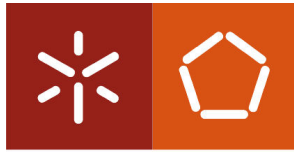




Universidade do Minho
Escola de Engenharia

Aloys Dushimimana

**Multi-scale and multi-physics investigation
on durability and long-term behavior of
concrete structures strengthened with
CFRP systems: Accelerated versus natural
ageing**



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Accelerated versus natural ageing**

Doctoral Thesis

Civil Engineering

Work conducted under supervision of

Professor Doctor José Manuel de Sena Cruz

Doctor Luís Luciano Gouveia Correia

Doctor João Miguel Pereira

April 25

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STATEMENT OF INTEGRITY

I hereby declare having conducted this academic work with integrity. I confirm that I have not used any sort of plagiarism or any form of undue use of information or falsification of results along the process leading to its elaboration.

I further declare that I have fully acknowledged the Code of Ethical Conduct of the University of Minho.

University of Minho, April 30, 2025

Aloys Dushimimana

“LOVE THE LIFE YOU LIVE.
LIVE THE LIFE YOU LOVE”

Bob Marley

ABSTRACT

All over the world, if one looks at built structures, most of them were made using concrete. In most cases, concrete is used with an embedded reinforcing material (e.g., steel) to improve its tensile strength, thus yielding to reinforced concrete (RC). The adoption of concrete dates to many centuries ago, but its current use has been a hindrance, as it emits the unwanted high levels of carbon dioxide (CO₂) into the atmosphere. Apart from this, aging, poor construction practices, need for changing the use of a structure, new regulations, among others, have led to questioning the sustainability and long-term performance of built RC structures. Rehabilitation in the form of “*strengthening systems*” can be a promising solution.

This PhD thesis aimed at addressing the durability and long-term performance of RC structures strengthened with Carbon Fiber Reinforced Polymer (CFRP) composites using non-prestressed Externally Bonded Reinforcement (EBR) and Near-Surface Mounted (NSM) techniques, and the EBR prestressed via mechanical and gradient anchorages. Multi-scale and multi-physics investigations on specimens exposed to laboratory-controlled (accelerated) and outdoor (natural) environments for up to 6 years were performed.

These are key findings for up to 6 years of aging: (i) *at material level*: concrete and CFRP laminates generally resisted against degradation; however, water diffusion through epoxy adhesives substantially decreased their tensile strengths and elastic moduli by 60-66 and 70-75 %, respectively; numerical and analytical models for the adhesive water diffusion simulated accurately the experimental results; (ii) *at bond level*: EBR and NSM generally resisted against degradation; but the latter degraded its bond strength under freeze-thaw cycling and water exposure by nearly 9 and 12 %, respectively; furthermore, numerical models for the EBR followed the experimental results; (iii) *at structural element level*: experimental data show that higher CFRP axial stiffness and presence of prestressed systems decreased the instantaneous deflection significantly, while the latter increased more the time-dependent deflections due to creep and shrinkage effects but led to total deflections much smaller compared with non-prestressed systems; analytical models for short-term flexural analysis showed an excellent agreement with experimental tests, owing to newly proposed equations; different mid-span deflection analytical models predicted well the experimental results, and time-dependent factor curves were established to predict deflections up to 50 years. Finally, environmental conversion factors were proposed at material and bond levels, being equal to 0.26 and 0.75, respectively, for the harshest environments.

KEYWORDS: Concrete, CFRP composites, Durability and long-term behavior, Epoxy adhesive, Numerical and analytical modeling

RESUMO

Em todo o mundo, quando se analisam as estruturas construídas, a maioria delas foi produzida com betão. Na maioria dos casos, no betão é incorporado um material de reforço (p.e., aço) para melhorar a resistência à tração, dando origem ao betão armado (BA). A adoção do betão remonta a muitos séculos, mas sua utilização mais recente tem sido um obstáculo, pois emite elevados níveis de CO₂ indesejado para a atmosfera. Além disso, práticas de construção inadequadas, a necessidade de alterar o uso de uma estrutura e o desenvolvimento de novas regulamentações, entre outros aspetos, têm posto em causa a sustentabilidade e o desempenho a longo prazo das estruturas em BA. A reabilitação na forma de “sistemas de reforço” pode ser uma solução promissora.

Esta tese de doutoramento teve como objetivo abordar a durabilidade e o desempenho a longo prazo de estruturas de BA reforçadas com compósitos de polímero reforçado com fibra de carbono (CFRP) utilizando a técnica de reforço externo não pré-esforçado (EBR), a técnica da inserção do reforço no betão de recobrimento (NSM), e as técnicas EBR pré-esforçadas por intermédio de ancoragens mecânicas e gradiente. Foram realizadas investigações multiescala e multifísicas em provetes/protótipos expostos a ambientes controlados em laboratório e em ambientes naturais por períodos até 6 anos.

Estas são as principais conclusões obtidas: (i) *ao nível do material*: betão e laminados de CFRP geralmente resistiram à degradação; no entanto, a difusão de água através de adesivos epóxi diminuiu substancialmente suas resistências à tração e módulos de elasticidade em 60-66 e 70-75 %, respectivamente; modelos numéricos e analíticos para a difusão de água simularam com rigor suficiente os resultados experimentais; (ii) *ao nível de ligação*: as técnicas EBR e NSM geralmente resistiram à degradação; mas no caso da NSM degradou a resistência de ligação sob exposição ao gelo-degelo e à imersão em água em quase 9 % e 12 %, respectivamente; modelos numéricos para a EBR concordaram com os resultados experimentais; (iii) *ao nível do elemento estrutural*: a maior rigidez axial do CFRP e a presença de pré-esforço, diminuíram significativamente a flecha instantânea; por sua vez o pré-esforço conduziu a maiores deformações diferidas; os modelos analíticos para análise das deformações mostraram uma excelente concordância com ensaios experimentais, devido às novas formulações propostas; os modelos de previsão da deformação até 6 anos permitiram a sua extrapolação até 50 anos. Fatores de redução ambientais foram propostos a nível do material e da ligação, com valores de 0.26 e 0.75, respetivamente, para os ambientes mais críticos.

PALAVRAS-CHAVE: Adesivo epóxi, Betão, Compostos de CFRP, Durabilidade e comportamento a longo prazo, Modelagem numérica e analítica

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LIST OF ABBREVIATIONS

AAT	Accelerated aging test
ACI	American concrete institute
ADH 1	Epoxy adhesive produced by S&P company
ADH 2	Epoxy adhesive produced by Sika company
AFRP	Aramid fiber reinforced polymers
APDL	A name of one of the outdoor environments
Avg	Average
AEMM	Age-adjusted effective modulus method
BFRP	Basalt fiber reinforced polymers
BS	Bond strength
C CS	Concrete cohesive failure combined with concrete splitting
CC	Concrete cohesive failure, Concrete crushing failure
A CS	Concrete splitting combined with adhesive failure
CC F/A FD	Mixed failure of concrete cohesive, adhesive failure, CFRP disbond
CC F/A FD FS	Mixed failure of concrete cohesive, adhesive failure, CFRP disbond, and CFRP splitting/longitudinal fracture failure
CCT	Concrete cohesive failure with a thick layer removal at the free end for small scale specimens with EBR strengthening
CDCD	Critical diagonal crack debonding
CDP	Concrete damage plasticity model
CFRP	Carbon fiber reinforced polymer
CNR	The National Research Council of Italy
CO ₂	Carbon dioxide
CTE	Coefficient of thermal expansion
D×H	Diameter × height
DIC	Digital image correlation
DMA	Dynamic mechanical analysis
E1	Indoor environmental exposure characterized by 20 °C/55 % RH conditioning
E2	Indoor environment exposure characterized by tap water immersion at 20 °C

E3	Outdoor environmental exposure characterized by mild Mediterranean climate in urban area, resulting in elevated atmospheric CO ₂
E4	Outdoor environmental exposure characterized by presence of freeze-thaws, lowest temperatures in Portugal, and high thermal amplitudes
E5	Outdoor environmental exposure characterized by warm temperature climate, very high thermal amplitudes, and highest temperatures in Portugal
E6	Outdoor environmental exposure characterized by maritime moist environment-Atlantic Ocean, with high salinity, low levels of humidity and airborne chlorides
EBR	Externally bonded reinforcement
EBR CFRP	Externally bonded carbon fiber reinforced polymer
EBR-GA	Externally bonded carbon fiber reinforced polymer prestressed via gradient anchorage system
EBR-MA	Externally bonded carbon fiber reinforced polymer prestressed via mechanical anchorage system
ECF	Environmental conversion factor
ED	End debonding
EDP	Energias de Portugal, a name of one of the outdoor environments
EMM	Effective modulus method
Ew	External work
F/A	Adhesive failure
F/A CC	Adhesive failure combined with concrete cohesive failure
F/A CS	Adhesive failure combined with concrete splitting failure
FD	Failure of the CFRP in the form of superficial molecular disbond
FE	Finite element
FEM	Finite element method
FIB	International Federation for Structural Concrete
FM	Failure mode
FR	Failure due to FRP rupture
FRP	Fiber reinforced polymer
FRPLongDur	The name of the project on which the present thesis is based
FS	Failure due to FRP splitting/fracture in its longitudinal direction

FT	Freeze – thaws
Ft/A	Failure due to both adhesive and the FRP
Ft/A CS	A mixture of CFRP-Adhesive and concrete splitting failures
FTIR	Fourier Transform Infrared Spectroscopy
GA	Gradient anchorage
GFRP	Glass fiber reinforced polymer
GK	Kelvin chain
ICD	Intermediate crack debonding
IE	Internal energy
KE	Kinetic energy
L×W×H	Length×width×height
L10	CFRP laminate with 10×1.4 cross section [in mm ²]
L50	CFRP laminate with 15×1.4 cross section [in mm ²]
LL	Load level
LTNEC	National Laboratory of Civil Engineering
LVDT	Linear Variable Differential Transformer
LVDT1	Linear Variable Differential Transformer 1
LVDT2	Linear Variable Differential Transformer 2
MA	Mechanical anchorage
Max	Maximum
Min	Minimum
MTS	Material Testing System
N/A	Not Available
NA	Neutral axis
NAT	Natural aging test
NSM	Near-surface mounted system
NSM CFRP	Near-surface mounted carbon fiber reinforced polymer
OPC	Ordinary Portland cement
RC	Reinforced concrete
REF	Reference
RH	Relative humidity
S&P	Name of a company producing the adhesives and CFRP composites

SL_EBR_T0	Reinforced concrete slab strengthened with externally bonded reinforcement before exposure condition
SL_GA_T0	Reinforced concrete slab strengthened with externally bonded reinforcement that is prestressed via gradient anchorage system before exposure condition
SL_MA_T0	Reinforced concrete slab strengthened with externally bonded reinforcement that is prestressed via mechanical anchorage system before exposure condition
SL_NSM_T0	Reinforced concrete slab strengthened with CFRP laminate applied using near-surface technique before exposure condition
SL_REF_T0	Reinforced concrete slab without strengthening system before exposure condition
T0	Initial time before exposure condition (i.e., Year 0)
T1	Exposure duration of up to 1 Year
T2	Exposure duration of up to 2 Years
T3	Exposure duration of up to 3 Years
T4	Exposure duration of up to 4 Years
T6	Exposure duration of up to 6 Years
T_g	Glass transition temperature
UMinho	University of Minho
UTM	Universal testing machine
UV	Ultraviolet radiation

CHAPTER 1

INTRODUCTION

1.1. MOTIVATION

Concrete refers to a *material* made from a carefully mixed combination of water, aggregates, sand, cement, and other cementitious materials or components, such as additives and superplasticizers [1,2]. Although often simply referred to as a *material*, concrete can be more accurately described as a *composite material* because of its combination of various elements, as described above. While concrete typically exhibits medium to high compressive strength, its tensile strength is relatively low. Therefore, concrete is often reinforced with materials that possess excellent tensile properties, such as steel or fiber-reinforced polymers, in the form of rebars, to form what is known as *reinforced concrete* (RC). Concrete has been used in various forms for thousands of years, with ancient examples dating back to civilizations like the Romans, while the RC (commonly attributed to the 1854 patent by William B. Wilkinson [3]) has been widely used in the construction industry since the 19th century. In this PhD thesis, the terms *concrete* and *RC* will be used interchangeably.

Reinforced concrete is one of the most widely used materials for building structures worldwide. Many structures built using RC have proven to be durable and thus have stood the test of time; examples such

as the Alvord Lake Bridge (built in 1889 by Ernest L. RunSome) [4,5] or the 11-span flat slab RC bridge (built in 1914-1916 [6]), among others, serves as evidence of the durability of the RC. This demonstrates the reliability of concrete as a construction material, as also evidenced by its widespread adoption and acceptance. However, RC structures do not last indefinitely because they naturally age over time and their properties can degrade when exposed to harsh environments; particularly when degradation agents cause corrosion of steel rebars. Numerous infrastructures worldwide are aging, requiring the implementation of sustainable strategies to ensure their continued viability and longevity.

The construction of new RC structures significantly contributes to environmental issues, such as increase of carbon dioxide (CO_2) emission from Ordinary Portland Cement (OPC) production. The production of large amounts of OPC is required for making concrete; however, this production process emits about 8 % of the world's human-induced CO_2 [7], contributing heavily to global warming and climate change. That is, between 1990 and 2017, global CO_2 emissions from cement manufacture increased by 184 % [8]; and the global total emission from cement production increased from 157 megatons of carbon (MtC) in 1990 to 568 MtC in 2017 [8]. According to the UN environment [9], the annual global OPC production increased sharply between 1950 and 2023, with a rate much higher than other building materials such as asphalt, lime, wood, crushed stone, crude steel, and aluminum. To reduce the environmental impact of the construction industry, alternative solutions such as reusing, strengthening, or repairing existing structures, must be explored. With these solutions, the OPC production cycle can be delayed, which significantly contributes to reducing the CO_2 emission and energy consumption, among others.

When comparing the cost of demolishing and rebuilding RC structures to that of strengthening them, the latter is typically more economical. Moreover, the global construction industry reached a value of EUR 10 trillion in 2017 [10], with projections estimating an increase to EUR 16.2 trillion by 2030 [11], indicating that sustainable construction strategies are essential to keeping construction costs low and for low environmental impact. Existing buildings and bridges can be strengthened (rather than demolished) to extend their service life. Some structures, particularly those with significant social value, need to be preserved even if they were built long ago. This includes historical monuments, structures with tourist attractions, churches, cultural centers, residential buildings, and bridges/buildings of historical or aesthetic value, among others. Additionally, if there are issues such as poor construction practices or a need to change the use of a structure, strengthening becomes the most environmentally friendly and cost-effective solution. RC structures can be strengthened with specific materials to improve sustainability, structural integrity and durability. One prime example of strengthening solution is the use of fiber-reinforced polymer

(FRP) materials. These materials have been successfully applied in the past and are considered state-of-the-art in civil engineering [12].

FRP composites are ideal for strengthening reinforced concrete (RC) structures due to their exceptional properties, including high strength-to-weight ratio and corrosion resistance, among others [12–14]. In fact, unlike traditional materials such as steel, FRP composites are almost immune to corrosion, making them highly durable under harsh environmental conditions. Their lightweight nature also allows for easier and faster installation with minimal disruption to the structure's existing load, thereby reducing labour costs and downtime. Additionally, FRP composites can be tailored to meet specific design requirements, thereby allowing designers to introduce different desired (including complex ones) shapes that are usually difficult to achieve when using other materials. These advantages make FRP a state-of-the-art solution for enhancing the performance and durability of aging or underperforming RC structures.

The use of FRP composites in the construction industry has significantly increased over time, primarily due to their successful application as a strengthening solution for various structural components. This growing acceptance has led to the development and release of numerous guidelines and code provisions globally, including, for example, the American Concrete Institute (ACI) [15], the Italian National Research Council (CNR) [16], the *fib*90 [17], or from the European Committee for Standardization (CEN) [18]. These advancements underscore the industry's commitment to integrating innovative materials and methods to enhance structural performance and safety. In particular, the existing guidelines consider the durability and long-term performance aspects by introducing environmental conversion factors (ECF), which are used to quantify the environmental effects associated with various degradation agents. The ECF is expressed in terms of coefficients that typically convert a physical quantity into an equivalent environmental impact measure.

Four main types of FRP materials are commonly used in construction applications: glass FRP (GFRP), basalt FRP (BFRP), aramid FRP (AFRP), and carbon FRP (CFRP) composites. Each type has its own distinct characteristics and advantages, but CFRP stands out due to its superior mechanical properties, including higher stiffness, greater tensile strength, enhanced fatigue resistance, and longer lifespan [17]. These qualities make CFRP particularly suitable for demanding structural applications where durability and performance are critical. Additionally, the greater resistance of CFRP to aggressive environmental conditions, such as chemical exposure and temperature variations, further enhances its appeal, making it more promising and preferred over other FRP types for a wide range of civil engineering projects, particularly in strengthening systems [16,19–21].

CFRP composites have been widely adopted for strengthening RC structures, with great acceptance in the standards and code provisions. Despite this, the durability and long-term performance of these composites, including the bond between the concrete substrate and the applied CFRP strip/laminate have not yet been fully understood. The existing literature primarily presents data on the short-term performance of the bond-constituent materials (epoxy adhesive, CFRP laminate), while there is insufficient data on the bond performance of strengthened RC elements after many years in service. Additionally, the existing short-term data have been generally obtained using accelerated aging test protocols in laboratory environments, which may not accurately reflect the conditions in natural (outdoor) aging environments where RC structures are typically built. Consequently, these gaps in knowledge formed the basis for the motivation to conduct this PhD thesis.

1.2. FRPLongDur RESEARCH PROJECT

To contribute toward finding solutions to the knowledge gaps described in **Section 1.1**, FRPLongDur research project POCI-01-0145-FEDER-016900 (FCT PTDC/ECM-EST/1282/2014) [22] was initiated in 2016. The primary aim of the project is to conduct investigations (for up to 10 years) on the durability and long-term performance of RC structures strengthened with CFRP systems under accelerated and natural aging conditions. A comparative analysis between these two aging conditions (together with other related works already carried out in the existing literature) is expected to lead to appropriate definitions of the ECF. This PhD thesis was developed based on the FRPLongDur project; therefore, a detailed explanation of the project is provided in **Section 1.2.1**.

1.2.1. Preparation and initial testing of specimens for the FRPLongDur project

Three different classifications of the specimens involved in the FRPLongDur project were made: (i) specimens at *material level*; (ii) specimens at *bond level*; and (iii) specimens at *structural element level*. Further explanations about these classifications are provided in **Sections 1.2.1.1 - 1.2.1.3**.

1.2.1.1 Specimens at the material level

Three different groups of specimens were prepared: (i) concrete cylinders (**Figure 1.1a,b**), (ii) two types of CFRP laminates (denoted as L10 and L50, **Figure 1.1c,d**), and (iii) two types of epoxy adhesives (denoted as ADH1 and ADH2, **Figure 1.1e,f**).

The concrete used to produce the cylinders, and remaining specimens involving concrete material, had the following characteristics: a maximum aggregate size of 12.5 mm, cement type of CEM II/A-L 42.5R, water-

to-cement ratio of 0.4, slump ranging between 160 and 210 mm (i.e., slump class S4), exposure class XC4(P), and concrete class of 30/37 MPa (C30/37).

The laminate L10 had a thickness of 1.4 mm and width of 10 mm, while L50 had a thickness of 1.2 mm and width of 15 mm; and both laminates had a length of 250 mm as per EN ISO 527 5:2012 [23]; also, both laminates were produced by the S&P company [24].

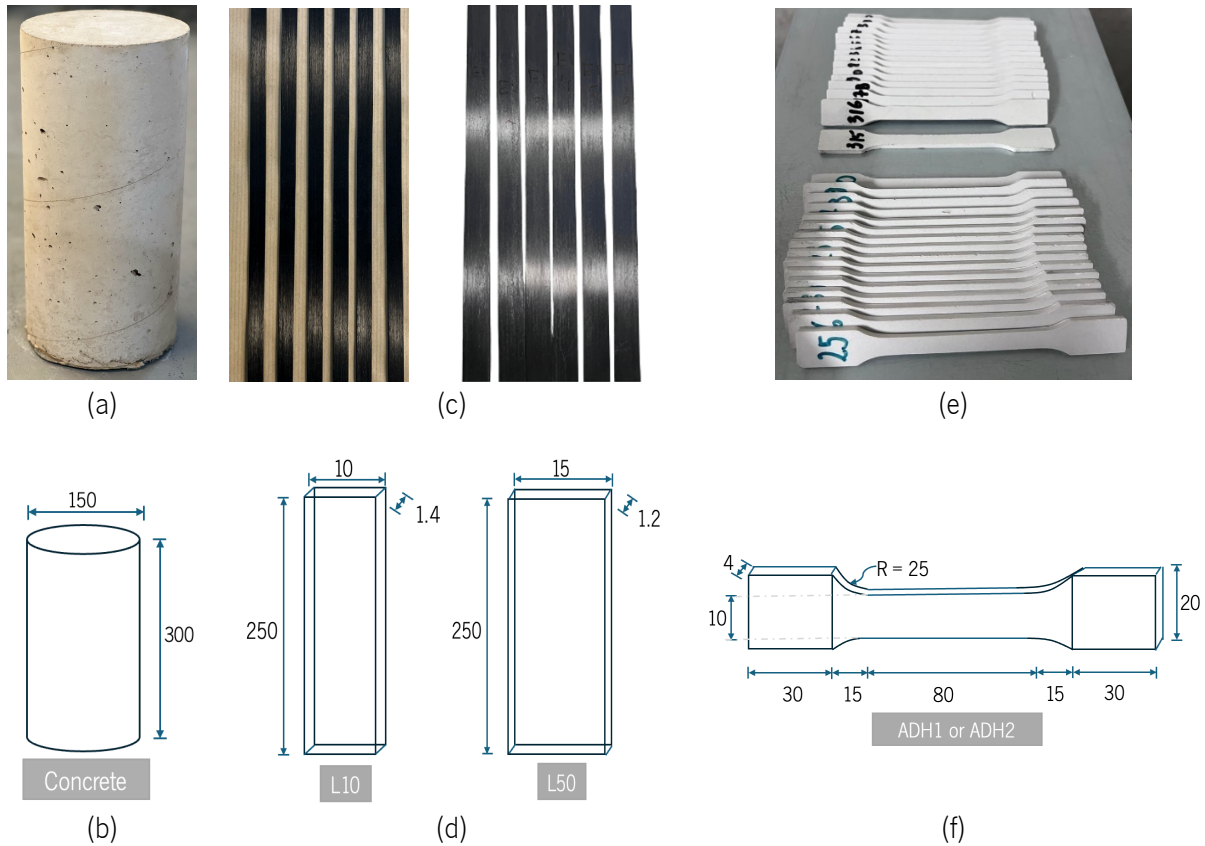


Figure 1.1: Materials used in the FRPLongDur project: typical (a) concrete cylinder, (b) CFRP laminate, and (c) epoxy adhesive specimens; dimensions and geometry of typical (d) concrete cylinder, (e) CFRP laminates, and (f) epoxy adhesives. Note: All dimensions in [mm].

Furthermore, ADH1 and ADH2 had similar geometry (dog-bone shaped as per EN ISO 527 2:2012 [22]) but different properties as the former was produced by S&P Company [24] and the latter by SIKA Company [25]. Additional details on the production and preparation of all the materials is given in [22].

A total of 130 concrete cylinders, 350 ADH1, 350 ADH2, 255 L10, and 135 L50 (each with 1.4x50 [mm] section area) specimens were prepared. All these specimens were categorized as *small-scale* specimens.

Initial tests performed on small-scale specimens

A total of 3 concrete cylinders, 5 ADH1, 5 ADH2, 6 L10 and 6 L50 were initially tested to characterize their properties before exposure to different environments. The results from these initial tests were considered as reference values (referred to as T0 throughout the thesis) and were compared with the results obtained in later tests after exposure. Concrete tests included compression tests to determine the initial compressive strength and elastic modulus, and pull-off tests to determine the initial concrete pull-off strength. Furthermore, both epoxy adhesives and CFRP laminates were tested under tension to determine their initial tensile strength and elastic modulus properties.

A proportion of the epoxy adhesives (3 ADH1 and 3 ADH2 specimens) and CFRP laminates (6 L10 and 6 L50 specimens) was additionally submitted to physical and chemical characterizations using the following methods: water absorption capacity was evaluated through gravimetric analysis over a period of up to 10,000 hours, by submerging the CFRP specimens in tap water at 20 °C and the adhesive specimens in tap water at three distinct temperatures: 20 °C, 40 °C, and 60°C. Test specimens were periodically taken out of the water; after gently wiping off surface moisture, they were promptly weighed and then placed back into their containers to continue the water absorption process. The chemical composition of the epoxy adhesives, cured at 23 °C for seven days, was analyzed using Fourier transform infrared spectroscopy (FTIR). Furthermore, dynamic mechanical analysis (DMA) was conducted on the adhesive specimens (cured at 23 °C for seven days) to assess their viscoelastic behavior and determine their glass transition temperature (T_g).

1.2.1.2 Specimens at the bond level

Two different types of joint/bond between a concrete substrate and a CFRP laminate were used: (i) a joint prepared using externally bonded reinforcement (EBR) technique (**Figure 1.2a**), and a joint prepared using near-surface mounted (NSM) technique (**Figure 1.2b**). Depending on the context, the terms “*joint*” and “*bond*” are used interchangeably in this PhD thesis. The concrete prisms and cubes used for the EBR and NSM specimens, respectively, were prepared using concrete with the same properties as those used to produce the concrete cylinders described in **Section 1.2.1.1**. An epoxy adhesive layer with the properties of ADH1 (previously described in **Section 1.2.1.1**) served as the bonding agent in both the EBR and NSM joints, and the bonded CFRP laminates were L50 and L10 (described in **Section 1.2.1.1**) for the EBR and NSM specimens, respectively (it is important to note that the L50 used herein had a larger width of 50 mm). A total of 90 NSM and 90 EBR specimens were prepared. Like the specimens at the material level, the EBR and NSM specimens were categorized as *small-scale* specimens.

During the preparation of the EBR/NSM CFRP-to-concrete joint, the following steps were followed:

- 1) Preparing the concrete substrate for the EBR through (i) sandblasting and (ii) cleaning the surface
- 2) Preparing the concrete substrate for the NSM through (i) creating a groove into concrete and (ii) cleaning the groove to remove loose particles.
- 3) The epoxy adhesive, prepared according to the supplier's recommendation, was poured onto the prepared concrete substrate for the EBR specimen (**Figure 1.2a**) and into the concrete groove for the NSM specimen (**Figure 1.2b**).
- 4) The CFRP laminate was carefully bonded on top of the poured epoxy adhesive for the EBR technique (**Figure 1.2a**), while the laminate was carefully inserted and aligned into the groove followed by pouring and filling the groove with the epoxy adhesive for the NSM technique (**Figure 1.2b**)

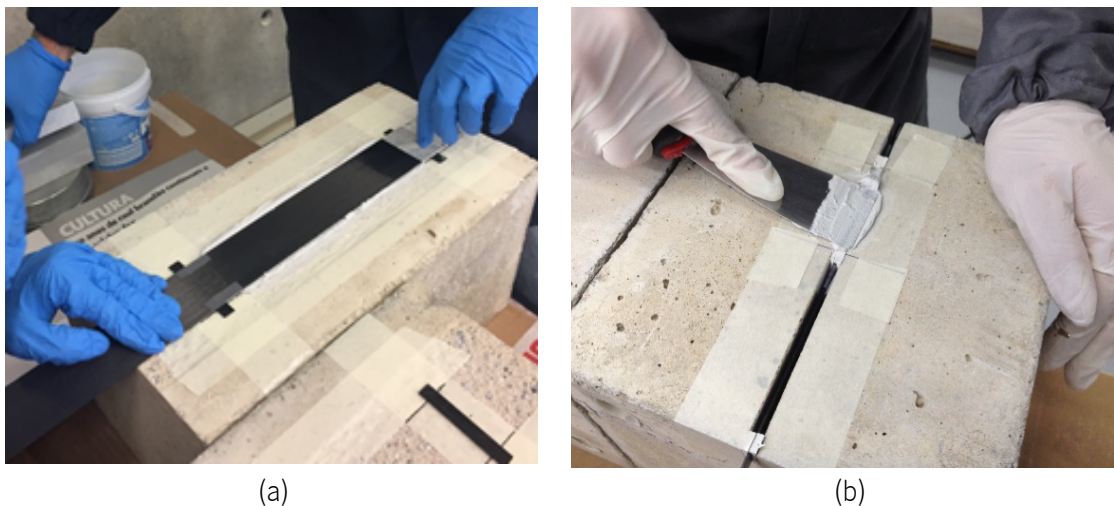


Figure 1.2: Specimen geometries and dimensions: (a) EBR specimen with EBR CFRP-to-concrete joint, (b) NSM specimen with NSM CFRP-to-concrete joint [25].

The geometry and dimensions of the EBR and NSM specimens are described in the following section, which mainly describes the initial tests conducted on the mentioned specimens.

Initial pull-out tests on small-scale EBR and NSM specimens

A total of 2 EBR and 2 NSM specimens were tested under single-lap shear tests (2 tests per each specimen) at the beginning of the FRPLongDur project [22], and the obtained results were considered as reference values (referred to as T0 throughout the thesis), which were compared later with the results obtained after exposing similar specimens to different environments. The specimen geometry and test setup for both the EBR and NSM specimens are shown in **Figure 1.3**. Further details about these tests can be found in [22].

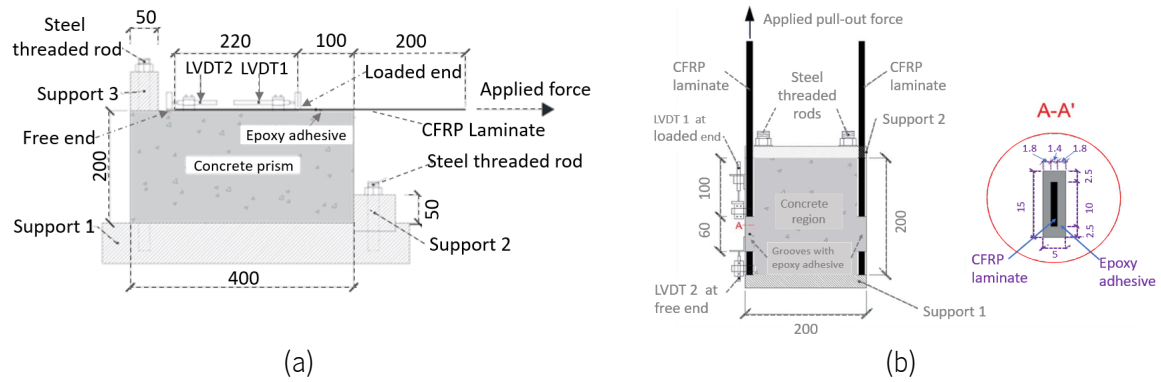


Figure 1.3: Specimen geometry and dimensions: (a) EBR and (b) NSM techniques. Note: all dimensions in [mm] (adapted from works previously published by the author [14,27]).

1.2.1.3 Specimens at the structural element level

Four different types of RC slabs with EBR, NSM, and EBR prestressed via mechanical anchorage—MA (denoted as EBR-MA) and EBR prestressed via gradient anchorage—GA (denoted as EBR-MA) systems were prepared as shown in **Figures 1.4a–d**, respectively.

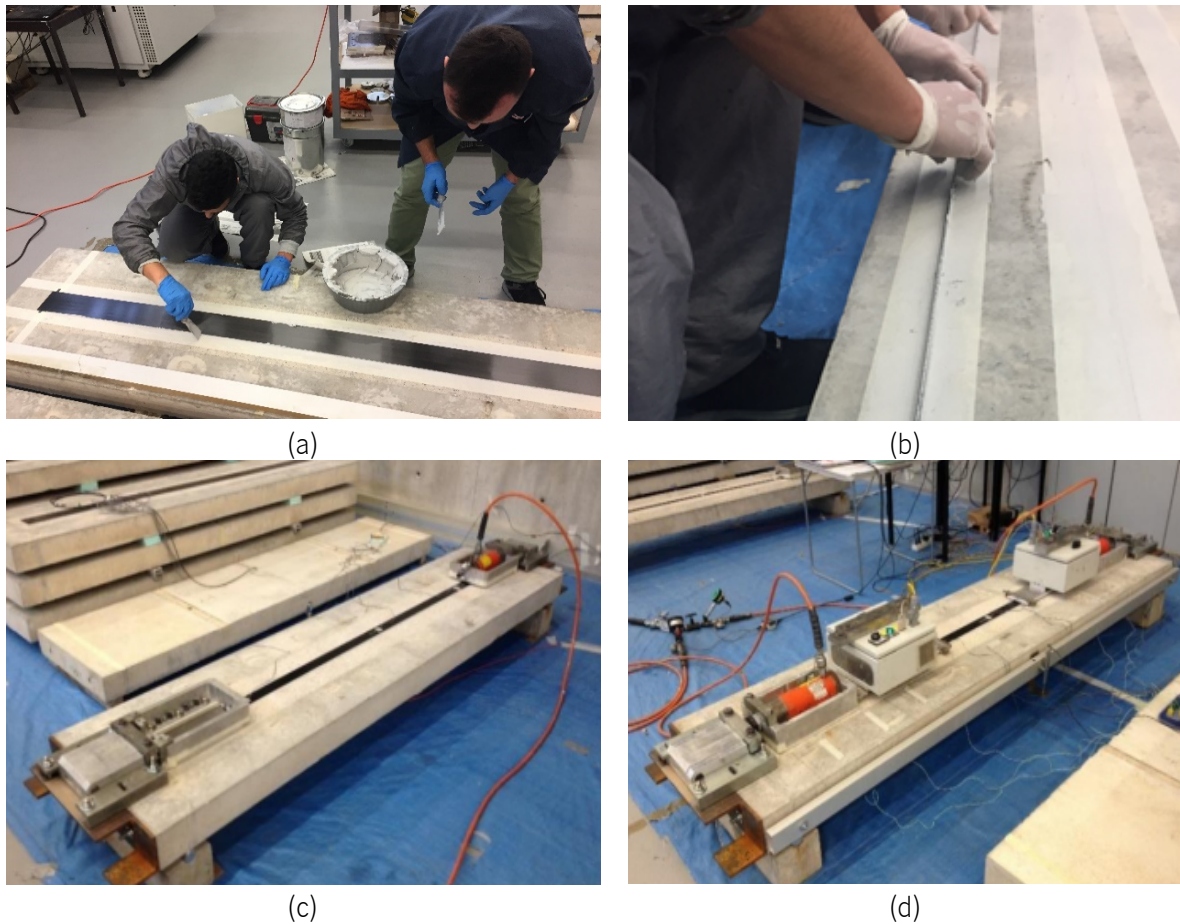


Figure 1.4: Strengthening techniques: (a) EBR, (b) NSM, (c) EBR-MA, and (d) EBR-GA systems (adapted from previous works [22,26]).

The properties of the materials used to produce these slabs were as previously described in **Section 1.2.1.1** (it is worth noting that adhesive layer with properties of the ADH1 was used in all slabs). **Table 1.1** lists the detailed properties of these slabs. Additionally, a reference RC slab without a strengthening system was also included in the study for comparison. This slab had similar properties as those for the RC slabs presented in **Table 1.1**, excluding those related to the CFRP laminate strip and anchorage systems. For the RC slabs with EBR and NSM (**Figure 1.4a,b**), the preparation steps and bonding procedures were like those for the small-scale specimens in **Section 1.2.1.2**. The materials and bonding procedures for the RC slabs strengthened with EBR prestressed CFRP systems (i.e., EBR-MA and EBR-GA) are depicted in **Figure 1.5**, where the required materials are highlighted using numerals from 1 to 10 and the preparation methods are listed using uppercase letters A to J.

Table 1.1: Types and properties of RC slabs with CFRP strengthening systems.

SLAB	CFRP LS [mm ²]	ANCHORAGE TYPE	ε_{fp} [%]	E_s [GPa]		f_y [MPa]		f_t [MPa]		E_c [GPa]	f_c [MPa]
				$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$		
EBR	100×1.2	–	–	219.9 (1.6 %)	209.3 (7.7 %)	543.9 (1.8 %)	551.2 (0.8 %)	670.0 (0.9 %)	635.4 (0.5 %)	187.2 (0.9 %)	2895.2 (4.4 %)
NSM	4x (10×1.4)	–	–	241.4 (5.3 %)	228.1 (3.2 %)	528.2 (0.7 %)	581.3 (0.6 %)	687.1 (0.7 %)	698.3 (0.3 %)	185.0 (1.5 %)	2858.2 (2.0 %)
EBR-MA	50×1.2	mechanical	0.4	219.9 (1.6 %)	209.3 (7.7 %)	543.9 (1.8 %)	551.2 (0.8 %)	670.0 (0.9 %)	635.4 (0.5 %)	184.3 (2.9 %)	2980.3 (4.1 %)
EBR-GA	50×1.2	gradient	0.4								

Notes:

- The values between parentheses are the corresponding coefficients of variation (CoV).
- ε_{fp} is the pre-strain applied to the prestressed slabs, and
- E_s is the modulus of elasticity of the steel rebar
- $\Phi 8$: diameter of longitudinal tensile bottom reinforcement rebars
- f_y is the yielding stress of the steel rebar
- f_t is the ultimate strength of the steel rebar
- E_c is the modulus of elasticity of the CFRP laminates:
- f_c is the ultimate strength of the CFRP laminates
- $\Phi 6$: diameter of longitudinal top reinforcement rebars
- LS: laminate strip

In addition to the data provided in **Table 1.1**, it should be noted that according to the supplier [24], the ADH1 used in the slabs attains a flexural elastic modulus higher than 7.1 GPa. Furthermore, the CFRP laminate strips contain 68 % of fiber content held together by a vinyl ester resin matrix [24]. The length of each CFRP laminate strip is 2200 mm. For internal steel reinforcement, the rebars shown in **Table 1.1** fall in the class A400 SD as per Eurocode 2 [17]. Each of the slabs had these dimensions: 600 mm × 120 mm × 2600 mm. The shear span, loading span and span length for each slab were 900 mm, 600 mm, and 2400 mm, respectively. Other characteristics of the slabs and their constituent materials were provided in the FRPLongDur project report [22]. A total of 30 RC slabs were produced. These slabs were categorized as *full-scale* specimens.

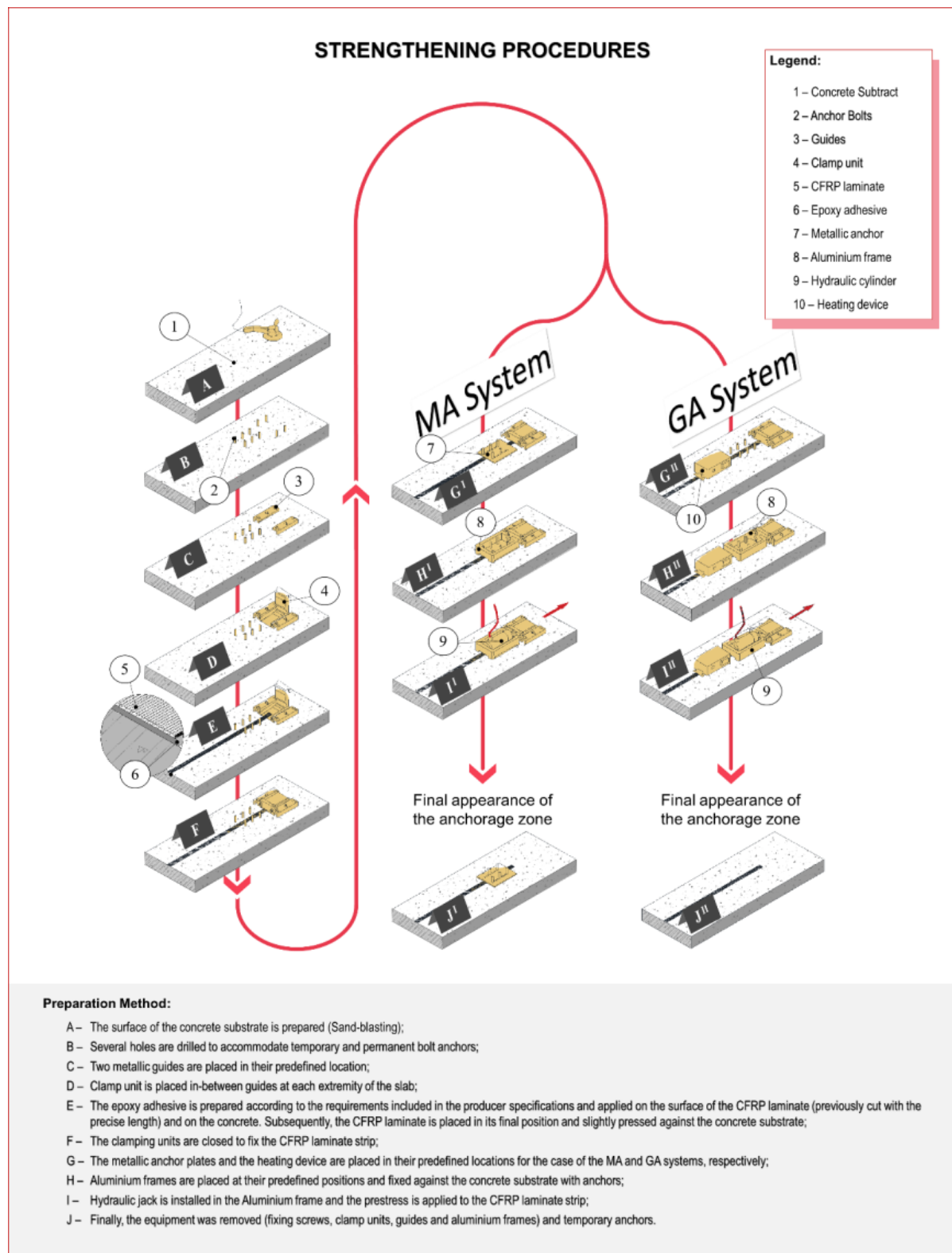


Figure 1.5: Strengthening procedures for RC slabs with EBR-MA (denoted as MA system) and EBR-GA (denoted as GA system) strengthening.

Further details on the geometry and dimensions of each of the mentioned RC slabs are presented in the following section, along with a relevant detailed description of the initial tests.

Initial flexural tests on full-scale RC slabs

The previously described RC slabs were initially tested under 4-point configuration (**Figure 1.6a**) to assess their flexural performance. Five linear variable differential transformer (LVDT) and strain gauges (SG) were used during the tests to measure important parameters as shown in **Figure 1.6a**. The cross-sections and dimensions of the slabs are shown in **Figure 1.6b-d**. The parameters recorded during these tests were: strains (in concrete, steel, and CFRP laminate) and force *versus* mid-span deflection. The efficiency and ductility parameters were obtained by post-processing the experimental data. The results from these tests served as reference values (referred to as T0 throughout the thesis) and were further compared with results obtained later after performing relevant analytical models.

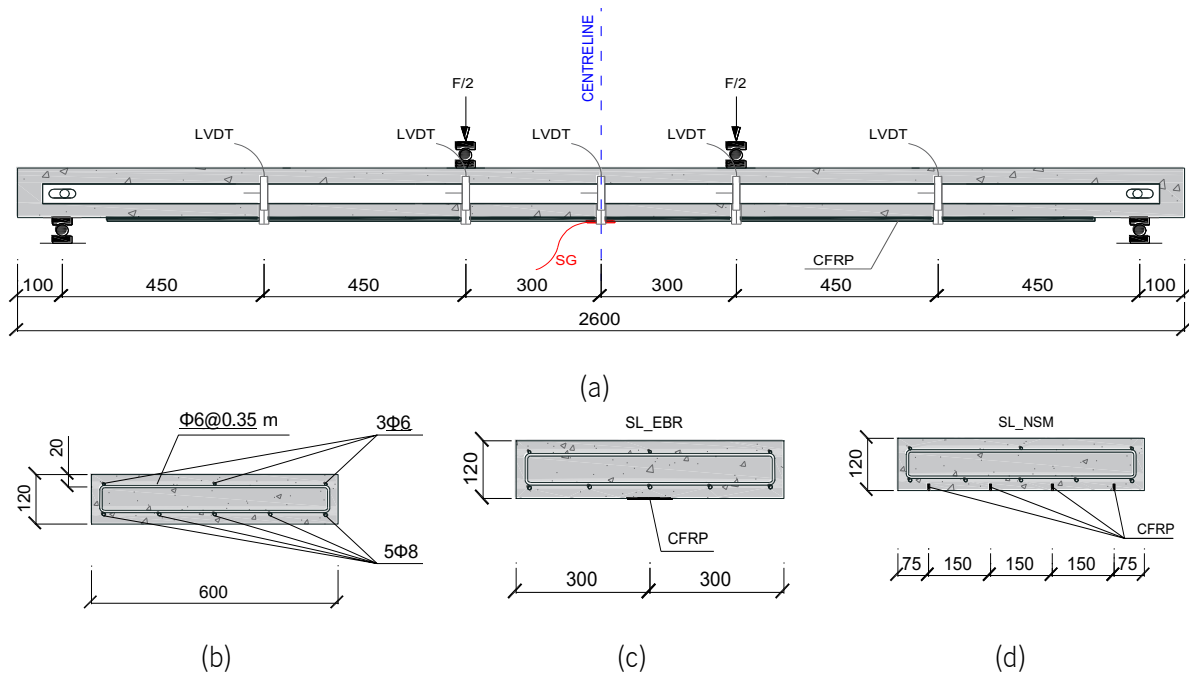


Figure 1.6: Geometry and dimensions of RC slabs (a) full-scale of typical slab with instrumentation, (b) reference slab without any strengthening system, (c) typical slab with EBR, EBR-MA and EBR-GA systems; (d) slab with NSM. Note: all dimensions in [mm] [22,26].

1.2.2. Environmental exposures used in the FRPLongDur project

Following the initial tests on both small-scale and full-scale specimens, as outlined in **Section 1.2.1**, a portion of the remaining specimens was stored in controlled laboratory conditions, while the rest were transported and exposed to different natural (outdoor) environments to assess the effects of various degradation agents. This section outlines the specifics of these environmental conditions.

Specimens at the material, bond, and structural element levels were exposed to different environments, labelled E1 to E6 in **Figure 1.7**, to evaluate how these conditions influence their properties over time. The

geographical distribution of these environments is shown on a Portugal map (Figure 1.7). Detailed information on the environments is also presented in Table 1.2.

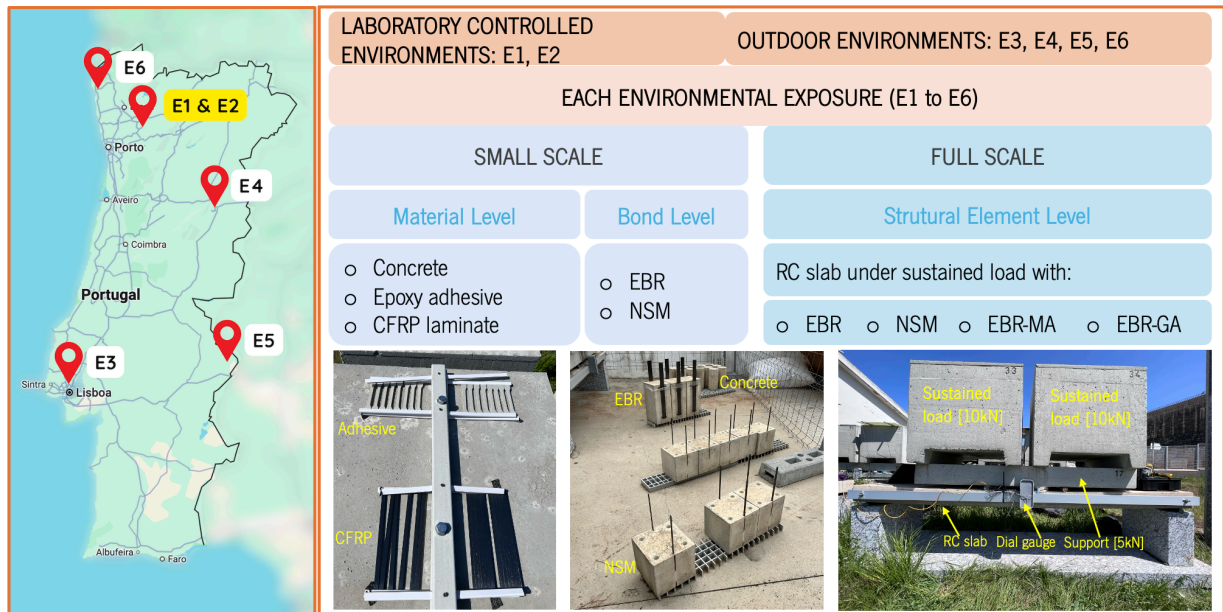


Figure 1.7: Environmental exposures, scales, and typical specimens used in each environment.

Table 1.2: Environmental exposures used in the FRPLongDur project [12].

Environment	Location	Description
E1	University of Minho, Guimarães (41°27'11.5" N 8°17'26.8" W)	Controlled hygrothermal conditions, 20 °C and 50 % relative humidity (climatic chamber)
E2	University of Minho, Guimarães (41°27'08.1" N 8°17'33.8" W)	Controlled hygrothermal conditions, 20 °C and 100 % relative humidity (tap water immersion)
E3	National Laboratory of Civil Engineering, Lisbon (38°45'41.7" N 9°08'30.6" W)	Mild subtropical Mediterranean climate with short and mild winters and warm to hot summers; high levels of CO ₂
E4	Lagoa Comprida Dam (EDP), Seia (40°21'55.8" N 7°38'52.0" W)	Mild subtropical Mediterranean climate with low temperatures, snow during winter, and warm to hot summer
E5	S&P Clever and Reinforcement, Elvas (38°53'33.5" N 7°08'46.0" W)	Hot summer months in Mediterranean climates with high temperatures and droughts, especially during summer
E6	Port of Viana do Castelo (APDL), V. do Castelo (41°40'57.0" N 8°49'28.3" W)	Mild subtropical Mediterranean climates with short and mild winters and warm to hot summers; high chloride concentrations and relative humidity

Both small-scale (i.e., at material and bond levels, see Figure 1.7) and full-scale (i.e., at structural element level, see Figure 1.7) specimens were placed in two different laboratory-controlled environments (E1 where the specimens were conditioned at 20 °C temperature and 55 % relative humidity (RH), and E2 where the specimens were fully immersed in tap water controlled at a temperature of 20 °C). Similar specimens were placed in each of the four outdoor environments to mainly promote aging due to exposure to carbonation (denoted as E3), freeze-thaw attacks (E4), elevated temperatures (E5), and airborne chlorides from the

ocean (E6). The small-scale specimens are highlighted in **Figure 1.7** as follows: concrete cylinders (denoted as concrete), epoxy adhesive, CFRP laminate, and specimens with CFRP-to-concrete joints (denoted as EBR and NSM). The full-scale specimens were 4 RC slabs in each environment, strengthened using different techniques (**Table 1.1**) as previously described in **Section 1.2.1.3**, and each slab was continuously loaded with a sustained load of around 40 % of the failure load (a typical slab with the sustained load is shown in **Figure 1.7**). The evolution of concrete strains, CFRP strains, concrete cracks, and mid-span deflection of these RC slabs were monitored annually for up to 6 years of environmental exposures.

It is worth noting that each of the selected outdoor environments represents the exposure of specimens to more than a single degradation agent because outdoor environments are uncontrollable. Nevertheless, these environments were strategically selected based on the expected dominance of a single degradation agent. Hence, although it is believed that synergistic effects took place in each environment, these effects might have been dominated by the expected/targeted dominant agent

1.3. OBJECTIVES AND RESEARCH METHODOLOGIES

Section 1.2 established the groundwork for this PhD thesis by presenting the specimens, environmental conditions, and exposure scenarios that form the basis for the subsequent detailed investigations. In other words, the present work embarks on an in-depth exploration of the specimens described in **Section 1.2**, mainly addressing the durability and long-term behavior of both the small-scale and full-scale specimens illustrated in **Figure 1.7**. This section presents the journey undertaken to address the above aspects, describing the carefully crafted objectives and the involved methodologies.

1.3.1. Objectives

This research aimed to perform multi-scale and multi-physics investigations on the durability and long-term performance of concrete structures strengthened with CFRP systems, considering both accelerated and natural aging conditions. The term “*multi-scale*” was related to the use of small-scale and full-scale specimens (described in **Section 1.2**); the term “*multi-physics*” was related to the state of addressing a simultaneous physical and mechanical behavior of the studied strengthening systems. Furthermore, the term “*accelerated aging*” referred to the artificial speeding up of the degradation of specimens using extreme conditions, while the term “*natural aging*” referred to the gradual degradation of the specimens under real environmental conditions. To achieve the above-defined aim, key research questions were first established, guiding the formulation of specific objectives.

The research questions were as follows:

- How are the properties of strengthening materials affected by long-term environmental exposure?
- How does the bond between the concrete substrate and strengthening system respond to different degradation agents over extended exposure periods?
- How does the aging process influence the durability and long-term performance of strengthened RC slabs?
- What correlations do exist between natural and accelerated aging?
- How reliable are the environmental conversion factors provided by different standards?

Based on the research questions, the specific objectives and scope for each objective were established as follows:

- To investigate the durability of epoxy adhesives and CFRP laminates typically used for strengthening RC structures under both accelerated and natural aging conditions. This included experimental studies on epoxy adhesives and CFRP laminates and analytical and numerical simulations of water diffusion through epoxy adhesives.
- To examine the durability of the bond between concrete and CFRP laminate (using the epoxy adhesive as a bonding agent) under accelerated and natural aging conditions. This included experimental works on small-scale specimens with externally bonded reinforcement (EBR) and near-surface mounted (NSM) techniques.
- To numerically model the short- and long-term behaviors of the bond between concrete and CFRP laminate. This included short-term modeling of the CFRP-to-concrete bond behavior for the small-scale specimens using the EBR technique, followed by long-term modeling of the bond considering water diffusion effects over time (also EBR technique).
- To assess the short- and long-term performance of RC slabs strengthened with CFRP systems. This included the development of analytical models for the short-term flexural behavior of the slabs, long-term monitoring works (on the slabs with sustained load), and analytical models for predicting the instantaneous and long-term mid-span deflections of the slabs.
- To examine correlations between accelerated and natural aging conditions and provide design recommendations. This included examining the correlations at both *material* and *bond* levels using

the data obtained from the FRPLongDur project (up to 4 years) and those from the existing database, which led to proposing new environmental conversion factors.

To address the highlighted objectives, collaboration among multiple institutions was essential, providing access to laboratory facilities for both accelerated and natural aging tests, and supporting the experimental testing program. This collaborative framework had already been established through the FRPLongDur research project described in **Section 1.2**, which allowed for a seamless continuation of the work with minimal obstacles. A previous work [26] had already addressed the first two years of the above-described project, focusing on experimental work. In the present PhD thesis, significant contributions were made to the current body of knowledge by performing investigations for up to four more years on the durability and long-term behavior of concrete structures strengthened with CFRP systems, through testing of specimens at both small- and full-scale and comparing the obtained results with those from the previous work [26] and from existing literature, along with extensive numerical and analytical simulations. The methodologies employed to make these contributions are described in the following section (**Section 1.3.2**).

1.3.2. Research methodologies

To achieve the objectives defined in **Section 1.3.1**, specimens at small- and full-scale, as well as accelerated and natural aging environments, were the first necessity; however, these were predefined in the FRPLongDur research project (see **Section 1.2**). Therefore, additional tasks were undertaken at the initial stage, including a strategic review of the existing literature, which was conducted promptly and continued throughout the development of the PhD thesis.

This review primarily covered the following: (i) literature on the strengthening of concrete structures using CFRP laminates; (ii) the durability of each bond constituent material, as well as the durability of the bond between concrete and CFRP laminate; (iii) existing numerical and analytical models used to simulate the long-term behavior of the joint between concrete and CFRP laminate, as well as those used to model the global performance of prestressed or non-prestressed strengthened elements; (iv) experimental methodologies for testing prestressed/non-prestressed RC elements strengthened with CFRP systems; and (v) literature on accelerated and natural aging test protocols. This comprehensive review was instrumental in achieving the predefined objectives. The methods used for each specific objective are described in the next section.

1.3.2.1 Durability of the CFRP-to-concrete bond constituent materials

The durability of materials constituting the CFRP-to-concrete joint was investigated by performing different experimental tests on concrete, epoxy adhesive, and CFRP laminate, and by simulation of water diffusion through the epoxy adhesive. The key characteristics of these materials were previously provided in detail in **Section 1.2**. The following activities were performed:

- (i) Collection of small-scale specimens from different environments (described in detail in **Section 1.2.2**) after 3 and 4 years of exposure;
- (ii) Experimental testing on the collected specimens for mechanical characterization as shown in **Figure 1.8**. The testing program included concrete compression, concrete tensile (pull-off), and concrete carbonation tests. Epoxy adhesives and CFRP laminates were characterized using tensile tests. Further details on these tests, including the properties investigated for each material are provided in **Table 1.3**;
- (iii) Numerical and analytical simulations of water diffusion through the epoxy adhesive, following the workflow presented in **Figure 1.9**. The numerical simulation employed the mass diffusion analysis available in ABAQUS software, where the moisture diffusion problem was reformulated as a heat-flux problem to facilitate the solution process. For the analytical simulation, two models were applied: a Fickian-based equation to capture only physical aging effects, and a newly developed equation proposed in the present work to account for both physical and chemical aging. The results from both simulations were then validated by the experimentally obtained data;
- (iv) Comparison of the durability properties obtained from the specimens used in the FRPLongDur project at initial (T0), one (T1) and two (T2) years (carried out in the previous works [26], marked as *1st phase* in **Figure 1.8**), and three (T3) and four (T4) years (carried out in the scope of this PhD thesis, marked as *2nd phase* in **Figure 1.8**), with those existing in the literature under accelerated aging test protocols;
- (v) Derivation of environmental conversion factors based on comparisons from (iv). Further details are provided in **Section 1.3.2.4**.

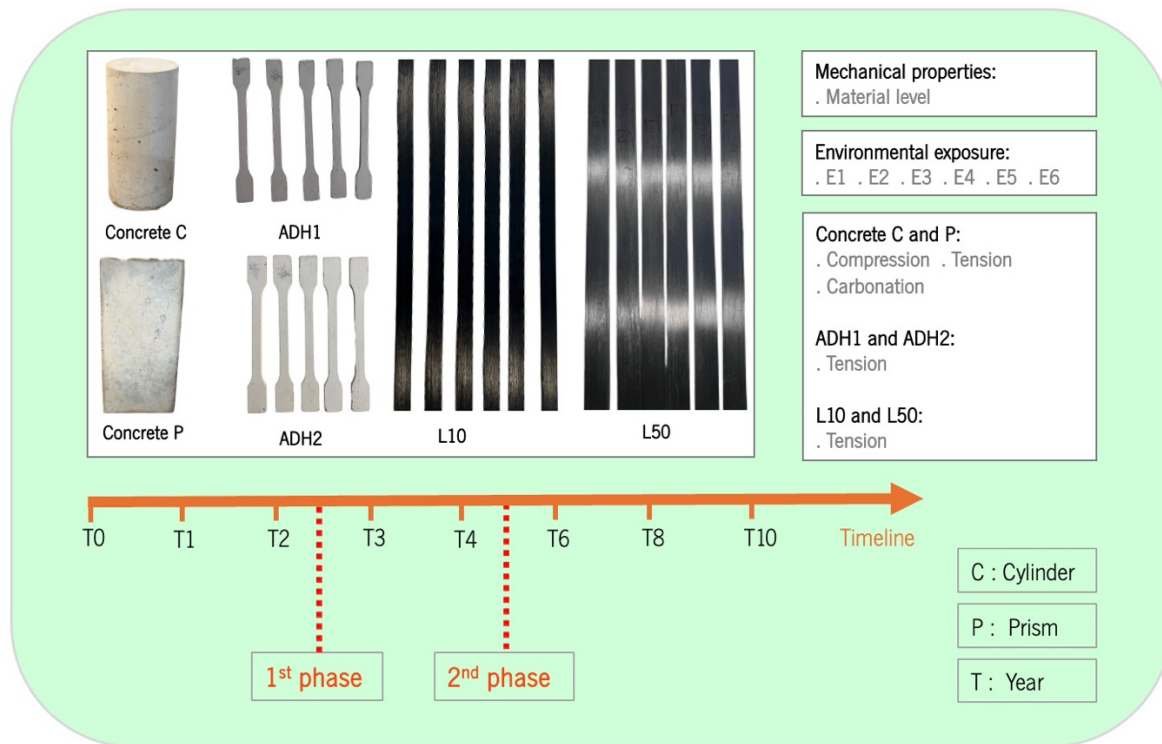


Figure 1.8: Investigated materials, timeline and works conducted at the material level.

Table 1.3: Tests and standards adopted for the involved materials.

Material	Type of test	Standard adopted
Epoxy Adhesives	Tensile tests	The tensile elastic modulus, tensile strength, and strain at the peak stress were measured as per ISO 527-2:2012 [23].
CFRP Laminates	Tensile tests	Tensile elastic modulus, tensile strength, and strain at peak stress, determined as per ISO 527-5:2009 [23].
Concrete	Compression tests	The modulus of elasticity and compressive strength were determined according to NP EN 1239013:2014 [28] and NP EN 12390-3:2011 [28] standards, respectively.
	Tensile tests (pull-off tests)	The concrete tensile strength was measured by pull-off tests using concrete prisms.
	Carbonation tests	The carbonation depth in the concrete was determined according to the LNEC E391:1993 [29].

It is important to note that the material properties obtained at T3 and T4 in the present study were further compared and correlated with those reported in previous work [26] for T0 to T2, to enable a comprehensive and consistent interpretation of long-term performance trends.

Regarding the simulations in (iii), it is acknowledged that coupled hygrothermal-mechanical modeling could have offered a more comprehensive understanding of the degradation mechanisms affecting the mechanical properties—such as tensile strength and elastic modulus—of the epoxy adhesives under water immersion. However, such advanced modeling was beyond the scope of the present PhD thesis.

Nonetheless, the adopted simulation approach in (iii), when integrated with the experimentally observed mechanical property data, yielded meaningful insights into the influence of water on the mechanical performance of the adhesives.

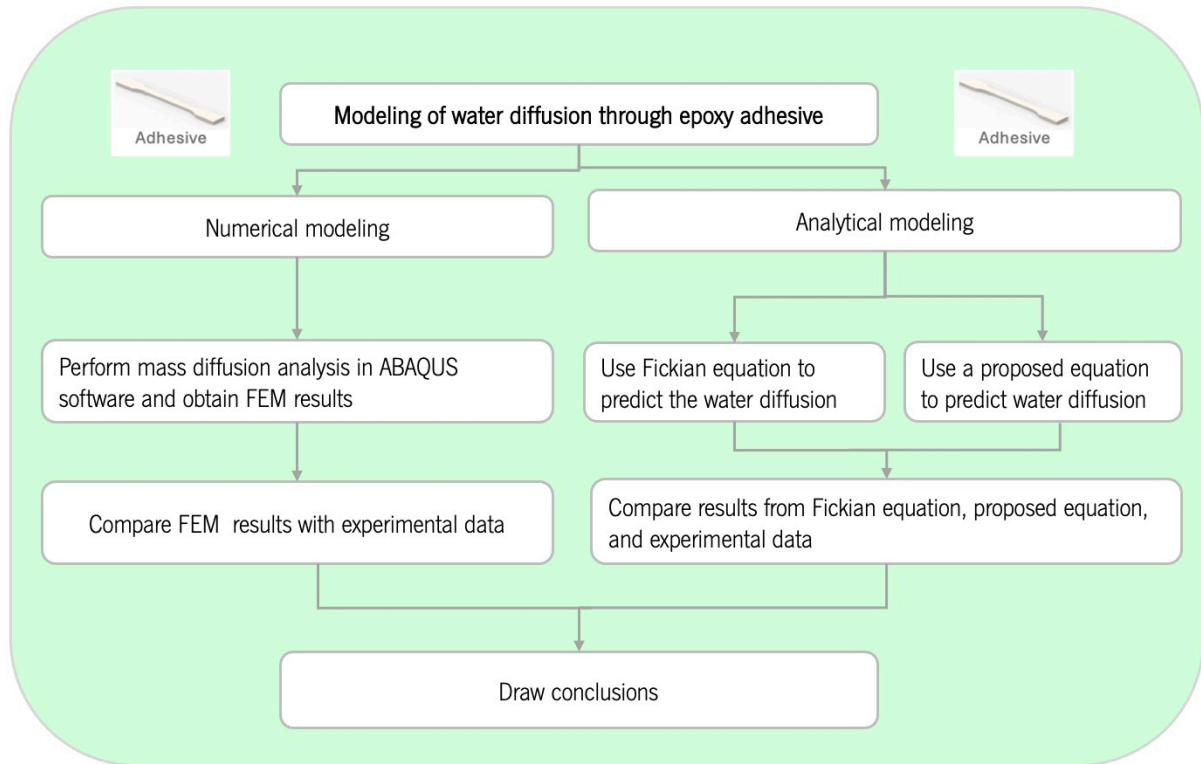


Figure 1.9: Workflow for numerical and analytical modeling of water diffusion through epoxy adhesive.

1.3.2.2 Durability of CFRP-to-concrete bond

To assess the durability of the bond between concrete and the CFRP laminate, the following activities were performed:

- (i) Collection of small-scale EBR and NSM specimens from the environments described in **Section 1.2.2** (E1 to E6) after 3 and 4 years of exposure;
- (ii) Experimental testing (pull-out tests) of the collected specimens for assessing the degradation of the CFRP-to-concrete bond properties over time, measured in terms of pull-out force, loaded end slip, interfacial bond strength and stiffness, and observed failure mechanisms, as depicted in **Figure 1.10**;
- (iii) Comparative analysis of the experimental bond behavior of EBR and NSM specimens with the intrinsic material properties discussed in **Section 1.3.2.1**;

- (iv) Finite element modeling (FEM) of short- and long-term bond behavior of EBR systems, following the computational framework outlined in **Figure 1.11**. The short-term simulations involved the definition of constitutive relationships for the constituent materials under both tension and compression, based on different existing models and use of experimental data. The resulting models were implemented in the ABAQUS software to simulate the pull-out response and identify the most accurate model. For long-term simulations, the calibrated model during short-term analysis was utilized to assess the evolution of bond performance under hygrothermal exposure. Key structural response variables—including pull-out force, slip distribution, interfacial damage evolution, and stress-strain fields—were extracted and compared to experimental observations for validation;
- (v) Comparison of the durability properties obtained from the specimens used in the FRPLongDur project at initial (T0), one (T1) and two (T2) years (carried out in the previous work [26], marked as *1st phase* in **Figure 1.10**), and three (T3) and four (T4) years (carried out in the scope of this thesis, marked as *2nd phase* in **Figure 1.10**), with those existing in the literature under accelerated aging test protocols;
- (vi) Derivation of environmental conversion factors based on the comparative analysis described in (v). Further details are provided in **Section 1.3.2.4**.

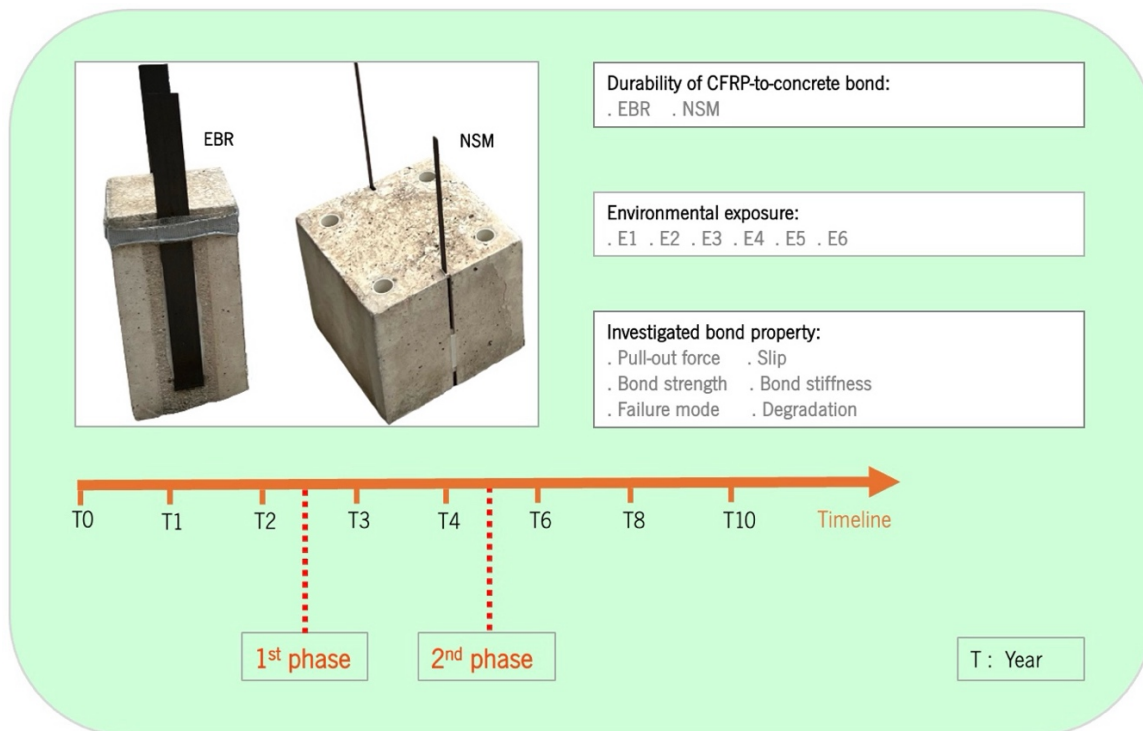


Figure 1.10: Specimens, timeline, and properties investigated at bond level (for both EBR and NSM techniques).

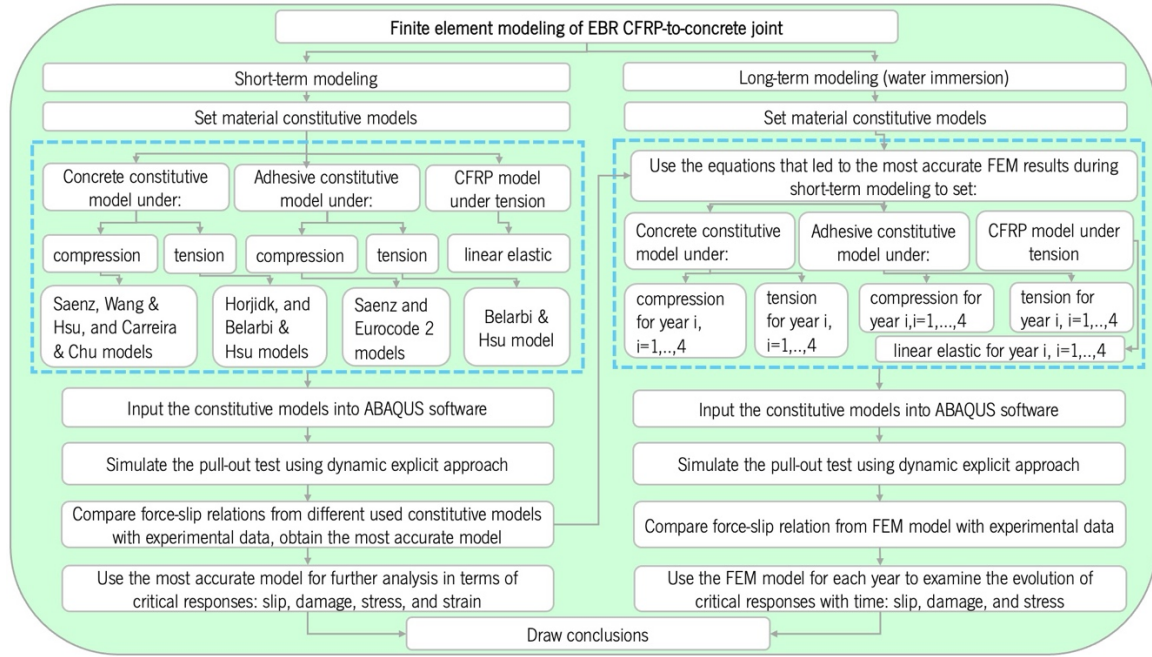


Figure 1.11: Workflow for short- and long-term modeling of specimens with externally bonded CFRP-to-concrete joint.

1.3.2.3 Short- and long-term performance of RC slabs with CFRP systems

In the scope of this PhD thesis, three aspects were addressed to assess the performance of the full-scale RC slabs before exposure (denoted as *short-term*) and after exposure (denoted as *long-term*):

- (i) Monitoring of the evolution of mid-span deflection for up to 6 years (T6);
- (ii) Analytical modeling of the short-term flexural performance of the slabs (the model results being validated by experimental results obtained from initial flexural tests as described in **Section 1.2.1.3**);
- (iii) Analytical prediction of mid-span instantaneous and long-term deflections of the slabs with a sustained load under different environmental exposures for up to 6 years (validated with results in (i)), and extrapolation of the deflections for up to 50 years.

Further details on these aspects are provided below.

Monitoring of various parameters of the investigated RC slabs over time

As previously described in **Section 1.2.2**, four RC slabs with EBR, NSM, EBR-MA, and EBR-GA systems were loaded with a sustained load and placed in different environmental exposure. Each year, monitoring works were performed on these slabs, where various parameters (mid-span deflection, CFRP strains, concrete strains and crack growth) were recorded, as shown in **Figure 1.12**.

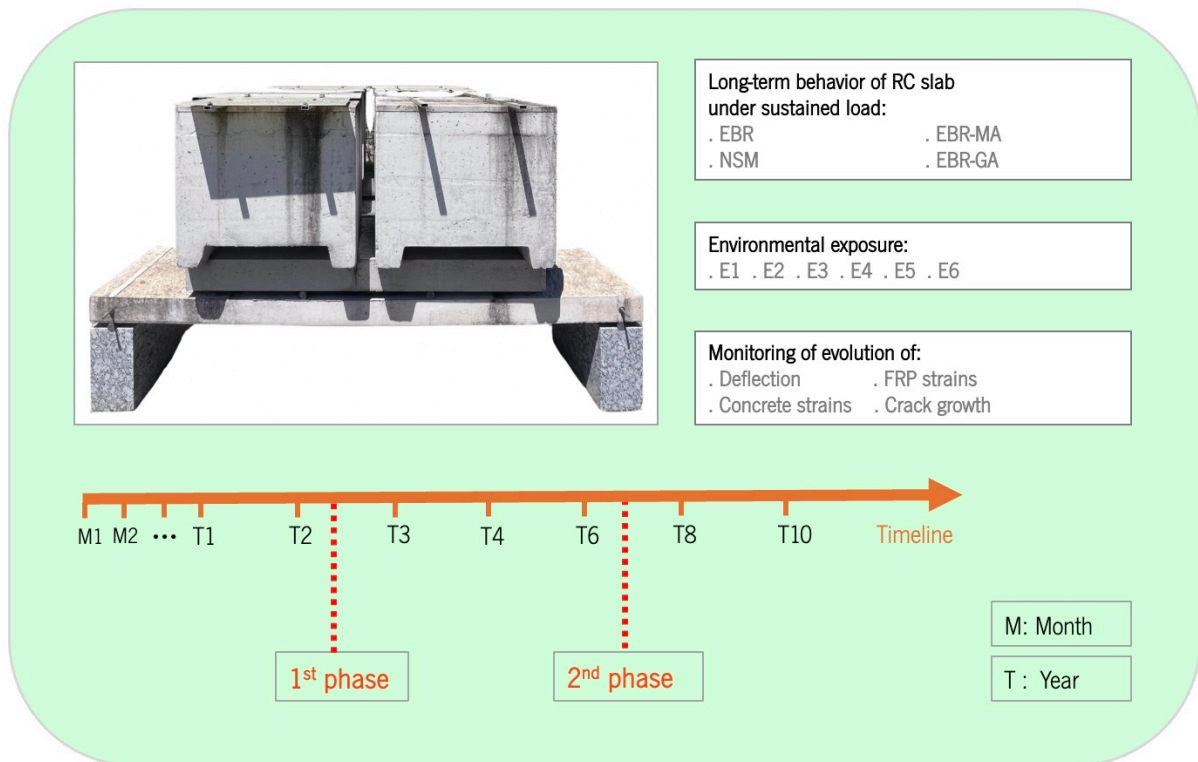


Figure 1.12: Typical strengthened RC slab with sustained load, long-term monitored properties, and timeline.

The first 2 years being recorded in the previous work [26] (marked as *1st phase* in **Figure 1.12**) and the next 4 years being recorded in the scope of this PhD thesis (marked as *2nd phase* in **Figure 1.12**). However, due to limited time, only instantaneous and long-term mid-span deflections were included in this PhD thesis.

Short- and long-term modeling of flexural behavior of investigated RC slabs

The analytical models for predicting the short-term flexural behavior of five different RC slabs (previously described in **Section 1.2.1.3**) were developed using moment-curvature relationships. In each of the models, four critical considerations were made:

- 1) Adopting *fib90* [17] approach for determining the ultimate moment capacity,
- 2) Using two different existing equations for predicting the mid-span deflection with the purpose of comparing their effectiveness at different loading states (the first equation being based on integration approach by incorporating the current moment and curvature, and the second being based on determinant section approach);

- 3) Using various existing equations from standards/guidelines to predict the FRP strain at failure, and finally, comparing results from those standards/guidelines to assess their effectiveness depending on the type of the considered strengthened RC slab.
- 4) Newly established equations for defining the curvature considering initial strains in the materials were presented and proposed for the case of RC slabs with prestressed CFRP systems, which was one of the major contributions.

The constitutive model adopted for concrete followed the recommendations of *fib90* [17]. Steel was modelled as a perfectly elasto-plastic material, while CFRP was assumed to exhibit linear elastic behavior up to failure. The results from the short-term flexural tests, conducted as previously described in **Section 1.2.1.3**, were used to validate the outcomes of the analytical modeling presented above. The workflow followed during the short-term modeling of the flexural tests is shown in **Figure 1.13**. That is, the moment-curvature relationship was first established for each slab, incorporating the prevailing failure mode, followed by adopting different formulae for predicting the FRP failure strain and the midspan deflection; and furthermore, by performing comparative analysis for the used formulae; and thereafter, by proposing the formulae leading to the most agreements with experimental data for future use.

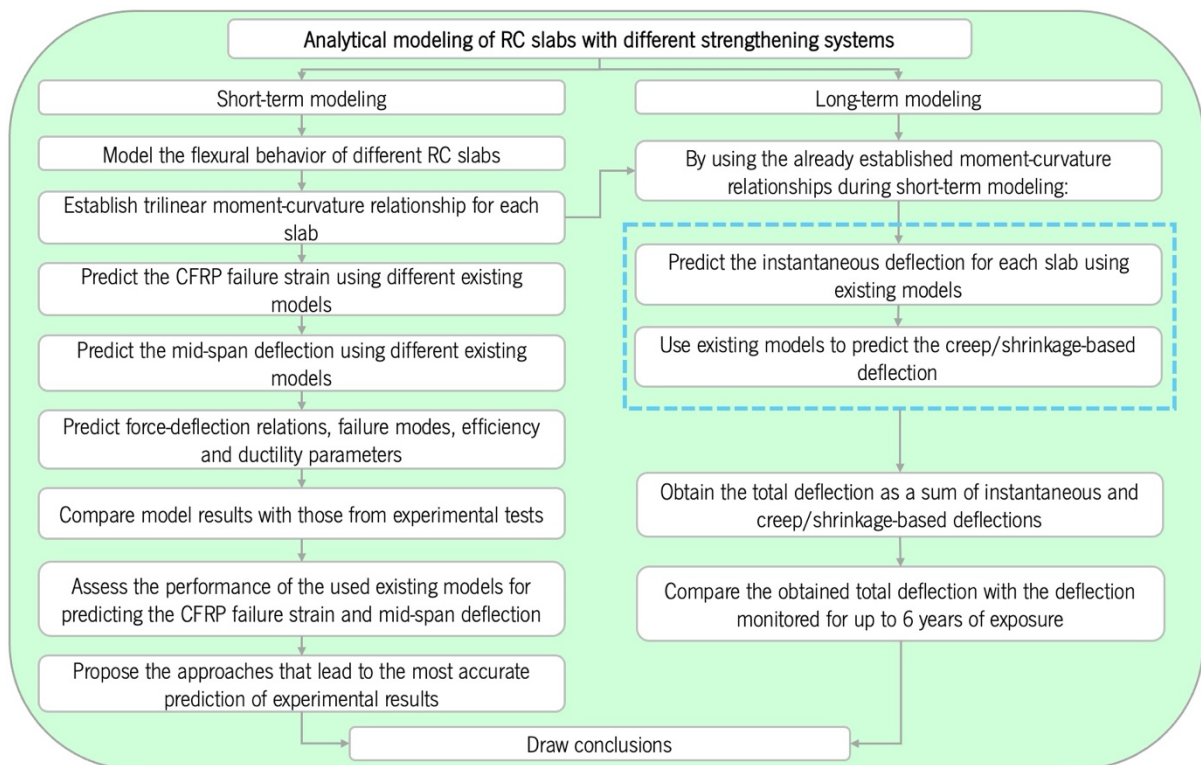


Figure 1.13: Workflow for analytical modeling of the short- and long-term flexural behaviors of the investigated RC slabs.

In addition, long-term analytical modeling of midspan deflections of the RC slabs with different CFRP systems was also performed following the workflow shown in **Figure 1.13**. That is, with the moment-curvature relationships already established during short-term modeling of the flexural behavior of the slabs and existing models incorporating time-dependent effects, the instantaneous deflections resulting from the sustained load and those resulting from creep and shrinkage effects were predicted. The developed analytical predictive models were validated by experimentally monitored instantaneous and long-term deflections; one of the models was used to extrapolate deflections for up to 50 years.

1.3.2.4 Environmental conversion factors and design recommendations

The environmental conversion factors (ECF) were derived from the data obtained from the FRPLongDur project (i.e., for up to 4 years of exposure), and those existing in the literature. These ECF were proposed to be incorporated in the existing design standards (or guidelines) at both *the material level* (i.e., for the epoxy adhesives and CFRP laminates) and at *the bond level* (i.e., for the case of EBR and NSM bond specimens). The derivation of the ECF followed the procedure highlighted in (v) of **Section 1.3.2.1** at *the material level* and in (vi) of **Section 1.3.2.2** at *the bond level*.

For clarity, **Figure 1.14** shows the workflow for deriving the ECF.

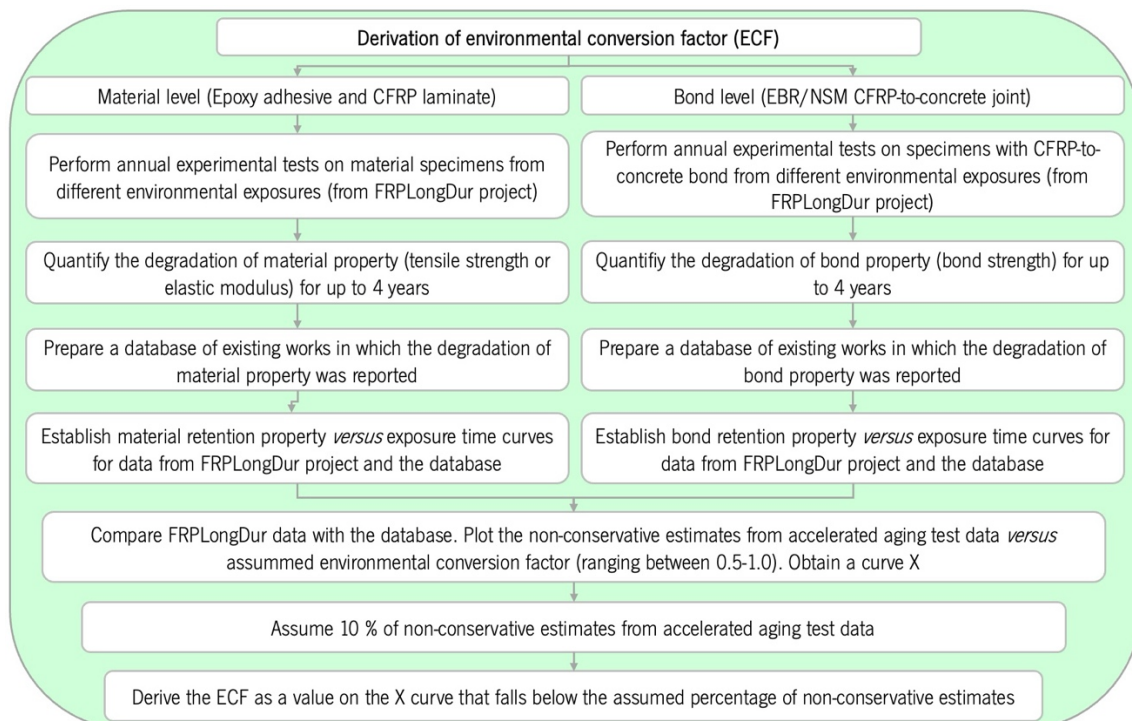


Figure 1.14: Workflow for determining environmental conversion factors (ECF).

Two different relationships were first established: the material/bond retention property *versus* exposure time relationship and the non-conservative estimates from accelerated aging test data *versus* assumed conversion factors relations. The former helped to compare the data obtained in the scope of the FRPLongDur project with those from the existing literature, while the latter helped to estimate the ECF based on the assumed percentage of the non-conservative estimates from accelerated aging test protocols.

1.4. OUTLINE OF THE DISSERTATION

This section delineates the primary components of this PhD thesis, which is structured into five chapters.

Chapter 1 presents the motivation behind this research, introduces the FRPLongDur project that forms the foundation of this PhD thesis, outlines the objectives, and describes the research methodologies employed to achieve the defined objectives. Finally, it provides an overview of the structure of this dissertation.

Chapter 2 presents CFRP composites in construction industry. It starts with a short overview of FRP materials including glass FRP (GFRP), basalt FRP (BFRP), and aramid FRP (AFRP), and carbon FRP (CFRP), the latter being the central focus. This is followed by a discussion on different existing strengthening techniques using CFRP laminates, with particular emphasis on EBR, NSM, and EBR prestressed via mechanical and gradient anchorage systems. Furthermore, an in-depth discussion on the durability and long-term performance of RC elements with CFRP strengthening systems is provided, focusing on the durability of materials constituting the bond between the concrete and the CFRP laminate and the durability of the bond itself, as well as the long-term behavior of RC elements with CFRP systems under coupled effects of sustained loading and different environmental conditioning. Furthermore, a discussion on the numerical and analytical modeling techniques for different strengthening systems is provided. The chapter ends with overall final remarks about the CFRP composites in construction, highlighting the gaps and needs for further relevant research.

Chapter 3 presents a summary of the papers authored within the scope of this PhD thesis. Throughout the research, seven journal articles were prepared, three of which have already been published in high-impact journals, while the remaining four are potential candidates for future publication. Each paper is summarized with a focus on the specific problem addressed, methodologies employed, and the results obtained.

Chapter 4 presents conclusions and recommendations derived from this PhD thesis. Regarding the conclusions, the first section presents insights into the durability of the specimens at material level, including concrete, epoxy adhesives, and CFRP laminates. The second and third sections focus on the experimental

and numerical works on the behavior of the bond between concrete and CFRP laminates at bond level. The fourth to sixth sections provide insights into the monitoring works and analytical models developed for the short- and long-term flexural behavior of different RC slabs strengthened by CFRP systems (structural level). Finally, the last section introduces the proposed environmental conversion factors at material and bond levels. The chapter ends with a separate section providing insights on future developments.

Chapter 5, the final chapter, comprises the seven full journal papers associated with this PhD thesis, organized in a logical sequence. The first paper (**PAPER I**) addresses the durability of the studied materials. This is followed by the second (**PAPER II**) and third (**PAPER III**) papers, which present experimental investigations on the durability of the bond between concrete and the CFRP laminate applied using the EBR and NSM techniques, respectively. The fourth paper (**PAPER IV**) focuses on developing various numerical models to assess the short-term behavior of the bond between concrete and the CFRP laminate using the EBR technique. The fifth paper (**PAPER V**) presents an extension of one of the models developed in **PAPER IV** to evaluate the bond behavior under water immersion for up to 4 years, also within the EBR framework. The sixth paper (**PAPER VI**) presents developed analytical models for short-term flexural behavior of each of the five different RC slabs investigated in this PhD thesis (i.e., the slabs previously described in **Section 1.2.1.3**). Finally, **PAPER VII** addresses the experimental monitoring of instantaneous and long-term deflections of each of the strengthened RC slabs investigated in **PAPER VI**, under sustained load and different environmental conditioning. Furthermore, the paper also presents the developed analytical models for predicting the instantaneous and long-term deflections of the slabs. These models were eventually validated against the experimentally monitored deflections for up to 6 years and one of them was extended to estimate deflections up to 50 years.

1.5. REFERENCES

- [1] P. Shen, L. Lu, W. Chen, F. Wang, S. Hu, Efficiency of metakaolin in steam cured high strength concrete, *Constr. Build. Mater.* 152 (2017) 357–366. <https://doi.org/10.1016/j.conbuildmat.2017.07.006>.
- [2] M. Mazloom, A.A. Ramezani pour, J.J. Brooks, Effect of silica fume on mechanical properties of high-strength concrete, *Cem. Concr. Compos.* 26 (2004) 347–357. [https://doi.org/10.1016/S0958-9465\(03\)00017-9](https://doi.org/10.1016/S0958-9465(03)00017-9).
- [3] J.M. Brown, W.B. Wilkinson (1819-1902) and his place in the history of reinforced concrete, *Early Reinf. Concr.* 39:1 (1966) 129–142. <https://doi.org/10.1179/tns.1966.009>.

- [4] American Society of Civil Engineers (ASCE), Alvord Lake Bridge. History & Heritage. Historic landmarks, ASCE Publ. (1889). <https://www.asce.org/about-civil-engineering/history-and-heritage/historic-landmarks/alvord-lake-bridge> (accessed December 30, 2024).
- [5] P. Gaudette, D. Slaton, 15 PRESERVATION BRIEFS: Preservation of Historic Concrete, Natl. Park Serv. (2007). <https://doi.org/ISBN: 978-0-16-078946-5>.
- [6] P. Wolert, A. Nowak, RELIABILITY ANALYSIS FOR A 100-YEAR-OLD REINFORCED CONCRETE FRAMED BRIDGE, in: 8th Int. Conf. Arch Bridg., 2016: pp. 1241–1250.
- [7] A. Sivakrishna, A. Adesina, P.O. Awoyera, K.R. Kumar, Green concrete: A review of recent developments, Mater. Today Proc. 27 (2020) 54–58. <https://doi.org/10.1016/j.matpr.2019.08.202>.
- [8] D. Gilfillan, G. Marland, CDIAC-FF: Global and national CO2 emissions from fossil fuel combustion and cement manufacture: 1751-2017, Earth Syst. Sci. Data. 13 (2021) 1667–1680. <https://doi.org/10.5194/essd-13-1667-2021>.
- [9] K.L. Scrivener, V.M. John, E.M. Gartner, Eco-efficient cements: Potential economically viable solutions for a low-CO2 cement-based materials industry, Cem. Concr. Res. 114 (2018) 2–26. <https://doi.org/10.1016/j.cemconres.2018.03.015>.
- [10] CIC, Construction-Intelligence-Center. Global Construction Outlook to 2022: Q3 2018 Update., (2018).
- [11] GCP-G, GCP-G. GCP-Global, Oxford-Economics. Global Construction 2030: A Global Forecast for the Construction Industry to 2030. Global/Oxford Economics, London. 2015, (2015).
- [12] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fiber reinforced polymer laminates used in strengthening systems: Accelerated ageing versus natural ageing, Materials (Basel). 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [13] A. Dushimimana, A.P. Vassilopoulos, J. Sena-Cruz, J.M. Pereira, L. Correia, S. Cabral-Fonseca, R. Cruz, Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-controlled environments, Constr. Build. Mater. 438 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.137201>.
- [14] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of

- bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024).
<https://doi.org/10.1016/j.conbuildmat.2024.136213>.
- [15] ACI 440.2R-08, American Concrete Institute (ACI). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures., USA, 2008.
 - [16] CNR-DT 200 R1, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures; CNR-DT 200 R1/2013;, Rome, Italy, 2013.
 - [17] CEB-FIP, fib90 Bulletin:Externally Applied FRP reinforcement for Concrete structures, 2019.
 - [18] EN 1992-1-1 (2004), Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, 2004.
 - [19] M. Zoghi, "The International Handbook of FRP Composites in Civil Engineering," Boca Raton, FL, USA, 2013.
 - [20] FIB Bulletin 90, "Externally Applied FRP Reinforcement for Concrete Structures, Federation International du Beton Technical Report Prepared by a Working Party of the T5.1 FRP Reinforcement for Concrete Structures, Lausanne, Switzerland, 2019.
 - [21] Committee 440, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures," Farmington Hills, MI, USA, 2017.
 - [22] J.M. Sena-Cruz, M.Â.D. Azenha, S. Cabral-Fonseca, P. Fernandes, M. Rezazadeh, L. Correia, R. Cruz, FRPLongDur – Long-term structural and durability performances of reinforced concrete elements strengthened in flexure with CFRP laminates, Guimaraes, Portugal, 2020.
<https://doi.org/PTDC/ECM-EST/1282/2014>.
 - [23] ISO 527-2:2012, ISO 527-2:2012—Plastics—Determination of Tensile Properties, Genève, Switzerland, 2012.
 - [24] S&P, S&P CFRP Laminates. Technical Datasheet, Seewen, Switzerland, 2014.
 - [25] SIKa, "Sikadur®-30. Product Data Sheet ", Dublin, Ireland, 2017.
 - [26] J. Cruz, Multi-scale investigation on durability and long-term behaviour of concrete elements strengthened with CFRP laminates according to the EBR and NSM techniques, University of Minho, 2023.

- [27] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174. <https://doi.org/10.1016/j.compstruct.2024.118174>.
- [28] IPQ (Instituto Português da Qualidade), NP EN 12390. Testing Hardened Concrete, Caparica, Portugal, 2013.
- [29] LNEC E397-1993, Concrete - determination of the elasticity young modulus under compression [Portuguese specification from LNEC], 1993.

CHAPTER 2

CFRP COMPOSITES IN CONSTRUCTION INDUSTRY

2.1. OVERVIEW

Fiber reinforced polymer (FRP) materials have found widespread application across various industries, particularly in construction, aeronautics, or wind energy sectors [1], due to their unique properties that surpass those of many conventional materials [2]. FRP materials result from a combination of fibers and polymer matrix [3] (**Figure 2.1**). Commonly, FRP materials are designed as a function of the type of fiber, namely, glass, basalt, aramid, or carbon fiber. These fibers when combined with the polymer matrix result in glass fiber reinforced polymer (GFRP), basalt fiber reinforced polymer (BFRP), aramid fiber reinforced polymer (AFRP), and carbon fiber reinforced polymer (CFRP) composites.

To better understand the benefits of FRP composites, the stress-strain behavior of their constituent fiber is compared to that of mild steel in **Figure 2.2** [4]. The behavior of steel, including steel tendons and steel rebars, typically exhibits a distinct yielding point followed by a region of plastic deformation before failure.

This ductile behavior allows steel to absorb significant energy before fracturing. In contrast, FRP materials such as ones using high-strength carbon (HS), high-modulus carbon (HM), glass, and aramid fibers display a linear-elastic stress-strain relationship up to failure, characterized by a brittle failure mode without yielding. Carbon HS and HM FRPs offer high tensile strength, but lower ultimate tensile strain compared to steel, with HM carbon fibers providing a higher modulus of elasticity (**Figure 2.2**). GFRP composites also shows linear behavior but with a lower modulus and tensile strength than carbon FRPs. AFRP composites combine high tensile strength with moderate modulus and exhibit slightly higher strain at failure compared to CFRP composites, yet still lack the ductility of steel [5,6]. The fact that CFRP materials possess superior mechanical properties (higher stiffness, strength, fatigue and life among others) and greater resistance to aggressive environments renders them more promising and more preferred than others in some specific applications [7-9].

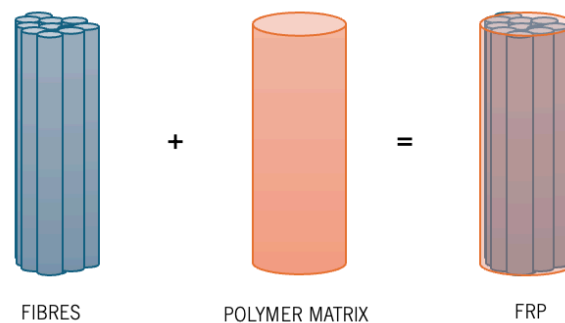


Figure 2.1: Constituent materials of FRP composites (adapted from [3])

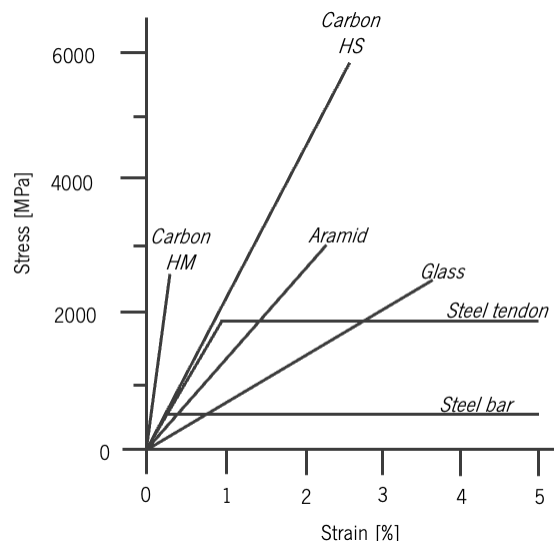


Figure 2.2: Typical stress-strain relations of fibers versus mild steel (adapted from [4])

Over the last decades, FRP materials have gained popularity in the construction industry. According to existing studies [10,11] as shown in **Figure 2.3**, the construction sector ranks second (26 %) in the application of FRP materials, just after the automotive industry (31 %). This indicates that FRP materials have great potential to become one of the most promising construction materials in the future. FRP composites are used in Civil Engineering in two main forms: as tensile components (e.g., laminates, sheets, bars, cables) and as bending components (e.g., profiles, cellular, or sandwich slab panels). These materials are applied for internal reinforcement of new or existing constructions, repair or strengthening of structures, all-FRP structures, and hybrid structures combining FRP with traditional materials like concrete. CFRP composites are the focus of this PhD thesis; therefore, they are addressed pertinently in the remainder of this chapter. The following sections present a discussion on different techniques for strengthening RC structures using CFRP strips/laminates, followed by a section presenting an in-depth literature review on the durability and long-term behavior of RC elements strengthened with CFRP systems. Finally, the chapter ends with a discussion on analytical and numerical modeling of short/long-term behavior of RC elements with CFRP systems.

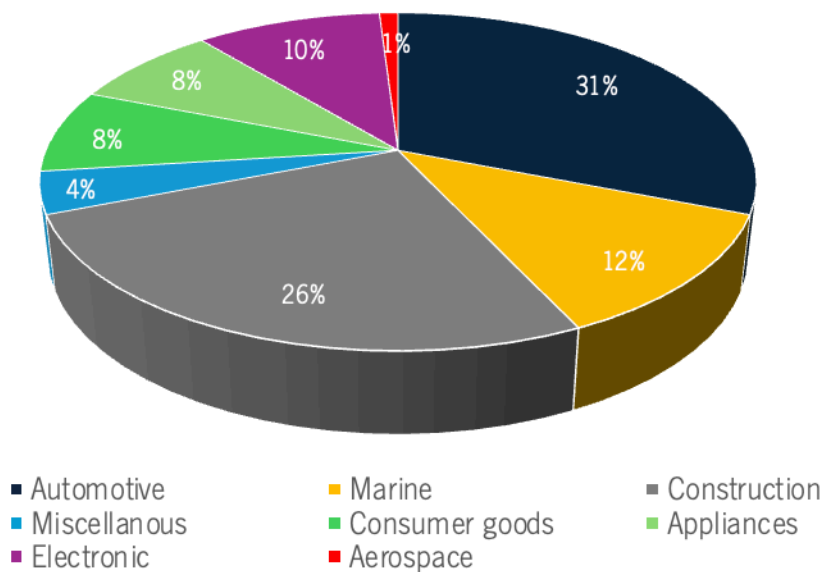


Figure 2.3: Market share of Fiber Reinforced Polymer (FRP) by application (adapted from [10])

2.2. CFRP STRENGTHENING TECHNIQUES FOR RC STRUCTURES

CFRP strips/laminates have shown promising structural performance when bonded to the reinforced concrete structure (RC) as a mean of strengthening system [12]. Various strengthening techniques have been investigated, including the externally bonded reinforcement (EBR) [13,14], near-surface mounted

(NSM) [15,16], externally bonded reinforcement on grooves (EBROG) [17], externally bonded reinforcement in grooves (EBRIG) [17], and embedded through-section (ETS) [18]. In the scope of the present PhD thesis, only the EBR and NSM techniques have been addressed, hence the rest of the discussion focuses on these two techniques.

