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**PERFORMANCE OF FRP BARS IN ULTRA-
HIGH STRENGTH SEA-SAND CONCRETE IN
COASTAL ENVIRONMENTS UNDER NATURAL
AND ACCELERATED AGING CONDITIONS**

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The Hong Kong Polytechnic University
Department of Civil and Environmental Engineering

**Performance of FRP bars in ultra-high strength
sea-sand concrete in coastal environments under
natural and accelerated aging conditions**

CUI Chuying

A thesis submitted in partial fulfilment of the requirements for the
degree of Doctor of Philosophy

August 2025

CERTIFICATE OF ORIGINALITY

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ABSTRACT

To address the challenge of the shortage of river sand, non-desalted sea-sand has emerged as a promising alternative aggregate in ultra-high strength concrete (UHSC), leading to the development of sustainable sea-sand UHSC (UHSSC). However, the use of steel reinforcement in sea-sand concrete is limited by severe corrosion risks. High-strength, corrosion-resistant fiber-reinforced polymer (FRP) offers a viable reinforcement solution for UHSSC, particularly in coastal and marine construction. To ensure the effective and safe use of FRP-reinforced UHSSC structural members, this thesis presents a comprehensive investigation into the short-term and durability performance of FRP bars in UHSSC under coastal environmental conditions.

The thesis is structured into two main parts. The first part presents an experimental investigation into the short-term bond behavior between FRP bars and UHSSC, followed by the development of a widely applicable bond strength model for FRP bars in concrete. The second part focuses on the durability of FRP bars in UHSSC under both accelerated aging and natural coastal exposure conditions. Furthermore, a bond degradation model was developed, and long-term predictions of the tensile performance of FRP bars embedded in concrete were conducted.

The first part begins with a systematic experimental study on the bond performance of FRP bars in UHSSC. Direct pullout tests were performed on 107 pullout specimens, covering four concrete strength grades and five types of FRP bars from three manufacturers. In addition to concrete strength, bar diameter, and fiber type, this study examined the effects of horizontal shear properties and surface topography of FRP bars on their bond performance in UHSSC, factors that have not been quantitatively evaluated in previous research. To this end, two series of companion tests were performed: topographical measurements of bar surface and short-beam shear tests to assess the physical surface geometries and shear properties of FRP bars. The experimental results reveal that the surface topographical regularity, fiber type, and horizontal shear performance of FRP bars are the three key factors influencing their bond performance in UHSSC, which are not adequately addressed in current design guidelines. In general, in UHSC, carbon FRP (CFRP) bars exhibit substantially higher bond strength than

glass FRP (GFRP) bars produced using the same fabrication technology, and the bond strength increases with the horizontal shear strength of FRP bars. Surface imperfections, however, are detrimental to the bond strength. It is therefore recommended to incorporate a fiber factor, a surficial reduction factor, and an upper limit based on the horizontal shear strength of FRP bars when predicting their bond strength with UHSC. Based on these findings, design recommendations for the bond of FRP bars in UHSC were provided, laying the foundation for the reliable bond strength model developed in the subsequent part of this thesis.

Following the experimental study, this thesis then presents the development of a new bond strength model for commercially available FRP bars through a large test database assembled by the author. The database comprises 687 pullout test results, sourced from 29 published papers as well as the test data generated in this PhD study. Statistical analyses, including analysis of variance (ANOVA) and corresponding post hoc tests, were performed on the assembled database. The results highlight the significant effects of bar type, bar diameter, concrete strength, and fiber type, as well as their two-way interactions. Existing bond strength models were first evaluated against the assembled database. Subsequently, a new model was developed based on the general size effect law, incorporating the effects of these four key parameters and their statistically significant coupling effects. The proposed bond strength model demonstrates broader applicability than existing models, as it is valid for a wide range of concrete grades, from normal concrete to UHSC. Importantly, the new bond strength model accurately captures both the size effect and the two-way interaction effects of key parameters, which are overlooked in existing models. Comparative assessments confirm that the proposed model offers superior accuracy in predicting bond strength.

The second part of this thesis presents a comprehensive investigation into the durability performance of FRP bars embedded in UHSSC in coastal environments, using both accelerated aging and field exposure tests over a period of around one year. In the accelerated aging study, two batches of UHSSC mixtures and four types of FRP bars were tested. In total, 554 specimens, including 117 for pullout tests, 199 for tensile tests, and 238 for short-beam shear tests, were immersed in artificial seawater at room temperature (about 23°C), 40°C, and 60°C. The results show that the saline-alkalinity environment within UHSSC does not adversely affect the tensile and shear properties of FRP bars and may even slow their degradation. Furthermore, the FRP bar-UHSSC system demonstrates reliable bond durability under seawater immersion, with no significant changes observed in failure mode or key bond

characteristics, such as bond strength, bond stiffness, and relative post-peak strength. However, in UHSSC with a low compressive strength of around 120 MPa, FRP bars exhibited an increase of around 10% in bond strength but a notable 30% decrease in bond stiffness. This is attributed to the increased compressive strength of the UHSSC mixture and a subsequent change in failure mode from debonding at the bar-concrete interface to shear failure within the FRP bars after one year of seawater immersion.

In the field exposure tests, four types of FRP bars were conditioned at a wharf in Zhanjiang, China, across four distinct coastal environments: atmospheric, splash, tidal, and underwater zones. Comparative tests, including 148 specimens, were performed on both bare FRP bars and bars embedded in UHSSC to evaluate their tensile, horizontal shear, and bond properties before and after exposure. The results show that the tidal and underwater zones were identified as the most critical conditions, causing significant degradation in the tensile strength and horizontal shear strength of bare FRP bars. In contrast, the FRP bars embedded in UHSSC exhibited minimal degradation in tensile, shear, and bond performance after one year of natural coastal exposure.

Finally, based on these experimental observations, a new bond degradation model was developed that accounts for the changes in the material properties of both the FRP bars and the surrounding concrete. The model demonstrates reasonable agreement with experimental results. In addition, this thesis synthesized durability data from 430 accelerated aging tests on FRP bars. Based on this comprehensive database, long-term predictions were carried out using the Arrhenius relation, considering the effects of bar type, concrete wrap, alkalinity level, relative humidity (RH), and mean annual temperature (MAT) of exposure. The field durability of internal FRP reinforcement was further estimated based on short-term accelerated aging test results reported in the literature.

In summary, the investigation into the short-term and bond durability performance of FRP bars establishes a reliable foundation for ensuring adequate bonding of FRP bars in concrete, particularly in UHSSC, thereby enabling effective control of cracking and deflection in FRP bar-reinforced UHSSC members throughout their service life. Furthermore, durability studies and long-term predictions of the tensile properties of FRP bars help quantify the effect of environmental exposure on FRP-reinforced UHSSC structures.

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LIST OF ACRONYMS

3D	Three-dimensional
ANOVA	Analysis of variance
At	Atmospheric
BB	Bare bar
C1	CFRP bars from P1
C2	CFRP bars from P2
CAC	Coral aggregate concrete
CC	Conventional concrete
CFRP	Carbon fiber reinforced polymer
CoV	Coefficient of variation
DSC	Differential scanning calorimetry
EB	Embedded bar
EDX	Energy dispersive x-ray
ERF	Environmental reduction factor
FRC	Fiber-reinforced concrete
FRP	Fiber reinforced polymer
FTIR	Fourier transform infrared spectroscopy
G1	GFRP bars from P1
G2	GFRP bars from P2
G3	GFRP bars from P3
GDP	Gross domestic product
GFRP	Glass fiber reinforced polymer
GPC	Geopolymer cement concrete
HSC	Higher strength concrete
HW	Helically wrapped
HWSC	Helically wrapped with sand coating
IC	Ion chromatography
ILSS	Interlaminar shear strength
In	Indented
LAC	Low-alkalinity concrete

LC	Limestone calcined clay cement concrete
M	Mean
MAPE	Mean absolute percent error
MAT	Mean annual temperature
NC	Normal concrete
NSC	Normal strength concrete
P1	Producer 1 (Dextra Building Products Co., Ltd, Guangzhou, China)
P2	Producer 2 (Pulwell Composites Co., Ltd, Zhongshan, China)
P3	Producer 3 (Pultrall Inc., Thetford Mines, Canada)
PO	Pullout
RAC	Recycled aggregated concrete
Rb	Ribbed
RC	Reinforced concrete
RH	Relative humidity
RMSE	Root mean squared error
SC	Sand-coated
SD	Standard deviation
SEM	Scanning electron microscopy
Sp	Splash
SPS	Simulated pore solution
SW/SSC	Seawater and/or sea-sand concrete
SWC	Seawater concrete
Ti	Tidal
TS	Tensile
TSF	Time-shift factor
UHSC	Ultra-high strength concrete
UHSSC	Ultra-high strength sea-sand concrete
Un	Underwater

NOTATION

A	Constant related to the material and degradation process
A_f	Nominal cross-sectional area of FRP bars calculated according to ASTM D7205/D7205M
$(\alpha\beta)_{ij}$	Non-additive interaction effects of factor α and β
$(\alpha\beta\gamma)_{ijk}$	Non-additive 3-way interaction effect from three factors for samples h ($h = 1, \dots, n_{ijk}$)
$(\alpha\gamma)_{ik}$	Non-additive interaction effects of factor α and γ
$(\beta\gamma)_{jk}$	Non-additive interaction effects of factor β and γ
$\bar{\tau}_u$	Bond strength normalized by concrete strength
Δ_1	Projected tensile strength reduction of design life at service temperature
R_{10}	Standard reduction of tensile strength in percent per decade in fib (2007)
Y_∞	Residual strength retention at exposure time of infinity
Y_s	Horizontal shear strength retention
Y_τ	Bond strength retention
d_{cs}	Smaller value of the 2/3 of center-to-center spacing of the bar or the space between closest concrete surface and bar center
d_o	Outer diameter of FRP bars
f'_c	Cylindrical compressive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{cr}	Cracking strength of concrete
f_{cu}	Cubic compressive strength of concrete
f_{fkd}	Long-term design value of tensile strength of FRP bars
f_{tsp}	Splitting tensile strength
k_f	Fiber coefficient accounting for the contribution of fibers in concrete
n_{SL}	Desired service life factor in fib (2007)
n_T	Exposed temperature factor in fib (2007)
n_d	Diameter correction factor in fib (2007)
n_{mo}	Moisture condition factor in fib (2007)
r_0	Initial radius of the bar cross-section
y_{hijk}	Observed value in 3-way ANOVA linear function
α_f	Fiber factor of proposed bond strength model
α_i	Additive main effects of level i from factor α
β_j	Additive main effects of level j from factor β
γ_f	Partial safety factor for FRP in fib (2007)
γ_k	Additive main effects of level k from factor γ
ε_{hijk}	Error in 3-way ANOVA linear function

ρ_v	Stirrup ratio
a	Regression constant
b	Regression constant
B	Regression coefficient in general size effect law
C	Cover thickness
C	Concentration of solution
C_E	Environmental reduction factor
d	Diameter of FRP bars
D	Diffusion coefficient
d_0	Transitional size
d_{eff}	Effective diameter of FRP bars
df	Degree of freedom
E_a	Activation energy of the chemical reaction
E_f	Tensile modulus of FRP bars
E_s	Tensile modulus of deformed steel bars
F	Pullout load
f	Strength parameter in general size effect law
f_F	Design stress in FRP tension reinforcement at ultimate limit state
$f_{fk1,000h}$	Characteristic value of tensile strength after 1000 h exposure test of FRP bars
h_r	Rib height
k	Degradation rate
K_1	Location factor of the FRP bar in CSA S806-12
k_1	Bar location factor in CSA S6-19
K_2	Density factor of concrete in CSA S806-12
K_3	Size factor of the FRP bar in CSA S806-12
K_4	Fiber factor of the FRP bar in CSA S806-12
k_4	Bar surface profile factor in CSA S6-19
K_5	Surface treatment factor of the FRP bar in CSA S806-12
K_{tr}	Transverse reinforcement index
L_e	Embedment length of FRP bars
l_r	Evaluation length
n	Sum of influencing terms in fib (2007)
P	Shear load
R	Nominal radius of FRP bars
R	Universal gas constant
R_a	Surface roughness
S	Interlaminar shear strength
S	Coefficient for the fiber and surface type in Basaran and Kalkan (2021)
s_1	Slip corresponding to the bond strength

s_r	Rib spacing
T	Temperature in Kelvin
w_c	Width of the concrete lug
w_r	Rib width
Y	Strength retention (%)
Y_{BB}	Tensile strength retentions of bare bars
Y_{EB}	Tensile strength retentions of embedded bars
$Z(x)$	Height of the surface profile deviating from the mean line
β_d	Relative size
β_H	RH correction factor
λ	Time correction component
τ	fitted parameter related to the degradation rate
τ_m	Adhesion strength between the resin layer and the core fiber bar
τ_s	Friction strength between smooth FRP bars and UHSC
τ_u	Bond strength
α	Regression constant related to surface type
α_1	Confinement factor in JSCE 2007
α_2	Modification factor for bond strength in JSCE 2007
β	Regression constant related to surface type
θ	Regression coefficient of proposed bond degradation model
μ	Grand mean in 3-way ANOVA linear function
ϖ	Empirical constant related to bar type

CHAPTER 1. INTRODUCTION

1.1. Research background

A recent survey reported that global cement production has reached approximately 4 billion tonnes in recent years (see Figure 1.1), highlighting the substantial consumption of concrete and, consequently, river sand as a main raw material in construction (U.S. Geological Survey 2022). To alleviate the heavy burden on river sand, non-desalted sea-sand has been considered as an alternative (Xiao *et al.* 2017). Guo *et al.* (2020) noted that the current annual production of marine aggregate was 20 million tonnes, accounting for 20% and 48% aggregates used in England and Welsh, respectively, while in Japan, marine aggregate has been taken as a major source of construction materials (Katano *et al.* 2013).

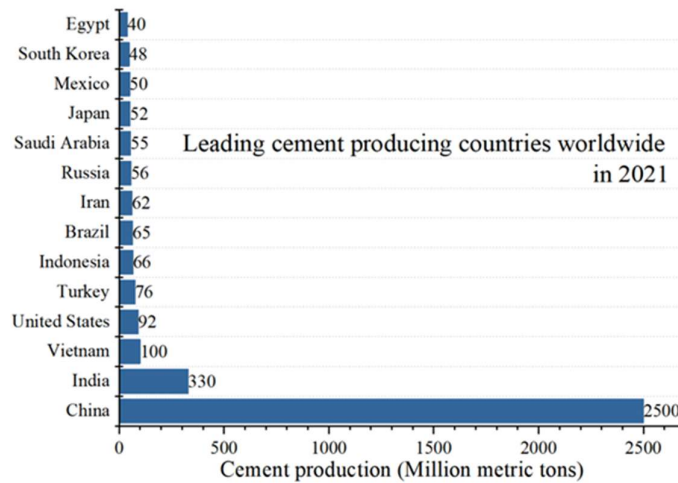


Figure 1.1. Annual output for worldwide cement production

Regarding the application of sea sand concrete, however, the corrosion issue is the main sticking point, in that the chloride ions in sea sand can induce corrosive damage on steel reinforcement (Broomfield 2003; Angst 2018). Currently, steel corrosion in concrete has been a critical issue that limits the service life of reinforced concrete (RC) structures and results in enormous economic losses. As shown in Figure 1.2, different reviews indicated that corrosion is the predominant factor in 70-90% of the evaluated RC structures subjected to premature deterioration (Schiebl and Mayer 2007; fib 2011). According to a comprehensive survey in China, the overall cost attributable to corrosion was 3.34% of the gross domestic product

(GDP) in 2014 (Hou *et al.* 2017). Notably, the corrosion of infrastructure accounted for around half of the direct costs caused by metallic corrosion (Koch *et al.* 2002). As a result, the corrosion-resistant FRP reinforcement has shown promise as a great alternative to steel (see Figure 1.3), particularly in coastal and marine environments (Xiao *et al.* 2017; Ahmed *et al.* 2020).

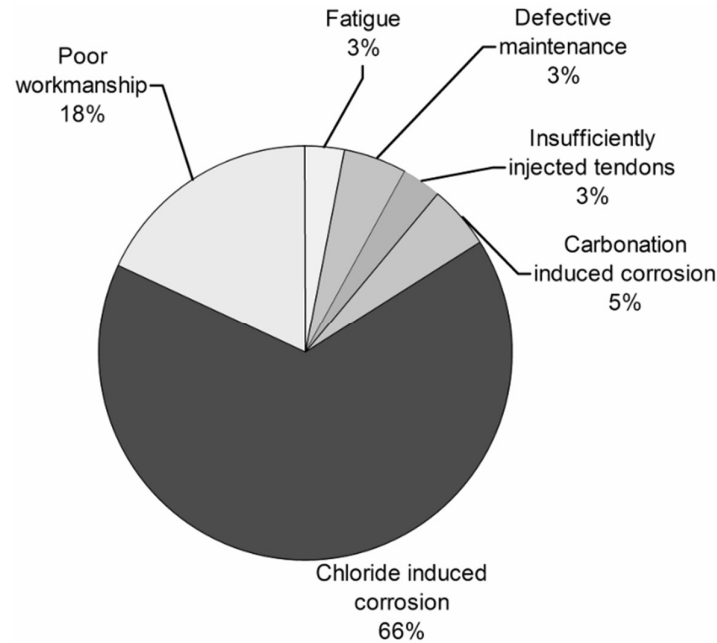


Figure 1.2. Causes of damage to bridge structures of the German motorway network (Schiebl and Mayer 2007)

For constructions in aggressive environments, UHSC, characterized by relatively low permeability and superior mechanical properties, offers an attractive solution. The enhanced durability and high strength-to-weight ratio of UHSC enable extended service life and reduced cross-sections of components compared to conventional concrete (Yoo and Yoon 2016). Since the late 20th century, UHSC has been used in various structural components of infrastructure construction across Canada, Germany, Japan, South Korea, and other regions, demonstrating its global adoption (Farhat *et al.* 2007; Graybeal 2011; Russell and Graybeal 2013; Wang *et al.* 2015; Yoo and Yoon 2016; Abbas *et al.* 2016). Recently, in response to the scarcity of freshwater and river sand, UHSC mixed with sea-sand and/or seawater has been developed (Teng *et al.* 2019; Liao *et al.* 2021; Saleh *et al.* 2022; Zeng *et al.* 2022). This sustainable material, when combined with corrosion-resistant FRP reinforcements, significantly enhances the mechanical and durability performance of structural members in chloride-rich environments. Consequently, combining sea-sand-mixed UHSSC with FRP

reinforcement emerges as an efficient and sustainable solution for coastal and marine construction.



Figure 1.3. Demonstration project of GFRP reinforced seawater sea sand concrete structures (Xiao *et al.* 2017)

1.2. Research significance and objectives

For a confident use of FRP bars reinforced UHSSC structural members, a comprehensive study of both their short-term and durability performance is essential. In terms of short-term behavior, the bond strength of FRP bars embedded in UHSSC needs to be experimentally determined in the absence of a commonly accepted bond strength model. Existing design guidelines for the bond of FRP bars in conventional concrete (ACI 2022; CSA 2012, 2019; JSCE 1997) tend to provide conservative estimates for bars in UHSC (Ametrano 2011), thereby limiting the full utilization of the superior mechanical properties of both FRP bars and UHSSC. Furthermore, although FRP bar-UHSSC structures hold significant potential for use in aggressive environments, their safe application faces challenges due to their susceptibility to aggressive environmental attacks in coastal conditions. The degradation of

FRP bars in UHSSC directly affects the durability of these structures. However, to date, no research has systematically addressed the durability performance of FRP bars embedded in UHSSC.

This thesis aims to investigate the fundamental issues related to the short-term and durability performance of FRP bars in UHSSC substrate. The main objectives of this thesis are given as follows:

- (a) to elucidate the short-term bond behavior of FRP bars embedded in UHSSC and to identify the critical key factors influencing the bond performance;
- (b) to develop a predictive bond strength model that can accurately capture the effects of these critical factors;
- (c) to evaluate the durability of tensile, shear, and bond properties of FRP bars in UHSSC in coastal environments;
- (d) to make reliable predictions of bond strength degradation and tensile strength durability of FRP bars.

1.3. Layout of the thesis

Based on the aforementioned research objectives, the following chapters start with an extensive literature review on relevant topics, including bond performance of FRP bars, existing predictive models and design equations, durability performance of FRP bars, and design and prediction methods concerning environmental attacks on FRP bars. all of which are instrumental in identifying research needs. The short-term bond tests are given in the follow-up chapter, emphasizing the effects of critical parameters on the bond performance of FRP bars in UHSSC. A bond tests database, compiled from both test results from the previous chapter and an intensive review of available literature, is then presented. The thesis proceeds with the statistical analysis of test data and the development of a new bond strength model. Finally, the long-term performance of FRP bars embedded in UHSSC in coastal environments was evaluated through accelerated aging and field tests. Long-term predictions of residual bond strength and tensile strength are also discussed. The thesis structure, as outlined in Figure 1.4, is organized as follows:

Chapter 2 presents a review of the available literature concerning the bond behavior of FRP

bars in concrete, including design equations and models proposed in existing studies and design guidelines. This chapter also assesses the durability of FRP bars in concrete under accelerated aging and field conditions, followed by a review of prediction methods of the long-term performance of FRP bars in concrete environments. Against this review, several research gaps are identified.

Chapter 3 presents a comprehensive investigation of the bond performance of FRP bar-UHSSC structural members. A series of direct pullout tests were performed on various types of FRP bars embedded in UHSSC and HSSC. This study highlights the influence of the mechanical properties (i.e., the horizontal shear performance) and surface topography (i.e., surface type and relevant physical variation) of the FRP bars on bond performance. A total of 107 pullout specimens, featuring diverse concrete strengths (denoted as S60, S120, S140, and S170), bar diameters ($\Phi 6$, $\Phi 10$, $\Phi 13$, and $\Phi 16$), and FRP bars differing in fiber type (glass fiber and carbon fiber), surface type (helically wrapped with sand coating and sand-coated), and resin type (epoxy and vinyl ester) were tested.

Chapter 4 focuses on predicting the bond strength of FRP bars in concrete. A large database was established, including 678 pullout test results of three types of commercially available FRP bars: helically wrapped with sand coating (HWSC), sand-coated (SC), and indented (In) FRP bars. Through extensive literature review and statistical analysis, the influences of four key factors (i.e., bar type, bar diameter, concrete strength, and fiber type) and their significant coupled effects were identified. Considering the effects of these factors and their interaction effects, a new model was proposed based on the general size effect law.

Chapter 5 investigates the durability of FRP bars in UHSSC subjected to a simulated marine environment. Specimens were conditioned under artificial seawater immersion at both room temperature and elevated temperatures (40 °C and 60 °C) for up to 13 months. Comparative tests were conducted on UHSSC-wrapped and bare FRP bars to evaluate the tensile, horizontal shear, and bond properties before and after exposure.

In Chapter 6, a systematic investigation of the long-term behavior of FRP bars embedded in UHSSC under field coastal environments is presented. The FRP bar-UHSSC specimens were conditioned in four different field environments at a wharf in Zhanjiang, China, over one year, including the atmospheric, splash, tidal, and underwater zones. Four types of GFRP and

CFRP bars were tested to evaluate changes in macroscopic mechanical properties.

Chapter 7 presents the long-term predictions on the bond durability and tensile strength of FRP bars in concrete structures exposed to natural environments. Based on these experimental observations, a new bond degradation model was first developed that accounts for the changes in the material properties of both the FRP bars and the surrounding concrete. In addition, this chapter synthesized durability data from 430 accelerated aging tests on FRP bars. Based on this comprehensive database, long-term predictions on tensile strength of internal FRP reinforcement were carried out using the Arrhenius relation, considering the effects of bar type, concrete wrap, alkalinity level, relative humidity (RH), and mean annual temperature (MAT) of exposure. The field durability of internal FRP reinforcement was further estimated based on short-term accelerated aging test results reported in the literature.

Finally, the thesis closes with Chapter 8 by summarizing the main conclusions from previous chapters and discussing the research needs for the future.

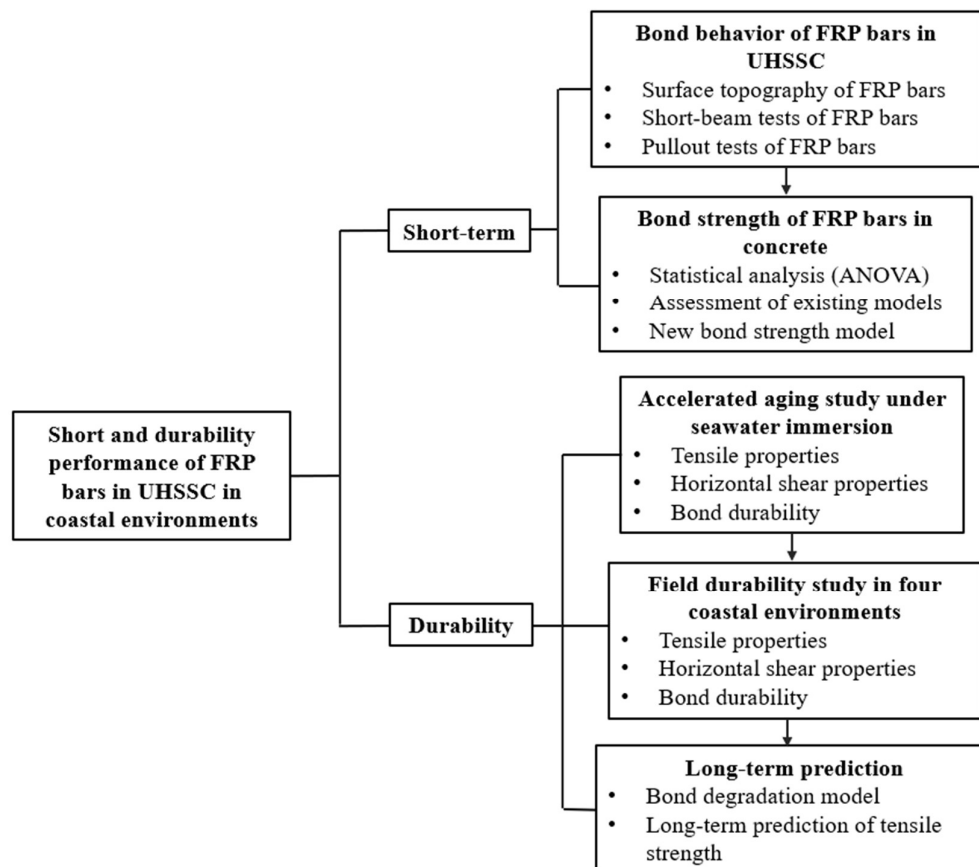


Figure 1.4. Thesis outline

CHAPTER 2. LITERATURE REVIEW

2.1. Introduction

In this chapter, an extensive literature review is presented concerning the short-term and long-term performance of FRP bars in concrete. It commences by reviewing the short-term bond performance of FRP bars, with the focus on the bond mechanism, critical parameters influencing the bond strength, and prediction models. The durability of FRP bars in concrete under accelerated aging conditions and field conditions was subsequently reviewed, including the long-term predictions of the field durability of FRP bars used as internal reinforcement in concrete structures. Based on the state-of-the-art review in this chapter, the research needs can be identified.

2.2. Short-term bond performance of FRP bars in concrete

In flexural RC members, the bond between reinforcements and the surrounding concrete is crucial for controlling the tension force transfer to the reinforcement, which balances the compression force in concrete (ACI 2022). At the serviceability limit state, the degree of bond affects spacing width of cracks, tension stiffening, and curvature. Therefore, for a safe and reliable design, sufficient bond should be mobilized in RC structures. The bond of FRP reinforcement in concrete is exploited through mechanisms similar to that of steel reinforcement but is generally considered more complex because of the distinct manufacturing processes and properties of FRP compared to steel (Cosenza *et al.* 1997; Tighiouart *et al.* 1998).

2.2.1. Bond mechanisms

Over the past several decades, the bond properties of FRP bars have been extensively studied (Nepomuceno *et al.* 2021). Researchers have identified that the development of bond of both steel and FRP bars in concrete originates from chemical adhesion, frictional resistance, and mechanical interlocking (Tepfers 1979; Tighiouart *et al.* 1998). However, the contributions of these mechanisms differ between the two types of reinforcement. It is widely accepted that the bond between FRP bars and concrete primarily relies on chemical adhesion and friction,

while mechanical interlocking contributes less to bond resistance in FRP bars than in steel bars due to the lower shear strength of surface deformations of FRP bars (*fib* 2007; Kim and Wang 2022).

Moreover, studies have observed that these mechanisms make various contributions to the bond performance of FRP bars with different surface configurations (Cosenza *et al.* 1997; Baena *et al.* 2009; Harajli and Abouniaj 2010; Soong *et al.* 2011; Saleh *et al.* 2019; Xiong *et al.* 2021; Fahmy *et al.* 2021). Generally, FRP bars with sand coatings show higher adhesion bond and initial bond stiffness than other surface configurations due to the dominant chemical resistance and static friction. For FRP bars finished with surface deformations, especially those with apparent ribs, mechanical interlocking has a more pronounced effect. Researchers have further noted that the contributions of the three bond mechanisms can vary significantly within the same surface category (e.g., sand-coated surfaces with different sand fineness and coating density, and ribbed surfaces with different rib geometries) (Baena *et al.* 2009; Soong *et al.* 2011; Basaran and Kalkan 2020a; Fahmy *et al.* 2021). However, due to the lack of standardization in the surface treatment of FRP bars and the resulting variability in surface conditions, the quantitative correlation between the surface configuration of FRP bars and the contributions from these mechanisms remains inconclusive. This underscores the necessity for a comprehensive study of the bond performance of FRP bars.

2.2.2. Failure modes

For normal strength concrete (NSC, for concrete grade $\leq$ C50 as stipulated in *fib* (2013)) with low compressive strength, FRP bars fail in a pattern similar to deformed steel bars (Achillides and Pilakoutas 2004; Lee *et al.* 2008; Tekle *et al.* 2016; Soares *et al.* 2020; Peng *et al.* 2022). Bond failure typically occurs within the concrete matrix, where the concrete substrate crushes near the FRP bar-concrete interface, leaving no apparent damage on the FRP bar surface, as shown in Figure 2.1. In this case, the bond strength is primarily governed by the concrete strength.

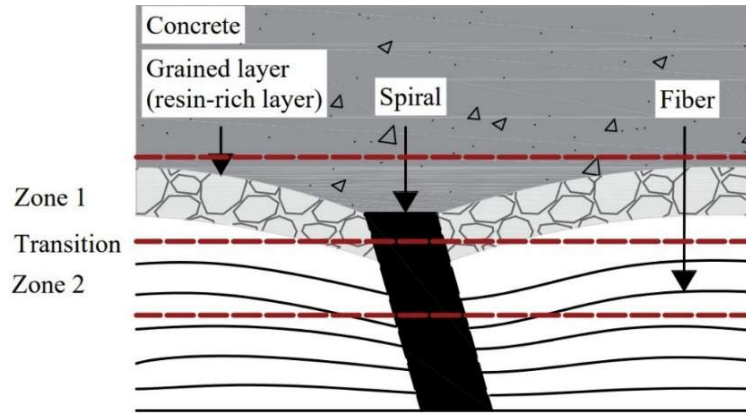


Figure 2.1. Schematic of debonding surfaces

In contrast, for high strength concrete (HSC, for concrete grade $> C50$ but $\leq C120$), the FRP bar-concrete bonded joints tend to fail by peeling off the surface layer of the FRP bars, in addition to the crushing of the surrounding concrete observed in NSC. Previous experimental studies revealed that bond failure occurs partially within the concrete matrix and partially within the surface layer of the FRP bars, as depicted in Zone 1 of Figure 2.1 (Achillides and Pilakoutas 2004; Lee *et al.* 2008; Solyom and Balázs 2020; Chen *et al.* 2023). A closer investigation of the debonding surface showed that a thin layer of concrete matrix adheres to the bar surface, while the superficial layer of the FRP bar, including coating sands, surficial resins, and winding spirals, is partially peeled off. Further analysis by Lee *et al.* (2008, 2017), as shown in Figure 2.2, indicated that the peeling off area ratio of the FRP bar surface layer increases with concrete strength. These observations suggest that as concrete strength increases, the contribution of concrete strength to bond resistance diminishes, and the shear resistance of the surface layer of FRP bars becomes more significant.

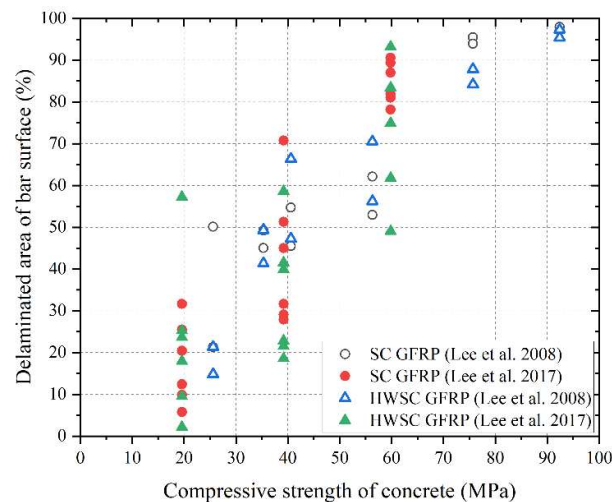


Figure 2.2. Ratio of the delaminated area of the surface layer of FRP bars

In UHSC (for concrete grade $> C120$), many researchers have found that bond failure generally occurs at a critical interface within the FRP bar itself (Firas *et al.* 2011; Yoo *et al.* 2015; Kim and Wang 2022; Pan *et al.* 2022; Liang *et al.* 2023; Zhu *et al.* 2023; Hu *et al.* 2024; Jiang *et al.* 2024). They reported that FRP bars are seriously damaged with the resin-rich surface layer completely sheared off and, in some cases, internal fibers near the bar surface being chopped. This indicates that bond strength is governed by the horizontal shear capacity of the resin-rich layer and inner bar core, rather than by concrete strength. In this case, the failure mode remains unchanged with further increases in concrete strength.

Overall, as concrete strength increases, the occurrence of bond failure changes from the surrounding concrete to the surface layer of the FRP bar, and eventually approaches the bar core. However, identifying those failure modes based solely on concrete strength is challenging. The development of the debonding surface is controlled by the relative shear capacity of the concrete and the resin-rich surface layer, as well as the subsequent core layer of the FRP bars. This is, in fact, dependent on the coating quality and characteristics of the resin and resin-fiber interface of the FRP bars.

2.2.3. Factors affecting the bond

Previous studies (Yan *et al.* 2016; Basaran and Kalkan 2020b) have reported that the bond performance between FRP bars and concrete depends on (i) the properties of FRP bars, on (ii) the properties of surrounding concrete, and (iii) the confinement conditions of FRP bar-concrete bond joints.

2.2.3.1. Effect of concrete type

For the safe use of RC structural members using materials other than normal concrete, the bond between FRP bars and various types of concrete has been extensively studied. This section analyzes the influence of concrete type, including several innovative concrete mixtures (i.e., seawater and/or sea-sand concrete (SW/SSC), recycled aggregated concrete (RAC), coral aggregate concrete (CAC), limestone calcined clay cement concrete (LC), and geopolymers cement concrete (GPC)) and conventional concrete (CC), on the bond performance.

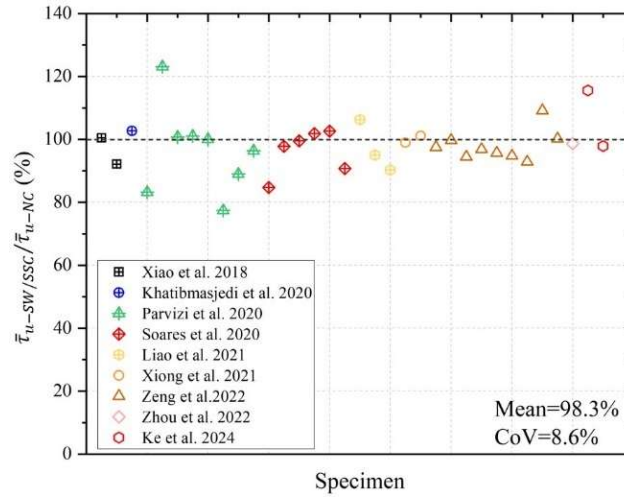
Previous studies have indicated that the replacement of CC with seawater and/or sea-sand appears to have no distinct effect on the short-term bond-slip response or the bond strength (Xiao *et al.* 2018; Khatibmasjedi *et al.* 2020; Parvizi *et al.* 2020; Soares *et al.* 2020; Liao *et al.* 2021; Xiong *et al.* 2021; Zeng *et al.* 2022; Zhou *et al.* 2022; Ke *et al.* 2024). Experimental observations have demonstrated that SW/SSC specimens exhibit a bond-slip curve shape similar to that of CC specimens. Figure 2.3 (a) compares the average bond strength of SW/SSC specimens with their conventional counterparts. It is noteworthy that some variations in concrete strength were reported between the SW/SSC specimens and the counter CC specimens. To account for this, and in accordance with long-standing use in design codes (ACI 2022; ACI 2019), the bond strength of all specimens is normalized with respect to concrete compressive strength to eliminate the effect of variations in concrete strength, as described by Eq. (2.1)

$$\bar{\tau}_u = \tau_u / \sqrt{f_c} \quad (2.1)$$

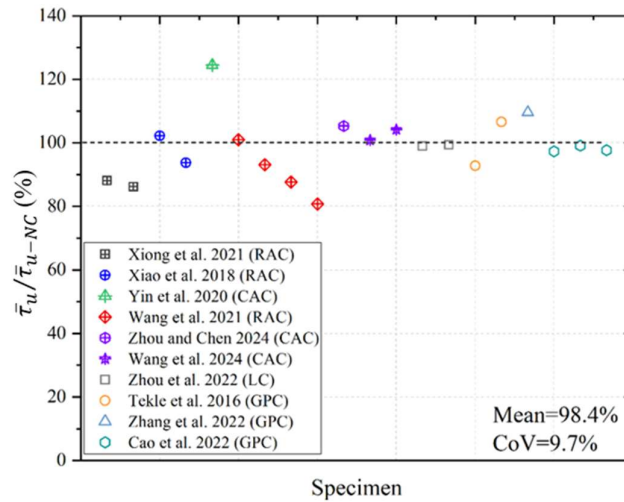
where τ_u is the measured bond strength and f_c is the specified compressive strength of concrete. Although the use of $\sqrt{f_c}$ in Eq. (2.1) may not always accurately represent the contribution of various concrete strengths, particularly for UHSC as discussed in section 2.2.2, the normalization is acceptable here for the following reasons: (1) the UHSC mixtures used in the SW/SSC specimens were of the same concrete grade as those in the control specimens, rendering the effect of concrete strength negligible; (2) the variation in concrete strength among the collected SW/SSC and CC specimens with normal and high compressive strength is not large enough to alter the failure mode of two groups of specimens, thus preventing misestimation of the variation in concrete strength between SW/SSC specimens and their counterparts. Figure 2.3 (a) shows that the normalized bond strength of SW/SSC specimens is approximately equivalent to that of the control CC specimens, with a mean ratio of 98% and a coefficient of variation (CoV) of 9%.

In this section, a one-way ANOVA test was conducted to analyze differences in normalized bond strengths among groups of different concrete types, providing a statistical test to determine whether the mean normalized bond strengths of these groups are all equal. The significance level was 0.05. The basic assumptions of the ANOVA test have been fulfilled: (1) the dependent variable (i.e., normalized bond strength) is normally distributed, and (2)

the population variances are roughly equal. The ANOVA test results are summarized in Table 2.1. It can be found that for all groups, there is no statistically significant difference in the normalized bond strength between the SW/SSC and CC specimen groups, with all p -values higher than 0.05. This suggests that the bond performance of FRP bars is identical in concrete mixed with either seawater and/or sea-sand or conventional raw materials (i.e., fresh water and river sand).



(a) SW/SSC versus CC



(b) Other types of concrete versus CC

Figure 2.3. Comparison of normalized bond strength of different concrete types ($\bar{\tau}_u^*$: normalized bond strength of RAC, LC, and GPC)

Table 2.1. ANOVA test results between SW/SSC and CC specimen groups

Ref.	Specimen groups	Concrete type	Normality test	One-way ANOVA test	
			p-value	F	Sig
Parvizi <i>et al.</i> 2020	SP-12-CC-CA	CC	0.0752	3.5095	0.1032
	SP-12-SW-CA	SWC	0.6828		
	SP-12-CC-CH	CC	0.5306	5.8882	0.0723
	SP-12-SW-CH	SWC	0.5359		
	SP-16-CC-CA	CC	0.2088	0.3188	0.5878
	SP-16-SW-CA	SWC	0.8159		
	SP-16-CC-CH	CC	0.5428	0.0138	0.9123
	SP-16-SW-CH	SWC	0.1191		
	SC-12-CC-CA	CC	0.2255	0.0007	0.9800
	SC-12-SW-CA	SWC	0.3515		
	SC-12-CC-CH	CC	0.1760	4.3058	0.1066
	SC-12-SW-CH	SWC	0.1694		
	SC-16-CC-CA	CC	0.2807	1.1324	0.3183
	SC-16-SW-CA	SWC	0.5262		
Soares <i>et al.</i> 2020	SC-16-CC-CH	CC	0.3639	0.0757	0.7968
	SC-16-SW-CH	SWC	0.1225		
	TW_7D_Ø8_Lb5Ø	CC	0.5240	5.3442	0.0819
	SW_7D_Ø8_Lb5Ø	SWC	0.3351		
	TW_28D_Ø12_Lb5Ø	CC	0.0782	0.0070	0.9372
	SW_28D_Ø12_Lb5Ø	SWC	0.6161		
	TW_7D_Ø8_Lb10Ø	CC	0.3441	0.0640	0.8127
Liao <i>et al.</i> 2021	SW_7D_Ø8_Lb10Ø	SWC	0.6288		
	TW_28D_Ø12_Lb10Ø	CC	0.5652	3.7647	0.1243
	SW_28D_Ø12_Lb10Ø	SWC	0.2774		
	N80-PE0%-5D	CC	0.1096	1.5817	0.2769
	S80-PE0%-5D	SWSSC	0.4334		
	N80-PE0.5%-5D	CC	0.5968	0.2104	0.6703
Xiong <i>et al.</i> 2021	S80-PE0.5%-5D	SWSSC	0.6212		
	N80-PE1%-5D	CC	0.0610	0.4641	0.5331
	S80-PE1%-5D	SWSSC	0.5888		
	R8r-G	RAC	0.4558	0.0570	0.8231
Zeng <i>et al.</i> 2022	R8sr-G	SSRAC	0.6258		
	R8c-G	CC	0.1719	0.0106	0.9231
	R8s-G	SSC	0.0592		
	C-0PE-φ10-5D	CC	0.6290	0.1072	0.7598
Zhou <i>et al.</i> 2022	S-0PE-φ10-5D	SWSSC	0.4648		
	C-5PE-φ10-5D	CC	0.2634	0.0076	0.9348
	S-5PE-φ10-5D	SWSSC	0.0615		
	C-10PE-φ10-5D	CC	0.5817	2.4117	0.1954
	S-10PE-φ10-5D	SWSSC	0.4950		
	C-0PE-φ6-5D	CC	0.6307	0.1727	0.6990
	S-0PE-φ6-5D	SWSSC	0.2733		
	C-5PE-φ6-5D	CC	0.3667	0.2699	0.6309
	S-5PE-φ6-5D	SWSSC	0.0599		
	C-5PE-φ16-5D	CC	0.2421	0.6995	0.4500
	S-5PE-φ16-5D	SWSSC	0.4278		
	C-10PE-φ16-5D	CC	0.1363	0.0002	0.9903
	S-10PE-φ16-5D	SWSSC	0.3152		
Zhou <i>et al.</i> 2022	R-4.5	RAC	0.5265	0.1155	0.7511
	S-4.5	SWSSRAC	0.2661		

A comparison of the mean values of normalized bond strength among other types of innovative concrete and CC is presented in Figure 2.3 (b). In general, the normalized bond strengths of RAC, CAC, LC, and GPC specimens are comparable to those of CC specimens, with a mean ratio of 98%. Except for the specimen groups in Yin *et al.* (2020), the one-way ANOVA test results in Table 2.2 confirm that replacing conventional aggregate or cement with other sustainable materials does not lead to a statistically significant difference in normalized bond strength. In Yin *et al.*'s study (2020), the actual concrete strength of CAC during testing was higher than the nominal value, due to the relatively low actual water/cement ratio of the CAC mixture, caused by the high water absorption rate of coral aggregates that did not undergo saturated surface dry treatment.

Table 2.2. ANOVA results between innovative and conventional concrete specimen groups

Ref.	Specimen groups	Concrete type	Normality test	One-way ANOVA test	
			p-value	F	Sig
Xiong <i>et al.</i> 2021	R8c-G	CC	0.1719	1.3555	0.3090
	R8r-G	RAC	0.4558		
	R8s-G	CC	0.0592	3.8402	0.1216
	R8sr-G	RAC	0.6258		
Yin <i>et al.</i> 2020	Z1-BB-PC40	CC	0.6277	8.3430	0.0203
	H5-BB-SC40	SWCAC	0.7721		
Zhou and Chen 2024	NC-			1.6372	0.2699
	D14L12T4C1d0S2	CC	0.40761		
	D14L12T4C1d0S2	SWSSCAC	0.62649		
Zhou <i>et al.</i> 2022	R-4.5	CC	0.5265	0.0437	0.9576
	A(35)-4.5	LC1	0.6267		
	A(50)-4.5	LC2	0.3821		
Tekle <i>et al.</i> 2016	O13-6d	CC	0.4549	1.0915	0.3551
	G13-6d	GPC	0.5359		
	O16-3d	CC	0.2558	0.4504	0.5389
	G16-3d	GPC	0.5359		
Zhang <i>et al.</i> 2022	CAC-B7	CC	0.4728	1.3884	0.3040
	AACAC-B7	GPC	0.6286		
	AACAC-B7	GPC	0.6286		

From these results, it can be concluded that the concrete type is not a critical factor to consider when evaluating the bond performance of FRP bars. Instead, the influence of concrete type on bond performance is primarily dependent on changes in concrete strength resulting from

the replacement of raw materials in the matrix.

2.2.3.2. Effect of concrete strength

In existing studies, the compressive strength measured at 28 days is commonly referred to as concrete strength, and this designation will also be used in this thesis. Regarding the bond between reinforcement and concrete, it is widely accepted that an increase in concrete strength generally enhances bond strength (*fib* 2013). However, unlike steel bars, this is not always the case for the bond strength of FRP bars. Early studies (Achillides 1998; Pecce *et al.* 2001; Nanni *et al.* 1995) suggested that there is an upper limit of concrete strength beyond which the bond strength of FRP bars is governed by the shear properties of the bars and increases barely with further increases in concrete strength. Based on experimental investigations, they recommended a relatively low critical value of concrete strength, between 30 and 50 MPa. However, as shown in Figure 2.4, recent research has shown that even beyond this limit, the bond strength of FRP bars increases substantially with increasing concrete strength (Lee *et al.* 2008; Won *et al.* 2008; Hu *et al.* 2023; Yoo *et al.* 2023; Ke *et al.* 2024). These contradictory observations can be attributed to significant advancements in the properties and fabrication processes of FRP bars over the past decade (Solyom and Balázs 2020). As a result, the shear strengths of the resin-rich surface layer and subsequent core layers of FRP bars have been enhanced, leading to the change of failure mode from delamination within the FRP bars to concrete crushing, even in high strength concrete (HSC), and thus allowing for a significantly higher threshold value of concrete strength. For FRP bars with improved fabrication, a comprehensive understanding of how concrete strength affects bond performance is essential.

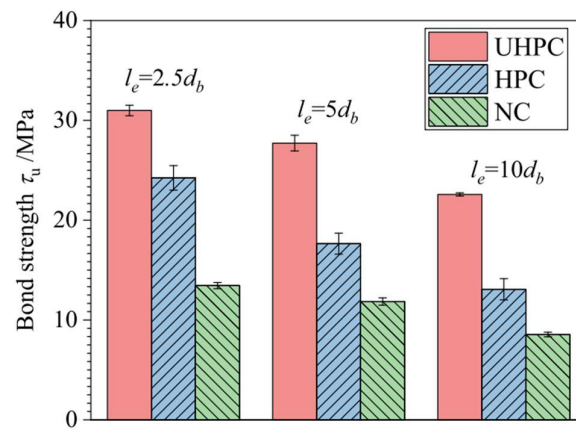


Figure 2.4. Effect of concrete strength on bond strength (Hu *et al.* 2023)

In terms of bond strength with increasing concrete strength, previous research has compared the SC FRP bars as well as HWSC FRP bars with steel bars (Lee *et al.* 2008, 2012). They concluded that the increasing rate of FRP bars with respect to concrete strength is much smaller than that of steel bars (see Figure 2.5). Moreover, in UHSC classes, a relatively low dependency of bond resistance on concrete strength has been reported: the bond strength of FRP bars varies marginally as concrete strength increases, as shown in Figure 2.6 (Firas *et al.* 2011; Chen *et al.* 2022; Liang *et al.* 2023). As mentioned in section 2.2.2, this is ascribed to the less pronounced contribution of concrete in FRP bars in HSC and especially in UHSC, where shear damage of FRP bars is also involved, in addition to damage in the concrete substrate. Currently, the influence of concrete strength on bond strength is represented using $\sqrt{f_c}$ in existing design expressions for FRP bars (ACI 2022; CSA 2012; CSA 2019). However, Lee *et al.* (2008, 2012) found that $\sqrt{f_c}$ significantly overestimates the contribution of concrete strength. Several research teams have provided an improved match with test data using lower powers of concrete strength, ranging from 0.25 to 0.41, for SC and HWSC FRP bars (Lee *et al.* 2008, 2012; Tang *et al.* 2008; Nepomuceno *et al.* 2021; Wang *et al.* 2024).

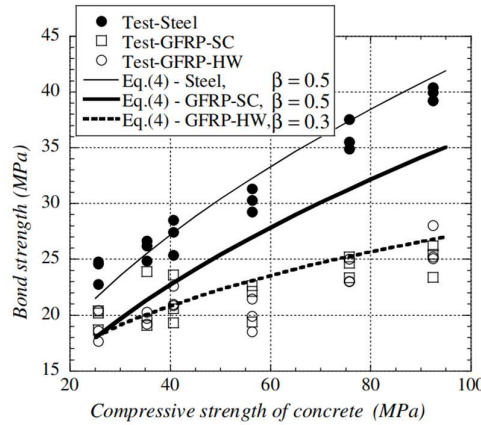


Figure 2.5. Increase in bond strength of steel and GFRP bars (Lee *et al.* 2008)

According to several comprehensive studies of various surface configurations of FRP bars, surface configuration has a great impact on the effect of concrete strength on bond strength (Tang *et al.* 2008; Solyom and Balázs 2021; Nepomuceno *et al.* 2021; Wang *et al.* 2024). These studies observed that the increase in bond strength with respect to concrete strength varies with the surface characteristics of FRP bars, probably due to different contributions from the three bond mechanisms associated with various surface configurations.

Consequently, in their proposed bond strength models, factors are provided to account for the contribution of concrete strength depending on surface type derived from their datasets. However, the proposed concrete strength factors for each surface type vary significantly across different studies. Based on these experimental observations, it appears that the effect of concrete strength on bond strength interacts with that of surface configuration. Further discussion on this interaction effect is required.

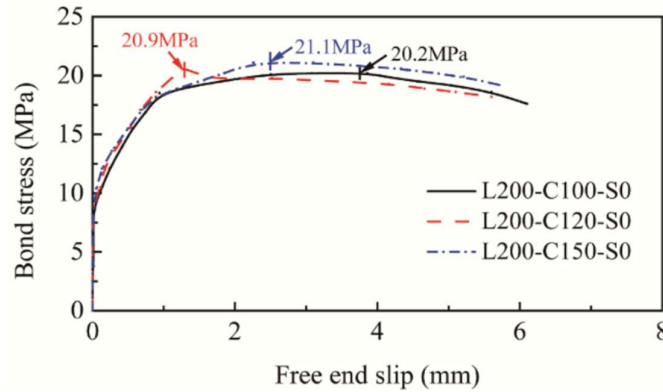


Figure 2.6. Effect of concrete strength in UHSC class on bond strength (Chen *et al.* 2022)

2.2.3.3. Effect of surface configuration

To enhance bond performance, the surface of FRP bars is commonly finished with surface deformation and sand coating (Cosenza *et al.* 1997; Hao *et al.* 2009; El-Nemr 2016). Based on surface configuration, commercially available FRP bars in Figure 2.7 can be generally categorized as SC bars, helically wrapped (HW) bars, ribbed (Rb) bars, In bars, and HWSC bars.

An early comprehensive study was unable to explore the effect of bar surface type on bond strength due to limited test results for different surface configurations in its created database (Wambeke and Shield 2006). As a result, the design equation in the ACI code (ACI 2022), based on the work of Wambeke and Shield (2006), did not consider the effect of surface configuration on development length. In contrast, CSA S806-12 (CSA 2012) valued the significance of surface profile and specified a factor for different surface types of FRP bars when evaluating development length. However, the contributions of different surface configurations stipulated in CSA S806-12 (2012) are inconsistent with some experimental observations. Some researchers have suggested that FRP bars with surface coating sands

achieve better bond performance (El-Nemr *et al.* 2016; Solyom and Balázs 2020; Parvizi *et al.* 2020; Basaran and Kalkan 2020b; Yin *et al.* 2020; Lu *et al.* 2021), while other studies advocate for deformed bars over sand-coated ones for superior bond strength (Aiello *et al.* 2007; Mazaheripour *et al.* 2013; Huang *et al.* 2020; Fahmy *et al.* 2021; Xiong *et al.* 2021; Yoo *et al.* 2023).

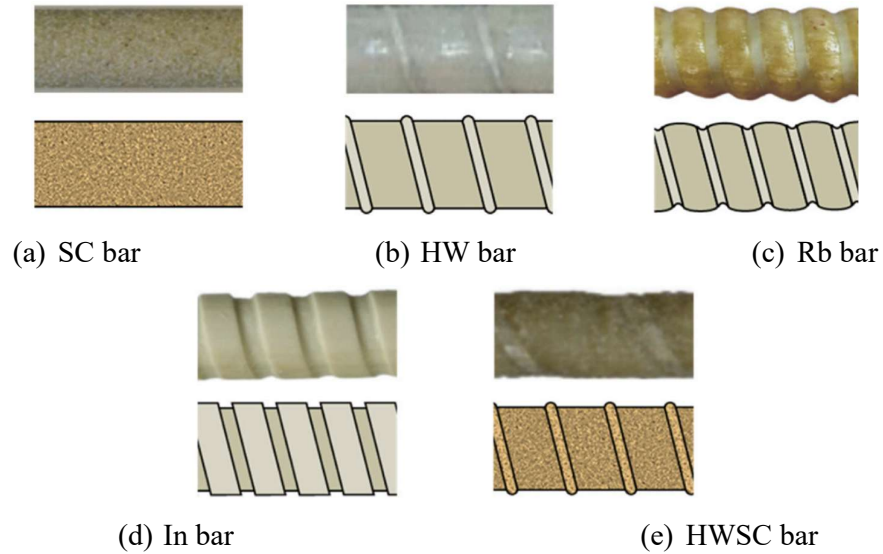


Figure 2.7. Schematic view of FRP bars with various surface types

In addition, many studies reported significant variations in bond strength within the same surface type (Hao *et al.* 2009; Arias *et al.* 2012; Lämmlein *et al.* 2017; Henin *et al.* 2019; Basaran and Kalkan 2020b; Solyom and Balázs 2020; Shan *et al.* 2023; Liang *et al.* 2023; Jiang *et al.* 2024). For instance, in the case of the Rb FRP bars, those with optimal combinations of rib geometries (i.e., rib height and rib spacing in Figure 2.8) presented higher bond strength by a maximum of 21% to 80%, as observed in previous research (Hao *et al.* 2009; Solyom and Balázs 2020; Shan *et al.* 2023; Liang *et al.* 2023; Jiang *et al.* 2024). For the FRP bars with sand coating, bond strength varied within wide limits depending on the size and density of sand, as well as the quality and production variability of the coating. Several researchers (Arias *et al.* 2012; Solyom and Balázs 2020; Basaran and Kalkan 2020b) concluded that bars coated with coarse sand possessed bond strength ranging from 1.2 to 1.6 times that of bars with fine sand coating, due to the more effective interlocking and friction of coarse sands. In addition, Henin *et al.* (2019) and Toumpanak *et al.* (2018) reported that the bond strength values of densely sand coated FRP bars were 24% to 80% higher than those of FRP bars with thin sand coating. Importantly, these research groups (Toumpanak *et al.*

2018; Henin *et al.* 2021) also highlighted the effect of production variability in the surface condition on bond behavior, reporting large scatter in the pullout test results of sand-coated bars.

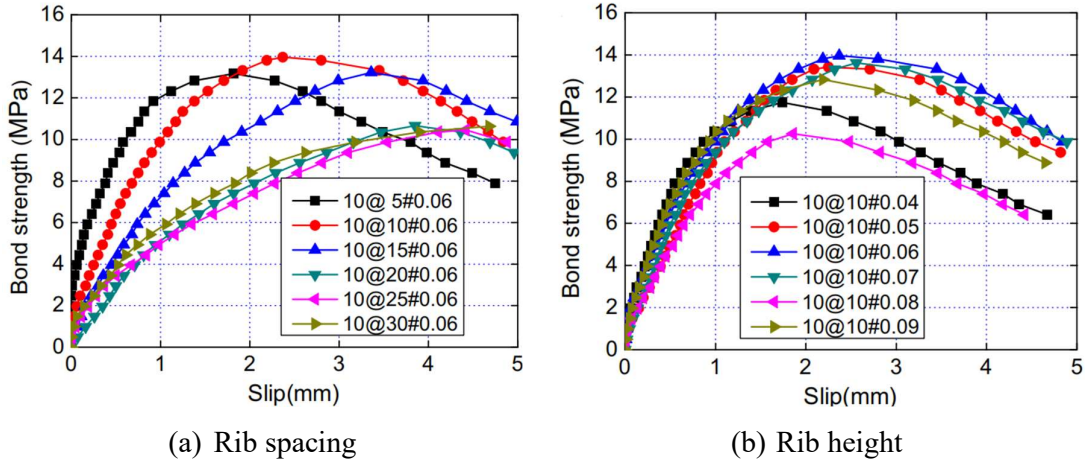


Figure 2.8. Effect of rib geometries on bond strength (Hao *et al.* 2009)

All these observations indicate that the bond strength of FRP bars is influenced not only by surface type but also by the physical properties of the surface condition (i.e., the size and density of sand adhered to the bar surface, the geometry of deformation, and the surface finishing process) and their corresponding variations. As a result, Solyom and Balázs (2021) suggested that specifying the surface factor based solely on surface type, as stipulated in the Canadian standard (CSA 2012), is impractical. Inevitably, the non-standardized fabrication of FRP bars complicates the examination of the effect of surface configuration, necessitating an interpretation of extensive test data.

2.2.3.4. Effect of bar diameter

The size effect on the bond of reinforcement in concrete has been well documented through experimental evidence, showing that bond strength decreases as bar diameter increases. The main explanations are given as follows: (a) bleeding effect—for larger diameter bars, more defects are accumulated at the vicinity of the bar-concrete interface caused by bleeding water, resulting in the reduced contact area between bar and concrete and thus a decrease in bond strength (Benmokrane and Tighiouart 1996); (b) Poisson's effect—the reduction in bar diameter caused by Poisson's effect is more pronounced in large diameter bars, leading to reduced bearing stress (Davalos *et al.* 2008); (c) shear lag effect—large diameter bars exhibit

more evident normal stress differences in cross section, resulting in less shear stress developed near the bar surface (Tepfers 2006). Bažant *et al.* (1995) and Bamonte *et al.* (2003) indicated that the effect of bar diameter on bond follows the general size effect law due to the localization of concrete damage at the bar-to-concrete interface, and they have demonstrated the applicability of this law in evaluating the impact of bar diameter on bond strength. However, the influence of bar diameter is not always considered in bond strength equations (Nepomuceno *et al.* 2021; Wang *et al.* 2024).

Significantly, the effect of bar diameter on bond strength is inconsistent for all grades of concrete strength. Previous studies (Bamonte and Gambarova 2007; Lee *et al.* 2017) stated that the decrease degree of bond strength with increasing bar diameter moderates as concrete strength increases. As shown in Figure 2.9, Lee *et al.* (2017) reported that the reduction rates of bond strength of SC and HWSC GFRP bars decreased from 50% and 55% in NSC to 30% and 33% in HSC, when the nominal bar diameter increased from 13 mm to 25 mm. In UHSC, results from Jiang *et al.* (2024) and Hu *et al.* (2024) showed that when specimens were subjected to pullout failure, the decline in bond strength was insignificant with increasing diameters of FRP bars, with reductions of less than 10%. This indicates that the size effect on bond strength may become less pronounced as concrete strength increases. Consequently, it can be preliminarily confirmed that the bar diameter and concrete strength may have a synergic influence on bond strength, which needs further study.

Furthermore, many experimental studies have investigated the possible interaction between the effect of bar diameter and surface type (Mazaheripour *et al.* 2013; Basaran and Kalkan 2020b; Parvizi *et al.* 2020; Soares *et al.* 2020; Solyom and Balázs 2021). Solyom and Balázs (2021) observed that the effect of bar diameter significantly varied with the surface type and suggested different equations to define the influence of bar diameter depending on the surface type, as given in Table 2.3. Unlike other types of FRP bars, researchers (Mazaheripour *et al.* 2013; Parvizi *et al.* 2020; Soares *et al.* 2020; Solyom and Balázs 2021) pointed out that for the FRP bars finished with notable ribs and lugs (i.e., Rb and In bars), the bond strength of FRP bars with a larger diameter was higher than that of smaller-diameter bars. For example, Soares *et al.*'s (2020) study showed that the bond strength increased by 21%-37% when 12-mm-diameter bars were used instead of 8-mm-diameter bars. This is due to the fact that larger bars have thicker ribs and lugs, producing greater bearing force and contact area with the surrounding concrete, which increases bond strength. In addition, for bars with surficial sand

coating, the work done by the author's group and Toumpanaki *et al.* (2018) demonstrated that smaller bars were more likely to yield large variations in pullout results. They concluded that this is attributed to irregularities in the surficial sand coating during manufacturing, due to the limited coating area of smaller bars. The variability in the sand-coating layer seems to affect bond performance and, consequently, overshadows the effect of bar diameter. Due to the interaction between the effect of bar diameter and surface condition, the relationship between bar diameter and bond strength is not always straightforward.

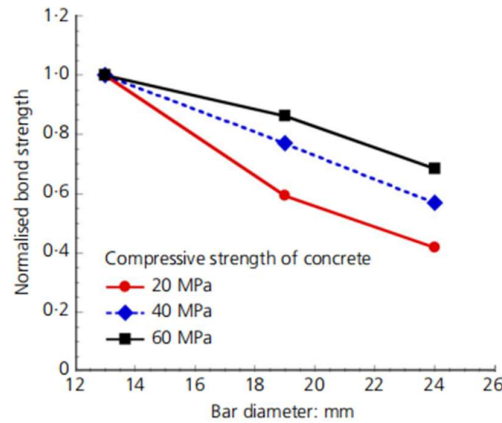


Figure 2.9. Effects of bar diameter together and concrete strength on bond strength (Lee *et al.* 2017)

Table 2.3 Formulae to define the influence of bar diameter on the bond strength depending on the surface type (Solyom and Balázs 2021)

Surface type	Equation	R ²
HW	$\frac{1}{2.34 + 0.0181 \cdot d^2 - 0.309 \cdot d}$	0.997
In	$\frac{1}{1.84 - 0.0034 \cdot d^2}$	0.718
Rb	$\frac{1}{0.747 + 0.0038 \cdot d^2}$	0.468
SC	$\frac{1}{0.934 + 0.0015 \cdot d^2}$	0.638
HWSC	$\frac{1}{3.6 + 0.0314 \cdot d^2 - 0.583 \cdot d}$	0.628

2.2.3.5. Effect of horizontal shear strength of FRP bars

Since bond failure occurs within the FRP bars in UHSC and in HSC (Firas *et al.* 2011; Kim and Wang 2022; Pan *et al.* 2022; Liang *et al.* 2023; Tong *et al.* 2023; Hu *et al.* 2024; Jiang *et*

al. 2024), the nature of bond performance in these concrete matrixes can be expected to depend primarily on the shear characteristics of the FRP bars (Chen *et al.* 2022; Liang *et al.* 2023). However, few studies have attempted to explore the relationship between the horizontal shear strength (i.e., interlaminar shear strength (ILSS)) of FRP bars and bond strength in this failure pattern. For an accurate assessment of the contribution of the horizontal shear strength of FRP bars, more work is needed.

2.2.3.6. Effect of fiber type

Generally, the effect of fiber type on bond strength is represented by the modulus of elasticity of FRP bars. Some studies found that the bar modulus appeared to influence bond strength (Okelo and Yuan 2005; Refai *et al.* 2015; Wei *et al.* 2019). They claimed that FRP bars of higher modulus developed stronger bonds with concrete. However, other studies showed that the bond strength was only marginally affected by the modulus of elasticity of FRP bars, and it was not uncommon for bars with a lower modulus to produce nearly identical or slightly higher bond strength (Achillides and Pilakoutas 2004; Lämmlein *et al.* 2017; Hossain *et al.* 2017; Yin *et al.* 2020; Zhang *et al.* 2021; Zhang *et al.* 2022; Zhou *et al.* 2023). In the former cases, bars with a higher modulus were often produced with stronger bearing ribs (i.e., stronger mechanical interaction) or higher adhesion of the sand-coating layer, which contributed more significantly to the higher bond strength than the modulus itself. As observed in the latter cases, the effect of bar modulus, or fiber type, on bond strength appeared to be insignificant when bars were manufactured with the same surface configuration and fabrication process.

For bars embedded in HSC, the test results of Zhang *et al.* (2021) claimed that the influence of fiber type on bond strength needs to be re-examined. Although bars of different fiber types showed similar bond performance in NSC, they observed that in HSC, the bond strength of GFRP bars was higher than that of BFRP bars, as illustrated in Figure 2.10. More importantly, the difference in bond strength between GFRP bars and BFRP bars became more pronounced as concrete strength increased. However, more experimental work is needed for validation, particularly in UHSC.

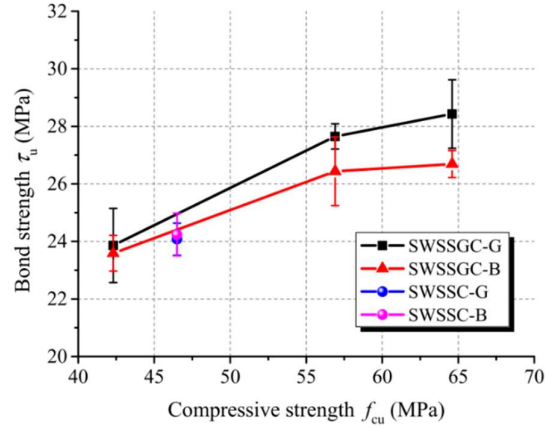
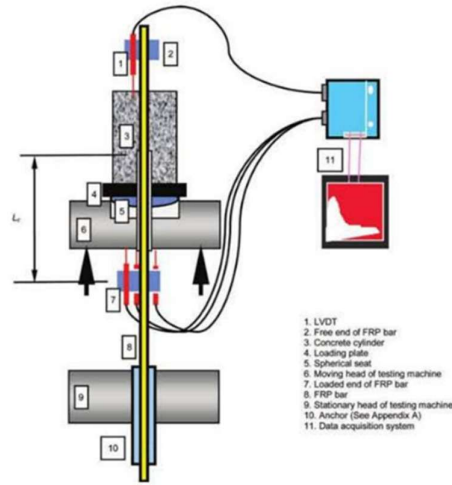


Figure 2.10. Effect of fiber type on bond strength in different concrete matrices (Zhang *et al.* 2021)

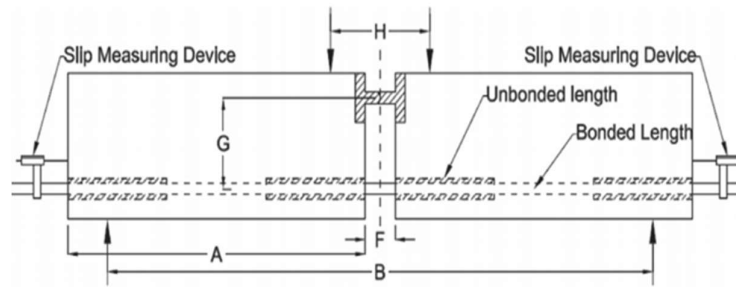
These studies reported that the influence of fiber type on bond strength may be influenced by concrete strength. This is probably because bond failure in NSC generally occurs within the concrete matrix and adjacent surficial layer of FRP bars, where the shear capacity of surrounding concrete and the surficial layer of specific FRP bar type dominate the bond. However, for higher-strength concrete, especially UHSC, it can be anticipated that bond failure near Zone 2 causes the rupture of internal fibers as discussed in section 2.2.2, indicating that the fiber rupture strength plays an important role in the bond performance. To properly evaluate the effect of fiber type on bond strength across various concrete strengths, further statistical analysis is needed.

2.3. Existing models and design equations

In this section, two types of models and equations will be reviewed. The first part will focus on models for the prediction of bond strength of the local bond-slip relation. These models are commonly regressed from results of pullout tests in Figure 2.11 (a), which allows the execution of extensive experimental investigations to carry out comparative studies on the effects of various parameters on bond characteristics due to its convenience. Within a short bond length in the pullout test, the experimentally determined values are commonly treated as local bond strength. Later in this section, those models and equations that were developed for the calculation of design bond stress will be discussed. For this purpose, these bond values are determined based on the results of beam tests that simulate the state of bond stress developed in a flexural member, as shown in Figure 2.11 (b).



