experimental campaign, this thesis delves into the influence of different embedment lengths on the bond behaviour of GFRP rebars. The findings help establish optimal embedment practices to ensure reliable and efficient load transfer between GFRP rebars and the surrounding concrete matrix.

Implications for Practical Applications

The outcomes of this research carry significant implications for the construction industry, offering new insights into the design and implementation of GFRP-reinforced concrete structures. The knowledge gained from this study can guide engineers, architects, and policymakers in adopting sustainable and resilient construction practices.

By addressing these research novelties, this thesis aims to enrich the existing body of knowledge surrounding GFRP rebars, and their integration into reinforced concrete structures. The results presented herein contribute to the ongoing advancement of civil engineering practices and lay the groundwork for further exploration in the field of composite materials for structural applications.

1.4 Thesis organization

Following this introductory section, Chapter 2 will focus on the literature review. This chapter will provide an overview of a new type of reinforcing bars, covering various aspects such as manufacturing, mechanical properties, bond mechanisms, and bond-slip behaviour. Furthermore, a dedicated section will focus on the pull-out method, a widely used approach for evaluating the bond relationship between concrete and reinforcing bars embedded within concrete prisms or cylinders. The concept of strain penetration effects will be elucidated, followed by an investigation into the use of optical fiber sensors, which will play a vital role in a specific segment of the experimental tests.

Chapter 3 will delve into the experimental campaign, divided into three parts. The initial portion is dedicated to the creation of cubic samples, serving as the foundation for characterizing the bond behaviour of each rebar type and identifying the factors influencing this behaviour. The methodology for this part adheres to the guidelines stipulated in the ISO 10406-1 standard. A second part emerged due to operational oversight, necessitating the fabrication of additional specimens to ascertain shear resistance of the silicon utilized in the first phase. The final segment revolves around a description of cylindrical specimens, meticulously crafted to investigate the impact of varying embedment lengths. Within each part, a comprehensive presentation and elucidation of the samples are provided, accompanied by an overview of the testing setup and instrumentation used.

Chapter 4 is dedicated to the meticulous analysis and comprehensive discussion of the experimental findings. The chapter's trajectory commences with a thorough investigation into the shear resistance of the silicon material, subsequently transitioning to a meticulous exploration of the behaviour exhibited by each rebar type. The final segment of this chapter delves into an in-depth analysis of the influence exerted by varying embedment lengths. The juxtaposition of the obtained results against those accessible within the existing literature forms an integral part of this chapter's endeavour, enabling the derivation of overarching conclusions regarding the bond behaviour exhibited by Glass-Fiber Reinforced Polymers (GFRPs).

Chapter 5 will center around the bond-slip numerical model. It will provide a general introduction to the proposed model, outlining the input parameters it requires. This section will compare the results obtained from numerical simulations with experimental findings from various authors documented in the literature. Finally, by analyzing the results obtained from conducted experiments, the input data for each GFRP rebar will be adjusted accordingly.

In the concluding chapter, the essential results and observations will be summarised. The section will also discuss potential future developments to further enhance and expand upon the research presented in this thesis.

Chapter 2

Literature review

2.1 Introduction to Glass-Fiber Reinforced Polymer (GFRP)

For decades, steel has been the primary reinforcement material used in reinforced-concrete (RC) structures, owing to concrete's low tensile strength. To withstand tensile loads, a material with excellent tensile behaviour is necessary. To transfer forces between concrete and steel rebars, a strong bond between the two materials must be established. This can be achieved by deforming the rebar's surface or adjusting the embedment length to reach its full capacity (Liao et al., 2023). One significant challenge associated with steel reinforcing bars is corrosion, which can cause concrete structure degradation and decrease its service life. Corrosion can be caused by external loading, freeze-thaw cycles, salty air near coastal areas, or an alkali environment. Corrosion impacts the mechanical properties of rebars and can result in significant structural damages, ultimately leading to structural failure. Various techniques exist to prevent or reduce the corrosion process, including coating of steel rebar with epoxy, increasing concrete cover, or adding agents to the concrete mixture to improve its material properties such as its durability or permeability (Moon et al., 2007).

As durability has become a critical concern, alternatives to steel rebars are being sought today. This task is challenging because it requires the identification of a material with excellent mechanical properties that is resistant to corrosion. Fibre-reinforced polymers (FRPs) have emerged as a promising replacement for steel reinforcing bars due to their superior mechanical properties, low cost, and high strength-to-weight ratio, as evidenced by Zhao's research in 2022. FRPs are primarily composed of fibers as structural elements and a polymer binder such as polyester, vinyl-ester, or epoxy, according to Liao et al (2023). Since FRPs effectively address the corrosion problem, they enable the use of seawater and sea sand in the concrete mixture, which reduces freshwater and river sand consumption (Liao et al., 2023; X. Zhao et al., 2022).

The fields of application of this type of reinforcing bars are very diverse. According to various authors, such rebars were used in bridge decks where the use of de-icing salts induces rapid corrosion of steel rebars or where there is a presence of an electromagnetic field like tunnel lining, structures serving rail transportation, or hospitals rooms to avoid electromagnetic interference (Spagnuolo et al., 2021; X. Zhao et al., 2022). Furthermore, in highly aggressive regions, such as coastal places or in salty waters, it is recommended to use them instead of steel rebars.

The four types of FRPs available are glass-fiber reinforced polymers (GFRP), carbon-fiber reinforced polymers (CFRP), basalt-fiber reinforced polymers (BFRP), and aramid fiber-reinforced polymers (AFRP) (Liu et al., 2021). GFRPs are the most commonly used due to their cost-effectiveness and diverse range of properties. Additionally, a variety of GFRP bars can be found, such as Sand-Coated or deformed with a ribbed surface GFRP, and more.

2.2 Manufacture process of GFRP rebars

The primary method used in the manufacturing of Glass-Fiber Reinforced Polymers is pultrusion. This process has proven itself the best method for obtaining lengths of high quality and offers flexibility to produce a wide range of cross-sectional shapes, including circular rebars, I-section profiles, hollow profiles, and many more (Saribiyik and Saribiyik, 2011). As noted in Moon's research in 2007, pultrusion is the preferred manufacturing method for commercial GFRPs because of its efficient operation, high-quality output, and low equipment costs (Moon et al., 2007). Figure (2.1) provides an illustration of the pultrusion process used to fabricate such profiles.

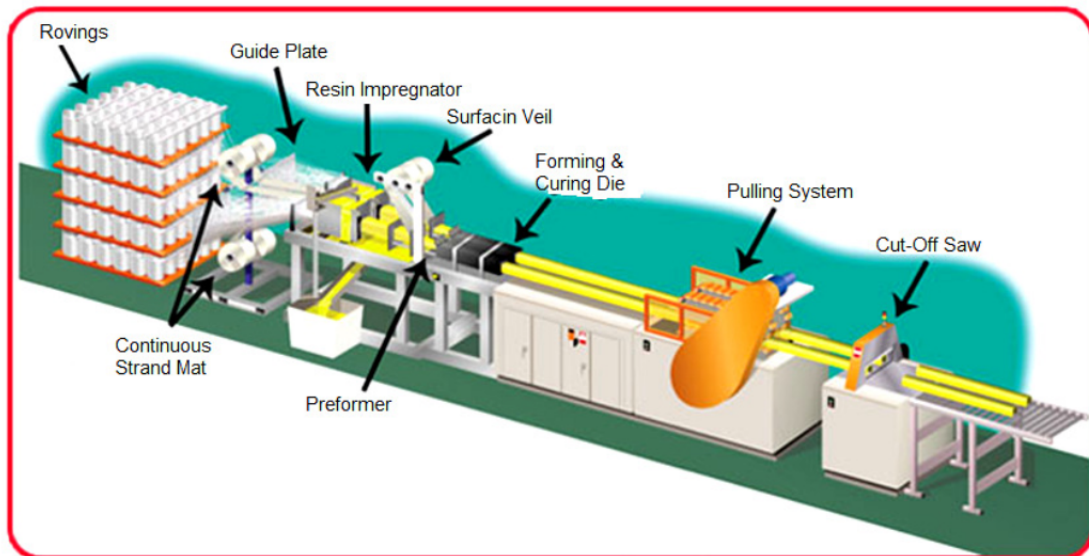


Figure 2.1: Pultrusion process of GFRP (adapted from Saribiyik and Saribiyik, 2011)

Fiber-Reinforced Polymers (FRPs) consist of continuous glass filaments embedded within a matrix material, which can be composed of epoxy, vinyl ester, thermoplastic, or polyester resin. These filaments are available in various types commonly used in FRP applications, such as E-glass (low alkali glass), E-CR glass (Electric/Chemical Resistance), S-glass (high tensile strength), or AR-glass (Alkali Resistant). The matrix material serves two main purposes: first, it holds individual filaments together, allowing the load to be distributed among them, and then, it provides protection to fibers against external aggressors (Spagnuolo et al., 2021).

The manufacturing process for these composite elements is depicted in Figure 2.1. The process begins with glass fiber rovings, which are unrolled to form parallel filaments. These filaments are then passed through a resin bath and subsequently through a heated die, where they are shaped into the desired continuous cross-sectional form and cut to the desired length (Saribiyik and Saribiyik, 2011).

In the existing literature, the two common types of GFRPs are sand-coated (SC) rebars and helically wrapped (HW) rebars. SC-GFRP rebars are manufactured by securely attaching sand particles to the smooth surface of a pultruded rebar using resin. On the other hand, GFRP-HW rebars are produced by deforming a smooth surface with a helical winding of glass fiber strings before the resin sets (J. Y. Lee et al., 2012).

2.3 Mechanical and bond properties

Fiber-reinforced polymers have been studied for a long time but are not yet used as the main reinforcement in RC structures. Their mechanical properties are still unknown and a general formulation to predict the bond behaviour between FRP rebars and concrete has not yet been reached. Experimental tests are therefore performed such as pull-out or beam tests.

Compared to steel reinforcing bars, GFRPs have a smaller modulus of elasticity. According to Zhao et al. (2022), this can be four to five times smaller, which for the same reinforcing ratio leads to a lower stiffness of the GFRP reinforcement causing larger deformations and more major concrete crackings when loaded (X. Zhao et al., 2022). Furthermore, glass-fiber reinforced polymers seem to have a linear stress-strain relationship and thus no yielding point, leading to a more fragile behaviour at failure. Moreover, the fire resistance of such rebars is very poor, giving issues that need to be solved before they can actually be used as a main reinforcement (Liu et al., 2021).

Despite a higher tensile strength compared to steel, another problem is that mechanical properties are different under compression and tension. Studies have shown that the compressive strength is about 50% to 60% of the tensile strength and even less under cyclic loading. Kobayashi and Fujisaki (1995) have conducted some experiments and concluded that the compressive strength under cyclic loading was 20% to 50% of that under monotonic loading (Kobayashi and Fujisaki, 1995).

According to Vint's thesis in 2012, the average bond strength of Glass-Fiber Reinforced Polymer (GFRP) rebars is approximately 55% to 90% of steel reinforcement with the same diameter, which can be attributed to a lower shear strength (Vint, 2012). Lee et al. (2017) examined the influence of rebar diameter on bond behaviour. Their findings indicated that as the rebar diameter increases, the bond strength of GFRP decreases at a higher rate compared to steel bars. This phenomenon can be attributed to several factors, including (i) the bleeding effect (Benmokrane et al., 1996), (ii) a more favourable ratio between the external surface area of the rebar and its cross-section, or (iii) the size effect. However, this phenomenon was mitigated as the concrete compressive strength increased (J.-Y. Lee et al., 2017).

In contrast to steel reinforcement, an increase in concrete compressive strength does not significantly affect the bond strength of GFRP rebars, as it is primarily governed by the shear strength of the resin. Instead, the concrete compressive strength tends to influence the failure mode during the pull-out (Vint, 2012). For concrete compressive strengths greater than 30 MPa, the bond failure occurs at the surface of the FRP rebar. On the other hand, for lower concrete compressive strengths (around 15 MPa), the failure occurs at the interface between the FRP rebar and the concrete matrix. In other words, as the concrete compressive strength increases, more damage occurs in the GFRP rebar, and less damage occurs in the surrounding concrete, and vice versa (Baena et al., 2009).

2.4 Bond mechanisms and failure mode

A bond mechanism between a reinforcing bar and surrounding concrete is needed to combine the action of the two materials and to assure the transfer of the forces between them. This bond is due to three main mechanisms, as illustrated in figure 2.2.

- Chemical adhesion between the rebar and surrounding concrete
- Frictional forces
- Mechanical interlocking between lugs and concrete

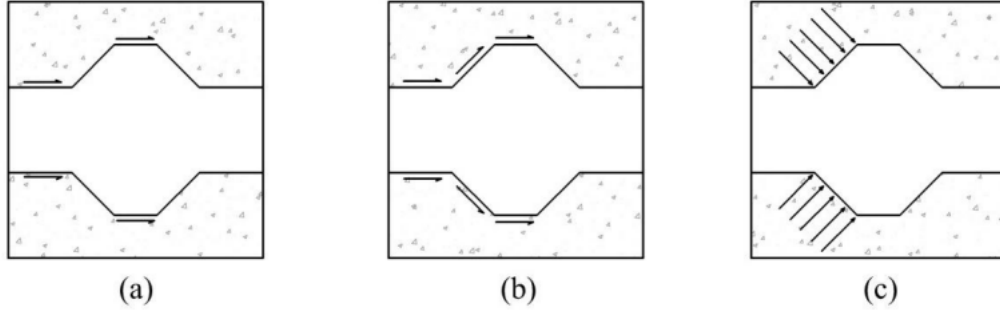


Figure 2.2: Bond mechanisms: a)chemical adhesion, b)friction, and c)interlocking (adapted from Vint, 2012)

When a reinforcing bar is embedded in concrete and undergoes tensile forces, the rebar will tend to become loose and slip out of the concrete. Splitting and pullout failure are two main failure mechanisms that can occur and depend on the level of confinement and concrete cover (Fig. 2.3).

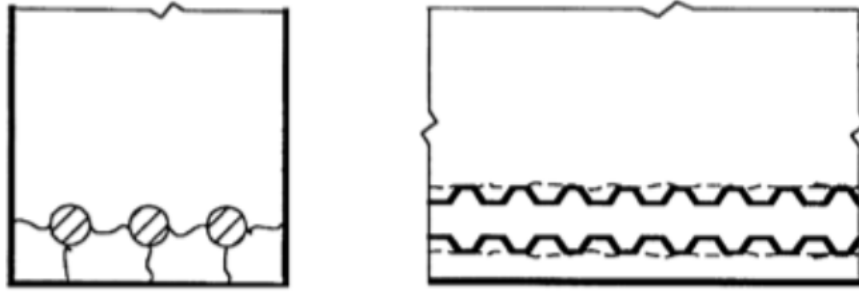


Figure 2.3: Failure mode: splitting (left) and pullout (right) failure (adapted from Sousa, 2015)

2.4.1 Pullout failure

Pullout failure is defined as the pulling of the rebar out of the concrete without any cracks in the surrounding concrete (Waleed and Bilal, 2022). If transverse pressure and concrete cover are sufficient, the system will fail by shearing either (i) at the interface between concrete and the rebar's surface or (ii) at the interface between the rebar's core and the outer surface as illustrated in figure 2.4. Pullout failure of GFRP can thus occur if one or more than these following conditions are full-filled (J. Y. Lee et al., 2012).

- Concrete near the deformed surface crushes and shear cracks form
- Surface modification of the FRP rebar shears off (Interface 1 delamination)

- Rupture of the interface between the fiber core and outer resin of the FRP (Interface 2 delamination)

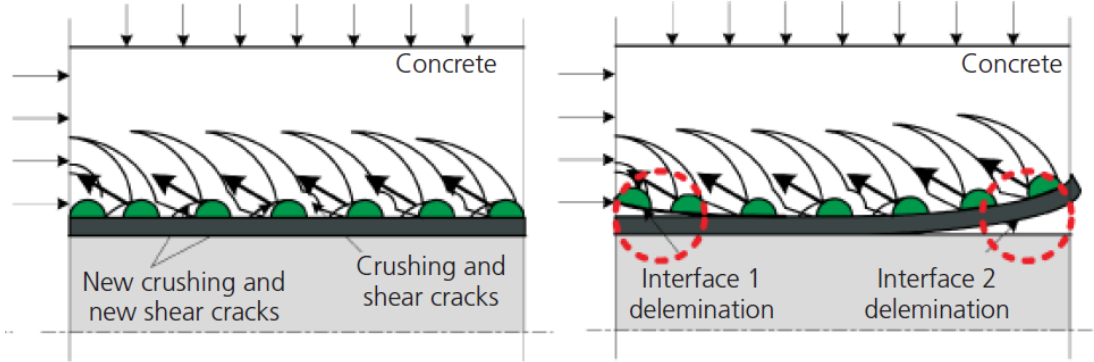


Figure 2.4: Failure mechanisms (adapted from J. Y. Lee et al., 2012)

2.4.2 Splitting failure

Splitting failure of surrounding concrete is the failure mode where radial stresses exceed concrete tensile strength. Studies have proven that a longer embedment length will lead to splitting failure, as well as increase the rebar's diameter (Waleed and Bilal, 2022). If transverse reinforcement and concrete tensile strength are insufficient, this failure mode is more likely to occur.

2.5 Pullout method for bond evaluation

Pullout tests are typically used to assess bond resistance between two materials. The operating principle is illustrated in figure 2.5, where we see a block, generally made of concrete, with a reinforcing bar embedded in it. A sleeve, often made of PVC, is used to create an unbounded region, to reduce the amount of normal stress at this point, and to prevent concrete from cracking. To evaluate shear resistance, a pullout load is applied to the rebar to pull it out of the concrete at a certain rate. Among others, those tests are normalized by the ISO standard, which stands for International Organisation for Standardisation, or the ASTM standard, which stands for American Society for Testing Materials. Both standards have their own version of the pull-out test and differ in a number of respects. For instance, the samples dimensions or the tested length which is $4d_b$ for the ISO 10406-1 procedure (ISO, 2015) and $5d_b$ for the ASTM D7913 (ASTM, 2020).

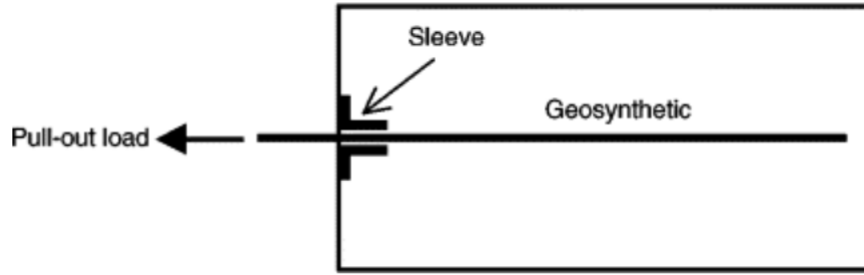


Figure 2.5: Illustration of a pullout test principle

One typical pullout setup is shown in Figure 2.6 where a Linear Variable Differential Transformer (LVDT) is often used to measure the free-end slip of the bar. However, this method may not provide highly reliable results for determining the real bond behaviour, since the loading pad induces compression stresses in the concrete, whereas, in reality, concrete will also be subjected to tension variations (Vint, 2012). This method tends to overestimate the bond stress and is therefore more suitable for comparison purposes and investigating the influence of different parameters, such as the rebar's diameter (d_b), surface finishing, and concrete strength. Another limitation is splitting failure that can occur during the testing, leading to inaccurate results (Vint, 2012; Waleed and Bilal, 2022).

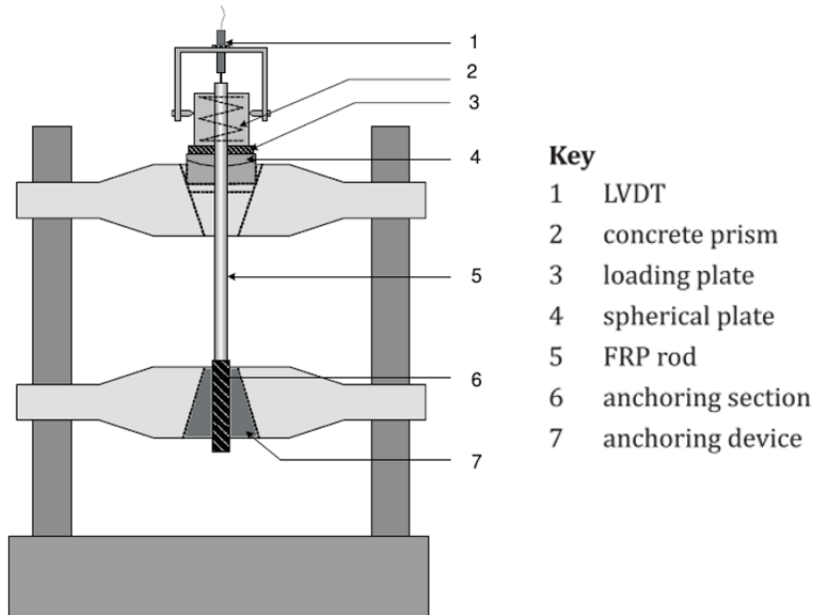


Figure 2.6: Typical pullout test setup (adapted from ISO, 2015)

Results from pullout tests are typically presented with the average bond stress, which can be expressed by equation 2.1, where P is the applied load, d_b the rebar's nominal diameter and L_e the embedment length. Since the stresses have a non-uniform distribution along the embedment length, an average is thus calculated between the free-end and the loaded-end of the rebar as illustrated in figure 2.7.

$$\tau = \frac{P}{\pi d_b L_e} \quad (2.1)$$

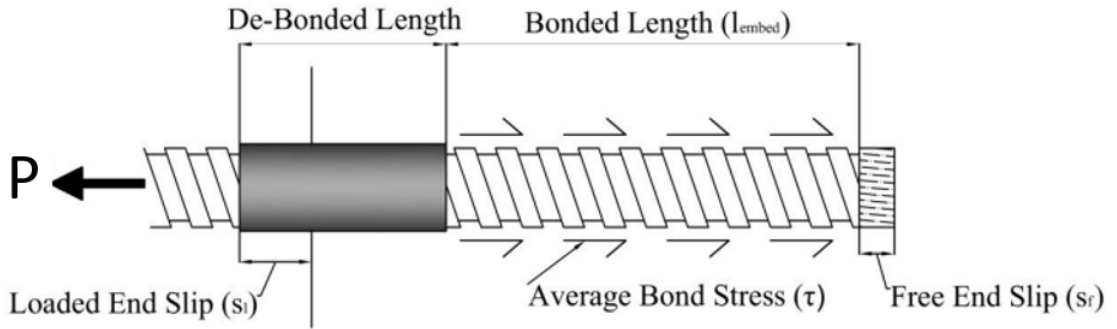


Figure 2.7: Illustration of the average bond stress (adapted from Vint, 2012)

2.6 Bond stress-slip relation

In the past years, many studies have been made on the development of a bond stress-slip model on which FRP design should be based. The bond between FRP and the surrounding concrete depends on various parameters such as the embedment length, rebar's diameter, concrete strength, confining stress, surface condition, and many more. A lot of these models are based on the one proposed by Eligehausen et al. in 1983 for deformed steel rebars, known as the Bertero-Popov-Eligehausen (BPE) model (See Figure 2.8). Malvar (1995), Cosenza et al. (1995), and Tighiouart et al. (1998) have each proposed a different version of a bond stress-slip relation between FRP and concrete (J. Y. Lee et al., 2012).

2.6.1 BPE model for steel rebars

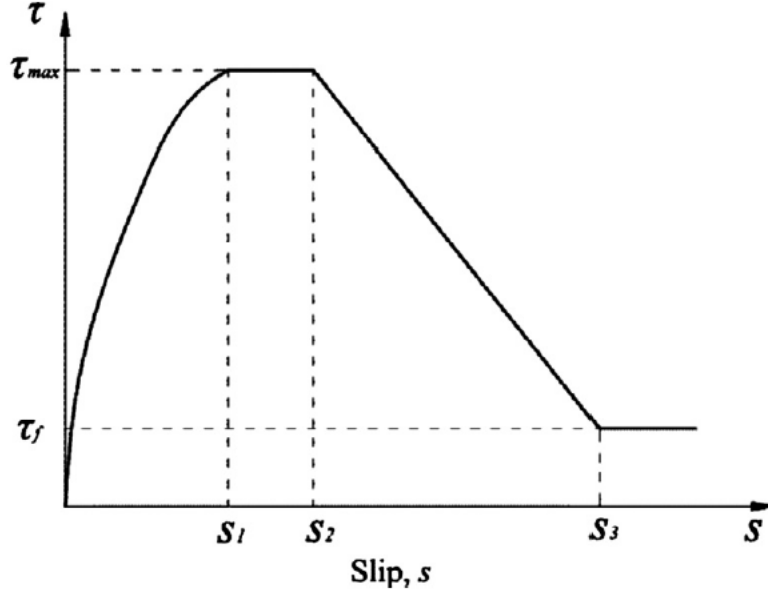


Figure 2.8: BPE model (adapted from Lin and Zhang, 2014)

This model, developed by Eligehausen et al. in 1983, expresses the relationship between the slip (S) and the average bond stress (τ) for steel rebars. This relation can be decomposed into four parts and each part is expressed by the following equations.

- $0 \leq S \leq S_1$

$$\tau(S) = \tau_{max} \left(\frac{S}{S_1} \right)^\alpha \quad (2.2)$$

- $S_1 \leq S \leq S_2$

$$\tau(S) = \tau_{max} \quad (2.3)$$

- $S_2 \leq S \leq S_3$

$$\tau(S) = \tau_{max} - (\tau_{max} - \tau_f) \left(\frac{S - S_2}{S_3 - S_2} \right) \quad (2.4)$$

- $S \leq S_3$

$$\tau(S) = \tau_f \quad (2.5)$$

In the above equations, τ_{max} represents the average peak bond stress, the slip S_1 is the slip associated with the chemical adhesion, the slip S_2 is associated with the peak average bond stress, and the slip S_3 with the residual or frictional bond stress τ_f .

The first part of this relationship, when the slip range between 0 and S_1 , is an ascending curve that described the chemical adhesion of the steel rebar with the surrounding concrete (Lin and Zhang, 2014), where the fitting parameter α affects the curvature of this branch. The next part is a plateau where the bond stress is at its peak, called the bond strength (τ_{max}), followed by a linear descending branch and a final horizontal branch with a constant response equal to τ_f .