2.2.1. Strengthening using externally bonded reinforcement (EBR) technique

Strengthening RC structures using externally bonded reinforcement (EBR) has been widely accepted as a promising method to increase both their structural and durability performance. In this technique, the CFRP laminates/strips are glued to the concrete substrate using a bonding agent such as epoxy adhesive (see **Figure 1.2a**). With this technique, the cracking, yielding, and ultimate loads are increased compared to RC structural elements without strengthening [19–22]. According to existing studies [23–25], the procedures for strengthening using the EBR technique are as follows: (i) The existing condition of an RC structure is assessed for the need of strengthening, then a strengthening design is developed in which the type and amount of CFRP reinforcement is specified; (ii) Concrete surface is treated to enhance the bond between the concrete substrate and the CFRP laminate, which is followed by cleaning the surface to ensure complete removal of dust and loose particles; (iii) CFRP laminate is cut into small parts according to the design requirements, followed by cleaning the CFRP surface with a solvent to ensure removal of any dust; (iv) The epoxy adhesive is mixed according to the instructions from manufacturer; (v) A layer of epoxy adhesive is applied to the prepared concrete surface or to the CFRP laminate surface, followed by the placement of the CFRP laminate onto the concrete meanwhile pressing firmly to ensure proper adhesion (the adhesive in excess is removed and the adhesive layer is levelled); (vi) A period of time is needed for reaching a minimum curing level of the epoxy adhesive according to the instructions from manufacturer, followed by inspection to ensure the CFRP laminates are properly bonded and aligned; (vii) If necessary, tests are conducted to assess the effectiveness of the strengthening system, followed by documentation of the entire process including the design calculations, material specifications, and test results.

2.2.2. Strengthening using near-surface mounted (NSM) technique

The application of CFRP laminate strips using Near-Surface Mounted (NSM) technique has been shown to enhance the flexural strength of RC structures more effectively than the counterpart EBR technique and ensures a more efficient load transfer [26–32]. Strengthening RC structures using the NSM technique involves embedding CFRP strips into grooves pre-cut into the concrete cover (see **Figure 1.2b**). Here are the key steps involved [33]: (i) The existing condition of an RC structure is assessed for the need of strengthening, then a strengthening design is developed in which the type and amount of CFRP

reinforcement is specified; (ii) Next is surface preparation, which includes cutting grooves where the CFRP laminate strip will be inserted, followed by cleaning the grooves thoroughly to remove dust and loose particles; (iii) After groove preparation, the strip is cut according to design requirements, and its surface is cleaned with a solvent to remove any dust; (iv) The epoxy adhesive is then mixed according to the instructions from manufacturer; (v) A layer of epoxy adhesive is poured into the pre-cut groove, the strip is placed into the groove pressing firmly to ensure proper alignment and adhesion, the adhesive in excess is removed followed by levelling the adhesive layer; (vi) A period of time is needed for reaching a minimum curing level of the epoxy adhesive according to the instructions from manufacturer, followed by an inspection to verify proper bonding and alignment of the CFRP laminates; (vii) If necessary, tests are conducted to assess the effectiveness of the strengthening system, followed by documentation of the entire process including the design calculations, material specifications, and test results.

2.2.3. Strengthening using EBR technique with prestressed CFRP laminates

2.2.3.1 Mechanical anchorage system

When strengthening RC structures, the previously mentioned EBR technique can be prestressed to typically benefit from the full utilization of the FRP strain [34]. One of the methods used relies on the mechanical anchorage (MA) systems clamped at the CFRP extremities (see **Figure 1.4c** and **Figure 1.5**). For more details about the MA systems, previous research works dedicated to this topic can be found [19,35], in which the key steps when implementing these systems are as follows: (i) At first, the assessment and design stage begin with evaluating the RC structure to determine the extent of strengthening required, followed by designing the prestressed CFRP system, including the type and amount of CFRP laminates and the anchorage system; (ii) The next stage is surface preparation (as previously described in **Section 2.2.1**); (iii) Installation of the CFRP laminates using epoxy adhesive (as previously described in **Section 2.2.1**); (iv) The prestressing process starts by first fixing one end of the CFRP laminate using a mechanical anchorage system, followed by applying the required prestressing force on the other end using a hydraulic jack to ensure even distribution within the design specifications, and finally by anchoring the latter end with bolts to maintain the prestressed state; (v) A period of time is needed for reaching a minimum curing level of the epoxy adhesive according to the instructions from manufacturer to ensure a strong bond formation; (vi) If necessary, tests are conducted to assess the effectiveness of the strengthening system, followed by documentation of the entire process including the design calculations, material specifications, and test results; (vii) As a final stage, if necessary, a protective coating may be applied to the CFRP laminates if needed to shield them from environmental factors.

2.2.3.2 Gradient anchorage system

Gradient anchorage (GA) system is also one of the systems used to apply a prestress to the CFRP laminates externally bonded to an RC element (see **Figure 1.4d** and **Figure 1.5**). Briefly, a gradient anchorage system involves gradually reducing the prestressing force along the length of the laminate. This method distributes the stress more evenly, enhancing the bond between the CFRP and the concrete. According to existing studies [35,36], the key steps when applying the GA systems are as follows: steps (i) to (v) for the case of MA system are similar to those of the GA system; after the hydraulic jacks are used to apply the required prestressing force, heat is applied to the epoxy adhesive in a controlled manner to accelerate curing, starting from the inner zones and gradually moving towards the ends in segments, as shown in **Figure 2.4**. The prestressing force (F_p) is then reduced step by step towards the ends of the CFRP strip to distribute the force ($\Delta F_{p i=1, \dots, n-1}$) over multiple segments (n) [35,36]; (vi) The epoxy adhesive is allowed to cure to ensure a strong bond formation; (vii) If necessary, tests are conducted to assess the effectiveness of the strengthening system, followed by documentation of the entire process including the design calculations, material specifications, and test results; (viii) Finally, like MA systems, a protective coating may be applied to the CFRP laminates if needed to shield them from environmental factors.

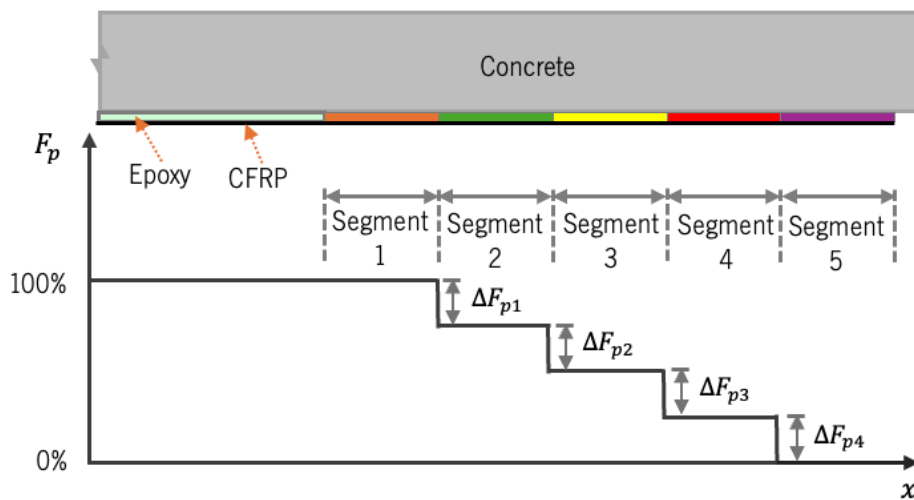


Figure 2.4: Gradient anchorage working mechanism demonstrated at the end of a concrete beam (adapted from [36]).

2.3. DURABILITY AND LONG-TERM BEHAVIOR OF RC ELEMENTS WITH CFRP STRENGTHENING

Understanding the durability and long-term performance of the strengthening techniques described in **Section 2.2** is fundamental, as it can help in building trust and reliability when applied. This section presents a detailed literature review about this topic, paving the way to various works developed in this PhD thesis. To begin with, critical terms shaping the behavior of RC elements when exposed to different exposure conditions are first defined. Furthermore, the durability of materials constituting the bond between CFRP laminate and concrete is addressed in detail, followed by a thorough discussion on the data addressing the durability and long-term behavior of RC elements with CFRP systems at small-scale and full-scale levels.

Durability can have different meanings depending on the area in which it is used. In the context of the present PhD thesis, durability of a material, component or structure is defined as its ability to resist cracking, oxidation, chemical degradation, delamination, wear, and/or the effects of foreign object damage for a specified period, under the appropriate load conditions, and under specified environmental conditions.

Long-term behavior of a structure refers to the performance of a structure over time under various environmental, operational, and loading conditions. It includes factors such as material degradation (due to aging, corrosion, or weathering), time-dependent deformations like creep and shrinkage, deflections and settlements, fatigue from repetitive loading, and the impact of environmental effects like temperature and humidity.

Degradation agent/factor refers to any agent/factor that acts on a material, component, or structure, and that may cause alterations on its performance. The main degradation factors can be classified according to two categories: (i) environmental degradation factors and (ii) mechanical degradation factors. The former category includes factors such as moisture, chemicals, thermal effects, and UV exposure. The latter category includes factors such as static loading (creep, relaxation) and dynamic loading (fatigue, vibration, impact).

Degradation mechanism refers to a sequence of chemical, mechanical and/or physical changes that lead to alteration of one or more mechanical properties of a material, component, or structure, in a harmful way after being exposed to a degradation factor or a combination of them. Examples of degradation mechanisms may include physical (plasticization, relaxation), mechanical (debonding) and chemical (hydrolysis, leaching).

Accelerated aging test (AAT) refers to testing protocols of a material, component, or structure that is initially (before testing) exposed to extreme conditions (such as high temperatures, UV radiation, humidity, or

mechanical stress) to simulate long-term aging in a shorter period. These protocols typically involve higher stress levels, extreme environmental actions, and adjustments in sample size, such as reducing the sample thickness, speeding up the degradation process and shortening the experimental duration.

Natural aging test (NAT) refers to testing protocols where aging occurs under real-world environmental conditions, in which the testing of a material, component, or structure is performed after exposure to normal environmental factors like temperature fluctuations, moisture, UV radiation, and mechanical wear over an extended period. This type of aging reflects the actual, gradual degradation that happens in service.

2.3.1. Durability of materials constituting the bond between CFRP laminate and concrete

2.3.1.1 Durability of concrete

Concrete durability is essential for ensuring the longevity and performance of structures exposed to environmental conditions, mechanical stresses, and chemical influences. As a widely used material, its ability to resist deterioration from degradation factors like moisture, temperature fluctuations, chemical attacks, and mechanical loading directly affects safety, maintenance, and cost-effectiveness. A thorough understanding of concrete durability can lead to optimized material selection, reduced maintenance costs, enhanced structural resilience, and improved service life, ultimately contributing to more sustainable and reliable infrastructure.

The *fib* MC 2020 [37] provides a comprehensive framework for evaluating concrete durability by addressing the above degradation factors. It details key degradation mechanisms—including carbonation, chloride ingress, freeze–thaw cycles, alkali-silica reaction, and leaching—alongside predictive models for service life based on exposure conditions and material properties. Time-dependent behaviors like creep, drying shrinkage, and autogenous shrinkage are also modelled, with emphasis on their interaction with sustained loading and their impact on serviceability, such as long-term deflections and cracking.

The use of concrete in the construction industry has taken a central role owing to its properties, namely its compression performance, and its tensile capacity improvement when reinforced with other materials. However, concrete properties change with time mainly resulting from the effects of exposure conditions. Building on the general principles of concrete durability, numerous studies have specifically examined how various degradation agents contribute to the degradation of concrete over time. For example, studies show that concrete develops cracks over time, which leads to ingress of various degradation agents including CO₂, water, fluids, chemicals, and harmful gases, among others [38]. Additionally, early age frost, especially

when used in winter or in cold environments, affects the development of hydration process and concrete performance thereby leading to reduced service of concrete structures [39], later- age freeze-thaw attacks, however, do not significantly affect the concrete strength properties [39]. Concrete durability properties were measured up to 90 days of exposure to different degradation agents such as carbonation and chlorides attacks, the findings indicate that compressive strength, static modulus of elasticity, splitting tensile strength, and flexural strength increased with time [40]. In a study by Chalee et al. [41], it was found that the compressive strength of Ordinary Portland cement (OPC)-based concretes experienced a slight decrease after exposure to marine environments for up to 5 years.

Compressive strength, concrete carbonation depth and concrete relative dynamic modulus of elasticity were measured after exposure to temperate plain region and cold mountain region for up to 17 years [42]. The results indicate that the freeze-thaw attacks from the cold mountain region did not affect the properties significantly. Specimens in the cold mountain regions tended to have a smaller carbonation depth than those in the temperate plain region, which was attributed to prolonged periods of high humidity from cold climate with heavy snowfall and short periods of clear sky. Concrete strength increased with the first 10 months of exposure, thereafter, levelling off until 17 years of exposure [42]. Hence, concrete generally maintained its strength properties for up to the exposure time (17 years), also, no significant drop in the relative dynamic modulus of elasticity was detected. In a study by Frenzer et al. [43], non-destructive tests of different concrete properties (including moisture and carbonation) were performed on field during assessment of concrete durability on 30 different concrete structures aged up to 60 years. The results indicated that age could improve durable concrete while damage may impair non-durable concrete.

In a study conducted by Wang and Yue [44], the time variation law of concrete compressive strength was evaluated, and it was noted that the strength loss of concrete can be better understood if the coupled effect of multiple factors (carbonation, freeze-thaw attack, temperature effect, and surface attack) is analyzed rather than the effect of a single factor. The main factors affecting the concrete mechanical properties at low temperatures is the temperature and moisture content; concrete strength increases significantly with decrease in temperature and with increase in moisture content [45].

According to a recent state-of-the-art review by RILEM TC289-DCM [46], during drying-wetting cycles in marine environments, capillary absorption becomes active, efficiently transporting external ions into the concrete. The intensity of this process is highly influenced by the saturation level of concrete and the duration of the drying and wetting periods [47]. Freeze-thaw cycles come into action for concrete that is

both saturated and exposed to freezing conditions. Under these cycles, external liquids can also penetrate the concrete [48] with the number of cycles significantly impacting chloride penetration [49].

From the above selected existing literature, it can be noted that concrete generally maintains its strength over long periods, with some variations depending on environmental conditions and exposure to degradation agents.

2.3.1.2 Durability of epoxy adhesive

Epoxy adhesives are widely used in structural strengthening and repair applications due to their high bond strength, ease of application, and compatibility with fiber-reinforced polymer (FRP) systems. However, their long-term performance is critically influenced by environmental exposure and loading conditions, making durability a key consideration in design and assessment. Over time, factors such as elevated temperatures, moisture ingress, ultraviolet (UV) radiation, freeze-thaw cycles, and chemical attack can alter the physical and mechanical properties of epoxy adhesives, potentially compromising bond integrity and structural performance. To better understand these effects, numerous studies have investigated the degradation behavior of epoxy systems under various exposure conditions.

According to existing studies, the penetration of water into polymers and adhesives can alter their physical, chemical, and mechanical properties [50]. Physical aging involves reversible changes in material properties that can be partially restored after drying, with these changes also being influenced by temperature [12]. Chemical aging can occur after prolonged exposure to moisture, leading to an irreversible degradation process. This can affect all components of the epoxy-FRP based system, including not only the epoxy adhesive, but also the fiber, matrix, and fiber-matrix interface [51].

The process of physical aging includes a mechanism known as plasticization, which leads to a decrease in the modulus of elasticity and strength, while increasing ductility [52–54]. Another physical mechanism is swelling, which involves volumetric changes due to moisture content alone, independent of thermal expansion. Swelling can potentially weaken the fiber-matrix interface bond, causing premature cracking or fiber separation [55,56]. Water can also lower the glass transition temperature (T_g), which marks the temperature range above which the mechanical properties, such as stiffness and strength, of the epoxy adhesive or polymer matrix significantly deteriorate [53,55]. The main factors contributing to the strength reduction in epoxy resins are water absorption and ongoing curing reactions [57].

Moisture and water immersion of epoxy adhesives affect significantly the properties of the epoxy adhesives. Significant reductions in tensile strength (up to 38 % of the initial value) and flexural strength (up to 44 %

of the initial value) were observed over time [57]. The primary causes of this degradation were water absorption and hydrolysis, which softened the adhesive and accelerated deterioration. Additionally, internal stresses and defects formed during the curing process contributed to strength loss by causing cracking and peeling. In accelerated aging tests conducted by Silva [32], significant reductions were observed after 480 days of full immersion in water: the $\bar{\gamma}_g$ decreased by 14 %, the elastic modulus by 47 %, and the tensile strength by 38 %. Furthermore, in a study by Fernandes et al. [58], immersion in pure water was found to be especially detrimental to epoxy adhesives, resulting in a strength reduction of up to 38 % after two years of exposure. Also, in a study by Sousa et al. [59], significant reductions in flexural properties of a commercial epoxy adhesive were found after two years of immersion in water at 40 °C; specifically, the flexural strength decreased by 24 %, and the elastic modulus dropped by 30 %.

Freeze-thaw cycles have shown a significant impact on the properties of the epoxy adhesive. The impact of freeze-thaw cycles (ranging from -18 °C to 20 °C for up to 240 days) was investigated [32]. After freeze-thaw aging, there was a 23 % reduction in the $\bar{\gamma}_g$ due to the interruption of the curing process by low temperatures. Exposure of epoxy adhesive to wet-dry cycles also led to a reduction in both elastic modulus and tensile strength [60].

In a recent study by Ren et al. [61], it was found that the mechanical properties of epoxy resin adhesive (ERA) initially improved but subsequently degraded when subjected to freeze-thaw, chlorine dry-wet, and combined freeze-thaw/dry-wet environmental exposures. Their findings indicated that: (i) after 120 cycles of exposure to freeze-thaw (F-T), chlorine dry-wet (D-W), and combined F-T/D-W environments, the tensile strength and elastic modulus of the ERA increased; (ii) the stress-strain behavior of the ERA transitioned from linear to nonlinear after exposure to these environmental conditions; (iii) deformation capacity decreased for the ERA after environmental exposure, except for specimens subjected to 120 D-W cycles; (iv) both the strain energy and the tensile strength retention rate of the ERA increased initially but then declined over the 120 cycles of environmental exposure. However, exposure to chlorides showed minimal impact on the properties of epoxy adhesives in a study by Fernandes et al. [62]. The duration of exposure, particularly during wet-dry cycles, and the exposure temperature were found to be the primary factors influencing the elastic modulus of epoxy resin in a chloride environment [63].

The post-curing process, which happens when the cured resin is subjected to temperatures at or above its curing temperature for an extended period, can enhance the properties of the epoxy adhesive [64]. For example, thermal cycles led to an increase in the tensile properties of the epoxy adhesive, with strength improving between 18 % to 50 % and elastic modulus between 5 % to 25 % compared to initial reference

values, which was attributed to the post-curing phenomenon. The effects of thermal cycles (ranging from -15 °C to 60 °C and 20 °C to 80 °C for up to 180 and 240 days, respectively) were also reported [32] with insignificant improvements of adhesive properties. In a study conducted by Yang et al. [54], it was found that hygrothermal aging, which involves moisture and temperature, led to a decrease in the glass transition temperature of the adhesive, nearing the recommended minimum threshold.

The curing process of epoxy adhesive depends significantly on the temperature. In a study by Moussa et al. [65], lower temperatures significantly extended the curing time of a cold-curing epoxy adhesive: at higher temperatures (60 °C to 35 °C), full curing was achieved within a few hours (3.7 to 1.6 hours), whereas at a lower temperature (10 °C), the curing period extended to 3 days. Additionally, exposing the epoxy adhesive to high levels of CO₂ has been shown to speed up its curing rate [66].

Long-term exposure to outdoor conditions can impact the durability of epoxy adhesives. Negligible changes in the mechanical properties of two epoxy adhesives exposed to outdoor conditions, including UV radiation, temperature, and humidity fluctuations in Salerno, Italy, were observed [67]. Furthermore, inconsistent changes were observed in the properties of commercial epoxy adhesives exposed to UV radiation, moisture, and temperature cycles in a Mediterranean climate, with a slight increase in shear modulus and a decrease in shear strength [59].

Table 2.1 summarizes the reviewed existing studies on the durability of epoxy adhesive, highlighting the environmental conditioning, the addressed adhesive property, major findings/observations, and the quantification of the property retention. It should be noted that the existing literature mainly addresses the effect of accelerated aging test (AAT) conditions with insufficient knowledge of the effects of natural aging test (NAT) conditions. Moreover, current literature generally lacks comprehensive studies on the long-term durability of epoxy adhesives using predictive models. While several studies have analytically and numerically modelled the behavior of the epoxy adhesive under exposure to different degradation agents (e.g., [68–72]), these investigations typically focus on short-term exposure (days or a few months), mainly using the AAT protocols to mimic what is expected to take place in outdoor environments. This is concerning, given that epoxy adhesives are expected to function effectively as bonding fillers and as polymeric matrix in reinforcing materials for many years in service (outdoor).

Table 2.1: Key effects of environmental conditions on epoxy adhesive properties

Ref.	Aging condition	Property investigated	Findings & observations	Retention (%)
[57]	AAT (water immersion)	Tensile strength, Flexural strength	Water absorption and hydrolysis led to significant reductions.	Tensile: 62 %, Flexural: 56 %
[32]	AAT (water immersion)	Glass transition temperature (T_g), Elastic modulus, Tensile strength	480 days of water immersion caused major reductions.	T_g : 14 %, Elastic modulus: 53 %, Tensile strength: 62 %
[62]	AAT (water immersion, and water immersion with 3.5 % NaCl)	Tensile strength, Elastic modulus	Pure water immersion was highly detrimental.	Tensile strength: 62 % Elastic modulus: 53 %
[62]	AAT (wet-dry cycles in tap water (22 ± 2 °C) with 3.5 % NaCl, 240 (WD240), 480 (WD480) days)	Tensile strength, Elastic modulus	General reduction in tensile properties.	Tensile strength: 62 % Elastic modulus: 53 %
[59]	AAT (water immersion)	Flexural strength, Elastic modulus	Immersion at 40 °C led to significant reductions.	Flexural strength: 76 %, Elastic modulus: 70 %
[32]	AAT (freeze-thaw cycles)	Glass transition temperature (T_g)	Freeze-thaw cycles reduced T_g due to curing interruptions.	T_g : 23 %
[61]	AAT (freeze-thaw and dry-wet cycles)	Tensile strength, Elastic modulus, Stress-strain behavior	Initially increased but later degraded under freeze-thaw and dry-wet cycles.	Varied impact
[62]	AAT (chlorides)	Tensile strength	Chloride exposure had minimal impact.	Negligible
[64]	AAT (thermal cycles)	Tensile strength, Elastic modulus	Thermal cycles improved properties due to post-curing.	Tensile strength: 50-82 %, Elastic modulus: 5-25 % (increase)
[54]	AAT (moisture and temperature)	Glass transition temperature (T_g)	Hygrothermal aging reduced T_g significantly.	Near minimum threshold
[65]	AAT (temperature influence on curing)	Curing time	Low temperatures extended curing time drastically.	Not quantified
[66]	AAT (CO ₂ exposure on curing)	Curing rate under CO ₂ exposure	High CO ₂ levels accelerated curing rate.	Not quantified
[67]	NAT (natural outdoor exposure)	Long-term mechanical properties	Minimal changes observed in mechanical properties.	Negligible

2.3.1.3 Durability of CFRP laminate

CFRP laminates have become a popular choice for strengthening and retrofitting reinforced concrete structures due to their high strength-to-weight ratio, corrosion resistance, and ease of installation [12]. Despite these advantages, the long-term durability of the CFRP laminates under service and environmental conditions remains a critical concern, particularly in outdoor or aggressive environments. Over extended periods, exposure to moisture, temperature variations, ultraviolet (UV) radiation, freeze-thaw cycles, and

chemical agents can induce physical and chemical changes in both the carbon fibers and the polymer matrix, potentially affecting the laminate's structural performance. To assess the reliability of CFRP systems over their service life, a wide body of research has focused on evaluating the degradation of key mechanical and thermal properties—including tensile strength, elastic modulus, and glass transition temperature—under various environmental conditions. This section reviews these findings, offering insight into the durability characteristics of the CFRP laminates across different exposure conditions.

Existing studies show that CFRP materials exhibit a high resistance to moisture-related damage [53,55]. Indeed, a recent study on the durability of CFRP laminate strips found that after 240 days of full immersion in tap water, there was only a 3 % decrease in both tensile strength and elastic modulus [12]. However, the combination of moisture and other degrading agents can have a synergistic effect on CFRP laminates. A study explored this by examining the impact of different water-based environments on the durability of three commonly used CFRP laminates [73]. These environments included immersion in demineralized water, water with 35 g/L of sodium chloride, and an alkaline solution, at various temperatures (23 °C, 40 °C, and 60 °C). A reduction in flexural strength was observed across all conditions, with the highest degradation rates occurring at the elevated temperature of 60 °C, showing a decrease between 11 % and 32 % for the demineralized water.

In a study by Choi et al. [74], the durability of the CFRP laminate was highly dependent on the specific epoxy system used, with different commercial CFRP systems exhibiting varying degrees of strength loss under environmental exposure. Wet lay-up CFRP systems generally demonstrated high durability when exposed to water and fluids at different temperatures. However, one system experienced significant strength loss under high temperatures and extended exposure. In contrast, a precured laminate system showed a notable reduction in strength with environmental exposure. Interestingly, exposure to brackish water increased flexural strength due to barnacle buildup, which provided protective reinforcement. Overall, the durability was highly dependent on the type of epoxy system, with different CFRP systems showing varying levels of strength loss under exposure.

Thermal effects on composites can arise from various conditions, including sub-zero temperatures, freeze-thaw cycles, thermal cycles, and elevated temperatures [12]. Exposure to elevated temperatures primarily causes the resin in composite materials to soften, reducing their stiffness and strength. These mechanical properties are temperature-dependent, and temperatures near the T_g lead to increased viscoelasticity of the polymeric matrix in fiber polymer (FRP) composites. At their typical service temperatures, CFRP laminates are generally resistant to thermal cycles. However, due to the differing coefficients of thermal expansion

(CTEs) between carbon fibers and the matrix resin, this can result in matrix microcracking and potentially increase the degradation rate [32]. The impact of thermal cycles (ranging from -15 °C to 60 °C and 20 °C to 80 °C for up to 180 and 240 days, respectively) on CFRP laminate strips was investigated [32]. The study found negligible changes in the tensile strength, ultimate strain, or stiffness of the laminates.

In contrast, high temperatures can also induce a positive post-cure effect, potentially increasing the T_g , as shown in a study by Cromwell et al. [75], where improved tensile properties in CFRP strips were found after exposure to dry heat at 60 °C for 6 and 18 weeks. However, elevated temperatures can also enhance moisture absorption and diffusion, and the combination of heat and moisture can lead to more severe degradation than either factor alone, significantly impacting the durability of composite materials [53]. Generally, thermal cycles cause only minor changes in the stiffness and strength of fiber-reinforced polymer (FRP) materials, unless the temperature variations are exceptionally large [12]. Conversely, in composite materials that use high modulus resins, thermal cycles can result in the formation of microfractures [12]. In a study by Liu et al. [76], subjecting CFRP laminates to 270 thermal cycles in air did not alter their properties while those in water reduced the elastic modulus by only 2 %, which showed resistance of both the resin and fibers against this type of exposure.

Regarding freeze-thaw cycles, the performance of reinforcing fibers is generally unaffected [12]. However, the resin and the fiber-resin interface experience reduced performance when FRP materials are exposed to these cycles. This reduction is due to the differing coefficients of thermal expansion (CTE) between the polymer matrix and the reinforcing fibers. Sub-zero temperatures can enhance the performance of polymer resin-based systems by increasing their strength and stiffness. However, when freeze-thaw cycles occur in the presence of moisture, they can accelerate the degradation of CFRP materials. This degradation is due to the repeated incursions to sub-zero temperatures combined with moisture exposure [12]. The impact of freeze-thaw cycles (ranging from -18 °C to 20 °C for up to 240 days) on CFRP laminate strips was investigated [32]. The study found negligible changes in the tensile strength, ultimate strain, or stiffness of the laminates.

Ultraviolet (UV) radiation can impact CFRP structures used outdoors. UV damage primarily affects the polymer matrix components [12], which are crucial for transferring stresses to and between the reinforcing fibers. Consequently, degradation in these components can significantly impair the mechanical properties of the composite material [77]. The effects of UV radiation exposure, known as photodegradation, on composite materials are primarily confined to the top few microns of the surface, which mainly impacts the aesthetic properties, hence leading to a loss of gloss and discoloration, often referred to as yellowing [12].

Surface damage from UV radiation disproportionately impacts the thermomechanical properties of CFRP composites. This damage can lead to flaws and initiate fractures at lower stress levels compared to unexposed material. Additionally, these flaws can allow moisture to penetrate the material. In outdoor environments, UV effects are often compounded by other factors such as temperature fluctuations, moisture, wind-borne abrasives, and freeze-thaw cycles, further exacerbating the degradation. Carbon fibers are largely unaffected by UV radiation, and the mechanical properties of composites are only slightly influenced by UV exposure. In fact, while photodegradation occurred on the surface of CFRP laminate strips, their flexural strength remained unchanged after 2000 hours of UV exposure [73].

A 10-year outdoor exposure (with three exposure sites exhibiting very different climatic conditions: two exposures in Japan and one in Canada) of FRP products was initiated in a study by Nishizaki et al. [78], and the findings from the first five years were presented. The findings from the study showed that after five years of exposure, CFRP laminates retained 88-100 % of their original tensile strength, with no significant changes observed. However, in-plane shear strength experienced a slight reduction, leading to 80 % of its initial value, with similar outcomes across different exposure sites. Additionally, a decrease in the bending strength of the matrix resin was noted in the early stages of exposure, suggesting a possible reduction in the bonding properties between the carbon fibers and the resin.

In a recent study by Alsuhaibani et al. [79], a long-term prediction model was developed to assess the durability of CFRP laminates exposed to various laboratory and outdoor environmental conditions, including water immersion, temperature, and aging. The research involved immersing samples of a new CFRP laminate in heated water at 23, 45, and 60 °C for 224 days, followed by testing their residual tensile properties. The results from the model were compared with the environmental reduction factor from ACI 440.2R-17 and other common guidelines. The study found that the ACI-specified reduction factor of 0.75 did not account for the age of the CFRP and overestimated the design tensile strength by about 13 %, potentially compromising the safety of retrofitted bridges. Reduction factors from other sources ranged between 0.51 and 0.85. These changes in CFRP properties were reported: (i) the tensile strength decreased by up to 33 % after 224 days of laboratory exposure, with higher temperatures causing more significant degradation, and the field samples aged 12 years showed a 21 % reduction compared to new samples; (ii) the elastic modulus dropped by up to 26 % in the laboratory, also worsening with higher temperatures, and the 12-year-old field samples exhibited a 13 % reduction in elastic modulus; (iii) the failure strain declined by up to 12 % after 224 days, with temperature having minimal effect.

Table 2.2 summarizes the reviewed existing studies on the durability of CFRP laminates, highlighting the aging condition, investigated property, major findings, and the property retention. It is noted that most of existing works were conducted under AAT protocols, while the application of the CFRP in strengthening is typically exposed to outdoor environments. Also, the ECFs in existing standards e.g., [8,9] were derived generally from AAT data, hence, further works are needed, particularly those addressing the NAT conditions.

Table 2.2: Key effects of environmental conditions on the properties of CFRP laminates.

Ref.	Aging condition	Property investigated	Findings & observations	Retention (%)
[12]	AAT (water immersion, 240 days)	Tensile strength (TS), Elastic modulus (EM)	Synergistic effects may impact CFRP performance.	97 % (for both TS and EM)
[73]	AAT (water, NaCl, alkaline at 60 °C)	Flexural strength	Highest degradation in demineralized water at 60°C.	68 % - 89 %
[74]	AAT (water and fluids, various temperatures)	Flexural strength, Tensile strength	Durability depended on epoxy system; precured laminate degraded more.	Varied by system
[32]	AAT (thermal cycles, -15 °C to 80 °C, 180-240 days)	Tensile strength, Strain, Stiffness	Negligible changes in properties.	Negligible
[62]	AAT (wet-dry cycles in tap water (22 ± 2 °C) with 3.5 % NaCl, 240 (WD240), 480 (WD480) days)	Tensile strength, Elastic modulus	Small reductions of the tensile properties.	93 % of strength for WD480, 97.4 % of modulus for WD240
[75]	AAT (dry heat at 60 °C, 6-18 weeks)	Tensile strength (TS), Elastic modulus (EM)	Post-cure effect improved tensile properties.	No significant degradation, ECF: 0.9 proposed for both TS and EM
[76]	AAT (270 thermal cycles, -18 °C to 30 °C, 2160 hours): in air or water	Elastic modulus for thermal cycles in air and water	No significant degradation observed.	No degradation (air), 98 % (water)
[77]	AAT (UV radiation)	Surface property, Mechanical properties	UV mainly affected surface, leading to discoloration and microdamage.	Minor (aesthetic)
[78]	NAT (5-year of outdoor exposure)	Tensile strength, Shear strength	Shear strength reduced. Minor reduction in tensile strength	Shear strength: 80 %, Tensile strength: 88-100 %
[79]	NAT (12-year of outdoor exposure)	Tensile strength, Elastic modulus	Tensile strength and elastic modulus decreased with time.	Tensile: 79 %, Elastic modulus: 87 %
[79]	AAT (heated water immersion, 224 days)	Tensile strength, Elastic modulus, Failure strain	Tensile strength and elastic modulus decreased with time.	Tensile: 67 %, Modulus: 84 %, Strain: 88 %

Figure 2.5 presents a summary of key findings related to the contents discussed in **Sections 2.3.1.2** and **2.3.1.3**. The figure provides a comparative analysis of property degradation in epoxy adhesives and CFRP laminates, focusing on tensile strength (TS), elastic modulus (EM), flexural strength (FS), glass transition temperature (T_g), and shear strength (SS) after exposure to various environmental conditions. These conditions include water immersion (WI), wet-dry cycles (WD), freeze-thaw cycles (FT), thermal cycles (TC), thermal cycles in water (TCW), heated water immersion (HTW), and results from NAT protocols. The findings indicate that epoxy adhesive properties are more susceptible to degradation than those of CFRP laminates.

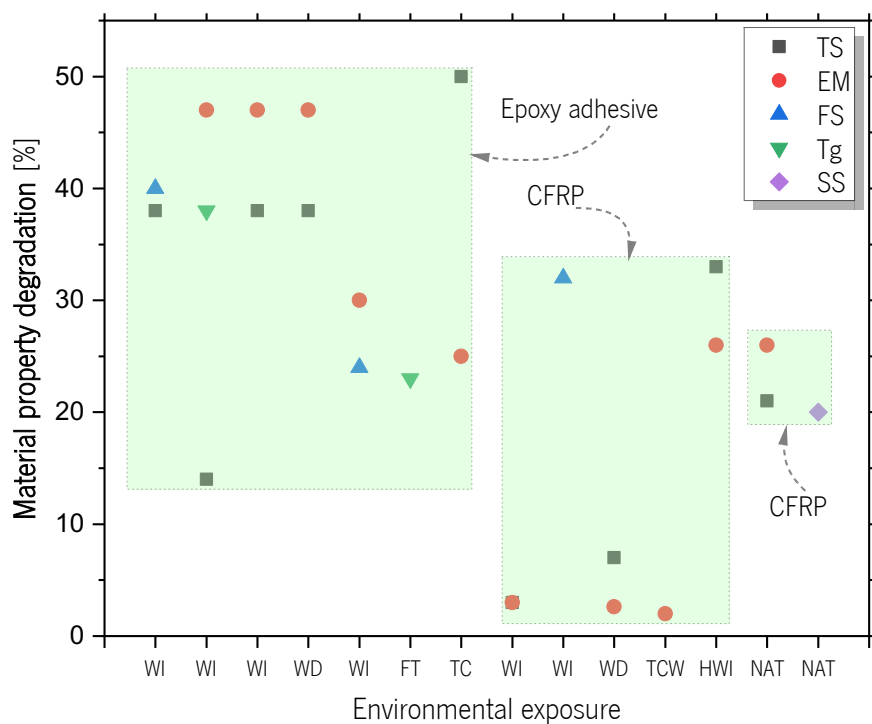


Figure 2.5: Comparative perspectives on the property degradation of epoxy adhesive and CFRP laminate after exposure to different environments (WI: water immersion, WD: wet-dry, FT: freeze-thaw, TC: thermal cycles, TCW: thermal cycles in water, HWI: heated water immersion, NAT: natural aging test, TS: tensile strength, EM: elastic modulus, FS: flexural strength, T_g : glass transition temperature, SS: shear strength).

The adhesive experiences degradation percentages predominant in the range of 25–50 % under AAT protocols, whereas CFRP laminates generally exhibit degradation percentages below 25 %. This suggests that the adhesive undergoes more significant deterioration due to moisture absorption, thermal expansion, and cyclic stress conditions compared to the more resilient CFRP laminates. Hence, the existing literature highlights the critical role of environmental exposure in the long-term performance of bonded composite systems. However, as previously mentioned (also obviously seen in **Figure 2.5**), most of the available data come from AAT protocols, which may not fully capture real-world degradation behavior. Variations in

formulations, testing methods, and environmental factors may further limit definitive conclusions. Therefore, additional studies under NAT protocols for prolonged periods are necessary to enable comparative analysis and establish appropriate environmental conversion factors.

2.3.2. Durability and long-term behavior of concrete elements with CFRP systems

2.3.2.1 Durability of EBR technique

The application of EBR using CFRP laminates has become a widely adopted technique for strengthening and rehabilitating RC structures due to its high efficiency, minimal added weight, and ease of application [2]. However, its long-term durability is a critical aspect influencing the effectiveness and safety over the service life of the strengthened RC structure. The performance of these systems is governed not only by the intrinsic properties of the CFRP laminate but also by the integrity of the adhesive bond and the interaction with the concrete substrate [2]. Environmental exposures—such as moisture, elevated temperatures, ultraviolet (UV) radiation, freeze–thaw cycles, and aggressive chemical agents—can lead to degradation in both the laminate and the adhesive interface, potentially resulting in reduced bond strength, delamination, or premature failure. Numerous experimental studies have been conducted to assess the degradation of mechanical, thermal, and interfacial properties of EBR CFRP-to-concrete bond under various conditioning regimes. This section presents a systematic review about this topic, with a focus on property evolution such as bond strength, fracture energy, and failure mode.

The durability of the EBR CFRP-to-concrete bond has been extensively studied using AAT protocols. The existing relevant studies include [75,76,80–87]. The findings from these studies indicate that certain conditions can affect or enhance the bond strength. For instance, freeze-thaw cycles (FT) of 20 cycles [75], 50, 150, and 300 cycles [80], and 100 and 200 cycles (for specimens with a 400 mm bond length) [82] increased bond strength. However, a decrease in bond strength was noted after 300 cycles [84] and after 30, 60, and 90 cycles [86]. Also, weakening of concrete surface under the action of 300 freeze-thaws was responsible for the loss of the bond strength and led to a 25.3 % decrease in the load-carrying capacity of the CFRP strengthened concrete element [84].

Water immersion tests also showed varied results. Immersion for 1000 and 3000 hours at temperatures of 22 °C and 38 °C [75], and for 2880 hours [84], increased bond strength. Conversely, immersion for 330 to 1340 hours [85] and for 10,000 hours at 22 °C [75] led to a reduction in bond strength.

Temperature effects on the behavior of EBR can increase or decrease the EBR properties. For example, exposure to 80 °C decreased bond strength by 25 % for the case of CFRP laminates and up to 54 % for

the case of CFRP sheets, and increasing the temperature from 50 to 80 °C changed the failure mode from being concrete cohesive to adhesive failure [81], but exposure to 60 °C for 1000 and 3000 hours [75] decreased it. Elevated temperatures higher than the T_g of the epoxy adhesive did not deteriorate the bond strength, which was attributed to the friction forces and chemical adhesion at the concrete-adhesive interface, hence indicating that a critical temperature could be defined in practical applications (depending on the stress level installed in the strengthening system) for both service and fire conditions [88]. Thermal cycling between -18 and +30 °C for 2160 hours in air increased bond strength by 35 %, while the same cycles in water decreased it by 9 %, thereby proposing a reduction factor of 0.7 [76]. Hygrothermal conditions at 40 °C and 95 % relative humidity (RH) for 4200 hours increased bond strength, but a decrease was observed when the duration was extended to 8472 hours [83].

In a study by Cruz et al. [89], the bond strength between CFRP laminates and concrete using the EBR technique exhibited minimal degradation under both real-time field conditions and accelerated laboratory aging conditions. The maximum pull-out force, indicating bond strength, ranged from a 4 % decrease to a 16 % increase during real-time field exposure, while specimens immersed in water showed an approximate 8 % reduction.

Table 2.3 summarizes the major findings from the reviewed existing works, focusing on the aging condition, exposure duration, and the resulting effect on the EBR bond performance. The bond property is found to increase or decrease, and most of studies are AAT-based just as the epoxy and CFRP laminates discussed in the previous sections. The studies demonstrate considerable variability in the findings, underscoring the intricate nature of the durability of EBR CFRP-to-concrete bond under various accelerated aging conditions. It is found crucial to quantify and correlate the effects observed in AAT-based tests with those from NAT to establish a consistent criterion between the two, particularly leading to appropriate deductions of the environmental conversion factors. However, this could be reached only if the NAT data are as sufficient as AAT data, which therefore raises an important gap in the literature. Hence, future works can mainly focus on developing works that address the NAT protocols, given that the AAT-based works are significant in the existing literature. With sufficient NAT data, appropriate correlations can then be achieved, thereby leading to appropriate derivations of the environmental conversion factors.

Table 2.3: Key effects of environmental conditions on EBR CFRP-to-Concrete bond properties.

Ref.	Aging condition	Property investigated	Exposure duration	Effect on bond property
[76]	Thermal cycling in air (AAT)	Interfacial fracture energy	-18 °C to +30 °C for 2160 hours	Increased by 35 %
[76]	Thermal cycling in water (AAT)	Bond strength	-18 °C to +30 °C for 2160 hours	Decreased by 9 %, a reduction factor of 0.7
[75]	Freeze-thaw (AAT)	Bond strength (BS), ECF	20 cycles	Slight increase in BS, ECF of 0.9 proposed
[75]	Water immersion (AAT)	Bond strength	1000 (WI1K), 3000 (WI3K) hours (22 °, 38 °C)	Increased by 9 % (WI1K) and 14 % (WI3K)
[75]	Saltwater (AAT)	Bond strength	3000 (SW3K) and 10000 (SW10K) hours (22 °C, 38 °C)	Increased by 30 % (SW3K) and decreased by 6 % (SW10K)
[75]	Temperature exposure (AAT)	Bond strength	60 °C for 1000 (WI1K), and for 3000 (WI3K) hours	Decreased by 9 % (WI1K) and by 2 % (WI3K)
[80]	Freeze-thaw (AAT)	Bond strength	Up to 300 cycles	Increased up to 43 %
[81]	Temperature exposure (AAT)	Bond strength (BS), Failure mode (FM)	Exposure to 50 °C and 80 °C	BS decreased by 25 % at 80°C, and the FM changed from CC to A/F
[82]	Freeze-thaw cycles from -18 °C to +4 °C (AAT)	Bond strength (BS), Bond length (BL), Concrete compressive strength (CS)	100, 200 cycles (400 mm BL) for 5 hours each cycle	Marginal effect on BS, 2 % reduction of CS
[83]	Hygrothermal conditioning (AAT)	Bond strength (BS), Failure mode (FM)	40 °C, 95 % RH for 400 days	10 % increase of BS, FM changed from substrate to polymer cohesive failure
[84]	Freeze-thaw cycles: 16h freezing at -18 °C and 8h thawing at +15 °C (AAT)	Bond strength (BS), Failure mode (FM)	300 cycles	BS decreased by 25.3 %, FM was concrete cohesive
[84]	Saltwater immersion (AAT)	Bond strength (BS), Failure mode (FM)	2880 hours	Slight increase of BS by 4.6 %, No change in FM
[85]	Water immersion: 20 °C and 50 °C (AAT)	Bond strength (BS), Failure mode (FM)	336 hours (2W) – 1008 hours (6W)	Decreased by 30 % at 20°C for 6W and by at 50°C for 2W. FM changed from CC to A/F
[86]	AAT: Freeze-thaw cycles: -20 °C for 10h and +30 °C for 10h in water and 90 % RH	Bond strength (BS), Fracture energy (FrEg), Failure mode (FM)	Up to 90 cycles	BS decreased by 61 %, FrEg decreased by 51 %, FM remains concrete cohesive (CC)
[86]	Freeze-thaw cycles: -20 °C for 10h and +30 °C for 10h, 90 % RH (AAT)	Bond strength (BS), Fracture energy (FrEg), Failure mode (FM)	Up to 90 cycles	BS decreased by 43 %, FrEg decreased by 32 % FM shifts from concrete cohesive (CC) to adhesive failure (A/F)
[89]	Real-time field conditions (NAT)	Bond strength	2 years	16 % increase

Notes: CC: cohesive failure, A/F: adhesive failure, ECF: environmental conversion factor

2.3.2.2 Durability of NSM technique

The NSM technique has emerged as a durable and efficient method for strengthening reinforced concrete (RC) structures using CFRP reinforcements embedded in pre-cut grooves and bonded with epoxy adhesives [2]. Compared to EBR systems, NSM offers enhanced protection of the CFRP material and the adhesive from direct environmental exposure [15,90], improving bond performance and reducing the risk of premature debonding. Nevertheless, the long-term durability of NSM CFRP-to-concrete bond remains a key concern, particularly under harsh environmental conditions. Factors such as moisture ingress, elevated temperatures, freeze-thaw cycles, and chemical exposure can still impact the epoxy adhesive performance, and in some cases, the bonded CFRP itself, thereby resulting in overall poor bond performance. Several studies have examined the degradation of mechanical and bond-related properties—such as bond strength and failure mode to assess the durability of NSM systems. This section summarizes the findings from existing relevant works.

Several studies, e.g., [62,84,91,92], have been conducted to assess the performance of the NSM CFRP-to-concrete bond under AAT conditions. In a study by Sena-Cruz et al. [91], concrete elements strengthened with CFRP laminates were subjected to 90 wet-dry cycles over 2160 hours. These elements had bond lengths of 60 mm and 90 mm, CFRP laminate thickness of 1.4 mm, groove depths of 15 mm and 25 mm, and groove widths of 4 mm and 8 mm. Generally, the bond strength decreased after this exposure, except for specimens with a groove depth of 25 mm and a groove width of 4 mm. Furthermore, subjecting specimens to 300 freeze-thaw cycles or immersing them in saltwater at 20 °C reduced the bond strength of concrete elements reinforced with a 1.3 mm thick CFRP laminate [84].

In a study by Fernandes et al. [62], immersing concrete elements strengthened with a 1.4 mm CFRP laminate in tap water at 22 °C (or tap water with 3.5 % NaCl) for 5760 and 11,520 hours increased bond strength. Similarly, 240 or 480 wet-dry cycles in tap water (or tap water with 3.5 % NaCl) also enhanced bond strength. Temperature cycles between -15 °C and +60 °C increased the bond strength, whereas cycles between +20 °C and +80 °C had no effect. Additionally, 120 freeze-thaw cycles between -18 °C and +20 °C for up to 5760 hours did not affect the bond strength, but doubling the cycles over 4320 hours caused a slight decrease. In a study by Mitchell [3], the epoxy adhesive strengthened pull-out bond tests experienced a 27 % average drop in ultimate load after 150 freeze-thaw cycles.

A few of existing studies, e.g., [30,93], have explored the durability of NSM CFRP-to-concrete bond using NAT protocols. These NAT-based studies generally found that the bond is significantly impacted by moisture and freeze-thaw cycles.

Table 2.4 puts together important findings observed in the reviewed literature about the NSM CFRP-to-concrete bond. It is noted that the bond strength decreased in many studies as compared to those that reported an increase in the strength; furthermore, the failure mode is dominated by adhesive failure. Like what is previously mentioned for the case of the EBR technique, the literature on NSM CFRP-to-concrete bond reveals several knowledge gaps. All the mentioned gaps for the case of the EBR in **Section 2.4.2.1** also apply for the case of the NSM technique. Therefore, further research is needed, particularly those addressing the effects of outdoor exposures.

Figure 2.6 presents a summary of key findings related to the contents discussed in **Sections 2.3.2.1** and **2.3.2.2**. The figure provides a comparative analysis of bond property degradation for the EBR CFRP-to-concrete and NSM CFRP-to-concrete bond systems, focusing on the bond strength (BS), fracture energy (GF), and ultimate load (UL) properties. The considered environmental conditions include water immersion (WI), saltwater (SW), high temperatures of 60 °C (T60) and 80 °C (T80), freeze-thaw cycles (FT), elevated temperatures (HT), thermal cycles (TC), wet-dry cycles (WD), and natural outdoor exposure (NAT).

From the above comparative analysis, it can be noted that out of 15 studies on the bond strength variation in the EBR CFRP-to-concrete system, 8 reported degradations (positive values on the degradation y-axis), while 7 reported a resistance to degradation (negative values on the degradation y-axis), suggesting that the effects of environmental conditioning on the BS in this system are not entirely uniform. This indicates that the performance of the EBR CFRP-to-concrete system may be influenced by different factors, such as environmental conditions, the specific constituent materials used in the studies, or the application methods, leading to either degradation or resistance to degradation in different instances.

Contrary to the EBR CFRP-to-concrete system, the NSM CFRP-to-concrete system tends to present only degradation for all considered literature, with only one study (out of 12 studies) reporting resistance to degradation (see **Figure 2.6**). This suggests that environmental conditioning consistently leads to a deterioration of bond strength in the NSM CFRP-to-concrete system. The uniformity of degradation across the studies may indicate a more significant vulnerability of the NSM CFRP-to-concrete bond to environmental factors, possibly due to differences in bonding mechanisms or the materials used compared to the EBR CFRP-to-concrete system. It should be noted that effects from accelerated aging test (AAT) and NAT protocols are similar for this system, with a bond strength degradation varying between 1-10 % degradation.

This suggests that environmental conversion factors can easily be deduced from the observed similar trends, as compared to the EBR CFRP-to-concrete system.

Table 2.4: Key effects of environmental conditions on NSM CFRP-to-concrete bond properties.

Ref.	Aging condition	Property investigated	Exposure duration	Effect on bond property
[3]	Freeze-thaw (AAT)	Ultimate load (UL)	150 cycles	UL decreased by 27 %
[30]	Freeze-thaw cycles (NAT), Water immersion at 20 °C (AAT)	Bond strength (BS), Failure mode (FM)	2 years	BS decreased by 8 % (NAT), by 12 % (AAT), FM by A/F
[62]	Temperature cycles, between -15 °C and +60 °C (AAT)	Bond strength (BS), Failure mode (FM)	2 years	BS increased by 8 %, FM by A/F
[62]	Warm and temperate environment with temperature between -1.9 and 40 °C (NAT)	Bond strength (BS)	2 years	BS decreased by 11.6 %, FM by A/F
[62]	Tap water immersion at 22 °C or with 3.5 % NaCl (AAT)	Bond strength	5760, 11520 hours	Negligible
[62]	Wet-dry cycles in marine environment with temperature between 3.8 - 36.6 °C (NAT)	Bond strength (BS), Failure mode (FM)	2 years	BS decreased by 11.5 %, FM by A/F
[62]	Thermal cycles: -15 °C to 60 °C (TCA) and +20 °C and +80 °C (TCB) (AAT)	Bond strength (BS)	Each cycle lasting for 24h for up to 240 days	BS increased by 8 % (TCA) and negligible (TCB)
[62]	Freeze-thaw (-18 °C to +20 °C) (AAT)	Bond strength, Failure mode (FM)	120 cycles (-18 °C to 20 °C, 5760 hours)	Slight increase of 7 %, FM from A/F to CC
[62]	Freeze-thaw cycles (AAT): 240 cycles (-18°C for 13hours to +20°C for 3hours), 5h for transition from - to + and 3h from + to -	Bond strength (BS), Failure mode (FM)	4320 hours	Slight decrease of BS by 4.8 %, FM from A/F to CC
[84]	Freeze-thaw (AAT)	Bond strength (BS), Failure mode (FM)	300 cycles	Slight decrease of BS by 1.8 %, FM by F/A
[84]	Saltwater immersion (AAT)	Bond strength (BS), Failure mode (FM)	20 °C for 120 days	Slight decrease of BS by 3 %, FM by F/A
[91]	Wet-dry cycles (AAT)	Bond strength (BS) for groove sections of width/depth of 4/15 [mm] (GrA) and 4/25 [mm] (GrB)	90 cycles (2160 hours)	BS decreased by 11 % for GrA, BS increased by 10 % for GrB
[92]	Wet-dry cycles (AAT): 90 wet-dry (WD) cycles, each lasting 24 hours: 12 hours of immersion in 3 % NaCl solution at 20 ± 1 °C, followed by 12 hours of drying at about 30 ± 1 °C	Bond strength (BS) for groove sections of width/depth of 4/15 [mm] (GrA) and 4/25 [mm] (GrB), bond length of 60 mm (Bl.1) and 90 mm (Bl.2)	Between 150 and 600 days	BS decreased by 12 % for GrA and Bl.1; BS decreased about 6 % for GrA and Bl.2; mixed failure mode

Notes: CC: cohesive failure, A/F: adhesive failure

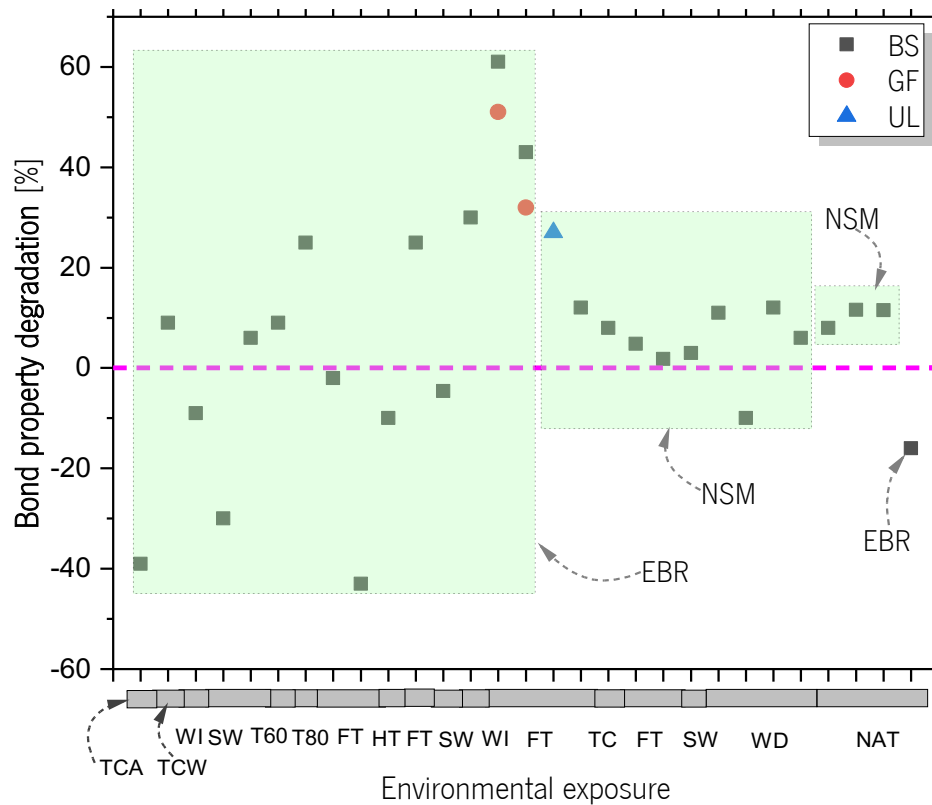


Figure 2.6: Comparative analysis on the bond property degradation in EBR and NSM CFRP-to-concrete systems (WI: water immersion, SW: saltwater, T60: 60 °C temperature, T80: 80 °C temperature, FT: freeze-thaw, WD: wet-dry, NAT: natural aging test, BS: bond strength, GF: fracture energy, UL: ultimate load, EBR: externally bonded reinforcement, NSM: near-surface mounted).

On the other hand, the fracture energy and ultimate capacity of the EBR/NSM CFRP-to-concrete bond degrade after exposure to freeze-thaw cycles, highlighting susceptibility of the systems to cyclic environmental conditions. Freeze-thaw cycles lead to repeated expansion and contraction, which can cause micro cracking and weakening of the bond between the CFRP and concrete.

2.3.2.3 Durability and long-term performance of RC beams/slabs with CFRP systems

CFRP systems, applied through the EBR CFRP or NSM CFRP techniques, have become common solutions for strengthening RC beams and slabs. These systems significantly enhance structural performance, particularly in terms of load-carrying capacity and resistance to cracking. However, the durability and long-term performance of RC elements strengthened with CFRP laminates has not been fully understood yet, due to limited number of relevant works on full-scale specimens in the literature [94]. The long-term behavior of these strengthened elements under sustained loads, particularly in diverse environmental conditions, remains a critical area of study. Environmental factors such as moisture, temperature fluctuations, freeze-thaw cycles, and exposure to chemicals can degrade both the CFRP laminate and the adhesive bond,

potentially affecting the overall performance of the system. Additionally, literature highlights the influence of sustained loading on concrete behavior, with factors such as creep, shrinkage, and long-term deflections contributing to the evolution of structural performance over time. This section explores the durability and long-term behavior of RC beams and slabs strengthened with CFRP systems, considering the interplay between environmental exposures and sustained load effects.

Several existing studies address effects of environmental conditions on the behavior of RC beams/slabs with CFRP laminates under prolonged exposures, e.g., [3,19,20,35,95,96]. For the case of freeze-thaw cycling, repeated freezing and thawing can pose potential problems for concrete, such as cracking, scaling, and spalling, particularly if proper amounts of air entrainment are not achieved during mixing, casting, and consolidation [3]. The durability of RC slabs with CFRP strengthening was examined [20] after 8 months of exposure to different environments, namely, (i) water immersion at 20 °C; (ii) water immersion with 3.5 % of dissolved chlorides at 20 °C; (iii) wet-dry cycles in a tank with a water temperature of 20 °C. In addition, half of the used slabs were subjected to a sustained load corresponding to the 1/3 of the ultimate load, with the occurrence of cracking. The results showed that the slabs resisted the imposed conditions with slight losses of performance and ductility. In another study by Lu et al. [95], the durability of RC beams with prestressed CFRP sheets under wet-dry cycling in a chloride-containing environment also showed insignificant change in the flexural performance of the beams for up to 90 days of exposure. Exposure to chloride-containing environments increased prestress losses in CFRP-strengthened RC beams. However, these losses stabilized after 30 days at room temperature, with a 15 % prestress loss recorded after 60 days.

The behavior of slabs with CFRP strips and with epoxy adhesive as a bonding agent under various environmental conditions, including sustained loads (ranging from 59 % to 78 % of the non-strengthened beams' ultimate static capacities) was studied deeply in a study by Mitchell [3]. The experiments involved testing RC slabs subjected to sustained loads under various environmental conditions: (i) in laboratory premises; (ii) immersed in water at 20 °C with no chlorides; (iii) immersed in water at 20 °C with 3.5 % chlorides; and (iv) exposed to wet/dry cycles at 20 °C with 3.5 % chlorides. Continuous monitoring was conducted for mid-span vertical deflection. Observations included creep, degradation, and their relationship with temperature. Furthermore, the slabs showed more pronounced creep effects which were reduced when immersed in water. The slab displayed minor changes in ultimate load (less than 3 %) after exposure to freeze-thaw cycles or a period of sustained load. However, a reduction of up to 8 % was observed for the slabs with the combined effect of freeze-thaw cycles and sustained load.

Other existing works on the long-term behavior of RC beams/slabs under different environmental conditioning can be found, e.g., [97–105], from which the important findings in terms of property investigated, exposure duration, and major findings are summarized in **Table 2.5**. From this table, it can be noted that the exposure time varied from hours to years, hence covering a considerable time range to investigate effects of conditioning on different properties including the ultimate load, mid-span instantaneous and long-term deflections, creep coefficient, CFRP strains, and failure modes. The table also highlights the methods used in some studies to predict analytically the long-term behavior of the beams/slabs, such as the effective modulus method (EMM) and age-adjusted effective modulus method (AEMM). It should be noted that the EMM and AEMM methods are based on modification of the time-dependent concrete modulus of elasticity using a time-dependent concrete creep coefficient, where in the latter method the creep coefficient is multiplied by a reduction factor (usually 0.8), namely, the aging coefficient to account for the fact that the creep coefficient presents different values for each stress increment when the stress is applied gradually on a concrete. However, it should also be noted that the AEMM is not limited to concrete, as it can also be applicable to other materials to examine the change of their modulus over time using relevant constitutive models.

Figure 2.7 presents a summary of key data from the reviewed studies on the long-term performance of RC beams/slabs after extended exposure periods for different environmental conditioning. The conditioning considered include effects of constant sustained load (SL), increased sustained load (SL+), increased temperature (T+), wet-dry cycles (WD), water immersion (WI), saltwater (SW), direct sunlight (DS), varied temperature and relative humidity (VTR), high temperature (HT), room temperature (RT), and finally natural outdoor conditioning (NAT).

The data shown in **Figure 2.7** resulted from a combination of two or more of the above conditioning, that is, the resulting combinations are shown on y-axis of the figure. For example, SL+/VTR indicates a combination of increased sustained load and a varied temperature and relative humidity, SL/HT indicates a combination of increased sustained load and high temperature, and SL/T+ indicates a combination of constant sustained load and increased temperature. The parameters investigated are the evolution of CFRP strain, long-term deflection, ultimate load (obtained after exposure), prestress loss (after exposure), instantaneous deflection, and crack initiation and growth over time. These parameters are classified as either property increase (if the related study reported an increase after exposure) or property decrease (if the related study reported a decrease after exposure).

Table 2.5: Key effects of aging conditions on flexural behavior of CFRP strengthened RC beams/slabs.

Ref.	Aging condition	Property investigated	Exposure duration	Findings & observations
[3]	Sustained load and/or 300 freeze-thaw cycles (AAT)-NSM	Ultimate load	24 hours	Minor reduction (up to 3 %); 8 % reduction under combined conditions
[19]	Accelerated aging exposure (AAT): water immersion (WI), 20 °C and 55 % relative humidity	Mid-span deflection (MD) and creep coefficient (CrCt) for different types of strengthening	3 years	Varied CrCt depending on the strengthening and exposure type
[19]	Natural aging exposure (NAT): freeze-thaw attacks, carbonation, chlorides, and high temperatures	Mid-span deflection (MD) and creep coefficient (CrCt) for different types of strengthening	3 years	Varied CrCt depending on the strengthening and exposure type
[20]	AAT: water immersion in tap water at 20 °C, and tap water with 3.5 % of chlorides, and wet-dry cycles in tap water at 20 °C, Sustained load (SL)	CFRP strains (S), Failure mode (FM), Ductility (D) of both mechanical (MA) and gradient (GA) anchorage systems	8 months	Higher S due to prestressing systems, Debonding FM dominated, reduced D for MA due to SL
[94]	Full-scale RC beams under different levels of sustained load (SL) applied after 6-12 months (AAT), temperature variation between 19-24 °C, relative humidity variation between 30-59 %	Creep behavior: long-term deflection (LTD) and creep coefficient (CrCt), Ultimate load capacity (ULC), Concrete compressive strength (CCS)	Up to 573 days	Strengthening increase UCL but no improvement in LTD, CrCt is inversely proportional to LTD; strengthening presents lower LTD compared to non-strengthened beams, CCS and SL are the major factors of CrCt and LTD
[95]	Wet-dry cycles in a chloride environment (AAT)	Flexural performance, Failure mode	90 days	Insignificant change in flexural performance, Debonding, FM for prestressed CFRP without anchorages, CFRP rupture FM for prestressed CFRP with anchorage or U-jackets systems
[95]	Wet-dry cycles in a chloride environment (AAT)	Prestress loss in CFRP after different stages of exposure	60 days	Prestress losses of CFRP stabilized after 30 days, 15 % prestress loss in post-releasing stage after 60 days
[96]	Coupled hot-wet and uncoupled hot-wet environments (AAT)	Durability of CFRP strengthening	Not specified	Hot-wet environment presented significant effect on performance of CFRP strengthened RC beam
[97]	Saline water-SW (AAT), Direct sunlight-DS (AAT)- CFRP sheets	Bond reduction factor (BRF), Overall flexural reduction factor (OFRF), Ultimate load (UL)	Up to 24 months	BRF of 0.75 (SW) and 0.95 (DS); OFRF of 0.85 (SW) and 0.95 (DS); the UL of RC with CFRP sheets increased by 47 %

Table 2.5: Key effects of aging conditions on flexural behavior of CFRP strengthened RC beams/slabs (Cont.).

Ref.	Aging condition	Property investigated	Exposure duration	Findings & observations
[98]	Effects of long-term exposures to service temperatures (20 °C and 50 °C)	Deflection (experimental and analytical studies-using AEMM)	Up to 200 days	Deflection increased with increased temperature under sustained load
[99]	Effect of different levels of sustained loading (60 and 80 % of nominal strength)	Deflection (experimental and analytical studies-using AEMM)	Up to 100 days	Deflection increased with increased sustained load, Modified AEMM predicted well the experimental data
[100]	Effect of levels of sustained loading (40 and 71 % of flexural capacity)	Instantaneous (ID) and long-term (LT) deflection (experimental and analytical studies-using modified AEMM and EMM)	600 days	Strengthening reduced the ID and crack width but had no effect on LT and crack width, LT increased with increased sustained load, AEMM with CEB-FIP predicted better while EMM with ACI predicted better the experimental results
[101]	Wet or dried environments (AAT)	Effects on 20 % and 40 % prestressed CFRP	170 days	40 % prestress hinders crack initiation and growth, prestress loss can be compensated by concrete cracks, 20 % prestress was more susceptible to moisture
[102]	High temperatures and solar radiation and sustained load (NAT)	CFRP strains, Failure mode (experimental study)	4 years	CFRP strains increased substantially with time for non-prestressed EBR systems, Prestressed EBR systems experienced premature failure (after 2 years) attributed to elevated temperatures and sustained loads
[103]	Natural exposure (NAT): hygrothermal exposure in subtropical environments	Ultimate load (UL) of RC strengthened with CFRP systems under sustained loading, Interface behavior, Failure mode	6 years	No significant difference between UL of non-strengthened and strengthened RC beams, Reduced fatigue (30-80 %) for the strengthened beams
[104]	Effect of different levels of sustained loading (38 and 56 % of flexural capacity)	Instantaneous (ID) and long-term (LT) deflection (experimental and analytical studies-using AEEM and EMM as per ACI committee)	200 days	Strengthening reduced the deflection for up to 28 % and 46 % for uncracked and cracked beams, respectively Higher CFRP reinforcement ratio led to smaller long-term deflection, CFRP reduces both ID and LT
[105]	Effect of different levels of sustained loading: 21 % (uncracked state) and 30 % (cracked state) of flexural capacity	Instantaneous (ID) and long-term (LT) deflection (experimental and analytical studies-using modified EMM as per ACI 440)	6 months	The LT after 6 months reached 42 % and 46 % of ID for uncracked and cracked beams, respectively. CFRP strengthening controlled both ID and LT, tension stiffening model is conservative with 18 % difference compared to experimental data

Notes: EMM: effective modulus method, AEMM: age-adjusted effective modulus method, Ref.: reference/citation

The letters S and P shown in **Figure 2.7** stand for the studies in which the CFRP strengthening system was applied without prestressing systems and with prestressing systems, respectively.

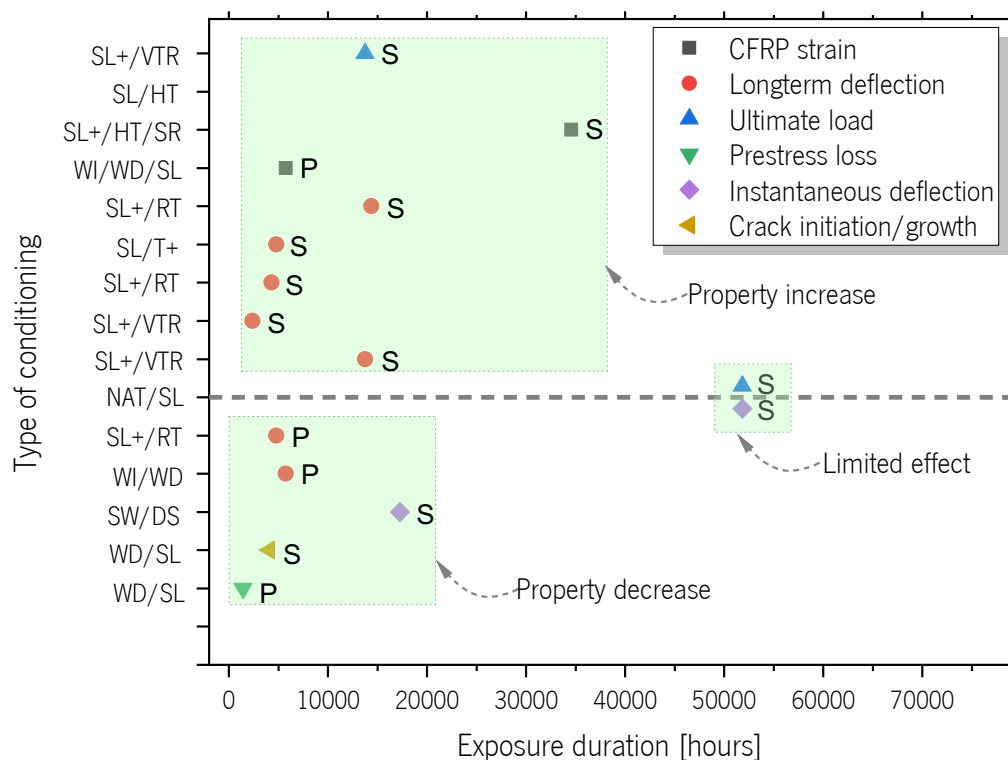


Figure 2.7: Comparative analysis on benefits of CFRP strengthening systems under different environmental conditions (SL: sustained load, SL+: increased SL, T+: increased temperature, VTR: varied temperature and relative humidity, HT: high temperature, SR: solar radiation, WI: water immersion, WD: wet-dry, RT: room temperature, NAT: natural aging test, SW: saltwater, DS: direct sunlight, P: strengthening with prestressed CFRP laminate, S: strengthening with non-prestressed CFRP laminate).

It can be noted that most of existing reviewed works have generally been conducted using non-prestressed CFRP systems, highlighting limited knowledge about long-term performance of prestressed CFRP systems. Furthermore, the figure shows that the long-term deflection is the major variable reported in most of the reviewed works. The long-term deflection is found to increase with time for both non-prestressed and prestressed CFRP systems but for the latter the increase is less compared to the former (which is the reason why it is classified as a property decrease in **Figure 2.7** to showcase this difference). The observed increase of deflection explains the effects of creep over time. The CFRP strains and ultimate load (after exposure) are also observed to increase with time due to conditioning type and exposure duration. Furthermore, the crack initiation (and growth) and instantaneous deflection are found to decrease thanks to the applied strengthening systems, and finally, the prestress loss is decreased as its increase is limited by the concrete cracking over time. For the studies where a combination of NAT and SL was used, a limited effect in terms

of ultimate load and instantaneous deflection was reported; however, very limited works exist, and hence this cannot be a representative of what normally takes place in outdoor environments; therefore, there is a need to increase related works.

Overall, the creep effects in RC beams/slabs with CFRP strengthening systems are evident from the reviewed studies (increased deflection over time) and the benefits of CFRP strengthening systems are also clear in terms of limiting instantaneous deflection and crack growth over time. Further analysis indicates that prestressed CFRP systems tend to perform better over time in terms of deflection control and load-bearing capacity, as compared to their non-prestressed counterparts. This could be attributed to the additional preloading provided by the prestressing process, which helps mitigate the effects of environmental conditions and reduces the development of excessive deflection. However, the increase in CFRP strains over time suggests that both systems, particularly the non-prestressed ones, are still susceptible to long-term deflection increase due to environmental exposure. It is also important to note that the benefits of strengthening can be even more pronounced in CFRP prestressed systems due to reduced instantaneous deflection and delayed crack initiation and growth. Nonetheless, the long-term performance of both systems is influenced by various factors, including exposure conditions, strengthening techniques, and exposure duration, warranting further investigation into the long-term durability and performance of non-prestressed/prestressed CFRP systems under diverse environmental conditions.

2.4. MODELING OF SHORT- AND LONG-TERM BEHAVIOR OF RC ELEMENTS WITH CFRP STRENGTHENING

In this section numerical modeling of the EBR CFRP-to-concrete joint (small-scale) and flexural behavior of RC beams/slabs (full-scale) are addressed. **Section 2.4.1** discusses the available data on the numerical short- and long-term modeling of the EBR CFRP-to-concrete joint mainly considering studies in which small-scale specimens were used. This is followed by **Section 2.4.2** in which analytical modeling of both short- and long-term flexural behavior of RC beams/slabs is addressed. It is worth noting that the short-term modeling of flexural behavior of RC beams/slabs refers to the analytical simulation of the flexural tests performed initially before exposure, while the long-term modeling solely focuses on analytical predictions of both instantaneous and time-dependent deflections.

2.4.1. Numerical modeling of short- and long-term performance of EBR CFRP-to-concrete joint

Numerical modeling of the CFRP-to-concrete joint in externally bonded reinforcement (EBR) systems is essential for understanding the long-term performance of strengthened structures, particularly under

environmental degradation. Water immersion and moisture exposure are among the most significant factors that can degrade the bond between the CFRP laminate and the concrete substrate, leading to a reduction in bond strength and potential delamination. Finite element models (FEM), whether in 2D or 3D, are commonly employed to simulate the behavior of the CFRP-to-concrete joint. These models incorporate material constitutive laws for each component of the system: the concrete is typically modelled using a nonlinear elastic-plastic model that accounts for cracking and softening, while the CFRP is generally assumed to a linear elastic behavior up to failure. The adhesive layer, crucial for bond performance, is usually modelled with a viscoelastic or cohesive zone model to simulate its behavior under varying conditions and its role in load transfer between the concrete and CFRP. This section explores the application of FEM in modeling the EBR CFRP-to-concrete joint, with particular emphasis on the impact of water immersion and moisture exposure on the behavior of concrete, CFRP, and adhesive layers.

Existing literature presents a substantial number of experimental tests addressing the small-scale EBR CFRP-to-concrete joint (also, known as single lap or pull-out tests), e.g., [89,106,107] from which the failure mode was generally reported to be debonding with a very thin concrete layer or concrete cohesive failure. Before going deeper into numerical modeling of the EBR CFRP-to-concrete bond, a brief discussion on the existing analytical models (from which important inputs to the numerical models are derived) is first presented herein. Existing literature presents a substantial number of analytical studies in which bond stress-slip laws were developed to address the bond behavior, e.g., [108–114]. In a study by Matthys et al. [108], a model was proposed which is based on bilinear bond stress-strain relation, requiring the knowledge of three main empirical parameters: the maximum bond stress, the corresponding slip, and the maximum slip, all of which could simply be obtained from the concrete compressive strength.

A database of 311 experimental FRP-to-concrete bond data was created by Wu et al. [111] to calibrate and validate a proposed model. The performance of the model was then compared to other bond strength models, demonstrating its validity and superiority. In a study by Zhou et al. [112], the performance of the proposed model depended on the effective length (l_e) and length of the joint, for long joints with lengths greater than l_e , the infinite bond length model was roughly accurate beyond l_e from the free end. However, for shorter joints or areas near the free end, the bond-slip relation varied significantly. In a study by Wu and Jiang [113], A database comprising 628 shear tests of EBR CFRP-to-concrete joints was built for model development, from which it was found that the FRP-to-concrete width factor is an important parameter, due to this, a new model for the width factor as a function of both the width ratio and concrete strength was developed, leading to more accurate and rational bond strength and fracture energy models.

From the above brief review, it can be noted that existing literature presents advanced and reliable data on the bond stress-slip relations. As previously mentioned, these relations are key input parameters when developing a finite element model. Accurate modeling of the EBR CFRP-to-concrete joint is essential for simulating the behavior of small-scale specimens strengthened with EBR systems. These models help capture the complex interactions at the interface, which are critical for predicting bond performance under various exposure conditions. Different numerical studies are discussed in the following paragraphs, highlighting the approaches to numerically modeling of the bond stress-slip behavior and the findings on the performance of the EBR CFRP-to-concrete joint under different exposure conditions.

Several numerical models to predict the short-term behavior of the EBR CFRP-to-concrete joint (e.g., [115–119]) exist in which a cohesive or connector element is used to model the CFRP-to-concrete interfaces. The element used at the interfaces incorporates the bond stress-slip properties from the analytical models previously discussed. In a study by Mukhtar and Shehadah [115], both 2D and 3D numerical models for EBR CFRP-to-concrete joints was developed and compared. It was shown that the 3D model outperformed the 2D model by accurately capturing the failure load, which the 2D model failed to do. It also highlighted the need for an improved double shear test setup over the traditional one. In a study by Obaidat et al. [116], the developed model with a cohesive model to represent the EBR CFRP-to-concrete bond behavior showed a desirable accuracy by predicting ultimate loads and debonding failure mode that closely matched experimental results.

The existing literature shows very limited number of studies on numerical models addressing the EBR CFRP-to-concrete bond durability, examples include those addressing the temperature effects [120–122]. In addition to this, these numerical models are generally based on very short-term exposures (generally less than a year), hence lacking data in terms of longer exposures. Due to this, further relevant works on this topic can be done to better understand the behavior of the joint after long-term exposure to different environments. For the cases when the failure takes place within concrete (concrete cohesive failure), which generally occurs for the EBR CFRP-to-concrete joints, assuming the interfaces to be perfectly connected may be thought appropriate, since concrete properties will control the failure.

Overall, it is noted that numerical studies addressing both short- and long-term bond performance are still few and do not incorporate cases such as natural outdoor conditioning or effects of longer exposure times; hence, further research is needed, particularly for the long-term NAT case, where a little or no relevant numerical studies can be found.

2.4.2. Analytical modeling of short- and long-term performance of beams/slabs with CFRP systems

Predicting the flexural behavior of RC beams/slabs strengthened with non-prestressed or prestressed CFRP systems can help understand the behavior of these systems even without having to perform experimental tests. Additionally, such predictions can also help to examine other important parameters impossible to capture during experimental works, just to mention a few. Relevant analytical studies exist on the prediction of the short-term flexural behavior of RC elements strengthened with different techniques such as EBR, NSM, EBR-GA, and EBR-MA systems, e.g., [123–128]. However, there are still some points that need to be further addressed, examples of these include, the incorporation of the prevailing failure mode in the models, and assessment of the reliability of the FRP failure strains prediction from different guidelines and standards such as CNR [9], ACI [129] and *fib* 90 bulletin [2]. All these gaps in knowledge show that further research is needed to better understand the effectiveness of using FRP composites in construction Industry.

On the other hand, studies also exist about analytical modeling of long-term flexural behavior of structural elements strengthened with CFRP systems, e.g., [32,96,98,105]. These works generally addressed the long-term flexural performance of beams/slabs subjected to a sustained load and environmental conditioning, and the general findings is that creep of concrete is the leading factor for increase of deflections over time. In a study by Silva [32], concrete model was established using the well-known Generalized Kelvin Chain (GKC) model, with this model the concrete creep compliance was estimated; also, the age-adjusted effective modulus (AEMM), adopted by other researchers, e.g., [98], can be adopted to predict the long-term deflections. The AEMM is based on adjusting the properties of concrete with time, particularly its modulus of elasticity being updated by the time-dependent concrete creep coefficient [130]. The *fib* MC (2020) [37] recommends solely adjusting the concrete modulus which in turn leads to adjustment of steel and CFRP modular ratios and the corresponding moments of inertia. However, studies have also shown that the CFRP modulus of elasticity can be age-adjusted through its the CFRP creep coefficient model [100]. The adhesive layer can also be incorporated by additionally scaling the age-adjusted CFRP modulus with the adhesive creep coefficient model [99].

The creep and shrinkage of concrete are very detailed in standards such as ACI 318R-19 [131] and *fib* MC (2020) [37], where creep formulations are provided including the effects environmental exposure such as humidity and temperature. Furthermore, analytical approaches for predicting the instantaneous and long-term deflections of beams under sustained load are also provided in ACI 318R-19 [131] and *fib* MC (2020) [37]. ACI 318R-19 [131] provides a simplified deflection formula for non-prestressed members that depends on the compression reinforcement ratio and a time-dependent coefficient. This formula does not account

for the effects of CFRP systems, or effects of environmental conditioning. Furthermore, *fib* MC (2020) [37] also provides a simplified deflection formula for determining the mid-span deflection using section analysis and age-adjusted effective modulus to account for change of modular ratios of reinforcements (steel and CFRP) over time. These modular ratios are basically obtained using the age-adjusted effective modulus of concrete, which in turns depends on the concrete creep coefficient.

The concrete creep coefficient model detailed in *fib* MC (2020) [37] is straightforward; however, the fact that the formulae in this model do not account for all types of exposure conditioning (e.g., freeze-thaw or synergistic effects taking place in outdoor environments) may limit its applicability, particularly, for the case when different outdoor environments are studied (e.g., outdoor environments in the present PhD thesis). The *fib* MC (2020) [37] specifically states that the long-term behavior of a strengthened RC element under sustained is primarily governed by the creep of the concrete in compression, since the bond stresses at the interface with the EBR are usually low under service loads and within the linear elastic part of the bond stress-slip curve. However, even by assuming that the above statement remains correct, the creep coefficient, which depends on the type of environmental conditioning, can still play a central role, as it is the critical parameter when determining the so-called "age-adjusted effective modulus of concrete", on which the modular ratios of steel and CFRP laminates depend. As a result, the creep coefficient model may need further improvements to incorporate effects from other environments, particularly synergistic effects in outdoor environments, where the structures are typically built. In addition, the *fib* 90 bulletin [2] states that externally bonded FRP systems reduce the time-dependent deflection, compared to non-strengthened systems, which somehow may sound contradicting to the previously statement from the *fib* MC (2020) [37] where effects of applied strengthening systems are minimal, but can well indicate how the creep of concrete can be different in both systems. Hence, further relevant works may play a major role in better understanding of the effects of creep on the long-term behavior of RC elements with/without the CFRP systems.

Considering that there is currently a lack of guidelines or accurate analytical models for predicting long-term deflections of the slabs investigated in the scope of the present PhD thesis, one can think of establishing a creep deflection model that is based on the well-known Generalized Kelvin (GK) model (e.g., [32,132]), assuming that the long-term deflection of an RC element strengthened with CFRP system is primarily governed by the viscoelastic behavior of concrete. This assumption would be in line with the recommendations from both ACI 318R-19 [131] and *fib* MC (2020) [37], where the concrete properties are the major driving factor for the creep behavior. Such a creep deflection model can be established referring to the existing long-term deflection data and the resulting model parameters can be used to establish

formulae that incorporate the complex effects of outdoor environments, when determining the creep coefficient. The established creep deflection model can furthermore be used to extend the applicability of the ACI 318R-19 [131] formula for design practice, because of its simplified nature, where one only needs to know the time-dependent factor and the compression reinforcement ratio.

Figure 2.8 highlights selected standards/guides that are currently used to model both short- and long-term flexural behavior of RC slabs with CFRP strengthening systems.

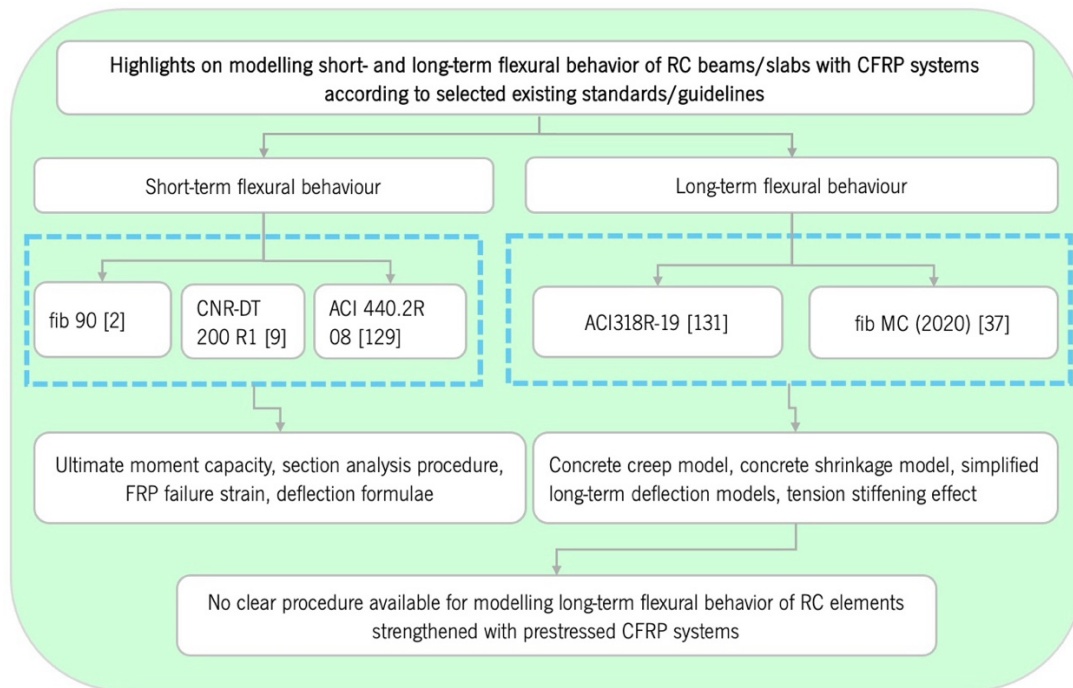


Figure 2.8: Existing guides on analytical and numerical models highlighting knowledge gaps

It can be noted that the procedure for modeling the short-term flexural behavior is straightforward. However, the shown standards may need improvements in terms of topics such as guidelines on how to model long-term behavior of RC elements with prestressed CFRP systems in future releases.

2.5. FINAL REMARKS

CFRP composites have gained popularity in construction industry in recent decades. Particularly, these composites have been widely used to strengthen RC structures to increase both sustainability, structural, and durability performances. The confidence in this strengthening solution has increased after its successful application to different existing buildings and bridges worldwide. Despite the effectiveness of the CFRP strengthening solution, its broader adoption has been hindered by a lack of sufficient knowledge on,

particularly, the durability and long-term performance of CFRP materials and the structures reinforced with them [78].

The research on the durability and long-term performance at material level (i.e., epoxy adhesive and CFRP laminate), at bond level (e.g., EBR, NSM pull-out testing) and at structural element level (e.g., CFRP strengthened RC beam/slab) has predominantly been carried out under AAT conditions, with only a limited number of studies focusing on NAT conditions. Moreover, current literature generally lacks comprehensive studies on the long-term durability using predictive models, where the existing models can only address the durability generally for short-term periods of exposure. This is concerning, given that the CFRP systems are expected to function effectively for many years in service.

The findings from the accelerated tests are typically extrapolated to predict the behavior of structures under real outdoor conditions. However, the correlation AAT and NAT outcomes remains poorly understood. This lack of understanding poses challenges in accurately assessing the long-term durability of RC structures strengthened with CFRP under real-world conditions. Furthermore, the existing environmental conversion factors (ECF) are heavily based on AAT studies; in addition, these ECF are generally intended to represent all possible degradation agents, but in practice, RC structures are exposed to a combination of specific agents (with synergistic effects) depending on their location. Appropriate ECF can be obtained through correlations between AAT and NAT data; however, such correlations would only be realistic if there exist sufficient works that were performed under NAT protocols. Therefore, further works should focus on increasing the literature addressing the NAT protocols.

The above highlighted knowledge gaps formed the basis of the development of the present PhD thesis. That is, the present PhD thesis addressed: (i) the durability of materials composing an RC structure that is strengthened with CFRP systems (concrete, epoxy adhesives, and CFRP laminates), where the specimens were exposed to different AAT and NAT environments for up to 4 years, in addition, modeling of water diffusion in epoxy adhesive was also addressed; (ii) the durability of the EBR CFRP-to-concrete and NSM CFRP-to-concrete joints, where the specimens were also exposed to the same conditioning as in (i) for up to 4 years, in addition, numerical modeling of short- and long-term behavior of the EBR CFRP-to-concrete joint was addressed; (iii) the long-term monitoring of mid-span deflections (up to 6 years) of the RC slabs with different CFRP strengthening systems (prestressed and non-prestressed), analytical modeling of short-term flexural behavior of the RC slabs to predict the results obtained during initial flexural tests before exposure, analytical modeling of long-term flexural behavior (mid-span deflections) of the RC slabs, from which the predicted results were validated by the deflections monitored as previously mentioned.

Furthermore, deflection predictions for longer periods of exposures (up to 50 years) were also achieved through the use of the GK model and the ACI 318R-19 [131] approach, and the derived time-dependent factor curves can play a significant role towards establishing accurate predictive models, particularly for those incorporating effects of complex outdoor environments.

2.6. REFERENCES

- [1] D.K. Rajak, D.D. Pagar, P.L. Menezes, E. Linul, Fiber-reinforced polymer composites: Manufacturing, properties, and applications, *Polymers* (Basel). 11 (2019). <https://doi.org/10.3390/polym11101667>.
- [2] CEB-FIP, FIB Bulletin 90, “Externally Applied FRP Reinforcement for Concrete Structures”, Federation International du Beton, Technical Report Prepared by a Working Party of the T5.1 FRP Reinforcement for Concrete Structures, Lausanne, Switzerland, 2019.
- [3] P.A. Mitchell, Freeze-Thaw and Sustained Load Durability of Near Surface Mounted Frp Strengthened Concrete, Queen’s University Kingston, Canada, 2010. <http://qspace.library.queensu.ca/handle/1974/5653>.
- [4] I.S. Abbood, S.A. Odaa, K.F. Hasan, M.A. Jasim, Properties evaluation of fiber reinforced polymers and their constituent materials used in structures - A review, *Mater. Today Proc.* 43 (2021) 1003–1008. <https://doi.org/10.1016/j.matpr.2020.07.636>.
- [5] T.A. Kinjawadekar, S. Patil, G. Nayak, A Critical Review on Glass Fiber-Reinforced Polymer Bars as Reinforcement in Flexural Members, *J. Inst. Eng. Ser. A.* 104 (2023) 501–516. <https://doi.org/10.1007/s40030-023-00729-6>.
- [6] J. Qureshi, A Review of Fibre Reinforced Polymer Structures, *Fibers.* 10 (2022) 4–6. <https://doi.org/10.3390/fib10030027>.
- [7] M. Zoghi, “The International Handbook of FRP Composites in Civil Engineering,” Boca Raton, FL, USA, 2013.
- [8] Committee 440, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures,” Farmington Hills, MI, USA, 2017.
- [9] CNR-DT 200 R1, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures; CNR-DT 200 R1/2013;, Rome, Italy, 2013.

- [10] J.R. Duflou, Y. Deng, K. Van Acker, W. Dewulf, Do fiber-reinforced polymer composites provide environmentally benign alternatives? A life-cycle-assessment-based study, *MRS Bull.* 37 (2012) 374–382. <https://doi.org/10.1557/mrs.2012.33>.
- [11] R.D. Anandjiwala, S. Blouw, Composites from bast fibres - Prospects and potential in the changing market environment, *J. Nat. Fibers.* 4 (2007) 91–109. https://doi.org/10.1300/J395v04n02_07.
- [12] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fibre reinforced polymer laminates used in strengthening systems: Accelerated ageing versus natural ageing, *Materials (Basel).* 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [13] A. Bilotta, F. Ceroni, M. Di Ludovico, E. Nigro, M. Pecce, G. Manfredi, Bond Efficiency of EBR and NSM FRP Systems for Strengthening Concrete Members, *J. Compos. Constr.* 15 (2011) 757–772. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000204](https://doi.org/10.1061/(asce)cc.1943-5614.0000204).
- [14] FIB, Externally bonded FRP reinforcement for RC structures. Bulletin no. 14, Lausanne, Switzerland, 2001.
- [15] L. De Lorenzis, J.G. Teng, Near-surface mounted FRP reinforcement: An emerging technique for strengthening structures, *Compos. Part B Eng.* 38 (2007) 119–143. <https://doi.org/10.1016/j.compositesb.2006.08.003>.
- [16] J.A.O. Barros, R.K. Varma, J.M. Sena-Cruz, A.F.M. Azevedo, Near surface mounted CFRP strips for the flexural strengthening of RC columns: Experimental and numerical research, *Eng. Struct.* 30 (2008) 3412–3425. <https://doi.org/10.1016/j.engstruct.2008.05.019>.
- [17] D. Mostofinejad, S.M. Shameli, A. Hosseini, EBROG and EBRIG methods for strengthening of RC beams by FRP sheets, *Eur. J. Environ. Civ. Eng.* 18 (2014) 652–668. <https://doi.org/10.1080/19648189.2014.900523>.
- [18] M. Breveglieri, A. Aprile, J.A.O. Barros, Embedded Through-Section shear strengthening technique using steel and CFRP bars in RC beams of different percentage of existing stirrups, *Compos. Struct.* 126 (2015) 101–113. <https://doi.org/10.1016/j.compstruct.2015.02.025>.
- [19] J. Cruz, Multi-scale investigation on durability and long-term behaviour of concrete elements strengthened with CFRP laminates according to the EBR and NSM techniques, University of Minho, 2023.

- [20] L. Correia, J. Sena-Cruz, J. Michels, P. França, E. Pereira, G. Escusa, Durability of RC slabs strengthened with prestressed CFRP laminate strips under different environmental and loading conditions, *Compos. Part B*. 125 (2017) 71–88. <https://doi.org/10.1016/j.compositesb.2017.05.047>.
- [21] A. Dushimimana, L. Correia, R. Cruz, J.M. Pereira, J. Sena-cruz, Analytical approaches for flexural analysis of RC slabs strengthened with prestressed or non-prestressed CFRP laminates, in: 15th Int. Conf. Fibre-Reinforced Polym. Reinf. Concr. Struct. 8th Asia-Pacific Conf. FRP Struct. 10-14 December 2022, Shenzhen, China, 2022: pp. 10–14.
- [22] A. Dushimimana, L. Correia, R. Cruz, S. Cabral-fonseca, J. Pereira, J. Sena-Cruz, Analytical Prediction of Flexural Behaviour of Rc Slabs Strengthened With Non-Prestressed and Prestressed Cfrp Strips, in: 11th Int. Conf. FFP C. Civ. Eng. (CICE 23), 2023: pp. 1–11.
- [23] T.C. Triantafillou, Shear Strengthening of Reinforced Concrete Beams Using Epoxy-Bonded FRP Composites, *Struct. J.* 95 (1998). <https://doi.org/10.14359/531>.
- [24] J.G. Teng, J.F. Chen, S.T. Smith, L. Lam, *FRP: Strengthened RC Structures*, Wiley, 2001. <https://www.wiley.com/en-us/FRP%3A+Strengthened+RC+Structures-p-9780471487067>.
- [25] Committee 440, *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*, Farmington Hills, MI, USA, 2017.
- [26] J.M. Sena-Cruz, Strengthening of concrete structures with near-surface mounted CFRP laminate strips *Reforço de estruturas com laminados de CFRP inseridos no betão de recobrimento* José Manuel de Sena Cruz, University of Minho, 2005.
- [27] F.A. Megahed, M.H. Seleem, A.A.M. Badawy, I.A. Sharaky, The flexural response of RC beams strengthened by EB/NSM techniques using FRP and metal materials: a state-of-the-art review, Springer International Publishing, 2023. <https://doi.org/10.1007/s41062-023-01245-z>.
- [28] K. Mallikarjuna, P.M. Ravindra, D.P. Archana, M. Daniyal, A.N.M. Asiri, M.A. Khan, S. Islam, Employing Carbon Fiber Reinforced Polymer Composites toward the Flexural Strengthening of Reinforced Concrete T-Beams, *ACS Omega*. 8 (2023) 18830–18838. <https://doi.org/10.1021/acsomega.3c00988>.
- [29] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174.

- <https://doi.org/10.1016/j.compstruct.2024.118174>.
- [30] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of Bond between NSM CFRP Strips and Concrete under Real-Time Field and Laboratory Accelerated Conditioning, *J. Compos. Constr.* 26 (2022) 1–15. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001262](https://doi.org/10.1061/(asce)cc.1943-5614.0001262).
 - [31] I.G. Costa, Prestressed carbon fibre laminates applied according to near surface mounted technique to increase the flexural resistance of reinforced concrete beams, University of Minho, 2014.
 - [32] P.M. Silva, Time-dependent behaviour and durability of RC slabs strengthened with NSM CFRP strips. Doctoral Thesis, Universidade do Minho., University of Minho, 2017.
 - [33] H.S. AL-Ameedee, H.M. Al-Khafaji, Improving the flexural behavior of RC beams strengthening by near - surface mounting, *J. Mech. Behav. Mater.* 2022; 31 (2022) 701–709.
 - [34] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B Eng.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
 - [35] L. Correia, Durability and long-term behaviour of RC slabs strengthened in flexure with prestressed CFRP laminate strips., University of Minho, 2018. <https://repositorium.sdum.uminho.pt/handle/1822/60062>.
 - [36] Y.E. Harmanci, Long-Term Resistance of Gradient Anchorage for Prestressed CFRP Strips in Structural Concrete Retrofitting, ETH, 2018. <https://doi.org/https://stresshead.ch>.
 - [37] FIB MC (2020). International Federation for Structural Concrete (fib), fib Model Code for Concrete Structures (2020), (2020) 782.
 - [38] S. Luhar, I. Luhar, F.U.A. Shaikh, Review on performance evaluation of autonomous healing of geopolymer composites, *Infrastructures.* 6 (2021) 1–24. <https://doi.org/10.3390/infrastructures6070094>.
 - [39] D. Liu, Y. Tu, P. Shi, G. Sas, L. Elfgren, Mechanical and durability properties of concrete subjected to early-age freeze-thaw cycles, *Mater. Struct. Constr.* 54 (2021) 1–18. <https://doi.org/10.1617/s11527-021-01802-x>.
 - [40] V.S. Kashyap, G. Sancheti, J.S. Yadav, U. Agrawal, Smart sustainable concrete: enhancing the strength and durability with nano silica, *Smart Constr. Sustain. Cities.* 1 (2023) 1–17.

- <https://doi.org/10.1007/s44268-023-00023-1>.
- [41] W. Chalee, T. Sasakul, P. Suwanmaneechot, C. Jaturapitakkul, Utilization of rice husk-bark ash to improve the corrosion resistance of concrete under 5-year exposure in a marine environment, *Cem. Concr. Compos.* 37 (2013) 47–53. <https://doi.org/10.1016/j.cemconcomp.2012.12.007>.
 - [42] K. Kobayashi, W. Koyanagi, 17-Year-Long Sewage Sludge Ash Concrete Exposure Test, *Infrastructures*. 6 (2021). <https://doi.org/10.3390/infrastructures6050074>.
 - [43] G. Frenzer, F. Jacobs, R.J. Torrent, In situ durability characteristics of new and old concrete structures, *J. Adv. Concr. Technol.* 19 (2021) 53–66. <https://doi.org/10.3151/JACT.19.53>.
 - [44] W. Wang, Q. Yue, The Time Variation Law of Concrete Compressive Strength: A Review, *Appl. Sci.* 13 (2023). <https://doi.org/10.3390/app13084947>.
 - [45] Y. Huo, H. Sun, D. Lu, Z. Chen, Y. Yang, Mechanical properties of concrete at low and ultra-low temperatures- a review, *J. Infrastruct. Preserv. Resil.* 3 (2022). <https://doi.org/10.1186/s43065-022-00063-4>.
 - [46] K. Li, J. Zeng, L. Tang, H.E. Sørensen, P. Castro Borges, M.R. Geiker, M.T. Pedersen, P. Zhang, S. Surana, R. Maddalena, J. Wang, C. Andrade, V. Baroghel-Bouny, F. Martirena-Hernández, G. Geng, K. Kovler, S. Wang, Long-term field exposure of structural concretes in marine environment: state-of-the-art review by RILEM TC 289-DCM, *Mater. Struct. Constr.* 55 (2022). <https://doi.org/10.1617/s11527-022-02027-2>.
 - [47] K. Li, C. Li, Modeling hydroionic transport in cement-based porous materials under drying-wetting actions, *J. Appl. Mech. Trans. ASME.* 80 (2013) 1–10. <https://doi.org/10.1115/1.4007907>.
 - [48] M.J. Setzer, Micro-ice-lens formation in porous solid, *J. Colloid Interface Sci.* 243 (2001) 193–201. <https://doi.org/10.1006/jcis.2001.7828>.
 - [49] L. jie Li, Q. feng Liu, L. Tang, Z. Hu, Y. Wen, P. Zhang, Chloride penetration in freeze–thaw induced cracking concrete: A numerical study, *Constr. Build. Mater.* 302 (2021). <https://doi.org/10.1016/j.conbuildmat.2021.124291>.
 - [50] F.R. JONES, Durability of reinforced plastics in liquid environments, in: *Reinf. Plast. Durab.*, 1999: pp. 70–110. <https://doi.org/10.1533/9781845694876.70>.
 - [51] C. Helbling, V. Karbhari, Durability of composites in aqueous environments, in: *Durab. Compos. Civ.*

- Struct. Appl., Elsevier, Amsterdam, The Netherlands, 2007: pp. 31–71.
<https://doi.org/https://doi.org/10.1533/9781845693565.1.31>.
- [52] K.W. Thomson, T. Wong, L.J. Broutman, The plasticization of an epoxy resin by dibutylphthalate and water, *Polym. Eng. Sci.* 24 (1984) 1270–1276. <https://doi.org/10.1002/pen.760241610>.
- [53] S. Cabral-fonseca, J.R. Correia, J. Custódio, H.M. Silva, A.M. Machado, J. Sousa, Durability of FRP - concrete bonded joints in structural rehabilitation : A review, *Int. J. Adhes. Adhes.* 83 (2018) 153–167. <https://doi.org/10.1016/j.ijadhadh.2018.02.014>.
- [54] Q. Yang, G. Xian, V.M. Karbhari, Hygrothermal Ageing of an Epoxy Adhesive Used in FRP Strengthening of Concrete, *J. Appl. Polym. Sci.* 107 (2008) 2607–2617. <https://doi.org/10.1002/app>.
- [55] R. Sen, Developments in the durability of FRP-concrete bond, *Constr. Build. Mater.* 78 (2015) 112–125. <https://doi.org/10.1016/j.conbuildmat.2014.12.106>.
- [56] C.A. Wood, W.L. Bradley, Determination of the effect of seawater on the interfacial strength of an interlayer E-glass/graphite/epoxy composite by in situ observation of transverse cracking in an environmental SEM, *Compos. Sci. Technol.* 57 (1997) 1033–1043. [https://doi.org/10.1016/S0266-3538\(96\)00170-4](https://doi.org/10.1016/S0266-3538(96)00170-4).
- [57] Y. Zhang, J. Ma, C. Wu, X. Han, W. Zhang, Effects of moisture ingress on the mesoscale mechanical properties of epoxy adhesives under elevated temperature, *Polym. Test.* 94 (2021). <https://doi.org/10.1016/j.polymertesting.2020.107049>.
- [58] P. Fernandes, P. Silva, L. Correia, J. Sena-cruz, Durability of an epoxy adhesive and a CFRP laminate under different exposure conditions, in: *Third Conf. Smart Monit. Assess. Rehabil. Civ. Struct.* 2015, 2015: p. 9.
- [59] J.M. Sousa, J.R. Correia, S. Cabral-Fonseca, Durability of an epoxy adhesive used in civil structural applications, *Constr. Build. Mater.* 161 (2018) 618–633. <https://doi.org/10.1016/j.conbuildmat.2017.11.168>.
- [60] E. Cui, S. Jiang, J. Wang, X. Zeng, Bond behavior of CFRP-concrete bonding interface considering degradation of epoxy primer under wet-dry cycles, *Constr. Build. Mater.* 292 (2021) 123286. <https://doi.org/10.1016/j.conbuildmat.2021.123286>.