(a) Pullout test (ACI 108T)



(b) Beam test (Hossain *et al.* 2017)

Figure 2.11. Test methods for the determination of bond strength

2.3.1. Local bond strength

Table 2.4 summarizes existing bond strength models developed in previous research. Numerous bond strength models have been empirically derived for FRP bars using different data sets from pullout tests (i.e., bars with various surface configurations in NSC, HSC, UHSC) (Yan *et al.* 2016; Nepomuceno *et al.* 2021; Wang *et al.* 2024). Most models have been proposed by modifying the ACI's multiple linear regression equation (ACI 2022) or CSA's multiplicative equation (CSA 2012) to account for one or more influencing factors, such as concrete strength, bar size, surface type, embedment length, and confinement condition, as shown in Table 2.4. However, only one model developed in Liang *et al.*'s (2023) study considered the effect of shear strength of FRP bars on the bond in UHSC by setting an upper limit on bond strength based on the adhesion strength of the interface between the resin layer and the core fiber. In addition, the potential interaction effects of the aforementioned

parameters were not taken into account in these models.

Table 2.4 also shows that due to the long standing use of the $\sqrt{f_c}$ in existing design guidelines, most bond strength models use $\sqrt{f_c}$ to represent the contribution of concrete strength to bond strength of FRP bars in concrete. However, as reviewed in section 2.2.3.2, the bond strength developed in FRP bars increases at a significantly different rate than $\sqrt{f_c}$ with increasing concrete strength. Consequently, several studies (Lee *et al.* 2012; Nepomuceno *et al.* 2021; Chen *et al.* 2022; Wang *et al.* 2024) have changed the exponent of the concrete strength adopted to predict the bond strength. For instance, Lee *et al.*'s team (2012) suggested a lower exponent of 0.33 for SC and HWSC bars in NSC and HSC, and Chen *et al.* (2022) claimed that the bond strength of FRP bars was proportional to $f_{cu}^{0.122}$. In addition, two research teams (Nepomuceno *et al.* 2021; Wang *et al.* 2024) adopted different exponents of the concrete strength depending on the surface type of FRP bars, based on experimental observations mentioned in section 2.2.3.2 that the surface configuration has a great impact on the effect of concrete strength on bond strength.

In general, due to limited test results in previous studies, the effect of a single factor varies significantly with different types of FRP bars and different concrete substrates. The bond strength equations in Table 2.4 are consequently limited to specific FRP bar types. For developing a comprehensive model, a reliable database interpreting extensive data is of vital importance.

Table 2.4 . Summary of existing bond strength models

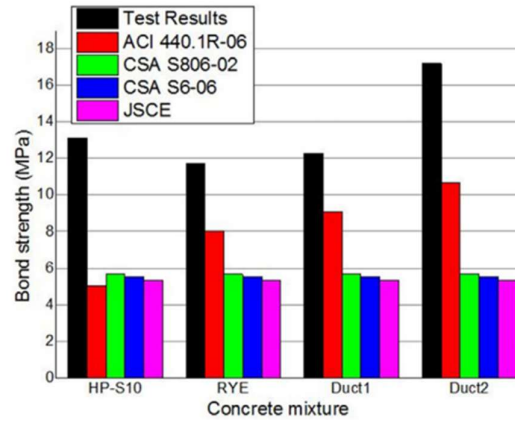
Ref.	Models	Limitations
Okelo and Yuan 2005	$\tau_u = 14.7 \sqrt{f_c}/d$ (V)	f_c : NSC, HSC d : 6-19 mm
Lee <i>et al.</i> 2012	$\tau_u = 4f_c^{0.33}$	f_c : NSC, HSC d : 13 mm
Xue <i>et al.</i> 2014; Hu <i>et al.</i> 2023	$\tau_u = \alpha' \phi(8.00 - 0.17d) \left(\frac{L}{d}\right)^{-0.4} f_t$ for NSC HSC $\tau_u = \alpha(0.9 + 0.4 \frac{C}{d} + 3.0 \frac{d}{L}) \sqrt{f_c}$ for UHSC	f_c : NSC, HSC, UHSPC d : 6-19 mm
Yoo <i>et al.</i> 2015	$\tau_u = (1.05 + 0.85 d/L) \sqrt{f_c}$	f_c : HSC, UHSC d : 13-19 mm
Shen <i>et al.</i> 2016	$\tau_u = (0.72 + 0.17 C/d) \sqrt{f_c}$	f_c : NSC, HSC d : 10-19 mm
Nepomuceno <i>et al.</i> 2021	$\tau_u = \alpha f_c^\beta$	f_c : NSC, HSC, UHSC d : 6-25 mm
Chen <i>et al.</i> 2022	$\tau_u = (52.73 \times (1 + 0.36 \rho_v^{0.816}) \frac{d}{L_e} + 8.77) f_{cu}^{0.122}$	d : 10 mm L_e : 20-30d f_{cu} : 100-150 MPa Bar type: In-CFRP
Pan <i>et al.</i> 2022	$\tau_u = (0.425d - 0.958)(0.264 \frac{d}{L_e} + 0.091 \frac{C}{d} + 0.126) \sqrt{f_{cu}}$	d : 10, 14, 18 mm C/d : ≤ 5 L_e : 1-2d Bar type: Rb-GFRP
Peng <i>et al.</i> 2022	$\tau_u = (1.4 + 3/d + 5d/L) \sqrt{f_c}$ (V)	f_c : NSC, HSC d : 3-10 mm
Kim and Wang 2023	$\tau_u = \sqrt{f_c}/(4\varpi)$ (V)	f_c : UHSC d : 10 mm
Liang <i>et al.</i> 2023	$\tau_u = 0.82E_f \frac{h_r^2 \sqrt{4h_r^2 + w_r^2}}{4rw_r s_r} + \frac{w_c + 0.25w_r}{s_r} \tau_s$ $\leq \tau_m = 40$ MPa	d : 10, 12 mm L_e : 3.5d f'_c : 64.5-122 MPa Bar type: Rb-GFRP, HW-GFRP
Tong <i>et al.</i> 2023	$\tau_u = k_f(0.5 + 0.3 \frac{C}{d} + 1.5 \frac{d}{L_e}) \sqrt{f_{cu}}$	d : 12, 16, 20 mm L_e : 3-5d f_{cu} : 90-130 MPa Bar type: Rb-GFRP, HW-GFRP
Jiang <i>et al.</i> 2024	$\tau_u = \frac{d_{cs} \sqrt{f_c}}{1.15k_1 k_2 k_3 k_4 k_5 \pi d} \times 14.23 \frac{d}{L}$	f_c : HSC, UHSC d : 8-16 mm
Wang <i>et al.</i> 2024	$\tau_u = f_c^{0.278} (5.536 + 0.15 \frac{C}{d})$ for HWSC $\tau_u = f_c^{0.249} (1.503 + 0.204 \frac{C}{d} + 20.169 \frac{d}{L})$ for SC	f_c : NSC, HSC d : 6-20 mm

Note: expressions marked with “(V)” are the models derived from vertically cast pullout specimens.

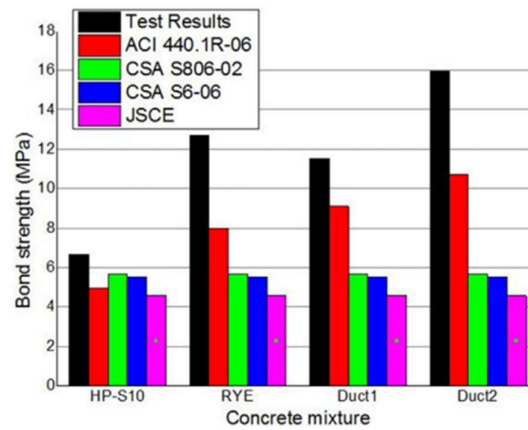
2.3.2. Design bond strength

Currently, several design equations for FRP bars reinforced concrete have been specified in design guidelines to determine the design bond strength in NSC, including ACI 440.1R-22 from the ACI Committee (2022), CSA S806-12 and CSA S6-2019 from the Canadian Standard Association (2012, 2019), and Recommendation from JSCE (1997). All these design equations take into account several key parameters, including concrete strength, concrete cover thickness, and the location of FRP bars. The ACI equation (Eq. (2.2) in Table 2.5) is the only one that considers the effect of embedment length on design bond strength, as the equation was derived from a comprehensive study conducted by Wambeke and Shield (2006) through 240 beam tests. That is the main reason why some research (Hossain *et al.* 2017; Michaud *et al.* 2021; Saleh *et al.* 2019; Basaran and Kalkan 2020; Zeng *et al.* 2022) claimed that ACI equation gives closest prediction on design bond strength, while other three design equations (Eq. (2.3), Eq. (2.4), and Eq. (2.5) in Table 2.5) that were developed based on a relatively long bond length tended to show comparatively conservative estimations for safety design. In addition, it can be noted that the existing design provisions haven't thought through the effects of bar type and corresponding surface type. Even though a surface profile factor has been incorporated in Canadian standards (CSA 2012, 2019), it does not suffice.

To date, there is no established design equation for the design bond strength for FRP bars in UHSC (JSCE 2004; KCI 2012; French Standard Institute 2016). As demonstrated by previous studies in Figure 2.12 (Ametrano 2011; Hossain *et al.* 2017a; Saleh *et al.* 2019; Basaran and Kalkan 2020a; Michaud *et al.* 2021; Chen *et al.* 2022; Xue *et al.* 2023), the aforementioned design provisions for NSC produced relatively unrealistic predictions for FRP bars in UHSC. They reported that the average experimental-predicted design bond strength ratios were up to 2.6 for the ACI equation and 6 for the CSA equations, and that these design provisions provided, on average, 2.5 times higher development lengths (calculated based on the design bond stress) than experimental results. This discrepancy inevitably hinders the full utilization of the superior mechanical properties of both FRP bars and UHSC.



(a) $\Phi 16$ FRP bars



(b) $\Phi 19$ FRP bars

Figure 2.12. Bond strength calculated based on design codes (Ametrano 2011)

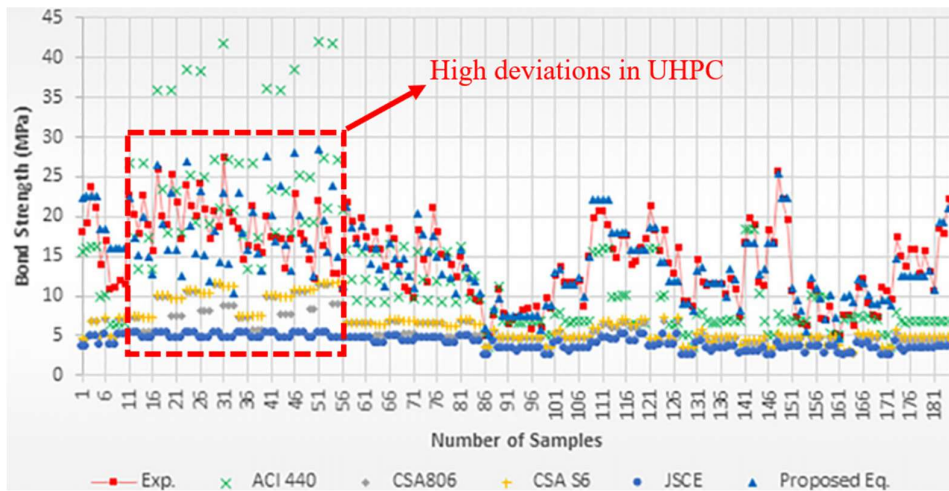


Figure 2.13. Experimental VS predictive values using Eq. (2.6) (Basaran and Kalkan 2020)

Table 2.5. Design provisions and models from existing studies

Design provisions and models		Symbols
Eq. (2.2)	$\frac{\tau_u}{0.083\sqrt{f_c}} = 4.0 + 0.3\frac{C}{d} + 100\frac{d}{l_e}$	C : lesser of cover to the center of bar or one-half of the center to spacing of bars being developed f_c : cylindrical compressive strength of concrete
Eq. (2.3)	$\tau_u = \frac{d_{cs}\sqrt{f_c}}{1.15(K_1K_2K_3K_4K_5)\pi d}$	K_1 : bar location factor; K_2 : concrete density factor; K_3 : bar size factor; K_4 : bar fibre factor (1.0 for CFRP and GFRP; 1.25 for AFRP); K_5 : surface profile factor (1.0 for roughened, SC, and braided; 1.05 for spiral pattern, and Rb; 1.80 for In) d_{cs} : lesser of closest concrete surface to the center of bar or two-thirds of the center to spacing of bars being developed f_{cr} : cracking strength of concrete $f_{cr} = 0.4\sqrt{f_c}$
Eq. (2.4)	$\tau_u = \frac{f_{cr}(d_{cs} + K_{tr} E_f / E_s)}{0.45\pi d k_1 k_4}$	k_1 : bar location factor k_4 : bar surface profile factor, depending on the ratio of bond strength of FRP bar to that of deformed steel bar, $k_4 \leq 1$ E_f : modulus of elasticity of FRP bar E_s : modulus of elasticity of deformed steel bar K_{tr} : transverse reinforcement index
Eq. (2.5)	$f_{bod} = \alpha_2 \left(\frac{\tau_u = f_{bod} / \alpha_1}{\frac{0.28 f_{ck}^{\frac{2}{3}}}{1.3}} \right) \leq 3.2 \text{ MPa}$	α_1 : confinement factor α_2 : modification factor for bond strength $\alpha_2 \leq 1$ f_{ck} : characteristic compressive strength of concrete
Eq. (2.6)	$\tau_u = (S - 0.2817 f_{tsp} + 6.706 \frac{d}{l_e} - 0.1237 d_o) f_{tsp}$	d_o : outer diameter of FRP bars f_{tsp} : splitting tensile strength of concrete S : coefficient for the reinforcement fiber and surface type
Eq. (2.7)	$\tau_u = 3.61\sqrt{f_c} - 27.3$	
Eq. (2.8)	$\tau_u = (-0.5 + 0.03\frac{C}{d} + 4.5\frac{d}{l_e} + \frac{21.6}{d})\sqrt{f_c}$	

Recently, several researchers (Basaran and Kalkan 2020; Michaud *et al.* 2021; Zhu *et al.* 2023) have proposed models to predict the design bond strength based on regression analysis on beam tests. In Basaran and Kalkan's model (2020) (Eq. (2.6) in Table 2.5), the effects of fiber type and surface type were synthesized into a single coefficient S . However, their model,

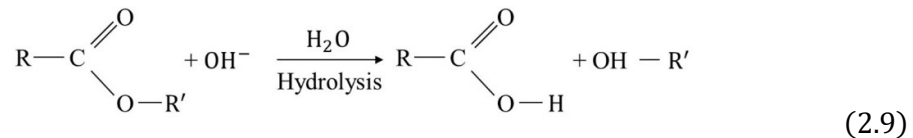
in spite of an acceptable Pearson Correlation of 0.86, did not accurately capture the bond strength in UHSC (see Figure 2.13), with high deviations in UHSC specimens. This is chiefly because this model misestimated the contribution of concrete strength and the horizontal shear resistance of FRP bars, which controls the bond behavior of FRP bars in UHSC, but may vary from different producers against previous discussion. The equation (Eq. (2.7) in Table 2.5) in Michaud *et al.*'s (2021) study aiming at predicting the bond in UHSC cannot be considered as a robust model, as only a single type of GFRP bar ($\Phi 17.2$ sand coated bar) was used in their beam tests where the effect of bar diameter and surface type cannot be accurately evaluated. That's the same as the situation in Zhu *et al.*'s model (2023) (Eq. (2.8) in Table 2.5). It was developed through limited data fitting and overlooked the influence of the properties of FRP bars, such as surface type and shear strength. Therefore, further comprehensive investigation is required for the development of a reliable design equation in terms of the bond of FRP bars in UHSC.

2.4. Long-term performance of FRP bars

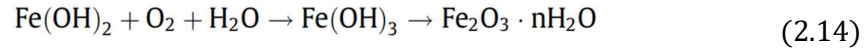
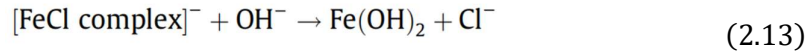
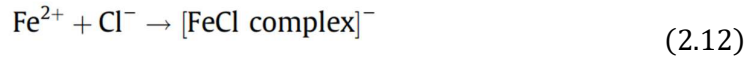
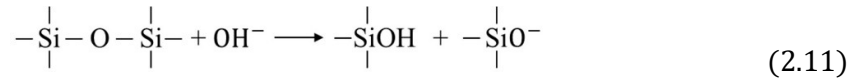
2.4.1. Degradation mechanisms

To ensure the long-term safety of FRP reinforced concrete structures, it is essential to assess the durability characteristics of FRP reinforcement under field conditions, particularly in coastal and marine environments. With the diffusion of hydroxyl ions and water molecules, the damage of FRP bars can be classified into three main processes: (1) physical and chemical attack, including hydrolysis and plasticization of resin, as well as leaching and etching of fibers; (2) fiber-resin interface degradation; (3) embrittlement and notching of fibers resulting from the formation and nucleation of hydration products.

Specifically, the diffused free hydroxyl ions react with the resin as shown in Eq. (2.9), leading to degradation of the ester group in the polymer by hydrolysis (Chen *et al.* 2007). The deterioration of the resin matrix also occurs by plasticization with the diffusion of water molecules by hindering the integrity of Van Waals bonds, which results in cracking and swelling of the matrix (Benmokrane *et al.* 2015).



Regarding the glass fiber, the glass can be dissolved by leaching as shown in Eq. (2.10). This reaction causes diffusion of alkali ions out of the glass structure in water and eventually evolves in the alkali attack, called etching. As expressed in Eq. (2.11), the silica network is damaged by breaking the Si-O-Si structure during the etching process (Charles 1959). For the basalt fiber, in addition to leaching and etching, iron that exists only in the basalt fiber can react with the chloride ions from the natural environment as presented in Eq. (2.12) to Eq. (2.14) (Wei *et al.* 2011). As a consequence of water and/or alkaline attack, the resulting formation and nucleation of hydration products may induce embrittlement and notching of fibers (Chen *et al.* 2007).



Finally, the interphase region between the fibers and the matrix is highly vulnerable to water exposure, which provides a convenient channel for the ingress of water molecules (Wang and Hahn 2007). The interfacial debonding, differential swelling, and propagation of microcrackings occur at the interface by water absorption (Liu *et al.* 2020).

Therefore, the fibers, resin matrix, and fiber-matrix interface of FRP are susceptible to moist and alkaline environments found in concrete, resulting in significant degradation in tensile, shear, and bond performance.

2.4.2. Durability of FRP bars under accelerated aging conditions

As the internal reinforcement in concrete, the FRP bars often experience moisture and alkaline environments, and the saline environment when the SW/SSC are used or when the RC structures are exposed to the coastal and marine conditions. This section reviews existing studies on the durability of FRP under such conditions.

2.4.2.1. Durability of bare and concrete-wrapped FRP bars

Hu *et al.* (2016) exposed glass FRP (GFRP) laminates to tap water and salt water at 60 °C. They found that the GFRP laminates exhibited high water absorption and significant plasticization of epoxy matrix, as well as fiber and fiber-matrix interface degradation, resulting in an average decline of 60% in tensile and interlaminar shear strength retention after one-year thermal aging. Benmokrane *et al.* evaluated the mechanical properties of various types of FRP bars in an alkaline solution simulating a concrete environment (Benmokrane *et al.* 2017; Ali *et al.* 2020; Benmokrane *et al.* 2020). They reported that three months of alkaline conditioning at 60 °C could severely degrade the tensile, interlaminar shear, and transverse shear strength of bare FRP bars by up to 43%, 25%, and 39%, respectively. Wang's team did similar research work on GFRP bars conditioned in tap water and saline and alkaline solutions, noting that the tensile strength retention varied from 30% to 50% after 12 months of exposure at 60 °C, depending on the pH values of the solution, due to fiber erosion (Wang *et al.* 2022; Wang *et al.* 2023). In addition, immersion in alkaline and saline environments and tap water resulted in bond strength retention as low as around 75% after 1 to 3 months of exposure, as reported by a comprehensive review (Nepomuceno *et al.* 2021).

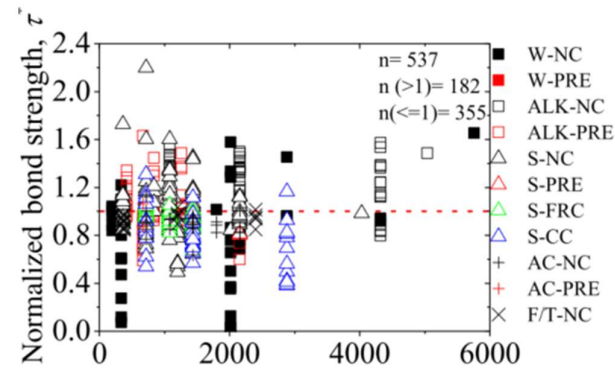
Several researchers have claimed that the traditional aging of bare bars in a simulated pore solution is too harsh compared to real concrete environments. For instance, Robert *et al.* (2009, 2013) reported minor deterioration of concrete-wrapped GFRP bars in tensile strength of only 10% after 240 days and 365 days of aging in tap water and saline solution, respectively, at 40 °C. However, in a similar study, Rolland *et al.* (2021) presented a significant loss of tensile strength of 32% of the same bars after aging in a simulated pore solution of concrete for 240 days at 40 °C. In Rifai *et al.*'s research (2020), bare FRP bars immersed in an alkaline solution exhibited a 29% tensile strength reduction, while those wrapped in moist concrete showed only a 15% reduction after nine months at 60 °C. Zhou *et al.* (2024) compared the loss of tensile strength of cementitious composites-covered FRP bars with bare bars exposed directly to an alkaline solution and observed that the wrapped FRP bars had 36% higher tensile strength retention after 180 days of conditioning at 40 °C. Therefore, it was concluded that the degradation rate of bare bars in accelerated alkaline conditions is higher than that of bars in most concrete at the same temperature, and predictions based on traditional aging data of bare bars are considered conservative. For a

more representative prediction of the long-term performance of FRP reinforcements in RC structures under actual service conditions, it is suggested to assess the properties of concrete-wrapped FRP bars using accelerated aging (Chen *et al.* 2007; Davalos *et al.* 2011; Almusallam *et al.* 2012; Fergani *et al.* 2018; Manalo *et al.* 2020; Morales *et al.* 2021). However, no research has focused on the long-term performance of UHSSC-wrapped FRP bars in marine conditions.

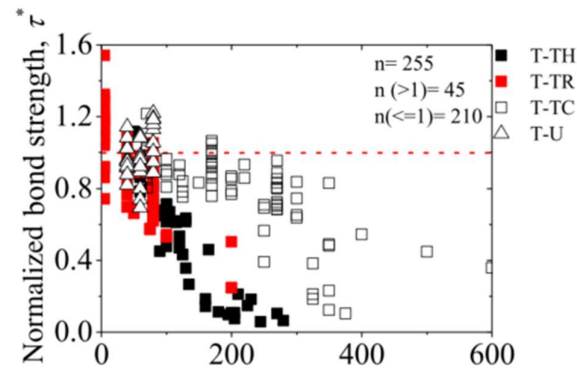
2.4.2.2. Bond durability of FRP bars

Studies available in the existing literature that investigated the bond durability of FRP bars in concrete commonly simulated the long-term field environments by exposing the specimens to accelerated aging conditions. Many researchers indicated that the bond strength was generally the mechanical property that measured the least significant degradation compared to the shear and especially tensile strength after conditioning (Chen *et al.* 2007; Khatibmasjedi *et al.* 2020; Nassar *et al.* 2021; Rolland *et al.* 2021; Emparanza *et al.* 2022; Li *et al.* 2023). For instance, Khatibmasjedi *et al.* (2020) reported that the bond strength showed the least reduction of less than 10% after 24 months of exposure, while reductions in shear and tensile strength were around 25% after the same exposure. In Emparanza *et al.*'s study (2022), the 60°C seawater exposure significantly degraded the tensile strength by up to 45%, followed by the bond strength with a maximum of 14%.

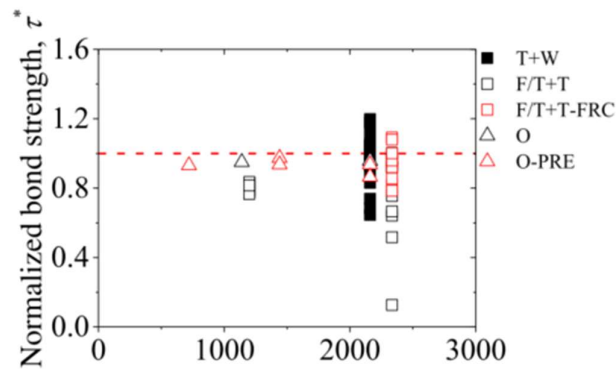
However, current test results observed that no clear trend was visible for the reduction in bond strength as shown in Figure 2.14, and that different fiber and resin combinations, together with various surface textures of FRP bars, could lead to different levels of degradation (Gravina *et al.* 2020). Previous studies demonstrated that there was a potential correlation between the decrease in bond strength and material degradation of FRP bars (Bank *et al.* 1998; Chen *et al.* 2007; Dong *et al.* 2018; Lu *et al.* 2021; Hao *et al.* 2023). Bank *et al.*'s (1998) mechanistic model related the observed physical degradation in the microstructure of FRP bars, in the form of matrix cracking, fiber chipping, and debonding at the fiber-resin interface, to the changes in bond strength after conditioning. However, there has been limited focus on quantitatively analyzing bond strength retention based on the material degradation (Lu *et al.* 2021; Hao *et al.* 2023).



(a) Exposure duration, t (Hours)
(a) hygrothermal conditions



(b) Exposure temperature, T (°C)
(b) thermal conditions



(c) Exposure duration, t (Hours)
(c) other conditions

Figure 2.14. Bond strength retention of tests from a comprehensive database (Gravina *et al.* 2020)

Table 2.6. Experimental research studies on bond durability in seawater environments

Ref.	Bar type		Concrete type	Concrete strength	Duration (days)
	Fiber type / resin type	Surface type			
Emparanza <i>et al.</i> 2022	GFRP1 (E-CR/V), GFRP2 (E-CR/V), GFRP3 (E/V), GFRP4 (E-CR/V), GFRP5 (E-CR/E), GFRP6 (E-CR/V),	HWSC, HW, Rb, SC	NC	NSC	Up to 360
Zheng <i>et al.</i> 2020	GFRP1 (/V), GFRP2 (/E)	Rb	NC	NSC	Up to 270
Emparanza <i>et al.</i> 2018	GFRP1 (E-CR/V), GFRP2 (E-CR/V), GFRP3 (E/V)	HWSC, HW, Rb	NC	NSC	Up to 120
Dong <i>et al.</i> 2016	GFRP (/V), CFRP (/E)	Rb	NC	NSC	Up to 60
Zhou <i>et al.</i> 2012	GFRP (E/P)	Rb	NC	NSC	Up to 90
Shakiba <i>et al.</i> 2023	GFRP (E/V)	SC	NC, FRC	NSC	Up to 150
Yan <i>et al.</i> 2017	GFRP (E/V)	HWSC	NC, FRC	NSC	Up to 60
Kazemi <i>et al.</i> 2022	GFRP (E/E)	SC	NC, SWC	NSC	Up to 250
Khatibmasjedi <i>et al.</i> 2020	GFRP (E-CR/V)	SW	NC, SWC	NSC	Up to 720
Chang <i>et al.</i> 2021	GFRP (E/V)	Rb	SWSSC	NSC	Up to 270
Ke <i>et al.</i> 2024	GFRP (E/V)	HWSC	NC, SWSSC	NSC, HSC	Up to 360
Bazli <i>et al.</i> 2017	GFRP (E/V)	SW	NC	NSC, HSC	Up to 150
El Refai <i>et al.</i> 2015	GFRP	In	NC	HSC	Up to 90
Altalmas <i>et al.</i> 2015	GFRP	In	NC	HSC	Up to 90

Note: ECR: ECR-glass; E: E-glass; V: vinyl ester; E: epoxy; P: polyester; FRC: fiber-reinforced concrete.

Regarding the bond between FRP reinforcement and concrete, recent studies have indicated that SW/SSC-wrapped FRP bars showed comparable normalized bond strength with their counterparts in normal concrete (Parvizi *et al.* 2020; Soares *et al.* 2020; Liao *et al.* 2021; Xiong *et al.* 2021; Zeng *et al.* 2022; Zhou *et al.* 2022). Through statistical analysis in section 2.2.3.1 using the ANOVA tests, the author's team further indicated that the substitution of SW/SSC in the concrete matrix has no statistically significant influence on the short-term

bond performance of FRP bars. However, Table 2.6 summarized that research on bond durability in SW/SSC is limited and has primarily focused on normal and high strength concretes (Khatibmasjedi *et al.* 2020; Chang *et al.* 2021; Kazemi *et al.* 2022; Ke *et al.* 2024). Additionally, only one study was performed on CFRP bars exposed to the seawater environment, and to date, no research has been devoted to the bond durability of FRP bars in UHSSC.

2.4.3. Durability of FRP bars in concrete exposed to field conditions

The accelerated aging test has been widely adopted in most of the available literature to assess the durability of FRP bars (D'Antino *et al.* 2018), since it is possible to get sufficient test data in a short space of time. The long-term performance of bars under field exposure can then be predicted based on accelerated aging test results and the Arrhenius relation theory (Nelson 2009). However, it has been reported that the field conditions are different from the idealized conditions at the laboratory, and that the predictions using the short-term accelerated test results were conservative compared to the data in field concrete structures (Wang *et al.* 2017a, b; Wang *et al.* 2018). Consequently, field investigation of the long-term performance of FRP bars is essential to develop a more representative prediction methodology and more appropriate environmental reduction factors for design purposes.

2.4.3.1. In-service GFRP bars in concrete structures

Limited studies were devoted to the long-term performance of in-service FRP bars in existing concrete structures (Mufti *et al.* 2007; Gooranorimi and Nanni 2017; Benmokrane *et al.* 2018; Al-Khafaji *et al.* 2021). The information and test results of the in-service cases are summarized in Table 2.7. As shown in Figure 2.15, the GFRP bars were extracted from concrete cores drilled from 17 concrete structures after 5-20 years of service in the United States and Canada. Most extracted samples were from normal concrete structures subjected to outdoor exposure, and only one sample set was from a marine structure (i.e., Hall's Harbor Wharf). These structures generally experience frequent wetting-and-drying cycles, freezing-and-thawing cycles, and exposure to chloride ions from deicing salts and salt water at the harbor for the wharf. The MAT and RH in the regions, where the concrete structures are, vary from 4 °C to 20 °C and 54% to 85%, respectively.

The physicochemical properties and microstructure of extracted GFRP bars were assessed

through scanning electron microscopy (SEM), energy dispersive x-ray (EDX), Fourier transform infrared spectroscopy (FTIR), and differential scanning calorimetry (DSC) analysis. The test results indicated that there was no physicochemical degradation of GFRP bars extracted from the in-service concrete structures, with no significant changes in the glass transition temperature of resin matrices and the chemical composition of the fibers and resin. The SEM observations revealed minimal debonding at the fiber-matrix interface and cracking in both the fibers and resin, demonstrating negligible changes in the microstructure of FRP bars after service exposure.

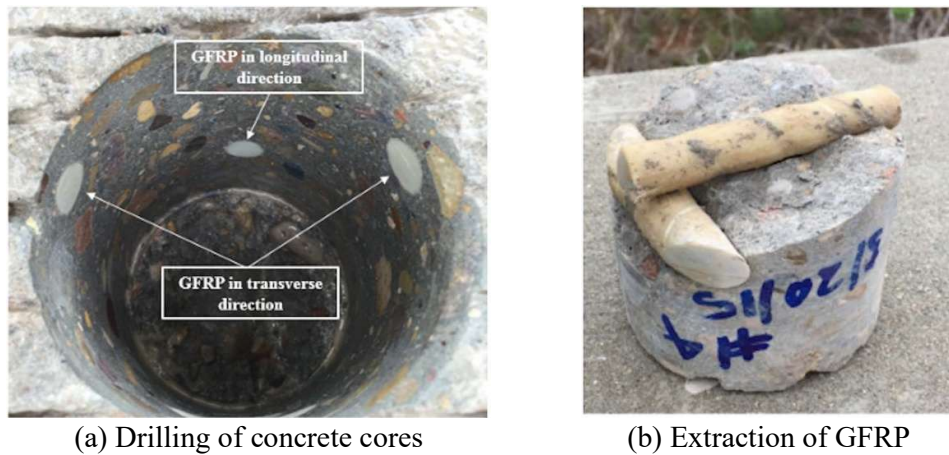


Figure 2.15. Extracted samples from in-service concrete structures (Gooranorimi and Nanni 2017)

The mechanical tests were also performed on extracted FRP bars in some bridges to examine their durability. Generally, the extracted FRP bars were not of enough length to test their tensile properties according to ASTM D7205 (ASTM 2016). In most cases, consequently, only horizontal shear strength could be evaluated using the short-beam test per ASTM D4475 (ASTM 2021). As shown in Table 2.7, there were certain decreases of around 7% to 27% in the horizontal shear strength of in-service GFRP bars compared to the reference bars tested at the time of bridge construction. However, these results were inconclusive, due to limited specimens extracted for each type of FRP bar. More importantly, the long-term horizontal shear strength is insufficient to provide necessary insight into the durability of FRP bars. This is mainly due to the fact that different mechanical properties having diverse degradation mechanisms may deteriorate at different rates. As reported in previous studies (Dejke and Tepfers 1997; Chen *et al.* 2007; Benmokrane *et al.* 2020; Emparanza *et al.* 2022; Wang *et al.* 2022; Hosseini *et al.* 2024), the tensile strength degraded much faster than the horizontal

shear strength of the same FRP bars. Therefore, the long-term performance of one mechanical property is unrepresentative of the overall durability of the in-service FRP bars. In previous studies, FRP bars from only one bridge (Sierrita de la Cruz Creek Bridge) were tested to evaluate both the tensile strength and horizontal shear strength after 17 years of service. The results showed that the extracted FRP bars presented satisfactory tensile durability with a minor tensile strength reduction of 2.5% based on a modified test method. The residual tensile strength, in that case, was determined as the average strength value of flat coupons extracted from different portions of tested bars, including the edge and core parts of FRP bars, as shown in Figure 2.16. However, this result is somewhat inconclusive due to limited test data measured using this non-standard method.

To improve the understanding of the long-term performance of FRP bars in field conditions, especially in coastal and marine structures in aggressive situations, additional information on the durability of the mechanical properties of FRP bars after exposure to realistic situations is required.

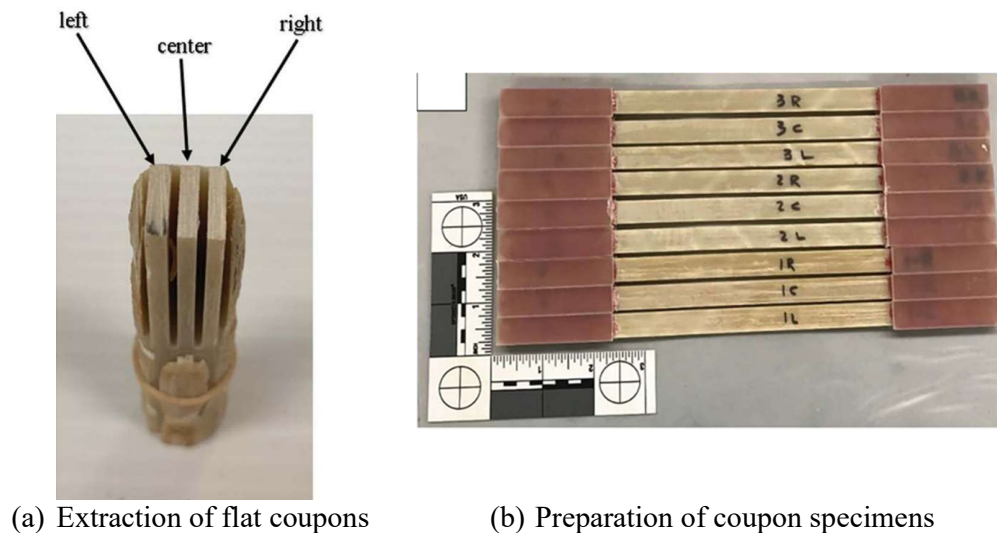


Figure 2.16. Modified tensile test method (Al-Khafaji *et al.* 2021)

2.4.3.2. Concrete-wrapped FRP bars at ambient conditions

To generate more valuable long-term data on the residual mechanical properties of FRP bars in concrete structures, several studies have performed field exposure tests on bars cast in concrete members and subjected to natural environments and/or sustained loading (Bakis *et*

al. 2005; Mukherjee and Arwikaar 2005; Davalos *et al.* 2011; Trejo *et al.* 2011; Almusallam *et al.* 2012; Zhu *et al.* 2014; Morales *et al.* 2021; Feng *et al.* 2023). The relevant information on specimens and test results of these studies are summarized in Table 2.8.

For the time being, the exposure time of test programs in Table 2.8 is much shorter than that of in-service specimens from existing concrete structures, with a maximum conditioning period of 7 years in Trejo *et al.*'s study (2011). The concrete-wrapped FRP bars (see Figure 2.17 (a)) were conditioned to various environments in temperate and tropical regions, encompassing a wide range of MATs varying from 9 °C to 34 °C and RH in the range of 26% to 100%. Most previous studies exposed specimens in the open air (indoor or outdoor exposure), while only two examined the bars subjected to tap water and seawater immersion. To simulate the stress level that the embedded FRP bars will be seen in service, some concrete specimens reinforced with FRP bars on the tension side were applied with sustained load, as shown in Figure 2.17 (b). In two of the collected experimental studies (Trejo *et al.* 2011; Mukherjee and Arwikaar 2005), bending cracks were introduced in concrete specimens, exposing the internal bars to the environment, and were kept open by the sustained load during conditioning. Note that only two studies were undertaken to assess the in-field durability of FRP bars in SW/SSC other than CC.

As shown in Table 2.8, limited tests have evaluated the residual horizontal shear strength of concrete-wrapped FRP bars after field exposure. Feng *et al.* (2023) and Morales *et al.* (2021) concluded that the FRP bars embedded in SW/SSC did not show significant degradation in horizontal shear strength when subjected to hot and wet weather and seawater immersion. The maximum strength reduction was 14% for the BFRP bars in an aggressive environment, which is embedded in SWSSC and continuously immersed in the coastal area.

For tensile durability in most cases, the concrete-wrapped FRP bars retained a large proportion of their tensile strength after exposure to open air and tap water, even when a sustained load lower than the creep-rupture stress limit of GFRP bars given in ACI 440.1R (ACI 2015) was applied. For instance, Davalos *et al.* (2011) reported a reduction in tensile strength of less than 10% of GFRP bars after 150 days of exposure to open air and tap water. Similarly, a marginal change (less than 2%) in tensile strength was observed for the concrete-wrapped bars stored outdoors in Saudi Arabia for 1.5 years (Almusallam *et al.* 2012). In addition, previous test results suggested that the sustained load at a low level had minimal

impact on the degradation of FRP bars, without severe tensile strength reduction in Bakis *et al.*'s (2005) and Davalos *et al.*'s (2011) research.

However, for the bars in pre-cracked concrete members with sustained load, the reported losses of tensile strength of GFRP bars conditioned in the open air were up to 39% and 26%, respectively, in Mukherjee and Arwika (2005) and Trejo *et al.*'s research (2011). In addition to the influence of exposing the bars to the environment by open cracks, the high sustained load appears to mainly contribute to the significant degradation of tensile strength. As reported in previous studies, the sustained stress of these bars was in the range of 20% to 29% of the mean ultimate strength of FRP bars, which exceeded the creep-rupture stress limit of first-generation GFRP bars (ACI 2015) and thus induced certain damage in the bars. The stress-induced damage consequently accelerated the degradation of FRP bars. However, due to the improved technology with respect to fibers, resins, and manufacturing processes in the recent decade, ACI 440.11 (ACI 2022) has recommended a higher creep-rupture stress limit of GFRP bars based on recent test results (Keller *et al.* 2017; Sayed-Ahmed *et al.* 2017; Benmokrane *et al.* 2019). Consequently, a better performance can be expected for the FRP bars produced in recent years, which needs further investigation as more long-term test data becomes available.

In all, limited data are available for both the long-term tensile and shear performance of new-generation FRP bars in concrete after field exposure. Additional information on the durability of FRP bars in structural members using new concrete types (e.g., SW/SSC and UHSC) when exposed to various conditions is needed to implement the potential benefits of using FRP bars in concrete structures in harsh environments.



(a) Concrete-wrapped FRP bars



(b) Reinforced concrete beam with sustained load

Figure 2.17. Concrete-wrapped specimens (Davalos *et al.* 2011)

Table 2.7. Summary of in-service durability GFRP bars in existing concrete structures

Ref.	Surface type	Bar diameter (mm)	Location	In-field condition		Exposure time (year)	Physicochemical test	Strength retention (%)
				MAT	RH			
Al-Khafaji <i>et al.</i> 2021	HWSC	19	Thayer Road Bridge, Indiana, USA	12 °C	79%	15-20	Minimal damage to the microstructure, and no leaching of fibers	73 ⁺
	HWSC		Gill's Creek Bridge, Virginia, USA	16 °C	74%	15-20		
	Sp		Roger's Creek Bridge, Kentucky, USA	14 °C	78%	15-20		
	Sp		McKinleyville Bridges, West Virginia, USA	16 °C	79%	15-20		
	HWSC	19	Cuyahoga County Bridge, Ohio, USA	11 °C	81%	15-20		
	Rb		Bettendorf Bridge, Iowa, USA	11 °C	76%	15-20		
	HWSC	19-22	O'Fallon Bridge, Colorado, USA	3.5 °C	66%	15-20		
	HWSC	19	Southview Bridge, Missouri, USA	15 °C	75%	15-20		
	Rb		Salem Ave. Bridge, Ohio, USA	11 °C	81%	15-20		
	HWSC	6	Walker Box Culvert Bridge, Missouri, USA	15 °C	75%	15-20		
	HWSC	16-19	Sierrita de la Cruz Creek Bridge, Texas, USA	15 °C	75%	17		76 ⁺ -93 ⁺ , 97 [*]
Benmokrane <i>et al.</i> 2018	SC	16-19	Val-Alain Bridge, Quebec, Canada	5.3 °C	84%	11	No changes in the physicochemical properties as well as microstructure	84 ⁺ - 89 ⁺
Gooranorimi and Nanni 2017	HWSC	16-19	Sierrita de la Cruz Creek Bridge, Texas, USA	20 °C	54%	15	No changes in the physicochemical properties as well as microstructure	77 ⁺ -104 ⁺
Mufti <i>et al.</i> 2007			Hall's Harbor Wharf, Nova Scotia, Canada	7.6 °C	81%	5	No changes in the physicochemical properties as well as microstructure	
			Joffre Bridge, Sherbrooke, Canada	4.1 °C	84%	7		
			Chatham Bridge, Chatham, Canada	4.6 °C	81%	8		
			Crowchild Trail Bridge, Calgary, Canada	3.9 °C	79%	8		
			Waterloo Creek Bridge, Vancouver Island, Canada	9.9 °C	85%	6		

Note: XX* is the tensile strength retention; XX⁺ is the horizontal shear strength retention. The mean annual temperature and relative humidity data were collected from an online database (<https://weatherandclimate.com/>).

Table 2.8. Summary of long-term performance of concrete-wrapped FRP bars exposed to field conditions

Ref.	Bar type	Surface type	d (mm)	Concrete type	Location	Exposure condition			Exposure time (year)	Retention (%)	
						MAT	RH	Stress level		TS	ILSS
Feng <i>et al.</i> 2023	B/E	Rb	8	SWSSC1 SWSSC2	Submerged in the coastal area, Qingdao, China	10 °C	100%		1/2		86 97
Morales <i>et al.</i> 2021	ECR/V	HW	19	SWC1 SWC2	Open air, subtropical region of South Florida, USA	25 °C	71%		1/12-2	82-89 97-100	101-106 97-101
Zhu <i>et al.</i> 2014	B/E	Rb	6	CC	Open air of indoor laboratory, Jiangsu, China	17 °C	73%		1/12-2/3	96-99	
Almusallam <i>et al.</i> 2012	E/V	In	19	CC	Outdoor exposure, Riyadh area, Saudi Arabia	34 °C	26%		1/2-3/2	98-99	
					Outdoor exposure, Gulf area, Saudi Arabia	27 °C	52%		1/2-3/2	98	
Trejo 2011	E/V	HWSC	16-19	CC	Outdoor exposure, Texas, USA	23 °C	68%	PC +	7		79-88
	E/V	SC						20% -			74-79
	E/P	Rb						29% f_u			87-93
Davalos <i>et al.</i> 2011											100
	E/V	HWSC	10	CC	Open air of indoor laboratory, Morgantown, USA	16 °C	79%	11% - 14% f_u	5/12	98	
	E/V	SC								100	
										90	
	E/V	HWSC						11% - 14% f_u		81	
			10	CC	Tap water immersion at the ambient condition	16 °C	100%		5/12	90	
Bakis <i>et al.</i> 2005								12 - 15% f_u		92	
	E/V ^B	HWSC	10	CC	Open air of indoor laboratory, Pennsylvania, USA	23 °C	60%	12% f_u	1/2-3	88-92	
					Outdoor exposure, Pennsylvania, USA	9 °C	63%	12% f_u	1-3	96-100	
Mukherjee and Arwkar 2005	E/V	HWSC	10	CC	Open air, Mumbai, India	30 °C	71%	PC + 20% f_u	3/2-5/2	61-65	

Note: B is basalt fiber; ECR: is ECR-glass fiber; E; E-glass fiber; E is epoxy; V is vinyl ester; P is polyester; PC is pre-cracking; f_u is the mean ultimate tensile strength of FRP bars. The mean annual temperature and relative humidity data were collected from an online database (<https://weatherandclimate.com/>).

2.5. Prediction and design for durability of FRP bars

2.5.1. Prediction of durability of FRP bars

2.5.1.1. Long-term prediction using Arrhenius relation

The long-term prediction based on the short-term data from the accelerated aging test using the Arrhenius relation is considered as a time- and labor-saving method, which has been widely used by many researchers (Chen *et al.* 2006; Davalos *et al.* 2011; Dong *et al.* 2017; Wang *et al.* 2017a; Benmokrane *et al.* 2020; Hosseini *et al.* 2024). The accelerated aging test that uses elevated temperatures as an accelerator is favored for its ability to generate sufficient data in a short space of time. In the Arrhenius relation, the degradation is assumed to be dominated by only one single mechanism, which will not change with aging time and temperature. However, the degradation rate can be accelerated with elevated temperature as presented in Eq. (2.15)

$$k = A \cdot e^{\frac{-E_a}{RT}} \quad (2.15)$$

where k is the degradation rate; A is a constant related to the material and degradation process; E_a is the activation energy of the chemical reaction; R is the universal gas constant; and T is the temperature in Kelvin. According to Eq. (2.15), the time-shift factor (TSF) used to project the time required in the accelerated aging condition (t_0) to the required time under the service exposure (t_1) for a given strength retention is expressed as follows

$$TSF = \frac{t_0}{t_1} = \frac{k_1}{k_0} = \frac{A \cdot e^{\frac{-E_a}{RT_1}}}{A \cdot e^{\frac{-E_a}{RT_0}}} = e^{\frac{E_a}{R}(\frac{1}{T_0} - \frac{1}{T_1})} \quad (2.16)$$

where T_0 and T_1 are temperatures of short-term accelerated aging condition and service condition, respectively, and k_0 and k_1 are degradation rates at temperature T_0 and T_1 , respectively. The detailed calculation procedure can be found in Bank *et al.* (2003). After the prediction results are determined through the Arrhenius relation, Dong *et al.* (2016) and Huang and Aboutaha (2010) suggested that the long-term performance of FRP bars in concrete structures should be refined with correction factors accounting for the effects of

exposed RH, temperature fluctuation, and concrete wrap.

2.5.1.2 Degradation laws

To describe the relationship between strength retention and exposure time under different accelerated aging conditions, in general, three different types of degradation models have been adopted by researchers (Bank *et al.* 2003; Katsuki and Uomoto 1995; Davalos *et al.* 2011; Wang *et al.* 2017a).

The first type of model was proposed by Bank *et al.* (2003), which adopted a linear correlation between the property retention and the logarithmic aging time, as shown in Figure 2.18. The expression of the logarithmic model can be written as follows

$$Y = a \log(t) + b \quad (2.17)$$

where Y is the strength retention (%); a and b represent the regression constants; and t is the exposure time. This model has been widely used due to its simple form, but it is only a phenomenological representation of test data and does not hypothesize on the degradation mechanism. Moreover, according to the expression in Eq. (2.17), the infinite residual strength will be observed at time zero, contradicting the experimental observations. Some studies found that the fitted Arrhenius plots based on the logarithmic model violated the basic assumption in the Arrhenius theory of maintaining a consistent single degradation mechanism.

The second type of model is proposed based on the exponential function, with the degradation mechanism assumed to be debonding at the fiber/matrix interface (Phani and Bose 1986, 1987). The two predicted equations in this model can be described as follows

$$Y = 100 \exp\left(-\frac{t}{\tau}\right) \quad (2.18)$$

$$Y = (100 - Y_{\infty}) \exp\left(-\frac{t}{\tau}\right) + Y_{\infty} \quad (2.19)$$

where τ is the fitted parameter related to the degradation rate and Y_{∞} is the residual strength retention at exposure time of infinity. As shown in Figure 2.19, the model considers that the

degradation rate decreases with exposure time and approaches zero and Y_{∞} in Eq. (2.18) and Eq. (2.19), respectively.

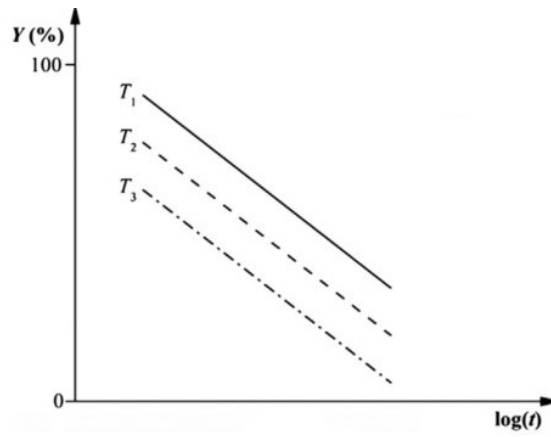


Figure 2.18. Schematic of the logarithmic model (Wang *et al.* 2017a)

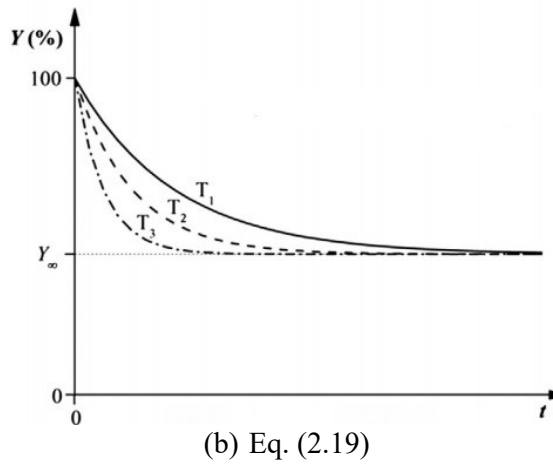
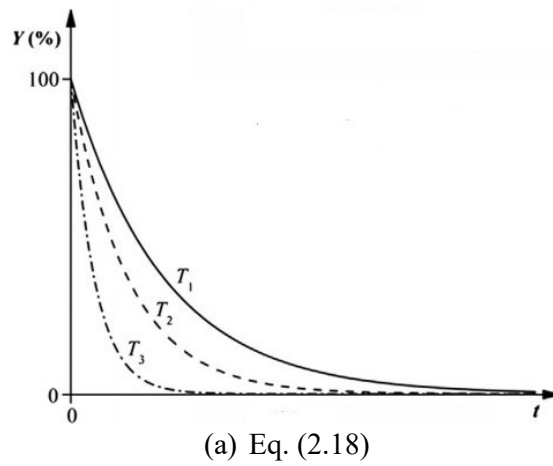


Figure 2.19. Schematic of the exponential model (Wang *et al.* 2017a)

Katsuki and Uomoto (1995) assumed that the reduction in capacity of FRP bars is proportional to the affected cross-sectional area after conditioning and thus developed a degradation model based on Fick's diffusion law in Eq. (2.20) to relate the strength retention with the moisture absorption of bars.

$$Y = 100 \left(1 - \frac{\sqrt{2DCt}}{r_0} \right)^2 \quad (2.20)$$

where D is the diffusion coefficient; C is the concentration of solution; and r_0 is the initial radius of the bar cross-section. Since the FRP bars do not degrade instantaneously upon contact with a solution, Aguioniga *et al.* (2004) claimed that Eq. (2.20) yielded relatively conservative prediction results. Consequently, Aguioniga *et al.* (2004) introduced a time correction component λ in the diffusion-based model as follows

$$\lambda = f(c, t) \quad (2.21a)$$

$$Y = 100 \left(1 - \frac{\sqrt{2D\lambda t}}{r_0} \right)^2 \quad (2.21b)$$

where λ can be determined by regression.

2.5.2. Design for durability of FRP bars

Currently, several design guidelines have recommended environmental reduction factors (ERF) accounting for the effects of long-term exposure to the environment on durability-related strength of FRP bars. Table 2.9 summarizes the ERFs of GFRP bars used as internal reinforcement due to environmental actions in existing guidelines (ACI 2022; CSA 2012; JSCE 2007; Norway Standard 1998). It can be noted that the ERF values of different guidelines vary within large limits. Additionally, the ERF is stipulated as a single value in these guidelines without considering the effects of moisture, alkalinity, and temperature, which have been identified in the literature (fib 2007) as being significant to the durability of FRP bars. Therefore, these guidelines may result in a conservative or aggressive design depending on the exposure conditions of FRP bars.

Table 2.9. Environmental reduction factors in the existing design guidelines

	ACI 440.11-22	CSA S806-12	JSCE	NS3473
ERF	0.85	0.75	0.77	0.50

To address these issues, fib (2007) proposed a refined approach for durability specification for FRP bars, which is conducive to producing more economic and safer results if the field exposure conditions are available. The design strength equation for FRP bars multiplies the characteristic strength by an environmental factor quantifying the effects of exposed RH, MAT, and service life, as follows

$$f_{fkd} = f_{fk1,000h} / [(100 - R_{10})/100]^n / \gamma_f \quad (2.22a)$$

$$n = n_{mo} + n_T + n_{SL} + n_d \quad (2.22b)$$

where f_{fkd} = long-term design value of tensile strength; $f_{fk1,000h}$ = characteristic value of tensile strength after 1000 h exposure test; R_{10} = standard reduction of tensile strength in percent per decade; γ_f = partial safety factor for FRP; n = sum of influencing terms, including n_{mo} for moisture condition, n_T for exposed temperature, n_{SL} for desired service life, and n_d for diameter correction. In this equation, the logarithmic property retention and exposure time in logarithmic scale are linearly dependent, as depicted in Figure 2.20. Obviously, it is a phenomenological model that is able to represent part of the test data. Previous research demonstrated that the calculated strength retention at the primary exposure period was significantly higher than 100% based on the R_{10} determined from the short-term data (Li *et al.* 2023; Zhou *et al.* 2024).

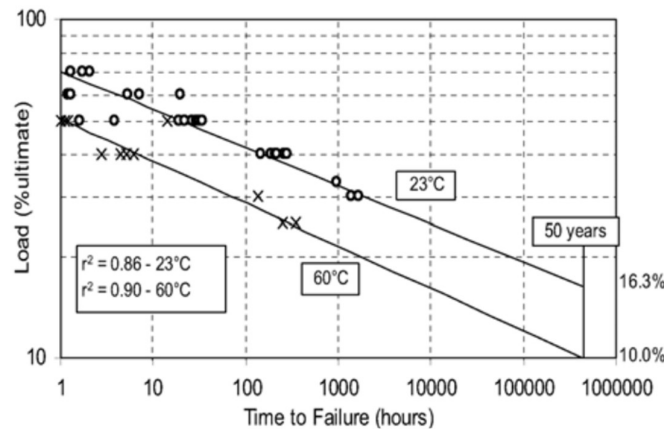


Figure 2.20. Strength retention curves in double log scale (fib 2007)

2.6. Conclusions

This chapter provides a comprehensive review of existing studies on the short- and long-term performance of FRP in concrete environments, with primary concerns on the bond, tensile, and horizontal properties of FRP bars. Against this review, several research gaps have been identified as follows

- (1) The critical factors that influence the bond behavior of FRP bars in UHSC have not been systematically investigated. The quantitative effects of the properties of FRP bars, especially the horizontal shear capacity of FRP bars, on the bond strength in UHSC need to be experimentally determined.
- (2) This review suggests that there appear to be 2-way interaction effects between every two aforementioned factors, namely, concrete strength, surface configuration, and bar diameter. Moreover, the effect of fiber type seems to interact with that of concrete strength. These potential interaction effects among several key factors need further investigation.
- (3) There is currently no bond strength model applicable to all concrete strengths that can consider the possible interaction effects of the critical factors.
- (4) No research has focused on the durability of UHSSC-wrapped FRP bars in coastal and marine conditions, including the long-term tensile, shear, and bond performance.
- (5) The relationship between the material degradation of FRP bars and the changes in bond strength has not been quantitatively studied.

CHAPTER 3. BOND BEHAVIOR OF FRP BARS IN UHSSC

3.1. Introduction

Recent research has indicated that the bond mechanism of FRP bars differs between NSC and HSC members. Lee *et al.* (2008, 2017) revealed that in HSC, bonded joints tend to fail by peeling off the surface layer of FRP bars, rather than by crushing of the surrounding concrete commonly observed in NSC. Additionally, the peeling off area ratio of the FRP bar surface layer increases with the concrete strength. Similar experimental observations have been reported by other researchers (Soares *et al.* 2020; Solyom and Balázs 2020; Peng *et al.* 2022). In UHSC, many researchers have found that the major failure occurs inside the FRP bar core, with some cases showing that fibers near the surface of the bar are chopped (Firas *et al.* 2011; Yoo *et al.* 2015, 2017; Liao *et al.* 2021; Kim *et al.* 2022; Zeng *et al.* 2022; Liang *et al.* 2023; Hu *et al.* 2024; Jiang *et al.* 2024). Consequently, it is reasonably expected that the shear properties of the FRP bars, particularly of the surface layer, significantly influence the bond performance in UHSC. To the best of the authors' knowledge, the influence of mechanical properties of FRP bars on their bond behavior in UHSC has not been quantitatively examined.

In contrast to the steel reinforcement, the surface configuration of FRP bars, which is associated with different bond mechanisms, significantly affects bond performance (Cosenza *et al.* 1997; Hao *et al.* 2009; El-Nemr 2016; Pour *et al.* 2016; Yan *et al.* 2016; Nepomuceno *et al.* 2021). However, the contributions of different surface types remain inconclusive. Some research suggested that deformed FRP bars with outer coating sands achieve favorable bond performance (Firas *et al.* 2011; Solyom and Balázs 2020; Lu *et al.* 2021; Nepomuceno *et al.* 2021). Conversely, other studies advocated for ribbed bars and helically wrapped bars over sand-coated ones for superior bond strength (Mazaheripour *et al.* 2013; El-Nemr *et al.* 2016; Basaran and Kalkan 2020a; Yoo *et al.* 2023). In addition, other studies have shown that bond strength can significantly vary within the same surface type (Achillides and Pilakoutas 2004; Hao *et al.* 2009; Arias *et al.* 2012; Lämmlein *et al.* 2017; Henin *et al.* 2019; Basaran and Kalkan 2020a; Solyom and Balázs 2020; Nepomuceno *et al.* 2021). Their experimental results showed that the bond strength of FRP bars is influenced not only by the surface type but also by the detailed physical variations of the bar surface, including the size and quantity

of sand adhered on the bar surface, the geometry of deformation, and the quality of the coating process. As a result, Solyom and Balázs (2021) suggested that the effect of bar surface should not be considered as a stand-alone coefficient, as stipulated in Canadian standard (CSA 2012). In a recent study, Henin and Morcoux (2021) evaluated the bond behavior variability among FRP bars from the same producer but different production lots, highlighting the statistical significance of production variability, particularly in surface conditions. This underscores the need for comprehensive research to elucidate the effect of surface properties and their variations on bond behavior, typically associated with the fabrication technology and material properties of specific types of FRP bars.

As reviewed in Chapter 2, there are currently no design equations specifically for the bond strength and development length of FRP bars in UHSC. Although numerous studies have developed predictive equations for the bond strength of FRP bars in UHSC, no existing models have considered the effect of the shear properties of FRP bars, except for Liang *et al.*'s model (2023) who were the first to incorporate the adhesion strength between the resin layer and core fiber in a frictional model to represent the complete peeling-off failure in UHSC. Nonetheless, for UHSC applications where FRP bars failed due to the delamination between the surficial resin layer and core fiber, a bond strength model requires extensive investigation into the specific FRP bar type, which may vary in horizontal shear strength. Consequently, these equations are limited to specific FRP bar types due to limited experimental studies of FRP bars in UHSC in each dataset.

This chapter conducts a comprehensive investigation on the bond performance of FRP bar-UHSSC structural members, aiming to develop a reliable predictive equation. For this purpose, a series of direct pullout tests are performed on five types of FRP bars embedded in UHSSC. Comparative tests are also conducted on these bars when embedded in HSC. In addition to concrete strength, bar diameter, and fiber type, this thesis aimed to examine the effects of horizontal shear properties and surface topography of FRP bars on bond performance in UHSSC, which were not quantitatively evaluated in previous research. Accordingly, a total of 107 pullout specimens, encompassing four concrete strength grades (denoted as S60, S120, S140, and S170), and FRP bars with four bar diameters ($\Phi 6$, $\Phi 10$, $\Phi 13$, and $\Phi 16$), two fiber types (glass fiber and carbon fiber), two surface types (helically wrapped with sand coating and sand-coated), and two resin systems (epoxy and vinyl ester), are tested. Additionally, topographical measurements of the bar surface and short-beam shear

tests are conducted to examine the physical surface geometries and shear properties of the FRP bars. In short, the chapter is divided into three main parts. The first two parts present the results of the measurement of surface topography and short-beam shear tests, respectively. Then, an analysis of the effects of several parameters, including horizontal shear performance and surface topography of FRP bars discussed in the previous two sections, on the FRP-UHSSC bond performance is carried out.

3.2. Experimental materials and methods

3.2.1. Materials

3.2.1.1. FRP bars

As given in Figure 3.1, five types of commercially available pultruded FRP bars from three different producers, denoted as P1 (Dextra Building Products Co., Ltd, Guangzhou, China), P2 (Pulwell Composites Co., Ltd, Zhongshan, China), and P3 (Pultrall Inc., Thetford Mines, Canada), were used in this research. The surface characteristics and properties are summarized in Table 3.1. FRP bars from P1 and P2 are categorized as the same surface type, designated as HWSC bars. This surface configuration is achieved by wrapping the bars with a resin-impregnated spiral prior to the hardening of the matrix, followed by an application of a sand particle coating. Notably, there exists a variance in the quality of sand coating between bars from P1 and P2, which is discussed in Section 3.3.1. Conversely, the bars from P3, which lack surface undulations, are exclusively treated with sand blasting to achieve a roughened texture. The FRP bars from P3 are thus categorized as SC bars. Apart from the surface type, these FRP bars were different in fiber type and resin system. Both GFRP (G1) and CFRP (C1) bars from P1 adopted the same vinyl ester resin (vinyl ester 1 in Table 3.1), whereas P2 employs distinct vinyl ester (vinyl ester 2) and epoxy resin for its GFRP (G2) and CFRP (C2) bars, respectively. The GFRP (G3) bars from P3 were pultruded with a unique vinyl ester system (named vinyl ester 3), differentiating from those used in P1 and P2. Note that, as reported by the producer, the GFRP bar and CFRP bar from the same producer adopt identical manufacturing and coating processes.

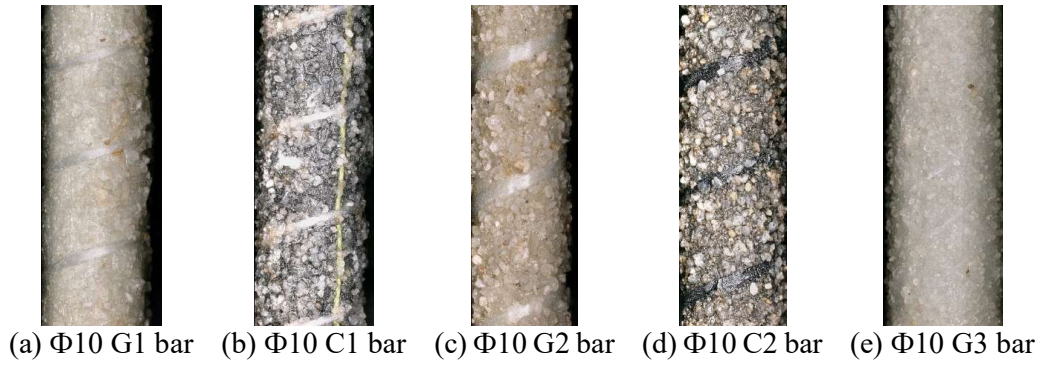


Figure 3.1. Five types of FRP bars

3.2.1.2. Concrete

The UHSSC mixture developed by Teng *et al.* (2019) was adopted in this study. For comparison, HSSC was also used to cover G1 bars for the pullout test and short-beam shear test. The mix proportions of the UHSSC and HSSC mixtures, respectively, are provided in Table 3.2. All concrete mixtures were prepared without coarse aggregate and were mixed with non-desalted sea-sands with particle sizes smaller than 1.18 mm. The particle size distribution of the sea-sands is presented in Figure 3.2, with a fineness modulus of 1.99, according to JGJ 52-2006 (2006). The chemical compositions of non-desalted sea-sand were analyzed using Ion Chromatography (IC) tests, and the slit and shell contents were measured per JGJ 52-2006 (2006). The results are shown in Table 3.3. Notably, the sea-sand used in this study had comparable chloride (Cl^-) and slit contents but a lower shell content with dredged marine sand collected from the mouth of the Pearl River (Liu *et al.* 2016). The lower content of the shell is probably due to the elimination of sand particles larger than 1.18 mm in this study. The two concrete mixtures were prepared using a drum mixer in the laboratory. All concrete specimens were cured in a moist condition at ambient temperature (around 23°C) for 28 days. The average 28-day compressive strength of the concrete was determined according to ASTM C1856/C1856M-17 (2017), as given in Table 3.2. The concrete with cylindrical compressive strength of over 120 MPa is defined as UHSC, according to ASTM C1856/C1856M-17 (2017). Note that the compressive strength of concrete is referred to as concrete strength for short in this thesis.

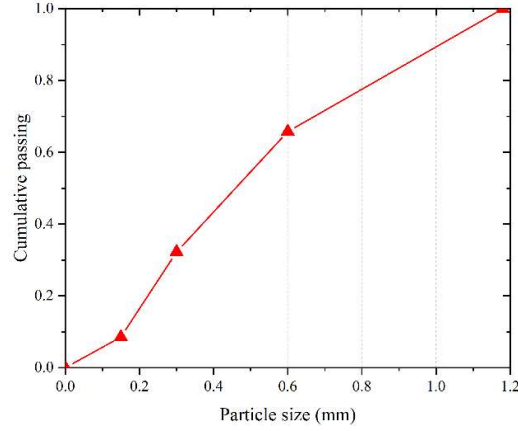


Figure 3.2. Particle size distributions of sea-sands

3.2.2. Experimental methods

To evaluate the influences of horizontal shear properties and surface topography on the bond performance of FRP bars in UHSSC, two companion tests, including topographical measurements of bar surface and short-beam shear tests, were first conducted to examine the physical surface geometries and shear properties of FRP bars. Then, the direct pullout tests were carried out to assess the effects of the aforementioned two parameters, bar diameter, fiber type, and concrete strength.

3.2.2.1. Measurement of surface topography

To evaluate the surface condition and quantify the topographical irregularity resulting from the surface finishing process, a three-dimensional (3D) digital microscope VHX-6000 was used to picture the surface profile of FRP bars (see Figure 3.3). In total, the 3D measurement was performed on nine batches of FRP bars with five different bar types and four different bar diameters. For each type of FRP bar, eight bars were mapped. By ISO4287-1997 (ISO 1997), the profile height along the longitudinal bar axis was measured over a 30 mm evaluation length. This length is considered representative, covering at least twice the clear spacing between two adjacent wrapped spirals on deformed bars. A 30x optical zoom lens was employed, with a sample interval of 6.66 μm .

The arithmetic mean deviation of the surface profile from the mean line, R_a , quantified by Eq. (3.1) is treated as the surface roughness

$$R_a = \frac{1}{l_r} \int_0^{l_r} |Z(x)| dx \quad (3.1)$$

where $Z(x)$ is the height of the surface profile deviating from the mean line, and l_r is the evaluation length.

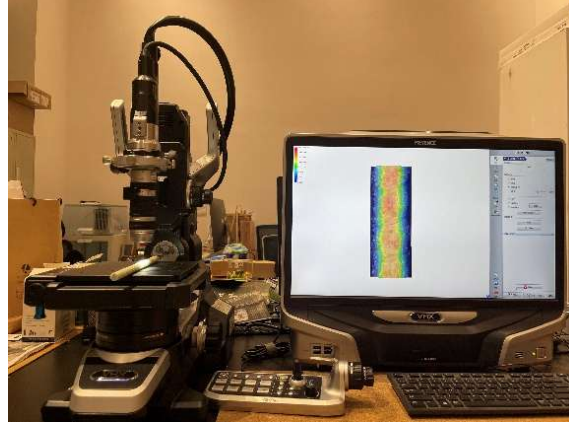


Figure 3.3. Measurement of surface topography

3.2.2.2. Short-beam shear test

The hypothesis that the ILSS of FRP bars is correlated with the bond performance of the FRP-UHSC interface was investigated through the short-beam tests, according to ASTM D4475-21 (2021). The three-point bending set-up for the short-beam test was conducted using a Wance testing machine (Shenzhen, China) with a 50 kN capacity, as given in Figure 3.4. Based on the preliminary tests, the support span-to-bar diameter ratio of 5.0 to 1.0 was determined to obtain the desired mid-plane horizontal shear fracture. The loading rate was 1.2 mm/min.