Some analytical expressions have been derived to determine the bond strength. The American Concrete Institute design code ACI-440 suggested equation 2.6 and Fujii in 1992 proposed the equation 2.7 (J. Y. Lee et al., 2012).

$$\tau_{max} = \frac{13.5(f_c)^{0.5}}{d_b} \quad (2.6)$$

$$\tau_{max} = 1.69(f_c)^{0.85} \quad (2.7)$$

In 1992, Fujii also proposed an analytical expression for the slip value S_1 for steel reinforcing bars.

$$S_1 = 0.011d_b \left(\frac{2}{(f_c/33) + 1} \right) \frac{d_b}{h} \left(\frac{l}{d_b} \right)^{1.6} \quad (2.8)$$

2.6.2 Malvar's Model for FRP rebars

The first FRP bond stress-slip relation was introduced by Malvar in 1995 (Malvar, 1995). The bond stress-slip curve is a two-step procedure. The first one aimed at defining the average peak bond stress (τ_{max}) and its associated slip (S_1) derived by equations 2.9 and 2.10 (Lin and Zhang, 2014).

$$\frac{\tau_{max}}{f_t} = A + B(1 - e^{-C\sigma_r/f_t}) \quad (2.9)$$

$$S_1 = D + E\sigma_r \quad (2.10)$$

Where A, B, C, D, and F are empirical constants for different bar types, f_t is the concrete tensile strength, and σ_r is the confining axisymmetrical radial pressure.

The second part is represented by equation (Eq. 2.11), which finalizes the bond stress-slip curve. This curve is determined by the slip S and σ_r , with F and G being the empirical constants.

$$\tau = \tau_{max} \frac{F(\frac{S}{S_1}) + (G - 1)(\frac{S}{S_1})^2}{1 + (F + 2)(\frac{S}{S_1}) + G(\frac{S}{S_1})^2} \quad (2.11)$$

This model was specifically developed to represent the local bond stress-slip relationship of FRPs. The empirical constants used in this model were derived from experimental results, making it valid only for certain types of bars. As a result, this model cannot be applied in a generalized manner (Lin and Zhang, 2014).

2.6.3 BPE-modified model for FRP rebars

Cosenza et al. (1995) proposed a modified model based on the BPE model initially introduced by Elgehausen et al. (1983). This modification was achieved through extensive experimental studies, which led to the adaptation and adjustment of parameters. The representation of the model can be observed in Figure 2.9, and can be divided into three distinct parts, with the plateau segment being eliminated. The first part corresponds to the ascending branch and is mathematically described by Equation 2.12, the second segment forms a descending linear branch represented by Equation 2.13 and finally there is a constant segment, expressed through Equation 2.14.

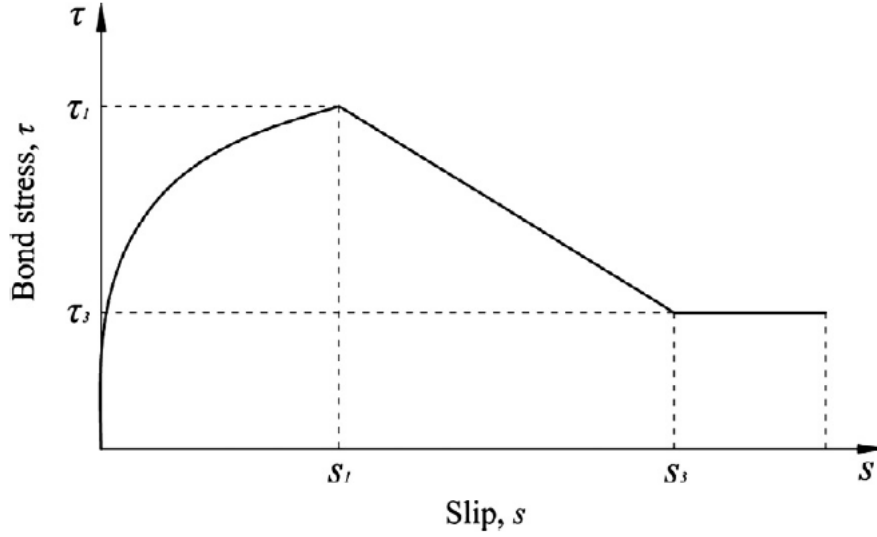


Figure 2.9: BPE modified model (adapted from Lin and Zhang, 2014)

- $0 \leq S \leq S_1$

$$\tau(S) = \tau_{max} \left(\frac{S}{S_1} \right)^\alpha \quad (2.12)$$

- $S_1 \leq S \leq S_3$

$$\tau(S) = \tau_{max} - (\tau_{max} - \tau_f) \left(\frac{S - S_3}{S_1 - S_3} \right) \quad (2.13)$$

- $S \leq S_3$

$$\tau(S) = \tau_f \quad (2.14)$$

Bond strength (τ_{max})

Lee et al. (2012) conducted some experiments on two commonly used types of GFRP rebars, known as sand-coated (SC) and helically wrapped (HW) rebars. They have shown that the maximal bond stress (τ_{max}) increases proportionally with an increase of f_c but at different rates due to the associated failure mechanism following the rebar's type (Steel, SC-GFRP or HW-GFRP). The ascending branch of the BPE-modified model is mainly governed by the adhesion of the GFRP surface with the concrete matrix. Multiple experimental studies have been made to predict the influence of concrete strength on the maximal bond stress of GFRP rebars and Okelo and Yuan (2005) have come across the following equation, with f_c as the concrete strength:

$$\tau_{max} = 0.749f_c^{0.5} \quad (2.15)$$

It is evident that equations 2.6, 2.7, and 2.15 are derived from the fundamental equation 2.16.

$$\tau_{max} = \kappa f_c^x \quad (2.16)$$

All these empirical formulas led Lee et al. (2012) to derive their own expression by comparing all the GFRP bond strengths available in the literature and came across equation 2.17.

$$\tau_{max} = 4f_c^{0.33} \quad (2.17)$$

Slip at the peak bond stress (S_1)

Lee et al. (2012) have also developed an expression for the slip S_1 , based on observations and least-square error analysis for pullout failure.

- For SC-GFRP rebars

$$S_1 = \left(\frac{\tau_{max}}{f_c} \right) \left(\frac{L_e}{60} \right) \quad (2.18)$$

- For HW-GFRP rebars

$$S_1 = \left(\frac{\tau_{max}}{f_c} \right) \left(\frac{L_e}{8} \right) \quad (2.19)$$

Residual bond strength (τ_f) and it's associated slip (S_3)

Following the bond peak, there is a linear decrease in bond stress until reaching S_3 , a point depending on the rebar type, where the residual bond strength (τ_f) is achieved. On average, Lee et al. (2012) demonstrated that this residual bond strength is approximately half of τ_{max} .

$$\tau_f = 0.5\tau_{max} \quad (2.20)$$

Unfortunately, no expression has yet been found to evaluate the slip S_3 associated with the residual stress for FRP rebars.

2.7 Strain penetration effects

When reinforced concrete (RC) members are subjected to flexural loading, local deformations occur and the more sensitive regions are beam-column or footing-column joints (See fig 2.10).

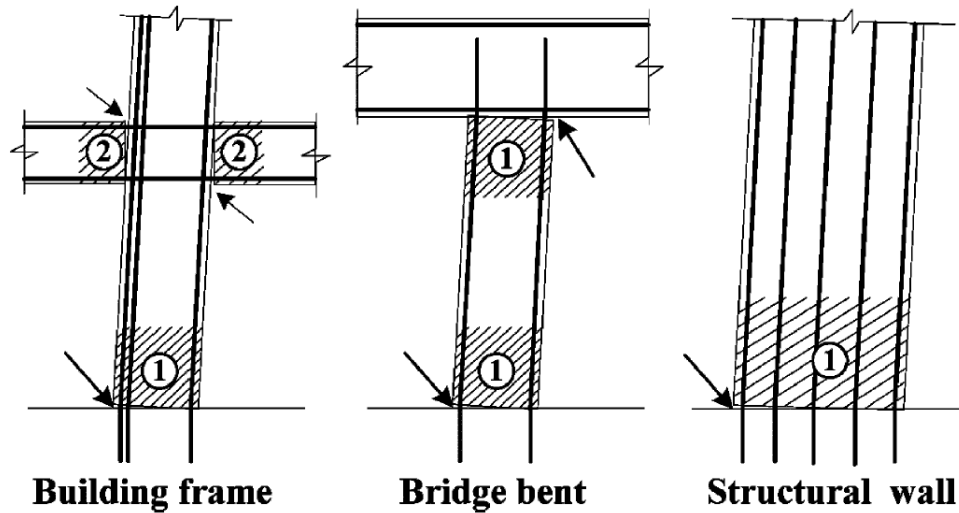


Figure 2.10: Typical regions affected by strain penetrations (adapted from J. Zhao and Sritharan, 2007)

The reinforced bar (rebar) should be sufficiently embedded in order to transfer its axial load to the surrounding concrete. If an adequate anchorage length is provided, deformations result only from the tensile strains along a portion of the anchorage but a limited embedment length will cause the rebar to experience an increase of the slip and can lead to a failure of concrete in the anchorage region (Sousa, 2015; Sousa et al., 2018). Figure 2.11 illustrate the variation of bond stress, tensile stress, and slip along the embedment length for a sufficient and a limited embedment length, where the slip represents the relative displacement between the surrounding concrete and the rebar. Penetration of the stresses and strains along the anchorage region is called the strain penetration (SP) effect.

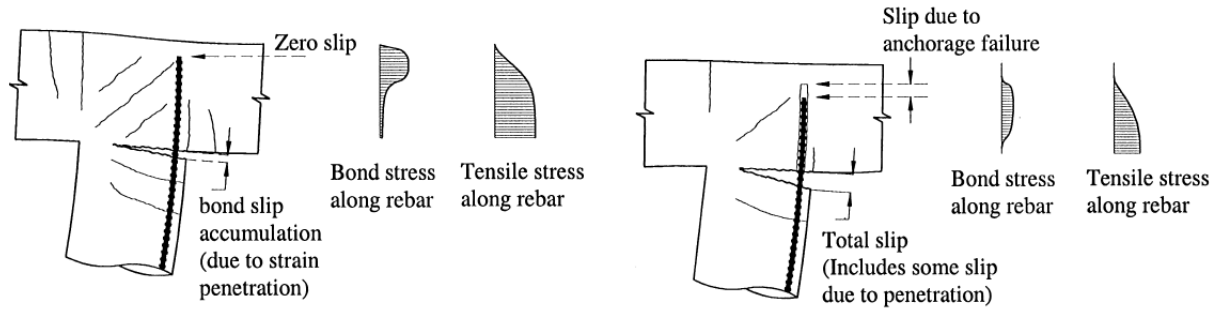


Figure 2.11: Behaviour of anchorage region with sufficient (left) and limited (right) embedment length (adapted from Sousa, 2015)

Sezen and Moehle (2004) and Goodnight et al. (2015) (Sousa et al., 2018) demonstrated that SP effects can contribute up to 40% of the total lateral displacement and are not to be neglected. Unfortunately, SP effects are still neglected in numerical simulation of large structures due to heavy and demanding computational time. 3D finite element (FE) software with fine mesh are computationally too expensive to reproduce the interaction between anchored longitudinal bars and concrete. Different approaches with fiber-based beam-column analyses have thus focused on the cross-sectional level to directly account for these effects at the member formulation and avoiding to model more elements. Despite all these efforts, a better way is to assume that SP effects can be lumped into a zero-length element which is located at the member's end, like the interface between a column and a footing for example (See fig 2.12).

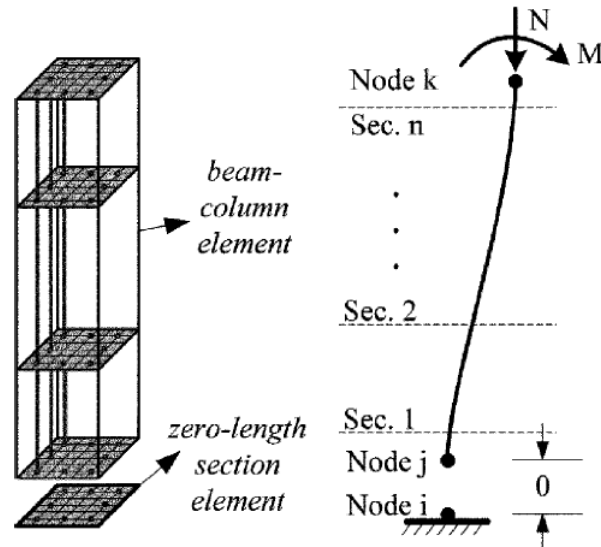


Figure 2.12: Fiber-based modeling of strain penetration effects with a zero-length element (adapted from J. Zhao and Sritharan, 2007)

A zero-length section element is a fiber discretization of the cross-section of a structural member (J. Zhao and Sritharan, 2007). This element is characterized by having a duplicated node where nodes i and j are located at a distance of zero from each other. The properties associated with this zero-length element, such as bond stress or slip, are determined either through empirical data or through local bond-slip and stress-strain approaches.

Three different approaches can be used to take SP effects into account. The first one is to reduce Young's modulus of the reinforcing bar to simulate greater flexibility of the RC member. The second one is to extend the member to an estimated strain penetration length, based on the assumption that they developed along this length (See figure 2.13). In Sousa et al. (2018) study, it is stipulated that different equations for steel reinforcement (Eq. 2.21 and 2.22) have been developed for this SP length, such as the one by Priestley et al. (2007) and the Eurocode 8 - Part 3 (Sousa et al., 2018).

$$L_{SP} = 0.022f_yd_b \quad (2.21)$$

$$L_{SP} = k \frac{f_y d_b}{\sqrt{f_c}} \quad (2.22)$$

The last approach (See figure 2.13) consist to consider an elastic rotational spring at the base of the column where the elastic stiffness of the spring can be calculated with the following formula

$$K_{\Theta} = 3EI \frac{L_c^2}{(L_c + L_{sp})^3 - L_c^3} \quad (2.23)$$

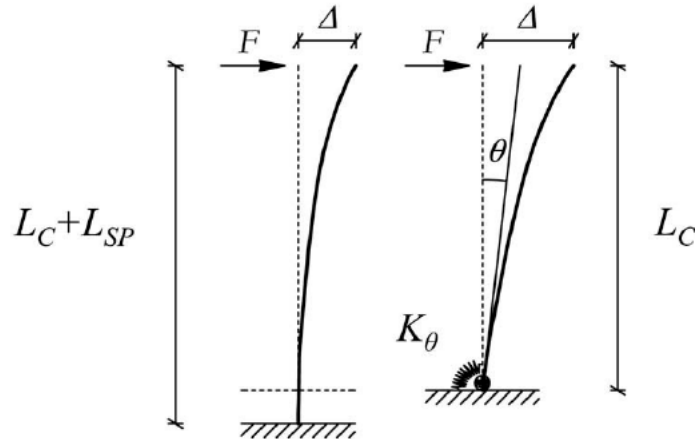


Figure 2.13: Strain penetration modelling approaches: elongated element(left) and base spring (right) (adapted from Sousa et al., 2018)

2.8 Distributed Fiber Optical Sensor (DFOS)

Nowadays, it is difficult to calculate or predict the evolution of strain distribution or even slip along the embedded length of a reinforcing bar in concrete with conventional measurement devices such as extensometers and strain gauges. Due to technological progress, DFOS has become a solution to this problem. An optical fiber is a very sensitive wire capable of transmitting a large amount of data with great precision. Fiber optical sensors can be divided into six categories, which are localized, distributed, environmental, thermal, mechanical, and chemical sensors but the most commonly used are the distributed ones (Zhou et al., 2023).

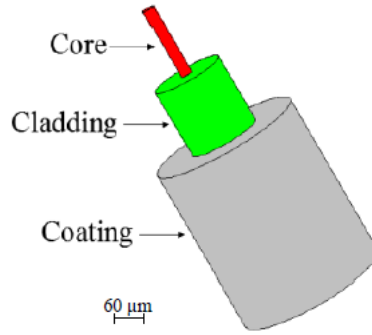


Figure 2.14: Schematic representation of a Distributed Fiber Optical Sensor (adapted from Zhou et al., 2023)

Figure 2.14 depicts the structure of a Distributed Fiber Optical Sensor (DFOS), consisting of three distinct layers: the core, the cladding, and the coating. The core is a slender glass wire with a diameter ranging from 4 to 600 μm and possesses a refractive index n_1 . Enveloping the core is the cladding, characterized by a refractive index n_2 smaller than n_1 , thereby confining the light within the core. The cladding serves as a protective layer, which can be fabricated from materials like metal or plastic, guarding against physical harm. Lastly, the coating, also fashioned from metallic or plastic materials, offers an additional layer of safeguarding against external physical influences. Commonly employed coatings include polyimide and acrylate coatings (Zhou et al., 2023).

Within civil engineering structures, optical fiber serve as a valuable sensor for gauging strain or temperature evolution within reinforced concrete (RC) elements. This involves emitting light of a specific wavelength into the fiber's core prior to the application of any loads. As the fiber is subjected to loading, it either extends or contracts, inducing deformation that alters the frequency of the reflected light. The ensuing deformations are quantified by gauging the loss in the emitted backscattered light, enabling a precise assessment of strain or temperature variations. To be more precise, strain data is obtained by analyzing the Rayleigh scattered profile, which is a spectral representation of the reflected signal. This analysis is conducted using an Optical Backscattered Reflectometer (OBR) in conjunction with Fourier

Transform techniques (Berrocal et al., 2020; Hoult et al., 2023; Malek et al., 2019). The fundamental working principle of Rayleigh scattering is depicted in Figure 2.15.

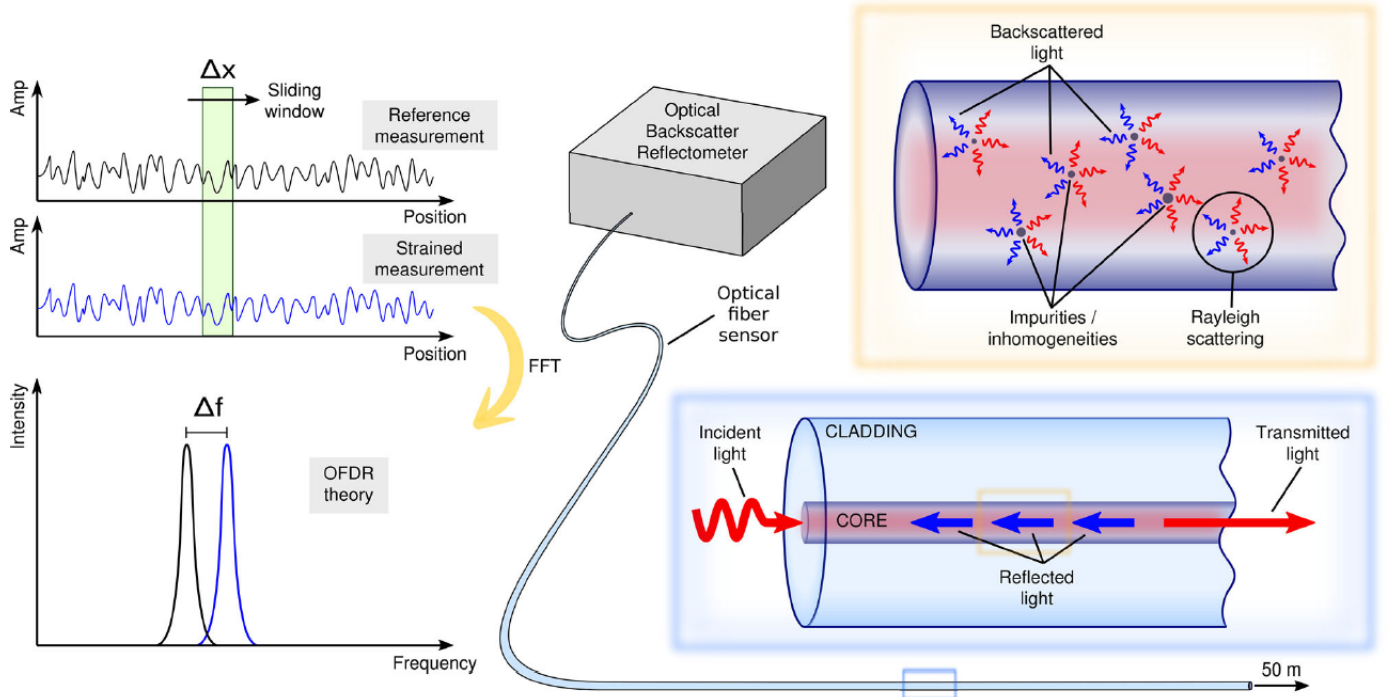


Figure 2.15: Rayleigh Backscattering principle (adapted from Berrocal et al., 2020)

Chapter 3

Experimental tests on GFRP rebars

3.1 Introduction

This chapter will focus on the experimental part of the thesis, which serves a dual purpose. The first objective is to characterise the bond-slip behaviour of GFRP rebars with different surface conditions, while the second objective is to analyse the influence of various embedment lengths in concrete. Due to an operational error, it became necessary to fabricate an additional six specimens in order to ascertain the shear strength of a silicon material and understand its potential impact on the bond-slip behaviour. This silicon material was used during the manufacturing process of the samples designed to evaluate the bond behaviour of GFRP rebars.

The chapter will be divided into five sections following this introduction. The first section will present and describe the Glass-Fiber Reinforced Polymers rebars used in the tests. Since specimens are made of concrete, a special section will be devoted to the study of the concrete mixture because it has to fulfil one requirement related to its compressive strength. The subsequent three sections will provide detailed information on different specimens, including the necessary manipulations involved. These sections will cover topics such as concrete quality and quantities, the creation of forms, and the setup and instrumentation used.

3.1.1 Characterization of the bond behaviour

The procedure for the test is based on the ISO 10406-1 standard, where it is recommended to conduct a minimum of three tests to validate the results. In case a test fails or slips out from the anchoring device, an additional test should be performed. To ensure reliability, we prepared 4 samples for each type of rebar. Additionally, one extra sample was created, serving as a preliminary test to evaluate the setup. In total, 25 cubic samples were produced on the same day, ensuring uniformity in the concrete mixture across all specimens. Furthermore, standard concrete cylinders

were fabricated using the same mixture to assess the compressive strength of concrete after 28 days of casting. The concrete shall have a compressive strength of 30 ± 3 MPa 28 days after casting.

The preliminary test was conducted 28 days after the samples were produced. Before going any further in the tests, results were processed to verify that every measure was as expected. The remaining tests were carried out the rest of the day and the day after. Additionally, compressive tests on concrete cylinders were conducted the same day to verify that the results fell within the required range of approximately 30 MPa. The inclusion of those tests is necessary as the compressive strength of concrete is an input parameter in the numerical bond-slip model, which will be introduced in chapter 5.

3.1.2 Influence of the embedment length

To investigate the influence of several embedment lengths, cylindrical samples were made based on the study realized by Zhao et al. in 2022 (X. Zhao et al., 2022). The three different embedment lengths that were tested were $10d_b$, $20d_b$, and $30d_b$. In total, 10 samples were made, including 3 samples for each configuration and one additional sample to verify the setup. Tests were performed only using one rebar type due to a quantity issue with other rebars.

3.2 Presentation of the tested rebars

In this section, our focus will be on examining the various bars that will undergo testing. These bars consist of six different types, distinguished by their diameter, surface condition, and resin composition. Each bar shares a common length of 2 meters, although the quantities available for each type may differ. A comprehensive overview of the bars, including their specific details, can be found in Table 3.1.

The Calendering rebars (Cal) possess one of the smoothest surface finishes, featuring fiber strips running along the length of the bar. In contrast, Peel-ply (PL) bars are characterized by fibers enveloping the entire circumference of the bar and have a nominal diameter of 16mm, slightly larger than the 13mm diameter of the Calendering bars. Peel-ply/grid (PLG) bars also exhibit a smooth surface finish with a distinct fiber arrangement. Notably, glass fibers on PLG bars appear to be only partially adhered to the surface, creating a unique texture. The Machining GFRPs (Mach) are ribbed in nature, providing a distinct surface profile. Lastly, sand-coated rebars (SC) boast a surface adorned with sand particles and are equipped with a helical strand wrapped around them. Among the sand-coated rebars, there are variants with diameters of 13mm (SC13) and 16mm (SC16), each displaying a coarser surface texture compared to the other rebar types.

Resin	Surface condition	d_b [mm]	L [mm]	Quantity
Elium® C599 E	Calandering	13	2000	20
Elium® C599 E	Peel-ply/Grid	13	2000	4
Elium® C599 E	Peel-ply	16	2000	5
Elium® C599 E	Machining	13	2000	2
Vinyl Ester	Sand-coating	16	2000	5
Vinyl Ester	Sand-coating	13	2000	4

Table 3.1: Summary of the tested GFRPs.

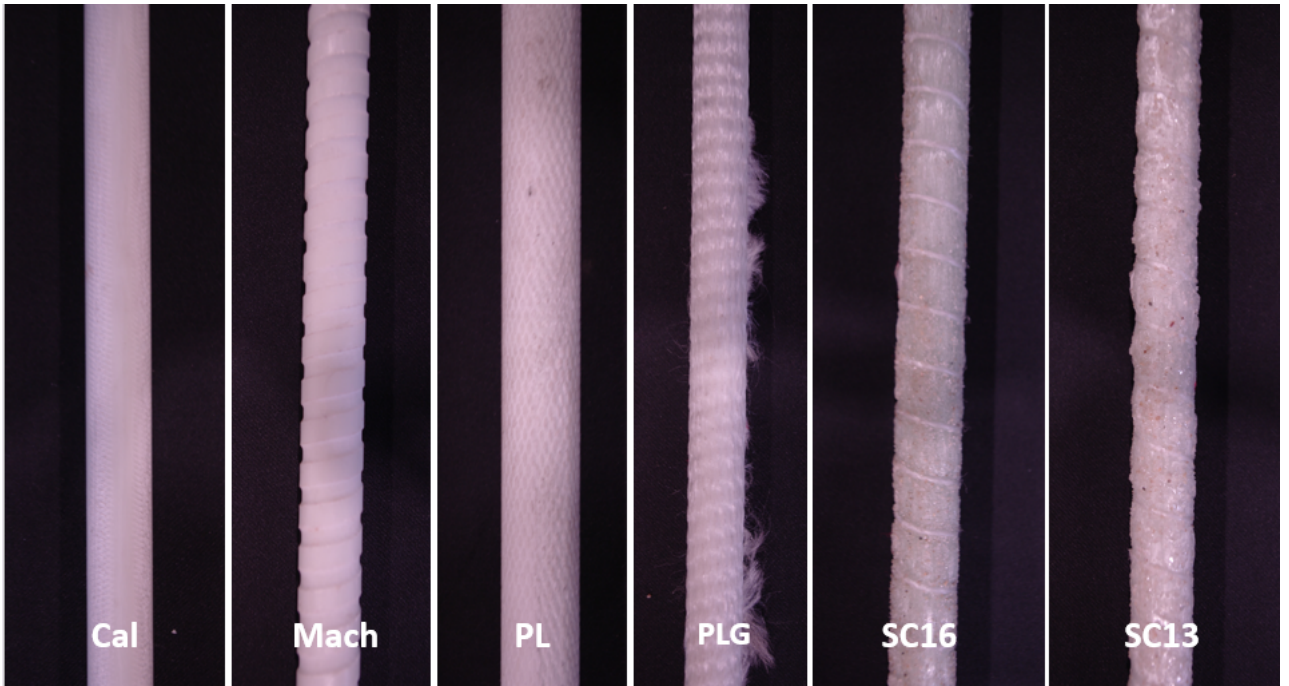


Figure 3.1: Surface finishing

The specimens are named using the notation X-Y-Z. The first letter X makes reference to the surface condition of the rebar and will be abbreviate like this:

- Calandering $\Rightarrow$ Cal
- Peel-ply/grid $\Rightarrow$ PLG
- Machining $\Rightarrow$ Mach
- Sand-coating 13mm $\Rightarrow$ SC13
- Sand-coating 16mm $\Rightarrow$ SC16
- Peel-ply $\Rightarrow$ PL

The second letter Y makes reference to the embedment length. Since the second part of the experimental campaign is to analyze the influence of different embedment lengths, this letter will vary and will either be equal to $4d_b$, $10d_b$, $20d_b$, or $30d_b$. The last letter will be the number of the specimen since there are multiple samples of the same kind.