- [61] X. Ren, Y. Wang, Y. Zhang, Y. Sun, Q. Liua, S. Chen, Effect of freeze-thaw/dry-wet environment on mechanical properties of epoxy resin adhesive, *J. Adhes. Sci. Technol.* 37 (2023). <https://doi.org/https://doi.org/10.1080/01694243.2022.2111189>.
- [62] P. Fernandes, J. Sena-Cruz, J. Xavier, P. Silva, E. Pereira, J. Cruz, Durability of bond in NSM CFRP-concrete systems under different environmental conditions, *Compos. Part B Eng.* 138 (2018) 19–34. <https://doi.org/10.1016/j.compositesb.2017.11.022>.
- [63] J. Li, J. Xie, F. Liu, Z. Lu, A critical review and assessment for FRP-concrete bond systems with epoxy resin exposed to chloride environments, *Compos. Struct.* 229 (2019) 111372. <https://doi.org/10.1016/j.compstruct.2019.111372>.
- [64] S.A. Grammatikos, R.G. Jones, M. Evernden, J.R. Correia, Thermal cycling effects on the durability of a pultruded GFRP material for off-shore civil engineering structures, *Compos. Struct.* 153 (2016) 297–310. <https://doi.org/10.1016/j.compstruct.2016.05.085>.
- [65] O. Moussa, A.P. Vassilopoulos, T. Keller, Effects of low-temperature curing on physical behavior of cold-curing epoxy adhesives in bridge construction, *Int. J. Adhes. Adhes.* 32 (2012) 15–22. <https://doi.org/10.1016/j.ijadhadh.2011.09.001>.
- [66] D. dong Hu, J. xun Lyu, T. Liu, M. dong Lang, L. Zhao, Solvation effect of CO₂ on accelerating the curing reaction process of epoxy resin, *Chem. Eng. Process. - Process Intensif.* 127 (2018) 159–167. <https://doi.org/10.1016/j.cep.2018.01.027>.
- [67] M. Frigione, C. Naddeo, D. Acierno, Cold-curing epoxy resins: Aging and environmental effects. I- Thermal properties, *J. Polym. Eng.* 21 (2001) 23–52. <https://doi.org/10.1515/POLYENG.2001.21.1.23>.
- [68] M. Sawilotidou, A.P. Vassilopoulos, T. Keller, Durability and Fatigue Performance of Cold-Curing Structural Adhesives in Bridge Construction, *École Polytechnique Fédérale de Lausanne (EPFL)*, 2017. <https://infoscience.epfl.ch/record/230442?ln=en>.
- [69] S. Amidi, J. Wang, Deterioration of the FRP-to-concrete interface subject to moisture ingress: Effects of conditioning methods and silane treatment, *Compos. Struct.* 153 (2016) 380–391. <https://doi.org/https://doi.org/10.1016/j.compstruct.2016.06.035>.
- [70] M. Heshmati, R. Haghani, M. Al-Emrani, Effects of moisture on the long-term performance of adhesively bonded FRP/steel joints used in bridges, *Compos. Part B.* 92 (2016) 447–462.

- <https://doi.org/http://dx.doi.org/10.1016/j.compositesb.2016.02.021>.
- [71] Y. Joliff, L. Belec, M.B. Heman, J.F. Chailan, Experimental, analytical and numerical study of water diffusion in unidirection composite materials - Interphase impact, *Comput. Mater. Sci.* 64 (2012) 141–145. <https://doi.org/http://dx.doi.org/10.1016/j.commatsci.2012.05.029>.
 - [72] J.C. Arnold, S.M. Alston, F. Korkees, An assessment of methods to determine the directional moisture diffusion coefficients of composite materials, *Compos. Part A.* 55 (2013) 120–128. <https://doi.org/http://dx.doi.org/10.1016/j.compositesa.2013.08.012>.
 - [73] S. Cabral-Fonseca, J.P. Nunes, M.P. Rodrigues, M.I. Eusébio, Durability of carbon fibre reinforced polymer laminates used to reinforced concrete structures, *Sci. Eng. Compos. Mater.* 18 (2011) 201–207. <https://doi.org/10.1515/SECM.2011.041>.
 - [74] S. Choi, A.L. Gartner, N. Van Etten, H.R. Hamilton, E.P. Douglas, Durability of Concrete Beams Externally Reinforced with CFRP Composites Exposed to Various Environments, *J. Compos. Constr.* 16 (2012) 10–20. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000233](https://doi.org/10.1061/(asce)cc.1943-5614.0000233).
 - [75] J.R. Cromwell, K.A. Harries, B.M. Shahrooz, Environmental durability of externally bonded FRP materials intended for repair of concrete structures, *Constr. Build. Mater.* 25 (2011) 2528–2539. <https://doi.org/10.1016/j.conbuildmat.2010.11.096>.
 - [76] S. Liu, Y. Pan, H. Li, G. Xian, Durability of the bond between CFRP and concrete exposed to thermal cycles, *Materials (Basel)*. 12 (2019). <https://doi.org/10.3390/ma12030515>.
 - [77] J.W. Chin, Durability of composites exposed to ultraviolet radiation, *Durab. Compos. Civ. Struct. Appl.* (2007) 80–97. <https://doi.org/10.1533/9781845693565.1.80>.
 - [78] I. Nishizaki, P. Labossière, B. Sarsaniuc, Durability of CFRP sheet reinforcement through exposure tests, *Am. Concr. Institute, ACI Spec. Publ. SP-230* (2005) 1419–1427. <https://doi.org/10.14359/14901>.
 - [79] E. Alsuhaibani, N. Yazdani, E. Beneberu, Durability and Long-Term Performance Prediction of Carbon Fiber Reinforced Polymer Laminates, *Polymers (Basel)*. 14 (2022) 1–17. <https://doi.org/10.3390/polym14153207>.
 - [80] M.F. Green, L.A. Bisby, Y. Beaudoin, P. Labossière, Effect of freeze-thaw cycles on the bond durability between fibre reinforced polymer plate reinforcement and concrete, *Can. J. Civ. Eng.* 27

- (2000) 949–959. <https://doi.org/10.1139/I00-031>.
- [81] M. Leone, S. Matthys, M.A. Aiello, Effect of elevated service temperature on bond between FRP EBR systems and concrete, *Compos. Part B Eng.* 40 (2009) 85–93. <https://doi.org/10.1016/j.compositesb.2008.06.004>.
- [82] P. Colombi, G. Fava, C. Poggi, Bond strength of CFRP-concrete elements under freeze-thaw cycles, *Compos. Struct.* 92 (2010) 973–983. <https://doi.org/10.1016/j.compstruct.2009.09.044>.
- [83] K. Benzarti, S. Chataigner, M. Quiertant, C. Marty, C. Aubagnac, Accelerated ageing behaviour of the adhesive bond between concrete specimens and CFRP overlays, *Constr. Build. Mater.* 25 (2011) 523–538. <https://doi.org/10.1016/j.conbuildmat.2010.08.003>.
- [84] F. Al-Mahmoud, J.M. Mechling, M. Shaban, Bond strength of different strengthening systems - Concrete elements under freeze-thaw cycles and salt water immersion exposure, *Constr. Build. Mater.* 70 (2014) 399–409. <https://doi.org/10.1016/j.conbuildmat.2014.07.039>.
- [85] Y. Pan, G. Xian, M.A.G. Silva, Effects of water immersion on the bond behavior between CFRP plates and concrete substrate, *Constr. Build. Mater.* 101 (2015) 326–337. <https://doi.org/10.1016/j.conbuildmat.2015.10.129>.
- [86] Y. Pan, G. Xian, H. Li, Effects of Freeze-Thaw Cycles on the Behavior of the Bond between CFRP Plates and Concrete Substrates, *J. Compos. Constr.* 22 (2018) 1–14. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000846](https://doi.org/10.1061/(asce)cc.1943-5614.0000846).
- [87] J. Li, R. Gravina, P. Visintin, S.T. Smith, Durability and Long-Term Performance of FRP-to-Concrete Joints under Environmental Conditioning: Experimental and Analytical Study, *J. Compos. Constr.* 24 (2020). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001023](https://doi.org/10.1061/(asce)cc.1943-5614.0001023).
- [88] J.P. Firmo, J.R. Correia, D. Pitta, C. Tiago, M.R.T. Arruda, Experimental characterization of the bond between externally bonded reinforcement (EBR) CFRP strips and concrete at elevated temperatures, *Cem. Concr. Compos.* 60 (2015) 44–54. <https://doi.org/10.1016/j.cemconcomp.2015.02.008>.
- [89] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of bond of EBR CFRP laminates to concrete under real-time field exposure and laboratory accelerated ageing, *Constr. Build. Mater.* 377 (2023). <https://doi.org/10.1016/j.conbuildmat.2023.131047>.
- [90] M.R.F. Coelho, J.M. Sena-Cruz, L.A.C. Neves, A review on the bond behavior of FRP NSM systems

- in concrete, *Constr. Build. Mater.* 93 (2015) 1157–1169. <https://doi.org/10.1016/j.conbuildmat.2015.05.010>.
- [91] J. Sena-Cruz, P. Fernandes, P. Silva, J. Barros, M.R.F. Coelho, Bond behaviour of concrete elements strengthened with NSM CFRP laminate strips under wet-dry cycles, in: J.W. Cairns, G. Metelli, G. A. Plizzari (Eds.), *New Mater. under Sev. Cond.*, 2012 Publisher creations, 2012.
- [92] J. Garzón-Roca, J.M. Sena-Cruz, P. Fernandes, J. Xavier, Effect of wet-dry cycles on the bond behaviour of concrete elements strengthened with NSM CFRP laminate strips, *Compos. Struct.* 132 (2015) 331–340. <https://doi.org/10.1016/j.compstruct.2015.05.053>.
- [93] A. Dushimimana, L. Correia, R. Cruz, S. Cabral-fonseca, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-concrete bond and corresponding involved materials under different natural environmental exposures for a period of four years, in: *FRPRCS-15 and APFIS-2022*, Shenzhen, China, 2022: pp. 10–14.
- [94] G. Al Chami, M. Thériault, K.W. Neale, Creep behaviour of CFRP-strengthened reinforced concrete beams, *Constr. Build. Mater.* 23 (2009) 1640–1652. <https://doi.org/10.1016/j.conbuildmat.2007.09.006>.
- [95] Z. Lu, J. Li, J. Xie, P. Huang, L. Xue, Durability of flexurally strengthened RC beams with prestressed CFRP sheet under wet-dry cycling in a chloride-containing environment, *Compos. Struct.* 255 (2021) 1–15. <https://doi.org/10.1016/j.compstruct.2020.112869>.
- [96] G. Qin, P. Huang, H. Zhou, X. Guo, X. Zheng, Fatigue and durability behavior of RC beams strengthened with CFRP under hot-wet environment, *Constr. Build. Mater.* 111 (2016) 735–742. <https://doi.org/10.1016/j.conbuildmat.2016.02.131>.
- [97] K. Douier, R. Hawileh, J.A. Abdalla, N. Al Nuaimi, M. Ghous Sohail, Durability performance of RC beams strengthened with CFRP laminates and galvanized steel mesh bonded with epoxy and mortar systems, *Structures*. 55 (2023) 338–353. <https://doi.org/10.1016/j.istruc.2023.06.024>.
- [98] Y. Jahani, M. Baena, A. Codina, C. Barris, L. Torres, Time-dependent behavior of NSM CFRP-strengthened RC beams under different service temperatures, *Compos. Struct.* 300 (2022). <https://doi.org/10.1016/j.compstruct.2022.116106>.
- [99] H. Van Tran, S.Y. Seo, M.U. Hanif, K. Senghong, Time-dependent flexural behavior of the RC beams strengthened with FRP strips using different methods, *Structures*. 65 (2024) 106655.

- <https://doi.org/10.1016/j.istruc.2024.106655>.
- [100] A.K. El-Sayed, R.A. Al-Zaid, A.I. Al-Negheimish, A.B. Shuraim, A.M. Alhozaimy, Long-term behavior of wide shallow RC beams strengthened with externally bonded CFRP plates, *Constr. Build. Mater.* 51 (2014) 473–483. <https://doi.org/10.1016/j.conbuildmat.2013.10.055>.
 - [101] X. Li, J. Deng, Y. Wang, Y. Xie, T. Liu, K. Rashid, RC beams strengthened by prestressed CFRP plate subjected to sustained loading and continuous wetting condition: Time-dependent prestress loss, *Constr. Build. Mater.* 275 (2021) 122187. <https://doi.org/10.1016/j.conbuildmat.2020.122187>.
 - [102] M. Breveglieri, C. Czaderski, RC slabs strengthened with externally bonded CFRP strips under long-term environmental exposure and sustained loading. Part 2: Laboratory experiments, *Compos. Part C Open Access*. 6 (2021) 100210. <https://doi.org/10.1016/j.jcomc.2021.100210>.
 - [103] J. Bai, Y. Yang, P. Huang, X. Guo, Z. Chen, R. Wan-Wendner, W. Li, Durability of CFRP strengthened RC beam after six years exposure to natural hygrothermal environment with sustained loads, *Int. J. Fatigue*. 181 (2024) 108126. <https://doi.org/10.1016/j.ijfatigue.2023.108126>.
 - [104] E. Ahmed, H.R. Sobuz, Immediate and long-term deflection of carbon fiber reinforced polymer (CFRP) concrete beams, *Key Eng. Mater.* 471–472 (2011) 73–78. <https://doi.org/10.4028/www.scientific.net/KEM.471-472.73>.
 - [105] H.R. Sobuz, E. Ahmed, N.M. Sutan, N.M. Sadiqul Hasan, M.A. Uddin, M. Jahir Uddin, Bending and time-dependent responses of RC beams strengthened with bonded carbon fiber composite laminates, *Constr. Build. Mater.* 29 (2012) 597–611. <https://doi.org/10.1016/j.conbuildmat.2011.11.006>.
 - [106] J. Tatar, H.R. Hamilton, Bond Durability Factor for Externally Bonded CFRP Systems in Concrete Structures, *J. Compos. Constr.* 20 (2016). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000587](https://doi.org/10.1061/(asce)cc.1943-5614.0000587).
 - [107] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>.
 - [108] H. Ko, S. Matthys, A. Palmieri, Y. Sato, Development of a simplified bond stress-slip model for bonded FRP-concrete interfaces, *Constr. Build. Mater.* 68 (2014) 142–157.

- <https://doi.org/10.1016/j.conbuildmat.2014.06.037>.
- [109] K. Nakaba, T. Kanakubo, T. Furuta, H. Yoshizawa, Bond Behavior between Fiber-Reinforced Polymer Laminates and Concrete, *Struct. J.* 98 (2001). <https://doi.org/10.14359/10224>.
 - [110] J. Dai, T. Ueda, Y. Sato, Development of the Nonlinear Bond Stress-Slip Model of Fiber Reinforced Plastics Sheet-Concrete Interfaces with a Simple Method, *J. Compos. Constr.* 9 (2005) 52–62. [https://doi.org/10.1061/\(asce\)1090-0268\(2005\)9:1\(52\)](https://doi.org/10.1061/(asce)1090-0268(2005)9:1(52)).
 - [111] Z. Wu, S.M. Islam, H. Said, A three-parameter bond strength model for FRP-concrete interface, *J. Reinf. Plast. Compos.* 28 (2009) 2309–2323. <https://doi.org/10.1177/0731684408091961>.
 - [112] Y.W. Zhou, Y.F. Wu, Y. Yun, Analytical modeling of the bond-slip relationship at FRP-concrete interfaces for adhesively-bonded joints, *Compos. Part B Eng.* 41 (2010) 423–433. <https://doi.org/10.1016/j.compositesb.2010.06.004>.
 - [113] Y.-F. Wu, C. Jiang, Quantification of Bond-Slip Relationship for Externally Bonded FRP-to-Concrete Joints, *J. Compos. Constr.* 17 (2013) 673–686. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000375](https://doi.org/10.1061/(asce)cc.1943-5614.0000375).
 - [114] K. Park, K. Ha, H. Choi, C. Lee, Prediction of interfacial fracture between concrete and fiber reinforced polymer (FRP) by using cohesive zone modeling, *Cem. Concr. Compos.* 63 (2015) 122–131. <https://doi.org/10.1016/j.cemconcomp.2015.07.008>.
 - [115] F.M. Mukhtar, M.E. Shehadah, Experimental verification of 2- and 3-D numerical models for bond-slip behavior of CFRP-concrete, *Constr. Build. Mater.* 287 (2021) 122814. <https://doi.org/10.1016/j.conbuildmat.2021.122814>.
 - [116] Y.T. Obaidat, S. Heyden, O. Dahlblom, The effect of CFRP and CFRP/concrete interface models when modeling retrofitted RC beams with FEM, *Compos. Struct.* 92 (2010) 1391–1398. <https://doi.org/10.1016/j.compstruct.2009.11.008>.
 - [117] T. Cai, Z. Zhang, X. Wang, Z. Yang, C. Zhou, Y. Li, A mesoscale approach for the three-dimensional modelling of interfacial debonding between FRP and concrete, *Eng. Struct.* 327 (2025) 119663. <https://doi.org/10.1016/j.engstruct.2025.119663>.
 - [118] C. Mazzotti, M. Savoia, B. Ferracuti, An experimental study on delamination of FRP plates bonded to concrete, *Constr. Build. Mater.* 22 (2008) 1409–1421.

- <https://doi.org/10.1016/j.conbuildmat.2007.04.009>.
- [119] F. Bencardino, A. Condello, L. Ombres, Numerical and analytical modeling of concrete beams with steel, FRP and hybrid FRP-steel reinforcements, *Compos. Struct.* 140 (2016) 53–65. <https://doi.org/10.1016/j.compstruct.2015.12.045>.
- [120] M.R.T. Arruda, J.P. Firmo, J.R. Correia, C. Tiago, Numerical modelling of the bond between concrete and CFRP laminates at elevated temperatures, *Eng. Struct.* 110 (2016) 233–243. <https://doi.org/10.1016/j.engstruct.2015.11.036>.
- [121] J.P. Firmo, M.G. Roquette, J.R. Correia, A.S. Azevedo, Influence of elevated temperatures on epoxy adhesive used in CFRP strengthening systems for civil engineering applications, *Int. J. Adhes. Adhes.* 93 (2019) 102333. <https://doi.org/10.1016/j.ijadhadh.2019.01.027>.
- [122] A. Al-Saoudi, R. Al-Mahaidi, R. Kalfat, J. Cervenka, Finite element investigation of the fatigue performance of FRP laminates bonded to concrete, *Compos. Struct.* 208 (2019) 322–337. <https://doi.org/10.1016/j.compstruct.2018.10.001>.
- [123] M. Badawi, K. Soudki, Flexural strengthening of RC beams with prestressed NSM CFRP rods – Experimental and analytical investigation, *Constr. Build. Mater.* 23 (2009) 3292–3300. <https://doi.org/10.1016/j.conbuildmat.2009.03.005>.
- [124] M.R. Mostakhdemin Hosseini, S.J.E. Dias, J.A.O. Barros, Behavior of one-way RC slabs flexurally strengthened with prestressed NSM CFRP laminates – Assessment of influencing parameters, *Compos. Struct.* 245 (2020) 112259. <https://doi.org/10.1016/j.compstruct.2020.112259>.
- [125] M. Rezazadeh, J. Barros, I. Costa, Analytical approach for the flexural analysis of RC beams strengthened with prestressed CFRP, *Compos. Part B Eng.* 73 (2015) 16–34. <https://doi.org/10.1016/j.compositesb.2014.12.016>.
- [126] J. Sena-Cruz, J. Michels, Y.E. Harmanci, L. Correia, Flexural strengthening of RC slabs with prestressed CFRP strips using different anchorage systems, *Polymers (Basel)*. 7 (2015) 2100–2118. <https://doi.org/10.3390/polym7101502>.
- [127] W.T. Jung, J.S. Park, J.Y. Kang, M.S. Keum, Flexural behavior of concrete beam strengthened by near-surface mounted CFRP reinforcement using equivalent section model, *Adv. Mater. Sci. Eng.* 2017 (2017) 16. <https://doi.org/10.1155/2017/9180624>.

- [128] Y. Deng, Z. Li, H. Zhang, A. Corigliano, A.C.C. Lam, C. Hansapinyo, Z. Yan, Experimental and analytical investigation on flexural behaviour of RC beams strengthened with NSM CFRP prestressed concrete prisms, *Compos. Struct.* 257 (2021) 113385. <https://doi.org/10.1016/j.compstruct.2020.113385>.
- [129] ACI 440.2R-08, American Concrete Institute (ACI). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures., USA, 2008.
- [130] R.I. Gilbert, Time effects in concrete structures, 1989. [https://doi.org/10.1016/0262-5075\(89\)90042-0](https://doi.org/10.1016/0262-5075(89)90042-0).
- [131] Building Code Requirements for Structural Concrete (ACI 318-19) Commentary on, Building Code Requirements for Structural Concrete (ACI 318R-19), An ACI Standard, ACI Comm. 318. (2019).
- [132] H.F. Brinson, L.C. Brinson, Polymer Engineering Science and Viscoelasticity An Introduction, Springer, New York, USA, 1982.

CHAPTER 3

SUMMARY OF APPENDED PAPERS

In this chapter, a summary of the journal papers constituting this PhD thesis is provided. As previously mentioned in **Chapter 1**, different specific objectives were defined, and therefore, the journal papers were prepared in line with those predefined objectives. The first objective was to address the durability of epoxy adhesives and CFRP laminates used in strengthening systems. **PAPER I** addressed this objective. The second objective was to address the durability of bond between the CFRP laminate and concrete. Since two different strengthening techniques to bond the CFRP to concrete were studied, two different papers were prepared for addressing the experimental works. That is, **PAPER II** addressed the durability of the bond for the case when the CFRP laminate is externally bonded to concrete, commonly known as, Externally Bonded Reinforcement (EBR) technique, while **PAPER III** addressed the durability of the bond for the case when the CFRP laminate is inserted into a groove pre-cut into concrete, commonly known as Near-Surface Mounted (NSM) technique. The third objective was about developing simulations of the short- and long-term behavior of the bond between the CFRP laminate and concrete for the case of the EBR technique.

PAPER IV addressed the finite element modeling of the EBR CFRP-to-concrete joint without considering durability (short-term behavior). **PAPER V** addressed the finite element modeling of EBR CFRP-to-concrete joint under water immersion for up to 4 years (long-term behavior). On the other hand, two papers were developed in the context of the fourth objective, to predict both short- and long-term flexural performances of the studied RC slabs. That is, **PAPER VI** addressed the development of analytical models for predicting the short-term flexural behavior of five different RC slabs: (i) without a strengthening system, (ii) strengthened with EBR technique, (iii) strengthened with NSM technique, (iv) strengthened with EBR prestressed via mechanical anchorage (MA) system, and (v) strengthened with EBR prestressed via gradient anchorage (GA) system.

In **PAPER VII**, the approaches developed in **PAPER VI** were further extended to predict the instantaneous, and long-term creep/shrinkage-based deflections of the mentioned strengthened RC slabs under sustained loading and varying environmental exposures; the predicted deflections were compared with the deflections monitored for up to 6 years. Furthermore, the developed models were used to estimate deflections for up to 50 years. Finally, the fifth objective was to examine correlations between accelerated and natural aging conditions and provide design recommendations. This objective was addressed in **PAPER I**, **PAPER II**, and **PAPER III**.

The main content of this chapter is summarized in **Figure 3.1**, which categorizes the papers into experimental, numerical, and analytical studies.

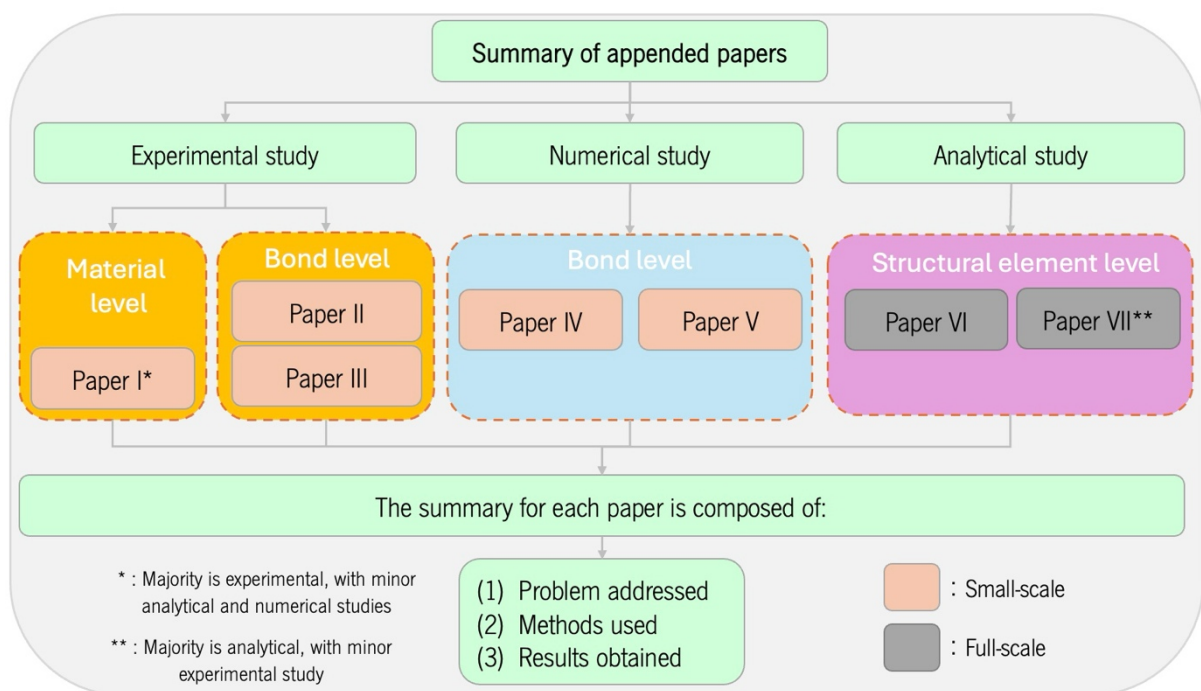


Figure 3.1: A graphical summary of appended papers

The figure, furthermore, classifies each paper based on the specimen's scale (small-scale or full-scale) and level (material, bond, or structural). Each paper summarizes: the problem addressed, the methodologies employed, and the results obtained.

3.1. PAPER I

- *Aloys Dushimimana, Anastasios P. Vassilopoulos, José Sena-Cruz, João M. Pereira, Luís Correia, Susana Cabral-Fonseca, Ricardo Cruz. Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-controlled environments. Construction and Building Materials 438 (2024); <https://doi.org/10.1016/j.conbuildmat.2024.137201>.*

Impact Factor: 7.4 (April 2025)

3.1.1. Problem addressed

With existing literature generally focusing on behavior of materials constituting the CFRP-to-concrete bond when exposed to laboratory-based accelerated aging environments, there is a gap in comprehensive studies addressing outdoor environments, which could help to better assess the environmental conversion factors. Hence, the durability of CFRP-to-concrete bond constituent materials (epoxy adhesives and CFRP laminates) for up to 4 years of exposure to different laboratory-based and outdoor environments was addressed in this publication.

3.1.2. Methodologies employed

Two different types of epoxy adhesives produced by different companies, and two different types of CFRP laminates produced by the same company, were exposed to both laboratory-based and outdoor environments. Laboratory-based environments included: (i) conditioning the specimens at 20 °C and 55 % relative humidity (denoted as E1) and tap water immersion of the specimens at a controlled temperature of 20 °C (denoted as E2). The outdoor environments included 4 different types of outdoor environments. These environments were selected to promote mainly the degradation due to carbonation (denoted as E3), freeze-thaw attacks (E4), high temperatures (E5) and airborne chlorides (E6). Both materials were tested yearly to assess the effects of the above environments on their mechanical properties. It is important to note that although these outdoor environments were selected based on regions where a specific degradation agent is expected to dominate, outdoor conditions remain uncontrollable. This challenge is further compounded by the growing impacts of climate change. As a result, multiple degradation agents may be present in each environment, though the intended dominant agent likely remains the most significant.

3.1.3. Results obtained

Both types of epoxy adhesives showed substantial decreases in their tensile properties in E2, which led to concluding that long-term exposure to tap water can significantly deteriorate the adhesives' ability under tension. Other environments also affected the adhesive properties but not as significant as in E2.

For the case of CFRP laminates, one type of CFRP laminate showed insignificant changes in tensile properties with time in all environments, while the other showed significant changes, which led to concluding that the width and thickness of the CFRP laminate can play a major role in terms of long-term performance properties.

Different environmental conversion factors were derived and proposed for both the epoxy adhesives and CFRP laminates.

3.2. PAPER II

- Aloys Dushimimana, José Sena-Cruz, Luís Correia, João M. Pereira, Susana Cabral-Fonseca, Ricardo Cruz. *Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated aging exposures*, *Construction and Building Materials*. 427 (2024); <https://doi.org/10.1016/j.conbuildmat.2024.136213>

Impact factor: 7.4 (April 2025)

3.2.1. Problem addressed

The study addressed the durability of the EBR CFRP-to-concrete bond after 4 years of exposure to various laboratory-based and outdoor environments. Existing literature often focuses on laboratory conditions, using an environmental conversion factor to estimate real outdoor effects. However, results from outdoor-exposed specimens can better predict appropriate factors, especially when correlated with laboratory-based (accelerated aging) results. Since the existing literature presents very few works developed within the context of investigating the behavior of the bond under real outdoor environments, the present paper contributed to increasing the data from different NAT protocols, as well as, to establishing relationships between the effects of exposure conditions from the AAT and NAT protocols.

3.2.2. Methodologies employed

To address the knowledge gap mentioned in **Section 3.2.1**, concrete prisms strengthened with CFRP laminates according to the EBR technique (denoted as EBR specimens) were subjected to both laboratory and outdoor environments. These environmental exposures were like those used in **PAPER I**. The EBR

specimens were investigated annually by performing pull-out tests to evaluate the impact of these environments on the EBR CFRP-to-concrete bond, focusing on bond strength and bond stiffness. In addition, concrete compression and pull-off tests, along with tensile tests of epoxy adhesive and CFRP laminate, were conducted to evaluate the effects of material degradation on bond performance.

3.2.3. Results obtained

The compressive strength (f_c) and elastic modulus (E_c) of concrete exhibited notable improvements in outdoor settings (E3-E6), while only slight changes were noted in laboratory conditions (E1 and E2). Considering all studied environments, the most significant increases were around 21 % of f_c in environment E3, and 12 % of the E_c in environment E4. In contrast, the pull-off strength (f_a) of the concrete consistently decreased over time across all environments examined. The behaviors for the epoxy adhesive and CFRP laminate are reported in **PAPER I**.

The bond strength and stiffness exhibited significant variations over time and across different environments. Bond stiffness generally degraded faster than bond strength in most conditions. This was especially evident in elevated temperature environments (E5), where bond stiffness decreased by 39 %, while bond strength showed only minor changes. In laboratory conditions (E1 and E2), bond strength tended to decrease over time. In outdoor environments, bond strength in E3 and E4 generally increased over time but with notable fluctuations, likely due to exposure duration, severity, and molecular adjustments in the bonded region. Conversely, bond strength remained relatively stable in E5 and E6, indicating minimal impact from the degradation agents in these settings. Bond stiffness, however, consistently decreased across all environments, with the most significant reductions observed in E4 (50 %), followed by E2 (40 %), E5 (39 %), and E3 (35 %). Smaller reductions were noted in E1 and E6.

Different failure modes were observed after exposure, indicating that environmental conditions influence the failure mode. Generally, higher pull-out forces were associated with specimens exhibiting a combination of adhesive-CFRP interaction layer failure and concrete cohesive failure, rather than concrete cohesive failure alone. The larger the area of adhesive-CFRP interaction failure, the higher the pull-out force. Additionally, the specimens with some fracture or splitting of CFRP along its longitudinal direction consistently resulted in the highest pull-out forces, making it the most desirable failure mode. It was additionally found that multiple degradation agents may have had more effect than a single degradation agent, hence, strengthening the presence of synergistic effects, particularly in outdoor environments.

3.3. PAPER III

- *Aloys Dushimimana, José Sena-Cruz, Luís Correia, João M. Pereira, Susana Cabral-Fonseca, Ricardo Cruz. Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, Composite Structures. (2024) 118174; <https://doi.org/10.1016/j.compstruct.2024.118174>*
Impact factor: 6.3 (April 2025)

3.3.1. Problem addressed

The study examined the durability of the NSM CFRP-to-concrete bond over a period of up to 4 years in various laboratory and outdoor environments. This area of research is underrepresented in existing literature, which predominantly focuses on laboratory-based AAT protocols. Consequently, there is a lack of understanding of effects from the NAT protocols, which is essential since built structures are typically exposed to natural outdoor conditions. Moreover, there is little or no existing literature that correlates both the AAT and NAT results, raising questions about the validity of current environmental conversion factors that are based solely on the AAT protocols.

3.3.2. Methodologies employed

Concrete cubes strengthened with CFRP laminates according to NSM technique (denoted as NSM specimens) were exposed to both laboratory and outdoor conditions, these conditions were like those described in **PAPER I**. To assess the bond performance over time in each environment, pull-out tests were performed yearly on the NSM specimens from which the bond strength and bond stiffness were obtained.

3.3.3. Results obtained

The behaviors of materials constituting the NSM CFRP-to-concrete bond were as reported in **PAPER I** and **PAPER II**.

Bond strength and stiffness varied over time and across environments. The greatest reductions in bond strength occurred under continuous water immersion (E2) and freeze-thaw cycles (E4), with decreases of nearly 12 % and 9 %, respectively. In contrast, bond strength in E1 improved by about 10 % in the final year. Outdoor environments (E5 and E6) also showed improvements in bond strength, suggesting benefits from high temperatures and airborne chlorides, with concrete property changes enhancing bond strength in E6. Atmospheric CO₂ had minimal impact. Bond stiffness generally decreased in the first year

(approximately 0.7 retentions) but increased in subsequent years, especially in laboratory-controlled environments.

Visual examination of failure modes revealed changes after exposure. Initially (T0), failure at the adhesive-CFRP interface (F/A) was prevalent in E1 and E5. In E2, a combination of adhesive failure (A) with concrete cohesive failure (CC) or concrete splitting (CS) was dominant. E3 primarily showed F/A, while E4 exhibited a mix of F/A and CS due to the prolonged negative effects of freeze-thaw cycles. In E6, either F/A or a combination of F/A and CS was observed.

3.4. PAPER IV

- *Aloys Dushimimana, João M. Pereira, Luís Correia, José Sena-Cruz, Anastasios P. Vassilopoulos. Finite element modeling of bond between concrete and CFRP laminate. Manuscript to be submitted to a journal.*

3.4.1. Problem addressed

Finite element modeling of the EBR CFRP-to-concrete joint was developed, addressing the relative scarcity of research in this area. Most of existing studies have generally adopted the bond stress-slip law to study the bond behavior between concrete and CFRP laminate, where a cohesive or connector element is used at the concrete-CFRP interfaces. An alternative simplified approach with a perfect bond at the interfaces was introduced by incorporating the geometries and properties of all materials constituting the joint, and by simulating the nonlinear behavior of both concrete and epoxy adhesive layer using damage plasticity model. The perfect bond was chosen given that many existing works reported a cohesive failure rather than interface failure.

3.4.2. Methodologies employed

An approach to modeling the interface between concrete and CFRP laminate, incorporating the real properties of epoxy adhesive in the well-known concrete damage plasticity (CDP) model, was developed as an alternative to the bond stress-slip law. Experimental data and various mathematical formulations were used to determine input parameters for the CDP model available in ABAQUS software. The reliability of this approach was verified using seven different combinations of mathematical formulations to define the constitutive models of concrete and epoxy adhesive under tension and compression. The most promising combination was further examined to understand the model's behavior, focusing on the evolution of damages, displacements, stresses, strains, and failure modes.

3.4.3. Results obtained

Two different responses were analyzed, at (i) material level and at (ii) interface level, described as follows. At the *material level*, concrete experienced much higher damage under compression and tension compared to the adhesive, reaching limit values in almost the entire bonded regions at failure. Adhesive damage was minor, occurring only at a small region of the loaded end and not reaching limit values. Thus, failure was attributed to concrete damage. Both compressive and tensile damages in concrete propagated deeply, indicating concrete cohesive failure. Stress concentrations in concrete were higher at the loaded end and progressively moved towards the free end. For the adhesive, slip and stress profiles resembled those of the CFRP laminate, aligning well with previous experimental studies, hence, confirming the model's reliability. At the *interface level*, the slip at the lower concrete-adhesive interface was smaller than that at the upper adhesive-CFRP interface up to approximately 80 mm from the loaded end during initial loading, becoming uniform at later stages. Higher slip values were found at the loaded end. Stress trends differed at the concrete-adhesive interface due to the distinct stress profiles of concrete and adhesive resulting from different load transfer mechanisms, unlike the similar profiles observed for the adhesive and CFRP laminate.

3.5. PAPER V

- *Aloys Dushimimana, João M. Pereira, Luís Correia, Anastasios P. Vassilopoulos, José Sena-Cruz. Modeling of EBR CFRP-to-concrete joint under long-term water immersion. Manuscript to be submitted to a journal.*

3.5.1. Problem addressed

Finite element models were developed to assess the durability of the EBR CFRP-to-concrete joint under water. The models specifically addressed the impact of continuous exposure of the joint to tap water controlled at a temperature of 20 °C. Such long-term exposure models, spanning up to 4 years, are rare in existing literature. Thus, this work helped fill that knowledge gap.

3.5.2. Methodologies employed

Experimental results and existing mathematical equations were utilized to develop constitutive models for concrete and epoxy adhesive. These models were then input into the concrete damage plasticity (CDP) model in ABAQUS to simulate the behavior of the EBR CFRP-to-concrete joint over time. The developed models were validated using experimental data.

3.5.3. Results obtained

The yearly predicted pull-out force *versus* loaded end slip curves matched well with experimental observations. The predicted failure mode (concrete cohesive failure) also aligned with experimental results. The model explains the observed removal of a thick concrete layer at the free end region: this removal is due to high concrete tensile damage occurring at the final loading stage, leading to a sharp increase in its compressive damage across the bonded region. This tensile damage resistance at the free end causes failure through stress release, producing a loud sound, just after compressive damage has spread throughout the bonded area.

The obtained strain profiles were like those typically found in existing literature, indicating the ability of the developed model to produce the responses commonly observed in experimental tests. Additionally, CFRP strains showed consistent trends from the 1st to the 4th year of exposure, despite changes in their maximum values over time, suggesting that water exposure did not alter the trends.

3.6. PAPER VI

- *Aloys Dushimimana, Luís Correia, João M. Pereira, José Sena-Cruz. Analytical approaches for flexural analysis of RC slabs strengthened with CFRP laminates. Manuscript to be submitted to a journal.*

3.6.1. Problem addressed

Several analytical models for predicting the flexural behavior of RC beams and slabs have been developed in the existing literature. However, some aspects still need further research, namely,

- Existing analytical predictions using the trilinear moment-curvature relationship often exclude the effect of the failure mode, though some studies partially address it. The ultimate capacity is influenced by the type of failure, necessitating a strategy to select the prevailing failure mode for estimating ultimate moment, curvature, and load.
- Curvature for deflection calculations depends on the failure mode as it affects the neutral axis location, which varies with different failure modes.
- In prestressed systems, curvature at all stages is influenced by initial prestress due to initial camber. Additionally, current literature lacks an analytical approach using different deflection formulae at various loading states, which could improve stiffness prediction.

- Various formulations predict FRP failure strain, but comparative studies are needed to assess their reliability.

This investigation aimed to address these four points.

3.6.2. Methodologies employed

Five different RC slabs were analytically modelled using a trilinear moment-curvature relationship with pre-cracking, post-cracking, and post-yield stages, established incorporating the prevailing failure mode, and initial strain-curvature dependence for the case of prestressed systems. For each RC slab, various analytical formulations were used. In these formulations, the determination of the FRP failure strain was calculated referring to existing standards corresponding to each RC slab; since many standards were adopted for each RC slab, their performances were also compared. Two existing formulations for predicting the mid-span deflection were adopted and their performances were compared for each slab. For RC slabs with EBR prestressed via MA and GA systems, new formulations for predicting curvature were developed. Their performances were then compared with existing curvature equations.

3.6.3. Results obtained

The analytical approaches were found promising, accurately predicting the responses of the RC slabs at cracking, yielding, and ultimate stages, aligning well with experimental test results. Additionally, the efficiency and ductility parameters, as well as the failure modes, matched those observed in experimental tests.

The applicability of the two deflection prediction formulas depends on the loading state, making a hybrid approach more promising. Both formulae are effective at cracking state. The integration-based formula is better when predicting responses at yielding state for all slabs, and at ultimate state for slabs failing by debonding. Conversely, the determinant section formula is more effective at ultimate state for slabs failing by FRP rupture.

Failure load predictions from different code provisions and existing literature vary, with some being more beneficial based on the strengthening technique and failure mode. Generally, *fib*90, CNR, and ACI provisions equally predict the CFRP failure strain. For prestressed systems, new curvature formulations lead to promising predictions of the mid-span deflection, thereby matching the experimental deflection results.

3.7. PAPER VII

- *Aloys Dushimimana, José Sena-Cruz, João M. Pereira, Luís Correia. Short- and long-term deflections of RC slabs strengthened with CFRP laminate under sustained loading in different environments. Manuscript to be submitted to a journal.*

3.7.1. Problem addressed

Long-term behavior of RC slabs strengthened with CFRP laminates under a sustained load when exposed to different environments is still a research topic that needs further investigations due to insufficient data available in the literature. **PAPER VII** aimed at experimentally and analytically examining the instantaneous and long-term creep/shrinkage-based deflections of the slabs exposed to laboratory-based and outdoor environments for up to 6 years, and estimations of deflections for longer periods (50 years).

3.7.2. Methodologies employed

Four different RC slabs strengthened with CFRP laminates applied according to the EBR, NSM, EBR prestressed via MA, and EBR prestressed via GA were exposed to different environments, each slab being subjected to a sustained load of approximately 40 % of its ultimate load. The environments used were like those previously described in **PAPER I**. For each RC slab, the experimental long-term deflections were monitored for up to 6 years. Furthermore, the established moment-curvature relationships from the short-term flexural analysis of the RC slabs in **PAPER VI** were extended to predict the monitored long-term deflections using: (i) age-adjusted effective modulus method (AEMM), (ii) ACI318R-19 simplified approach, and (iii) extension of (ii) to predict deflections up to 50 years and establish the time-dependent factor curves governing the studied slabs.

3.7.3. Results obtained

The monitoring works showed that the evolution of deflections of strengthened RC slabs under a sustained load presented two phases: instantaneous deflection and long-term creep/shrinkage-related deflection. The RC slabs under continuous water immersion (E2) showed insignificant increase of creep/shrinkage-based deflection which is attributed to the fact that creep of concrete under water is minimal as moisture migration (which is a major driver of creep) within concrete is prevented by water bodies; also, thermal stresses are minimal due to stabilized temperature. For RC slab in E1, the deflection increased with time due to sustained load causing both basic and drying creep effects; similarly, in outdoor environments, all RC slabs showed an increasing trend of deflection over time. The RC slabs strengthened with EBR prestressed via GA system under high temperature regions (E3 and E5) showed a debonding failure, highlighting the negative effects

of high levels of temperatures and CO₂ emissions on the performance of the adhesive in this type of prestressing system. Also, the prestressed systems presented higher creep/shrinkage-based deflection compared to non-prestressed ones, indicating that the increased compression zone for the former partially increased the creep over time.

The instantaneous deflection can well be predicted using the mathematical formulations from existing literature. The long-term deflection can be also well predicted using the AEMM but with appropriate creep coefficient that incorporates the effect of strengthening and conditioning type, and with appropriate bond-dependent coefficient. Use of ACI318R-19 approach can lead mostly to overpredictions; however, it can accurately predict the deflections if the time-dependent factor curve is improved to incorporate the effects of strengthening and conditioning types. Given its simplified form, the ACI318R-19 can further be used to extrapolate deflections for longer periods such as 50 years or more, thereby serving an important role in terms of design practice

CHAPTER 4

CONCLUSIONS AND FUTURE DEVELOPMENTS

Prior to presenting the main conclusions and recommendations, an overview of the investigated specimens, exposure conditions, and study types is provided. Small-scale specimens included concrete cylinders and prisms, two epoxy adhesives (ADH1, ADH2), two unidirectional CFRP laminates (L10, L50), and CFRP-strengthened concrete elements using EBR (L50) and NSM (L10) techniques, evaluated for both short- and long-term performance. Full-scale specimens comprised unstrengthened RC slab and slabs strengthened with EBR, NSM, and prestressed EBR systems, assessed for initial flexural behavior, and long-term performance under sustained loading in varying environments.

The study considered both laboratory (E1: 20 °C/55 % RH; E2: water immersion at 20 °C) and outdoor exposures (E3: carbonation; E4: freeze-thaw; E5: high temperature; E6: marine chlorides). While specific degradation mechanisms were targeted, uncontrolled outdoor conditions likely introduced additional synergistic effects.

The study involved: (i) mechanical and durability tests on small-scale specimens before exposure (T0) and after 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of exposure conditions; (ii) numerical and analytical modeling of water diffusion in adhesive; and numerical modeling of the EBR joint behavior; (iii) long-term monitoring (up to 6 years) of mid-span deflections of full-scale slabs; and (iv) development of predictive models for short-term flexural behavior and long-term deflections (up to 50 years).

4.1. CONCLUSIONS

The conclusions are structured hierarchically from material to structural element levels. The first two sections provide findings from experimental works at material and bond levels, followed by numerical modeling of the EBR system. The next section provides insights on the experimentally monitored slab deflections, followed by two sections addressing the findings from developed analytical models for the flexural performance of the slabs. The final section proposes environmental conversion factors at the material and bond levels.

4.1.1. Durability of CFRP-to-concrete bond constituent materials

Concrete

Environmental exposure significantly influenced the long-term mechanical performance of concrete. Outdoor conditions promoted continued cement hydration, resulting in marked increases in compressive strength (up to 21 % at T2) and elastic modulus (up to 12 % at T2). In contrast, specimens in controlled laboratory environments exhibited limited or negative mechanical development. Despite these gains, pull-off strength consistently declined across all exposure conditions, indicating progressive near-surface degradation. These findings underscore the critical role of environmental factors on the durability and mechanical evolution of concrete over time. The failure modes observed in concrete testing remained consistent across exposure conditions, indicating that exposure had no effect on failure mode.

Epoxy adhesives

Adhesive tensile strength (f_a) and elastic modulus (E_a) generally declined across all environments, but with some improvements in E_a observed during the first year in E1, E3, E4, and E5 associated with post-curing. In E2, ADH1 exhibited substantial reductions reaching up to approximately 66 % (f_a) at T2 and 75 % (E_a) at T2, while ADH2 showed similar losses of approximately 60 % at T3 and 70 % at T3, respectively, indicating that E_a deteriorated more rapidly than f_a . ADH1 demonstrated minimal benefit from post-curing, leading to progressive degradation over time. In contrast, ADH2 experienced substantial post-curing effects in E6, with initial gains of up to 40 % (f_a) at T1 and 50 % (E_a) at T1, effectively mitigating long-term degradation. Even

after four years, ADH2 maintained superior properties relative to unexposed specimens, highlighting the efficacy of post-curing in enhancing long-term adhesive performance.

All tested adhesives exhibited abrupt brittle failure, indicating that exposure did not affect the failure mode.

Modeling of water diffusion through epoxy adhesive

Analytical modeling of ADH1 exposed to tap water at 20, 40, and 60 °C was conducted using two diffusion equations. While the Fickian-based model captured only physical aging, a newly proposed equation accounted for both physical and chemical aging, showing better alignment with experimental results, particularly at higher temperatures. Additionally, numerical modeling using ABAQUS at 20 °C simulated mass diffusion through the adhesive, with results indicating rapid water penetration within approximately 6100 hours. This modeling helped explain the experimentally observed reductions in tensile strength and elastic modulus during the first year, attributing them primarily to the plasticization effect induced by fast water diffusion.

CFRP laminates

Exposure of the CFRP laminate L10 (10 mm wide, 1.4 mm thick) to different environments improved both tensile strength (f_t) and elastic modulus (E) due to the continuation of the post-curing phenomenon of the fiber matrix resin, mainly occurring during the first year. In the following years, properties started to decrease as degradation agents increased and the post-curing rate decreased. However, because of substantial post-curing during the first year, the properties, although decreasing over time, remained higher than those of non-exposed specimens after 4 years. Exposure of specimens in the regions with high levels of CO₂ emissions and freeze-thaw attacks increased f_t , the highest being approximately 16 and 15 % at T1, respectively. Additionally, regions with high levels of temperatures and CO₂ emissions caused the E to increase with time, the highest increase being about 10 % at T4.

In contrast, the exposure of the CFRP laminate L50 (50 mm wide, 1.2 mm thick) did not improve its elastic modulus (E); instead, a significant decrease was observed during the first year. However, its tensile strength (f_t) improved initially but started to decrease in later years, with f_t values at T4 being lower than those of non-exposed specimens. Comparing L10 and L50, it was concluded that the laminate with smaller width and slightly larger thickness had better durability performance. Therefore, the width and thickness of the CFRP laminate are crucial parameters when designing structures with CFRP composites.

All tested CFRP laminates exhibited abrupt brittle failure, indicating that exposure did not affect the failure mode.

4.1.2. Durability of CFRP-to-concrete bond

EBR CFRP-to-concrete bond

Bond strength and bond stiffness varied significantly with time and environment. The latter was found to degrade faster than the former in most of the environments. This was particularly noticeable when the EBR CFRP-to-concrete joint was conditioned in regions with elevated temperatures (E5), which led to a 39 % (T4) reduction in the bond stiffness, whereas marginal change was observed for the bond strength. A tendency for bond strength to decrease over time was generally observed in E1 and E2.

In outdoor environments, bond strength in E3 and E4 exhibited a general increasing trend over time, albeit with notable fluctuations likely due to varying exposure severity and time-dependent material interactions. In contrast, bond strength remained largely stable in E5 and E6, suggesting limited impact from the associated degradation agents. Bond stiffness, however, consistently declined across all environments, with the most severe reductions being observed in E4 (50 % at T4), followed by E2 (40 % at T4), E5 (39 % at T4), and E3 (35 % at T4). Minimal stiffness degradation occurred in E1 and E6.

Initially at T0, failure modes were mainly pure concrete cohesive or mixed with adhesive cohesive. Environmental exposure led to various failure modes, mostly due to interfacial separation between the adhesive and concrete or CFRP. Concrete cohesive failure with a thin layer dominated in E1 and E5, while E2 showed concrete cohesive failure with or without a thick layer at the free end. E3 and E4 had mixed failures of concrete cohesive with a thin layer and interfacial adhesive failure. E6 showed both mixed failure modes and concrete cohesive failure with a thin layer. Higher pull-out forces were linked to mixed adhesive-CFRP and concrete cohesive failures, with larger areas of the former resulting in higher forces. CFRP fracture/splitting always resulted in the highest pull-out force, making it the most desirable failure mode.

NSM CFRP-to-concrete bond

Bond strength and stiffness exhibited time- and environment-dependent variations. The greatest bond strength reductions occurred under continuous water immersion (E2) and freeze-thaw exposure (E4), with average decreases of approximately 12 % and 9 % from T1 to T4, respectively. Conversely, bond strength increased in E1 (approximately 10 % at T4) and improved in E5 and E6, suggesting that high temperatures and marine environments may enhance NSM bond performance—likely aided by concurrent improvements

in concrete properties, particularly in E6. The influence of elevated atmospheric CO₂ was minimal, resulting in negligible bond strength changes. Bond stiffness generally declined during the first year but increased thereafter, particularly in specimens exposed to laboratory-controlled conditions.

Visual examination of failure modes showed that the failure mode at T0 changed after exposure. In general, the failure at adhesive-CFRP interface (F/A) dominated in E1 and E5; the combination of adhesive failure (A) with concrete cohesive failure (CC) or with concrete splitting (CS) dominated in E2; F/A dominated in E3, a combination of F/A and CS dominated in E4 mainly because of the long-lasting negative effects from freeze-thaw attacks on both adhesive and CFRP properties; F/A or a combination of F/A and CS dominated in E6.

4.1.3. Finite element modeling of short- and long-term behavior of EBR CFRP-to-concrete joint

Modeling short-term behavior of EBR CFRP-to-concrete joint

A finite element model was developed in ABAQUS to simulate the short-term behavior of EBR CFRP-to-concrete joints, incorporating nonlinearities in both concrete and epoxy adhesive while assuming linear elastic behavior of the CFRP laminate up to failure. Various constitutive models were employed to accurately represent the material properties.

The model demonstrated high accuracy in predicting pull-out force-slip responses and failure modes across seven different constitutive model combinations, all aligning well with experimental observations. Analysis using the optimal model revealed key insights at both the material and joint levels. At the material level, concrete exhibited extensive damage in both tension and compression, indicating it as the primary source of failure, while adhesive damage remained minimal and localized near the loaded end. The predicted CFRP stress profiles were consistent with experimental results, validating the model's reliability. At the joint level, slip and stress profiles showed non-uniform displacement between interfaces initially, becoming uniform thereafter. The epoxy adhesive played a crucial role in load transfer, with its mechanical properties significantly influencing stress distribution and deformation across the joint.

Modeling long-term behavior of EBR CFRP-to-concrete joint under water

A finite element model was developed in ABAQUS to simulate the long-term durability of externally bonded reinforcement (EBR) CFRP-to-concrete joints subjected to water immersion at 20 °C. The model was validated against experimental data and demonstrated high predictive accuracy in reproducing bond behavior under environmental exposure.

The model successfully captured the annual pull-out force-slip responses and concrete cohesive failure patterns, including the characteristic removal of a thick concrete layer at the free end, as observed experimentally. It revealed a higher susceptibility to damage in the adhesive layer near the loaded end, with more pronounced material degradation in the early stages of exposure. Bond stress-slip and material degradation increased more rapidly in the first than in the fourth year of water exposure, explained by a delayed damage progression due to an improvement in bond performance in the latter year from ongoing cement hydration.

Additionally, the model accurately predicted CFRP strain distributions over time, aligning with experimental trends reported in the literature. While strain magnitudes varied annually, the overall profiles remained consistent, indicating that water exposure does not significantly affect the strain distribution trend. These findings affirm the robustness of the developed model and its potential utility in simulating long-term durability of CFRP-concrete joints.

4.1.4. Monitored short- and long-term deflections of RC slabs with CFRP systems under sustained load and environmental conditioning

The instantaneous deflections of the RC slabs with CFRP systems under a sustained stress and environmental conditioning varied depending on the strengthening system. The slabs strengthened with EBR-MA and EBR-GA systems showed smaller instantaneous deflections compared to those with non-prestressed EBR systems; hence concluding that the prestressed systems can be a solution for designs requiring reduced deflections. The effect of CFRP axial stiffness was also revealed on the instantaneous deflection control: the larger the stiffness the smaller the instantaneous deflection.

The monitored time-dependent deflections due to creep and shrinkage revealed that non-prestressed strengthening systems (EBR and NSM) exhibit smaller deflections than prestressed systems (EBR-MA and EBR-GA). This suggests that the initial prestressing increased the compression zone, thereby amplifying creep effects. Environmental conditions significantly influenced the deflection trends: slabs in E2 showed minimal deflection progression due to the absence of drying creep and stable thermal conditions, while those in E5 exhibited the highest deflections, likely due to elevated temperatures weakening the bond over time. Notably, prestressed systems—particularly EBR-GA—showed debonding under high-temperature environments (E3 and E5), consistent with findings from previous studies, highlighting its thermal sensitivity.

The adoption of the Generalized Kelvin (GK) chain model proved effective in establishing analytical curves that fit experimentally monitored deflections of CFRP-strengthened RC members. Despite the model's

inherent assumption of material viscoelasticity—originally not intended for composite RC systems—its application is justified, as concrete’s long-term behavior, which primarily governs RC performance, is viscoelastic in nature. The model yielded well-fitted curves and enabled the development of a creep deflection model, with extracted parameters that not only enhance understanding of slab performance under sustained loading and environmental conditioning but also support the formulation of analytical models. Furthermore, these parameters allowed for the prediction of deflections up to 50 years, producing realistic long-term estimates.

4.1.5. Analytical modeling of short-term flexural performance of RC slabs with CFRP systems

A comprehensive trilinear moment–curvature relationship was developed for each investigated RC slab, encompassing pre-cracking, post-cracking, and post-yield stages, and incorporating the prevailing failure mode and initial strain–curvature dependence for prestressed systems. Predictions were made for structural responses at key limit states—cracking, yielding, and ultimate—using two methods for deflection prediction and various existing models for estimating CFRP failure strain. The analytical models demonstrated strong agreement with experimental observations, accurately capturing failure modes, efficiency, and ductility parameters.

The suitability of deflection prediction formulae is dependent on the loading state. While both studied formulae performed well at the cracking state, a hybrid approach proved more robust across different limit states. The integration-based method was more accurate at the yielding state and for ultimate state predictions involving debonding failure, whereas the determinant section-based method was more effective when predicting the responses at ultimate state for slabs failing by FRP rupture. Furthermore, predictions of ultimate state varied among design code provisions, with *fib*90, CNR, and ACI offering comparable accuracy in estimating CFRP failure strains. For prestressed systems, newly developed curvature expressions yielded reliable deflection predictions agreeing with the experimentally recorded data. In summary, the study identified the most suitable analytical approaches for each slab configuration and failure mode, offering a validated framework for future research and design guidance.

4.1.6. Analytical modeling of short- and long-term deflections of RC slabs with CFRP systems under sustained load and environmental conditioning

The use of section analysis incorporating the Gilbert formula alongside the tension stiffening model proposed by *fib* MC (2020) proved effective in predicting the instantaneous deflections of CFRP-strengthened RC

slabs, showing good agreement with experimental observations and confirming the reliability of this approach.

The long-term deflection of CFRP-strengthened RC slabs was predicted using several analytical approaches. The combination of the age-adjusted effective modulus method (AEMM) and the modified tension stiffening model yielded predictions in close agreement with experimental results, demonstrating its reliability across various slab types. However, its accuracy depends on well-calibrated creep coefficients; also, it depends on bond-dependent coefficient which still needs further refinement for generalized applications, particularly for the quite new systems such as the prestressed CFRP strengthening systems. Additionally, while the ACI 318R-19 approach tends to overestimate deflections, given its simplified form ideal for design practice, it offers potential for deriving refined time-dependent factor curves to enhance its applicability to FRP-strengthened elements under complex exposure conditions. The GK model enabled long-term extrapolation of ACI 318R-19 predictions, revealing ongoing deflection growth beyond five years (mostly reaching up to 20 years) and supporting the development of revised time-dependent factors that account for strengthening systems and environmental exposure.

4.1.7. Proposed environmental conversion factors

Derived environmental conversion factors (ECF) were compared with those from existing standards. For epoxy adhesives, the ECF under water immersion were lower, likely due to shorter exposure time or differing adhesive types addressed in the standards; however, outdoor values were comparable. CFRP laminate ECFs were generally higher, reflecting early-stage degradation; convergence with standard values can be expected over time. **Table I.6** provides all values obtained for both the adhesives and laminates.

For the EBR CFRP-to-concrete bond, an ECF of 0.75 is proposed for accelerated aging tests (AAT) when all degradation agents are considered collectively. Specific ECF of 0.70, 0.75, and 0.90 are suggested for water immersion, freeze-thaw cycles, and high-temperature exposure, respectively. Comparing AAT with natural aging tests (NAT), ECFs of 0.75 and 0.95 are recommended, respectively, indicating that AAT may overestimate degradation relative to actual outdoor conditions over a four-year period.

For the NSM CFRP-to-concrete bond, bond strength retention values from AAT reported in the literature show considerable variability, ranging from approximately 0.8 to 1.2, with limited agreement with NAT results from the present work. Based on maximum allowable 10 % of the non-conservative estimates from the AAT and NAT data, the ECF equal to 0.88 and 0.93 is suggested for the former and the latter,

respectively, indicating that AAT data generally overestimates bond degradation for exposure durations up to four years.

4.2. FUTURE DEVELOPMENTS

The author recommends that future works should focus on the following topics:

- Although the environmental exposures employed in this PhD thesis were carefully designed to promote specific degradation mechanisms, real outdoor conditions are inherently variable. Extending exposure durations would allow for a more accurate interpretation of the degradation trends. Future research should use advanced statistical methods and simulate combined degradation mechanisms to better understand the long-term performance of CFRP-strengthened RC structures under real environmental conditions.
- Given the prevalence of accelerated aging test (AAT) protocols in evaluating the durability of epoxy adhesives, CFRP laminates, and EBR/NSM CFRP-to-concrete bonds, future research should prioritize natural aging test (NAT) protocols. This shift is essential for establishing correlations between AAT and NAT results, enabling the derivation of reliable environmental conversion factors.
- The development of numerical and analytical models to assess the durability of the NSM CFRP–concrete bond—particularly in predicting the degradation of bond strength and stiffness under various degradation agents—is strongly recommended due to the current lack of literature.
- Research on the time-dependent deflection behavior of RC members strengthened with prestressed CFRP systems remains limited, and such systems are generally not addressed in current design codes. Future research should prioritize this area to help develop relevant robust predictive models, thereby supporting the advancement and revision of existing code provisions.
- The extrapolation of the ACI 318R-19 approach to derive time-dependent factor curves beyond the currently recommended 6-year period appears justifiable. Future research should aim to develop more representative curves supported by robust experimental data, enabling the establishment of generalized formulations. Such developments could significantly enhance design practices by addressing the synergistic effects inherent in real environmental exposures.

- The application of AEMM for predicting time-dependent deflections in CFRP-strengthened RC members aligns well with experimental results when a suitable bond-dependent coefficient is used in the modified tension stiffening model. Future research should focus on refining this coefficient for broader applicability across different member types and exposure conditions.

CHAPTER 5

APPENDED PAPERS

PAPER I

DURABILITY OF CFRP-TO-CONCRETE BOND CONSTITUENT MATERIALS: ACCELERATED *VERSUS* NATURAL AGING

REFERENCE: *Aloys Dushimimana, Anastasios P. Vassilopoulos, José Sena-Cruz, João M. Pereira, Luís Correia, Susana Cabral-Fonsesca, Ricardo Cruz. Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-controlled environments. Construction and Building Materials 438 (2024);<https://doi.org/10.1016/j.conbuildmat.2024.137201>*

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ABSTRACT

Carbon fiber reinforced polymer (CFRP) composites have shown desirable properties in aerospace, automotive, and construction industries. Focusing on the latter and in the context of strengthening existing structures, CFRP

composites are typically bonded to concrete structures using epoxy adhesive as a bonding agent. To date, there is a lack of literature on the durability of both the CFRP composite and epoxy adhesive. In this regard, the durability of two different commercially available CFRP laminates and epoxy adhesives was assessed after exposure to different indoor and outdoor environments for up to 4 years. Experimental, numerical, and analytical studies were conducted. In the latter, a new formulation was introduced to model water diffusion through the epoxy adhesive. The results from the experimental tests showed that water immersion substantially affected the two studied epoxy adhesives, resulting in a reduction of approximately 60-66 % and 70-75 % of their tensile strengths and elastic moduli, respectively. In contrast, the CFRP laminates improved both tensile strengths and elastic moduli in outdoor environments, with the highest increases being approximately 16 % and 10 %, respectively. Furthermore, both the developed numerical and analytical models for epoxy adhesives corroborated well with the experimental results. In particular, the former revealed that water diffusion through the adhesive was faster and reached its maximum in less than a year, and the proposed formulation in the latter led to a promising prediction.

KEYWORDS: Epoxy adhesive, CFRP laminate, Durability, Tensile strength, Elastic modulus, Numerical modeling, Analytical modeling.

1. INTRODUCTION

Fiber-reinforced polymer (FRP) composites have been widely adopted in aerospace, automotive, and construction industries. In the construction industry, FRP composites have been widely adopted, for example, in strengthening existing reinforced concrete (RC) structures, such as existing buildings, bridges, tunnels, tanks, and silos [1]. Different types of FRP composites have been adopted in construction, the most common being carbon, glass, aramid, and basalt. This study focuses on the former (i.e., carbon FRP, denoted as CFRP). The application of CFRP composites as a strengthening material has been significantly investigated (e.g., [2–5] [6]). When strengthening existing RC structures with CFRP composites, a bonding filler is required to bond the CFRP laminate strips to the concrete substrate. This bonding filler must have properties that lead to strong bonds between concrete and the filler, and between the filler and the CFRP laminate; additionally, the filler should have desirable properties such that failure through itself is limited. In fact, according to the existing literature, epoxy adhesives have been successfully used as bonding fillers [2,3][7], as their use leads to the formation of strong covalent bonds that tie them to the CFRP laminates, thus creating a hard-to-break bond between the two

materials [8]. In addition, epoxy adhesives form some covalent and hydrogen bonds when bonded to the concrete substrate [4].

To better understand the structural and sustainability performance of the applied strengthening system using CFRP laminate and epoxy adhesive, it is necessary to conduct research on the durability of these two materials. Some of the existing studies aimed at investigating the durability of these materials under accelerated aging conditions (e.g., [9,10]). In those studies, the CFRP laminate or epoxy adhesive was exposed to one or more degradation agents to degrade its properties under accelerated laboratory-controlled conditioning. With this type of conditioning, for example, the epoxy adhesive was observed to significantly decrease its properties after exposure to moisture [9,10] and after immersion in water [9,10]. Exposure of epoxy adhesive to wet-dry cycles also led to a reduction in both elastic modulus and tensile strength [11]. In contrast, post-curing phenomenon, which occurs when the cured resin is exposed (for an extended period) to temperatures at or above the curing temperature can improve the properties of the epoxy adhesive, e.g., [3,12]. Furthermore, exposure of the epoxy adhesive to high concentrations of CO₂ accelerated its curing rate [13], and exposure to chlorides did not significantly affect its properties [9].

Studies have also been conducted on the durability of CFRP laminates under accelerated aging conditions. It has been shown that the exposure of the CFRP laminate to various degradation agents does not significantly affect its properties. In fact, in a previous study, the CFRP laminate withstood exposure to water immersion for up to 2 years [10]. In addition, exposure of the CFRP laminate to thermal cycles in air did not affect its properties [14]. However, the CFRP tensile strength and elongation decreased after exposure to freeze-thaw cycles. Furthermore, the CFRP laminate suffered from microcracking of its fiber matrix due to exposure to moisture, and UV radiation affected a few microns of the CFRP laminate surface [15]. Overall, it can be noted that studies addressing the durability of either the epoxy adhesive or the CFRP laminate have generally been conducted under accelerated aging conditions, and a very few studies have been conducted under natural aging conditions, e.g., [3,16].

In addition, the existing literature generally lacks sufficient studies that address the long-term durability of epoxy adhesives using predictive models. In fact, there are several studies in which the adhesive has been modelled analytically and numerically to predict its behavior when exposed to degradation agents (e.g., [17–22]). However, in these studies, only short-term exposure (a period of less than one year) was investigated (except for [17]),

which is of concern because the application of epoxy adhesive as a bonding filler or CFRP laminate as a reinforcing material is generally expected to last for many years. Therefore, it is of paramount importance to evaluate the reliability of existing models that incorporate the effects of long-term exposure of the epoxy adhesive and CFRP laminate to different types of degradation agents, and if necessary, improve existing models or develop new ones. Another major concern is that the environmental conversion factors recommended in different standards (e.g., [23]), are entirely based on data from studies that have been generally conducted using accelerated aging protocols, as there is a lack of data from natural aging protocols. These protocols may not represent what normally occurs in outdoor environments, where the epoxy adhesive and CFRP laminate are normally used, thus creating a knowledge gap that needs to be addressed. Most importantly, studies that include specimens exposed to natural outdoor environments and comparative perspectives between the results of the existing accelerated and natural outdoor aging are critical in determining the most appropriate methods to use when predicting environmental conversion factors.

To address the above research gap, the durability of two different types of epoxy adhesives (denoted as ADH1 and ADH2) and two different types of CFRP laminates (denoted as L10 and L50) were investigated. Both adhesives and CFRP laminates were conditioned under the same conditions. Two different types of environmental exposure were studied: i) the first was accelerated aging, in which both the epoxy adhesives and the CFRP laminates were conditioned in tap water tanks controlled at 20 °C, and the others were conditioned in a climatic chamber at 20 °C and approximately 55 % relative humidity (RH); ii) the second was natural outdoor aging, in which four different geographical regions were selected based on their meteorological characteristics to promote degradation of materials properties due to their exposure to carbonation, freeze-thaw attack, elevated temperatures, and airborne chlorides. In all environments, the exposure lasted for up to 4 years. However, tests of aged specimens were conducted each year to assess the degree of degradation over time. These durability tests were performed to evaluate the effect of environmental exposure on the tensile strength and elastic modulus. Furthermore, analytical, and numerical models were developed to simulate the behavior of the epoxy adhesive specimens when continuously immersed in tap water. For the analytical model, an attempt was made to develop a new formulation capable of incorporating the effects of both physical and chemical aging of the epoxy adhesive, while for the numerical model, the well-known mass diffusion analysis was performed. Experimental results validated both the analytical and numerical models.

2. RESEARCH METHODS

2.1. Experimental program

2.1.1. Properties of epoxy adhesives and CFRP laminates

A total of 250 specimens of the epoxy adhesives were cast into dog-bone shapes, as shown in **Figure I.1a**, according to ISO 527-2. Two different types of adhesives were used (one labelled ADH1 and the other labelled ADH2). S&P® Clever Reinforcement Ibérica Lda. Company (Seixal, Portugal) produced the ADH1 adhesive, while SIKA Schweiz AG (Zurich, Switzerland) produced the ADH2 adhesive. These epoxy adhesives are commonly used to bond CFRP laminates to concrete. The relevant properties of both ADH1 and ADH2, as provided by the suppliers, are shown in **Table I.1**.

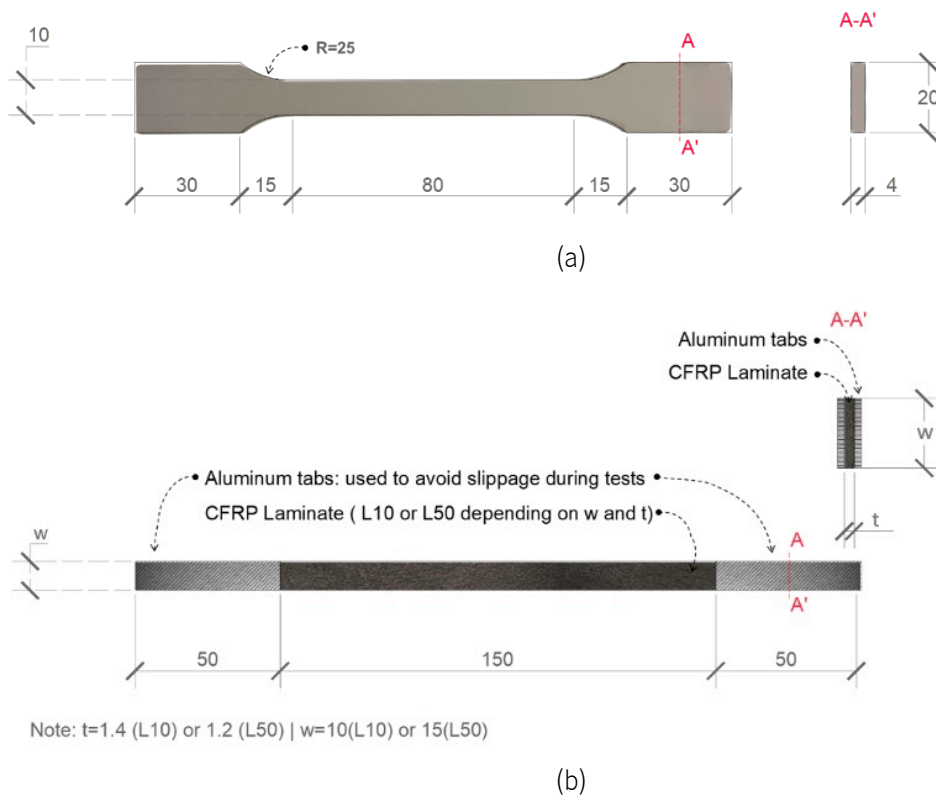


Figure I.1: Geometry of specimens: (a) epoxy adhesive and (b) CFRP laminate. Note: all dimensions in [mm].

In addition, 300 CFRP laminate specimens were prepared. Examples of L10 specimens are shown in **Figure I.2b**. These laminates (L10 and L50) were produced by the same company that produced the ADH1 adhesive,

and their properties according to the supplier's specifications are shown in **Table I.1**. It is worth noting that L10 and L50 are typically used in the reinforcement of concrete structures according to the Near-Surface Mounted (NSM) and Externally Bonded Reinforcement (EBR), respectively. For L50, specimens of 250 mm × 15 mm were cut from the original samples (see **Figure I.1b**).

Table I.1: Properties of the material under investigation (provided by the suppliers).

Property	Epoxy adhesive: ADH1	Epoxy adhesive: ADH2
Type of adhesive	Cold-curing S&P Resin 220	Cold-curing Sikadur-30
Flexural elastic modulus [GPa]	> 7.1	N/A
Tensile strength [MPa]	19.9 (after 7 days of curing at 20 °C)	26 (after 7 days of curing at +10 °C)
Glass transition temperature [°C]	46.2 (after 7 days of curing at 23 °C)	52 (after 30 days of curing at 30 °C)
Density, at 23 °C [g/cm ³]	1.7–1.8	1.65
Compressive strength [MPa]	> 70	> 75, 90 (after 7 days of curing at +10 °C)
Shear strength [MPa]	> 26	> 18
Bond strength by pull-off on concrete [MPa]	3 (after 3 days of curing at 20 °C)	> 4 (after 7 days of curing at 23 °C)
Property	CFRP laminate: L10	CFRP laminate: L50
Type and trademark	S&P clever (CFK 150/2000)	S&P clever (CFK 150/2000)
Prefabricated by	Pultrusion	Pultrusion
Fiber orientation	Unidirectional	Unidirectional
Fiber content [%]	68	N/A
Fiber matrix	Vinyl ester resin	Vinyl ester resin
External surface	Black, smooth	Black, smooth
Elastic modulus [GPa]	> 170	> 170
Tensile strength [MPa]	> 2000	> 2000
Dimensions [mm]	1.4 (thickness), 10 (width)	1.2 (thickness), 50 (width)

2.1.2. Environmental exposure and testing program

A total of 5 specimens for each adhesive and 6 specimens for each CFRP laminate were first tested at the initial time (T₀) to be considered as reference (i.e., before exposure). The remaining 240 epoxy adhesive specimens (**Figure I.2a**) and 192 CFRP laminate specimens (**Figure I.2b**) specimens (144 of L10 type and 48 of L50 type) were then exposed to six different environments. As previously mentioned, the first two environments were laboratory-based (one environment, denoted as E1, characterized by conditioning of the specimens around 20 °C/55 % RH, and the other, denoted as E2, characterized by immersing the specimens in tap water with a controlled temperature of 20 °C, see **Figure I.2c**). The remaining four environments were of outdoor exposure, where the stations were selected in different regions of Portugal in an attempt to promote aging due to carbonation (i.e., the environment denoted as E3), freeze–thaw attack (denoted as E4), elevated temperatures (denoted as E5), and airborne chloride attack (denoted as E6). An example of an outdoor environment (E4) is shown in **Figure I.2d**. Each year, 10 epoxy adhesive specimens (i.e., 5 ADH1 and 5 ADH2) and 8 CFRP

laminates (6 L10 and 2 L50) were collected from each of the aforementioned environments (E1-E6) and tested; however, prior to testing of the L50, 3 smaller specimens with dimensions as shown in **Figure I.1b** were cut from each of the L50 specimens (thus, 6 specimens were also tested for L50). Four different testing times, corresponding to four different exposure times, were considered. The tensile tests were performed at testing times of one year (denoted as T1), two years (T2), three years (T3), and four years (T4) of exposure. It is important to note that this is ongoing work that is planned to last for 10 years. The experimental results for the first 2 years of exposure have been published by the authors' group in [3].

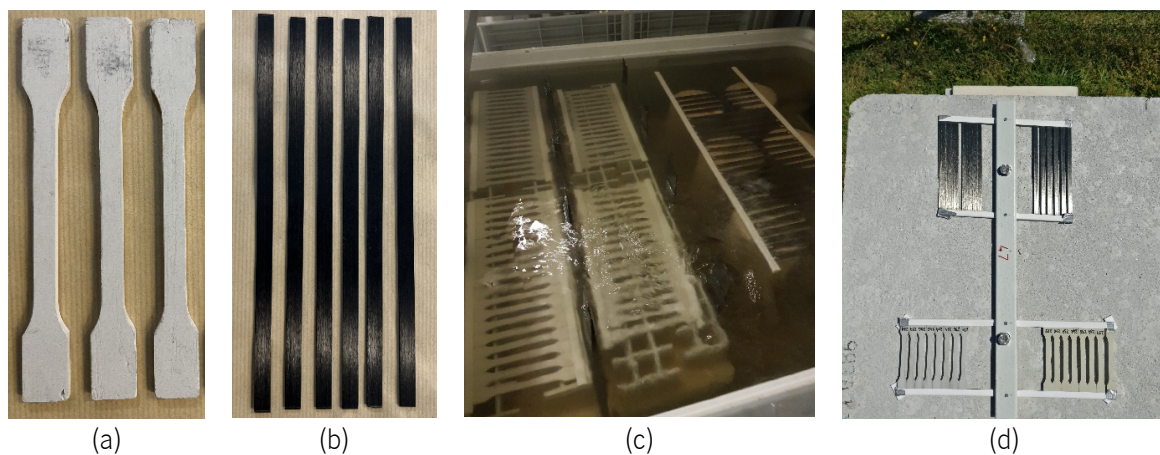


Figure I.2: Materials and examples of the environments studied: (a) epoxy adhesive specimens; (b) CFRP laminate specimens; (c) station E2 – tap water immersion conditioning; (d) station E4 – freeze-thaw outdoor environment.

Typical changes of temperature and relative humidity in the studied environments are shown in **Figure I.3**. More details about these environments can be found in [24] and all data about the studied environments can be found in [25,26]. Briefly, the main characteristics of the studied environments are as follows: E3 and E5 are generally characterized by peaks of high temperatures (and high levels of atmospheric CO₂ mostly in the case of E3), while E4 and E6 are generally characterized by high relative humidity and peaks of low temperatures.

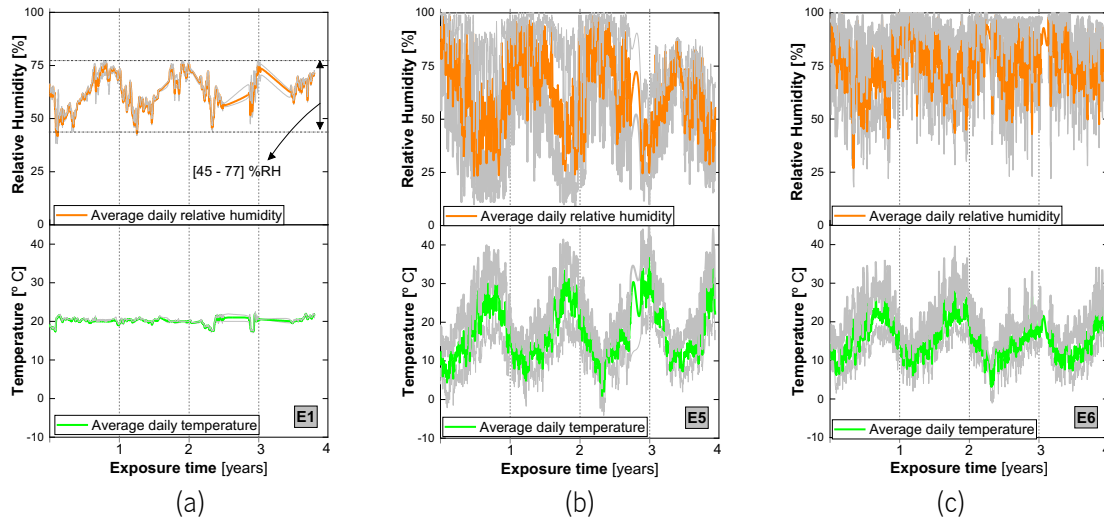


Figure I.3: Typical environments used in this study (variation of temperature and relative humidity) in: (a) E1, (b) E5, and (c) E6.

2.1.3. Tensile tests of epoxy adhesives and CFRP laminates

Tensile tests for both the epoxy adhesive and CFRP laminate specimens were performed using an MTS testing machine (model Exceed E45.104) with a maximum capacity of 100 kN. An example of the tensile test of the epoxy adhesive is shown in **Figure I.4a**, while that of the CFRP laminate is shown in **Figure I.4b**.

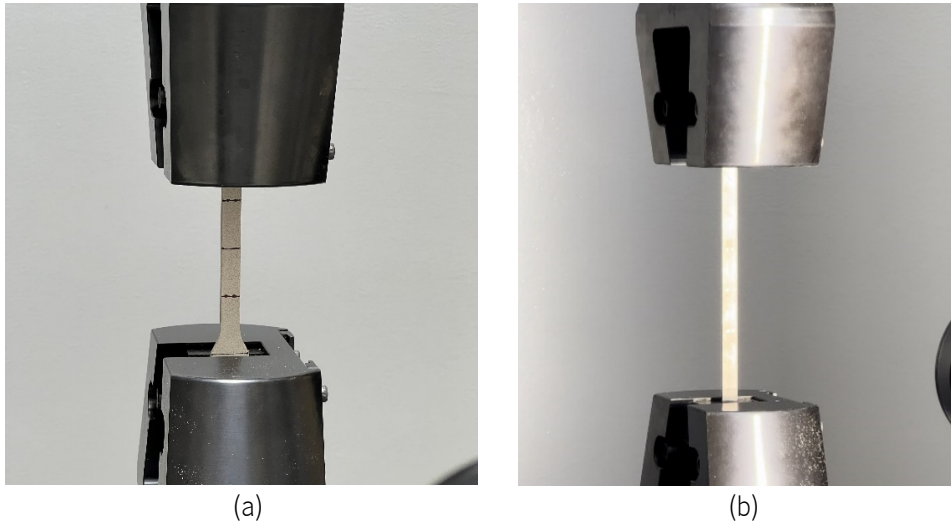


Figure I.4: Tensile tests: (a) typical epoxy adhesive testing, (b) typical CFRP laminate testing.

The testing of the epoxy adhesive followed the recommendations of EN ISO 527-2:2012 [27], and the elastic modulus was determined as recommended by the same standard, using the slope of the best fit line on the stress-strain curve between two points corresponding to 0.05 % and 0.25 % of the strains. Strains were

determined by dividing the extensometer-measured strains by the specimen length. Conversely, the CFRP laminates were tested according to ISO 527-5:2009 [28] using the same test configuration as that of the adhesives.

2.1.4. Mass uptake of epoxy adhesives

Water uptake of both ADH1 and ADH2 in the fully immersion state was measured over a period of up to 10752 hours (448 days). Both adhesives were continuously immersed in tap water at different temperatures (at 20, 40, and 60 °C) to study the effects of water diffusion over time and to assess the effects of temperature on the diffusion process. Since conditioning at higher temperatures within a shorter time can help predict diffusion at longer time, e.g., [29,30], the diffusion trends from epoxies conditioned at 40 and 60 °C can help better understand longer-term diffusion process in E2, i.e. after 4 years of exposure. The interval between measurements was short for the first few weeks and then relaxed in the later weeks. Due to the fact that both ADH1 and ADH2 can be modelled using similar analytical and numerical techniques, one of these adhesives (ADH1) was selected and modelled, as will be seen in Section 4 of the present work.

2.2. Analytical modeling

In this work, analytical modeling of water diffusion through an epoxy adhesive (conditioned at 20, 40, and 60 °C) was performed to help predict the experimental results obtained in **Section 2.1.4**. To predict the time-dependent mass uptake of the adhesive, **Eq. (I.1)** is generally used (e.g., [17]). **Eq. (I.1)** has its simplified approximate closed-form solution, as shown in **Eq. (I.2)**, which has been adopted in existing studies (e.g., [31] [32]), and was therefore adopted in the present work. It is worth noting that these equations are Fick's second law (also known as the general diffusion law) and are assumed for a one-dimensional diffusion process [32]. Furthermore, it is assumed that diffusion occurs through isotropic substances [33], and its mathematical theory is based on the hypothesis that the rate of transfer of the diffusing substance through a unit area of a section is proportional to the concentration gradient measured perpendicular to the section [33].

$$\frac{M(t)}{M_{\infty}} = 1 - \frac{8}{\pi^2} \sum_{n=0}^{\infty} \frac{1}{(2n+1)^2} \exp \left[-\frac{D(2n+1)^2 \pi^2 t}{4h^2} \right] \quad (\text{I.1})$$

$$\frac{M(t)}{M_{\infty}} \approx 1 - \exp \left[-7.3 \left(\frac{Dt}{4h^2} \right)^{\frac{3}{4}} \right] \quad (\text{I.2})$$

$$D = \frac{\pi}{16} \left(\frac{h}{M_{\infty}} \right)^2 \left(\frac{M_{t_1} - M_{t_2}}{\sqrt{t_1} - \sqrt{t_2}} \right)^2 \quad (I.3)$$

where, $M(t)$ is the total mass uptake at a given time t , M_{∞} is the mass uptake at infinite time, h is the thickness of the diffusing material, D is the diffusion coefficient defined by Eq. (I.3) and is given by the initial slope when weight gain $M(t)$ is plotted against the square root of time, M_{t_1} and M_{t_2} are the mass uptakes at given times t_1 and t_2 , respectively.

In the existing literature, many studies ([31,34–36]) have shown that most polymers exhibit non-Fickian behavior. Specifically, water diffusion through an epoxy adhesive cannot be fully defined by the above Fickian-based equations [19,31], as these equations solely address the physical aging behavior of the adhesive (i.e., reversible mechanisms such as plasticization and swelling, leading to a reduction in modulus of elasticity, strength and increased ductility [37,38]), whereas water diffusion through polymers and adhesives is known to affect not only the physical, but also the chemical and mechanical properties [30]. Chemical property changes are mostly irreversible and occur usually after long-term exposure to moisture [3], leading to degradation of material constituents (fiber, matrix and matrix-fiber interface [39]). Mechanical property changes can result from the combined effects of physical and chemical property changes, where swelling can lead to microcracking (due to weakened resin after hydrolysis) and debonding effects at the interface [40].

Chemical aging can also be speeded up by exposing the adhesive to high temperatures that accelerate the chemical reactions, resulting in faster moisture ingress than at low temperatures [30]. As mentioned above, the Fickian-based equations cannot capture the effects of physical and chemical aging simultaneously; therefore, in the present work, an equation capable of including the chemical aging effect is combined with the existing Fickian-based Eq. (I.2) to fully capture both the physical and chemical aging phenomena as follows:

$$\frac{M(t)}{M_{\infty}} = \begin{cases} 1 - \exp \left[-7.3 \left(\frac{Dt}{4h^2} \right)^{3/4} \right], & t < t_{max} \\ M_{max} \left(\frac{t_{max}}{t} \right)^{1/15}, & t \geq t_{max} \end{cases} \quad (I.4)$$

where M_{max} is the maximum mass uptake and t_{max} is the time at which the M_{max} is reached. The other variables are as previously defined in Eqs. (I.1)- (I.3).

According to existing literature, chemical aging is an irreversible change of chemical decomposition and molecular structure [41] and occurs after long-term exposure to moisture [3] with three different degradation mechanisms: thermal, thermo-oxidative, and hydrolytic [29,41]. Thermal degradation (found to be the most

degradation mechanism by [29]) is due to chain-scission reactions resulting from strong molecular oscillations at high temperatures and/or additional cross-linking; thermo-oxidative degradation results from chain-scission by oxidation [42]; and, hydrolytic degradation results from chain reduction due to water diffusion that increases the weight as a result of water embedment in the polymer [43]. The dominant mechanisms among the above three are governed by the resin type, environment, and temperature [29]. For the case addressed in this work where the epoxy adhesive is continuously immersed in water at different temperatures, both thermal and hydrolytic degradations can dominate. In the frequent cases, an increase in the cross-linking density changes the material densification and increases the $\bar{T}_g$, thereafter influencing the mechanical properties [41].

At a certain stage of diffusion process, the above material densification from cross-linking can lead to pushing out the diffused water hence paving the way to generation of negative slope in the water uptake *versus* time relationship. In this context, in **Eq. (I.4)**, it is assumed that the chemical aging effect starts at the point when the maximum mass uptake (M_{max}) is reached (i.e., the start of irreversible mechanisms resulting from material densification [41]). Therefore, the first part of **Eq. (I.4)** is exactly the same as in **Eq. (I.2)** and is used to define the region of the mass uptake *versus* time curve before the M_{max} is reached. Furthermore, the second part of **Eq. (I.4)** is a newly proposed equation to include the effects of chemical aging, which is assumed to occur after M_{max} is reached, i.e., at the start of material densification [41].

The proposed equation is a power function (to include the effects of chemical aging effects due to irreversible mechanisms) developed such that its softening curve is mainly governed by the power divisor; that is, the slope of the curve decreases as the divisor increases. It is worth noting that the power divisor was set through curve fitting to experimental data, and does not have phenomenological meaning, hence it may only be relevant to the current work. However, with the knowledge that the power divisor controls the negative slope just after reaching the maximum water uptake, the equation can be applied for other future works but with some adjustment of the divisor to match the slope obtained from experimental data. However, it is believed that such adjustment can be minor owing to the fact that the divisor varied irrespective with temperature in this work. The results from **Eq. (I.4)** were compared with those of the Fickian-based equations, i.e., **Eq. (I.2)**, to obtain a clear understanding of the effectiveness of the proposed **Eq. (I.4)**. Furthermore, the results obtained from the above analytical model were compared with those obtained from the experimental program previously described.

2.3. Finite element modeling

In the existing literature, finite element models have been developed to model the behavior of composite joints using ABAQUS software [18,19]. In some studies, e.g., [18], the moisture diffusion problem is converted into a heat-flux problem in order to obtain the desired results from the software. In fact, the mass diffusion analysis available in the software cannot output the average moisture concentration required for comparison with experimental results, unless post-processing of the data is performed. In this work, mass diffusion analysis was performed on the ADH1 epoxy adhesive (conditioned at 20 °C water immersion) to investigate water diffusion over time. This is exactly the same type of conditioning used during an experimental campaign (see E2) to allow diffusion predictions for the case of specimens in E2. A Python code (see ANNEX I.1) was developed to extract the average normalized concentration. The results from the performed simulations were then compared with those obtained from the experimental program. Detailed information on the modeling procedure is described as follows.

The dimensions and geometry of the epoxy adhesive are the same as previously shown in **Figure I.1a** for comparison purposes. This adhesive was modelled as a 3D deformable solid (**Figure I.5**). The properties of the epoxy adhesive are listed in **Table I.1**. The mass diffusion analysis step was used, in which the simulation time was set equal to the water immersion time used during the experimental program (i.e., 10752 hours). A transient response was assumed to allow the normalized concentration to change with time. The incrementation type was automatically applied. The initial, minimum, and maximum increment sizes were set to 1000, 500, and 10 000 s, respectively.

Boundary conditions were first created on the largest faces of the epoxy adhesive to allow water penetration in one direction through the thickness of the entire adhesive (denoted as Model 1D), while in the second model (denoted as Model 3D), the boundary conditions were set to allow water penetration in all directions. After performing a mesh sensitivity analysis, meshing with approximate global size of 4 mm, curvature control with maximum deviation factor of 0.1, and minimum size control by fraction of global size set to 0.1 were used (**Figure I.5**). Hex and sweep were selected as element shape and technique, respectively; and advanced front algorithm with mapped meshing applied where appropriate was selected. An 8-node linear heat-transfer brick element (DC3D8) was selected as the element type. Furthermore, a temperature of 20 °C was applied as a predefined field at the initial step and propagated to the mass diffusion step with instantaneous amplitude. In addition, the absolute zero temperature and the Stefan-Boltzmann constant of -273.15 K and 5.66e-08,

respectively, were applied using edit attributes. The other parameters used in both models are listed in **Table I.2**. It is worth noting that the diffusivity was obtained from **Eq. (I.3)**.

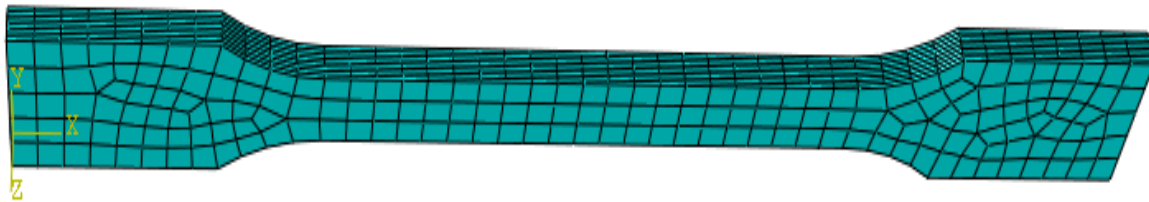


Figure I.5: Finite element model of the experimentally tested epoxy adhesive specimen.

Table I.2. Parameters used for numerical modeling of the epoxy adhesive.

Model	Diffusivity [mm ² /s]	Solubility	Density [kg/mm ³]	Material type	Law	Time [s]
1D, 3D	1.8433e-7	1.0	1.9e-9	isotropic	Fick	3.87e7

Notes: 1D: numerical modeling of water diffusion through epoxy adhesive in one direction (x), 3D: modeling of water diffusion through epoxy adhesive in 3 directions (x,y,z), Fick: Fickian diffusion model.

3. RESULTS AND DISCUSSION

3.1. Experimental results

3.1.1. Tensile loading of epoxy adhesives

The two different epoxy adhesives (ADH1 and ADH2) were examined. Firstly, the results from the glass transition temperature ($\bar{T}_g$) previously published by the authors' group in [3] are shown in **Figure I.6** and are used to help analyze the trends of other properties investigated in this work. It can be noted that the $\bar{T}_g$ generally increased after exposure, which can be attributed to the post-curing phenomenon according to [3].

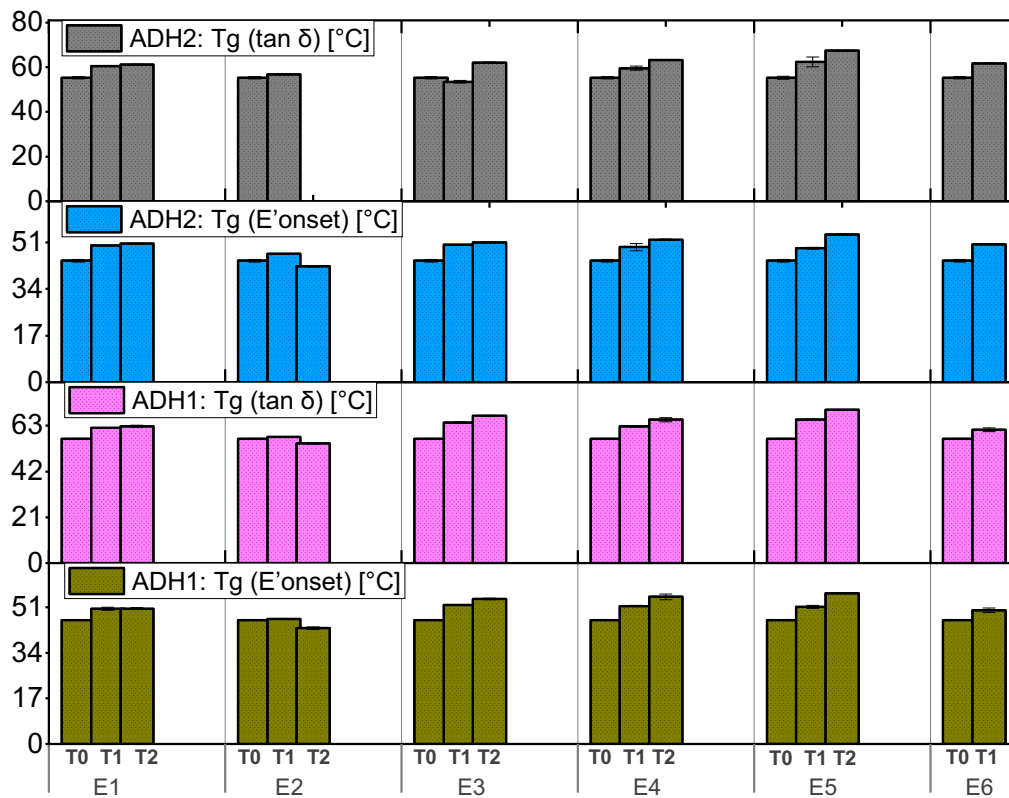


Figure I.6: Variation of glass transition temperature (T_g) of adhesive ADH1 and ADH2 after exposure to different environments [3].

3.1.1.1. Epoxy adhesive ADH1

The stress-strain relationships of ADH1 before and after exposure in environments E1 to E6 are provided in **Figure I.7a** to **Figure I.7f**, and its maximum mechanical properties in **Figure I.8** and **Table I.3**. In addition, stress-strain curves for all tested ADH1 specimens from different environments (E1-E6) for up to 4 years (T0-T4) are shown in **ANNEX I.2**. The degradation of the tensile strength (f_t) and elastic modulus (E_s) is generally evident for most of the environments, except for some increases observed in the first year for some specimens in the outdoor environments. The highest degradation of both f_t and E_s was found for the specimens immersed in tap water (i.e., environment E2), with retentions of approximately 0.34 and 0.26, corresponding to reductions of 66 % and 74 %, respectively (**Figure I.9a,b,c**). This also shows that there was a difference of about 10 % between the degradation of f_t and E_s , indicating that the latter degraded faster than the former. The stiffness also decreased substantially after just one year of exposure (**Figure I.7b**). It is worth noting that the specimens from E2 were tested just after being removed from water to avoid water loss that could significantly affect the results due to the reversibility of the physical mechanisms of the epoxy adhesive [3]. The observed substantial decrease

in E2 can be attributed to the plasticization effect, which is already known to significantly degrade the properties of the epoxy adhesive [9].

Further analysis shows that during the first year of exposure (T1), both f_s and E_s increased slightly in outdoor environments, except in E2 (water exposure) and E6 (exposure to airborne chlorides from the ocean), with the highest increase being observed in E5 (exposure to high temperatures) with approximately 1.10 and 1.15 retentions (**Figure I.9a,b,c**), respectively. This increase in E5 is attributed to the presence of elevated temperatures in this region, which led to a post-curing phenomenon (see **Figure I.6**), thereby improving the adhesive properties. In fact, existing literature shows that post-curing can improve the adhesive properties, e.g., in [12]. In the last years of exposure, the effects of degradation agents dominated those of post-curing, resulting in a reduction of both the f_s and E_s properties.

The outdoor environments E4 and E6 showed the largest decreases, which can be attributed to the effects of freeze-thaw in the former and high humidity in the latter. This observation is consistent with that of [9], where the presence of moisture significantly degraded the epoxy adhesive properties. Besides, water immersion increased the strain (**Figure I.7b**, **Table I.3** and **Figure I.9c**) during the first year with minor changes in the later years. Overall, the properties of the ADH1 adhesive generally showed a decrease in both its tensile strength and elastic modulus after exposure to different environments, and this decrease was generally dependent on the time and type of exposure. The decrease was substantial in the case of water immersion during the first year of exposure. Considering the observed results at the fourth year of exposure, a retention of 0.8 can be suggested for the case of outdoor exposure.