The interlaminar shear strength S of each bar type can be calculated as follows

$$S = 0.849 \frac{P}{d^2} \quad (3.2)$$

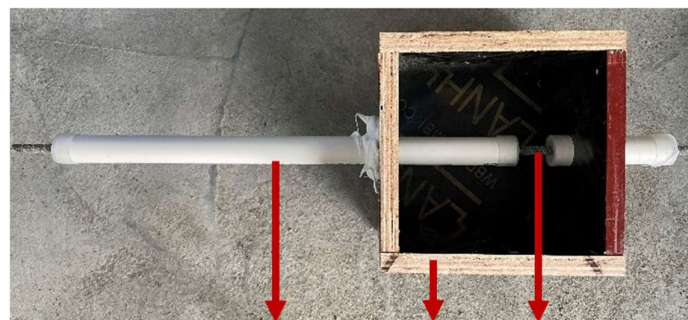
where P is the peak load of the short-beam shear test.



Figure 3.4.Set-up for the short-beam test

3.2.2.3. Pullout test and specimen preparation

The pullout tests were carried out according to CSA S806-12 (CSA 2012) and ASTM D7913/D7913M-14 (ASTM Committee 2020). For these tests, concrete cubes measuring $150 \times 150 \times 150 \text{ mm}^3$ were utilized, with a bond length of four times the bar diameter ($L=4d$). The FRP bars were concentrically positioned in the cubic mold, as shown in Figure 3.5. To prevent bonding beyond the designated bond length, a PVC tube was installed near the loaded end as a bond breaker, encasing the FRP bar outside the bond zone. Preliminary tests showed that, due to the considerable pullout load, both 16-mm-diameter G1 bars and the 10-mm-diameter G3 bars failed prematurely by concrete splitting when embedded in UHSSC prisms with the sizing $150 \times 150 \times 150 \text{ mm}^3$. The UHSSC prism size for those FRP bars was consequently increased to 200 mm to ensure the pullout failure.



PVC tube Wood mold Bond length
Figure 3.5.Specimen preparation

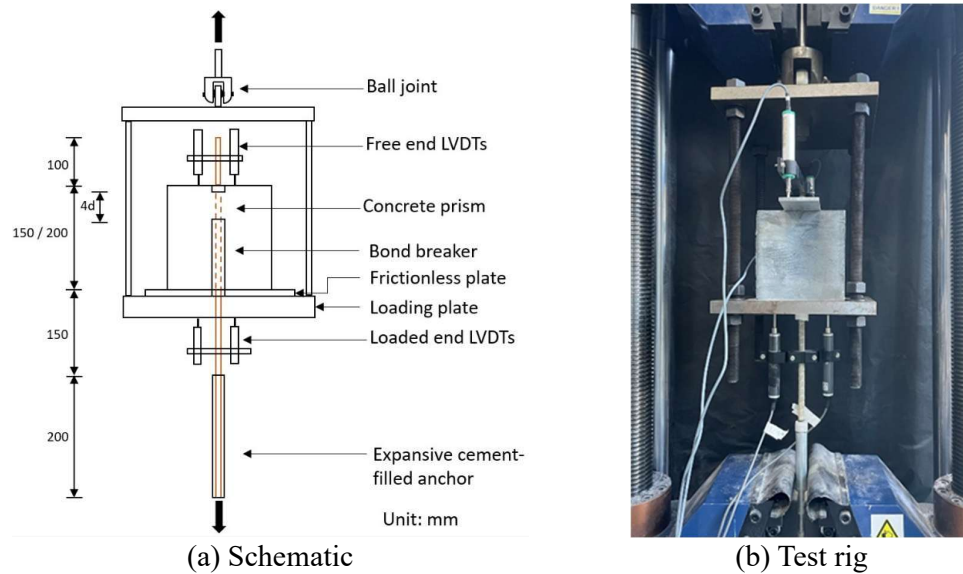


Figure 3.6. Pullout test arrangement

The pullout test fixture can be found in Figure 3.6. In addition to the loading frame, the test fixture comprised two free-end LVDTs and two loaded-end LVDTs to measure the relative free-end slip and loaded-end deformation. Moreover, a 0.1 mm-thick frictionless plate was placed between the concrete prism and reaction plate to eliminate the lateral confinement caused by friction. The pullout tests were performed using a Matest Model H001A universal testing machine with a loading rate of 1.2 mm/min.

In general, the test matrix is divided into four groups, each having three to four identical specimens. The first two groups aimed to analyze the influence of material properties, including concrete strength and ILSS of FRP bars, on bond behavior. In these groups, five different FRP bars (i.e., G1 and C1, G2 and C2, and G3) with a nominal diameter of 10 mm were tested. Note that the preliminary test results of G2 bars in HSC with an average compressive strength of 60 MPa were also used for analysis. Another group of specimens was used to investigate the effect of bar diameter depending on concrete strength. Accordingly, three nominal diameters of the GFRP and CFRP bars from P1, namely, 10, 13, and 16 mm for the P1 GFRP bar and 6, 10, and 13 mm for the P1 CFRP bar, were used. In addition, among the FRP bars from P1, only G1 and C1 bars of 10 mm and 13 mm in diameter were involved in exploring the effect of fiber type. Finally, the investigation on the effect of surface topography was carried out on all types of FRP bars from three producers. The pullout specimens are designed as a letter-number set. The first letter stands for the bar type (G for

GFRP and C for CFRP), followed by the number representing the corresponding producer (1, 2, 3 for P1, P2, and P3). The last two letter-number combinations denote the concrete strength and bar diameter. Note that the same nomenclature was used in the surface topography measurement and short-beam shear tests.

The average bond stress is evaluated as the local bond stress with a short bond length of 4 times the diameter. The bond stress is given by Eq. (3.3), where F is the pullout force and d_{eff} is the effective diameter according to ASTM D7205/D7205M-06 (2016).

$$\tau = \frac{F}{\pi d_{eff} L_e} \quad (3.3)$$

The loaded-end slip is calculated as the average readings of two loaded-end LVDTs minus the elastic elongation of the FRP bar between the supporting point of LVDTs and the nearest edge of the bond part. The elastic elongation of FRP bars can be obtained using Eq. (3.4).

$$S_c = \frac{F L_c}{E_f A_f} \quad (3.4)$$

where S_c is the elastic elongation in mm, L_c is the length from the supporting point of loaded-end LVDTs to the nearest edge of the bond part of FRP bars, E_f is the modulus of elasticity of FRP bars, and A_f is the nominal cross-sectional area of FRP bars calculated in accordance with ASTM D7205/D7205M-06 (2016).

Table 3.1. The surface characteristics and properties of the FRP bars

Bar type	Producer	Surface treatment	Fiber type	Resin system			Nominal diameter (mm)	Effective diameter (mm)	Nominal cross-sectional Area (mm ²)	Ultimate tensile strength (MPa)	Ultimate tensile strain	Tensile modulus of elasticity (GPa)
				Type	Tensile strength (MPa)	Ultimate Strain						
GFRP	P1	HWSC	ECR-glass	Vinyl ester 1	69-89 ^a	6.0-8.0% ^a	10	10.11	80.3	1208	2.25%	53.7
							13	13.36	140.2	972	1.93%	52.0
							16	17.28	234.5	946	1.83%	54.9
CFRP	P1	HWSC	Carbon fibre	Vinyl ester 1	69-89 ^a	6.0-8.0% ^a	6	6.95	38	2496	1.93%	140.2
							10	10.26	82.7	2285	1.88%	121.8
							13	13.62	145.6	2131	1.72%	126.9
GFRP	P2	HWSC	ECR-glass	Vinyl ester 2	88 ^a	5.0% ^a	10	10.46	86.6	1236	2.20%	56.3
CFRP	P2	HWSC	Carbon fibre	Epoxy	--	--	10	10.46	86.6	2746	1.75%	157.0
GFRP	P3	SC	--	Vinyl ester 3	--	--	10	10.33	83.8	1100	1.83%	60.0

Note: P1, P2, and P3 stand for three different producers, namely Dextra Building Products Co., Ltd, Pulwell Composites Co., Ltd, and Pultrall Inc., respectively; superscript ^a: the information is provided by producers.

Table 3.2. Mix proportions and compressive strength of HSSC and UHSSC (in kg/m³)

Concrete type/mix proportion	OPC	SF	QP	FS	TW	SP (100% solid content)	Compressive strength (MPa)
S170	830	207	207	913	187	13	175.41±1.10
S140	830	207	207	913	228	11	149.30±4.03
S120	830	207	207	913	260	9	127.45±4.75
S60	600	207	0	1314	206	8	68.36±2.57

Note: OPC: ordinary portland cement; SF: silica fume; QP: quartz powder; FS: fine sand; SP: superplasticizer.

Table 3.3. Physical properties of non-desalted sea-sand

Properties	Fineness modulus	Chloride content (wt%)	Sulfate content (wt%)	Silt content (wt%)	Shell content (wt%)
Mean	1.99	0.283	0.050	1.85	1.78

3.3. Experimental investigation

3.3.1. Surface topography of FRP bars

Figure 3.7 and Figure 3.8 present several representative surface profiles and the corresponding surface elevations along the longitudinal axis of $\Phi 10$ G1 bars. It is evident that the surface coating condition varies significantly even among bars of the same type. As shown in Figure 3.7 (c), some FRP bars received a sparser sand coating compared to their counterparts from the same producer in Figure 3.7 (a) and (b). Figure 3.8 further demonstrates that the sparsely coated bar exhibited less pronounced variation in surface elevation than those with dense and moderate coating, resulting in the lowest R_a among the three surface conditions.

Figure 3.9 and Table 3.4 compare the surface roughness of different types of FRP bars. The surface elevations of P3 bars, which were finished with fine sands, were, on average 41%-51% lower than those of P1 and P2 bars. This difference is attributed to the use of coarser sands for coating and the deeper surface deformation resulting from the spiral wrapping technique applied to P1 and P2 bars. As shown in Figure 3.9, P1 bars demonstrated the greatest variation in R_a , with a CoV of approximately 53%, while P2 bars had a lower CoV of around 30%. Notably, the $\Phi 6$ and $\Phi 13$ C1 bars showed the highest CoV, reaching up to 60%. In contrast, P3 bars showed minimal variation among samples (CoV=16%). It is important to note that a higher CoV in surface roughness indicates greater variation in surface condition, and thus, less consistent quality control of surface coating.

These measurement results show that the production variability in the surface condition of FRP bars can differ significantly among producers, probably due to the lack of standardized specifications for surface coating. P1 bars, especially the $\Phi 6$ and $\Phi 13$ C1 bars, exhibited greater variability in surface coating topography compared to bars from the other two producers. The surface variability was notably less pronounced for P2 bars and appeared

negligible for P3 bars. These findings suggest that P1 bars had inferior quality control of surface coating in contrast to P2 and P3 bars, with the $\Phi 6$ and $\Phi 13$ C1 bars being the most inconsistent. The influence of regularity in the surface condition on the bond performance is further discussed in Section 3.3.3.6.



(a) densely sand-coated (b) moderately sand-coated (c) sparsely sand-coated

Figure 3.7. Representative surface conditions of $\Phi 10$ G1 bars

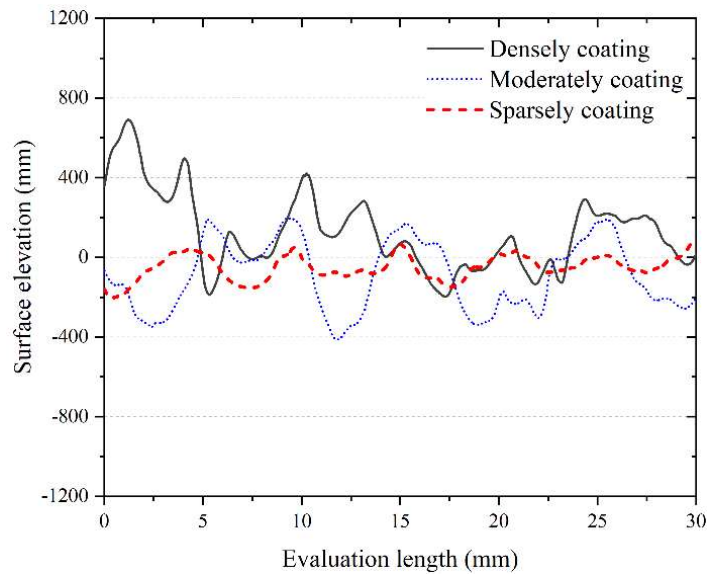


Figure 3.8. Representative surface profiles of $\Phi 10$ G1 bars

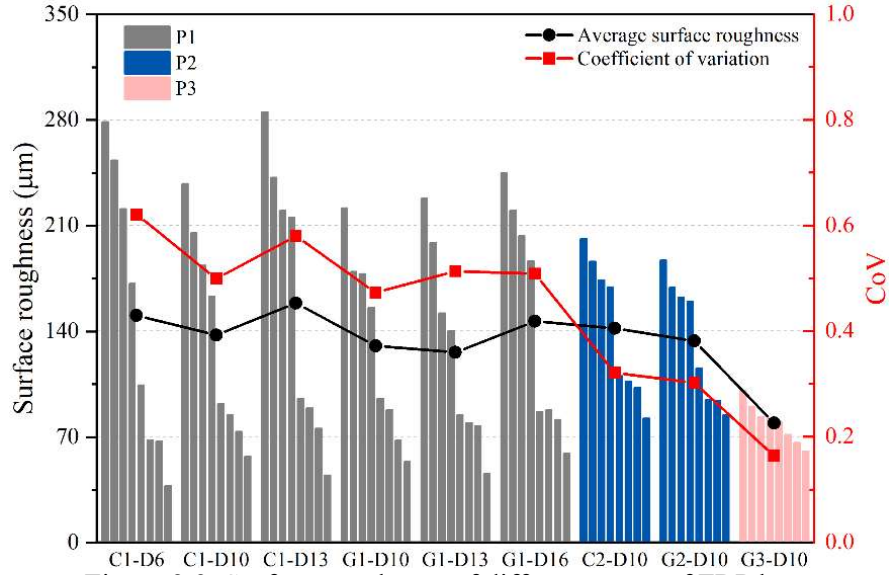


Figure 3.9. Surface roughness of different types of FRP bars

Table 3.4. Summary of the surface topography

Producer	Bar type	Surface roughness R_a		
		Mean (μm)	SD (μm)	CoV
P1	C1-D6	150	93	62.09%
	C1-D10	137	69	49.90%
	C1-D13	158	91	57.68%
	G1-D10	130	62	47.29%
	G1-D13	126	65	51.35%
	G1-D16	146	74	50.87%
P2	C2-D10	142	46	32.12%
	G2-D10	134	40	30.24%
P3	G3-D10	79	13	16.36%

Note: SD: standard deviation; CoV: coefficient of variation.

3.3.2. Horizontal shear performance of FRP bars

The typical failure mode and load-deflection curves are shown in Figure 3.10 and Figure 3.11. All specimens failed by the interlaminar shear, characterized by a major horizontal crack along the mid-plane. As reported in previous studies (Janus *et al.* 2022; Chen *et al.* 2007), FRP bars exhibited two primary failure patterns depending on their type. Most FRP bars experienced a quite brittle failure, marked by a dramatic load drop upon reaching the peak load, accompanied by the sudden formation of a horizontal shear crack on one side. The

failure of P2 (i.e., G2 and C2) and P3 (i.e., G3) FRP bars fell within this pattern, as depicted in Figure 3.11. Conversely, for the P1 (i.e., G1 and C1) bars, the shear failure generally initiated at the midspan and gradually propagated to one side. This ductile failure mode was characterized by a less pronounced decline in the applied force after the peak load. Janus *et al.* (2022) attributed these different failure behaviors to variations in resin type, fiber sizing, and production method.

Table 3.5. Summary of the short-beam test results

Producer	Bar type	Interlaminar shear strength (ILSS)			
		Measured (MPa)	Mean (MPa)	SD (MPa)	CoV
P1	C1-D10	24.43	24.83	1.28	5.17%
		26.58			
		24.78			
		23.52			
	G1-D10	28.91	29.75	0.60	2.01%
		30.33			
		29.89			
		29.88			
	C1-D13	23.35	24.74	1.01	4.06%
		25.23			
		25.66			
		24.70			
	G1-D13	28.69	27.68	0.90	3.24%
		27.33			
		26.62			
		28.07			
P2	C2-D10	44.94	44.25	2.29	5.18%
		40.86			
		45.31			
		45.89			
	G2-D10	31.86	32.71	0.73	2.24%
		33.64			
		32.57			
		32.76			
P3	G3-D10	38.32	39.69	2.21	5.56%
		42.96			
		39.09			
		38.38			

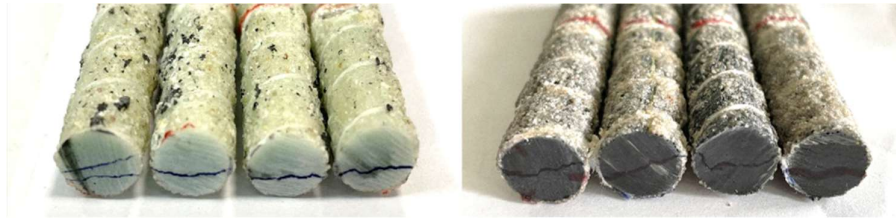


Figure 3.10. Typical failure mode of short-beam tests

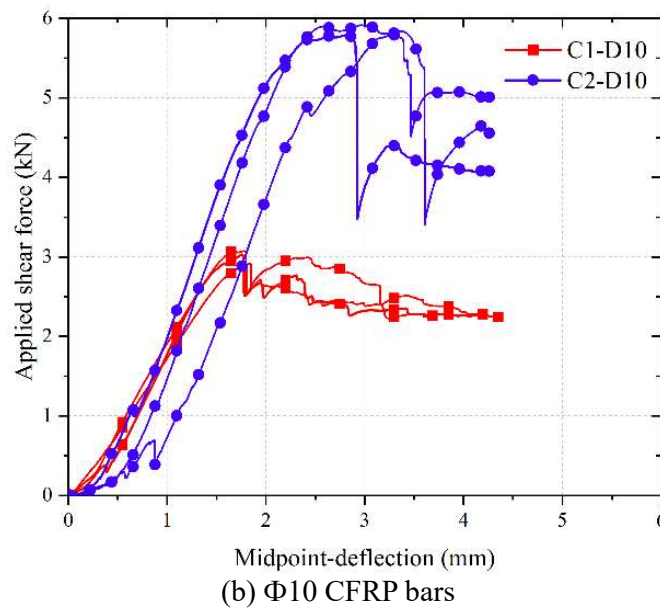
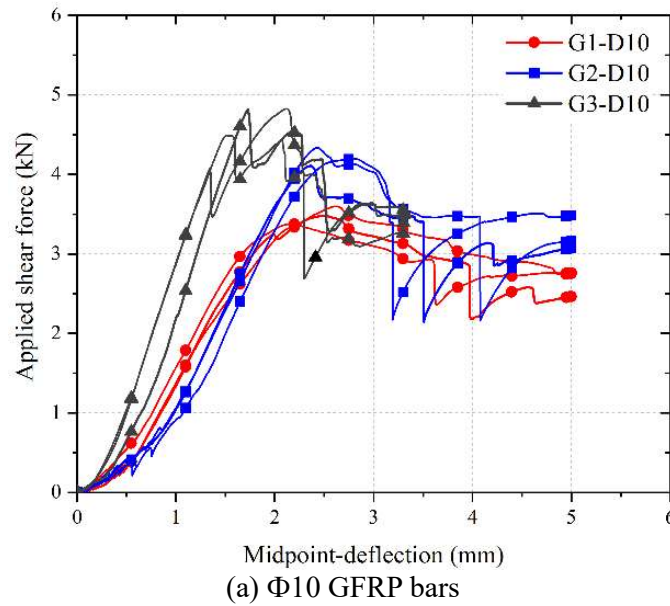


Figure 3.11. Shear force-deflection curves of FRP bars from three producers

Table 3.5 and Figure 3.12 compare the ILSS of the FRP bars. Specifically, P1 bars fabricated using the same vinyl ester resin system demonstrated 11%-17% lower ILSS in C1 bars

compared to G1 bars. This experimental observation was also reported by Benmokrane *et al.* (2020), who attributed the superior glass fiber-resin interface to the better compatibility of glass fibers with the vinyl ester matrix and sizing compared to carbon fibers. Remarkably, the ILSS of C2 bars, pultruded with an epoxy matrix, was 35% greater than that of G2 bars from the same producer and 78% higher than that of C1 bars, due to the superior mechanical properties of the epoxy resin. Among GFRP bars utilizing different vinyl ester resin systems, the G3 bars exhibited the highest ILSS (39.69 ± 2.21 MPa), followed by G2 bars (32.71 ± 0.73 MPa) and G1 bars (29.75 ± 0.60 MPa) with the lowest. This was reported to be attributed to the effect of resin chemistries, proprietary sizing of fiber in the FRP bars (Wang *et al.* 2017; Benmokrane *et al.* 2020; Janus *et al.* 2022). The potential correlation between the shear resistance of FRP bars and their bond performance is discussed in Section 3.3.3.2.

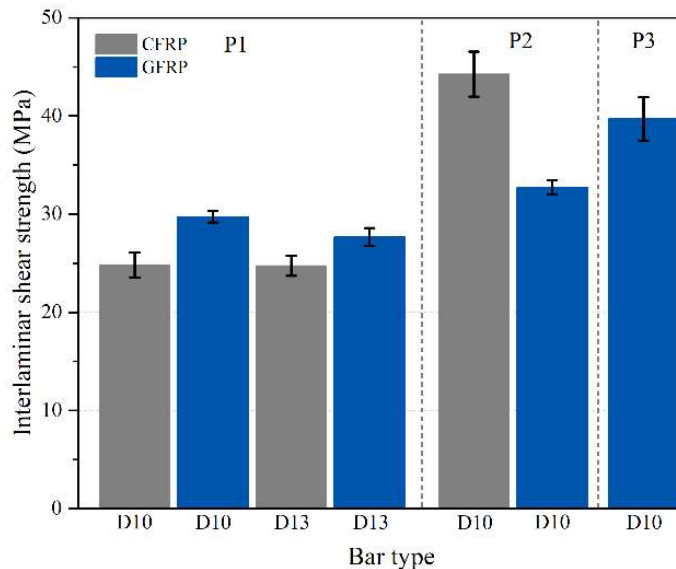


Figure 3.12. Comparison of interlaminar shear strength of different types of FRP bars

3.3.3. Bond behavior of FRP bars in UHSSC

A total of 107 pullout specimens were tested to analyze the influence of concrete strength, size, and properties of FRP bars on bond performance of FRP bars in UHSSC. The experimental results are given in Table 3.6.

3.3.3.1. Failure mode

All specimens failed by pulling out of the FRP bars as expected. Notably, no apparent

splitting cracks were observed in the concrete prisms. After the pullout tests, all concrete blocks were split for visual examination of the debonding surface along the bond length. Figure 3.13 displays several typical debonding surfaces of different types of FRP bars. For HSC (around 60 MPa), as seen in Figure 3.13 (a) and (h), substantial concrete pieces adhered to the bar surface near the loaded end. Additionally, the thin superficial layer of the FRP bar, including coating sands, surficial resins, and winding spirals, was partially peeled off. These observations suggest that the bond failure occurred partially within the concrete matrix and partially at the sand-blasted layer, referred to as Zone 1 in Figure 2.1. Consequently, the shear resistance of the concrete and the surface layer of the FRP bar controlled the bond capacity. This is a common failure mode in both NSC and HSC, as confirmed by many studies (Achillides and Pilakoutas 2004; Lee *et al.* 2008; Kim *et al.* 2013; Tekle *et al.* 2016; Soares *et al.* 2020; Bediwy and El-Salakawy 2021; Chen *et al.* 2023; Yoo *et al.* 2023).

For UHSSC, the FRP bar failed in a different way, as shown in Figure 3.13 (c), (f), and (j). The outer layer of the FRP bar was completely sheared off without apparent damage to the surrounding concrete, resulting in an eliminated surface undulation and texture of the FRP bars. The concrete prism was covered with a quite thick layer of white (for GFRP bars) or black (for CFRP bars) residues of the outer layer of FRP bars along the entire bond length. It is noteworthy that, in addition to the delamination of coating sands and the exterior resin, the rupture of fibers adjacent to the bar core was noticed. These observations indicate that the bond of FRP bars failed primarily between the surficial resin-rich layer and the fiber at the successive core layers (i.e., Zone 2 in Figure 2.1). In this case, the bond strength is governed by the interlaminar shear capacity between the resin and fiber peripheral to the bar core rather than the concrete strength. Therefore, it can be expected that a further increase in concrete strength does not change the failure pattern, as noticed in the S170 specimen set (see Figure 3.13 (d), (g), and (k)). This failure mechanism is consistent with previous research concerning the interaction between UHSC and FRP bars (Firas *et al.* 2011; Yoo *et al.* 2015, 2017; Kim and Wang 2022; Pan *et al.* 2022; Liang *et al.* 2023; Tong *et al.* 2023; Hu *et al.* 2024; Jiang *et al.* 2024).

For the lower concrete strength in the UHSC classes (i.e., the S120 set), bond failure manifested as a transition phase between Zone 1 and Zone 2 in Figure 2.1. As shown in Figure 3.13 (b) and (e), fewer concrete pieces were scratched and clung between the light ribs of P1 and P2 bars compared with the counterparts in the S60 set. In addition, a thinner layer of

attached concrete residues was observed on the P3 bar surface in Figure 3.13 (i) in contrast to the G3-S60 specimens. For the S120 set, the grained layer was also partially peeled off, resulting in a light gray debonding surface on the concrete matrix for the GFRP bars and a light brown-gray debonding surface for the CFRP bars. Therefore, the shear resistance of the concrete matrix contributed less to the bond capacity compared with the shear capacity of the grained layer of the FRP bar. Similar experimental findings have been reported for HSC and UHSC in previous studies (Lee *et al.* 2008; Baena *et al.* 2009; Chen *et al.* 2022; Zeng *et al.* 2022; Liang *et al.* 2023; Yoo *et al.* 2023).

In general, the failure mode changed with concrete strength. The bond failure occurred in the surrounding concrete in S60 specimen sets. As concrete strength increased, the bond tended to fail near the bar-to-concrete interface in S120 specimen sets, and between the surface-grained layer and the subsequent core layer of the FRP bar in UHSC with higher compressive strength. This change in debonding surface is also reported by Lee *et al.* (2008) and Hossain *et al.* (2017). However, it is challenging to draw clear lines for the concrete strength of demarcation between these failure zones. The distinction between these zones is predominantly influenced by the shear capacity of the grained layer and the subsequent core layer of the FRP bar, which depends on the production quality and technology, as well as the resin and fiber characteristics of the FRP bar.

Table 3.6. Summary of the pullout test results

Specimen	Maximum bond stress (MPa)			Peak slip measured at the free end (mm)			Residual bond strength (MPa)		
	Measure d	Mean	SD	Measure d	Mean	SD	Measure d	Mean	SD
C1-S120-D6	19.04			0.242			7.24		
	17.42	18.91	1.43	0.000	0.081	0.140	5.05	6.56	1.31
	20.27			0.000			7.39		
C1-S140-D6	24.92			0.046			14.83		
	20.89	22.21	2.34	0.148	0.077	0.062	11.86	11.56	3.43
	20.83			0.037			7.99		
C1-S170-D6	26.25			0.105			11.50		
	27.78	27.17	0.81	0.108	0.086	0.036	14.980	12.68	1.99
	27.48			0.044			11.570		
C1-S120-D10	22.71			0.000			12.88		
	25.21	23.91	1.25	0.000	0.000	0.000	12.72	12.06	1.28
	23.80			0.000			10.58		
C1-S140-D10	28.18			0.000			13.38		
	29.66	29.11	1.66	0.171	0.189	0.142	12.84	12.68	0.56
	31.17			0.254			12.38		

	27.43			0.332			12.11		
C1-S170-D10	31.13			0.296			12.53		
	30.53	30.01	1.45	0.094	0.166	0.113	17.29	13.20	3.81
	28.38			0.108			9.77		
C1-S120-D13	22.85			0.331			11.04		
	22.68	21.41	1.46	0.000	0.543	0.674	8.79	10.72	1.80
	19.73			1.298			12.33		
C1-S140-D13	28.94			--			10.49		
	26.60			0.000			10.38		
	25.81	27.01	1.35	0.188	0.517	0.739	12.52	11.69	1.49
	26.68			1.363			13.37		
C1-S170-D13	28.94			0.328			11.08		
	27.88	28.38	0.53	0.353	0.378	0.066	12.20	12.32	1.30
	28.31			0.452			13.67		
C2-S120-D10	26.76			0.000			6.99		
	30.02	27.89	1.84	0.000	0.000	0.000	7.53	7.26	0.38
	26.90			0.000			--		
C2-S140-D10	32.83			0.063			10.82		
	33.34	32.56	0.74	0.123	0.107	0.039	9.52	10.90	1.39
	31.58			--			10.44		
	32.51			0.135			12.81		
C2-S170-D10	32.74			0.096			10.01		
	35.91	33.10	2.17	0.146	0.112	0.024	9.85	10.53	1.04
	33.14			0.109			10.19		
	30.62			0.095			12.08		
G1-S60-D10	16.10			0.090			6.85		
	18.88	18.01	1.66	0.300	0.177	0.110	8.70	8.08	1.07
	19.06			0.140			8.70		
G1-S120-D10	21.69			0.118			9.97		
	22.37	21.89	0.33	0.060	0.146	0.103	8.56	10.56	1.66
	21.65			0.295			12.40		
	21.84			0.110			11.29		
G1-S140-D10	24.51			--			13.12		
	24.28	24.87	0.77	0.181	0.133	0.056	10.91	12.51	1.08
	24.68			0.071			13.24		
	25.99			0.146			12.75		
G1-S170-D10	23.92			0.014			13.32		
	26.21	25.01	1.16	0.144	0.144	0.098	12.24	12.67	0.52
	24.11			0.170			12.86		
	25.80			0.249			12.26		
G1-S60-D13	14.09			0.910			4.90		
	15.62	14.50	0.99	0.865	0.867	0.043	6.64	6.37	1.35
	13.78			0.825			7.56		
G1-S120-D13	20.80			0.351			9.73		
	22.84	21.20	1.46	0.654	0.643	0.287	10.73	10.27	0.44
	21.75			0.925			10.52		
	19.40			--			10.11		
G1-S140-	27.90	25.49	1.76	--	0.523	0.368	14.00	13.72	1.53

D13	23.77			0.197			11.64		
	24.79			0.922			13.90		
	25.50			0.451			15.34		
G1-S170-D13	26.88			--			13.53		
	27.19			0.916			14.52		
	24.49	26.05	1.24	0.331	0.509	0.353	13.83	13.96	0.51
	25.63			0.280			--		
G1-S120-D16	20.36			1.382			--		
	23.00	21.72	1.32	1.164	0.904	0.648	13.75	12.65	1.56
	21.81			0.167			11.54		
G1-S140-D16	24.50			0.452			15.37		
	25.25			1.652			13.92		
	27.20	25.54	1.16	0.848	1.085	0.492	16.93	15.09	1.38
	25.21			1.388			14.15		
G1-S170-D16	27.02			0.323			14.45		
	24.38	25.93	1.38	0.995	0.833	0.451	13.46	13.62	0.77
	26.40			1.180			12.94		
G2-S60-D10	18.04			0.200			6.34		
	18.53	18.30	0.25	0.100	0.147	0.050	4.21	5.57	1.18
	18.34			0.140			6.15		
G2-S120-D10	23.36			0.000			7.14		
	20.28			0.000			7.76		
	21.96	21.79	1.27	0.000	0.000	0.000	7.02	7.31	0.40
	21.57			0.001			--		
G2-S140-D10	22.13			0.200			10.17		
	25.66			--			12.23		
	23.11	24.08	1.10	0.184	0.159	0.057	10.16	10.32	1.44
	23.73			0.094			8.73		
G2-S170-D10	23.31			0.139			9.73		
	23.78			0.227			9.30		
	25.11	23.77	0.96	0.128	0.157	0.047	8.59	9.10	0.52
	22.89			0.133			8.77		
G3-S60-D10	20.55			0.120			6.56		
	19.84	18.78	2.48	0.140	0.130	0.010	6.38	6.73	0.46
	15.95			0.130			7.26		
G3-S120-D10	24.61			0.152			9.41		
	25.65	25.13	0.74	0.111	0.129	0.021	8.76	9.08	0.46
	24.26			0.125			--		
G3-S140-D10	32.71			0.108			--		
	31.59	32.69	1.10	0.153	0.132	0.023	13.97	14.08	0.16
	33.78			0.135			14.19		
G3-S170-D10	33.57			0.124			12.39		
	33.42	33.21	0.50	0.093	0.105	0.016	12.60	12.43	0.15
	32.63			0.099			12.30		



Figure 3.13. Typical debonding surfaces of FRP bars from three producers

3.3.3.2. Bond stress-slip response

The bond stress-slip curves are shown in Figure 3.14. All FRP bars demonstrated a notably stiff ascending branch until reaching the maximum bond stress τ_u . For the majority of specimens, the bond stress of up to $0.9\tau_u$ did not cause any noticeable interfacial slip, with slip values at bond strength generally less than 1.0 mm (a recommended slip at bond strength for steel reinforcement (fib 2013)). This can be attributed to the applied sand-coated layer on

all tested FRP bars. Previous research investigating the correlation between surface type and the contributions from different bond mechanisms (Soong *et al.* 2011; Fahmy *et al.* 2021) reported that chemical adhesion and static friction govern the bond mechanism of FRP bars with sand coating. These factors effectively hindered slip, thereby achieving high initial bond stiffness. As adhesion and static friction were overwhelmed, the micro-slip stage occurred. However, this second stage was inconspicuous as the bearing resistance was relatively insignificant for the tested FRP bars due to the less pronounced surface deformation that can result in mechanical interlocking.

When the bond stress reached its maximum, the descending branch occurred, after which the bond-slip curve gradually leveled off at a slip of around 8 mm to 12 mm. In general, two different descending stages can be identified. The P1 bars softened gradually with the increasing slip (see Figure 3.14 (a) to (f)). In contrast, a dramatic drop in load to around zero after the peak load was observed for the P2, as depicted in Figure 3.14 (g) and (h). The pullout force then increased rapidly and gradually stabilized at 30%-40% of τ_u . The abrupt failure was also noticed for the P3 bars in UHSPC and became less pronounced in HSC (i.e., S60 set in Figure 3.14 (i)).

Several researchers have reported these variations in the softening segment (Aiello *et al.* 2007; Baena *et al.* 2009; Firas *et al.* 2011; Soong *et al.* 2011; Lee *et al.* 2012; Refai *et al.* 2015; Lämmlein *et al.* 2017; Peng *et al.* 2022). They claimed that the brittle bond failure was typically observed for SC FRP bars. It was suggested that the abrupt decay of bond stress was attributed to the sand coating layer's sudden peeling off the bar core, which is generally related to the dramatic release of a large amount of energy. However, similar failure modes were also observed in FRP bars without surficial sand coating, which failed by the delamination of the outer resin-fiber layer and complete shearing of the bar ribs (Firas *et al.* 2011; Soong *et al.* 2011). An alternative explanation is that the brittle horizontal shear performance of FRP bars causes sudden bond failure when the debonding surface occurs inside the bar core. Comparing Figure 3.11 and Figure 3.14, it can be found that P1 bars, which showed ductile horizontal shear failure, debonded gradually, whereas P2 and P3 bars, featuring brittle shear failure under the short-beam test, yielded a sudden load drop in the post-peak bond behavior in UHSSC. Therefore, in addition to the sudden delamination of the surficial sand-coating layer, the horizontal shear performance of FRP bars emerges as another critical factor influencing the post-peak bond behavior of bars in UHSC.

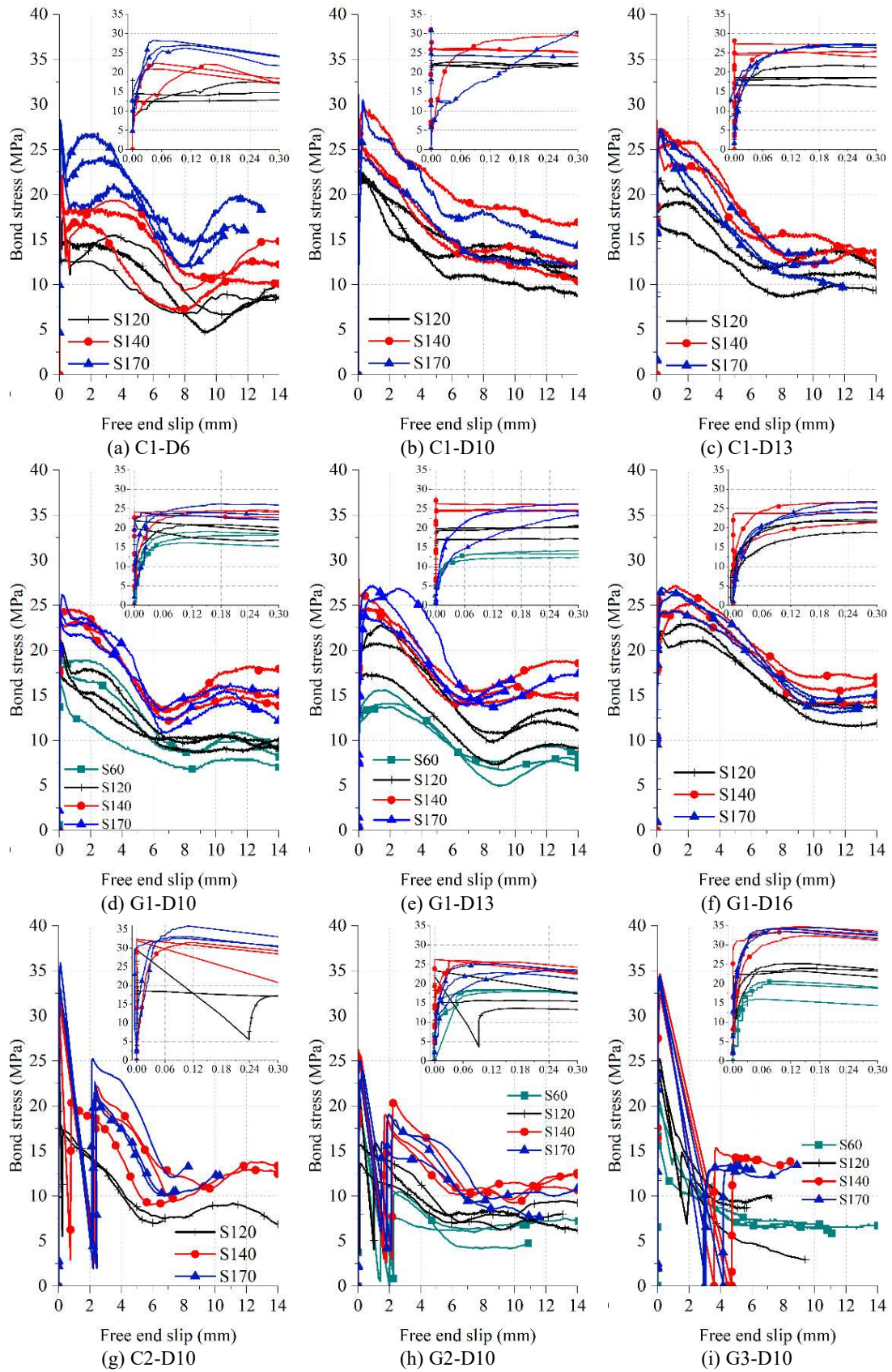


Figure 3.14. Bond-slip curves for FRP bars at the free end

3.3.3.3. Effect of concrete strength on bond strength depending on bar type

Figure 3.15 compares the bond strength of $\Phi 10$ CFRP and GFRP bars from three producers. It is evident that bond strength increased substantially with the increase in concrete strength from the HSC to UHSC classes. Specifically, when the compressive strength increased from S60 to S170, a 30% to 80% improvement in bond strength was observed. This finding contradicts previous studies, which suggested that the bond strength of FRP bars hardly increases with increasing f_c greater than 30-50 MPa (Achillides and Pilakoutas 2004; Achillides 1998; Nanni *et al.* 1995). This supports Solyom and Balázs (2020), who state that the properties and fabrication processes of FRP bars have made great progress over the past decade. Consequently, the suggested upper limit of concrete strength in previous research may no longer be applicable.

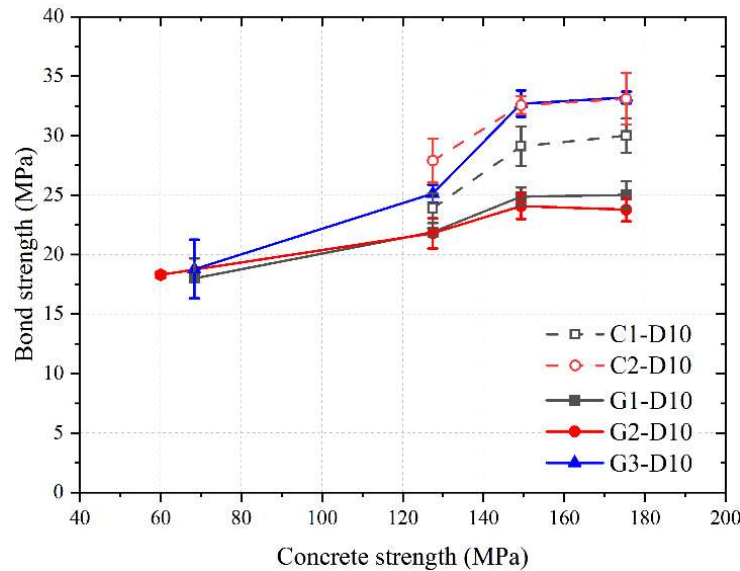


Figure 3.15. Effect of concrete strength on bond strength dependent on bar type

As reported in many studies (Lee *et al.* 2008; Tang *et al.* 2008; Arias *et al.* 2012; Lee *et al.* 2012; Basaran and Kalkan 2020b; Wang *et al.* 2021; Peng *et al.* 2022), the growth of bond strength is much slower than that of concrete strength. Specifically, an average 20% increase in bond strength was observed as the compressive strength increased from 129 MPa to 149 MPa, while a marginal increase (less than 3%) was noted when the compressive strength varied from 149 MPa to 175 MPa. This can be explained by the fact that the horizontal shear resistance of the peripheral layer of FRP bars, rather than the concrete strength, controls the bond properties within the UHSC classes, as bond failure occurs solely inside the bar core,

as mentioned earlier. Consequently, this study identifies an upper limit of concrete strength of between 129 MPa and 149 MPa. However, for certain FRP bar types, a substantial increase in bond strength was observed when the concrete strength enlarged to 175 MPa (Hossain *et al.* 2017a, b), while in other cases, the bond strength reached its maximum with a compressive strength of less than 120 MPa (Firas *et al.* 2011; Chen *et al.* 2022; Liang *et al.* 2023). Therefore, it is impractical to establish a universal critical value of concrete strength for all FRP bars, as this depends on the shear properties of the specific FRP bar type. The use of FRP bars with improved shear strength between the resin and fiber core alters the failure mode from delamination inside the bar core to concrete crushing, allowing for a significantly higher upper limit of concrete strength.

3.3.3.4. Effect of horizontal shear strength and fiber type on bond strength

This section discusses the effect of the horizontal shear strength of FRP bars on bond strength in UHSSC. Figure 3.16 shows the correlation between the shear strength of FRP bars and bond strength in UHSSC based on fiber type. Despite some inconsistencies, bond strength generally increased with the shear strength of FRP bars. As observed for both CFRP and GFRP bars in Figure 3.16, an increase in shear strength of FRP bars was reported to improve bond strength by 10% to 40% (23% on average). Notably, the increasing rate of bond strength was much lower than that of shear strength. For instance, with an increase of 26.7 MPa in shear strength from C1 to C2 bars, the resulting bond strength increased by an average of 3.9 MPa. The horizontal shear strength is, as a result, a critical governing factor that should be taken into consideration in the bond strength of FRP bars in UHSSC. For design purpose, it is recommended to include an additional coefficient accounting for the shear strength of specific types of FRP bars on the bond strength model. However, the quantitative contribution of the shear strength of FRP bars to the bond strength in UHSSC cannot be determined yet unless more test data becomes available.

Figure 3.17 compares the effect of fiber type on the bond strength in UHSSC based on the same producer. Analysis of $\Phi 10$ P1 bars reveals that the bond strengths of CFRP bars were respectively 17.0% and 20.0% higher than those of the GFRP bars in UHSSC specimen sets, despite the lower horizontal shear strength of CFRP bars. This can be attributed to the difference in fiber type, considering that both C1 and G1 bars were manufactured using the same fabrication technology and vinyl ester resin system. As detailed in Figure 3.13, the P1 bars in UHSSC, especially in S140 and S170 concrete matrices, failed inside the bar core

with rupture of internal fiber bundles, indicating that the fiber rupture strength greatly affects the bond performance in UHSSC. Therefore, it can be concluded that fiber type is also of great significance to the bond strength, in addition to the horizontal shear strength of FRP bars. However, for NSC and HSC, existing research showed that fiber type had a negligible influence on bond strength (Achillides and Pilakoutas 2004; Park *et al.* 2016; Yin *et al.* 2020; Zhang *et al.* 2021; Zhang 2022; Zhou *et al.* 2023). This is because bond failure in NSC and HSC generally occurs within the concrete matrix and its adjacent grained layer, where the shear capacity of the surrounding concrete and the superficial grained layer of the specific FRP bar type dominate the bond. Accordingly, unlike in NSC and HSC, evaluating the bond of FRP bars in UHSSC necessitates considering the effect of fiber type.

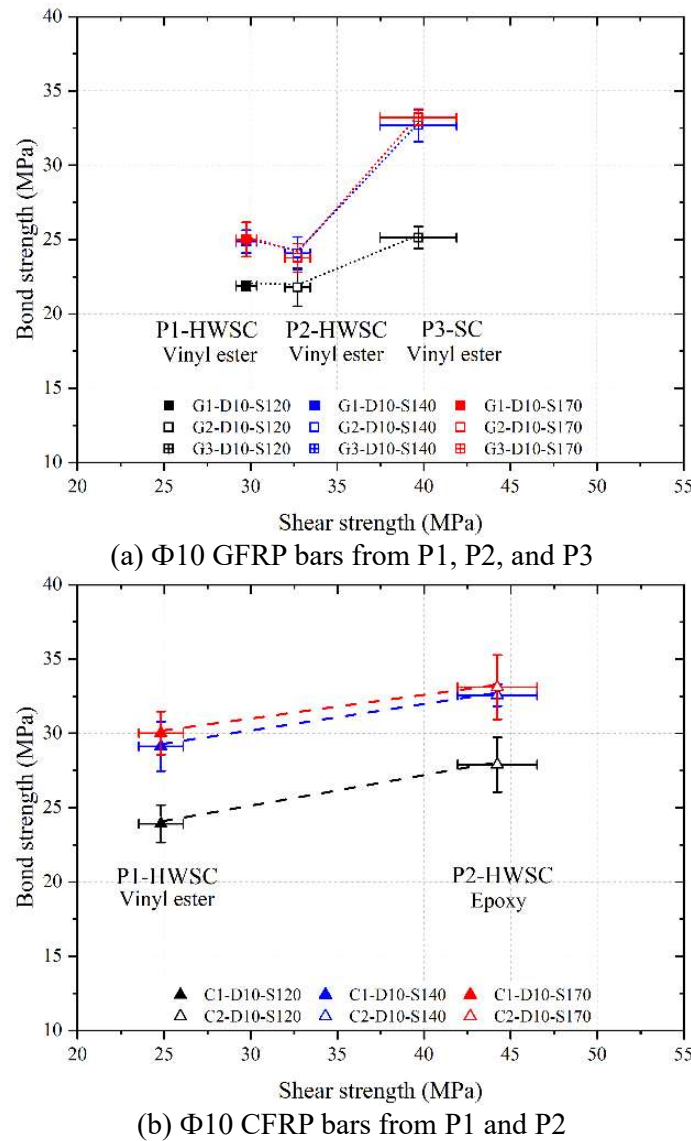
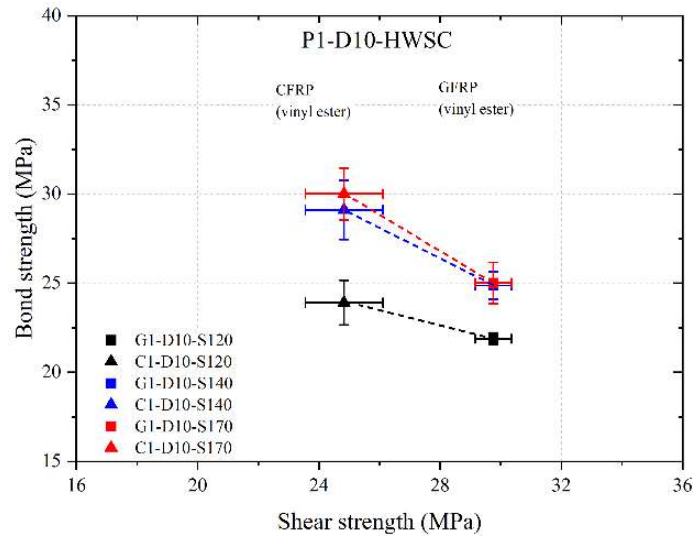
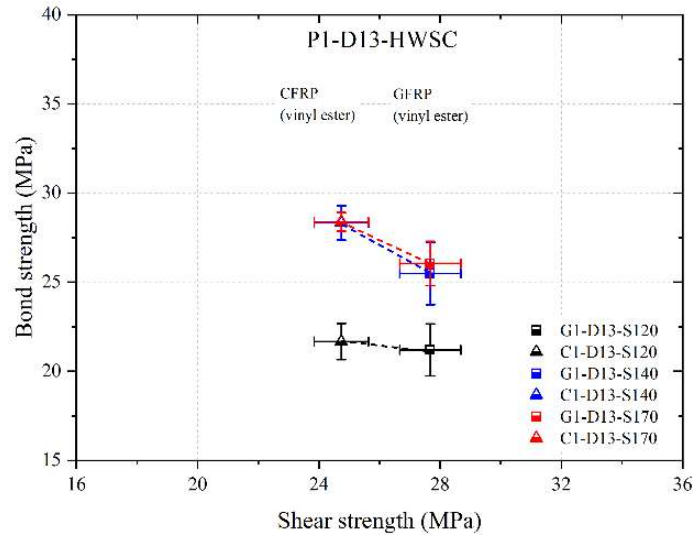


Figure 3.16. Effect of shear strength of FRP bar on bond strength dependent on fiber type



(a) Φ10 CFRP and GFRP bars from P1



(b) Φ13 CFRP and GFRP bars from P1

Figure 3.17. Effect of fiber type of FRP bar on bond strength from the same producer

3.3.3.5. Effect of bar diameter on bond strength depending on concrete strength

The increase in bond strength with decreasing bar diameter has been confirmed by many researchers (Benmokrane and Tighiouart 1996; Cosenza *et al.* 1997; Achillides and Pilakoutas 2004; Okelo and Yuan 2005; Davalos *et al.* 2008). They concluded that the size effect in bond strength resulted from the bleeding effect, Poisson's effect, and shear lag effect. All these effects were more pronounced for bars with larger diameters. In addition, Bažant *et al.* (1995) and Bamonte *et al.* (2003) noted that the effect of bar diameter on bond adheres to

the general size effect law due to the localization of concrete damage at the bar-to-concrete interface.

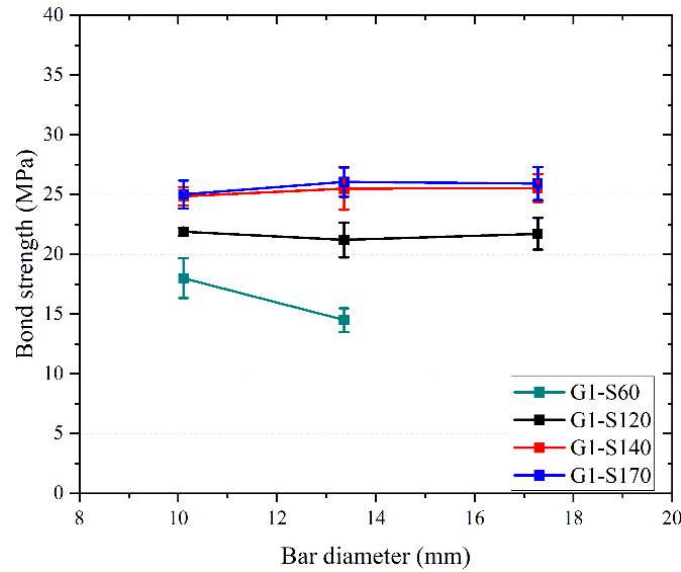


Figure 3.18. Effect of the bar diameter on bond strength depending on concrete strength

The bond strength versus the bar diameter for P1 bars in different concrete strength sets is shown in Figure 3.18. In this study, test results of the S60 specimen set adhered to the aforementioned finding, showing a 24% increase in bond strength for the $\Phi 10$ GFRP bars in contrast to the $\Phi 13$ bars. Conversely, for the S120, S140, and S170 specimen sets, the test results diverged from the expected trend. It can be observed that bond strength showed negligible variation across different bar diameters. Specifically, the $\Phi 10$ G1 bars yielded bond strength around -4.0% - 3.3% larger than those developed by the $\Phi 13$ G1 bars for concrete strengths of 120, 140, and 170 MPa, respectively. These percentages were around -3.6% - 0.8% for the $\Phi 16$ G1 bars in the same concrete strengths in comparison to the $\Phi 10$ G1 bars. A minor variation of 1.78 MPa in bond strength was also reported for CFRP bars of 6, 10, and 13 mm in diameter in the S170 sets. The observations are consistent with Jiang *et al.* (2024) and Hu *et al.* (2024). They reported insignificant bond strength reductions of less than 10% with increasing diameters of FRP bars in UHSC, when specimens were subjected to pullout failure.

A possible explanation is that the bleeding effect is less significant in UHSC prisms due to the more homogenous and compact mesostructure in these concrete matrices, which benefits the formation of a stronger bond at the bar-to-concrete interface. Furthermore, since bond

failure occurs mainly outside the concrete matrix in these cases, the size effect resulting from the localization of concrete damage is less pronounced. Consequently, the bond of FRP bars in UHSSC appears to be size-independent. This difference in size effect in HSC and UHSC may confirm the 2-way interaction between the influence of concrete strength and that of bar diameter on bond strength, as illustrated in previous studies (Bamonte and Gambarova 2007; Lee *et al.* 2017). That is, the reduction rate of bond strength with increasing bar diameter decreases as the concrete strength increases. However, more test data are required to consolidate this law for concrete in all strength classes.

3.3.3.6. Effect of the variation in surface condition on bond strength

In contrast to the size-independent bond performance in UHSSC outlined in the previous section, the bond of C1 bars in UHSC specimen sets exhibited a different trend (see Figure 3.19). The bond strength of 10-mm-diameter C1 bars experienced an increase of 10% to 31% and 6% to 12% compared to the $\Phi 6$ and $\Phi 13$ C1 bars, respectively. This discrepancy is probably due to the remarkable imperfections of the surface treatment of the $\Phi 6$ and $\Phi 13$ C1 bars during the fabrication process, which abated the bond performance. As discussed in Section 3.3.1, the quality of surface treatment of $\Phi 6$ and $\Phi 13$ C1 bars was inferior to that of $\Phi 10$ C1 bars, with the highest variation in surface topography in $\Phi 6$ and $\Phi 13$ C1 bars. The irregular surface topography was characterized by extensive missing of the sand-coating layer of $\Phi 6$ and $\Phi 13$ C1 bars (see Figure 3.20), leading to a significant reduction in contact surface area and a substantial decrease in the resulting chemical bond. Consequently, Figure 3.19 shows that the bond strength decreased as the surface irregularity became pronounced. Against these discussions, it can be inferred that the production variability in surface condition greatly affects bond strength, and the influence of bar size on bond strength can be overshadowed by the influence of possible imperfections of surface finish.

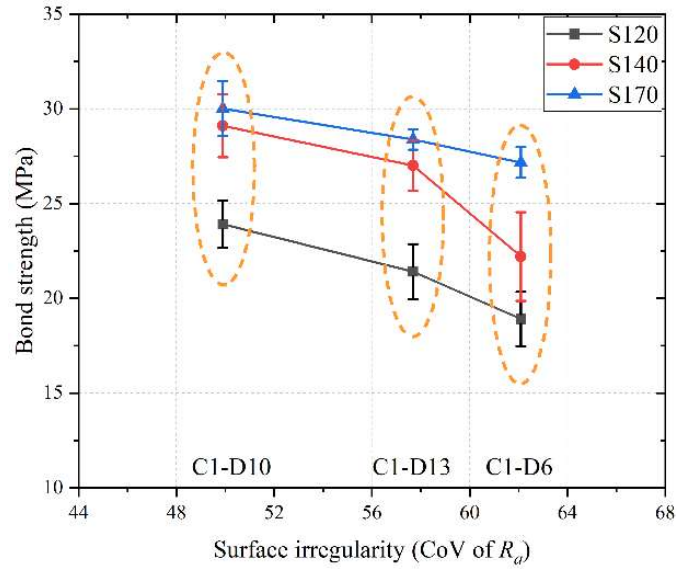


Figure 3.19. Effect of the variation in surface condition on bond strength

3.3.3.7. Factors that influence the slip s_1 corresponding to bond strength

Since the free-end slip measured at the peak load denotes the characteristic slip corresponding to the local bond strength, s_1 , based on the interpretation of pullout test results in previous analyses (Focacci *et al.* 2000; Vaculik *et al.* 2018), the values of free-end slips are directly used for analysis. In this section, the influences of concrete strength, bar diameter, surface type, and variation in surface condition are evaluated.

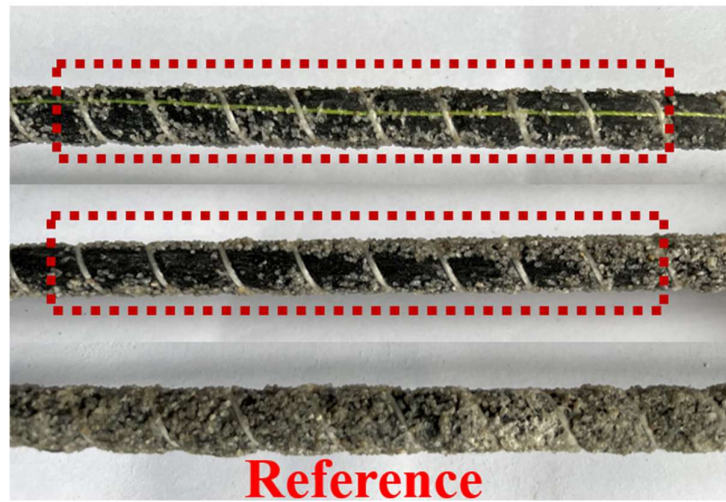


Figure 3.20. Typical imperfections of surface treatment of C1-D6 bars

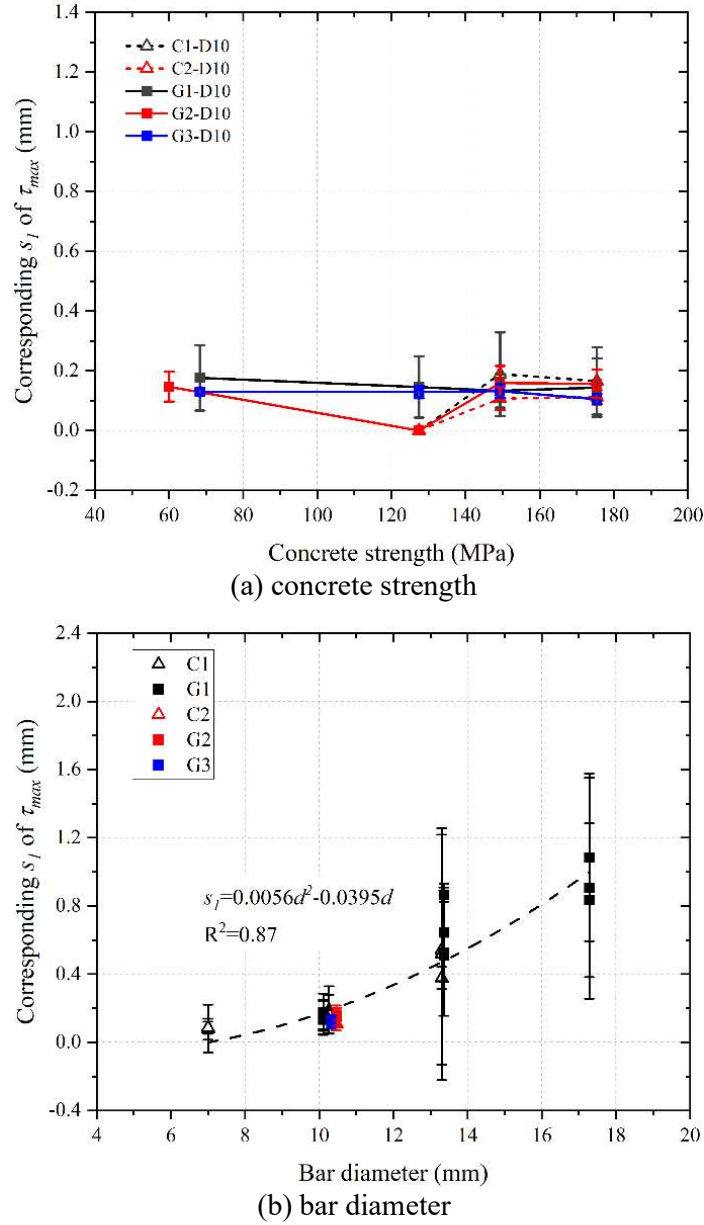


Figure 3.21. Factors that influence s_1 dependent on bar type

Figure 3.21 (a) shows that an increase in concrete strength from 60 to 170 MPa did not significantly affect the value of s_1 . This observation aligns with findings in HSC and UHSC in several previous studies (Shen *et al.* 2016; Fahmy *et al.* 2021; Zhao *et al.* 2021). However, the value of s_1 is found to be much larger in NSC than in HSC (Lee *et al.* 2008; Lee *et al.* 2012). This can be explained by the change in failure pattern from concrete damage in NSC, resulting in larger values of s_1 , to damage in FRP bars in higher strength concrete, which causes smaller values of s_1 . Furthermore, the value of s_1 was insensitive to the surface type of FRP bars in HSC and UHSSC in this test. In this study, all 10-mm-diameter FRP bars with

sand coating mainly fell in the range of 0.1-0.2 mm, irrespective of surface type. This contradicts the previously reported findings in NSC, where FRP bars with surface deformation yielded larger values of s_1 due to the mechanical interlocking caused by bar lugs (Aiello *et al.* 2007; Lee *et al.* 2008; Refai *et al.* 2015; Solyom and Balázs 2020). For FRP bars in HSC and UHSSC, the interlocking mechanism is less developed in HWSC bars (i.e., P1 and P2 bars) due to the damage to the bar surface.

For pullout failure, earlier studies on steel reinforcement found that the s_1 is mainly dependent on rib spacing (Harajli *et al.* 1995; Zhao and Zhu 2018; Lin *et al.* 2019), which is closely related to the formation of the internal bond cracks in concrete due to the interlocking mechanism. By comparison, few experimental studies have focused on the relationship between the spiral spacing C_{clear} of the FRP bar (equivalent to rib spacing in steel rebar) and s_1 (Xiong *et al.* 2021). Nevertheless, many researchers (Baena *et al.* 2009; Yoo *et al.* 2015; Shen *et al.* 2016; Xue *et al.* 2016) have documented an increase in s_1 with bar diameter. A similar conclusion can be derived from this study. As given in Figure 3.21 (b), s_1 is proportional to the bar diameter, irrespective of bar type. Unlike conventional steel bars, s_1 , in the authors' opinion, may not be strongly correlated with spiral spacing due to the weak interlocking force developed in P1 and P2 bars in HSC and UHSSC, as mentioned in the previous section.

Figure 3.22 highlights significant variability in s_1 , as indicated by high CoVs across the specimen sets. Notably, the s_1 of P1 bars exhibited the largest scatter, with the highest CoVs. For P1, P2, and P3 bars, the median CoVs were 68%, 30%, and 16%, respectively. Referring to the variations in surface condition as detailed in Section 3.3.1., it is interesting to find that the median CoV of s_1 increased with the corresponding CoV of R_a . Test results also show that the C1-S120-D6, C1-S120-D13, and C1-S140-D13 sets, where embedded FRP bars yielded the unexpectedly highest variation in surface coating condition, exhibited the largest CoV in s_1 . This suggests that extensive variability in surface topography can amplify the scatter of s_1 because of the prominent local irregularities on the bar surface caused by poor coating quality. The substantial missing of coating sands on bar surface, which decreases the surface roughness of these FRP bars, diminishes the contribution of chemical adhesion and static friction caused by sand-coated layer (Soong *et al.* 2011; Fahmy *et al.* 2021). The initial bond stiffness of these bars consequently declines with reduced chemical adhesion and static friction, thereby achieving significantly high s_1 . As a result, it can be inferred that the

topographical regularity of the FRP bar surface significantly affects the characteristic slip, demonstrating the necessity for careful coating of the bar surface during production for a consistent bond performance of FRP bars.

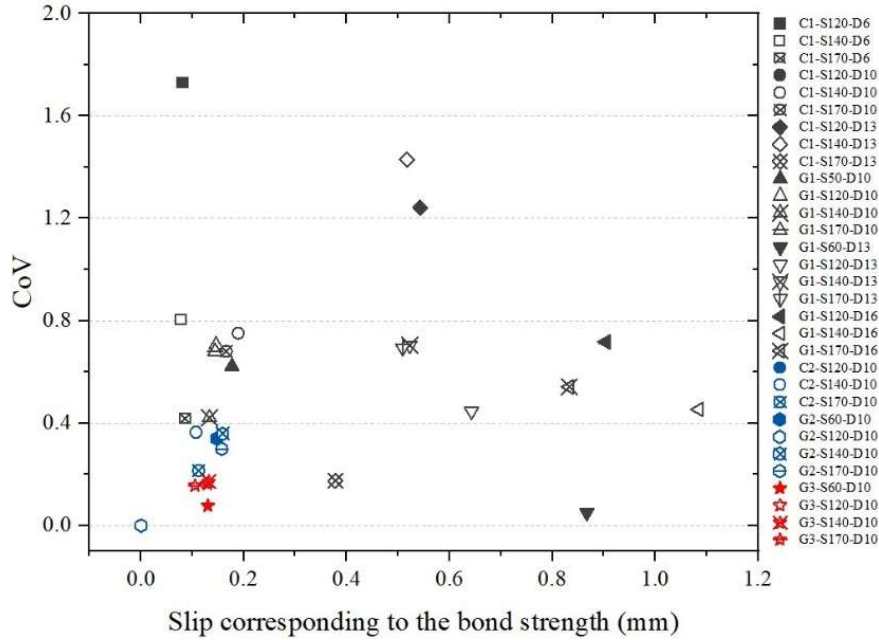


Figure 3.22. Coefficient of variation versus mean values of s_l

3.4. Recommendations to develop a bond strength model in UHSC

3.4.1. Existing bond strength model in UHSC

In general, the bond strength of FRP bars in UHSC depends more on the horizontal shear strength and fiber type of FRP bars than on the concrete strength and bar diameter, when the shear resistance of the concrete substrate exceeds that of FRP bars. Table 3.7 summarizes existing predictive bond strength models and the dataset used for their development. Regarding the models developed for UHSC, most defined the contribution of concrete strength as a multiplier, either in the form of f_c^n (i.e., the n -th power of f_c) or f_{tsp} , without specifying an upper limit, and none have considered the effect of the horizontal shear performance of FRP bars except the model proposed by Liang *et al.* (2023). Clearly, such models did not conform to the experimental observations and did not appear to be generally applicable to the whole range of UHSC classes. In contrast, the introduction of an upper bound τ_m (adhesion strength between the resin layer and the core fiber bar) to the maximum bond strength τ_u in Liang *et al.*'s model (2023) is shown to be a more viable practice. However,

determining the upper bound τ_m requires extensive investigation, taking into account the specific type of FRP bars.

Furthermore, only three of these models (i.e., models developed by Basaran and Kalkan (2020a), Kim and Wang (2023), and Jiang *et al.* (2024)) proposed a coefficient accounting for the effect of fiber type on bond strength in UHSC. However, Kim and Wang's (2023) model was derived without consideration of carbon fiber. In Jiang *et al.*'s (2024) model, the effect of fiber type on bond strength in UHSC was not accurately estimated, as it adopted the same fiber type factor as CSA's design equation (CSA 2012) (1.0 for both GFRP and CFRP bars), which is only valid for NSC and HSC. Basaran and Kalkan (2020a) specified different fiber factors for GFRP and CFRP bars depending on their surface type. However, this model is less practical due to complex limits on the dependent variables.