However, samples used to assess the shear strength of the silicon were named "Glue-13" or "Glue-16", depending if they were made with a 13mm or 16mm rebar diameter. The following table provides a summary of all the samples that will be manufactured.

Calandering	Peel-ply/grid	Machining	Sand-coating	Peel-ply	Glue samples
Cal- $4d_b$ -1	PLG- $4d_b$ -1	Mach- $4d_b$ -1	SC13- $4d_b$ -1	PL- $4d_b$ -1	Glue-13-1
Cal- $4d_b$ -2	PLG- $4d_b$ -2	Mach- $4d_b$ -2	SC13- $4d_b$ -2	PL- $4d_b$ -2	Glue-13-2
Cal- $4d_b$ -3	PLG- $4d_b$ -3	Mach- $4d_b$ -3	SC13- $4d_b$ -3	PL- $4d_b$ -3	Glue-13-3
Cal- $4d_b$ -4	PLG- $4d_b$ -4	Mach- $4d_b$ -4	SC13- $4d_b$ -4	PL- $4d_b$ -4	Glue-16-1
Cal- $4d_b$ -5			SC16- $4d_b$ -1		Glue-16-1
Cal- $10d_b$ -1			SC16- $4d_b$ -2		Glue-16-2
Cal- $10d_b$ -2			SC16- $4d_b$ -3		Glue-16-3
Cal- $10d_b$ -3			SC16- $4d_b$ -4		
Cal- $10d_b$ -4					
Cal- $20d_b$ -1					
Cal- $20d_b$ -2					
Cal- $20d_b$ -3					
Cal- $30d_b$ -1					
Cal- $30d_b$ -2					
Cal- $30d_b$ -3					

Table 3.2: Summary of the samples notation

3.3 Concrete study

In accordance with the ISO standard, the concrete used for testing required a compressive strength of 30 ± 3 MPa after a curing period of 28 days. To ensure adherence to this requirement, two different concrete mixtures were tested, aiming to achieve results as close as possible to this specified range. A reference mixture (C1) was used, known to possess an average compressive strength of approximately 33.6 MPa after 28 days. This mixture included 400 kg/m^3 of cement, with a water-to-cement ratio of approximately 0.45/1, along with fine and coarse aggregates in the ratio of 2/3. The second mixture (C2) was derived from the aforementioned composition, with the exception of a reduced cement content of 350 kg/m^3 instead of 400 kg/m^3 . The constituents of these concrete mixtures are detailed below.

- Cement CEM II/B - M 32,5 N
- Fine aggregate: The sand was however wet because it was stored outside and thus brings more water in the mixture.
- Coarse aggregate 7/14: aggregate's minimum and maximum diameter were 7mm and 14mm respectively.

Three standard cubes of $150\text{mm} \times 150\text{mm} \times 150\text{mm}$, were made for each mixture to test their compressive strength (See Fig 3.2). The samples were tested after 14 days instead of 28 due to time constraints. The reason cubes were used instead of cylinders is because it is easier to test a cube, whereas the cylinder's surface must be rectified before testing. The quantities had to be pre-calculated in advance to ensure ample material for filling three standard cubic molds. Each standard cube has a volume of 3.375L, summing up to a total of 10.125L for three cubes. To account for potential material losses, a safety margin was considered, resulting in the preparation of 15L of concrete. The quantities of each mixture required for this 15-liter volume are outlined below:

	Concrete mix C1 ($400\text{kg}/\text{m}^3$)	Concrete mix C2 ($350\text{kg}/\text{m}^3$)
Water	2.6L	2.36L
Cement	5.88 kg	5.25 kg
Fine aggregate	11.76 kg	13.17 kg
Coarse aggregate 7/14	17.64 kg	19.75 kg

Table 3.3: Required quantities for 15L of concrete



Figure 3.2: Standard cubes for concrete quality tests

The test outcomes are displayed in the subsequent tables, where $f_{c,14,cube}$ represents the compressive strength of the cube at a 14-day age, and $f_{cm,14,cube}$ denotes the mean compressive strength of the cube at a 14-day age.

concrete mixture C1 ($400kg/m^3$)			concrete mixture C2 ($350kg/m^3$)		
Sample	$f_{c,14,cube}$ [MPa]	$f_{cm,14,cube}$ [MPa]	Sample	$f_{c,14,cube}$ [MPa]	$f_{cm,14,cube}$ [MPa]
C1-A	29.5		C2-A	25.5	
C1-B	30.06	29.6	C2-B	25.3	24.9
C1-C	29.21		C2-C	24	

Table 3.4: Results of the compressive strength for the concrete mixture with $400kg/m^3$ (left) and $350kg/m^3$ (right) of cement at a 14-day age

As per the ISO standard, the stipulated mean compressive strength of $30MPa \pm 3MPa$ applies to cylinders at 28 days, not cubes. Consequently, it is necessary to compute the concrete's compressive strength based on cube results at 14 days. The initial step involves converting cube strength to its equivalent cylinder strength. It is recognized that the ratio between the compressive strength of cubes and cylinders is approximately 0.8 Tslb, 2021. Thus, the mean compressive strength of the cylinder at 14 days can be determined using the following formula:

$$f_{cm,14} = 0.8f_{cm,14,cube} \quad (3.1)$$

The subsequent stage involves approximating the concrete's age at the 28-day mark. This estimation will be based on the graph depicted in Figure 3.3, which illustrates the progressive evolution of compressive strength over time. Notably, nearly 90% of the anticipated strength is achieved by the 14th day, leaving only a 10% differential to be attained. The subsequent equation facilitates the computation of the compressive strength on a cylinder at the 28-day milestone.

$$f_{cm} = 1.11f_{cm,14} \quad (3.2)$$

where f_{cm} and $f_{cm,14}$ are the mean resistance at 28 and 14 days respectively.

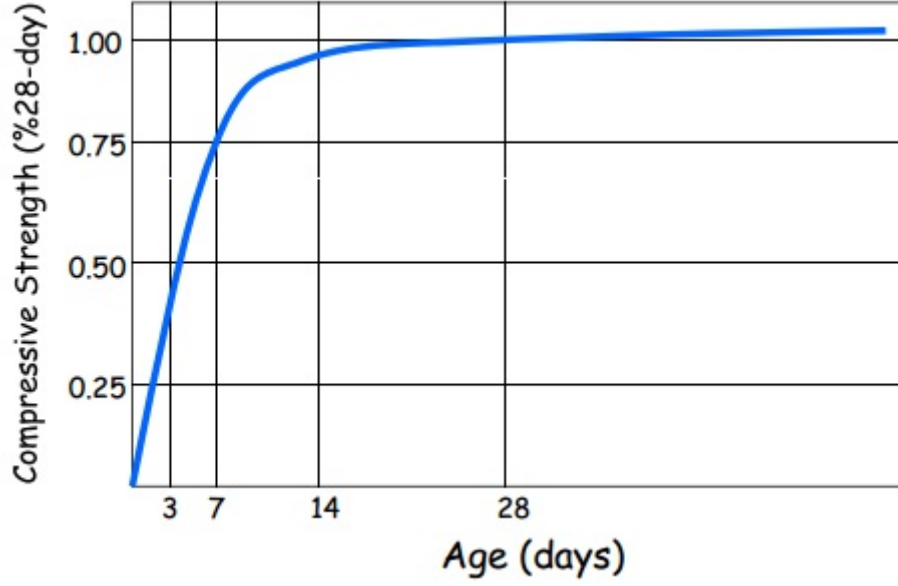


Figure 3.3: Evolution of concrete compressive strength over time (adapted from “Properties of normal strength Portland cement concrete - CivilEngineeringBible.com”, n.d.)

Upon the application of equations 3.2 and 3.1, the outcomes are summarized in the table provided below. A thorough inspection of the data highlights that the reference mixture (C1), which encompasses $400\text{kg}/\text{m}^3$ of cement, closely adheres to the stipulations outlined in the ISO standard. Hence, this particular mixture will be used for the fabrication of all specimens.

Mixture	$f_{cm,14,cube}$ [MPa]	$f_{cm,14}$ [MPa]	f_{cm} [MPa]
C1	29.6	23.68	26.05
C2	24.9	19.92	21.9

Table 3.5: Mean compressive strength of concrete on the cylinder at a 28-day age for the two mixtures.

3.4 Preparation of cubic samples

3.4.1 Presentation of the samples

According to the ISO 10406-1 standard (ISO, 2015), concrete cubes are used as samples. Since the GFRP rebars used in this study have a maximum diameter of 16mm, it is stipulated to use concrete cubes with dimensions of $100\text{mm} \times 100\text{mm} \times 100\text{mm}$. The samples will be provided with a spiral reinforcement to prevent splitting failure of the concrete, an unbounded region and an anchoring device.

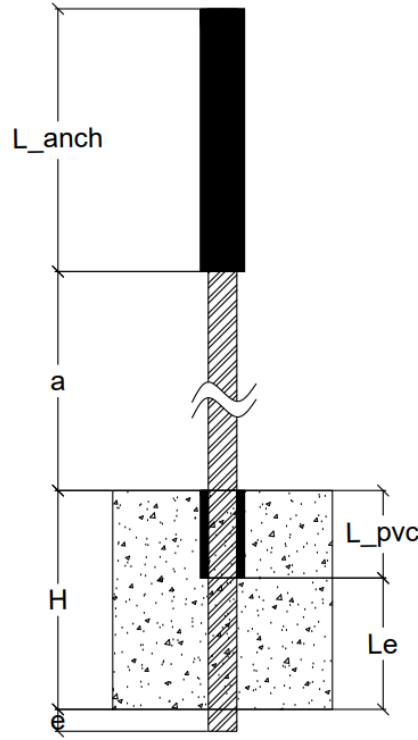


Figure 3.4: Representation of a cubic sample

Dimensions of the GFRP rebar

In accordance with the experimental protocol, the length of the Glass-Fiber Reinforced Polymer (GFRP) rebar must adhere to specific requirements, which are contingent upon its diameter. To illustrate, figure 3.4 displays the embedment length (L_e), which denotes the segment that we intend to evaluate and is equivalent to four times the nominal diameter of the rebar (d_b). It is also noteworthy that there is a designated area referred to as the "anti-adhesive" or "unbounded" region, which serves to impede the adhesion between the GFRP and concrete. This region's length (L_{PVC}) will vary in accordance with the diameter of the rebar. Moreover, there are specific requirements that must be met in regard to the free length (a). Specifically, the free length must be a minimum of 300mm or at least $40d_b$, whichever value is greater. The rebar is equipped with a free-end length (e) of 10mm, regardless of its diameter, and the anchoring length (L_{anch}) is fixed at 120mm. All those elements will be discussed further in their dedicated section. The following table 3.6 summarise all the required lengths imposed by the ISO standard.

As evidenced in table 3.6, the minimum prescribed length amounts to 870mm. Given that we possess bars measuring 2m in length, the rebars were truncated to 1m. The cutting process necessitated the use of a specialized disc, designed to avert the burning of GFRP, in addition to a smoke extractor that served to mitigate the dispersion of glass-fiber particles in the air.

Surface condition	d_b [mm]	e [mm]	$L_e = 4d_b$ [mm]	L_{PVC} [mm]	H [mm]	a [mm]	L_{anch} [mm]	Total [mm]
Calandering	13	10	52	48	100	520	120	750
Peel-ply/Grid	13	10	52	48	100	520	120	750
Peel-ply	16	10	64	36	100	640	120	870
Machining	13	10	52	48	100	520	120	750
Sand-coating	16	10	64	36	100	640	120	870
Sand-coating	13	10	52	48	100	520	120	750

Table 3.6: Summary of the minimum length required for all the GFRP rebars

Spiral reinforcement

To prevent splitting failure, the concrete samples were provided with a spiral reinforcement. These spirals possess an external diameter of 80mm and a pitch of 40mm, crafted from 6mm steel rebars. The spirals were meticulously formed by manually winding a bar around a cylindrical tube. An image illustrating the spiral reinforcement can be found in Figure 3.5.



Figure 3.5: Spiral reinforcement

Anti-adhesive or unbounded region

To confine the test length to $4d_b$, an anti-adhesive zone within the concrete cube is necessary. This zone will be created using a PVC tube fixed with silicon to the rebar. The length of these components varies based on the rebar's diameter, as outlined in Table 3.6. As depicted in the image, PVC tubes vary in size for different rebars. This discrepancy is attributed to the distinct external diameters resulting from differences in rebar nominal diameter and surface finishing.



Figure 3.6: FRPs with their anti-adhesive region made with a PVC tube.

Anchoring device

The anchoring device used in the experiments consists of an aluminum hollow cylinder, measuring 12cm in length, which is positioned at the end of the GFRP rebar. To create a strong adhesion between the rebar and the cylinder, epoxy was injected into the tube. Prior to the injection, the tubes were carefully cut and sanded to remove any imperfections and ensure optimal adhesion of the epoxy. Since each type of rebar had a different external diameter, the inner and outer diameters of the tube were chosen accordingly to accommodate the specific rebar being used.

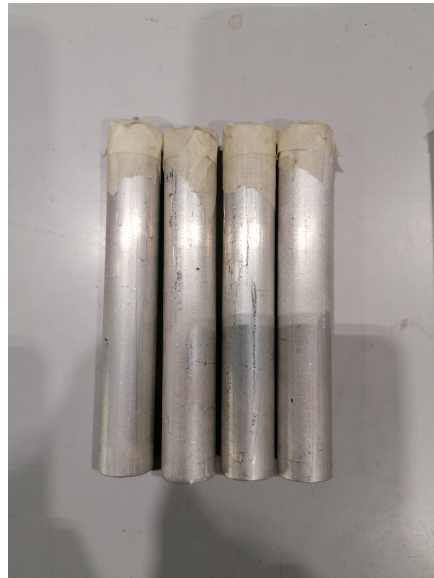


Figure 3.7: Aluminum tubes used for the anchoring device

Concrete

With the concrete mixture determined in the previous section, the next step is to calculate the quantities needed for the production of 25 samples (25L) and 6 standard cylinders (32L). The standard cylinders, of 300mm height and 150mm diameter, are essential for determining the compressive strength of concrete after 28 days of casting. To ensure an adequate margin of safety, 60L of concrete was prepared instead of the initially planned 55L. However, despite the margin, only 5 cylinders were successfully produced instead of the intended 6. The quantities required for the concrete mixture are listed below:

concrete mixture C1 ($400\text{kg}/\text{m}^3$)	
Water	10.4L
Cement	23.52 kg
Fine aggregate	44.04 kg
Coarse aggregate 7/14	64.68 kg

Table 3.7: Quantities required for 60L of the concrete

After the casting, the samples were placed in a tank filled with tempered water until the day of testing, as recommended in the ISO (See Fig 3.8).



Figure 3.8: Storage of the samples in a tank filled with tempered water.

Among the five standard cylinders that were produced, only three were deemed suitable for assessing the compressive strength. We obtained an average strength of 38 MPa, which is higher than the 26 MPa achieved when studying the concrete mixtures. The summarized results are presented in the following table:

Sample	f_c [MPa]	f_{cm} [MPa]
Cyl_1	43.23	
Cyl_2	34.48	38
Cyl_3	37.03	

Table 3.8: Results from the compressive test

Forms

To facilitate the casting process and ensure easy removal of the concrete once it has dried, suitable forms were required. These forms needed to be designed in a manner that allowed for convenient pouring and demolding. Given the need to produce 25 samples, a total of 5 forms, similar to the one depicted in Figure 3.9, were created. For practicality, specimens were cast horizontally, as this positioning offered ease of handling and operation during the casting process.

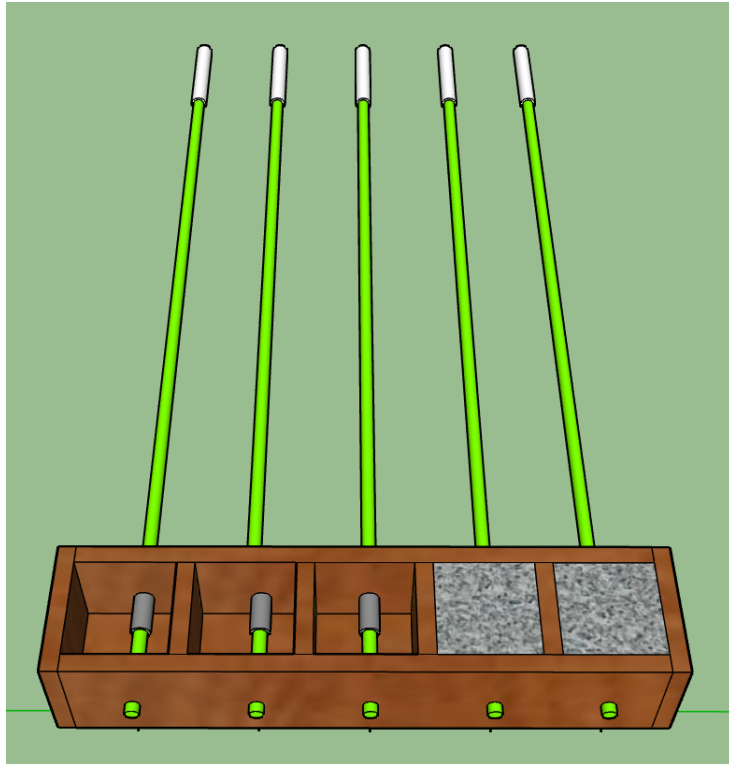


Figure 3.9: 3D representation of the cubic form

3.4.2 Setup and instrumentation

The experimental setup used in this study followed the guidelines outlined in the ISO standard, as depicted in Figure 3.10. A 100 kN MTS machine was utilized, along with a Linear Variable Differential Transformer (LVDT) to measure the displacement of the free-end of the rebar. To ensure accurate measurements and mitigate the risk of localized load concentration caused by irregularities on the concrete surface, the specimen was placed on a loading plate, allowing for even load distribution and minimizing the potential for splitting failures. According to the ISO standard, the loading rate should be situated between $10\text{N/mm}^2/\text{min}$ and $20\text{N/mm}^2/\text{min}$, which gives an average speed of 2500N/min for all specimens.

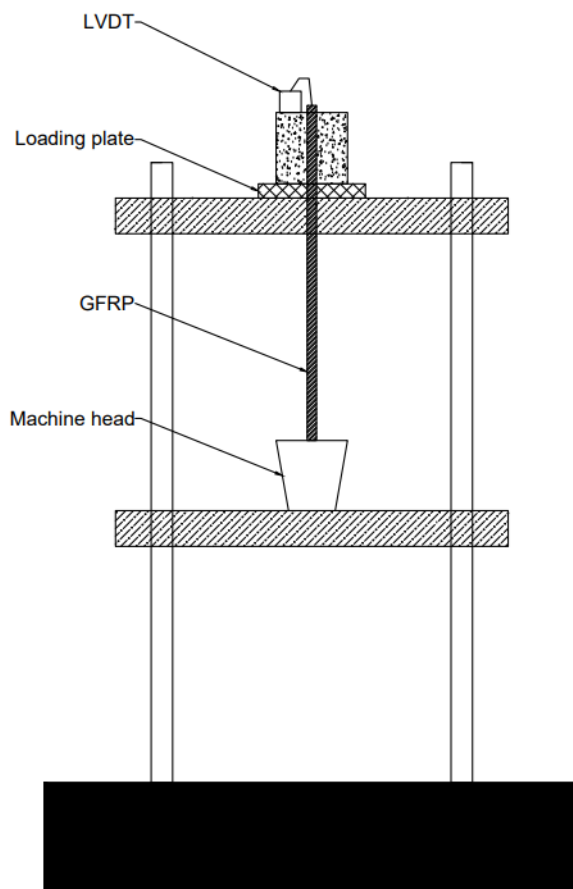


Figure 3.10: Schematic representation of the setup

Figure 3.11: Illustration of the setup

3.5 Determination of the shear strength of the silicon

Initially unplanned, an additional part of the experiment was conducted to measure the shear strength of the silicon used to bond the PVC tube to the GFRP rebar. This was prompted by an operating error, and the purpose of those samples is to assess the impact of silicon on the bond behaviour of GFRPs. To carry out this assessment, new samples were prepared, and a modified setup was used due to dimensional requirements. For this specific test, sand-coated rebars with diameters of 13mm and 16mm were chosen, as they possess a rougher surface condition and are expected to exhibit better adherence with the silicon.

3.5.1 Presentation of the samples

To evaluate the adhesive strength of silicon, which lays between the rebar and the PVC tube, we prepared the samples shown in Figure 3.12, which share the same dimensions as the previously presented ones, except for the free-length (a) which is shorter. For these tests, sand-coated GFRP rebars with diameters of 13mm and 16mm were used and in order to eliminate any effect of adhesion between the concrete and the GFRP, the rebar was intentionally not embedded within the concrete. The forms and the anchoring device will not be detailed in this section because they are the same as the ones used in the section 3.4.

Dimensions of the GFRP rebar

The cube has a dimension of $100mm \times 100mm \times 100mm$ and the anchorage device is also one aluminum tube of $120mm$ long with an external diameter of $20mm$, which is the same as the one used with the cubic samples presented in the previous section. Depending on the rebar's diameter, the length of the PVC tube (L_{PVC}) and the free length of the rebar (a) will vary. The specific values are outlined in the table 3.9.

Surface condition	d_b [mm]	L_{PVC} [mm]	L_{anch} [mm]	a [mm]	H [mm]
Sand-coated	13	48	120	162	100
Sand-coated	16	36	120	174	100

Table 3.9: Summary of the dimensions of the samples

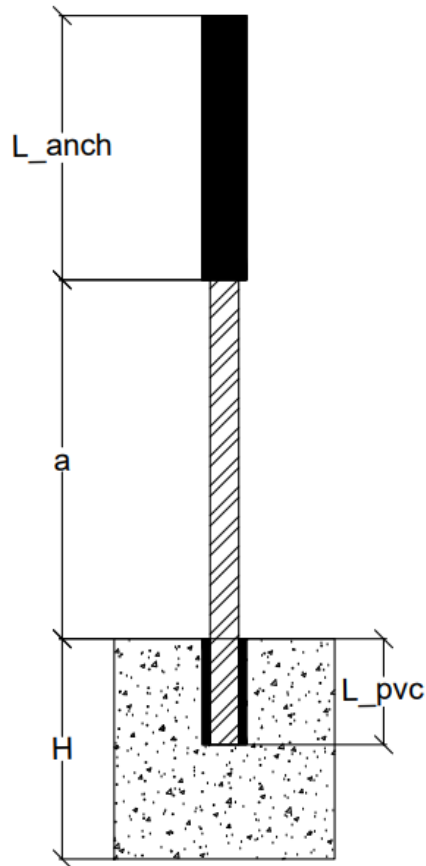


Figure 3.12: Schematic representation of a sample

Concrete

The concrete mixture used was the same as the one used for the previous cubic samples, but the quantities were adjusted. A total of 10L of concrete was prepared to cast the 6 samples, and the specific quantities are summarised in the following table:

concrete mixture C1 ($400\text{kg}/\text{m}^3$)	
Water	1.7L
Cement	3.92 kg
Fine aggregate	7.34 kg
Coarse aggregate 7/14	10.78 kg

Table 3.10: Quantities required for 10L of the concrete mixture



Figure 3.13: Illustration of the samples

3.5.2 Setup and instrumentation

To test these samples, a MTS 100kN testing machine was used, but with a slightly different setup. Due to the shorter length of the samples, they were placed inside a metallic box, as depicted in Figure 3.14. In this setup, only the applied force and the stroke of the machine were measured, whereas the measurement of rebar slip was not conducted. The loading rate for these tests remained consistent with the previous ones, set at 2500 N/min .

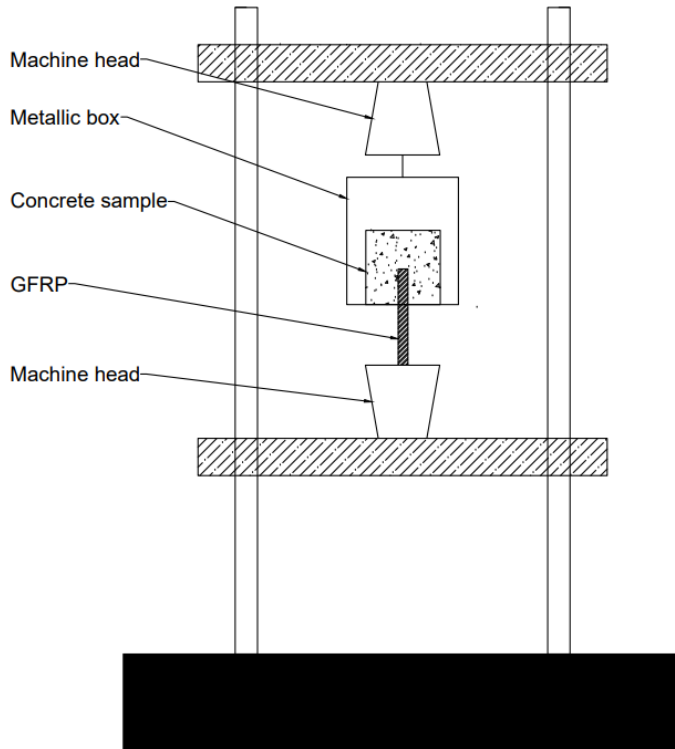


Figure 3.14: Schematic representation of the setup Figure 3.15: Illustration of the setup

3.6 Preparation of the cylindrical samples

3.6.1 Presentation of the samples

To investigate the influence of different embedment lengths on the bond stress, cylindrical samples will be fabricated in this study. Specifically, three embedment lengths will be examined, namely $10d_b$, $20d_b$, and $30d_b$, using the calendering rebars. The diameter of the base of the cylinder will be 200mm, while the height will vary depending on the specific length being tested. These samples will be designed to incorporate an anti-adhesive zone, and the concrete composition will remain consistent with that used for cubic samples. Figure 3.16 provides a visual representation of the sample and its dimensions.