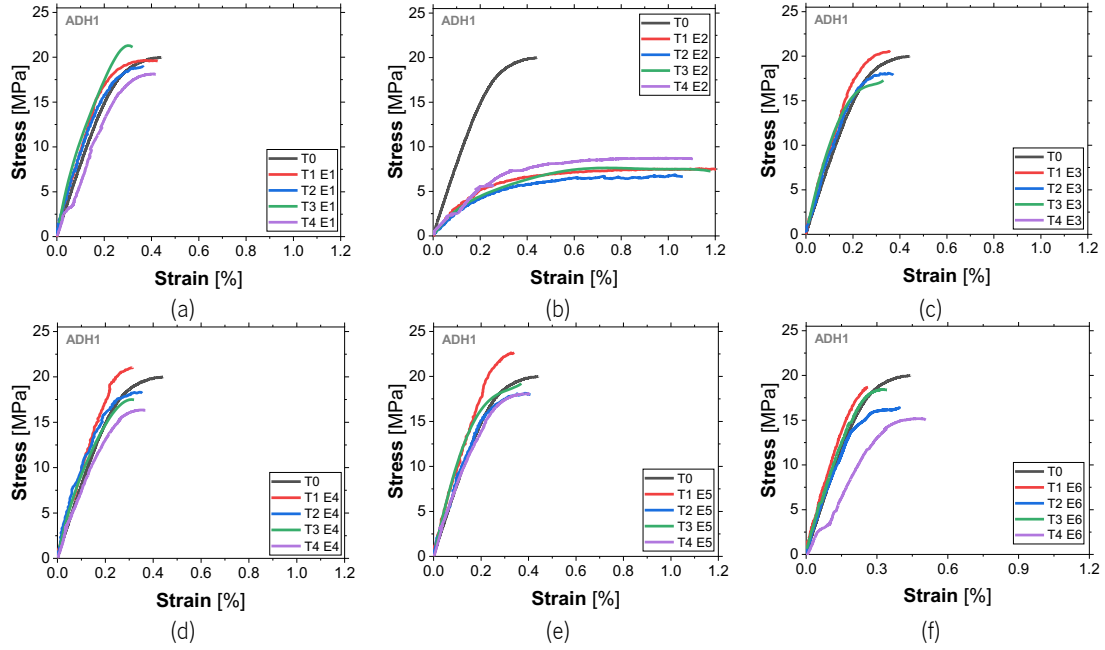


Figure I.7: Evolution of the stress-strain relationships with time and environmental exposure for ADH1: (a) to (f) correspond to data results from E1 to E6, respectively.

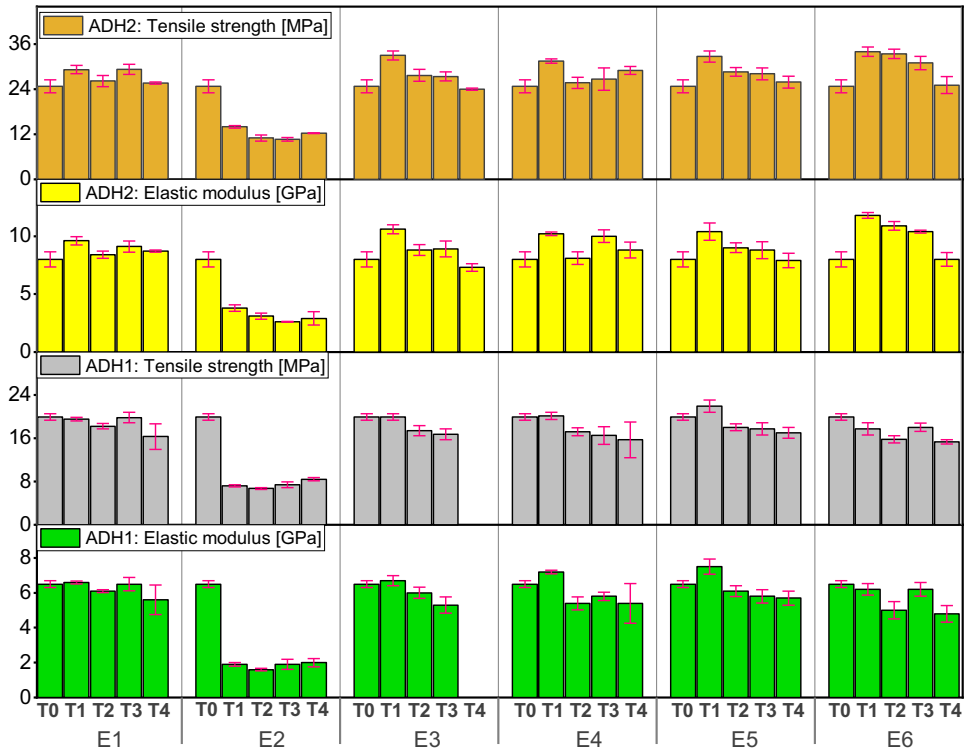


Figure I.8: Variation of the maximum mechanical properties of ADH1 and ADH2 with time and environmental exposure.

DURABILITY OF CFRP-TO-CONCRETE BOND CONSTITUENT MATERIALS:
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Table I.3: Average values of tensile strength, elastic modulus, and tensile strain of epoxy adhesives after 0 (T0), 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of various environmental exposures (E1 to E6).

Environment	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
	ADH1: Tensile strength [MPa]					ADH2: Tensile strength [MPa]				
REF	19.9 (3.0)					24.8 (7.0)				
E1	-	19.5 (1.8)	18.2 (2.8)	19.8 (4.9)	16.3 (14.6)	-	29.2 (3.8)	26.2 (5.7)	29.3 (4.6)	25.6 (10.7)
E2	-	7.2 (3.1)	6.7 (2.7)	7.4 (7.1)	8.4 (3.6)	-	14.0 (2.5)	11.0 (7.6)	10.7 (4.5)	12.3 (10.6)
E3	-	19.9 (3.1)	17.4 (5.3)	16.7 (5.9)	-	-	33.0 (3.6)	27.7 (5.8)	27.4 (4.3)	24.0 (12.5)
E4	-	20.1 (3.4)	17.2 (4.3)	16.5 (9.8)	15.7 (20.1)	-	31.5 (1.8)	25.7 (5.7)	26.7 (11.0)	29.0 (3.6)
E5	-	21.9 (5.2)	18.0 (3.6)	17.7 (6.5)	17.0 (5.8)	-	32.7 (4.6)	28.6 (4.0)	28.1 (5.6)	25.9 (6.1)
E6	-	17.7 (6.4)	15.8 (4.3)	18.0 (4.2)	15.3 (2.6)	-	34.0 (3.8)	33.4 (3.8)	31.0 (5.7)	25.1 (9.1)
	ADH1: Elastic modulus [GPa]					ADH2: Elastic modulus [GPa]				
REF	6.5 (3.0)					8.0 (8.2)				
E1	-	6.6 (1.3)	6.1 (1.4)	6.5 (6.0)	5.6 (14.6)	-	9.6 (3.7)	8.4 (3.6)	9.1 (5.3)	8.7 (10.5)
E2	-	1.9 (5.1)	1.6 (4.0)	1.9 (14.6)	2.0 (12.4)	-	3.8 (7.4)	3.1 (8.8)	2.6 (11.5)	2.9 (20.0)
E3	-	6.7 (4.4)	6.0 (5.4)	5.3 (8.9)	-	-	10.6 (3.7)	8.8 (5.3)	8.9 (7.6)	7.3 (4.4)
E4	-	7.2 (1.4)	5.4 (6.9)	5.8 (4.0)	5.4 (21.1)	-	10.2 (1.6)	8.1 (6.6)	10.0 (5.3)	8.8 (7.8)
E5	-	7.5 (5.7)	6.1 (5.0)	5.8 (6.5)	5.7 (7.0)	-	10.4 (7.2)	9.0 (4.7)	8.8 (8.3)	7.9 (7.7)
E6	-	6.2 (5.4)	5.0 (10.0)	6.2 (6.2)	4.8 (10.0)	-	11.8 (2.1)	10.9 (3.4)	10.4 (1.2)	8.0 (7.4)
	ADH1: Tensile strain [%]					ADH2: Tensile strain [%]				
REF	0.4 (6.2)					0.4 (20.0)				
E1	-	0.4 (13.0)	0.3 (11.7)	0.3 (8.8)	0.4 (11.3)		0.3 (18.6)	0.4 (18.2)	0.4 (16.0)	0.38 (8.1)
E2	-	1.1 (21.4)	1.1 (11.9)	1.0 (25.5)	0.9 (15.3)	-	0.3 (10.8)	0.3 (11.7)	0.2 (11.4)	0.21 (17.3)
E3	-	0.3 (11.1)	0.3 (19.1)	0.3 (10.1)	-	-	0.4 (14.7)	0.3 (8.7)	0.3 (13.2)	0.34 (14.2)
E4	-	0.3 (11.3)	0.3 (12.8)	0.3 (24.6)	0.3 (24.8)	-	0.4 (9.1)	0.3 (15.8)	0.3 (23.0)	0.37 (9.9)
E5	-	0.3 (11.2)	0.4 (13.1)	0.3 (11.5)	0.4 (10.5)	-	0.4 (10.3)	0.4 (14.4)	0.3 (7.6)	0.31 (25.4)
E6	-	0.3 (4.3)	0.3 (12.9)	0.3 (17.4)	0.4 (14.2)	-	0.3 (11.3)	0.3 (3.5)	0.3 (13.1)	0.34 (27.6)

Notes: all values in parentheses express *the coefficient of variation* in percentage; ADH1: S&P, ADH2: Sika, REF: Reference values from the specimens tested at the beginning, i.e., at T0

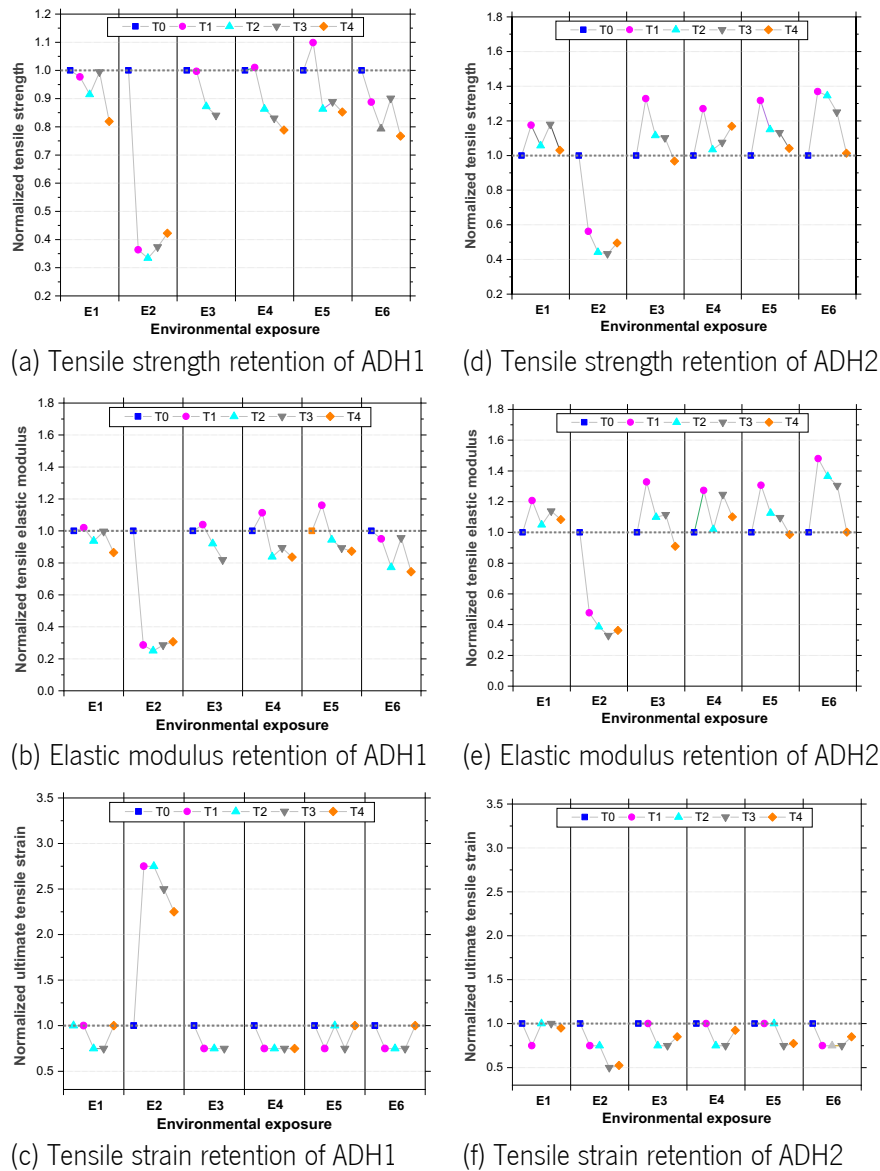


Figure I.9: Epoxy adhesive test results from different environmental exposures (E1-E6) for up to 4 years (T0-T4).

3.1.1.2. Epoxy adhesive ADH2

The stress-strain relationships of ADH2 before and after exposure in environments E1 to E6 are provided in **Figure I.10a** to **Figure I.10f**, respectively, and its maximum mechanical properties in **Figure I.8** and **Table I.3**. In addition, stress *versus* strain curves for all tested ADH2 specimens from different environments (E1-E6) for up to 4 years (T0-T4) are shown in **ANNEX I.3**. Like ADH1, ADH2 also showed a substantial decrease in both its tensile strength (f_t) and elastic modulus (E_t) after being immersed in tap water (E2) during the first year.

However, contrary to what is observed for ADH1, ADH2 initial stiffness in E2 (**Figure I.10b**) did not change significantly, while larger changes in the stiffness in other environments from time to time are evident as compared to the case of ADH1, which also showcases the differences in these two epoxy adhesives. As previously explained for ADH1, the effects of water diffusion through ADH2 led to plasticization, thereby causing both f_a and E_a to decrease substantially. After 3 years of exposure, f_a and E_a decreased by approximately 60 % and 70 %, respectively. Like what is observed for ADH1, there was a 10 % difference in the degradation of the f_a and E_a , hence indicating that the degradation of the latter was faster than that of the former. With the exception of the samples immersed in tap water (E2), the specimens in the other environments showed a significant increase in the first year (T1) of exposure, due to the higher post-curing rate (as compared to that of the ADH1, see **Figure I.6**) that is thought to have dominated the negative effects of the degradation agents during T1.

The outdoor environments showed the most significant improvements; as compared to indoor environments; in particular, the E3 and E5 environments showed approximately 1.39 and 1.40 f_a and 1.38 and 1.50 E_a retention (**Figure I.9d,e,f**), respectively. The substantial post-curing effect observed in environments E3 and E5 is believed to have been facilitated primarily by the presence of elevated temperatures in both environments, referring to **Figure I.6**, and existing studies [3,12] where the exposure of epoxy adhesive to high temperatures led to increased T_g , which in turn improved the adhesive properties. In addition, a previous study by [13] showed that carbonation can also contribute to increasing the curing rate of the resin matrix; therefore, the inevitable presence of the CO_2 in both environments may have catalyzed the post-cure phenomenon. After one year of exposure, the post-cure rate began to decrease; hence, the effects of the degradation agents began to dominate those of the post-cure, which led to a decreasing phase in successive years in the E3, E5, and E6 environments. However, there were some fluctuations in the E4 environment, where the ADH2 tensile strength f_a decreased after one year (T1) of exposure, then showed an increase again in the later years of exposure. There was also an increase in the elastic modulus during T1, followed by a decrease during T2, and again an increase during T3, followed by a decrease during T4. This may show the effect of exposures dissimilarity from year to year, resulting from the effects of exposure duration (e.g., difference in the number of months of freezing/thawing from one year to another) and exposure severity (i.e., one year may experience severe freezing/thawing compared to the previous or the next one). This is in agreement with [44]. On the other hand, the variation in tensile strains (**Table I.3**) did not change significantly with time. Overall, water diffusion significantly degraded

the properties of the ADH2 adhesive. However, the post-curing phenomenon of the ADH2 greatly increased both the tensile strength and the elastic modulus properties (except E2) during the first year, followed by a progressive decrease in the properties in the later years, although this decrease was still not significant enough to cause the properties to fall below those observed before the exposure (i.e., at T0).

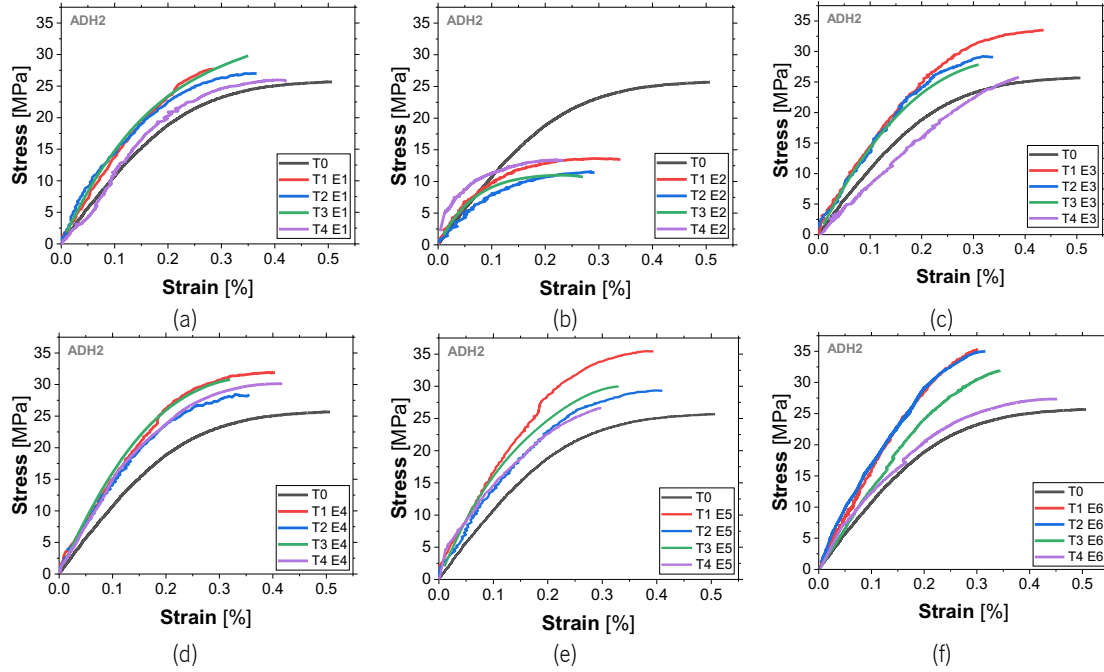


Figure I.10: Evolution of the stress-strain relationships with time and environmental exposure for ADH2: (a) to (f) correspond to data results from E1 to E6, respectively.

Comparing ADH1 and ADH2, it can be noted that the post-curing in the latter contributed significantly to improving its tensile strength and elastic modulus, whereas that in the former was minimal with insignificant post-curing effects. Therefore, environmental degradation agents attacked ADH1 much faster than ADH2. That is, after one year (T1), the degradation agents had already dominated the post-curing effects, and thus the properties began to fall below those originally recorded at the beginning (i.e., before exposure T0). In contrast, the degradation agents were not able to lower the properties of ADH2 because of the substantial post-curing that occurred during the first year, as a result, the ADH2 properties were still higher than those recorded at the beginning (T0) even after 4 years of exposure. Overall, although ADH1 and ADH2 are epoxy adhesives with the same purpose, i.e., structural bonding of CFRP laminates to concrete, they exhibit significantly different durability performances.

3.1.2. Tensile loading of CFRP laminates

Carbon fiber reinforced polymer (CFRP) laminates with different thicknesses and sizes (L10 and L50) were also tested in tension. The tensile strength (f_t), elastic modulus (E_t), and ultimate tensile strain (ϵ_t) from these tests are examined. The test results for each laminate type are as follows.

3.1.2.1. CFRP laminate L10

The tensile strength (f_t), elastic modulus (E_t), and ultimate tensile strain (ϵ_t) obtained from testing of the L10 CFRP laminate are presented in **Figure I.11** and maximum values in **Table I.4**. The trends for retentions of the mechanical properties are presented in **Figure I.12a,b,c**. In addition, stress-strain curves for all tested L10 specimens from different environments (E1-E6) for up to 4 years (T0-T4) are shown in **ANNEX I.4**, where a linear elastic behavior in each environment is observed for all testing. Looking at these results, the properties of L10 did not decrease with time for all the studied environments for up to 4 years of exposure; instead, there was a remarkable increase in the properties during the first year of exposure (T1). This increase in f_t and E_t can be attributed to the post-curing of the polymeric matrix joining fibers (referring to ADH1 in **Figure I.6**, as both ADH1 and L10 possess a similar polymeric matrix). In general, the retention values (**Figure I.12a,b,c**) in all environments ranged between 1.0–1.15 and 1.0–1.10 for f_t and E_t , respectively, over the 4 years of exposure.

Further analysis of the results from each environment shows that the specimens in E1 had a significant post-curing of the polymeric matrix during the first year (T1), which led to improved f_t and E_t properties, followed by a progressive decrease in the properties in subsequent years; however, despite this decrease, both f_t and E_t were still higher than the values recorded at the initial time before exposure (i.e., T0) because of the post-curing effect during T1. The specimens in E2 generally showed fluctuations from year to year. An increase in f_t and E_t properties was observed in the first year, followed by a fluctuating decrease and increase in subsequent years for up to 4 years of exposure. The observed increase in the properties can be attributed to the effects of post-curing, which improved the performance of the polymeric matrix (as evidenced by ADH1 $\bar{\gamma}_g$ growth in **Figure I.6**). Furthermore, the observed fluctuations in the following years may indicate the competing effects between the post-curing and plasticization mechanisms of the polymeric matrix. A similar observation was also reported in [3]. Nevertheless, despite the observed fluctuations, the f_t and E_t properties after 4 years were still higher than those at T0, thus showing the ability of the CFRP laminate to withstand long-term water immersion.

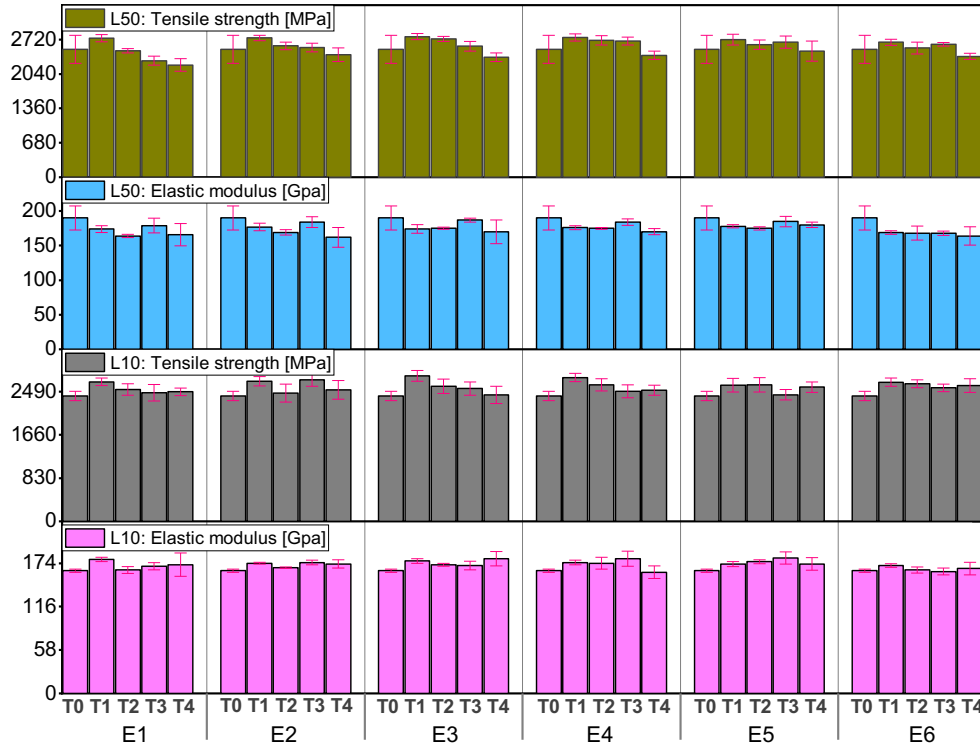


Figure I.11: CFRP L10 and L50 tensile strength and elastic modulus variations with time and environmental exposure.

Regarding the specimens in E3, during the first year of exposure, the specimens had the highest post-curing of the polymeric matrix (evidenced by ADH1 T_g growth in **Figure I.6**), which can be attributed to the effects of elevated temperatures, already known to cause post-cure of the resin [12]; additionally, the improvement may also be linked to high carbonation found to promote the curing of the resin [13], although this might not have a significant effect due to the fact that effect of compressed CO_2 (used in [13]) may differ from that of the atmospheric CO_2 . In the later years, the f_t and E_t properties started to decrease with time; however, after 4 years of exposure, the properties were still higher than those at T0.

The rate of post-curing in E4 was the second highest during T1 compared to the other environments, which significantly improved both the f_t and E_t properties; however, the competing mechanisms between the post-curing and the freeze-thaw attack resulted in a progressive decrease of the properties over time. In fact, the negative effects of the freeze-thaw attack dominated the positive effects from the post-curing, which may have led to the formation of polymeric matrix microcracks, thereby decreasing the f_t and E_t properties. This agrees with the findings in [45], where the formation of polymeric matrix microcracks led to a reduction in the CFRP properties as a result of exposure to freeze-thaw cycles.

For the specimens in E5, the combination of elevated temperatures and the presence of carbonation substantially improved the properties of the L10 laminate. The E_t continued to increase until the third year, where it showed a slight decrease in the 4th year, although the final value was still much higher than that at the beginning (T0). In addition, the f_t showed a remarkable improvement compared to other environments. This leads to inferring that the synergistic effect between the elevated temperatures and carbonation may have led to improved CFRP laminate properties over time.

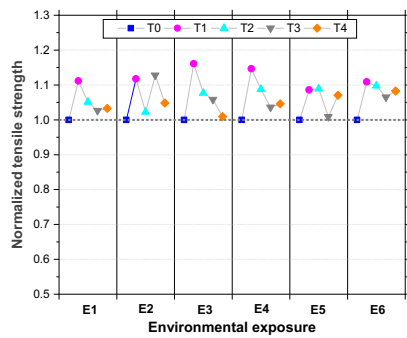
In E6, the tensile strength of L10 in this environment was significantly improved compared to the elastic modulus. This shows that the presence of airborne chlorides may have a significant effect on the strength properties but minimal effect on the elastic modulus. On the other hand, the variation of tensile strain (**Figure I.12c** and **Table I.4**) showed significant variations with time, with retentions ranging between 1.0 and 1.1. Overall, the CFRP laminate L10 generally showed improvements in its properties with time, indicating that the negative effects of various environmental exposures were minimal. This is mainly because of the post-curing phenomenon, which delayed the effects of degradation agents from different environments for up to 4 years. However, there is a tendency for the properties to keep decreasing in the next few years due to the minimized post-curing rate and the dominance of the degradation agent's attack.

Table I.4: Average values of tensile strength, elastic modulus, and ultimate tensile strain of CFRP laminates after 0 (T0), 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of various environmental exposures (E1 to E6).

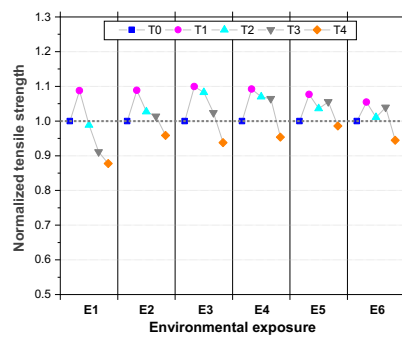
Environment	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
	L10: Tensile strength [MPa]					L50: Tensile strength [MPa]				
REF	2405 (3.8)					2527 (11)				
E1	-	2674 (2.72)	2528 (4.4)	2469 (6.4)	2484 (3.0)	-	2748 (2.6)	2497 (1.7)	2302 (3.9)	2217 (5.7)
E2	-	2688 (3.4)	2460 (7.1)	2713 (4.5)	2522 (7.1)	-	2750 (2.0)	2594 (2.8)	2562 (3.2)	2422 (5.6)
E3	-	2792 (3.7)	2590 (5.4)	2546 (5.1)	2427 (6.8)	-	2778 (2.1)	2735 (1.8)	2587 (3.6)	2369 (3.6)
E4	-	2757 (2.9)	2617 (4.5)	2492 (5.0)	2516 (3.9)	-	2760 (2.5)	2703 (3.4)	2690 (2.9)	2409 (3.5)
E5	-	2611 (5.0)	2619 (5.3)	2427 (4.1)	2575 (3.9)	-	2720 (3.9)	2618 (3.6)	2667 (4.6)	2491 (8.1)
E6	-	2667 (3.0)	2640 (2.9)	2561 (2.8)	2605 (5.1)	-	2665 (2.2)	2554 (4.6)	2626 (1.3)	2386 (2.6)
	L10: Tensile elastic modulus [GPa]					L50: Tensile elastic modulus [GPa]				
REF	164 (1.2)					190 (9.3)				
E1	-	179 (1.6)	165 (2.7)	170 (2.7)	172 (9.1)	-	174 (2.8)	164 (1.3)	179 (5.9)	166 (9.7)
E2	-	174 (0.7)	168 (0.6)	175 (1.8)	173 (3.2)	-	177 (3.2)	169 (2.3)	184 (4.3)	162 (8.9)
E3	-	177 (1.8)	172 (1.1)	171 (3.3)	180 (5.3)	-	174 (3.6)	175 (0.9)	187 (1.5)	170 (10.0)
E4	-	175 (1.8)	174 (4.5)	180 (5.7)	162 (5.3)	-	176 (1.5)	175 (0.8)	184 (2.6)	170 (2.6)
E5	-	173 (2.0)	176 (1.5)	181 (4.5)	173 (4.8)	-	178 (1.2)	175 (1.3)	185 (4.1)	180 (2.2)
E6	-	171 (1.4)	165 (2.5)	163 (2.8)	167 (5.0)	-	169 (1.6)	168 (6.0)	168 (1.9)	164 (8.2)
	L10: Ultimate tensile strain [x10 ⁻³]					L50: Ultimate tensile strain [x10 ⁻³]				
REF	14.6 (3.8)					13.3 (13.6)				
E1	-	14.9 (3.2)	15.3 (6.1)	14.5 (7.3)	14.8 (5.3)	-	15.8 (3.6)	15.3 (1.9)	12.9 (6.3)	13.7 (5.5)
E2	-	15.5 (2.9)	16.0 (12.5)	15.5 (6.1)	14.9 (1.3)	-	15.6 (3.9)	15.4 (3.0)	13.9 (3.9)	14.6 (2.6)
E3	-	15.8 (3.8)	15.1 (5.1)	14.6 (8.1)	14.3 (7.0)	-	16.0 (3.6)	15.6 (1.2)	13.6 (4.9)	14.7 (5.1)
E4	-	15.78 (2.3)	15.0 (4.5)	13.8 (4.3)	15.4 (1.1)	-	15.7 (2.8)	15.4 (3.3)	14.6 (1.6)	15.0 (4.4)
E5	-	15.1 (4.6)	14.9 (2.5)	13.5 (7.0)	15.4 (3.1)	-	15.3 (4.1)	14.9 (2.6)	14.5 (7.7)	15.1 (6.0)
E6	-	15.6 (2.9)	16.0 (1.9)	15.7 (2.8)	15.9 (5.0)	-	15.7 (2.6)	15.2 (6.1)	15.7 (2.6)	14.8 (8.2)

Notes: all values in parentheses express *coefficient of variation* in percentage; L10: CFRP with a 1.4 mm × 10 mm cross section; L50: CFRP with a 50 mm × 1.2 mm cross section (tested coupons had 15 mm × 1.2 mm); REF: Reference values from the non-exposed specimens tested at the beginning i.e., at T0.

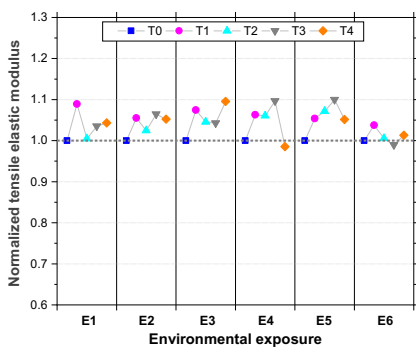
DURABILITY OF CFRP-TO-CONCRETE BOND CONSTITUENT MATERIALS: ACCELERATED *VERSUS* NATURAL AGING



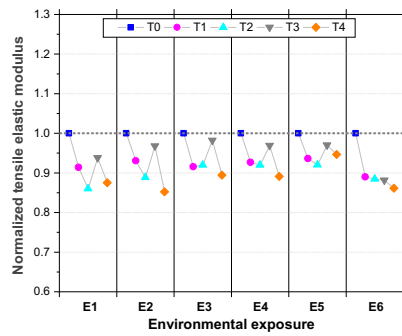
(a) Tensile strength retention of L10



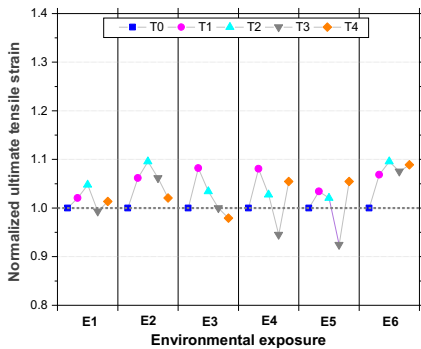
(d) Tensile strength retention of L50



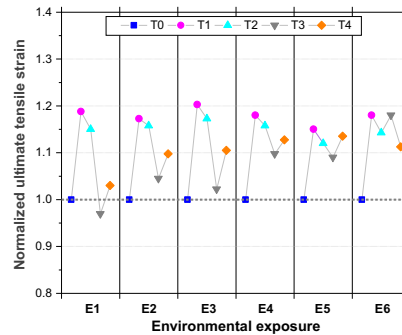
(b) Elastic modulus retention of L10



(e) Elastic modulus retention of L50



(c) Tensile strain retention of L10



(f) Tensile strain retention of L50

Figure I.12: CFRP laminate test results from different environmental exposures (E1-E6) for up to 4 years (T0-T4) of exposure.

3.1.2.2. CFRP laminate L50

The results obtained after testing the CFRP laminate L50 are presented in **Figure I.11**, **Figure I.12d,e,f** and **Table I.4**. In addition, stress *versus* strain curves for all tested L50 specimens tested from different environments (E1-E6) for up to 4 years (T0-T4) are shown in **ANNEX I.5**, which shows a linear elastic behavior for all testing.

A quick look at these results shows that the CFRP L50 showed a different behavior than that of the CFRP L10 previously described, although both were exposed to the same environmental conditions. In particular, the elastic modulus (E_t) of both laminates had opposite trends in all environments, i.e., the L10 had an increasing trend while the L50 had a decreasing trend over time. The tensile strength (f_t), however, showed a somewhat similar trend for both the L10 and L50 laminates, although the former had a higher post-curing effect than the latter, resulting in less pronounced effects of the degradation agents for the case of the L10 compared to that of L50. It is also important to note that the E_t of the L50 did not benefit from the post-curing phenomenon (evidenced by ADH1 $\bar{\gamma}_g$ growth in **Figure I.6**, as both ADH1 and L50 possess a similar polymeric matrix), which shows that the width and thickness of the CFRP laminate may have a significant effect on the E_t property after post-curing. The tensile strain variation is shown in **Figure I.12f** and **Table I.4**, and it can be noted that there were significant changes in all the environments studied during the 4 years of exposure, with retentions ranging between 1.0 and 1.2.

Overall, the effects of the post-curing were different between L10 (10 mm wide and 1.4 thick) and L50 (15mm wide and 1.2 thick) CFRP laminates. L10 laminate benefited more from post-curing than L50 laminate. This can be attributed to the fact that the agents promoting the post-curing may have been able to propagate across the smaller L10 width, whereas the larger L50 width and its fiber matrix may have not been fully post-cured. This is mainly because the L50 width was cut from a larger CFRP laminate with 50 mm width (see **Figure I.1d**); hence, some of the tested L50 coupons had some regions not fully exposed already. Furthermore, the effect of post-curing diminished in the later years, whereas the effect of degradation agents increased. Hence, the properties of both the L10 and L50 started to decrease progressively; however, since the polymeric matrix of L10 had had more post-curing, L10 properties decreased in the later years but were still higher than those at the initial testing before exposure. In contrast, the post-curing phenomenon was not very significant for the polymeric matrix of L50; hence, the effects of the degradation agents appeared faster so that at the 4th year, the properties were already lower than those at the beginning (T0). This leads to the conclusion that the performance of the smaller CFRP laminate width was better than that of the larger width, mainly because their thickness was comparable and hence had no significant effect on the observed trends.

Failure modes:

Typical failure modes for both the epoxy adhesives and CFRP laminates are shown in **Figure I.13**. The adhesive failed by abruptly breaking within the calibrated cross section (region of interest, see **Figure I.13a**), resulting in a minimal sound. For the case of the CFRP laminate, an abrupt progressive brittle failure was observed, where the fibers at the width edges were the ones that started breaking, and then progressively moved toward the fibers in the center of the laminate until the complete failure of all fibers occurred (**Figure I.13b**) with a massive sound. According to ASTM D3039/D3039M [46], this failure mode falls in the category of XGM (failure type: explosive - failure area; gage, failure location - middle). These types of failures did not change with environmental exposure or exposure time, hence it leads to concluding that the failure modes for epoxy adhesives and CFRP laminates is neither affected by type of exposure nor exposure duration.

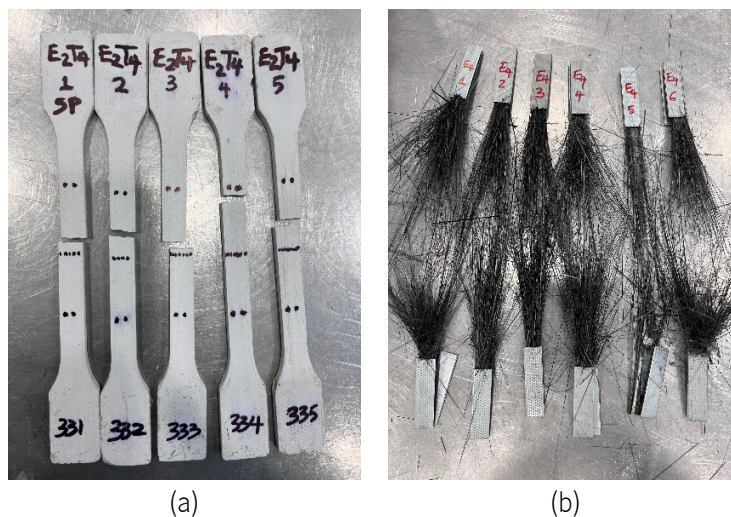


Figure I.13: Typical failure modes from tensile tests: (a) abrupt rupture in the region of interest of the epoxy adhesives, (b) progressive rupture of fibers composing the CFRP laminates.

4. RESULTS FROM MODELING OF EPOXY ADHESIVE IN MOISTURE CONDITIONING

4.1. Analytical modeling

The results obtained using the existing formulation (**Eq. (I.2)**) for predicting water diffusion through the epoxy adhesive thickness are described. It is worth noting that the specimens exposed to water immersion in the E2 environment were conditioned at 20 °C; hence, to facilitate the comparison of the analytical with experimental results, **Eq. (I.2)** was adopted for the epoxy cured at 20 °C for only up to 10752 hours (448 days) of exposure

(i.e., the same time as that of experimental measurements). **Table I.5** summarizes the parameters used to solve Eqs. (I.1) – (I.4). All these parameters are extracted from the experimental data recorded as described in Section 2.1.4.

Table I.5: Parameters used for analytical modeling of epoxy adhesive using Eqs. (I.1) – (I.4).

Temperature	$M_{r1}(\%)$	$M_{r2}(\%)$	t_1 (s)	t_2 (s)	$D = D1 = D2 = D3$ (mm ² /s)
20 °C	0.405	0.932	4.899	12.962	1.8433e-7
40 °C	0.877	2.003	4.899	12.962	7.222 e-7
60 °C	1.268	2.076	4.899	8.485	2.00 e-6

Furthermore, the results from the analytical modeling of the epoxy adhesive at 20 °C, i.e., solving Eq. (I.2) and using the parameters given in **Table I.5**, are plotted together with those from the experimental program in **Figure I.14a**, and a good agreement can be seen. In addition, to extrapolate the effect of water diffusion for up to 4 years, **Figure I.14b** is presented. The effects of water after 10 000 hours were minimal, which is in good agreement with the properties observed experimentally, where the change in both the f_t and E_t properties was also minimal after the first year of exposure. This confirms that the decrease in these properties was mainly due to plasticization caused by water diffusion during the first year of exposure.

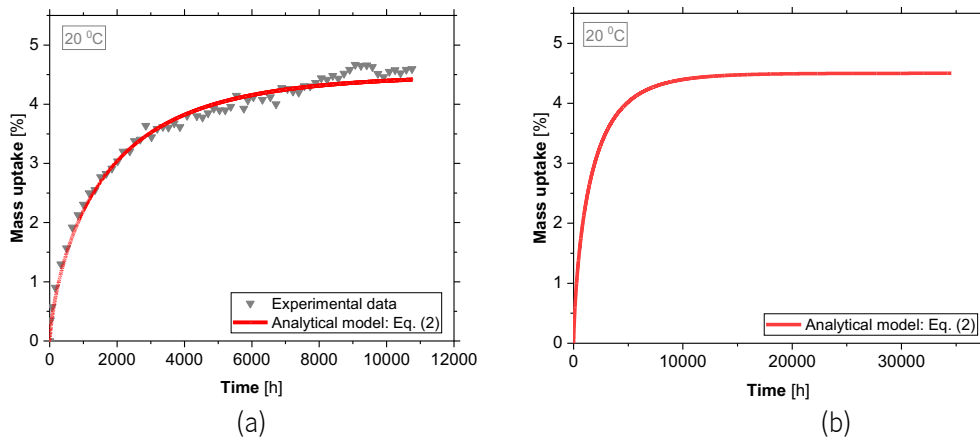


Figure I.14: Mass uptake with exposure time: (a) experimental *versus* analytical results and (b) analytical results extended for up to 4 years of exposure.

In addition, the results of the proposed equation (i.e., Eq. (I.4)) are also presented and compared with those from the existing formulation (Eq. (I.2)) and the experimental program in **Figure I.15**. In other words, to assess the effectiveness of the proposed new formulation, experimental data recorded for water immersion at different temperatures (40 °C and 60 °C) are used. At 20 °C, the chemical aging effect was not yet reached, due to the reasons previously explained in **Section 2.2** (i.e., chemical aging is known to occur after long-term exposure to

moisture [3] if the exposure is not done at high temperatures), because the temperature was not high enough; therefore, only **Eq. (I.2)** was used (i.e., to model physical aging effect only), mainly because the experimental curve continued to increase until the last few hours of exposure. In fact, the mass uptake reached its maximum just closer to the end of the curve (see **Figure I.14a**); hence, consideration of **Eq. (I.4)** would have less effect, i.e., the chemical aging effect may have started to appear at about 9000 hours of exposure.

On the other hand, for the adhesives immersed at higher temperatures, the importance of using **Eq. (I.4)** with parameters shown in **Table I.5** to predict responses for longer-term exposures (e.g., specimens in E2 exposed up to 4 years) became evident (i.e., by highlighting the well-known time temperature superposition effect where the chemical aging is speeded up by high temperatures despite short-term exposure (e.g., 448 days in this work), according to existing literature [29,30,41]). That is, the effect of **Eq. (I.4)** is compared with that of **Eq. (I.2)** in **Figure I.15a** and **Figure I.15b** for cases where the adhesive is continuously immersed at 40 °C and 60 °C, respectively.

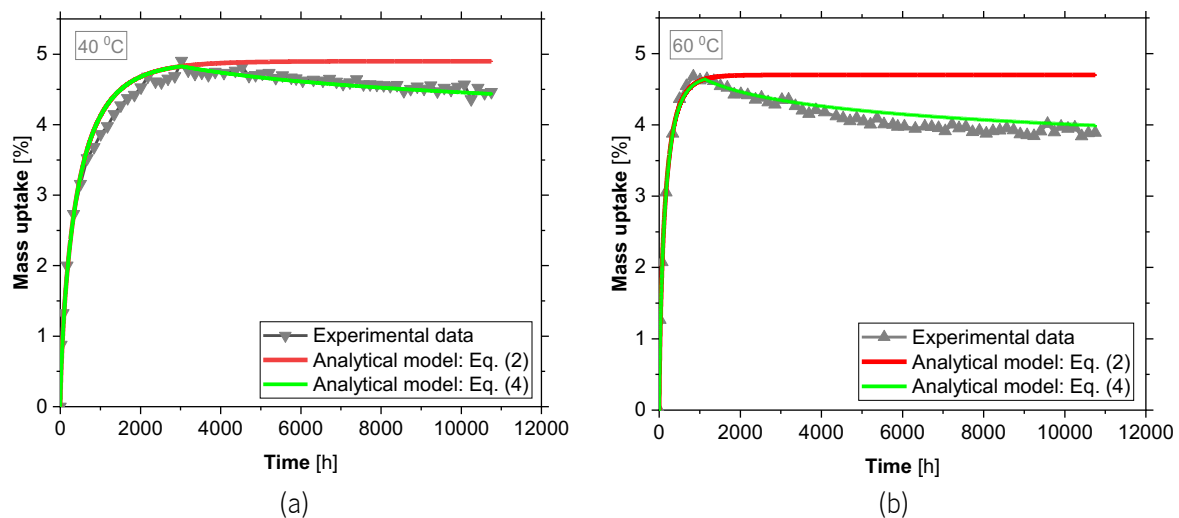


Figure I.15: Mass uptake with exposure time for epoxy adhesive ADH1: Experimental *versus* analytical results from the existing formulation and the proposed formulation at (a) 40 °C and (b) 60 °C of temperature.

In both cases, the failure of **Eq. (I.2)** to predict the experimental curve after reaching the maximum mass uptake is evident, while **Eq. (I.4)** accurately predicts the entire curve up to the final exposure time. Therefore, the proposed formulation surpasses the existing Fickian-based formulation significantly by incorporating the influence of both physical and chemical aging degradation mechanisms. It is worth noting that other formulations with detailed information exist in the literature, for example, those using the Fickian and Langmuir-based

diffusion models [47] or dual Fickian diffusion and delayed dual-Fickian diffusion [48]; however, these are complex compared to the proposed formulation (Eq. (I.4)) which is simply a system of the Fickian and simplified exponential equation.

4.2. Finite element model

The results from the mass diffusion analysis of ADH1 over time are shown in **Figure I.16**. It can be noted that after 10,000 hours, the moisture had already penetrated through the entire adhesive thickness. This finding agrees with the observed experimental results, where the effects of water diffusion (or plasticization) on the properties of the adhesive were much more pronounced during the first year of immersion. Similar to what was previously observed in the case of analytical modeling, the numerical model also reveals the reason for the slight changes observed experimentally in the later years of exposure. That is, after one year of exposure, water had already diffused throughout the adhesive; therefore, water level did not increase in the following years, rather decreased due to crosslinking generated from chemical aging effects, as previously explained in **Section 2.2**. As a result, further exposure did not significantly affect the adhesive's tensile strength or elastic modulus in subsequent years.

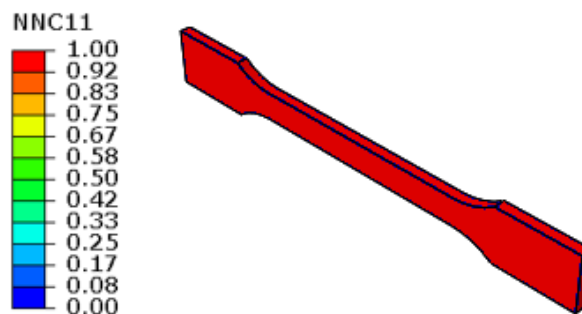


Figure I.16: Average normalized water concentration in the epoxy adhesive after 10 000 h of exposure.

To gain a better understanding of the rate of water penetration, **Figure I.17** and **Figure I.18** are shown for the case of Models 1D and 3D, respectively. In the case of Model 1D (**Figure I.17**), water had already penetrated through almost the entire thickness of the epoxy adhesive after 6100 hours. Similarly, in the case of Model 3D (**Figure I.18**), in which water was allowed to penetrate in all three directions, water had diffused through almost the entire thickness at 6100 hours, although the diffusion is somewhat less than that of Model 1D.

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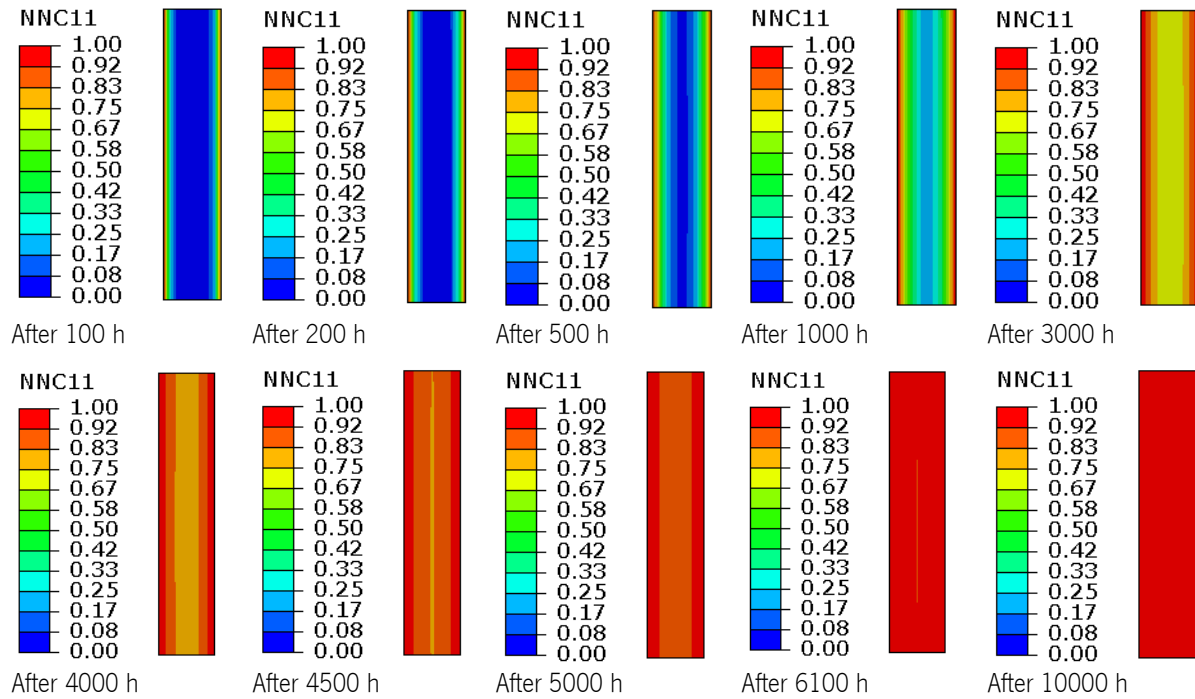


Figure I.17: Average normalized water concentration through epoxy adhesive thickness over time (FEM Model 1D).

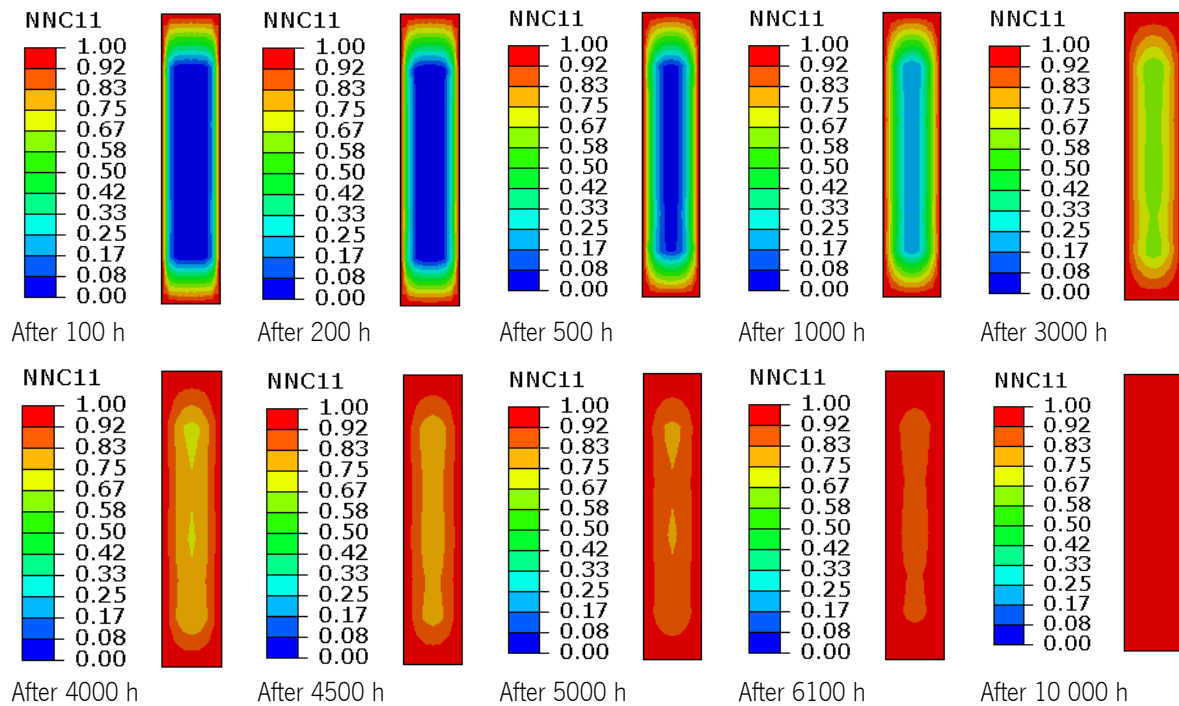


Figure I.18: Average normalized water concentration through epoxy adhesive thickness over time (FEM Model 3D).

Nevertheless, the diffusion in both models is consistent with the tensile test results recorded during the experiment for specimens in the E2 environment, where the epoxy adhesive significantly reduced both its tensile strength and elastic modulus after one year, while the variation in these two properties was minimal in the later years (see **Figure I.8**). This strengthens the hypothesis that plasticization due to water ingress was the main reason for the observed decrease in the aforementioned properties.

The average normalized concentration from the FEM Models 1D and 3D is plotted together with that from the experimental test in **Figure I.19a** and **Figure I.19b**, respectively. In the former, it can be noted that the obtained results from the FE model corroborated well with those obtained from the experimental program, thus strengthening the capability and reliability of the developed model. In fact, while the adopted mass diffusion analysis assumes a uniform level of moisture uptake (which is an oversimplification given that higher concentrations typically occur at the surface layers), the results obtained still showed a reasonable match with experimental data. This is also justified by comparing the developed FE model with the analytical model results, as shown in **Figure I.19c**. An excellent agreement between both models can be seen. In addition, these two models show good agreement with the experimental results (**Figure I.19c**); however, both models (i.e., Model 1D and analytical model) tend to slightly overestimate the experimental data in the range of 4000-9000 hours of exposure. Nevertheless, this overestimation is not very pronounced and hence can be considered not relevant.

For the case of Model 3D, an overprediction is observed when compared with experimental data (see **Figure I.12b**), and a discrepancy between this model and analytical (and experimental) data is also depicted in **Figure I.19d**, indicating that Model 1D can be better than Model 3D. However, both numerical models lead to results in reasonable agreement with the experimental data. It is worth noting again that the experimental data mainly show the physical degradation region, as the curve kept increasing until 9000 hours of exposure, after which it reached a maximum concentration with a tendency to start decreasing, mainly because the chemical aging effect could have started (start of formation of additional cross-linking that changes material densification by pushing out the already diffused water [41]) at this stage. However, since the chemical aging region is not large enough, only **Eq. (I.2)** was adopted.

Overall, the developed numerical and analytical models were in good agreement with the experimental data. In addition, the numerical model revealed unhidden mechanisms, notably the time at which the epoxy adhesive reached its almost full water uptake (i.e., approximately after 6100 h (254 days) of water immersion exposure). The analytical model revealed the importance of separating the physical aging effect from the chemical aging

effect, as this allows better predictions to be obtained compared to the existing case (Fickian-based models) where only the physical aging mechanism is considered. The above importance becomes very crucial as the curing temperature increases; hence, **Eq. (I.4)** is recommended for epoxy adhesive conditioned at high temperature and exposed for short-term periods, where the chemical aging mechanisms may start earlier.

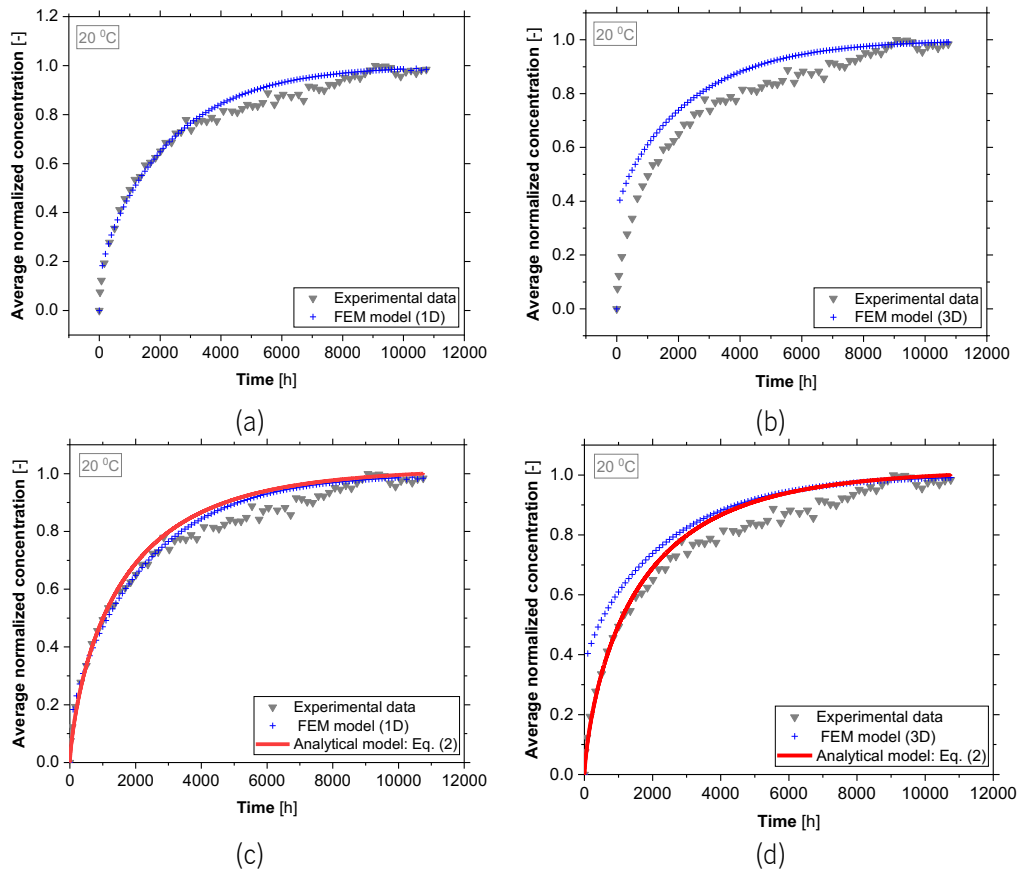


Figure I.19: Average normalized water concentration *versus* exposure time for epoxy adhesive: (a) experimental *versus* FE model (1D), (b) experimental *versus* FE model (3D), (c) experimental, FEM model (1D), and analytical model, and (d) experimental, FEM model (3D), and analytical model.

4.3. Environmental conversion factors

Existing standards and guidelines, e.g., [23,49–51], have recommended environmental conversion factors (ECF), i.e., retention values, to account for the effects of degradation agents from different environmental exposures on the properties of epoxy adhesive and CFRP laminate. A summary of these recommended ECFs is presented in **Table I.6** along with those obtained in this study (for up to 4 years). A comparative perspective

shows that for continuous water immersion, the values obtained in the present work for both ADH1 and ADH2 are lower than those recommended in CEN/TS 19901:2022 [23]. This is probably because the type of epoxy adhesive referred to in this specification is prepared for applications other than structural strengthening of concrete. However, in the case of outdoor exposure, comparable ECF can be observed between the present work and the CEN/TS 19901:2022 [23].

Table I.6: Comparisons between the environmental conversion factors obtained from this study and those from existing standards/guidelines.

	ASCE/ACMA	Dutch CUR96	German BUeV	CEN/TS 19901:2022	This work
Epoxy adhesive					
Continuous water immersion (III)	-	-	-	0.60	ADH1:0.37 ^c , 0.26 ^d ADH2:0.40 ^c , 0.30 ^d
Outdoor exposure (II) ^c	-	-	-	0.85	ADH1:0.80 ^c , 0.80 ^d ADH2:1.0 ^c , 1.0 ^d
CFRP laminate					
Continuous water immersion (III)	-	0.70	-	0.60	L10:1.05 ^c , 1.08 ^d L50:0.95 ^c , 0.85 ^d
Outdoor exposure (II) ^c	0.85 ^a , 0.80 ^b	0.90	0.91	0.85	L10:1.12 ^c , 1.10 ^d L50:0.95 ^c , 0.85 ^d

Notes: Values in parentheses stand for the exposure class, ^a: the resin type of the fiber matrix is vinylester for strength property, ^b: the resin type of the fiber matrix is polyester for strength property, ^c: tensile strength property, ^d: elasticity modulus property. In the present work, the outdoor exposure is considered as environments E3 to E6.

In particular, for the case of ADH2, the value of 1.0 is obtained and is expected to decrease over time, probably reaching the value of 0.85 recommended in the CEN/TS 19901:2022 [23].

For the case of the CFRP laminates, the values in the presented work are generally higher than those recommended in the Dutch CUR96 [50] and CEN/TS [23]. For the case of water immersion exposure, the obtained factors are expected to decrease and approach those recommended in the standards in later years, which is reasonable because of the shorter exposure time considered. For the case of outdoor exposure, the values in the present work (ranging from 0.85 to 1.12) are also slightly higher than those recommended in the ASCE/ACMA [49], Dutch CUR96 [50], German BUeV [51], and CEN/TS 19901:2022 [23], which range from 0.80 to 0.91. It can therefore be expected that they will decrease over time and approach the values recommended in the standards and guidelines. Overall, the obtained parameters are a first step towards establishing the most appropriate ones, as they only represent exposure of the specimens for up to 4 years.

5. CONCLUSIONS

Experimental, analytical, and numerical studies were conducted on the behavior of two different epoxy adhesives (one denoted as ADH1 and the other as ADH2) and two different CFRP laminates (one denoted as L10 and the other as L50) under different environmental exposures. Both materials (i.e., the adhesives and the CFRP laminates) were exposed to the same types of environments for up to 4 years. For the laboratory-controlled environments, specimens were immersed in tap water controlled at 20 °C and others conditioned at 20 °C and 55 % RH, while different environments were selected for outdoor exposure to promote aging induced by carbonation, elevated temperatures, freeze–thaw attack, and airborne chlorides from the ocean. Furthermore, the specimens were tested annually to assess the degradation of their tensile strength and elastic modulus. Analytical and numerical models were also developed and validated by the results of the experimental tests. The key findings are highlighted as follows.

- Water immersion substantially affected the properties of both ADH1 and ADH2 adhesives during the first years of exposure. In fact, ADH1 lost approximately 66 % and 75 % of its tensile strength (f_a) and elastic modulus (E_a), respectively, whereas ADH2 lost approximately 60 % and 70 % of its f_a and E_a , respectively. From these decreases, it was revealed that the elastic modulus degraded much faster than the tensile strength, with an approximate difference of 10 % between f_a and E_a .
- In the other environments studied, ADH1 did not significantly benefit from post-curing during the first year of exposure; therefore, its tensile strength (f_a) and elastic modulus (E_a) increased slightly. Due to these slight increases, degradation agents caused significant decreases in the later years. In contrast, ADH2 showed a significant post-curing during the first year of exposure in all environments. This resulted in increased f_a and E_a , with the highest increases of nearly 40 % and 50 %, respectively, for the specimens exposed to airborne chlorides from the ocean. Because of the above substantial post-curing, the effect of the degradation agents was delayed so that even after 4 years of exposure, the ADH2 properties were still higher than those recorded for non-exposed specimens. Hence, the post-curing of ADH2 played a significant role in counteracting the effects of degradation agents.
- Exposure of the CFRP laminate L10 (with 10 mm wide and 1.4 mm thick) to different environments improved both the tensile strength (f) and the elastic modulus (E) properties, as a result of the

continuation of the post-curing phenomenon of the fiber matrix resin that mainly occurred during the first year. In the following years, the properties started to decrease as the effect of the degradation agents increased, while the post-curing rate decreased. However, because of the substantial post-curing during the first year, the properties decreased with time but were still higher than those of the non-exposed specimens after 4 years. There were approximately 16 % and 15 % increases in f_t as a result of exposure of L10 to carbonation and freeze-thaw, respectively. In addition, high temperatures and presence of carbonation improved E_t by approximately 10 % in both cases. In contrast, the exposure of the CFRP laminate L50 (50 mm wide and 1.2 mm thick) did not improve its E_t ; rather, a significant decrease was observed only during the first year of exposure. However, its f_t improved during the first year and then started to decrease in the later years of exposure so that in the fourth year, the f_t values were already lower than those recorded for the non-exposed specimens. Comparing L10 and L50 led to the conclusion that the laminate with smaller width and slightly larger thickness had better durability performance than that with larger width and slightly smaller thickness. Hence, the width and thickness of the CFRP laminate are important parameters when designing structures with CFRP composites.

- Analytical modeling of ADH1 immersed in water at 20, 40 and 60 °C was conducted. Two different formulations were used to study the water diffusion process through the adhesive thickness. The first formulation is based on the well-known Fickian diffusion process, whereas the second formulation was newly proposed. The former formulation was ideal for predicting the physical aging effect; however, it failed to capture the chemical aging effect. In contrast, the newly proposed formulation was able to capture both the physical and chemical aging effects, thus leading to a much better agreement with the experimental results, particularly at higher temperatures. Therefore, the new formulation (i.e., Eq. (I.4)) can be proposed in future works.
- Numerical modeling of ADH1 immersed in water at 20 °C was conducted by performing mass diffusion analysis using the ABAQUS software. The results from the developed model corroborated well with the experimental results. The developed model showed that water diffusion was fast, such that it completely diffused the whole adhesive in just a few months (approximately after 6100 hours of exposure). This revealed that the substantial decrease in both the tensile strength and elastic modulus, observed in the experimental test after one year of exposure, was significantly due to the plasticization effect caused by the above full water diffusion.

- Based on the results of this work, the environmental conversion factors (ECF) were derived and compared with those from existing standards and guidelines. The derived ECFs are still smaller than those recommended for the case of water immersion, probably because of the shorter exposure time compared to that considered in the standards and guidelines, and because the type of epoxy adhesive referred to in these specifications is prepared for applications other than structural strengthening. However, for outdoor exposure, comparable ECFs were found. In the case of the CFRP laminates, the ECF values are generally higher than those recommended for water immersion exposure and slightly higher for outdoor exposure, which is reasonable since the exposure time is shorter; therefore, comparable values can be expected as the years of exposure increase.

In future studies, numerical and analytical models capable of quantifying the effect of water diffusion on the tensile strength and elastic modulus are recommended. In addition, models that consider the effects of other types of environments studied here (degradation due to carbonation, elevated temperatures, freeze-thaw attack, and airborne chlorides from the ocean) are highly recommended.

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REFERENCES

- [1] I.S. Abbood, S.A. Odaa, K.F. Hasan, M.A. Jasim, Properties evaluation of fiber reinforced polymers and their constituent materials used in structures - A review, *Mater. Today Proc.* 43 (2021) 1003–1008. <https://doi.org/10.1016/j.matpr.2020.07.636>.
- [2] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B Eng.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
- [3] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fibre reinforced polymer laminates used in strengthening systems: Accelerated ageing versus natural ageing, *Materials (Basel)*. 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [4] J. Tatar, H.R. Hamilton, Bond Durability Factor for Externally Bonded CFRP Systems in Concrete Structures, *J. Compos. Constr.* 20 (2016). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000587](https://doi.org/10.1061/(asce)cc.1943-5614.0000587).
- [5] J. Sena-Cruz, J. Michels, Y.E. Harmanci, L. Correia, Flexural strengthening of RC slabs with prestressed CFRP strips using different anchorage systems, *Polymers (Basel)*. 7 (2015) 2100–2118. <https://doi.org/10.3390/polym7101502>.
- [6] A. Dushimimana, R. Cruz, S. Cabral-fonseca, Natural Outdoor and Laboratory-Controlled Ageing of Epoxy Adhesives and Cfrp Laminates After Four Year of Exposure, 11th Int. Conf. FRP Compos. Civ. Eng. (2023) 1–10. <https://doi.org/https://doi.org/10.5281/zenodo.8109646>.
- [7] A. Dushimimana, L. Correia, R. Cruz, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-Concrete Bond in EBR and NSM Sysytems under Natural Ageing for a Period of Three Years, in: *Proc. 20th Eur. Conf. Compos. Mater., CC BY-NC 4.0, Lausanne, Switzerland, 2022: pp. 1–8*. https://doi.org/https://doi.org/10.5075/epfl-298799_978-2-9701614-0-0.
- [8] H. Yamane, M. Oura, N. Yamazaki, T. Ishihara, K. Hasegawa, T. Ishikawa, K. Takagi, T. Hatsui, Visualizing interface-specific chemical bonds in adhesive bonding of carbon fiber structural composites

- using soft X-ray microscopy, *Sci. Rep.* 12 (2022) 1–8. <https://doi.org/10.1038/s41598-022-20233-4>.
- [9] P. Fernandes, J. Sena-Cruz, J. Xavier, P. Silva, E. Pereira, J. Cruz, Durability of bond in NSM CFRP-concrete systems under different environmental conditions, *Compos. Part B Eng.* 138 (2018) 19–34. <https://doi.org/10.1016/j.compositesb.2017.11.022>.
- [10] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of bond of EBR CFRP laminates to concrete under real-time field exposure and laboratory accelerated ageing, *Constr. Build. Mater.* 377 (2023). <https://doi.org/10.1016/j.conbuildmat.2023.131047>.
- [11] E. Cui, S. Jiang, J. Wang, X. Zeng, Bond behavior of CFRP-concrete bonding interface considering degradation of epoxy primer under wet-dry cycles, *Constr. Build. Mater.* 292 (2021) 123286. <https://doi.org/10.1016/j.conbuildmat.2021.123286>.
- [12] S.A. Grammatikos, R.G. Jones, M. Evernden, J.R. Correia, Thermal cycling effects on the durability of a pultruded GFRP material for off-shore civil engineering structures, *Compos. Struct.* 153 (2016) 297–310. <https://doi.org/10.1016/j.compstruct.2016.05.085>.
- [13] D. dong Hu, J. xun Lyu, T. Liu, M. dong Lang, L. Zhao, Solvation effect of CO₂ on accelerating the curing reaction process of epoxy resin, *Chem. Eng. Process. - Process Intensif.* 127 (2018) 159–167. <https://doi.org/10.1016/j.cep.2018.01.027>.
- [14] S. Liu, Y. Pan, H. Li, G. Xian, Durability of the bond between CFRP and concrete exposed to thermal cycles, *Materials (Basel)*. 12 (2019). <https://doi.org/10.3390/ma12030515>.
- [15] C. Helbling, V. Karbhari, Durability of composites in aqueous environments, in: *Durab. Compos. Civ. Struct. Appl.*, Elsevier, Amsterdam, The Netherlands, 2007: pp. 31–71. <https://doi.org/https://doi.org/10.1533/9781845693565.1.31>.
- [16] J. Tatar, N.R. Brenkus, Performance of FRP-Strengthened Reinforced Concrete Bridge Girders after 12 Years of Service in Coastal Florida, *J. Compos. Constr.* 25 (2021). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001134](https://doi.org/10.1061/(asce)cc.1943-5614.0001134).
- [17] M. Savilotidou, A.P. Vassilopoulos, T. Keller, Durability and Fatigue Performance of Cold-Curing Structural Adhesives in Bridge Construction, *École Polytechnique Fédérale de Lausanne (EPFL)*, 2017. <https://infoscience.epfl.ch/record/230442?ln=en>.
- [18] S. Amidi, J. Wang, Deterioration of the FRP-to-concrete interface subject to moisture ingress: Effects of conditioning methods and silane treatment, *Compos. Struct.* 153 (2016) 380–391.

- <https://doi.org/https://doi.org/10.1016/j.compstruct.2016.06.035>.
- [19] M. Heshmati, R. Haghani, M. Al-Emrani, Effects of moisture on the long-term performance of adhesively bonded FRP/steel joints used in bridges, *Compos. Part B.* 92 (2016) 447–462. <https://doi.org/http://dx.doi.org/10.1016/j.compositesb.2016.02.021>.
 - [20] Y. Joliff, L. Belec, M.B. Heman, J.F. Chailan, Experimental, analytical and numerical study of water diffusion in unidirection composite materials - Interphase impact, *Comput. Mater. Sci.* 64 (2012) 141–145. <https://doi.org/http://dx.doi.org/1016/j.commatsci.2012.05.029>.
 - [21] J.C. Arnold, S.M. Alston, F. Korkees, An assessment of methods to determine the directional moisture diffusion coefficients of composite materials, *Compos. Part A.* 55 (2013) 120–128. <https://doi.org/http://dx.doi.org/10.1016/j.compositesa.2013.08.012>.
 - [22] N. Roe, Z. Huo, K. Chandrashekhara, A. Buchok, R. Brack, Advanced moisture modeling of polymer composites, *J. Reinf. Plast. Compos.* (2012) 62. <https://doi.org/10.1177/0731684412470182>.
 - [23] CEN/TS 19101, Design of fibre-polymer composite structures, 2022.
 - [24] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of Bond between NSM CFRP Strips and Concrete under Real-Time Field and Laboratory Accelerated Conditioning, *J. Compos. Constr.* 26 (2022) 1–15. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001262](https://doi.org/10.1061/(asce)cc.1943-5614.0001262).
 - [25] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>.
 - [26] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174. <https://doi.org/10.1016/j.compstruct.2024.118174>.
 - [27] ISO, ISO 527-2:2012—Plastics—Determination of Tensile Properties—Part 2: Test conditions for Moulding and Extrusion Plastics, Genève, Switzerland, 2012.
 - [28] ISO, ISO 527-5:2009 Part 5: Test Conditions for Unidirectional Fibre-Reinforced Plastic composites. Plastic—Determ. Tensile Prop., Inte Genève, Switzerland, 2009; Volume 1, 2009.
 - [29] D. Flore, K. Wegener, D. Seel, C.C. Oetting, T. Bublat, Investigation of chemical ageing and its effect on static and fatigue strength of continuous fibre reinforced plastics, *Compos. Part A Appl. Sci. Manuf.* 90 (2016) 359–370. <https://doi.org/10.1016/j.compositesa.2016.08.001>.
 - [30] F.R. JONES, Durability of reinforced plastics in liquid environments, in: *Reinf. Plast. Durab.*, 1999: pp.

- 70–110. <https://doi.org/10.1533/9781845694876.70>.
- [31] G. LaPlante, A. V. Ouriadov, P. Lee-Sullivan, B.J. Balcom, Anomalous Moisture Diffusion in an Epoxy Adhesive Detected by Magnetic Resonance Imaging, *J. Appl. Polym. Sci.* 109 (2008) 1351–1359. <https://doi.org/10.1002/app>.
- [32] V. Bonilla Mora, M. Mieloszyk, FE Model of Moisture Absorption by Adhesive Joints between Composites, GDANSK UNIVERSITY OF TECHNOLOGY, 2016.
- [33] J. Crank, The mathematics of diffusion. 2nd Edn., Second edi, CLARENDON PRESS, Oxford, 1975. <https://doi.org/10.1088/0031-9112/26/11/044>.
- [34] Y.J. Weitsman, Fluid effects in polymers and polymeric composites, Springer, 2011. <https://doi.org/10.1007/978-1-4614-1059-1>.
- [35] F. Müller-Plathe, S.C. Rogers, W.F. van Gunsteren, Computational evidence for anomalous diffusion of small molecules in amorphous polymers, *Chem. Phys. Lett.* 199 (1992) 237–243. [https://doi.org/10.1016/0009-2614\(92\)80112-O](https://doi.org/10.1016/0009-2614(92)80112-O).
- [36] A. Mubashar, I.A. Ashcroft, G.W. Critchlow, A.D. Crocombe, Modelling cyclic moisture uptake in an epoxy adhesive, *J. Adhes.* 85 (2009) 711–735. <https://doi.org/10.1080/00218460902997224>.
- [37] K.W. Thomson, T. Wong, L.J. Broutman, The plasticization of an epoxy resin by dibutylphthalate and water, *Polym. Eng. Sci.* 24 (1984) 1270–1276. <https://doi.org/10.1002/pen.760241610>.
- [38] S. Cabral-fonseca, J.R. Correia, J. Custódio, H.M. Silva, A.M. Machado, J. Sousa, Durability of FRP - concrete bonded joints in structural rehabilitation : A review, *Int. J. Adhes. Adhes.* 83 (2018) 153–167. <https://doi.org/10.1016/j.ijadhadh.2018.02.014>.
- [39] C. Helbling, V. Karbhari, Durability of composites in aqueous environments, in: *Durab. Compos. Civ. Struct. Appl.*, Elsevier, Amsterdam, The Netherlands, 2007: pp. 31–71. <https://doi.org/https://doi.org/10.1533/9781845693565.1.31>.
- [40] C.A. Wood, W.L. Bradley, Determination of the effect of seawater on the interfacial strength of an interlayer E-glass/graphite/epoxy composite by in situ observation of transverse cracking in an environmental SEM, *Compos. Sci. Technol.* 57 (1997) 1033–1043. [https://doi.org/10.1016/S0266-3538\(96\)00170-4](https://doi.org/10.1016/S0266-3538(96)00170-4).
- [41] T. Gates, The physical and chemical ageing of polymeric composites, in: *Ageing Compos.*, 2008: pp. 3–33. <https://doi.org/10.1533/9781845694937.1.3>.

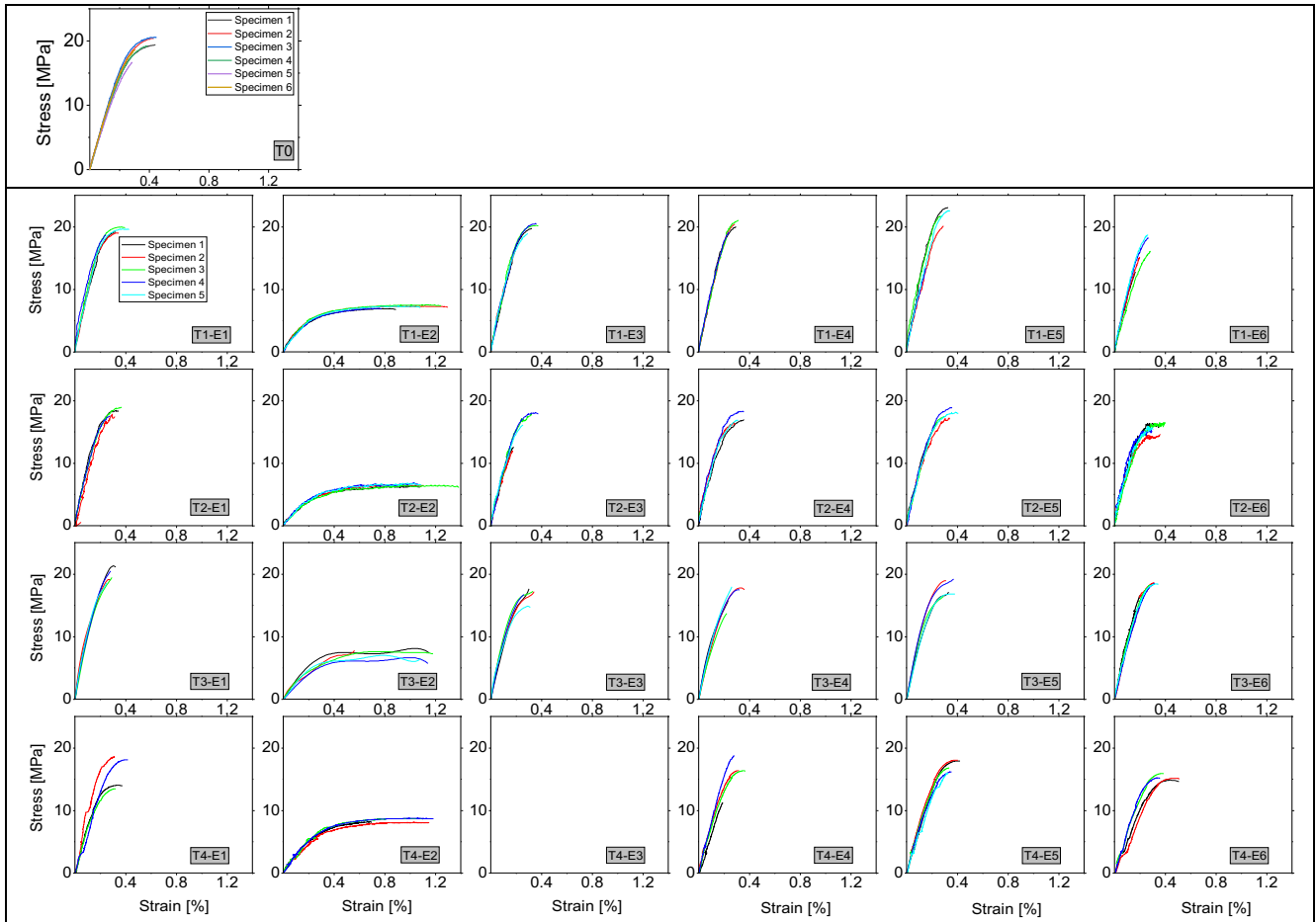
- [42] G.A. Schoeppner, G.P. Tandon, K. V Pochiraju, W.A.F. Base, Predicting Thermooxidative Degradation and Performance of High-Temperature Polymer, in: T. R, K. YW, A. DH (Eds.), *Multiscale Model. Simul. Compos. Mater. Struct.*, Springer, New York, 2008: p. 630. https://doi.org/https://doi.org/10.1007/978-0-387-68556-4_9.
- [43] P. Böer, L. Holliday, T.H.K. Kang, Independent environmental effects on durability of fiber-reinforced polymer wraps in civil applications: A review, *Constr. Build. Mater.* 48 (2013) 360–370. <https://doi.org/10.1016/j.conbuildmat.2013.06.077>.
- [44] A. Dushimimana, L. Correia, R. Cruz, S. Cabral-fonseca, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-concrete bond and corresponding involved materials under different natural environmental exposures for a period of four years, in: *FRPRCS-15 and APFIS-2022*, Shenzhen, China, 2022: pp. 10–14.
- [45] F. Jiang, X. Han, Y. Wang, P. Wang, T. Zhao, K. Zhang, Effect of freeze-thaw cycles on tensile properties of CFRP, bond behavior of CFRP-concrete, and flexural performance of CFRP-strengthened concrete beams, *Cold Reg. Sci. Technol.* 194 (2022) 103461. <https://doi.org/10.1016/j.coldregions.2021.103461>.
- [46] ASTM, “Standard test method for tensile properties of polymer matrix composite materials.” D3039/D3039M, West Conshohocken, PA, 2000.
- [47] V. La Saponara, Environmental and chemical degradation of carbon/epoxy and structural adhesive for aerospace applications: Fickian and anomalous diffusion, Arrhenius kinetics, *Compos. Struct.* 93 (2011) 2180–2195. <https://doi.org/10.1016/j.compstruct.2011.03.005>.
- [48] A. Mubashar, I.A. Ashcroft, G.W. Critchlow, A.D. Crocombe, Moisture absorption-desorption effects in adhesive joints, *Int. J. Adhes. Adhes.* 29 (2009) 751–760. <https://doi.org/10.1016/j.ijadhadh.2009.05.001>.
- [49] ASCE, Pre-standard for load and resistance factor design (LRFD) of pultruded (ACMA)., polymer (FRP) structures (final)., *Am. Compos. Manuf. Assoc.* (2010).
- [50] CROW-CUR Recommendation 96, Fiber-Reinforced Polymers in Buildings and Civil Engineering Structures. CROW, 2019.
- [51] BUeV Guideline, Structural plastic components in construction., 2014.

PAPER I ANNEXES

ANNEX I.1. A python code used during post-processing of data from mass diffusion analysis.

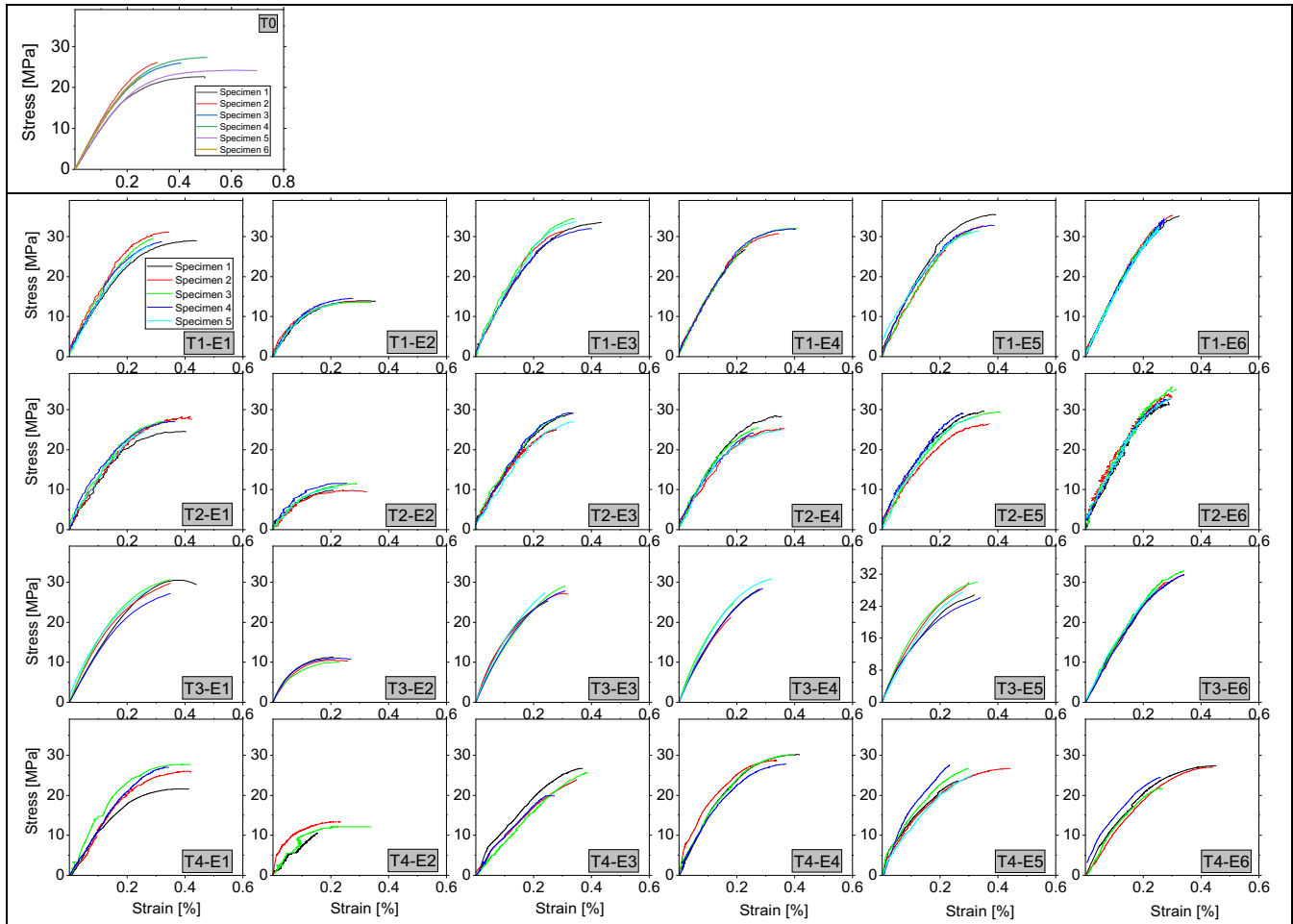
```
from odbAccess import *
from abaqusConstants import *
session.viewports['Viewport: 1'].setValues(displayedObject=None)
odb = odbAccess.openOdb(path='JobName.odb')
endSet = odb.rootAssembly.instances['PART-1-1']
myFile = open('FileName.txt', 'w+')
myFile.write("# Time Vol Sum ISOL SUM AVG CONCT n n\n")
for step in odb.steps.values():
    framesNum = len(step.frames)
    for frn in range(0, framesNum):
        frame = step.frames[frn]
        VolField = frame.fieldOutputs['IVOL']
        VolSubField = VolField.getSubset(region=endSet)
        SolField = frame.fieldOutputs['ISOL']
        SolSubField = SolField.getSubset(region=endSet)
        SolSum = 0
        VolSum = 0
        for num in range(len(VolSubField.values)):
            VolVal = VolSubField.values[num].data
            Vsum = VolSum + VolVal
            SolVal = SolSubField.values[num].data
            Ssum = SolSum + SolVal
        AVG = Ssum / Vsum
        frametotalTime = step.totalTime + frame.frameValue
        myFile.write(str(frametotalTime))
        myFile.write(" ")
        myFile.write(str(VolSum))
        myFile.write(" ")
        myFile.write(str(SolSum))
        myFile.write(" ")
        myFile.write(str(AVG))
        myFile.write("\n")
myFile.close()
```

ANNEX I.2. Stress *versus* strain curves for all tested ADH1 specimens from different environments (E1-E6) for up to 4 years (T0-T4) of exposure.

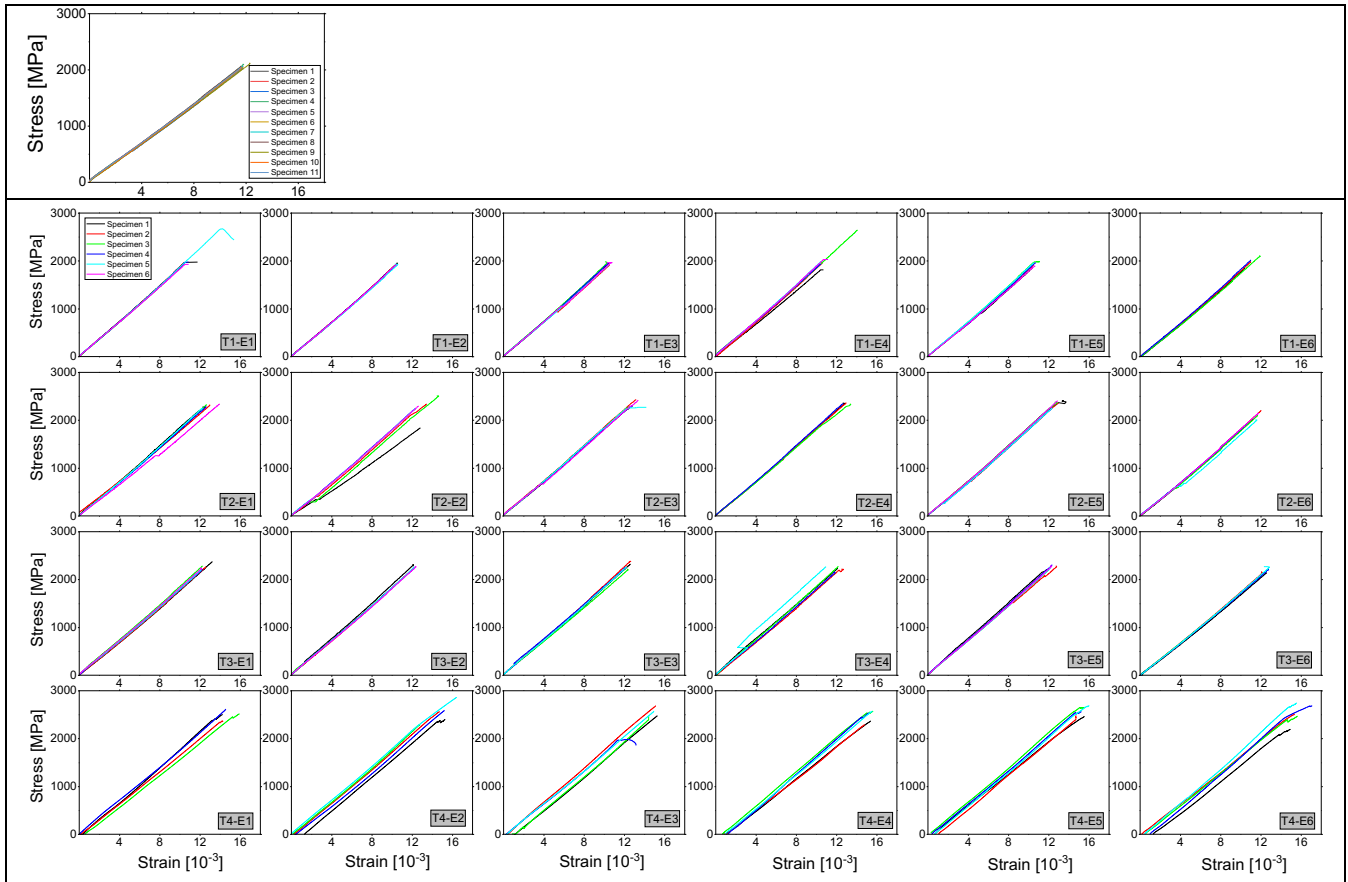


DURABILITY OF CFRP-TO-CONCRETE BOND CONSTITUENT MATERIALS:
ACCELERATED *VERSUS* NATURAL AGING

ANNEX I.3. Stress *versus* strain curves for all tested ADH2 specimens from different environments (E1-E6) for up to 4 years (T0-T4) of exposure.

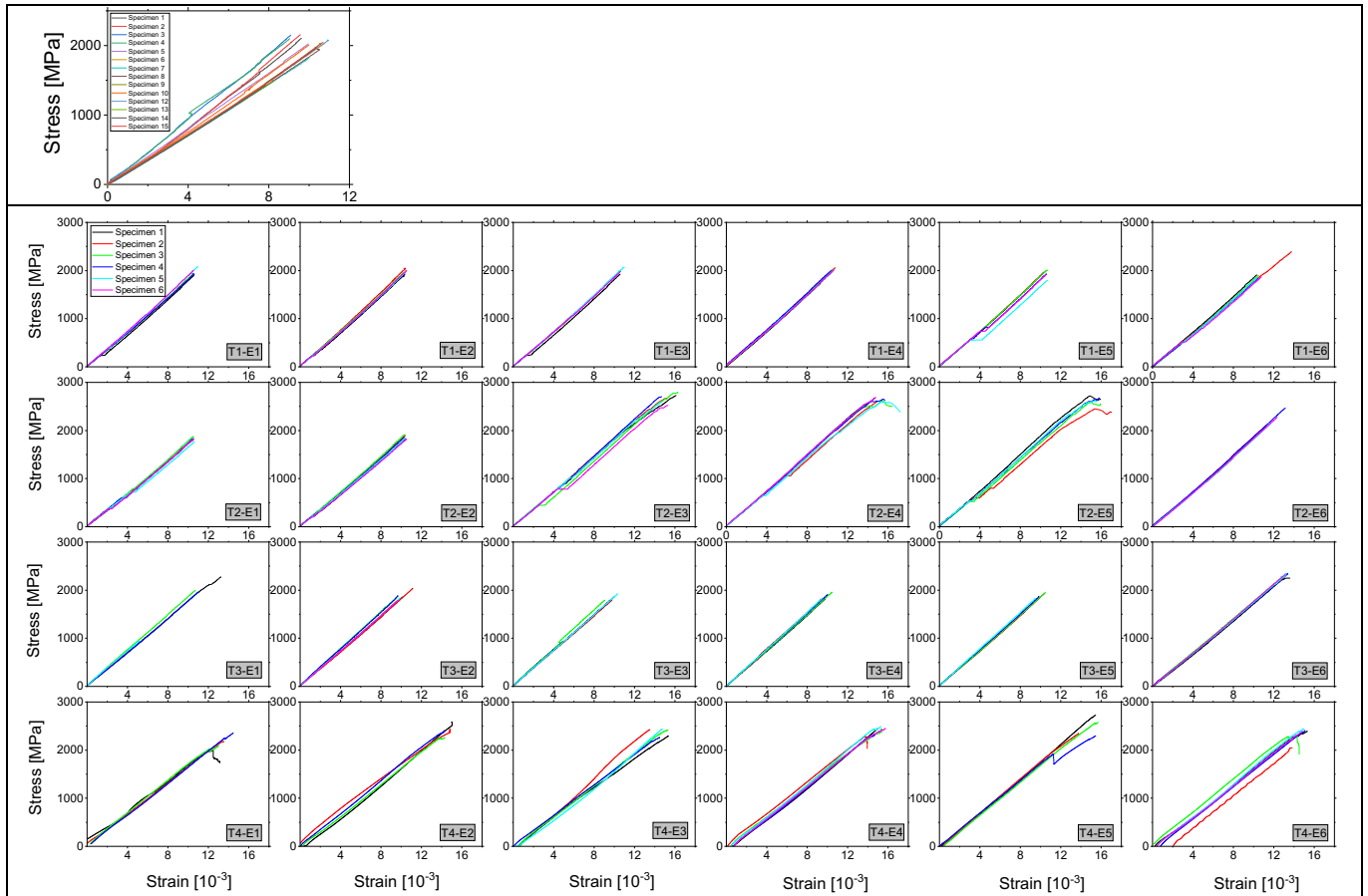


ANNEX I.4. Stress *versus* strain curves for all tested CFRP L10 specimens from different environments (E1-E6) for up to 4 years (T0-T4) of exposure.



DURABILITY OF CFRP-TO-CONCRETE BOND CONSTITUENT MATERIALS:
ACCELERATED *VERSUS* NATURAL AGING

ANNEX I.5. Stress *versus* strain curves for all tested CFRP L50 specimens from different environments (E1-E6) for up to 4 years (T0-T4) of exposure.



PAPER II

DURABILITY OF EXTERNALLY BONDED CFRP-TO- CONCRETE BOND: ACCELERATED *VERSUS* NATURAL AGING

REFERENCE: *Dushimimana A., Sena-Cruz J., Correia L., Pereira J.M., Cabral Fonseca S., Cruz R. Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated aging exposures, Construction and Building Materials. 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>*

Impact factor: 7.4 (April 2025)

ABSTRACT: Application of carbon fiber reinforced polymer (CFRP) composites in strengthening of existing reinforced concrete (RC) structures has been widely accepted. However, the durability of adhesively bonded CFRP-concrete joint has not yet been fully investigated, which therefore paves the way to the topic addressed

this work: the durability of the joint in concrete elements strengthened with CFRP laminate using externally bonded reinforcement (EBR) technique. Concrete strengthened elements were kept in laboratory-controlled environments (approximately 20 °C/55 % RH, and water immersion at 20 °C), while others were kept outdoor to mainly promote natural aging by carbonation, high temperatures, freeze-thaw attacks, and airborne chlorides. The results from durability tests after 4 years of exposure showed insignificant bond strength degradation but with a noticeable bond stiffness reduction, also, the stiffness degraded faster than the strength. Besides, environmental conversion factors of 0.75 and 0.95 were derived from a database of existing accelerated aging test data and the natural aging test data from the present work, respectively.

KEYWORDS: Bond, Concrete, Epoxy adhesive, CFRP, Aging, Moisture, Freeze-thaw, Temperature, Carbonation, Airborne chlorides

1. INTRODUCTION

A significant number of existing reinforced concrete (RC) structures around the globe are old, hence there is a growing demand for possible solutions that can be used to preserve and/or extend the service life of such structures. One of the possible solutions is the application of strengthening systems. The materials to be used should have desirable short- and long-term performance properties to improve the overall performance of the structure, and also be environmentally friendly, possessing a high sustainability index. One of the best candidates for such materials is carbon fiber reinforced polymer (CFRP) composites, which possess a lower environmental impact than glass FRP (GFRP) [1] and other alternative materials, if produced using a recycling approach [2]. CFRP composites have been shown to possess excellent properties including high strength to weight ratio, high durability, and high corrosion resistance [3], among others. It is worth noting that CFRP composites are typically produced as either in situ cured sheets or prefabricated (pultruded) laminates. The former is generally used for shear strengthening due to its flexibility to deform into any desired shape, e.g., [4], while the latter is commonly used for flexural strengthening, e.g., [5]. In this work, the CFRP laminates were selected as the focus of the study was on the flexural strengthening. The CFRP laminates can be bonded to existing RC structures using an epoxy adhesive as a bonding agent, and the application can be through externally bonded reinforcement (EBR) or near-surface mounted (NSM) strengthening technique [6]. Focusing on the EBR-CFRP technique, the CFRP is typically bonded on the surface of the element to be strengthened [7]. In general, the durability of EBR-CFRP

is still an open area of research, as there is a lack of knowledge on the long-term performance of CFRP-to-concrete bond and its constituent materials as well.

The durability of concrete when exposed to different degradation agents has been investigated. In fact, concrete carbonation depth can be affected by some agents, for example, low relative humidity (RH) and high temperatures can render concrete more porous, thereby allowing carbonation depth increase [8–10]. Contrary, low temperatures (below 9 °C) [11] and continuous cement hydration [12] will generally prevent CO₂ penetration. Besides, carbonation depth increases in environment with 50-70 % RH [8] and can increase optimally at 65 % RH [11]. Studies have shown that when concrete is exposed to carbonation [12] or to ultraviolet radiation (UV) with certain humidity and temperature [13], its compressive strength will increase. Both concrete compressive strength and elastic modulus increased after being exposed to carbonation [14]. Besides, carbonation reduced the effect of chlorides on the concrete surface by increasing the effect at the concrete inner region (>3mm depth) as exposure time increased [15]. This was also found in other studies [16], where carbonation released the bound chlorides (from Friedels salt) inward at greater depth.