Table 3.7 Predictive bond strength models from existing literature

Ref.	Bond strength models	Limitations
Yoo <i>et al.</i> 2015	$\tau_{\max} = (1.05 + 0.85 \frac{d}{L_e}) \sqrt{f'_c}$	d : 12.76, 15.9, 19.1 mm Bar type: HW-GFRP, SC-GFRP f'_c : HSC, UHSC L_e : 1-7d
Basaran and Kalkan 2020a	$\tau_{\max} = (S - 0.2817f_{tsp} + 6.706 \frac{d}{L_e} - 0.1237d_o)f_{tsp}$ $\tau_{\max} = (S - 0.4476f_{tsp} + 14.11 \frac{d}{L_e} - 0.4026d_o + 0.00497d_o^2 - 1.216f_{tsp} \frac{d}{L_e})f_{tsp}$	d : 6-25.4mm f'_c : 17.26-174.5 MPa L_e : 2.5-20d Bar type: SC-GFRP, Rb-GFRP, In-GFRP, HWSC-BFRP, SC-CFRP Several boundaries of f'_c , f_{tsp} , and d were defined for specific bar type
Nepomuceno <i>et al.</i> 2021	$\tau_{\max} = \alpha(f'_c)^\beta$ $\alpha = \begin{cases} 5.24 & \text{for sand coated and deformed} \\ 2.57 & \text{for deformed} \\ 3.33 & \text{for sand coated} \\ 0.18 & \text{for smooth} \end{cases}$ $\beta = \begin{cases} 0.33 & \text{for sand coated and deformed} \\ 0.54 & \text{for deformed} \\ 0.41 & \text{for sand coated} \\ 0.96 & \text{for smooth} \end{cases}$	L_e : 5d f'_c : 12-170 MPa

Pan <i>et al.</i> 2022	$\tau_{\max} = (0.425d - 0.958)(0.264 \frac{d}{L_e} + 0.091 \frac{C}{d} + 0.126)\sqrt{f_{cu}}$	d : 10, 14, 18 mm C/d : ≤ 5 L_e : 1-2d Bar type: Rb-GFRP
Chen <i>et al.</i> 2022	$\tau_{\max} = (52.73 \times (1 + 0.36\rho_v^{0.816}) \frac{d}{L_e} + 8.77)f_{cu}^{0.122}$	d : 10 mm L_e : 20-30d f_{cu} : 100-150 MPa Bar type: In-CFRP
Kim and Wang 2023	$\tau_{\max} = \frac{\sqrt{f'_c}}{4\varpi}$ $\varpi = a(\frac{S}{C})^2 + b\frac{S}{C} + c \text{ or }$ $\varpi = \begin{cases} 0.063 & \text{for steel bar} \\ 0.098 & \text{for GFRP bar} \\ 0.117 & \text{for BFRP bar} \end{cases}$	d : 9.5 mm f'_c : 123-148 MPa L_e : 2.5d Silica fume / cement ratio: 20% -40%
Hu et al 2023	$\tau_{\max} = \alpha(0.9 + 0.4 \frac{C}{d} + 3.0 \frac{d}{L_e})\sqrt{f'_c}$ $\alpha = \begin{cases} 1.0 & \text{HWSC} \\ 0.4 & \text{SC} \\ 0.4 & \text{SW} \\ 0.3 & \text{Rb} \end{cases}$	d : 10-25 mm L_e : 2.5d, 5d, 10d f'_c : 120-190MPa Bar type: HWSC-GFRP, SC-GFRP, In-GFRP, SW-GFRP
Liang <i>et al.</i> 2023	$\tau_{\max} = 0.82E_f \frac{h_r^2 \sqrt{4h_r^2 + w_r^2}}{4rw_r s_r} + \frac{w_c + 0.25w_r}{s_r} \tau_s$ $\leq \tau_m = 40 \text{ MPa}$	d : 10, 12 mm L_e : 3.5d f'_c : 64.5-122 MPa Bar type: Rb-GFRP, HW-GFRP
Tong <i>et al.</i> 2023	$\tau_{\max} = k_f(0.5 + 0.3 \frac{C}{d} + 1.5 \frac{d}{L_e})\sqrt{f_{cu}}$	d : 12, 16, 20 mm L_e : 3-5d f_{cu} : 90-130 MPa Bar type: Rb-GFRP, HW-GFRP
Zhu <i>et al.</i> 2023	$\tau_{\max} = (-0.5 + 0.03 \frac{C}{d} + 4.5 \frac{d}{L_e} + \frac{21.6}{d})\sqrt{f'_c}$	d : 8-19 mm L_e : 2.5-16d f'_c : 71-175 MPa Bar type: SC-GFRP, Rb-CFRP
Jiang et al .2024	$\tau_{\max} = \frac{d_{cs}\sqrt{f'_c}}{1.15k_1k_2k_3k_4k_5\pi d} \times 14.23 \frac{d}{L_e}$	d : 6-16 mm L_e : 1-10d f'_c : 93.7-181.3 MPa Bar type: Rb-BFRP, SW-BFRP, Sm-BFRP, HWSC-GFRP, Sm-GFRP, Rb-GFRP, Rb-CFRP, HW-CFRP, SC-CFRP

Based on the aforementioned discussion, although the variability in the surface coating condition has a great impact on the development of bond strength of FRP bars, no models

have introduced a reduction factor accounting for the topographical irregularity of the bar surface.

3.4.2. Recommendations

To account for the contributions of horizontal shear strength and fiber type of FRP bars to the bond in UHSSC in practical design, it is suggested to incorporate a fiber factor in the design equation and establish an upper limit on bond strength based on the horizontal shear strength of FRP bars. Based on the aforementioned discussions, an amplification fiber factor of 1.10-1.13 is recommendable for the bond strength of CFRP bars in UHSSC compared with that of GFRP bars of the same fabrication technology using the fiber factor of 1.0. When the upper limit based on the shear strength of FRP bars is applied to the design equation, the impact of concrete strength on bond strength will become insignificant in UHSC classes. This is consistent with the experimental observations in Section 3.3.3. However, an interpretation of extensive test data is required to determine the upper limit of various FRP bars due to the lack of standardization in FRP bar fabrication and variability in the mechanical characteristics of FRP components.

In addition, in contrast to steel bars, the topographical regularity of the surficial sand-coating layer of FRP bars is a key parameter that greatly affects the bond strength and corresponding slip value. The imperfection of bar surface condition caused by extensive missing of coating sands during manufacturing decreases the bond strength of FRP bars. For design purposes, the surficial reduction factors for bond strength predictive equations of 0.82, 0.92, and 1.0 are suggested for sparse, moderate, and dense surface conditions, respectively, based on the previous test results.

3.5. Conclusions

This chapter has examined the bond performance of different types of FRP bars embedded in HSC and UHSSC through direct pullout tests. Specifically, this study sheds light on the influence of mechanical properties, the surface conditions, and the corresponding variation of the FRP bar on bond behavior. The following concluding remarks can be drawn:

1. All the pullout test specimens failed by pulling out of the FRP bars. The failure mode changed with concrete strength. The bond failure commonly occurred in the surrounding

concrete in HSC. As concrete strength increased, the bond tended to fail near the bar-to-concrete interface, and between the surface and subsequent core layer of the FRP bar in UHSC. It is impractical to establish a universal threshold of concrete strength applicable to all types of FRP bars for demarcating these typical debonding zones. Such a threshold is controlled by the shear capacity of the grained layer and the subsequent core layer of the FRP bar, which, in turn, depends on the manufacturing technology and material properties of a particular type of FRP bar.

2. The topographical regularity of the FRP bar surface significantly influences the bond performance, including the bond strength and its corresponding slip value. The imperfection of the bar surface condition caused by extensive missing of coating sands during manufacturing decreased the bond strength by 6% to 31%, and resulted in a slip at the bond strength with a median CoV of up to 68%. To ensure the consistency of the bond performance of FRP bars, great attention should be paid to the quality control of surface treatment. Otherwise, a surficial reduction factor should be suggested for bars with poor surface conditions.
3. In UHSSC, the bond strength of FRP bars is not sensitive to concrete strength and bar diameter but to fiber type and horizontal shear performance of FRP bars. The bond strength of CFRP bars in UHSSC is 1.10-1.13 times as high as GFRP bars of the same fabrication technology. The horizontal shear performance of FRP bars affects post-peak bond behavior and bond strength in UHSSC. FRP bars featuring brittle shear failure under short-beam tests yield a sudden load drop in the debonding stage. In addition, an increase in the shear strength of the FRP bars correlates with an enhancement in bond strength. For design purpose, incorporating a fiber factor and establishing an upper limit based on the horizontal shear strength of FRP bars are recommendable for accurately predicting the bond strength of FRP bars in UHSC.
4. The test results revealed the potential 2-way interaction between the influence of bar size and concrete strength on bond strength. The size effect becomes less significant with higher concrete strength, and the bond in UHSSC appears to be size-independent.
5. For FRP bars in HSC and UHSC, the slip corresponding to the bond strength varies little with concrete strength and surface type. By comparison, the bar diameter greatly affects

s_1 , which increases with bar size.

Note that the FRP bars tested in the present study were finished without prominent surface deformation, for which the chemical adhesion and friction are two main bond mechanisms. The analysis in this chapter is therefore not applicable to the ribbed and indented FRP bars, since the contribution of mechanical interlocking to the bond mechanism is significant for these FRP bars. In addition, to clearly determine the upper limit on bond strength of different FRP bars, further research is needed to investigate other bar types with different horizontal shear strengths.

CHAPTER 4. ASSESSMENT OF BOND STRENGTH OF FRP BARS IN CONCRETE

4.1. Introduction

Considerable efforts have been made to investigate the factors influencing the bond behavior of reinforcement in concrete (Eligehausen *et al.* 1983; Harajli *et al.* 1995; *fib* 2013; Ehsani *et al.* 1996; Wambeke and Shield 2006; Yan *et al.* 2016; Wang *et al.* 2024). These studies have found that the compressive strength of concrete, bar diameter, concrete cover thickness, and confinement conditions significantly affect the steel bar-to-concrete bond properties. For FRP bars, additional factors such as surface treatment and mechanical properties are also of significant importance to bond performance (Aiello *et al.* 2007; Toumpanaki *et al.* 2018; Yin *et al.* 2020; Solyom and Balázs 2020; Hao *et al.* 2009; Zhao *et al.* 2021; Liang *et al.* 2023; Hu *et al.* 2024). For instance, Aiello *et al.* (2007) and Solyom and Balázs (2020) claimed that FRP bars using proper surface configurations can have up to 100% higher bond strength than deformed steel bars. Hao *et al.* (2009) and Zhao *et al.* (2021) reported that both bond-slip response and bond strength vary with the combination of surficial deformation parameters, including rib spacing and rib height. More importantly, recent studies indicated that the bond of FRP bars in high and ultra-high strength concrete fails due to shear damage within the bar (Liang *et al.* 2023; Hu *et al.* 2024). This suggests that the shear properties of FRP bars, particularly of the surface layer, significantly affect the bond performance. Based on experimental observations, accurately determining the bond strength between FRP bars and concrete requires considering all the aforementioned factors. However, FRP bars are fabricated not following a standardized manner. Their mechanical properties and surface configurations vary with producers. This hinders the development of a commonly accepted predictive model for the bond performance of FRP bars.

As reviewed in Chapter 2, the effects of various factors on the bond of FRP bars in concrete were widely investigated. As evident from experimental observations, the relative shear characteristics of the surrounding concrete and FRP bars determine the failure mode. Regarding concrete type, replacing conventional raw materials with seawater and/or sea-sand, and other sustainable materials does not have a statistically significant impact on the bond

strength.

Generally, the bond strength of FRP bars increases with concrete strength, but there is a threshold for concrete strength, beyond which the bond strength hardly increases. This threshold should be adjusted upward with improvements in FRP bars fabrication. The increase in bond strength with concrete strength varies with different surface types of FRP bars, and the common use of $\sqrt{f_c}$ cannot provide a good representation of the effect of concrete strength on bond strength.

Surface configuration greatly affects the bond strength of FRP bars, and the interpretation of the surface factor should consider not only the surface type but also the physical properties of the surface condition and their variations. In addition, large bars typically develop lower bond strength, and more importantly, the effect of bar diameter on bond follows the general size effect law. It should be noted that the size effect on bond strength varies with concrete strength and surface type.

In higher-strength concrete, particularly in UHSC, bond strength is affected by the horizontal shear strength and fiber type of FRP bars. In UHSC, an increase in horizontal shear strength of FRP bars leads to an increase in bond strength, and CFRP bars exhibit higher bond strength than BFRP and GFRP bars of the same fabrication technology and surface type.

The review in Chapter 2 suggests that there are potential 2-way interaction effects between the aforementioned factors (i.e., concrete strength, surface configuration, and bar diameter). Moreover, the effect of fiber type seems to interact with that of concrete strength. These potential interaction effects among several key factors will be further explored using statistical analysis based on the database developed in the next section.

Against this background, this chapter aims to evaluate the bond strength between several commercially available FRP bars and concrete of different grades. Based on the critical review in section 2.2. and analysis in Chapter 3, a comprehensive database is assembled by collecting the pullout test results from published studies. Through statistical analysis using the ANOVA and post hoc tests, the significance of several key factors, as well as their coupled effects, on the bond strength is further evaluated. Finally, a new bond strength predictive

equation considering the influence of key factors and their interaction effects is proposed based on the general size effect law, and it is assessed and compared with existing models using the database.

4.2. Bond test database

To assess the contributions of key factors and the possible interactions among the effects of these factors on the bond strength, and to develop a new bond strength model that considers these effects, a comprehensive database of pullout test was developed through an extensive literature review. The bond test data are statistically analyzed in the next section.

4.2.1. Overview of bond test database

In this study, a database containing 687 pullout test results has been assembled from 29 published papers (Ahmed and Sennah 2014; Alves *et al.* 2011; Baena *et al.* 2009; Calvet *et al.* 2015; Davalos *et al.* 2008; Emparanza *et al.* 2022; Hao *et al.* 2007; Hao *et al.* 2023; Henin *et al.* 2021; Hu *et al.* 2023; Ke *et al.* 2024; Lee *et al.* 2008; Lee *et al.* 2012; Lee *et al.* 2017; Liao *et al.* 2021; Okelo and Yuan 2005; Park *et al.* 2016; Parvizi *et al.* 2020; Peng *et al.* 2022; Refai *et al.* 2015; Robert *et al.* 2010; Soares *et al.* 2020; Solyom and Balázs 2020; Xue *et al.* 2013; Xue *et al.* 2014; Xue *et al.* 2016; Yan *et al.* 2017a; Yan *et al.* 2017b; Zeng *et al.* 2022) together with the experiment program conducted by the author's team. A summary of the pullout test results is given in Table 4.1.

Although properties of FRP materials and the fabrication of FRP bars that influences the physical properties of surface condition are crucial to the bond, detailed information is often missing in the literature and inaccessible to bar producers. Test results of commercial FRP bars from producers with similar fabrication and properties were collected in the database and grouped for each surface type. This allows the effect of surface type and the horizontal shear characteristics of FRP bars on bond strength to be consolidated as the effect of “bar type” on the bond strength. The database includes test results of three bar types (Figure 4.1): HWSC bars, SC bars, and In bars.

The detailed information of all the pullout specimens in the database is given in Table 4.15. All specimens in the database exhibited pure pullout failure, with bond lengths varying from 4 to 5 times the diameter of FRP bars. Within this short bond length, the experimentally

determined maximum bond stress can be treated as the bond strength of the local bond-slip relationship. In addition, the effects of concrete cover thickness and confinement are negligible in this analysis, as all specimens in the database failed due to FRP pullout without splitting cracks in the concrete prisms. Note that specimens tested with nonstandard test equipment and method as stipulated in ASTM D7913/D7913-14 (ASTM 2020) and CSA-S806 (CSA 2012) were excluded from the database, as highly inaccurate bond strength measurements may result from unexpected bending caused by system misalignment, etc.

The collected specimens in the database used various concrete matrices, and the concrete compressive strength was determined using samples with different sizes and shapes. The concrete type, as discussed in section 2.2.3.1., is not an influencing factor when assessing bond strength. The concrete compressive strength determined from 50 x 50 x 50 mm cubes ($f_{cm, cu-50}$), 100 x 100 x 100 mm cubes ($f_{cm, cu-100}$), 150 x 150 x 150 mm cubes ($f_{cm, cu-150}$), Φ 75 x 150 mm cylinders ($f_{cm, cyl-75}$), and Φ 150 x 300 mm cylinders ($f_{cm, cyl-150}$) were converted to that of Φ 100 x 200 mm cylinders using conversion factors listed in Table 4.2 (Fládr and Bílý 2018; Graybeal and Davis 2008; Riedel *et al.* 2019).

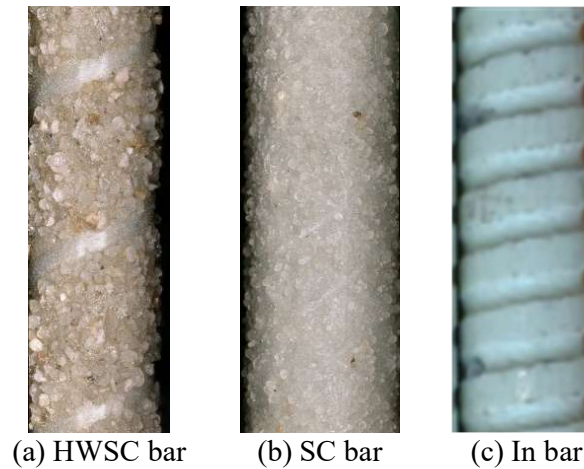


Figure 4.1. Three types of FRP bars in the database

Table 4.1. Summary of pullout specimens in the database

Ref.	Bar type	Fiber type	Bar diameter (mm)	Concrete strength (MPa)	L_e/d	C/d
Okelo and Yuan 2005	HWSC, SC	CFRP, GFRP	6, 10, 16, 19	30-50	5	4.8-16
Hao <i>et al.</i> 2007	HWSC	GFRP	6, 10, 13	30	4	5.3-11
Davalos <i>et al.</i> 2008	HWSC	GFRP	10, 13	60	5	5.3-7
Lee <i>et al.</i> .2008	HWSC, SC	GFRP	13	25-90	4	5.3
Baena <i>et al.</i> 2009	HWSC, SC, In	CFRP, GFRP	8, 10, 12, 13, 16, 19	25-60	5	4.8-12
Robert <i>et al.</i> 2010	SC	GFRP	19	60	5	4.8
Alves <i>et al.</i> 2011	SC	GFRP	16, 19	50	5	2-5.8
Lee <i>et al.</i> 2012	HWSC, SC	GFRP	13	25-75	5	5.3
Xue <i>et al.</i> 2013	HWSC	GFRP	13, 16	25	5	4.5
Ahmed and Sennah 2014	SC	GFRP	19	140	4	2
Xue <i>et al.</i> 2014	HWSC	GFRP	6, 10, 13, 16, 19	25	5	4.5
Calvet <i>et al.</i> 2015	SC	CFRP	6, 13	35-45	5	7.2-11.6
Refai <i>et al.</i> 2015	SC	GFRP	10	50	5	7
Park <i>et al.</i> 2016	SC	GFRP	13	30	4	5.3
Xue <i>et al.</i> 2016	HWSC	GFRP	6, 10, 13, 19	25	5	4.5
Lee <i>et al.</i> 2017	HWSC, SC	GFRP	19, 25	20-60	5	5.3
Yan <i>et al.</i> 2017a	HWSC	GFRP	13	50	5	4.5
Yan <i>et al.</i> 2017b	HWSC	GFRP	13	45-50	5	3-4.5
Parvizi <i>et al.</i> 2020	SC, In	GFRP	12, 13, 16	35-65	5	5.8-7.8
Soares <i>et al.</i> 2020	In	GFRP	8, 12	55-70	5	7.8-12
Solyom and Balázs.2020	HWSC, SC, In	GFRP	6, 8, 10, 12, 13	30-60	5	5.3-12
Liao <i>et al.</i> 2021	HWSC	GFRP	10	80-120	5	9.5
Henin <i>et al.</i> .2021	HWSC	GFRP	10, 16, 19, 25	40-50	5	4.8-9.5
Emparanza <i>et al.</i> 2022	HWSC, SC	GFRP	10	40	5	9.5
Peng <i>et al.</i> 2022	HWSC	GFRP	3, 6, 10	70	5	7-24.5
Zeng <i>et al.</i> 2022	HWSC	GFRP	6, 10, 16	110-120	5	5.8-16.0
Hao <i>et al.</i> 2023	HWSC	GFRP	10	60	4	7
Hu <i>et al.</i> 2023	HWSC	GFRP	16	45-160	5	2-4.5
Ke <i>et al.</i> 2024	HWSC	GFRP	10	40-85	5	7
This test	HWSC, SC	CFRP, GFRP	6, 10, 13, 16, 25	50-180	4, 5	3.4-12

Existing studies indicate that the bond strength is greatly affected by the casting position (Rossetti *et al.* 1995; Park *et al.* 2016). By comparing specimens from two different casting

positions in Figure 4.2, it shows that the bond strength of horizontally cast specimens is 82% of those cast vertically. Because it is most common to use horizontally cast specimens, the bond strength values of vertically cast specimens in the database have been adjusted by multiplying a reduction factor of 0.82. For ease of comparison, the bond strengths in the database have been recalculated using the nominal diameter of FRP bars, rather than the effective diameter, which varies significantly from one producer to another.

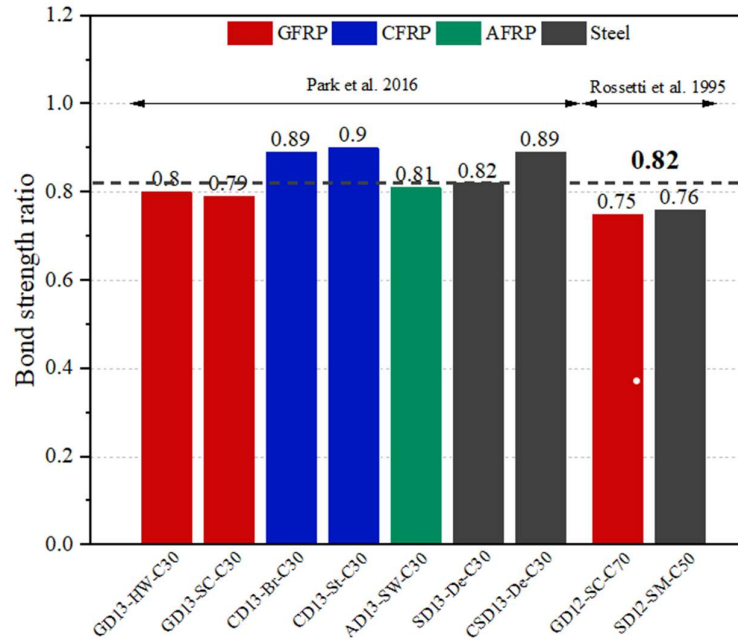


Figure 4.2. The ratio of bond strength measured from horizontally cast specimens to that from vertically cast specimens

Table 4.2. Conversion factors of concrete strength

Specimen	Concrete grade		
	NSC	HSC	UHSC
$f_{cm, cu-50}$	--	0.80 ^{A, B}	0.96 ^{A, B}
$f_{cm, cu-100}$	0.78 ^C	0.84 ^C	1.00 ^C
$f_{cm, cu-150}$	0.82 ^C	0.87 ^C	1.00 ^C
$f_{cm, cyl-75}$	--	--	1.00 ^{B, C}
$f_{cm, cyl-150}$	1.06 ^C	1.04 ^C	1.00 ^C

Note: A: Fládr and Bílý 2018; B: Graybeal and Davis 2008; C: Riedel *et al.* 2019

4.2.2. Distribution of key parameters

Figure 4.3 presents the number of specimens of different bar types for each fiber type (GFRP and CFRP). In total, 446 and 159 of these tests were conducted using HWSC and SC bars,

respectively, while only 82 were performed with In bars. Among the three bar types, HWSC bars accounted for approximately 65% of the test data. Detailed distributions of key parameters in the database are shown in Figure 4.4 and Figure 4.5. The concrete strength range from 20 MPa to 180 MPa, and the nominal diameter of bars varies from 6 mm to 25 mm. In the GFRP bar dataset, the majority of pullout tests were conducted on median-sized bars (Φ 13 to 16 mm) in an NSC matrix, comprising 27%, 47%, and 37% for HWSC, SC, and In bars, respectively. In comparison, fewer experimental studies have concerned with the bond of large-sized GFRP bars (Φ 19 to 25 mm) and those in a UHSC matrix, which account for 15% and 9% of the GFRP dataset, respectively.

As shown in Figure 4.3 (b), fewer tests were carried out with CFRP bars, which included only 86 specimens, representing 14% of the GFRP bars in the database. In addition, Figure 4.5 shows that no large-diameter and indented bars were included in the CFRP bars dataset. Thus, the CFRP bars dataset does not contain sufficient representation of different key factors. Therefore, the analysis of the database primarily focuses on the GFRP dataset in the next section.

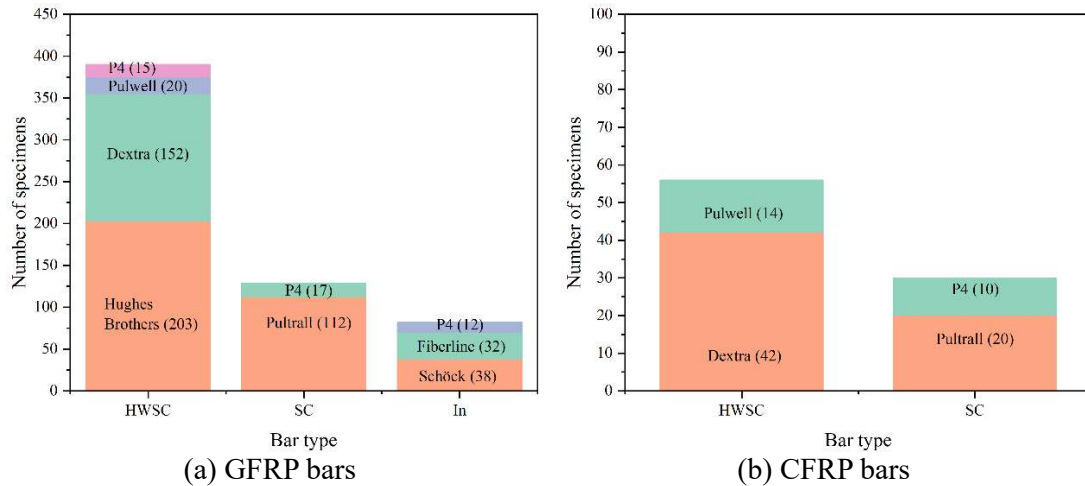


Figure 4.3. Distribution of bar type

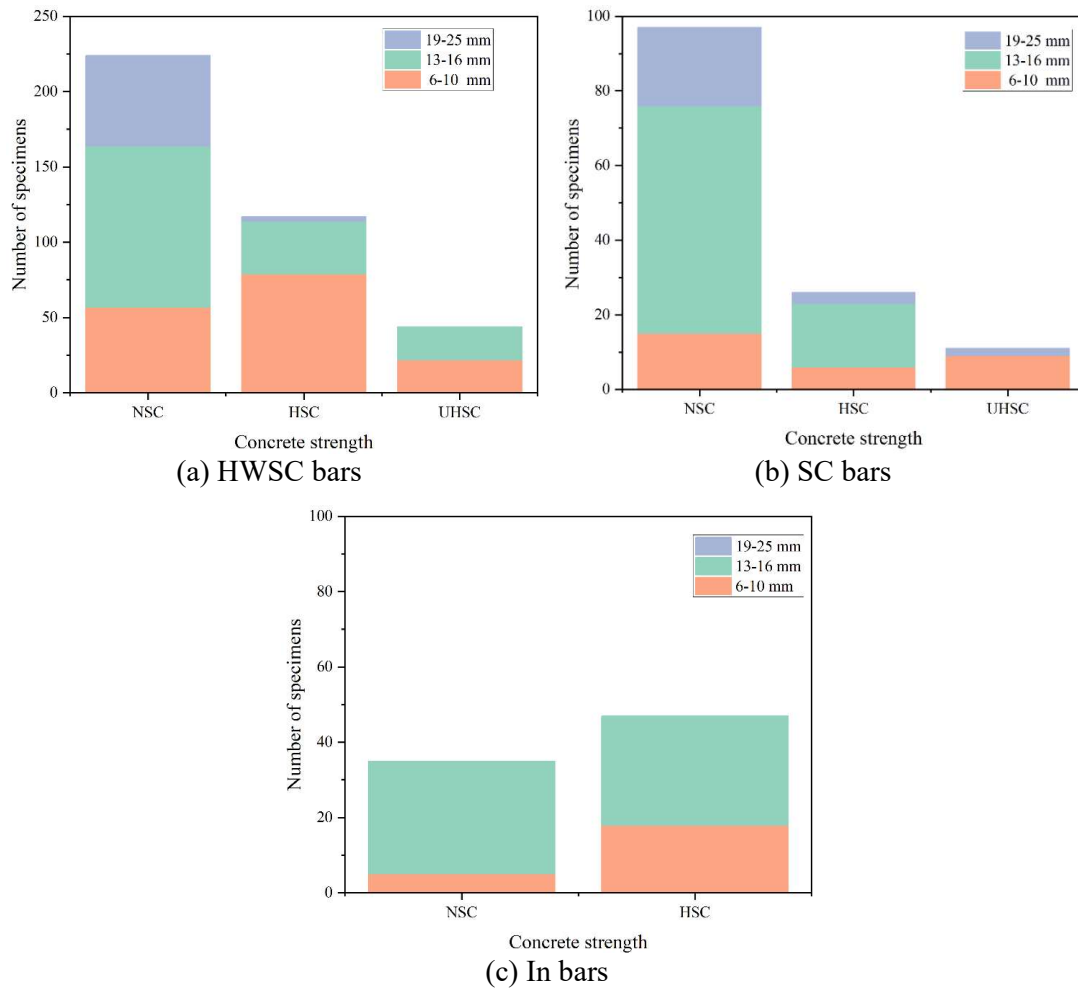


Figure 4.4. Distribution of GFRP bars

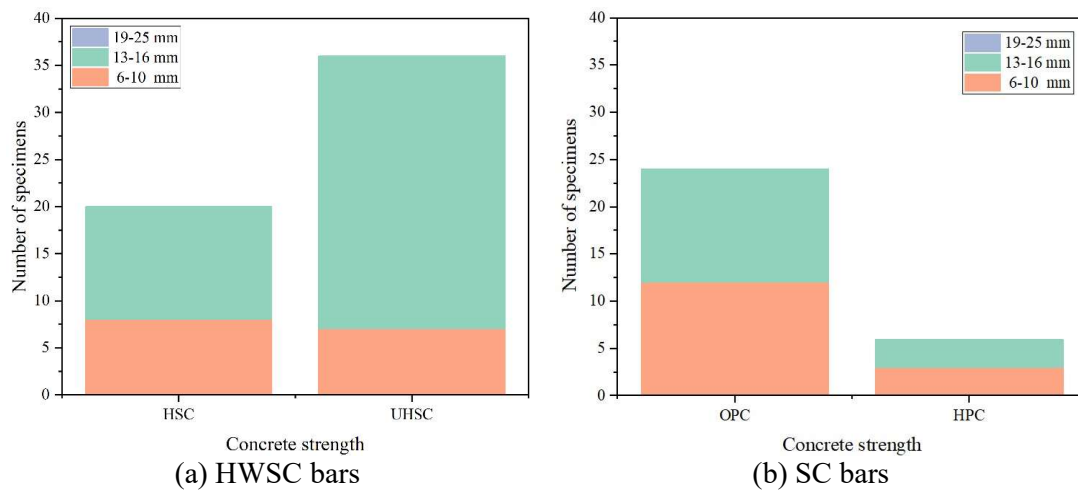


Figure 4.5. Distribution of CFRP bars

4.3. Statistical analysis

A statistical analysis using the ANOVA test was conducted to identify the interaction effect of key factors on bond performance, that is, whether the influence of a given parameter on bond strength is significantly affected by another. The independent factors considered are concrete strength, bar type, bar diameter, and fiber type. Each of the first three independent variables was categorized into three levels, as shown in Table 4.3. The fiber type was divided into two groups: glass fiber (G) and carbon fiber (C).

Table 4.3. Categories of independent variables

	Bar type	Concrete strength		Bar diameter	
HWSC (W)	Dextra Group.; Pulwell Composites Co., Ltd.; Hughes Brothers Inc.	NSC (O)	0-55 MPa	Small (X)	6-10 mm
SC (S)	Pultrall Inc.	HSC (H)	55-120 MPa	Median (M)	13-16 mm
In (I)	Schöck Canada Inc.; Fiberline Composites Inc.	UHSC (U)	120-200 MPa	Large (L)	19-25 mm

Note: the letter in the bracket denotes the level of independent variables.

4.3.1. Three-way ANOVA test

A three-way ANOVA test was first performed to investigate the interaction between the effects of the three factors (i.e., concrete strength, bar type, and bar diameter) on the bond strength in the GFRP dataset. Specifically, the response was modeled using the following linear function

$$y_{hijk} = \mu + \alpha_i + \beta_j + \gamma_k + (\alpha\beta)_{ij} + (\alpha\gamma)_{ik} + (\beta\gamma)_{jk} + (\alpha\beta\gamma)_{ijk} + \varepsilon_{hijk} \quad (4.1)$$

where y_{hijk} is the observed value, μ is the grand mean, α_i , β_j , and γ_k are respectively the additive main effects of level i, j , and k from the first, second and third factor, $(\alpha\beta)_{ij}$, $(\alpha\gamma)_{ik}$, and $(\beta\gamma)_{jk}$ are non-additive interaction effects of each two factors, $(\alpha\beta\gamma)_{ijk}$ is the non-additive 3-way interaction effect from three factors for samples h ($h = 1, \dots, n_{ijk}$), and ε_{hijk} is the error.

The null hypothesis is that the effect of each factor on the bond strength does not have a

statistically significant influence on the effects of the other two factors. It can be rejected if the significance level is lower than 0.05. Note that due to the absence of some test data, as mentioned in section 4.3.2., there are 12 groups in this 3-way test (see Table 4.4), representing the combination of two levels of bar diameter, three levels of bar type, and two levels of concrete strength.

Table 4.4. Descriptive statistics of the three-way ANOVA test

Group	df	Normality test		Variance
		Statistic	p-value	
XWO	57	0.983	0.580	4.736
XWH	81	0.979	0.214	6.295
XSO	20	0.953	0.418	3.677
XSH	8	0.832	0.063	14.092
XIO	5	0.896	0.390	2.705
XIH	14	0.912	0.168	2.200
MWO	107	0.986	0.356	5.577
MWH	35	0.939	0.053	6.759
MSO	63	0.975	0.238	8.243
MSH	20	0.942	0.267	6.187
MIO	30	0.956	0.246	4.097
MIH	20	0.957	0.479	8.455

Note: *df* means the degree of freedom.

Table 4.5. Three-way ANOVA test results

	df	SS	MS	F	p	Power
Bar diameter	1	187.737	187.737	31.195	0.000	1.000
Bar type	2	1352.115	676.058	112.336	0.000	1.000
Concrete strength	1	1061.375	1061.375	176.361	0.000	1.000
Bar diameter * bar type	2	134.283	67.141	11.156	0.000	1.000
Bar diameter * concrete strength	1	42.889	42.889	7.127	0.008	0.965
Bar type * concrete strength	2	84.533	42.267	7.023	0.001	0.999
Bar diameter * bar type * concrete strength	2	8.240	4.120	0.685	0.505	0.424
Model	11	5457.573	496.143	82.440		
Error	445	2678.098	6.018			
Corrected Total	456	8135.670				

Note: SS = sum of squares of deviation; MS = mean square; F = value of F-test; p = p-value.

The boxplot in Figure 4.6 shows that no extreme points but only a few outliers are presented in the test groups. The outliers were not weeded out from the dataset, as they lay only slightly beyond the box whiskers and do not significantly impact the robustness of the ANOVA test.

As shown in Table 4.4, the dependent variables were normally distributed within each group, with all p -values exceeding 0.05. In addition, Table 4.4 shows no statistically significant differences in variance for each group, with the p -value higher than 0.05 in the Levene test. Table 4.5 presents the results of the 3-way ANOVA test. It shows that the p -value of 3-way interaction term was higher than the significance level of 0.05 ($F(2, 445) = 0.685, p = 0.505$). This indicates an insignificant 3-way interaction between the effects of concrete strength, bar type, and bar diameter on bond strength.

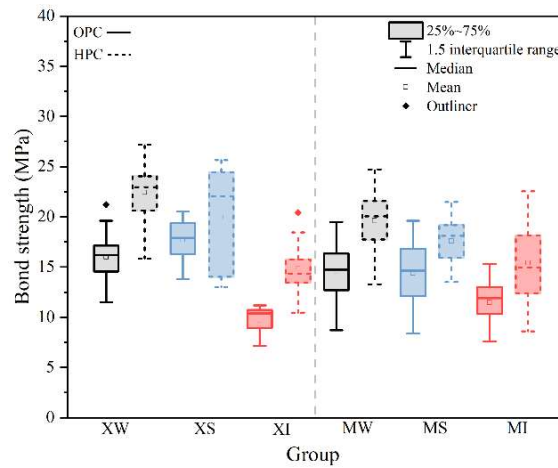


Figure 4.6. Boxplot of the dependent variable of different groups

4.3.2. Two-way ANOVA test

Since the 3-way joint effect was not statistically significant, the 3-way interaction term was removed from the analysis model (Eq. (4.1)) in the following ANOVA test. A 2-way ANOVA was then implemented to determine whether there are significant 2-way interactions between the effects of key parameters on bond strength. The analysis results are presented in Table 4.6. It shows that the p -values of the three interaction terms were all lower than 0.05 ($p(\text{bar diameter} * \text{bar type}) = 0.000$, $p(\text{bar diameter} * \text{concrete strength}) = 0.012$, and $p(\text{bar type} * \text{concrete strength}) = 0.000$). This indicates that the three 2-way interaction effects of every two factors on bond strength are statistically significant, that is, between the effects of bar type and concrete strength, the effects of bar type and bar diameter, as well as the effects of diameter and concrete strength.

After the ANOVA test, post hoc tests were conducted to find out how the effect of a key factor

interacts with that of another one on bond strength by determining whether there are significant differences between the mean measured bond strengths with multiple groups of four key factors. Post hoc tests were performed using the Bonferroni method on groups of every two key factors at a 95% confidence interval. The test results are shown in Table 4.7 to Table 4.9 and Figure 4.7. Note that the difference in bond strength of groups with a p value of less than 0.05 in Table 4.7 to Table 4.9 and groups within the dashed ellipse in Figure 4.7 isn't statistically significant.

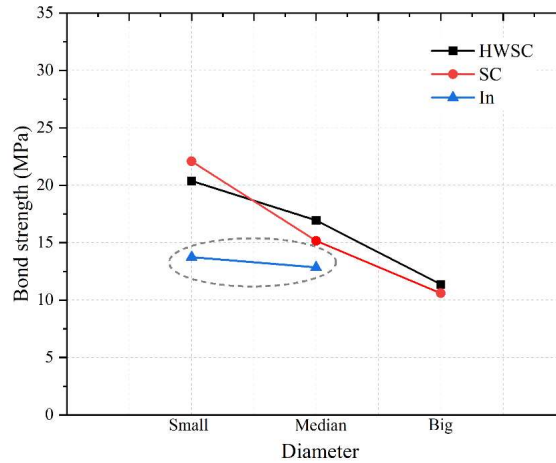
Table 4.6. ANOVA test result without 3-way interaction

	<i>df</i>	SS	MS	F	p	Power
Bar diameter	1	234.957	234.957	39.096	0.000	1.000
Bar type	2	1512.663	756.332	125.852	0.000	1.000
Concrete strength	1	1212.829	1212.829	201.812	0.000	1.000
Bar diameter * bar type	2	136.314	68.157	11.341	0.000	1.000
Bar diameter * concrete strength	1	38.462	38.462	6.400	0.012	0.947
Bar type * concrete strength	2	113.468	56.734	9.440	0.000	1.000
Model	9	5449.332	605.481	100.751		
Error	447	2686.338	6.010			
Corrected Total	456	8135.670				

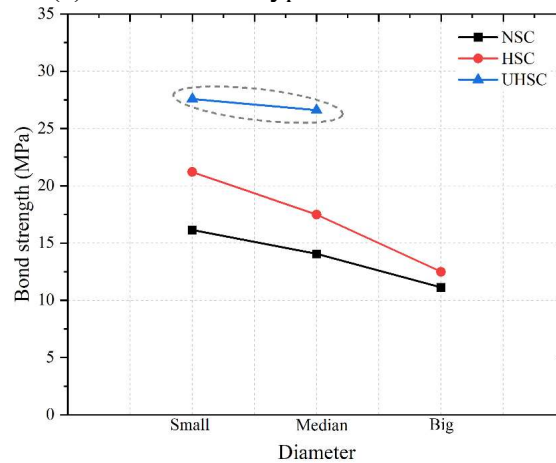
For both HWSC and SC bars, Table 4.7 reveals that the bond strength of bars with small diameters was statistically significantly higher than that of median diameter bars ($p = 0.000$), followed by that of large diameter bars ($p = 0.000$). Figure 4.7 (a) also shows that the increasing rate of bond strength with respect to bar diameter differed in HWSC and SC bars. However, for In bars, small bars did not yield significantly higher bond strength than median bars ($p = 1.000$). These results indicate that the bond strength of both HWSC and SC bars showed statistically remarkable size dependence, and the impact of bar diameter on bond strength varied from HWSC to SC bars, whereas the bar diameter of In bars dose not have a statistically significant effect on bond strength. For In bars, as discussed in section 2.2.3.4., this is attributed to the pronounced effect of bar rib or lug size, rather than the bar diameter. Consequently, it can be concluded that the influence of bar diameter on bond strength significantly differs by bar type, which is consistent with the experimental observations in section 2.2.3.4.

For the coupled effect of bar diameter and concrete strength, the statistical analyses in Figure 4.7 (b) and Table 4.8 confirm the discussion in sections 2.2.3.4 and 3.3.3.5 that the effect of bar diameter on bond strength varies with concrete strength, although the size effect on bond

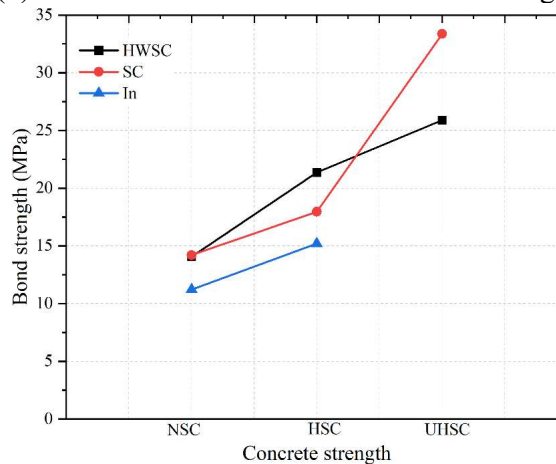
strength is sometimes obscured by irregularities of surface finish. In UHSC, there is no statistically significant size effect ($p = 1.000$), while the significant effect of bar diameter can be observed in both NSC and HSC, with all p values being lower than 0.05.



(a) Effects of bar type and bar diameter



(b) Effects of bar diameter and concrete strength



(c) Effects of bar type and concrete strength

Figure 4.7. Post hoc test results: pairwise comparisons of bond strength of FRP bars

Table 4.7 Post hoc test: joint effect of bar type and bar diameter on bond strength

Bar type	Bar diameter	Comparison diameter	Mean difference	t value	p	95% Confidence interval	
						Lower bound	Upper bound
HWSC	Small	Median	3.42	7.12	0.000	2.01	4.84
		Large	9.01	14.50	0.000	7.18	10.84
	Median	large	5.59	8.99	0.000	3.75	7.42
SC	Small	Median	6.92	8.35	0.000	4.47	9.36
		Large	11.49	10.72	0.000	8.33	14.65
	Median	large	4.57	4.80	0.000	1.76	7.38
In	Small	Median	0.89	0.84	1.000	-2.21	3.99

Table 4.8 Post hoc test: joint effect of concrete strength and bar diameter on bond strength

Concrete strength	Bar diameter	Comparison diameter	Mean dif.	t	p	95% Confidence interval	
						Lower bound	Upper bound
NSC	Small	Median	2.09	5.03	0.000	0.86	3.31
		Large	4.87	9.61	0.000	3.37	6.36
	Median	large	2.93	7.21	0.000	1.73	4.13
HSC	Small	Median	3.72	7.79	0.000	2.31	5.13
		Large	8.24	6.41	0.000	4.45	12.03
	Median	large	5.00	3.83	0.002	1.15	8.84
UHSC	Small	Median	0.97	1.10	1.000	-1.62	3.56

Table 4.9 Post hoc test: joint effect of concrete strength and bar type on bond strength

Bar type	Concrete strength	Comparison concrete strength	Mean dif.	t	p	95% Confidence interval	
						Lower bound	Upper bound
HWSC	NSC	HSC	-7.28	-20.36	0.000	-8.34	-6.23
		UHSC	-11.83	-23.36	0.000	-13.32	-10.34
	HSC	UHSC	-4.55	-8.39	0.000	-6.14	-2.95
SC	NSC	HSC	-3.76	-5.89	0.000	-5.64	-1.88
		UHSC	-19.17	-17.97	0.000	-22.32	-16.03
	HSC	UHSC	-15.42	-13.31	0.000	-18.83	-12.00
In	NSC	HSC	-4.00	-5.67	0.000	-6.07	-1.92

Table 4.9 shows that FRP bars in concrete with different strength grades yield different bond strengths that are statistically significant ($p \leq 0.05$). For the three types of FRP bars, the bond

strength of specimens with larger concrete strength is significantly higher than that of specimens with lower concrete strength. However, the increasing rate of bond strength with respect to concrete strength for these three types of FRP bars is statistically significantly different, as shown in Figure 4.7 (c). This means the influence of concrete strength on bond strength depends on bar type. This analysis thus verifies the interaction effect between concrete strength and bar type on bond strength, as suggested in section 2.2.3.2.

Table 4.10. Descriptive statistics of the two-way ANOVA test

Group	df	Normality test		Variance
		Statistic	p-value	
GO	240	0.061	0.334	6.864
GH	137	0.08	0.338	10.915
GU	44	0.134	0.373	6.571
CO	24	0.947	0.237	4.104
CH	25	0.965	0.532	10.924
CU	36	0.978	0.687	9.825

Table 4.11. ANOVA test results of bond strength

	df	SS	MS	F	p	Power
Fiber type	1	139.248	139.248	16.947	0.000	0.984
Concrete strength	2	7274.640	3637.322	442.688	0.000	1.000
Fiber type * concrete strength	2	101.916	50.958	6.202	0.002	0.996
Model	5	11705.300	2341.053	284.923		
Error	497	4083.570	8.216			
Corrected Total	502	15788.800				

Table 4.12 Post hoc test results: joint effect of concrete strength and fiber type

Concrete strength	Fiber type	Comparison fiber type	Mean dif.	t	p	95% Confidence interval	
						Lower bound	Upper bound
NSC	GFRP	CFRP	-0.03	-0.06	1.000	-1.84	1.78
HSC	GFRP	CFRP	-1.28	-2.06	0.605	-3.12	0.56
UHSC	GFRP	CFRP	-3.16	-4.89	0.000	-5.07	-1.25

Due to inadequate test data on CFRP bars with different bar diameters and bar types in the CFRP bar dataset, the statistical analysis was performed considering only the interaction effect of fiber type and concrete strength. Figure 4.8 shows only a few outliers in the “GU” group that are close to the upper limit of 1.5 times the interquartile range. They were not excluded from the ANOVA test as before. The normality and homogeneity of variances are

examined in Table 4.10, with p-values higher than 0.05.

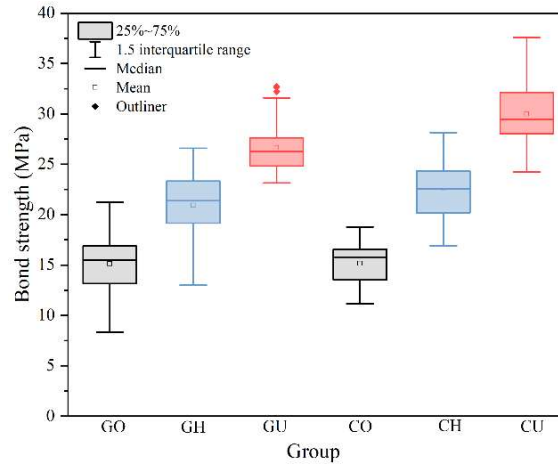


Figure 4.8. Boxplot of dependent variable

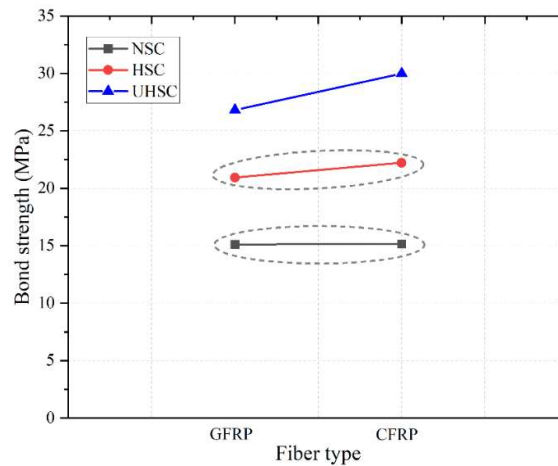


Figure 4.9. Pairwise comparison of the joint effect of fiber type and concrete strength

The results in Table 4.11, verify that the 2-way interaction effect of fiber type and concrete strength on bond strength is significant ($p = 0.002$). The pairwise comparison in Figure 4.9 and Table 4.12 reveals no statistically significant difference in the bond strength of GFRP and CFRP bars in both NSC ($p = 1.000$) and HSC ($p = 0.605$) matrices. However, in UHSC, CFRP bars yield significantly higher bond strength (by 12%) than GFRP bars, with the p-value being lower than 0.05. These statistical results are consistent with experimental observations in sections 2.2.3.6 and 3.3.3.4. Consequently, to account for the 2-way interaction effect between fiber type and concrete strength on bond strength of FRP bars, it is recommended to use fiber multiplicative factors of 1.12 and 1.00 for CFRP bars in UHSC and the other cases, respectively, in the bond strength equation.

4.4. Assessment of existing bond strength models

4.4.1. Assessment indicators

The statistical analysis in the previous section has confirmed strong 2-way correlations between the impacts of independent factors on bond strength. This suggests that interpreting the impacts of these factors on bond strength as standalone coefficients, a common practice in previous research (Yan *et al.* 2016; Wang *et al.* 2024), is not reasonable. Thus, several existing bond strength models were first evaluated in this section using the test results in the created database. For comparison, statistical indicators, including the mean (M), CoV, root mean squared error (RMSE), and mean absolute percent error (MAPE), are used. They are calculated as follows

$$M = \frac{1}{n} \sum_{i=1}^n \frac{\tau_{u-exp,i}}{\tau_{u-pre,i}} \quad (4.2)$$

$$CoV = \frac{100}{M} \sqrt{\frac{1}{n-1} \left(\sum_{i=1}^n \left[\frac{\tau_{u-exp,i}}{\tau_{u-pre,i}} \right]^2 - nM^2 \right)} \quad (4.3)$$

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^n [\tau_{u-pre,i} - \tau_{u-exp,i}]^2} \quad (4.4)$$

$$MAPE = \frac{1}{n} \sum_{i=1}^n \left| \frac{\tau_{u-pre,i} - \tau_{u-exp,i}}{\tau_{u-exp,i}} \right| \quad (4.5)$$

where n is the number of specimens, $\tau_{u-pre,i}$ and $\tau_{u-exp,i}$ represent the i -th predicted and experimental bond strength, respectively. The existing bond strength models studied are summarized in Table 4.13. It should be noted that a reduction factor of 0.82, accounting for the effect of casting orientation, was applied to three models (Okelo and Yuan 2005; Peng *et al.* 2022; Kim *et al.* 2023), which were developed based on test results from vertically cast specimens.

Table 4.13. Summary of existing bond strength models

Ref.	Models	Limitations
Okelo and Yuan 2005	$\tau_u = 14.7 \sqrt{f_c}/d$ (V)	f_c : NSC, HSC d : 6-19 mm
Lee <i>et al.</i> 2012	$\tau_u = 4f_c^{0.33}$	f_c : NSC, HSC d : 13 mm
Xue <i>et al.</i> 2014; Hu <i>et al.</i> 2023	$\tau_u = \alpha' \phi(8.00 - 0.17d) \left(\frac{L}{d}\right)^{-0.4} f_t$ for NSC HSC $\tau_u = \alpha(0.9 + 0.4 \frac{C}{d} + 3.0 \frac{d}{L}) \sqrt{f_c}$ for UHSC	f_c : NSC, HSC, UHSC d : 6-19 mm
Yoo <i>et al.</i> 2015	$\tau_u = (1.05 + 0.85 d/L) \sqrt{f_c}$	f_c : HSC, UHSC d : 13-19 mm
Shen <i>et al.</i> 2016	$\tau_u = (0.72 + 0.17 C/d) \sqrt{f_c}$	f_c : NSC, HSC d : 10-19 mm
Nepomuceno <i>et al.</i> 2021	$\tau_u = \alpha f_c^\beta$	f_c : NSC, HSC, UHSC d : 6-25 mm
Peng <i>et al.</i> 2022	$\tau_u = (1.4 + 3/d + 5d/L) \sqrt{f_c}$ (V)	f_c : NSC, HSC d : 3-10 mm
Kim <i>et al.</i> 2023	$\tau_u = \sqrt{f_c}/(4\varpi)$ (V)	f_c : UHSC d : 10 mm
Jiang <i>et al.</i> 2024	$\tau_u = \frac{d_{cs} \sqrt{f_c}}{1.15 k_1 k_2 k_3 k_4 k_5 \pi d} \times 14.23 \frac{d}{L}$	f_c : HSC, UHSC d : 8-16 mm
Wang <i>et al.</i> 2024	$\tau_u = f_c^{0.278} (5.536 + 0.15 \frac{C}{d})$ for HWSC $\tau_u = f_c^{0.249} (1.503 + 0.204 \frac{C}{d} + 20.169 \frac{d}{L})$ for SC	f_c : NSC, HSC d : 6-20 mm

Note: expressions marked with “(V)” are the models derived from vertically cast pullout specimens.

4.4.2. Assessment results

Figure 4.10 and Figure 4.11 show plots of test bond strength versus predicted values for HWSC and SC bars, respectively. The assessment results, including M, CoV, RMSE, and MAPE, for each model, are presented in Table 4.14.

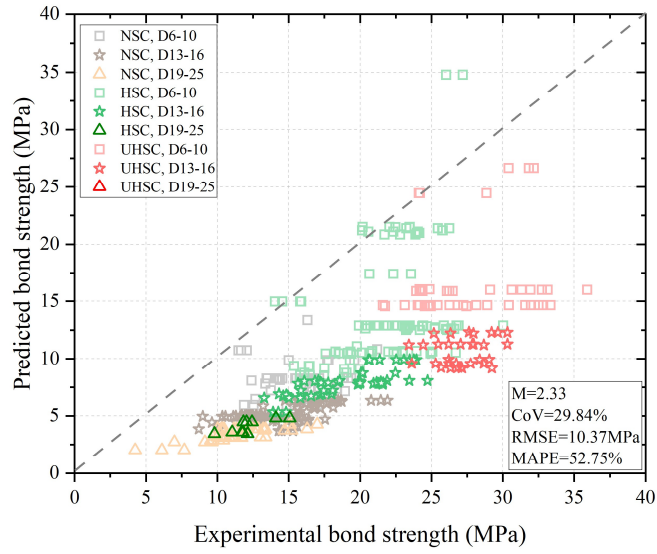
It can be found that models of Okelo and Yuan (2005), Yoo *et al.* (2015), Shen *et al.* (2016), and Jiang *et al.* (2024) all provided conservative predictions for both types of FRP bars. Notably, the model developed by Jiang *et al.* (2024) had the highest mean experimental-to-predicted ratio ($M = 4.79$ and 4.97 for HWSC and SC bars, respectively) and the largest prediction errors, with RMSE of 11.89 MPa to 14.37 MPa and MAPE of 72.37%-72.58%. For Shen *et al.*'s (2016) model, a minority of overestimations were observed for bars of small

diameters. This is attributed to the incorrect evaluation of the effect of cover thickness in their model. In standard pullout specimens, small bars generally have quite thick concrete covers, whose contribution to bond strength is overestimated without an upper limit in the predictive model.

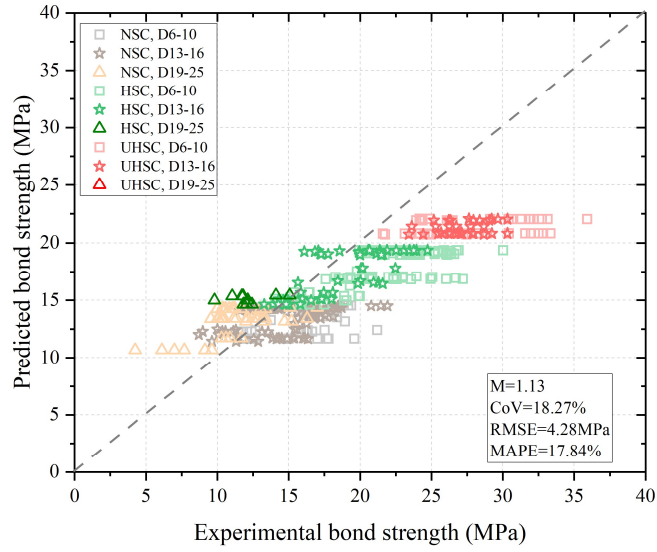
Xue *et al.*'s team evaluated the bond performance of GFRP bars in NSC, HSC, and UHSC, and proposed two models for different concrete grades (Xue *et al.* 2014; Hu *et al.* 2023). Due to limited test data, the effect of bar type was not accurately assessed in their models, resulting in an obvious overestimation for HWSC bars and underestimation for SC bars. Recently, Wang *et al.* (2024) developed several predictive equations considering the geometrical factors of surface deformation based on a comprehensive database. However, Wang *et al.*'s (2024) models significantly overestimated the bond strength of most HWSC and SC bars, probably due to the marginal effect of surface deformation for these bar types.

In comparison, closer agreement was achieved with the predictive models proposed by Lee *et al.* (2012), Nepomuceno *et al.* (2021), and Kim *et al.* (2023), with mean predicted-to-experimental ratios ranging from 0.86 to 1.11. However, these models showed significant differences for FRP bars of different sizes, leading to considerable prediction errors, as the influence of bar diameter on bond strength is ignored. These models generally underestimated the bond strength of small-diameter bars ($\Phi 6$ to $\Phi 10$) and overestimated that of large-diameter bars ($\Phi 19$ to $\Phi 25$). For Peng *et al.*'s model, even though the size effect is considered, overestimation is also observed for the large-diameter bars, as no large bars were included in their dataset for developing the bond strength model.

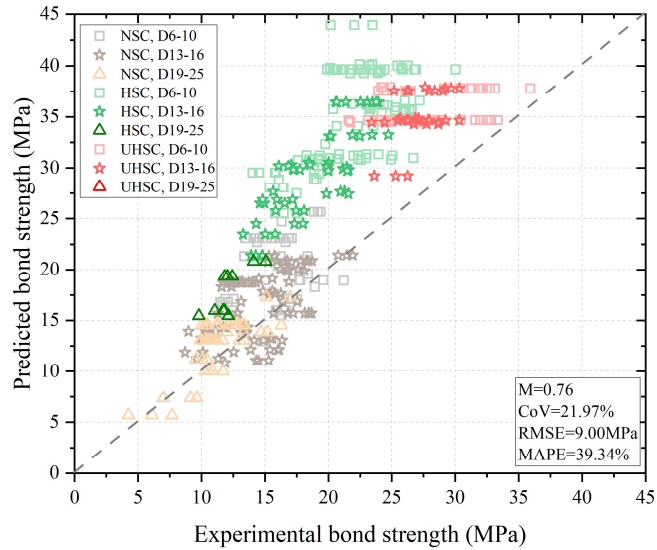
Figure 4.10 and Figure 4.11 demonstrate that the bond strength of FRP bars in UHSC cannot be accurately predicted by most existing models, except for the models developed by Kim *et al.* (2023) and Wang *et al.* (2024). It is evident that there is no upper limit to bond strength predicted by all existing bond strength models summarized in Table 4.13. Consequently, even in the UHSC matrix, the predicted bond strength continuously increases with the strength of concrete, which is contrary to the experimental observation discussed in sections 2.2.3.2 and 3.3.3.2: the bond strength in UHSC has almost no dependency on concrete strength.