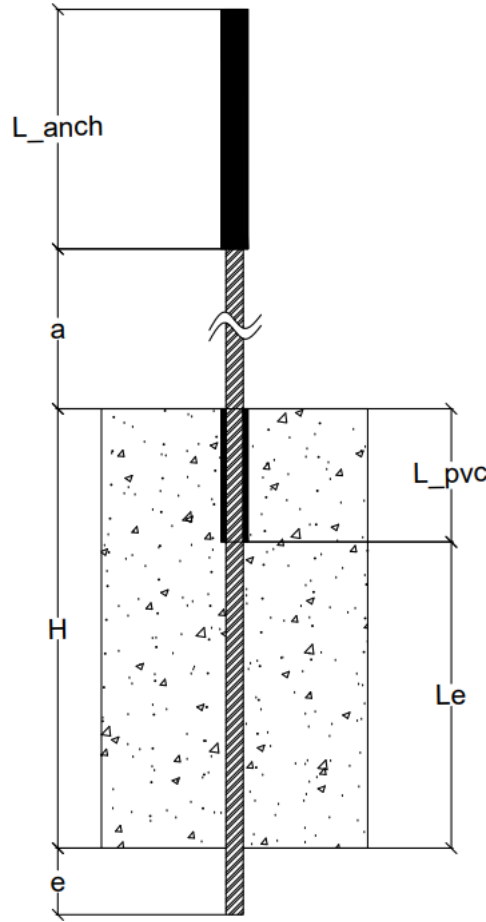


Figure 3.16: Representation of the cylindrical samples

Dimensions of the GFRP rebar

The rebar in the cylindrical samples features a free-end length (e) of $50mm$, extending throughout the concrete cylinder. The embedment length (L_e) varies depending on the specific length being tested, as will the free length of the rebar (a). The unbonded length (L_{PVC}) is achieved using a PVC tube, and the anchorage length (L_{anch}) remains constant regardless of the embedment length. Table 3.11 provides a summary of the specific values for the samples.

Surface condition	L_e [mm]	e [mm]	L_{PVC} [mm]	H [mm]	a [mm]	L_{anch} [mm]	Total [mm]
Calandering	130 ($=10d_b$)	50	100	230	600	180	700
Calandering	260 ($=20d_b$)	50	100	360	470	180	830
Calandering	390 ($=30d_b$)	50	100	490	340	180	960

Table 3.11: Summary of the dimensions of the samples

Anchoring device

Considering that the anticipated force levels during the pullout tests would be higher for longer embedment lengths, it was necessary to conduct tests using an alternative anchoring device. An initial experimental test was conducted using an aluminum tube measuring 180 mm in length, and this test resulted in the failure of the rebar. As a result, an aluminum tube of 180 mm in length will be affixed to the rebars using epoxy adhesive.

Concrete

In order to have no influence of the concrete quality on the results, the same mixture as for the cubic samples will be used. Because 155L of concrete was too much to prepare all at once, the concrete was realized in two mixes. Since the samples were not made on the same day as the cubes, 5 standard cylinders were made to assess the compressive strength on the day of testing, 3 for the first mix (1A, 1B and 1C) and only 2 for the second mix (2A and 2B) due to an insufficiency of the quantity. The needed quantities for 155L of concrete are resumed in the following table.

concrete mixture C1 ($400\text{kg}/\text{m}^3$)	
Water	26.78L
Cement	60.76 kg
Fine aggregate	113.77 kg
Coarse aggregate 7/14	137.09 kg

Table 3.12: Quantities required for 155L of the concrete mixtures

The compressive tests were conducted slightly before the intended 28-day mark due to technical issues. Specifically, cylinders 1A, 1B, and 1C were tested after 25 days of casting, while cylinders 2A and 2B were tested after 24 days. However, considering the trend depicted in Figure 3.3, it can be assumed that the difference in strength at these slightly earlier testing points is negligible compared to the expected resistance at the standard 28-day period.

Sample	f_c [MPa]	f_{cm} [MPa]
1A	31.1	32
1B	30.9	
1C	33.6	
2A	41.2	41
2B	40.6	

Table 3.13: Results from the compressive test

Unfortunately, the mean compressive strengths are quite different and the results will be treated taking this difference into account.

Forms

For cylindrical forms, a PVC tube with a diameter of 200mm was used, with varying heights depending on the embedment length required. To create the bottom of the form, a square wooden plate measuring $30\text{cm} \times 30\text{cm}$ was employed. A 15mm diameter hole was drilled into the plate to allow for the passage of the GFRP rebar. Additionally, a groove was incorporated into the bottom plate to securely hold the PVC tube in place and facilitate assembly.

To ensure proper alignment of the rebar, a wooden plank with a 15mm diameter hole was positioned on top of the cylinder. This plank was connected to the bottom plate using two vertical rafters, which were securely fastened to both plates using screws. This arrangement aided in centering the rebar within the cylindrical form and provided structural stability during the casting process.

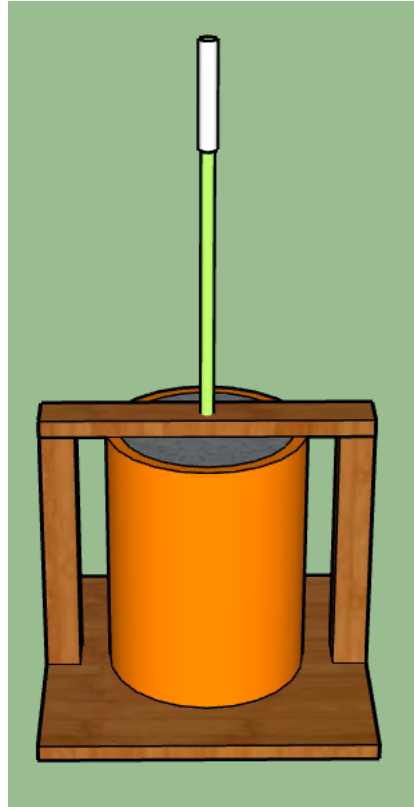


Figure 3.17: 3D representation of the cylindrical form

3.6.2 Setup and instrumentation

The tests were conducted using an MTS hydraulic machine with a maximum capacity of 1000 kN. In contrast to the preceding tests, the loading rate for this phase will be governed by displacement control, specifically set to 0.02 mm/s. As illustrated in Figure 3.16, samples will be placed between two metallic plates that can be adjusted according to the specimen's height. These two plates are connected using four threaded rods. The bottom plate is grooved to facilitate placement of the specimen, as shown in Figure 3.20. Since the bars were not always perfectly centered in the cylinder and were sometimes misaligned due to imperfections on the concrete surface, the alignment of the two plates could be adjusted to center the rebar in the machine jaw. This adjustment helps prevent bending forces during the test. The setup includes two Linear Variable Differential Transformers (LVDTs): LVDT1 (Fig. 3.21) for measuring the free-end slip and LVDT2 (Fig. 3.22) for measuring the loaded-end displacement. LVDT1 was magnetized on a metallic plate placed directly on the specimen. The second LVDT was attached to the rebar and, through the bottom plate groove, pointed on the concrete surface of the specimen. Additionally, an extensometer (Fig. 3.23) was positioned to measure the local elongation of the rebar.

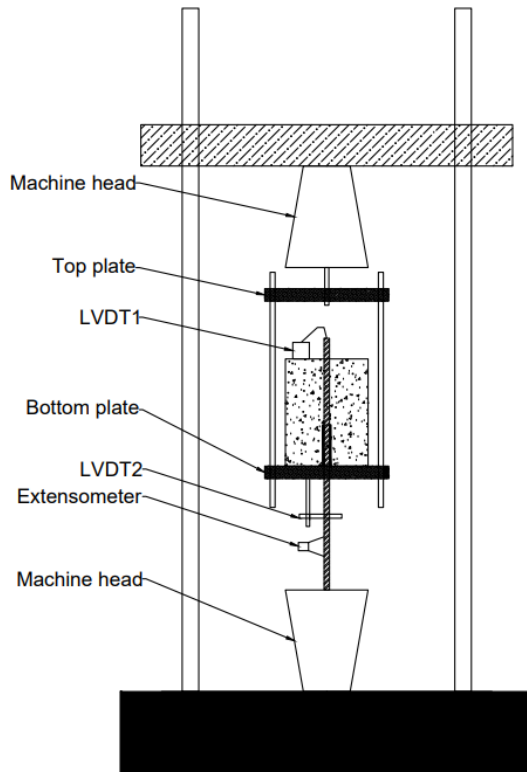


Figure 3.18: Schematic representation of the setup



Figure 3.19: Illustration of the setup

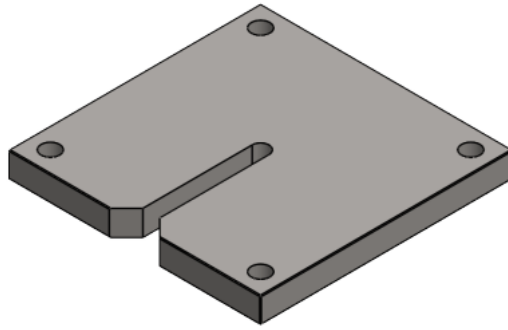


Figure 3.20: Bottom plate

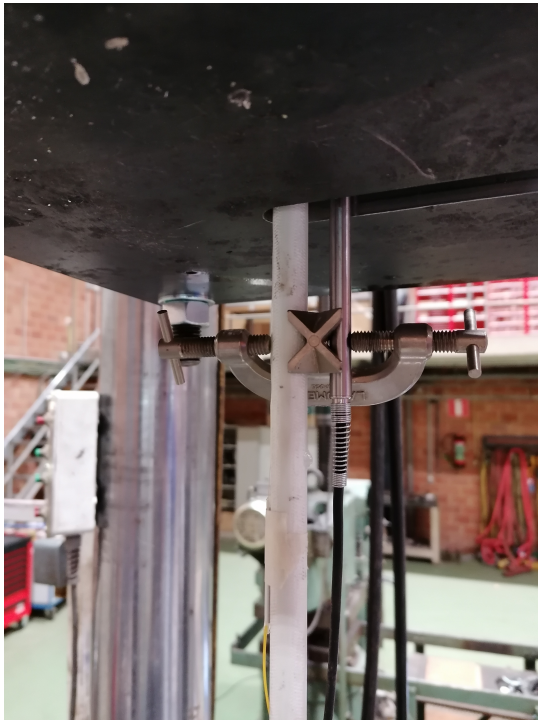


Figure 3.21: Bottom LVDT



Figure 3.22: Top LVDT

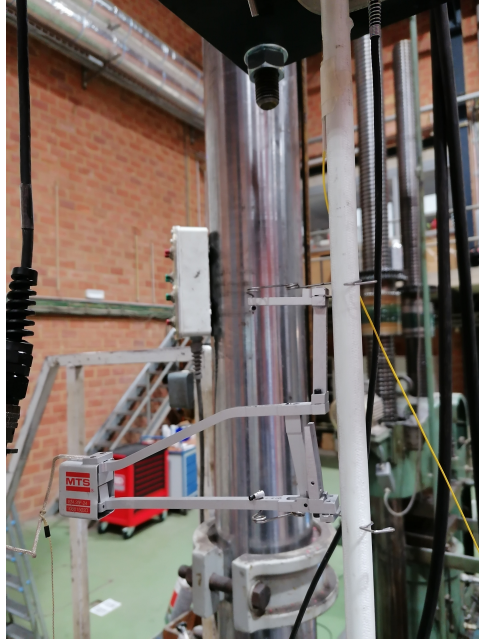


Figure 3.23: Extensometer

Distributed optical fiber sensors (DFOSs)

Additionally to the LVDTs, three specimens (Cal-10 d_b -3, Cal-20 d_b -1 and Cal-30 d_b -1) will be equipped with Distributed Optical Fiber Sensor (DFOS) glued along the rebar in order to measure the strain evolution along the GFRP. This will allow us on the one hand to have some data along the embedment length and on the other hand to validate the measure obtained by the LVDTs.

As in the paper of R. Hoult in 2023, the DFOS was implemented using the LUNA optical distributed sensor interrogator 4-channel ODiSI 6104 Series system. A spatial resolution of 0.65mm with a frequency of 12.5Hz was used, providing a big amount of data. As it is common in other research, the data will be filtered using a moving average over a length of approximately 10mm (Hoult et al., 2023).

Chapter 4

Analysis and discussion of the experimental results

This chapter is devoted to the results of the pullout tests carried out on the samples presented in the chapter 3. Firstly, we will analyse and discuss the influence of the shear strength of the silicon, then the bond stress-slip behaviour of every type of GFRP and finally the influence of the embedment length.

4.1 Results from the silicon shear strength

The outcomes of the pullout tests aimed at assessing the maximum shear stress of the silicon adhesive and its impact on the bond strength of the examined GFRP rebars are displayed in Figure 4.1. Evidently, all the samples experienced pullout failure, denoting that the PVC remained embedded within the concrete while the rebar disengaged from the PVC tube. The mode of failure was observed to occur at the interface between the silicon and the PVC.

It can be noted that irrespective of the rebar's diameter, both blue and green curves reached an approximate maximum force of 0.60 kN. This implies that the diameter of the rebar does not impact shear resistance of silicon adhesive. This observation is further supported by the data in table 4.1, where samples Glue-13-1 and Glue-16-1 both exhibited a peak force of 0.61 kN.

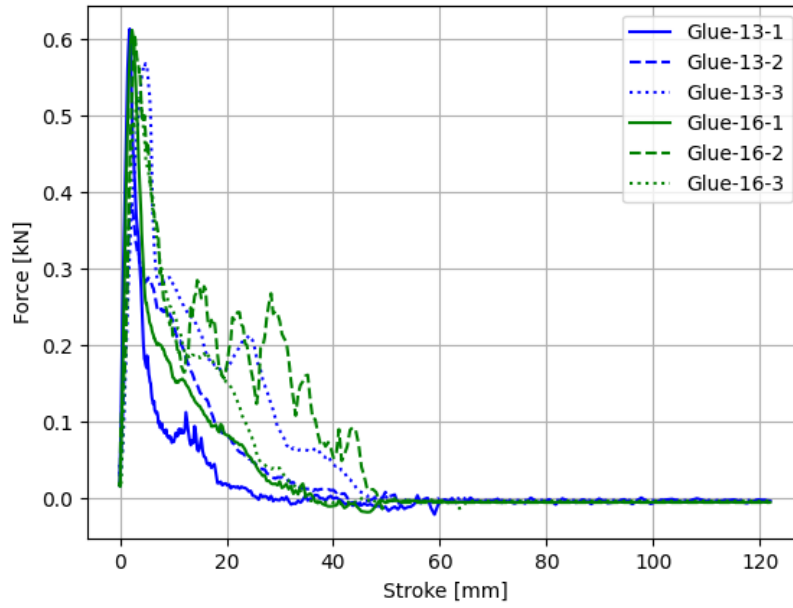


Figure 4.1: Results of the shear strength of the silicon

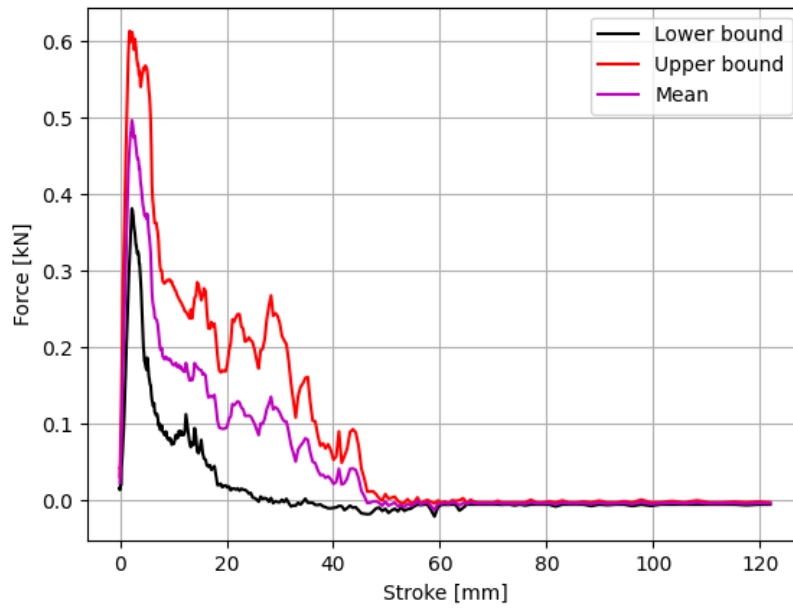


Figure 4.2: representation of the upper bound, lower bound and mean behaviour of the silicon

In Figure 4.2, the upper bound (red curve), lower bound (black curve), and mean value (violet curve) of all obtained results are illustrated. The upper and lower bounds signify the maximum and minimum values from the entirety of the tested samples, while the mean curve represents an average behaviour of silicon adhesive.

Sample	Peak [kN]	Average peak [kN]
Glue-13-1	0.61	0.5
Glue-13-2	0.38	
Glue-13-3	0.57	
Glue-16-1	0.61	
Glue-16-2	0.6	
Glue-16-3	0.53	

Table 4.1: Experimental results

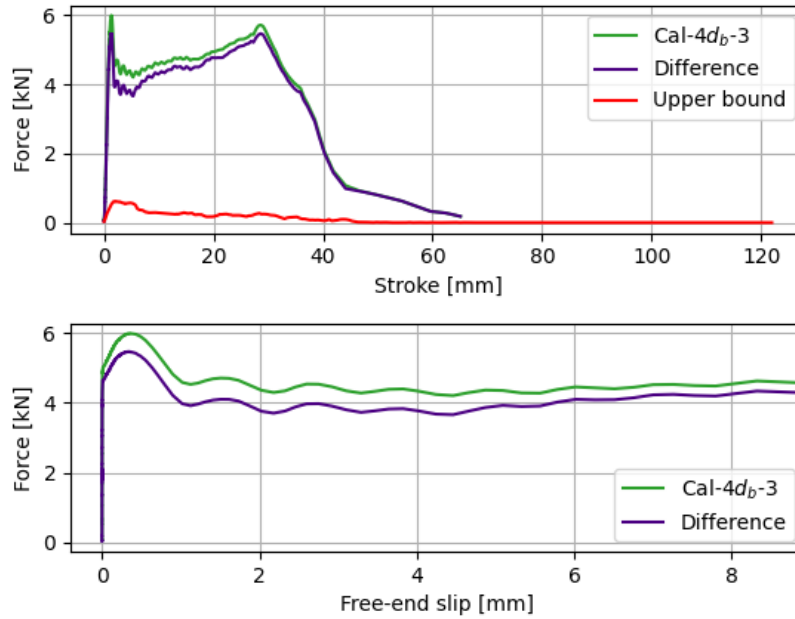


Figure 4.3: Effect of the silicon on the on the pullout results

In order to assess the maximal effect of silicon on the bond stress-slip relationship of the tested GFRPs, we selected sample Cal-4 d_b -3. Among all valid results, this sample exhibited the lowest applied force, as we will see later. Figure 4.3 presents two graphs. The first graph displays the upper bound curve of silicon effect (red curve), the relation between the applied force and the stroke for specimen Cal-4 d_b -3 (green curve), and the curve obtained by subtracting these two curves (purple curve).

The second graph depicts only the green and purple curves, but this time expressed in terms of the free-end slip of the rebar, which is of primary interest. Since the free-end slip was not directly measured, the upper bound curve is absent from this graph.

The impact of the silicon adhesive on the bond strength of the rebar is apparent, as evidenced by the displacement offset observed between the green and purple curves. Notably, for specimen Cal-4 d_b -3, where the maximum applied force reached 6kN, the influence of the silicon is estimated to be a maximum of 10%. However, this percentage is expected to be even lower for other specimens. As our primary goal is to depict the average bond stress-slip behavior of GFRPs, we will account for the silicon's average response across all specimens. Therefore, in the subsequent graphs, the effect of the silicon will already be considered.

4.2 Results from the pullout test with the cubic samples

In this section, a comprehensive analysis and discussion of the results will be conducted for each type of rebar. All specimens experienced failure through pullout, where the rebars dislodged from concrete while the PVC tubes remained embedded. Throughout the tests, various parameters were measured, including the applied force, machine's stroke, and free-end slip of the rebar. The following figure depicts the surface condition after being extracted from the concrete cubes. It is evident that sand-coated rebars exhibit more noticeable damage, while the other types appear to have suffered relatively less.

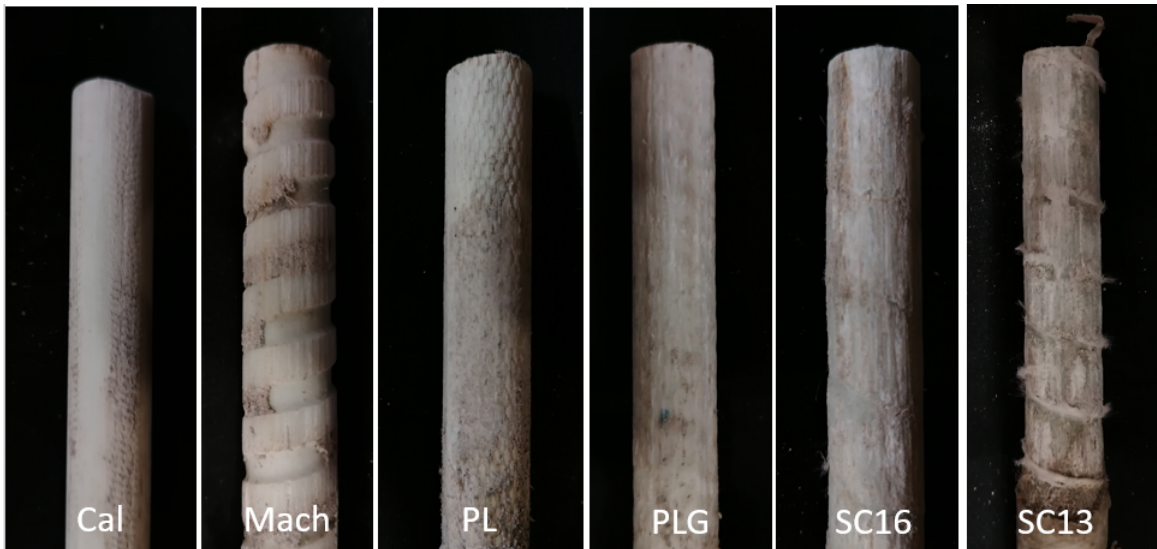


Figure 4.4: Surface finishing after test

Calandering

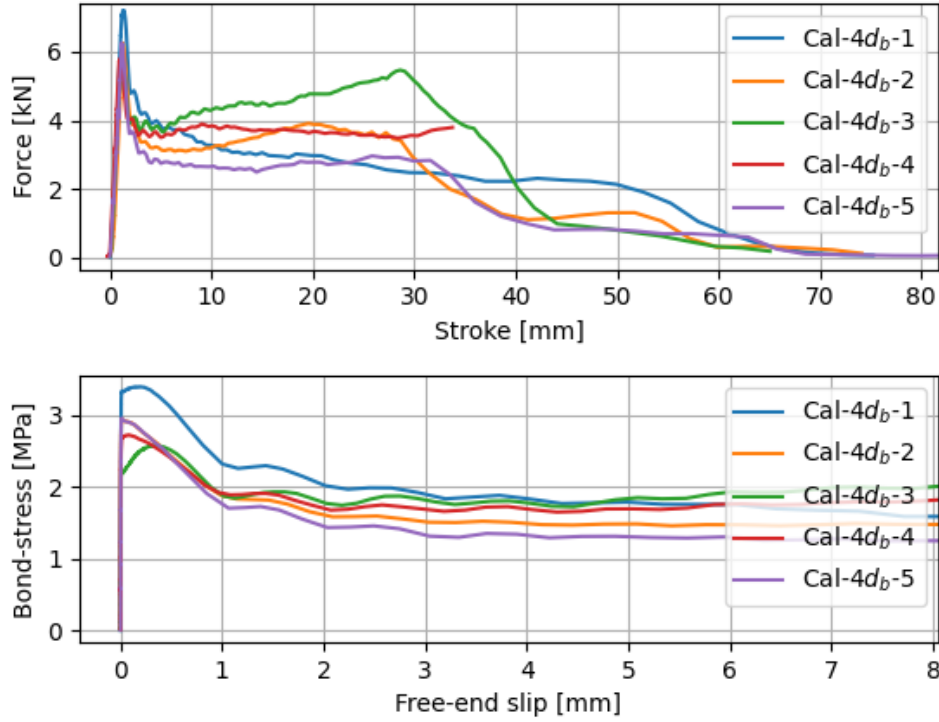


Figure 4.5: Results from the pullout tests on the Calandering rebar

The results for Calandering rebars are illustrated in Figure 4.5. Notably, we observe both a maximal (blue line) and a minimal peak (green line) at 7.19 kN and 5.46 kN, respectively, corresponding to a bond strengths of 3.38 MPa and 2.57 MPa. Concerning the shape of the bond-slip curves, a steep ascending branch is evident, where minimal free-end slip is observed until reaching the bond peak. This is followed by a rapid decrease of the bond stress after peak up to a free-end slip of approximately 2 mm, after which a constant residual stress between 1.3 MPa and 2 MPa is maintained. Sample Cal-4d_b-3 (green line) exhibits an offset in its peak compared to the other samples. Additionally, we observe for this specimen an unusual trend where, as the free-end slip increases, the bond stress also increases, which contradicts the expected behaviour. This phenomenon is more pronounced in the force versus stroke graph. Upon the bar slipping out of the concrete, it is noticeable (Fig. 4.4) that only the glass-fiber strips were damaged, remaining anchored to the concrete. This suggests a detachment failure of the fiber from the core.

Peel-ply/Grid

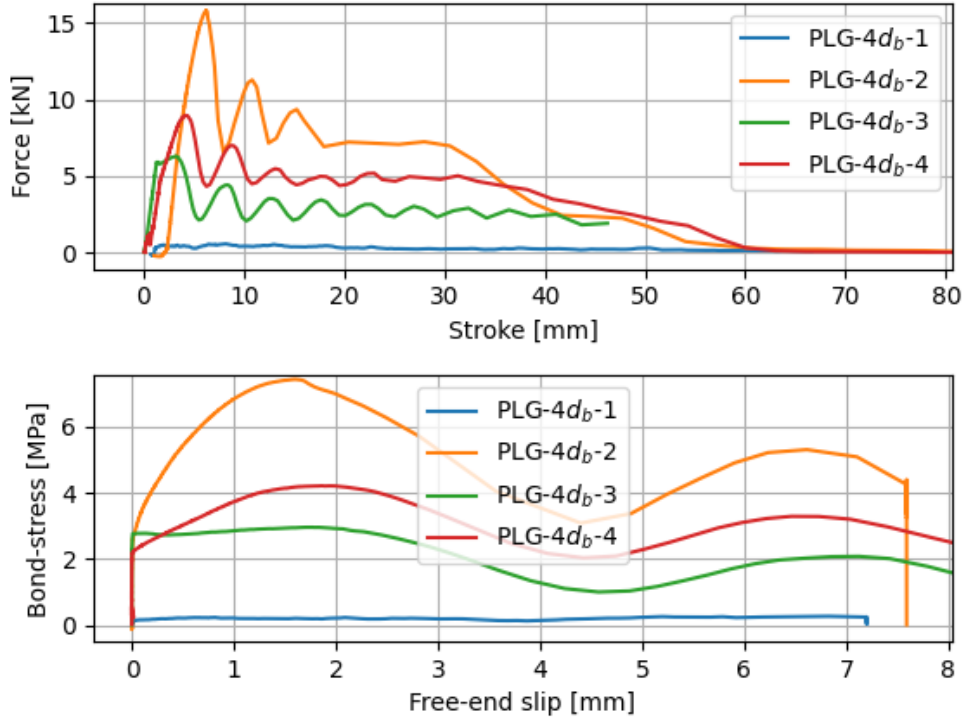


Figure 4.6: Results from the pullout tests on the Peel-ply/grid rebar

The first observation that we make is that specimen PLG-4 d_b -1 (blue line) can not be taken into account and will be excluded from the analysis. During the test, the bar immediately slipped out, generating no resistance with a maximal bond stress of nearly 0.1MPa. We can not explain what happened but it could be linked with the vibration of the concrete around the rebar during the casting. Since the embedment length is relatively short, an aggregate could have prevented the concrete from correctly settling on the rebar. Secondly, waves can be observed in the bond versus free-end slip relationship, unlike all the others. This phenomenon has already been observed in Lee et al. (2012) study but for helically wrapped rebar. In our case, this event is hard to explain, given that the surface finishing is smooth. Finally, specimen PLG-4 d_b -1 (green line) did not exhibit a peak but maintained constant bond stress. Subsequently, at approximately 2mm of slip, its behaviour shifted to resemble that of the other two specimens.