On the other hand, epoxy adhesive properties can also be affected by various degradation factors. In fact, studies show that exposure of epoxy adhesive to moisture and water diffusion can significantly deteriorate its properties [15, 16]. Exposure to temperatures closer to the adhesive's glass transition temperature (T_g) can lead to deterioration of its properties [18] through softening of the polymeric matrix viscoelasticity; moreover, a synergy between moisture and temperature may lead to higher degradation effects [19]. However, elevated temperatures will positively lead to improved adhesive properties (e.g. higher strength and stiffness) thanks to post-curing phenomenon [20]; wet-dry cycles can reduce both the adhesive tensile strength and elastic modulus [21], while exposure to chlorides does not lead to detrimental effect [22]. Furthermore, a study by [23] reported that high carbonation can increase the curing of epoxy resin.

Regarding the durability of CFRP laminate, existing studies show that CFRP is immune against chloride exposure [18], and thermal cycles [24] although microcracking of the fiber matrix can develop as a result of different coefficient of thermal expansion (CTE) between the fiber and matrix [18]. Besides, freeze-thaw (FT) cycles can decrease both tensile strength and elongation of CFRP [25], moisture can affect the properties, particularly at fiber-matrix level [26], UV may also affect a few microns from the CFRP surface [26]. Additionally, in outdoor environment, UV is combined with effects of temperature, moisture, and FT, among others [18]. Hence, more

studies are needed to be able to better understand the behavior of CFRP laminate, particularly in outdoor exposure.

The durability of EBR CFRP-to-concrete bond has also been investigated using accelerated aging test (AAT) protocols, e.g., [24,27–34]. According to these studies, 20 FT [30], 50, 150, and 300 FT [27], and 100 and 200 FT (for specimens with 400 mm bond length) [29] increased the bond strength; however, a reduction of bond strength was observed after 300FT [32] and 30, 60, 90 FT in [34]. Furthermore, water immersion for durations of 1000 and 3000 h at 22 and 38°C [30], 2880 h [32] increased the bond strength, while immersion for 330-1340 h [33] and 10000h at 22 °C [30] decreased the strength. Additionally, elevated temperatures of 50 °C [28] increased the bond strength but 60 °C for 1000 h and 3000 h [30] decreased it; and thermal cycles between -18 and 30 °C for 2160 h in air increased the bond strength, while those in water decreased it [24]. Hygrothermal at 40 °C and 95 % RH for 4200 h increased the bond strength, whereas a decrease was observed using the same conditioning but for up to 8472 h [31]. Overall, a significant variability of the results from these studies is evident.

From the above literature, some knowledge gaps still exist: (i) most of existing studies were conducted under accelerated aging test (AAT) protocols, while in reality strengthened structures are exposed to natural aging; ii) there is a lack of studies under natural aging test (NAT) protocols and the correlation between the AAT and NAT; (iii) environmental conversion factors (ECF) recommended in existing standards are based on studies solely performed under AAT; (iv) the recommended ECF ideally represents all degradation agents that are thought to promote aging; however, in real applications, a structure is exposed to a combination of certain agents (not a single one) depending on its location. In this regard, the main objective of this work was to try to address the above gaps by investigating the bond performance in concrete elements strengthened according to EBR-CFRP technique for the case when the bond is exposed to different accelerated (laboratory-controlled) and natural (carbonation, high temperatures, freeze-thaw attacks, and airborne chlorides) aging environments. Besides, the ECF for the bond strength from the existing AAT data were derived, and some comparative studies between the ECF from existing AAT data and NAT data from the present work were performed.

2. EXPERIMENTAL PROGRAM

This experimental campaign focused on the assessment of the EBR CFRP-to-concrete bond durability over time. In addition, specimens of the bond constituent materials (concrete, epoxy adhesive, and CFRP laminate) were also tested to have an advanced understanding of their effects on the bond behavior.

2.1. Properties of the tested specimens

The properties of concrete, epoxy adhesive, and CFRP laminate are first described followed by those of the EBR CFRP-to-concrete bond.

Concrete: the concrete used to prepare all specimens for determining the concrete compressive strength (f_{cc}), elastic modulus (E_c), pull-off strength (f_{ct}), and carbonation depth (C_a), was cast using the same batch as that of the single lap shear tests specimens. The concrete properties were as shown in **Table II.1**. Concrete cylinders (**Figure II.1a**) were produced to determine the E_c and f_{cc} properties. Concrete prisms (**Figure II.1b**) were used to assess the f_{ct} , the C_a , as well as the bond strength.

Epoxy adhesive: a two-component commercial cold-curing epoxy resin-based adhesive with properties shown in **Table II.1** was used to cast dog-bone shaped specimens (**Figure II.1c**) for determining the adhesive tensile strength (f_a) and elastic modulus (E_a). This adhesive is a solvent free, thixotropic, in grey colour, developed for bonding CFRP laminate to concrete.

CFRP laminate: the CFRP laminate (**Figure II.1d**) with properties as shown in **Table II.1** was used to assess its tensile strength (f_l) and elastic modulus (E_l) over time, and as a bond constituent material for strengthening purpose.

EBR CFRP-to-concrete bond: The EBR CFRP-to-concrete bond had properties as in **Table II.1**.

Table II.1: Materials and EBR CFRP-to-concrete bond properties.

Concrete		Epoxy adhesive	
Specimen type	Cylinder with D×H:150×300 [mm ²]	Type of adhesive	Cold-curing S&P Resin 220
Max aggregate size [mm]	12.5	Flexural elastic modulus [GPa]	>7.1
Cement type	CEM II/A-L 42.5R	Tensile strength [MPa]	19.9 (after 7d curing at 20 °C)
Slump [mm]	160-210 (slump class S4)	Glass transition temperature (T_g) [°C]	46.2 (after 7d curing at 23°C)
Concrete class	30/37 MPa (C30/37)	Density, at 23 °C [g/cm ³]	1.7 - 1.8
Water-to-cement ratio	0.4	Compressive strength [MPa]	>70
f_c [MPa], E [GPa]	41.5, 29.1 (after 28 days)	Shear strength [MPa]	>26
Exposure class	XC4(P)	BS by pull-off, on concrete [MPa]	3 (after 3d curing at 20 °C)
CFRP laminate		EBR CFRP-to-concrete Bond	
Type and trademark	S&P clever (CFK 150/2000)	CFRP cross-section [mm ²]	50×1.2
Prefabricated by	Pultrusion	Concrete prism dimensions [mm ³]	400×200×200
Fiber orientation	Unidirectional	Bond length [mm]	220
Fiber content [%]	68	Effective bond length [mm]	101
Fiber matrix	Vinyl ester resin (with $T_g \approx 23$ °C)	Number of laminates per specimen	2
External surface	Black, smooth	Epoxy adhesive thickness [mm]	1.5 - 2.0
Elastic modulus [GPa]	>170	Type of test	Single-lap shear test
Tensile strength [MPa]	>2000		

Notes: f_c : compressive strength (average value), E : elastic modulus (average value), BS: bond strength, D: diameter, H: height, T_g : glass transition temperature, d: days.

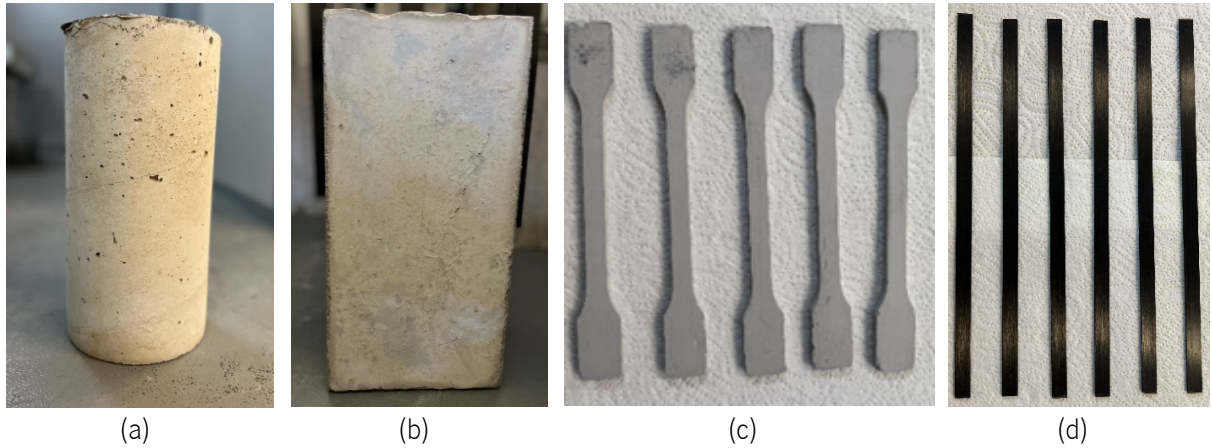


Figure II. 1: Materials specimens used: (a) concrete cylinder, (b) concrete prism, (c) epoxy adhesive dog-bone shaped specimens and (d) CFRP laminates.

2.2. Description of experimental stations

Six different environmental exposures (E1-E6) were used to promote degradation of both the EBR CFRP-to-concrete bond and its constituent materials. Different tests were performed on the specimens at initial time (denoted as T0) and on the specimens collected from the aforementioned environments after 1 year (T1), 2 years (T2), 3 years (T3) and 4 years (T4) of exposure. A description of the conditioning in each environment is provided as follows.

2.2.1. Laboratory accelerated conditioning

Two laboratory-controlled environments were considered. The first environment (denoted as E1) had the specimens conditioned at 20 °C /55 % RH. The second environment (E2), see **Figure II.2a**, contained the specimens fully and continuously immersed in tap water at 20 °C. More detailed information can be found in [35].

2.2.2. Real-time field conditioning

Four different outdoor environmental exposures were selected with the aim to mainly promote degradation of the EBR CFRP-to-concrete bond and its constituent materials. The first outdoor environment (denoted as E3) was selected to promote the ingress of atmospheric carbon dioxide (CO_2) through the bond constituent materials, also known as *carbonation*. The second environment (E4) was selected to mainly promote the freeze-thaw attacks. The third environment (E5), see **Figure II.2b**, was selected to mainly promote degradation due to outdoor high temperatures. The fourth outdoor environment (E6) was selected to mainly promote the ingress of airborne chlorides (through the bond constituent materials) from the Atlantic Ocean, and high levels of humidity.

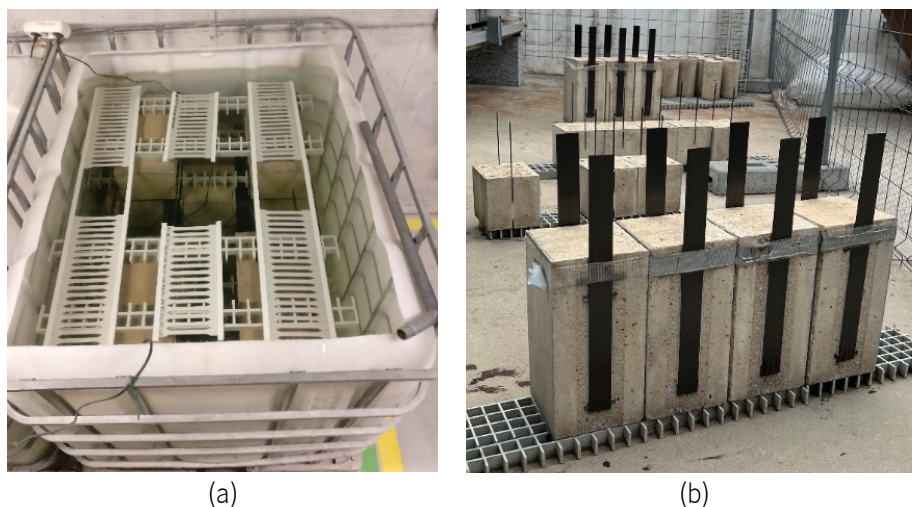


Figure II.2: Examples of environments considered: (a) laboratory-controlled (E2), and (b) outdoor conditioning (E5).

It is worth noting that despite the target of conditioning the specimens with a single degradation agent, it is inevitable for the specimens in outdoor environments to experience more than one degradation agent. Hence there was a combination of degradation agents in each outdoor environment, e.g., in E3, there was both carbonation and elevated temperatures as dominant degradation agents, while in E4 both freeze-thaws and moisture were dominant.

The yearly recorded temperature and relative humidity (RH) are shown in **Table II.2**, for all studied environments. It can be noted that the average of the yearly maximum temperatures in both E3 and E5 were the highest when compared to other outdoor environments. For example, in the first year (T1), the averages of the yearly maximum temperatures were 23.1 °C and 24.0 °C, and the yearly peak maximum values reached 46.2 °C and 44.6 °C, in E3 and E5, respectively. Furthermore, the RH in E6 was the highest for outdoor environments, followed by E3 and E4, respectively; hence confirming the significance effect of both temperature and relative humidity on the properties of the specimens in these environments.

Additionally, typical records of seasonal waves of temperature and relative humidity in both outdoor and laboratory conditioning are shown in **Figure II.3** for up to 4 years.

Table II.2: Variation of yearly temperatures and relative humidity in the studied environments.

Variable / Environment		E1	E2	E3	E4	E5	E6
Temp [°C] Year 1 (T1)	Max	20.2 (22.5)	20.2 (20.3)	23.1 (46.2)	17.8 (32.4)	24.0 (44.6)	21.6 (36.0)
	Min	19.8 (13.5)	20.1 (19.0)	13.4 (3.3)	11.1 (-4.7)	11.1 (-1.9)	11.8 (1.5)
	Avg	19.9	20.0	17.4	14.3	17.4	16.15
RH [%] Year 1 (T1)	Max	62.8 (79.5)	100.0	87.9 (100.0)	83.3 (100.0)	84.9 (100.0)	88.7 (100.0)
	Min	57.8 (31.0)	100.0	48.8 (11.0)	50.9 (4.0)	40.3 (9.0)	59.8(18.0)
	Avg	60.0	100.0	70.7	67.2	63.5	76.1
Temp [°C] Year 2 (T2)	Max	20.4 (22.0)	20.2 (20.4)	22.7 (39.7)	14.0 (29.6)	23.6 (39.9)	22.5 (39.5)
	Min	20.1 (19.0)	20.5 (19.5)	13.9 (4.3)	7.4 (-4.6)	8.9 (0.3)	13.0 (2.0)
	Avg	20.2	20.6	17.5	10.6	17.3	17.2
RH [%] Year 2 (T2)	Max	63.1 (77.5)	100.0	90.1 (100.0)	87.6 (100.0)	87.9 (100.0)	90.5 (100.0)
	Min	60.1 (41.5)	100.0	52.9 (16.0)	54.7 (4.0)	42.0 (11.0)	63.5 (28.5)
	Avg	61.8	100.0	74.4	72.8	67.1	78.6
Temp [°C] Year 3(T3)	Max	20.1 (22.0)	21.0 (22.0)	22.8 (40.7)	13.2 (29.5)	22.8 (41.4)	16.9 (31.1)
	Min	19.8 (17.0)	20.2 (19.6)	13.5 (1.4)	4.0 (-6.4)	11.4 (-4.1)	9.1 (-1.1)
	Avg	19.9	20.3	17.3	9.7	17.1	13.0
RH [%] Year 3 (T3)	Max	63.9 (77.5)	100.0	87.9 (100.0)	86.0 (100.0)	86.2 (100.0)	93.0 (99.0)
	Min	61.7 (43.0)	100.0	48.8 (16.0)	50.5 (6.0)	45.6 (9.0)	61.5 (23.0)
	Avg	62.9	100.0	70.7	71.3	66.5	79.4
Temp [°C] Year 4 (T4)	Max	20.5 (23.0)	20.2 (20.3)	-	20.7 (31.1)	25.5 (46.0)	19.9 (34.0)
	Min	20.1 (17.0)	20.3 (19.5)	-	12.7 (2.0)	13.9 (2.0)	11.1 (2.0)
	Avg	20.3	20.4	-	16.5	19.3	15.1
RH [%] Year 4 (T4)	Max	62.8 (77.5)	100.0	-	83.3 (100.0)	84.9 (88.5)	88.7 (100.0)
	Min	57.8 (47.0)	100.0	-	50.9 (4.0)	40.3 (10.0)	59.8 (22.0)
	Avg	60.0	100.0	-	67.2	63.4	76.1

Notes: RH: Relative humidity, Temp: Temperature, Max/Min/Avg: yearly maximum/ minimum/average value, the value in parentheses stands for the yearly peak value.

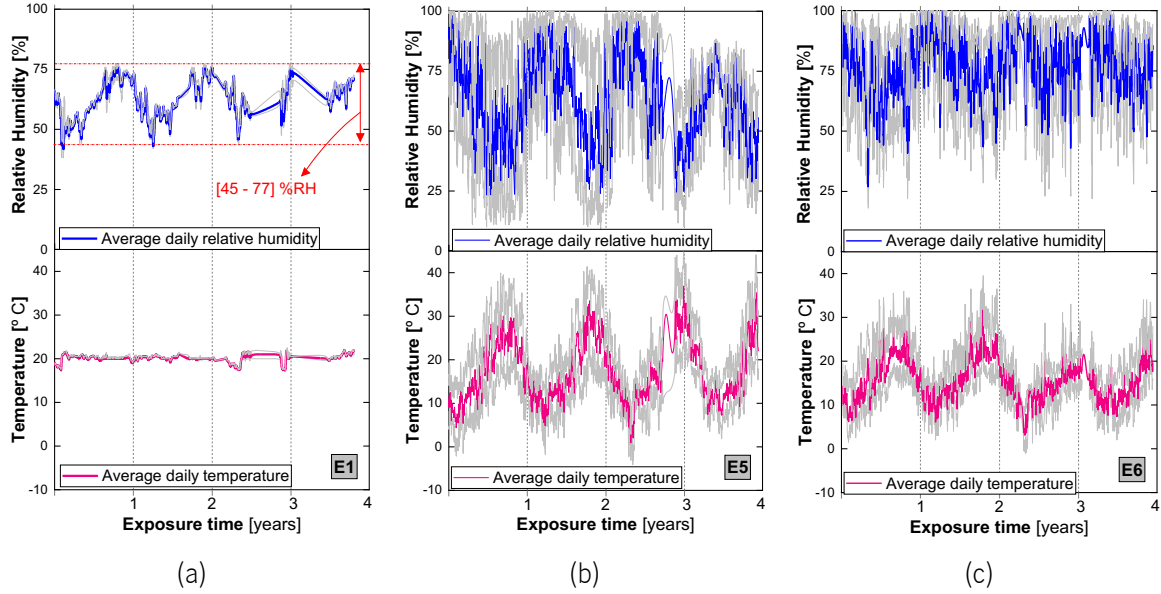


Figure II.3: Typical meteorologic records collected from (a) laboratory environment (E1) and from (b, c) outdoor environments (E5 and E6).

It can be noted that, in E1, despite the aim to condition the relative humidity at 55 %, it varied in the range [45 - 77 % RH], which can favour carbonation depth increase according to existing studies [8,11]. Besides, E5 shows increasing temperature and decreasing RH trends, while E6 shows decreasing temperature and almost constant RH trends over time. Also, higher temperatures can be noted in E5 than in E6. This shows differences in different outdoor environments.

2.3. Testing methods

2.3.1. Compression, tensile and carbonation tests of concrete

Concrete cylinders were tested to examine the E_c and the f_{cc} variation over time, using a universal testing machine (UTS) with a maximum load capacity of 2000 kN. During a non-destructive test (**Figure II.4a**), three LVDTs spaced at 120° were installed on the specimen to determine the E_c as per NP EN 12390-13:2013 [36]. The destructive compression test (**Figure II.4b**) was performed on the same specimens to determine the f_{cc} as per NP EN 12390-3:2011 [37]. A total of 90 specimens were tested for determining the E_c and the f_{cc} variations for up to 4 years. The pull-off tests were manually performed on specimens (**Figure II.4c**) using the DYNA Z5 testing machine (**Figure II.4d**) to determine the f_{ct} as per EN1542:1999 [38]. A total of 104 tests were performed to

determine the variation for up to 4 years. On the other hand, carbonation tests were also performed on 52 specimens using phenolphthalein indicator to examine the evolution of the C_0 over time.

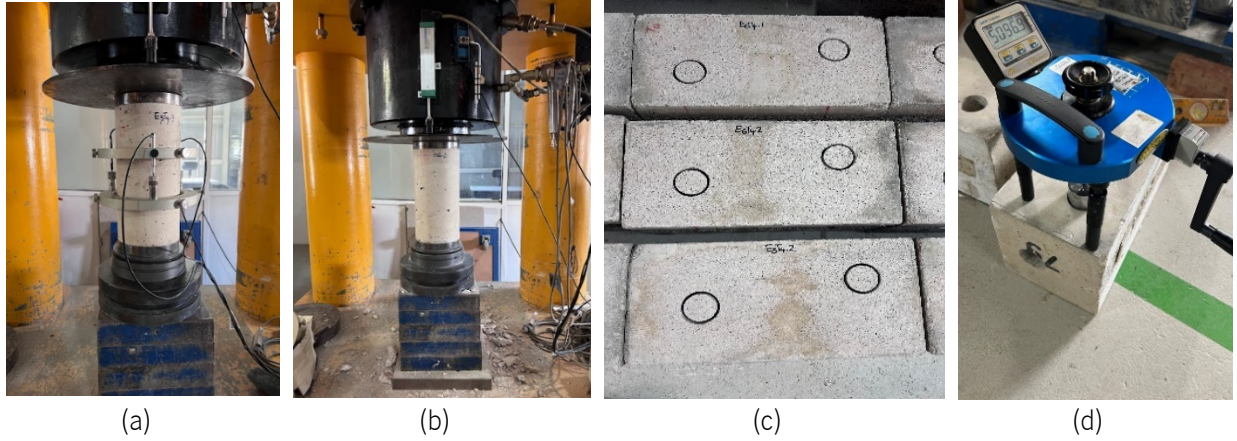


Figure II.4: Tests for characterization of concrete: (a) elastic modulus, (b) compression, and (c) sandblasting the concrete surface and creating circular grooves for bonding the metal dollies, and (d) concrete pull-off test setup.

2.3.2. Tensile tests of epoxy adhesives and CFRP laminates

The MTS UTS machine (**Figure II.5a**) was used for testing both epoxy adhesives and CFRP laminates. This MTS is additionally equipped with DIC (**Figure II.5b**) that helped in visualizing real-time strain evolution.

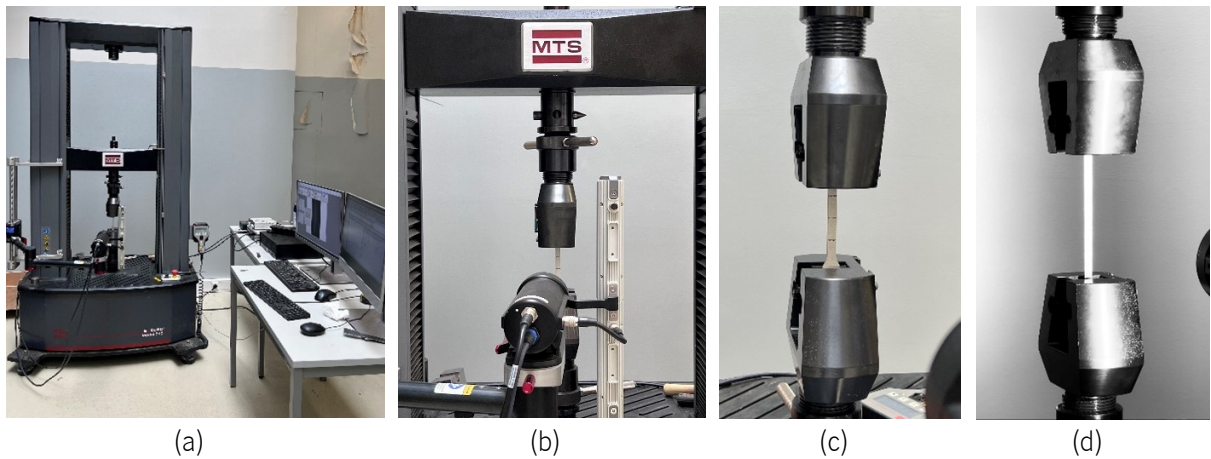


Figure II.5: Tensile tests using MTS UTS: (a) overview of the MTS equipment, (b) DIC camera used in the tests, (c) an epoxy adhesive specimen, (d) CFRP laminate specimen.

Direct tensile tests of 125 epoxy adhesive specimens (**Figure II.5c**) were performed as per EN ISO 527-2:2012 standard [39] to determine the f_s . The E_s was determined as per EN ISO 527-2:2012 [39] by calculating the slope of the secant line on the stress-strain curve from 0.05 % to 0.25 % of the strains. Similarly, testing of 150

CFRP laminate specimens (Figure II.5d) was carried out as per ISO 527-5:2009 [40] standards to determine both the f_t and E_t .

2.3.3. Single lap shear tests

A total of 50 concrete prisms each with 400×200×200 [mm] was strengthened according to EBR technique using 2 CFRP laminates, each with a cross-section of 50×1.2 [mm], being applied to two opposing sides (each in the side parallel to the casting direction) of each specimen. A bond length of 220 mm was adopted for each specimen, with 100 mm free from the extremity of the concrete prism to prevent the unwanted premature failure by concrete rip-off ahead of the loaded end. The used bond length was higher than the theoretical effective length, l_e , of 101 mm, as per [41]. The geometry and dimensioning of the EBR specimens and its test set-up are shown in (Figure II.6a), followed by a picture of the test setup (Figure II.6b), where further details are given as follows: i) the horizontal and vertical restraints, and the fixation of the LVDTs supports (Figure II.6c), ii) mounting of LVDTs (Figure II.6d) and, iii) checking of the alignment and flatness of the specimen and the clamp (Figure II.6e).

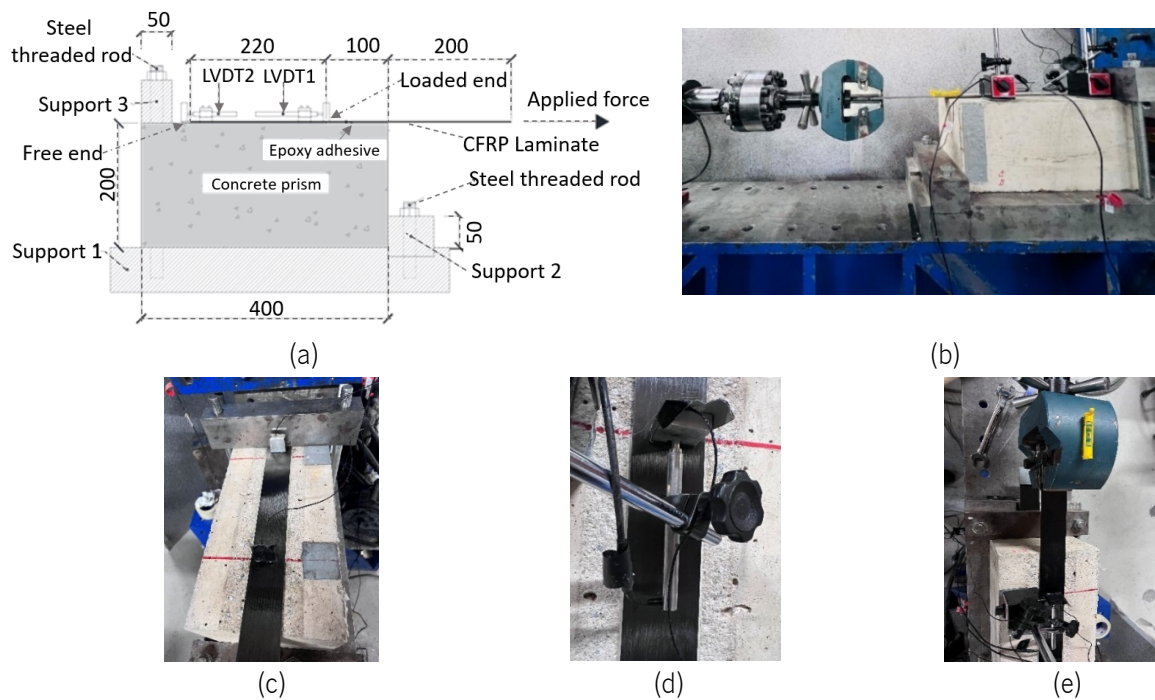


Figure II.6: Single-lap shear tests: (a) specimen's geometry and test set-up, (b) photograph of the full test configuration, (c) mounting LVDTs supports, (d) mounting LVDTs and (e) verification of the clamp and CFRP alignment. Notes: all units in [mm].

A total of 100 direct single-lap shear tests were performed using a servo-controlled equipment. The applied force was measured through a load cell of 200 kN maximum load carrying capacity (linearity error of 0.05 % F.S.) placed between the actuator and the grip (used to pull the CFRP laminate during the test, see **Figure II.6b**).

The tests were performed under displacement control at the loaded end with a rate of 2 $\mu\text{m/s}$, the displacement being measured through LVDT placed at the loaded end section (LVDT1 in **Figure II.6a**) as a control variable. The displacement at the free end was also recorded using the LVDT denoted as LVDT2 in **Figure II.6a**.

3. RESULTS AND DISCUSSION

The results from the tests of bond constituent materials (concrete, epoxy adhesive, and CFRP laminate) are first presented and discussed, followed by those from the single-lap shear test performed on the specimens strengthened with EBR technique.

3.1. EBR CFRP-to-concrete bond constituent materials

The results from concrete, epoxy adhesive and CFRP laminate testing are presented and discussed.

3.1.1. Concrete

The results from the tests conducted on the concrete are shown in **Figure II.7a** and **Table II.3**. The concrete carbonation depth (C_d) increased linearly with time in E1 mainly because of the relative humidity range [50-70 % RH] known to favor the ingress of CO_2 [42], while in E3 and E4 it mainly decreased over time. In general, trends with lower values of C_d can be noted from E2 and E4. This agrees with the findings by [43] where an increased humidity, rain, and snow was reported to slow the ingress of CO_2 after exposing concrete to outdoor environments for up to 4 years. In E2 and E4 the specimens were partially carbonated (**Figure II.8a**), hence having negligible effects on the concrete properties [44,45]. The highest C_d was observed in E3 as expected. However, even in other outdoor environments (except E4) the C_d was observed to be comparable to that in E3, which reflects the abundance of CO_2 everywhere in the atmosphere; the highest average values of C_d of 9.5 mm and 8.5 mm were found in E3 and E5, respectively.

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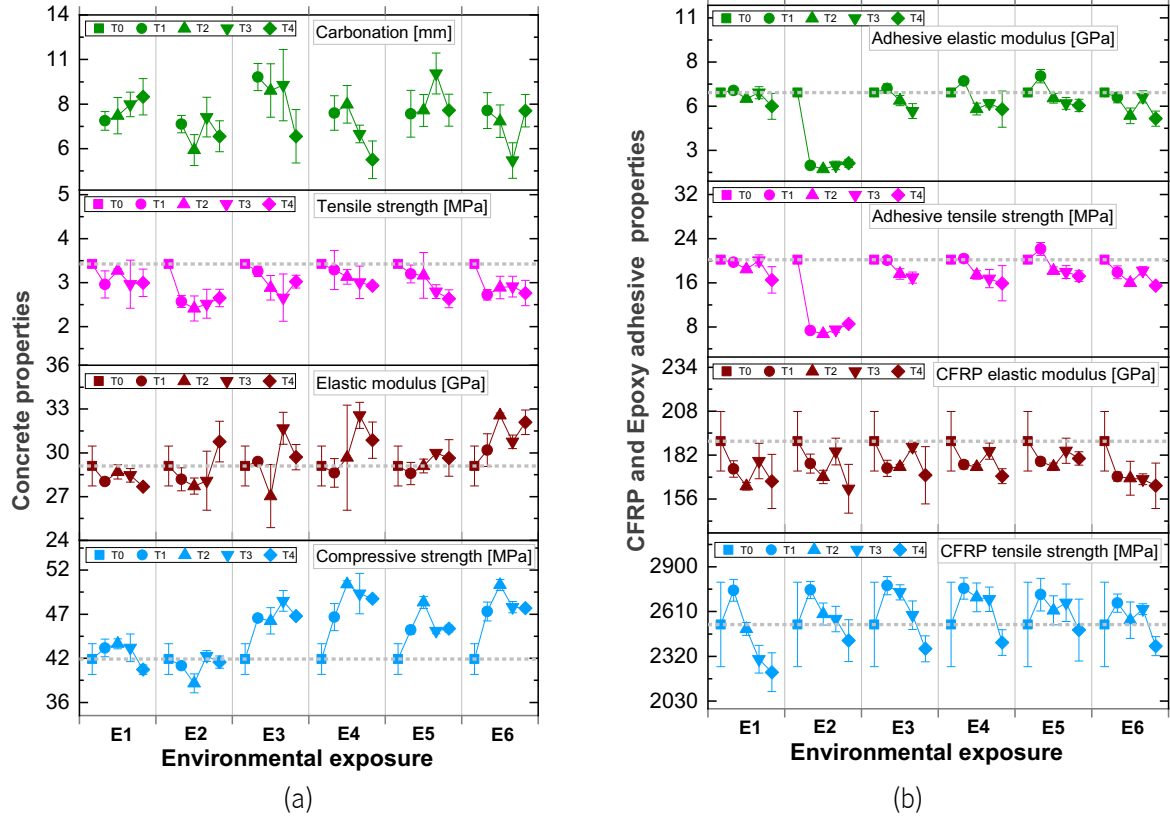


Figure II.7: Variation of the bond constituent material properties with time under different environmental exposures: (a) concrete, and (b) epoxy adhesive and CFRP laminate

The concrete compressive strength (f_{cc}) increased in all outdoor environments as compared to the value recorded at initial time T0. In E1, the f_{cc} increased from T0 to T2 by 4.3 % and decreased from T3 to T4 by 6.2 %. In E2, the f_{cc} decreased from T0 to T2 by 7.2 % followed by an increase from T2 to T3 by 8.3 % and levelled off thereafter. The observed increase can be thought to have resulted from the continuation of cement hydration with time, which increased the number of C-S-H silicates that occupied the empty voids, thereby reducing the porosity and improving the strength. The increase in the C-S-H silicates is already known to improve the f_{cc} [46,47]. Furthermore, since carbonation increases the f_{cc} [12,15], this can confirm the increased f_{cc} in E3 as a result of high carbonation. Also, increased curing humidity was found to increase the f_{cc} and E_c by [14]. Hence, the increased f_{cc} in E4 can be attributed to the effect of freeze-thaws that promoted the continuation of cement hydration, which is in line with the findings by [48].

Table II.3: Average values of concrete compressive strength, elastic modulus, splitting tensile strength, and carbonation depth before exposure (T0), and after 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of different exposures (E1 to E6).

	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
Environment	Compressive strength f_c [MPa]					Elastic modulus E_c [GPa]				
REF	41.5 (4.4)	-	-	-	-	29.1 (5.0)	-	-	-	-
E1	-	42.8 (2.4)	43.3 (1.4)	42.8 (3.8)	40.3 (1.4)	-	28.0 (0.7)	28.7 (1.7)	28.5 (1.6)	27.7 (0.4)
E2	-	40.7 (0.7)	38.7 (2.9)	41.9 (1.5)	41.1 (1.8)	-	28.2 (2.8)	27.7 (2.0)	28.1 (7.2)	30.8 (4.5)
E3	-	46.3 (0.9)	46.0 (3.4)	48.4 (2.5)	46.6 (0.6)	-	29.4 (0.8)	27.0 (8.0)	31.7 (3.4)	29.7 (2.9)
E4	-	46.5 (3.4)	50.3 (0.8)	49.2 (4.8)	48.6 (0.7)	-	28.6 (3.4)	29.7 (12.1)	32.6 (2.7)	30.9 (4.0)
E5	-	44.9 (1.0)	48.2 (1.5)	44.8 (0.7)	45.1 (0.9)	-	28.6 (2.6)	29.1 (1.6)	30.0 (0.3)	29.6 (4.2)
E6	-	47.1 (2.4)	50.2 (1.3)	47.6 (1.4)	47.5 (0.5)	-	30.2 (3.7)	32.6 (0.5)	30.8 (1.5)	32.1 (2.6)
	Pull-off strength f_t [MPa]					Carbonation depth C_a [mm]				
REF	3.4 (13.3)	-	-	-	-	0.0	-	-	-	-
E1	-	2.9 (10.4)	3.2 (1.3)	2.9 (18.4)	2.9 (10.5)	-	7.4 (19.9)	7.7 (14.8)	8.4 (9.2)	8.9 (12.8)
E2	-	2.5 (5.3)	2.3 (11.8)	2.5 (13.1)	2.6 (7.4)	-	7.2 (15.2)	5.5 (17.7)	7.6 (16.4)	6.4 (15.3)
E3	-	3.2 (3.6)	2.8 (9.7)	2.6 (20.1)	3.0 (4.7)	-	10.1 (5.5)	9.3 (18.2)	9.6 (23.4)	9.0 (18.8)
E4	-	3.2 (13.5)	3.1 (5.3)	2.9 (12.2)	2.9 (0.1)	-	7.8 (7.5)	8.4 (14.1)	6.5 (8.4)	4.9 (24.1)
E5	-	3.1 (6.3)	3.1 (16.4)	2.7 (5.7)	2.6 (7.7)	-	7.8 (10.9)	8.0 (12.5)	10.3 (12.4)	8.0 (12.5)
E6	-	2.7 (4.4)	2.8 (8.8)	2.9 (8.1)	2.7 (10.5)	-	8.0 (14.1)	7.3 (13.9)	4.9 (22.9)	8.0 (12.8)

Notes: all values in parentheses are *coefficients of variation*, REF: Reference values from the specimens tested at T0.

Briefly, the continuation of cement hydration in E4 and E6 with the presence of high outdoor humidity can be thought to have been the main governing factor of the increased concrete f_{cc} and E_c . Furthermore, the observed increase in E6 may further be attributed to the synergic effect between carbonation and chlorides. In E5, elevated temperatures as previously shown in **Figure II.3** led to increased f_{cc} , which agrees with a study by [49], also, elevated temperatures have been found to make concrete more porous thereby facilitating the CO₂ penetration [8–10]. Since there was a high C_a in E5 (**Figure II.7a**), it can be inferred that the increased f_{cc} resulted from a synergic effect between elevated temperatures and carbonation. On the other hand, the f_t showed a decreasing trend over time. It is worth noting that pull-off tests are also usually used to characterize FRP-to-concrete systems; despite that, they may not accurately represent the mechanisms occurring at the concrete near-surface region in FRP strengthened RC structural elements because the failure of the specimens occurred at the inner region

of concrete rather than at the interface region. However, in the present case, pull-off tests were only used to assess the tensile properties of the concrete near-surface region. Overall, the highest increases in outdoor environments were approximately 21 %, 16 %, 21 %, and 17 % in the f_{cc} , and 9 %, 12 %, 3 %, and 12 % in E_c for E3, E4, E5, E6, respectively.

Typical FMs for both concrete compression and pull-off tests are shown in **Figure II.8b** and **Figure II.8d**, respectively. The FM from the compression tests are as per NP EN 12390-3:2011 [37]. Furthermore, the FM is of cohesive failure in deeper regions of the concrete, from which it can be inferred that the pull-off force at such a failure may not be the actual representation of that leading to failure at concrete near-surface region (i.e., the region of interest, as it is where the CFRP is bonded).

3.1.2. Epoxy adhesive

The results from the tensile test of the epoxy adhesive are shown in **Figure II.7b** and **Table II.4**. Referring to **Figure II.7b**, and **Table II.4**, marginal changes in both adhesive elastic modulus (E_a) and its strength (f_a) can be noted in E1. Contrary, there was a substantial decrease in both E_a and f_a in E2. This decrease can be attributed to the effects of plasticization. Furthermore, a general progressive decrease in E_a during the later years can be noted in all environments. A similar trend for f_a and E_a can be seen in all environments, which shows that predictive models considering both the f_a and E_a can reasonably be developed. The highest decreases in E_a and f_a were 75.4 % at T2 and 66.3 % in E2 at T2, respectively, as compared to the value at initial stage (T0).

The justifications for the observed reductions in the adhesive properties can be as follows: (i) the observed marginal variations in E1 resulted from carbonation that may have caused the epoxy resin to cure faster [23]; (ii) water ingress in E2 and moisture effects in both E4 and E6 were the main degradation agents, as also found by [17,18]; (iii) In E3, synergic effects between elevated temperatures, carbonation, and UV radiations can be thought to be the main agents; iv) synergic effects between elevated temperatures and carbonation; and iv) a slight decrease observed in E6 can be attributed to moisture effects mainly because chlorides effects on epoxy adhesive are negligible [22]. The adhesive failure modes (**Figure II.8e**) were of an abrupt break in the region of the tensioned adhesive part (**Figure II.8f**). The adhesive tensile strains are also shown in **Table II.4**. Marginal variations with time can be noted, except in E2 where the specimens showed a substantial increase during T1 followed by a level-off in the later years.

Table II.4. Average values of epoxy adhesive and CFRP tensile strength, elastic modulus, maximum tensile strain before exposure (T0), and after 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of different environmental exposures (E1 to E6).

	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
Environment	Epoxy adhesive: Tensile strength [MPa]					CFRP laminate: Tensile strength [MPa]				
REF	19.9 (3.0)	-	-	-	-	2527 (11)	-	-	-	-
E1	-	19.5 (1.8)	18.2 (2.8)	19.8 (4.9)	16.3 (14.6)	-	2748 (2.6)	2497 (1.7)	2302 (3.9)	2217 (5.7)
E2	-	7.2 (3.1)	6.7 (2.7)	7.4 (7.1)	8.4 (3.6)	-	2750 (2.0)	2594 (2.8)	2562 (3.2)	2422 (5.6)
E3	-	19.9 (3.1)	17.4 (5.3)	16.7 (5.9)	-	-	2778 (2.1)	2735 (1.8)	2587 (3.6)	2369 (3.6)
E4	-	20.1 (3.4)	17.2 (4.3)	16.5 (9.8)	15.7 (20.1)	-	2760 (2.5)	2703 (3.4)	2690 (2.9)	2409 (3.5)
E5	-	21.9 (5.2)	18.0 (3.6)	17.7 (6.5)	17.0 (5.8)	-	2720 (3.9)	2618 (3.6)	2667 (4.6)	2491 (8.1)
E6	-	17.7 (6.4)	15.8 (4.3)	18.0 (4.2)	15.3 (2.6)	-	2665 (2.2)	2554 (4.6)	2626 (1.3)	2386 (2.6)
	Epoxy adhesive: Elastic modulus [GPa]					CFRP laminate: Elastic modulus [GPa]				
REF	6.5 (3.0)	-	-	-	-	190 (9.3)	-	-	-	-
E1	-	6.6 (1.3)	6.1 (1.4)	6.5 (6.0)	5.6 (14.6)	-	174 (2.8)	164 (1.3)	179 (5.9)	166 (9.7)
E2	-	1.9 (5.1)	1.6 (4.0)	1.9 (14.6)	2.0 (12.4)	-	177 (3.2)	169 (2.3)	184 (4.3)	162 (8.9)
E3	-	6.7 (4.4)	6.0 (5.4)	5.3 (8.9)	-	-	174 (3.6)	175 (0.9)	187 (1.5)	170 (10.0)
E4	-	7.2 (1.4)	5.4 (6.9)	5.8 (4.0)	5.4 (21.1)	-	176 (1.5)	175 (0.8)	184 (2.6)	170 (2.6)
E5	-	7.5 (5.7)	6.1 (5.0)	5.8 (6.5)	5.7 (7.0)	-	178 (1.2)	175 (1.3)	185 (4.1)	180 (2.2)
E6	-	6.2 (5.4)	5.0 (10.0)	6.2 (6.2)	4.8 (10.0)	-	169 (1.6)	168 (6.0)	168 (1.9)	164 (8.2)
	Epoxy adhesive: Tensile strain [%]					CFRP laminate: Maximum tensile strain [$\times 10^{-3}$]				
REF	0.4 (6.2)	-	-	-	-	13.3 (13.6)	-	-	-	-
E1	-	0.4 (13.0)	0.3 (11.7)	0.3 (8.8)	0.4 (11.3)	-	15.8 (3.6)	15.3 (1.9)	12.9 (6.3)	13.7 (5.5)
E2	-	1.1 (21.4)	1.1 (11.9)	1.0 (25.5)	0.9 (15.3)	-	15.6 (3.9)	15.4 (3.0)	13.9 (3.9)	14.6 (2.6)
E3	-	0.3 (11.1)	0.3 (19.1)	0.3 (10.1)	-	-	16.0 (3.6)	15.6 (1.2)	13.6 (4.9)	14.7 (5.1)
E4	-	0.3 (11.3)	0.3 (12.8)	0.3 (24.6)	0.3 (24.8)	-	15.7 (2.8)	15.4 (3.3)	14.6 (1.6)	15.0 (4.4)
E5	-	0.3 (11.2)	0.4 (13.1)	0.3 (11.5)	0.4 (10.5)	-	15.3 (4.1)	14.9 (2.6)	14.5 (7.7)	15.1 (6.0)
E6	-	0.3 (4.3)	0.3 (12.9)	0.3 (17.4)	0.4 (14.2)	-	15.7 (2.6)	15.2 (6.1)	15.7 (2.6)	14.8 (8.2)

Notes: all values in parentheses express *coefficient of variation* in percent, the CFRP laminate had a 50x1.2 [mm] cross-section (during testing coupons of 15x1.2 [mm] section was used), REF: Reference values at the beginning i.e., at T0.

3.1.3. CFRP laminate

The results from the tensile test of the CFRP laminate are shown in **Figure II.7b** and **Table II.4**. The CFRP laminate tensile strength (f_t) generally improved in the first year in all environments, while its elastic modulus (E_t) showed a progressive decrease as compared to the initial value at T0. The highest f_t occurred at T1 which reflects the effects of post-curing of the fiber resin matrix, followed by a general decrease in the consecutive years. After 4 years of exposure, in comparison with the initial value (T0), the f_t showed a decrease of 12.3 %, 4.2 %, 6.3 %, 4.7 %, 1.4 %, and 5.6 % in E1 to E6, respectively, while the E_t showed a decrease of 12.6 %, 14.7 %, 10.5 %, 10.5 %, 5.3 %, and 13.7 % in E1 to E6, respectively. These variations show that E_t degradation was faster than that of the f_t . The failure modes were characterized by a quick and progressive rupture of individual fibers that started from the CFRP longitudinal edges and moved towards the center until the complete failure occurred with a massive sound (**Figure II.8g,h**).

The improved CFRP properties in E1 may have resulted from the post-curing of the resin, mainly catalyzed by the presence of carbonation that caused the epoxy resin to cure faster, as was observed in [23]. In E2, the observed marginal variations indicate that f_t was immune to water diffusion; however, its E_t was remarkably affected. Similar trend was observed in [50] after 6 months of the CFRP laminate immersion in seawater with a reduction of its elastic modulus from 248.5 GPa to 240.5 GPa, hence water diffusion through the laminate may be thought to affect the elastic modulus but with insignificant effects on the tensile strength. In E3, both carbonation and high temperatures may have led to a higher resin matrix post-curing rate during T1; however, the curing rate reduced in the later years, which led to decreasing f_t trends. In E4, the post-curing rate was also high during T1 and then slowed down thereafter which allowed the freeze-thaws to start causing fiber matrix microcracks and progressively decreasing the f_t , this agrees with the findings by [25]. In E5, a synergic effect between high temperatures and the presence of carbonation can be attributed to the observed increased f_t with marginal decrease after 4 years. Furthermore, in E6 chlorides do not affect the CFRP properties [18], hence, the post-curing rate during T1 was favored by carbonation, but the rate was outweighed by the moisture effects in the later years. Finally, the maximum CFRP strain evolution is also shown in **Table II.4**, and a general increase as compared to the value at T0 can be noted.

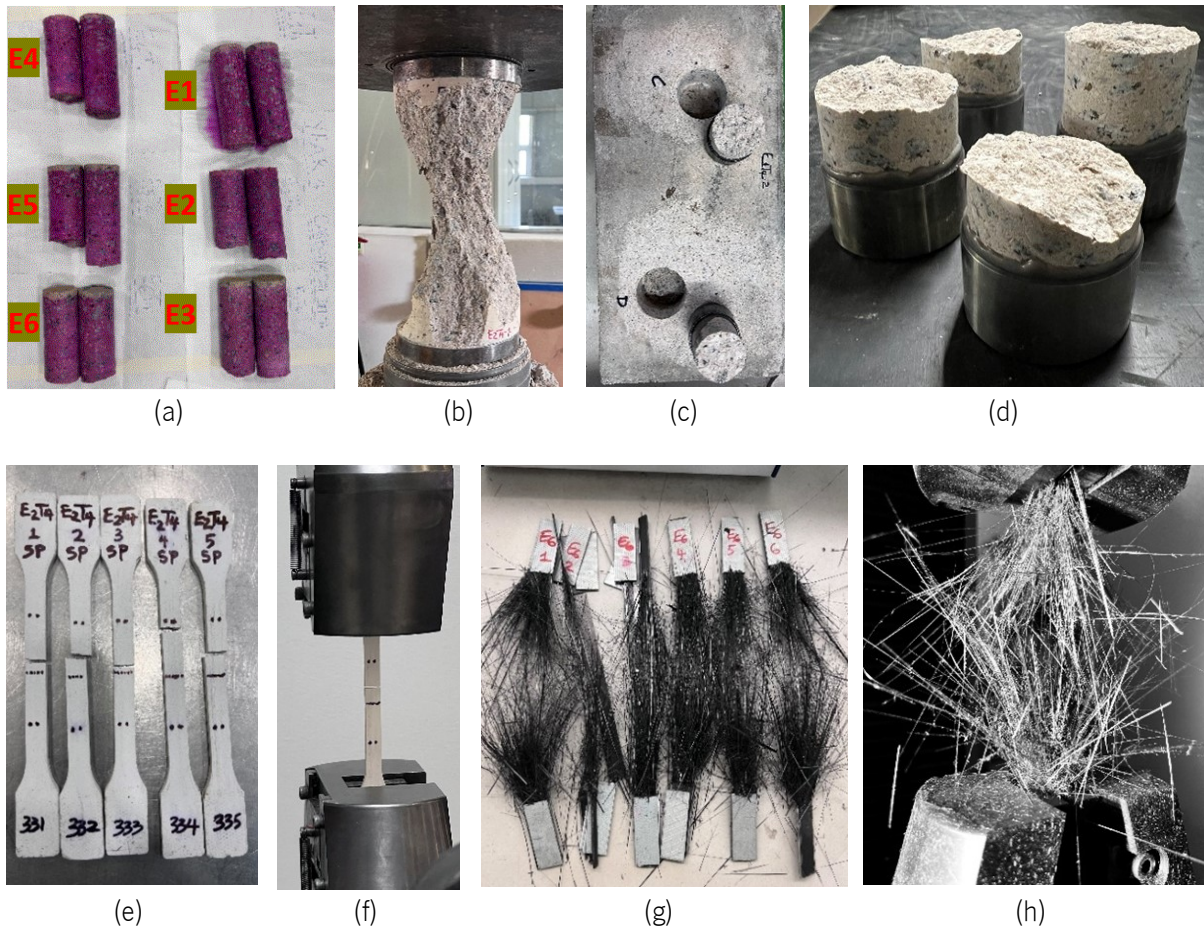


Figure II.8: Typical aspects/failure modes of some specimens after testing: (a) carbonation of concrete after T4; (b) concrete under compression, (c,d) concrete under tension (pull-off), (e,f) epoxy adhesive and (g,h) CFRP laminate under tension.

3.2. EBR CFRP-to-concrete bond

3.2.1. Pull-out force *versus* loaded end slip curves

The relationships between the pull-out force and the loaded end slip from T0 to T4 are shown in **Figure II.9a-f** for the studied environments (E1-E6), respectively. Additionally, the average maximum pull-out forces achieved during tests are shown in **Table II.5**. For more details, the maximum force-slip values and the force-slip curves for each tested specimen are shown in **ANNEX II.1** and **ANNEX II.2**, respectively. It can be noted that the pull-out force and slip changed from year to year in each environment. There was also a significant change in the stiffness that depended on the exposure time and type. Because of these changes in both the pull-out force, slip and stiffness, an attempt to establish the relationship between the variation of the maximum force and the slip

as time passes was carried out. In fact, it can be noted from **Figure II.9a-f** that after the maximum pull-out force attained its maximum there was a rapid increase of the loaded end slip with slight variation in the pull-out force, hence a tendency to form a plateau of which its length was controlled by the loaded end slip (i.e., the slip measured from the point corresponding to the maximum pull-out force, to the end or ultimate slip). However, due to change in the force from time (year) to time (year), the starting point of the above-mentioned plateau also changed thereby possessing a varying height (from the location of the minimal value of the maximum force to that of maximal value). This varying height is denoted as the *plateau height*. Similarly, the slip from the point (location) where the maximum pull-out force occurred to the end of the formed plateau also changed with time of aging, and with the plateau height, hence possessing a varying length (width) over time, denoted as *plateau width*. A comparative plot on the *plateau height* and *plateau width* is presented (see **Figure II.9g**) and more explanation is given as follows.

The *plateau height* in E2 and E6 decreased significantly as compared that at T0, followed by that in E5 and E3, which indicates that bond-slip curves' stabilization was faster in these environments as compared to that in E1 and E4. Mainly, water or chlorides ingress reduced the dispersion of the pull-out force with time, which therefore can help in developing better predictions over a long-term period. Furthermore, the lowest plateau width was found in E2 followed by E4, thereby indicating that the effective bond length was significantly affected by water immersion and freeze-thaws. The plateau width in E2 decreased by approximately 25 %, while that in E4 decreased by 19 %, as compared to that at T0. According to [51], 200 and 500 freeze-thaw cycles decreased the bond length by 3 and 9 % respectively, hence the effects of 200 and 500 freeze thaw cycles may represent 4 yearly outdoor (natural) freeze thaw attacks if multiplied by a certain matching factor. Other studies have also showed that freeze-thaw cycles will reduce the effective bond length [52]. Also, in a study by [24], the effective bond strength decreased by 1.5 % due to thermal cycles in air, which agrees with the observed decrease in the plateau width in E5. On the other hand, the maximum pull-out force (F_{max}) and its standard variation is shown in **Figure II.9h** for all environments from T0 to T4. For outdoor exposures, it can be noted that E3 and E4 showed significant variations with increasing tendency, while E5 and E6 showed marginal variations with stabilizing tendency. Furthermore, the laboratory conditioning (E1 and E2) showed insignificant variations, but with a decreasing tendency.

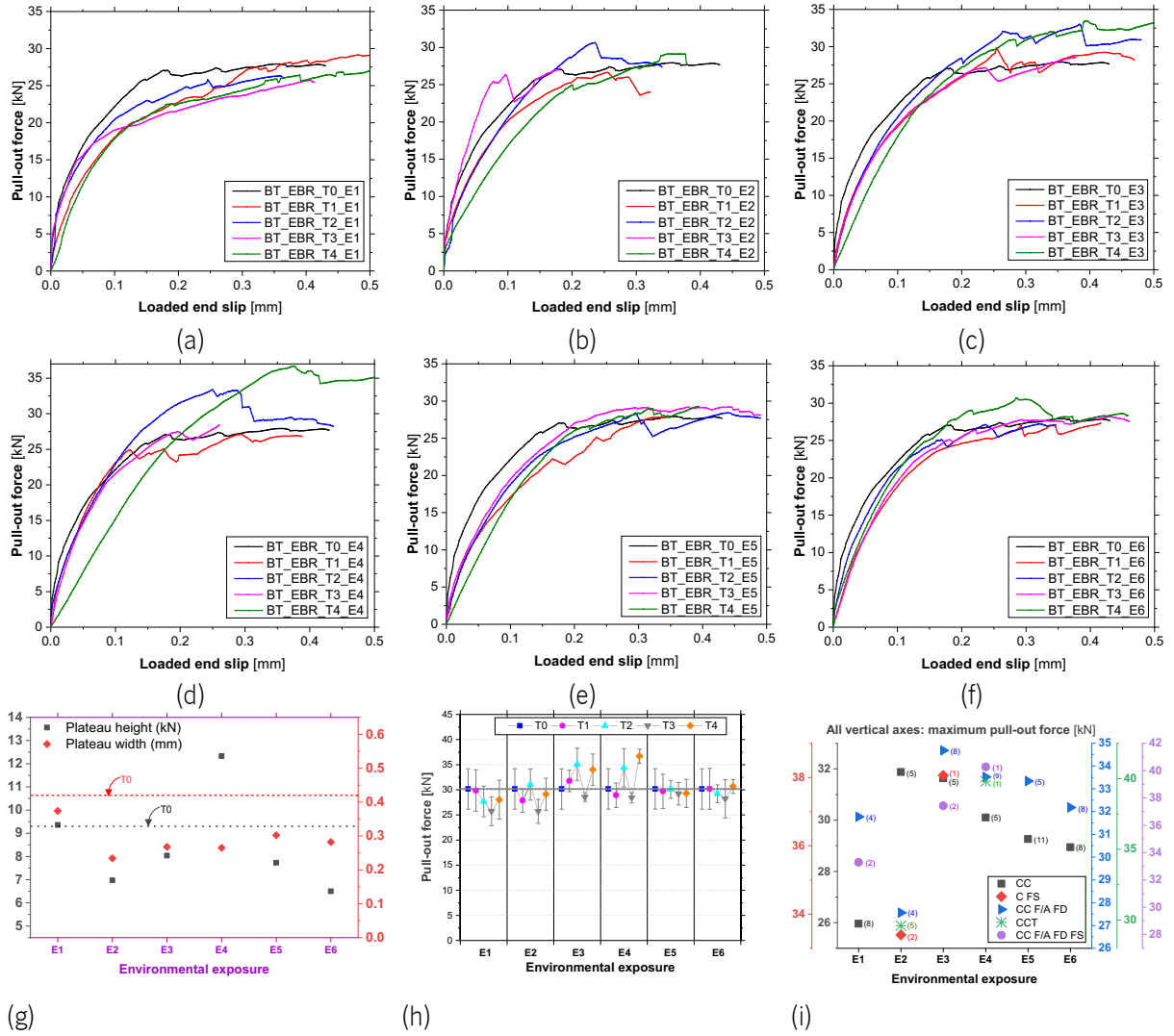


Figure II.9: EBR CFRP-to-concrete bond test results: (a-f) pull-out force versus loaded end slip, (g) plateau width and plateau height variations, (h) maximum pull-out force (F_{max}) variation, (i) failure modes variability with environment and F_{max} .

Table II.5: Maximum pull-out force and initial stiffness values for all studied environments for up to 4 years (each value is the average of four tests).

Environment	Maximum pull-out force F_{max} [kN]					Initial stiffness [kN/mm]				
	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
REF	30.2 (13.3)					299.4				
E1	-	29.88 (13.8)	27.67 (11.0)	25.74 (10.9)	28.05 (13.2)	-	252.6	284.8	278.7	228.5
E2	-	27.86 (8.4)	31.06 (10.0)	27.18 (8.6)	29.14 (9.5)	-	230.0	220.8	336.4	165.6
E3	-	31.78 (6.7)	35.10 (9.1)	28.56 (3.0)	34.02 (8.8)	-	281.4	221.0	232.2	181.3
E4	-	28.92 (8.4)	34.40 (11.1)	28.41 (3.4)	36.70 (3.7)	-	243.4	232.5	299.4	152.2
E5	-	29.71 (11.5)	30.10 (5.9)	29.24 (7.3)	29.28 (9.2)	-	270.0	191.8	192.4	172.1
E6	-	30.21 (13.5)	29.23 (6.0)	28.23 (12.4)	30.68 (4.3)	-	254.9	261.1	235.1	252.5

Notes: all values in parentheses express *coefficient of variation* in percent; REF: Reference values at the beginning i.e., at T0

3.2.2. Failure modes

Failure modes at T0: At the initial time (T0), four specimens were tested and two failed by concrete cohesive (CC) failure (**Figure II.10a,c**) while the remaining two showed a mixed failure (**Figure II.10b,d**). Two reasons can be attributed to these failures. First, the CC failure may demonstrate that the concrete was the weakest part probably because of the incomplete cement hydration, or due to its lower strength as compared to other bond constituent materials. Second, the mixed failure mode may indicate that adhesive joint was not yet fully cured with relatively low cross-linking degree.

Failure modes after exposure: It can be noted that generally either CC or a mixed failure of concrete, adhesive and CFRP occurred (**Figure II.10.e-j**). More details on these failures are provided in **Figure II.10.k-n**, where the CCT denotes the CC failure with a thick layer of concrete detached at free end region; F/A denotes the adhesive failure, FD denotes the CFRP disbond, and FS denotes the CFRP splitting/fracture. For example, a mixed failure mode denoted as “CC F/A FD” indicates that the specimen failed by the concrete cohesive failure in some bonded regions, the adhesive failure and the CFRP disbond in other regions (example of this type of failure is shown in **Figure II.10h**). The failure mode denoted as “CC F/A FD FS” indicates the mixed failure defined above for “CC F/A FD” but also with CFRP splitting (i.e., a failure like that one shown in **Figure II.10h** but with CFRP longitudinal splitting shown in **Figure II.10m** as FS).

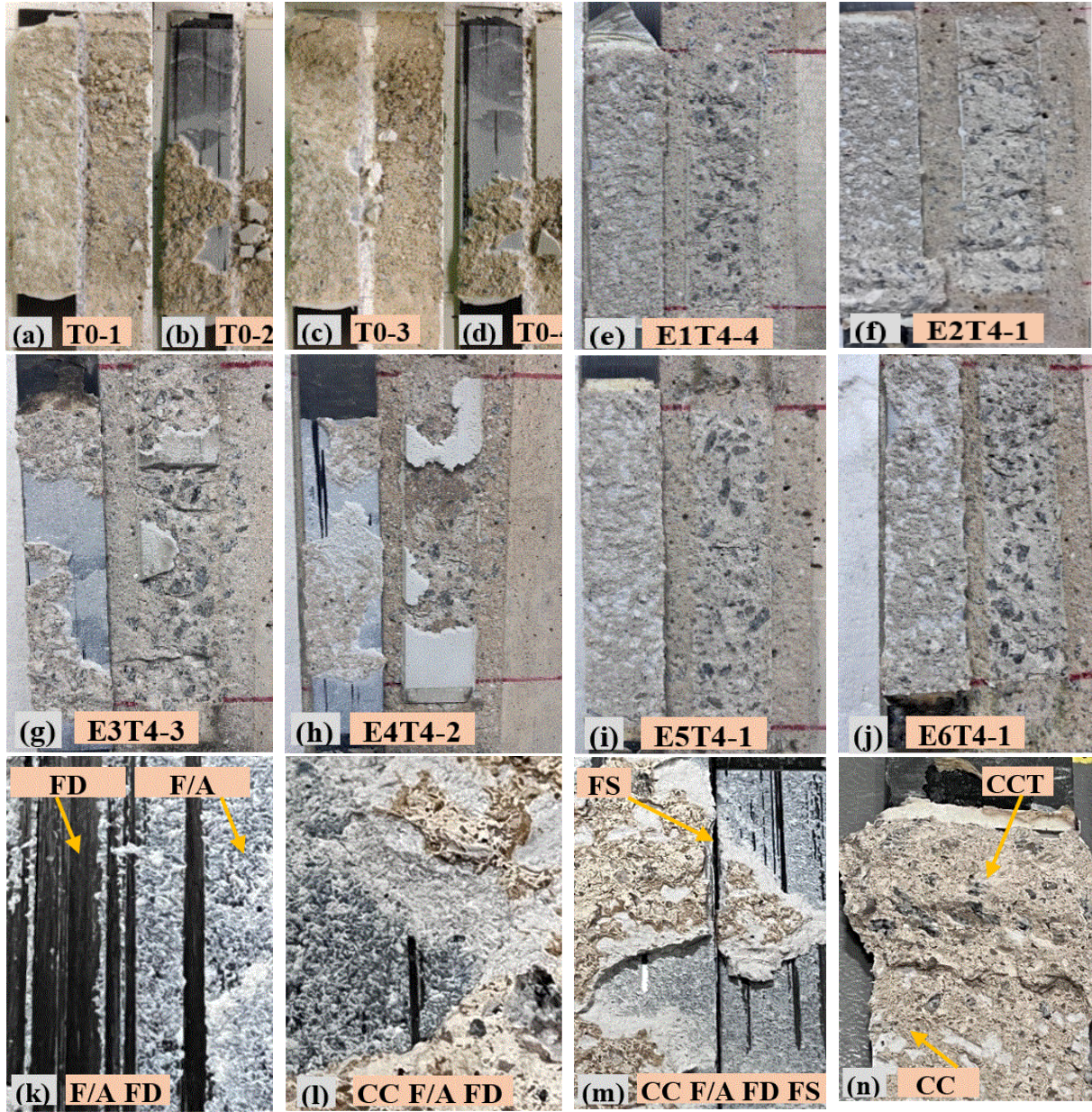


Figure II.10: Failure modes of EBR CFRP-to-concrete bond: (a-d) failure modes before exposure: concrete cohesive (CC) in T0-1 and T0-3, and mixed failure in T0-2 and T0-4 specimens; (e-j) typical predominant failure modes after exposure in each environment (E1 - E6): (e,i,j) concrete cohesive (CC), (f) CC with a thick layer at free end (CCT), (g,h) mixed failure (CC F/A FD) i.e., CC mixed with adhesive failure (F/A) and CFRP disbond (FD); (k-n) main regions (i.e., FD, F/A, FS, CC, and CCT) of the failures shown in (a-j) together with their corresponding notations used in text and in **Figure II.9i**. FS is when the CFRP laminate fractured in the longitudinal direction.

Visual observations led to noting that concrete region with fewer aggregates was more likely to fail before that with more aggregates, hence resulting in the higher pull-out force in the latter. Furthermore, the mixed failure mode generally led to higher pull-out force than the sole CC (**Figure II.9i**). This former failure occurred probably due to the adhesive being bonded to concrete with unequal distribution of aggregate at the concrete near-surface region, which caused unequal distribution of stresses, eventually leading to concrete failures in some regions (i.e., regions with fewer aggregates) and adhesive failures in other regions (i.e., regions of adhesive bonded to concrete with more aggregates). The mixed failure may also reflect the degradation of the materials themselves as time passes. It is worth noting that existing models for the CFRP-concrete bond generally consider the CC failure [53] without including other types of failure mode such as those observed in this work, hence further improvements by incorporating all types of failure modes can be crucial.

3.3. Constituent materials *versus* EBR CFRP-concrete bond properties

In this section, the change of EBR CFRP-to-concrete bond properties with time, in each studied environments, are analyzed taking into consideration of effects of the change of properties of the materials constituting the bond. The bond properties considered are the bond strength and bond stiffness. The former was obtained directly from the pull-out tests, while the latter was derived from each of the pull-out force *versus* loaded end slip curve by performing a linear fitting considering the force range between 0 to 10kN.

Bond strength variation in E1: The pull-out force consistently decreased from T0 to T3 and then increased from T3 to T4; however, the value at T4 was still lower than that at T0. Also, the bond stiffness generally showed a decrease. The lowest bond strength and bond stiffness retentions were around 0.9 at T3 and 0.7 at T4, respectively, which shows that the stiffness degradation was faster than that of bond strength. As can be seen from **Figure II.11a**, the concrete tensile strength (f_{ct}) and the CFRP laminate tensile strength (f_f) reductions would be attributed to the observed bond strength reduction, while in **Figure II.11b**, the CFRP laminate elastic modulus (E_f) reduction would also be attributed to the observed bond stiffness reduction. However, since the dominant failure mode was by concrete cohesive (CC), see **Figure II.9i**, concrete was still the weakest part of the bond, and hence the bond strength/stiffness decrease can be due to altered concrete properties, particularly its f_{ct} , or bonding redistributions that resulted from improved properties. It is worth noting that the highest maximum pull-out force occurred from the specimens that failed by CC F/A FD FS failure as can be seen from **Figure II.9i**.

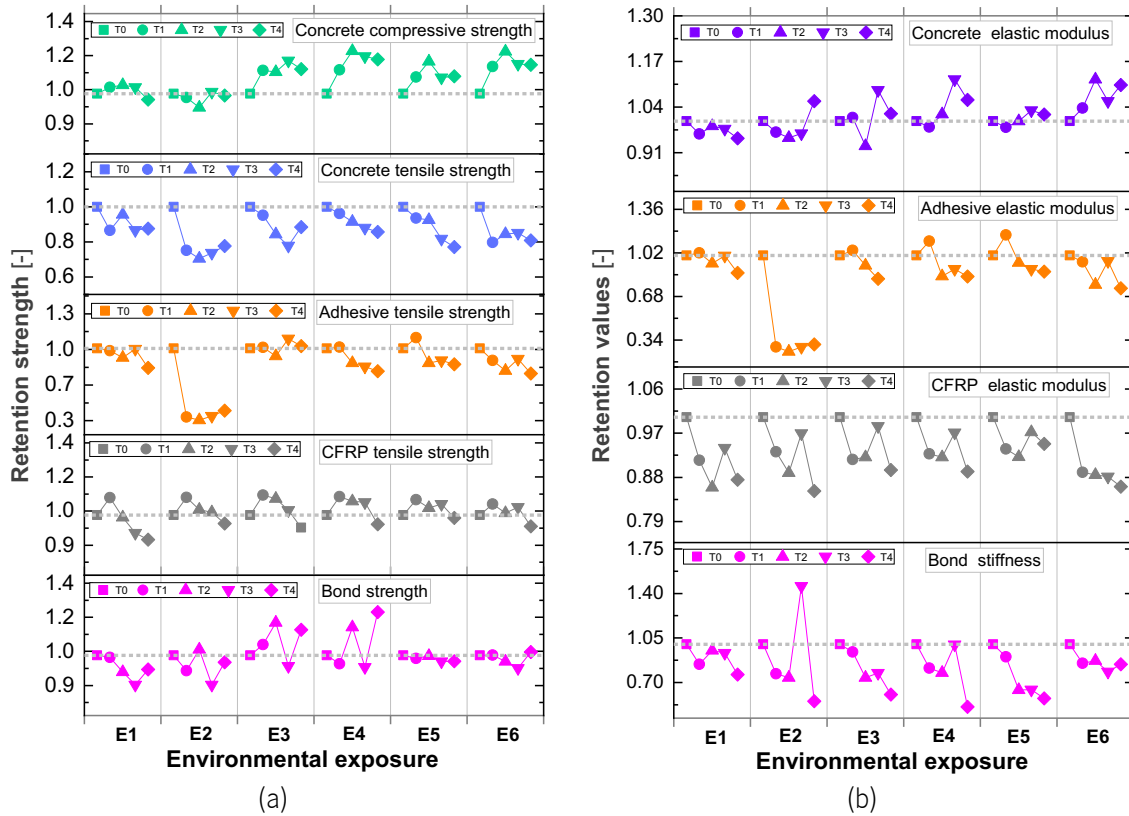


Figure II.11: Comparative perspectives on the effect of change of material properties on (a) the bond strength and (b) bond stiffness behavior.

Bond strength variation in E2. From **Figure II.11a,b**, the bond strength showed significant fluctuations over time, and the bond stiffness showed a general decrease except at T3. Like what is observed in E1, the lowest bond strength and stiffness retentions were around 0.9 at T3 and 0.7, respectively, again revealing that the stiffness degraded faster than the strength. The bond strength variations can be thought of mainly be related to the degradation adhesive tensile strength (f_a) and concrete pull-off strength (f_c), whereas the degradation of the bond stiffness can be attributed mainly to the E_a and E_c reductions (see **Figure II.11a,b**). In fact, water ingress can disrupt the interchange bonds thereby leading to plasticization effects [54]. It was shown that concrete and adhesive are bonded via chemical bond and mechanical interlock and the primary chemical interaction between the two adherends is hydrogen bonding, very weak compared to the covalent bonding [55]. The mechanical interlock will be loosened by plasticization thereby reducing the adhesive stiffness and water molecules will tend to react with the loosened interlock and breaks the hydrogen bond [55], hence leading to reduced bond properties. The predominant failure mode in E2 was CC at an average pull-out force of 32 kN and CCT at 29 kN (**Figure II.9i**). However, there was a tendency for the failure to be a “CC F/A FD”, with a lower average force

(27.5 kN) than that at CC (see **Figure II.9i**), thereby indicating that the negative effects of degraded f_a were more pronounced than those of degraded f_{ct} . Besides, the specimens with larger region of aggregate bonding to adhesive failed at the highest pull-out force, which confirms the significant positive effect of the aggregate bonding to the adhesive as the aggregate strength is neither affected by water nor partial carbonation.

Bond strength variation in E3: A tentative increase in bond strength is observed (**Figure II.11a**) which may be attributed to the post-curing of both adhesive and CFRP that increased polymer cross-linking due to high temperatures, also the post-curing was speeded up by carbonation as described in **Section 3.1**. Since the bond strength generally increased when the f_{cc} and f_t significantly increased with almost unchanged f_a , it seems the degraded f_{ct} had no effect on the bond strength. On the other hand, the bond stiffness showed a progressive decrease over time (**Figure II.11b**), the lowest retention being nearly 0.65. This decrease may have resulted from the degraded E_a and E_t , as the E_c generally did not decrease. The stiffness degradation may also be attributed to the chemical bond change at the interface with time between the concrete and adhesive (e.g., break of hydrogen bonds linking concrete to the adhesive) and/or between the adhesive and CFRP laminate (e.g., break of covalent bonds linking adhesive to the CFRP). The predominant failure mode was “CC F/A FD” (**Figure II.10g**), which was more beneficial compared to CC as it led to the higher average maximum pull-out force (34.5 kN). In fact, the improved concrete properties shifted the failure mode from being of the sole CC to “CC F/A FD”. Also, the tensile properties of adhesive reduced, which confirms the observed predominant failure mode. Comparing all failures from T0 to T4, the maximum pull-out force (38.4 kN) occurred for the case when the failure was “CC FS” (see **Figure II.9i**), which shows that the FS occurrence imposed higher pull-out force, hence the failure with splitting of the CFRP laminate would be the desired failure mode. Another perspective is that the climate change led to conditioning dissimilarity from year to year, hence, both the duration of exposure and the exposure severity significantly changed with time. Hence, the bond strength and bond stiffness may have depended on the above dissimilarity, where the longer the exposure the higher the variation or the more the severity the more the variation in a particular year.

Bond strength variation in E4: The bond strength showed fluctuations with a general increasing tendency (**Figure II.11a**); contrary, the bond stiffness showed a general decrease with time (**Figure II.11b**). The lowest bond stiffness retention was 0.51. Both the concrete E_c and f_{cc} retentions increased but with some reduction in its f_{ct} . However, both the adhesive E_a and f_a reduced. A slight reduction of the CFRP E_t and f_t was also observed. Hence, the decreased adhesive properties may have led to the observed variation of the bond properties. However, the

increased cement hydration in concrete and the post-curing of the CFRP may also have caused adjustment in the bonded region which either caused the bond strength to decrease or increase. Hence, this phenomenon can also be thought to have contributed to the fluctuations observed. Another perspective aligns with that described for E3, where the conditioning dissimilarity may have contributed to the observed fluctuations. A study by [27] showed that 300 FT cycles consisting of 16h of freezing and 8h of thawing in water bath did not significantly damage the bond between CFRP and concrete, which agrees with the results from E4 (**Figure II.11a**). Also, in a study by [25], it was found that the greater the number of freeze-thaw cycles the lower the sliding stiffness and loaded end slip of the CFRP-concrete interface, which is similar to what is observed in E4 (**Figure II.11b**). The predominant failure mode was “CC F/A FD” at an average of pull-out force of approximately 33.5 kN, and the failure that led to the highest pull-out force was “CC F/A FD FS” with 41 kN, which shows that a mixed failure with the occurrence of CFRP splitting/fracture can be suggested to be the desired failure mode.

In general, the pull-out force increased for mixed failure (**Figure II.10h**) as compared to when there was a sole CC. Also, the larger the CFRP-adhesive interface failure region the higher the pull-out force. Mixed failure modes were also reported in [56,57]. The combined effects of freeze-thaw attacks and the UV exposure can be thought to have taken place which resulted in some CFRP fracture/splitting failure, i.e., the microcracks may have formed thereby allowing the entrance of UV that caused more harmful effects to the fiber matrix. This agrees with the findings by [30]. Hence, the damaged fiber matrix is likely to be attributed to the observed mixed failure mode, particularly that containing FD or FS or both.

Bond strength variation in E5: A stable and slightly decreasing trend in the bond strength is observed (**Figure II.11a**). This trend can show the competing mechanisms between the temperature effects on the matrix post-curing for both CFRP and adhesive, and concrete carbonation or cement hydration effects (or other effects such as UV radiation) all of which tend to balance each other over time. However, the bond stiffness showed a progressive decrease over time (**Figure II.11b**), the highest decrease being 39 %, which indicates that the stiffness degraded faster than the bond strength. The stiffness decrease with increased temperature was also reported in a study by [28]. In fact, the increase in f_{cc} and f_t may have neutralized the negative effects from the degraded concrete f_{ct} and the f_a (**Figure II.11a**), thereby leading to marginal changes of the bond strength. On the other hand, the bond stiffness degradation can be attributed to the degraded E_t and the E_a (**Figure II.11b**). The CC was the predominant failure mode (**Figure II.9i**) occurring at an average pull-out force of approximately 28.5 kN, while the other sole competing failure mode was “CC F/A FD” which occurred at the force of

approximately 33.5 kN, hence suggesting that the latter failure would be the desired failure mode in E5. Existing studies show that high temperatures can cause the fiber-matrix interface to develop microcracks at the fiber/polymer interface [58], thereby degrading both the matrix and fibers [59]. Hence, some minor CFRP/adhesive interface failures observed in E5 may have resulted from the developed microcracks at the fiber matrix level. In general, the change of failure mode with time was less pronounced, hence the effects of degradation agents in E5 did not significantly affect the failure mode. This agrees with the pull-out force trend which also was somehow stable with time. Overall, synergic effects between high temperatures and carbonation were the main degradation agents.

Bond strength variation in E6: Marginal variations of the bond strength and a significant decrease in the bond stiffness can be seen from **Figure II.11a,b**. The almost non-variation of the bond strength with time can be attributed to the improved f_{cc} and f_t that counteracted the loss of f_a and f_{ct} . The CC and the “CC F/A FD” were the only two competing failure modes (**Figure II.9i**) at an average force of 29 kN and 32.5 kN, respectively. In the former, a very thin layer of concrete detached, which shows that the effects of chlorides on concrete were very minimal, as previously described in **Section 3.1**. Also, since the exposure of epoxy adhesive to chlorides does not lead to detrimental effect [22], and the f_t is generally immune to chloride exposure [18], the major degradation agents were the interaction of the moisture and carbonation. However, the carbonation played a more role than moisture because of its ability to substantially reduce concrete porosity in the superficial region [31] thereby preventing the rapid ingress of moisture. This is confirmed by the carbonation (C_a) trend (**Figure II.7a**) that tends to match well with the bond strength trend (**Figure II.11a**), thereby indicating that the more the C_a the better the bond performance.

Like what is noted in E2, the aggregate bonding to adhesive seemed to control the maximum pull-out force in E6. The more the aggregates were bonded to the adhesive the higher was the pull-out force (**Figure II.10.j**). This is in line with the findings by [60], where the shape, size and strength of the coarse aggregate played an important role in the pull-off strength.

3.4. Accelerated aging *versus* natural aging

To compare the existing accelerated aging test (AAT) data with those of natural/outdoor aging test (NAT) from the present work, a database from different existing studies [24,27–34], was prepared with the following characteristics: i) the database comprised of 99 specimens strengthened with pultruded CFRP laminate strips

tested either in a direct or beam single lap shear test configuration, and the environmental conditioning adopted were freeze-thaw cycles, wet-dry cycles, immersion in salt or tap water, moisture, temperature cycles, and splash exposure; ii) the exposure duration was up to 18,000 h (2 years); iii) only epoxy adhesives, and CFRP laminate thickness between 1.0 and 1.4 mm with elastic modulus between 155 and 176 GPa were considered; iv) bond length ranged between 100 and 600 mm, v) concrete compressive strength varied between 25 and 50 MPa.

The bond strength retentions over time for both the above-described database and the data from the present work (denoted as FRPLongDur-E2-E6) are presented in Figure II.12a.

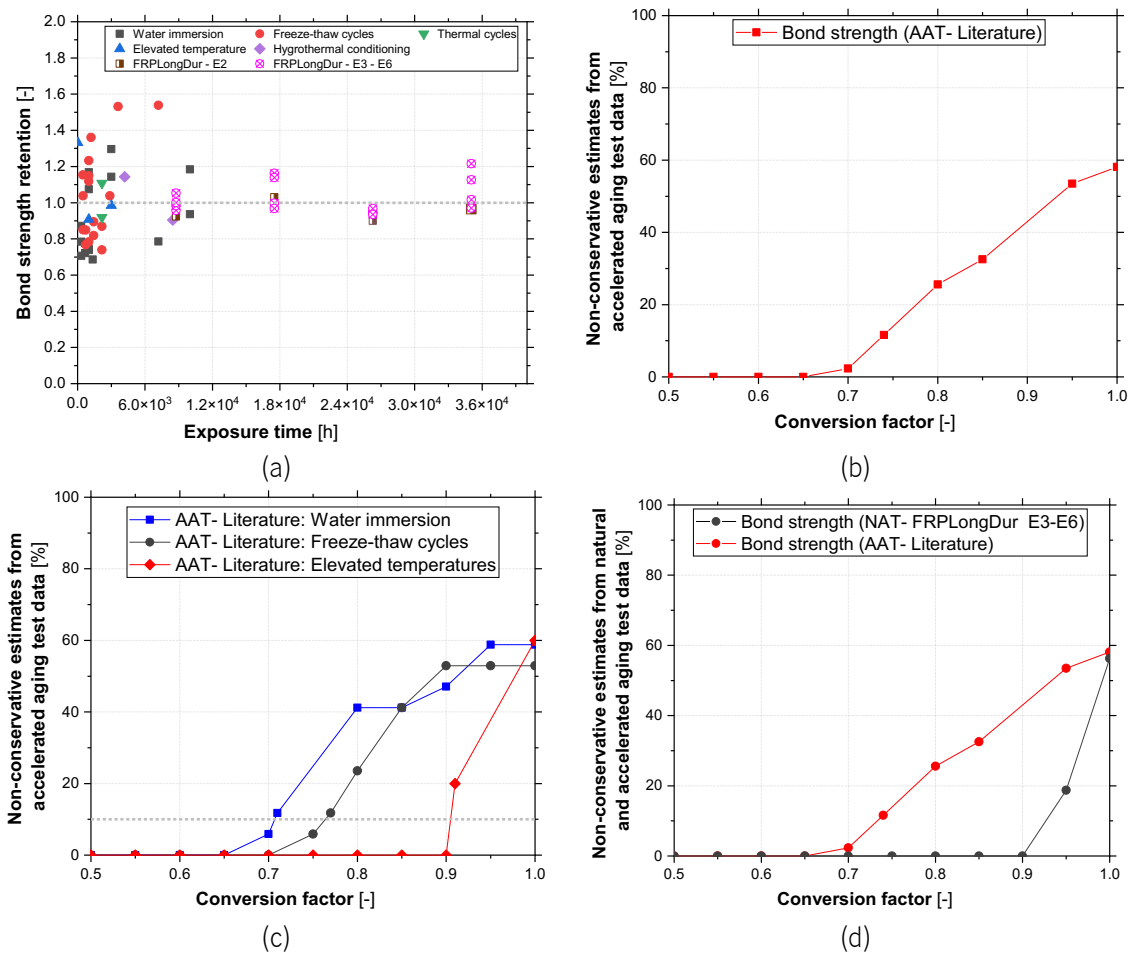


Figure II.12: Natural *versus* accelerated aging durability results for EBR technique: (a) bond strength retention from the AAT and NAT data, (b) derivation of the ECF considering all studied degradation agents in AAT data, (c) derivation of the ECF considering a single degradation agent in AAT, (d) comparison of the ECF from the AAT and NAT data.

It can be observed that the retentions from the existing literature are very scattered ranging between 0.7 and 1.6, while those from the present work lie between 0.9 and 1.2. In fact, after 4 years of exposure, the retentions

from NAT data (FRPLongDur-E3-E6) are still generally higher than those from AAT data in the existing literature. On the other hand, assuming 10 % as the maximum allowable non-conservative estimates from all the AAT data in the database, it can be seen from **Figure II.12b** that the environmental conversion factor (ECF), considering all types of degradation agents, can be suggested to be approximately 0.75. Furthermore, based on the results from **Figure II.12c**, the ECF is suggested to be approximately 0.70, 0.75, or 0.90 for structures immersed in water, exposed to freeze-thaw cycles, or elevated temperatures, respectively. This is reasonable because both the bond and its constituent materials are expected to degrade more when exposed to water and freeze-thaws as compared to when exposed to elevated temperatures.

Comparing both the AAT and NAT (**Figure II.12d**), the ECF for the AAT and NAT is approximately 0.75 and 0.95, respectively, which leads to stating that the exposure of EBR CFRP-to-concrete bond to various outdoor degradation agents for up to 4 years can lead to less degradation as compared to when the bond is exposed to artificially accelerated degradation agents.

4. CONCLUSION AND RECOMMENDATIONS

In this work, the durability of EBR CFRP-concrete bond and its constituent materials was addressed. Specimens were conditioned in 6 different environments. The first two were laboratory-based (i.e., E1 with 20 °C/55 % RH, and E2 with water immersion at 20 °C) and the remaining were outdoor (i.e., E3, E4, E5, and E6 in which the main degradation agents were carbonation, freeze-thaw attacks, elevated temperatures, and airborne chlorides, respectively). The specimens were tested at initial time (T0), and after 1 (T1), 2 (T2), 3 (T3) and 4 (T4) years of exposure. The key findings are as follows:

- Although the outdoor environments were selected to mainly promote a single degradation agent in each environment, multiple degradation agents occurred; thus, synergistic effects were dominant. That is, carbonation, moisture, and airborne chlorides were the three-competing agents in E6; carbonation and elevated temperature effects were dominant in E3 and E5, and moisture and freeze-thaw effects were dominant in E4.
- Constituent materials: Concrete compressive strength (f_c) and elastic modulus (E_c) showed a significant increase in outdoor environments (E3-E6), whereas marginal variations were observed in laboratory environments (E1 and E2). The highest increases were approximately 16 % (T3), 21 % (T2), 17 % (T2), and 21 % (T2) in f_{cc} , and 9 % (T3), 12 % (T3), 3 % (T3), and 12 % (T2) in E_c for E3, E4, E5, and E6,

respectively. Conversely, the concrete pull-off strength (f_{ct}) showed a progressive decrease with time in all the environments studied. On the other hand, the epoxy adhesive generally decreased both its tensile strength (f_a) and tensile elastic modulus (E_a) in all environments, with the lowest values observed in water with reductions of 66.3 % and 75.4 %, respectively. Similarly, the tensile elastic modulus of the CFRP laminate (E_f) also showed a decreasing trend in all environments, the lowest being observed in E6 with a reduction of 13.7 %; however, the CFRP tensile strength (f_f) increased during the first year in all environments as a result of post-curing, followed by a progressive decrease in the later years. The f_f reduced by 12.3 %, 4.2 %, 6.3 %, 4.7 %, 1.4 %, and 5.6 % and its E_f by 12.6 %, 14.7 %, 10.5 %, 10.5 %, 5.3 %, and 13.7 % in E1 to E6, respectively. This showed that E_f degraded faster than f_f .

- EBR CFRP-to-concrete bond: Bond strength and bond stiffness varied significantly with time and environment. The latter was found to degrade faster than the former in most of the environments. This was particularly noticeable when the bond was exposed to elevated temperatures (E5), which led to a 39 % reduction in the bond stiffness, whereas there was a marginal change in the bond strength. A tendency for bond strength to decrease over time was generally observed in E1 and E2. In outdoor environments, the bond strength in E3 and E4 showed a tendency to increase with time, but with significant fluctuations that may have resulted from the effects of exposure duration and severity, or from the molecular adjustments in the bonded region resulting from the change in the properties of the constituent materials over time. In contrast, the bond strength was almost unchanged in E5 and E6, indicating that the considered degradation agents did not significantly affect the bond strength in these environments. On the other hand, the bond stiffness showed a progressive-significant decrease in all studied environments. The lowest bond stiffness was observed in E4 with 50 % reduction followed by E2 with 40 % reduction, and 39 % and 35 % reductions in E5 and E3, respectively. Smaller reductions were observed in E1 and E6.
- Failure modes: Visual examination of the failure modes in EBR showed that the failure mode at the initial time (T0) changed mainly from pure concrete cohesive failure within the concrete alone (or mixed with cohesive failure within the adhesive) to various failures depending on the environmental exposure. Most of the failure modes from different environments were due to interfacial separation between either the adhesive and the concrete substrate or the adhesive and the CFRP. Concrete cohesive failure with a thin concrete layer dominated in E1 and E5; concrete cohesive failure with or without a very thick

concrete layer at the free end was predominant in E2; the mixed failure mode of concrete cohesive failure with a thin concrete layer, interfacial adhesive failure at the interface between adhesive and CFRP (at interface regions closest to the adhesive surface or closest to the CFRP laminate surface) dominated in E3 and E4; the mixed failure mode and the concrete cohesive failure with a thin concrete layer were the two competing and predominant failures observed in E6. In general, higher pull-out forces corresponded to specimens with a failure mode that is a mixture of the adhesive-CFRP interaction layer failure and the concrete cohesive failure (than the concrete cohesive failure alone), the larger the area of the former failure the higher the pull-out force. Besides, the fracture/splitting of CFRP always resulted in the highest pull-out force, making it the most desirable failure mode.

- The environmental conversion factor (ECF) for the accelerated aging test (AAT) based studies is proposed to be 0.75 when all degradation agents are considered as one entity, whereas ECFs of 0.7, 0.75, or 0.90 are proposed to account for the effects of water immersion, freeze-thaws, or high temperature effects, respectively. When comparing both the AAT and natural aging test (NAT) protocols, ECF values of 0.75 and 0.95 are suggested for the former and the latter protocols, respectively, leading to the conclusion that the accelerated aging test protocols may overestimate the degradation that normally occurs in outdoor environments for 4 years of exposure.

As main recommendation, the authors suggest that studies that attempt to quantify the effects of UV radiations in outdoor environments, in addition to the degradations agents considered in the present work (carbonation, high temperatures, airborne chlorides, and freeze-thaw attacks), can lead to a better understanding on the degradation process of the structures in outdoor environments. Further research comprising both laboratory-based and outdoor environments with comparative perspectives on both environments are highly recommended as they can lead to a better understanding of the durability of the bond between concrete and CFRP laminate.

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REFERENCES

- [1] F. Hermansson, S. Heimersson, M. Janssen, M. Svanström, Can carbon fiber composites have a lower environmental impact than fiberglass? *Resour. Conserv. Recycl.* 181 (2022). <https://doi.org/10.1016/j.resconrec.2022.106234>.
- [2] S. Maiti, M.R. Islam, M.A. Uddin, S. Afroj, S.J. Eichhorn, N. Karim, Sustainable Fiber-Reinforced Composites: A Review, *Adv. Sustain. Syst.* 6 (2022). <https://doi.org/10.1002/adsu.202200258>.
- [3] I.S. Abbood, S.A. Odaa, K.F. Hasan, M.A. Jasim, Properties evaluation of fiber reinforced polymers and their constituent materials used in structures - A review, *Mater. Today Proc.* 43 (2021) 1003–1008. <https://doi.org/10.1016/j.matpr.2020.07.636>.
- [4] H.H. Mhanna, R.A. Hawileh, J.A. Abdalla, Shear strengthening of reinforced concrete beams using CFRP wraps, *Procedia Struct. Integr.* 17 (2019) 214–221. <https://doi.org/10.1016/j.prostr.2019.08.029>.
- [5] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B Eng.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
- [6] CEB-FIP, fib90 Bulletin:Externally Applied FRP reinforcement for Concrete structures, 2019.
- [7] J.S. Cruz, J. Barros, Modeling of bond between near-surface mounted CFRP laminate strips and concrete, *Comput. Struct.* 82 (2004) 1513–1521. <https://doi.org/10.1016/j.compstruc.2004.03.047>.
- [8] N.M. Huang, J.J. Chang, M.T. Liang, Effect of plastering on the carbonation of a 35-year-old reinforced concrete building, *Constr. Build. Mater.* 29 (2012) 206–214. <https://doi.org/10.1016/j.conbuildmat.2011.08.049>.
- [9] V. Talakokula, S. Bhalla, R.J. Ball, C.R. Bowen, G.L. Pesce, R. Kurchania, B. Bhattacharjee, A. Gupta,

- K. Paine, Diagnosis of carbonation induced corrosion initiation and progression in reinforced concrete structures using piezo-impedance transducers, *Sensors Actuators, A Phys.* 242 (2016) 79–91. <https://doi.org/10.1016/j.sna.2016.02.033>.
- [10] X. han Shen, Q. feng Liu, Z. Hu, W. qiang Jiang, X. Lin, D. Hou, P. Hao, Combine ingress of chloride and carbonation in marine-exposed concrete under unsaturated environment: A numerical study, *Ocean Eng.* 189 (2019) 106350. <https://doi.org/10.1016/j.oceaneng.2019.106350>.
- [11] S.K. Roy, D.O. Northwood, K.B. Pohs, Effect of plastering on the carbonation of a 19-year-old reinforced concrete building, *Constr. Build. Mater.* 29 (2012) 206–214. <https://doi.org/10.1016/j.conbuildmat.2011.08.049>.
- [12] A.F. Al Fuhaid, A. Niaz, Carbonation and Corrosion Problems in Reinforced Concrete Structures, *Buildings*. 12 (2022) 1–20. <https://doi.org/10.3390/buildings12050586>.
- [13] J. Zhao, G. Cai, L. Cui, A. Si Larbi, K.D. Tsavdaridis, Deterioration of basic properties of the materials in FRP-strengthening RC structures under ultraviolet exposure, *Polymers (Basel)*. 9 (2017). <https://doi.org/10.3390/polym9090402>.
- [14] Z. Mi, Y. Hu, Q. Li, Z. An, Effect of curing humidity on the fracture properties of concrete, *Constr. Build. Mater.* 169 (2018) 403–413. <https://doi.org/10.1016/j.conbuildmat.2018.03.025>.
- [15] Y. Wang, S. Nanukuttan, Y. Bai, P.A.M. Basheer, Influence of combined carbonation and chloride ingress regimes on rate of ingress and redistribution of chlorides in concretes, *Constr. Build. Mater.* 140 (2017) 173–183. <https://doi.org/10.1016/j.conbuildmat.2017.02.121>.
- [16] H. Kuosa, R.M. Ferreira, E. Holt, M. Leivo, E. Vesikari, Effect of coupled deterioration by freeze-thaw, carbonation and chlorides on concrete service life, *Cem. Concr. Compos.* 47 (2014) 32–40. <https://doi.org/10.1016/j.cemconcomp.2013.10.008>.
- [17] P. Fernandes, P. Silva, L. Correia, J. Sena-cruz, Durability of an epoxy adhesive and a CFRP laminate under different exposure conditions, in: *Third Conf. Smart Monit. Assess. Rehabil. Civ. Struct.* 2015, 2015: p. 9.
- [18] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fibre reinforced polymer laminates used in strengthening systems: Accelerated ageing versus natural ageing, *Materials (Basel)*. 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [19] S. Cabral-fonseca, J.R. Correia, J. Custódio, H.M. Silva, A.M. Machado, J. Sousa, Durability of FRP -

concrete bonded joints in structural rehabilitation : A review, *Int. J. Adhes. Adhes.* 83 (2018) 153–167. <https://doi.org/10.1016/j.ijadhadh.2018.02.014>.

[20] S.A. Grammatikos, R.G. Jones, M. Evernden, J.R. Correia, Thermal cycling effects on the durability of a pultruded GFRP material for off-shore civil engineering structures, *Compos. Struct.* 153 (2016) 297–310. <https://doi.org/10.1016/j.compstruct.2016.05.085>.

[21] E. Cui, S. Jiang, J. Wang, X. Zeng, Bond behavior of CFRP-concrete bonding interface considering degradation of epoxy primer under wet-dry cycles, *Constr. Build. Mater.* 292 (2021) 123286. <https://doi.org/10.1016/j.conbuildmat.2021.123286>.

[22] P. Silva, P. Fernandes, J. Sena-Cruz, J. Xavier, F. Castro, D. Soares, V. Carneiro, Effects of different environmental conditions on the mechanical characteristics of a structural epoxy, *Compos. Part B Eng.* 88 (2016) 55–63. <https://doi.org/10.1016/j.compositesb.2015.10.036>.

[23] D. dong Hu, J. xun Lyu, T. Liu, M. dong Lang, L. Zhao, Solvation effect of CO₂ on accelerating the curing reaction process of epoxy resin, *Chem. Eng. Process. - Process Intensif.* 127 (2018) 159–167. <https://doi.org/10.1016/j.cep.2018.01.027>.

[24] S. Liu, Y. Pan, H. Li, G. Xian, Durability of the bond between CFRP and concrete exposed to thermal cycles, *Materials (Basel)*. 12 (2019). <https://doi.org/10.3390/ma12030515>.

[25] F. Jiang, X. Han, Y. Wang, P. Wang, T. Zhao, K. Zhang, Effect of freeze-thaw cycles on tensile properties of CFRP, bond behavior of CFRP-concrete, and flexural performance of CFRP-strengthened concrete beams, *Cold Reg. Sci. Technol.* 194 (2022) 103461. <https://doi.org/10.1016/j.coldregions.2021.103461>.

[26] C. Helbling, V. Karbhari, Durability of composites in aqueous environments, in: *Durab. Compos. Civ. Struct. Appl.*, Elsevier, Amsterdam, The Netherlands, 2007: pp. 31–71.

[27] M.F. Green, L.A. Bisby, Y. Beaudoin, P. Labossière, Effect of freeze-thaw cycles on the bond durability between fibre reinforced polymer plate reinforcement and concrete, *Can. J. Civ. Eng.* 27 (2000) 949–959. <https://doi.org/10.1139/I00-031>.

[28] M. Leone, S. Matthys, M.A. Aiello, Effect of elevated service temperature on bond between FRP EBR systems and concrete, *Compos. Part B Eng.* 40 (2009) 85–93. <https://doi.org/10.1016/j.compositesb.2008.06.004>.

[29] P. Colombi, G. Fava, C. Poggi, Bond strength of CFRP-concrete elements under freeze-thaw cycles, *Compos. Struct.* 92 (2010) 973–983. <https://doi.org/10.1016/j.compstruct.2009.09.044>.

[30] J.R. Cromwell, K.A. Harries, B.M. Shahrooz, Environmental durability of externally bonded FRP materials

intended for repair of concrete structures, *Constr. Build. Mater.* 25 (2011) 2528–2539. <https://doi.org/10.1016/j.conbuildmat.2010.11.096>.

[31] K. Benzarti, S. Chataigner, M. Quiertant, C. Marty, C. Aubagnac, Accelerated ageing behaviour of the adhesive bond between concrete specimens and CFRP overlays, *Constr. Build. Mater.* 25 (2011) 523–538. <https://doi.org/10.1016/j.conbuildmat.2010.08.003>.

[32] F. Al-Mahmoud, J.M. Mechling, M. Shaban, Bond strength of different strengthening systems - Concrete elements under freeze-thaw cycles and salt water immersion exposure, *Constr. Build. Mater.* 70 (2014) 399–409. <https://doi.org/10.1016/j.conbuildmat.2014.07.039>.

[33] Y. Pan, G. Xian, M.A.G. Silva, Effects of water immersion on the bond behavior between CFRP plates and concrete substrate, *Constr. Build. Mater.* 101 (2015) 326–337. <https://doi.org/10.1016/j.conbuildmat.2015.10.129>.

[34] Y. Pan, G. Xian, H. Li, Effects of Freeze-Thaw Cycles on the Behavior of the Bond between CFRP Plates and Concrete Substrates, *J. Compos. Constr.* 22 (2018) 1–14. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000846](https://doi.org/10.1061/(asce)cc.1943-5614.0000846).

[35] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of Bond between NSM CFRP Strips and Concrete under Real-Time Field and Laboratory Accelerated Conditioning, *J. Compos. Constr.* 26 (2022) 1–15. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001262](https://doi.org/10.1061/(asce)cc.1943-5614.0001262).

[36] IPQ, IPQ (Instituto Português da Qualidade). 2013. NP EN 12390-13. Testing Hardened Concrete. Part 13: Determination of Secant Modulus of Elasticity in Compression, Instituto Português da Qualidade (IPQ)., Caparica, Portugal, 2013.

[37] IPQ, 2011. NP EN 12390-3. Testing Hardened Concrete. Part 3: Compressive Strength of Test Specimen, Instituto Português da Qualidade (IPQ), Caparica, Portugal, 2011.

[38] BSI, BSI (British Standards Institution). Products and systems for the protection and repair of concrete structures – Test methods – Measurement of bond strength by pull-off. EN 1542., London: BSI, 1999.

[39] ISO, ISO 527-2:2012—Plastics—Determination of Tensile Properties—Part 2: Test conditions for Moulding and Extrusion Plastics, Genève, Switzerland, 2012.

[40] ISO, ISO 527-5:2009 Part 5: Test Conditions for Unidirectional Fibre-Reinforced Plastic composites. Plastic—Determ. Tensile Prop., Inte Genève, Switzerland, 2009; Volume 1, 2009.

[41] CNR-DT 200 R1, Guide for the Design and Construction of Externally Bonded FRP Systems for

Strengthening Existing Structures; CNR-DT 200 R1/2013; Rome, Italy, 2013.