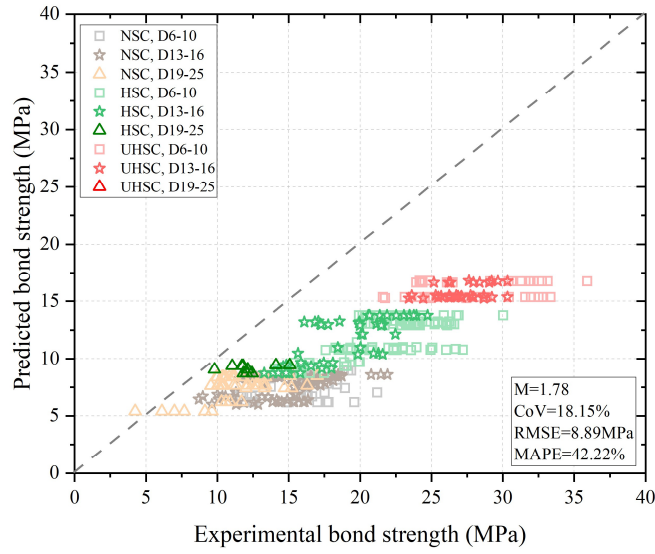
(a) Okelo and Yuan 2005



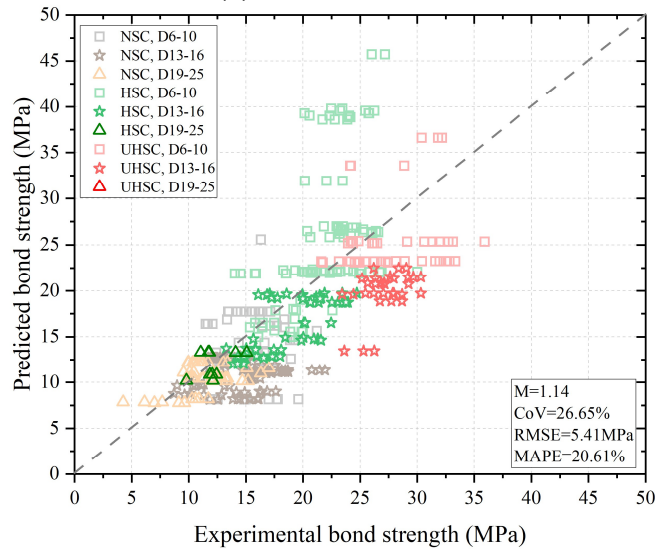
(b) Lee *et al.* 2012



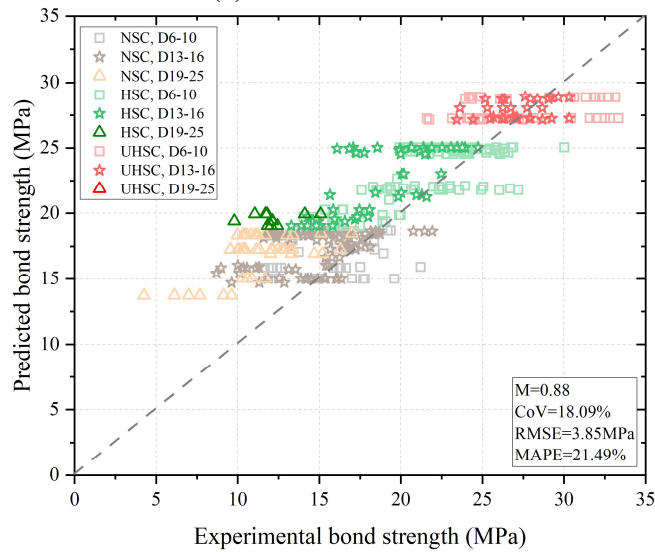
(c) Xue *et al.* 2014; Hu *et al.* 2023



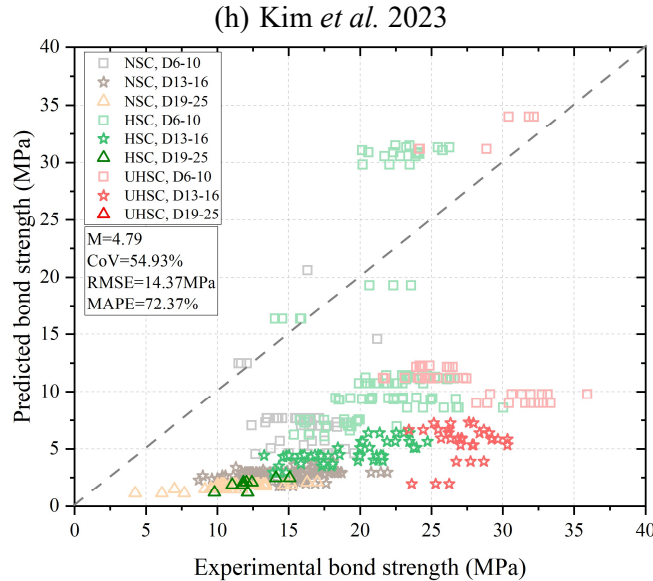
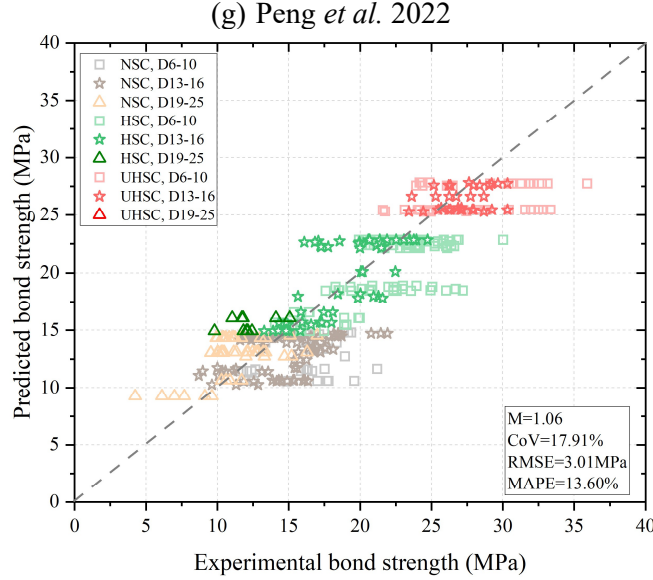
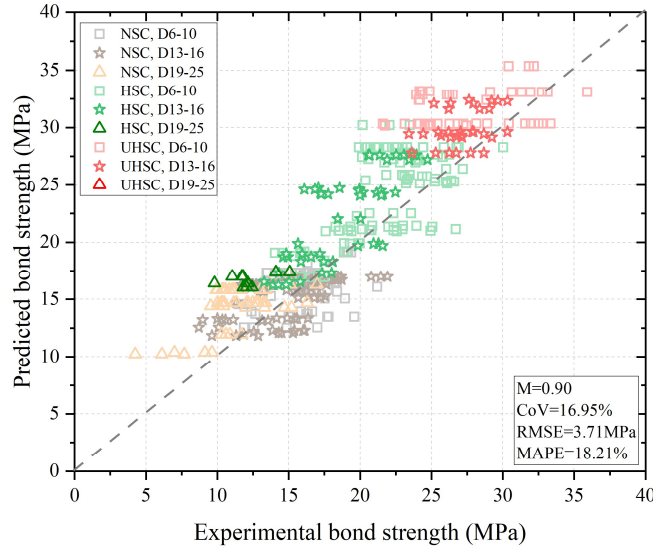
(d) Yoo *et al.* 2015

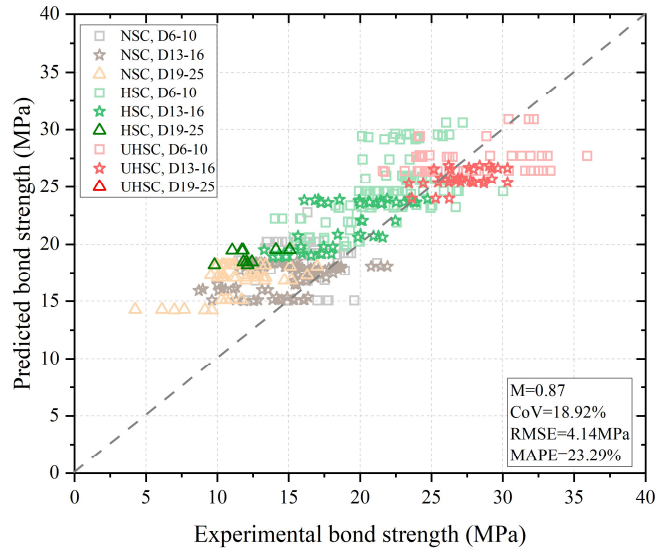


(e) Shen *et al.* 2016

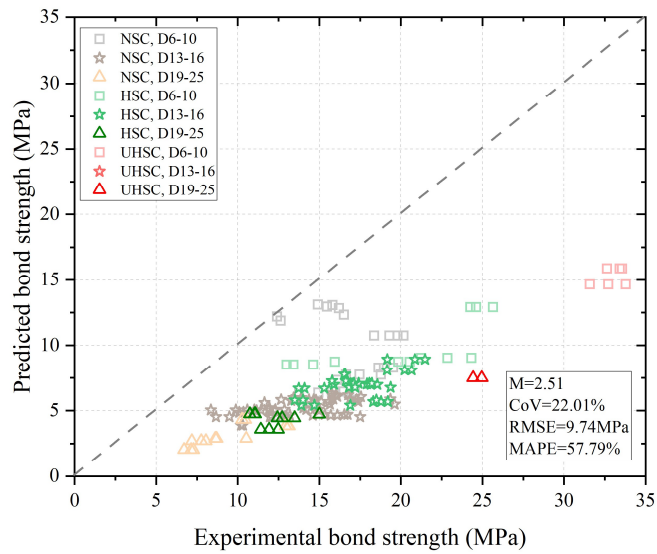


(f) Nepomuceno *et al.* 2021

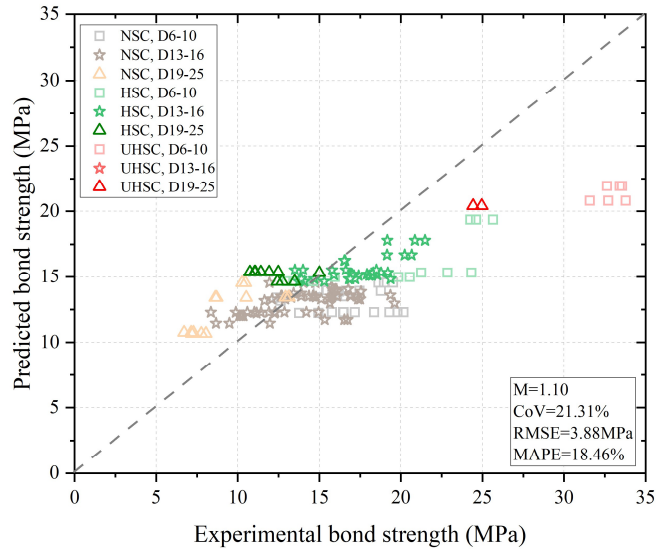




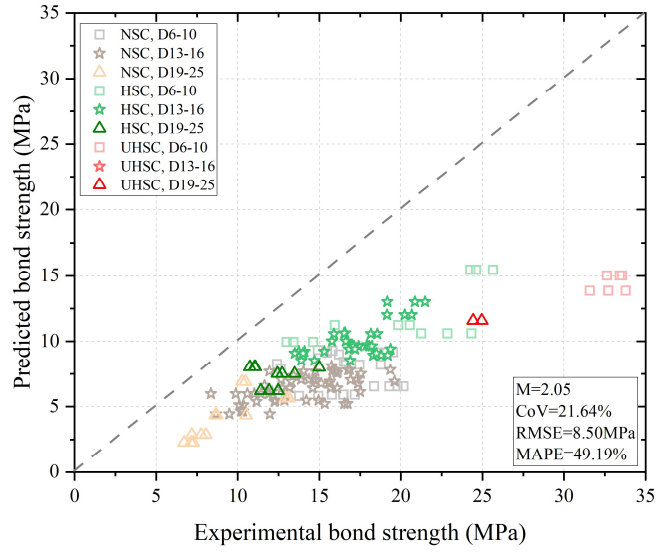
(j) Wang *et al.* 2024
Figure 4.10. Experimental versus predicted bond strength of HWSC bars



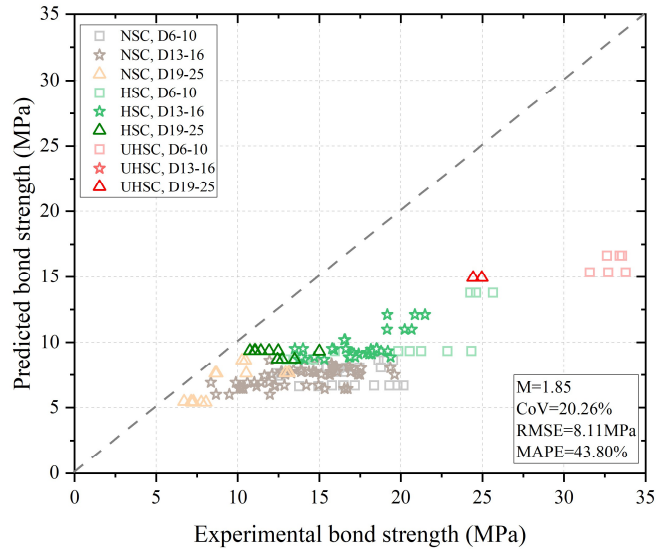
(a) Okelo and Yuan 2005



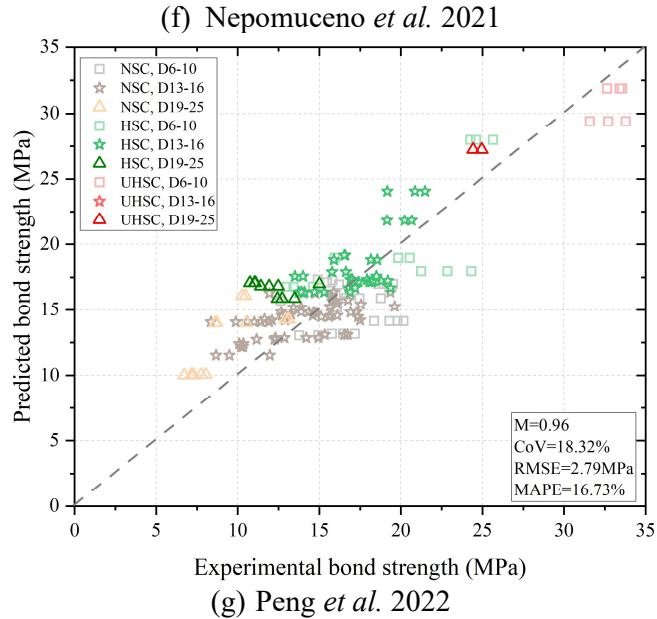
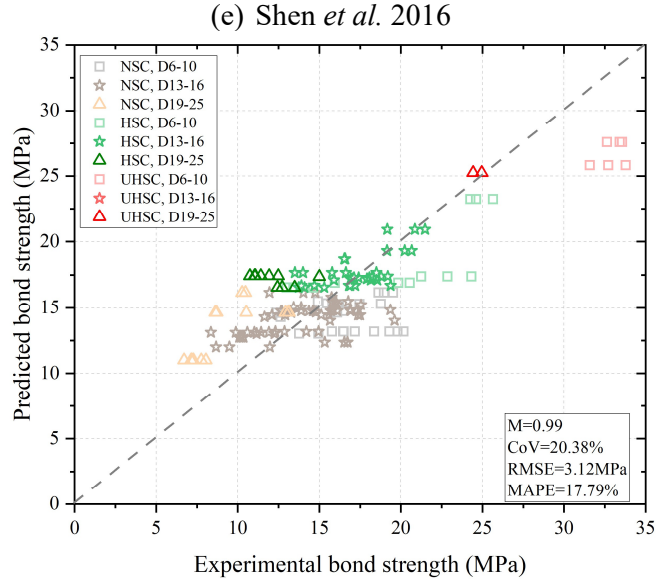
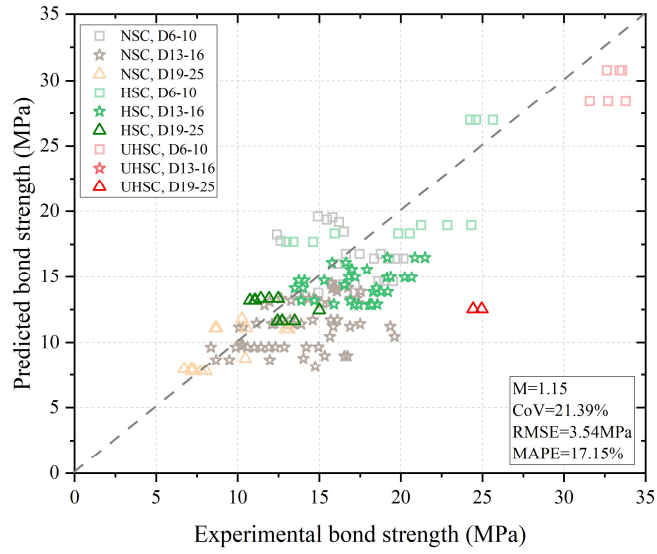
(b) Lee *et al.* 2012

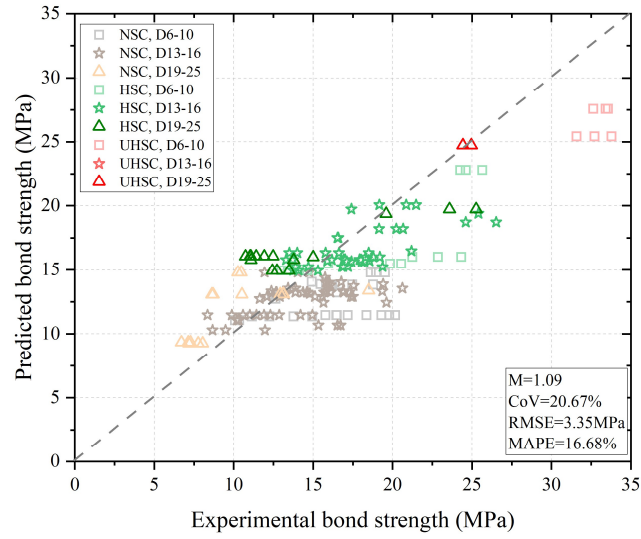


(c) Xue *et al.* 2014; Hu *et al.* 2023

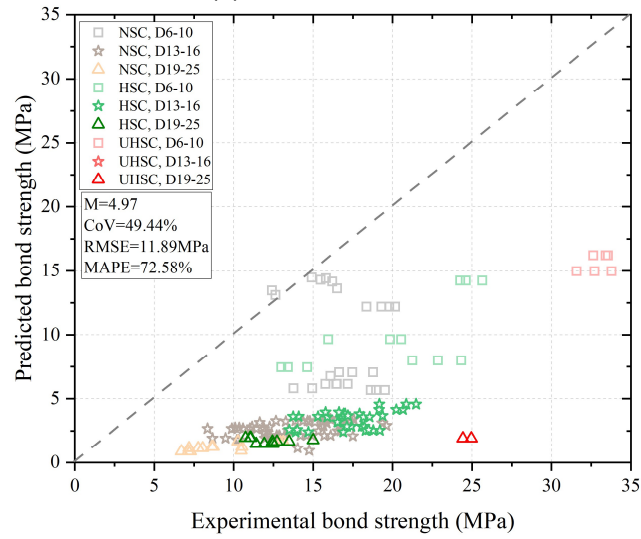


(d) Yoo *et al.* 2015

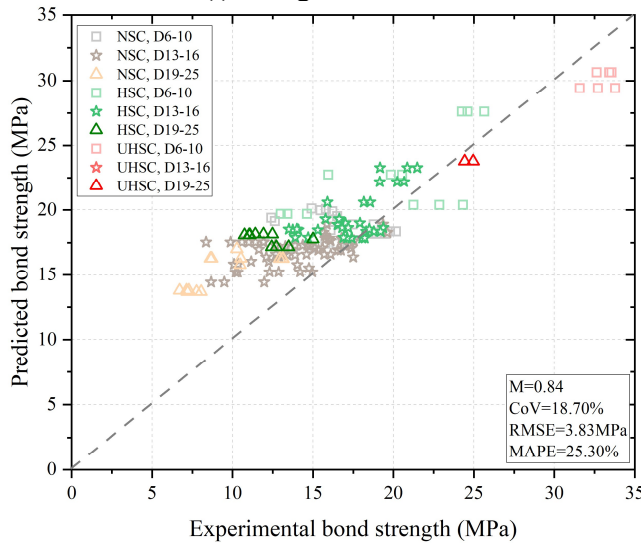




(h) Kim *et al.* 2023



(i) Jiang *et al.* 2024



(j) Wang *et al.* 2024

Figure 4.11. Experimental versus predicted bond strength of SC bars

4.5. Development of a new bond strength predictive equation

To accurately assess the bond strength of FRP bars, this study considers these statistically significant 2-way interaction effects. Given the significant interaction effects of bar type and bar diameter, as well as that of bar type and concrete strength, bond strength is examined separately for each bar type. Since the test results for In GFRP bars in the database are limited, as described in section 4.2.2, and detailed information about rib geometries that have a greater impact on bond strength than bar diameter is scarce, the following analysis of bond strength focuses only on HWSC and SC bars.

Based on the discussion in section 2.2.3.4., the general size effect law is applied to evaluate the influence of bar diameter on bond strength as follows

$$\tau_u = Bf(1 + \beta_d)^{-\frac{1}{2}} \quad (4.6)$$

where τ_u is the bond strength; f is a strength parameter; B is a regression coefficient; and β_d is the relative size defined as

$$\beta_d = d/d_0 \quad (4.7)$$

in which d is the characteristic size treated as the bar diameter; d_0 is a transitional size, determined by regression of test data. Note that both B and d_0 are material characteristics irrelevant to the size of structural members (Bažant 1984). To calculate the regressed parameters Bf and d_0 , Eq. (4.6) can be rearranged in a linear plot as follows

$$Y = A_\tau d + C_\tau \quad (4.8)$$

where $Y = l / \tau_u^2$; $C_\tau = l / (Bf)^2$; $A_\tau = C_\tau / d_0$. For each concrete grade i , the slope $A_{\tau i}$ and the intercept $C_{\tau i}$ can be calculated by linear regression of Eq. (4.8) using the test data (d_i , τ_{ui}) for two types of GFRP bars, as illustrated in Figure 4.12. Then, as shown in Figure 4.13 and Figure 4.14, $(Bf)_i$ and d_{0i} can be obtained for different concrete grades.

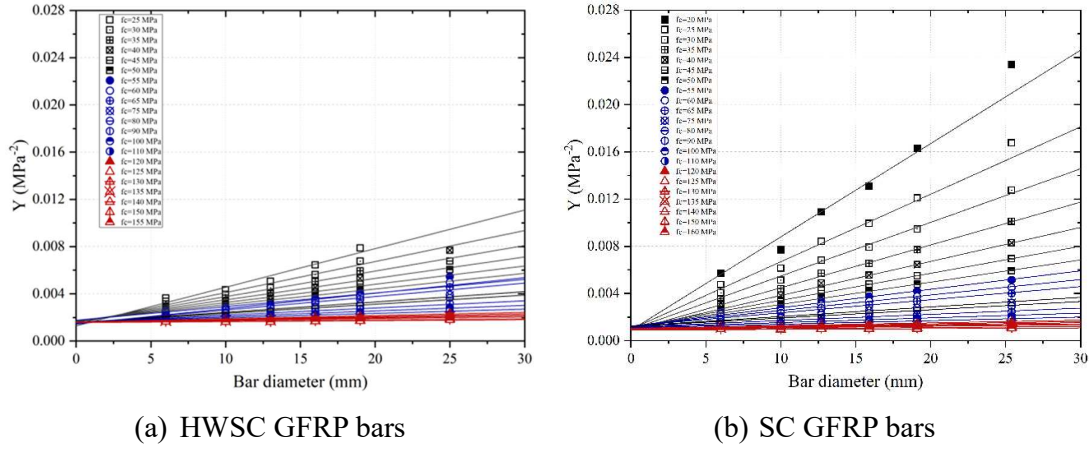


Figure 4.12. Determination of regression parameters

From Eq. (4.6), it can be found that when the bar diameter is much smaller than the transitional size d_0 , the bond strength levels off at Bf . Clearly, Bf represents the upper limit of the bond strength for a specific type of FRP bars. Figure 4.13 shows that the mean values of Bf do not vary significantly with concrete strength. On average, SC bars exhibit higher Bf values than HWSC bars. This suggests that Bf depends mainly on the properties, specifically the shear resistance as discussed in sections 2.2.3.5 and 3.3.3.4, of a particular bar type rather than the strength of the concrete. Consequently, regardless of concrete strength, Bf is identified as 27 MPa for HWSC GFRP bars and 32 MPa for SC GFRP bars.

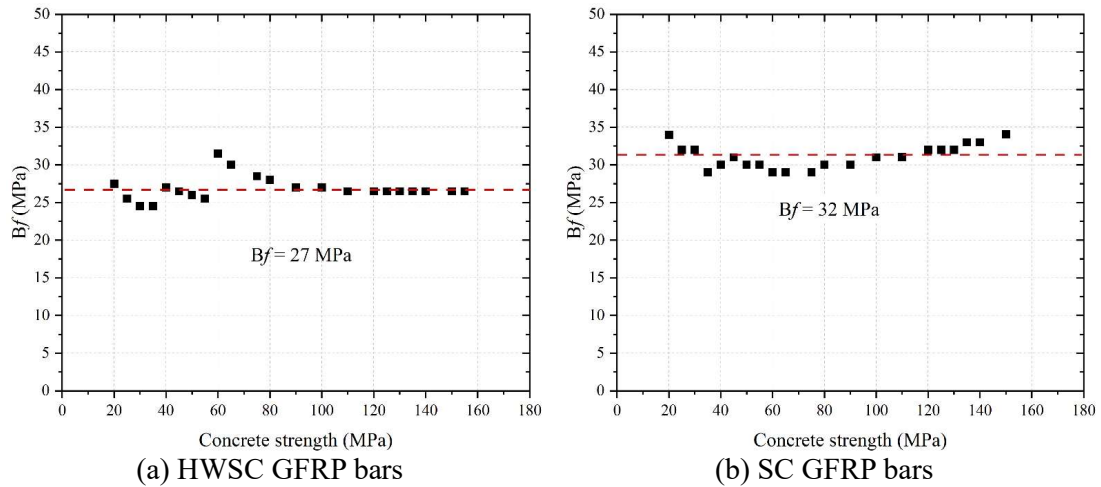


Figure 4.13. Regressed parameter Bf

Figure 4.14 presents the mean values of the transitional size d_0 with concrete strength, regressed from two datasets. d_0 shows a strong positive correlation with the compressive

strength of concrete for each bar type, as fitted in Eq. (4.9) and (4.10) for HWSC and SC bars respectively

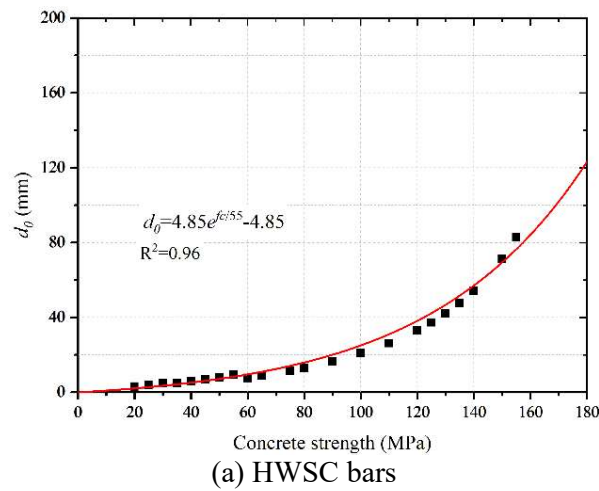
$$d_0 = 4.85(e^{f_c/55} - 1) \quad (4.9)$$

$$d_0 = 1.47(e^{f_c/36} - 1) \quad (4.10)$$

where f_c is the mean compressive strength of $\Phi 100 \times 200$ concrete cylinders. These fitting equations represent the joint influence of bar diameter and concrete strength on bond strength: an increase of f_c increases d_0 , which leads to a reduction of relative size β in Eq. (4.7) for a given bar diameter and thus a reduced size effect as reflected in Eq. (4.6). The bond strength for HWSC and SC GFRP bars is thus proposed as follows

$$\tau_u = 27 \left(1 + \frac{0.21d}{e^{f_c/55} - 1} \right)^{-0.5} \quad \text{for HWSC GFRP bar} \quad (4.11)$$

$$\tau_u = 32 \left(1 + \frac{0.68d}{e^{f_c/36} - 1} \right)^{-0.5} \quad \text{for SC GFRP bar} \quad (4.12)$$



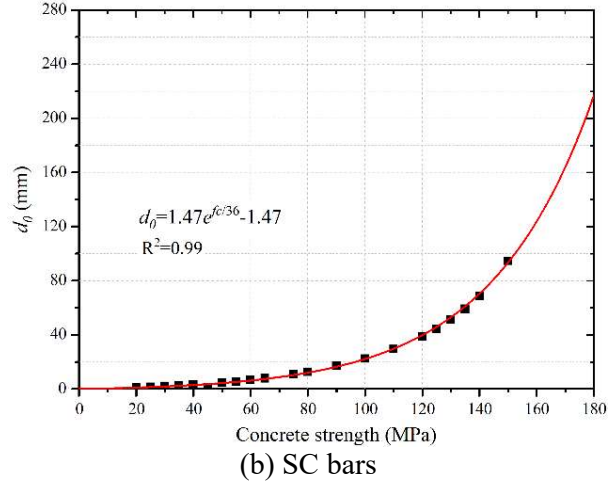


Figure 4.14. Correlation between transitional size and compressive strength of concrete

In addition to the interaction effect of bar diameter and concrete strength, the new model indicates that the influences of bar diameter and concrete strength on bond strength vary with bar type. This is consistent with the ANOVA test results in the previous section, which demonstrate the statistically significant coupled effects of bar type with both bar diameter and concrete strength.

Due to the limited test results for CFRP bars in the database, a fiber factor α_f is applied in Eqs. (4.11) and (4.12) to account for the effect of fiber type for simplification. The fiber factor α_f is preliminarily defined based on the interaction effect of fiber type and concrete strength, as discussed in the statistical analysis in section 4.3.2.

$$\alpha_f = \begin{cases} 1.12 & \text{for CFRP bar in UHSC} \\ 1.00 & \text{for other cases} \end{cases} \quad (4.13)$$

Consequently, the proposed equation for different types of FRP bars is given as follows

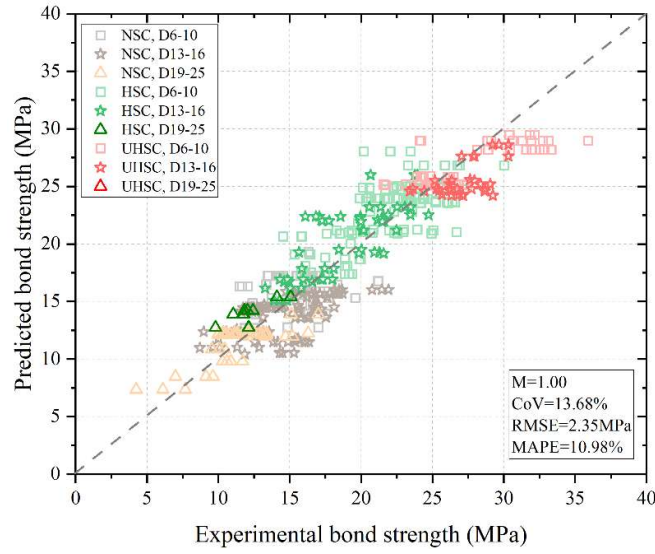
$$\tau_u = 27\alpha_f \left(1 + \frac{0.21d}{e^{\frac{f_c}{55}} - 1} \right)^{-0.5} \quad \text{for HWSC FRP bar} \quad (4.14)$$

$$\tau_u = 32\alpha_f \left(1 + \frac{0.68d}{e^{\frac{f_c}{36}} - 1} \right)^{-0.5} \quad \text{for SC FRP bar} \quad (4.15)$$

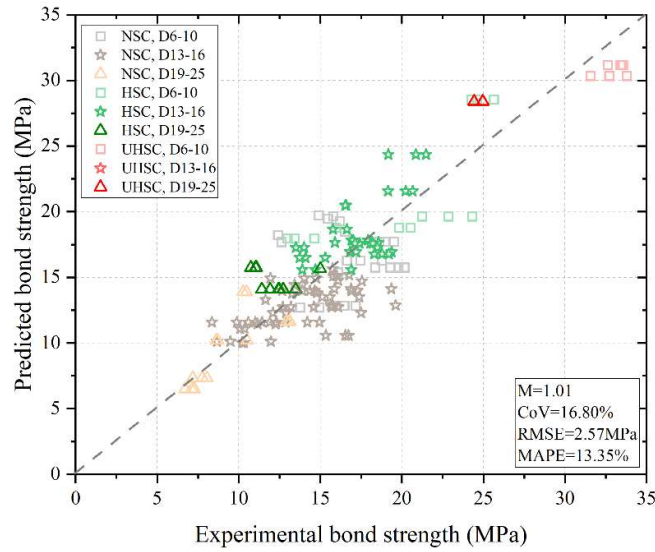
As shown in Table 4.14, the proposed equation presented the highest accuracy compared to other selected models, with M values of 1.00 and 1.01 for the two types of FRP bars. Furthermore, the consideration of the influence of four key factors and their interaction effects made the comparison results of bars of different sizes and types, as well as bars in different concrete strengths, evenly distributed around the diagonal perfect-prediction line, as shown in Figure 4.15. This resulted in the lowest predicted errors, with RMSE of 2.35 MPa and 2.57 MPa and MAPE of 10.98% and 13.35% for HWSC and SC bars, respectively. Thus, the new equation provides better predictions than other models investigated. However, the limited test results of CFRP bars necessitate further investigation for a more accurate assessment of the effect of fiber type.

Table 4.14. Assessment results of existing models and the proposed model

Ref.	HWSC				SC			
	M	CoV (%)	RMSE (MPa)	MAPE (%)	M	CoV (%)	RMSE (MPa)	MAPE (%)
Okelo and Yuan 2005	2.33	29.84	10.37	52.75	2.51	22.01	9.74	57.79
Lee <i>et al.</i> 2012	1.13	18.27	4.28	17.84	1.10	21.31	3.88	18.46
Xue <i>et al.</i> 2014; Hu <i>et al.</i> 2023	0.76	21.97	9.00	39.34	2.05	21.64	8.50	49.19
Yoo <i>et al.</i> 2015	1.78	18.15	8.89	42.22	1.85	20.26	8.11	43.80
Shen <i>et al.</i> 2016	1.14	26.65	5.41	20.61	1.15	21.39	3.54	17.15
Nepomuceno <i>et al.</i> 2021	0.88	18.09	3.85	21.49	0.99	20.38	3.12	17.79
Peng <i>et al.</i> 2022	0.90	16.95	3.71	18.21	0.96	18.32	2.79	16.73
Kim <i>et al.</i> 2023	1.06	17.91	3.01	13.60	1.09	20.67	3.35	16.68
Jiang <i>et al.</i> 2024	4.79	54.93	14.37	72.37	4.97	49.44	11.89	72.58
Wang <i>et al.</i> 2024	0.87	18.92	4.14	23.29	0.84	18.70	3.83	25.30
Proposed models	1.00	13.68	2.35	10.98	1.01	16.80	2.57	13.35



(a) HWSC bars



(b) SC bars

Figure 4.15. Experimental versus predicted bond strength using proposed model

4.6. Conclusions

This chapter has evaluated the bond strength of commercially available FRP bars in concrete. Through an extensive literature review, the effects of key parameters and their possible joint effects on bond strength have been discussed. Subsequently, a statistical analysis was conducted to identify the statistically significant interaction effects of key factors. Considering these interaction effects, a new bond strength predictive equation with high

accuracy has been proposed and compared with some existing models. The following conclusions can be drawn:

1. A total of 687 pullout specimens exhibiting pure pullout failure were collected in the database. Most data were tested on HWSC and SC GFRP bars embedded in NSC and HSC, with nominal diameters ranging from 6 mm to 16 mm. The analysis of data distribution highlighted the research need for the bond of CFRP bars and that of large-diameter GFRP bars in HSC and UHSC.
2. ANOVA test results revealed statistically significant 2-way interactions between the effects of bar type and concrete strength, bar type and bar diameter, concrete strength and bar diameter, as well as fiber type and concrete strength. Further post hoc analysis indicated that the influence of bar diameter on bond strength is dependent on bar type. Specifically, the bond strength of both HWSC and SC bars shows statistically remarkable size dependence, whereas for In bars, bar diameter does not significantly affect the bond strength. In addition, the effect of bar diameter on bond strength becomes less pronounced as the concrete strength increases, and in UHSC, there is no statistically significant size effect. The increasing rates of bond strength with respect to concrete strength vary with bar types. Regarding the effect of fiber type, CFRP bars demonstrate significantly higher bond strength than GFRP bars in UHSC, but no statistically significant difference can be observed between the bond strengths of GFRP and CFRP bars in NSC and HSC matrices.
3. A new bond strength predictive equation has been for HWSC and SC bars, following the general size effect law and incorporating the 2-way interaction effects of bar type and concrete strength, bar type and bar diameter, concrete strength and bar diameter, as well as fiber type and concrete strength. In comparison to other existing models, the proposed equation provides more accurate predictions of the test data. However, due to limited test results of CFRP bars, the accuracy of fiber factor accounting for the effect of fiber type should be further examined.

Table 4.15. Test database of pullout specimens

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Okelo and Yuan 2005	HWSC	GFRP	6	45	5	16	16.32
	HWSC	GFRP	10	41	5	9.5	16.48
	HWSC	GFRP	10	41	5	9.5	17.71
	HWSC	GFRP	10	41	5	9.5	17.14
	HWSC	GFRP	10	45	5	9.5	16.24
	HWSC	GFRP	10	45	5	9.5	13.37
	HWSC	GFRP	10	45	5	9.5	18.37
	HWSC	GFRP	16	32	5	5.8	15.58
	HWSC	GFRP	16	32	5	5.8	15.42
	HWSC	GFRP	16	32	5	5.8	10
	HWSC	GFRP	16	42	5	5.8	17.55
	HWSC	GFRP	16	42	5	5.8	13.12
	HWSC	GFRP	16	42	5	5.8	16.4
	HWSC	GFRP	16	45	5	5.8	15.58
	HWSC	GFRP	16	45	5	5.8	14.84
	HWSC	GFRP	16	49	5	5.8	14.51
	HWSC	GFRP	16	49	5	5.8	15.58
	HWSC	GFRP	19	49	5	4.8	15.1
	HWSC	GFRP	19	49	5	4.8	17
	HWSC	GFRP	19	37	5	4.8	15.2
	HWSC	GFRP	19	37	5	4.8	13.3
	HWSC	GFRP	19	37	5	4.8	14.7
	HWSC	GFRP	19	37	5	4.8	12
Hao <i>et al.</i> 2007	SC	CFRP	10	44	5	9.5	18.78
	SC	CFRP	10	44	5	9.5	17.47
	SC	CFRP	10	44	5	9.5	16.65
	HWSC	GFRP	6	32	4	11	21.21
	HWSC	GFRP	10	31	4	7	17.52
	HWSC	GFRP	13	31	4	5.3	11.3
	HWSC	GFRP	10	62	5	7	16.47
Davalos <i>et al.</i> 2008	HWSC	GFRP	10	62	5	7	16.42
	HWSC	GFRP	10	62	5	7	15.37
	HWSC	GFRP	13	63	5	5.3	17.46
	HWSC	GFRP	13	63	5	5.3	15.85
	HWSC	GFRP	13	63	5	5.3	18.09
	HWSC	GFRP	13	26	4	5.3	14.16
Lee <i>et al.</i> 2008	HWSC	GFRP	13	26	4	5.3	16.35
	HWSC	GFRP	13	26	4	5.3	14.89
	HWSC	GFRP	13	36	4	5.3	15.38
	HWSC	GFRP	13	36	4	5.3	15.78
	HWSC	GFRP	13	36	4	5.3	16.25

Table 4.15 continued-1

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Lee <i>et al.</i> 2008	HWSC	GFRP	13	41	4	5.3	18.12
	HWSC	GFRP	13	41	4	5.3	16.72
	HWSC	GFRP	13	41	4	5.3	16.81
	HWSC	GFRP	13	56	4	5.3	17.18
	HWSC	GFRP	13	56	4	5.3	14.83
	HWSC	GFRP	13	56	4	5.3	15.94
	HWSC	GFRP	13	76	4	5.3	18.45
	HWSC	GFRP	13	76	4	5.3	20.03
	HWSC	GFRP	13	76	4	5.3	18.44
	HWSC	GFRP	13	92	4	5.3	20.07
	HWSC	GFRP	13	92	4	5.3	22.47
	HWSC	GFRP	13	92	4	5.3	20.19
	SC	GFRP	13	26	4	5.3	16.73
	SC	GFRP	13	26	4	5.3	15.33
	SC	GFRP	13	26	4	5.3	16.54
	SC	GFRP	13	36	4	5.3	15.65
	SC	GFRP	13	36	4	5.3	19.61
	SC	GFRP	13	41	4	5.3	15.86
	SC	GFRP	13	41	4	5.3	16.88
	SC	GFRP	13	41	4	5.3	19.35
	SC	GFRP	13	57	4	5.3	18.16
	SC	GFRP	13	57	4	5.3	15.9
	SC	GFRP	13	57	4	5.3	18.56
	SC	GFRP	13	75	4	5.3	19.16
	SC	GFRP	13	75	4	5.3	20.23
	SC	GFRP	13	75	4	5.3	20.66
	SC	GFRP	13	90	4	5.3	19.17
	SC	GFRP	13	90	4	5.3	20.86
	SC	GFRP	13	90	4	5.3	21.48
	HWSC	GFRP	10	51	5	9.5	17.49
	HWSC	GFRP	10	51	5	9.5	16.48
	HWSC	GFRP	10	51	5	9.5	15.81
	HWSC	GFRP	13	51	5	7.2	13.28
	HWSC	GFRP	13	51	5	7.2	15.03
Baena <i>et al.</i> 2009	HWSC	GFRP	13	51	5	7.2	15.81
	HWSC	GFRP	16	51	5	5.8	14.82
	HWSC	GFRP	16	51	5	5.8	13.91
	HWSC	GFRP	16	51	5	5.8	14.27
	HWSC	GFRP	16	28	5	5.8	8.7
	HWSC	GFRP	16	28	5	5.8	10.1
	HWSC	GFRP	19	51	5	4.8	11.83

Table 4.15 continued-2

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Baena <i>et al.</i> 2009	HWSC	GFRP	19	51	5	4.8	12.05
	HWSC	GFRP	19	51	5	4.8	12.43
	SC	GFRP	10	50	5	9.5	13
	SC	GFRP	10	50	5	9.5	14.63
	SC	GFRP	10	50	5	9.5	13.42
	SC	GFRP	13	50	5	7.2	15.3
	SC	GFRP	13	50	5	7.2	14.09
	SC	GFRP	13	50	5	7.2	13.74
	SC	GFRP	13	28	5	7.2	10.09
	SC	GFRP	13	28	5	7.2	10.35
	SC	GFRP	16	59	5	5.8	18.78
	SC	GFRP	16	59	5	5.8	18.31
	SC	GFRP	16	59	5	5.8	19.2
	SC	GFRP	16	28	5	5.8	10.32
	SC	GFRP	16	28	5	5.8	10.2
	SC	GFRP	19	50	5	4.8	13.48
	SC	GFRP	19	50	5	4.8	12.73
	SC	GFRP	19	50	5	4.8	12.43
	SC	CFRP	10	59	5	9.5	21.24
	SC	CFRP	10	59	5	9.5	24.33
	SC	CFRP	10	59	5	9.5	22.85
	SC	CFRP	10	30	5	9.5	14.94
	SC	CFRP	10	30	5	9.5	13.77
	SC	CFRP	13	57	5	7.2	17.92
	SC	CFRP	13	57	5	7.2	16.92
	SC	CFRP	13	57	5	7.2	17.01
	SC	CFRP	13	30	5	7.2	11.16
	SC	CFRP	13	30	5	7.2	12.25
	In	GFRP	8	59	5	12	14.37
	In	GFRP	8	59	5	12	15.51
	In	GFRP	8	31	5	12	11.18
	In	GFRP	8	31	5	12	10.72
	In	GFRP	12	59	5	7.8	13.63
	In	GFRP	12	59	5	7.8	13.63
	In	GFRP	12	28	5	7.8	8.52
	In	GFRP	12	31	5	7.8	7.95
	In	GFRP	16	61	5	5.8	13.67
	In	GFRP	16	59	5	5.8	13.84
	In	GFRP	16	28	5	5.8	10.34
	In	GFRP	16	28	5	5.8	8.7
Robert <i>et al.</i> 2010	SC	GFRP	19	59	5	4.8	15

Table 4.15 continued-3

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Alves <i>et al.</i> 2011	SC	GFRP	16	50	5	5.8	11.94
	SC	GFRP	16	50	5	2	14.75
	SC	GFRP	16	50	5	2.5	14.03
	SC	GFRP	19	50	5	4.8	10.27
	SC	GFRP	19	50	5	2.5	10.47
	HWSC	GFRP	13	24	5	5.3	9.62
	HWSC	GFRP	13	24	5	5.3	11.32
	HWSC	GFRP	13	24	5	5.3	12.86
	HWSC	GFRP	13	40	5	5.3	16.8
	HWSC	GFRP	13	40	5	5.3	17.22
	HWSC	GFRP	13	40	5	5.3	15.68
	HWSC	GFRP	13	74	5	5.3	21.29
	HWSC	GFRP	13	74	5	5.3	15.65
	HWSC	GFRP	13	74	5	5.3	20.95
Lee <i>et al.</i> 2012	HWSC	GFRP	13	30	5	5.3	13.55
	HWSC	GFRP	13	30	5	5.3	13.13
	HWSC	GFRP	13	42	5	5.3	16.9
	HWSC	GFRP	13	42	5	5.3	13.22
	HWSC	GFRP	13	73	5	5.3	21.58
	HWSC	GFRP	13	73	5	5.3	19.85
	SC	GFRP	13	24	5	5.3	9.49
	SC	GFRP	13	24	5	5.3	11.97
	SC	GFRP	13	24	5	5.3	8.67
	SC	GFRP	13	42	5	5.3	13.42
	SC	GFRP	13	42	5	5.3	13.89
	SC	GFRP	13	70	5	5.3	16.59
	SC	GFRP	13	70	5	5.3	16.53
	HWSC	GFRP	13	26	5	4.5	12.55
	HWSC	GFRP	16	26	5	4.5	11.85
Ahmed and Sennah 2014	SC	GFRP	19	140	4	2	24.96
	SC	GFRP	19	140	4	2	24.42
Xue <i>et al.</i> 2014	HWSC	GFRP	6	26	5	4.5	17.79
	HWSC	GFRP	10	26	5	4.5	14.87
	HWSC	GFRP	13	26	5	4.5	15.4
	HWSC	GFRP	16	26	5	4.5	14.5
	HWSC	GFRP	16	26	5	4.5	15.3
	HWSC	GFRP	16	26	5	4.5	14.3
	HWSC	GFRP	19	26	5	4.5	10.88
	SC	CFRP	13	44	5	7.2	16.09
Calvet <i>et al.</i> 2015	SC	CFRP	13	40	5	7.2	16.92
	SC	CFRP	13	45	5	7.2	16.77

Table 4.15 continued-4

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Calvet <i>et al.</i> 2015	SC	CFRP	13	45	5	7.2	15.8
	SC	CFRP	13	44	5	7.2	17.55
	SC	CFRP	13	37	5	7.2	11.64
	SC	CFRP	13	47	5	7.2	15.75
	SC	CFRP	13	39	5	7.2	13.28
	SC	CFRP	13	40	5	7.2	14.79
	SC	CFRP	13	45	5	7.2	16.01
	SC	CFRP	6	39	5	11.6	16.52
	SC	CFRP	6	44	5	11.6	14.91
	SC	CFRP	6	44	5	11.6	15.81
	SC	CFRP	6	43	5	11.6	16.21
	SC	CFRP	6	37	5	11.6	12.63
	SC	CFRP	6	37	5	11.6	12.41
	SC	CFRP	6	44	5	11.6	15.48
	SC	GFRP	10	50	5	7	18.63
Refai <i>et al.</i> 2015	SC	GFRP	10	50	5	7	19.52
	SC	GFRP	10	50	5	7	18.97
	SC	GFRP	10	50	5	7	18.97
Park <i>et al.</i> 2016	SC	GFRP	13	30	4	5.3	10.59
	SC	GFRP	13	30	4	5.3	9.88
	SC	GFRP	13	30	4	5.3	11.43
Park <i>et al.</i> 2016	SC	GFRP	13	30	4	5.3	8.36
	SC	GFRP	13	30	4	5.3	11.01
	SC	GFRP	13	30	4	5.3	11.91
Xue <i>et al.</i> 2016	HWSC	GFRP	6	26	5	4.5	19.6
	HWSC	GFRP	6	26	5	4.5	15
	HWSC	GFRP	6	26	5	4.5	17.7
	HWSC	GFRP	10	26	5	4.5	17.01
	HWSC	GFRP	10	26	5	4.5	11.88
	HWSC	GFRP	10	26	5	4.5	12.54
	HWSC	GFRP	13	26	5	4.5	13.87
	HWSC	GFRP	13	26	5	4.5	16.12
	HWSC	GFRP	13	26	5	4.5	16.02
	HWSC	GFRP	19	26	5	4.5	10.6
	HWSC	GFRP	19	26	5	4.5	10.3
	HWSC	GFRP	19	26	5	4.5	11.7
	HWSC	GFRP	19	20	5	5.3	9.13
	HWSC	GFRP	19	20	5	5.3	9.63
	HWSC	GFRP	19	20	5	5.3	6.99
Lee <i>et al.</i> 2017	HWSC	GFRP	19	39	5	5.3	12.46
	HWSC	GFRP	19	39	5	5.3	12.15
	HWSC	GFRP	19	39	5	5.3	16.3

Table 4.15 continued-5

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Lee <i>et al.</i> 2017	HWSC	GFRP	19	59	5	5.3	15.08
	HWSC	GFRP	19	59	5	5.3	14.1
	HWSC	GFRP	25	20	5	5.4	7.68
	HWSC	GFRP	25	20	5	5.4	6.12
	HWSC	GFRP	25	20	5	5.4	4.25
	HWSC	GFRP	25	39	5	5.4	9.54
	HWSC	GFRP	25	39	5	5.4	10.44
	HWSC	GFRP	25	39	5	5.4	10.08
	HWSC	GFRP	25	59	5	5.4	11.8
	HWSC	GFRP	25	59	5	5.4	11.05
	HWSC	GFRP	25	59	5	5.4	11.71
	SC	GFRP	19	20	5	5.3	7.77
	SC	GFRP	19	20	5	5.3	8.04
	SC	GFRP	19	20	5	5.3	7.18
	SC	GFRP	19	40	5	5.3	13.14
	SC	GFRP	19	40	5	5.3	12.93
	SC	GFRP	19	40	5	5.3	13.1
	SC	GFRP	19	59	5	5.3	11.03
	SC	GFRP	19	59	5	5.3	10.74
	SC	GFRP	19	59	5	5.3	11.08
	SC	GFRP	25	20	5	5.4	7.28
	SC	GFRP	25	20	5	5.4	6.72
	SC	GFRP	25	20	5	5.4	7.13
	SC	GFRP	25	39	5	5.4	8.63
	SC	GFRP	25	39	5	5.4	10.52
	SC	GFRP	25	39	5	5.4	8.71
	SC	GFRP	25	59	5	5.4	11.92
	SC	GFRP	25	59	5	5.4	11.42
	SC	GFRP	25	59	5	5.4	12.48
	HWSC	GFRP	13	49	5	4.5	15.32
	HWSC	GFRP	13	49	5	4.5	15.74
	HWSC	GFRP	13	49	5	4.5	14.74
	HWSC	GFRP	13	48	5	4.5	18.71
	HWSC	GFRP	13	48	5	4.5	17.81
Yan <i>et al.</i> 2017a	HWSC	GFRP	13	48	5	4.5	18.43
	HWSC	GFRP	13	50	5	4.5	20.77
	HWSC	GFRP	13	50	5	4.5	21.92
	HWSC	GFRP	13	50	5	4.5	21.45
	HWSC	GFRP	13	48	5	4.5	15.27
	HWSC	GFRP	13	48	5	4.5	17.1
	HWSC	GFRP	13	48	5	4.5	16.55

Table 4.15 continued-6

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Yan <i>et al.</i> 2017a	HWSC	GFRP	13	48	5	4.5	15.94
	HWSC	GFRP	13	48	5	4.5	16.76
	HWSC	GFRP	13	48	5	4.5	18.26
	HWSC	GFRP	13	46	5	3	16.86
	HWSC	GFRP	13	46	5	3	17.62
Yan <i>et al.</i> 2017b	HWSC	GFRP	13	46	5	3	16.21
	HWSC	GFRP	13	50	5	4.5	18.67
	HWSC	GFRP	13	50	5	4.5	18.14
	HWSC	GFRP	13	50	5	4.5	18.45
	SC	GFRP	13	38	5	7.2	17.39
	SC	GFRP	13	38	5	7.2	15.12
	SC	GFRP	13	38	5	7.2	15.59
	SC	GFRP	13	40	5	7.2	13.89
	SC	GFRP	13	40	5	7.2	14.65
	SC	GFRP	13	40	5	7.2	12.66
	SC	GFRP	13	40	5	7.2	14.36
	SC	GFRP	13	40	5	7.2	17.29
	SC	GFRP	13	53	5	7.2	19.37
	SC	GFRP	13	53	5	7.2	16.82
	SC	GFRP	13	53	5	7.2	17.2
	SC	GFRP	13	61	5	7.2	15.78
	SC	GFRP	13	61	5	7.2	16.63
	SC	GFRP	16	38	5	5.8	12.1
	SC	GFRP	16	38	5	5.8	12.9
Parvizi <i>et al.</i> 2020	SC	GFRP	16	38	5	5.8	17.5
	SC	GFRP	16	40	5	5.8	16.1
	SC	GFRP	16	40	5	5.8	15.7
	SC	GFRP	16	40	5	5.8	12.7
	SC	GFRP	16	40	5	5.8	14.6
	SC	GFRP	16	40	5	5.8	13.2
	SC	GFRP	16	53	5	5.8	13.9
	SC	GFRP	16	53	5	5.8	14.7
	SC	GFRP	16	53	5	5.8	16.9
	SC	GFRP	16	61	5	5.8	18.5
	SC	GFRP	16	61	5	5.8	13.5
	SC	GFRP	16	61	5	5.8	14
	In	GFRP	12	40	5	7.8	13
	In	GFRP	12	40	5	7.8	11.9
	In	GFRP	12	40	5	7.8	9.5
	In	GFRP	12	40	5	7.8	12.5
	In	GFRP	12	40	5	7.8	12.4

Table 4.15 continued-7

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Parvizi <i>et al.</i> 2020	In	GFRP	12	42	5	7.8	10.1
	In	GFRP	12	42	5	7.8	7.6
	In	GFRP	12	42	5	7.8	10.5
	In	GFRP	12	42	5	7.8	10.6
	In	GFRP	12	42	5	7.8	10.9
	In	GFRP	12	55	5	7.8	9.6
	In	GFRP	12	55	5	7.8	11.3
	In	GFRP	12	55	5	7.8	8.6
	In	GFRP	12	63	5	7.8	13.3
	In	GFRP	12	63	5	7.8	11.7
	In	GFRP	12	63	5	7.8	12.7
	In	GFRP	16	40	5	5.8	13.4
	In	GFRP	16	40	5	5.8	12.8
	In	GFRP	16	40	5	5.8	13.4
	In	GFRP	16	40	5	5.8	13.6
	In	GFRP	16	40	5	5.8	12.6
	In	GFRP	16	42	5	5.8	13
	In	GFRP	16	42	5	5.8	11.9
	In	GFRP	16	42	5	5.8	13.2
	In	GFRP	16	42	5	5.8	15.3
	In	GFRP	16	42	5	5.8	14.1
	In	GFRP	16	55	5	5.8	12.4
	In	GFRP	16	53	5	5.8	13.7
	In	GFRP	16	55	5	5.8	11.6
	In	GFRP	16	63	5	5.8	12.4
	In	GFRP	16	61	5	5.8	12.1
	In	GFRP	16	63	5	5.8	15.3
	In	GFRP	8	53	5	12	13.91
	In	GFRP	8	53	5	12	15.08
	In	GFRP	8	53	5	12	14.33
	In	GFRP	8	53	5	12	13.46
	In	GFRP	8	53	5	12	12.7
	In	GFRP	8	53	5	12	10.44
Soares <i>et al.</i> 2020	In	GFRP	8	69	5	12	18.47
	In	GFRP	8	69	5	12	20.43
	In	GFRP	8	69	5	12	20.41
	In	GFRP	8	60	5	12	13.79
	In	GFRP	8	60	5	12	13.02
	In	GFRP	8	60	5	12	13.66
	In	GFRP	12	53	5	7.8	16.45
	In	GFRP	12	53	5	7.8	19.39

Table 4.15 continued-8

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Soares <i>et al.</i> 2020	In	GFRP	12	53	5	7.8	16.72
	In	GFRP	12	53	5	7.8	18.17
	In	GFRP	12	53	5	7.8	17.97
	In	GFRP	12	69	5	7.8	22.44
	In	GFRP	12	69	5	7.8	19.85
	In	GFRP	12	69	5	7.8	22.55
	In	GFRP	12	60	5	7.8	20.55
	In	GFRP	12	60	5	7.8	19.9
	In	GFRP	12	60	5	7.8	19.36
	HWSC	GFRP	8	28	5	8.9	16.65
	HWSC	GFRP	8	28	5	8.9	12.41
	HWSC	GFRP	8	28	5	8.9	15.87
	HWSC	GFRP	6	28	5	12	11.5
	HWSC	GFRP	6	28	5	12	11.67
	HWSC	GFRP	6	28	5	12	12.08
	HWSC	GFRP	6	56	5	12	14.55
	HWSC	GFRP	6	56	5	12	15.8
	HWSC	GFRP	6	56	5	12	15.88
	HWSC	GFRP	10	28	5	7	12.67
	HWSC	GFRP	10	28	5	7	16.33
Solyom and Balázs. 2020	HWSC	GFRP	10	28	5	7	16.24
	HWSC	GFRP	13	28	5	5.3	8.98
	HWSC	GFRP	13	28	5	5.3	10.34
	HWSC	GFRP	13	28	5	5.3	10.57
	HWSC	GFRP	13	28	5	5.3	11.12
	HWSC	GFRP	13	56	5	5.3	17.33
	HWSC	GFRP	13	56	5	5.3	14.31
	HWSC	GFRP	13	56	5	5.3	18
	SC	GFRP	6	31	5	12	20.16
	SC	GFRP	6	31	5	12	18.36
	SC	GFRP	6	31	5	12	19.3
	SC	GFRP	6	31	5	12	19.8
	SC	GFRP	10	31	5	10	16.46
	SC	GFRP	10	31	5	10	16.5
	SC	GFRP	10	31	5	10	17.18
	SC	GFRP	10	31	5	10	15.79
	SC	GFRP	13	31	5	5.3	14.95
	SC	GFRP	13	31	5	5.3	12.86
	SC	GFRP	13	31	5	5.3	12.3

Table 4.15 continued-9

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Solyom and Balázs. 2020	SC	GFRP	13	31	5	5.3	14.2
	SC	GFRP	13	56	5	5.3	18.06
	SC	GFRP	13	56	5	5.3	18.22
	SC	GFRP	13	56	5	5.3	17.41
	SC	GFRP	13	56	5	5.3	17.13
	In	GFRP	8	30	5	8.9	8.93
	In	GFRP	8	30	5	8.9	10.38
	In	GFRP	8	30	5	8.9	7.14
	In	GFRP	8	59	5	8.9	15.75
	In	GFRP	8	59	5	8.9	15.9
	In	GFRP	8	59	5	8.9	11.55
	In	GFRP	8	59	5	8.9	14.84
	In	GFRP	12	30	5	5.8	10.59
	In	GFRP	12	30	5	5.8	10.65
	In	GFRP	12	30	5	5.8	10.84
	In	GFRP	12	30	5	5.8	8.04
	In	GFRP	12	59	5	5.8	14.95
	In	GFRP	12	59	5	5.8	17.52
	In	GFRP	12	59	5	5.8	16.97
	In	GFRP	12	59	5	5.8	10.9
Liao <i>et al.</i> 2021	HWSC	GFRP	10	79	5	9.5	22.35
	HWSC	GFRP	10	79	5	9.5	20.66
	HWSC	GFRP	10	79	5	9.5	20.8
	HWSC	GFRP	10	81	5	9.5	23.95
	HWSC	GFRP	10	81	5	9.5	23.01
	HWSC	GFRP	10	81	5	9.5	20.86
	HWSC	GFRP	10	119	5	9.5	22.96
	HWSC	GFRP	10	119	5	9.5	21.84
	HWSC	GFRP	10	119	5	9.5	23.49
	HWSC	GFRP	10	79	5	9.5	26.71
	HWSC	GFRP	10	79	5	9.5	22.97
	HWSC	GFRP	10	79	5	9.5	20.15
	HWSC	GFRP	10	78	5	9.5	24.94
	HWSC	GFRP	10	78	5	9.5	18.5
	HWSC	GFRP	10	78	5	9.5	18.56
	HWSC	GFRP	10	81	5	9.5	21.98
	HWSC	GFRP	10	81	5	9.5	25.06
	HWSC	GFRP	10	81	5	9.5	19.32
	HWSC	GFRP	10	81	5	9.5	18.29
	HWSC	GFRP	10	81	5	9.5	22.35

Table 4.15 continued-10

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Liao <i>et al.</i> 2021	HWSC	GFRP	10	116	5	9.5	23.35
	HWSC	GFRP	10	116	5	9.5	23.33
	HWSC	GFRP	10	116	5	9.5	25.27
	HWSC	GFRP	10	112	5	9.5	24.2
	HWSC	GFRP	10	112	5	9.5	26.1
	HWSC	GFRP	10	112	5	9.5	23.23
	HWSC	GFRP	10	49	5	9.5	14.56
	HWSC	GFRP	10	49	5	9.5	13.46
	HWSC	GFRP	10	49	5	9.5	16.45
	HWSC	GFRP	10	49	5	9.5	17.13
	HWSC	GFRP	10	49	5	9.5	16.22
	HWSC	GFRP	10	49	5	9.5	16.87
	HWSC	GFRP	10	49	5	9.5	16.91
	HWSC	GFRP	10	49	5	9.5	14.16
	HWSC	GFRP	10	49	5	9.5	15.82
	HWSC	GFRP	10	49	5	9.5	13.51
	HWSC	GFRP	10	49	5	9.5	15.55
	HWSC	GFRP	10	49	5	9.5	15.47
	HWSC	GFRP	10	49	5	9.5	16.13
	HWSC	GFRP	10	49	5	9.5	14.28
Henin <i>et al.</i> 2021	HWSC	GFRP	10	49	5	9.5	15.77
	HWSC	GFRP	16	47	5	5.8	12.15
	HWSC	GFRP	16	47	5	5.8	13.11
	HWSC	GFRP	16	47	5	5.8	12.34
	HWSC	GFRP	16	47	5	5.8	12.37
	HWSC	GFRP	16	47	5	5.8	14.3
	HWSC	GFRP	16	47	5	5.8	12.96
	HWSC	GFRP	16	47	5	5.8	12.7
	HWSC	GFRP	16	47	5	5.8	12.5
	HWSC	GFRP	16	47	5	5.8	13.74
	HWSC	GFRP	16	47	5	5.8	12.05
	HWSC	GFRP	16	47	5	5.8	14
	HWSC	GFRP	16	47	5	5.8	12.75
	HWSC	GFRP	16	47	5	5.8	12.29
	HWSC	GFRP	16	47	5	5.8	12.44
	HWSC	GFRP	16	47	5	5.8	13.32
	HWSC	GFRP	16	47	5	5.8	13.65
	HWSC	GFRP	16	47	5	5.8	11.93
	HWSC	GFRP	16	47	5	5.8	11.74
	HWSC	GFRP	19	39	5	4.8	11.14

Table 4.15 continued-11

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Henin <i>et al.</i> 2021	HWSC	GFRP	19	39	5	4.8	10.47
	HWSC	GFRP	19	39	5	4.8	10.36
	HWSC	GFRP	19	39	5	4.8	10.01
	HWSC	GFRP	19	39	5	4.8	10.27
	HWSC	GFRP	19	39	5	4.8	12.06
	HWSC	GFRP	19	39	5	4.8	12.7
	HWSC	GFRP	19	39	5	4.8	11.32
	HWSC	GFRP	19	39	5	4.8	12.88
	HWSC	GFRP	19	39	5	4.8	13.22
	HWSC	GFRP	19	39	5	4.8	12.83
	HWSC	GFRP	19	39	5	4.8	13.38
	HWSC	GFRP	25	47	5	5.4	11.24
	HWSC	GFRP	25	47	5	5.4	10.59
	HWSC	GFRP	25	47	5	5.4	10
	HWSC	GFRP	25	47	5	5.4	10
	HWSC	GFRP	25	47	5	5.4	11.15
	HWSC	GFRP	25	47	5	5.4	10.39
	HWSC	GFRP	25	47	5	5.4	9.99
	HWSC	GFRP	25	47	5	5.4	10.83
	HWSC	GFRP	25	47	5	5.4	10.33
	HWSC	GFRP	25	47	5	5.4	10.82
	HWSC	GFRP	25	47	5	5.4	11.05
	HWSC	GFRP	25	47	5	5.4	10.45
	HWSC	GFRP	25	47	5	5.4	13.39
	HWSC	GFRP	25	47	5	5.4	13.02
	HWSC	GFRP	25	47	5	5.4	11.69
	HWSC	GFRP	10	40	5	9.5	18.4
	SC	GFRP	10	40	5	9.5	16.1
Emparanza <i>et al.</i> 2022	HWSC	GFRP	6	72	5	12	22.33
	HWSC	GFRP	6	72	5	12	23.58
	HWSC	GFRP	6	72	5	12	20.65
	HWSC	GFRP	10	72	5	7	17.6
	HWSC	GFRP	10	72	5	7	19.4
	HWSC	GFRP	10	72	5	7	22.58
	HWSC	GFRP	6	114	5	16	24.05
Zeng <i>et al.</i> 2022	HWSC	GFRP	6	114	5	16	20.59
	HWSC	GFRP	6	114	5	16	22.33
	HWSC	GFRP	6	119	5	16	23.46
	HWSC	GFRP	6	119	5	16	22.48
	HWSC	GFRP	6	118	5	16	23.24
	HWSC	GFRP	6	118	5	16	25.45

Table 4.15 continued-12

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Zeng <i>et al.</i> 2022	HWSC	GFRP	6	118	5	16	26.26
	HWSC	GFRP	6	113	5	16	23.99
	HWSC	GFRP	6	113	5	16	24.15
	HWSC	GFRP	6	116	5	16	20.14
	HWSC	GFRP	6	116	5	16	25.8
	HWSC	GFRP	6	116	5	16	25.76
	HWSC	GFRP	6	112	5	16	21.72
	HWSC	GFRP	6	112	5	16	23.89
	HWSC	GFRP	6	112	5	16	22.8
	HWSC	GFRP	10	114	5	9.5	20.38
	HWSC	GFRP	10	114	5	9.5	26.39
	HWSC	GFRP	10	114	5	9.5	23.29
	HWSC	GFRP	10	119	5	9.5	22.96
	HWSC	GFRP	10	119	5	9.5	21.84
	HWSC	GFRP	10	119	5	9.5	23.49
	HWSC	GFRP	10	118	5	9.5	24.36
	HWSC	GFRP	10	118	5	9.5	24.69
	HWSC	GFRP	10	118	5	9.5	23.11
	HWSC	GFRP	10	113	5	9.5	26.59
	HWSC	GFRP	10	113	5	9.5	25.24
	HWSC	GFRP	10	113	5	9.5	26.03
	HWSC	GFRP	10	116	5	9.5	23.35
	HWSC	GFRP	10	116	5	9.5	23.33
	HWSC	GFRP	10	116	5	9.5	25.27
	HWSC	GFRP	10	112	5	9.5	24.2
	HWSC	GFRP	10	112	5	9.5	26.1
	HWSC	GFRP	10	112	5	9.5	23.23
	HWSC	GFRP	16	113	5	5.8	17.23
	HWSC	GFRP	16	113	5	5.8	21.16
	HWSC	GFRP	16	117	5	5.8	17.08
	HWSC	GFRP	16	117	5	5.8	18.57
	HWSC	GFRP	16	117	5	5.8	16.85
	HWSC	GFRP	16	117	5	5.8	16.09
	HWSC	GFRP	16	117	5	5.8	19.95
	HWSC	GFRP	16	112	5	5.8	17.27
	HWSC	GFRP	16	112	5	5.8	21.61
	HWSC	GFRP	16	112	5	5.8	17.76
	HWSC	GFRP	16	115	5	5.8	19.91
	HWSC	GFRP	16	115	5	5.8	21.45
	HWSC	GFRP	16	111	5	5.8	19.99
	HWSC	GFRP	16	111	5	5.8	21.49

Table 4.15 continued-13

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
Hao <i>et al.</i> 2023	HWSC	GFRP	10	59	4	7	18.89
	HWSC	GFRP	10	59	4	7	19.9
	HWSC	GFRP	10	59	4	7	19.99
	HWSC	GFRP	16	162	5	4.5	27.74
	HWSC	GFRP	16	162	5	4.5	28.67
	HWSC	GFRP	16	162	5	4.5	26.74
Hu <i>et al.</i> 2023	HWSC	GFRP	16	162	5	2	26.25
	HWSC	GFRP	16	162	5	2	23.62
	HWSC	GFRP	16	162	5	2	25.3
	HWSC	GFRP	16	45	5	4.5	11.59
	HWSC	GFRP	16	45	5	4.5	11.59
	HWSC	GFRP	16	45	5	4.5	12.37
	HWSC	GFRP	10	48	5	7	14.39
	HWSC	GFRP	10	48	5	7	16.63
	HWSC	GFRP	10	75	5	7	19.17
	HWSC	GFRP	10	84	5	7	19.77
Ke <i>et al.</i> 2024	HWSC	GFRP	10	37	5	7	13.74
	HWSC	GFRP	10	41	5	7	16.06
	HWSC	GFRP	10	54	5	7	17.5
	HWSC	GFRP	10	67	5	7	19.29
	HWSC	GFRP	10	50	4	7	18.87
	HWSC	GFRP	10	50	4	7	19.38
	HWSC	GFRP	10	50	4	7	19.18
	HWSC	GFRP	10	119	4	7	24.43
	HWSC	GFRP	10	119	4	7	21.21
	HWSC	GFRP	10	119	4	7	22.97
	HWSC	GFRP	10	119	4	7	22.56
	HWSC	GFRP	10	146	4	7	27.45
	HWSC	GFRP	10	146	4	7	24.89
	HWSC	GFRP	10	146	4	7	21.75
	HWSC	GFRP	10	148	4	7	27.24
	HWSC	GFRP	10	148	4	7	23.14
	HWSC	GFRP	10	148	4	7	26.84
	HWSC	GFRP	10	148	4	7	24.17
This test	HWSC	GFRP	10	148	4	7	24.82
	HWSC	GFRP	10	148	4	7	25.44
	HWSC	GFRP	10	177	4	7	24.38
	HWSC	GFRP	10	177	4	7	24.87
	HWSC	GFRP	10	174	4	7	26.26
	HWSC	GFRP	10	174	4	7	23.94
	HWSC	GFRP	10	54	4	7	16.1
	HWSC	GFRP	10	54	4	7	18.88

Table 4.15 continued-14

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L/d	C/d	τ_u (MPa)
This test	HWSC	GFRP	10	54	4	7	19.06
	HWSC	GFRP	10	119	4	7	20.36
	HWSC	GFRP	10	119	4	7	20.73
	HWSC	GFRP	10	119	4	7	21.65
	HWSC	GFRP	10	119	4	7	21.84
	HWSC	GFRP	10	119	4	7	21.16
	HWSC	GFRP	10	119	4	7	21.93
	HWSC	GFRP	10	119	4	7	22.62
	HWSC	GFRP	10	119	4	7	20.45
	HWSC	GFRP	10	119	4	7	19.92
	HWSC	GFRP	10	146	4	7	24.78
	HWSC	GFRP	10	146	4	7	24.55
	HWSC	GFRP	10	146	4	7	24.95
	HWSC	GFRP	10	148	4	7	26.28
	HWSC	GFRP	10	148	4	7	21.59
	HWSC	GFRP	10	148	4	7	24.8
	HWSC	GFRP	10	148	4	7	24.04
	HWSC	GFRP	10	177	4	7	24.18
	HWSC	GFRP	10	177	4	7	24.29
	HWSC	GFRP	10	174	4	7	26.5
	HWSC	GFRP	10	174	4	7	24.38
	HWSC	GFRP	10	174	4	7	26.08
	HWSC	GFRP	13	119	4	5.3	23.07
	HWSC	GFRP	13	119	4	5.3	22.53
	HWSC	GFRP	13	119	4	5.3	20.58
	HWSC	GFRP	13	54	4	5.3	14.95
	HWSC	GFRP	13	54	4	5.3	16.57
	HWSC	GFRP	13	54	4	5.3	14.62
	HWSC	GFRP	13	119	4	5.3	21.38
	HWSC	GFRP	13	119	4	5.3	23.47
	HWSC	GFRP	13	146	4	5.3	28.67
	HWSC	GFRP	13	146	4	5.3	24.43
	HWSC	GFRP	13	146	4	5.3	23.42
	HWSC	GFRP	13	148	4	5.3	25.48
	HWSC	GFRP	13	148	4	5.3	26.21
	HWSC	GFRP	13	177	4	5.3	27.62
	HWSC	GFRP	13	174	4	5.3	27.95
	HWSC	GFRP	13	174	4	5.3	25.17
	HWSC	GFRP	13	174	4	5.3	26.34
	HWSC	GFRP	16	119	4	5.8	21.89

Table 4.15 continued-15

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
	HWSC	GFRP	16	119	4	5.8	24.73
	HWSC	GFRP	16	119	4	5.8	23.45
	HWSC	GFRP	16	148	4	5.8	26.34
	HWSC	GFRP	16	148	4	5.8	27.14
	HWSC	GFRP	16	148	4	5.8	29.24
	HWSC	GFRP	16	150	4	5.8	27.1
	HWSC	GFRP	16	150	4	5.8	26.86
	HWSC	GFRP	16	150	4	5.8	25.67
	HWSC	GFRP	16	174	4	5.8	29.05
	HWSC	GFRP	16	174	4	5.8	26.2
	HWSC	GFRP	16	174	4	5.8	28.38
	HWSC	GFRP	25	51	5	3.4	9.79
	HWSC	GFRP	25	51	5	3.4	12.13
	HWSC	GFRP	25	51	5	3.4	12.11
	HWSC	CFRP	6	119	4	12	22.05
	HWSC	CFRP	6	119	4	12	20.18
	HWSC	CFRP	6	119	4	12	23.48
	HWSC	CFRP	6	148	4	12	28.87
	HWSC	CFRP	6	148	4	12	24.2
	HWSC	CFRP	6	148	4	12	24.13
	HWSC	CFRP	6	176	4	12	30.41
This test	HWSC	CFRP	6	176	4	12	32.18
	HWSC	CFRP	6	176	4	12	31.83
	HWSC	CFRP	10	119	4	7	26.76
	HWSC	CFRP	10	119	4	7	30.02
	HWSC	CFRP	10	119	4	7	26.9
	HWSC	CFRP	10	148	4	7	32.83
	HWSC	CFRP	10	148	4	7	33.34
	HWSC	CFRP	10	148	4	7	31.58
	HWSC	CFRP	10	148	4	7	32.51
	HWSC	CFRP	10	176	4	7	32.74
	HWSC	CFRP	10	176	4	7	35.91
	HWSC	CFRP	10	176	4	7	33.14
	HWSC	CFRP	10	176	4	7	30.62
	HWSC	CFRP	10	119	4	7	23.3
	HWSC	CFRP	10	119	4	7	25.87
	HWSC	CFRP	10	119	4	7	24.42
	HWSC	CFRP	10	148	4	7	28.91
	HWSC	CFRP	10	148	4	7	30.43
	HWSC	CFRP	10	148	4	7	31.98
	HWSC	CFRP	10	148	4	7	28.14
	HWSC	CFRP	10	176	4	7	31.94
	HWSC	CFRP	10	176	4	7	31.32

Table 4.15 continued-16

Ref.	Bar type	Fiber type	d (mm)	f_c (MPa)	L_e/d	C/d	τ_u (MPa)
This test	HWSC	CFRP	10	176	4	7	29.12
	HWSC	CFRP	13	119	4	5.3	23.94
	HWSC	CFRP	13	119	4	5.3	23.76
	HWSC	CFRP	13	119	4	5.3	20.67
	HWSC	CFRP	13	148	4	5.3	30.32
	HWSC	CFRP	13	148	4	5.3	27.87
	HWSC	CFRP	13	148	4	5.3	27.04
	HWSC	CFRP	13	148	4	5.3	27.95
	HWSC	CFRP	13	176	4	5.3	30.32
	HWSC	CFRP	13	176	4	5.3	29.21
	HWSC	CFRP	13	176	4	5.3	29.66
	SC	GFRP	10	54	4	9.5	20.55
	SC	GFRP	10	54	4	9.5	19.84
	SC	GFRP	10	54	4	9.5	15.95
	SC	GFRP	10	119	4	9.5	24.61
	SC	GFRP	10	119	4	9.5	25.65
	SC	GFRP	10	119	4	9.5	24.26
	SC	GFRP	10	148	4	9.5	32.71
	SC	GFRP	10	148	4	9.5	31.59
	SC	GFRP	10	148	4	9.5	33.78
	SC	GFRP	10	174	4	9.5	33.57
	SC	GFRP	10	174	4	9.5	33.42
	SC	GFRP	10	174	4	9.5	32.63

CHAPTER 5. LONG-TERM PERFORMANCE OF FRP BARS IN UHSSC UNDER ACCELERATED AGING CONDITIONS

5.1. Introduction

Previous research has shown that chloride ions introduced by marine sands can decrease the total porosity of hardened cement paste and optimize the pore distribution (Liu *et al.* 2016). In addition, a certain amount of chloride ions from raw materials can increase the early strength of concrete, and the capillary and gel pores of concrete can be filled with salt crystals (Lanas *et al.* 2006; Schere 1999; Younis *et al.* 2018). These observations thus suggest that the use of seawater and/or sea-sand in concrete may result in lower permeability and thus higher resistance to foreign matter compared to normal concrete, which has some beneficial effects on the durability of SW/SSC-wrapped FRP bars. Some studies found tensile and shear strength reductions of around or less than 20% when the concrete-wrapped specimens were conditioned at high temperatures (El-Maaddawy *et al.* 2016; Khatibmasjedi *et al.* 2020; Feng *et al.* 2023). However, findings about the durability of SW/SSC-wrapped FRP bars are inconclusive. A maximum of 34% to 50% reduction in tensile strength and 60% in horizontal shear strength has also been reported in existing studies (Morales *et al.* 2021; Li *et al.* 2020; Li *et al.* 2023). Although several studies have examined the degradation of FRP bars exposed to normal marine-concrete (i.e., saline-alkaline) environments, no research has focused on the long-term performance of UHSSC-wrapped FRP bars in coastal conditions.

This chapter presents a systematic study aiming to evaluate the long-term performance of FRP bars embedded in UHSSC in a simulated marine condition using accelerated aging tests. In the accelerated aging study, two batches of UHSSC mixtures and four types of FRP bars were tested. In total, 554 specimens, including 117 for pullout tests, 199 for tensile tests, and 238 for short-beam shear tests, respectively, were immersed in artificial seawater at room temperature (about 23°C), 40°C, and 60°C. For comparison, bare bars directly exposed to external conditions are also tested. After aging, various mechanical tests, including tensile test, short-beam shear test, and pullout test, are performed to evaluate the evolution of their mechanical properties and the possible correlation between the material degradation of FRP bars. These properties are considered sufficient to provide the necessary insight into the durability performance of the FRP bars and their synergy with the surrounding concrete,

thereby providing useful guidelines for practical applications.