However, the Peel-ply/grid rebars still demonstrate favorable characteristics, as one sample nearly reached a bond strength of 7MPa with a slip of approximately 1.6mm, and the frictional stress is estimated to be around 60% of the bond strength.

Peel-ply

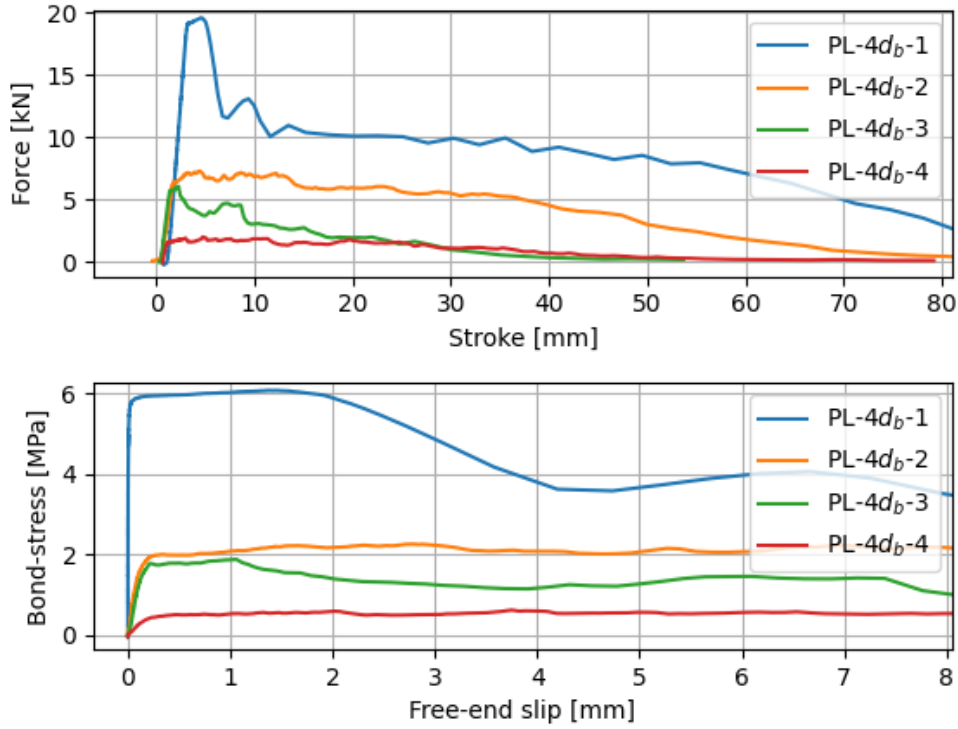


Figure 4.7: Results from the pullout tests on the Peel-ply rebar

Upon observing the outcomes for Peel-ply (PL) rebars, as depicted in Figure 4.7, a noticeable dispersion in the results becomes evident. Curiously, an anomaly was observed in three out of four specimens, primarily concerning the concrete. Specifically, the concrete surrounding the reinforcing bar appeared damp, as if it hadn't completely dried. This phenomenon could potentially be attributed to specimens being stored within a water tank two days after their casting. This timing, combined with insufficient compaction of the concrete, might have allowed water infiltration into voids, hampering proper drying. Furthermore, since the samples were cast horizontally, it's conceivable that aggregate segregation occurred, leading to different qualities of concrete layers around the rebar. Additionally, the bond stress-slip curves of the three failed samples (PL-4d_b-2, PL-4d_b-3, and PL-4d_b-4) deviate from the anticipated trend, as illustrated in Figure 2.9. With only one valid result available, establishing an average bond behaviour for these rebars becomes a challenging endeavour.

Machining

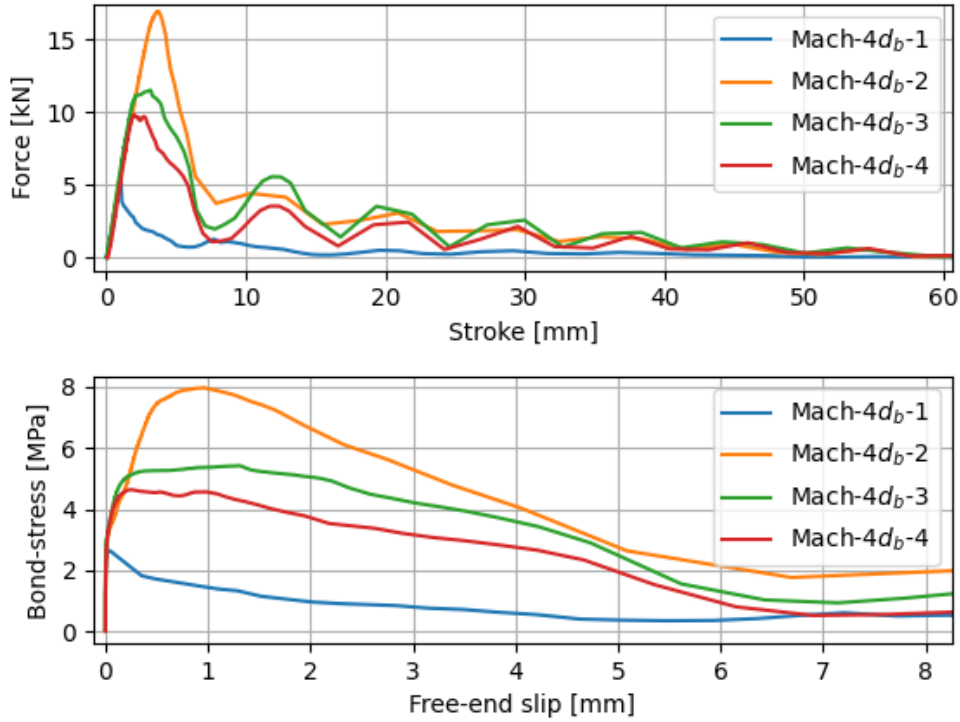


Figure 4.8: Results from the pullout tests on the Machining rebar

The machining rebars' response curves generally follow a consistent pattern, except for specimen Mach-4d_b-1 (blue curve). This deviation is evident in Figure 4.8, particularly in the force versus stroke graph. Here, around a stroke of 8mm, specimen Mach-4d_b-1's force trajectory experiences an upward trend, reaching a peak, while the other specimens exhibit a decreasing phase, eventually reaching their minimum values. Due to this unusual behaviour, and the presence of wet concrete around the rebar when it slipped out, specimen Mach-4d_b-1 will be excluded from the analysis of the average bond behaviour for machining rebars. The three other samples are following a shape such that the ascending branch has a pronounced curvature and reaches a maximum value around a free-end slip of 1mm, followed by a linear decreasing line until 6.5mm of slip and finally a residual stress between 1 and 2 MPa. In comparison to the Calandering rebars, the descending branch is more gradual, decreasing from approximately 1mm to 7mm.

As shown in Figure 4.4, there is concrete present between the ribs of the rebar. This observation implies that the failure mode occurred between the concrete and the surface of the rebar, where the concrete sheared off. This phenomenon aligns with what was observed in Vint's thesis for concrete with low strength (below 50MPa).

However, for high-strength concrete, it was the ribs of the rebar that were sheared off. Interestingly, the strength of the concrete did not significantly influence the bond strength of the rebar. In comparison, ribbed rebars in Vint’s study reached a bond strength of approximately 21MPa, whereas, in our study, we only achieved 8MPa (Vint, 2012). It’s important to note that these results might not be directly comparable due to variations in bond mechanisms stemming from differences in surface geometry, resin composition, and mechanical properties of the materials involved. Comparable outcomes from the existing literature are condensed in the table below, where mean bond strength values align, although the free-end slip at the mean bond strength appears to be greater in our tests compared to other studies.

Specimen	d_b [mm]	L_e [mm]	f_c [MPa]	τ_{max} [MPa]	S_1 [mm]
Baena et al. (2009)					
C-G/R5-12-1-C1	12	63.5	30	9.88	0.242
C-G/R5-12-2-C1	12	63.5	28.3	9.78	0.29
C-G/R5-16-1-C1	16	80	30	10.47	0.327
C-G/R5-16-2-C1	16	80	28.3	12.23	0.334
Vint Lisa (2012)					
CS5-1	16	80	30.2	9.58	0.644
CS5-2	16	80	30.2	10.10	0.458
CS5-3	16	80	30.2	7.78	0.365
Dandois Christophe (This study)					
Mach-4 d_b -2	13	52	38	7.97	0.96
Mach-4 d_b -3	13	52	38	5.42	1.3
Mach-4 d_b -4	13	52	38	4.64	0.24

Table 4.2: Bond strength and slip data of ribbed GFRP rebars from the literature

Sand-coated

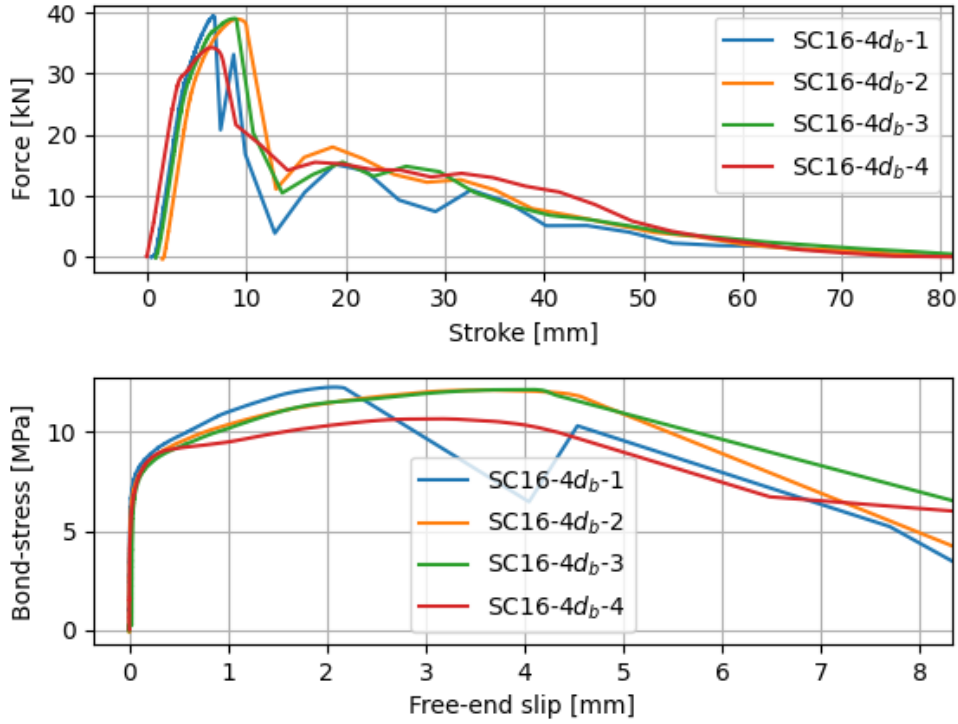


Figure 4.9: Results from the pullout tests on the Sand-coated 16mm rebar

The results indicate that all four tested samples follow the same pattern. The ascending branch exhibits an increment up to a free-end slip value of about 4mm, reaching a peak bond stress of 12MPa, and characterized by a pronounced curvature. The subsequent segment shows a linear decline until a free-end slip of 8.5mm, concluding with a constant frictional bond stress situated between 4 and 7MPa. Comparable findings have been observed in previous investigations by researchers like Baena et al. (2009) and Lee et al. (2012), along with various other sources Baena et al., 2009; J. Y. Lee et al., 2012. However, it's important to note that there is a discrepancy concerning the slip value S_1 , which is considerably larger than what is commonly reported in the existing literature. A summary of available results found in the literature as well as results from our experiments is presented in Table 4.3. Additionally, it's notable that specimen SC16-4d_b-1 (blue curve) initiated its descending branch earlier than the others. Nonetheless, around a slip of 4mm, the bond abruptly increases, returning to the anticipated behaviour.

After examining the outcomes obtained from sand-coated rebars with a nominal diameter of 16mm, one might anticipate that SC13 rebars would exhibit a higher bond strength, as discussed in the literature review chapter.

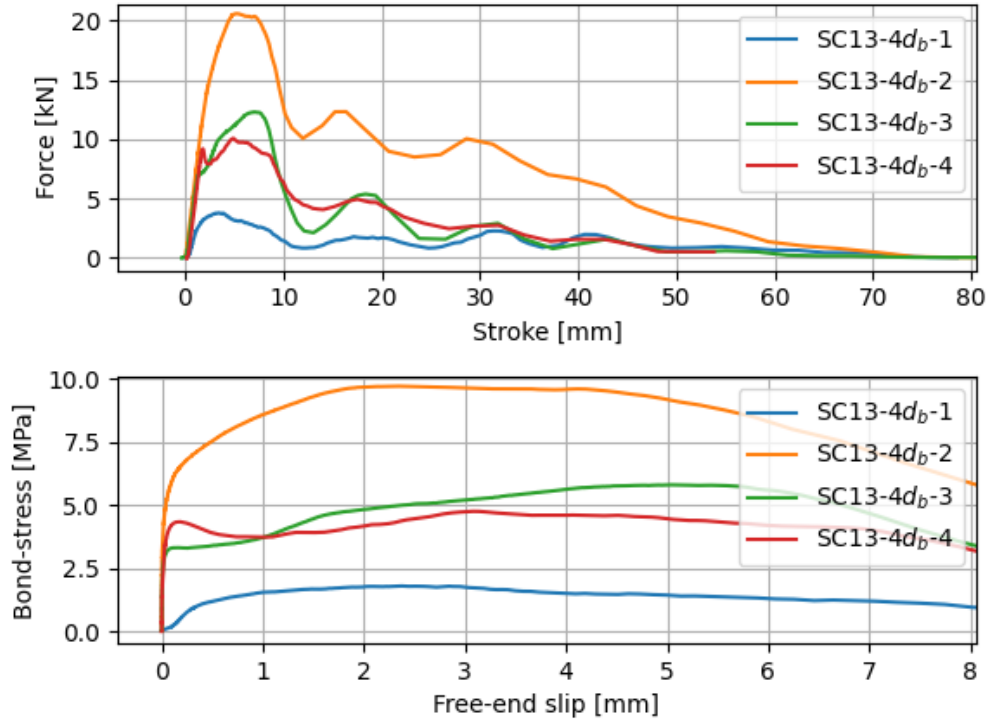


Figure 4.10: Results from the pullout tests on the Sand-coated 13mm rebar

Upon examining the graphs in Figure 4.10, a certain level of dispersion in the results becomes evident. Firstly, it's important to note that specimen SC13-4d_b-1 seems to have encountered an issue similar to the previous samples regarding the concrete, making it unsuitable for analysis. Secondly, specimens SC13-4d_b-3 and SC13-4d_b-4 (green and red curves) exhibit a shape that diverges from the anticipated pattern. Specifically, the initial segment of these two curves shows a decrease in bond stress followed by a gradual increase as the free-end slip expands. In contrast, specimen SC13-4d_b-2 (orange curve) appears to follow the expected trend, with an initial increase of the bond stress, culminating in a peak of approximately 10MPa at a slip of 2mm, followed by a declining segment and ultimately converging to consistent residual stress. Despite the unexpected nature of the results for SC13-4d_b-3 and SC13-4d_b-4, given that no anomalies were observed during the tests, it's difficult to disregard these outcomes entirely.

Results taken in the literature are resumed in the table below. For the same type of rebars, Lee et al. (2012) obtained higher values for the average bond strength and associated slip S_1 .

Specimen	d_b [mm]	L_e [mm]	f_c [MPa]	τ_{max} [MPa]	S_1 [mm]
Baena et al. (2009)					
B-G/R4-#4-1-C1	12.7	63.5	30	9.88	5.028
B-G/R4-#4-2-C1	12.7	63.5	28.3	9.78	4.189
B-G/R4-#5-1-C1	16	80	30	10.47	1.907
B-G/R4-#5-2-C1	16	80	28.3	12.23	4.947
Lee et al. (2008)					
F35C1	12.7	99.2	35.3	19.20	7.00
F35C2	12.7	99.2	35.3	19.70	3.00
F35C3	12.7	99.2	35.3	20.28	5.70
F40C1	12.7	99.2	40.6	22.62	4.83
F40C2	12.7	99.2	40.6	20.87	5.10
F40C3	12.7	99.2	40.6	20.99	4.60
Lee et al. (2012)					
HW40-1	12.7	63.5	39.3	17.2	6.25
HW40-2	12.7	63.5	39.3	17.63	4.06
HW40-3	12.7	63.5	39.3	16.05	5.95
Lee et al. (2017)					
GSP-D19-C40-1	19	124.5	39.2	15.12	6.172
GSP-D19-C40-2	19	124.5	39.2	14.74	7.846
GSP-D19-C40-3	19	124.5	39.2	19.77	4.122
Dandois Christophe (This study)					
SC16-4 d_b -1	16	64	38	12.24	2.2
SC16-4 d_b -2	16	64	38	12.09	4.6
SC16-4 d_b -3	16	64	38	12.12	4.2
SC16-4 d_b -4	16	64	38	10.64	3.2
SC13-4 d_b -2	13	52	38	9.71	2
SC13-4 d_b -3	13	52	38	-	-
SC13-4 d_b -4	13	52	38	4.34	0.2

Table 4.3: Bond strength and slip data of Sand-coated GFRP rebars from the literature

Figure 4.4 illustrates the extent of damage sustained by Sand-coated rebars after being extracted from the concrete, contrasting with the other types of rebars. The sand particles have been entirely sheared off, yet the helical strand enveloping them remains intact, albeit impaired.

4.2.1 Summary of the first set of tests

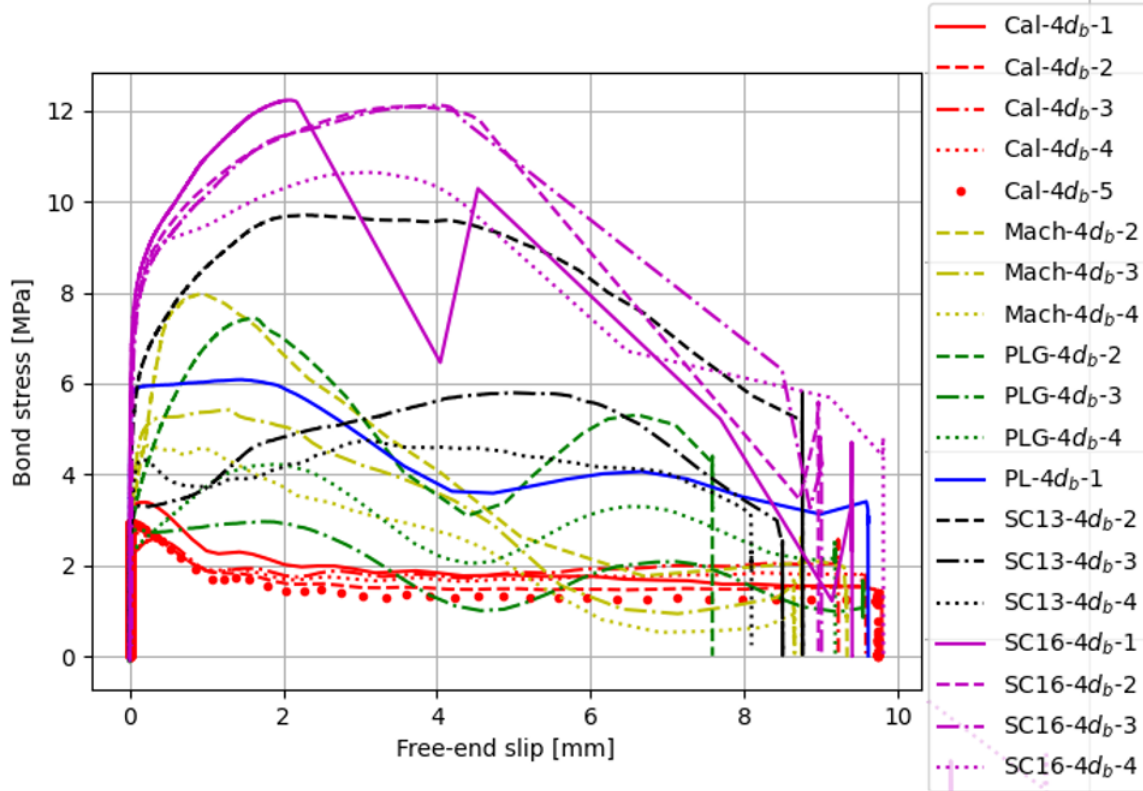


Figure 4.11: Results of all valid tested GRRP

For a comprehensive overview, Figure 4.11 presents exclusively the valid outcomes from the pullout tests conducted using the cubic samples. It's noteworthy that certain specimens were excluded due to concrete-related complications, rendering them unsuitable for establishing an average behaviour pattern of the tested rebars. This illustration facilitates a direct comparison between various surface finishes and rebar diameters. A succinct summary of the valid results obtained from the experiments is provided in Table 4.5.

The observations and conclusions of the tests are listed hereunder :

- We can conclude that rebars with roughest surfaces (Sand-Coated and Machining) perform better due to mechanical interlocking effects. They are capable of reaching high bond strength values with relatively high slip compared to the others.
- Contrary to the expected trend where an increase in rebar diameter would lead to a decrease in bond strength, an opposite phenomenon has been observed with Sand-Coated rebars.

- In the case of Peel-Ply reinforcing bars, a notable disparity among the results introduces a greater level of complexity in determining a definitive mean bond-slip behaviour for these rebars. This challenge is exacerbated by the absence of comparable results in the existing literature for reference.
- Besides having the lowest bond resistance, Calandering rebars also demonstrate a relatively rapid decline in bond stress after reaching the peak value.
- Machining rebars display the lowest frictional stress, which accounts for approximately 20% of the bond strength (τ_{max}).
- Oscillations were observed in the Peel-ply/Grid response, which is difficult to explain because of the smoothness of the surface.

Sample	d_b [mm]	L_e [mm]	f_c [MPa]	F_{max} [kN]	τ_{max} [MPa]	τ_{av} ¹ [MPa]	S_1 [mm]	Failure mode ²
Cal-4 d_b -1	13	52	38	7.2	3.38		0.2	PO
Cal-4 d_b -2	13	52	38	6.25	2.94		0.02	PO
Cal-4 d_b -3	13	52	38	5.46	2.57	2.96	0.4	PO
Cal-4 d_b -4	13	52	38	5.78	2.72		0.02	PO
Cal-4 d_b -5	13	52	38	6.25	2.95		0.02	PO
PL-4 d_b -1	16	64	38	19.58	6.08	6.08	1.5	PO
PLG-4 d_b -2	13	52	38	15.8	7.44		1.7	PO
PLG-4 d_b -3	13	52	38	6.3	2.96	5.2	-	PO
PLG-4 d_b -4	13	52	38	8.96	4.22		1.8	PO
Mach-4 d_b -2	13	52	38	16.93	7.97		0.96	PO
Mach-4 d_b -3	13	52	38	11.52	5.42	6.27	1.3	PO
Mach-4 d_b -4	13	52	38	9.85	4.64		0.24	PO
SC13-4 d_b -2	13	52	38	20.6	9.7		2	PO
SC13-4 d_b -3	13	52	38	12.31	-	7.2	-	PO
SC13-4 d_b -4	13	52	38	10.08	4.75		0.2	PO
SC16-4 d_b -1	16	64	38	39.37	12.24		2.2	PO
SC16-4 d_b -2	16	64	38	38.75	12.05		4.6	PO
SC16-4 d_b -3	16	64	38	38.98	12.12	11.3	4.2	PO
SC16-4 d_b -4	16	64	38	33.44	10.4		3.2	PO

Table 4.4: Summary of all valid experimental results

¹Average bond strength

²PO: pullout

4.3 Results from the pullout tests with the cylindrical samples

4.3.1 Analysis of measurements taken by the LVDTs

Due to the different casting dates, the compressive strengths of the concrete samples vary (32 MPa and 41 MPa), as mentioned in Section 3.6. However, a clear pattern emerges despite the difference in compressive strength: bond strength decreases as the embedment length increases. This is because longer embedment lengths result in more defects along the bar. This has been previously discussed by Acchillides and Pilakoutas in 2004, who attributed it to the non-linear dispersion of bond stress along the bar (Achillides and Pilakoutas, 2004). Figure 4.13 illustrates the effect of concrete on the average bond strength, with dimensionless axes. Interestingly, this trend remains consistent regardless of the compressive strength of the concrete.