- [42] T.P. Hills, F. Gordon, N.H. Florin, P.S. Fennell, Statistical analysis of the carbonation rate of concrete, *Cem. Concr. Res.* 72 (2015) 98–107. <https://doi.org/10.1016/j.cemconres.2015.02.007>.
- [43] N. Bouzoubaâ, A. Bilodeau, B. Tamtsia, S. Foo, Carbonation of fly ash concrete: Laboratory and field data, *Can. J. Civ. Eng.* 37 (2010) 1535–1549. <https://doi.org/10.1139/L10-081>.
- [44] L. Jiang, Y. Zhang, Analysis and calculation of the length of half – carbonated zone in concrete [in Chinese], *Ind Constr.* 29 (1999) 4–8.
- [45] H.. Tseng, J.. Hsia, Concrete carbonation and durability prediction[in Chinese], *Hunan Commun Sci Technol.* 32 (2006) 121–6.
- [46] A. Dushimimana, A.A. Niyonsenga, F. Nzamurambaho, A review on strength development of high performance concrete, *Constr. Build. Mater.* 307 (2021) 124865. <https://doi.org/10.1016/j.conbuildmat.2021.124865>.
- [47] S.N. Shoukry, G.W. William, B. Downie, M.Y. Riad, Effect of moisture and temperature on the mechanical properties of concrete, *Constr. Build. Mater.* 25 (2011) 688–696. <https://doi.org/10.1016/j.conbuildmat.2010.07.020>.
- [48] H. Peng, Y. Liu, C.S. Cai, J. Yu, J. Zhang, Experimental Investigation of Bond between Near-Surface-Mounted CFRP Strips and Concrete under Freeze-Thawing Cycling, *J. Aerosp. Eng.* 32 (2019) 1–10. [https://doi.org/10.1061/\(asce\)as.1943-5525.0000937](https://doi.org/10.1061/(asce)as.1943-5525.0000937).
- [49] M.I. Kabir, R. Shrestha, B. Samali, Effects of applied environmental conditions on the pull-out strengths of CFRP-concrete bond, *Constr. Build. Mater.* 114 (2016) 817–830. <https://doi.org/10.1016/j.conbuildmat.2016.03.195>.
- [50] M. Massou, N. Babu, G. Xian, Experimental study on the mechanical properties of CFRP/epoxy composite plates under seawater immersion, *Structures.* 54 (2023) 48–57. <https://doi.org/10.1016/j.istruc.2023.05.042>.
- [51] D. Mostofinejad, M. Mohammadi, Effect of Freeze–Thaw Cycles on FRP-Concrete Bond Strength in EBR and EBROG Systems, *J. Compos. Constr.* 24 (2020) 1–12. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001010](https://doi.org/10.1061/(asce)cc.1943-5614.0001010).
- [52] L. Hong, R.M. Duo, S.Y. Wang, L.X. Li, Influence of freeze-thaw cycles on bonded interface performance between CFRP and high strength concrete, *Appl. Mech. Mater.* 638–640 (2014) 1516–1520. <https://doi.org/10.4028/www.scientific.net/AMM.638-640.1516>.

- [53] R.M. Faysal, M.M.H. Bhuiyan, K. Al Momin, T. Tafsirojjaman, Y. Liu, A review on the advances of the study on FRP-Concrete bond under hygrothermal exposure, *Constr. Build. Mater.* 363 (2023) 129818. <https://doi.org/10.1016/j.conbuildmat.2022.129818>.
- [54] M. Savvilotidou, A.P. Vassilopoulos, T. Keller, *Durability and Fatigue Performance of Cold-Curing Structural Adhesives in Bridge Construction*, École Polytechnique Fédérale de Lausanne (EPFL), 2017. <https://infoscience.epfl.ch/record/230442?ln=en>.
- [55] J. Tatar, S. Milev, Durability of externally bonded fiber-reinforced polymer composites in concrete structures: A critical review, *Polymers (Basel)*. 13 (2021) 1–26. <https://doi.org/10.3390/polym13050765>.
- [56] J. Lee, J. Kim, C.E. Bakis, T.E. Boothby, Durability assessment of FRP-concrete bond after sustained load for up to thirteen years, *Compos. Part B Eng.* 224 (2021) 109180. <https://doi.org/10.1016/j.compositesb.2021.109180>.
- [57] A.K. Al-Tamimi, R.A. Hawileh, J.A. Abdalla, H.A. Rasheed, R. Al-Mahaidi, Durability of the Bond between CFRP Plates and Concrete Exposed to Harsh Environments, *J. Mater. Civ. Eng.* 27 (2015) 04014252. [https://doi.org/10.1061/\(asce\)mt.1943-5533.0001226](https://doi.org/10.1061/(asce)mt.1943-5533.0001226).
- [58] L.T. Drzal, M.J. Rich, M.F. Koenig, P.F. Lloyd, Adhesion of Graphite Fibers to Epoxy Matrices: II. The Effect of Fiber Finish, *J. Adhes.* 16 (1983) 133–152. <https://doi.org/10.1080/00218468308074911>.
- [59] C. Bockenheimer, D. Fata, W. Possart, New aspects of aging in epoxy networks. I. Thermal aging, *J. Appl. Polym. Sci.* 91 (2004) 361–368. <https://doi.org/10.1002/app.13092>.
- [60] E. Bonaldo, J.A.O. Barros, P.B. Lourenço, Bond characterization between concrete substrate and repairing SFRC using pull-off testing, *Int. J. Adhes. Adhes.* 25 (2005) 463–474. <https://doi.org/10.1016/j.ijadhadh.2005.01.002>.

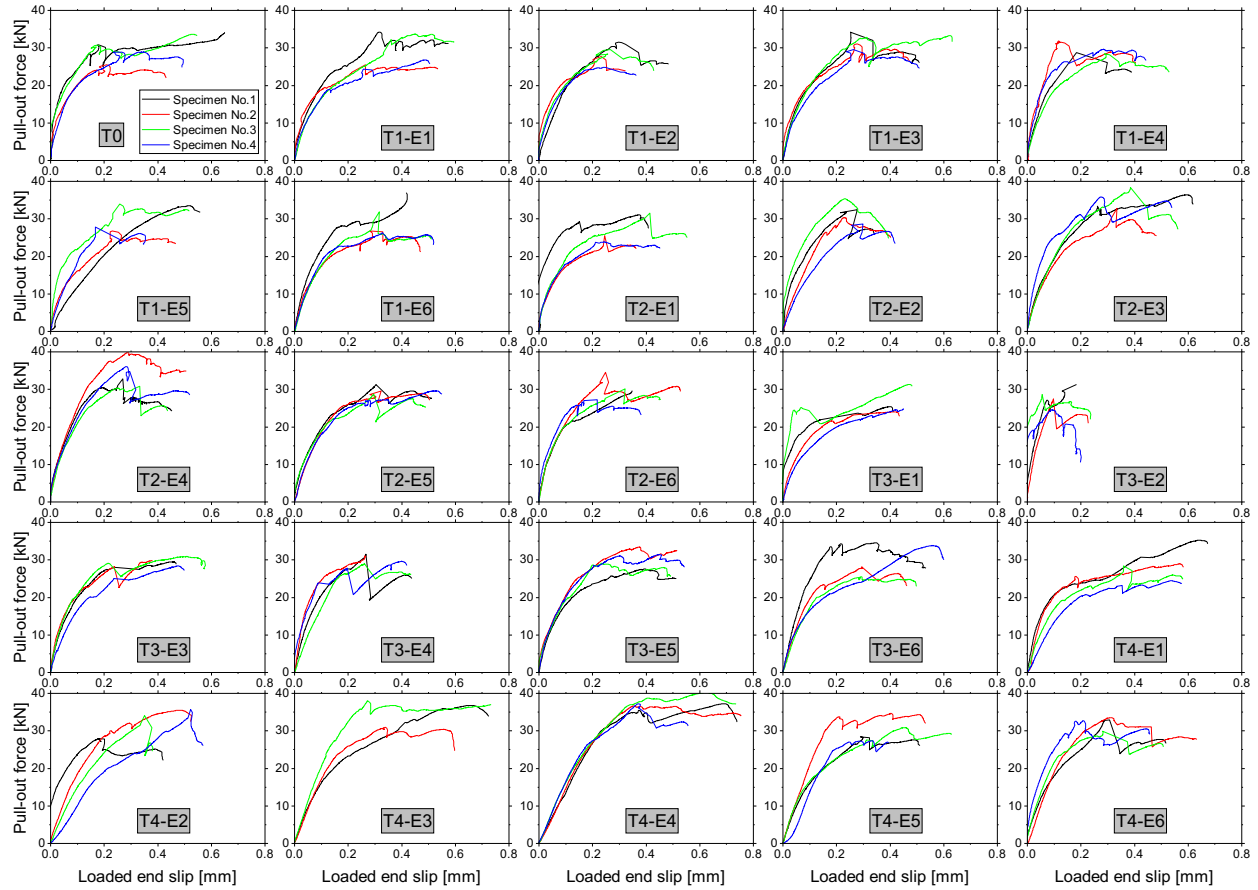
ANNEX II.1: Single-lap shear test results for all the specimens tested from T0 to T4.

Time	Specimen	Force (kN)	Slip (mm)	Force (kN)	Slip (mm)	Force (kN)	Slip (mm)
T0	No.1	34.06	0.65	-	-	-	-
	No.2	25.47	0.43	-	-	-	-
	No.3	33.62	0.55	-	-	-	-
	No.4	28.92	0.50	-	-	-	-
		E1	E2		E3		
T1	No.1	34.18	0.31	31.55	0.30	34.17	0.25
	No.2	26.3	0.20	27.74	0.23	31.06	0.28
	No.3	33.69	0.45	29.53	0.27	33.24	0.62
	No.4	26.77	0.49	28.09	0.24	29.58	0.26
T2	No.1	31.13	0.37	32.43	0.28	36.43	0.59
	No.2	26.36	0.21	30.37	0.23	32.82	0.33
	No.3	31.58	0.41	35.40	0.23	38.32	0.39
	No.4	26.32	0.22	28.63	0.30	35.87	0.27
T3	No.1	25.43	0.40	31.28	0.18	29.49	0.45
	No.2	24.02	0.40	27.45	0.10	29.86	0.38
	No.3	31.31	0.47	28.65	0.06	30.90	0.51
	No.4	24.84	0.45	24.57	0.09	28.41	0.48
T4	No.1	35.27	0.67	27.91	0.18	36.80	0.72
	No.2	28.97	0.58	35.51	0.52	30.93	0.60
	No.3	28.44	0.58	34.22	0.38	38.06	0.27
	No.4	24.47	0.58	35.67	0.57	-	-
		E4	E5		E6		
T1	No.1	28.81	0.18	33.52	0.52	37.04	0.42
	No.2	31.87	0.12	26.82	0.23	26.74	0.28
	No.3	28.35	0.29	34.21	0.24	31.89	0.32
	No.4	30.89	0.16	27.88	0.17	28.39	0.34
T2	No.1	32.79	0.27	34.92	0.27	29.72	0.35
	No.2	39.81	0.28	29.52	0.32	34.56	0.25
	No.3	30.83	0.33	29.18	0.24	30.32	0.25
	No.4	36.11	0.28	29.61	0.52	27.19	0.22
T3	No.1	31.49	0.27	27.65	0.39	34.59	0.35
	No.2	31.20	0.26	33.46	0.37	28.15	0.29
	No.3	29.09	0.26	29.08	0.23	25.56	0.29
	No.4	29.63	0.40	31.45	0.45	33.83	0.56
T4	No.1	37.27	0.74	28.4	0.29	32.90	0.31
	No.2	36.68	0.35	34.65	0.40	33.55	0.31
	No.3	40.27	0.63	30.87	0.46	29.93	0.28
	No.4	37.21	0.37	27.38	0.33	32.74	0.21

Notes: E1-E6: studied environments, T1-T4: exposure periods (in years)

DURABILITY OF EXTERNALLY BONDED CFRP-TO-CONCRETE BOND:
ACCELERATED *VERSUS* NATURAL AGING

ANNEX II.2: Single-lap shear test results: Pull-out force *versus* loaded end slip curves for all specimens tested up to 4 years.



PAPER III

DURABILITY OF NEAR-SURFACE MOUNTED CFRP-TO- CONCRETE BOND: ACCELERATED *VERSUS* NATURAL AGING

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ABSTRACT: Carbon fiber reinforced polymer (CFRP) composites can be used to strengthen existing reinforced concrete (RC) structures. The CFRP laminate can be bonded to RC structure using epoxy adhesive via near-surface mounted (NSM) strengthening technique. However, existing literature generally lacks data about

durability of NSM CFRP-to-concrete bond. In this study, strengthened concrete elements were exposed to laboratory-controlled environments (at approximately 20 °C/55 % RH, and water immersion at 20 °C) and natural field environments (to promote natural aging induced mainly by carbonation, high temperatures, freeze-thaw attack, and airborne chlorides) for up to four years. Durability tests were conducted yearly for the bond and its constituent materials. The highest bond strength degradations were nearly 12 % and 9 % for the specimens immersed in water and those exposed to freeze-thaw attack, respectively. Besides, environmental conversion factors of 0.88 and 0.93 were derived from a database of existing accelerated, and natural aging data from the present work, respectively.

KEYWORDS: Durability, Natural aging, Accelerated aging, NSM CFRP-to-concrete bond, Epoxy adhesive, Concrete, Carbon fiber

1. INTRODUCTION

It is undoubtedly realistic to state that concrete has been proven to be a reliable construction material since its invention. Concrete has very good compressive behavior but generally poor tension properties; however, it can usually be reinforced with e.g., steel bars to become reinforced concrete (RC) with improved tensile properties. This constructive solution has been used for several decades and the growing need to extend and/or maintain the service life of these structures has led to significant investments in the strengthening of RC structures. Fiber Reinforced Polymer (FRP) composites can be used to strengthen RC structures. In particular, carbon FRP (CFRP) can be a good option, owing to its advantageous properties including high strength to weight ratio, high durability, high fatigue resistance [1], and high corrosion resistance [2], among others.

CFRP composites in the form of laminate strips and sheets have generally been applied for flexural and shear strengthening of existing RC structures, respectively, where a bonding agent, such as epoxy adhesive, is typically used to bond the CFRP to concrete substrate. Near-surface mounted (NSM) is known as one of the existing strengthening techniques [3], and its durability is mainly addressed in the present work. During application of the NSM technique, the CFRP strip is basically inserted into a groove pre-cut on concrete cover [4], and is bonded to concrete with an adhesive. The NSM technique is more recent and has been repeatedly reported to possess more advantages than other techniques, such as, the externally bonded reinforced (EBR) technique [5–8]. Furthermore, NSM can additionally be prestressed, mainly to benefit from the full usage of the CFRP strain, which leads to efficient use of materials, thereby resulting in reduced crack width and increased cracking

and yield response [9,10]. However, the durability of the bond in NSM technique, either non-prestressed or prestressed, still lacks literature under both accelerated aging test (AAT) and natural aging test (NAT) protocols. Besides, the durability of the materials constituting the bond in NSM technique (i.e., concrete, epoxy adhesive, and CFRP) has also not been fully investigated yet, considering NAT protocols (e.g., [11,12]), thus there is a need to conduct further related research.

A significant number of studies have been conducted on the AAT of the NSM CFRP-to-concrete bond constituent materials. Starting from concrete, carbonation of concrete is an important durability factor. Studies show that high temperatures can make concrete more porous thereby leading to carbonation depth increase, which thereby can improve concrete properties [13–15], although the carbonation is known to have negative effects on the reinforcing steel bar [16]. In contrast, low temperatures and high relative humidity (RH) [17], as well as continuous cement hydration (leading to micropore closure) [18] can hinder the carbonation depth increase. On the other hand, carbonation depth can be high if concrete is exposed to environments with 50-70 % RH [13] (the optimum increase being at 65 % RH [17]). In a study by [19], both concrete elastic modulus and compressive strength increased after carbonation. Besides, only the compressive strength of concrete increased after exposure to carbonation in [18], and to ultraviolet (UV) radiation at certain humidity and temperature in [20]. Studies have also shown that when concrete is exposed to both carbonation and chlorides, the carbonation can release bound chlorides inward at greater depths [21,22], thereby minimizing the effects of chlorides at the concrete surface and hence protecting the concrete surface region, where FRP is normally bonded during strengthening applications.

In studies addressing the durability of epoxy adhesives, it has been found that the properties of the adhesives reduce significantly after exposure to moisture or in the case of full immersion in water [11,23]. Wet-dry cycles were also found to decrease both the tensile strength and elastic modulus of the adhesive [24]. However, exposure to high temperatures improved the adhesive properties through post-curing phenomenon [25], while temperatures close to the glass transition temperature resulted in softening of the polymeric matrix [11]. Besides, high carbonation can accelerate the curing of the epoxy resin [26]. However, exposure of the adhesive to chlorides has no harmful effect on the adhesive properties [27].

Regarding the durability of CFRP composites, studies show that CFRP is generally not affected by degradation agents such as chloride exposure [11], and thermal cycles (TC) in air [28]. However, freeze-thaw (FT) cycles

may reduce both tensile strength and elongation of CFRP [29], UV radiation can affect a few microns from the CFRP surface [30], and some matrix microcracks may form due to different thermal expansion coefficients between the fiber and matrix after exposure to TC.

It is worth noting that existing literature on the durability of the bond constituent materials has generally been conducted using AAT protocols and basically addresses the effect of a single degradation agent; however, in outdoor environment the materials are exposed to more than one degradation agent simultaneously. Hence there is a need to conduct studies that address the combined or synergic effect of different agents to be able to mimic what normally occurs in outdoor environments.

Regarding the NSM CFRP-to-concrete bond, there are some studies [7,31–33] that address the durability of the bond under AAT conditioning. In [31], 90 wet-dry cycles lasting for 2160 hours were applied on strengthened concrete elements with 60 mm and 90 mm bond lengths, 1.4 mm CFRP laminate thickness, 15 mm and 25 mm groove depths, and 4 mm and 8 mm groove widths. The bond strength generally decreased after exposure, except for specimens with 25 mm groove depth and 4 mm groove width. In [33], conditioning specimens to 300 freeze-thaw cycles or immersion in salt water at 20 °C decreased the bond strength for concrete elements strengthened with a 1.3 mm CFRP laminate thickness. In [7], immersion in tap water at 22 °C (or tap water with 3.5 NaCl) for 5760 hours and 11520 hours increased the bond strength of concrete elements strengthened with 1.4 mm CFRP laminate thickness. Furthermore, applying 240 or 480 wet-dry cycles on the specimens in tap water (or in tap water with 3.5 NaCl) also increased the bond strength. Besides, conditioning the specimens with temperature cycles between -15 °C and +60 °C increased the bond strength; however, temperature cycles between +20 °C and +80 °C did not affect the bond strength. Finally, conditioning the specimens to 120 freeze-thaw cycles between -18 °C to +20 °C for 5760 hours also did not affect the bond strength, but doubling the number of freeze-thaw cycles between -18 °C to +20 °C for 4320 hours led to a slight decrease in the bond strength. On the other hand, a few studies, e.g., [8,34,35] addressed the durability of NSM with NAT protocols. In these NAT-based studies, it was generally found that the NSM CFRP-to-concrete bond is significantly affected by moisture and freeze-thaw attack.

Based on the above mentioned literature on both the NSM CFRP-to-concrete bond and the material constituting the bond, it can be noted that there is a knowledge gap in several aspects, namely: (i) existing durability data are largely based on the AAT protocols with very little knowledge on the behavior of the bond and materials under real outdoor environments; (ii) existing data, apart from being heavily dependent on the AAT protocols,

are still insufficient to lead to appropriate predictions of environmental conversion factors (ECF); and, (iii) currently some standards [36,37] recommend ECFs to account for degradation of epoxy/carbon systems due to some generic exposures; however, these ECFs are neither derived from sufficient data, nor from data including both AAT and NAT protocols. Furthermore, there is no information about the factors for specific types of exposures, which is generally the real situation for most of the RC strengthened structures.

Regarding the above aspects, the present work intends to contribute to increasing the number of existing data related to the durability of NSM CFRP-to-concrete bond and attempt to address the above-mentioned aspects. This is achieved by firstly presenting durability data from AAT (laboratory-controlled environments: specimens conditioned at 20 °C/55 % RH or immersed in water at 20 °C) and NAT (outdoor environments: specimens kept in outdoor regions with characteristics mainly promoting carbonation, elevated temperatures, freeze-thaw attacks, and airborne chlorides) protocols. Secondly, the ECF from a database of existing AAT data and the ECF from NAT data in the present work, are derived and compared.

2. EXPERIMENTAL PROGRAM

This section first describes the properties of the materials composing the NSM CFRP-to-concrete bond followed by detailing the environments studied, and, finally, the testing methods used.

2.1. Constituent materials

Three different materials are involved in the bond of NSM CFRP-to-concrete system: the concrete, the epoxy adhesive and the CFRP laminate. Details of each are given below.

Concrete: The concrete used in the preparation of all specimens was in accordance with the recommendations as per [38]. All concrete properties were as shown in **Table III.1**. Concrete cylinders (**Figure III.1a**) were produced and used to investigate the variation of concrete elastic modulus (E_c) and compressive strength (f_{cc}). Concrete prisms (**Figure III.1b**) were used to assess the variation of concrete pull-off strength (f_{ct}) and the concrete carbonation depth (C_d).

Epoxy adhesive: A two-component commercial cold-curing epoxy resin-based adhesive with properties as shown in **Table III.1** was used to cast the dog-bone shaped specimens (**Figure III.1c**) and to investigate the variation of the adhesive elastic modulus (E_a) and tensile strength (f_a). This adhesive was then used as a bonding agent in NSM CFRP-to-concrete bond specimens. The adhesive is a solvent-free, thixotropic, grey colour, developed

for bonding carbon fiber laminates to concrete substrates. The properties of this adhesive according to the supplier [39] are shown in **Table III.1**.

Table III.1. Properties of the NSM CFRP-to-concrete bond specimens and their constituent materials.

Concrete		Epoxy adhesive	
Concrete class	C30/37 (cylinder/cube in [MPa])	Type of adhesive	Cold-curing S&P Resin 220
Max aggregate size [mm]	12.5	Density, at 23 °C [g/cm ³]	1.7-1.8
Cement type	CEM II/A-L 42.5R	T_g [°C]	> 7.1
Slump [mm]	160-210 (slump class S4)	Tensile strength [MPa]	19.9 (after 7d curing at 20 °C)
Water-to-cement ratio	0.4	Flexural elastic modulus [GPa]	46.2 (after 7d curing at 23 °C)
Exposure class	XC4(P)	Compressive strength [MPa]	> 70
E [GPa]	29.1 (at 28 days)	Shear strength [MPa]	> 26
f_c [MPa]	41.5 (at 28 days)	BS by pull-off, on concrete [MPa]	3 (after 3d curing at 20 °C)
CFRP laminate		CFRP-to-concrete bond	
Type and trademark	S&P clever (CFK 150/2000)	CFRP cross-section [mm ²]	10 × 1.4
Prefabricated by	Pultrusion	Concrete cube dimensions [mm ³]	200 × 200
Fiber orientation	Unidirectional	Bond length [mm]	60
Fiber content [%]	68	Groove depth [mm]	15
Fiber matrix	Vinyl ester resin (with $T_g \approx 85$ °C)	Groove width [mm]	5
External surface	Black, smooth	Epoxy adhesive thickness [mm]	5 / 3.6 at the CFRP level
Elastic modulus [GPa]	> 170	Epoxy adhesive depth [mm]	15
Tensile strength [MPa]	> 2000	Type of test	Single-lap shear test

Notes: f_c : average compressive strength; E : average elastic modulus; BS : bond strength; T_g : glass transition temperature; 3d: three days; 7d:

seven days.

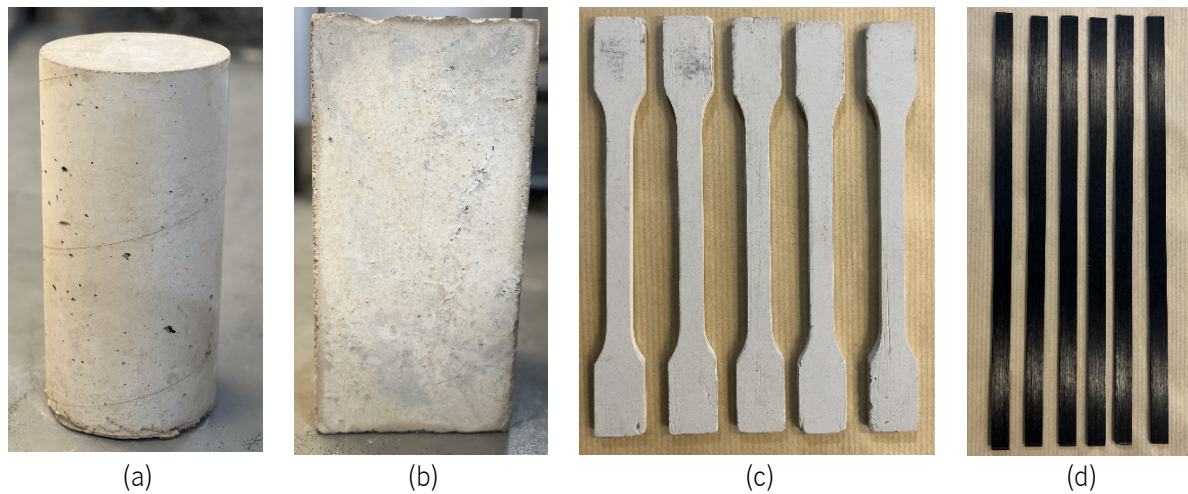


Figure III.1. NSM CFRP-to-concrete bond constituent materials: (a) concrete cylinder, (b) concrete prism, (c) epoxy adhesive dog bone shaped specimens and (d) CFRP laminate strips.

CFRP laminate: The CFRP laminate (**Figure III.1d**) produced by S&P [40] was used to strengthen the concrete elements and its properties are as shown in **Table III.1**. This CFRP laminate has a rectangular cross-section of 10 mm × 1.4 mm (width × thickness). Besides, the variations of both its elastic modulus (E) and tensile strength (f_t) were also investigated using CFRP strips.

NSM CFRP-to-concrete bond: The properties of the specimens used to characterize the bond between the CFRP laminate and concrete are also shown in **Table III.1**.

2.2. Environmental exposure conditions

Six different environmental exposure conditions (E1-E6) were used to study the degradation of both the NSM CFRP-to-concrete bond and its constituent materials. Different characterization tests were performed on the specimens before exposure (referred to as T0) and on the specimens collected from the different environments after one year (T1), two years (T2), three years (T3) and four years (T4) of exposure. Description of the exposure conditions in each environment is provided as follows.

2.2.1. Laboratory-controlled environments

Two laboratory-controlled environments were considered. In the first environment, denoted as E1, the specimens were kept in an in-house walk-in climatic chamber with a targeted temperature and relative humidity of 20 °C and 55 % RH, respectively. The second environment, denoted as E2, consisted of specimens fully and continuously immersed in water at 20 °C (see **Figure III.2a** and **Figure III.2b**). The specimens in E2 were tested under wet state. More details about these two environments can be found in [8,11].

2.2.2. Natural outdoor environments

Four different outdoor environments were selected in different regions of Portugal with different meteorological conditions. These environments were selected with the aim of promoting the degradation of the NSM CFRP-to-concrete bond and its constituent materials. The first outdoor environment, denoted as E3, was chosen to mainly promote concrete carbonation, by exposing the specimens to air pollution (all along the year) from highway with heavy traffic and the international airport of Lisbon. The second environment (E4) was chosen to mainly promote the freeze-thaw attacks, and therefore, the specimens were placed on the highest mountain of Portugal (at the altitude of 1600 meters) with some seasonal snowfall. The third environment (E5) was chosen to mainly promote the degradation due to high temperatures, so the specimens were placed in a region with yearly elevated temperatures (**Figure III.2c** and **Figure III.2d**). The fourth environment (E6) was chosen to be close to the Atlantic Ocean to promote airborne chloride attack, and the effects of high seasonal humidity.

Although these outdoor environments were chosen to primarily promote the mentioned degradation agents, the specimens kept outdoors are likely to be affected by more than one degradation agent. For example, in E3, in

addition to carbonation, high temperatures can also play an important role as the region generally experiences hot weather throughout the year. Typical variations in temperature and relative humidity in the laboratory and outdoor environments are shown in **Figure III.3**. Particularly, in E1, the relative humidity varies in the range of 45-77 % RH, which promotes concrete carbonation [13,17]. The RH variations are mainly due to inevitable opening/closing the door of the climatic chamber for maintenance and data collection and fall within the moderate humidity range (exposure class XC3), according to the Eurocode 1992-1-1:2004 (E) [41]. These actions may have promoted the flow of CO₂ into the climatic chamber from the lab environment. Furthermore, a significant difference in terms of temperature and relative humidity can also be observed in outdoor environments (E4 and E5). More details can be found in [42], where a general view is that E4 and E6 tend to experience low temperatures and high relative humidity; while E3 and E5 experience high temperatures and low relative humidity. The variation of annual temperatures and relative humidity is tabulated in **Table III.2**, where it can be noted that the maximum temperatures from E3 and E5 were the highest, as expected due to their high daily temperatures as compared to other environments.

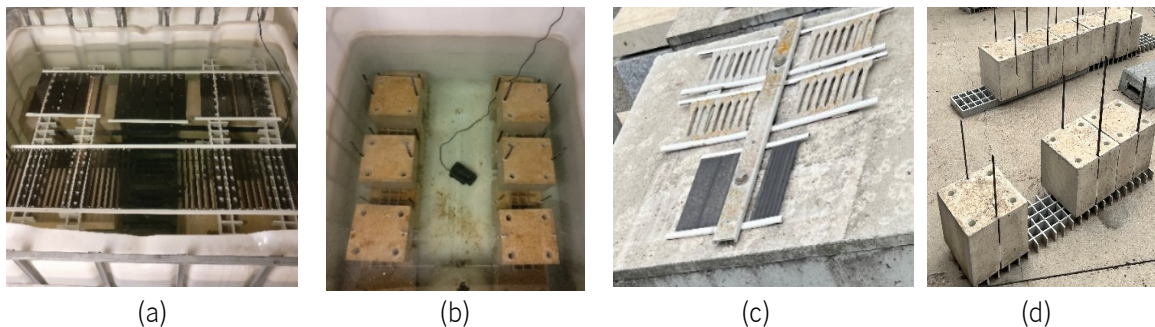


Figure III.2. Typical examples of studied environments: (a) exposure of the constituent materials specimens to water immersion (concrete, epoxy adhesive and CFRP laminate); (b) exposure of the specimens of NSM CFRP-to-concrete bond to water immersion (E2); exposure of (c) constituent materials and (d) NSM CFRP-to-concrete bond specimens to natural outdoor environments (E5).

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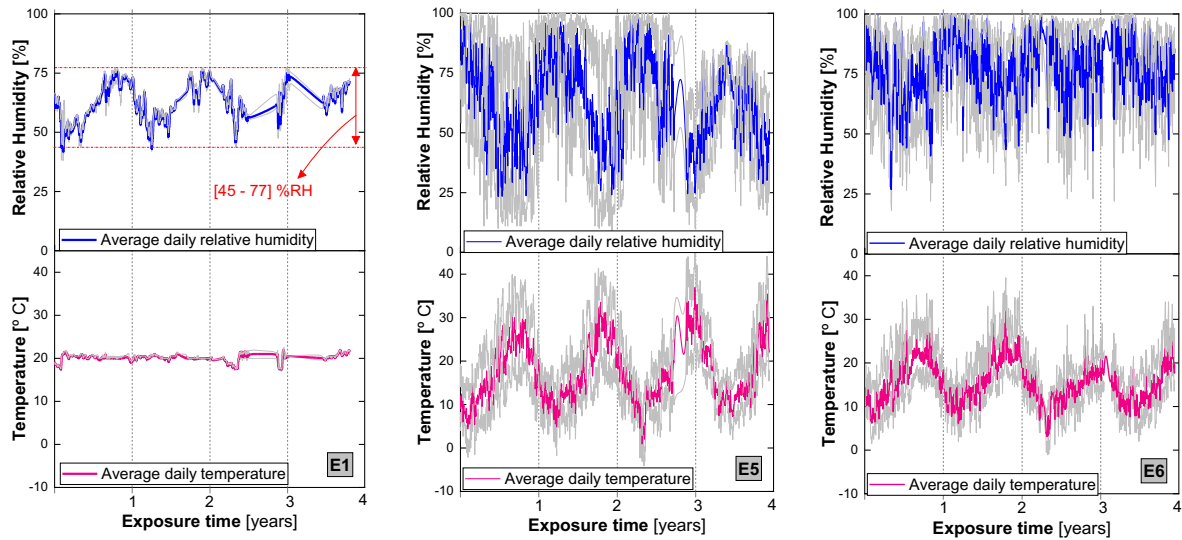


Figure III.3. Typical meteorological records from laboratory (E1) and outdoor (E5, E6) environments.

Table III.2. Variation of annual temperatures and relative humidity in the studied environments.

Variable / Environment		E1	E2	E3	E4	E5	E6
Temp [°C] Year 1 (T1)	Max	20.2 (22.5)	20.2 (20.3)	23.1 (46.2)	17.8 (32.4)	24.0 (44.6)	21.6 (36.0)
	Min	19.8 (13.5)	20.1 (19.0)	13.4 (3.3)	11.1 (-4.7)	11.1 (-1.9)	11.8 (1.5)
	Avg	19.9	20.0	17.4	14.3	17.4	16.15
RH [%] Year 1 (T1)	Max	62.8 (79.5)	100.0	87.9 (100.0)	83.3 (100.0)	84.9 (100.0)	88.7 (100.0)
	Min	57.8 (31.0)	100.0	48.8 (11.0)	50.9 (4.0)	40.3 (9.0)	59.8 (18.0)
	Avg	60.0	100.0	70.7	67.2	63.5	76.1
Temp [°C] Year 2 (T2)	Max	20.4 (22.0)	20.2 (20.4)	22.7 (39.7)	14.0 (29.6)	23.6 (39.9)	22.5 (39.5)
	Min	20.1 (19.0)	20.5 (19.5)	13.9 (4.3)	7.4 (-4.6)	8.9 (0.3)	13.0 (2.0)
	Avg	20.2	20.6	17.5	10.6	17.3	17.2
RH [%] Year 2 (T2)	Max	63.1 (77.5)	100.0	90.1 (100.0)	87.6 (100.0)	87.9 (100.0)	90.5 (100.0)
	Min	60.1 (41.5)	100.0	52.9 (16.0)	54.7 (4.0)	42.0 (11.0)	63.5 (28.5)
	Avg	61.8	100.0	74.4	72.8	67.1	78.6
Temp [°C] Year 3 (T3)	Max	20.1 (22.0)	21.0 (22.0)	22.8 (40.7)	13.2 (29.5)	22.8 (41.4)	16.9 (31.1)
	Min	19.8 (17.0)	20.2 (19.6)	13.5 (1.4)	4.0 (-6.4)	11.4 (-4.1)	9.1 (-1.1)
	Avg	19.9	20.3	17.3	9.7	17.1	13.0
RH [%] Year 3 (T3)	Max	63.9 (77.5)	100.0	87.9 (100.0)	86.0 (100.0)	86.2 (100.0)	93.0 (99.0)
	Min	61.7 (43.0)	100.0	48.8 (16.0)	50.5 (6.0)	45.6 (9.0)	61.5 (23.0)
	Avg	62.9	100.0	70.7	71.3	66.5	79.4
Temp [°C] Year 4 (T4)	Max	20.5 (23.0)	20.2 (20.3)	-	20.7 (31.1)	25.5 (46.0)	19.9 (34.0)
	Min	20.1 (17.0)	20.3 (19.5)	-	12.7 (2.0)	13.9 (2.0)	11.1 (2.0)
	Avg	20.3	20.4	-	16.5	19.3	15.1
RH [%] Year 4 (T4)	Max	62.8 (77.5)	100.0	-	83.3 (100.0)	84.9 (88.5)	88.7 (100.0)
	Min	57.8 (47.0)	100.0	-	50.9 (4.0)	40.3 (10.0)	59.8 (22.0)
	Avg	60.0	100.0	-	67.2	63.4	76.1

Notes: RH: Relative humidity; Temp: Temperature; Max/Min/Avg: yearly maximum/minimum/average value; the value in parentheses stands for the annual peak value.

2.3. Characterization test methods

This section provides information about the tests adopted for the characterization of both the NSM CFRP-to-concrete bond and its constituent materials. In all the cases, specimens were tested before aging (T0) and after being exposed to the environments during one (T1), two (T2), three (T3) and four (T4) years. In the case of environment E2, all the specimens were tested in wet state.

2.3.1. Concrete

A universal testing machine (UTM) with a maximum load capacity of 2000 kN was used to perform compression tests for the concrete elastic modulus (E_c) and compressive strength (f_{cc}). Under non-destructive test (**Figure III.4a**), three LVDTs spaced at 120 ° were used to measure the data required to estimate the E_c according to EN 12390-13:2013 [43]. Furthermore, a destructive test (**Figure III.4b**) was performed on the same specimen to determine the f_{cc} according to NP EN 12390-3:2011 [44]. A total of 75 specimens were tested to determine both the E_c (series of 2 specimens) and the f_{cc} (series of 3 specimens). Additionally, the pull-off test was performed using the DYNA Z5 testing machine according to EN 1542:1999 [45]. After allowing the epoxy adhesive to cure for 7 days at room temperature, metal dollies bonded to the concrete cores (using the epoxy adhesive) were manually loaded (**Figure III.4c**) to pull-off the formed cylinder, until the split occurred (**Figure III.4d**). A total of 100 specimens were tested (series of 4 tests). Further details of this type of test can be found in [8].

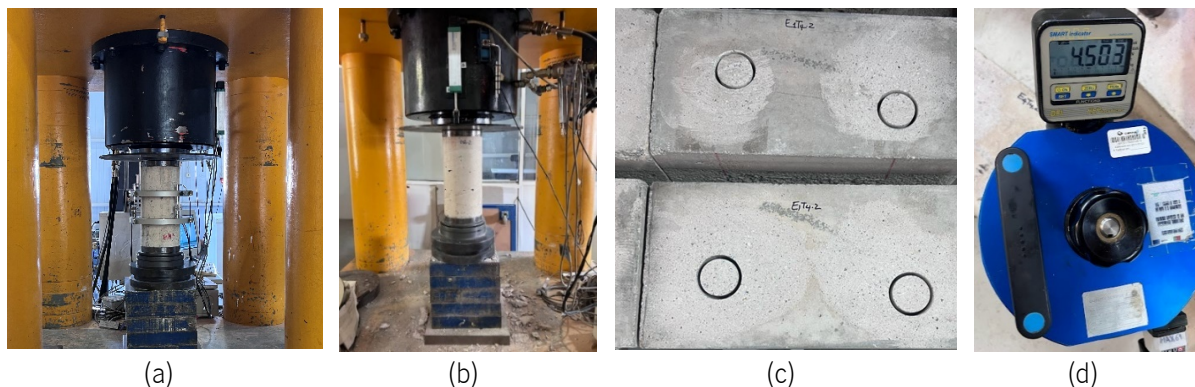


Figure III.4. Tests for concrete characterization: (a) elastic modulus setup; (b) compression tests setup; (c) sandblasting and creating the cylindrical cores; (d) concrete pull-off test machine.

In addition to these tests, the carbonation ingress of concrete was also evaluated by spraying phenolphthalein indicator on the cylindrical cores drilled from concrete prisms (series of 4 specimens). After spraying, the clear

areas of the specimens indicated the concrete carbonated areas, while the purple-red areas indicated the concrete non-carbonated areas. These tests were performed according to [8], to measure the carbonation depth.

2.3.2. Epoxy adhesive and CFRP laminate

Tensile tests of both epoxy adhesives and CFRP laminates were performed using MTS UTS machine (**Figure III.5a**). A total of 125 epoxy adhesive specimens (series of at least 5 specimens) were tested according to EN ISO 527-2:2012 [46] as shown in **Figure III.5b** and the elastic modulus was determined according to EN ISO 527-2:2012 [46] by calculating the slope of the secant line on the stress-strain curve between 0.05 % and 0.25 % of the strains. On the other hand, a total of 150 CFRP laminate specimens were also tested in the same configuration (**Figure III.5c**) as that of the adhesives and according to EN ISO 527-5:2009 [47].

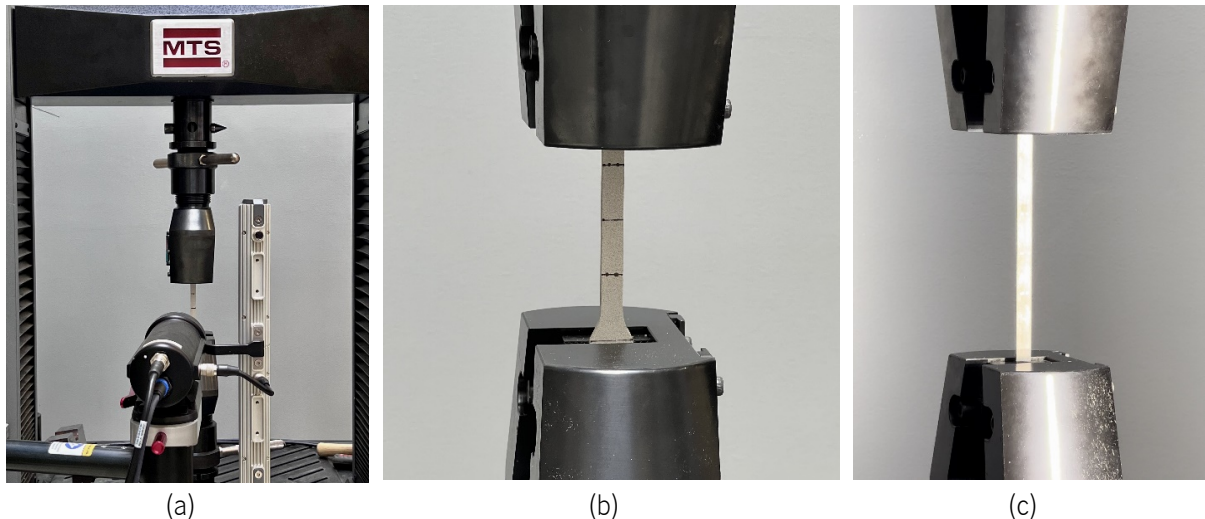


Figure III.5. Tensile tests using an MTS UTS machine: (a) overview of MTS with video extensometer; (b) epoxy adhesive test; (c) CFRP laminate tensile test.

2.3.3. NSM CFRP-to-concrete bond

A total of 60 concrete cubes, each with 200 mm, were used to prepare the specimens strengthened according to NSM technique. The concrete cubes were strengthened with CFRP strips, each being strengthened by 2 CFRP laminates with a bond length of 60 mm (**Figure III.6a**). For that purpose, two opposed faces parallel to the casting direction were used. The bond length adopted aimed at avoiding the CFRP laminate failure and also to be large enough to represent the system and minimize some inevitable effects such as geometric irregularities [8]. Two of these concrete blocks (4 strengthening systems) were tested at the beginning to serve as reference

specimens, whereas the remaining ones were exposed to different environments and tested after exposure. A total of 100 pull-out tests were performed.

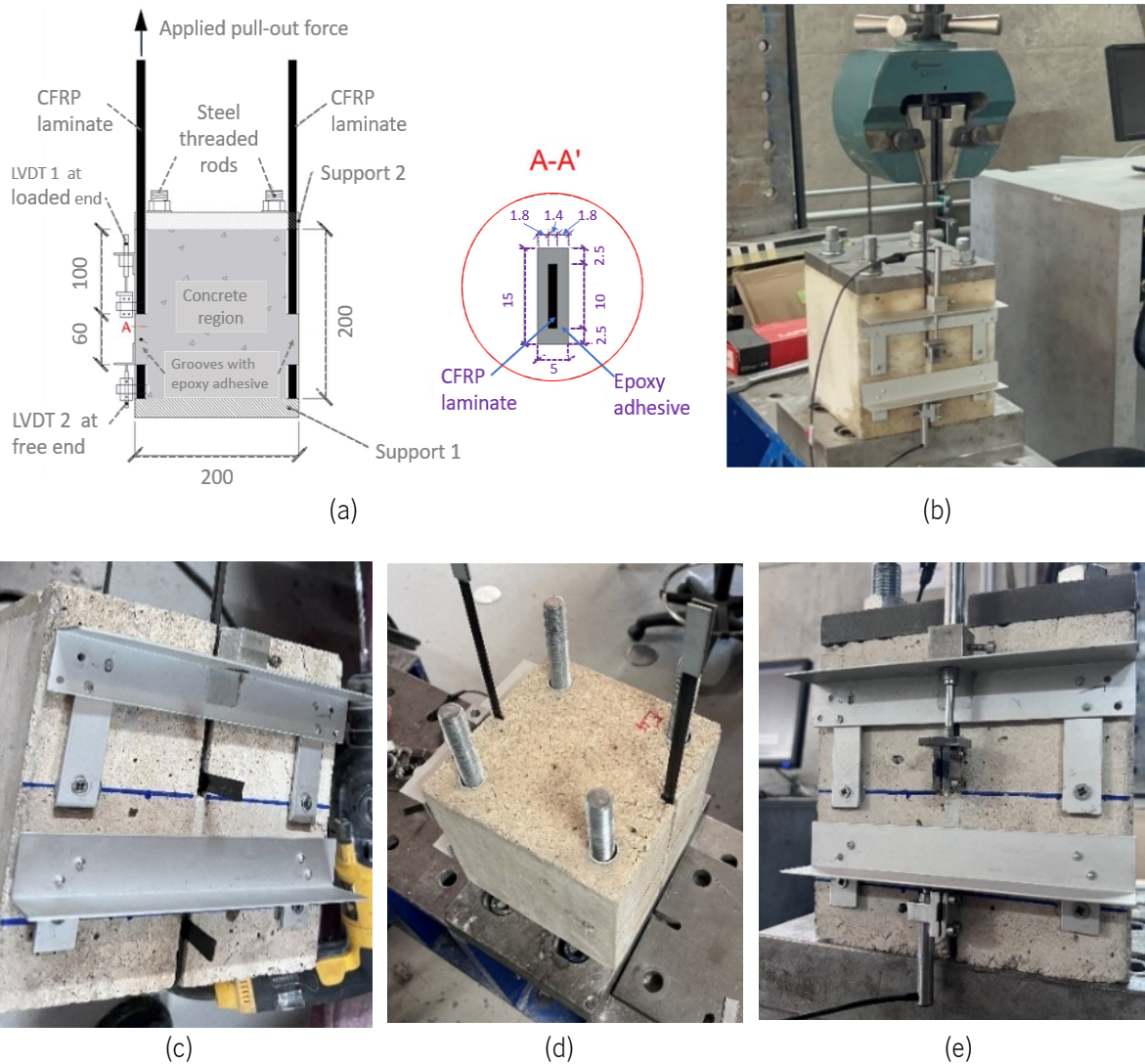


Figure III.6. NSM specimen and single-lap shear test setup: (a) specimen's geometry and test configuration (NSM); (b) photograph of the test; (c) mounting LVDTs supports; (d) installing the specimen; (e) positioning of LVDTs. Notes: all units in [mm].

The test setup is shown in **Figure III.6b**, from which the main parts can be highlighted: (i) LVDT support fixation (**Figure III.6c**); (ii) horizontal and vertical movements restraint (**Figure III.6d** and **Figure III.6e**); and (iii) LVDT installation (**Figure III.6e**). These tests were conducted on four specimens each year (i.e., from T1 to T4). A servo-controlled equipment was used, with the applied force measured by a load cell with a maximum capacity

of 200 kN. The tests were performed under displacement control at the loaded end with a rate of 2 $\mu\text{m/s}$, using a displacement measured through the LVDT1 (**Figure III.6a**) placed at the loaded end section as a control variable.

3. RESULTS AND DISCUSSION

This section presents the results from the different tests performed (as described in **Section 2**), followed by a comprehensive analysis and discussion. Furthermore, comparative studies are also performed between the results from this work and those from the existing literature to derive and propose the environmental conversion factors.

3.1. Test results from the bond constituent materials

3.1.1. Concrete

Concrete compressive strength (f_{cc}), elastic modulus (E_c), pull-off strength (f_a), and carbonation depth (C_a) for all testing time series (T0-T4) and all studied environments (E1-E6) are plotted in **Figure III.7a** while the values are shown in **Table III.3**. The results show that the concrete f_a decreased in all studied environments. On the other hand, both f_{cc} and E_c generally improved with time in all outdoor environments. The highest improvements in f_{cc} and E_c were found in E4 and E6 at T2, with increases of 21.2 % and 12.0 % in E4, and 20.9 % and 12.0 % in E6, respectively. The observed increase of f_{cc} and E_c in different environments can be attributed to different factors such as carbonation, temperature variations and continuation of cement hydration. In fact, the C_a is observed to increase linearly with time in E1 (**Figure III.7a**) due to the relative humidity range [45-77 % RH], see **Figure III.3**, which is already known to favor the ingress of CO_2 [48].

However, trends with lower C_a values can be observed from E2 and E4 probably due to increased humidity and moisture in these two environments. This agrees well with the findings by [49] who reported that the increased humidity, rain, and snow slowed down the ingress of CO_2 after exposing concrete to outdoor environments for up to 4 years. Most importantly, the specimens in E2 and E4 are generally found to be partially carbonated (**Figure III.8a**). Partial carbonation has negligible effects on the concrete properties [50,51] for high relative humidity environments, hence the effects of carbonation in E2 and E4 on the f_{cc} and E_c are negligible. The highest C_a is observed in E3 as expected although there is a surprising decrease in the last year T4 that may be due to climate change issues. Besides, even in other outdoor environments (E5, E6) the C_a is observed to be

comparable to that in E3, which may reflect the effects of climate change that causes the abundance of CO_2 in the atmosphere. On the other hand, the observed increase in f_{cc} and E_c in E6 may be further attributed to the synergistic effect between carbonation and chlorides as previously reported in [21].

Studies also show that high temperatures can make concrete more porous thereby increasing the CO_2 penetration [13,14], which may be the reason for the high C_d observed in E5. Hence, the presence of carbonation in E3, E5, E6 can be considered as one of the possible factors that increased the concrete f_{cc} and E_c . This is in line with other previous studies where carbonation was found to increase the f_{cc} [18] and both the f_{cc} and E_c [19]. On the other hand, as previously described for outdoor environments, high temperatures with low RH were recorded in E3 and E5, so this may also have played a major role in increasing the f_{cc} and E_c . This is in agreement with a study by [52], where high temperature variations increased the f_{cc} . Hence, it can be noted that there is a synergistic effect between high temperatures and carbonation, but high temperatures may have played a greater role than the presence of carbonation due to the inconsistent trend of the latter, as shown in Figure III.7a.

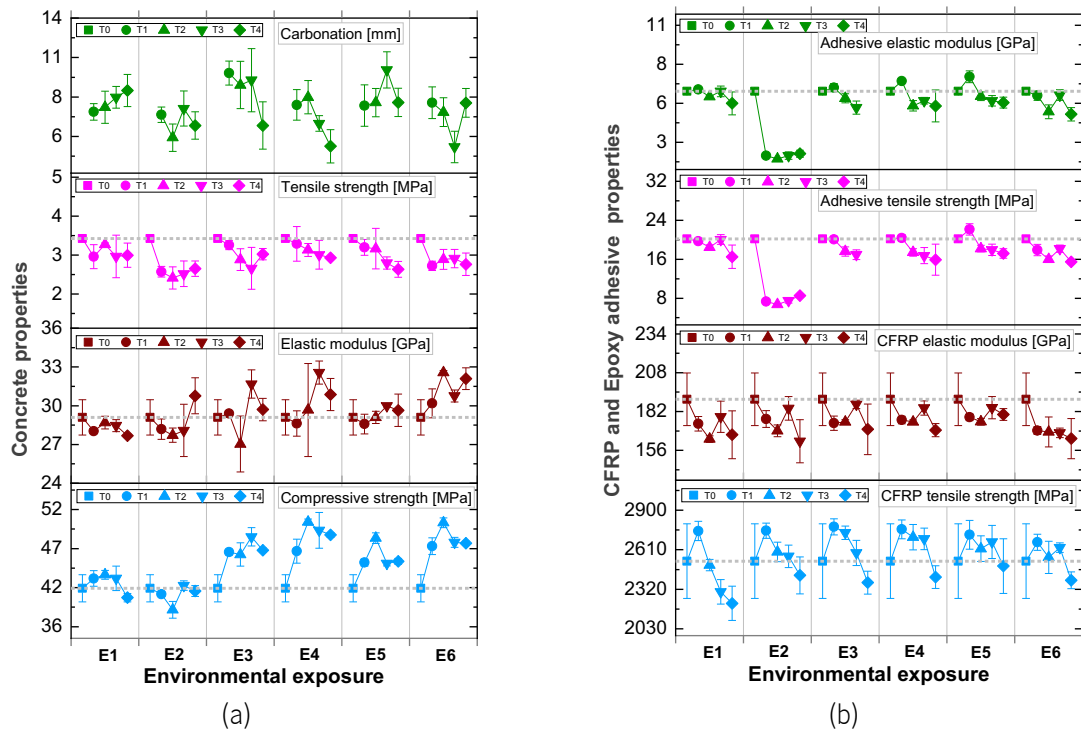


Figure III.7. Variation of material properties over time under different environmental exposures from 0 to 4 years of exposure: (a) concrete and (b) epoxy adhesive and CFRP laminate.

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Contrary, the effects of temperatures on the increase in the f_{cc} and E_c may be thought negligible in E4 and E6 as these environments experienced lower temperature values, hence other factors may have been attributed to the observed increased concrete properties. For example, as a result of the high RH in E4 and E6, continued hydration of cement may occur over time, with the increase of the number of C-S-H silicates, which then occupy the empty voids and push out the diffused moisture in the voids, thereby reducing the porosity and improving concrete properties. Increase of the C-S-H silicates is already known to improve the f_{cc} [53,54]. Increased curing moisture has also been found to increase the f_{cc} and E_c [19].

Table III.3. Average values of concrete compressive strength, elastic modulus, pull-off strength, and carbonation depth after 0 (T0), 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of different environmental exposures (E1 to E6).

Environment	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
Compressive strength f_c [MPa]						Elastic modulus E_c [GPa]				
REF	41.5 (4.4)	–	–	–	–	–	–	–	–	–
E1	–	42.8 (2.4)	43.3 (1.4)	42.8 (3.8)	40.3 (1.4)	29.1 (5.0)	28.0 (0.7)	28.7 (1.7)	28.5 (1.6)	27.7 (0.4)
E2	–	40.7 (0.7)	38.7 (2.9)	41.9 (1.5)	41.1 (1.8)	–	28.2 (2.8)	27.7 (2.0)	28.1 (7.2)	30.8 (4.5)
E3	–	46.3 (0.9)	46.0 (3.4)	48.4 (2.5)	46.6 (0.6)	–	29.4 (0.8)	27.0 (8.0)	31.7 (3.4)	29.7 (2.9)
E4	–	46.5 (3.4)	50.3 (0.8)	49.2 (4.8)	48.6 (0.7)	–	28.6 (3.4)	29.7 (12.1)	32.6 (2.7)	30.9 (4.0)
E5	–	44.9 (1.0)	48.2 (1.5)	44.8 (0.7)	45.1 (0.9)	–	28.6 (2.6)	29.1 (1.6)	30.0 (0.3)	29.6 (4.2)
E6	–	47.1 (2.4)	50.2 (1.3)	47.6 (1.4)	47.5 (0.5)	–	30.2 (3.7)	32.6 (0.5)	30.8 (1.5)	32.1 (2.6)
Pull-off strength f_a [MPa]						Carbonation depth C_a [mm]				
REF	3.4 (13.3)	–	–	–	–	0.0	–	–	–	–
E1	–	2.9 (10.4)	3.2 (1.3)	2.9 (18.4)	2.9 (10.5)	–	7.4 (19.9)	7.7 (14.8)	8.4 (9.2)	8.9 (12.8)
E2	–	2.5 (5.3)	2.3 (11.8)	2.5 (13.1)	2.6 (7.4)	–	7.2 (15.2)	5.5 (17.7)	7.6 (16.4)	6.4 (15.3)
E3	–	3.2 (3.6)	2.8 (9.7)	2.6 (20.1)	3.0 (4.7)	–	10.1 (5.5)	9.3 (18.2)	9.6 (23.4)	9.0 (18.8)
E4	–	3.2 (13.5)	3.1 (5.3)	2.9 (12.2)	2.9 (0.1)	–	7.8 (7.5)	8.4 (14.1)	6.5 (8.4)	4.9 (24.1)
E5	–	3.1 (6.3)	3.1 (16.4)	2.7 (5.7)	2.6 (7.7)	–	7.8 (10.9)	8.0 (12.5)	10.3 (12.4)	8.0 (12.5)
E6	–	2.7 (4.4)	2.8 (8.8)	2.9 (8.1)	2.7 (10.5)	–	8.0 (14)	7.3 (13.9)	4.9 (22.9)	8.0 (12.8)

Notes: all values in parentheses express coefficient of variation; REF: Reference values from the specimens tested before exposure (i.e., at T0).

To summarize, the above results show that the increase in concrete properties can result from a combination of more than one factor. In fact, these can be the leading factors in each environment: (i) carbonation in E1; (ii) the competing mechanisms between the increased porosity *versus* cement hydration in E2; (iii) the synergistic effect between carbonation and high temperatures in E3, (iv) the synergistic effect between the continuation of cement hydration and increased humidity in E4; (v) the synergistic effect between high temperatures and carbonation in E5; and, (vi) the synergistic effect of carbonation and chlorides and continuation of cement hydration in E6. Furthermore, comparing the degradation agents, the presence of cement hydration resulting from high relative humidity regions leads to the highest improvement in the concrete f_{cc} and E_c .

The typical failure modes from concrete compression and tensile tests are shown in **Figure III.8b**, **Figure III.8c** and **Figure III.8d**, respectively. In particular, it can be seen that the failure mode from tensile tests occurs deep inside the concrete, as mentioned earlier, which can lead to misleading values since the inner region is ideally not exposed. Finally, the duration and severity of exposure may have a greater impact on the carbonation depth and elastic modulus of the concrete compared to other properties, due to significant fluctuations rather than a gradual increase or decrease over time. It is possible that the exposure duration was shorter or longer in year i than in year $i-1$ (for $i = 0, \dots, 4$), which could have caused these fluctuations. Similarly, fluctuations may also result from differences in exposure severity (i.e., more severe exposures in year i can be thought to lead to higher changes in the properties than in year $i-1$).

3.1.2. Epoxy adhesive

The average values of the tensile strength (f_a) and average elastic modulus (E_a) of the epoxy adhesive, for all testing times and all environments are shown in **Figure III.7b**, their values in **Table III.4**, and their failure modes in **Figure III.8e**.

The E_a increased from T0 to T1 in E1, E3, E4, and E5, mainly due to the post-curing phenomenon. However, there was a significant decrease in the E_a in E2 which can be attributed to the plasticization effect, and a slight decrease in E6 due to moisture, as the effects of chlorides on the adhesive properties are negligible [27]. Furthermore, a general decrease in the E_a can be observed during the following consecutive years, except for E2 that showed marginal variations after T1. The adhesive f_a shows a similar trend as that of the E_a ; however,

the effects of the post-curing phenomenon are more pronounced for the latter. The highest decreases in E_a and f_a are found from the specimens immersed in water (E2), with reductions of 75.4 % (at T2) and 66.3 % (at T2), respectively, compared to the value recorded at the initial stage (T0). The reasons for the observed decrease in both the E_a and f_a may be as follows: (i) marginal variations in E1 are due to the effects of increased carbonation, which tends to cause the epoxy resin to cure faster as previously observed in [26], (ii) negative effects of water ingress in E2 and moisture effects in both E4 and E6, which was also found by [11,23], also, in a study by [24], where both the adhesive E_a and f_a decreased due to exposure to wet-dry cycles (case of E4), (iii) in E3, temperatures higher than T_g and UV (ultra-violet) radiation can be considered as the main factors that deteriorated the properties of the epoxy adhesive. The failure mode (**Figure III.8e**) of the adhesive was always an abrupt break in the testing zone of constant cross-section.

3.1.3. CFRP laminate

The average values of the tensile strength (f_t) and average elastic modulus (E_t) of the CFRP laminate, for all testing times and all environments are shown in **Figure III.7b**, their values in **Table III.4**, and their failure modes in **Figure III.8f** to **Figure III.8h**.

The CFRP laminate increased both its elastic modulus (E_t) and tensile strength (f_t) as compared to the initial value at T0 (**Figure III.7b**) and **Table III.4**. This is consistent with the existing literature under AAT where it was reported that CFRP is generally not affected by environmental exposure [11]. The highest increase is generally found at T1, which reflects the effects of post-curing of the fiber resin matrix, followed by a general decrease in subsequent years, but still with higher values than those at T0, except for E5 where the post-curing phenomenon can be expected to still improve both E_t and f_t after 4 years of exposure. The highest values of E_t are found for the specimens in E5 with 10.4 % (at T3) and in E3 with 9.8 % (at T4) increase, while the highest values of the f_t are found in E3 with 16.1 % (at T1) and E4 with 14.6 % (at T1), compared to the value recorded at the initial stage (at T0).

Table III.4. Average values of CFRP and epoxy adhesives' tensile strength, elastic modulus, tensile strain after 0 (T0), 1 (T1), 2 (T2), 3 (T3), and 4 (T4) years of different environmental exposures (E1 to E6).

Environment	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
Adhesive: Tensile strength f_t [MPa]					CFRP laminate: Tensile strength f_t [MPa]					
REF	19.9 (3.0)	–	–	–	–	2405 (3.8)	–	–	–	–
E1	–	19.5 (1.8)	18.2 (2.8)	19.8 (4.9)	16.3 (14.6)	–	2674 (2.72)	2528 (4.4)	2469 (6.4)	2484 (3.0)
E2	–	7.2 (3.1)	6.7 (2.7)	7.4 (7.1)	8.4 (3.6)	–	2688 (3.4)	2460 (7.1)	2713 (4.5)	2522 (7.1)
E3	–	19.9 (3.1)	17.4 (5.3)	16.7 (5.9)	–	–	2792 (3.7)	2590 (5.4)	2546 (5.1)	2427 (6.8)
E4	–	20.1 (3.4)	17.2 (4.3)	16.5 (9.8)	15.7 (20.1)	–	2757 (2.9)	2617 (4.5)	2492 (5.0)	2516 (3.9)
E5	–	21.9 (5.2)	18.0 (3.6)	17.7 (6.5)	17.0 (5.8)	–	2611 (5.0)	2619 (5.3)	2427 (4.1)	2575 (3.9)
E6	–	17.7 (6.4)	15.8 (4.3)	18.0 (4.2)	15.3 (2.6)	–	2667 (3.0)	2640 (2.9)	2561 (2.8)	2605 (5.1)
Adhesive: Elastic modulus E_t [GPa]					CFRP laminate: Elastic modulus E_t [GPa]					
REF	6.5 (3.0)					164 (1.2)				
E1	–	6.6 (1.3)	6.1 (1.4)	6.5 (6.0)	5.6 (14.6)	–	179 (1.6)	165 (2.7)	170 (2.7)	172 (9.1)
E2	–	1.9 (5.1)	1.6 (4.0)	1.9 (14.6)	2.0 (12.4)	–	174 (0.7)	168 (0.6)	175 (1.8)	173 (3.2)
E3	–	6.7 (4.4)	6.0 (5.4)	5.3 (8.9)	–	–	177 (1.8)	172 (1.1)	171(3.3)	180(5.3)
E4	–	7.2 (1.4)	5.4 (6.9)	5.8 (4.0)	5.4 (21.1)	–	175 (1.8)	174 (4.5)	180 (5.7)	162 (5.3)
E5	–	7.5 (5.7)	6.1 (5.0)	5.8 (6.5)	5.7 (7.0)	–	173 (2.0)	176 (1.5)	181 (4.5)	173 (4.8)
E6	–	6.2 (5.4)	5.0 (10.0)	6.2 (6.2)	4.8 (10.0)	–	171 (1.4)	165 (2.5)	163 (2.8)	167 (5.0)
Adhesive: Ultimate tensile strain [%]					CFRP laminate: Ultimate tensile strain [%]					
REF	0.4 (6.2)	–	–	–	–	14.6 (3.8)	–	–	–	–
E1	–	0.4 (13.0)	0.3 (11.7)	0.3 (8.8)	0.4 (11.3)	–	14.9 (3.2)	15.3 (6.1)	14.5 (7.3)	14.8 (5.3)
E2	–	1.1 (21.4)	1.1 (11.9)	1.0 (25.5)	0.9 (15.3)	–	15.5 (2.9)	16.0 (12.5)	15.5 (6.1)	14.9 (1.3)
E3	–	0.3 (11.1)	0.3 (19.1)	0.3 (10.1)	–	–	15.8 (3.8)	15.1 (5.1)	14.6 (8.1)	14.3 (7.0)
E4	–	0.3 (11.3)	0.3 (12.8)	0.3 (24.6)	0.3 (24.8)	–	15.78 (2.3)	15.0 (4.5)	13.8 (4.3)	15.4 (1.1)
E5	–	0.3 (11.2)	0.4 (13.1)	0.3 (11.5)	0.4 (10.5)	–	15.1 (4.6)	14.9 (2.5)	13.5 (7.0)	15.4 (3.1)
E6	–	0.3 (4.3)	0.3 (12.9)	0.3 (17.4)	0.4 (14.2)	–	15.6 (2.9)	16.0 (1.9)	15.7 (2.8)	15.9 (5.0)

Notes: all values in parentheses express coefficient of variation in percent; REF: Reference values from the specimens tested at the beginning (before exposure) i.e., at T0.

The improved CFRP properties in E1 and E2 during the first year of exposure may be due to the effects of post-curing of the polymeric matrix resin, also observed in [26]. In the following years, reductions on the mechanical

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properties were observed in both environments: while for the case of E2 the decrease can be supported by water diffusion through the polymeric matrix of the CFRP laminates (physical and chemical aging), for environment E1 no apparent reason could be found. In E3, both carbonation and high temperatures may also lead to a significant post-curing effect of the resin matrix, while in E4 post-curing may be the reason for the observed increased values and the matrix microcracking due to freeze-thaws cycles may be assumed to be the main reason for the slight decrease observed later, as freeze-thaw cycling are known to lead to microcracks of the polymeric matrix thereby decreasing the material property [29].



Figure III.8. (a) Carbonation of concrete; typical failure modes from: (b) concrete under compression; (c,d) concrete under tension (pull-off tests); (e) epoxy adhesive under tension; and (f to h) CFRP laminate under tension.

In E5, a synergistic effect between high temperatures and the presence of carbonation can be attributed to the observed increased CFRP properties. Furthermore, in E6 only post-curing and carbonation can be inferred to be the underlying reason for the increased CFRP properties, mainly because chlorides do not affect the CFRP properties according to [11]. The failure modes were characterized by a rapid and progressive fracture of

individual fibers starting from the edges and moving towards the center of the CFRP until complete fiber rupture (**Figure III.8g** and **Figure III.8h**) with a massive sound.

Overall, a general observation from the bond constituent materials is that the adhesive showed significant degradation, while concrete and CFRP laminate improved their properties after exposure to most environments (except concrete in E2).

3.2. NSM CFRP-to-concrete bond

This section presents the data from the single-lap shear tests performed on the specimens strengthened according to NSM technique. The average pull-out force *versus* loaded end slip curves, their maximum values and their variations are shown in **Figure III.9a** to **Figure III.9f** and **Table III.5**. Additionally, the maximum pull-out force (F_{max}) and the loaded end slip at the F_{max} , and the pull-out force *versus* loaded end slip curves for all tested specimens are shown in the **ANNEX III.1** and **ANNEX III.2**, respectively. Furthermore, the variations of both the maximum pull-out force and slip compared to the values at initial time (T0) are presented in **Table III.6**. Comparative plots of the maximum pull-out force *versus* loaded end slip at F_{max} , the observed failure modes with their corresponding F_{max} , and the typical observed failure mode are shown in **Figure III.9g**, **Figure III.9h**, and **Figure III.10**, respectively.

Figure III.9a to **Figure III.9f** shows an initial linear branch at smaller values of slip, followed by a nonlinear branch up to the peak load at larger slip values. Thereafter, starting from the peak, a descending branch develops as a result of the decay of the pull-out force. Similar trends were also found by [7] and [8]. After the peak, within the descending branch, the pull-out force decrease can be due to several factors depending on the failure mode. It can be inferred that when the failure mode is by concrete cohesive failure (CC), the behavior is closely related to the friction of aggregates and cement paste, while when the failure mode is by adhesive failure (F/A), the dominant factor is the frictional forces between adhesive and CFRP. A visual examination during the test showed that the cracks first appeared in the region near the loaded end section and gradually moved toward the free end region as the applied load increased, which also demonstrates that the maximum shear stresses moved from the loaded end to the free end region as the load increased. This is in good agreement with the examination performed using digital image correlation (DIC) in [7].

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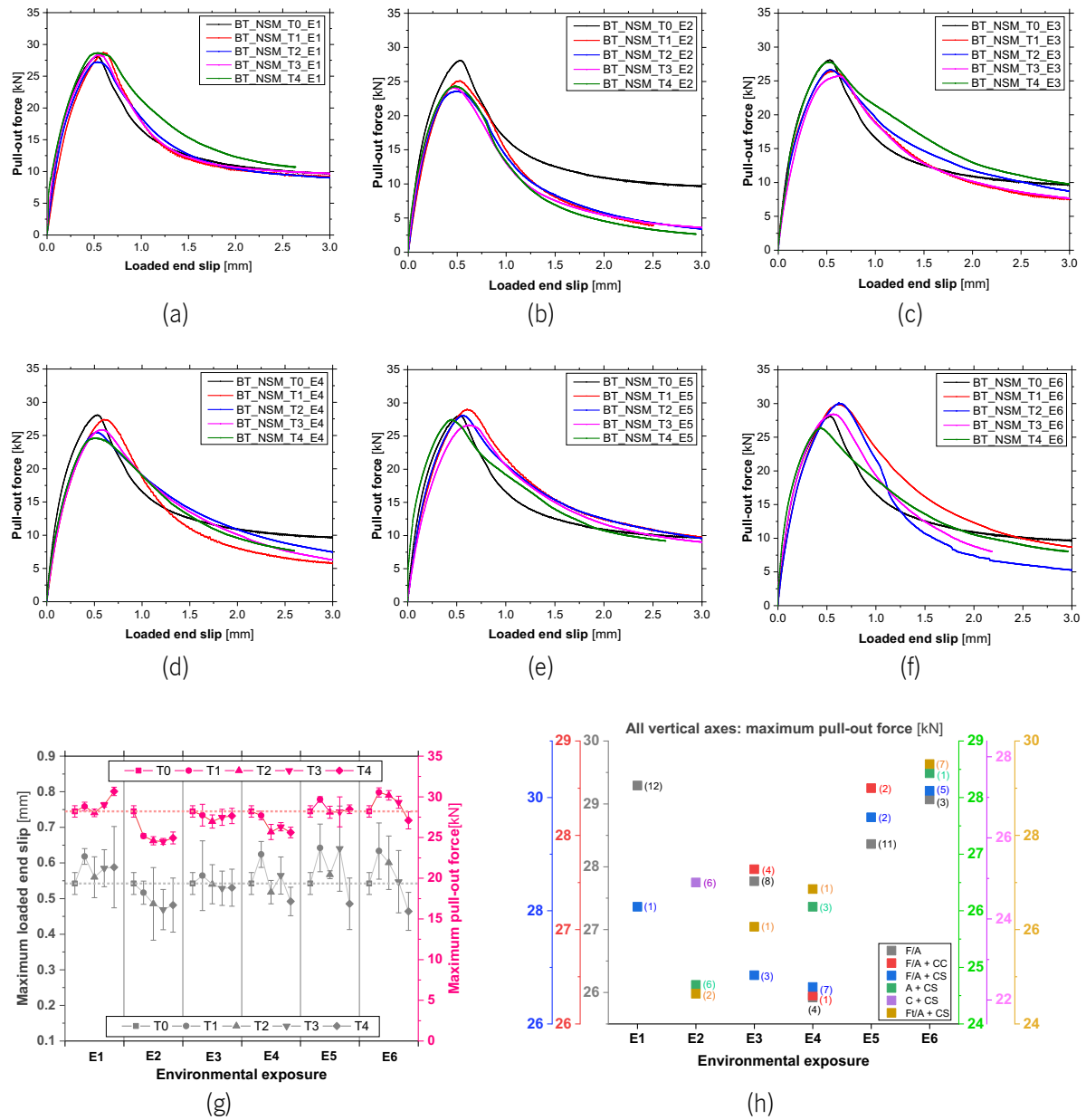


Figure III.9. NSM CFRP-to-concrete bond test results: (a-f) average curves of the loaded end slip *versus* pull-out force for all testing times and all studied environmental exposures; (g) comparison of maximum pull-out force and loaded end slip trends; (h) types of failure modes and their corresponding maximum pull-out forces (numbers in parentheses indicate the number of specimens failed with the shown failure mode, the images of the typical failure mode are shown in **Figure III.10** and the legend of **Figure III.9h** is explained in the caption of **Figure III.10**).

From **Table III.6**, it can be seen that the pull-out force decreased in E2, E3, and E4 for all test times (T1 to T4) as compared to the initial values at T0. The largest reductions were obtained for the specimens immersed in

water (E2), followed by those of E4, indicating that the NSM CFRP-to-concrete bond is significantly affected by water diffusion and freeze-thaw attack. In fact, there was an average reduction of nearly 12 % and 9 % in the pull-out forces from E2 and E4, respectively, considering all testing times (T1-T4). Besides, the largest reduction in the slip was also found in E2 with a 13.5 % reduction at T3 and an average of nearly 10 % reduction in E2 considering all testing times. In contrast, specimens in E1 generally showed an increase in the pull-out force with an increase of almost 9 % at T4 and an average of nearly 3 % increase over all test times, compared to T0.

Table III.5. Average maximum pull-out force and average loaded end slip of NSM technique for all testing times and all studied environments.

Environment	Fmax [kN]					Loaded slip [mm]				
	T0	T1	T2	T3	T4	T0	T1	T2	T3	T4
E1	28.21 (2.5)	28.81 (1.9)	28.03 (1.7)	29.02 (0.9)	30.67 (1.7)	0.54 (28.7)	0.62 (3.6)	0.56 (10.3)	0.59 (8.9)	0.59 (19.4)
E2	-	25.19 (1.1)	24.60 (2.0)	24.52 (1.5)	24.95 (2.9)	-	0.52 (6.3)	0.49 (21.0)	0.47 (12.1)	0.48 (15.8)
E3	-	27.75 (4.8)	26.96 (3.0)	27.49 (3.6)	27.65 (3.4)	-	0.56 (17.4)	0.54 (8.6)	0.53 (8.7)	0.54 (9.1)
E4	-	27.71 (1.9)	25.68 (3.6)	26.32 (1.9)	25.62 (2.6)	-	0.62 (5.8)	0.52 (6.3)	0.56 (9.2)	0.49 (8.2)
E5	-	29.68 (1.1)	28.11 (2.6)	28.15 (6.5)	28.47 (1.9)	-	0.64 (10.4)	0.57 (1.9)	0.64 (18.7)	0.48 (14.9)
E6	-	30.54 (1.8)	30.18 (4.8)	29.33 (2.5)	27.11 (3.9)	-	0.63 (12.5)	0.60 (12.5)	0.55 (16.0)	0.46 (11.5)

Notes: values in parentheses are coefficient of variation in %

Table III.6. Variation of pull-out force and loaded end slip referring to the initial value at T0 for all studied environments.

Environment	Variation of maximum force with time (%)				Variation of loaded end slip with time (%)			
	T1	T2	T3	T4	T1	T2	T3	T4
E1	(+) 2.13	(-) 0.64	(+) 2.87	(+) 8.72	(+) 14.05	(+) 3.23	(+) 7.91	(+) 8.47
E2	(-) 10.71	(-) 12.80	(-) 13.08	(-) 11.56	(-) 4.75	(-) 10.51	(-) 13.52	(-) 11.14
E3	(-) 1.63	(-) 4.43	(-) 2.55	(-) 1.99	(+) 4.06	(-) 0.37	(-) 2.46	(-) 2.16
E4	(-) 1.77	(-) 8.97	(-) 6.70	(-) 9.18	(+) 15.08	(-) 4.48	(+) 4.14	(-) 9.28
E5	(+) 5.21	(-) 0.35	(-) 0.21	(+) 0.92	(+) 18.45	(+) 4.56	(+) 18.07	(-) 10.50
E6	(+) 8.26	(+) 6.98	(+) 3.97	(-) 3.90	(+) 16.83	(+) 10.76	(+) 0.96	(-) 14.41

Notes: (+): increase in %, (-): decrease in %, E1-E6: studied environments, T1-T4: Exposure time from year 1 (T1) to year 4 (T4).

The comparative perspectives between the variation of the pull-out force and the variation of the slip (in various environmental exposures) over time are shown in **Figure III.9g**. In fact, the pull-out force has different variation trend when compared to that of the slip in E3, E4, and E5, while similar trends can be seen in E1, E2, and E6. Therefore, attempts to establish long-term predictions using the force *versus* slip relationship can be simpler in the latter environments (i.e., E1, E2, and E6) than in the former.

However, since the trends from the percentage increase/decrease of the force and slip are not similar (Table III.6), they cannot be used to establish such models; hence, the models based on the force-slip relationships may be better than those based on the percentage increase/decrease of the force-slip relationships.

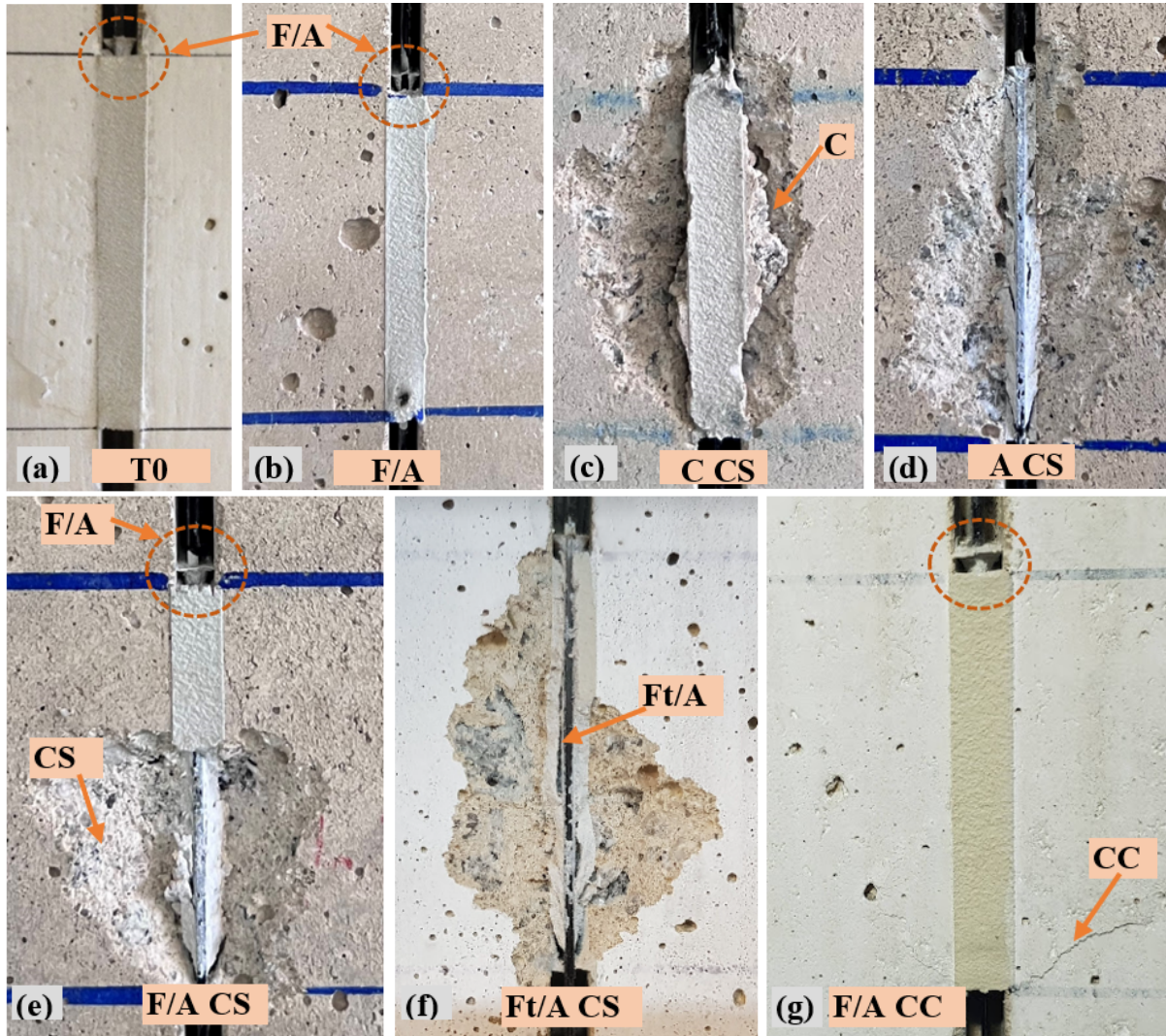


Figure III.10. Typical failure modes observed in the NSM CFRP-to-concrete bond specimens: (a,b) F/A: failure at the adhesive-CFRP interface; (c) C CS: mixed concrete cohesive and concrete splitting failures; (d) A CS: mixed adhesive cohesive and concrete splitting failures; (e) F/A CS: mixed adhesive-CFRP interface and concrete splitting failures; (f) Ft/A CS: mixed adhesive-CFRP interphase with debonding at CFRP bondline and concrete splitting failures; (g) F/A CC: mixed adhesive-CFRP interface and concrete cracking failures.

Furthermore, **Figure III.9h** is used to relate the failure mode (notations used **Figure III.9h** are defined in **Figure III.10**) with the maximum pull-out force, and to help predict the predominant failure mode in each environment: (i) the failure at the adhesive-CFRP interface (F/A); (ii) adhesive-CFRP interface with debonding at CFRP bondline (Ft/A); (iii) concrete cohesive (C); (iv) concrete cracking (CC); (v) concrete splitting (CS); and, (vi) adhesive cohesive (A). It can be seen that F/A failure is the leading failure mode in E1, E3 and E5, while A CS and C CS are the two competing failure modes in E2; F/A CS dominates in E4, while Ft/A CS dominates in E6.

It can also be seen that the same failure mode does not result in the same maximum pull-out force when considering different environments. For example, the F/A failure in E4 corresponds to a pull-out force of around 25.8 kN, while the corresponding force in E1 is approximately 29.3 kN. Classifying the dominant failure mode according to environmental exposures and their corresponding pull-out forces can pave the way towards developing appropriate analytical or numerical predictions that take into account the expected failure mode, the expected failure load and the type of environment.

3.3. NSM CFRP-to-concrete bond properties *versus* constituent materials properties

For a better understanding of the obtained results, presented in **Section 3.2**, the strength retentions for both the NSM CFRP-to-concrete bond and the bond constituent materials are plotted in **Figure III.11a**, and the retentions of the bond stiffness and elastic modulus of the bond constituent materials are plotted in **Figure III.11b**. In the present work, *retention* was defined as the ratio between the actual value of the material property (after exposure) and the original one (before exposure). It is worth noting that the bond stiffness was obtained from the linear fit of the experimental pull-out *versus* loaded end slip curve, considering the range of force from 0 to 10 kN. A detailed analysis of the data obtained here is given in the following paragraphs.

DURABILITY OF NEAR-SURFACE MOUNTED CFRP-TO-CONCRETE BOND:
ACCELERATED *VERSUS* NATURAL AGING

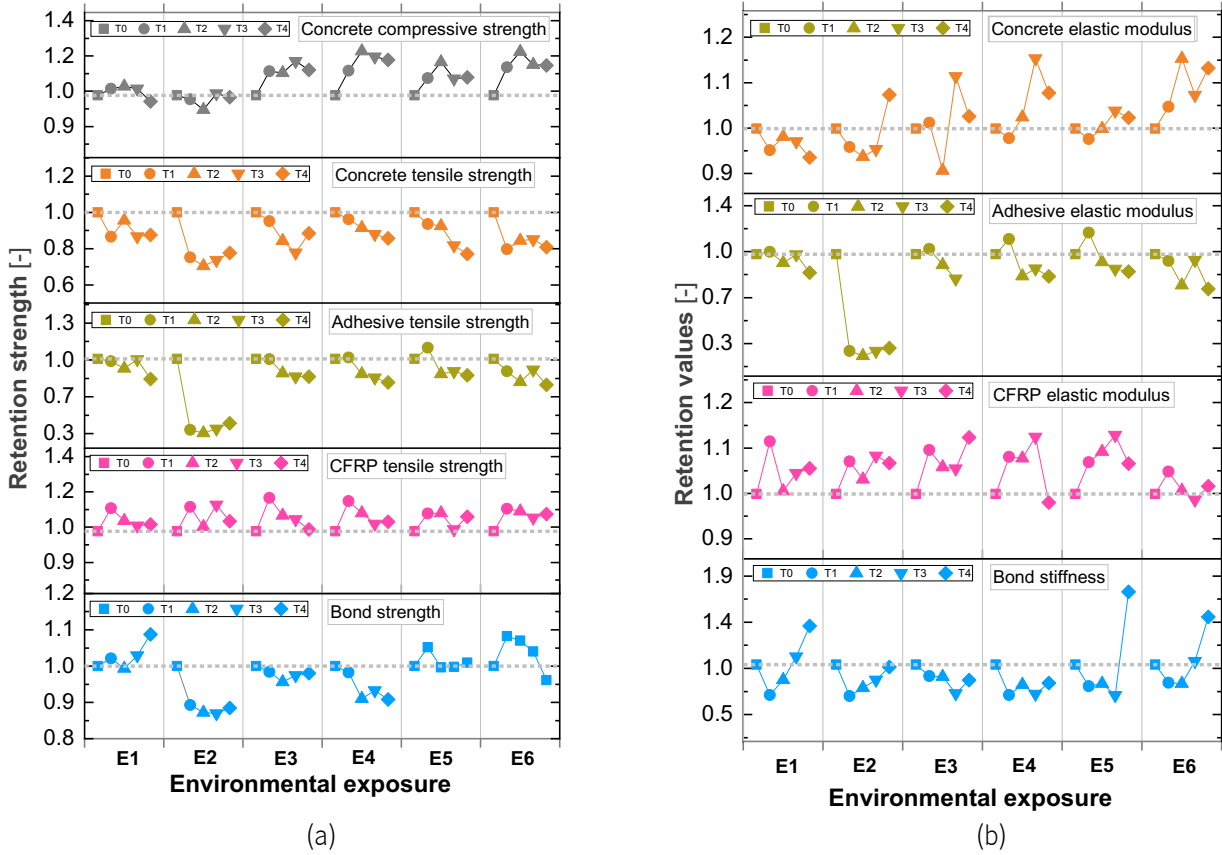


Figure III.11. Variation of bond properties and its constituent materials: (a) comparative perspectives between the bond strength and material strength properties, (b) comparative perspectives between the bond stiffness and material elastic modulus properties.

Environment E1: The bond strength generally increases in E1. As previously shown in **Section 3.2**, both the f_{cc} and f_t generally increased, while the f_{ct} and f_s slightly decreased, as shown in **Figure III.11a**. The variations in material properties can cause alteration of the bond properties. That is, the number of bonds (e.g., covalent or hydrogen bonds) linking the epoxy adhesive to the CFRP laminate or those linking the adhesive to concrete augmented, which consequently increased the bond strength (**Figure III.11a**). On the other hand, the bond stiffness and elastic modulus of the constituent materials are presented in **Figure III.11b**. The bond stiffness shows a decrease in the first year and a progressive increase in the later years. It can be noted that only the E_c decreased in the first year while the E_s and E_t increased, hence the decreased bond stiffness may be largely due to the E_c degradation. In the later years, the slight reductions in the E_c and E_s may have been outweighed by the increase in E_t , resulting in increased bond stiffness. Overall, it can be concluded that the improvements in f_{cc} and E_c and f_t and E_t outweighed the slight reductions in the f_{ct} and f_s , resulting in increased bond strength and

bond stiffness. However, it is worth noting that the bond properties may also be affected by other factors such as the interactive mechanisms taking place at the interface level, rather than the change of the sole properties of the bond constituent materials. Additionally, the materials specimens were fully exposed, while those for pull-out tests were partially exposed (i.e., epoxy adhesive was partially covered by concrete and CFRP laminate, and the CFRP was partially covered by the epoxy adhesive), hence poor correlations between the trends of the variations of the material properties and those of the bond properties are inevitable.

Environment E2: the bond strength decreases sharply from T0 to T1 and then shows marginal decreases until T4. As previously seen, the concrete and adhesive significantly decreased their tensile strength, while a slight decrease in f_{ct} and increase in f_t were observed (in **Figure III.11a**). Since the bond strength retentions follow the same trend as those of the f_{ct} and f_t , this may be an indicator that the bond strength is strongly dependent on the tensile properties of the concrete and adhesive tensile properties rather than the E_c and E_t properties. The f_{ct} retention (0.7) is comparable to that of the bond strength (0.85), while that of the f_t is much smaller (around 0.3). Hence, both the concrete and adhesive tensile properties degradation can be considered as the main cause of the observed bond strength degradation, with the latter contributing almost 2.5 times more than the former. Besides, E2 is the only environment where specimens showed C CS and A+CS as the equally dominant failure modes (**Figure III.9h**), hence analytical models can be established mainly focusing on these two failure modes. On the other hand, the bond stiffness shows a decreasing trend from T0 to T1 and an increasing trend up to T4. The E_c tends to show a similar trend as that of the bond stiffness, while the adhesive shows a different trend. This indicates that the trend of bond stiffness may depend more on E_c and E_t than on E_a . However, since there is a significant loss in the adhesive E_a retention (nearly 0.72 loss at T1) which almost corresponds to a retention of the bond stiffness (of about 0.45), it can be inferred that the decreased adhesive E_a retention may affect the bond stiffness during the water diffusion phase through the adhesive. Then, once the water diffusion is stabilized, the improvement in E_t and E_c may counteract the negative effects caused by the decreased E_a retention, allowing the bond to regain its stiffness over time. The predominant failure mode in E2 also confirms the significant loss of the concrete properties, f_{ct} and E_c (i.e., C CS failure) and the adhesive properties, f_a and E_a , leading to a complete detachment of the adhesive from the CFRP laminate (i.e., the Ft/A CS failure).

Environment E3: In E3, the bond strength is found to be nearly stable with time, with minor variations as can be seen in **Figure III.11a**. This is in good agreement with the findings by [7], where bond strength stability was observed. The f_t shows a progressive decrease, but with the last value at T4 still being slightly comparable to

that recorded at the beginning. Therefore, it can be concluded that the improvement of the concrete f_{cc} counteracted the negative effects of the deterioration of its tensile strength (f_{ct}), and the observed minor reductions in the bond strength may be largely attributed to the deterioration of the f_a , mainly because of the similarity of the trend for both the f_a and the bond strength. Similarly, a significant reduction in the E_a retention (up to 0.3) can be considered as the dominant factor for the observed reduction in bond stiffness, as increased elastic modulus retentions are observed for both concrete and the CFRP. The dominant failure mode in E3 (F/A) may also indicate that the degradation of adhesive properties (f_a and E_a) may be the primary cause of the reduced bond strength and bond stiffness retentions.

Environment E4: bond strength and stiffness generally showed a progressive decrease with exposure time. These decreases are closely related to the observed general decrease in the adhesive f_a and E_a with increasing exposure time (**Figure III.11a** and **Figure III.11b**). This is in agreement with the findings of [24], where both f_a and E_a decreased after conditioning the specimens in a wet-dry environment, which reduced the fracture energy at the CFRP-concrete bond interface. The freeze-thaw attack in E4 may have caused a mismatch in the coefficient of thermal expansion (CTE) between the fiber and resin, which thereafter led to microcracking that in turn led to a decrease in the chemical bond strength between adhesive and fiber matrix resin. Such a decrease then caused the failure mode to be mainly F/A CS (see **Figure III.9h**). The observed significant increase in f_{cc} can also explain why the failure mode was not CC. In general, both bond strength and bond stiffness seem to depend on the adhesive properties mainly because of the observed similar trends. However, the increase in f_{cc} over time may have prevented the severe degradation in bond strength observed in the case of E2. In a study by [29] using the AAT protocol, the bond strength first increased and then decreased, which is also the case in E4 from T1 to T2 (increase) and then from T2 to T3 (decrease).

Environment E5: There was a 5.2 % increase in the bond strength at T1 compared to the initial value at T0, followed by a decrease from T1 to T2, which then levelled off until T4 (**Figure III.11a**). Thus, in general, there were marginal changes in the bond strength. This is in good agreement with a study by [7], where the thermal cycles ranging between 20 °C and 80 °C, imposed on the NSM specimens for a period of 180 days, did not significantly affect the bond strength. The observed sharp increase during T1 may be due to the improvement of f_a and f_{ct} , and the level-off after T1 may be due to the continuation of f_a and f_{ct} degradation. The bond strength showed a similar trend as that of the f_a ; hence, it may be a good indicator that the bond strength is significantly dependent on the adhesive tensile strength. On the other hand, the bond stiffness decreased from year to year,

except for the last year where a surprisingly sharp increase can be seen (**Figure III.11b**). The reason for this sharp increase after 4 years is not clear yet; however, this behavior may be revealed in the future, as the testing campaigns will continue for up to 10 years. The observed stiffness decrease is consistent with the findings of [55], where the FRP-concrete interface stiffness decreased with increasing service temperatures; hence, the high seasonal temperatures in E5 can be considered as the reason for the varying bond stiffness. Besides, the reduced bond stiffness may be attributed to the degradation of the adhesive E_a , as the elastic modulus of the other constituent materials showed increasing trends. The predominant failure mode in E5 is adhesive failure at the CFRP-adhesive interface (F/A), which confirms the weakening of the adhesive bond to CFRP due to its reduced properties. The observed failure is also consistent with the findings in a study by [56], where the increased temperatures changed the failures of NSM specimens from concrete shear failure to F/A.

Environment E6: An opposite trend between bond strength and bond stiffness for specimens in E6 can be seen in **Figure III.11a** and **Figure III.11b**. There was a sharp increase in the bond strength during T1, followed by a progressive decrease from year to year until T4. The f_{cc} and f_t increased sharply during T1 and somehow levelled off thereafter. It can be inferred that this increase was due to the high rate of cement hydration and of post-curing of polymeric matrix resin which added more additional bonding to the adhesive, thereby increasing bond strength. In fact, it can be inferred that the above additional bonding served two important purposes: (i) by generating crosslinking that counteracted the negative effects from the f_s and f_{ct} deterioration; and (ii) by supplying additional crosslinking that served the role of increasing the bond strength. However, the continuation of the f_s degradation in the later years led to a progressive decrease in the bond strength, mainly because both f_{cc} and f_t levelled off in those years without any additional bonding being supplied. Besides, the increasing trend in bond stiffness may have been significantly dependent on the increased E_c , since the E_s showed reductions in retentions and CFRP showed insignificant variations. Although both concrete and CFRP laminate are accredited for the observed improved bond properties, the former might have played a more relevant role because of its higher retentions (nearly 1.30 for f_{cc} and 1.18 for E_c) than those of the latter (nearly 1.10 for f_t and 1.08 for E_s). This is mainly supported by the observed failure mode, i.e., a mixed failure with Ft/A CS is predominant (**Figure III.9h**), which shows that concrete played a more relevant role in increasing the maximum pull-out force because the main failure was not the concrete cohesive. Therefore, concrete properties can be thought to have played a greater role in controlling both the bond strength and bond stiffness in E6.

Overall, it is important to note that the response of the NSM CFRP-to-concrete bond can be thought to depend not only on the behavior of the constituent materials, but also on the behavior of the interface itself as well as the resisting mechanisms, as was previously observed in [4,57]. In addition, the properties of the solely tested constituent materials may differ from those of the region composing the interface, particularly in the case of the adhesive which is partially exposed because it is mostly covered by concrete, and the CFRP laminate, which is completely covered by the adhesive.

3.4. Natural outdoor *versus* laboratory-controlled environments

In outdoor application of CFRP composites as strengthening systems, the CFRP-to-concrete bond and the bond constituent materials are essentially exposed to the outdoor environment. It is of paramount importance to develop correlations between Accelerated Aging Test (AAT) and Natural Aging Test (NAT) data, as this can lead to appropriate recommendations on the reduction factors for quantifying the effects of different outdoor degradation agents on the durability of the materials. In this context, comparisons were made between the existing AAT data from literature and NAT data from the present work, and some environmental conversion (reduction) factors were derived. For this purpose, a database composed of AAT data from different existing studies [7,31–33] was prepared with the following characteristics:

- Database comprised of 84 specimens strengthened with CFRP strips according to NSM technique, bond length in the range 100-600 mm, specimens tested either in a direct or beam single-lap shear test configuration; type of environmental exposures being accelerated aging, with degradation agents as freeze-thaw cycles, wet-dry cycles, immersion in salt or tap water, moisture, and temperature cycles.
- Type of CFRP laminate: pultruded composites with thickness and elastic modulus in the range of 1.3-1.4 mm and 155-176 GPa, respectively.
- Adhesive epoxy-based adhesive type, the compressive strength in the range of 25-50 MPa.
- Exposure time: up to 2 years.

The variation of bond strength retention with exposure time is presented in **Figure III.12a** for the results from the above-described AAT database with those from both the AAT (FRPLongDur-E2) and NAT (FRPLongDur- E3-E6) data of the present work. It can be seen that the retentions of the AAT data from existing studies vary approximately between 0.8 and 1.2, thereby showing a significant dispersion. On the other hand, the retentions

from the AAT data of the present work show a marginal variation (between 0.8 and 0.9) with a decreasing trend. Besides, the NAT data also show a decreasing trend of the strength retentions with time.

In order to derive the environmental conversion factor (ECF) from both the existing AAT data (from the literature) and the NAT data (from the present work), the percentages of non-conservative estimates (i.e., the percentages for which it is assumed that the estimated test data did not result in conservative values, i.e., values that did not fall within the desired range) are plotted against the estimated environmental conversion factors in **Figure III.12b**. Considering 10 % of the non-conservative data as the maximum allowable percentage, the ECF of 0.88 and 0.93 can be proposed for the AAT and NAT data, respectively. When comparing these two factors, it can be seen that the existing accelerated aging data tends to overestimate the bond strength degradation rate for up to 4 years; however, NAT data covering longer durations, may help to predict more appropriate factors.

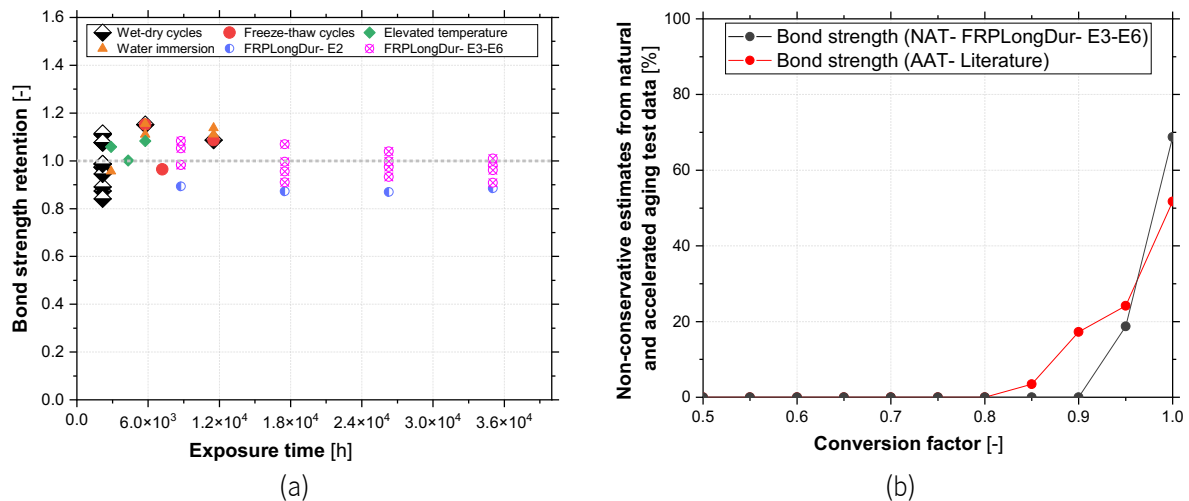


Figure III.12. Comparison of natural *versus* accelerated aging durability results for NSM technique: (a) bond strength retention with time for both AAT and NAT; (b) prediction of conversion factors for both AAT and NAT protocols.

4. CONCLUSION AND RECOMMENDATIONS

In this study, the durability of the NSM CFRP-concrete bond was addressed. The durability of the bond was assessed after exposure to two laboratory-controlled environments (i.e., one denoted as E1 with conditioning at around 20 °C/55 % RH and other as E2 with water immersion at 20 °C), and four outdoor environments to mainly promote aging due to carbonation (denoted as E3), freeze-thaw attack (E4), elevated temperatures (E5), and airborne chlorides (E6). Materials (concrete, adhesive and CFRP laminates), as well as NSM CFRP-Concrete bond specimens, were tested before exposure at initial time (T₀); and after being exposed to the above-

mentioned environments (E1-E6), that is, after one year (T1), two years (T2), three years (T3), and fourth years (T4) of exposure. The key findings are highlighted as follows.