5.2. Experimental program

5.2.1. Materials

This study utilized four types of 10-mm-diameter FRP bars, manufactured by two different producers: Dextra Building Products Co., Ltd, and Pulwell Composites Co., Ltd (denoted as P1 and P2, respectively). The physical and mechanical properties of unconditioned FRP bars can be found in section 3.2.1. Two UHSSC mixtures, referred to as S120 and S140, were used in this study. The information on raw materials in the two mix proportions was also detailed in section 3.2.1. Before conditioning, all concrete specimens were cured in a moist condition at ambient temperature (around 23°C) for 28 days.

5.2.2. Specimen preparation

Figure 5.1 presents three types of concrete-wrapped specimens (EB) prepared for the mechanical tests: pullout test specimens (PO specimens), short-beam shear test specimens (ILSS specimens), and tensile test specimens (TS specimens). All FRP bars were centrally embedded in concrete blocks with a cross-section of 150 × 150 mm. As shown in Figure 5.1 (a), the pullout specimens were prepared in accordance with CSA S806-12 (2012), with a bond length of four times the bar diameter ($L = 4d$). To ensure the specified bond length and to protect the bars from the external environment, both ends of the FRP bars outside the bonded part were encased with PVC tubes and sealants. For ILSS specimens, FRP bars were embedded in concrete prisms with a width of 6d and sealed at both ends. The FRP bars for tensile tests were embedded in 150-mm concrete blocks at the mid-length of the bars, as shown in Figure 5.1 (c). In a similar way, the protruded portions of TS specimens were covered with PVC tubes and end sealants. Note that the concrete-wrapped specimens were cast perpendicular to the longitudinal axis of the FRP bars. Regarding the bare bars (BB) for direct immersion in Figure 5.2, the cut ends were locally sealed with epoxy resin before aging to prevent the ingress of foreign matter from the cut sections. After conditioning, the UHSSC-wrapped specimens, except for PO specimens, were split and the embedded FRP bars were carefully extracted to avoid any damage to the bars for the following mechanical tests, as shown in Figure 5.3.

For a clear expression, the specimens are labeled using a similar letter-number system to that used in Chapter 3. The first letter indicates the bar type (G for GFRP and C for CFRP), followed by a number representing the producer (1, and 2, for P1, and P2). The letters in the second part represent the condition type of FRP bars (EB for embedded bars and BB for bare bars). The concrete grades are specified as “S140” and “S120”. The fourth and fifth parts denote, respectively, the aging temperature and duration of specimens (RT for room temperature).

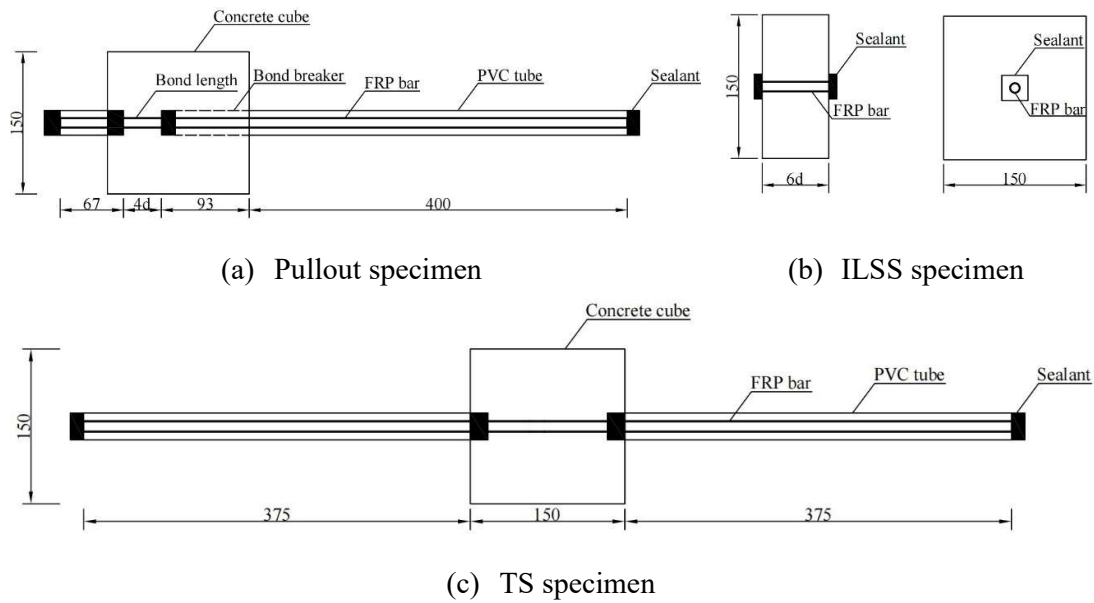
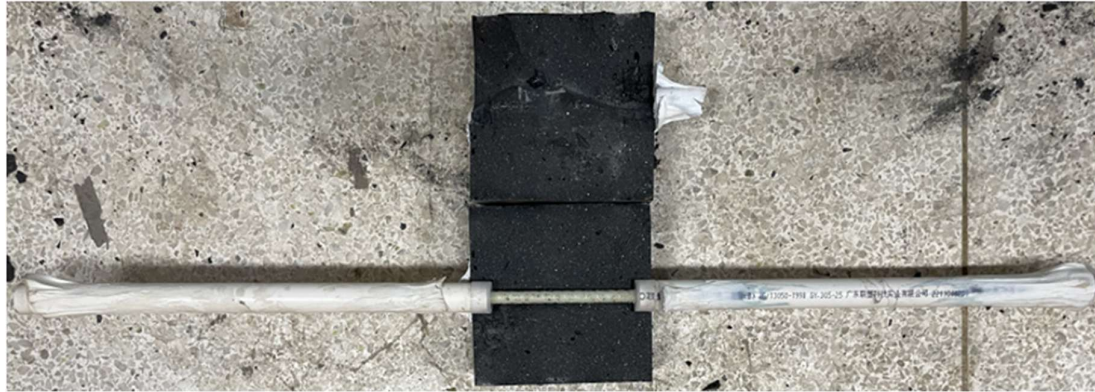


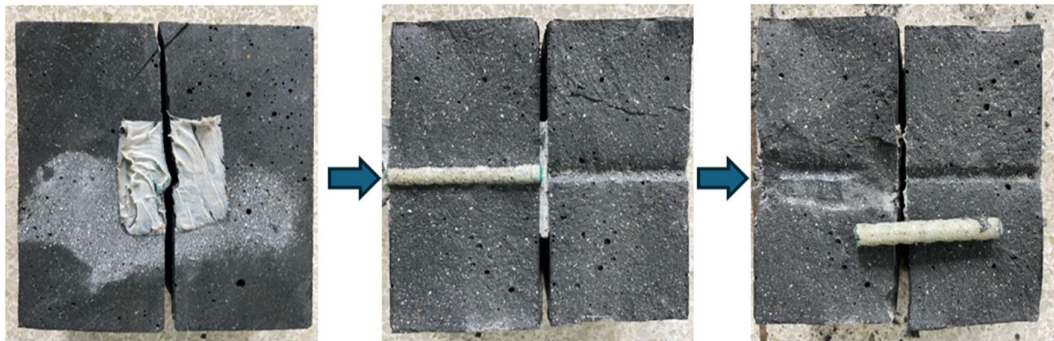
Figure 5.1. Schematic of concrete-wrapped specimens (unit: mm)



Figure 5.2. Sealing of bare bars



(a) TS specimen



(b) ILSS specimen

Figure 5.3 Extraction of embedded FRP bars

5.2.3. Accelerated aging conditions

After 28 days of curing, the control specimens were immediately tested for their benchmark properties. The remaining specimens were immersed in the artificial seawater at room temperature and two elevated temperatures, including 40 °C and 60 °C, using a temperature-controlled water tank, as shown in Figure 5.4. The artificial seawater used for conditioning was prepared according to ASTM D1141-98 (2013). For comparison, specimens of G2 and C2 bars, as well as of HSSC-wrapped G1 bars, were only immersed in seawater at 60 °C. The

specimens underwent conditioning for durations of 1, 3, 7, and 12 months, with some concrete-wrapped bars being immersed for up to 13.5 months.



Figure 5.4. Specimens in the conditioning tank

5.2.4. Test methods

5.2.4.1. Mechanical characterization of FRP bars

The tensile tests were conducted on FRP bars after conditioning to determine the residual tensile strength and tensile modulus of elasticity, according to ASTM D7205/D7205M-06 (ASTM 2016). All TS specimens were 900 mm long with a free length of at least 40d. Before testing, 200-mm-long steel-pipe anchors filled with epoxy resin were installed at both ends of GFRP bars, while CFRP bars used expansive cement-filled anchors with a length of 250 mm. Tests were performed using a universal testing machine from Matest (Model H001A) with a loading rate of 4 mm/min. To capture the elongation of FRP bars, each specimen was instrumented with an extensometer at midlength, as shown in Figure 5.5. The short-beam shear test and pullout test were performed per ASTM D4475-02 (ASTM 2021) and ASTM D7913/D7913M-14 (ASTM 2020), respectively, to examine the horizontal shear and bond durability of FRP bars. Detailed information on the two test methods can be found in Section 3.2.2.

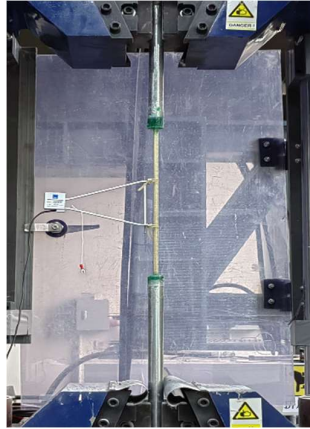
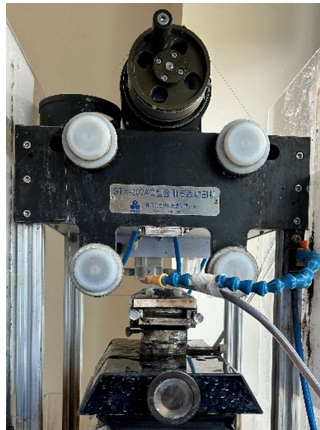


Figure 5.5. Tensile test setup

5.2.4.2. Microstructural analysis of FRP bars

In this Chapter, the SEM analysis was carried out on the control FRP bars to investigate their microstructure, including the porosity and the bond at the fiber-matrix interface. The 15 mm long sections of FRP bars were cut using a low-speed saw equipped with diamond wire at a moving speed of 1 mm/min, as shown in Figure 5.6 (a). The bar sections were then molded using the epoxy resin. After 24 hours of curing, the samples were polished on a grinding and polishing machine (see Figure 5.6 (b)) using four different grit levels (i.e., 180, 320, 600, and 1500), with each grit level lasting for two minutes. The fine polishing was then conducted using 9- μ m polycrystalline diamond paste. After polishing, the samples were dried in an oven at 50 °C for 24 hours after ultrasonic cleaning.



(a) Cutting using a low-speed saw



(b) Polishing bar sections

Figure 5.6. Preparation of SEM specimens

Before imaging, the surface of the polished samples was gold-coated using a sputter coater (Quorum, UK), as shown in Figure 5.7 (a). The SEM analysis was then performed using a

Merlin microscope (ZEISS, Germany) in Figure 5.7 (b).



(a) Gold coating



(b) SEM imaging

Figure 5.7. SEM analysis

5.2.4.3. Alkalinity in UHSSC

In addition to the compressive test, the pH measurement was conducted to estimate the alkalinity level within the UHSSC matrix. Following the test method proposed by Grubb *et al.* (2007), 5g of concrete powder was collected using a drill and mixed with 10 mL of fresh distilled water. After filtering the mixture using No. 40 filter paper, the alkalinity of the solution was measured using the pH probe, as shown in Figure 5.8. The pH measurement was performed on control S120 and S140 UHSSC mixtures after 28 days of curing and aged specimens after 12 months of conditioning at 60 °C. For aged specimens, the pH values of UHSSC substrates were measured at the level of FRP bars, as shown in Figure 5.9.

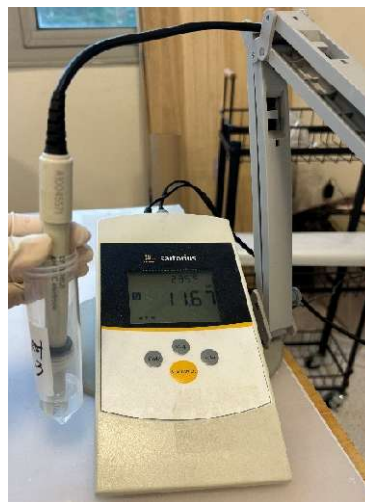


Figure 5.8. Measurement of alkalinity of the solution using the pH probe

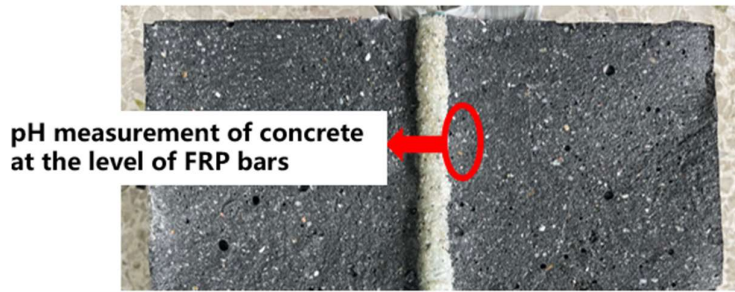


Figure 5.9. Sampling locations of aged specimens

5.3. Results and discussion

5.3.1. Properties of concrete

Figure 5.10 compares the compressive strength and modulus of elasticity of conditioned concrete specimens with their control groups after 28-day curing indicated by a purple dashed line as a reference value. It can be noted that the compressive strength of two UHSSC mixtures increased with time, irrespective of conditioning temperature, and only slight drops of less than 8% were observed in the S140 specimen set from 7-month aging to 12-month aging. Generally, all concrete specimens exhibited higher compressive strength after seawater immersion in comparison to the control groups. The main explanation is that the hydration of the cement paste continued when the specimens were immersed in artificial seawater. After 12 months of aging, the compressive strength reached 181.3 MPa, 172.5 MPa, and 200.9 MPa for the S140 specimen set conditioned at room temperature, 40 °C, and 60 °C, respectively. For the S120 specimen set aged for 12 months at 60 °C, the compressive strength increased to 155.5 MPa.

Similar to the changes in compressive strength, Figure 5.10 (b) shows that the elastic modulus of conditioned specimens was significantly higher than that of the control groups. The elastic modulus of two batches of conditioned UHSSC specimens generally increased over aging time. Specifically, increases of 18.0% to 26.8% for the S140 specimen set and 7.8% for the S120 specimen set were reported after 12-month aging. Based on these observations, it can be found that the UHSSC did not exhibit deterioration under seawater immersion, suggesting relatively good durability of UHSSC.

The pH measurement results show that the average pH values of the control S120 and S140 mixtures were 11.8 and 11.7, respectively, which are lower than the alkalinity level of mature ordinary portland cement concrete, typically having a pH value of 12.5 to 13 (ACI 2012; Behnood *et al.* 2016). This suggests that the UHSSC environment is less aggressive to FRP bars than the normal concrete environments. After 12 months of seawater immersion at high temperature, the alkalinity level at the depth of FRP bars remained almost unchanged, with the average values being 11.6.

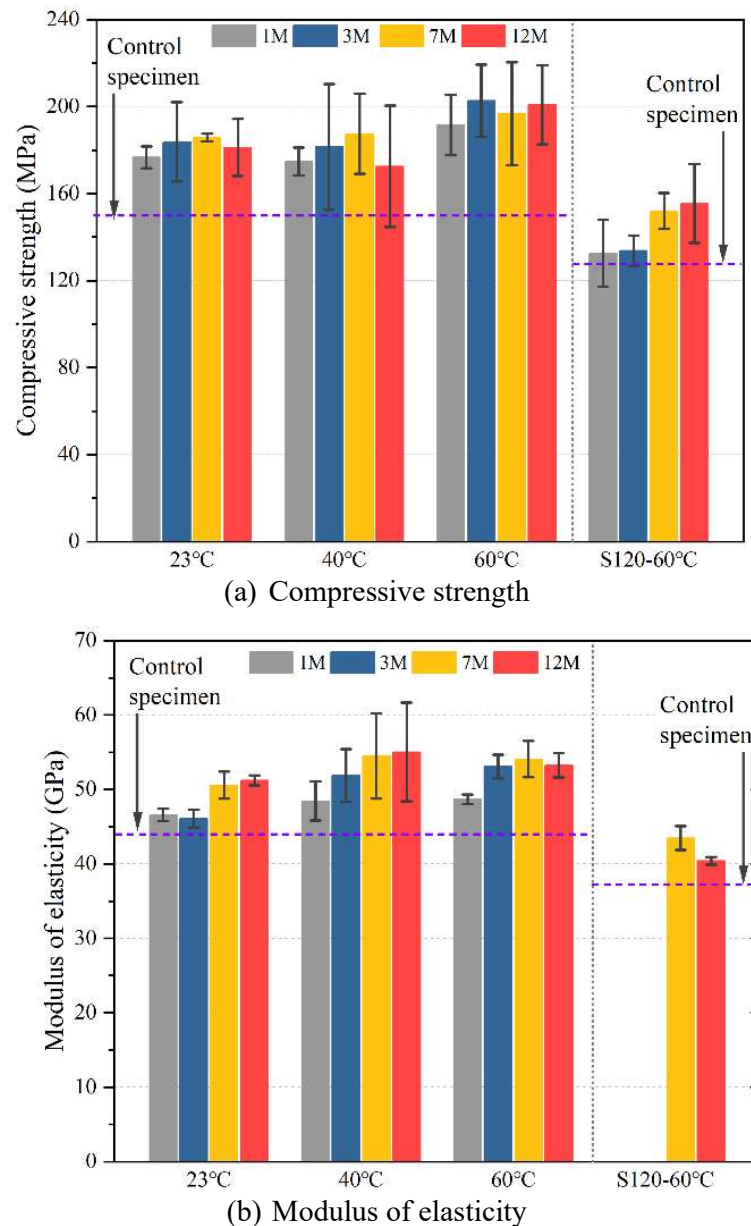


Figure 5.10. Concrete mechanical properties after exposure

5.3.2. Tensile properties of FRP bars

5.3.2.1. Failure modes

As shown in Figure 5.11 and Figure 5.12, all FRP bars failed by rupture at the middle of the free length before and after conditioning. Figure 5.11 illustrates that a violent burst, like a nest, was observed for both control and conditioned CFRP bars. However, that was not the case for GFRP bars. The GFRP bars aged at elevated temperatures exhibited a similar but less catastrophic failure, with more localized rupture compared to the control specimens and those immersed at room temperature. A similar failure mode has been observed in previous studies (Mukherjee and Arwikaar 2005; Robert *et al.* 2009; Davalos *et al.* 2011; Lu *et al.* 2021). This is probably due to the premature breakage of eroded fibers, resulting in the opening of cracks in the matrix and fiber-resin interface debonding at the peripheral region of conditioned GFRP bars when the tensile force is applied. This affects the stress transfer between fibers and resin, causing a sudden and localized rupture in the central part of the bars.

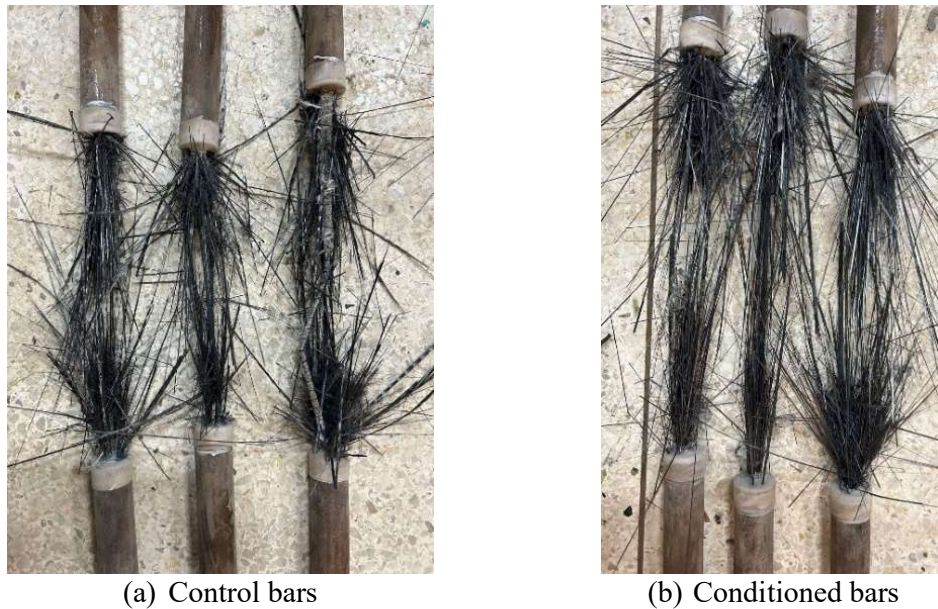


Figure 5.11. Typical failure modes of control and conditioned CFRP specimens

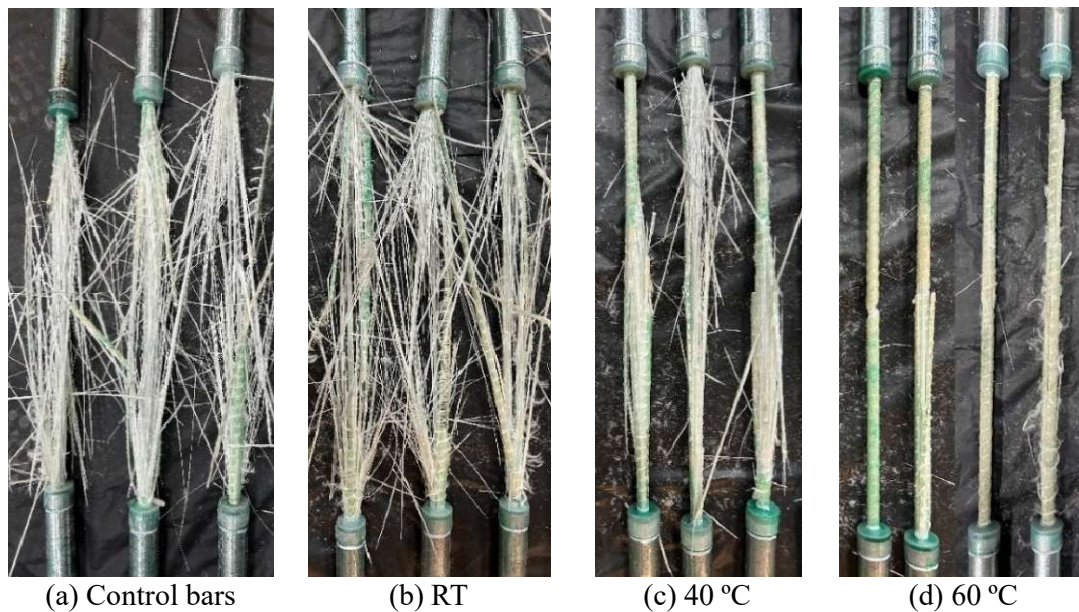


Figure 5.12. Typical failure modes of control and conditioned GFRP specimens aged at different temperatures

5.3.2.2. Tensile strength

The residual tensile strength and corresponding retention percentages of conditioned FRP bars are shown in Table 5.1, Figure 5.13, and Figure 5.14, respectively. As expected, neither bare CFRP bars nor CFRP bars in the saline-concrete environment experienced a noticeable reduction in tensile strength after around one year of aging. The maximum tensile strength reduction of CFRP bars was less than 6% under seawater immersion. In contrast, the GFRP bars underwent more significant degradation in tensile strength after the same conditioning. Specifically, bare G1 bars exhibited a maximum reduction of 7.1% in tensile strength after 12 months of seawater immersion at room temperature. Due to the accelerated diffusion rate of water molecules and other ions at higher temperatures, the maximum tensile strength reduction increased to 43.5% with an elevated aging temperature of 40 °C, and at 60 °C, the most significant degradation was reported, with a 60.3% tensile strength reduction. Since C1 and G1 bars had the same resin matrix and were manufactured following similar processes, the difference in tensile strength retention between CFRP and GFRP bars can be attributed to the inferior durability performance of glass fibers. This is also confirmed by Wang et al. (2023). The glass fibers in the edge of GFRP bars lost their tensile resistance due to erosion under environmental attack. When the GFRP bars were in tension, the peripheral eroded glass fibers experienced premature failure, which affected the stress transfer between the matrix

and fibers in the interior region of bars and thus caused the premature rupture of GFRP bars. For CFRP bars, the chemically inert carbon fibers exhibited excellent durability performance.

More importantly, Figure 5.14 (a) shows that the UHSSC-wrapped GFRP bars exhibited less significant tensile strength reduction compared to the corresponding bare bars. The UHSSC-encased G1 and G2 under seawater immersion retained 10% more tensile strength retention than their counterparts exposed directly to seawater when conditioned at elevated temperatures for seven months or more. Although it is well-documented that saline contents have a less significant effect on tensile strength degradation of GFRP than the alkalinity of concrete pore solution (Chen *et al.* 2007; Manalo *et al.* 2020; Lu *et al.* 2021; Wang *et al.* 2023), the impact mesostructure of UHSSC retards the ingress of external water molecules into the FRP bars, thus slowing the alkaline attack of the pore solution on FRP bars. This indicates that, compared to direct exposure to a seawater environment, the UHSSC environment does not adversely affect the tensile properties of FRP bars but decelerates their tensile strength degradation rate.

In addition, Figure 5.13 (a) and Figure 5.14 (a) compare the effect of bar type on the tensile durability of FRP bars under seawater immersion. In contrast to G1 bars, slightly better durability performance was observed in G2 bars. To be more specific, bare G2 bars presented a tensile strength retention of 49.1% after 12 months of seawater immersion at 60 °C, marking a 23.8% increase compared to bare G1 bars aged under the same condition. Similar observations were reported for the G1 and G2 bars embedded in UHSSC. These experimental findings can be attributed to the different chemical resistance of FRP bars with different fiber, resin types, and sizings (Benmokrane *et al.* 2020).

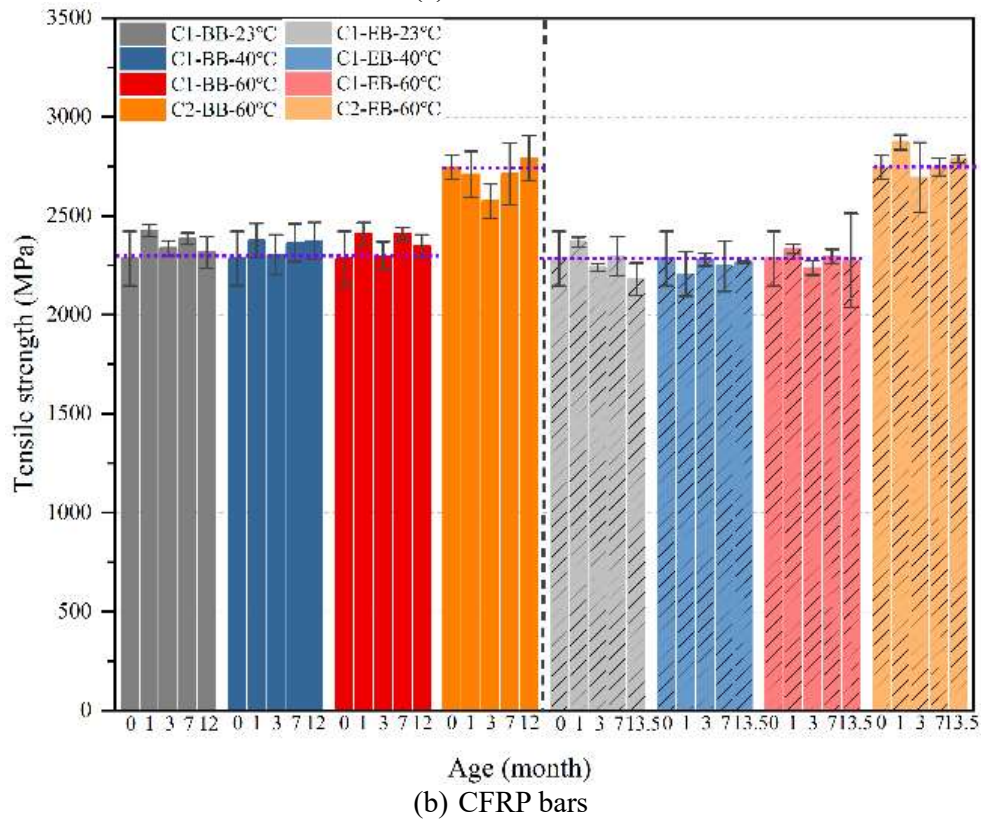
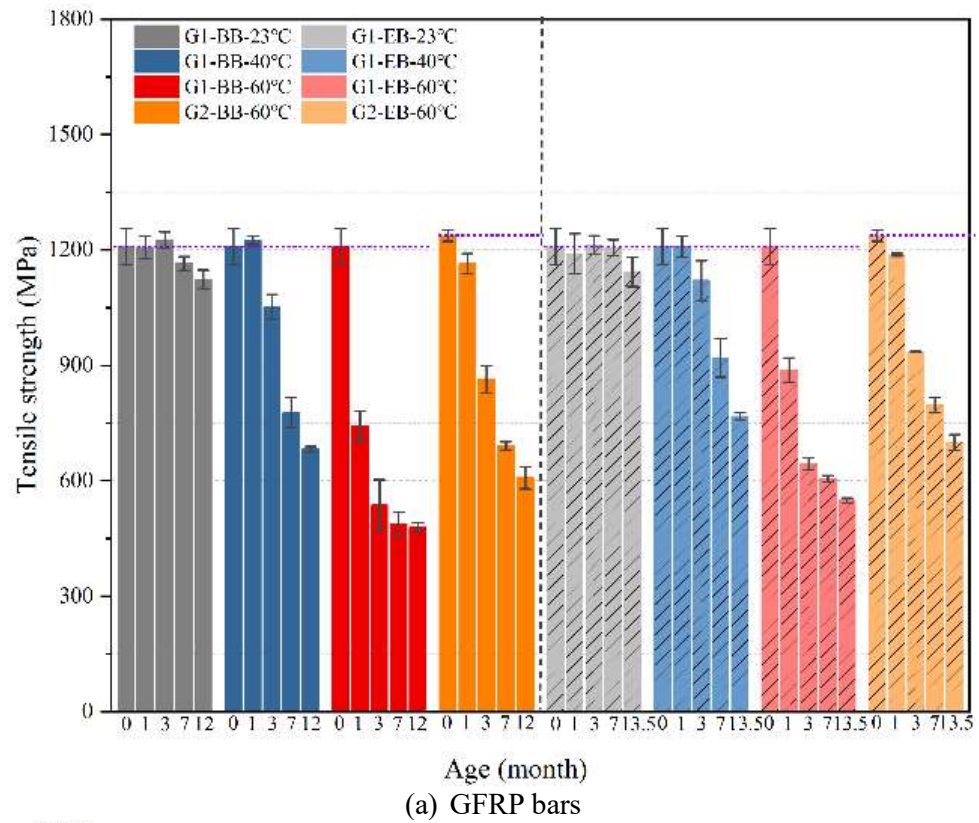
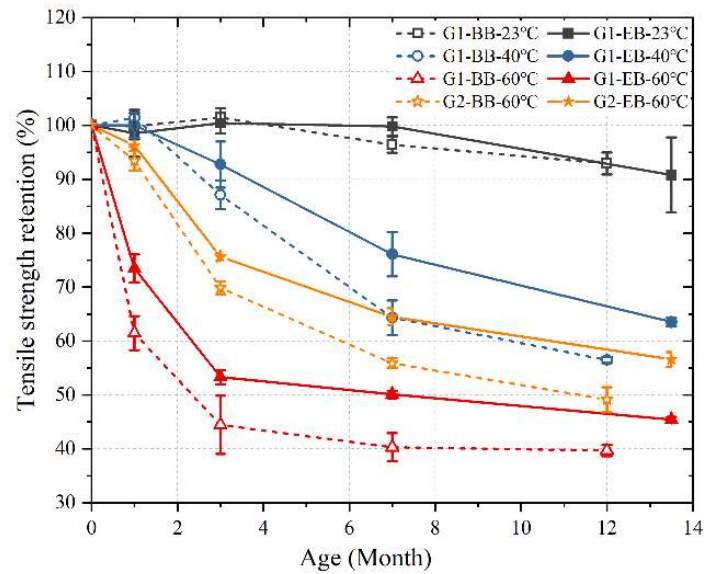
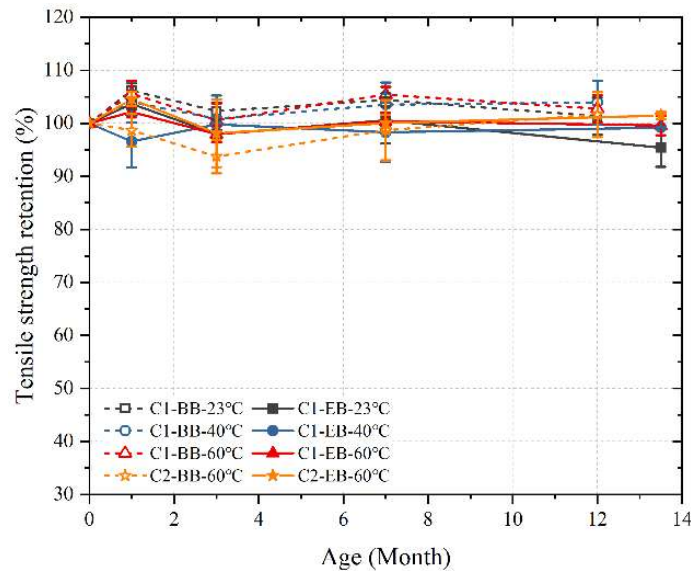


Figure 5.13. Residual tensile strength of conditioned FRP bars



(a) GFRP bars



(b) CFRP bars

Figure 5.14. Tensile strength retention of conditioned FRP bars

5.3.2.3. Tensile elasticity of modulus

Figure 5.15 and Table 5.1 present the change in tensile modulus of elasticity of bare FRP bars after 12 months of immersion at high temperature and UHSSC-wrapped FRP bars after 13.5 months of conditioning at 60 °C. Clearly, both bare bars and UHSSC-wrapped FRP bars showed negligible loss of modulus of elasticity, irrespective of bar type. This suggests that the modulus of elasticity of FRP bars was not affected by seawater immersion and saline-concrete environment within the UHSSC matrix, compared to the tensile strength. Similar experimental results for various FRP bars under different exposure conditions were also

reported in previous research (Robert *et al.* 2013; Wang *et al.* 2017a; Wang *et al.* 2018). This is attributed to the insignificant degradation of tensile modulus of fibers, which controls the modulus of unidirectional FRP bars.

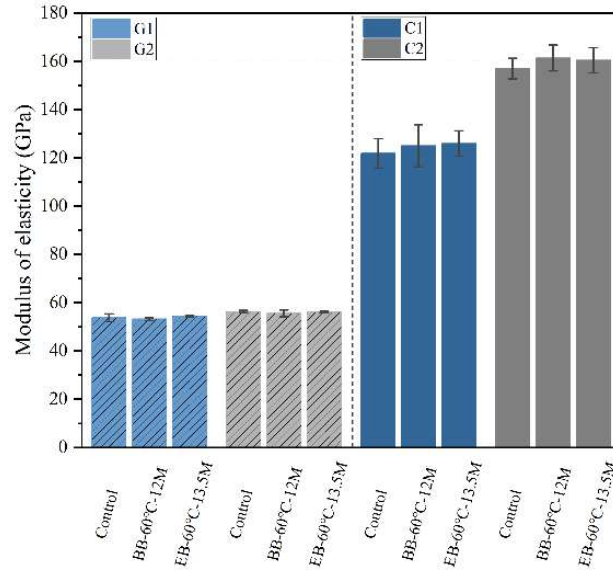


Figure 5.15. Modulus of elasticity of conditioned FRP bars

Table 5.1 Summary of residual tensile properties and horizontal shear strength

Bar type	Concrete cover	T (°C)	Age (m)	Tensile strength (MPa)		Modulus of elasticity (GPa)		Horizontal shear strength (MPa)	
				Mean	SD	Mean	SD	Mean	SD
G1	BB	23	1	1206	29	--	--	28.91	1.13
			3	1226	21	--	--	29.34	0.82
			7	1165	18	--	--	28.65	1.11
			12	1123	25	52.0	0.4	28.48	0.32
		40	1	1225	11	--	--	29.79	0.55
			3	1053	32	--	--	29.19	0.87
			7	777	39	--	--	28.42	0.85
			12	682	7	51.1	3.2	28.36	1.71
		60	1	742	38	--	--	30.17	0.37
			3	537	66	--	--	28.65	0.61
			7	487	32	--	--	27.50	1.01
			12	478	12	53.2	0.6	27.96	1.29
G2	BB	60	1	1165	26	--	--	34.17	3.36
			3	864	36	--	--	33.77	1.41
			7	691	11	--	--	34.38	0.96
			12	607	29	55.6	1.4	31.79	1.85
G1	EB-S140	23	1	1191	52	--	--	28.63	0.81
			3	1214	24	--	--	29.01	2.14
			7	1206	20	--	--	29.49	0.40
			12 (13.5)	1143	38	54.2	0.6	29.33	0.29
		40	1	1208	28	--	--	29.92	0.97

			3	1121	52	--	--	29.05	0.46
			7	920	50	--	--	29.41	0.64
			12 (13.5)	768	9	56.9	6.0	29.60	0.89
			1	888	32	--	--	29.62	1.16
		60	3	644	16	--	--	30.02	0.64
			7	605	8	--	--	30.12	1.85
			12 (13.5)	549	6	54.4	0.4	30.24	0.66
			1	--	--	--	--	30.08	1.65
	EB-S120	60	3	--	--	--	--	29.08	0.56
			7	--	--	--	--	29.70	0.34
			12	--	--	--	--	30.73	1.12
G2	EB-S140	60	1	1187	3	--	--	31.85	1.54
			3	935	2	--	--	33.07	0.89
			7	797	20	--	--	33.44	1.53
			12 (13.5)	700	20	56.1	0.3	33.56	2.32
			1	2426	31	--	--	24.62	0.72
		23	3	2337	37	--	--	25.83	0.60
			7	2385	29	--	--	25.05	1.46
			12	2316	80	126.3	4.0	29.36	0.91
			1	2376	86	--	--	29.94	1.16
C1	BB	40	3	2304	101	--	--	23.75	1.58
			7	2365	96	--	--	23.11	1.12
			12	2374	93	127.8	3.9	21.59	0.60
			1	2411	56	--	--	22.34	0.57
		60	3	2298	72	--	--	21.97	0.91
			7	2408	33	--	--	21.28	0.72
			12	2347	58	125.0	8.7	20.13	0.06
			1	2710	116	--	--	40.23	1.71
C2	BB	60	3	2575	87	--	--	38.74	1.67
			7	2711	156	--	--	39.57	2.60
			12	2792	116	161.4	5.3	38.61	0.86
			1	2368	24	--	--	24.02	1.47
		23	3	2238	18	--	--	23.81	1.03
			7	2297	99	--	--	24.54	0.79
			12 (13.5)	2180	83	120.2	3.4	24.36	0.34
			1	2207	112	--	--	25.40	1.37
C1	EB-S140	40	3	2280	34	--	--	23.48	1.09
			7	2246	126	--	--	23.85	1.35
			12 (13.5)	2266	4	122.8	11.3	23.40	2.17
			1	2334	23	--	--	24.50	0.86
		60	3	2238	36	--	--	22.33	0.94
			7	2294	37	--	--	23.33	2.44
			12 (13.5)	2275	239	125.9	5.2	22.94	0.81
			1	2871	38	--	--	43.34	1.51
C2	EB-S140	60	3	2695	176	--	--	41.57	0.85
			7	2747	44	--	--	42.01	2.26
			12 (13.5)	2787	21	160.4	5.2	40.31	0.07

Note: the fourth batch of UHSSC-wrapped TS specimens was tested after 13.5 months of conditioning.

5.3.3. Horizontal shear properties of FRP bars

5.3.3.1. Failure mode

Figure 5.16 illustrates the typical shear failure mode of FRP bars subjected to the short-beam test. All specimens exhibited horizontal shear failure before and after conditioning, with a major crack initiating and propagating horizontally along the mid-plane of the FRP bars.

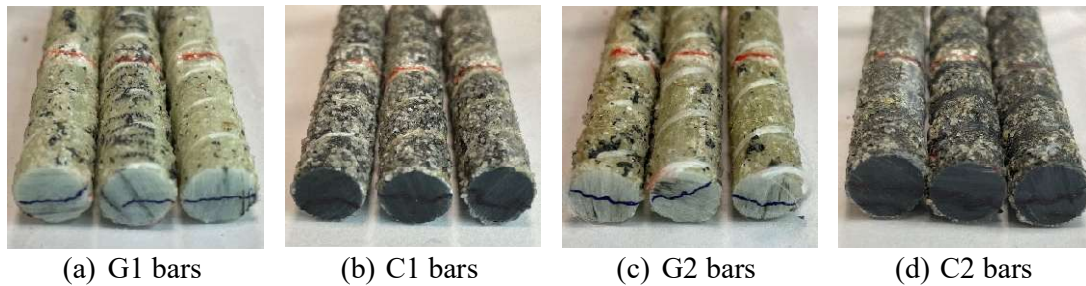


Figure 5.16. Typical failure mode of ILSS specimens

5.3.3.2. Horizontal shear strength

The experimental results of residual horizontal shear strength and corresponding retention for FRP bars after aging are summarized in Table 5.1, Figure 5.17, and Figure 5.18. Generally, the horizontal shear strength of conditioned bare bars declined over time, except for bare CFRP bars immersed at lower temperatures. Notably, about a 20% increase in shear strength was reported in bare C1 bars aged at room temperature for 12 months and 40 °C for 1 month. This can be attributed to the swelling effect of polymer matrix under direct immersion, which induces strong confinement to internal carbon fibers and consequently closes the gaps inside the CFRP bar core that will be further discussed below (Rolland *et al.* 2021). However, the greater material degradation of CFRP bars at high conditioning temperature counteracted this positive effect and thus resulted in the loss of horizontal shear strength at 60 °C.

For GFRP bars, the seawater environment caused less considerable degradation in the horizontal shear strength compared to the tensile strength. After 12 months of exposure to seawater, the horizontal shear strength retentions were 95.7%, 95.3%, and 94.0% for bare G1 bars conditioned at room temperature, 40 °C, and 60 °C, respectively. In addition, the bare G2 bars decreased by only 6% in horizontal shear strength (a reduction of 0.9 MPa) after 12-month immersion at 60 °C. This indicates that the seawater immersion had a marginal effect

on the horizontal shear strength of GFRP bars.

In contrast, relatively inferior shear durability was observed for CFRP bars, with much more pronounced degradation in horizontal shear strength than in tensile strength. Specifically, Figure 5.18 shows that the decreases in horizontal shear strength were 13.1% and 18.9% for bare C1 bars after 12 months of aging at 40 °C and 60 °C, respectively. For bare C2 bars, the horizontal shear strength was reduced by 5.6 MPa in Figure 5.17 (around 13%) after 12-month immersion at 60 °C. A similar trend was noted in previous studies (Wang *et al.* 2017; Benmokrane *et al.* 2020), which reported that the horizontal shear durability of CFRP bars was not as good as that of GFRP. This difference in horizontal shear durability in CFRP and GFRP bars can be explained by SEM analysis. As evidenced by the SEM observations in Figure 5.19, in comparison to GFRP bars, CFRP bars exhibited higher porosity with conspicuously long strips of voids densely distributed across the resin matrix within the bar section, which were formed during manufacturing. A closer micrograph in Figure 5.19 (c) also showed some debonding of the fiber-resin interface in control CFRP bars. These defects, as concluded in previous research, are mainly caused by the immature compatibility between fibers and resin due to the low surface energy of carbon fibers, indicating a poor bond between carbon fibers and the resin matrix (Wang *et al.* 2015; Tang and Kurdos 2004). Previous research reported that the horizontal shear strength depends on the shear resistance of matrix and the fiber-resin interface, not on fibers (Chen *et al.* 2007; Benmokrane *et al.* 2020). These defects facilitate the ingress of solutions that can induce more significant local damage in the form of resin deterioration and the extension of fiber-resin interface debonding, leading to a higher degradation rate of horizontal shear strength in CFRP bars.

When encased in UHSSC, all types of FRP bars yielded significantly lower degradation rates of horizontal shear strength compared to their counterparts directly immersed in seawater. To be more specific, after 12 months of exposure at 60 °C, almost no reduction was observed for the horizontal shear strength of both UHSSC-wrapped G1 and G2 bars. Lower reductions of 1.9% to 7.6% in horizontal shear strength were reported for UHSSC-wrapped C1 bars at the end of 12-month conditioning with three different temperatures. For C2 bars embedded in UHSSC, the maximum deterioration was only 8.9%, which showed a 30.5% reduction in degradation compared to the counterpart directly immersed in seawater. A similar positive effect of UHSSC cover on durability was also reported for S120 specimen sets: as shown in Figure 5.17 (a) and Figure 5.18 (a), the G1 bars embedded in the S120 set did not exhibit

significant degradation in horizontal shear strength. This observation confirms that neither saline-alkalinity environment in the two UHSSC mixtures is detrimental to the horizontal shear durability of FRP bars, but effectively protects the bars from seawater.

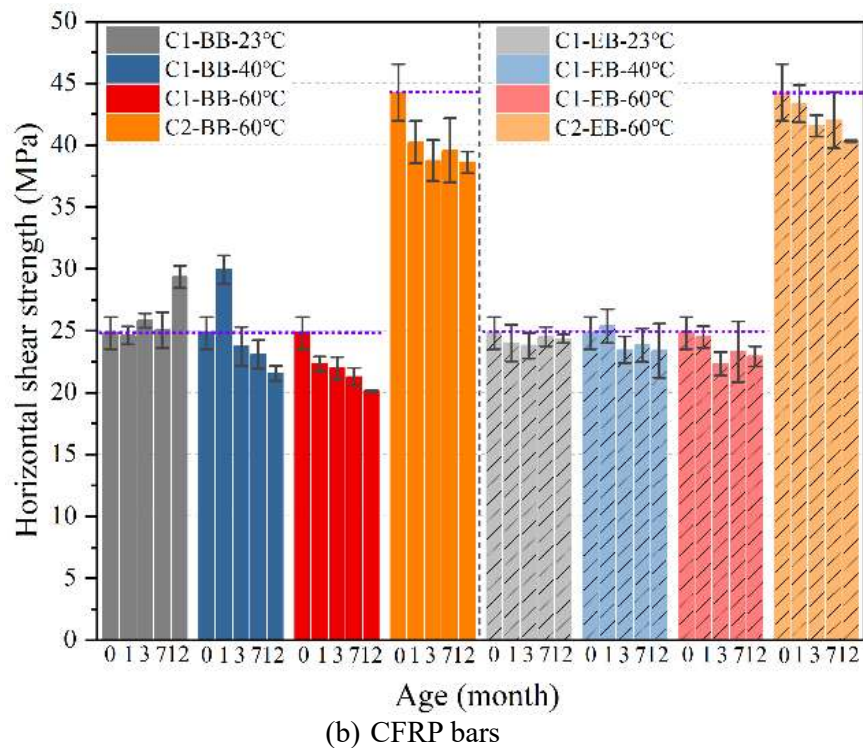
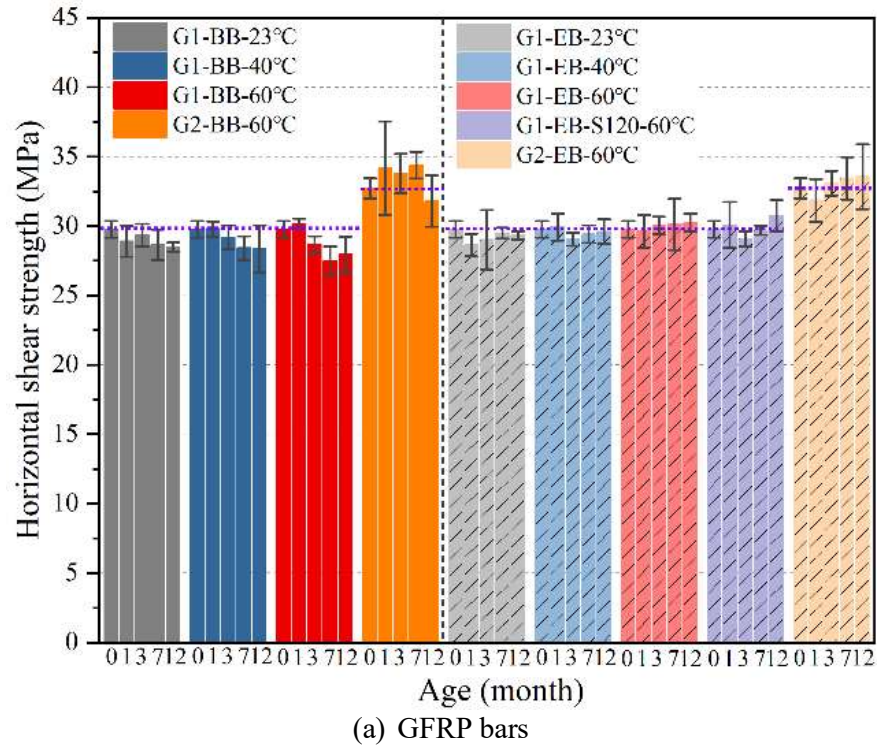


Figure 5.17. Residual horizontal shear strength of conditioned FRP bars

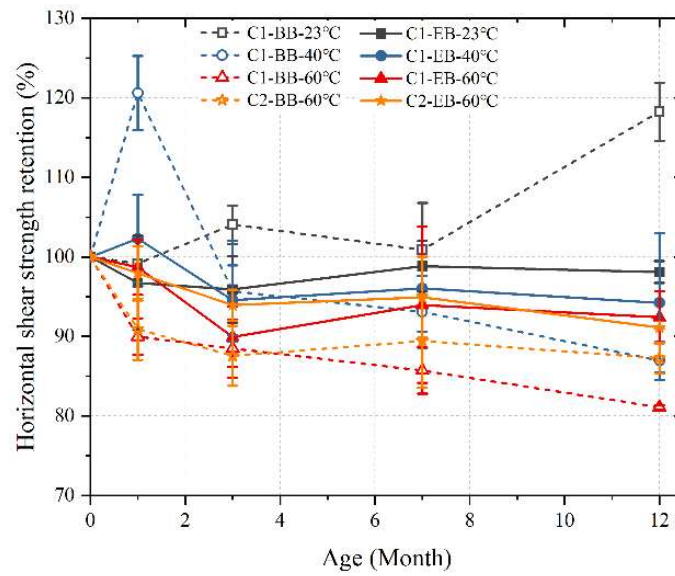
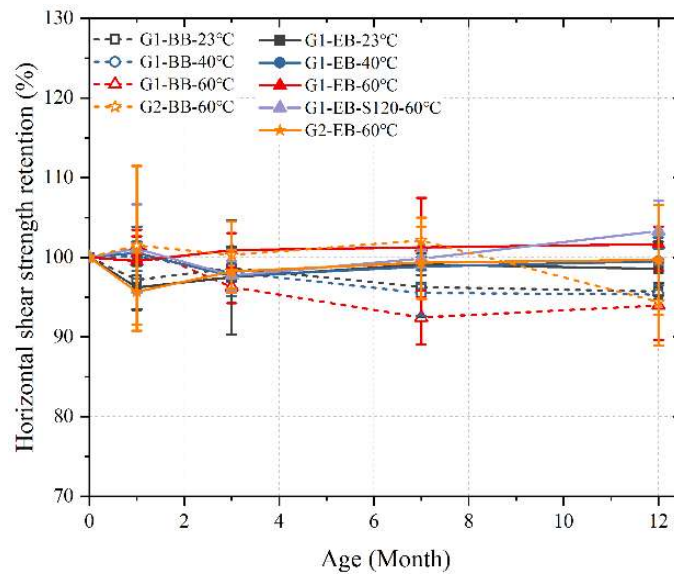
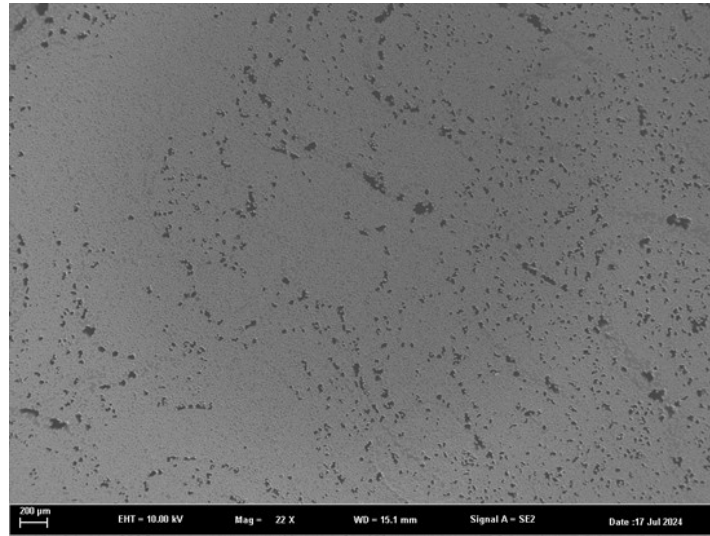
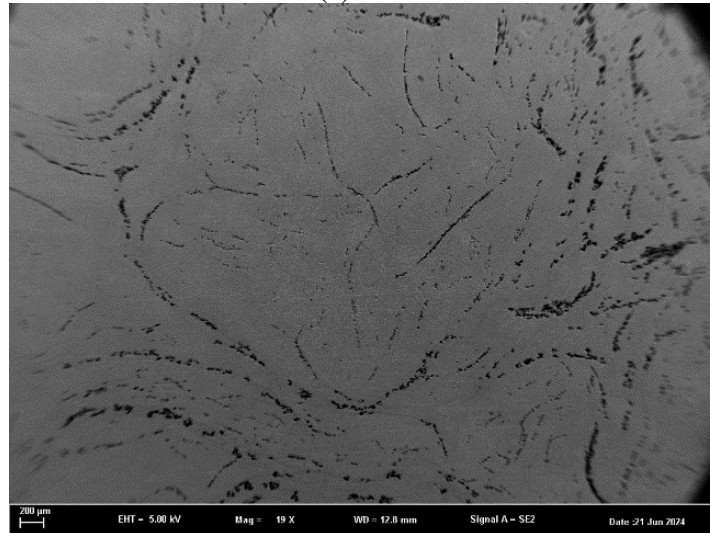


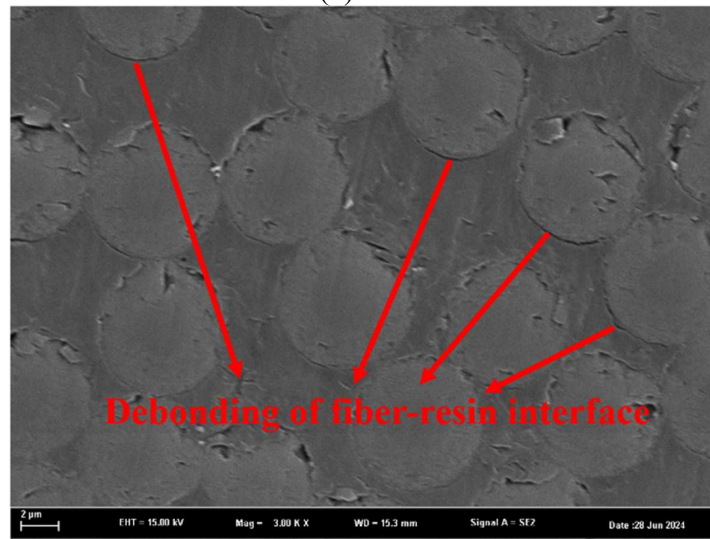
Figure 5.18. Horizontal shear strength retention of conditioned FRP bars



(a) G1 bars



(b) C1 bars



(c) Magnified image of CFRP bars

Figure 5.19. Typical micrographs of the cross-section of reference FRP bars

5.3.4. FRP bar-UHSSC bond properties

5.3.4.1. Failure mode

All specimens failed by pulling out of the FRP bars without splitting the concrete prisms. After pullout tests, the specimens were split for visual examination of the failure surface. Representative failure surfaces are shown in Figure 5.20. Both control and conditioned S140 specimen sets exhibited the same failure pattern: the concrete matrix remained intact, and all FRP bars were damaged, with delamination of the entire surface layer of FRP bars and rupture of some internal fibers at the bar core. This observation indicates that the bond failure primarily occurred between the surficial resin-rich layer and the fibers in the successive core layers. Consequently, the bond strength of the FRP bars in UHSSC is governed by the interlaminar shear capacity between the resin and fiber at the bar core rather than the concrete strength.

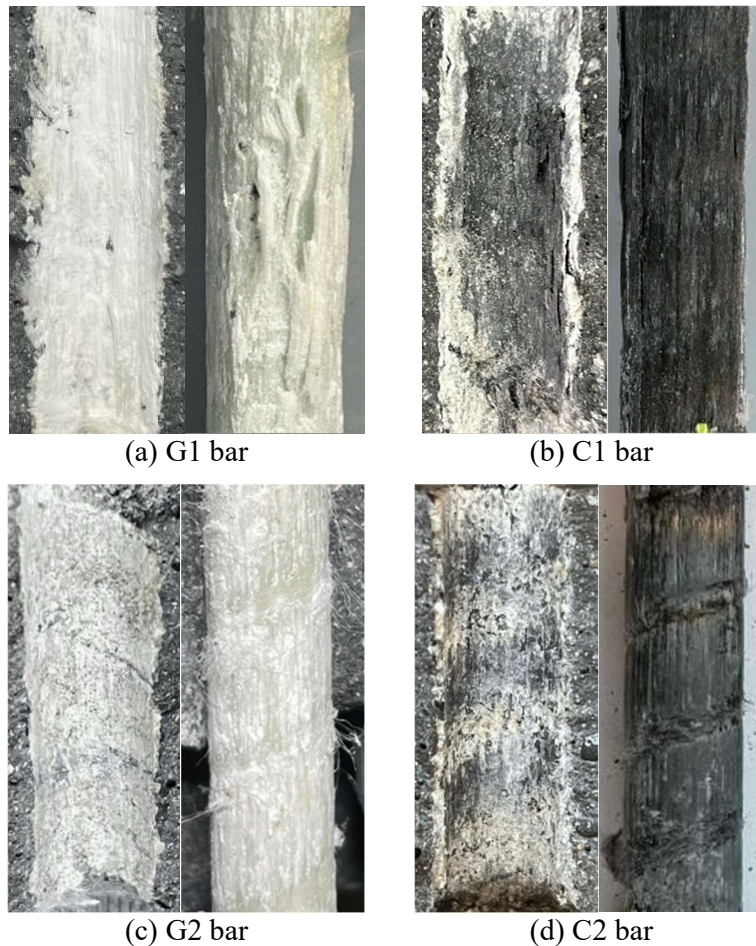


Figure 5.20. Typical failure of conditioned FRP bars in S140 sets

For the S120 specimen set, the failure surface changed after conditioning, as shown in Figure 5.21. The control specimens mainly failed at the bar-concrete interface, with partial delamination of the surface layer of FRP bars and partial crushing of concrete near the interface, while the bond failure occurred solely within the FRP bars for conditioned specimens. This change is due to the increase in concrete strength of the S120 specimen set during conditioning (see Figure 5.10), where the lower shear strength of FRP bars controlled the bond failure, rather than the increased concrete shear strength.

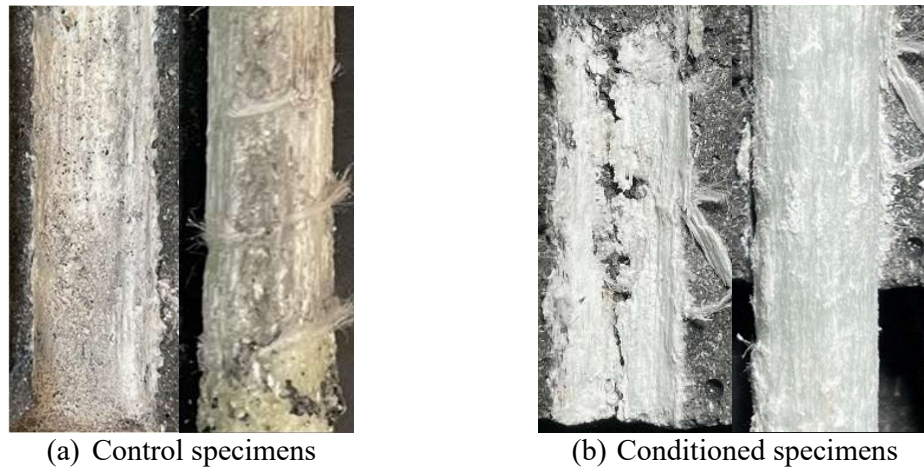


Figure 5.21. The debonding surface of control and conditioned bars in the S120 set

5.3.4.2. Bond stiffness

Bond stiffness is a critical component influencing the deflection of FRP bar reinforced concrete members. The bond stiffness was determined as the secant slope of the segment in the bond stress-loaded end slip curve between 25% to 50% τ_b . Figure 5.22 and Table 5.2 show the bond stiffness for both control and conditioned specimens aged at high temperature. For the S140 specimen sets, no significant deterioration in bond stiffness was observed for all types of FRP bars across varying conditioning periods. The changes in the S120 specimen set were also compared in Figure 5.22. It can be found that the control G1 bars in the S120 concrete matrix exhibited a significantly higher bond stiffness of 71 MPa/mm compared to their counterparts in the S140 matrix, which had a bond stiffness of 54 MPa/mm. However, immersion in seawater at 60 °C resulted in a decrease in the bond stiffness for the S120 specimen set, and after 12 months, the bond stiffness fell to 50 MPa/mm, comparable to that

of the same FRP bars in S140 specimen sets. This is probably attributed to the change of failure mode of the S120 specimen set with increasing concrete strength during conditioning, as mentioned earlier. The shear stiffness of the bar-concrete interface (the debonding surface of control S120 specimens) appeared to be higher than that of the internal layers of FRP bars, where the bond failure occurred for the conditioned S120 specimen set.

5.3.4.3. Bond strength

Figure 5.23, Figure 5.24, and Table 5.2 compare the bond strength of different types of FRP bars conditioned at accelerated aging environments. The results indicate that the bond durability between FRP bars and UHSSC under seawater immersion was not a critical issue. Specifically, the bond strength of conditioned G1 and G2 bars decreased by less than 2.4% after 12-month conditioning. The maximum bond strength reductions for C1 and C2 bars were 6.0% and 14% evidenced under the most aggressive condition (i.e., the highest aging temperature of 60 °C and the longest conditioning duration of 365 days), and the bond strength of degraded specimens tended to stabilize within 12 months of aging. For the S120 specimen set, an increase of approximately 12% in bond strength was found, mainly due to the increase in strength of the concrete matrices during thermal aging.

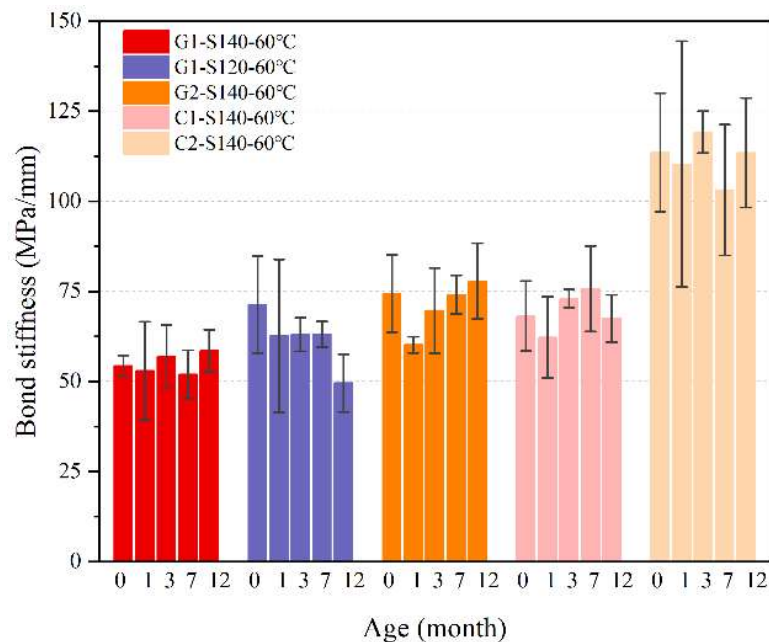
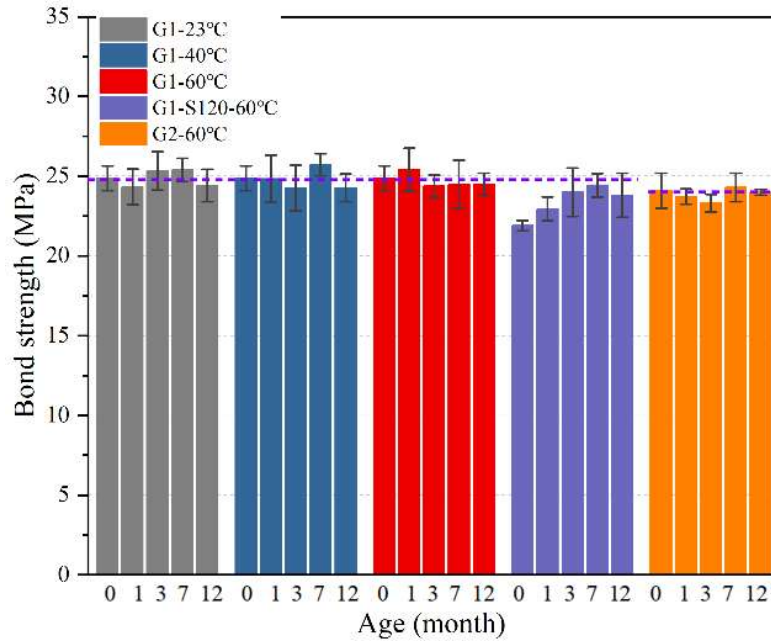
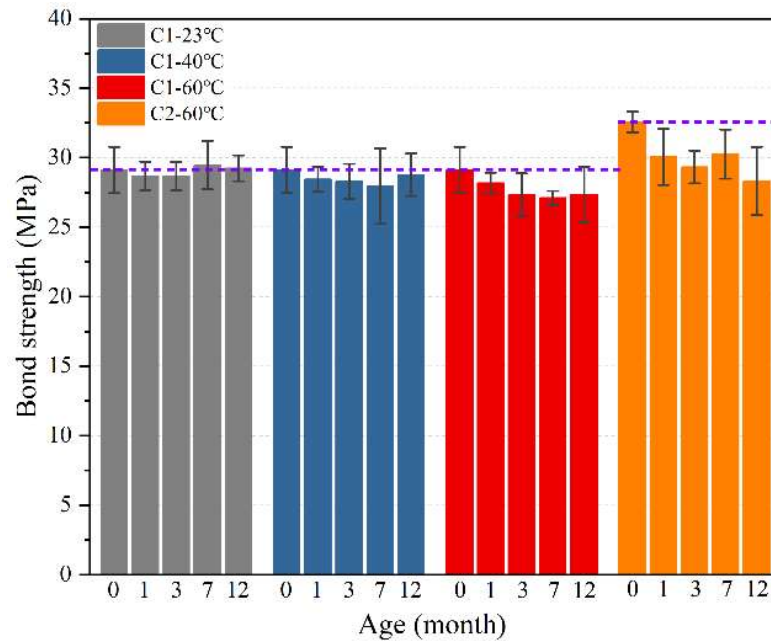


Figure 5.22. Bond stiffness of conditioned FRP bars



(a) GFRP bars



(b) CFRP bars

Figure 5.23. Bond strength of conditioned FRP bars

Interestingly, all GFRP bars in UHSSC demonstrated better bond durability under seawater immersion than the CFRP bars, which reported decreases of 6% to 14% for C1 and C2 bars. This is primarily because of the relatively poor durability of shear properties of CFRP bars, as discussed previously in section 5.3.3.2. In S140 specimen sets, the bond performance was controlled by the horizontal shear properties of FRP bars. A certain reduction in horizontal

shear strength of embedded CFRP bars consequently brought a corresponding decline in bond strength. The potential relationship between the bond strength degradation and the deterioration in horizontal shear strength of FRP bars will be analyzed in Chapter 7.