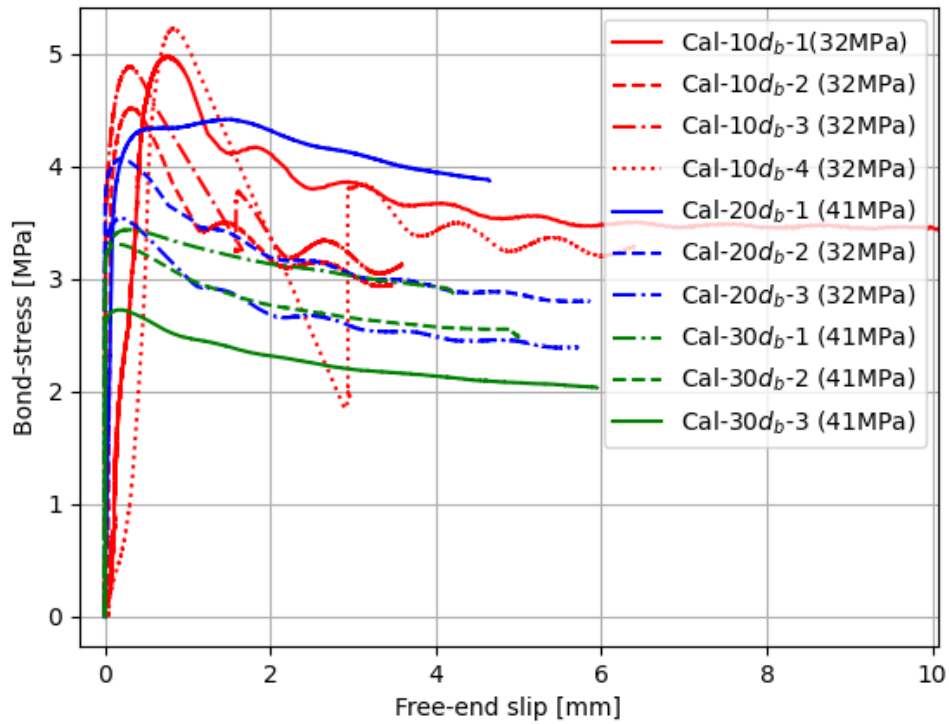


Figure 4.12: Influence of the embedment length on the relation between the free-end slip and the bond stress

When examining the behaviour of these rebars, it becomes clear that there is a noticeable similarity in their response patterns. A peak in bond stress is consistently seen when there is a free-end slip that ranges from around 0.15mm to 0.8mm. It should be mentioned that specimen Cal-20 d_b -1 is an exception to this trend, as it shows larger slip values at its peak. After reaching the peak, there is a rapid decrease in bond stress, followed by a steady residual stress level. As the embedment length increases, the ratio between the bond strength and the residual bond stress appears to increase. Specifically, for an embedment length of 10 d_b , this ratio is roughly 70% of the bond strength, and it increases to 80% for 20 d_b , ultimately reaching 90% for 30 d_b .

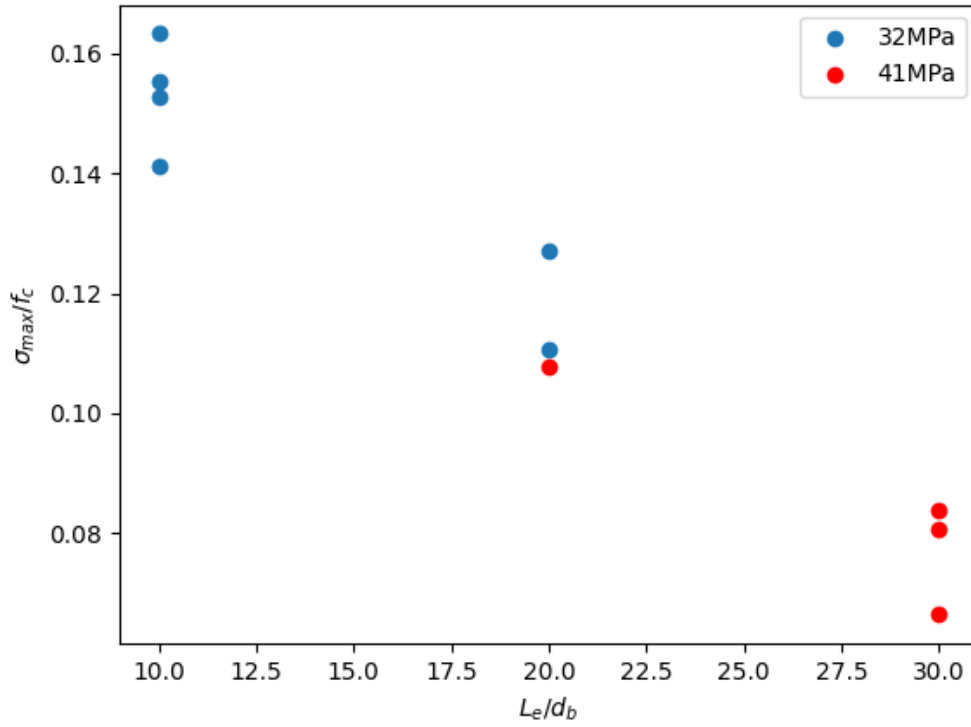


Figure 4.13: Influence of the concrete compressive strength on the average bond strength

4.3.2 Analysis of measurements taken by the DFOS

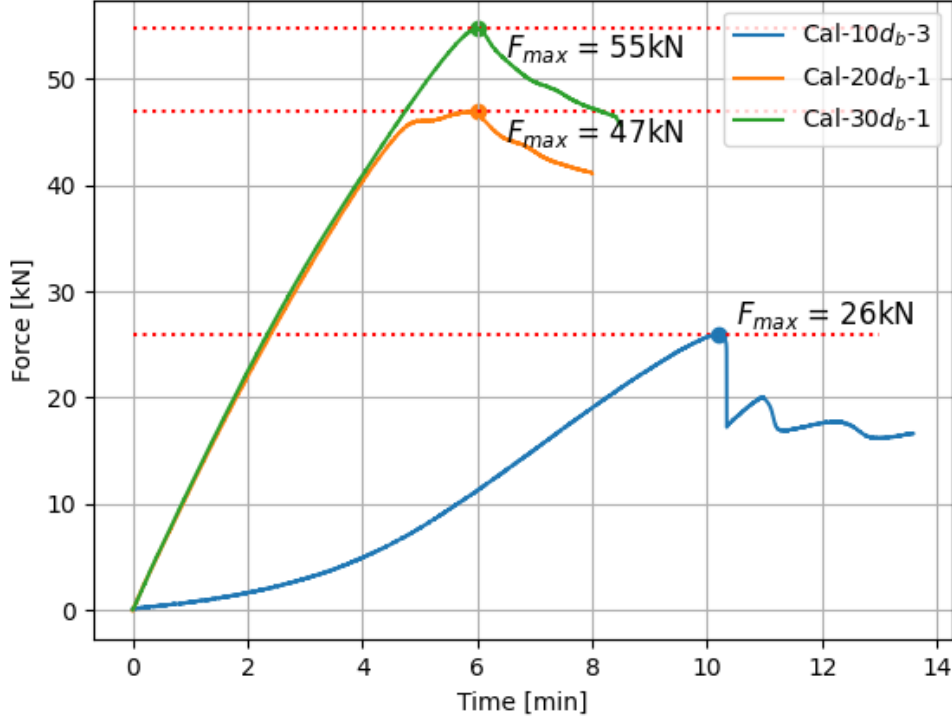


Figure 4.14: Evolution of the force versus the time

The graph above shows how the applied force changes over time for three samples that have Distributed Fiber Optical Sensors (DFOS). Our goal is to figure out the highest force each sample experienced, which will help us find the moment of peak strain. After looking at the graph, we can see that the curve for sample Cal-10 d_b -3 is different from the other two samples. This is because we put a rubber mat underneath Cal-10 d_b -3 to fix a problem with the support surface. Because of the rubber mat, it took 10 minutes for Cal-10 d_b -3 to reach its maximum force, which is longer than the 6 minutes it took for the other samples.

In the following graphs (Fig 4.15, Fig4.16 and Fig 4.17), we can see the strain evolution along the bonded length for an embedment length of 10 d_b , 20 d_b and 30 d_b . For each graph, the curves are plotted as a ratio of the peak force reached for the concerned specimen.

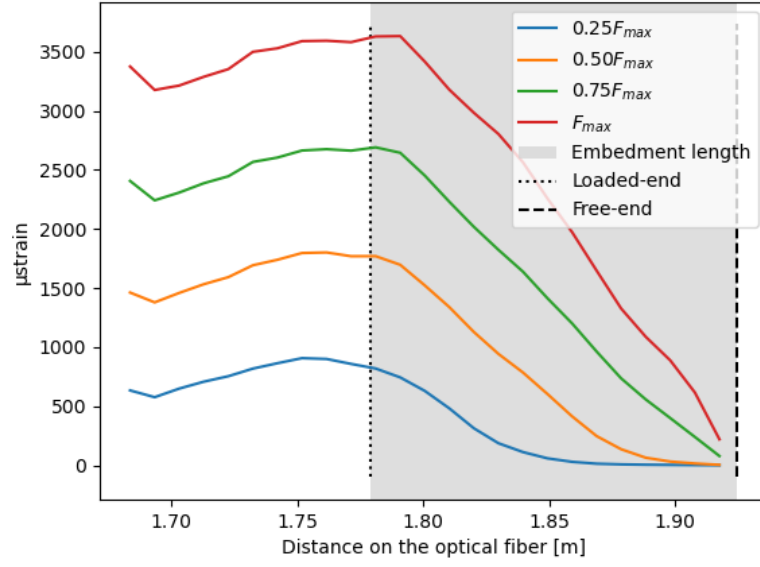


Figure 4.15: Evolution of the strain along the embedment length ($10d_b$) with time

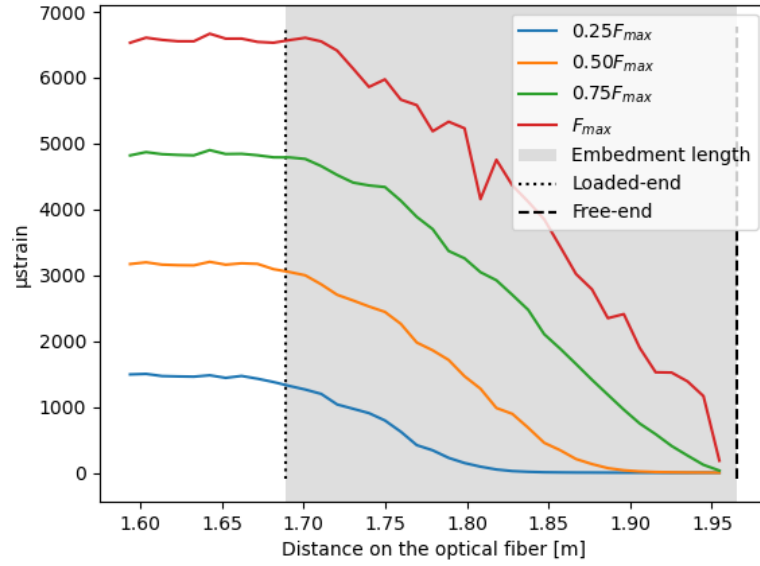


Figure 4.16: Evolution of the strain along the embedment length ($20d_b$) with time

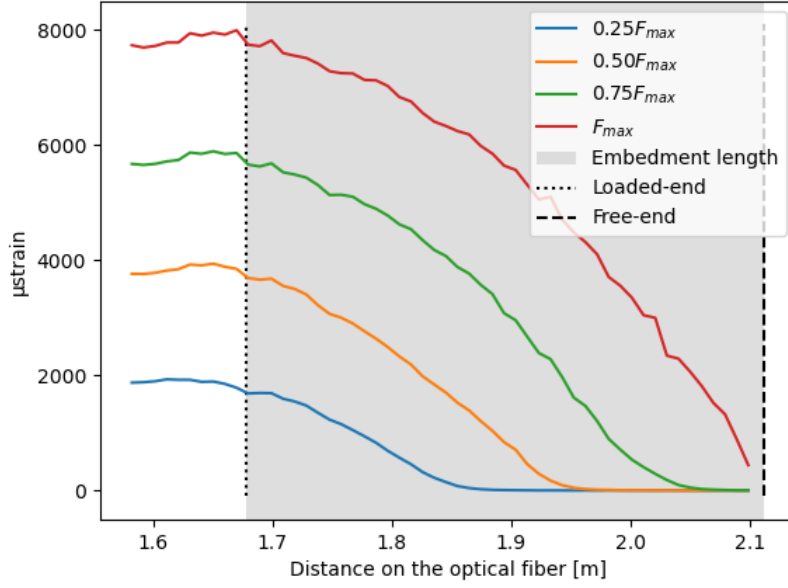


Figure 4.17: Evolution of the strain along the embedment length ($30d_b$) with time

A consistent pattern is evident, wherein the strain gradually amplifies from the loaded end and diffuses along the embedment length until reaching the free end. Notably, as the bonded length extends, strains intensify, while the slope between loaded and free ends becomes less steep. Moreover, we observe between an anchorage length of $10d_b$ and $20d_b$ that the strain peak escalates from 3500 μstrain to 7000 μstrain , effectively doubling. On the other hand, between anchorage lengths of $20d_b$ and $30d_b$, the augmentation in strains is comparatively restrained, registering an increase from 7000 to 8000 μstrain , equivalent to an approximate 15% upsurge.

Extensometer and DFOS

The elongation of the rebar, which was measured by the extensometer, will be compared with the data obtained from the DFOS. The extensometer was placed at the loaded end of the rebar, just below the optical fiber. Therefore, the highest deformation recorded by the extensometer should correspond to the maximum strain recorded by the DFOS at the loaded end.

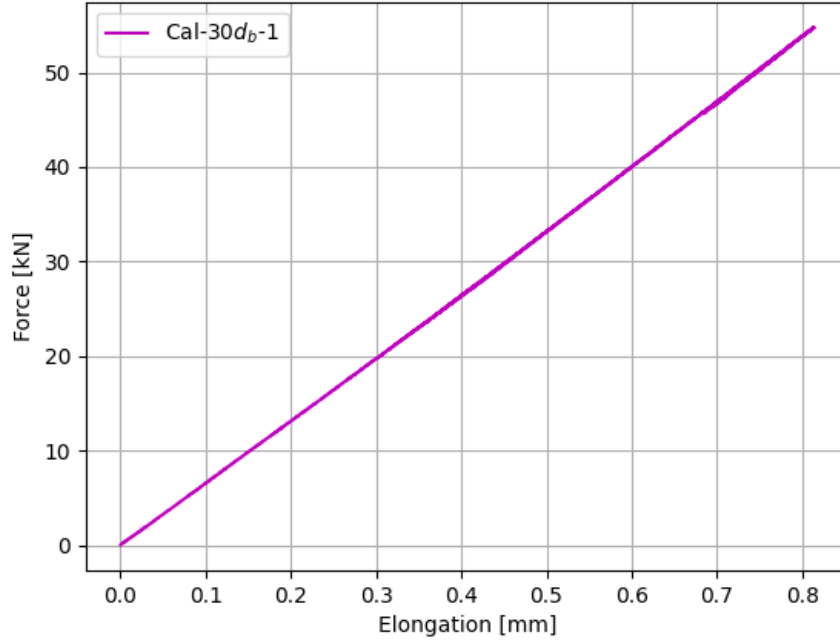


Figure 4.18: Data recorded from the extensometer for the specimen Cal-30 d_b -1

Let's take a look at an example, specifically specimen Cal-30 d_b -1. The maximum elongation recorded by the extensometer was 0.8mm, which you can see in Figure 4.18. To calculate the deformation of the bar, also known as the strain (ϵ), we use equation 4.1. The elongation of the bar, expressed in millimeters, is represented by ΔL , while L_0 is the initial length between two reference points. It's important to note that the specific length L_0 varies depending on the extensometer model chosen. For our study, we set this length at 100mm.

$$\epsilon = \frac{\Delta L}{L_0} \quad (4.1)$$

Upon using the aforementioned equation, the resulting deformation ϵ is calculated to be 0.8, which corresponds to 8000 μ strains, as determined by the DFOS measurements.

LVDT and DFOS

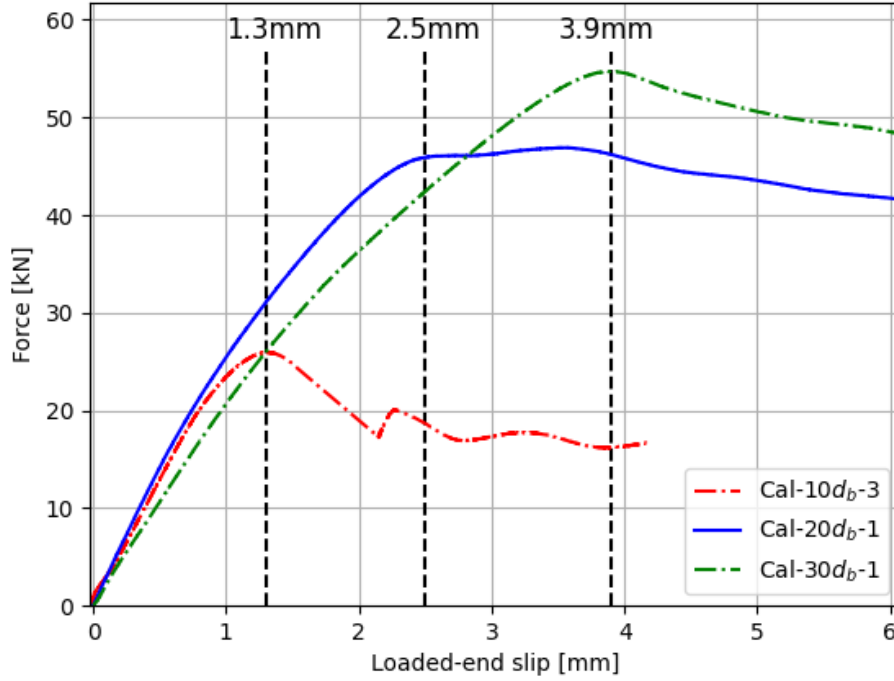


Figure 4.19: Influence of the embedment length on the relation between the loaded-end slip and the bond stress

A comparative analysis will be conducted between the data obtained from the LVDTs and the optical fibers. Specifically, by integrating the strain along the embedment length for the maximal applied force F_{max} , approximate equality should be observed with the loaded-end slip for the same applied force. The elongation of the bar (Δ_v) can be calculated by integrating the measured strain across the embedment length (L_e), as indicated in equation 4.2. Given that the gauges length is established at 0.65mm, the simplest approach for integration involves multiplying each data point by this gauge length and subsequently calculating the summation.

$$\Delta_v = \int_0^{L_e} \epsilon dx \quad (4.2)$$

Furthermore, Figure 4.19 provides a visualization of the loaded-end slip S_1 corresponding to the occurrence of the peak force. Evidently, there is a direct correlation between the increase in embedment length and the corresponding increase in loaded-end slip.

After integration, we obtain the following results:

- $\Delta_{v,10} = 0.648\text{mm} \leq 1.3\text{mm}$ for $L_e = 10d_b$
- $\Delta_{v,20} = 1.75\text{mm} \leq 2.5\text{mm}$ for $L_e = 20d_b$
- $\Delta_{v,30} = 2.215\text{mm} \leq 3.9\text{mm}$ for $L_e = 30d_b$

It is important to note that the integration of the strains across the embedment length yields values that are smaller than the loaded-end slip measured by the LVDT for the same applied force, represented by F_{max} in this context.

4.3.3 Summary of the second set of tests

In summary, our experiments have shown that increasing the embedment length leads to a decrease in bond strength, along with an increase in the ratio between bond strength and residual stress. This shift in ratio ranges from 55% at an embedment length of 4 times the diameter of the bar ($4d_b$), to 70% at $10d_b$, 80% at $20d_b$, and eventually 90% at $30d_b$. As for the strain distribution along the embedment length, the peak strain experiences a twofold increase between $10d_b$ and $20d_b$, while there is only a 15% increase between $20d_b$ and $30d_b$. It is worth noting that the results from the Distributed Fiber Optic Sensors (DFOS) align with the extensometer data, but they differ from the measurements taken by the Linear Variable Differential Transformer (LVDT), which yield higher displacement values.

Sample	d_b [mm]	L_e [mm]	f_c [MPa]	F_{max} [kN]	τ_{max} [MPa]	τ_{av} ³ [MPa]	S_1 [mm]	Failure mode ⁴
Cal- $10d_b$ -1	13	130	32	26.42	4.97		0.75	PO
Cal- $10d_b$ -2	13	130	32	24	4.52	4.89	0.32	PO
Cal- $10d_b$ -3	13	130	32	25.95	4.88		0.3	PO
Cal- $10d_b$ -4	13	130	32	27.75	5.23		0.82	PO
Cal- $20d_b$ -1	13	260	41	46.91	4.18		-	PO
Cal- $20d_b$ -2	13	260	32	43.16	4.06	3.93	0.2	PO
Cal- $20d_b$ -3	13	260	32	37.58	3.54		0.2	PO
Cal- $30d_b$ -1	13	390	41	54.75	3.44		0.3	PO
Cal- $30d_b$ -2	13	390	41	52.69	3.31	3.16	0.15	PO
Cal- $30d_b$ -3	13	390	41	43.38	2.72		0.2	PO

Table 4.5: Summary of all valid experimental results

³Average bond strength

⁴PO: pullout

Chapter 5

Bond-slip numerical model

5.1 General description of the bond-slip model

In chapter 2, it was observed that strain penetrations (SP) can account for up to 40% of the total lateral displacement of RC members. While finite element (FE) software can accurately replicate SP effects, they are computationally demanding. On the other hand, fiber-based beam-column software has limitations since it is generally based on simplified empirical constitutive relations (Sousa, 2015). The bond-slip numerical model, created by Romain Sousa in 2015, aims to replicate the physical phenomena of strain penetration between the concrete and a reinforcing bar in beam-column elements. The code was originally intended for steel rebar and later modified for glass-fiber reinforced polymer rebars.

To be as general as possible and to account for all the parameters and physical phenomena influencing the SP effects in the anchoring region, the model is based on an appropriate bond-slip local constitutive relation. Among others, the concrete strength, the geometry of the rebar surface, the embedment length, and the type of failure are to be accounted for.

To guarantee the simplicity and the efficiency of the model with a reasonable computational effort, the model was developed with a zero-length element that simulated the behaviour of the rebar along its anchorage region, as explained in the section 2.7. Since it is extremely difficult to anticipate how the different parameters (the slip, the bond stress, the strain, and the force) will evolve along the embedment length, the model defines a series of integration points (IPs) to keep track of all the responses along the embedment length as illustrated in the figure 5.1. Equilibrium and compatibility equations have to be verified for each IPs.

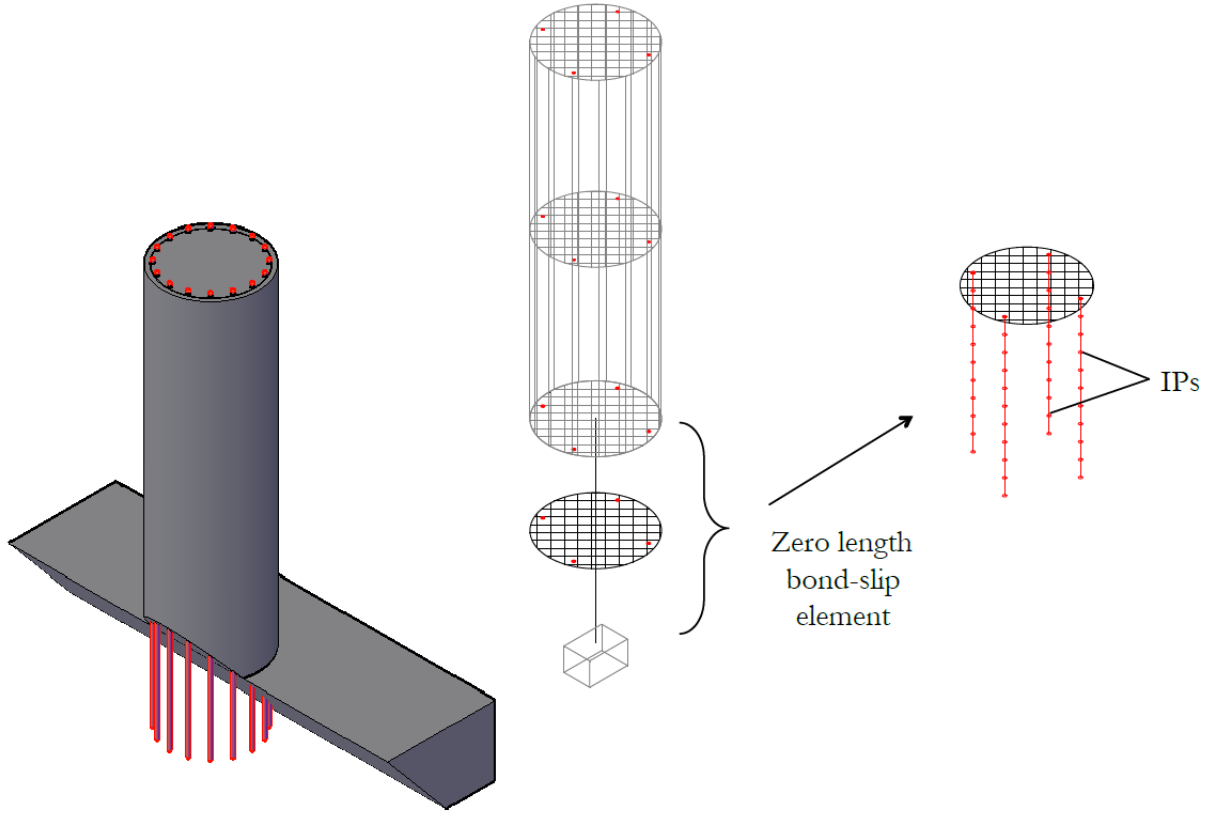


Figure 5.1: Representation of the IPs along the embedment length and the zero-length element (adapted from Sousa, 2015)

The flowchart (Fig. 5.2) shows the operating principle of the numerical model. Starting from an imposed slip (S_0) at the rebar's loaded-end, the model will calculate for every integration point (IPs) along the embedment length, the slip (S), the force (F), the stresses (σ) and strains (ϵ). Then, through an iterative procedure, the applied force (F_0) at the loaded-end of the rebar will be computed. Figure 5.3 illustrates what was explained as well as the evolution of the cited parameters along the embedment length. As you can see, L_d is called the development length, which is the length required for the rebar force at the rebar to reduce to zero (Sousa, 2015).

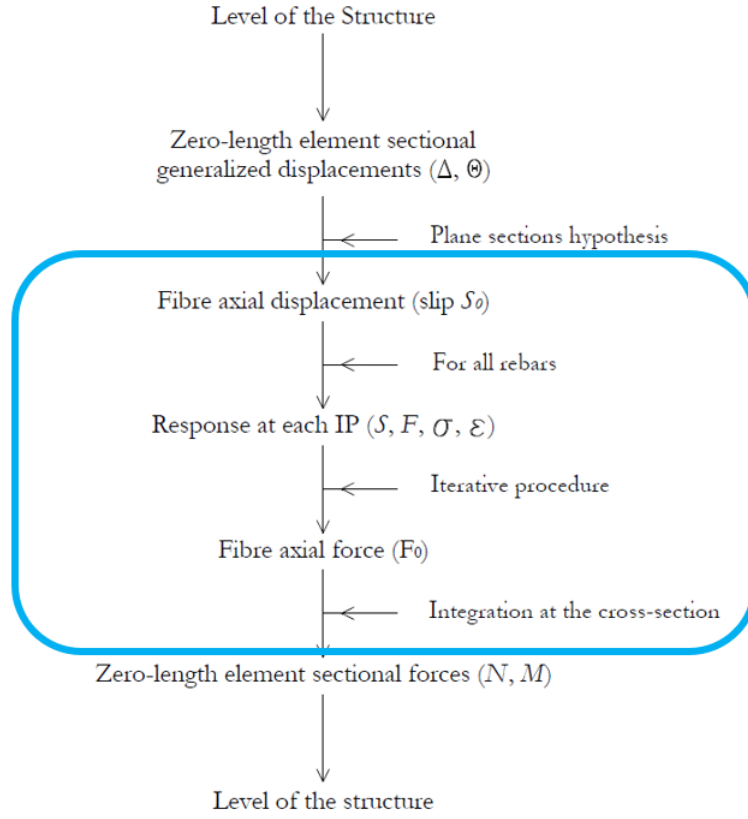


Figure 5.2: Simplified flowchart of the bond stress-slip numerical model (adapted from Sousa, 2015)

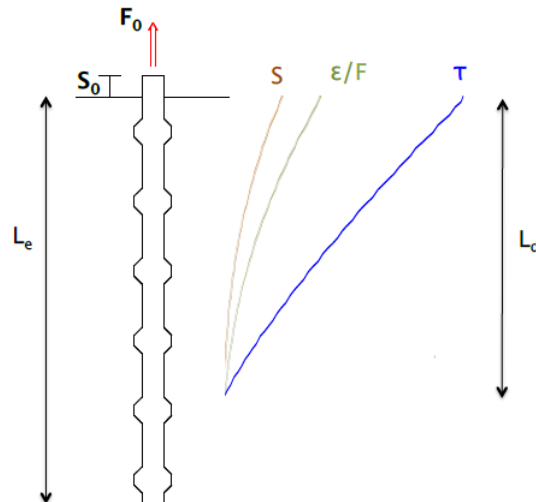


Figure 5.3: Evolution of the slip, the stresses, the force and the strains along the embedment length for steel rebars (adapted from Sousa, 2015)

5.2 Input parameters of the numerical model

The program requires many different input parameters that will influence the output responses. These input parameters can be divided into 3 categories. The first one is the geometric properties of the analyzed cross-section, followed by the material properties of the considered elements and the last category regroups the parameters defining the bond stress-slip constitutive relation (Sousa, 2015).