- Although the environmental exposures were carefully selected to promote mainly a single degradation agent, the effects of multiple degradation agents may have occurred within a single environment. Therefore, synergistic effects on the bond properties may have played an important role in the observed variations, particularly in outdoor environments. In fact, the combined effects of carbonation, moisture, and airborne chlorides were the three-competing agents in E6; carbonation and elevated temperature effects were dominant in E3 and E5, while in E4 the effects of moisture and freeze-thaws dominated.
- *Constituent materials.* The bond constituent materials showed different variations depending on the environmental exposures:
 - *Concrete:* the concrete compressive strength (f_{cc}) increased sharply during the first years due to the enhanced continuation of cement hydration and then began to level-off in the later years; however, marginal variations were observed in laboratory-controlled environments. Besides, the concrete elastic modulus (E_c) also showed a general improvement in outdoor and a decrease in laboratory-controlled environments. The highest improvements of the concrete f_{cc} and E_c were found in E3 and E5 with 21.2 % and 21.0 % increase, and in E4 and E6 with 12.0 % increase, respectively. Contrary, the concrete pull-off strength (f_{ct}) showed decreasing trends in all environments);
 - *Epoxy adhesive:* the adhesive tensile strength (f_a) and its elastic modulus (E_a) decreased in all environments, but with some improvements in E1, E3, E4, and E5 for the case of E_a during the first year as a result of post-curing. The lowest values of the E_a and f_a were obtained from water immersion, with reductions of 75.4 % and 66.3 %, respectively, as compared to the initial values;
 - *CFRP laminate:* the CFRP laminate tensile strength (f_l) and its elastic modulus (E_l) increased as a result of post-curing of the polymeric matrix, which can be attributed to the presence of carbonation in E1, synergistic effect between carbonation and high temperatures in E3, between moisture and freeze-thaw in E4, the synergistic effect between carbonation and high temperatures in E5, and synergistic effects between carbonation, moisture and airborne chlorides in E6. The highest values of E_l were found for the specimens in E5 with 10.4 % (at T3) and in E3 with 9.8 % (at T4), while the highest values of the

f_t were found in E3 with 16.1 % (at T1) and E4 with 14.6 % (at T1), as compared to the initial value recorded before exposure (at T0).

- *NSM CFRP-to-concrete bond:* the bond strength and bond stiffness changed over time and from environment to environment. The highest degradations of the bond strength were observed under continuous water immersion (E2) and freeze-thaw attack (E4), with decreases of nearly 12 % and 9 %, respectively. In contrast, the bond strength in E1 generally improved, showing an increase of approximately 10 % in the last year. Similarly, the bond strength in outdoor environments (E5 and E6) generally improved, indicating that exposure to high temperatures and airborne chlorides can generally be beneficial to the bond properties in NSM technique. Also, change of concrete properties played a major role in improving bond properties in E6. The effects of atmospheric CO₂ were minimal, with very insignificant changes in the bond strength. On the other hand, the bond stiffness generally showed a decrease (with approximately 0.7 retentions in all environments) during the first year, followed by a general increase in the following years, remarkable for the specimens in laboratory-controlled environments.
- *Failure modes:* visual examination of failure modes in NSM showed that the failure mode at initial time (T0) changed after exposure. In general, the failure at adhesive-CFRP interface (F/A) dominated in E1 and E5; the combination of adhesive failure (A) with concrete cohesive failure (CC) or with concrete splitting (CS) dominated in E2; F/A dominated in E3, a combination of F/A and CS dominated in E4 mainly because of the long-lasting negative effects of freeze-thaw on both adhesive and CFRP; F/A or a combination of F/A and CS dominated in E6.
- The bond strength retentions from the AAT data (from existing studies) vary significantly, ranging approximately between 0.8 and 1.2, which shows a remarkable dispersion. Also, most of AAT bond strength retentions from literature did not agree with those from natural aging test (NAT) in the present work. Attempts were made to derive the ECF. Considering 10 % of the non-conservative estimates from the AAT and NAT data as the maximum allowable percentage, the conversion factors of 0.88 and 0.93 are suggested for the former and the latter, respectively. Comparing these two factors, it can be seen that the existing accelerated aging test data tends to overestimate the bond strength degradation rate for up to 4 years.

As a main recommendation, the authors suggest that studies with NAT data for longer periods (e.g., more than 4 years) of exposure are required to be able to appropriately predict the conversion factors.

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REFERENCES

- [1] T. Westphal, P. Bortolotti, R.P.L. Nijssen, Carbon glass hybrid materials for wind turbine rotor blades, 2013. <http://resolver.tudelft.nl/uuid:f214ffc8-60c5-4855-a4b9-db3be9991193>.
- [2] I.S. Abbood, S.A. Odaa, K.F. Hasan, M.A. Jasim, Properties evaluation of fiber reinforced polymers and their constituent materials used in structures - A review, Mater. Today Proc. 43 (2021) 1003–1008. <https://doi.org/10.1016/j.matpr.2020.07.636>.
- [3] CEB-FIP, fib90 Bulletin: Externally Applied FRP reinforcement for Concrete structures, 2019.
- [4] J.S. Cruz, J. Barros, Modelling of bond between near-surface mounted CFRP laminate strips and concrete, Comput. Struct. 82 (2004) 1513–1521. <https://doi.org/10.1016/j.compstruc.2004.03.047>.
- [5] L. De Lorenzis, J.G. Teng, Near-surface mounted FRP reinforcement: An emerging technique for strengthening structures, Compos. Part B Eng. 38 (2007) 119–143. <https://doi.org/10.1016/j.compositesb.2006.08.003>.
- [6] M.R.F. Coelho, J.M. Sena-Cruz, L.A.C. Neves, A review on the bond behavior of FRP NSM systems in

- concrete, *Constr. Build. Mater.* 93 (2015) 1157–1169. <https://doi.org/10.1016/j.conbuildmat.2015.05.010>.
- [7] P. Fernandes, J. Sena-Cruz, J. Xavier, P. Silva, E. Pereira, J. Cruz, Durability of bond in NSM CFRP-concrete systems under different environmental conditions, *Compos. Part B Eng.* 138 (2018) 19–34. <https://doi.org/10.1016/j.compositesb.2017.11.022>.
- [8] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of Bond between NSM CFRP Strips and Concrete under Real-Time Field and Laboratory Accelerated Conditioning, *J. Compos. Constr.* 26 (2022) 1–15. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001262](https://doi.org/10.1061/(asce)cc.1943-5614.0001262).
- [9] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
- [10] R. El-Hacha, R. Wight, M. Green, Prestressed fibre-reinforced polymer laminates for strengthening structures, *Prog. Struct. Eng. Mater.* 3 (2001) 111–121. <https://doi.org/10.1002/pse.76>.
- [11] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fibre reinforced polymer laminates used in strengthening systems: Accelerated aging versus natural ageing, *Materials (Basel)*. 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [12] J. Tatar, N.R. Brenkus, Performance of FRP-Strengthened Reinforced Concrete Bridge Girders after 12 Years of Service in Coastal Florida, *J. Compos. Constr.* 25 (2021). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001134](https://doi.org/10.1061/(asce)cc.1943-5614.0001134).
- [13] N.M. Huang, J.J. Chang, M.T. Liang, Effect of plastering on the carbonation of a 35-year-old reinforced concrete building, *Constr. Build. Mater.* 29 (2012) 206–214. <https://doi.org/10.1016/j.conbuildmat.2011.08.049>.
- [14] V. Talakokula, S. Bhalla, R.J. Ball, C.R. Bowen, G.L. Pesce, R. Kurchania, B. Bhattacharjee, A. Gupta, K. Paine, Diagnosis of carbonation induced corrosion initiation and progression in reinforced concrete structures using piezo-impedance transducers, *Sensors Actuators, A Phys.* 242 (2016) 79–91. <https://doi.org/10.1016/j.sna.2016.02.033>.
- [15] X. han Shen, Q. feng Liu, Z. Hu, W. qiang Jiang, X. Lin, D. Hou, P. Hao, Combine ingress of chloride and carbonation in marine-exposed concrete under unsaturated environment: A numerical study, *Ocean Eng.* 189 (2019) 106350. <https://doi.org/10.1016/j.oceaneng.2019.106350>.
- [16] V. Medvedev, A. Pustovgar, A Review of Concrete Carbonation and Approaches to Its Research under Irradiation, *Buildings*. 13 (2023).

- [17] S.K. Roy, D.O. Northwood, K.B. Pohs, Effect of plastering on the carbonation of a 19-year-old reinforced concrete building, *Constr. Build. Mater.* 29 (2012) 206–214. <https://doi.org/10.1016/j.conbuildmat.2011.08.049>.
- [18] A.F. Al Fuhaid, A. Niaz, Carbonation and Corrosion Problems in Reinforced Concrete Structures, *Buildings*. 12 (2022) 1–20. <https://doi.org/10.3390/buildings12050586>.
- [19] Z. Mi, Y. Hu, Q. Li, Z. An, Effect of curing humidity on the fracture properties of concrete, *Constr. Build. Mater.* 169 (2018) 403–413. <https://doi.org/10.1016/j.conbuildmat.2018.03.025>.
- [20] J. Zhao, G. Cai, L. Cui, A. Si Larbi, K.D. Tsavdaridis, Deterioration of basic properties of the materials in FRP-strengthening RC structures under ultraviolet exposure, *Polymers (Basel)*. 9 (2017). <https://doi.org/10.3390/polym9090402>.
- [21] Y. Wang, S. Nanukuttan, Y. Bai, P.A.M. Basheer, Influence of combined carbonation and chloride ingress regimes on rate of ingress and redistribution of chlorides in concretes, *Constr. Build. Mater.* 140 (2017) 173–183. <https://doi.org/10.1016/j.conbuildmat.2017.02.121>.
- [22] H. Kuosa, R.M. Ferreira, E. Holt, M. Leivo, E. Vesikari, Effect of coupled deterioration by freeze-thaw, carbonation and chlorides on concrete service life, *Cem. Concr. Compos.* 47 (2014) 32–40. <https://doi.org/10.1016/j.cemconcomp.2013.10.008>.
- [23] P. Fernandes, P. Silva, L. Correia, J. Sena-cruz, Durability of an epoxy adhesive and a CFRP laminate under different exposure conditions, in: *Third Conf. Smart Monit. Assess. Rehabil. Civ. Struct.* 2015, 2015: p. 9.
- [24] E. Cui, S. Jiang, J. Wang, X. Zeng, Bond behavior of CFRP-concrete bonding interface considering degradation of epoxy primer under wet-dry cycles, *Constr. Build. Mater.* 292 (2021) 123286. <https://doi.org/10.1016/j.conbuildmat.2021.123286>.
- [25] S.A. Grammatikos, R.G. Jones, M. Evernden, J.R. Correia, Thermal cycling effects on the durability of a pultruded GFRP material for offshore civil engineering structures, *Compos. Struct.* 153 (2016) 297–310. <https://doi.org/10.1016/j.compstruct.2016.05.085>.
- [26] D. dong Hu, J. xun Lyu, T. Liu, M. dong Lang, L. Zhao, Solvation effect of CO₂ on accelerating the curing reaction process of epoxy resin, *Chem. Eng. Process. - Process Intensif.* 127 (2018) 159–167. <https://doi.org/10.1016/j.cep.2018.01.027>.
- [27] P. Silva, P. Fernandes, J. Sena-Cruz, J. Xavier, F. Castro, D. Soares, V. Carneiro, Effects of different

environmental conditions on the mechanical characteristics of a structural epoxy, *Compos. Part B Eng.* 88 (2016) 55–63. <https://doi.org/10.1016/j.compositesb.2015.10.036>.

[28] S. Liu, Y. Pan, H. Li, G. Xian, Durability of the bond between CFRP and concrete exposed to thermal cycles, *Materials (Basel)*. 12 (2019). <https://doi.org/10.3390/ma12030515>.

[29] F. Jiang, X. Han, Y. Wang, P. Wang, T. Zhao, K. Zhang, Effect of freeze-thaw cycles on tensile properties of CFRP, bond behaviour of CFRP-concrete, and flexural performance of CFRP-strengthened concrete beams, *Cold Reg. Sci. Technol.* 194 (2022) 103461. <https://doi.org/10.1016/j.coldregions.2021.103461>.

[30] C. Helbling, V. Karbhari, Durability of composites in aqueous environments, in: *Durab. Compos. Civ. Struct. Appl.*, Elsevier, Amsterdam, The Netherlands, 2007: pp. 31–71.

[31] J. Sena-Cruz, P. Fernandes, P. Silva, J. Barros, M.R.F. Coelho, Bond behaviour of concrete elements strengthened with NSM CFRP laminate strips under wet-dry cycles, in: J.W. Cairns, G. Metelli, G. A. Plizzari (Eds.), *New Mater. under Sev. Cond.*, 2012 Publisher creations, 2012.

[32] J. Garzón-Roca, J.M. Sena-Cruz, P. Fernandes, J. Xavier, Effect of wet-dry cycles on the bond behaviour of concrete elements strengthened with NSM CFRP laminate strips, *Compos. Struct.* 132 (2015) 331–340. <https://doi.org/10.1016/j.compstruct.2015.05.053>.

[33] F. Al-Mahmoud, J.M. Mechling, M. Shaban, Bond strength of different strengthening systems - Concrete elements under freeze-thaw cycles and saltwater immersion exposure, *Constr. Build. Mater.* 70 (2014) 399–409. <https://doi.org/10.1016/j.conbuildmat.2014.07.039>.

[34] A. Dushimimana, L. Correia, R. Cruz, S. Cabral-fonseca, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-concrete bond and corresponding involved materials under different natural environmental exposures for a period of four years, in: *FRPRCS-15 and APFIS-2022*, Shenzhen, China, 2022: pp. 10–14.

[35] A. Dushimimana, L. Correia, R. Cruz, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-Concrete Bond in EBR and NSM Systems under Natural Ageing for a Period of Three Years, in: *Proc. 20th Eur. Conf. Compos. Mater.*, CC BY-NC 4.0, Lausanne, Switzerland, 2022: pp. 1–8. https://doi.org/https://doi.org/10.5075/epfl-298799_978-2-9701614-0-0.

[36] Committee 440, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, Farmington Hills, MI, USA, 2017.

[37] CNR-DT 200 R1, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures; CNR-DT 200 R1/2013; Rome, Italy, 2013.

[38] IPQ, IPQ (Instituto Português da Qualidade). 1992. NP EN EN 1992-1-1. Eurocode 2: design of concrete

structures - Part 1-1: general rules and rules for buildings, Instituto Português da Qualidade (IPQ)., Caparica, Portugal, 2010.

[39] S&P, S&P 220 Resin epoxy adhesive. Technical Data Sheet, Seewen, Switzerland, 2015.

[40] S&P, S&P CFRP Laminates. Technical Datasheet, Seewen, Switzerland, 2014.

[41] EN 1992-1-1 Eurocode 2, Design of concrete structures - Part 1-1 : General rules and rules for buildings, Brussels, Belgium:, 2004. [https://doi.org/EN 1992-1-1:2004](https://doi.org/EN%201992-1-1:2004) (E).

[42] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of bond of EBR CFRP laminates to concrete under real-time field exposure and laboratory accelerated ageing, Constr. Build. Mater. 377 (2023). <https://doi.org/10.1016/j.conbuildmat.2023.131047>.

[43] IPQ, IPQ (Instituto Português da Qualidade). 2013. NP EN 12390-13. Testing Hardened Concrete. Part 13: Determination of Secant Modulus of Elasticity in Compression, Instituto Português da Qualidade (IPQ)., Caparica, Portugal, 2013.

[44] IPQ, 2011. NP EN 12390-3. Testing Hardened Concrete. Part 3: Compressive Strength of Test Specimen, Instituto Português da Qualidade (IPQ), Caparica, Portugal, 2011.

[45] BSI, BSI (British Standards Institution). Products and systems for the protection and repair of concrete structures – Test methods – Measurement of bond strength by pull-off. EN 1542., London: BSI, 1999.

[46] ISO, ISO 527-2:2012—Plastics—Determination of Tensile Properties—Part 2: Test conditions for Moulding and Extrusion Plastics, Genève, Switzerland, 2012.

[47] ISO, ISO 527-5:2009 Part 5: Test Conditions for Unidirectional Fibre-Reinforced Plastic composites. Plastic—Determ. Tensile Prop., Inte Genève, Switzerland, 2009; Volume 1, 2009.

[48] T.P. Hills, F. Gordon, N.H. Florin, P.S. Fennell, Statistical analysis of the carbonation rate of concrete, Cem. Concr. Res. 72 (2015) 98–107. <https://doi.org/10.1016/j.cemconres.2015.02.007>.

[49] N. Bouzoubaâ, A. Bilodeau, B. Tamtsia, S. Foo, Carbonation of fly ash concrete: Laboratory and field data, Can. J. Civ. Eng. 37 (2010) 1535–1549. <https://doi.org/10.1139/L10-081>.

[50] L. Jiang, Y. Zhang, Analysis and calculation of the length of half – carbonated zone in concrete [in Chinese], Ind Constr. 29 (1999) 4–8.

[51] H.. Tseng, J.. Hsia, Concrete carbonation and durability prediction [in Chinese], Hunan Commun Sci Technol. 32 (2006) 121–6.

[52] M.I. Kabir, R. Shrestha, B. Samali, Effects of applied environmental conditions on the pull-out strengths

- of CFRP-concrete bond, *Constr. Build. Mater.* 114 (2016) 817–830.
<https://doi.org/10.1016/j.conbuildmat.2016.03.195>.
- [53] A. Dushimimana, A.A. Niyonsenga, F. Nzamurambaho, A review on strength development of high-performance concrete, *Constr. Build. Mater.* 307 (2021) 124865.
<https://doi.org/10.1016/j.conbuildmat.2021.124865>.
- [54] S.N. Shoukry, G.W. William, B. Downie, M.Y. Riad, Effect of moisture and temperature on the mechanical properties of concrete, *Constr. Build. Mater.* 25 (2011) 688–696.
<https://doi.org/10.1016/j.conbuildmat.2010.07.020>.
- [55] M. Leone, S. Matthys, M.A. Aiello, Effect of elevated service temperature on bond between FRP EBR systems and concrete, *Compos. Part B Eng.* 40 (2009) 85–93.
<https://doi.org/10.1016/j.compositesb.2008.06.004>.
- [56] J.P. Firmo, J.R. Correia, D. Pitta, C. Tiago, M.R.T. Arruda, Bond Behaviour between Near-Surface-Mounted CFRP Strips and Concrete at High Temperatures, *J. Compos. Constr.* 19 (2015) 04014071.
[https://doi.org/10.1061/\(asce\)cc.1943-5614.0000535](https://doi.org/10.1061/(asce)cc.1943-5614.0000535).
- [57] J.S. Cruz, J. Barros, Bond Between Near-Surface Mounted Carbon-Fiber- Reinforced Polymer Laminate Strips and Concrete, *J. Compos. Constr.* 82 (2004) 1513–1521.
<https://doi.org/10.1016/j.compstruc.2004.03.047>.

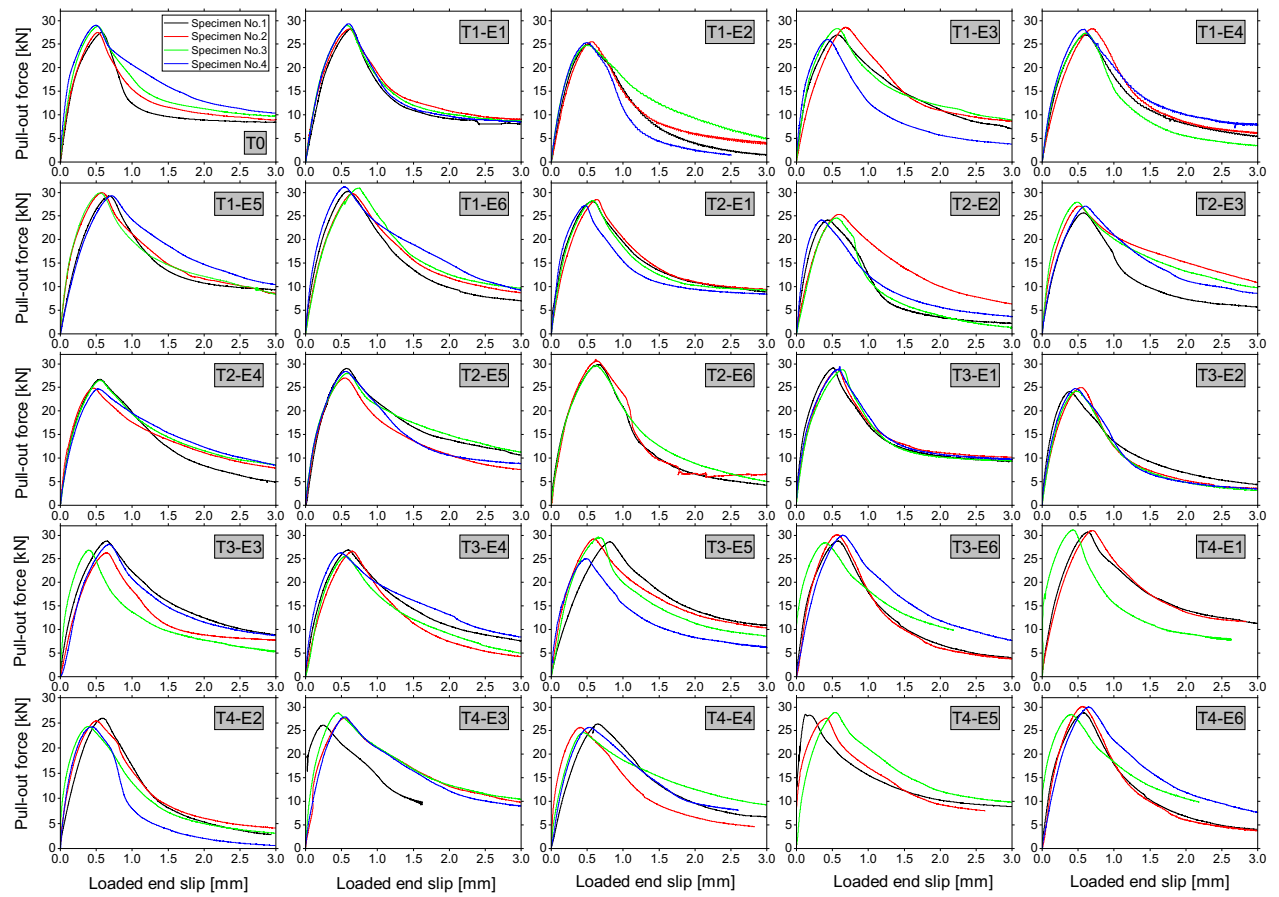
DURABILITY OF NEAR-SURFACE MOUNTED CFRP-TO-CONCRETE BOND:
ACCELERATED *VERSUS* NATURAL AGING

ANNEX III.1. Single-lap shear test results for all the specimens tested from T0 to T4.

Time	Specimen	F_{max} [kN]	s_{max} [mm]	F_{max} [kN]	s_{max} [mm]	F_{max} [kN]	s_{max} [mm]
T0	No.1	24.47	0.59	-	-	-	-
	No.2	27.55	0.53	-	-	-	-
	No.3	28.81	0.53	-	-	-	-
	No.4	28.99	0.50	-	-	-	-
E1		E2		E3			
T1	No.1	28.30	0.65	29.05	0.51	26.98	0.57
	No.2	28.26	0.63	25.53	0.57	29.54	0.71
	No.3	29.16	0.60	24.77	0.49	28.40	0.55
	No.4	29.50	0.60	25.36	0.49	26.06	0.43
T2	No.1	28.07	0.56	24.18	0.42	25.68	0.57
	No.2	28.48	0.63	25.41	0.61	27.11	0.51
	No.3	28.30	0.57	24.62	0.56	27.94	0.48
	No.4	27.27	0.47	24.18	0.35	27.11	0.60
T3	No.1	29.12	0.56	24.11	0.43	28.78	0.64
	No.2	28.76	0.63	24.99	0.61	26.28	0.64
	No.3	28.77	0.57	24.20	0.56	26.83	0.40
	No.4	29.43	0.47	24.76	0.47	28.07	0.68
T4	No.1	-	-	25.94	0.59	26.16	0.24
	No.2	30.66	0.63	25.36	0.50	27.70	0.55
	No.3	31.01	0.70	24.30	0.39	28.76	0.44
	No.4	31.18	0.43	24.20	0.44	27.96	0.55
E4		E5		E6			
T1	No.1	27.00	0.61	29.29	0.68	30.26	0.59
	No.2	28.28	0.68	29.98	0.60	29.77	0.66
	No.3	27.39	0.63	30.04	0.56	30.96	0.75
	No.4	28.16	0.58	29.39	0.73	31.17	0.54
T2	No.1	26.70	0.54	29.04	0.58	29.90	0.65
	No.2	24.86	0.46	27.01	0.55	27.11	0.47
	No.3	26.50	0.55	27.98	0.58	30.99	0.65
	No.4	24.66	0.52	28.40	0.56	29.65	0.54
T3	No.1	26.86	0.60	28.63	0.82	28.78	0.57
	No.2	26.59	0.65	29.26	0.60	30.11	0.57
	No.3	25.53	0.54	29.66	0.65	28.43	0.41
	No.4	26.30	0.52	25.02	0.49	30.01	0.65
T4	No.1	26.46	0.65	28.36	0.19	28.78	0.57
	No.2	25.68	0.41	27.65	0.41	30.11	0.57
	No.3	24.61	0.44	-	-	28.43	0.41
	No.4	25.73	0.53	28.86	0.54	30.01	0.65

Notes: F_{max} : maximum pull-out force; s_{max} : loaded end slip at F_{max} ; E1-E6: studied environments; T0: initial time before exposure; T1-T4: exposure time in years.

ANNEX III.2. Single-lap shear test results: Pull-out force *versus* loaded end slip curves for all specimens tested up to 4 years.



PAPER IV

FINITE ELEMENT MODELING OF SHORT-TERM BEHAVIOR OF EXTERNALLY BONDED CFRP-TO- CONCRETE JOINT

REFERENCE: *Aloys Dushimimana, João M. Pereira, Luís Correia, Anastasios P. Vassilopoulos, José Sena-Cruz.*
Finite element modeling of bond between concrete and CFRP laminate. To be submitted to a journal.

ABSTRACT: Existing reinforced concrete structures can be strengthened with carbon fiber reinforced polymer (CFRP) laminates using externally bonded reinforcement (EBR) technique. Several existing numerical models for predicting the behavior of CFRP-to-concrete joint with the EBR technique adopt bond stress-slip law approaches which require proper definition of the cohesive (or connector) behavior. In this work, an alternative approach that uses a perfect bond at the interfaces (concrete/adhesive and adhesive/CFRP) and a damage

plasticity model to simulate the nonlinear behavior of both concrete and epoxy adhesive is proposed. Evolutions of damage, slip, and stresses at material and joint levels are examined. The developed approach leads to a very good agreement with experimental results and shows consistency even when adopting different constitutive laws for concrete or adhesive.

KEYWORDS: Composites, Epoxy adhesive, Concrete, Numerical Modeling, EBR, Damage Plasticity Model.

1. INTRODUCTION

Carbon fiber-reinforced polymer (CFRP) composites have shown interesting properties that fulfil the requirements for structural rehabilitation. CFRP composites possess very high stiffness and high strength-to-weight ratios [1], and generally show promising durability properties compared to other materials [2–7]. Hence, they have usually been used to strengthen existing RC structures. Different strengthening techniques using CFRP composites already exist, the most common being externally bonded reinforcement (EBR) [4,8] and near-surface mounted (NSM) [1,9]. In both techniques, a bonding agent is required to bond the CFRP to concrete. One of the most used bonding agents is epoxy adhesive, examples of its use include studies by [3,5,10–12], among others. The adoption of the epoxy adhesive relies on its ability to form strong covalent bonds when bonded to CFRP composites [13]. Also, the adhesive forms chemical bonding and mechanical interlock when bonded to concrete (the primary chemical interaction being hydrogen bonding in the former [14], and the latter resulting from the flow of epoxy through the open pores, crevices and irregularities of roughened concrete [15]).

Focusing on the EBR technique, existing literature presents a substantial number of relevant experimental tests, e.g., [2,4,8,10,16–21], from which the failure mode was generally reported to be debonding with a very thin concrete layer or concrete cohesive failure. On the other hand, several numerical models to predict the EBR CFRP-to-concrete bond exist, e.g., [22–30]. However, many of these models have generally adopted the bond-slip law (using a cohesive or connector element) to model the CFRP-to-concrete joint, due to its popularity, e.g., [21,31–36]. The above existing models are generally based on the fact that a failure will initiate at the interface (i.e., at region where the cohesive or connector elements are defined), in addition, the input parameters to these models require complex derivations relying on a combination of experimental data and inverse analyses.

In this regard, as an alternative approach, the present work proposes a model that explicitly simulates the epoxy adhesive between the CFRP and the concrete, assuming a perfect bond between the different materials (concrete/adhesive and adhesive/CFRP). With this assumption, the interface failure is not captured,

nevertheless, the fact that cohesive failure has been generally observed leads to less interest in predicting the interface failure. Both concrete and epoxy adhesive were modelled using concrete damage plasticity (CDP) model, adopted in [37–39], while the CFRP laminate was modelled as a linear elastic material up to failure. Different existing mathematical formulations for establishing material constitutive models were adopted to examine the reliability of the developed approach. As a result, seven different models were used, and their results were compared with those from experimental tests.

2. RESEARCH METHODS

2.1. Experimental tests

2.1.1. Concrete

A single batch of concrete was used to produce concrete cylinders and prisms. The 28 days compressive strength and elastic modulus were 41.5 MPa and 29.1 GPa, respectively. Further details about this concrete can be found in [8].

2.1.2. Epoxy adhesive and CFRP laminate

The properties of the epoxy adhesive and CFRP laminate used in this work are given in **Table IV.1**, as provided on datasheets from the supplier (S&P® Clever Reinforcement Ibérica Lda Company, Seixal, Portugal) [40].

Table IV.1. Properties of the epoxy adhesive and CFRP laminate as per supplier [46].

Epoxy adhesive	
Type	Cold-curing S&P Resin 220
Elastic modulus [GPa]	>7.1
T_g [°C]	N/A
Density, at 23 °C [g/cm ³]	1.7 - 1.8
Compressive strength [MPa]	>70
Shear strength [MPa]	>26
BS_{by} pull-off, on concrete [MPa]	>3 (after 3d curing at 20 °C)
CFRP laminate	
Type and trademark	S&P clever (CFK 150/2000)
Prefabricated by	Pultrusion
Orientation	Unidirectional
Fiber content [%]	68
Fiber matrix	Vinyl ester resin (with $T_g \approx 85$ °C)
External surface	Black, smooth
Elastic modulus [GPa]	>170
Tensile strength [MPa]	>2000

Notes: T_g : glass transition temperature, BS : bond strength, d: days, N/A: not available

Additional experimental tests were performed to determine the missing information. Further details on these tests can be found in the previous work by the authors [41].

2.1.3. Concrete elements strengthened with CFRP laminates

Two CFRP laminates were bonded to a concrete prism (with $400 \times 200 \times 200$ [mm] dimensions) using an epoxy adhesive. The properties of the resulting EBR CFRP-to-concrete joint were as follows: CFRP laminate width of 50 mm and thickness of 1.2 mm, bond length of 220 mm, theoretical effective bond length of 101 mm as per CNR [43], and adhesive thickness varying between 1.5-2.0 mm. Four single lap shear tests were performed: the specimen's geometry and the test set-up being as shown in **Figure IV.1a** and **Figure IV.1b**, respectively. Three different supports (denoted as Support 1, 2, and 3, see **Figure IV.1a**) were used to restrain any movement of the concrete prism. Furthermore, three linear variable differential transformers (LVDT) were used to measure the specimen's movement at selected locations. That is, LVDT1 and LVDT2 were used to capture the loaded end and the free end movements, respectively (see **Figure IV.1a**); and the LVDT3 was installed on top of the Support 3 to verify if there is any vertical movement of the concrete prism (see **Figure IV.1b**). It is worth noting that due to the test setup used (**Figure IV.1b**), where the LVDT1 and LVDT2 were fixed at selected reference points on top face of concrete prism before loading, the term *slip* is defined as the global displacement of the CFRP laminate referring to those reference points, rather than the relative displacements between CFRP-concrete interfaces.

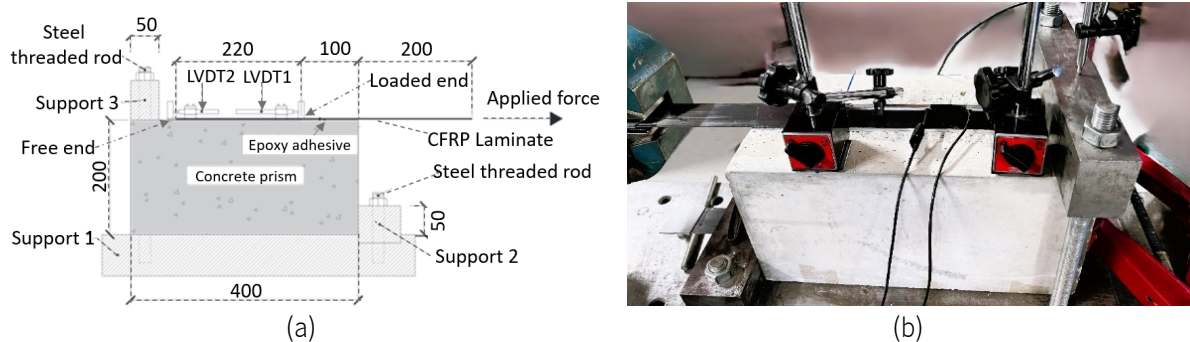


Figure IV.1. Single lap shear test: (a) specimen's geometry and test set-up (adapted from previous experimental work by authors' group in [8]), (b) photograph of the full test configuration. Notes: all units in [mm].

2.2. Experimental test results

The responses of concrete, epoxy adhesive, CFRP laminate, and EBR CFRP-to-concrete bond obtained from testing are shown in **Table IV.2**, and the pull-out force *versus* loaded-end slip curves in **Figure IV.2**. The curves exhibit a typical behavior, beginning with an initial phase dominated by chemical bonding, which is reflected by an increase in the pull-out force with minimal variation of the slip. As the applied load increases, the bond starts to deteriorate, leading to the onset of the debonding stage. However, complete debonding is delayed because the bond length used (220 mm) exceeds the theoretical effective length (101 mm). This allows for a continued increase in the slip with minimal variation in the pull-out force, resulting in the formation of a plateau-like region on the curve. Consequently, the joint failure occurs gradually rather than abruptly. Further detailed results can be found in [8,44].

Table IV.2. Results from concrete, epoxy adhesive, CFRP laminate, and EBR CFRP-to-concrete bond testing.

Material property	Concrete	Epoxy adhesive	CFRP laminate	EBR CFRP-to-concrete bond
Compressive strength [MPa]	41.5 (4.4)	-	-	-
Tensile strength [MPa]	-	19.9 (3.0)	2527 (11)	-
Tensile strain [%]	-	0.4 (6.2)	13.3 (13.6)	-
Elastic modulus [GPa]	-	-	190 (9.3)	-
Pull-out force [kN]	-	-	-	27.91*

Notes: values in () express coefficient of variation in %,*value from mean curve derived from 4 experimental curves.

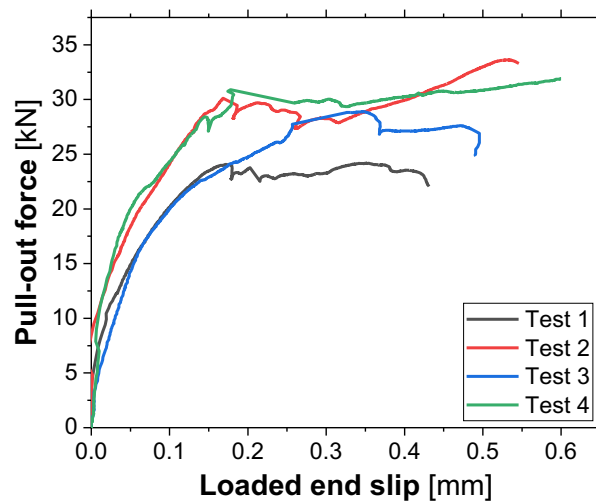


Figure IV.2. Pull-out force *versus* loaded end slip curves from single lap shear tests.

2.3. Finite element model

2.3.1. Development of material constitutive models

The concrete damage parameter (CDP) model [37–39] available in ABAQUS [45] was used to model concrete and epoxy adhesive. For a material under compression, CDP requires inputting material compressive stress, compressive inelastic strain, and compressive damage parameter; and under tension it requires tensile stress, tensile cracking strain (or crack opening displacement) and tensile damage parameter. All these inputs were obtained as follows.

2.3.1.1. Concrete model

Concrete under compression

Saenz model [46] defined by Eq. (IV.1) was adopted to model concrete compressive behavior. This model describes the complete stress-strain relationship including initial elasticity, nonlinear hardening, peak strength, and a post-peak softening. The model effectively captures both the ascending and descending phases, making it effective for predicting material's response through all stages including softening and failure. Using Eq. (IV.1) and results from Table IV.2, the stress-strain relationship was established and plotted in Figure IV.3a.

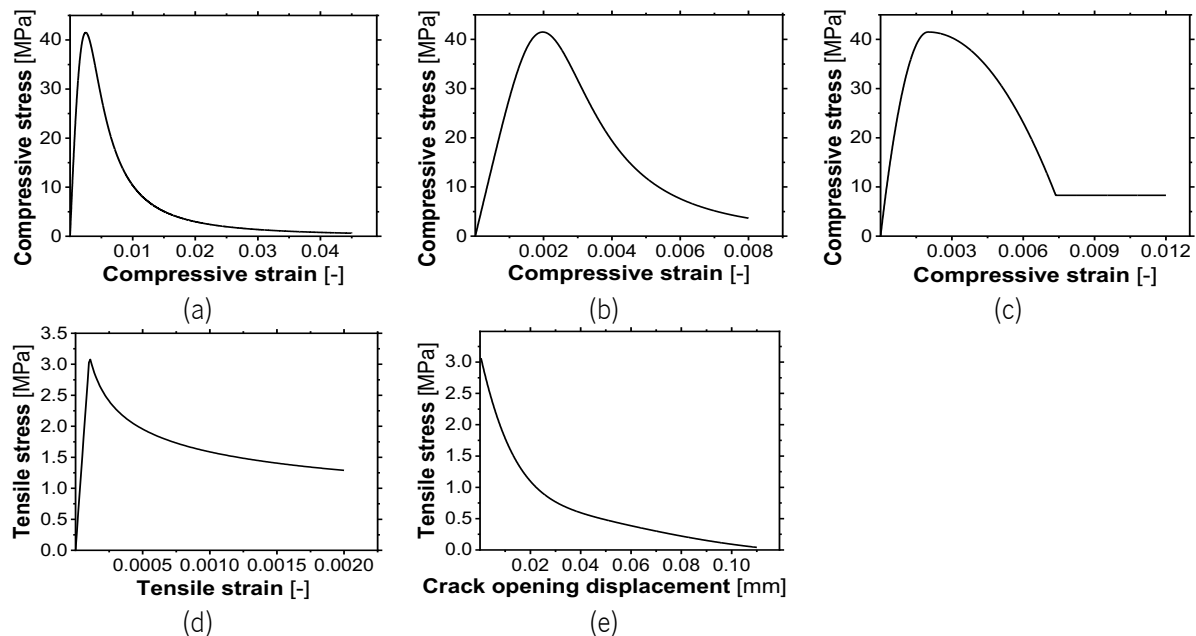


Figure IV.3. Concrete constitutive models under compression by (a) Saenz [46], (b) Carreira and Chu [47], (c) Wang and Hsu [48]; and, under tension by (d) Belarbi and Hsu [49], and (e) Horjick [50].

From Eq. (IV.1), concrete compressive inelastic strain (ε_{cin}) and concrete compressive damage parameter (d_c) were obtained using Eq. (IV.2) and Eq. (IV.3) [51], respectively.

$$\sigma_{cc} = \frac{E_c \varepsilon_{cc}}{1 + (R + R_E - 2) \frac{\varepsilon_{cc}}{\varepsilon_{cu}} - (2R - 1) \left(\frac{\varepsilon_{cc}}{\varepsilon_{cu}} \right)^2 + R \left(\frac{\varepsilon_{cc}}{\varepsilon_{cu}} \right)^3} \quad (IV.1)$$

where, $R = \left[(R_E(R_\sigma - 1)) / (R_E - 1)^2 \right] - 1/R_E$, $R_E = E_c/E_0$, $E_0 = \sigma_{cu}/\varepsilon_{cu}$, $R_\sigma = 4$, $R_\sigma = 4$

$$\varepsilon_{cin} = \varepsilon_{cc} - \varepsilon_{cel} \quad (IV.2)$$

$$d_c = 1 - \frac{\sigma_{cc}}{\sigma_{cu}} \quad (IV.3)$$

where, σ_{cu} , σ_{cc} , ε_{cc} , ε_{cu} , ε_{cel} , and E_c are the maximum compressive stress, compressive stress, compressive strain, compressive strain corresponding to the σ_{cu} , elastic compressive strain, and elastic modulus (calculated as per CEP-FIP [52], i.e., $E_c = 4700\sqrt{\sigma_{cu}}$, respectively).

Concrete under compression was also modelled using Eq. (IV.4)-(IV.5) by Carreira and Chu [47]. This model is a general form of a serpentine curve presenting both ascending and descending branches, and covers a wide spectrum of experimental data [47].

$$\frac{\sigma_{cc}}{\sigma'_c} = \frac{\beta(\varepsilon_{cc}/\varepsilon'_c)}{\beta - 1 + (\varepsilon_{cc}/\varepsilon'_c)^\beta} \quad (IV.4)$$

$$\beta = \frac{1}{1 - \frac{\sigma'_c}{\varepsilon'_c E_{it}}} \quad (IV.5)$$

for $\beta \geq 1$ and $\varepsilon_{cc} \leq \varepsilon_{cu}$

Where, σ_{cc} , ε_{cc} , and β are the compressive stress, compressive strain, and a material parameter that depends on the shape of the stress-strain diagram, ε_{cu} is the maximum compressive strain, σ'_c , ε'_c , and E_{it} are parameters from compression tests [47]. Using results from Table IV.2 and Eq. (IV.4)-(IV.5), the resulting stress-strain relationship is depicted in Figure IV.3b. The inelastic strain and the damage parameter were determined using Eq. (IV.2) and Eq. (IV.3), respectively.

The behavior under compression was also modelled using Eq. (IV.6)-(IV.8) following Wang and Hsu [48]. This model captures elastic phase, peak strength, softening, and a residual strength. Its piecewise formulation accurately represents all loading stages.

$$\sigma_{cc} = \zeta \sigma_{cu} \left[2 \left(\frac{\varepsilon_{cc}}{\zeta \varepsilon_0} \right) - \left(\frac{\varepsilon_{cc}}{\zeta \varepsilon_0} \right)^2 \right] \quad \varepsilon_{cc}/\zeta \varepsilon_0 \leq 1 \quad (IV.6)$$

$$\sigma_{cc} = \zeta \sigma_{cu} \left[1 - \left(\frac{\varepsilon_{cc}/\zeta \varepsilon_0 - 1}{4/\zeta - 1} \right)^2 \right] \quad \varepsilon_{cc}/\zeta \varepsilon_0 > 1 \quad (IV.7)$$

$$\text{if } \sigma_{cc} \leq 0.2 \zeta \sigma_{cu} \quad \text{then } \sigma_{cc} = 0.2 \zeta \sigma_{cu} \quad (IV.8)$$

where, σ_{cc} and ε_{cc} are the compressive stress and strain, respectively, σ_{cu} is the concrete cylinder compressive strength, ε_0 is taken as 0.002 [37], ζ is the softened coefficient. With results from **Table IV.2** and **Eq. (IV.6)-(IV.8)**, the resulting stress-strain relationship is as shown in **Figure IV.3c**. Inelastic strain and damage parameter were calculated as in **Eq. (IV.2)** and **Eq. (IV.3)**, respectively.

Concrete under tension

To model the behavior of concrete under tension, the models proposed by Belarbi and Hsu [49] and Horjick [50] were adopted. The stress-strain relationship from the former [49] is governed by **Eq. (IV.9)**, with the maximum tensile stress calculated using **Eq. (IV.11)** as per CEP-FIP [52]. Belarbi and Hsu [49] model accounts for the tension-stiffening effect, where cracked concrete continues to carry tensile stress due to its bond with reinforcing material. Using results from **Table IV.2** and **Eq. (IV.9)-(IV.10)**, the stress-strain relationship is as shown in **Figure IV.3d**. From this relationship, the tensile cracking strain (ε_{Tcr}) and the tensile damage parameter (d_t) were determined using **Eq. (IV.11)** and **Eq. (IV.12)**, respectively.

$$\sigma_{ct} = \begin{cases} E_0 \varepsilon_{ct} , & \varepsilon_{ct} \leq \varepsilon_{tcr} \\ \sigma_{tu} \left(\frac{\varepsilon_{tcr}}{\varepsilon_{ct}} \right)^r , & \varepsilon_{ct} > \varepsilon_{tcr} \end{cases} \quad (IV.9)$$

$$\sigma_{tu} = 1.4 ((\sigma_{cu} - 8)/10)^{2/3} \quad (IV.10)$$

$$\varepsilon_{Tcr} = \varepsilon_{ct} - \varepsilon_{Tel} \quad (IV.11)$$

$$d_t = 1 - \frac{\sigma_{ct}}{\sigma_{tu}} \quad (IV.12)$$

where, σ_{tu} , σ_{ct} , ε_{ct} , ε_{tcr} , σ_{cu} , ε_{Tel} , E_0 are the maximum tensile stress, tensile stress, tensile strain, tensile strain at initial crack, maximum compressive stress, elastic tensile strain, and initial elastic modulus, respectively. The empirical parameter r typically ranges between 0.4 and 0.5 [49]. In this work, r was set to 0.4, referring to [48].

Furthermore, the model by Horjick [50] is defined by **Eq. (IV.13)**. This model accurately captures both the peak tensile strength and the nonlinear stress decline as cracks propagate until stress reaches zero at a characteristic crack width. The tensile stress (σ_{ct})-crack opening displacement (w_t) relationship (**Figure IV.3e**) is established referring to results from **Table IV.2**, **Eq. (IV.10)** and **Eq. (IV.13)**. From this relation, the tensile damage parameter (d_t) was obtained using **Eq. (IV.14)**.

$$\frac{\sigma_{ct}}{\sigma_{tu}} = \begin{cases} \left[1 + \left(c_1 \frac{w_t}{w_{cr}} \right)^3 \exp \left(-c_2 \frac{w_t}{w_{cr}} \right) - \left(\frac{w_t}{w_{cr}} \right) (1 + c_1^3) \exp(-c_2) \right], & 0 < w_t < w_{cr} \\ 0, & w_{cr} < w_t < \infty \end{cases} \quad (\text{IV.13})$$

$$d_t = 1 - \frac{\sigma_{ct}}{\sigma_{tu}} \quad (\text{IV.14})$$

where, σ_{tu} and w_{cr} are the maximum tensile stress and maximum crack opening displacement, respectively; $w_{cr} = 5.14(G_f/\sigma_{tu})$, $G_f = (0.0469d_a^2 - 0.5d_a + 26)(\sigma_{cu}/10)^{0.7}$, $c_1=3.0$, and $c_2=6.93$ [37], $d_a=12.5$, where: G_f , σ_{cu} , and d_a are fracture energy, concrete compressive stress, and maximum aggregate size, respectively.

2.3.1.2. Epoxy adhesive model

Epoxy adhesive under compression

Modeling the behavior of epoxy adhesive under compression has not yet been fully established in the existing literature; a few existing works that attempted to establish the constitutive models under compression, e.g., [53–55], showed that the compressive behavior may significantly depend on how the load is applied. Some of these models, e.g. [54], present a stress-strain relation closely related to that obtained using Eurocode 2 [56]; hence, for simplification **Eq. (IV.15)-(IV.17)** as per Eurocode 2 [56] are adopted. A confined epoxy adhesive (like the case in the present work) may not present behaviors generally found when tested alone. Hence, this leads to adopting the already well-established models to assess their capability to model a confined epoxy adhesive.

$$\sigma_{ac} = \sigma_{cu} \frac{K\eta - \eta^2}{1 + (K - 2)\eta} \quad (\text{IV.15})$$

$$\eta = \varepsilon_{ac} / \varepsilon_{c1} \quad (\text{IV.16})$$

$$K = \frac{1.05E_c\varepsilon_{c1}}{\sigma_{cu}} \quad (\text{IV.17})$$

where σ_{ac} is the compressive stress, ε_{ac} is the compressive strain, ε_{c1} is the compressive cracking strain, σ_{cu} is the maximum compressive strength. With Eq. (IV.15)-(IV.17) and properties from Table IV.1 and IV.2, the resulting stress-strain relationship is shown in Figure IV.4a. From this relation, the inelastic strain and damage parameter were calculated using Eq. (IV.2) and Eq. (IV.3), respectively.

The behavior under compression was additionally modelled using Eq. (IV.1) by [46] previously described and Eq. (IV.2)-(IV.3) with inputs from Table IV.1 and IV.2, and the resulting stress-strain relation are shown in Figure IV.4b.

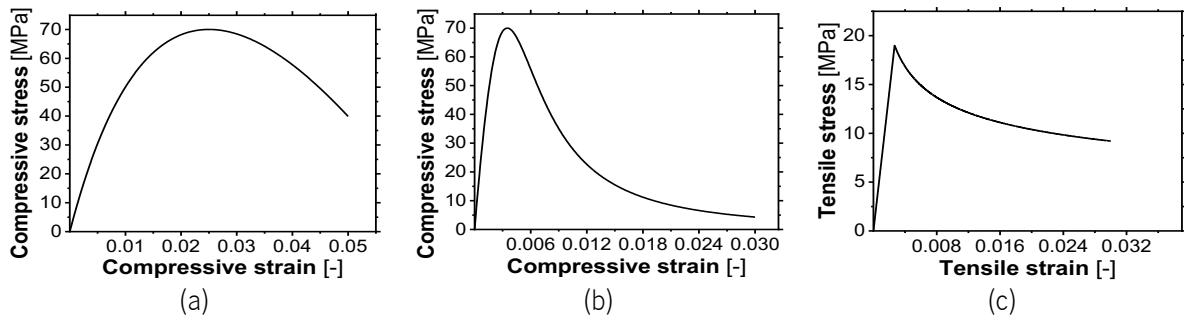


Figure IV.4. Epoxy adhesive constitutive models referring to Table IV.1 and Table IV.2, and existing formulae for behavior under compression from (a) Eurocode 2 [56] and (b) Saenz [46], and under tension from (c) Belarbi and Hsu [49].

Epoxy adhesive under tension

An attempt to model the epoxy adhesive behavior using Eq. (IV.9) [49] and Eqs. (IV.11)-(IV.12) was performed with inputs from Table IV.2; the resulting stress-strain curve is plotted in Figure IV.4c. In fact, the behavior observed in [57] was closely related to that shown in Figure IV.4c, hence strengthening the adoption of the above model.

2.3.1.3. CFRP laminate model

The CFRP laminate was modelled as linear elastic up to failure [25].

2.3.2. Modeling approach

A total of seven different approaches were developed as shown in Table IV.3. ABAQUS explicit [45] was used to model the EBR CFRP-to-concrete joint. The workflow during FEM modeling is provided in Figure IV.5. As previously explained in Section 2.1.3, the slip was experimentally defined as the global displacement of the CFRP laminate, rather than the relative displacement between materials, similarly, the slip in the developed

model is referred to as the global displacement in the direction of the applied force (i.e., U_3). Also, ABAQUS uses the terms DAMAGEC and DAMAGET to represent the compressive and tensile damage, respectively.

Table IV.3. FEM model combinations adopted for concrete and epoxy adhesive.

FE Models	Concrete under compression	Concrete under tension	Adhesive under compression	Adhesive under tension
<i>M1</i>	Eq.(1) [46]	Eq.(9) [49]	Eq.(15) [56]	Eq.(9) [49]
<i>M2</i>	Eq.(1) [46]	Eq.(13) [50]	Eq.(15) [56]	Eq.(9) [49]
<i>M3</i>	Eq.(4) [47]	Eq.(13) [50]	Eq.(15) [56]	Eq.(9) [49]
<i>M4</i>	Eq.(6)-(8) [48]	Eq.(13) [50]	Eq.(15) [56]	Eq.(9) [49]
<i>M5</i>	Eq.(1) [46]	Eq.(13) [50]	Eq.(1) [46]	Eq.(9) [49]
<i>M6</i>	Eq.(6)-(8) [48]	Eq.(9) [49]	Eq.(15) [56]	Eq.(9) [49]
<i>M7</i>	Eq.(1) [46]	Eq.(9) [49]	Eq.(1) [46]	Eq.(9) [49]

Note: the inputs to the equations mentioned are from **Table IV.1** and **Table IV.2**.

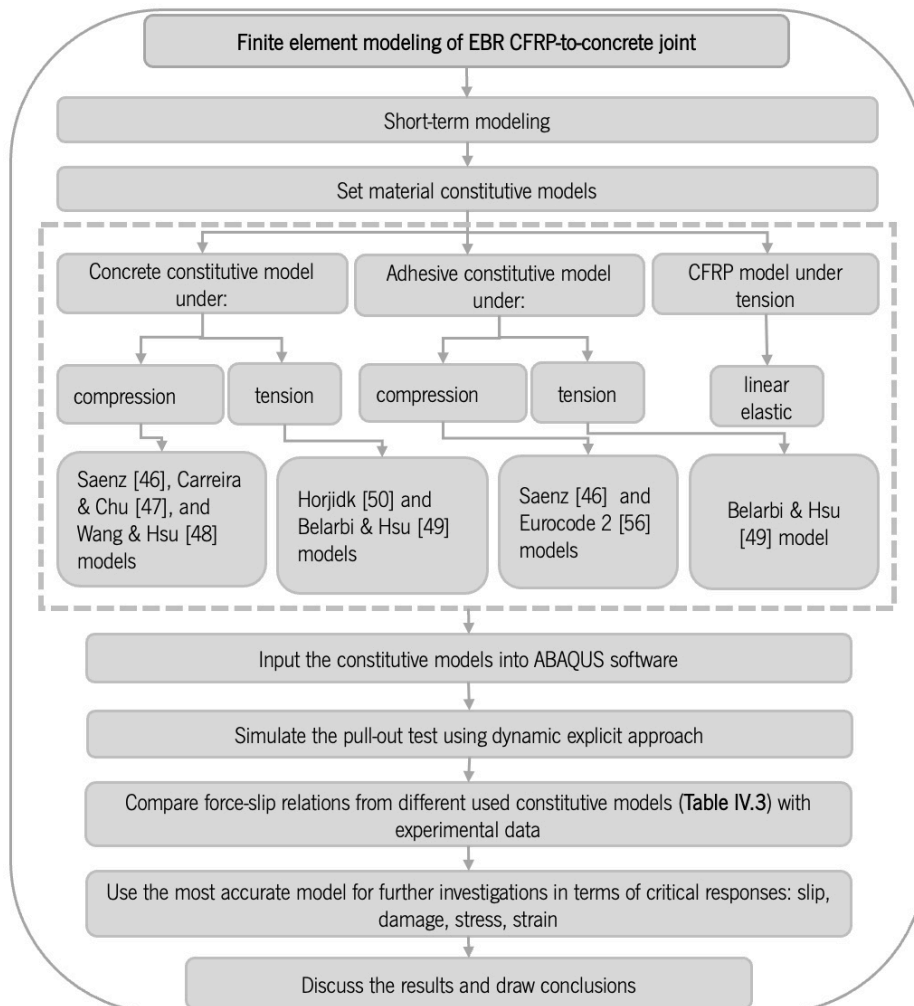


Figure IV.5. Workflow of finite element modeling of the EBR CFRP-to-concrete joint.

The geometry and dimensions (previously shown in **Figure IV.1a**) were used to construct parts of the model (i.e., concrete prism, adhesive layer and CFRP laminate). The material properties of concrete, epoxy adhesive and CFRP laminate are shown in **Table IV.4**. The plasticity flow parameters for both concrete and epoxy adhesive are: i) dilation angle of 38, eccentricity of 0.1, f_{b_0}/f_{c_0} ratio of 1.16, viscosity parameter of 0.00, and the value of K as 0.6667. These values were also used in previous studies for concrete, e.g., [39]. Although these parameters have been specifically derived from studies addressing concrete, it is assumed that they can equally be applied to any brittle material.

Table IV.4. Material properties for the developed FEM model

Material	Density (kN/m ³)	Young modulus (GPa)	Poisson ratio
Concrete	2.4	32.3	0.2
Epoxy adhesive	1.7	7.1	0.38
CFRP laminate	1.6	190	0.3

To create interaction between model parts, reference points were first created in the locations of the three supports shown in **Figure IV.1a**. Tie constraints were applied at the interfaces. Furthermore, boundary conditions were created at each of the created reference points. The displacement/rotation was used to apply the load at the CFRP laminate. Concrete and epoxy adhesive layer were modelled as a 3D deformable solid using a *3D Stress* (C3D8R) element, and the CFRP laminate as 3D deformable shell with S4R element. After performing a mesh sensitivity analysis, all bond constituent materials were given the same mesh size at their interaction surfaces, i.e., 5×5 [mm] hex shapes, as shown in **Figure IV.6**.

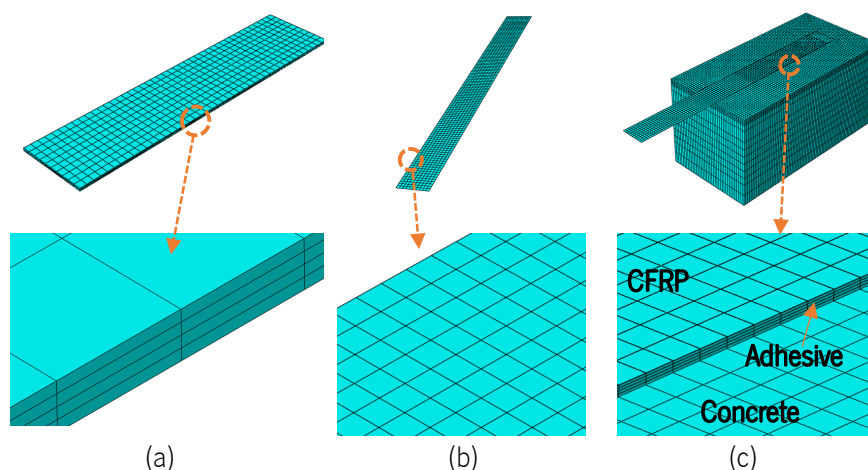


Figure IV.6. Finite element model of the EBR CFRP-to-concrete bond: (a) epoxy adhesive with its enlargement, (b) CFRP laminate with its enlargement, and (c) full size of the model with constituent materials highlighted.

3. RESULTS AND DISCUSSION

3.1. Pull-out force *versus* loaded end slip

Comparative pull-out force *versus* loaded end slip ($F_t - S_t$) curves between experimental results and those from FE model are shown in **Figure IV.7a**, along with maximum values in **Table IV.5**. It can be noted that all FEM model curves generally fall within the shaded area representing the envelope of experimental curves. The $F_t - S_t$ relationship and kinetic energy (KE)-internal energy (IE) relations are plotted in **Figure IV.7b**. The KE/IE ratios fall below the exceedance line as recommended in [45], hence strengthening the accuracy of the model. Furthermore, **Figure IV.7c** shows that models $M2$, $M4$, $M5$, and $M7$ led to predictions that perfectly fall within the aforementioned shaded area.

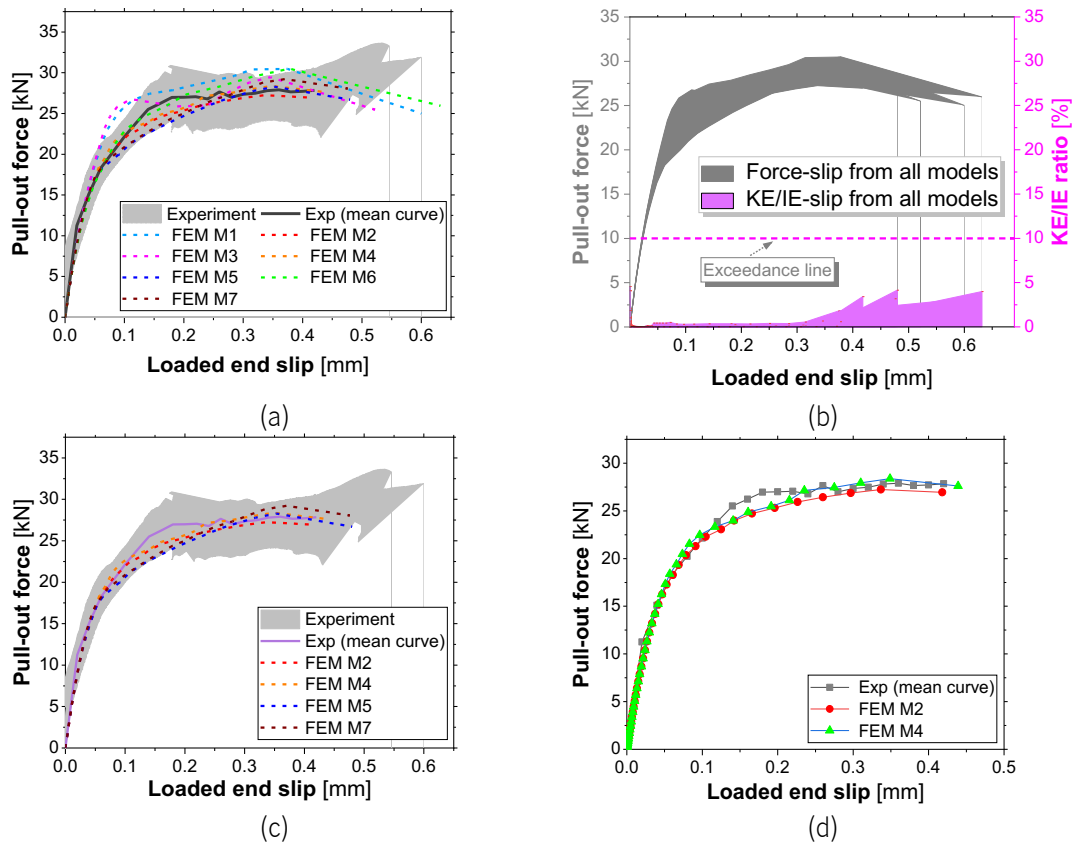


Figure IV.7. EBR CFRP-to-concrete bond responses: (a) experimental *versus* all FEM model data, (b) model validity check, (c) models showing superior prediction capacities, and (d) selected models ($M2$ and $M4$) with experimental mean curve.

Models $M2$ and $M4$ seem to be the most accurate since they simulate the experimental mean curve better than all other used models (see **Figure IV.7d**). Better performances from these two models may be attributed to the

material constitutive laws adopted. M2 is selected and used in the rest of the work owing to its better prediction accuracy in terms of both the maximum pull-out force and slip with only 2.4 % and 0.0 % differences, respectively (see **Table IV.5**). Additionally, while other models also lead to similar values of the external work (Ew) and IE as per [45], M2 shows a closer matching of the Ew and IE (**Figure IV.8a**), hence also proving its prioritization.

Table IV.5. Experimental versus FEM model: maximum pull-out force and maximum loaded end slip.

Experiment / FEM Models	$F_{l,max}$ [kN]	$S_{l,max}$ [mm]
<i>Experiment (mean curve)</i>	27.91	0.42
<i>M1</i>	30.47 (8.4 %)	0.60 (42.9 %)
<i>M2</i>	27.25 (2.4 %)	0.42 (0.0 %)
<i>M3</i>	29.45 (5.2 %)	0.52 (23.8 %)
<i>M4</i>	28.37 (1.6 %)	0.44 (4.8 %)
<i>M5</i>	28.30 (1.4 %)	0.48 (14.3 %)
<i>M6</i>	30.50 (9.3 %)	0.63 (50.0 %)
<i>M7</i>	29.23 (4.7 %)	0.48 (14.3 %)

Notes: $F_{l,max}$: maximum pull-out force, $S_{l,max}$: maximum loaded end slip, the percentages shown in () indicate differences between FEM model values and the experimental values.

3.2. Further analysis using model M2

The damage, slip, and von-Mises stress responses were examined at both material and joint levels, at different loading stages (see **Figure IV.8b**): at a loading level (LL) of 10 kN (denoted as LL1), 15 kN (LL2), 20 kN (LL3), 25 kN (LL4), 27.24 kN (LL5) and 26.90 kN (LL6).

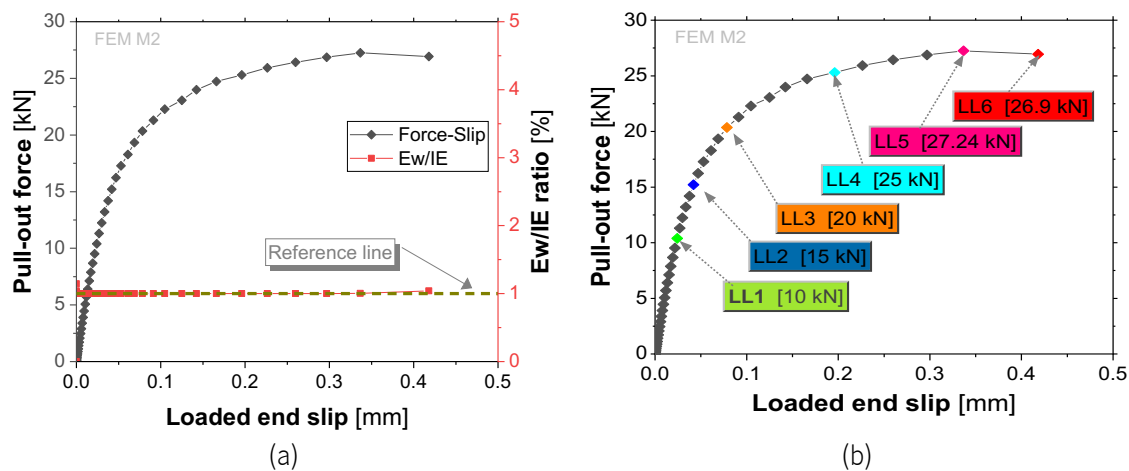


Figure IV.8. Details of the selected FEM model M2: (a) comparison of external work and internal energy with force-slip relation, (b) loading levels considered in the present work. Note: LL stands for load level.

3.2.1. Responses at material level

3.2.1.1. Concrete

Concrete damage at failure stage is plotted in **Figure IV.9a** and **Figure IV.9b**, respectively. The concrete substrate shows higher tensile damage up to approximately 80 mm from the loaded end, thereafter, decreasing progressively towards the free end, and again showing higher damage at regions close to the free end. On the other hand, higher compressive damage is observed across the entire bonded region. Higher slip is observed from the loaded end with progressive decreases towards the free end (**Figure IV.9c**), and higher stresses are observed at the loaded end and are more pronounced at the free end. Furthermore, higher stresses can also be seen at longitudinal edges of the bonded region (**Figure IV.9d**), due to easier cracking at edges [58].

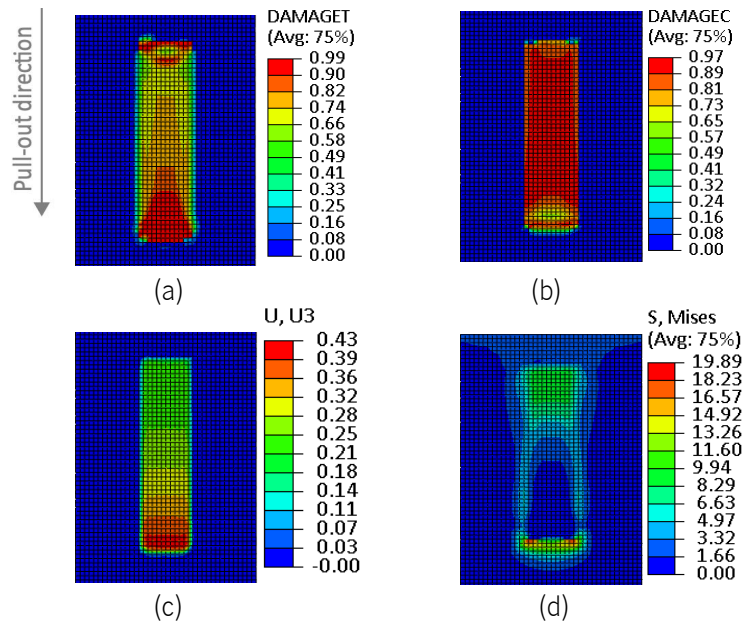


Figure IV.9. Concrete responses at failure stage: (a) damage under tension, (b) damage under compression, (c) slip in the loading direction (U3) in [mm], (d) von Mises stress at LL5 (S, Mises) in [MPa].

Figure IV.10 shows concrete damage at different loading stages and different concrete depths. The selected depths are positioned at a longitudinal section cut through the middle longitudinal axis of the CFRP laminate. The reference line (denoted as *Full damage*) is shown in **Figure IV.10a-f** to demonstrate the value at which the complete damage (failure) of concrete is predicted referring to the models described in **Section 2.3.1.1**. The compressive and tensile damage (**Figure IV.10a-f**) are dominant at the concrete-adhesive interface (**Figure IV.10a,d**) but also deep inside at approximately 3.125 mm (**Figure IV.10b,e**). However, at 6.250 mm deep the

compressive damage is minor (Figure IV.10f), while the tensile damage is only severe at the free end region (Figure IV.10c), which may have played a major role in the experimentally observed concrete thick layer removal at this region.

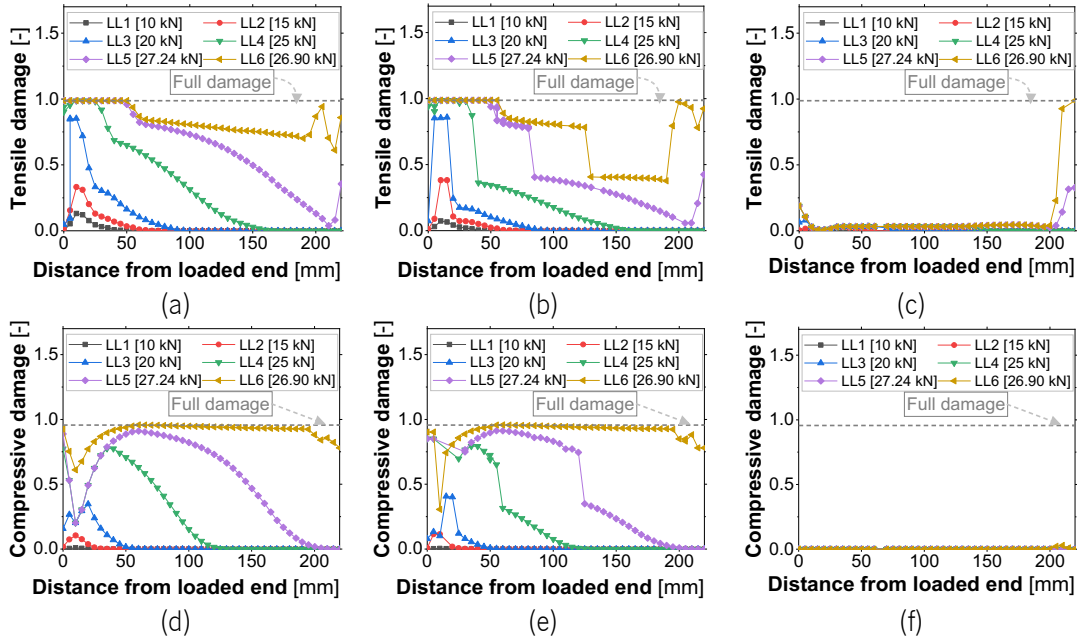


Figure IV.10. Concrete response at different loading stages and different depths measured from concrete-adhesive interface: damage under tension at (a) 0.000 mm, (b) 3.125 mm, and (c) 6.250 mm depths; damage under compression at (d) 0.000 mm, (e) 3.125 mm, and (f) 6.250 mm depths.

On the other hand, the slip at concrete-adhesive interface (Figure IV.11a) is nearly hundred and thousand times larger than that at 3.125 mm (Figure IV.11b) and 6.125 mm (Figure IV.11c) depths, respectively, indicating that concrete undergoes much larger slip at the interface level.

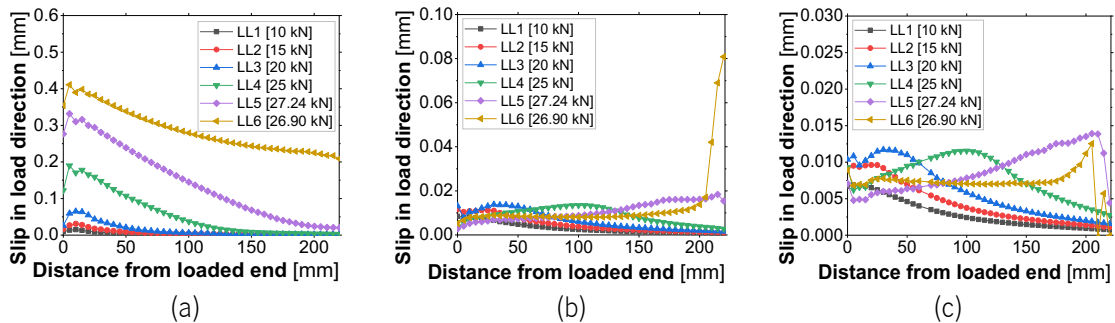


Figure IV.11. Concrete slip at different loading stages and different depths measured from concrete-adhesive interface: slip at (a) 0.000 mm, (b) 3.125 mm, and (c) 6.250 mm depths.

The smaller slips at deeper regions can be attributed to the load spreading and dissipation within the concrete, while the larger slips at the interface result from direct load transfer from the adhesive. On the other hand, similar stress profiles are observed across the depth (Figure IV.12a,b,c), with peaks at the interface (Figure IV.12a) and at 3.125 mm (Figure IV.12b) at the loaded end region but also pronounced peaks at free end in deeper regions (Figure IV.12b,c). The stress distribution in concrete at different loading levels at the interface region (Figure IV.12a) is similar to the interfacial bond stress distribution curves found from experimental work by [59], which strengthens the hypotheses that concrete behavior plays a major role in controlling the bond behavior, this is also reported in [37].

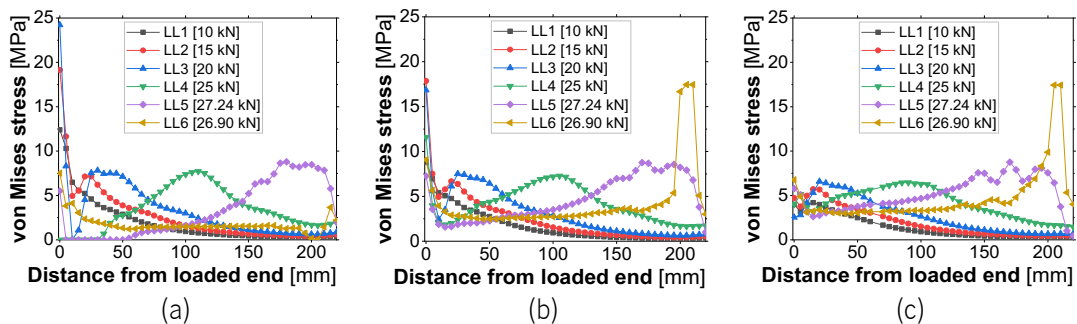


Figure IV.12. von Mises stress in concrete at different loading stages and different depths measured from concrete-adhesive interface: stress at (a) 0.000 mm, (b) 3.125 mm, and (c) 6.250 mm depths.

At initial loading levels, the loaded end regions experience higher stress, while the free end has almost zero stress (Figure IV.12a,b,c). As the load increases, stress transfers to the free end, explaining damage evolution. Significant stress transfer occurs after 20 kN, with peak stresses appearing at the loaded end, middle, and free end at 20, 25, and 27.24 kN, respectively (Figure IV.12a,b). At the maximum pull-out force (LL5), high stress is concentrated within 70 mm of the free end, while stress at the interface drops to zero at 50 mm from the loaded end. Initially, higher stress is required at the loaded end to distribute stress along the effective bond length (EBL), decreasing toward the free end. Later, sequential peak stresses emerge, explaining the plateau in pull-out force *versus* slip relations (Figure IV.2), likely due to the bond length (220 mm) exceeding the theoretical EBL (101 mm). Stress transfer is confirmed by the observed stress peaks across different regions as the load increases.

3.2.1.2. Epoxy adhesive

Epoxy adhesive responses at the failure stage are shown in **Figure IV.13**, with stress plotted at the maximum pull-out force (LL5), where stress is largest. Tensile damage is concentrated in a small region near the loaded end (**Figure IV.13a**), while compressive damage is nearly zero (**Figure IV.13b**), indicating minimal compression effects. Slip dominates at the loaded end but is also significant in other regions (**Figure IV.13c**). The stresses are also higher from the loaded end and progressively decrease towards the free end (**Figure IV.13d**), but also significant stresses at edges (peripheries) are obvious due to easier crack development [58].

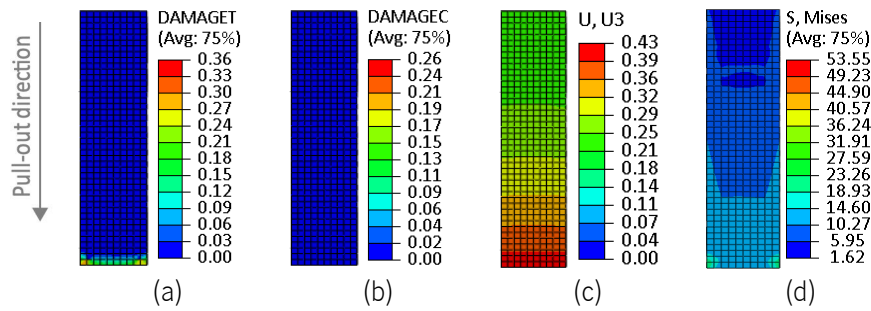


Figure IV.13. Epoxy adhesive responses at failure stage: (a) tensile damage, (b) compressive damage, (c) slip in the loading direction, (d) von Mises stress at LL5.

Figure IV.14 shows the epoxy adhesive damage across its thickness as the load level increases. Similarly to concrete, the selected locations for the adhesive responses are positioned at a longitudinal section cut through the middle longitudinal axis of the CFRP laminate.

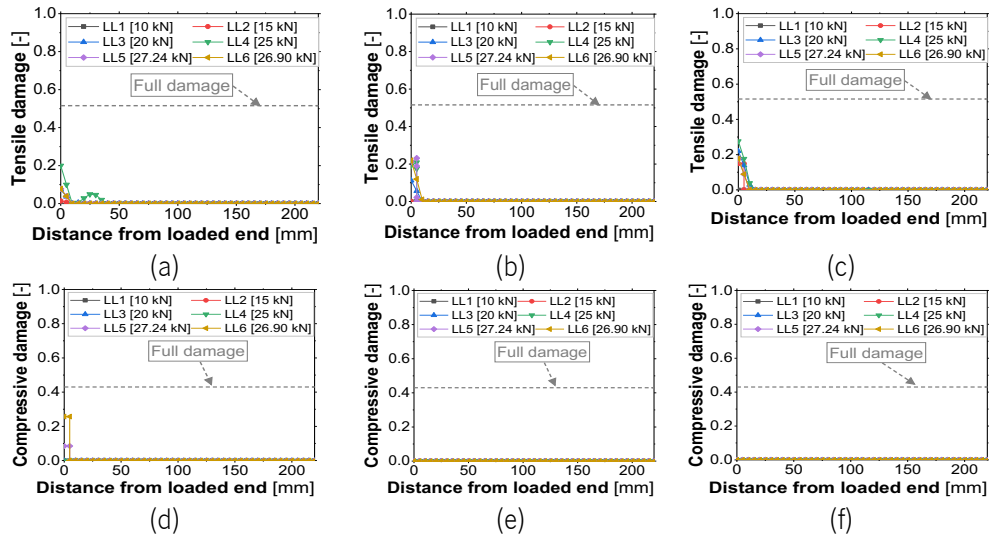


Figure IV.14. Epoxy adhesive damage at different loading stages and across its thickness: damage under tension at (a) adhesive-concrete interface, (b) middle region, and (c) adhesive-CFRP interface; damage under compression at (d) adhesive-concrete interface, (e) middle region, and (f) adhesive-CFRP interface.

The reference line (denoted as *Full damage*) is shown to indicate the ultimate damage value obtained from the used constitutive models (**Figure IV.4a**). The adhesive tensile and compressive damage across its thickness seem not to vary significantly (**Figure IV.15a-f**), the tensile damage dominates at the loaded end region with approximately 30 % of damage (**Figure IV.15a,b,c**). However, nearly 25 % of compressive damage is found only at the bottom region (i.e., at adhesive-concrete interface, see **Figure IV.15d**) and 0 % damage across the thickness towards the CFRP location (**Figure IV.15e,f**).

The slips across the adhesive thickness are plotted in **Figure IV.15**. The slips at bottom (**Figure IV.15a**) and top (**Figure IV.15c**) interfaces are slightly higher than those at the middle (**Figure IV.15b**). This can be attributed to higher CFRP and concrete stiffness that led to less deformation than that of the adhesive inner regions. The slip trends at the bottom and middle tend to present a different profile than that at the top (**Figure IV.15c**), particularly at the loaded end.

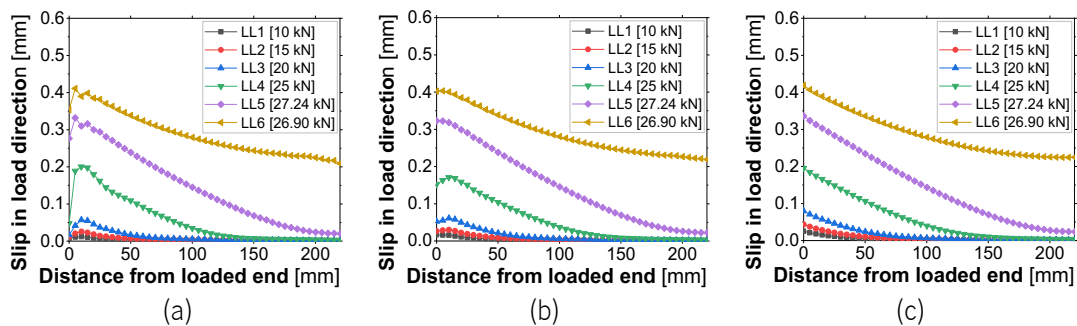


Figure IV.15. Epoxy adhesive slip at different loading stages and across its thickness: slip at (a) adhesive-concrete interface, (b) middle region, and (c) adhesive-CFRP interface.

The von Mises stress trends across the adhesive thickness are shown in **Figure IV.16**. The maximum stresses are located at the loaded end for all loading stages (with nearly 60, 35, and 30 MPa at bottom (**Figure IV.16a**), middle (**Figure IV.16b**) and top (**Figure IV.16c**) regions, respectively), and at other bonded regions they propagate towards the free end as the load level increases. These stress gradients across the adhesive thickness reflect how the adhesive transfers the load to concrete while also deforming.

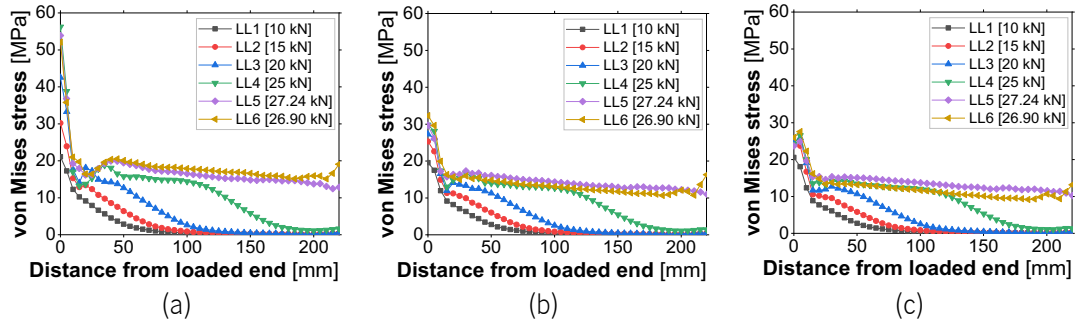


Figure IV.16. von Mises stress in epoxy adhesive at different loading stages and across its thickness: stress at (a) adhesive-concrete interface, (b) middle region, and (c) adhesive-CFRP interface.

3.2.1.3. CFRP laminate

The CFRP laminate von Mises stress and slip at failure are shown in **Figure IV.17a,b**, with higher values near the force application point, decreasing towards the bonded region. Stress and slip profiles at various loading levels are presented in **Figure IV.17c,d**. The maximum stress at the bonded region, approximately 480 MPa (**Figure IV.17c**), at the loaded end is much smaller than the ultimate tensile stress of the CFRP laminate, indicating no CFRP failure.

A sudden stress increase from 20 to 25 kN at approximately 100 mm from the loaded end (**Figure IV.17c**) confirms the reaching of effective bond length of 101 mm (**Section 2.1.3**). Hence, both numerical and the calculated effective bond lengths agree well. Similar stress profiles across the bonded region were also reported in the existing experimental works, e.g., [20,22,37,60,61]. This is also another indicator of the reliability of the developed model. At the final loading level (LL6), the predicted slip at the loaded end (0.42 mm) (**Figure IV.17d**) has excellent agreement with that recorded from the experiment (0.42 mm), thus, proving the reliability of the model.

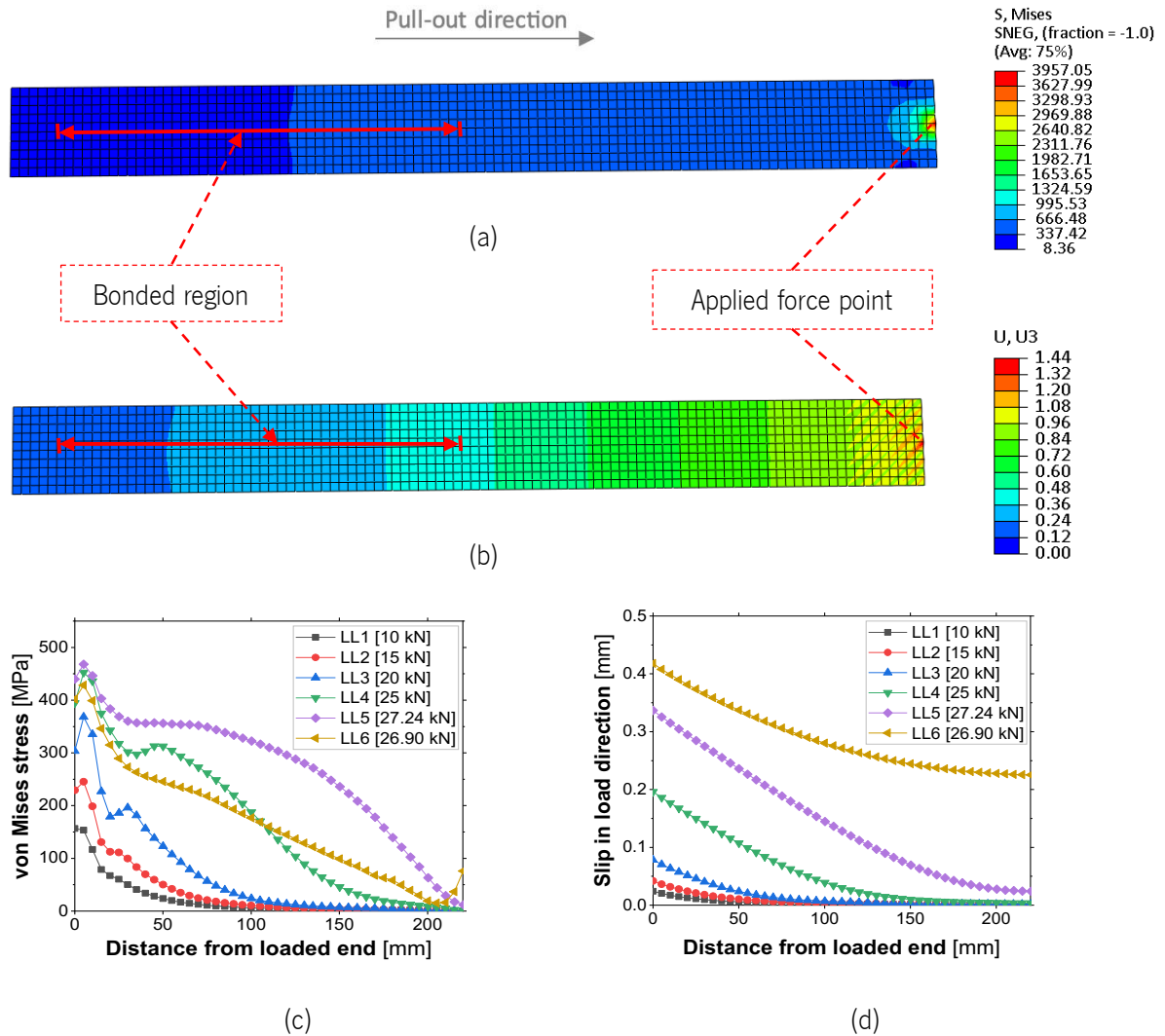


Figure IV.17. CFRP responses from FEM model: (a) von Mises stress (S, Mises) at the maximum pull-out force (LL5), (b) slip (U3) at failure (LL6), (c) von Mises stress and (d) slip profiles at different loading levels.

3.2.2. Response at joint level

The evolution of tensile and compressive damage for the CFRP-to-concrete joint as load level (LL) increases is depicted in **Figure IV.18**, where the concrete depth for each image is 22 mm and LL4* indicates the damage at the loading level of 26.87 kN, just before reaching the maximum pull-out force (27.24 kN). Both compressive and tensile damage propagate from the loaded end towards the free end as the load level (LL) increases. Once the previously described stress transfer at material (concrete) level reaches its maximum value at the free end, an additional load level causes removal of the concrete thick layer (**Figure IV.18a**, LL6).

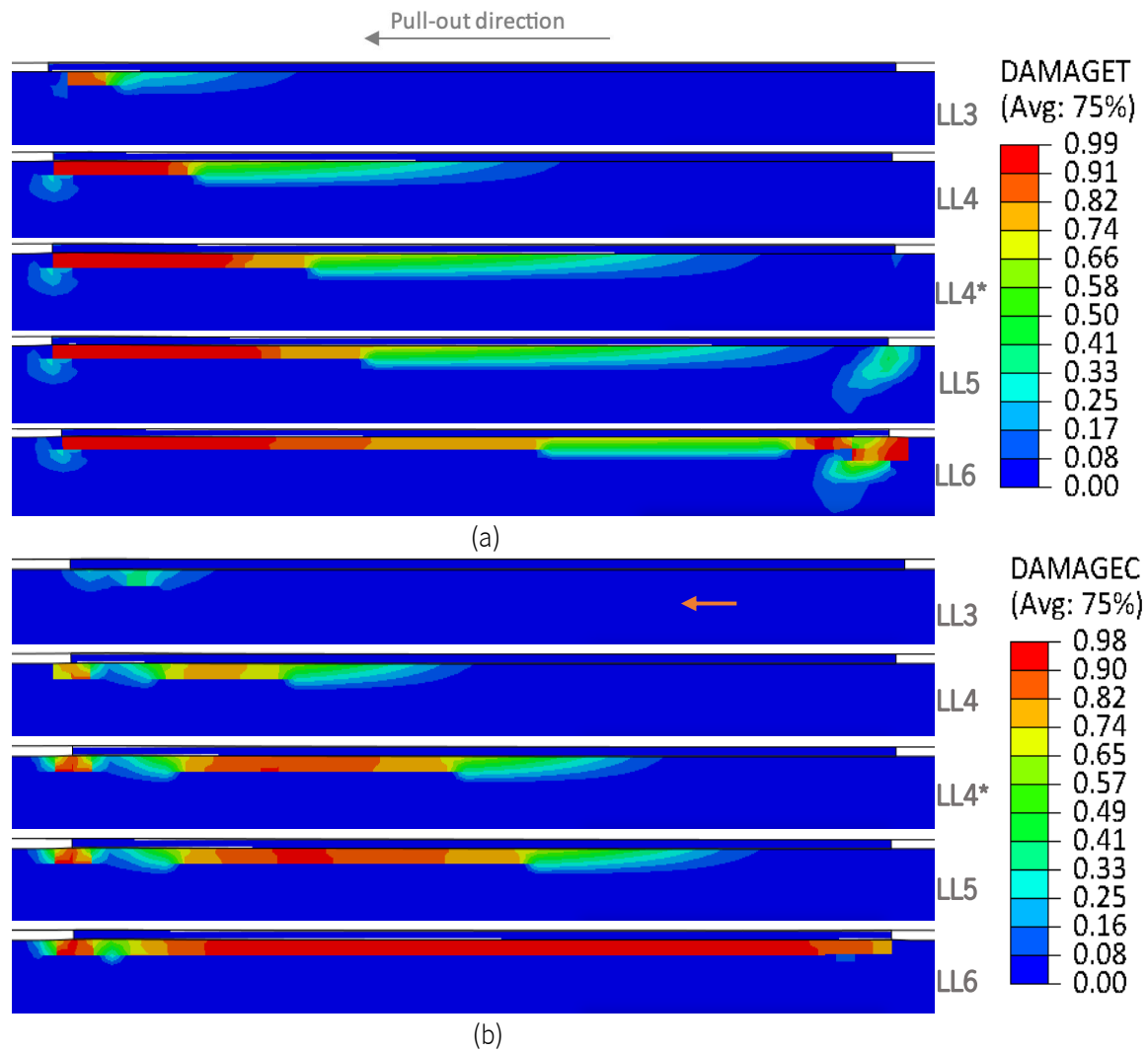


Figure IV.18. Evolution of damage at the bonded region: (a) tensile and (b) compressive damage at different loading levels as shown in **Figure IV.8**.

Evolution of slip and PE (plastic strains) is depicted in **Figure IV.19** (concrete depth for each image is 12 mm and LL4* is as previously defined). The slip is observed to propagate towards the free end as the load increases, and larger values are observed in the adhesive layer compared to concrete region (**Figure IV.19a**).

The maximum principal plastic strains (**Figure IV.19b**) show that the crack patterns tend to follow similar profiles as those of the concrete tensile damage, hence the cracks propagate from the loaded to the free end, but more cracks concentrate at the loaded end.

FINITE ELEMENT MODELING OF SHORT-TERM BEHAVIOR OF EXTERNALLY BONDED CFRP-TO-CONCRETE JOINT

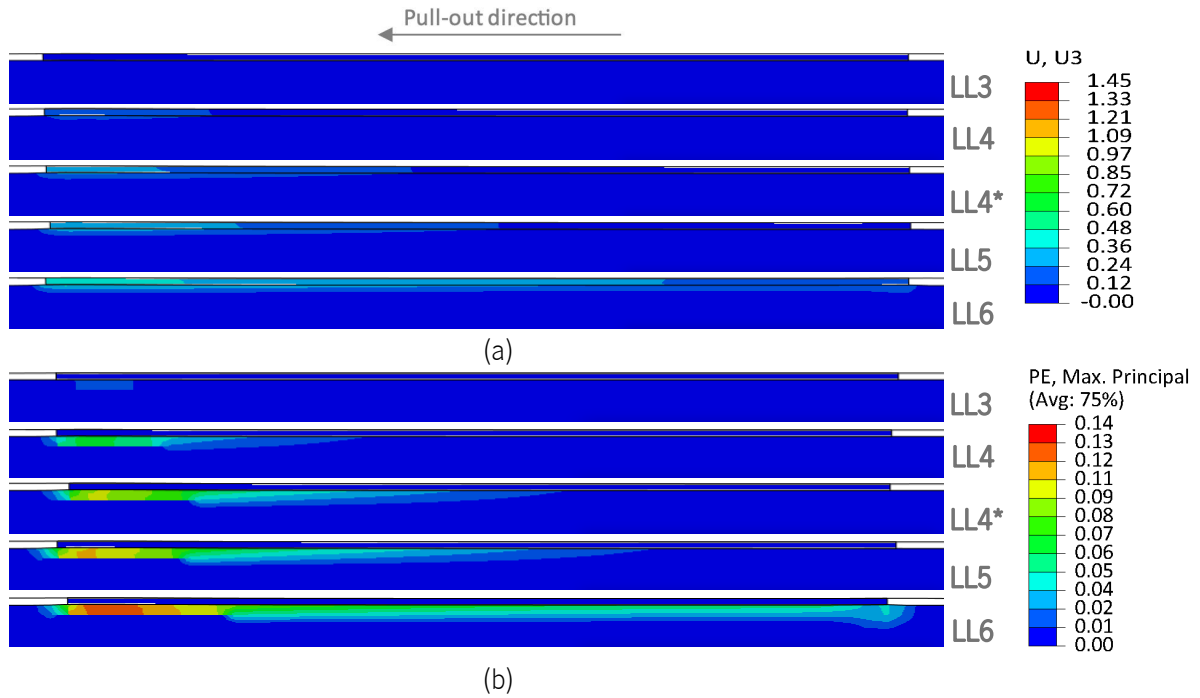


Figure IV.19. Evolution of (a) slip and (b) maximum principal plastic strain at different loading levels (Figure IV.8b).

The variation of slip at the joint is shown in Figure IV.20 for different loading levels.

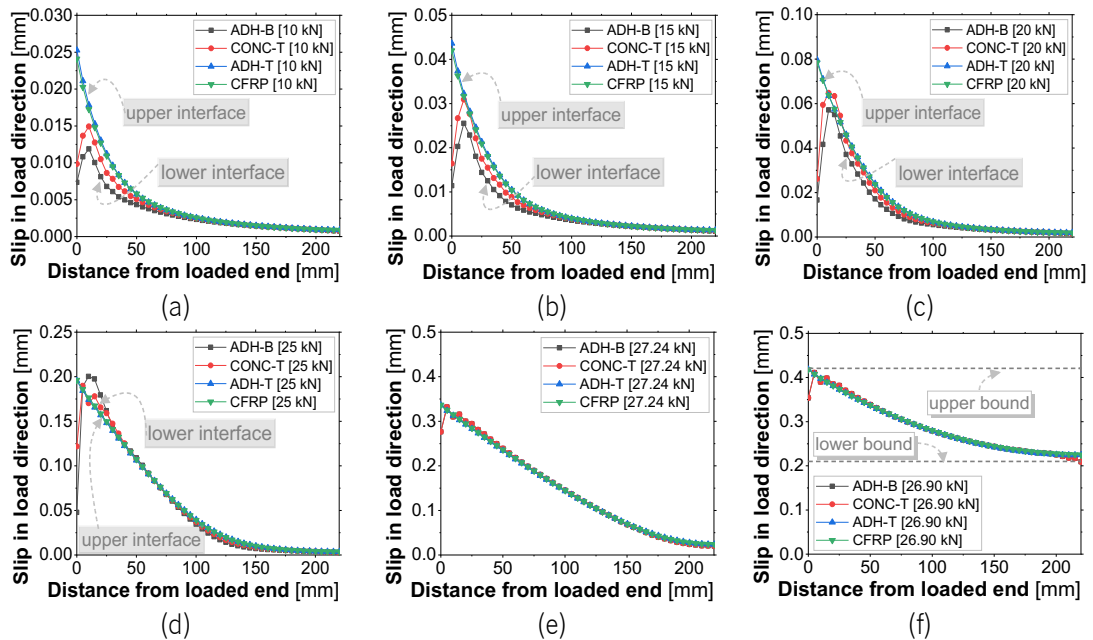


Figure IV.20. Predicted evolution of slip at the loading levels shown in Figure IV.8b: (a) LL1, (b) LL2, (c) LL3, (d) LL4, (e) LL5 and (f) LL6.

At the initial loading levels (**Figure IV.20a-d**), the slips at both interfaces are dissimilar (higher slip at upper interface than that at the lower one) particularly from the loaded end to nearly 50 mm towards the free end, then as the load level increases the interfaces tend to move uniformly (**Figure IV.20e,f**). This dissimilarity can be attributed to the higher compliance behavior of the adhesive layer at initial loadings caused by localized stresses. The observed apparent slip differences at the bottom interface (despite the assigned perfect bond) during the initial loading stages and the eventual convergence of these slips at higher loading levels can be explained by the mechanical behavior of the materials and the load transfer mechanism in the bonded system (particularly at the loaded end regions). Both concrete and adhesive presented high localized deformation at the loaded end mainly due to localized stress and boundary effects.

3.2.2.1. Failure modes

The failure modes from both experiments (**Figure IV.21a**) and FEM model (**Figure IV.21b,c**) are compared. The model predicts well the concrete cohesive failure mode previously observed during the experiment. Both tensile and compressive damage also propagate deep inside the concrete, hence agreeing well with the experimentally observed thick cohesive failure (**Figure IV.21a**).

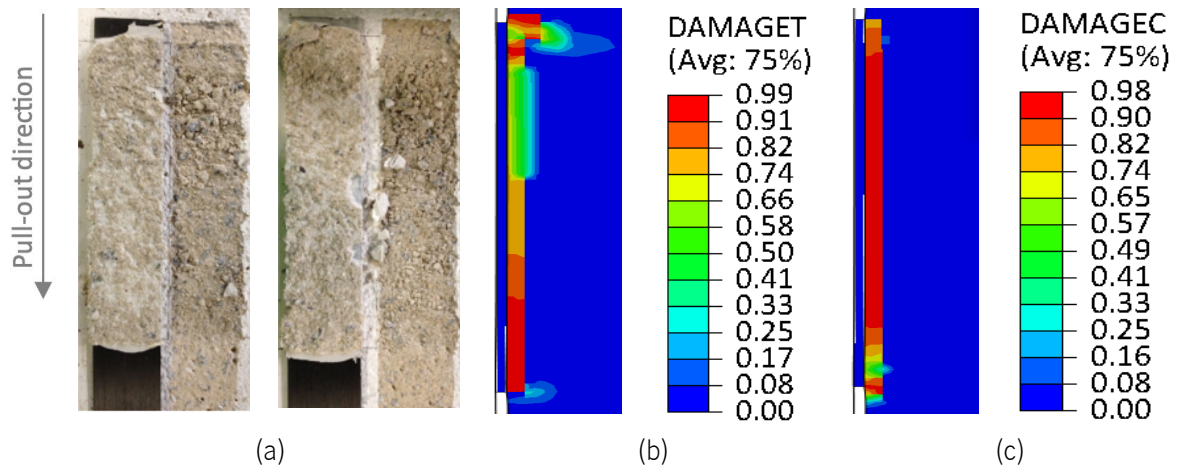


Figure IV.21. Failure modes: (a) concrete cohesive failure from experimental program; FEM model results for (b) tensile and (c) compressive damages (damages are predominant inside concrete with insignificant damages at interfaces or within adhesive).