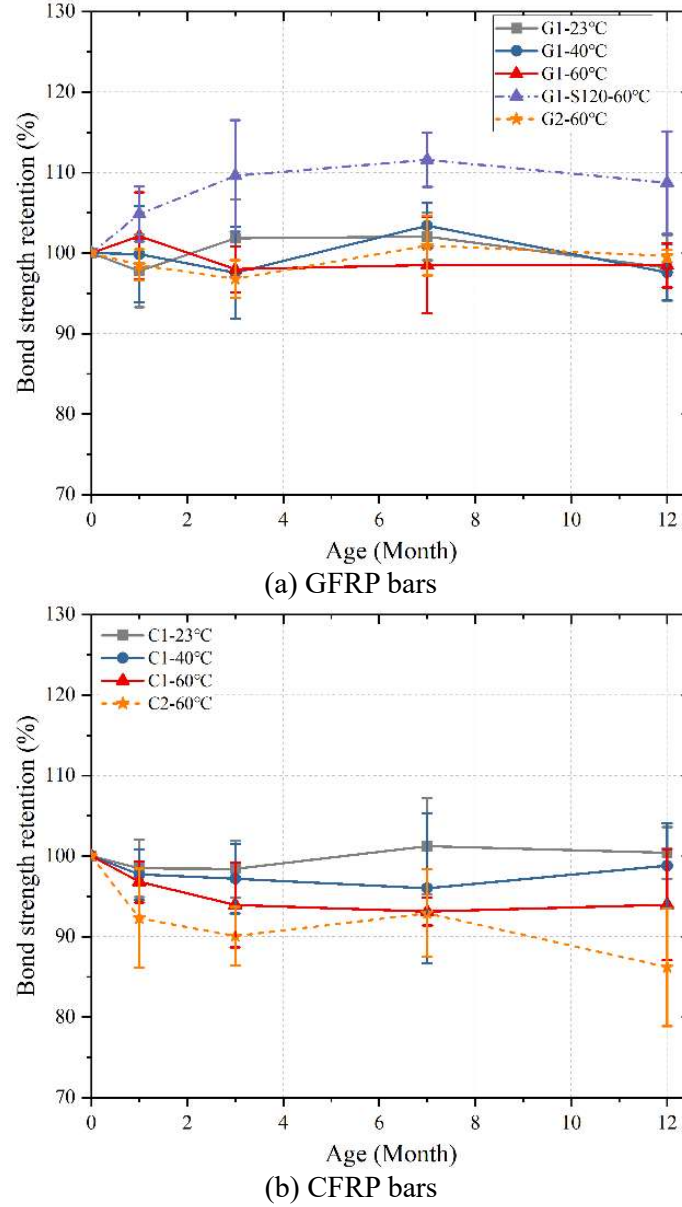


Figure 5.24. Bond strength retention of conditioned FRP bars

5.3.4.4. Slip at bond strength and relative post-peak strength

In addition to bond strength, two other bond characteristics, namely the corresponding slip at bond strength s_1 and the relative post-peak strength τ_{res}/τ_b , were investigated in this

section, as given in Figure 5.25, Figure 5.26, and Table 5.2. By convention, the ratio of residual strength to the corresponding bond strength is used to represent the post-peak behavior of FRP bars. Test results show that the relative post-peak strength of all UHSSC specimens did not vary significantly under seawater immersion at different temperatures, and no clear trend was observed. This indicates that the seawater-UHSSC environment has a marginal impact on the relative post-peak strength of FRP bars. However, more research is required to investigate the effect on characteristic slips at bond strength, since they exhibited high scatter (see Figure 5.25) due to irregularity of surface coating condition of tested FRP bars as discussed in Section 3.3.3.7.

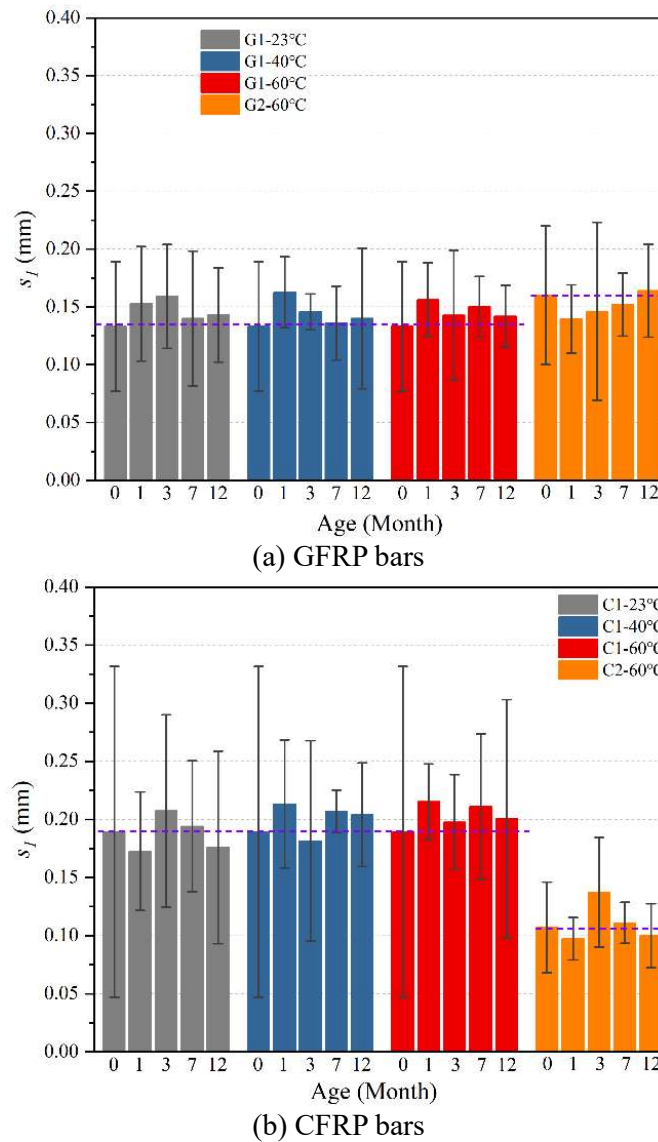
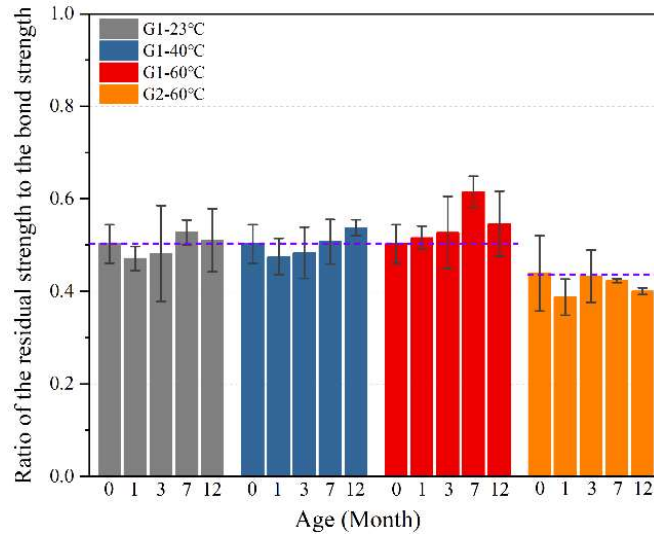
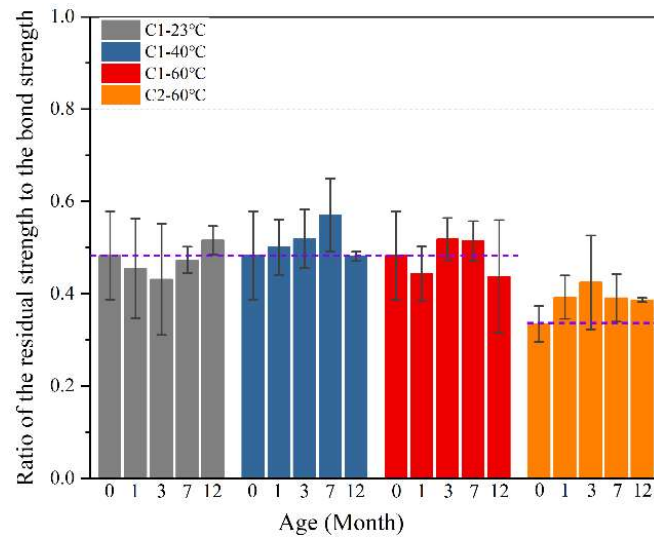


Figure 5.25. Corresponding slip at the bond strength of conditioned FRP bars



(a) GFRP bars



(b) CFRP bars

Figure 5.26. Ratio of residual strength to bond strength of conditioned FRP bars

Table 5.2 Summary of bond properties before and after conditioning

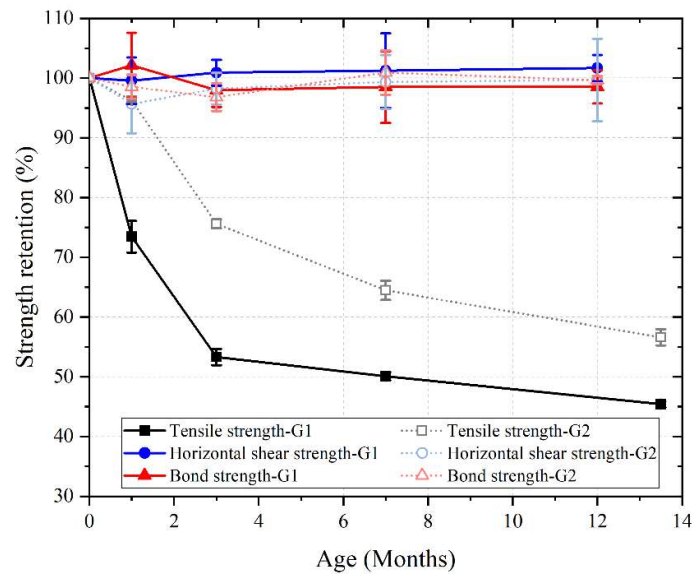
Specimen	T (°C)	Age (m)	Bond strength (MPa)		Bond stiffness (MPa/mm)		Slip at bond strength (mm)		Relative post-peak strength	
			Mean	SD	Mean	SD	Mean	SD	Mean	SD
G1-S140	23	Control	24.87	0.77	54.38	2.88	0.133	0.056	0.50	0.04
		1	24.32	1.12	--	--	0.153	0.049	0.47	0.03
		3	25.32	1.21	--	--	0.159	0.045	0.48	0.10
		7	25.38	0.73	--	--	0.140	0.058	0.53	0.03
		12	24.42	1.01	--	--	0.143	0.041	0.51	0.07
	40	1	24.83	1.48	--	--	0.163	0.031	0.48	0.04
		3	24.26	1.42	--	--	0.146	0.016	0.48	0.06

	60	7	25.72	0.70	--	--	0.136	0.032	0.51	0.05
		12	24.27	0.86	--	--	0.140	0.061	0.54	0.02
		1	25.40	1.35	52.98	13.58	0.156	0.032	0.52	0.02
		3	24.37	0.71	57.03	8.58	0.143	0.056	0.53	0.08
		7	24.50	1.49	51.98	6.71	0.150	0.026	0.62	0.03
		12	24.50	0.69	58.52	5.82	0.142	0.027	0.55	0.07
G1-S120	60	Control	21.89	0.33	71.43	13.51	--	--	--	--
		1	22.95	0.75	62.64	21.22	--	--	--	--
		3	23.99	1.52	63.03	4.69	--	--	--	--
		7	24.43	0.74	63.09	3.53	--	--	--	--
		12	23.80	1.39	49.57	8.00	--	--	--	--
G2-S140	60	Control	24.08	1.10	74.43	10.73	0.160	0.060	0.44	0.08
		1	23.72	0.47	60.18	2.25	0.140	0.029	0.39	0.04
		3	23.31	0.56	69.68	11.84	0.146	0.077	0.43	0.06
		7	24.30	0.89	74.14	5.35	0.152	0.027	0.42	0.00
		12	23.99	0.18	77.86	10.55	0.164	0.040	0.40	0.01
C1-S140	23	Control	29.11	1.66	68.13	9.74	0.189	0.142	0.48	0.10
		1	28.67	1.04	--	--	0.173	0.051	0.45	0.11
		3	28.65	1.03	--	--	0.207	0.083	0.43	0.12
		7	29.47	1.74	--	--	0.194	0.056	0.47	0.03
		12	29.23	0.93	--	--	0.176	0.083	0.52	0.03
	40	1	28.45	0.91	--	--	0.213	0.055	0.50	0.06
		3	28.29	1.26	--	--	0.182	0.086	0.52	0.06
		7	27.95	2.72	--	--	0.207	0.018	0.57	0.08
		12	28.77	1.53	--	--	0.204	0.045	0.48	0.01
	60	1	28.18	0.75	62.25	11.26	0.215	0.032	0.44	0.06
		3	27.34	1.53	73.07	2.47	0.198	0.041	0.52	0.05
		7	27.10	0.50	75.76	11.90	0.211	0.063	0.51	0.04
		12	27.35	2.01	67.47	6.58	0.201	0.102	0.44	0.12
C2-S140	60	Control	32.56	0.74	113.57	16.43	0.107	0.039	0.33	0.04
		1	30.05	2.01	110.35	34.11	0.097	0.018	0.39	0.05
		3	29.32	1.18	119.19	5.80	0.137	0.047	0.42	0.10
		7	30.25	1.77	103.13	18.16	0.111	0.018	0.39	0.05
		12	28.32	2.45	113.52	15.20	0.100	0.028	0.39	0.00

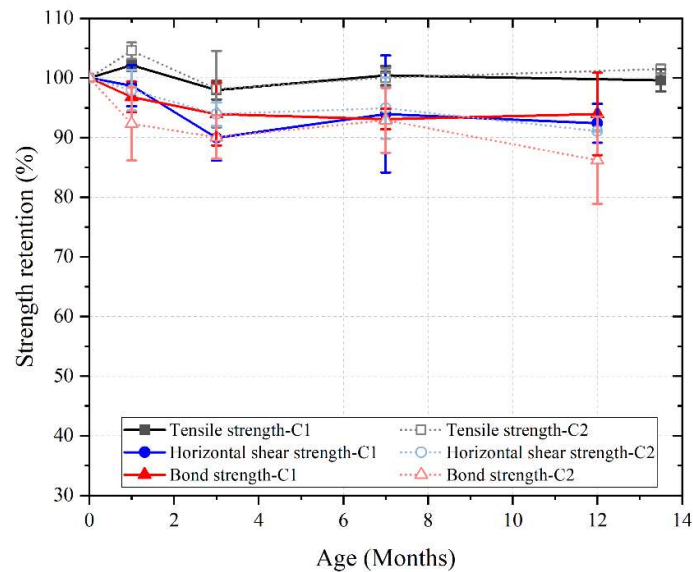
5.4. Comparison of mechanical properties of FRP bars in UHSSC

Figure 5.27 compares various mechanical properties under seawater immersion at 60 °C of different types of FRP bars in UHSSC. Generally, the durability performance in terms of various mechanical properties differs between the two types of FRP bars. For GFRP bars, the bond strength and horizontal shear strength showed the least deterioration and even a slight increase in the S120 specimen set, with strength reductions of less than 2.0% after 12 months of seawater immersion, while the tensile strength decreased rapidly to around 50% at 13.5

months. On the contrary, CFRP bars exhibited greater degradation of 8.1% to 13.8% in bond strength and horizontal shear strength after 12-month aging, with tensile strength remaining almost unchanged. Notably, it can be found that the change in horizontal shear strength of UHSSC-wrapped FRP bars showed a similar trend to that in bond strength during conditioning, irrespective of bar types. This further implies that the bond durability of FRP bars has a close correlation with their degradation in horizontal shear strength. In order to clarify this potential correlation, the quantitative evaluation will be conducted in Chapter 7.



(a) GFRP bar



(b) CFRP bar

Figure 5.27 Comparison of mechanical properties

5.5. Conclusions

The durability of four types of GFRP and CFRP bars in UHSSC exposed to the seawater environment was investigated using accelerated aging tests in this study and compared with the long-term performance of bare bars exposed to the same conditions. Based on the results, the following conclusions can be drawn:

1. The FRP bars embedded in UHSSC matrices showed comparable or superior performance to bare bars after seawater immersion. This indicates that the saline-alkalinity environments in UHSSC matrices have no adverse effect on the durability of FRP bars. Both types of concrete covers help decelerate the degradation rate of FRP bars under seawater exposure.
2. Despite having excellent tensile durability performance, CFRP bars exhibited greater degradation in horizontal shear strength compared to GFRP bars. The poor horizontal shear durability of CFRP bars is attributed to a large number of defects in bars caused by immature compatibility between the resin and carbon fibers with low surface energy. These defects facilitate the ingress of foreign matters, leading to local damage to the resin matrix and fiber-resin interface.
3. The bond durability between FRP bars and UHSSC under seawater immersion was not a critical issue. The failure mode of FRP-UHSSC specimens remained unchanged after conditioning, with the shearing-off of the surficial layer of FRP bars and internal fiber bundles at the bar core. No significant deterioration in long-term bond strength and bond stiffness was observed for FRP bars in UHSSC. The slip at bond strength and relative post-peak strength were minimally affected by the seawater-UHSSC environment. For FRP bars in UHSSC of lower compressive strength, bond strength improved by around 10% after one year of exposure at 60 °C, primarily due to increased concrete strength during conditioning. However, the bond stiffness decreased substantially by 30% because of a change in failure surface from debonding at the bar-concrete interface to shear failure within the FRP bars.

This chapter has provided valuable information on the durability behavior of FRP bars and the effect of UHSSC protection on embedded bars in the seawater environment under

accelerated conditions. However, it should be noted that the test results in this chapter are specific to the GFRP and CFRP bars tested. Due to the variations in the chemical resistance of FRP bars with different resin, fiber, and sizing combinations, other types of FRP bars should be investigated to recommend practical guidelines regarding their durability performance. Moreover, the FRP bars tested were embedded in UHSSC with a cover thickness of 70 mm, and a further study is required to examine the impact of UHSSC cover thickness on the degradation of FRP bars in aggressive environments.

CHAPTER 6. FIELD DURABILITY STUDY OF FRP BARS IN UHSSC

6.1. Introduction

Extensive literature is available on the durability of FRP bars embedded in concrete, primarily based on the accelerated aging test under aggressive conditions (Bakis *et al.* 2005; Chen *et al.* 2007; Davalos *et al.* 2011; Robert *et al.* 2009, 2013; Manalo *et al.* 2020; Rifai *et al.* 2020; Rolland *et al.* 2021; Yi *et al.* 2021, 2022; Feng *et al.* 2023). This type of test is favored for its ability to generate sufficient data in a short space of time. In addition, various researchers have developed methods to correlate accelerated aging test results with long-term field performance (Dejke and Tepfers 2001; Katsuki and Uomoto 1995; Huang and Aboutaha 2010; Dong *et al.* 2017). However, accurately predicting the performance of FRP-reinforced concrete structures requires in-field durability data for validation. Field durability study results can also provide users with direct evidence of the long-term performance of FRP bars in concrete structures.

In Chapter 5, research has been performed on UHSSC-wrapped FRP bars to evaluate the degradation of tensile, horizontal shear, and bond properties under accelerated conditions. However, as reviewed in Chapter 2, no studies have focused on the long-term performance of UHSSC-wrapped FRP bars in real coastal conditions, which is crucial for verifying predictive results from aging tests.

This chapter systematically investigates the long-term behavior of FRP bars embedded in UHSSC under coastal environments. The FRP bar-UHSSC specimens are conditioned in four different field environments at a wharf in Zhanjiang, China, over one year, including the atmospheric, splash, tidal, and underwater zones. Four types of GFRP and CFRP bars are tested to evaluate changes in macroscopic mechanical properties, namely bond properties, tensile properties, and horizontal shear strength. Comparative tests, including 148 specimens, were performed on both bare FRP bars and bars embedded in UHSSC. These properties are considered sufficient to provide essential insights into the durability performance of the FRP bars and their interaction with the surrounding concrete.

6.2. Experimental program

6.2.1. Environmental conditions

To evaluate the durability of FRP bars in UHSSC in coastal environments, the specimens were placed in four different field conditions at the wharf, as illustrated in Figure 6.1. In the atmospheric zone (At), the specimens were left in the open air on the stilt floor. The splash zone (Sp) was an underground area where specimens stayed above water but were subjected to splashing. In the tidal zone (Ti), specimens experienced periodic wetting and drying due to tidal streams, while in the underwater zone (Un), they were continuously immersed in seawater. The field study commenced in March 2023. Some specimens were removed from these zones for testing after one year of exposure, while others will remain conditioned at the wharf for at least five years, which is beyond the scope of this thesis. As shown in Figure 6.2, specimens were exposed to temperatures ranging from 5 °C to 35 °C, with a mean annual RH of 82%.

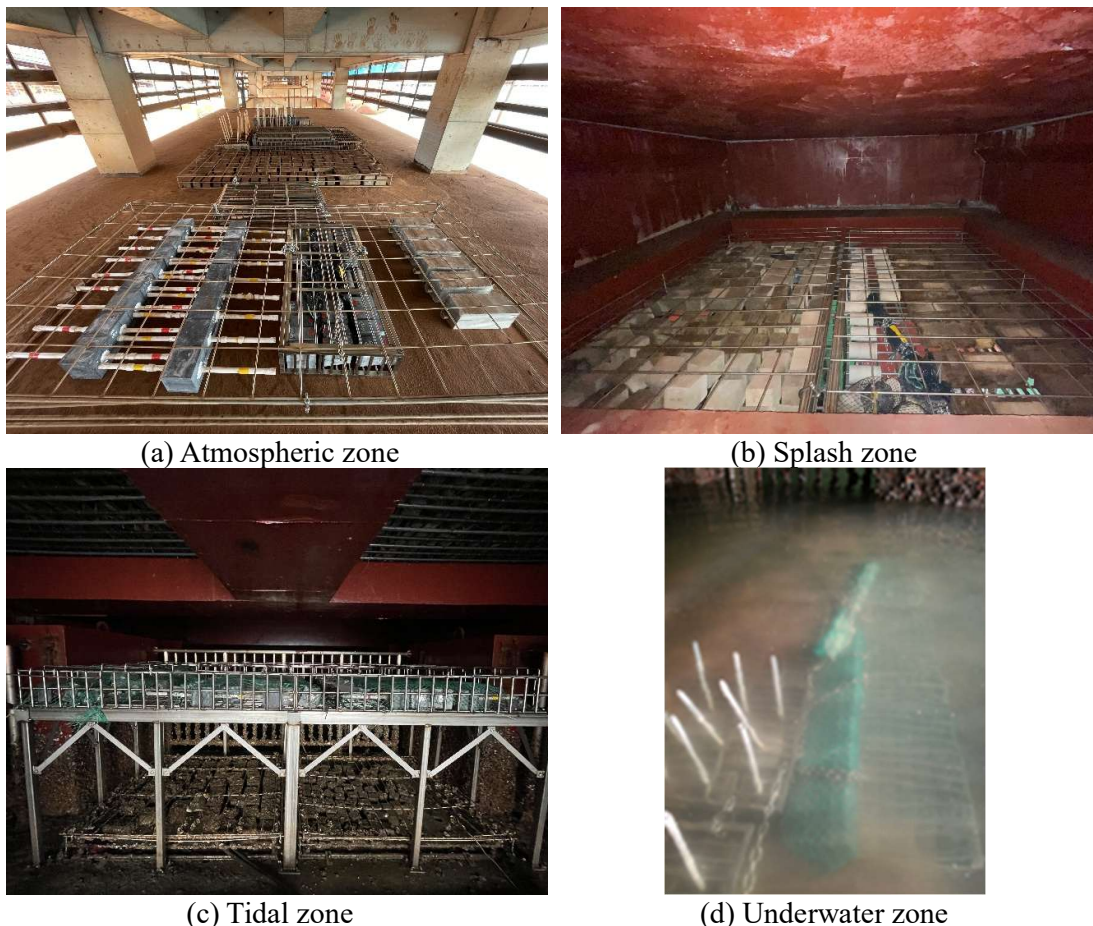


Figure 6.1. Four different field conditions

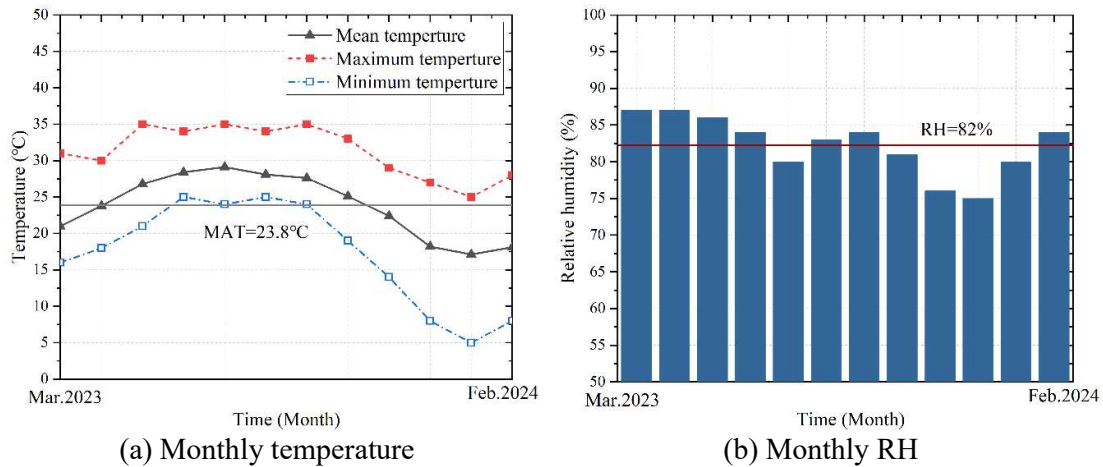


Figure 6.2. Weather data in Zhanjiang Harbor from March 2023 to February 2024 (Note: the data is provided by <https://www.qweather.com/>)

6.2.2. Test plan

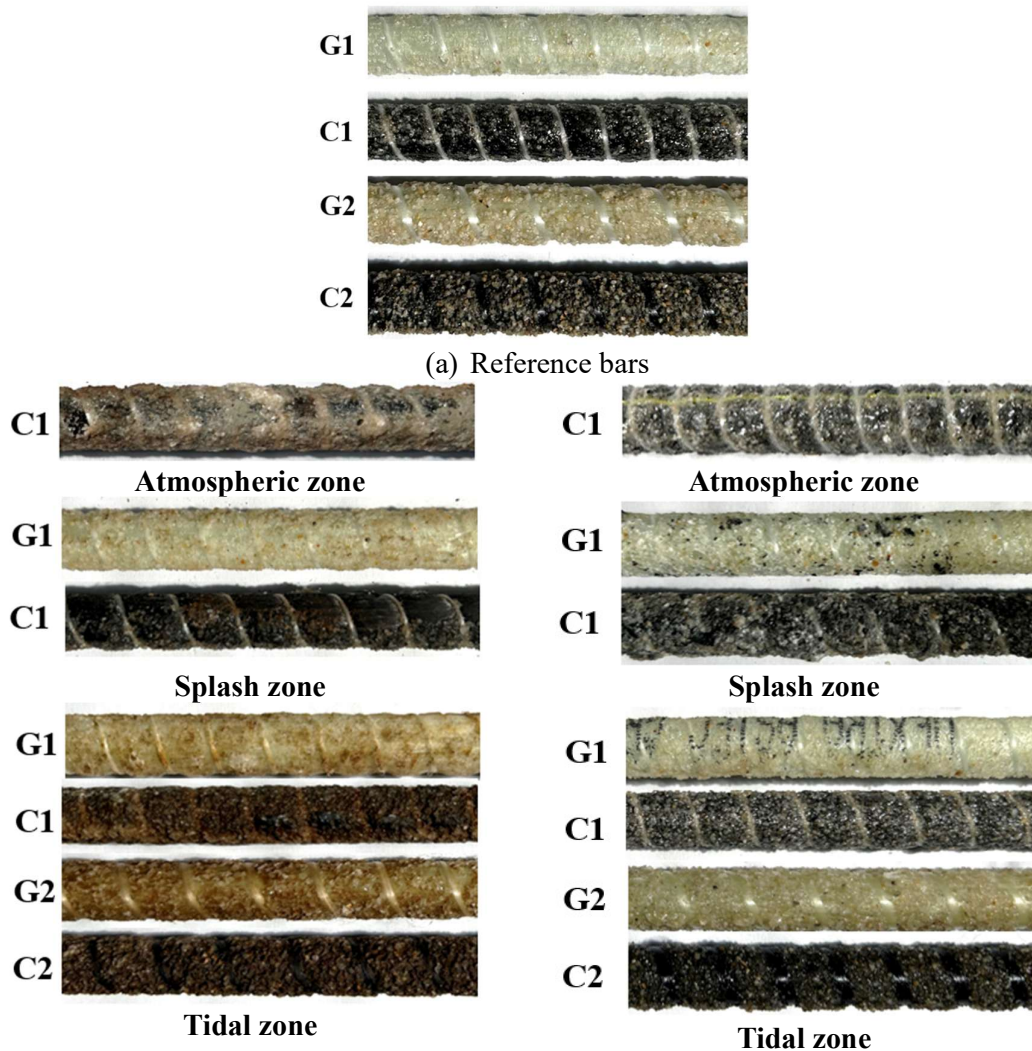
Four types of 10-mm-diameter FRP bars from two different producers were tested. The material properties of the FRP bars and UHSSC are detailed in Chapter 3. Three types of UHSSC-wrapped specimens, PO specimens, ILSS specimens, and TS specimens, were prepared as described in section 5.2. The protruded portions of the embedded bars, except for the concrete-wrapped portion, were protected with PVC tubes and sealant. For comparison, four types of bare bars were also exposed to coastal conditions and tested to evaluate their degradation. FRP bars in UHSSC-wrapped ILSS and TS specimens were carefully extracted from the concrete matrices before mechanical testing. The designations of four types of FRP bars are consistent with those in Chapter 3.

Tensile test and interlaminar shear test were performed per ATSM D7205M-06 (ASTM 2016) and ASTM D4475-02 (ASTM 2021), respectively. The pullout test followed ASTM D7913/D7913M-14 (ASTM 2020). The ILSS and TS specimens had lengths of 6d with a shear span of 5d, and 90d with a free length of 40d to 50d, respectively. For UHSSC substrates, the pH measurement and compressive test were performed after coastal exposures. Detailed information on these test methods is available in section 5.2.

6.3. Results and discussion

6.3.1. Surface appearance

Figure 6.3 and Figure 6.4 present the surface appearance of bare and concrete-wrapped bars after one year of field exposure. It is evident that the surface color of bare bars in the tidal and underwater zones has changed significantly compared to the reference bars, with more pronounced changes observed in the underwater zone, where bars were covered with a thick layer of marine organisms. This phenomenon was also observed in concrete specimens from the underwater zone, as shown in Figure 6.4. However, the surface appearance of bars extracted from concrete matrices remained unchanged in all environments. These observations suggest that continuous seawater immersion is more aggressive than other coastal conditions, and the FRP bars can be protected by the UHSSC matrix.



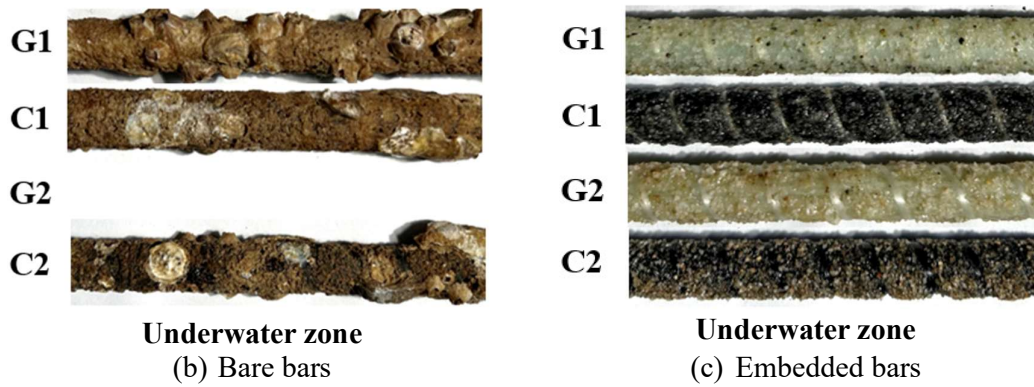


Figure 6.3. Appearance of FRP bars after field exposure

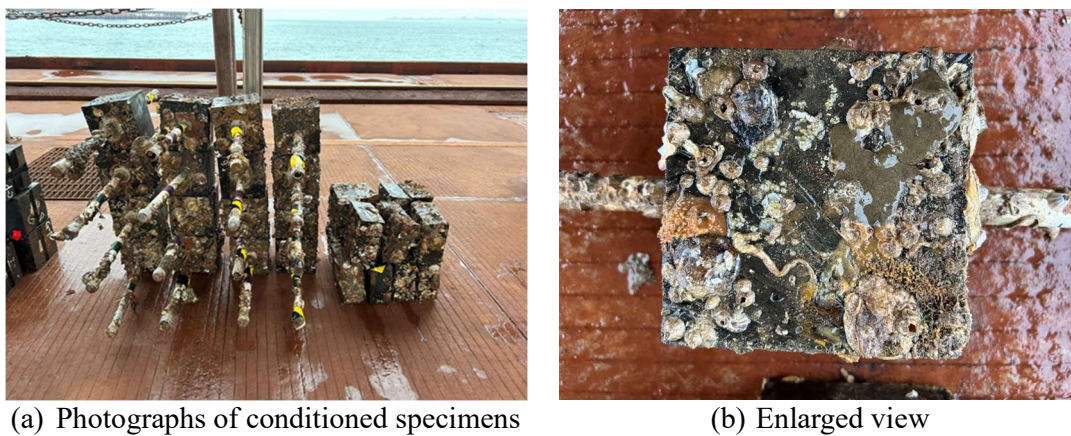


Figure 6.4. Appearance of concrete specimens in the underwater zone after field exposure

6.3.2. Properties of concrete

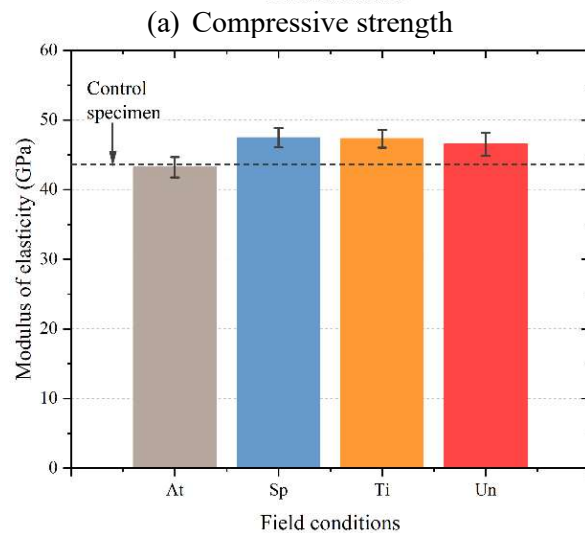
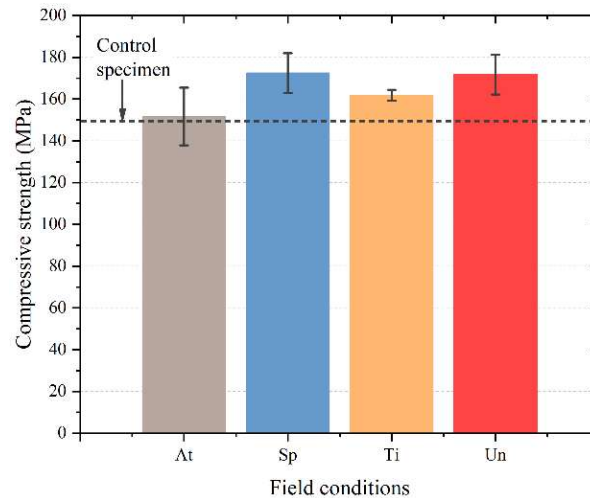
Figure 6.5 presents the changes in compressive strength and modulus of elasticity of concrete matrices after one year of exposure to four different field conditions. It can be found that the compressive strength and modulus of elasticity of specimens in the atmospheric zone remained nearly unchanged, whereas specimens exposed to the other three zones exhibited higher values. This increase can be attributed to further hydration during exposure to the moist and submerged environments. Specifically, the compressive strength of concrete matrices in the splash, tidal, and underwater zones increased by 8% to 16% after one year of exposure, while a 7% to 9% enhancement in elastic modulus was observed for the corresponding concrete specimens.

The pH measurement results are given in Table 6.1. After one year of coastal exposure, the average pH values at the depth of the embedded FRP bars ranged from 11.4 to 11.8 across

four different field environments, which were comparable to those of the control UHSSC substrate. The lower pH values in UHSSC after exposure confirm the less aggressive environment for FRP bars in the UHSSC substrate. On the other hand, it indicates almost no carbonation reaction and no decrease in the alkalinity within the UHSSC substrate at the level of FRP bars in the coastal environments. This is probably due to the low porosity and compact mesostructure of UHSSC substrate. Overall, the results of compressive tests and pH measurement reveal that the UHSSC mixture show good durability in the coastal environments.

Table 6.1 Alkalinity in UHSSC environment after coastal exposures

Field conditions	Atmospheric zone	Splash zone	Tidal zone	Underwater zone
Average pH	11.8	11.4	11.6	11.5



(a) Compressive strength
(b) Modulus of elasticity
Figure 6.5. Concrete mechanical properties after exposure

6.3.3. Failure modes

The failure modes of the three types of specimens did not change before and after conditioning. Figure 6.6 (a) and (b) show that both control and conditioned TS specimens failed by rupture, with a nested burst at the middle of the free length. In addition, ILSS specimens failed by interlaminar shear before and after conditioning, characterized by a major horizontal crack along the mid-plane, as presented in Figure 6.7 (c) and (d).

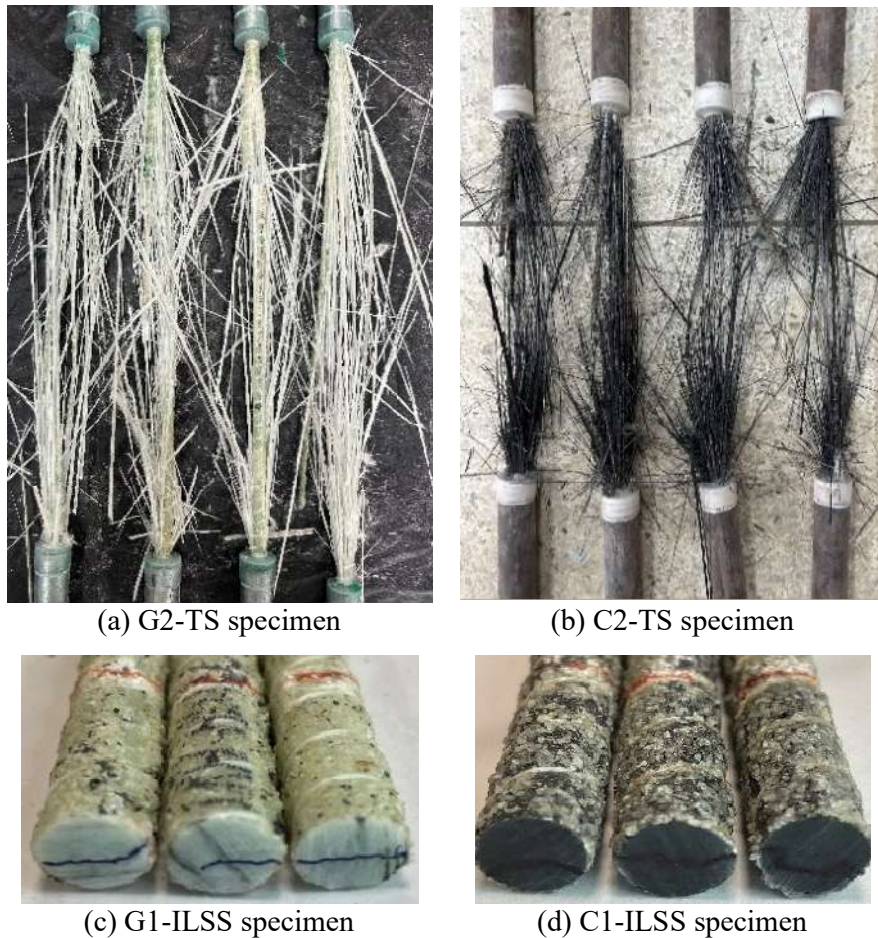


Figure 6.6. Typical failure modes of TS and ILSS specimens

The failure mode of all pullout specimens was characterized by the pullout failure of FRP bars without apparent damage to the concrete prisms. For a detailed visual examination of the debonding surface, specimens were split along the bond length following pullout tests. Figure 6.7 presents typical debonding surfaces of PO specimens. Similar to the control specimens in Chapter 3, debonding occurred between the fibers and resin matrix within the surface layer of FRP bars, demonstrating severe peeling off of the entire bar surface and

rupture of internal fibers. This indicates that the interlaminar shear capacity between the resin and fiber at the bar core controls the bond performance of FRP bars in UHSSC. Therefore, it can be anticipated that deterioration of FRP bars in UHSSC members may lead to bond degradation between FRP bars and the surrounding concrete.

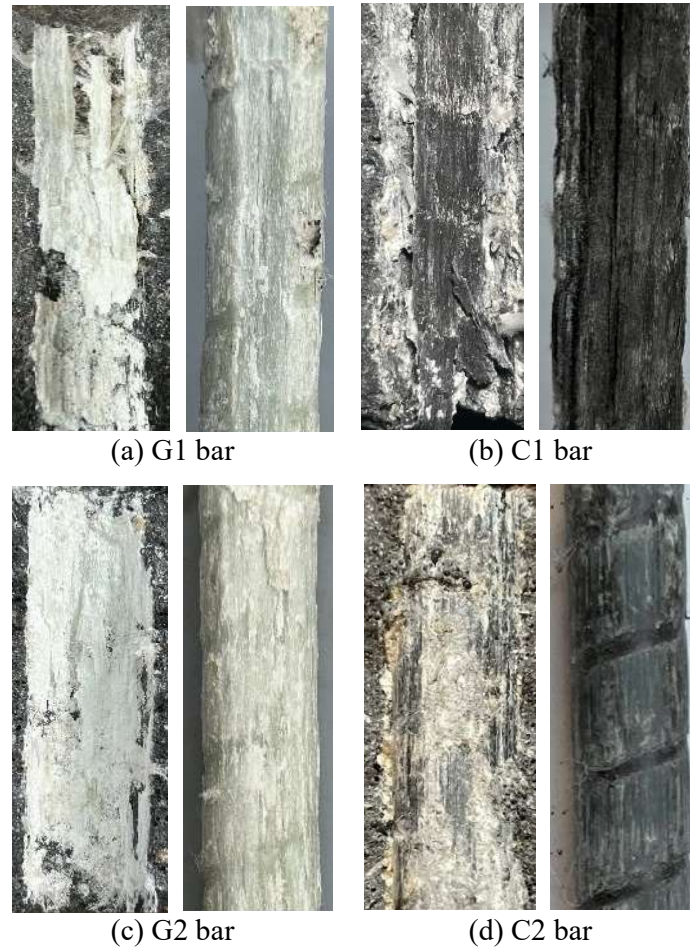


Figure 6.7. Typical failure of FRP bars in UHSSC

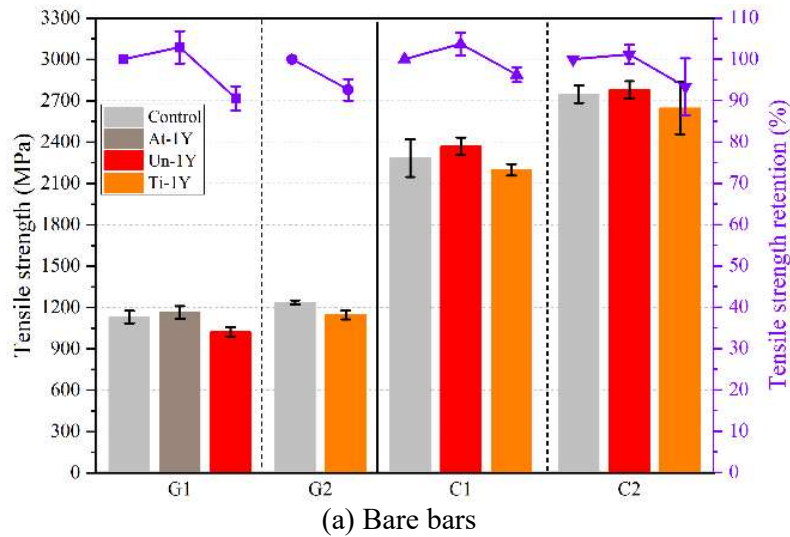
6.3.4. Degradation of mechanical properties of FRP bars

For a definite evaluation of performance, it is ideal to measure the mechanical properties of structural members directly. Figure 6.8-Figure 6.12 and Table 6.2 present the changes in several key mechanical parameters in various field conditions, including the tensile and interlaminar strength of FRP bars and the bond strength between FRP bars and UHSSC.

6.3.4.1. Tensile properties

As shown in Figure 6.8 (a) and Table 6.2, no significant deterioration in tensile strength was observed for bare FRP bars after one year of exposure. Among the field conditions, the tidal and underwater zones were critical in causing some reduction in tensile strength for bare bars. Specifically, a maximum tensile strength reduction of 10% was evident for bare G1 bars in the underwater zone. Under the same condition, the tensile strength of CFRP bars from two different producers remained almost constant, owing to the superior durability of carbon fibers. Interestingly, in the tidal zone, both bare GFRP and CFRP bars showed a comparable tensile strength reduction of around 6%. This is attributed to the improper securing of bare bars in the tidal zone, leading to damage from wave impact, as depicted in Figure 6.1(c).

Furthermore, compared to bare bars, almost no reduction in the tensile strength (less than 2%) was observed for UHSSC-wrapped FRP bars in four coastal conditions, as highlighted in Figure 6.8 (b) and Table 6.2. Although embedded FRP bars were surrounded by an alkaline-saline environment in UHSSC, erosion of FRP bars by external water molecules and detrimental foreign matter was limited to a dense-point type. Moreover, the UHSSC with compact microstructure acts as a drag on the penetration of alkali and salt ions into the FRP bars by inhibiting the ingress of water molecules from the external environment. It can be concluded that UHSSC effectively protects FRP bars in adverse environments.



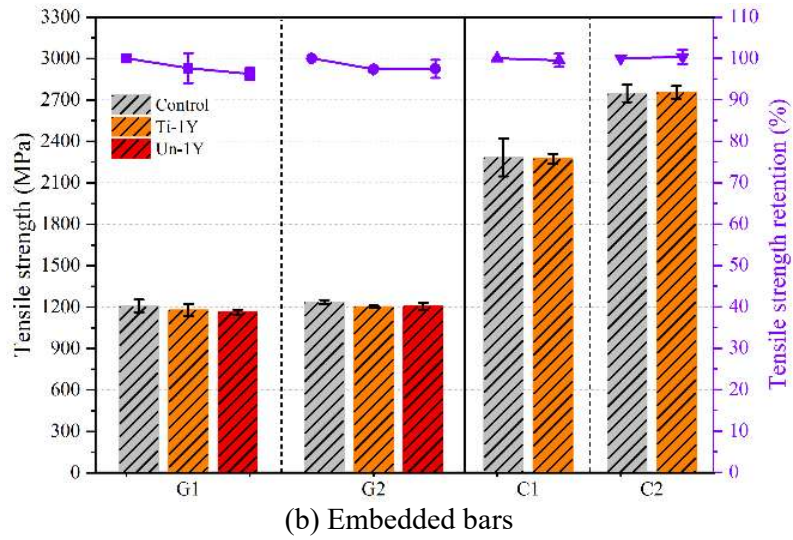


Figure 6.8. Tensile strength retention in various field conditions

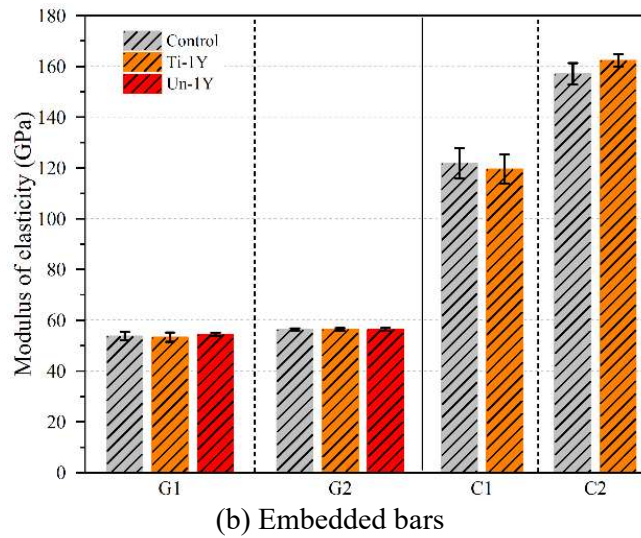
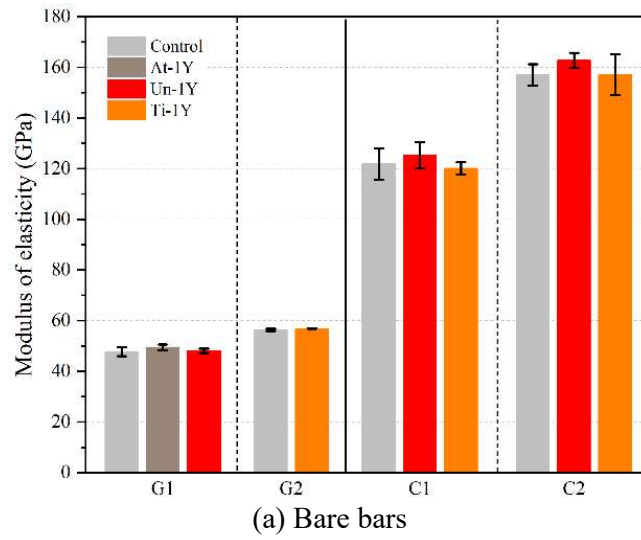


Figure 6.9. Change of modulus of elasticity in various field conditions

Figure 6.9 and Table 6.2 present the tensile modulus retention after one year of exposure to different field conditions. It can be noticed that the coastal conditions did not influence the modulus of elasticity of both bare bars or embedded bars.

Table 6.2 Summary of residual tensile properties and horizontal shear strength after exposure

Bar type	Concrete cover	Field condition	Tensile strength (MPa)		Modulus of elasticity (GPa)		Horizontal shear strength (MPa)	
			Mean	SD	Mean	SD	Mean	SD
G1	BB	Control	1131*	45*	47.7*	1.7*	--	--
		At	1164*	45*	49.4*	1.2*	--	--
		Sp	--	--	--	--	29.53	1.19
		Ti	--	--	--	--	29.80	0.65
		Un	1024*	33*	48.1*	0.9*	--	--
G2	BB	Ti	1145	32	56.8	0.0	33.59	4.19
C1	BB	At	--	--	--	--	25.46	1.47
		Sp	--	--	--	--	24.37	1.00
		Ti	2199	42	120.2	2.4	23.76	2.97
		Un	2369	64	125.3	5.1	21.82	0.51
C2	BB	Ti	2646	188	157.1	8.0	45.43	1.66
		Un	2779	63	162.7	3.0	40.66	1.28
G1	EB	At	--	--	--	--	28.86	1.22
		Sp	--	--	--	--	28.84	0.59
		Ti	1180	43	53.4	1.8	28.63	0.10
		Un	1163	17	54.5	0.6	29.15	1.41
G2	EB	Ti	1204	7	56.5	0.4	33.53	3.42
		Un	1205	27	56.5	0.5	32.53	3.18
C1	EB	At	--	--	--	--	23.37	0.76
		Sp	--	--	--	--	23.64	0.44
		Ti	2274	36	119.5	5.8	23.56	0.32
		Un	--	--	--	--	25.26	0.24
C2	EB	Ti	2755	47	162.3	2.5	44.16	2.16
		Un	--	--	--	--	42.24	0.13

Note: XX* is the residual tensile properties measured from another production lot of G1 bars.

6.3.4.2. Horizontal shear properties

The shot-beam test results of bare and embedded FRP bars exposed to different coastal conditions are compared in Figure 6.10 and Table 6.2. It can be noticed that one year of field exposure did not cause severe deterioration in the horizontal shear strength of different types of bare bars. The maximum horizontal shear strength reduction was observed for bare CFRP bars in the underwater zone, with reductions of 12% for bare C1 bars and 8% for bare C2 bars. This observation confirms that the underwater zone is the most aggressive environment

among the four coastal conditions.

In contrast, the embedded bars, regardless of bar type, exhibited only marginal degradation in all coastal conditions, with a maximum decrease in horizontal shear strength of around 4% after one-year field exposure. This further confirms that the UHSSC environment does not negatively impact the durability of FRP bars and can retard their deterioration by hindering the ingress of foreign matter due to its compact microstructure. Additionally, for embedded specimens, the alkalinity of the UHSSC environment may become less aggressive over exposure time, as the chloride ingress and carbonation can decrease the pH value of concrete (Grubb *et al.* 2007; Behnood *et al.* 2016).

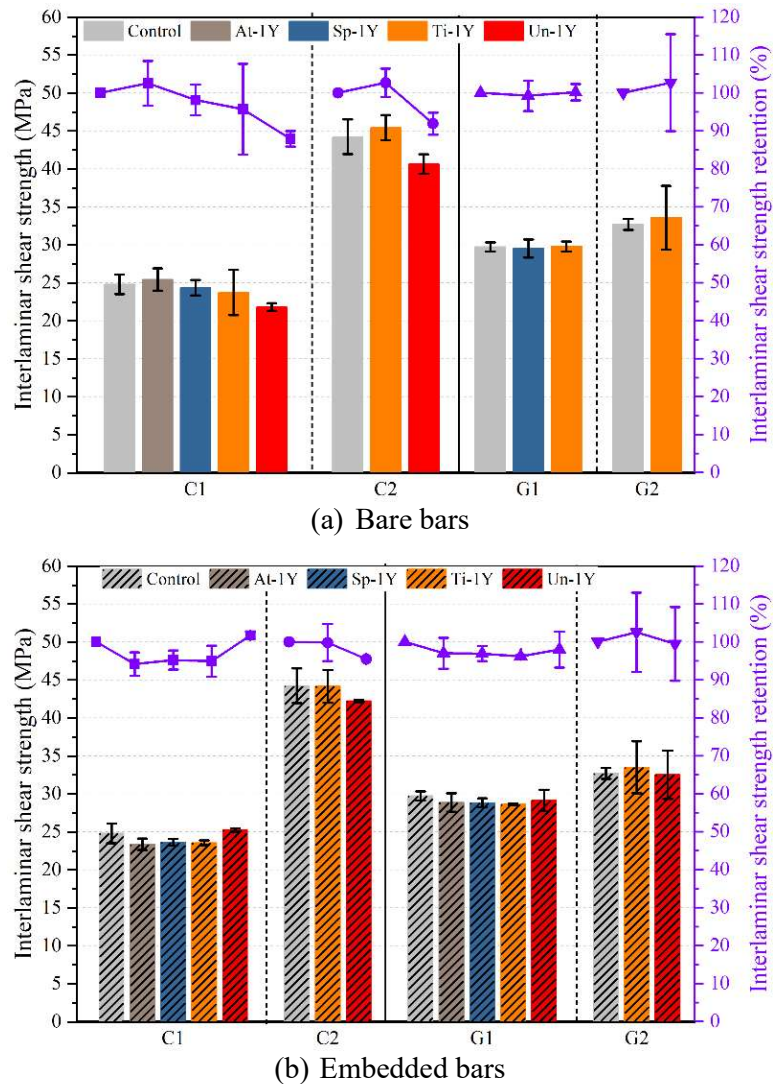


Figure 6.10. Interlaminar shear strength retention in various field conditions

6.4.3.3. Bond properties

Figure 6.11 and Table 6.3 compare the changes in bond strength of four different FRP bars exposed to various field conditions. FRP bars in UHSSC showed minimal deterioration in bond strength after one year of exposure. Even in the underwater zone, identified as the most aggressive condition based on the analysis in sections 6.3.4.1 and 6.3.4.2, the maximum reduction was only 7% for C2 bars. As discussed in Chapter 5, the long-term bond performance of FRP bars is correlated with changes in the shear properties of concrete and FRP bars, as well as potential increases in mechanical interlocking due to FRP bar swelling in moist conditions. However, mechanical interlocking contributes little to bond resistance for the four types of FRP bars used in this study due to their light surface deformations, allowing the potential influence of bar swelling during exposure to be neglected.

As shown in Figure 6.5, although the UHSSC matrix exhibited increased concrete strength after one year of field exposure, further increases in concrete strength have a negligible impact on bond strength, as discussed in the previous chapter. According to the discussion in section 5.4, the superior bond durability of FRP bars in UHSSC can be expected due to the insignificant horizontal shear strength degradation observed for UHSSC-wrapped FRP bars after the same exposure, as presented in the previous section.

Table 6.3 Summary of residual bond properties after exposure

Specimen	Field condition	Bond strength (MPa)		Bond stiffness (MPa/mm)		Slip at bond strength (mm)		Relative post-peak strength	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD
G1	At	24.16	0.75	60.58	3.23	0.154	0.046	0.45	0.07
	Sp	24.46	0.52	57.20	3.77	0.141	0.029	0.52	0.05
	Ti	24.71	0.49	56.32	3.99	0.151	0.029	0.49	0.04
	Un	23.99	1.19	56.07	4.15	0.128	0.025	0.48	0.04
G2	Ti	23.75	1.56	76.33	10.30	0.145	0.026	0.43	0.05
	Un	23.82	0.93	72.92	13.36	0.139	0.016	0.37	0.01
C1	Sp	28.41	1.23	70.16	3.66	0.200	0.070	0.42	0.04
	Ti	28.78	1.02	70.22	4.34	0.173	0.039	0.45	0.08
	Un	28.58	0.41	67.51	6.93	0.196	0.049	0.48	0.05
C2	Ti	32.03	1.17	115.63	3.27	0.102	0.012	0.37	0.09
	Un	30.37	1.52	118.79	18.60	0.107	0.024	0.40	0.07

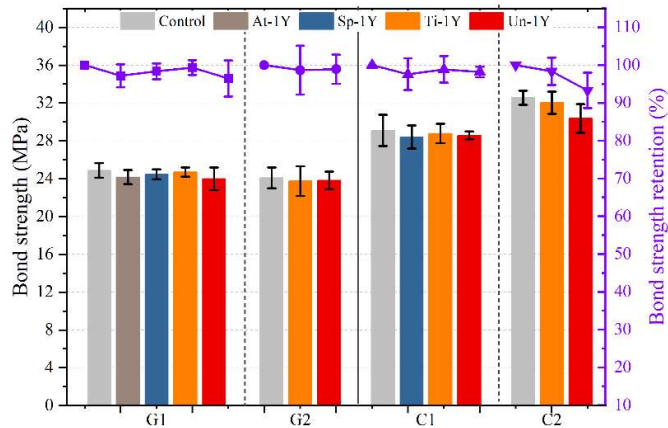
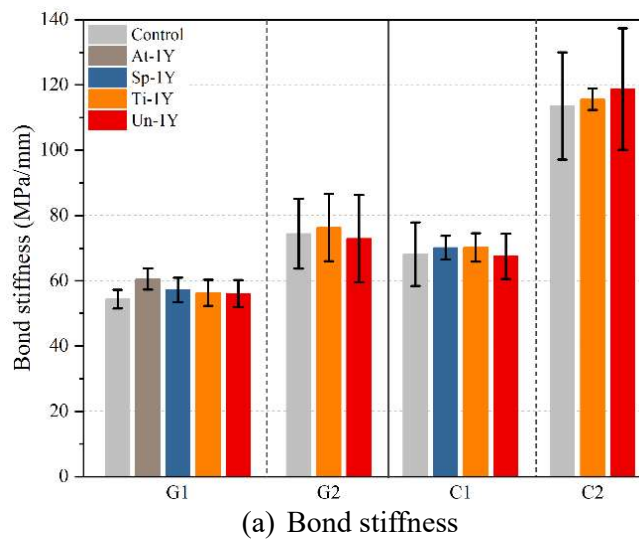


Figure 6.11. Bond strength retention in various field conditions

Figure 6.12 and Table 6.3 show the effect of coastal exposures on other bond characteristics of four types of FRP bars. In Figure 6.12 (a), no significant change can be found in the bond stiffness of all FRP bars after one year of coastal exposure. In addition, the slip corresponding to bond strength and the relative post-peak strength exhibited some variations after field exposure, primarily due to the test scatter, but no clear trend was evident in Figure 6.12 (b) and (c). Based on these observations, the one-year coastal exposures had minimal impact on the long-term bond performance of FRP bar-UHSSC members. Compared to control specimens, exposed FRP bars in UHSSC showed similar bond strength, bond stiffness, characteristic slip at bond strength, and relative post-peak strength.



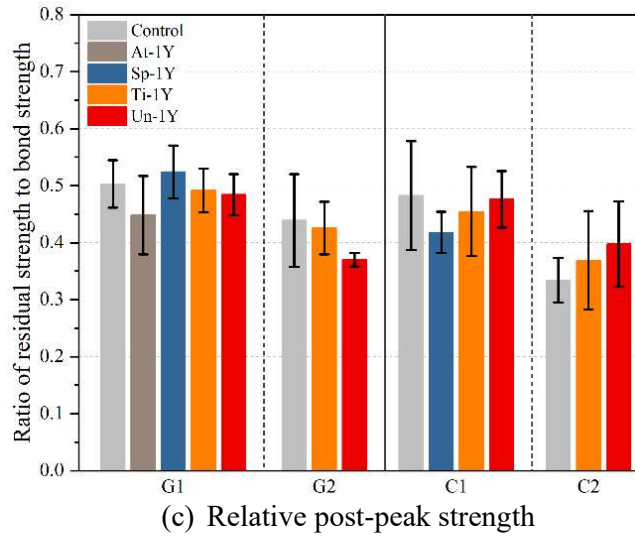
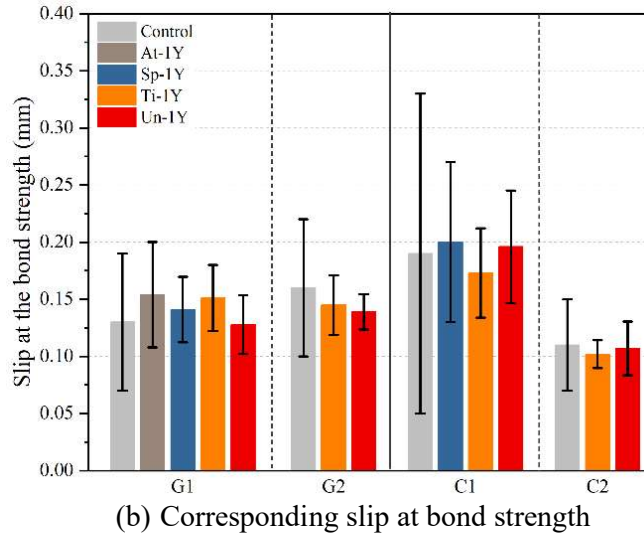


Figure 6.12. Changes in bond characteristics of FRP bars after field exposure

6.4. Conclusions

This research was part of an ongoing durability research program investigating the durability performance of different FRP bars embedded in UHSSC exposed to coastal environments. Based on the systematic results in this study, the following conclusions can be drawn:

1. After one year of exposure in the coastal environment, the failure modes of tensile, interlaminar shear, and pullout tests did not change. In general, the tidal and underwater zones are two critical conditions that can cause visible degradation in the mechanical properties of bare FRP bars. Test results reveal that the alkaline-saline environment in

UHSSC has no adverse impacts on the durability of FRP bars. Instead, due to its compact microstructure, the UHSSC cover protects the internal bars from severe deterioration by hindering the penetration of external water molecules and detrimental ions.

2. After one year of exposure to the coastal environment, the most prominent degradation occurred in bare bars conditioned in the underwater zone. The maximum reduction in tensile strength was 10% for submerged bare GFRP bars, and the reduction in horizontal shear strength was 8%-12% for submerged bare CFRP bars. After the same exposure, no significant reduction in tensile and horizontal shear strength was observed in UHSSC-wrapped FRP bars. Moreover, the coastal environment did not influence the tensile elastic modulus of either bare bars or UHSSC-wrapped bars.
3. The one-year coastal exposures had little impact on the bond durability of FRP bar-UHSSC members. The bond strength, bond stiffness, and relative post-peak strength of exposed FRP bar-UHSSC members were comparable to their control specimens.

Based on the current experimental observations, the one-year coastal conditions had no negative effect on the physical and mechanical properties of the UHSSC substrate and FRP bars, and caution should be exercised when extrapolating these findings to longer exposure years. A field study lasting for a longer period should be conducted to generate sufficient data before coming to solid conclusions.

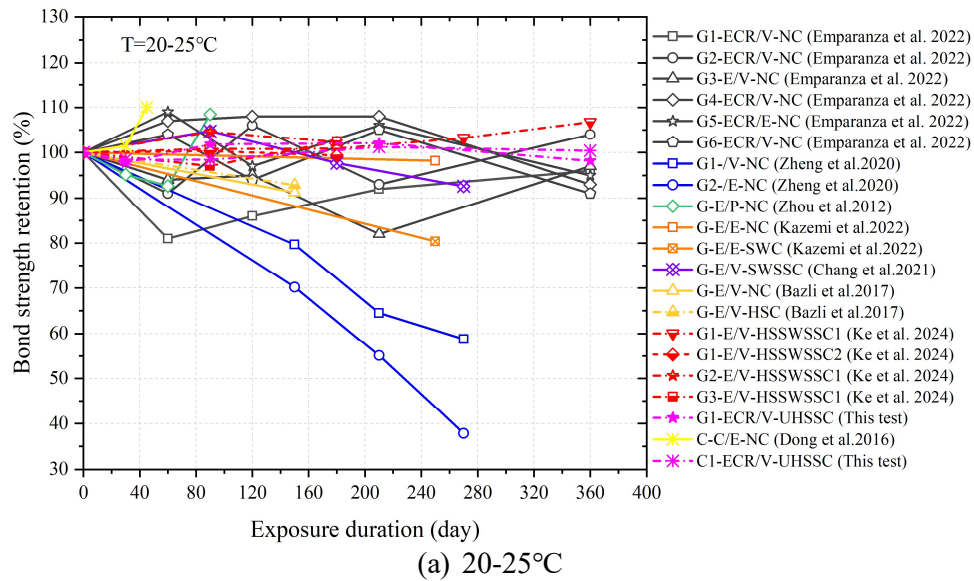
CHAPTER 7. LONG-TERM PREDICTION OF FRP BARS IN CONCRETE

7.1. Introduction

Chapter 5 and Chapter 6 have systematically investigated the durability of four types of FRP bars in simulated seawater and natural coastal environments. To have a comprehensive understanding of the long-term performance of FRP bars in concrete, an interpretation of extensive durability data is required, since the durability performance of FRP bars is significantly affected by the properties of their components (i.e., fiber and resin) and the manufacturing parameters (e.g., fiber sizings and catalyst) (Benmokrane *et al.* 2020).

Figure 7.1 summarizes the bond durability results from previous chapter and other 14 experimental studies (Zhou *et al.* 2012; Altalmas *et al.* 2015; El Refai *et al.* 2015; Dong *et al.* 2016; Bazli *et al.* 2017; Yan *et al.* 2017; Emparanza *et al.* 2018; Khatibmasjedi *et al.* 2020; Zheng *et al.* 2020; Chang *et al.* 2021; Kazemi *et al.* 2022; Emparanza *et al.* 2022; Shakiba *et al.* 2023; Ke *et al.* 2024). Obviously, the bond performance in seawater environments varied with bar type, which exhibited different levels of bond strength reduction after conditioning. It can be found that the seawater environments generally did not have a significantly negative impact on the bond durability of FRP bars in various types of concrete matrix. Most FRP bars retained at least 80% of their bond strength even after exposure at high temperatures (55 °C to 70 °C), and in some cases, the bond strength increased slightly. However, significant bond degradation with bond strength reduction of over 60% was also observed in some specimens. This may be attributed to the difference in the material degradation of different FRP bars, which needs further investigation. Bank *et al.* (1998) have proposed a descriptive model of the mechanism of the deterioration of the FRP material to relate the changes seen in the bond strengths to the observed material degradation in FRP bars. However, limited research has quantitatively clarified the effects of material degradation of FRP bars on their bond durability (Hao *et al.* 2023; Lu *et al.* 2021), which hinders the accurate prediction of bond strength degradation of FRP bars. This chapter, therefore, aims to quantitatively identify the key parameter in the degradation of the bond strength and thus to develop a predictive bond strength degradation model.

In terms of the long-term tensile strength, two approaches are currently adopted to predict the field durability of FRP bars embedded in concrete structures using the Arrhenius relation. The first prediction approach directly extrapolates the accelerated short-term performance of bare bars in an alkaline simulated pore solution (SPS) to predict the long-term performance of bars in a concrete environment. The second approach considers the effect of concrete wrap by applying a methodology similar to the first. Due to the convenience of directly immersing bare bars in the alkaline solution, the first prediction approach is often preferred by researchers (Wu *et al.* 2015; Wang *et al.* 2017; Wang *et al.* 2023; Hosseini *et al.* 2024). However, as reviewed in section 2.4.2, the degradation rate of bare bars in the alkaline solution is faster than that of embedded bars in moisture-saturated concrete. Consequently, several research teams have suggested that assessing the field durability of bars in concrete environments based on bare bars in accelerated conditions is overly conservative (Mufti *et al.* 2007; Gooranorimi and Nanni 2017; Benmokrane *et al.* 2018). Despite this, limited research has quantified the differences in field durability results of the two prediction methods (Zhu *et al.* 2014).