5.2.1 Geometric properties

The two geometric inputs are the embedment length L_e and the diameter of the bar (d_b) expressed in [m]. Then, we can compute the number of IPs needed with the following formula:

$$NoIPs = \frac{L_e}{0.01} + 1 \quad (5.1)$$

Equation 5.1 create one integration point every centimeter but can be adjusted to have a precision every millimeter by changing 0.01 to 0.001.

5.2.2 Material properties

Concrete properties

The numerical model takes the concrete compressive and tensile strength into account. The compressive strength can be chosen by the user and expressed in [KPa] but the tensile strength is calculated with the following formula:

$$f_{ct} = 0.75 \sqrt{\frac{f_c}{1000}} \times 1000 [KPa] \quad (5.2)$$

GFRP properties

The first parameter to consider for GFRP is the modulus of elasticity, E, which is measured in KPa. Due to its brittle behavior, GFRP can be represented as a straight line in a stress-strain diagram without yielding of the rebar. Therefore, we only take into account the ultimate stress (f_u) and ultimate strain (ϵ_u), both expressed in KPa and m respectively.

5.2.3 Bond properties

The model uses the local bond-slip constitutive relation of the BPE-modified model developed by Cosenza et al. (See Section 2.6.3).

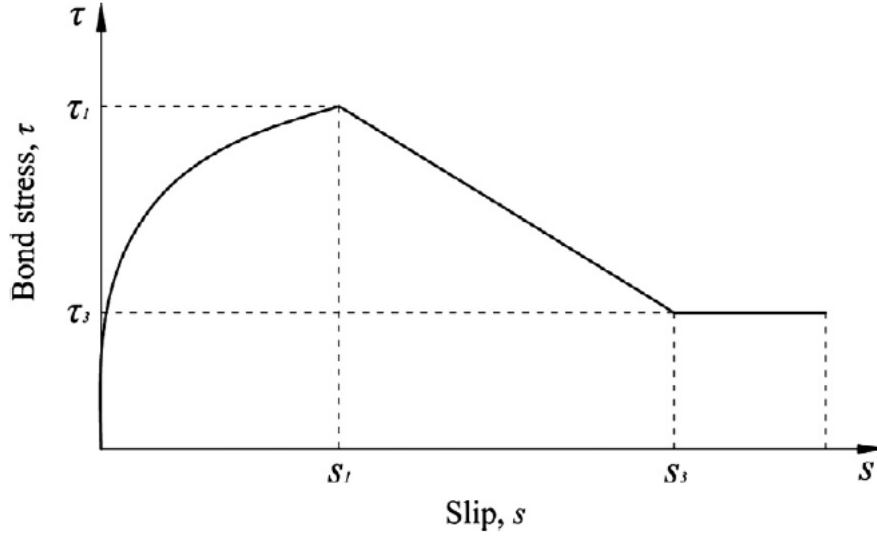


Figure 5.4: BPE modified model (adapted from Lin et al., 2014)

The bond properties are determined by considering slip limits such as S_1 , S_2 , and S_3 , as well as the maximum bond stress (τ_{max}) and residual bond stress (τ_f). Additionally, a bond stress parameter called α is taken into account, which affects the response in the ascending branch of the bond stress-slip relationship. The model was originally created for steel rebars, and as a result, S_2 is set to be equivalent to S_1 in order to remove the plateau from the steel behaviour, as shown in Figure 2.8.

The generic bond stress-slip response under pullout failure is represented in the figure 2.9 where all the bond parameters are illustrated. This constitutive law can be divided into three parts and can be calculated with the following equations:

- $0 \leq S \leq S_1$

$$\tau(S) = \tau_{max} \left(\frac{S}{S_1} \right)^\alpha \quad (5.3)$$

- $S_1 \leq S \leq S_3$

$$\tau(S) = \tau_{max} - (\tau_{max} - \tau_f) \left(\frac{S - S_1}{S_3 - S_1} \right) \quad (5.4)$$

- $S \leq S_3$

$$\tau(S) = \tau_f \quad (5.5)$$

In the proposed model, the maximal bond stress τ_{max} and the frictional bond stress will be of the following form:

$$\tau_{max} = \kappa f_c^{0.33} \quad (5.6)$$

$$\tau_f = \eta \tau_{max} \quad (5.7)$$

where κ and η will vary following the rebar's surface condition.

The slip limit, denoted as S_1 , is determined by factors such as τ_{max} , f_c , and L_e . As suggested by Lee et al. (2012), the slip limit can be expressed in the following way:

$$S_1 = \left(\frac{\tau_{max}}{f_c} \right) \left(\frac{L_e}{\xi} \right) \quad (5.8)$$

The parameter α will have an influence on the curvature of the ascending branch. The greater the value of alpha, the smaller the curvature.

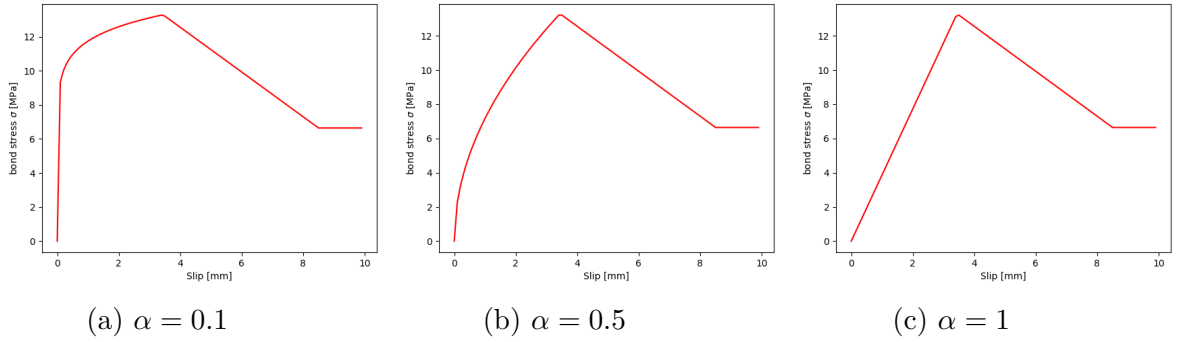


Figure 5.5: Influence of the input parameter α

5.3 Literature benchmark

A study made by Lee et al. in 2012 has shown the comparison of such equations with experimental results available in the literature. The GFRP rebars studied were sand-coated and helically wrapped with a rebar diameter of 12.7 mm and an embedment length of $5d_b$. Different concrete compressive strengths (25MPa, 40MPa and 70MPa) were also used in this study to analyze its influence. The results are presented in the following figure 5.6, with the bond stress for the y-axis and the slip for the x-axis expressed in [MPa] and in [mm] respectively (J. Y. Lee et al., 2012).

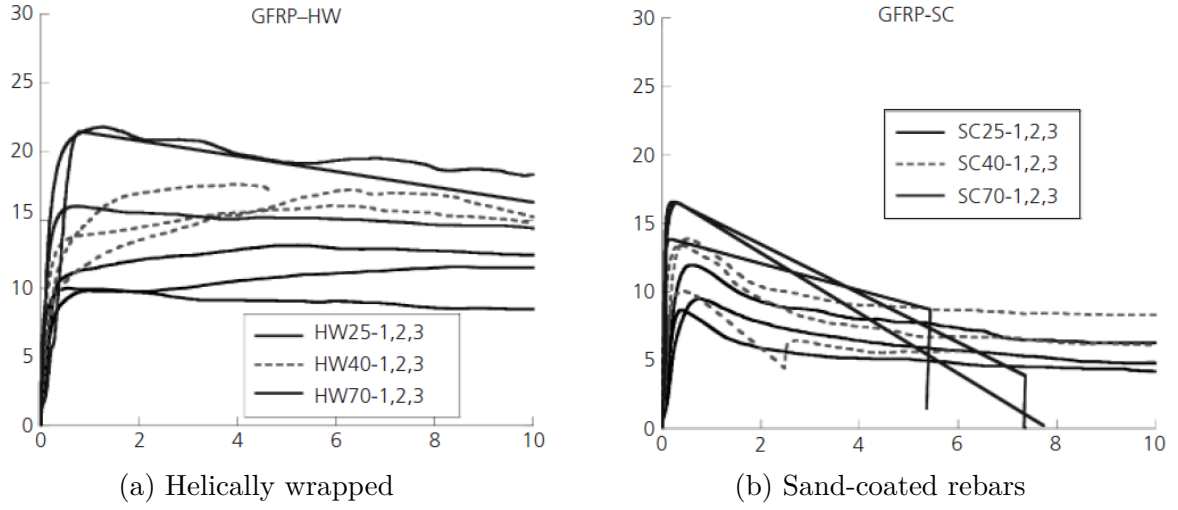


Figure 5.6: experimental results from Lee et al. (2012) for sand-coated and helically wrapped GFRP rebars

The obtained results have provided the necessary parameters to apply to the bond-slip model for both SC and HW rebars. These parameters are presented in the table below to match the results.

Sand-coated rebars	Helically wrapped rebars
$\alpha = 0.5$	$\alpha = 0.15$
$\tau_{max} = 4f_c^{0.33}$	$\tau_{max} = 4f_c^{0.33}$
$\tau_f = 0.5\tau_{max}$	$\tau_f = 0.5\tau_{max}$
$S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{60}$	$S_m = \frac{\tau_{max}}{f_c} \frac{L_e}{8}$

Table 5.1: Bond-slip input parameters proposed by J. Y. Lee et al., 2012

Based on the input values provided in Table 5.1, our developed bond-slip model for GFRP produces the following outcomes.

Based on the following figures, it appears that the model accurately replicates the behaviour of the SC rebars. The maximum values of bond stress align with Lee's findings, along with the slip limits. However, the numerical results for the HW rebars do not accurately reflect their bond behaviour. The bond strength is underestimated and the slip limit S_1 appears to be greater than it truly is.

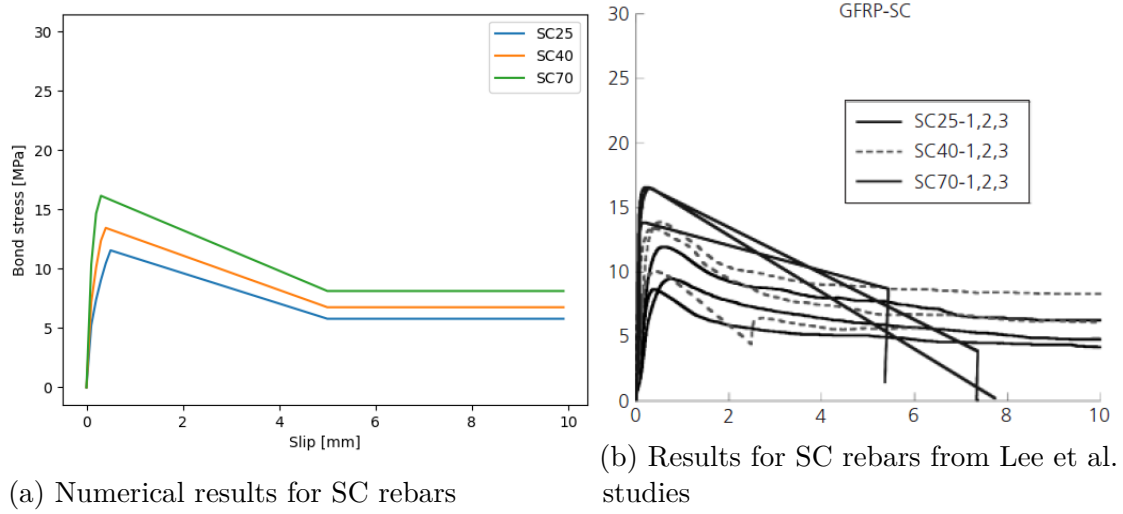


Figure 5.7: Comparison between the numerical and Lee's results for SC rebars

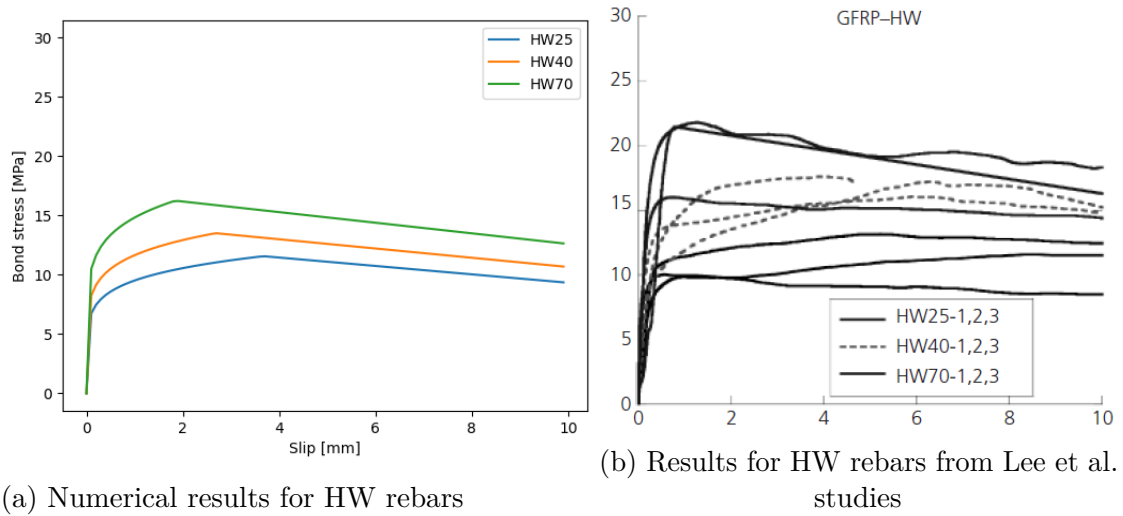


Figure 5.8: Comparison between the numerical and Lee's results for HW rebars

In 2022, Zhao et al. conducted a study on the impact of embedment length on GFRP rebars. They tested five different lengths, including $5d_b$ (red lines), $10d_b$ (black lines), $20d_b$ (dark blue lines), $30d_b$ (light blue lines), and $40d_b$ (green lines), with a rebar diameter of 12mm. The rebars were sand-coated and had a carbon yarn wrapped around the surface. The tests were conducted 28 days after casting, using concrete with an average compressive strength of 31MPa. Unlike Lee et al., Zhao et al. presented their results (seen in Fig 5.9) based on the slip at the loaded end as a function of the applied load, rather than the bond stress.

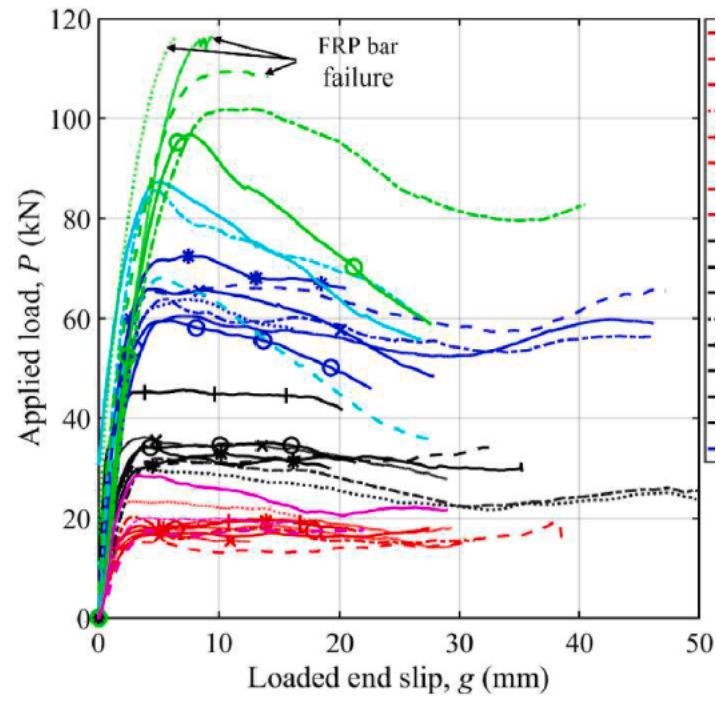


Figure 5.9: Results for sand-coated rebars from Zhao et al.

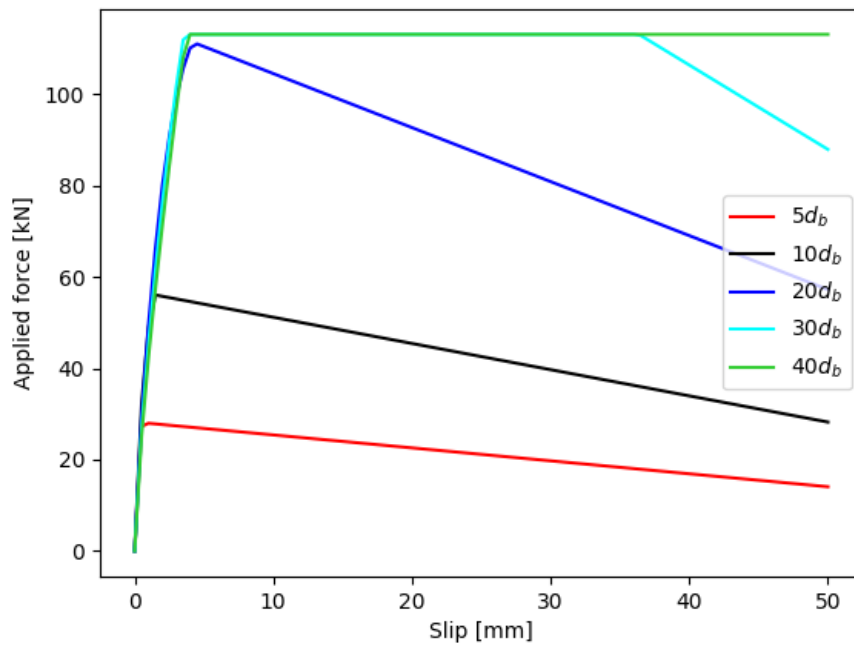


Figure 5.10: Prediction of the numerical model for sand-coated rebars at various embedment lengths

When we compare the results of Zhao et al. to the predictions of our numerical model, it is clear that the model can mimic the behaviour of GFRP with an embedment length of $5d_b$, as shown by the red curves. However, when we examine the other curves, it is clear that the model tends to overestimate the force. For example, in the case of the dark blue curve, the model predicts a force that exceeds 100 kN, while Zhao et al.'s results indicate a force of around 80 kN. Additionally, at embedment lengths of $30d_b$ and $40d_b$, the GFRP reaches its failure point, resulting in the horizontal sections of those curves.

5.4 Outputs of the numerical model

As mentioned previously, the model is also able to predict the slip, the bond stress, the strain, and the force along the embedment length for an imposed slip (S_0) at the rebar's loaded-end. To illustrate this, we will use the same parameters as the ones proposed in the study of Lee et al. (See table 5.1) with an imposed slip equal to 1mm. It is evident that when the embedment length is adequate, the slip, force, bond stress, and strain decrease to zero. The length required for these values to reach zero is known as the development length (L_d).

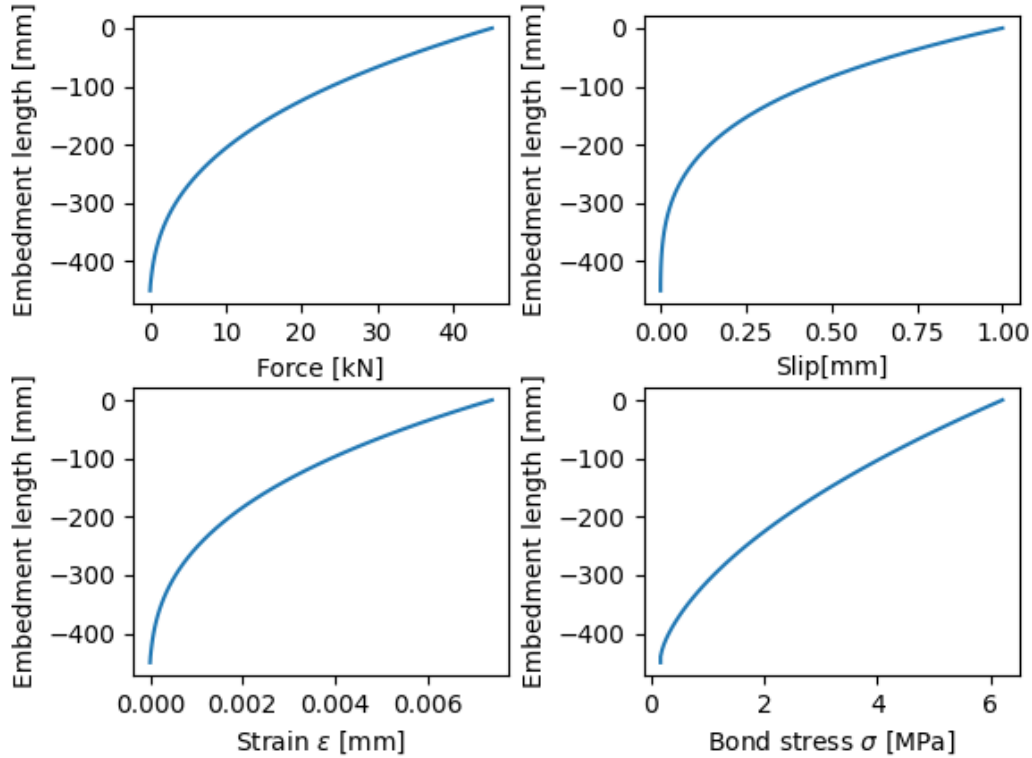


Figure 5.11: Prediction of the bond-slip numerical model along the embedment length with the input parameters proposed by Lee et al.

5.5 Adjustments and validation of the input parameters

The aim is to establish the typical behavior of each GFRP by making necessary adjustments to the numerical model inputs.

Calandering rebar

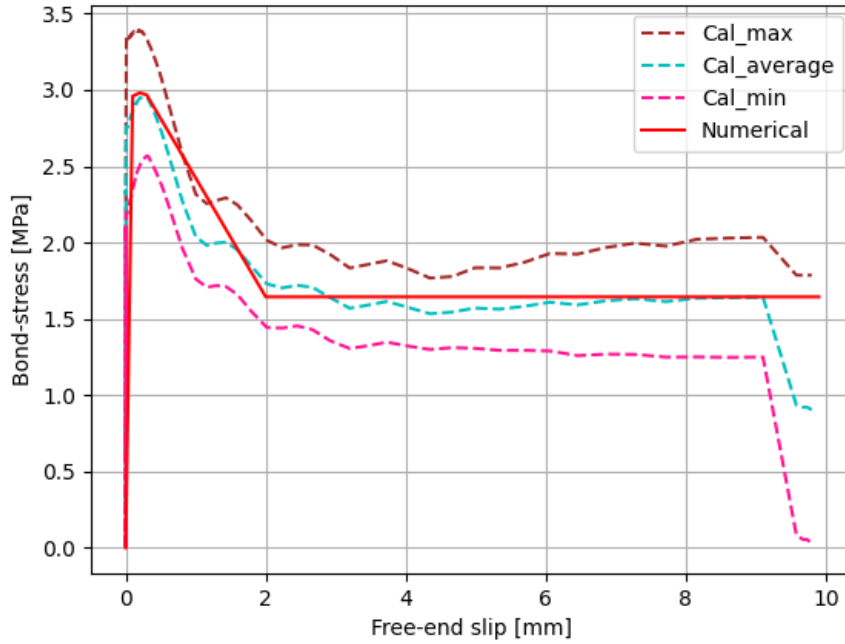


Figure 5.12: Average bond stress-slip behaviour of the calandering GFRPs

$$\alpha = 0.01 \quad \tau_{max} = 0.9f_c^{0.33} \quad \tau_f = 0.55\tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{15} \quad S_3 = 2\text{mm}$$

The diagram in Figure 5.12 shows the upper and lower limits, as well as the average response of the Calandering rebars, represented by dotted lines. The red line displays the most accurate average behavior obtained from the numerical model. By using the bond parameters that were determined manually and are listed below, an average bond strength of approximately 3 MPa was achieved. This was associated with a slip value of 0.28 mm, and an average residual bond stress of 1.64 MPa, which is 55% of the maximum bond strength.

Upon closer inspection of the ascending branch, it becomes evident that the average behaviour line exhibits a notably steep incline. Despite attempts to adjust the value of α to a smaller magnitude, the numerical code still struggles to precisely replicate this sharply ascending straight line.