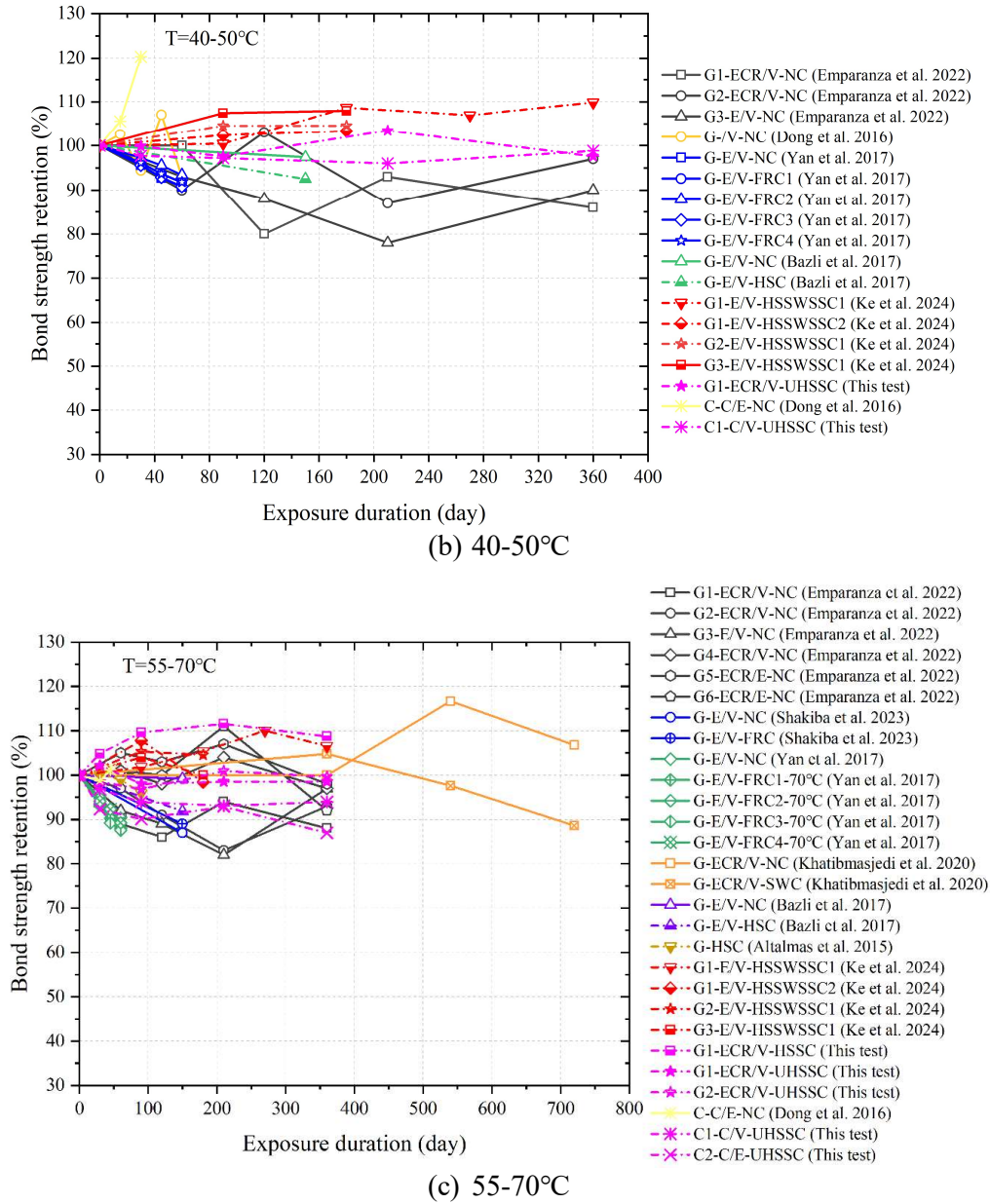


Figure 7.1. Bond durability of different types of FRP bars in seawater environments

This chapter concerns the long-term prediction of the residual bond strength and tensile strength of FRP bars in concrete. Accordingly, this chapter first focuses on the development of a new bond degradation model based on the changes in the material properties of both the FRP bars and the surrounding concrete. Then, the effect of concrete wrap on long-term predictions of tensile strength of internal FRP reinforcement is investigated using an improved prediction method based on short-term data from accelerated aging tests. Against the prediction results, the in-service performance of FRP reinforcement for a 100-year design life is also assessed.

7.2. Prediction of bond durability of FRP bars

7.2.1. Correlation between bond durability and materials degradation of FRP bars

The review in section 2.4.2.2 and the test results in the previous chapters indicate that the changes in bond strength may be related to the material degradation of FRP bars. Therefore, to develop a reliable predictive model for bond durability of FRP bars, it is necessary to investigate the correlation between the bond durability and material degradation of FRP bars in the first instance. The following analysis will be conducted on this issue, revealing the mechanism of bond degradation.

7.2.1.1. Changes in bond strength and tensile strength of FRP bars

Through comparison between the tensile strength retention of FRP bars and the bond strength retention of FRP bars in concrete, Hao *et al.* (2023) and Nepomuceno *et al.* (2021) reported that the degradations in both tensile strength and bond strength of FRP bars showed a similar trend during conditioning. Consequently, they claimed that the deterioration in the bond appeared to be correlated with the changes in the tensile strength of FRP bars. This finding, however, was inconsistent with the experimental observations in this study. As reported in section 5.4, the GFRP bars exhibited a substantial reduction in tensile strength (up to 55%), but almost no bond degradation was found under the same conditioning. In contrast, the CFRP bars that had superior tensile durability presented significant degradation in bond of up to 14%. Against these discussions, it can be concluded that the decrease in bond strength of FRP bars is not governed by their degradation in tensile strength.

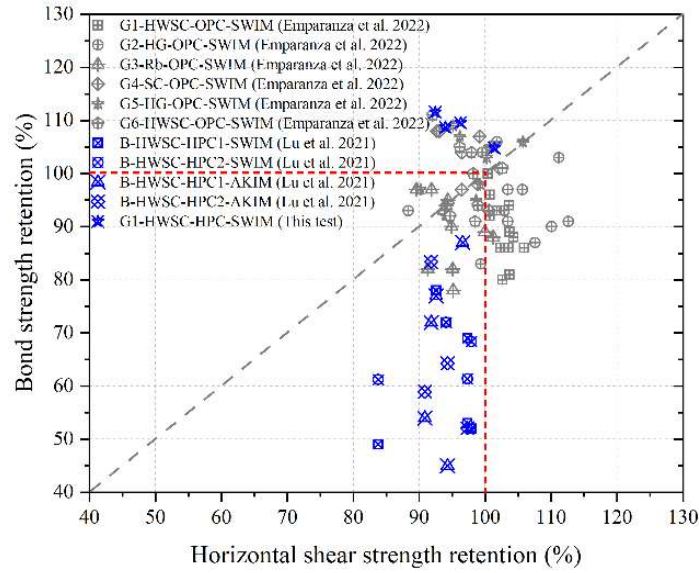
7.2.1.2. Changes in bond strength and horizontal shear strength of bare bars

Based on existing studies, Figure 7.2 compares the bond strength retention with the horizontal shear strength retention of bare bars when ILSS and PO specimens were subjected to identical environmental conditioning. It reveals that the changes in the horizontal shear strength of bare bars had no relevance to the bond strength degradation of the same FRP bars. In NSC, HSC and the S120 specimen set in Chapter 5, most FRP bars exhibited a decrease in bond strength, with only slight deterioration or even some increases in the horizontal shear strength of bare bars. Conversely, in UHSC with higher compressive strength, bond strength degraded more slowly than the horizontal shear strength of the same bare FRP bars. These observations

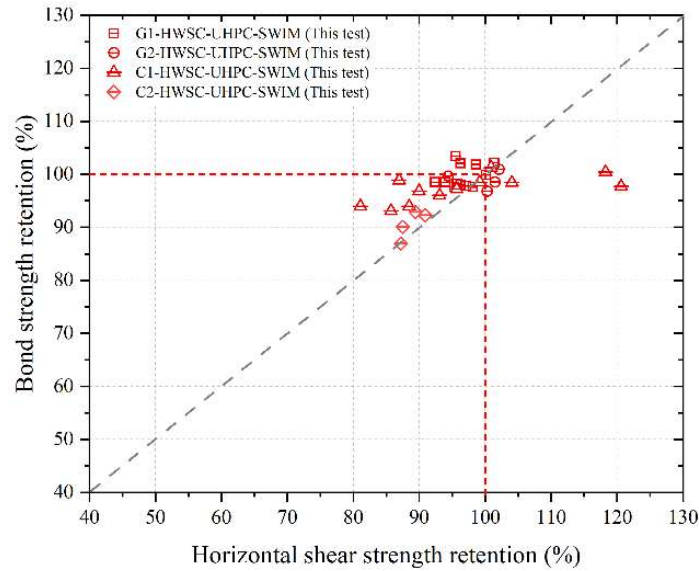
are not surprising due to the following reasons: bare FRP bars in ILSS specimens have more exposure to aggressive solutions than the embedded bars, leading to higher degradation; bare bars in some cases may show less pronounced degradation in environments with lower alkalinity compared to bars embedded in concrete; and for bars that are not fully cured, post-curing of the resin matrix in bare bars during hydrothermal conditioning can counteract to some extent the negative effects of physical aging, resulting in slight decreases or enhancements in horizontal shear strength. Consequently, the methodology adopted by Lu *et al.* (2021) for predicting bond strength degradation based on the material deterioration of corresponding bare bars is not universally applicable to all FRP bars.

7.2.1.3. Changes in bond strength and horizontal shear strength of embedded bars

Figure 7.3 illustrates the comparison of changes in bond strength with those in horizontal shear strength of bars embedded in the same concrete matrices as PO specimens after identical exposure. Note that ILSS specimens and corresponding PO specimens had identical concrete cover thickness in each experimental program, except for Li *et al.*'s (2023) study, where ILSS specimens were embedded in the concrete with a much smaller cover thickness than PO specimens. For FRP bars in UHSC with higher compressive strength, the same degradation levels were observed for both bond strength and horizontal shear strength of embedded ILSS specimens. This suggests that in UHSC, the bond strength retention is equivalent to the horizontal shear strength retention of embedded ILSS specimens, as the bond in UHSC generally has a dependency on the shear properties of FRP bars rather than the concrete matrix, as discussed in section 5.3.4. However, in NSC, HSC, and the S120 specimen set in Chapter 5, bond strength generally exhibited higher retention than the horizontal shear strength of embedded ILSS specimens. This is attributed to the influence of changes in concrete strength in addition to the material deterioration of FRP bars during conditioning. As summarized in previous studies (Yan *et al.* 2016; Nepomuceno *et al.* 2021; Wang *et al.* 2024), concrete strength significantly contributes to the bond strength of FRP bars in NSC and HSC. This indicates that long-term bond strength assessment should consider changes in concrete strength alongside material degradation of FRP bars.



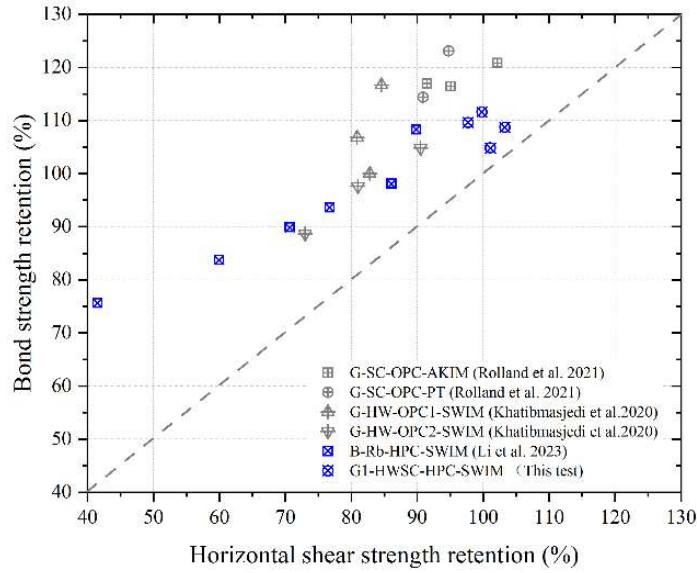
(a) NSC and HSC (including S120 specimen sets)



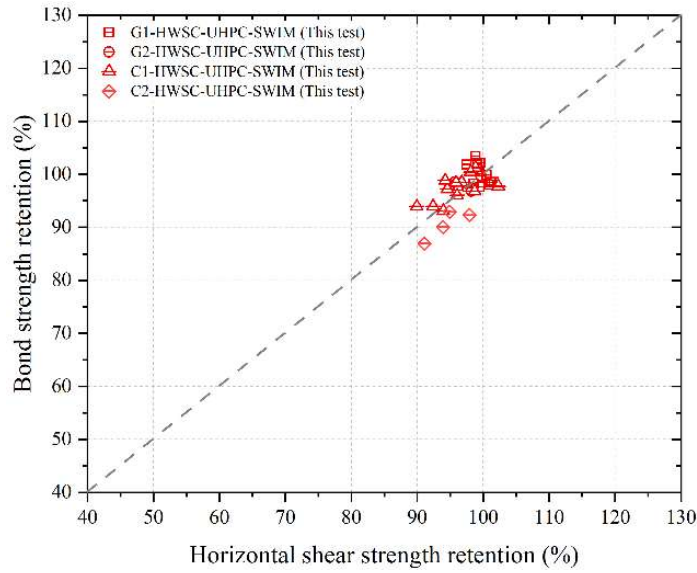
(b) UHSC (S140 specimen sets)

Figure 7.2. Comparison of degradation of bare FRP bars and change in bond strength of bars in different concrete matrices

Note: G, B, and C denote the GFRP, BFRP, and CFRP bars, respectively; HWSC is the helically wrapped and sand-coated bar; HG is the helically grooved bar; Rb is the ribbed bar; SC is the sand-coated bar; SWIM means immersion in the seawater; AKIM means immersion in the alkaline solution; the number following the notations of bar and concrete represent the types of FRP bar and concrete matrix used in the specimens.



(a) NSC and HSC (including S120 specimen sets)



(b) UHSC (S140 specimen sets)

Figure 7.3. Comparison of degradation of embedded FRP bars and change in bond strength of bars in different concrete matrices

Note: PT means pure thermal aging; HW is the helically wrapped bar; the other notations are the same as those in Figure 7.2.

Furthermore, previous research suggested that the bond strength may initially increase during conditioning due to enhanced mechanical interlocking between bars and concrete caused by FRP bar swelling (El Refai *et al.* 2015; Bazli *et al.* 2017; Yan *et al.* 2017; Chang *et al.* 2021; Kazemi *et al.* 2022). However, the impact of bar swelling and the resulting increased mechanical interlocking on bond durability cannot be quantitatively assessed. Additionally,

this effect is likely less significant for bars without prominent surface deformation, where mechanical interlocking contributes minimally to bond resistance (*fib* 2007; Soong *et al.* 2011). As a result, the potential positive effect of FRP bar swelling is neglected in the subsequent analysis.

7.2.2. Development of a new bond strength degradation model

Based on the bond strength model proposed in Chapter 4, the bond strength retention after conditioning can be given as

$$Y_\tau = \frac{\tau_{ui}}{\tau_{u0}} = \frac{\tau_{mi}(S_i) \cdot g_i(f_{ci}, d)}{\tau_{m0}(S_0) \cdot g_0(f_{c0}, d)} \quad (7.1a)$$

$$g(f_c, d) = \left(1 + \frac{\beta_d d}{e^{\frac{f_c}{\theta}} - 1} \right)^{-0.5} \quad (7.1b)$$

where Y_τ is the bond strength retention; τ_{u0} and τ_{ui} are bond strengths of control and aged FRP bars, respectively; τ_m is the upper limit of bond strength of a specific bar type; S_0 and S_i are horizontal shear strength of control and aged FRP bars, respectively; d is the nominal bar diameter; f_{c0} and f_{ci} are compressive strengths of Φ 100 x 200 concrete cylinders before and after conditioning; β_d and θ are two regression coefficients as given in Table 7.1.

For the control and conditioned S140 UHSSC mixtures of the accelerated aging test in Chapter 5, the ratio of $g_i(f_{ci}, d)/g_0(f_{c0}, d)$ approximates 1.0, as shown in Figure 7.4. The expression in Eq. (7.1) can then be simplified as Eq. (7.2). This is consistent with the finding in Chapter 3 that the bond strength in UHSC has minimal dependence on the increase in concrete strength.

$$Y_\tau = \frac{\tau_{ui}}{\tau_{u0}} = \frac{\tau_{mi}(S_i) \cdot g_i(f_{ci}, d)}{\tau_{m0}(S_0) \cdot g_0(f_{c0}, d)} \rightarrow \frac{\tau_{mi}(S_i)}{\tau_{m0}(S_0)} \quad (7.2)$$

Based on the experimental observation in Figure 7.3 that the changes in bond strength of the S140 specimen sets are identical with the horizontal shear strength deterioration of embedded bars, Eq. (7.3) can be obtained. By substituting Eq. (7.3) into Eq. (7.1), a bond strength

degradation model can be expressed as Eq. (7.4). In the proposed model, the effects of changes in properties of concrete substrate and material deterioration of FRP bars on the bond durability of FRP bars were considered.

$$\frac{\tau_{mi}(S_i)}{\tau_{m0}(S_0)} = Y_S \quad (7.3)$$

$$Y_\tau = Y_S \frac{g_i(f_{ci}, d)}{g_0(f_{c0}, d)} \quad (7.4)$$

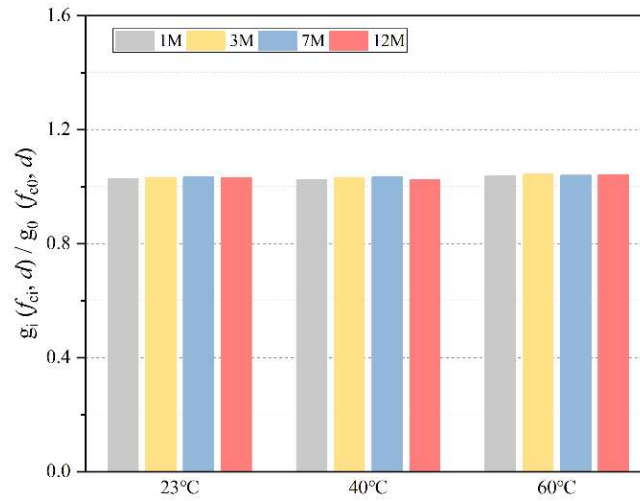


Figure 7.4. Ratio of $g_i(f_{ci}, d) / g_0(f_{c0}, d)$ for the S140 specimen set

Table 7.1. Coefficients in the bond strength degradation model

Bar type	Coefficient	
	β_d	θ
Deformed bar	0.21	55
Sand-coated bar	0.68	36

Figure 7.5 presents a comparison between the predicted bond strength retention and the experimentally determined values. The proposed models are evaluated using several statistical indicators, including the M, CoV, RMSE, and MAPE. It is evident that the proposed model does not provide accurate predictions for specimens from Li *et al.*'s study (2023). This discrepancy is attributed to the significantly smaller cover thickness of embedded ILSS specimens compared to PO specimens in Li *et al.*'s test program, as mentioned in section 7.2.1.3. The reduced cover thickness facilitated more rapid ingress of aggressive solutions,

leading to faster degradation of FRP bars in ILSS specimens than in PO specimens. Consequently, predictions based on the material degradation of these ILSS specimens significantly underestimated bond strength retention. Excluding the specimens from Li *et al.*'s study, the predictive model generally provided close agreement with the test results, with low predicted errors of 5.41% in RMSE and 3.51% in MAPE. Nonetheless, the proposed model requires further assessment as more test results are available, particularly those evaluating both the bond durability of FRP bars in concrete and the material deterioration of FRP bars and surrounding concrete.

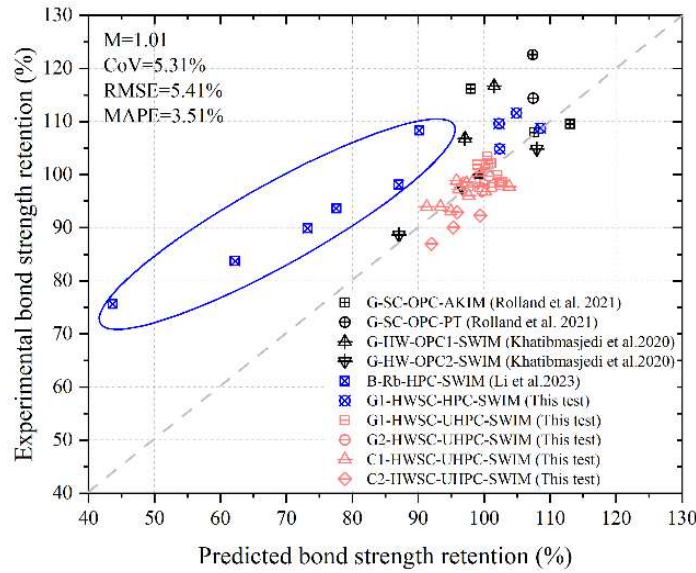


Figure 7.5. Comparison of test and predicted bond strength retention

7.3. Prediction of long-term tensile strength of FRP bars in concrete

7.3.1. Experimental database

A consolidated database of 430 accelerated aging tests, including the test data of FRP-UHSSC specimens in this thesis was created in this study. Table 7.2 provides a breakdown of the tests in this database by bar type, condition type (with/without concrete cover), alkalinity level, aging temperature, and aging duration. Each test was conducted with three to six repetitions. As summarized in Table 7.2, the collected data comprised three types of FRP bars: SC GFRP bars (from Pultrall Inc.), HWSC GFRP bars (from Dextra Building Products Co., Ltd, Hughes Brothers Inc., Pulwell Composites Co., Ltd), and Rb BFRP bars (from Jiangsu

Green Materials Valley New Material T&D Co., Ltd). For each bar type, the short-term data of both bare bars in alkaline solution and embedded bars in moist concrete were assembled.

Numerous researchers have highlighted the significant impact of surrounding alkali-ion content on the degradation rate and residual strength at infinite aging time of FRP bars (Chen *et al.* 2006; Chen *et al.* 2007; Wang *et al.* 2017; Yi *et al.* 2022; Wang *et al.* 2023; Feng *et al.* 2023). They have indicated that higher alkalinity causes faster and more damage to FRP bars. It has been reported that mature ordinary portland cement concrete typically has a pH value of 12.5 to 13 (Li *et al.* 2005; ACI 2012; Behnood *et al.* 2016), and in some cases, between 13 to 13.6 (Bruneaux and Benzarti 2006; Benzecry *et al.* 2021). When part of portland cement is replaced with silica fume or other pozzolanic materials, the alkalinity of concrete can be significantly reduced, with pH values of around 10 to 12.

As a result, the following analysis was performed based on two different alkalinity levels: normal concrete (NC) of normal-alkalinity or alkaline solution with a pH value higher than 12.5, and low-alkalinity concrete (LAC) or alkaline solution with a pH value lower than 12.5. It can be noticed that the sustained load was applied to some specimens. Nevertheless, the effect of sustained load on the degradation of FRP bars in these cases was neglected in this study, as the applied stress level in the collected tests was relatively low and hence showed minimal effect on degradation (Debaiky *et al.* 2006; Davalos *et al.* 2011; Wu *et al.* 2015).

7.3.2. Prediction approaches

7.3.2.1. Degradation laws

Long-term predictions of FRP bars in natural environments are commonly based on the Arrhenius relation using short-term accelerated aging test results (Nelson 2009). The Arrhenius relation is appropriate when the degradation of FRP is dominated by only one single mechanism that does not change with the exposure temperature and time. On the basis of the Arrhenius relation, three different types of degradation models were adopted by researchers (Bank *et al.* 2003; Katsuki and Uomoto 1995; Davalos *et al.* 2011; Wang *et al.* 2017a), namely the logarithmic model, exponential model and the diffusion-based model. As reviewed in Chapter 2.5, the logarithmic model is only a phenomenological representation of part of test data, and fitting curves using this model have violated the basic assumptions of the Arrhenius relation in some cases (Gonenc 2001; Davalos *et al.* 2011; Wu *et al.* 2015;

Hosseini *et al.* 2024). Moreover, the diffusion-based model requires coefficients that are unavailable due to the absence of moisture absorption data from the collected accelerated aging tests. Consequently, the exponential model (i.e., Eq. (7.4) and Eq. (7.5)) was adopted in this study.

Experimental observations in Figure 7.6 indicate that the residual tensile strength of FRP bars did not reach zero but stabilized at a certain level after aging in most cases. Consequently, the exponential equation in Eq. (7.5) with a given residual retention Y_{∞} , which can be simplified to Eq. (7.4) when Y_{∞} reduces to zero, is more applicable for the following analysis.

$$Y = 100 \exp\left(-\frac{t}{\tau}\right) \quad (7.4)$$

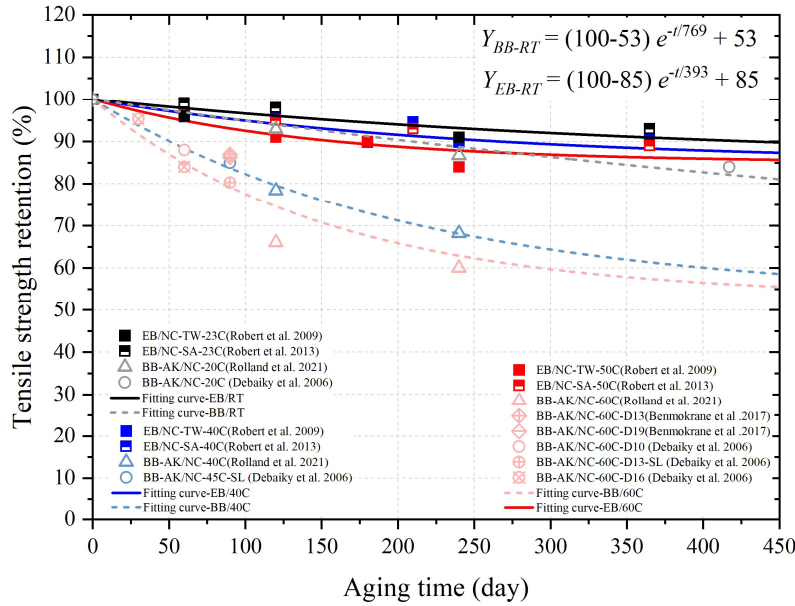
$$Y = (100 - Y_{\infty}) \exp\left(-\frac{t}{\tau}\right) + Y_{\infty} \quad (7.5)$$

where Y represents the tensile strength retention (%), Y_{∞} is the tensile strength retention (%) at the exposure time of infinity, and $1/\tau$ is the degradation rate coefficient which varies with aging temperature and can be obtained by regression.

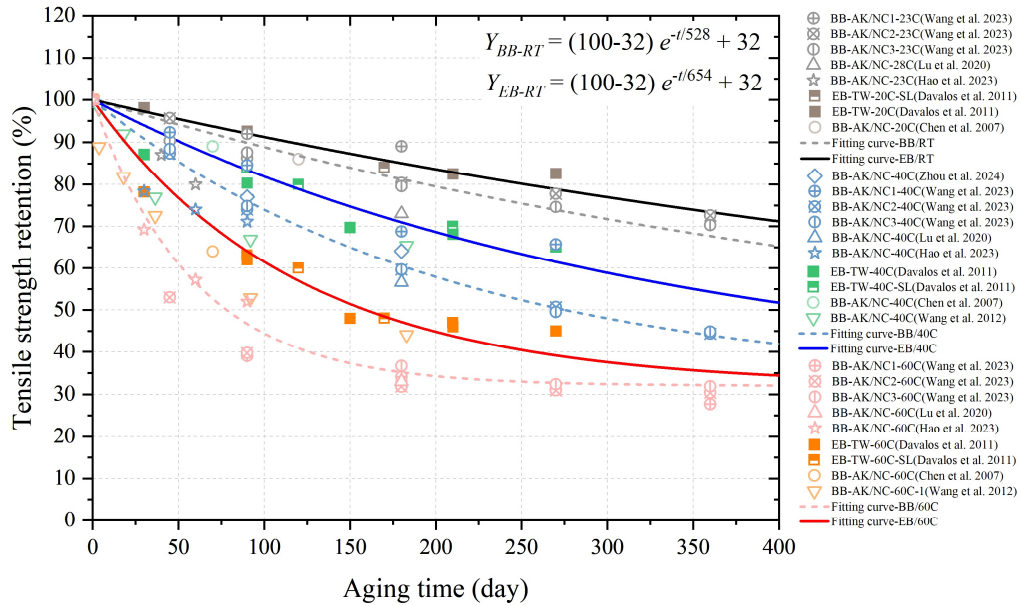
Although bare bars and the same type of bars embedded in concrete exposed to the same pH level exhibit different degradation rates, they are expected to have identical strength retention at infinite exposure time Y_{∞} . The fitting curves for three types of FRP bars in concrete environments with different alkalinity levels are presented in Figure 7.6. It can be found that the test results were well fitted with the exponential model in Eq. (7.5), with R^2 values of 0.8 or higher for all fitting curves. The regressed parameters for the exponential degradation model are given in Table 7.3 and Figure 7.7.

Figure 7.7 (a) to (c) confirm that the degradation rate of bare bars is faster than that of the same embedded bars at identical alkalinity levels, with the difference in degradation rates increasing with aging temperature. In addition, Figure 7.7 (d) indicates that tensile strength degradation is dominated by the alkalinity level of exposure conditions. The residual tensile strength retention Y_{∞} of bars exposed to low alkalinity is 34% to 172% higher than that in normal alkalinity. It is also noted that tensile durability varies with bar type. SC GFRP bars

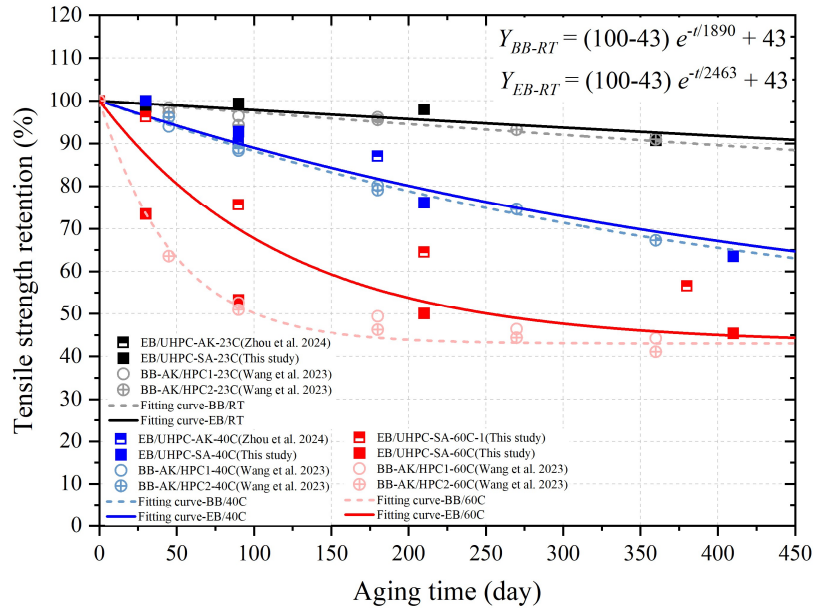
exhibited the highest residual tensile strength retention Y_{∞} of 53% and 85%, respectively, in the normal and low alkalinity environments, while the worst durability was observed for Rb BFPR bars in normal alkalinity and HWSC GFRP bars in the low alkalinity environments. Apart from different technologies with respect to fibers, resins, and manufacturing processes among the three types of FRP bars, the dense sand coating of the SC bar surface is believed to provide effective protection and restraint in bars, resulting in less deterioration, as confirmed by Yue *et al.*'s study (2024).



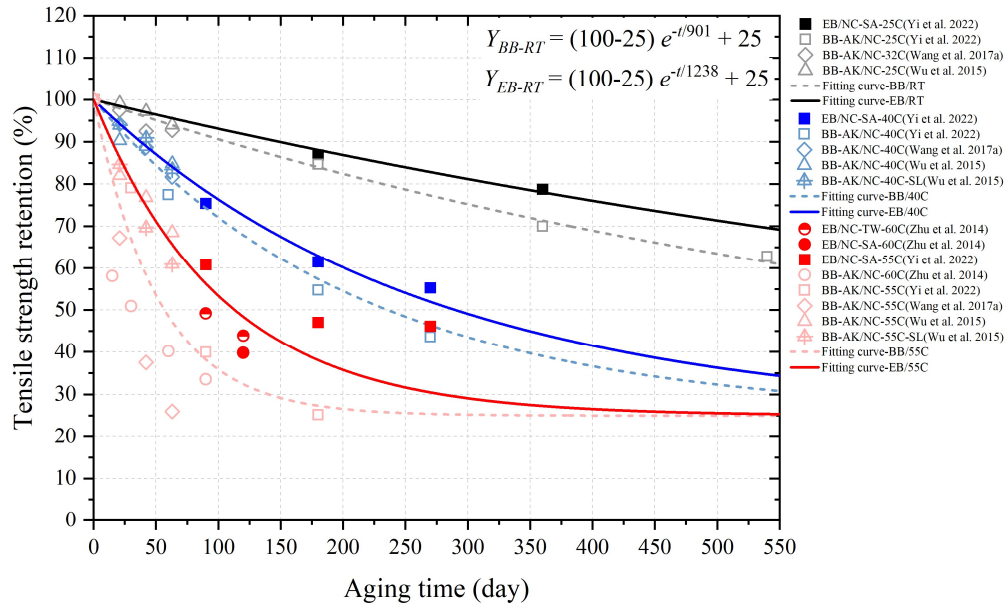
(a) SC/GFRP-NC/LAC



(b) HWSC/GFRP-NC



(c) HWSC/GFRP-LAC



(d) Rb/BFRP-NC

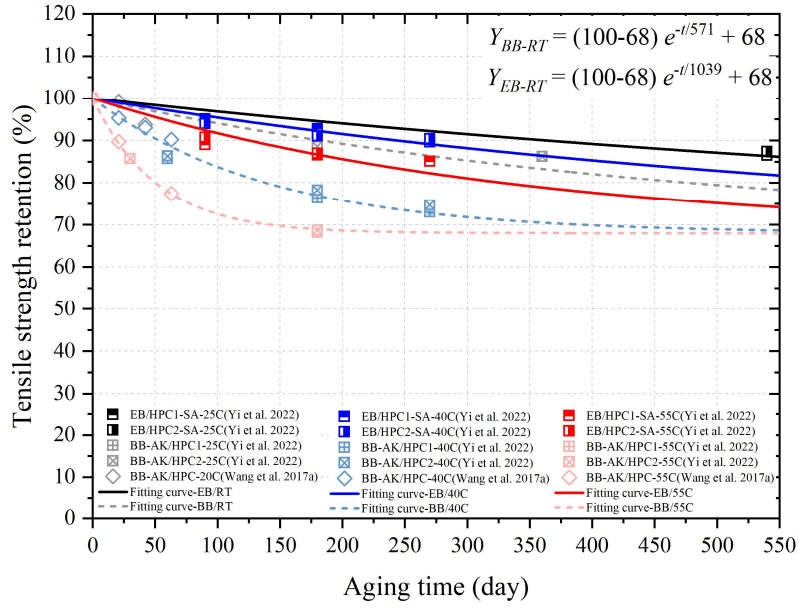


Figure 7.6. Tensile strength retention and fitting curves of bars in accelerated aging conditions

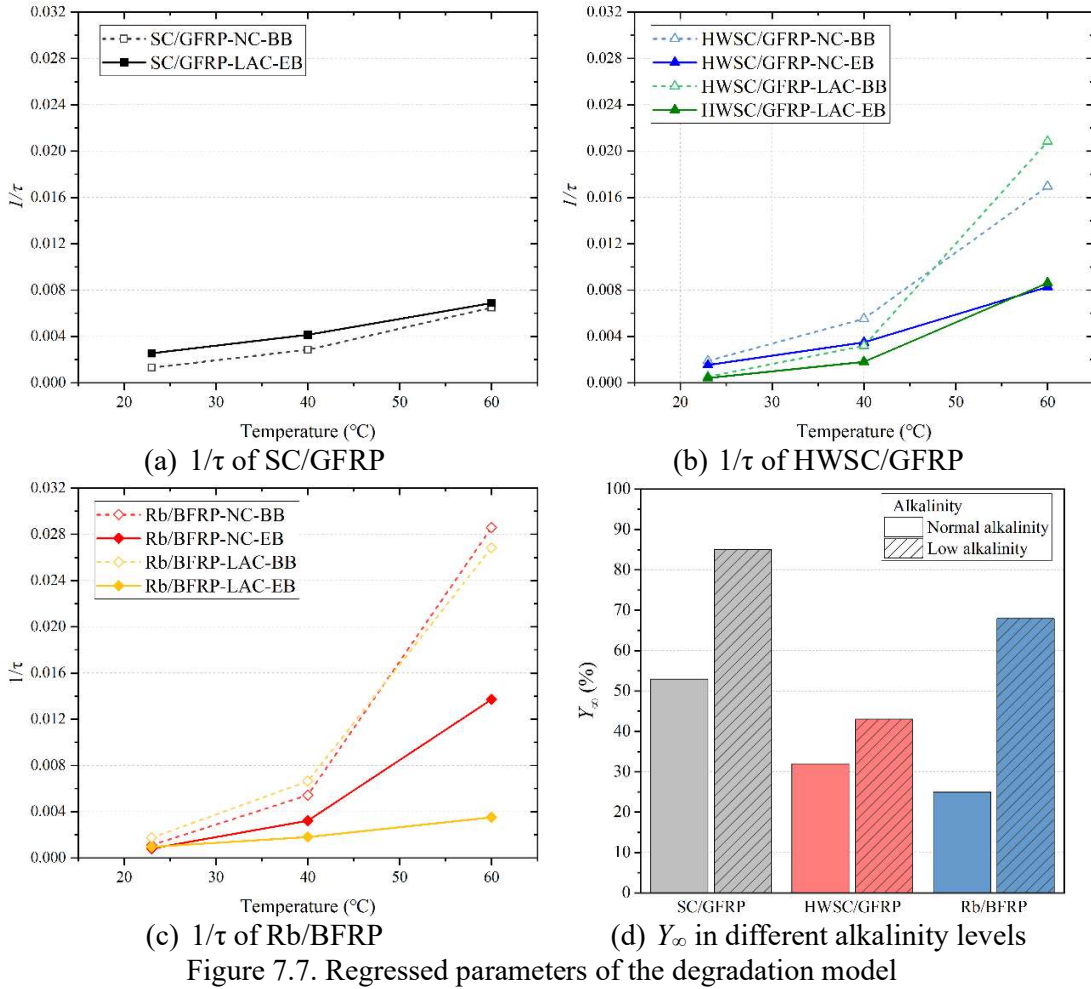


Figure 7.7. Regressed parameters of the degradation model

Table 7.2. Summary of accelerated aging tests

Ref.	Bar type (Producer)	Condition	Concrete type	pH concrete EB)	of (for	pH of solution (for BB)	Bar diameter (mm)	Applied stress level	Temperature (°C)	Aging duration (day)
Robert <i>et al.</i> 2009	GFRP (SC)	EB	LAC	12.15		--	13	0	RT, 40, 50	240
Robert <i>et al.</i> 2013		EB	LAC	12.15		--	13	0	RT, 40, 50	365
Debaiky <i>et al.</i> 2006		BB	--	--		12.8	10, 13, 16	0, 17-24% f_u	RT, 40, 60	417
Benmokrane <i>et al.</i> 2017		BB	--	--		12.6	10, 13, 16	0	60	90
Rolland <i>et al.</i> 2021		BB	--	--		13-13.6	13	0	RT, 40, 60	240
Davalos <i>et al.</i> 2011	GFRP (HWSC)	EB	NC	--		--	10	0, 11-15% f_u	RT, 40, 50, 60	270
Chen <i>et al.</i> 2006		BB	--	--		12.7	10	0	RT, 40, 60	120
Chen <i>et al.</i> 2007		BB	--	--		13.6	10	0	RT, 40, 60	120
Wang <i>et al.</i> 2012		BB	--	--		12.6-13	16	0	40, 60	183
This test		EB	LAC	--		--	10	0	RT, 40, 60	410
Zhou <i>et al.</i> 2024		EB	LAC	--		--	10	0	RT, 40	180
		BB	--	--		12.6	10	0	40	180
Wang <i>et al.</i> 2023		BB	--	--		11, 12, 13, 13.6	10	0	RT, 40, 60	365
Lu <i>et al.</i> 2020		BB	--	--		12.6-13	6	0	RT, 40, 60	180
Hao <i>et al.</i> 2023	BB	--	--		13.4	10	0	RT, 40, 60	90	
Yi <i>et al.</i> 2022	BFRP (Rb)	EB	LAC, NC	11.1-11.7, 12.5		--	8	0	RT, 40, 55	540
Zhu <i>et al.</i> 2014		EB	NC	12.7-12.75		--	6	0	60	120
Yi <i>et al.</i> 2022		BB	--	--		10.1-12.3, 13.2	8	0	RT, 40, 55	540
Zhu <i>et al.</i> 2014		BB	--	--		12.7	6	0	60	90
Wang <i>et al.</i> 2017		BB	--	--		12.7, 13.4	6	0	RT, 40, 55	63
Wu <i>et al.</i> 2015		BB	--	--		12.9-13.1	6	0, 20% f_u	RT, 40, 55	63

7.3.2.2. Long-term prediction using Arrhenius relation

The following long-term prediction of FRP bars in field conditions was conducted using the Arrhenius relation, as given in Eq. (7.6)

$$k = A \cdot e^{\frac{-E_a}{RT}} \quad (7.6)$$

where k is the degradation rate; A is the constant of the material and degradation process, E_a is the activation energy, R is the universal gas constant, and T is the kelvin temperature. According to Eq. (7.6), the TSF can then be expressed as Eq. (7.7)

$$TSF = \frac{t_0}{t_1} = \frac{k_1}{k_0} = \frac{A \cdot e^{\frac{-E_a}{RT_1}}}{A \cdot e^{\frac{-E_a}{RT_0}}} = e^{\frac{E_a}{R}(\frac{1}{T_0} - \frac{1}{T_1})} \quad (7.7)$$

where T_0 and T_1 are temperatures of short-term accelerated aging condition and service condition, respectively, and t_0 and t_1 are exposure days to reach a given strength retention at T_0 and T_1 , respectively. The projected tensile strength reduction (Δ_1) of design life at service temperature T_1 can then be calculated by substituting exposure time of t_0/TSF into Eq. (7.5) at T_0 .

Table 7.3. Fitting results of tensile strength retention of bars in accelerated aging conditions

Bar type-alkalinity	Condition	Fitting curve	R^2	E_a/R
SC/GFRP-NC/LAC	BB	$Y_{BB-23C} = (100 - 53)e^{\frac{-t}{769}} + 53$	0.94	4232
		$Y_{BB-40C} = (100 - 53)e^{\frac{-t}{351}} + 53$	0.80	
		$Y_{BB-60C} = (100 - 53)e^{\frac{-t}{154}} + 53$	0.92	
	EB	$Y_{EB-23C} = (100 - 85)e^{\frac{-t}{394}} + 85$	0.80	2631
		$Y_{EB-40C} = (100 - 85)e^{\frac{-t}{242}} + 85$	0.82	
		$Y_{EB-60C} = (100 - 85)e^{\frac{-t}{145}} + 85$	0.80	
HWSC/GFRP-NC	BB	$Y_{BB-23C} = (100 - 32)e^{\frac{-t}{529}} + 32$	0.80	5772
		$Y_{BB-40C} = (100 - 32)e^{\frac{-t}{181}} + 32$	0.90	
		$Y_{BB-60C} = (100 - 32)e^{\frac{-t}{59}} + 32$	0.97	
	EB	$Y_{EB-23C} = (100 - 32)e^{\frac{-t}{654}} + 32$	0.95	4440

HWSC/GFRP-LAC	BB	$Y_{EB-23C} = (100 - 32)e^{\frac{-t}{654}} + 32$	0.93	9666
		$Y_{EB-60C} = (100 - 32)e^{\frac{-t}{121}} + 32$	0.98	
		$Y_{BB-23C} = (100 - 43)e^{\frac{-t}{1890}} + 43$	0.80	
		$Y_{BB-40C} = (100 - 43)e^{\frac{-t}{313}} + 43$	0.82	
		$Y_{BB-60C} = (100 - 43)e^{\frac{-t}{48}} + 43$	0.99	
	EB	$Y_{EB-23C} = (100 - 43)e^{\frac{-t}{2463}} + 43$	0.81	8041
		$Y_{EB-40C} = (100 - 43)e^{\frac{-t}{552}} + 43$	0.92	
		$Y_{EB-60C} = (100 - 43)e^{\frac{-t}{116}} + 43$	0.80	
		$Y_{BB-23C} = (100 - 25)e^{\frac{-t}{901}} + 25$	0.96	
		$Y_{BB-40C} = (100 - 25)e^{\frac{-t}{184}} + 25$	0.96	
Rb/BFRP-NC	BB	$Y_{BB-60C} = (100 - 25)e^{\frac{-t}{35}} + 25$	0.80	8548
		$Y_{EB-23C} = (100 - 25)e^{\frac{-t}{1238}} + 25$	0.94	
		$Y_{EB-40C} = (100 - 25)e^{\frac{-t}{311}} + 25$	0.94	
		$Y_{EB-60C} = (100 - 25)e^{\frac{-t}{73}} + 25$	0.91	
		$Y_{BB-23C} = (100 - 68)e^{\frac{-t}{571}} + 68$	0.88	
	BB	$Y_{BB-40C} = (100 - 68)e^{\frac{-t}{151}} + 68$	0.97	7201
		$Y_{BB-60C} = (100 - 68)e^{\frac{-t}{37}} + 68$	1.00	
		$Y_{EB-23C} = (100 - 68)e^{\frac{-t}{1039}} + 68$	0.96	
		$Y_{EB-40C} = (100 - 68)e^{\frac{-t}{552}} + 68$	0.86	
		$Y_{EB-60C} = (100 - 68)e^{\frac{-t}{285}} + 68$	0.90	
Rb/BFRP-LAC	EB			3404

Note: Y_{BB-} is the tensile strength retention of bare bars conditioned at XX °C, and Y_{EB-} is the tensile strength retention of embedded bars conditioned at XX °C.

For the structures in moisture-unsaturated conditions, the effect of humidity should be considered by adopting the RH correction factor β_H , as discussed in Huang and Aboutaha's paper (2010). The correction coefficient β_H , accounting for the influence of RH in the exposure environment, is shown in Figure 7.8. The correction factor at RH values between those specified in Figure 7.8 can be obtained through linear interpolation. Thus, the strength retention of FRP bars under field exposure can be expressed as follows

$$Y = 100(1 - \beta_H \cdot \Delta_1) \quad (7.8)$$

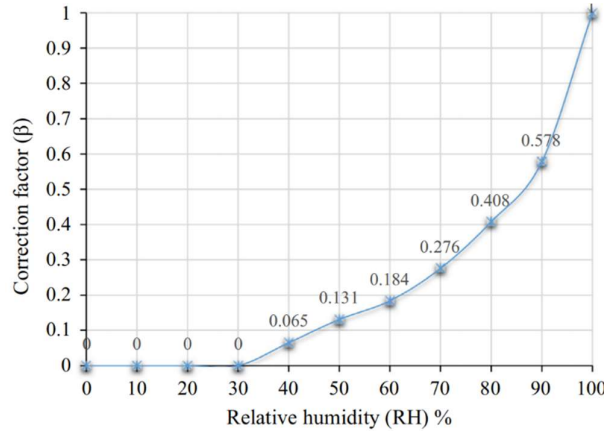


Figure 7.8. Correction factor β_H with varying RH (Dong *et al.* 2017)

7.3.3. Prediction results

7.3.3.1. Comparison of prediction results based on two approaches

To evaluate the effect of concrete wrap on long-term predictions, the tensile strength retentions based on short-term accelerated test results of bare bars (Y_{BB}) and embedded bars (Y_{EB}) were assessed at different RH values with a MAT ranging from 10°C to 33°C over various service years. The temperatures of 10°C and 33°C were selected to represent the MAT in the northern and tropical climates, respectively. Meanwhile, 23°C was chosen as the mean temperature for intermediate regions between these zones, based on field test data in Chapter 2.4.3. The differences between the two environments were compared using the ratio of Y_{BB} to Y_{EB} . The results are presented in Figure 7.9. Due to the lack of experimental short-term data for bare SC GFRP bars in low alkalinity solutions and corresponding embedded bars in normal alkalinity concrete matrices, SC GFRP bars were not included in this section.

Figure 7.9 shows that the difference between Y_{BB} and Y_{EB} for FRP bars exposed at two different alkalinity levels varied with RH and temperature in field conditions. For structures in dry conditions, the Y_{EB} values for both types of FRP bars were nearly identical to their Y_{BB} values at the same alkalinity level at all service temperatures. In conditions with a median RH of 60%-70%, the difference in long-term predictions based on both environments was less than 5%. The Y_{BB}/Y_{EB} ratio showed greater reductions as RH increased, with the most significant effect observed in saturated conditions, where the ratio was as low as 77%.

In addition, the discrepancy in long-term predictions based on both environments increased

with rising field temperature in high RH and saturated conditions. For instance, in moisture-saturated conditions, only Rb BFRP bars at normal alkalinity presented some differences between Y_{BB} and Y_{EB} (a maximum of 8%) at 10°C. At 23°C, all FRP bars at both alkalinity levels exhibited noticeable differences in Y_{BB} and Y_{EB} , with the most significant differences (13%-23%) reported at 33°C for each type of FRP bar at the two alkalinity levels.

It can be noticed that the discrepancy between the results of the two prediction approaches generally emerged within the first 30 years of field exposure. Specifically, apparent differences between Y_{BB} and Y_{EB} were observed in the first 5 years at 33°C and 1.5 to 7 years at 23°C, respectively, when structures experienced high RH of 80% to 90%. Moreover, in the moisture-saturated environments, during the first 10 years of exposure at 33°C, 1 to 20 years at 23°C, and 7 to 30 years at 10°C, predictions based on short-term data on bare bars presented obviously lower tensile strength retention than those based on embedded bars.

Against these discussions, it can be concluded that when structures are subjected to high RH and moisture-saturated conditions, especially in hot climates, long-term predictions of the field durability of internal FRP bars using traditional aging data of bare bars will yield conservative results during the aforementioned critical exposure times. In such cases, it is more practical to assess the long-term performance of internal FRP reinforcements based on the short-term durability of concrete-wrapped specimens. Otherwise, predictions using short-term data of bare bars in alkaline environments are acceptable.

7.3.3.2. Assessment of long-term performance for a 100-year design life

The discussion in the previous section indicated that the two different prediction approaches yielded nearly identical durability results for a 100-year design life. By substituting a service life of 100 years in Eq. (7.5) and considering the effect of environmental RH in Eq. (7.8), the long-term tensile strength of three types of FRP bars under field exposure at different RH values was evaluated. The predicted long-term performance is summarized in Table 7.4. Notably, after 100-year exposure, the tensile strength retention of each bar type for different RH values did not vary with exposure MAT ranging from 10°C to 33°C, so the temperature effect was not included in this assessment.

It can be found that internal FRP bars exhibited superior long-term performance in dry conditions (50% RH), with all tensile strength retentions Y exceeding 90% (lower limits for

a 95% confidence interval of 91%). Under indoor and outdoor exposure at 60%-80% RH, the tensile strength retentions of concrete-wrapped FRP bars at two different alkalinity levels ranged from 69% to 97%, with lower and upper limits for a 95% confidence interval of 82% and 89%, among which, the average tensile strength retention of bars in low alkalinity concrete was 90% (95% confidence interval, CI: 86%-94%). Some bars, particularly those in normal-alkalinity concrete, had tensile strength retentions lower than 85%, an environmental reduction factor recommended by ACI 440.11-22 (ACI 2022). However, it has been demonstrated that the alkalinity of concrete members can decrease over exposure time due to carbonation (Chang and Chen 2006). For instance, Grubb *et al.* (2007) recorded a pH value of 12 for a 2-month-old concrete specimen and a significant reduction to around 10.5 for a 20-year-old specimen. Similarly, Benzecry *et al.* (2021) reported average pH values 10-12.2 for 15- to 20-year old bridges using ordinary portland cement concrete, which is observably less than the general pH values in normal concrete mentioned earlier. Consequently, for structures in the open air, it is safe to assess FRP bars in normal concrete based on the normal alkalinity environment.

For concrete structures exposed to earth and weather (around 90% RH), the tensile strength retentions of bars at the two exposure alkalinity levels were between 62% and 82% (average of 72%) based on a confidence coefficient of 95%. Under submerged conditions (100% RH), all FRP bars degraded significantly, with a 95% confidence interval in the range of 33% to 69% and an average of 51%.

Based on the calculated tensile strength retentions, the reduction factor accounting for the effects of long-term exposure to the environment (C_E) stipulated in ACI 440.11-22 (ACI 2022) is aggressive for internal FRP reinforcement, particularly for those in contact with earth and those in submerged conditions. As given in Table 7.5, the results suggest that an environmental reduction factor for a 100-year design life of 0.82 is more appropriate for internal reinforcement not exposed to earth and weather, based on a 95% confidence interval level. For structures in contact with earth, this value should be reduced to 0.62 and 0.72 for FRP bars with moderate and good alkaline resistance, respectively, given the lower bound for the 95% confidence interval and the average strength retention in this exposure condition. Following the same methodology, the C_E value in the most aggressive environment (submerged condition) can be determined as 0.33 and 0.51 for the FRP bars showing moderate and good alkaline resistance, respectively.

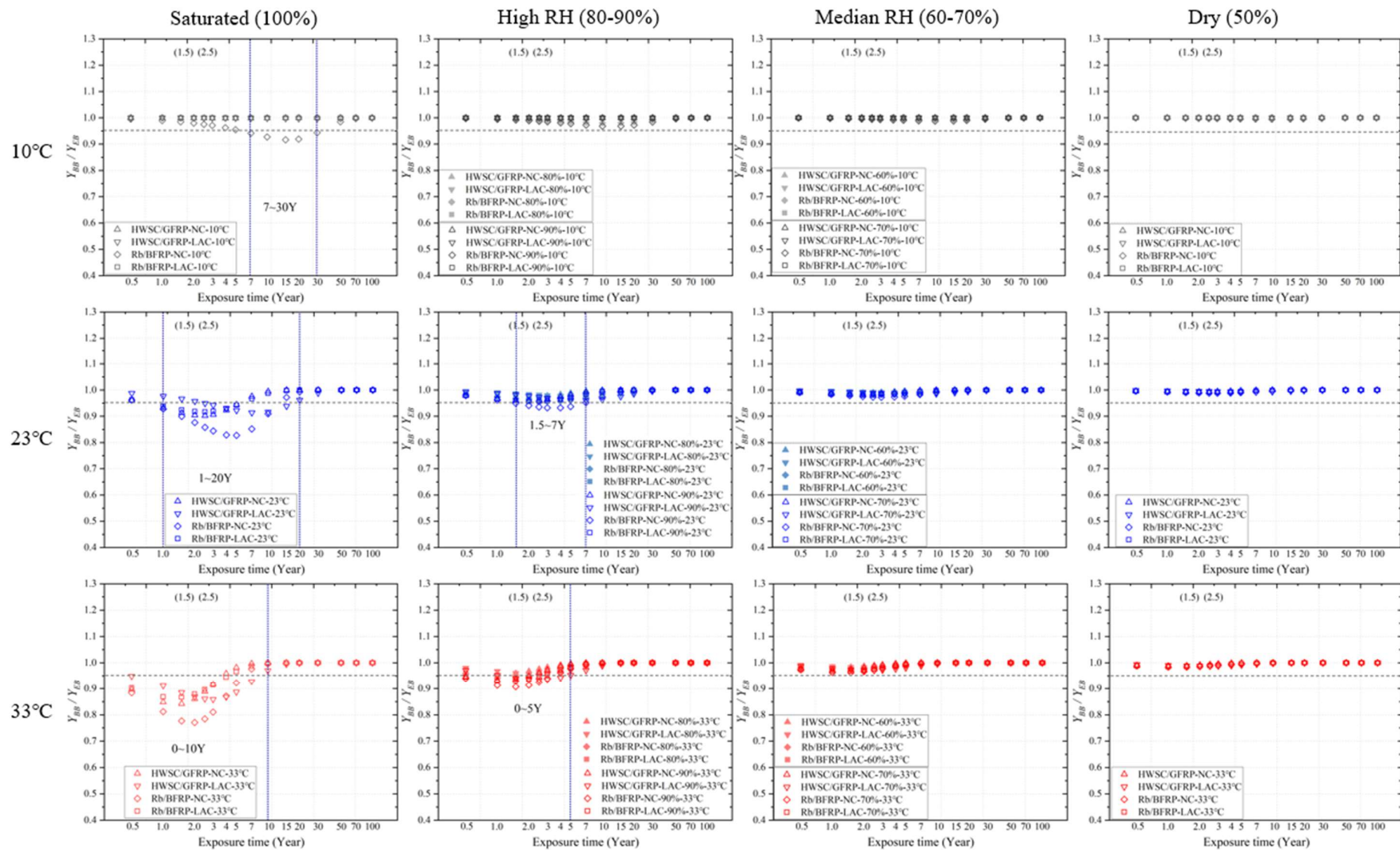


Figure 7.9. Comparison of tensile strength retention under different RH values and temperatures

Table 7.4. Tensile strength retention of three types of FRP bars for a 100-year design life

Bar type	Alkalinity level	Tensile strength retention					
		50% RH	60% RH	70% RH	80% RH	90% RH	100% RH
SC /	Normal	94	91	87	81	73	53
GFRP	Low	98	97	96	94	91	85
HWSC /	Normal	91	87	81	72	61	32
GFRP	Low	93	90	84	77	67	43
Rb /	Normal	90	86	79	69	57	25
BFRP	Low	96	94	91	87	82	68

Table 7.5 Environmental reduction factor under different field conditions for a 100-year design life

Field condition	RH	C_E (95% confidence interval level)
Dry condition	50%	0.91
Not exposed to earth and weather	60%-80%	0.82
Exposed to earth and weather	around 90%	0.62 (normal concrete) 0.72 (low-alkalinity concrete)
Submerged condition	100%	0.33 (normal concrete) 0.51 (low-alkalinity concrete)

7.4. Conclusions

This chapter focused on the long-term prediction of the residual bond strength and tensile strength of FRP bars in concrete. Based on experimental results in this thesis and previous studies, a prediction model was developed to assess the bond strength degradation of FRP bars. Moreover, this study summarized short-term data from 430 accelerated aging tests to predict the field durability of internal FRP reinforcement. The following concluding remarks can be made.

1. The deterioration in bond strength presented a direct correlation with changes in the surrounding concrete strength and the horizontal shear strength of FRP bars in the same concrete matrix after identical exposure. A quantitative model was developed to evaluate bond strength degradation based on changes in the material properties of FRP bars and the surrounding concrete. The results showed that the prediction of the proposed model is in close agreement with the test data.
2. Long-term predictions of tensile strength of internal FRP reinforcement in field

environments were performed considering the effects of bar type, concrete wrap, alkalinity level, RH, and MAT of exposure, based on the Arrhenius relation. The degradation results show that residual tensile strength retention greatly depended on the bar type and alkalinity level in the concrete environment. For structures in relatively moist conditions (80%-100% RH), especially in hot climates, the positive effect of concrete wrap on prediction results was significant within the first 30-year exposure. Estimations of FRP durability based on short-term data of bare bars yielded observably conservative results during this critical exposure duration, and otherwise, long-term predictions without considering the effect of concrete wrap were acceptable.

3. Long-term prediction results indicate that the environmental reduction factor (C_E) stipulated in the ACI design code was aggressive for internal FRP reinforcement in field conditions. Based on a 95% confidence interval level, this study recommended an environmental reduction factor for a 100-year design life of 0.82 for internal reinforcement not exposed to earth and weather. For structures in contact with earth, the C_E value of 0.62 and 0.72 can be implemented for FRP bars with moderate and good alkaline resistance, respectively, while these values were reduced to 0.33 and 0.51 in submerged conditions.

8. CONCLUSIONS AND FUTURE RESEARCH NEEDS

8.1. Introduction

In response to the shortage of river sand, non-desalted sea-sand has been used in UHSC, leading to the development of sustainable UHSSC. However, steel reinforcements cannot be used in UHSSC due to corrosion concerns. The use of high strength and corrosion-resistant FRP bars as reinforcements in UHSSC offers a viable solution for coastal environments. To have a good understanding and validate this new type of structural system, this thesis has presented a comprehensive investigation of the short- and durability performance of FRP bars in UHSSC under coastal conditions.

The first part of this thesis addressed the short-term bond behavior of several commercially available FRP bars. Chapter 3 begins with a systematic experimental study on the bond performance of FRP bars in UHSSC. This test program sheds light on the effects of horizontal shear properties and surface topography of FRP bars on their bond performance in UHSSC, factors that have not been quantitatively evaluated in previous research. Based on the experimental observations, several recommendations for the design equation for UHSC, which are not adequately addressed in current design guidelines, are provided.

The test results in Chapter 3 then lay the groundwork for developing a reliable bond strength model in Chapter 4. A comprehensive database containing 687 pullout specimens, compiled from 29 published studies and test data in Chapter 3, was presented in Chapter 4. Statistical analyses, including ANOVA and corresponding post hoc tests, were performed on the assembled database to identify the interactions between the effects of several critical factors. Subsequently, a new model was developed based on the general size effect law, incorporating the effects of these four key parameters and their statistically significant coupling effects. The proposed bond strength model demonstrates broader applicability than existing models, as it is valid for a wide range of concrete grades, from normal concrete to UHSC. Importantly, the new bond strength model accurately captures both the size effect and the two-way interaction effects of key parameters, which are overlooked in existing models.

The second part of this thesis (Chapter 5, Chapter 6, and Chapter 7) focused on the durability performance of UHSSC-wrapped FRP bars. This research was the first to investigate the durability performance of this new type of structural member (i.e., the FRP-UHSSC member) in coastal environments under both natural and accelerated aging conditions. In Chapter 5, a series of accelerated aging tests were conducted by immersing FRP bars in a simulated seawater environment at both room and elevated temperatures for up to 13.5 months. The experimental program involved four types of FRP bars, two types of concrete wrapping, and two bar conditions. Tensile tests, short-beam shear tests, and pullout tests were performed to evaluate the evolution of their mechanical properties and the correlation between the material degradation of FRP bars. The accelerated aging test in Chapter 5 was able to generate sufficient durability data within a short space of time, enabling extrapolation of the long-term performance of FRP bars under field conditions, as analyzed in Chapter 6.

Chapter 6 then investigated the durability performance of four types of bare GFRP and CFRP bars, as well as corresponding UHSSC-wrapped specimens, conditioned in four different field environments at a wharf in Zhanjiang, China, over one year, including the atmospheric, splash, tidal, and underwater zones. Three types of mechanical tests were also conducted to evaluate changes in bond properties, tensile properties, and horizontal shear strength after field exposure. These properties are considered sufficient to provide the necessary insight into the field durability of FRP bars and their interaction with the surrounding concrete, thereby providing useful guidelines for practical applications.

In Chapter 7, based on the durability studies in this thesis and previous research, a new bond degradation model was developed, which was the first one that can account for the changes in the material properties of both the FRP bars and the surrounding concrete. This chapter has summarized short-term data from 430 accelerated aging tests to predict the field durability of internal FRP reinforcement. In addition, the effect of concrete protection of FRP bars in traditional aging tests on the long-term predictions of internal FRP reinforcement was first examined in this research. Based on the comparison results, the simplified and appropriate aging test method for accurate long-term predictions of internal FRP reinforcement was identified.

Based on these systematic studies, this chapter summarizes the key findings of the thesis and offers recommendations for future research needs.

8.2. Short-term bond of FRP bars in UHSSC

The following concluding remarks can be drawn from Chapter 3:

1. The bond failure mode changed with concrete strength. The pullout specimens typically failed near the bar-to-concrete interface in HSC, exhibiting partial debonding within the surrounding concrete and partial at the surface layer of FRP bars. Conversely, in UHSSC, bond failure predominantly occurred at the surficial and subsequent core layers of FRP bars.
2. Consistency in bond performance is highly dependent on the quality control of surface treatment. The topographical regularity of the FRP bar surface significantly influences bond performance, including both bond strength and the corresponding slip value.
3. In general, the bond strength of FRP bars in UHSSC is not significantly affected by the effects of compressive strength of concrete and or the bar diameter. Instead, it is influenced by the fiber type and the horizontal shear performance of the FRP bars. In UHSSC, the horizontal shear performance of FRP bars impacts both the post-peak bond behavior and the bond strength. FRP bars with brittle shear failure characteristics tend to exhibit sudden bond failure, and the bond strength increases with the horizontal shear strength of FRP bars. Notably, CFRP bars demonstrate substantially higher bond strength than GFRP bars produced using the same fabrication technology.
4. In both HSSC and UHSSC, bar diameter is a critical factor that influences the slip at bond strength for FRP bars, whereas concrete strength and surface type are less significant. Generally, the slip at bond strength increases with the bar size.
5. For design purposes, incorporating a fiber factor and establishing an upper limit based on the horizontal shear strength of specific types of FRP bars are recommended for accurately predicting the bond strength of FRP bars in UHSC.

To quantify the effects of these critical factors and to develop a bond strength predictive equation of general validity, a broader interpretation of commercially available FRP bars, supported by a large database, is necessary. This topic is further explored in Chapter 4.

Chapter 4 focuses on the development of a commonly accepted bond strength predictive equation between commercially available FRP bars and concrete of different grades. The following conclusions can be drawn:

1. The ANOVA test results revealed statistically significant 2-way interactions between the effects of several factors affecting bond strength: bar type and concrete strength, bar type and bar diameter, concrete strength and bar diameter, as well as fiber type and concrete strength.
2. Further post hoc analysis indicated that the influence of bar diameter on bond strength is dependent on bar type. Specifically, the bond strength of both HWSC and SC bars shows statistically remarkable size dependence, whereas for In bars, bar diameter does not significantly affect the bond strength. In addition, the effect of bar diameter on bond strength becomes less pronounced as the concrete strength increases, and in UHSC, there is no statistically significant size effect. The increasing rates of bond strength with respect to concrete strength vary with bar types. Regarding the effect of fiber type, CFRP bars demonstrate significantly higher bond strength than GFRP bars in UHSC, but no statistically significant difference can be observed between the bond strengths of GFRP and CFRP bars in NSC and HSC matrices.
3. A new bond strength predictive equation (Eq. (4.14) and Eq. (4.15)) was developed for HWSC and SC bars, based on the general size effect law and incorporating the identified 2-way interaction effects. In comparison to existing models, the proposed predictive equation provides more accurate predictions of the test data, with mean experiment-to-predict ratios of 1.0 to 1.01, RMSE values of 2.35 to 2.57 MPa, and MPAE values of 10.98% to 13.35%.

8.3. Durability of FRP bars in UHSSC in coastal environments

Chapters 5 and 6 present a systematic study on the long-term performance of FRP bars embedded in UHSSC in coastal environments. The following conclusions can be drawn based on the study in Chapter 5:

1. The saline-alkalinity environments in UHSSC matrices do not adversely affect the durability of FRP bars. The UHSSC covers effectively decelerate the degradation rate of FRP bars under seawater exposure, resulting in comparable or even superior performance of UHSSC-wrapped FRP bars in contrast to bare bars.
2. Despite having excellent tensile durability, CFRP bars exhibited greater degradation in horizontal shear strength compared to GFRP bars. The poor horizontal shear durability

of CFRP bars is attributed to considerable defects in bars arising from immature compatibility between the resin and carbon fibers with low surface energy. These defects facilitate the ingress of foreign matters, leading to localized damage in the resin matrix and at the fiber-resin interface.

3. The bond durability between FRP bars and UHSSC under seawater immersion is not a critical concern. No significant changes in failure mode or long-term bond characteristics, including bond strength, bond stiffness, slip at bond strength, and relative post-peak strength, can be observed for FRP bars in UHSSC after seawater exposure.
4. After one year of seawater immersion at 60 °C, FRP bars in UHSSC with lower compressive strength exhibited an increase of around 10% in bond strength, primarily due to increased concrete strength. However, a substantial 30% decrease in bond stiffness was observed because of a change in failure mode from debonding at the bar-concrete interface to shear failure within the FRP bars.

Based on the systematic study in Chapter 6, the following conclusions can be drawn:

1. After one year of exposure in the coastal environments, the failure modes observed in tensile, interlaminar shear, and pullout tests did not change. In general, the tidal and underwater zones were identified as the most critical conditions, causing visible degradation in the mechanical properties of bare FRP bars.
2. The alkaline-saline environment in UHSSC does not negatively impact the durability of FRP bars. The compact microstructure of UHSSC cover effectively protects internal bars from severe deterioration by hindering the penetration of external water molecules and detrimental ions. After one year of coastal exposure, the most prominent degradation was observed in bare bars conditioned in the underwater zone, with maximum reductions in tensile and horizontal strength of around 10%. In comparison, UHSSC-wrapped FRP bars exhibited no significant reduction in these properties.
3. One-year coastal exposures had little impact on the long-term bond performance of FRP bar-UHSSC members. The bond strength, bond stiffness, characteristic slip at bond strength, and relative post-peak strength of exposed FRP bar-UHSSC members were comparable to those of control specimens.

Chapter 7 then is devoted to predicting the durability performance of FRP bars in concrete, and the main conclusions can be given as follows:

1. A quantitative model (Eq. (7.3)) was developed to evaluate bond strength degradation based on changes in the material properties of FRP bars and the surrounding concrete. The results showed that the prediction of the proposed model was in close agreement with the test data.
2. Long-term predictions for internal FRP reinforcement in field environments were performed considering the effects of bar type, concrete wrap, alkalinity level, RH, and MAT of exposure, based on the Arrhenius relation. The degradation results showed that residual tensile strength retention greatly depended on bar type and alkalinity level in the concrete environment. For structures in moist conditions (80%-100% RH), particularly in hot climates, the positive effect of concrete wrap on prediction results is significant within the first 30-year exposure. Estimations of FRP durability based solely on short-term data of bare bars are notably conservative for this critical exposure period; otherwise, long-term predictions without considering the effect of concrete wrap are acceptable.
3. Long-term prediction results suggest that the environmental reduction factor stipulated in the ACI design code is aggressive for internal FRP reinforcement in field conditions. At a 95% confidence interval level, this study recommends an environmental reduction factor for a 100-year design life of 0.82 for internal reinforcement not exposed to earth and weather. For structures in contact with earth, C_E values of 0.62 and 0.72 can be recommended for FRP bars with moderate and good alkaline resistance, respectively, and these values decrease to 0.33 and 0.51 for submerged conditions.

8.4. Future research needs

This thesis has provided a systematic investigation into the short- and long-term performance of FRP bars in UHSSC for potential use in coastal conditions. However, many research areas remain to be explored in the future for a confident application of FRP bar-UHSSC structural members in aggressive environments. For a comprehensive understanding of this new type of structural member, the future research needs are highlighted as follows:

1. The data distribution of the bond database in Chapter 4 revealed that experimental studies on the bond behavior of CFRP bars are limited. Future research is required to calibrate the fiber factor proposed in the bond strength model in Chapter 4. Additionally, more test results for large-diameter GFRP bars in HSC and UHSC matrices are needed to validate

the size effect reflected in the proposed model.

2. This study focused on the bond strength determined by the direct pullout test, which may not accurately represent the real stress state of FRP reinforcement in the flexural RC member. Future work is needed to extend the applicability of the proposed bond strength model to flexural structural members, with consideration of possible splitting failure by taking into account the effects of confinement conditions and concrete cover thickness.
3. Although UHSSC-wrapped FRP bars demonstrated favorable long-term performance after one year of coastal exposure, additional field durability data over longer exposure periods are necessary to reach robust conclusions and to develop reliable time-dependent degradation laws for concrete-wrapped FRP bars.
4. To better simulate real service conditions of concrete structures, future research should investigate the couple effects of environmental attacks and sustained mechanical loading on FRP bar-UHSSC members.

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