Peel-ply/grid rebar

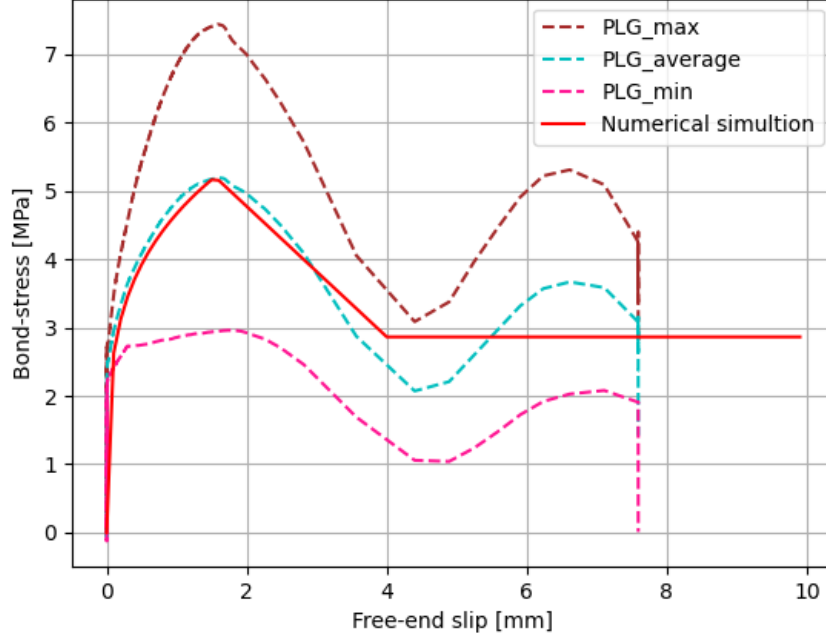


Figure 5.13: Average bond stress-slip behaviour of the peel/ply grid GFRPs

$$\alpha = 0.25 \quad \tau_{max} = 1.8f_c^{0.33} \quad \tau_f = 0.55\tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{7} \quad S_3 = 4\text{mm}$$

In the previous chapter, we analyzed the results and found significant variation among the outcomes. Figure 5.13 shows the highest and lowest behaviours plotted. The maximum stress, τ_{max} , shows a range of approximately 3 to 7.5 MPa, resulting in a gap of 4.5 MPa. The post-peak response also displays waves that were difficult to simulate using the numerical model. Therefore, we used an average value for modeling purposes.

Peel-ply rebar

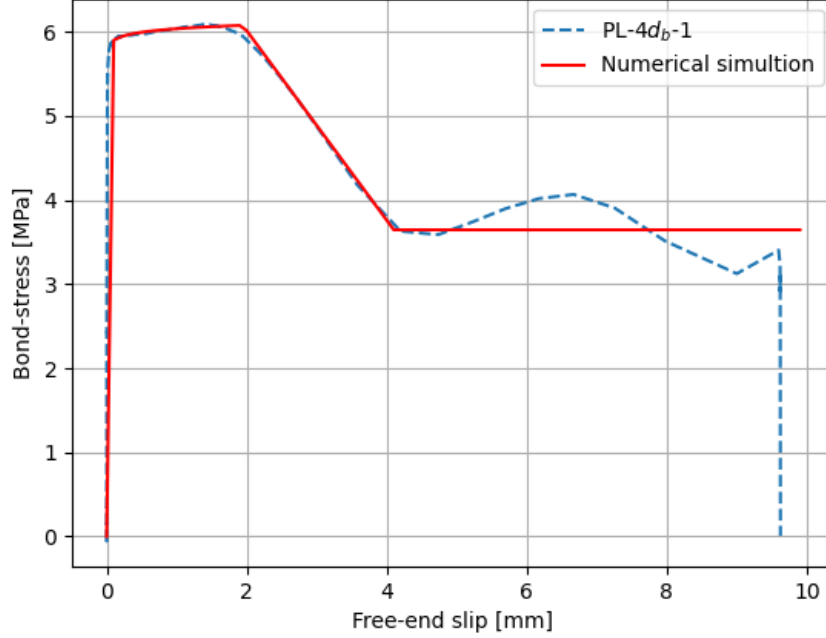


Figure 5.14: Average bond stress-slip behaviour of the peel-ply GFRPs

$$\alpha = 0.01 \quad \tau_{max} = 2.1 f_c^{0.33} \quad \tau_f = 0.6 \tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{8} \quad S_3 = 4\text{mm}$$

Considering the pullout tests conducted on the peel-ply rebars with a nominal diameter of 16mm, it's important to note that we were able to obtain only one valid result out of the four attempts. This limited dataset poses a challenge in estimating an average bond-slip behaviour for this specific case. From the sole successful test, we opted to configure the numerical input parameters in a manner that closely aligns with the curve observed in specimen PL-4d_b-1. As illustrated in Figure 5.14, this approach led to an average bond stress of 6 MPa, a slip S_1 of 2mm, and a residual stress of 3.58 MPa. The specific input values are provided above.

Machining rebar

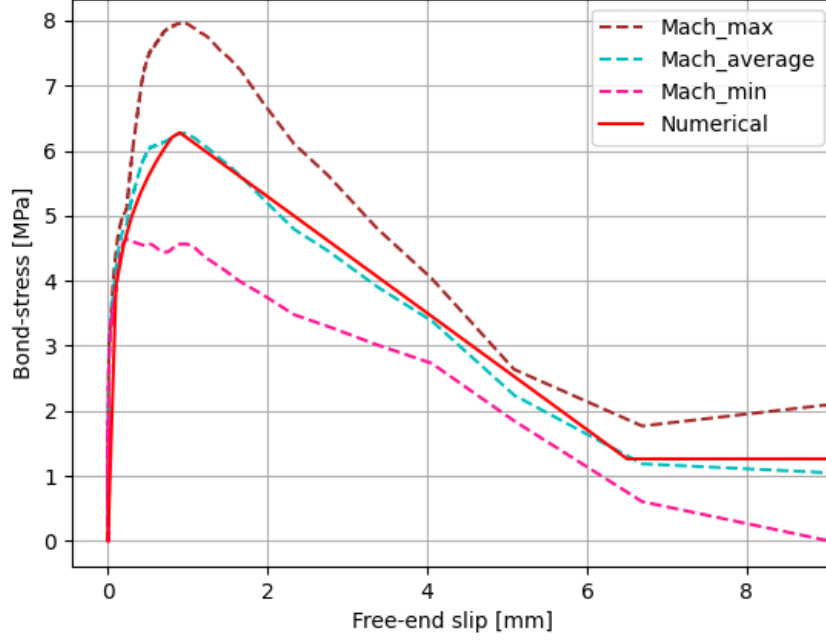


Figure 5.15: Average bond stress-slip behaviour of the machining GFRPs

$$\alpha = 0.22 \quad \tau_{max} = 1.9f_c^{0.33} \quad \tau_f = 0.2\tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{10} \quad S_3 = 6.5\text{mm}$$

As depicted in the figure above, the Machining rebars exhibit excellent performance with an average bond strength exceeding 6 MPa, even with a free-end slip of 0.8mm. As previously noted in the previous section, these glass-fiber reinforced polymers have relatively low residual stress amounting to approximately 20% of the maximum bond stress (τ_{max}). There is a noticeable difference in the size of the gap at the peak compared to the post-peak response, as the distance between the curve narrows.

Sand-coated rebar

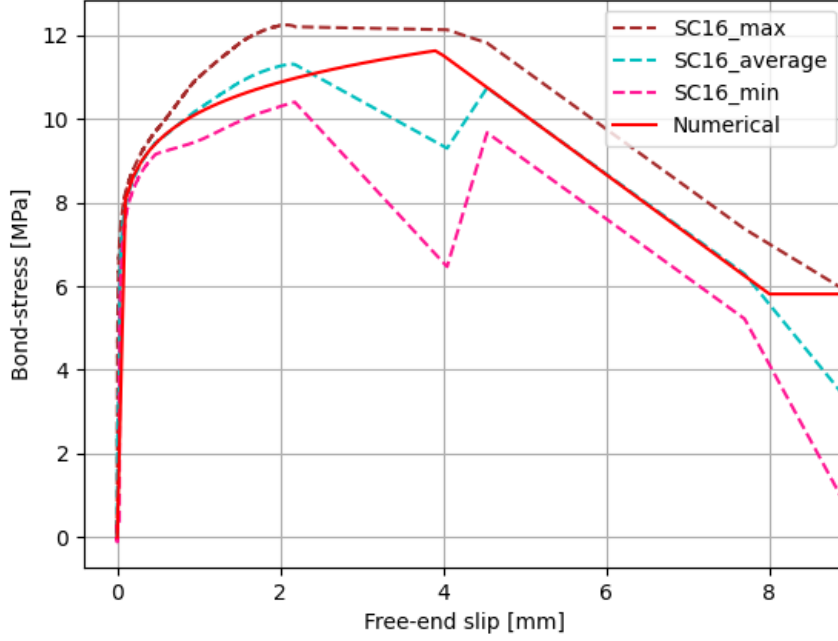


Figure 5.16: Average bond stress-slip behaviour of the sand-coated 16mm GFRPs

$$\alpha = 0.1 \quad \tau_{max} = 3.5f_c^{0.33} \quad \tau_f = 0.5\tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{5} \quad S_3 = 8\text{mm}$$

In Figure 5.17, you can see the results for sand-coated rebars that have a nominal diameter of 16mm. It's important to note that the blue curve, which represents the average behavior, shows an unusual pattern because of specimen SC16-4d_b-1 (you can find more information about this in Section 4.2). Despite this anomaly, the numerical output follows the trend of the average behavior closely and predicts where the curve is headed beyond this deviation. By using the numerical model and input parameters, we were able to calculate an average bond stress of 11.6 MPa, accompanied by a slip of 3.9mm, and an average frictional stress of 5.8 MPa.

Based on a study by Lee et al. in 2017, the bond strength is expected to decrease when there is an increase in the diameter of the rebar. Therefore, it is expected that SC13 specimens would have higher bond strength compared to SC16. We have compiled the following figure that shows the average behaviour of sand-coated rebar with a nominal diameter of 13mm, including its maximum and minimum values based on valid results. The red curve represents the attempt of the numerical simulation to closely replicate the average behaviour.

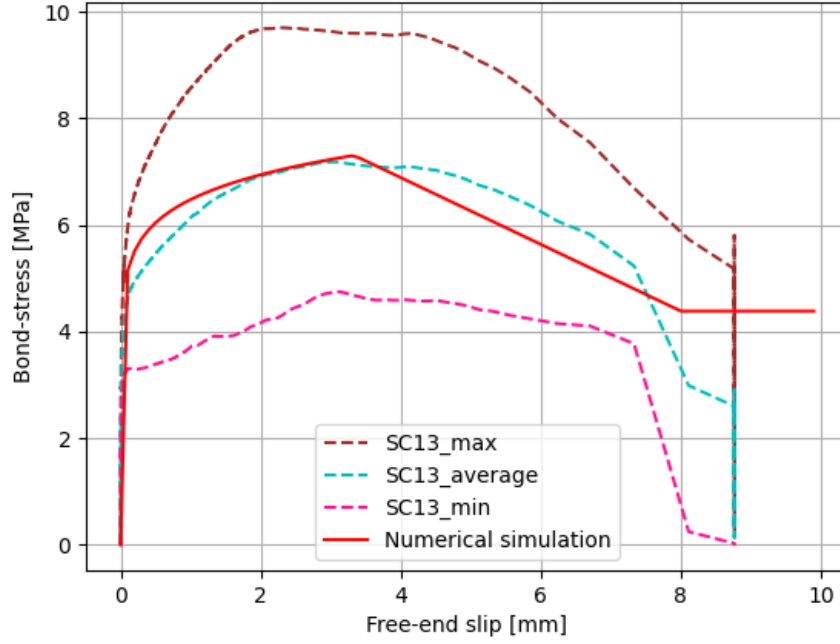


Figure 5.17: Average bond stress-slip behaviour of the sand-coated 13mm GFRPs

$$\alpha = 0.14 \quad \tau_{max} = 2.2f_c^{0.33} \quad \tau_f = 0.55\tau_{max} \quad S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{3} \quad S_3 = 8.5\text{mm}$$

Based on our observations, the bond behaviour of SC13 is lower compared to SC16. This finding contradicts the research conducted by Lee et al. However, it could be due to some of the specimens experiencing unfavourable conditions that affected their overall behaviour. Additionally, the numerical model's parameters may have been slightly adjusted since they did not correspond well with the bond-slip behaviour of SC16.

5.6 Contrasting Empirical Findings with Model Projections

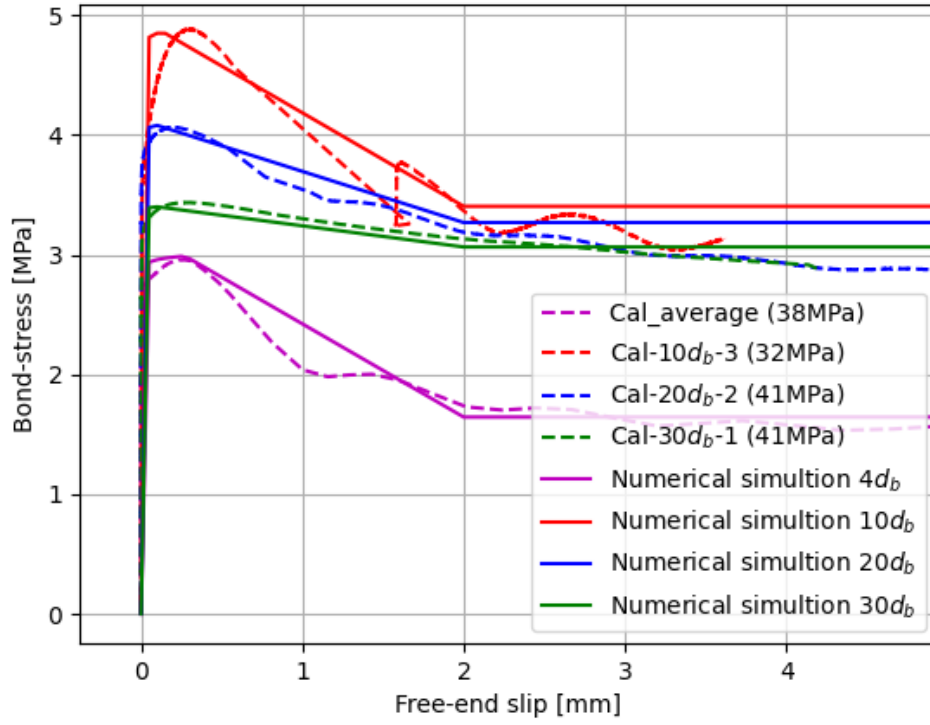


Figure 5.18: Numerical VS experimental results

To facilitate a comparison between the strain evolution along the embedment length for the maximum applied force and the numerical model, it is crucial to determine the experimental slip value at which the peak strain occurs. Given that the bond parameters have been calibrated based on the correlation between average bond stress and the free-end slip (See Fig 5.12), and considering the requirement of an imposed slip (S_0) for the numerical model, we must establish the slip value at which the maximum applied force occurs. In Figure 5.18, the experimental findings are juxtaposed with the numerical predictions, using the identical bond parameters established earlier for Calandering rebars, except for τ_{max} and S_1 . As the extension of the embedment length brought about a surge in bond strength, it necessitated a corresponding adjustment in this parameter. The slip S_1 where the peak occurs was also changed to keep the ratio $\frac{L_e}{15}$ identical.

The ensuing values were determined as follows:

- $\tau_{max} = 0.9f_c^{0.33}$ and $S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{15}$ for $L_e = 4d_b$
- $\tau_{max} = 1.5f_c^{0.33}$ and $S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{10*15}$ for $L_e = 10d_b$
- $\tau_{max} = 1.2f_c^{0.33}$ and $S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{20*15}$ for $L_e = 20d_b$
- $\tau_{max} = 1f_c^{0.33}$ and $S_1 = \frac{\tau_{max}}{f_c} \frac{L_e}{30*15}$ for $L_e = 30d_b$

The following graphs illustrate the best fitting that the numerical model could produce to simulate the evolution of the strain along the embedment length.

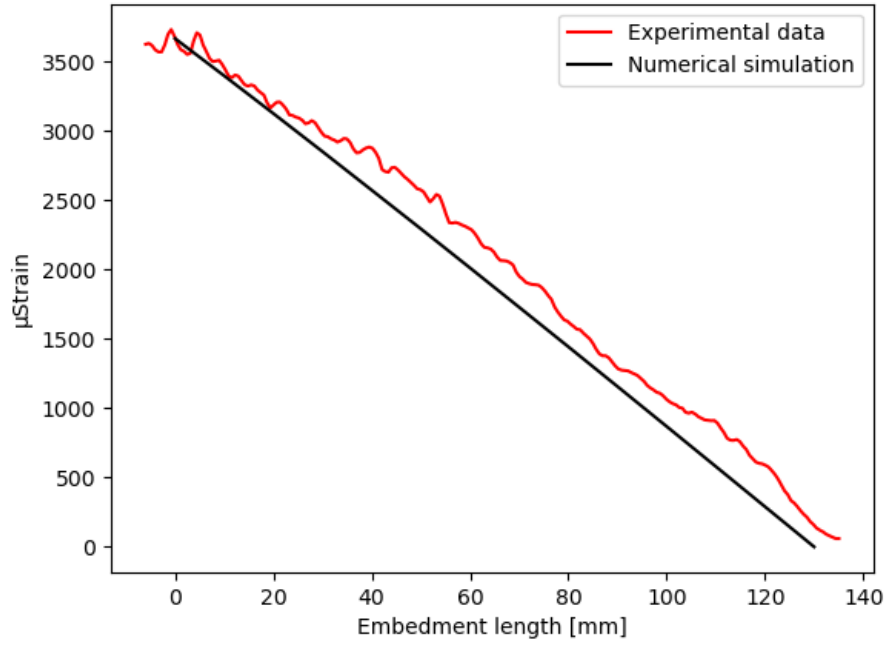


Figure 5.19: Strain evolution along the embedment length ($10d_b$)

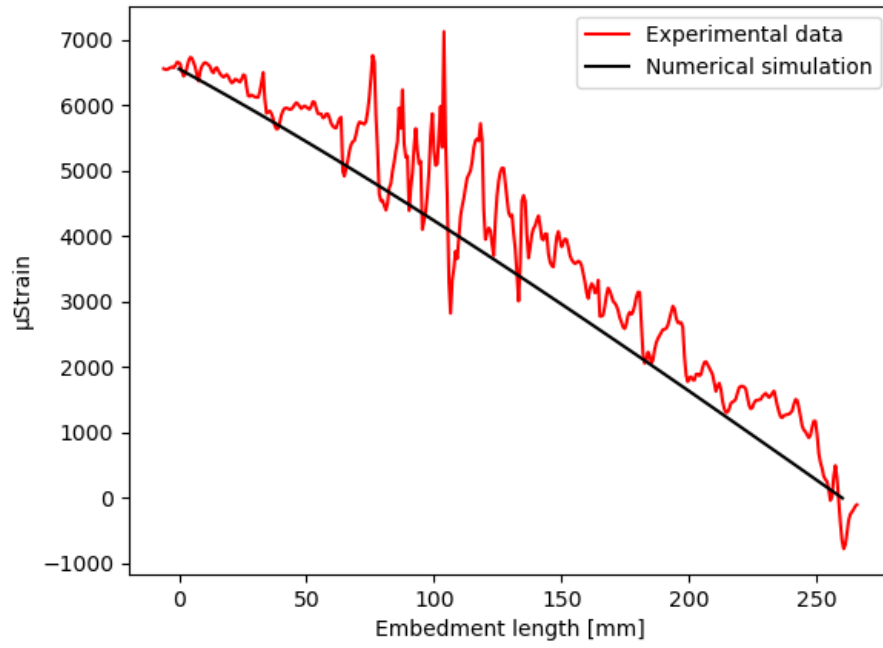


Figure 5.20: Strain evolution along the embedment length ($20d_b$)

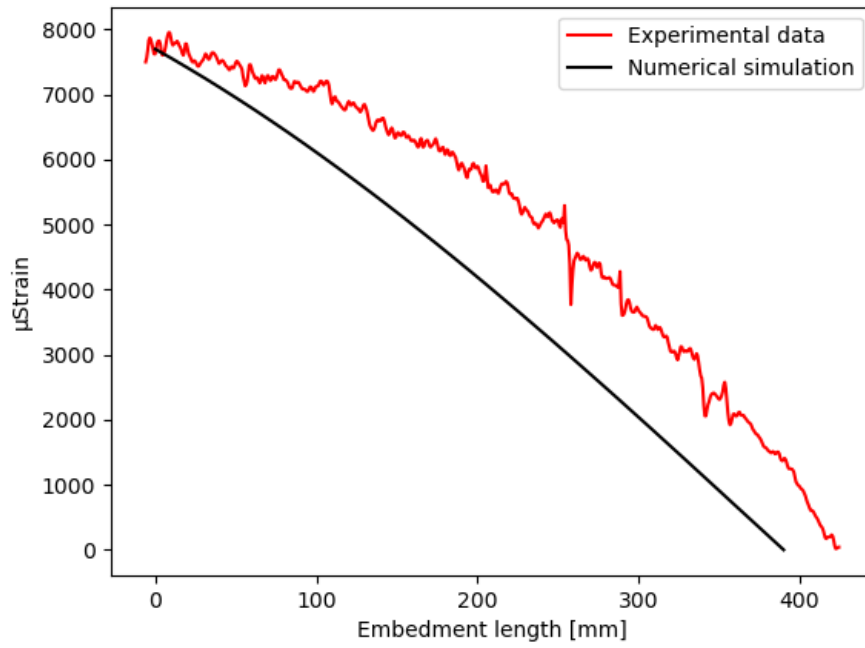


Figure 5.21: Strain evolution along the embedment length ($30d_b$)

It is important to note that the numerical model used input parameters that were not obtained directly from experimental observations. The required slip of the bar (S_0) and the computed maximal applied force were not the same as the actual slip and force values observed during the experiments. However, even with these adjusted inputs, the model was able to accurately approximate the strain distribution along the embedment length. The specific values for S_0 and the maximal applied force, as calculated by the numerical model, are listed below.

- $S_0 = 1\text{mm}$ and $F_{max} = 23.2\text{kN}$ for $L_e = 10d_b$
- $S_0 = 1\text{mm}$ and $F_{max} = 40\text{kN}$ for $L_e = 20d_b$
- $S_0 = 1.7\text{mm}$ and $F_{max} = 47\text{kN}$ for $L_e = 30d_b$

By adjusting the inputs, the model was able to accurately replicate the strain changes for an embedment length of $10d_b$. The strain showed a linear trend and peaked at $3500\mu\text{strain}$. For an embedment length of $20d_b$, the model reached a peak of $7000\mu\text{strain}$, despite some oscillation in the experimental data. The linear trend was still evident. However, for an embedment length of $30d_b$, there was some divergence between the numerical model and the experimental data. The numerical simulation reached a peak of $8000\mu\text{strain}$, but the curvature, which was more pronounced in the experimental curve, could not be aligned with the simulation.

5.7 Summary

	α	τ_{max}	τ_f	S_1	S_3
Calandering	0.01	$0.9f_c^{0.33}$	$0.55\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{15}$	2
PLG	0.25	$1.8f_c^{0.33}$	$0.55\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{7}$	4
PL	0.01	$2.1f_c^{0.33}$	$0.6\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{8}$	4.1
Mach	0.22	$1.9f_c^{0.33}$	$0.2\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{10}$	6.5
SC16	0.1	$3.5f_c^{0.33}$	$0.5\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{5}$	8
SC13	0.14	$2.2f_c^{0.33}$	$0.55\tau_{max}$	$\frac{\tau_{max}}{f_c} \frac{L_e}{3}$	8.5

Table 5.2: Summary of the bond parameters of the numerical model for GFRP rebars

Overall, once the inputs are adjusted, the numerical model can accurately replicate the behaviour of all the tested GFRPs. Based on the established parameters, we have observed the following:

- As shown in the table above, the residual bond stress (τ_f) accounts for approximately 50% to 60% of the bond strength, except for Machining rebars which have significantly lower residual stress of 20%. Additionally, smooth rebars generally exhibit a lower slip S_3 at the point where residual stress begins compared to GFRP with a rough surface.
- The curvature, represented by the parameter α , seems to be lower for smooth rebar than the rough ones, except for Peel-ply/grid which deviates from that observation.
- To accurately simulate the strain evolution along the embedment length, the inputs of the numerical model required modification. However, these modifications did not match the experimental data. The imposed slip (S_0) was greater than the actual slip (S_1) at which the peak occurred. Additionally, the maximal applied force calculated by the model was lower than what was observed in the experiments.
- When the inputs are adjusted, the numerical model's prediction for various embedment lengths is quite accurate, but there is room for improvement. Although the simulations can match the peak strain, they tend to diverge from the experimental data as the embedment length increases.
- The parameters established for the Peel-ply rebars are not representative, as they match only the behaviour of one specimen.

Chapter 6

Conclusion and future developments

This chapter will be divided into two parts. The first section will summarise all the relevant conclusions that we were able to draw from the experiments. Then, to fulfill the objectives of the thesis, we will see how we were able to validate the numerical model with the help of our experiments. The second section will be devoted to future development. We will discuss what could be put in place to extend the research.

6.1 Conclusion

From the pullout results, different conclusions were made on the behaviour of GFRP rebars depending on their surface finishing and diameter. Some conclusions could also be made depending on the embedment length. The last step was to adjust the bond parameter of the numerical model based on the experimental results and validate it. The conclusions will be listed here under.

- The pullout tests conducted on cubic samples showed that surface finishing has a significant impact on the bond behavior of GFRP rebars. Specifically, rough surfaces like Sand-Coated or Machining surfaces perform better than their smoother counterparts, showing higher bond strengths. These rough surfaces also have larger slip values at which the frictional bond stress initiates (S_3), which occurs at around 8mm for SC rebars and 6mm for Mach rebars. In contrast, smoother surfaces like Calendering and PeeL-ply/Grid have a lower transition point, occurring at about 2mm and 4mm, respectively.
- Machining rebars exhibit the lowest residual stress (τ_f), measuring at 20% of the bond strength (τ_{max}), in contrast to the other rebars which typically fall within the range of 50% to 60%.
- The study shows that when the embedment length is longer, the bond strength for Calendering GFRPs decreases. This could be due to an increase in defects and uneven stress distribution along the bar with longer embedment lengths.

Furthermore, Calandering rebars experience a rise in residual bond stress as the embedment length increases. For instance, at an embedment length of $4d_b$, the residual stress is roughly 55% of τ_{max} , but it increases to 70% at $10d_b$, 80% at $20d_b$, and ultimately 90% at $30d_b$.

- Despite calibrating the bond parameters of the numerical model to align with each specific type of GFRP, the model encounters challenges when accurately predicting the strain evolution for certain embedment lengths. However, by appropriately readjusting the input parameters, the model can provide a relatively close approximation of the strain distribution along the embedment length.

6.2 Future developments

Based on the results of that thesis, we observed that GFRP rebars have great potential but need more experimental data before we can actually use them in reinforced concrete structures. Here-under, a list with all the possible developments that could extend the work done on such rebars will be enumerated:

- Additional experimental tests could be conducted on other types of glass fiber-reinforced polymers to gather more data for the calibration of the bond-slip numerical model.
- Further experiments could be performed using specimens equipped with Distributed Optical Fiber Sensors. Since we only had one specimen for each configuration, the extra data acquired from these experiments could enhance the numerical model because it was observed that the model encountered challenges in replicating strain evolution with longer embedment lengths. If we truly desire more data, we should consider using different GFRPs instead of the Calandering bars that were used in previous tests.
- Additional experiments could be conducted on the same GFRPs, such as simple tensile tests, to establish the stress-strain constitutive relationship. This would enable the calculation of Young's Modulus of the bar. With this information, the evolution of stresses along the embedment length could be plotted using the relation $\sigma = E\epsilon$, along with the corresponding tensile force.

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ChatGPT was used for this master thesis to rephrase some parts in order to have a more scientific vocabulary. It was also used to write the section 'Research Novelities' in Chapter 1.

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