4. CONCLUSIONS AND RECOMMENDATIONS

In this work, finite element modeling of the EBR CFRP-to-concrete joint was developed by incorporating explicitly the epoxy adhesive layer and by considering nonlinearities of both concrete and epoxy adhesive, and assuming linear elastic behavior for the CFRP laminate. Experimental data and different existing mathematical formulations were used to determine the required input parameters when using the concrete damage plasticity (CDP) model in ABAQUS. Hence, different CDP based models were implemented and validated by experimental results. The key findings are as follows:

- Modeling the EBR CFRP-to-concrete bond (joint) behavior through incorporation of the epoxy adhesive layer is found to be an appropriate approach, as it accurately predicts the pull-out force *versus* loaded end slip relationship and failure mode observed during experimental tests. The accuracy of this approach is verified by using seven different types of combinations of existing mathematical formulations to define the constitutive models of involved materials, all leading to agreements with experimental tests.
- By using the most appropriate combination, the behavior of the EBR CFRP-to-concrete joint is analyzed at both material and joint levels in terms of the most critical responses:
 - (i) ***At material level:*** concrete damage under compression and tension are much higher than those of the adhesive, reaching the limit values in almost the entire bonded regions at failure, while the adhesive damage is minor and only occur at a small region of the loaded end and does not reach the limit values. Therefore, it is concluded that the failure is due to concrete damage. In addition, both concrete compressive and tensile damage propagate well deep inside the concrete, which evidences the occurring of the concrete cohesive failure mode. The CFRP laminate presents stress profiles that agree well with those previously found in experimental works by other researchers, thereby evidencing the reliability of the developed model.
 - (ii) ***At joint level:*** The slip profiles show that up to 60 % of the maximum pull-out force, concrete-adhesive interface displaces differently from adhesive-CFRP interface; however, as the load increases (from 80 % of the maximum pull-out force onward) the slip for both interfaces become uniform. Furthermore, concrete and adhesive present different stress profiles at their interface due to different

load transfer mechanism across the bonded materials. It is shown that the epoxy adhesive plays a crucial role in the load transfer and deformation where its properties significantly influence the stress and slip gradients, hence highlighting the importance of the adhesive compliance and stiffness.

The developed model is recommended for joints that present a cohesive failure rather than the interface failure, due to the fact that a perfect bond is assumed in the latter. For joints failing at their interfaces, future works can extend the model developed in this work by incorporating the bond-slip law at the interfaces, together with the adhesive layer modelled using the CDP model.

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REFERENCES

- [1] CEB-FIP, fib90 Bulletin:Externally Applied FRP reinforcement for Concrete structures, 2019.
- [2] J. Tatar, H.R. Hamilton, Bond Durability Factor for Externally Bonded CFRP Systems in Concrete Structures, J. Compos. Constr. 20 (2016). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000587](https://doi.org/10.1061/(asce)cc.1943-5614.0000587).
- [3] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of Bond between NSM CFRP Strips and Concrete under Real-Time Field and Laboratory Accelerated Conditioning, J. Compos. Constr. 26 (2022) 1–15. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001262](https://doi.org/10.1061/(asce)cc.1943-5614.0001262).
- [4] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of bond of EBR CFRP laminates to concrete under real-time field exposure and laboratory accelerated ageing, Constr. Build. Mater. 377 (2023). <https://doi.org/10.1016/j.conbuildmat.2023.131047>.

- [5] R. Cruz, L. Correia, A. Dushimimana, S. Cabral-Fonseca, J. Sena-Cruz, Durability of epoxy adhesives and carbon fibre reinforced polymer laminates used in strengthening systems: Accelerated ageing versus natural ageing, *Materials (Basel)*. 14 (2021). <https://doi.org/10.3390/ma14061533>.
- [6] A. Dushimimana, R. Cruz, S. Cabral-fonseca, Natural Outdoor and Laboratory-Controlled Ageing of Epoxy Adhesives and Cfrp Laminates After Four Year of Exposure, 11th Int. Conf. FRP Compos. Civ. Eng. (2023) 1–10. <https://doi.org/https://doi.org/10.5281/zenodo.8109646>.
- [7] N. Azimi, K. Schollbach, D. V. Oliveira, P.B. Lourenço, Effect of acidic environment exposure on mechanical properties of TRM composites, *Constr. Build. Mater.* 438 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.137142>.
- [8] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>.
- [9] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174. <https://doi.org/10.1016/j.compstruct.2024.118174>.
- [10] J.S. Cruz, J. Barros, Modeling of bond between near-surface mounted CFRP laminate strips and concrete, *Comput. Struct.* 82 (2004) 1513–1521. <https://doi.org/10.1016/j.compstruc.2004.03.047>.
- [11] J.A.O. Barros, S.J.E. Dias, J.L.T. Lima, Efficacy of CFRP-based techniques for the flexural and shear strengthening of concrete beams, *Cem. Concr. Compos.* 29 (2007) 203–217. <https://doi.org/10.1016/j.cemconcomp.2006.09.001>.
- [12] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B Eng.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
- [13] H. Yamane, M. Oura, N. Yamazaki, T. Ishihara, K. Hasegawa, T. Ishikawa, K. Takagi, T. Hatsui, Visualizing interface-specific chemical bonds in adhesive bonding of carbon fiber structural composites using soft X-ray microscopy, *Sci. Rep.* 12 (2022) 1–8. <https://doi.org/10.1038/s41598-022-20233-4>.

- [14] F. Djouani, C. Connan, M. Delamar, M.M. Chehimi, K. Benzarti, Cement paste-epoxy adhesive interactions, *Constr. Build. Mater.* 25 (2011) 411–423. <https://doi.org/10.1016/j.conbuildmat.2010.02.035>.
- [15] J. Tatar, S. Milev, Durability of externally bonded fiber-reinforced polymer composites in concrete structures: A critical review, *Polymers (Basel)*. 13 (2021) 1–26. <https://doi.org/10.3390/polym13050765>.
- [16] J. Tatar, H.R. Hamilton, Comparison of laboratory and field environmental conditioning on FRP-concrete bond durability, *Constr. Build. Mater.* 122 (2016) 525–536. <https://doi.org/10.1016/j.conbuildmat.2016.06.074>.
- [17] S. Cabral-fonseca, J.R. Correia, J. Custódio, H.M. Silva, A.M. Machado, J. Sousa, Durability of FRP - concrete bonded joints in structural rehabilitation : A review, *Int. J. Adhes. Adhes.* 83 (2018) 153–167. <https://doi.org/10.1016/j.ijadhadh.2018.02.014>.
- [18] A. Dushimimana, L. Correia, R. Cruz, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-Concrete Bond in EBR and NSM Systems under Natural Ageing for a Period of Three Years, in: *Proc. 20th Eur. Conf. Compos. Mater., CC BY-NC 4.0, Lausanne, Switzerland, 2022: pp. 1–8*. https://doi.org/https://doi.org/10.5075/epfl-298799_978-2-9701614-0-0.
- [19] F. Qu, H. Wei, H. Lu, D. Feng, Q. Meng, S. Zhao, Experimental Study on Bond Behavior between CFRP and Concrete with a Convex-Circular Arc Interface, *Buildings*. 13 (2023). <https://doi.org/10.3390/buildings13123077>.
- [20] S. Chataigner, J.F. Caron, K. Benzarti, M. Quiertant, C. Aubagnac, Use of a single lap shear test to characterize composite-to-concrete or composite-to-steel bonded interfaces, *Constr. Build. Mater.* 25 (2011) 468–478. <https://doi.org/10.1016/j.conbuildmat.2010.08.009>.
- [21] H. Ko, S. Matthys, A. Palmieri, Y. Sato, Development of a simplified bond stress-slip model for bonded FRP-concrete interfaces, *Constr. Build. Mater.* 68 (2014) 142–157. <https://doi.org/10.1016/j.conbuildmat.2014.06.037>.
- [22] C. Mazzotti, M. Savoia, B. Ferracuti, An experimental study on delamination of FRP plates bonded to concrete, *Constr. Build. Mater.* 22 (2008) 1409–1421.

- <https://doi.org/10.1016/j.conbuildmat.2007.04.009>.
- [23] F.M. Mukhtar, M.E. Shehadah, Experimental verification of 2- and 3-D numerical models for bond-slip behavior of CFRP-concrete, *Constr. Build. Mater.* 287 (2021) 122814. <https://doi.org/10.1016/j.conbuildmat.2021.122814>.
- [24] F. Bencardino, A. Condello, L. Ombres, Numerical and analytical modeling of concrete beams with steel, FRP and hybrid FRP-steel reinforcements, *Compos. Struct.* 140 (2016) 53–65. <https://doi.org/10.1016/j.compstruct.2015.12.045>.
- [25] Y.T. Obaidat, S. Heyden, O. Dahlblom, The effect of CFRP and CFRP/concrete interface models when modelling retrofitted RC beams with FEM, *Compos. Struct.* 92 (2010) 1391–1398. <https://doi.org/10.1016/j.compstruct.2009.11.008>.
- [26] J.W. Shi, W.H. Cao, Z.S. Wu, Effect of adhesive properties on the bond behaviour of externally bonded FRP-to-concrete joints, *Compos. Part B Eng.* 177 (2019). <https://doi.org/10.1016/j.compositesb.2019.107365>.
- [27] C.A. Coronado, M.M. Lopez, Numerical Modeling of Concrete-FRP Debonding Using a Crack Band Approach, *J. Compos. Constr.* 14 (2010) 11–21. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000044](https://doi.org/10.1061/(asce)cc.1943-5614.0000044).
- [28] F.C. An, W. Liu, B. Fu, A predictive 2D finite element model for modelling FRP-to-concrete bond behavior, *Compos. Struct.* 226 (2019). <https://doi.org/10.1016/j.compstruct.2019.111189>.
- [29] M.R.T. Arruda, J.P. Firmo, J.R. Correia, C. Tiago, Numerical modelling of the bond between concrete and CFRP laminates at elevated temperatures, *Eng. Struct.* 110 (2016) 233–243. <https://doi.org/10.1016/j.engstruct.2015.11.036>.
- [30] J.P. Firmo, M.G. Roquette, J.R. Correia, A.S. Azevedo, Influence of elevated temperatures on epoxy adhesive used in CFRP strengthening systems for civil engineering applications, *Int. J. Adhes. Adhes.* 93 (2019) 102333. <https://doi.org/10.1016/j.ijadhadh.2019.01.027>.
- [31] K. Nakaba, T. Kanakubo, T. Furuta, H. Yoshizawa, Bond Behavior between Fiber-Reinforced Polymer Laminates and Concrete, *Struct. J.* 98 (2001). <https://doi.org/10.14359/10224>.

- [32] J. Dai, T. Ueda, Y. Sato, Development of the Nonlinear Bond Stress–Slip Model of Fiber Reinforced Plastics Sheet–Concrete Interfaces with a Simple Method, *J. Compos. Constr.* 9 (2005) 52–62. [https://doi.org/10.1061/\(asce\)1090-0268\(2005\)9:1\(52\)](https://doi.org/10.1061/(asce)1090-0268(2005)9:1(52)).
- [33] Z. Wu, S.M. Islam, H. Said, A three-parameter bond strength model for FRP-concrete interface, *J. Reinf. Plast. Compos.* 28 (2009) 2309–2323. <https://doi.org/10.1177/0731684408091961>.
- [34] Y.W. Zhou, Y.F. Wu, Y. Yun, Analytical modeling of the bond-slip relationship at FRP-concrete interfaces for adhesively-bonded joints, *Compos. Part B Eng.* 41 (2010) 423–433. <https://doi.org/10.1016/j.compositesb.2010.06.004>.
- [35] Y.-F. Wu, C. Jiang, Quantification of Bond-Slip Relationship for Externally Bonded FRP-to-Concrete Joints, *J. Compos. Constr.* 17 (2013) 673–686. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000375](https://doi.org/10.1061/(asce)cc.1943-5614.0000375).
- [36] K. Park, K. Ha, H. Choi, C. Lee, Prediction of interfacial fracture between concrete and fiber reinforced polymer (FRP) by using cohesive zone modeling, *Cem. Concr. Compos.* 63 (2015) 122–131. <https://doi.org/10.1016/j.cemconcomp.2015.07.008>.
- [37] Y. Tao, J.F. Chen, Concrete Damage Plasticity Model for Modeling FRP-to-Concrete Bond Behavior, *J. Compos. Constr.* 19 (2015) 1–13. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000482](https://doi.org/10.1061/(asce)cc.1943-5614.0000482).
- [38] A.S. Genikomsou, M.A. Polak, Finite element analysis of punching shear of concrete slabs using damaged plasticity model in ABAQUS, *Eng. Struct.* 98 (2015) 38–48. <https://doi.org/10.1016/j.engstruct.2015.04.016>.
- [39] S.H. Lee, A. Abolmaali, K.J. Shin, H. Du Lee, ABAQUS modeling for post-tensioned reinforced concrete beams, *J. Build. Eng.* 30 (2020) 101273. <https://doi.org/10.1016/j.jobbe.2020.101273>.
- [40] S&P, S&P 220 Resin epoxy adhesive. Technical Data Sheet, Seewen, Switzerland, 2015.
- [41] A. Dushimimana, A.P. Vassilopoulos, J. Sena-Cruz, J.M. Pereira, L. Correia, S. Cabral-Fonseca, R. Cruz, Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-controlled environments, *Constr. Build. Mater.* 438 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.137201>.
- [42] S&P Clever Reinforcement Company AG, S&P C-Laminate Carbon fibre polymer plates for structural

- reinforcement, Seewen, Switzerland, 2020. https://www.sp-reinforcement.eu/sites/default/files/field_sup_dcmnt_file/2024/01/24/131247/r_c-laminates_pub_tds_prod_c-laminates_v4.102020_eu_en.pdf.
- [43] CNR-DT 200 R1, Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures; CNR-DT 200 R1/2013;, Rome, Italy, 2013.
- [44] A. Dushimimana, J. Sena-Cruz, A.P. Vassilopoulos, L. Correia, J.M. Pereira, Modelling of short- and long-term bond behavior in concrete structures strengthened with externally bonded CFRP composites, in: C. BINETRUY, F. JACQUEMIN (Eds.), Proc. 21st Eur. Conf. Compos. Mater. Vol. 3 - Mater. Struct. Behav. – Simul. Test., The European Society for Composite Materials (ESCM) and the Ecole Centrale de Nantes, Nantes, France, 2024: p. 683. <https://doi.org/10.60691/yj56-np80>.
- [45] Simulia, ABAQUS computer software and Online Documentation, v6.12;, 2012.
- [46] L. Saenz, Discussion of Equation for the stress-strain curve of concrete by Desayi, P. and Krishnan, S., ACI J. 61 (1964) 1229–1235.
- [47] D.J. Carreira, Kuang-Han Chu, Stress-Strain Relationship for Reinforced Concrete in Compression, ACI Struct. J. (1985) 797–804.
- [48] T. Wang, T.T.C. Hsu, Nonlinear finite element analysis of concrete structures using new constitutive models, Comput. Struct. 79 (2001) 2781–2791. [https://doi.org/10.1016/S0045-7949\(01\)00157-2](https://doi.org/10.1016/S0045-7949(01)00157-2).
- [49] A. Belarbi, T.T.C. Hsu, Constitutive Laws of Concrete in Tension and Reinforcing Bars Stiffened By Concrete, Struct. J. 91 (1994) 465–474.
- [50] D.A. Hordijk, Local approach to fatigue of concrete, Diss. Delft Univ. Technol. (1991) 41. <http://resolver.tudelft.nl/uuid:fa87147b-8201-47ed-83d7-b812b09c5fbb>.
- [51] A. Kaushik, G. Patnaik, A. Rajput, G. Prakash, Nonlinear Behaviour of Concrete Under Low-Velocity Impact by Using a Damaged Plasticity Model, Iran. J. Sci. Technol. - Trans. Civ. Eng. 46 (2022) 3655–3677. <https://doi.org/10.1007/s40996-021-00808-3>.
- [52] CEB-FIP, Model Code 90” CEB Bulletin No. 213/214, Lausanne, Switzerland, 1993.

- [53] X. Poulain, A.A. Benzerga, R.K. Goldberg, Finite-strain elasto-viscoplastic behavior of an epoxy resin: Experiments and modeling in the glassy regime, *Int. J. Plast.* 62 (2014) 138–161. <https://doi.org/10.1016/j.ijplas.2014.07.002>.
- [54] Y. Miao, W. Du, J. Yin, Y. Zeng, C. Wang, Characterizing multi mechanical behaviors for epoxy-like materials under wide strain rate range, *Polym. Test.* 116 (2022). <https://doi.org/10.1016/j.polymertesting.2022.107804>.
- [55] Z. Wang, P. Li, Visco-elasto-plastic constitutive model of adhesives under uniaxial compression in a range of strain rates, *J. Appl. Polym. Sci.* 137 (2020) 1–8. <https://doi.org/10.1002/app.48962>.
- [56] EN 1992-1-1 (2004), Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings, 2004.
- [57] Z. Jia, D. Hui, G. Yuan, J. Lair, K. tak Lau, F. Xu, Mechanical properties of an epoxy-based adhesive under high strain rate loadings at low temperature environment, *Compos. Part B Eng.* 105 (2016) 132–137. <https://doi.org/10.1016/j.compositesb.2016.08.034>.
- [58] Y. Liu, C. Zhou, L. Chen, Y. Wang, Experimental study on bond behavior between CFRP and concrete with longitudinal grooves, *Structures.* 65 (2024). <https://doi.org/10.1016/j.istruc.2024.106654>.
- [59] Z.X. Li, X. Zhang, Y. Shi, Experimental study on the dynamic bond behavior between CFRP and concrete under different slip rates, *Eng. Struct.* 216 (2020) 110788. <https://doi.org/10.1016/j.engstruct.2020.110788>.
- [60] Y.T. Obaidat, S. Heyden, O. Dahlblom, Evaluation of Parameters of Bond Action between FRP and Concrete, *J. Compos. Constr.* 17 (2013) 626–635. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000378](https://doi.org/10.1061/(asce)cc.1943-5614.0000378).
- [61] X.Z. Lu, J.G. Teng, L.P. Ye, J.J. Jiang, Bond-slip models for FRP sheets/plates bonded to concrete, *Eng. Struct.* 27 (2005) 920–937. <https://doi.org/10.1016/j.engstruct.2005.01.014>.

PAPER V

FINITE ELEMENT MODELING OF LONG-TERM BEHAVIOR OF EXTERNALLY BONDED CFRP-TO-CONCRETE JOINT UNDER WATER

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Modeling of EBR CFRP-to-concrete bond under long-term water immersion. To be submitted to a journal.

ABSTRACT: This work introduces predictive models for durability of bond between concrete and carbon fiber reinforced polymers (CFRP) applied using externally bonded reinforcement (EBR) technique under prolonged water immersion controlled at 20 °C for up to 4 years, as this topic lacks sufficient literature. The model

development is based on data from long-term experimental works performed by the authors' group. Using this data, finite element models are developed to predict the long-term behavior of EBR CFRP-to-concrete bond. Damage plasticity model is adopted to model concrete and epoxy adhesive nonlinearities, while linear elastic behavior is adopted for the CFRP laminate. Responses such as CFRP strains, pull-out force, slip, and damage propagation are examined at different loading levels. The predicted force-slip relations and failure modes agree well with experimental data; the models also help examine responses not captured during experimental tests.

KEYWORDS: Durability, Epoxy adhesive, Modeling of bond, EBR, concrete, CFRP laminate

1. INTRODUCTION

Strengthening reinforced concrete (RC) structures using carbon fiber reinforced polymer (CFRP) composites has been widely adopted [1], owing to the promising properties of these composites, including high-stiffness and high strength-to-weight ratios and corrosion and fatigue resistance [1,2], among others. Two main application techniques are usually implemented: externally bonded reinforcement (EBR) and near-surface mounted (NSM) [1]. For the case of EBR technique, the CFRP laminate is bonded to the concrete surface, while for the case of NSM the CFRP laminate is inserted in a groove pre-cut into concrete cover [3,4]. In both techniques, an epoxy adhesive is needed to bond the CFRP to concrete. The adoption of epoxy adhesive is largely due to its capacity to create strong covalent bonds when applied to CFRP composites [5]. Furthermore, when bonding to concrete, the adhesive not only forms chemical bonds, primarily through hydrogen bonding [6], but also establishes a mechanical interlock. This interlock occurs as the epoxy flows into the open pores, crevices, and surface irregularities of roughened concrete [1]. Several commercially available epoxy adhesives have been satisfactorily used, e.g., [7–10].

Despite the successful application of the above techniques, questions regarding the long-term performance of the bond between the CFRP laminate and the concrete, and the variation of the properties of the bond constituent materials (concrete, adhesive, CFRP laminate) when exposed to different environmental conditions have not been fully addressed yet. Evaluating the durability performance of the EBR CFRP-to-concrete system is complex and requires durability analysis at both the constituent materials and the system itself, as the former may present different behaviors than the latter [1,4]. Regarding the EBR CFRP-to-concrete system under water, at material level, if epoxy adhesive is not completely cured, unreacted polar sites can attract water molecules into the cross-linked epoxy network, resulting in an onset of plasticization [1]. Even when fully cured, the

adhesive plasticization can still take place after being immersed in water; however, this is a reversible mechanism [1,10]. The plasticization can then lead to reduction of epoxy adhesive elastic modulus and tensile strength. The CFRP is generally inert when exposed to moisture and all environments typically experienced by civil structure, and shows better resistance to chemical attacks compared to other FRP materials [1]. At the bond level, the plasticization of the epoxy adhesive reduces its stiffness, which in turn weakens the mechanical interlock. This, combined with the disruption of hydrogen bonds due to interactions with water molecules, leads to a loss of adhesion and, consequently, a reduction in bond strength after prolonged exposure to moisture [1,10]. The accumulations of water molecules at the EBR CFRP-to-concrete bond interface tend to seep into the gap between the adhesive and concrete, thereby causing debonding that starts from the local cracking at the loaded end and advances to the free end until full debonding occurs [10].

Limited experimental data about the durability of the EBR CFRP-to-concrete bond exist, e.g., [7,11,12]. Particularly, the EBR CFRP-to-concrete bond performance when exposed to water immersion lacks significant literature. Presence of water and moisture on the concrete surface can significantly affect the adhesion in the EBR CFRP-to-concrete systems [1]. The effect of water immersion of the EBR CFRP-to-concrete bond was investigated in [7], and it was observed that the bond strength was not significantly affected, even though it is well known that the adhesive layer can be significantly deteriorated under water [1,4]. Hence, further works are needed to better understand the effects of water exposure on the bond performance.

Furthermore, several numerical studies addressing short-term bond performance for the EBR technique can be found, e.g., [13–18], where concrete is generally modelled considering its nonlinearities, while the CFRP is assumed to present a linear elastic behavior up to failure. Furthermore, a cohesive or connector element is generally used to model the interfaces between concrete and CFRP laminate. For example, numerical models for both 2D and 3D were developed incorporating the above material constitutive models and compared in [14]. It was shown that the 3D model outperformed the 2D model by accurately capturing the failure load, which the 2D model failed to do. The material constitutive models and cohesive model were also adopted in [16] to model the effect of the CFRP-to-concrete interfaces in RC beams with strengthening systems, leading to excellent prediction of the short-term bond behavior.

However, existing literature presents very few numerical models addressing durability such as elevated temperatures, e.g., [19,20] which are additionally based on very short-term exposure; hence a limited amount

of data is available in terms of longer exposures. Furthermore, there is a little or no numerical studies addressing the EBR CFRP-to-concrete bond performance under prolonged water immersion, hence a need for further relevant research. Trying to address this gap, an attempt to develop a numerical model considering the durability of the EBR CFRP-to-concrete bond for up to 4 years of tap water exposure is made. Constitutive models for each bond constituent material (concrete, epoxy adhesive, CFRP laminate) are first developed considering the material properties obtained from yearly experimental tests and the existing mathematical formulations [21–23]. These models are then used in the finite element model to predict different responses such as the evolution of CFRP strains, slip, pull-out force, and damage over time.

2. RESEARCH METHODS

This section addresses the experimental campaign on the durability of EBR CFRP-to-concrete bond under water immersion for up to 4 years and the results obtained from the pull-out tests performed on the studied specimens. Furthermore, the methods used for numerical modeling of the EBR CFRP-to-concrete bond are described in detail.

2.1. Experimental investigations

2.1.1. Concrete

The properties for all concrete specimens are summarized **Table V.1**.

Table V.1. Concrete properties for the EBR CFRP-to-concrete bond specimens

Property	Value
Specimen dimension	D×H: 150×300 (concrete cylinder) L×W×H: 200×200×400 (concrete prism)
Maximum aggregate size [mm]	12.5
Cement type	CEM II/A-L 42.5R
Slump [mm]	160-210 (slump class S4)
Concrete class	30/37 MPa (C30/37)
Water-to-cement ratio	0.4
f_c [MPa], E_c [GPa]	41.5, 29.1 (after 28 days)
Exposure class	XC4(P)

Notes: f_c : compressive strength (average value), E_c : elastic modulus (average value),

D×H: diameter × height (units in mm³), L×W×H: Length × Wide × Height (units in mm³).

A total of 12 concrete cylinders were immersed in tap water at 20 °C and 3 of them were tested each year under compression to determine the compressive strength (f_{cc}) and the elastic modulus (E_c). The f_{cc} was

determined following the EN 12390-3:2011 [24], and E_c following the EN 12390-13:2013 [25]. Furthermore, 8 concrete prisms were used to assess concrete pull-off strength (f_a), following EN1542:1999 [26].

2.1.2. Epoxy adhesive and CFRP laminate

The utilized epoxy adhesive and CFRP laminate were produced by S&P® Clever Reinforcement Ibérica Lda Company (Seixal, Portugal) [27,28] with properties shown in **Table V.2**. Initially, 5 dog-bone shaped epoxy adhesive and 6 CFRP laminate specimens were tested under tension (before exposure) following ISO 527-2:2012 [29,30] to obtain the initial tensile strengths and elastic moduli from stress-strain curves, which served as reference values for comparison purposes. Furthermore, 20 dog-bone-shaped epoxy adhesive and 24 CFRP laminate specimens were immersed in tap water at a controlled temperature of 20 °C and tested later under tension (5 epoxy adhesive and 6 CFRP laminate specimens each year) following the same procedures as the initial tests to determine the variation of strength and elastic modulus for up to 4 years.

Table V.2. Properties of the epoxy adhesive and CFRP laminate as per supplier [40].

Epoxy Adhesive	
Type of adhesive	Cold-curing S&P Resin 220
Flexural elastic modulus [GPa]	>7.1
Tensile strength [MPa]	19.9 (after 7d curing at 20 °C)
Density, at 23°C [g/cm ³]	1.7 - 1.8
Compressive strength [MPa]	>70
Shear strength [MPa]	>26
BS by pull-off on concrete [MPa]	>3 (after 3d curing at 20 °C)
Type of adhesive	Cold-curing S&P Resin 220
CFRP laminate	
Type and trademark	S&P clever (CFK 150/2000)
Prefabricated by	Pultrusion
Fiber orientation	Unidirectional
Fiber content [%]	68
Fiber matrix	Vinyl ester resin (with $T_g \approx 85^\circ\text{C}$)
External surface	Black, smooth
Elastic modulus [GPa]	>170
Tensile strength [MPa]	>2000

Note: T_g : glass transition temperature, BS: bond strength, N/A: not available, d: days.

2.1.3. Concrete elements strengthened with EBR CFRP laminates

Concrete elements in the form of prisms, each strengthened with 2 CFRP laminates (the laminate being bonded on each of the concrete prism faces using epoxy adhesive), were used to assess the CFRP-to-concrete bond performance. These specimens were tested sequentially (two specimens tested each year) after being fully immersed in tap water at a temperature of 20 °C, for up to 4 years. The properties of the CFRP-to-concrete joint

are summarized in **Table V.3**. The test setup for the conducted single lap shear tests, the geometry and dimensioning of the specimens are all shown in **Figure V.1a**. Furthermore, a photograph of the full test is also provided in **Figure V.1b**. Further details can be found in [15].

Table V.3. Properties of the EBR CFRP-to-concrete joint.

CFRP cross-section	50×1.2 [mm ²]
Concrete prism	400×200×200 [mm ³]
Bond length	220 [mm]
Effective bond length	101 mm
Epoxy adhesive thickness	1.5 - 2.0 mm
Type of test	Single-lap shear test

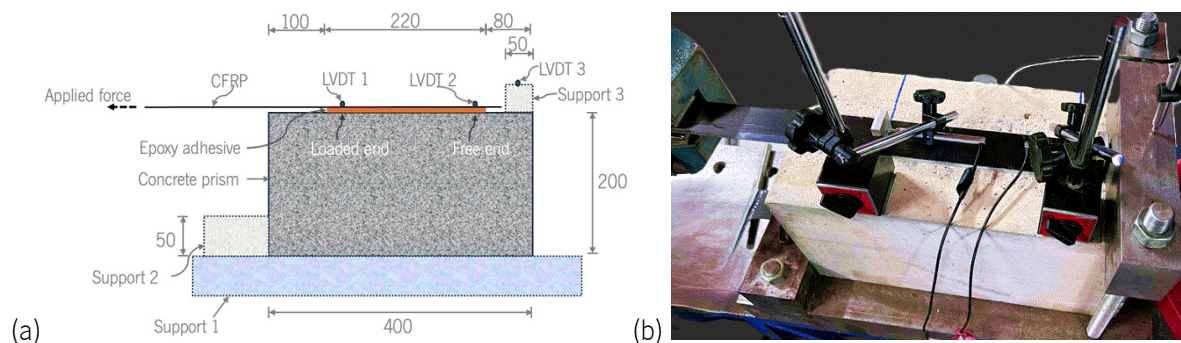


Figure V.1. Single lap shear test: (a) specimen's geometry and test set-up (adapted from previous work by the authors [15]), (b) photograph of the full test configuration. Note: All units in [mm].

2.2. Experimental results

2.2.1. Bond constituent materials

The estimated maximum values from the various experiments performed on concrete, epoxy adhesive, CFRP laminate and the EBR CFRP-to-concrete joint are summarized in **Figure V.2** and **Table V.4** for all water exposure times. The reference values (for specimens examined before exposure, i.e., at year 0) are also shown for better examination of the experimental result trends, and to assist the comprehension of the developed model results.

2.2.1.1. Concrete, epoxy adhesive, and CFRP laminate

Marginal variations in both concrete compressive strength and elastic modulus over time can be observed in **Figure V.2a,b**; however, a significant reduction of the pull-off strength from year 0 to year 1 can be noted, followed by minor fluctuations in later years (**Figure V.2c**).

FINITE ELEMENT MODELING OF LONG-TERM BEHAVIOR OF EXTERNALLY BONDED CFRP-TO-CONCRETE JOINT UNDER WATER

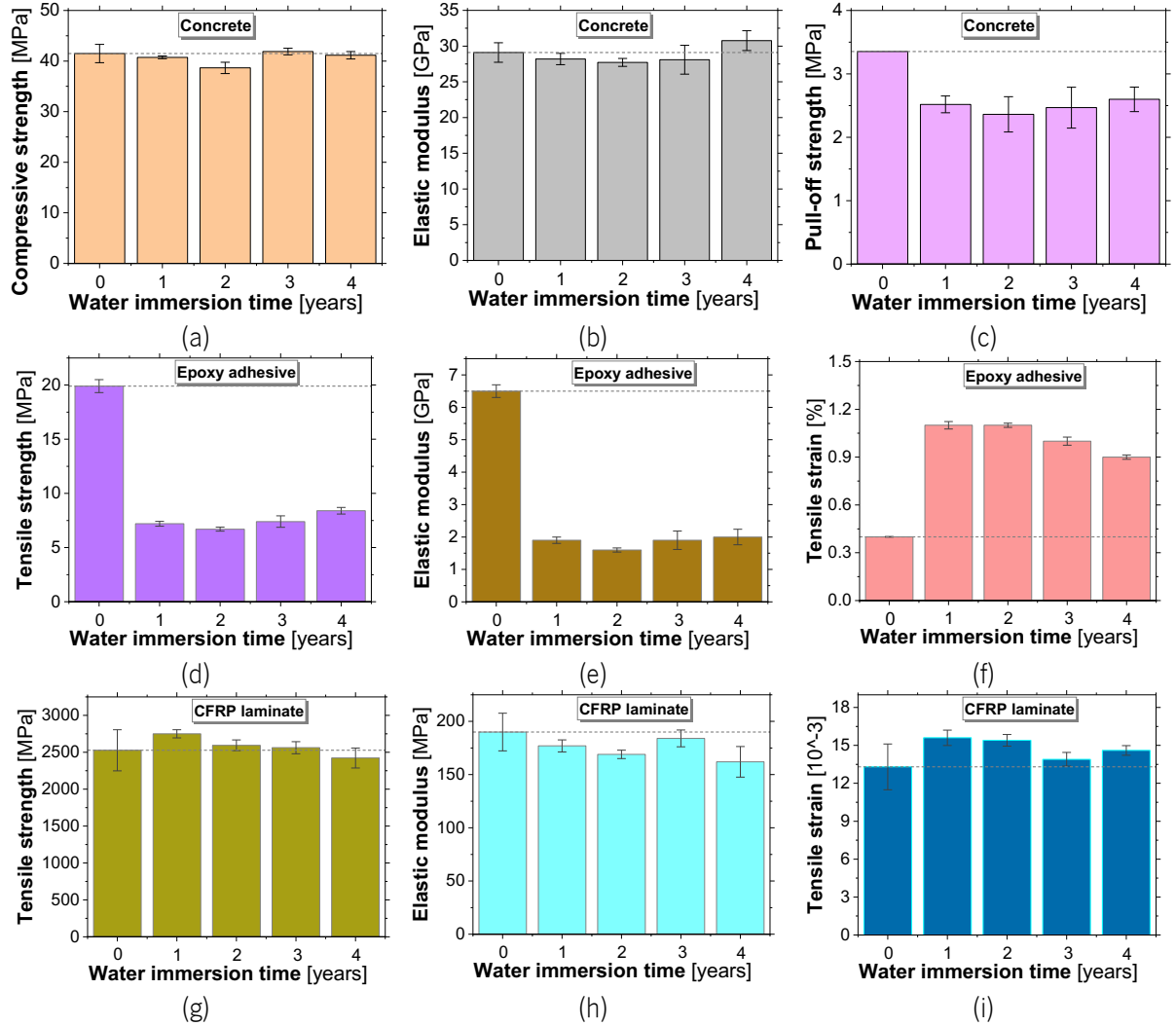


Figure V.2. Materials responses during 4 years of water immersion: concrete (a) compressive strength, (b) elastic modulus, and (c) pull-off strength; epoxy adhesive (d) tensile strength, (e) elastic modulus, and (f) tensile strain; and CFRP laminate (g) tensile strength, (h) elastic modulus, and (i) tensile strain.

Concrete did not show significant property degradation except the pull-off strength. Further details on the above concrete trends were provided in a previous work by the authors [7].

The epoxy adhesive tensile strength — f_t (Figure V.2d) and elastic modulus — E_a (Figure V.2e) were deteriorated by water diffusion from year 0 to year 1 with approximately 66 and 75 % reductions, respectively. These reductions can mainly be attributed to physical aging effects (plasticization and swelling mechanisms)[31,32]. From the above reductions, it can be noted that the E_a degraded faster than f_t and this is in line with the findings in [33] (with 30 and 24 % reductions in the E_a and f_t after exposure to demineralized water at 40 °C) and in [31]

(with Arrhenius predictions of 64 and 62 % reductions in the E_s and f_s after water immersion at 20 °C). Hence, microcracking and swelling from the physical aging effects may be thought to disrupt the ability of a material to resist deformation (elastic modulus) more than its ability to withstand the applied load up to failure (tensile strength). The tensile strength and elastic modulus responses showed minor variations in the later years, particularly with a tendency to regain their properties as time increases, which can be attributed to chemical aging effects leading to molecular densification [34]. On the other hand, the tensile strain (**Figure V.2f**) showed a remarkable increase of approximately 275 %, followed by minor decreases in the later years, which agrees with the findings in [35].

The CFRP laminate showed marginal tensile strength variations (**Figure V.2g**) but with a tendency for its elastic modulus (**Figure V.2h**) to decrease (approximately 15 % reduction after 4 years). Furthermore, insignificant changes in the tensile strain were observed (**Figure V.2i**). Further details on the CFRP trends were provided in a previous study by the authors [36].

Table V.4. Average values for materials and EBR CFRP-to-concrete joint for different water exposure times.

	Year 0	Year 1	Year 2	Year 3	Year 4
Concrete					
Compressive strength [MPa]	41.5 (4.4)	40.7 (0.7)	38.7 (2.9)	41.9 (1.5)	41.1 (1.8)
Pull-off strength [MPa]	3.4 (13.3)	2.5 (5.3)	2.3 (11.8)	2.5 (13.1)	2.6 (7.4)
Elastic modulus [GPa]	29.1 (5.0)	28.2 (2.8)	27.7 (2.0)	28.1 (7.2)	30.8 (4.5)
Epoxy adhesive					
Tensile strength [MPa]	19.9 (3.0)	7.2 (3.1)	6.7 (2.7)	7.4 (7.1)	8.4 (3.6)
Elastic modulus [GPa]	6.5 (3.0)	1.9 (5.1)	1.6 (4.0)	1.9 (14.6)	2.0 (12.4)
Tensile strain [%]	0.4 (6.2)	1.1 (21.4)	1.1 (11.9)	1.0 (25.5)	0.9 (15.3)
CFRP Laminate					
Tensile strength [MPa]	2527 (11)	2750 (2.0)	2594 (2.8)	2562 (3.2)	2422 (5.6)
Elastic modulus [GPa]	190 (9.3)	177 (3.2)	169 (2.3)	184 (4.3)	162 (8.9)
Tensile strain [%]	13.3 (13.6)	15.6 (3.9)	15.4 (3.0)	13.9 (3.9)	14.6 (2.6)
CFRP-to-concrete bond					
Pull-out force [kN]	30.2 (13.3)	27.68 (8.4)	31.06 (10.0)	27.18 (8.6)	29.14 (9.5)

Note: values in parentheses express the coefficient of variation in %.

2.2.1.2. EBR CFRP-to-concrete joint

The pull-out force *versus* loaded end slip relationships can be depicted in **Figure V.3a-d**. Generally, these relationships present typical behavior, starting with an initial phase where chemical bonding predominates. During this phase, the pull-out force increases with only slight changes in the slip. As the applied load continues to rise, the bond begins to weaken, marking the beginning of the debonding stage. However, complete debonding is delayed because the used bond length (220 mm) is greater than the theoretical effective length

(101 mm). This difference allows the slip to keep increasing with minimal change in the pull-out force, creating a plateau-like region on the curve. As a result, the joint failure occurs gradually rather than suddenly. The maximum pull-out force (**Figure V.3e**) shows fluctuations with time which might be due to the observed degradation-regain property of the epoxy adhesive, as well as concrete and CFRP resistance against exposure; however, a general observation is that the bond strength did not degrade significantly. More details on these data were previously provided in [7].

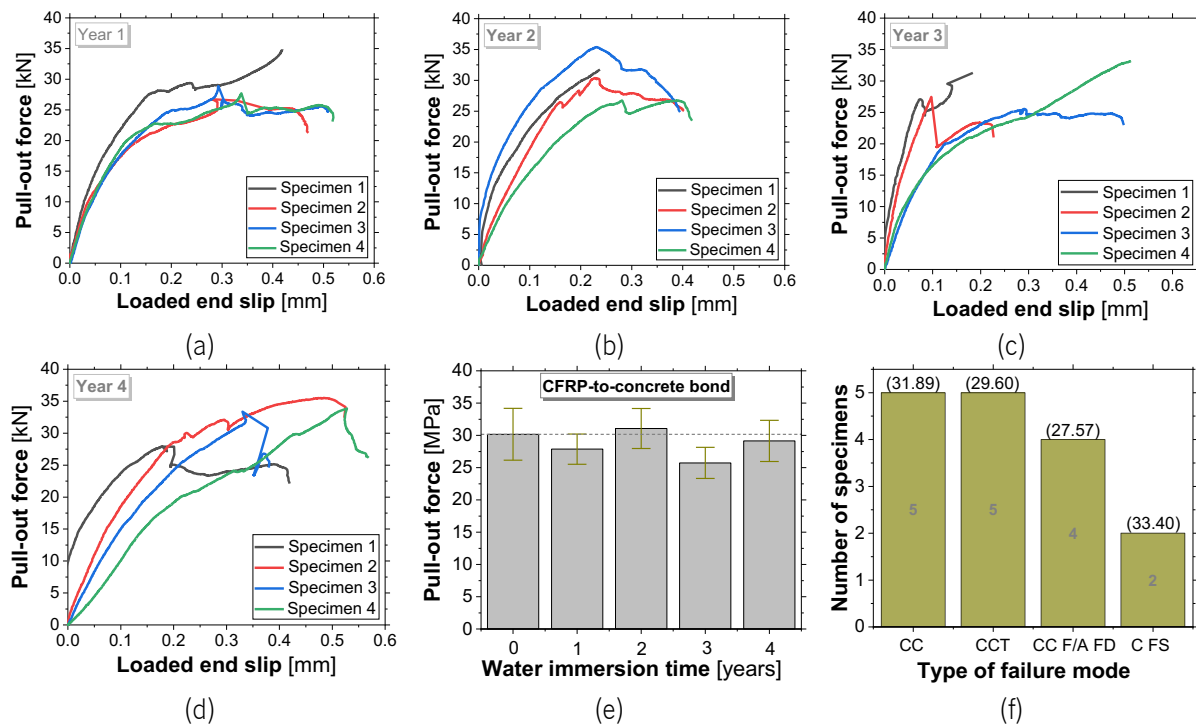


Figure V.3. Single lap shear test results for different water immersion times: variation of pull-out force *versus* loaded end slip curves after (a) 1, (b) 2, (c) 3, and (d) 4 years; (e) variation of maximum average pull-out force, and (f) failure modes from year 1 to year 4, the values in parentheses indicate the average maximum pull-out force in [kN].

The failure modes and their corresponding maximum pull-out forces, as well as the number of specimens with the same failure, are presented in **Figure V.3f**. The failure modes are as follows: CC is the concrete cohesive failure, CCT is the CC with a thick layer at the free end, CC F/A FD is a mixture of CC and adhesive failure (F/A) and FRP disbond (FD), and C FS is a mixture of concrete cracking (C) and FRP fracture (FS); CC and CCT are the major types of failures observed. Overall, among the three material constituents of the bond, the adhesive shows the highest degradation due to water exposure, concrete and CFRP laminate tend to resist the

degradation, and the EBR CFRP-to-concrete joint shows fluctuations around the reference value, which indicates that the bond is not yet significantly affected by water exposure after 4 years.

2.3. Finite element model

This section presents the material constitutive models adopted in the finite element analysis, followed by a detailed description of the modeling approach implemented using ABAQUS software [37].

2.3.1. Development of material constitutive models

2.3.1.1. Concrete model

The concrete damage plasticity (CDP) model, available in the ABAQUS software [37] provides a general capability for modeling concrete and other quasi-brittle materials, and it can describe the irreversible damage that occurs during fracturing process of a material [37]. This damage is expressed in terms of *compressive damage* (DAMAGEC) and *tensile damage* (DAMAGET) to indicate the material damage under compression and tension, respectively [37]. The CDP model is used to model concrete behavior due to its capability to capture the concrete nonlinearity, cracking and crushing, handle irreversible deformations, and track the evolution of damage. In the present work, the inputs to the CDP model for concrete under compression and tension were determined from **Eq. (V.1)** by Saenz [21] and **Eq. (V.4)** by Horjick [22], respectively, in which the required constant parameters (maximum compressive stress, pull-off stress, and elastic modulus) were obtained from experimental results in **Table V.4**. Then, the corresponding stress-strain (or crack opening displacement) relations were established as shown in **Figures V.4a,b, V.4c,d, V.4e,f, and V.4g,h** for year 1 to year 4 of water immersion, respectively.

$$\sigma_c = \frac{E_c \varepsilon_{cc}}{1 + (R + R_E - 2) \frac{\varepsilon_{cc}}{\varepsilon_{ccr}} - (2R - 1) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccr}} \right)^2 + R \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccr}} \right)^3} \quad (\text{V.1})$$

where, $R = [(R_E(R_\sigma - 1))/(R_E - 1)^2] - 1/R_E$, $R_E = E_c/E_0$, $E_0 = \sigma_{cu}/\varepsilon_{ccr}$, $R_E = 4$, $R_\sigma = 4$

$$\varepsilon_{cin} = \varepsilon_{cc} - \varepsilon_{0cel} \quad (\text{V.2})$$

$$d_c = 1 - \frac{\sigma_c}{\sigma_{cu}} \quad (\text{V.3})$$

where, σ_c , σ_{cu} , ε_{cc} , ε_{ccr} , $\varepsilon_{0_{cel}}$, ε_{cin} , d_c , E_0 , and E_c are the compressive stress, maximum compressive stress, compressive strain, compressive strain corresponding to the maximum compressive stress, elastic compressive strain, inelastic strain calculated as Eq. (V.2), compressive damage calculated as Eq. (V.3), initial elastic modulus, and elastic modulus, respectively.

$$\frac{\sigma_t}{\sigma_{tu}} = \begin{cases} \left[1 + \left(c_1 \frac{w_t}{w_{cr}} \right)^3 \exp \left(-c_2 \frac{w_t}{w_{cr}} \right) - \left(\frac{w_t}{w_{cr}} \right) (1 + c_1^3) \exp(-c_2) \right], & 0 < w_t < w_{cr} \\ 0, & w_{cr} < w_t < \infty \end{cases} \quad (V.4)$$

$$d_t = 1 - \frac{\sigma_t}{\sigma_{tu}} \quad (V.5)$$

where σ_t , σ_{tu} , w_t , w_{cr} , and d_t are the tensile stress, maximum tensile stress, crack opening displacement, maximum crack opening displacement, and tensile damage calculated as Eq. (V.5), respectively. For clarity, $w_{cr} = 5.14(G_F/\sigma_{tu})$, $G_F = (0.0469d_a^2 - 0.5d_a + 26)(\sigma_{cu}/10)^{0.7}$ $c_1=3.0$, $c_2=6.93$, $d_a=12.5$. G_F , σ_{cu} , and d_a are fracture energy, maximum compressive stress, and maximum aggregate size, respectively.

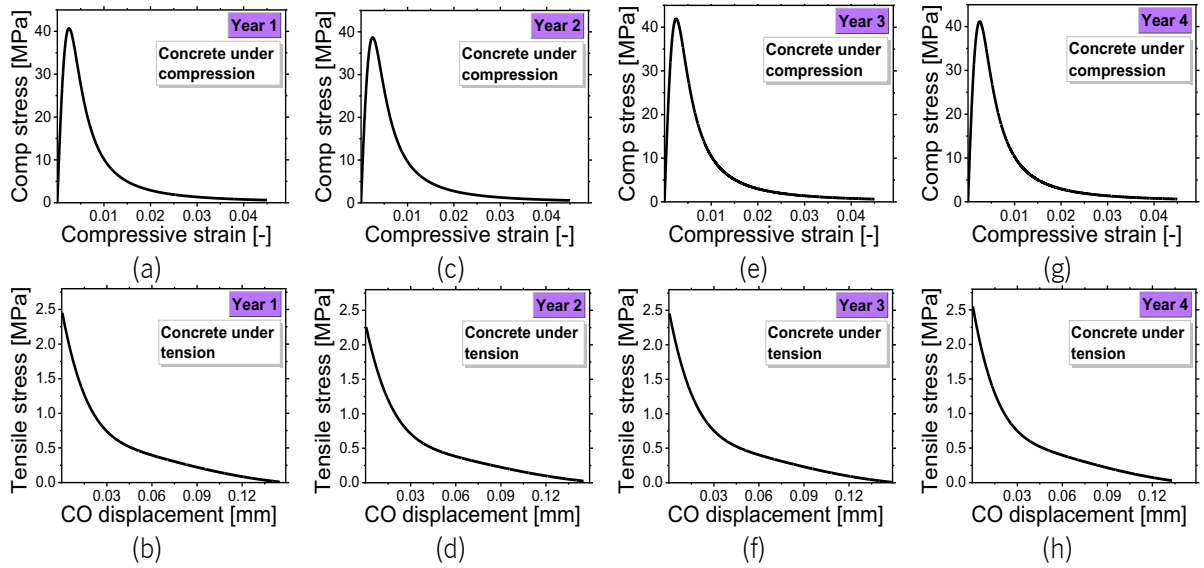


Figure V.4. Developed concrete constitutive models using experimental data and existing equations. Note: Comp and CO stand for compressive and crack opening, respectively.

The relations described above were furthermore used in the ABAQUS as the inputs to the developed finite element models. It is worth noting that this approach is of simplified nature, given the fact that it relies on the experimentally known (recorded) values, and additionally without inclusion of complex mechanisms that occur at the interface level.

2.3.1.2. Epoxy adhesive model

Modeling the epoxy adhesive follows the same procedure as that of concrete. The behavior under compression and tension is modelled using **Eq. (V.1)** by Saenz [21] and **Eq. (V.6)** by Belarbi and Hsu [23], respectively, and by using the properties from **Table V.2** and **Table V.4**. Although these models were typically established for concrete modeling, their capabilities to model the behavior of other materials such as a confined adhesive (i.e., surrounded by concrete and CFRP laminate in almost its regions) is assessed in this work, as adhesive models in a such confined state do not exist in the literature. The corresponding stress-strain relations are established as shown in **Figures V.5a,b, V.5c,d, V.5e,f, and V.5g,h**, for year 1 to year 4, respectively. Like for the case of concrete, it should be noted that the derived adhesive constitutive models heavily rely on the experimentally recorded values, rather than theoretical predictions, which therefore cannot be more generalised to adhesives other than those addressed in the present work.

$$\sigma_t = \begin{cases} E_0 \varepsilon_{tt} & \text{if } \varepsilon_{tt} \leq \varepsilon_{tcr} \\ \sigma_{tu} \left(\frac{\varepsilon_{tcr}}{\varepsilon_{tt}} \right)^{0.4} & \text{if } \varepsilon_{tt} > \varepsilon_{tcr} \end{cases} \quad (\text{V.6})$$

$$\varepsilon_{Tcr} = \varepsilon_{Tt} - \varepsilon_{0Tel} \quad (\text{V.7})$$

$$d_t = 1 - \frac{\sigma_t}{\sigma_{tu}} \quad (\text{V.8})$$

where, σ_t , σ_{tu} , ε_{tt} , ε_{tcr} , ε_{Tcr} , ε_{Tt} , ε_{0Tel} , d_t , and E_0 are the tensile stress, maximum tensile stress, tensile strain, tensile strain corresponding to the maximum tensile stress, cracking strain calculated as **Eq. (V.7)**, total tensile strain, elastic tensile strain, tensile damage calculated as **Eq. (V.8)** and initial elastic modulus, respectively.

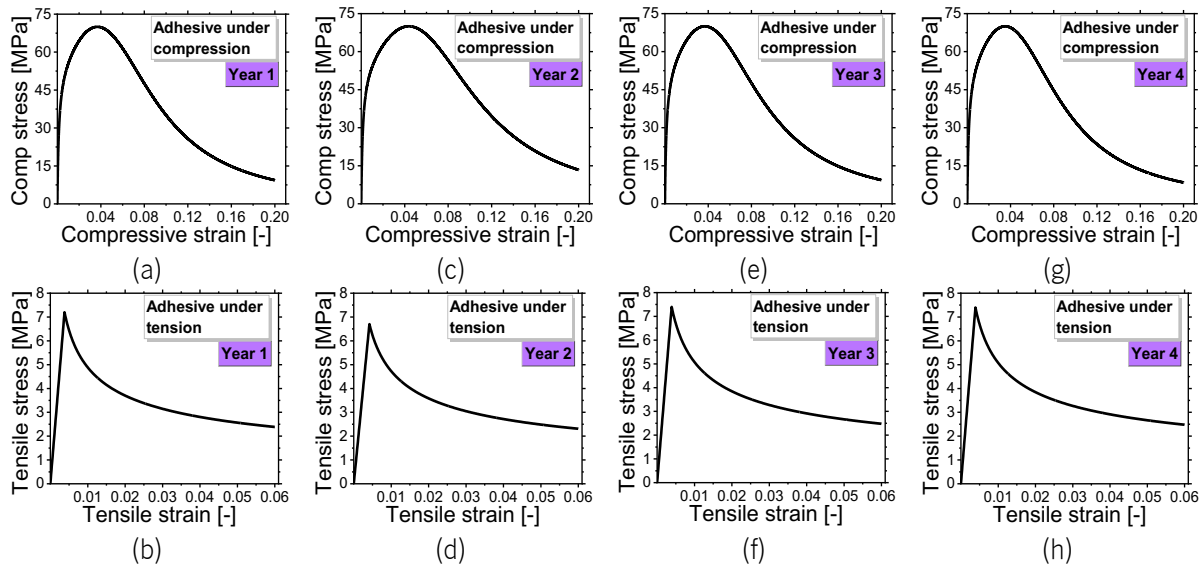


Figure V.5. Developed epoxy adhesive constitutive models using experimental data and existing equations. Note: Comp stands for compressive.

2.3.1.3. CFRP laminate model

The CFRP laminate is modelled as linear elastic up to failure. The properties used for each year are shown in **Table V.4**, and the Poisson ratio is assumed to be 0.3, a value in the range reported by [39,40]. This value is assumed to be constant during the exposure duration, since the CFRP laminate properties show insignificant change after 4 years of exposure (**Figure V.3**).

2.4. Modeling approach

A summary of the workflow for modeling the CFRP-to-concrete joint/bond using ABAQUS software [37] is provided in **Figure V.6**, in which the material constitutive models were set using data shown in **Table V.5**. It is important to note that the adhesive plasticity flow parameters are kept like those of concrete for model simplification, since such parameters do not exist in the literature for the adhesive. The model parts were constructed using the geometry and dimensions of the real specimens shown in **Figure V.1a**. The epoxy adhesive layer and concrete block were modelled as a 3D deformable solid using 3D Stress (C3D8R) element, and the CFRP laminate as a 3D deformable shell using S4R element. A tie constraint was applied to each interface, assuming a perfect bond at the material interfaces. With this assumption, the interface failure is not captured, nevertheless, the fact that concrete cohesive failure was predominant during experimental tests

(Figure V.3f) leads to less interest in predicting the interface failure. Boundary conditions were created at each of the reference points created at the location of the supports (see Figure V.1a). A displacement-controlled loading was applied to pull-out the CFRP laminate. After performing a mesh sensitivity analysis, all bond constituent materials were given the same meshing (5x5 [mm] hex shapes) at interfaces as shown in Figure V.7a,b).

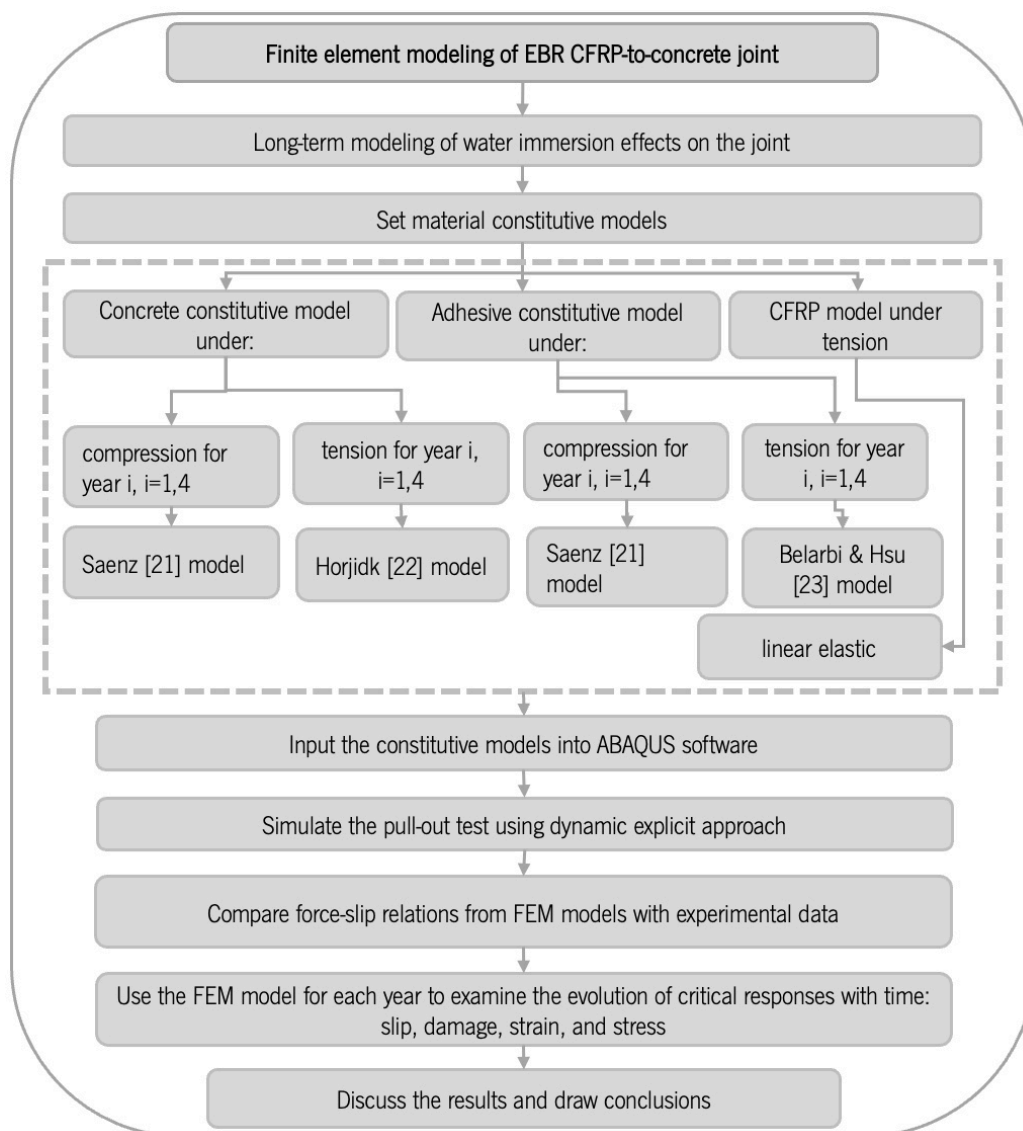


Figure V.6. Workflow of the numerical modeling of the EBR CFRP-to-concrete joint under water immersion exposure.

Table V.5 summarizes the concrete and epoxy adhesive parameters required for the CDP model.

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Table V.5. Damage plasticity model inputs for concrete and epoxy adhesive

Material	Concrete damage plasticity model (CDP) parameters										
	Plasticity flow parameters					Compression			Tension		
	D_a	e	f_{b_0}/f_c	K	V_p	CC_c	ϵ_{cin}	d_c	SS_t	w_t	d_t
Concrete	38	0.1	1.16	0.6667	0.0	Eq.(1)	Eq.(2)	Eq.(3)	Eq.(4)	Eq.(4)	Eq.(5)
Epoxy adhesive	38	0.1	1.16	0.6667	0.0	Eq.(1)	Eq.(2)	Eq.(3)	Eq.(6)	Eq.(7)	Eq.(8)
CFRP laminate	modelled as linear elastic up to failure										

Note: D_a : Dilation angle, e : eccentricity, f_{b_0}/f_c : the ratio of initial equibiaxial yield stress to initial uniaxial yield stress in compression, K : the ratio of the second stress invariant on the tensile meridian to the one on the compressive meridian, V_p : viscosity parameter, CC_c : constitutive law under compression, CC_t : constitutive law under tension, ϵ_{cin} : inelastic strain, d_c : compressive damage, SS_c : stress-strain curve under compression, SS_t : stress-strain (or displacement) curve under tension, w_t : crack opening displacement, d_t : tensile damage.

The model was used to predict force-slip relationships and to examine further responses not captured during experiment (evolution of slip, stresses, CFRP strains, and damage propagation).

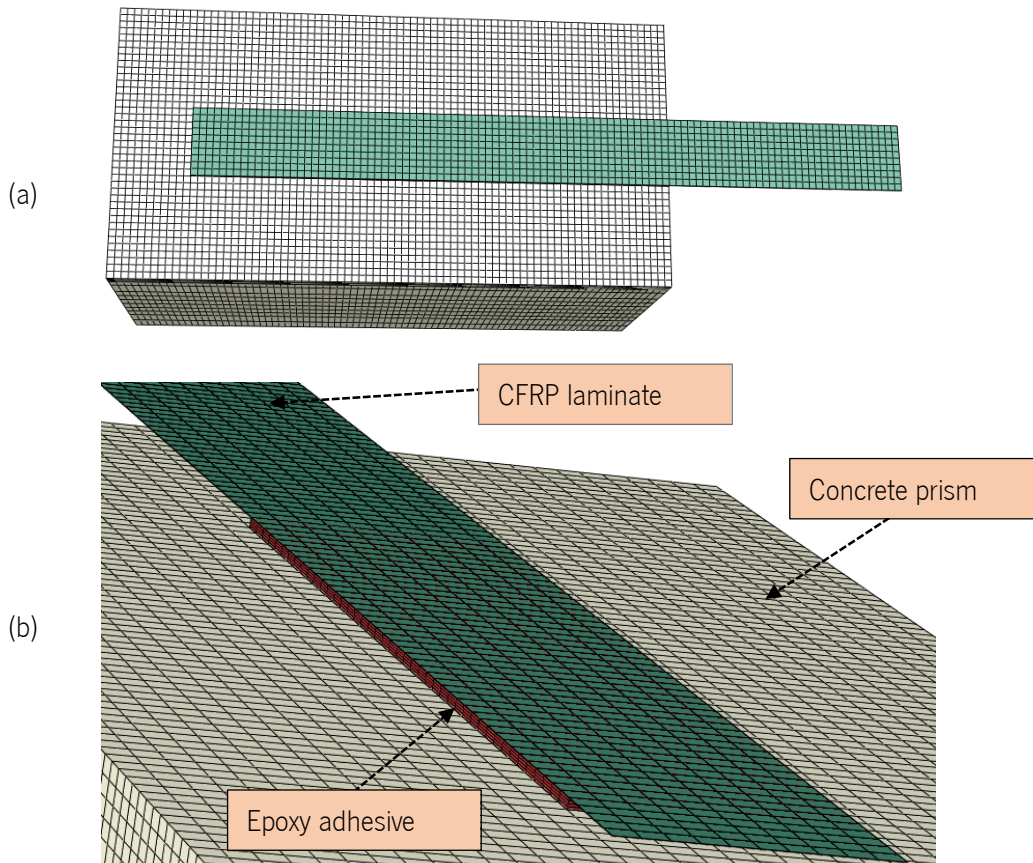


Figure V.7. Finite element model of the EBR CFRP-to-concrete joint: (a) entire model and (b) focus on the bond region.

3. NUMERICAL RESULTS AND DISCUSSION

The accuracy of the predicted force-slip curves is assessed following the recommendations from [37]. That is, when using ABAQUS dynamic explicit, two conditions need to be satisfied. The first condition is that the kinetic energy (KE) of the deforming material in the model should be less than 5 to 10 % of its internal energy (IE) [37], which is verified in **Figure V.8a**. The second condition is that the external work done (Ew) and the IE should be nearly the same [37] and this is also verified in **Figure V.8b**. With these two conditions met, the reliability of the model is assured. This paved the way to investigating other CFR-to-concrete bond properties at both material and interface levels as shown in **Sections 3.1** and **3.2**, respectively.

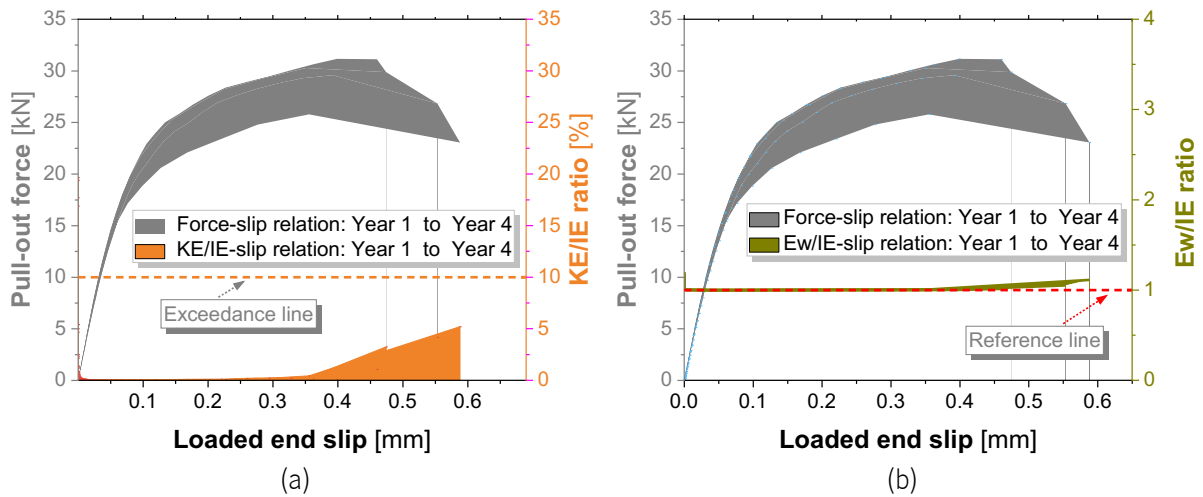


Figure V.8. Energy efficiency check for all developed models: (a) kinetic and internal energy *versus* force-slip relations and (b) external work and internal energy *versus* force-slip relations for all exposure times.

3.1. Responses of the constituent materials

3.1.1. Concrete

To examine concrete responses, two loading levels were considered: (i) loading level just before failure (denoted as *second-last loading stage*) and (ii) at failure (denoted as *last loading stage*). Concrete damage and slips at the second-last and last loading stages from the 1st year to the 4th year of exposure are plotted in **Figure V.9** to **Figure V.12**, respectively, with preliminary results already presented in [41]. For all years of exposure, the maximum values for both compressive and tensile damage have not been yet reached at the free end for the second-last loading stage (even though the maximum pull-out force has already been reached), as the maximum force is not proportional to the maximum damage. However, for the last loading stage, the maximum values for

the compressive damage suddenly propagate the entire bonded region while those for tensile damage advance mostly in its core regions and at the free end region. This is because as the CFRP is pulled-out the separation resistance at the adhesive-concrete interface causes the transferred stresses to be localized at the free end thereby causing this region to reach the maximum tensile damage first. After this, the additional loading tends to cause concrete detachment with a thick layer at the free end region. However, the stresses first redistribute backwardly in the form of compressive stresses towards the loaded end thereby leading to the full damage under compression. On the other hand, the slip profiles in each year (Figures V.9c, 10c, 11c, 12c) show 4 different regions with similar values in each region, where the highest values are at the region with the loaded end (i.e., the first 40 mm from the loaded end), and uniform progressive decreases (in each region) towards the free end. In fact, this shows the reason for the observed smaller slips at the free end than those at the loaded end. Overall, for all years of exposure, concrete is found damaged significantly.

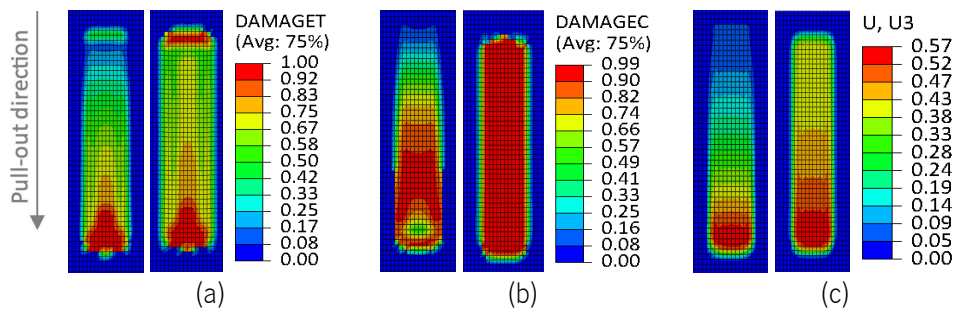


Figure V.9. Concrete responses at second-last (left image) and last (right image) stages after 1 year of water immersion: damage under (a) tension and (b) compression, (c) slip in the loading direction.

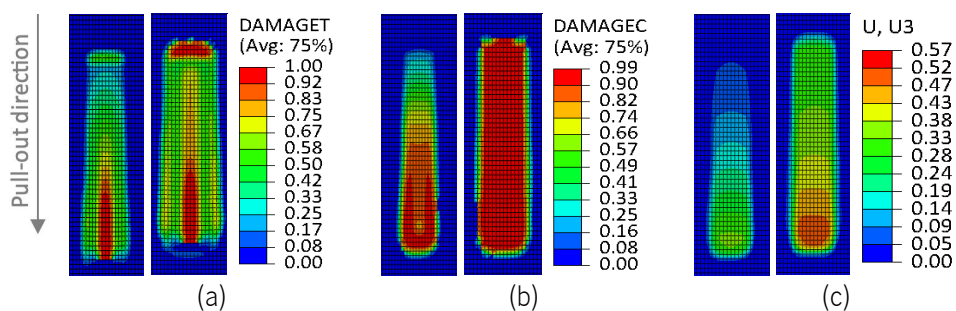


Figure V.10. Concrete responses at second-last (left image) and last (right image) loading stages after 2 years of water immersion: damage under (a) tension and (b) compression, and (c) slip in the loading direction.

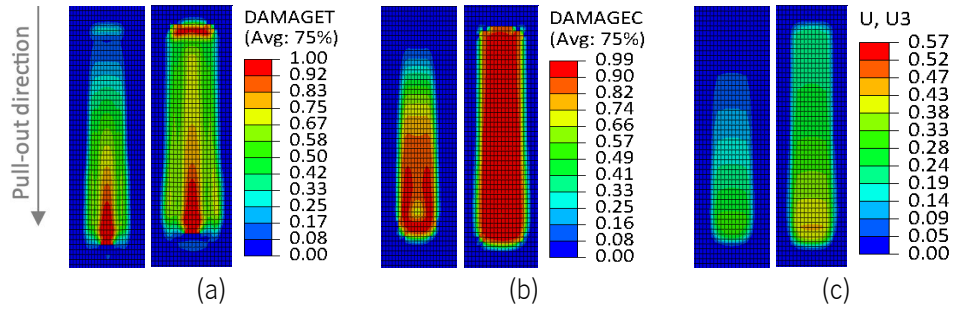


Figure V.11. Concrete responses at second-last (left image) and last (right image) loading stages after 3 years of water immersion: damage under (a) tension and (b) compression, and (c) slip in the loading direction.

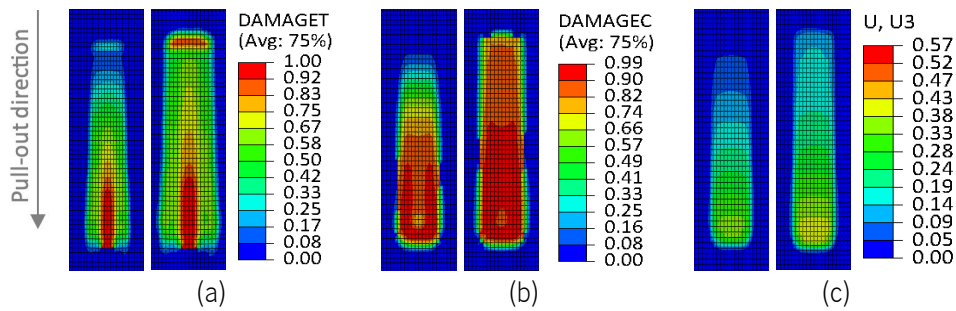


Figure V.12. Concrete responses at second-last (left image) and last (right image) loading stages after 4 years of water immersion: damage under (a) tension and (b) compression, (c) slip in the loading direction.

3.1.2. Epoxy adhesive

Epoxy adhesive tensile damage, compressive damage, and slip at the second-last and last loading stages for the top (adhesive-to-CFRP interface) and bottom (adhesive-to-concrete interface) bonded regions from year 1 to year 4 of water exposure are plotted in **Figure V.13** to **Figure V.16**, respectively; the first two images show top and bottom responses, respectively, at the second-last loading stage, and the last two images show the same responses at the last loading stage. For all years of water immersion, the top region only shows significant tensile damage (**Figures V.13a, V.14a, V.15a, V.16a**) and compressive damage (**Figures V.13b, V.14b, V.15b, V.16b**) at the loaded end and nearly zero damage at the free end; however, the bottom region indicates significant tensile damage along the longitudinal edges, mainly becoming substantial at the free end. No significant compressive damage is observed at the bottom region, except at the corners of the loaded end region at the 1st year of exposure. On the other hand, the slip profiles for all years of exposures (**Figures V.13c, V.14c, V.15c, V.16c**) indicate larger slips at the loaded end which gradually decrease towards the free end. Overall, the adhesive damage is minor, but the slips are slightly higher than those found in concrete mainly because of its closeness to the CFRP laminate compared to concrete.

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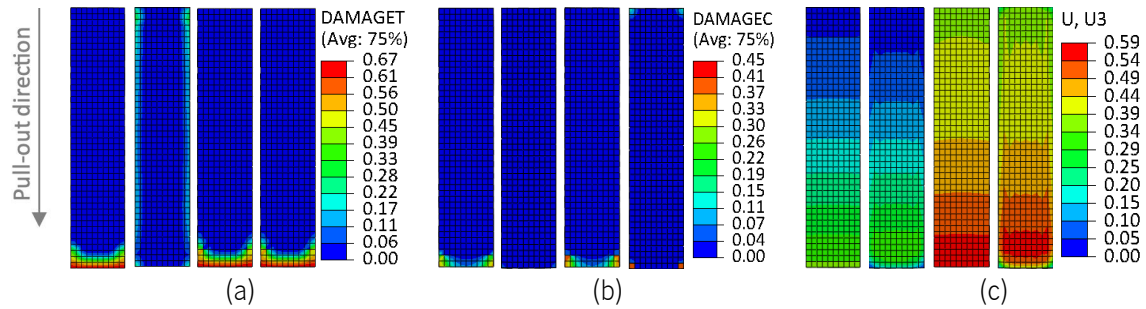


Figure V.13. Epoxy adhesive responses at second-last and last loading stages after 1 year of water immersion: damage (a) under tension and (b) compression, (c) slip in the loading direction.

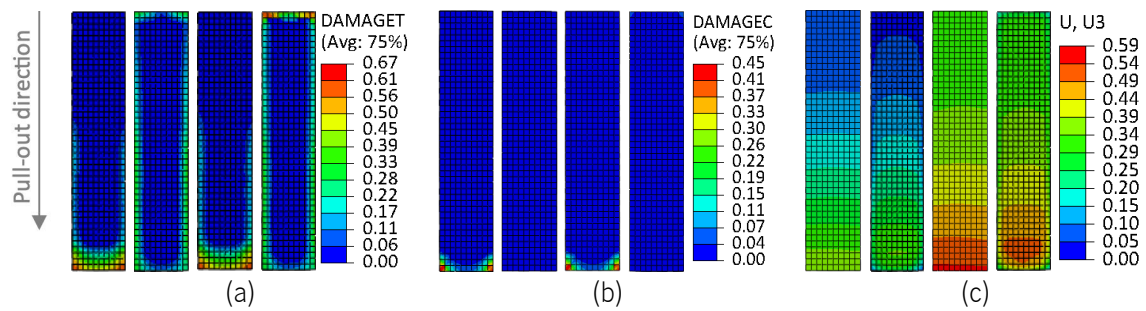


Figure V.14. Epoxy adhesive responses at second-last and last loading stages after 2 years of water immersion: damage (a) under tension and (b) compression, (c) slip in the loading direction.

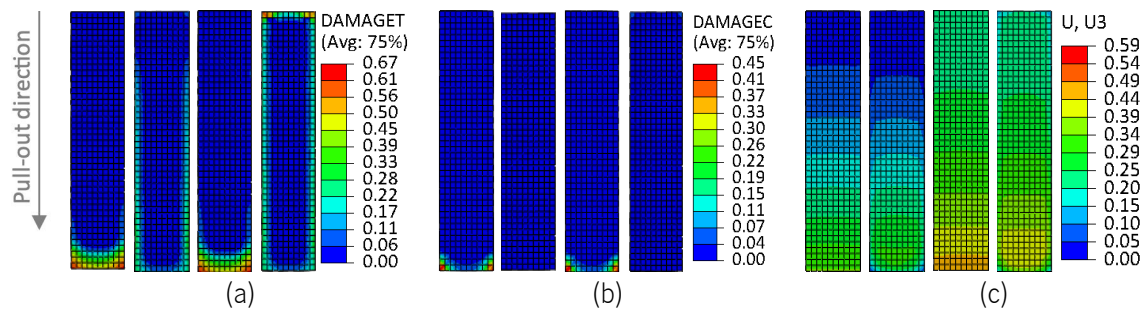


Figure V.15. Epoxy adhesive responses at second-last and last loading stages after 3 years of water immersion: damage (a) under tension and (b) compression, (c) slip in the loading direction.

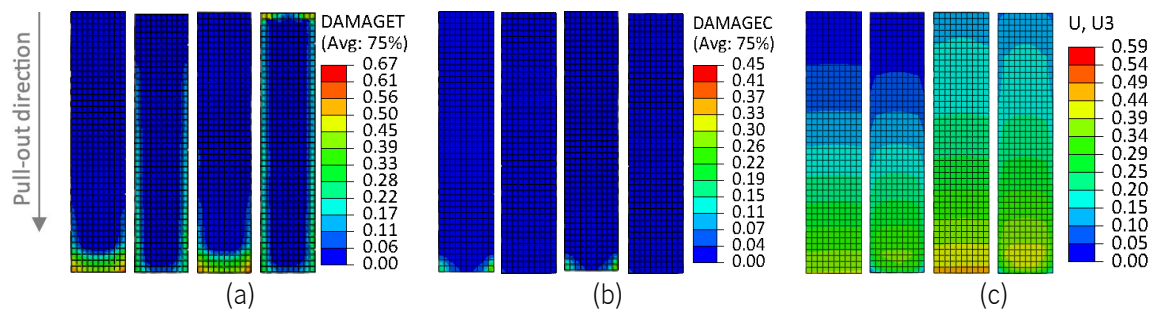


Figure V.16. Epoxy adhesive responses at second-last and last loading stages after 4 years of water immersion: damage (a) under tension and (b) compression, (c) slip in the loading direction.

3.1.3. CFRP laminate

The CFRP laminate stress and slip profiles at failure stage for all four years of water immersion are presented in Figure V.17.

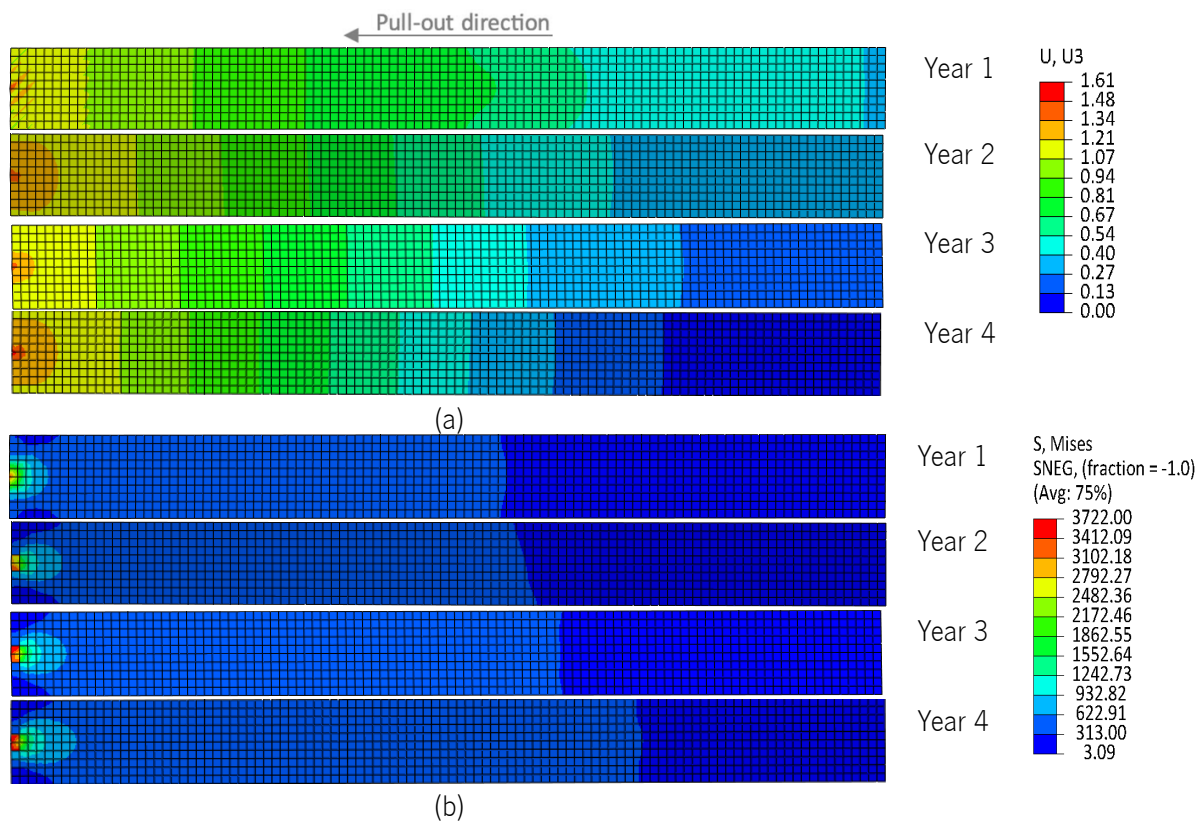


Figure V.17. CFRP laminate responses at failure stage: (a) slip and (b) von Mises stress variation for up to 4 years of water immersion.

The slip values are comparable to those obtained in experimental tests, and the von Mises stress (used referring to [42]) values are higher at the point of load application and tend to significantly decrease towards the bonded region, with increasing trends as the water immersion times increases, particularly at the bonded region.

3.2. EBR CFRP-to-concrete bond responses

The results from the EBR CFRP-to-concrete joint are analyzed, considering the behavior of the bond constituent materials previously discussed. Comparative graphs between experimental and FEM model force-slip curves from year 1 to year 4 are shown in **Figure V.18a** to **Figure V.18d**, respectively.

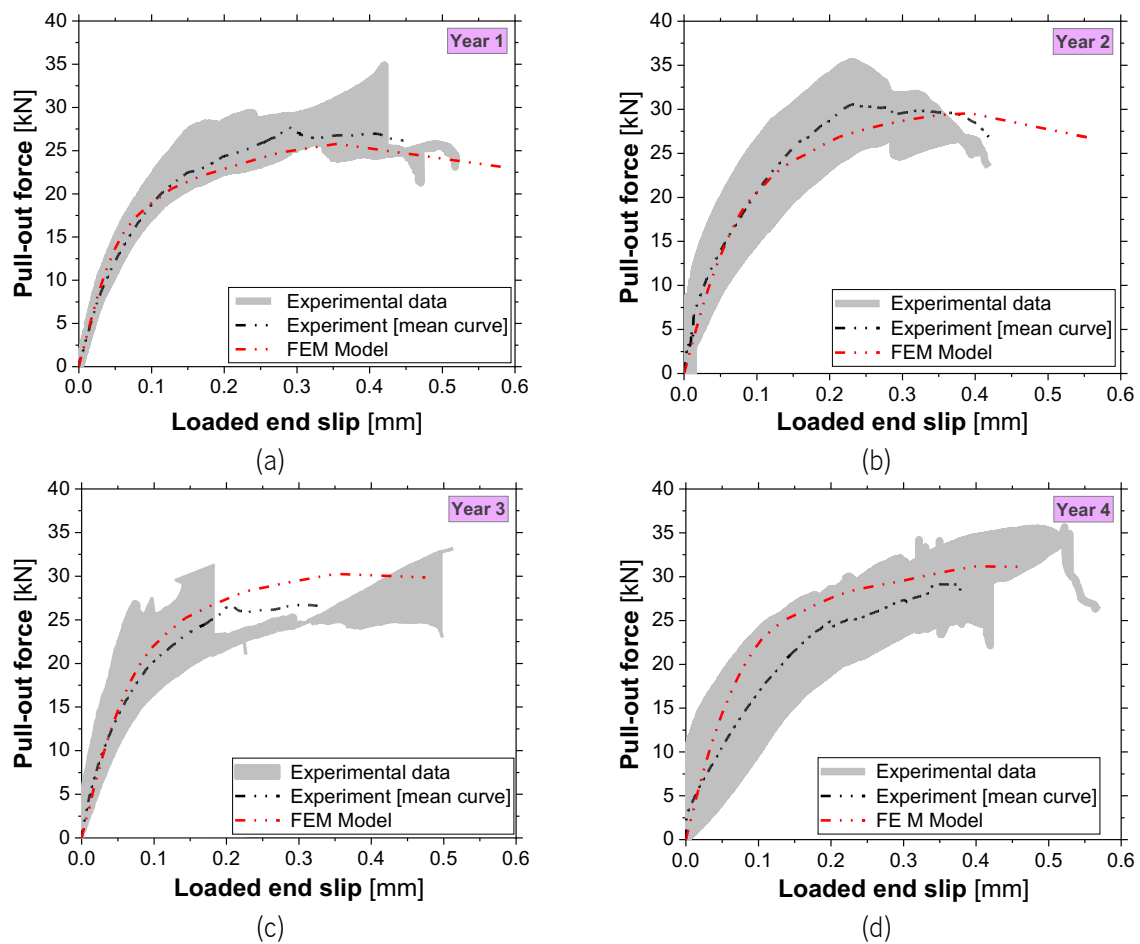


Figure V.18. EBR CFRP-to-concrete bond responses: experimental *versus* FEM model data (a) after 1 year, (b) after 2 years, (c) after 3 years, and (d) after 4 years of water immersion.

In general, the developed FEM model can predict the experimental results, however, initial stiffness prediction in the 4th year tends to deviate significantly from the experimental mean curve, which can be attributed to some

experimental curves that showed unusual trends with zero initial stiffness (see **Figure V.3d**). Hence, these differences between the predicted and experimental data may be more related to the scatter of the experimental results than the developed model.

To have a deeper understanding on the capability of the developed model, the tensile and compressive damage (at failure stage) for the concrete-epoxy adhesive and adhesive-CFRP interfaces are presented in **Figure V.19**. Concrete damage is more significant than that of the epoxy adhesive. Furthermore, the trends for the evolution of CFRP strains (**Figure V.20**) show that maximum values are well below the ultimate CFRP strain, which indicates that the CFRP did not rupture. Hence, the observed significant concrete damage (**Figure V.19**), insignificant adhesive damage (**Figure V.19**), and smaller CFRP strains (**Figure V.20**), confirms that the concrete cohesive failure is the prevailing failure mode. On the other hand, the evolution of CFRP strains (**Figure V.20**) is carefully examined.

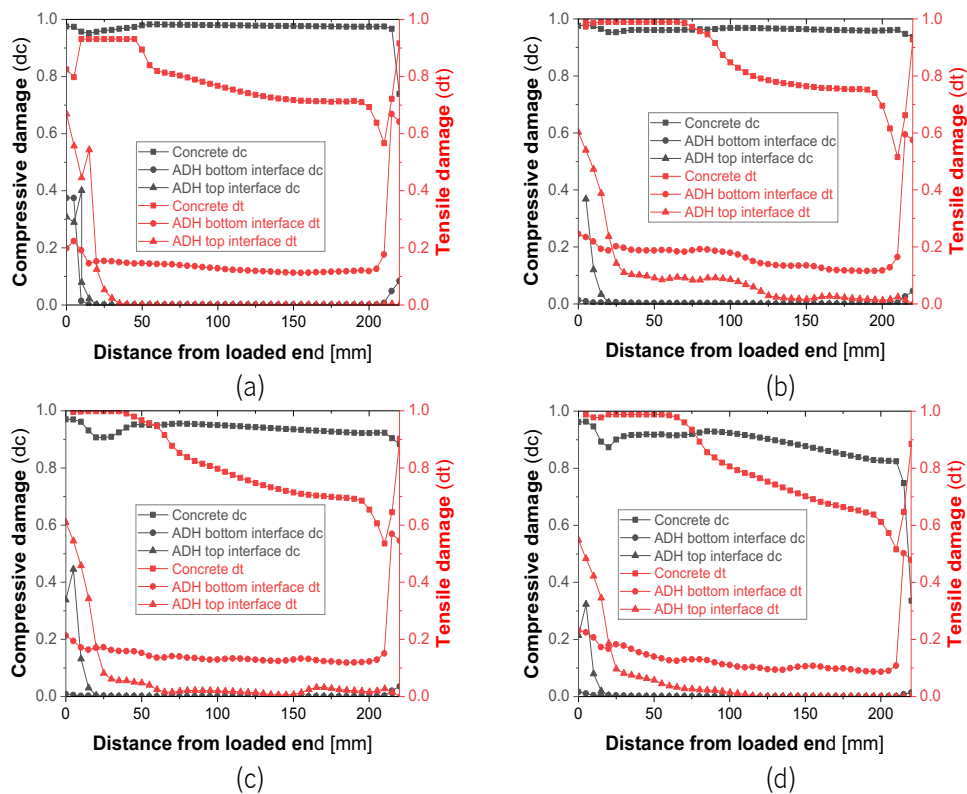


Figure V.19. EBR CFRP-to-concrete bond responses: compressive and tensile damage at bottom (concrete-adhesive) and top (adhesive-CFRP) interfaces at failure stage after (a) 1, (b) 2, (c) 3, and (d) 4 years of water immersion. Notes: ADH: epoxy adhesive, dc and dt stand for compressive and tensile damage, respectively.

A general observation is that up to a certain loading stage, higher strains still concentrate in the loaded end regions and asymptotically decrease to reach nearly zero strains at non-debonded-yet regions. After that stage, the strain evolution no longer follows the previous asymptotic trends. In fact, the maximum CFRP strains occur at regions close to the loaded end (nearly 15-20 mm from the loaded end) followed by sharp decreases ending approximately at 25 mm, thereafter, levelling off for up to nearly 50 mm, then slightly increasing, and then progressively decreasing towards the free end.

Similar strain profiles (that depend on the loading level) are commonly reported from experimental tests in the literature, e.g., [13,38,43–45], confirming the validity of the developed model and suggesting its adoption in future works. The implementation of the said FEM model would complement experimental works with strain gauges and facilitate the identification of damage along the entire region of the bond and individually in the bond constituent materials.

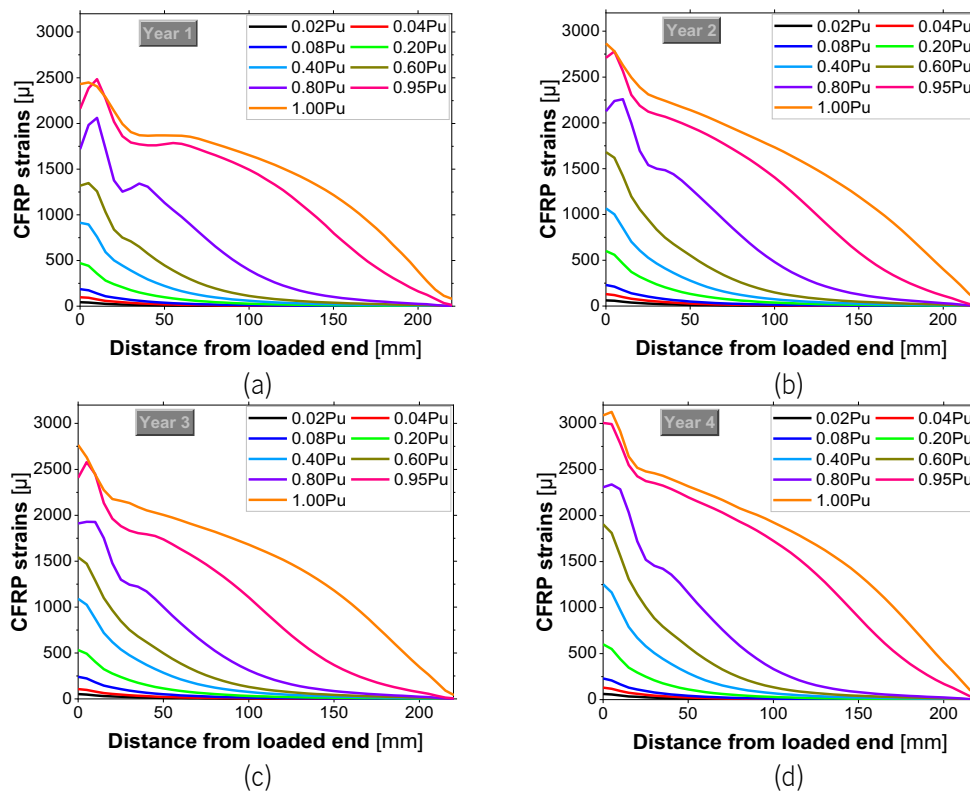


Figure V.20. EBR CFRP-to-concrete bond responses: CFRP strain distribution at different loading stages after (a) 1, (b) 2, (c) 3, and (d) 4 years of water immersion. Notes: Pu: maximum pull-out force.

To examine the effects of water immersion on the interface behavior, the evolution of slip, tensile damage, and compressive damage at different loading levels for the 1st and 4th years of exposure are compared in **Figure V.21-23**, respectively. The slips at a load level of 25 kN (**Figure V.21**) have not yet propagated through the whole effective length (101 mm). However, at the maximum pull-out force (LL3), the entire effective bond length has started slipping, and at the final loading stage (LL4), the entire bonded region has already slipped. Also, the slips in the 1st year are higher than those in the 4th year, regardless of the loading stage.

To examine the effects of water immersion on the interface behavior, the evolution of slip, tensile damage, and compressive damage at different loading levels for the 1st and 4th years of exposure are compared in **Figures V.21-23**, respectively. The slip at a load level of 25 kN (LL2, **Figure V.21**) has not yet propagated through the whole effective length (101 mm). However, at the maximum pull-out force (LL3), the entire effective bond length has started slipping, and at the final loading stage (LL4), the entire bonded region has already slipped. The slips in the 1st year are higher than those in the 4th year (regardless of the loading stage), showcasing water immersion effects.

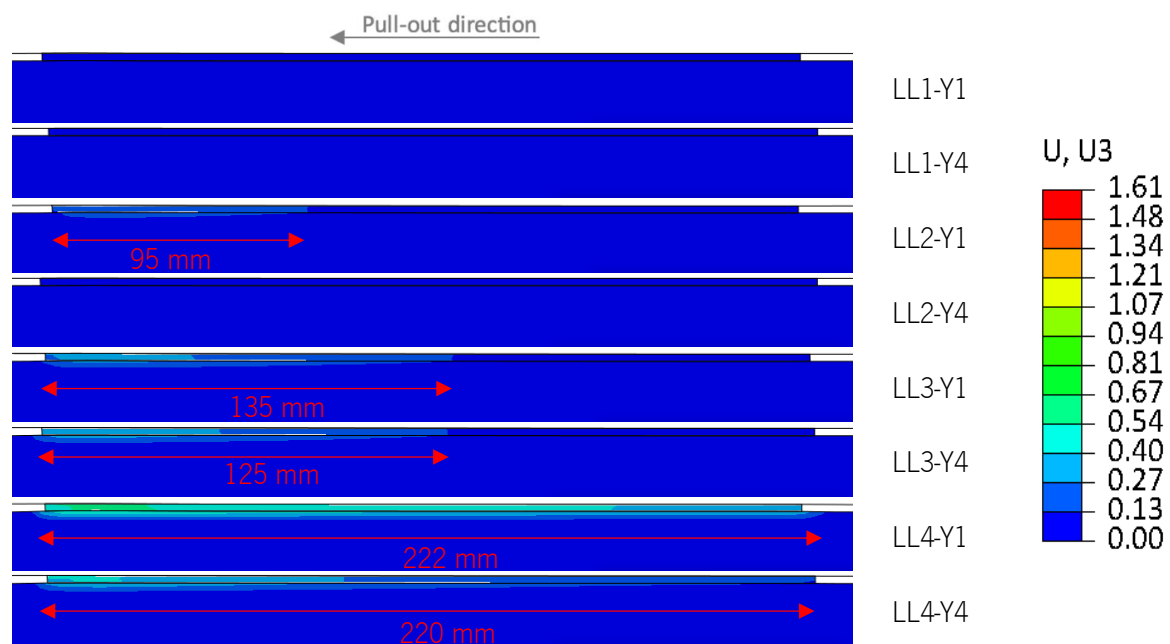


Figure V.21. Comparative perspectives on slip under different loading levels and exposure times. Notes: LL1 = 20 kN, LL2 = 25 kN, LL3 = maximum pull-out force for year 1 (25.8 kN) and year 4 (31.2 kN) denoted as Y1, LL4 = ultimate pull-out force for year 1 (23kN) and year 4 (31.1 kN) denoted as Y4.

On the other hand, both tensile (**Figure V.22**) and compressive (**Figure V.23**) damage generally propagate faster at the 1st year (Y1) than at 4th year (Y4). Furthermore, at LL3, the tensile damage propagates up to a maximum distance of 230 mm (**Figure V.22**), compared to 210 mm for the compressive damage (**Figure V.23**). From this, it is noted that concrete tensile damage propagates faster and reaches the free end before the compressive damage, and this may indicate that concrete first experienced higher localized tensile stresses at the free end, rather than compressive stresses.

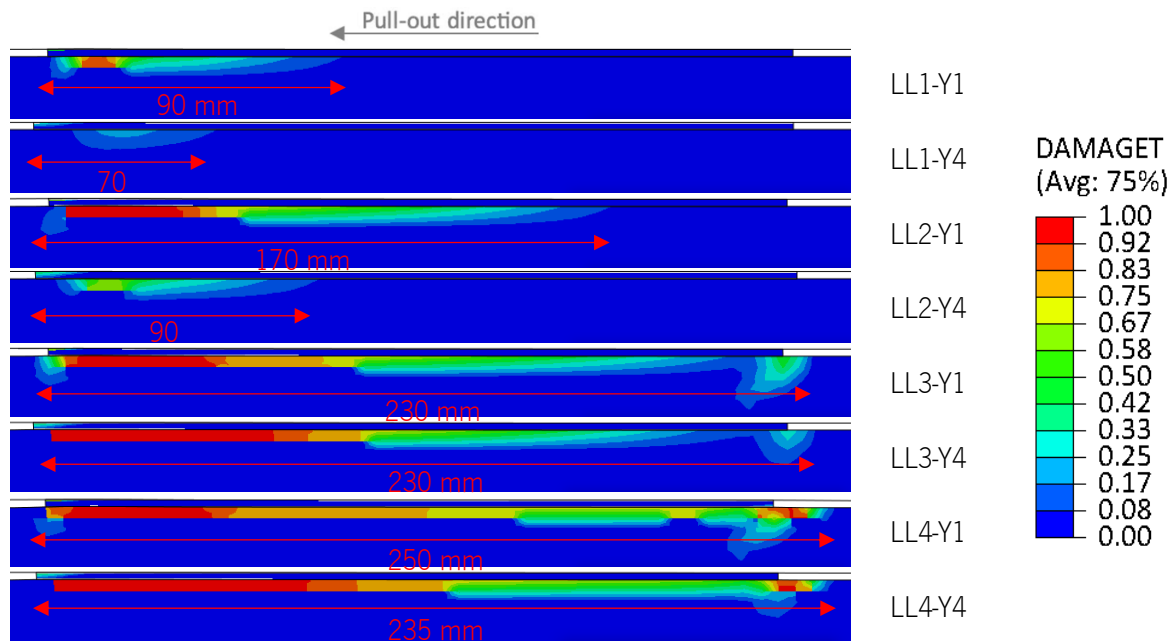


Figure V.22. Comparative perspectives on tensile damage under different loading levels and exposure times. Notes: LL1 = 20 kN, LL2 = 25 kN, LL3 = maximum pull-out force for year 1 (25.8 kN) denoted as Y1 and year 4 (31.2 kN) denoted as Y4, LL4 = ultimate pull-out force for year 1 (23kN) and year 4 (31.1 kN).

The concrete resistance (in a region just closer to the free end) against these high tensile stresses led to high compressive stress that eventually redistributed towards the loaded end, thereby causing full compressive damage nearly across all bonded region at the final loading stage (LL4), see **Figure V.21**. Hence, the experimentally observed thick layer removal can be attributed to the high localized (at the free end region) stress release just after reaching the successive full compressive damage (LL4) and significant tensile damage (LL4); this stress release was heard in the form of massive sound during the experiment at the failure stage.

The observed higher resistance of concrete slip (Figure V.19), compressive (Figure V.20) and tensile (Figure V.21) damages at the 4th year, compared to the 1st year, may explain the reason for the resulting higher maximum pull-out force (31.16 kN) compared to that at the 1st year (25.80 kN) as shown in Table V.6. Compared with experimental results, the percentage differences for the case of the pull-out force are within 4.8 and 11 % (Table V.6). Overall, the EBR CFRP-to-concrete bond behavior is found to significantly depend on the concrete properties. Also, in that context, longer water immersion may delay the slip and damage propagation, mainly being attributed to the concrete's cement hydration with time.

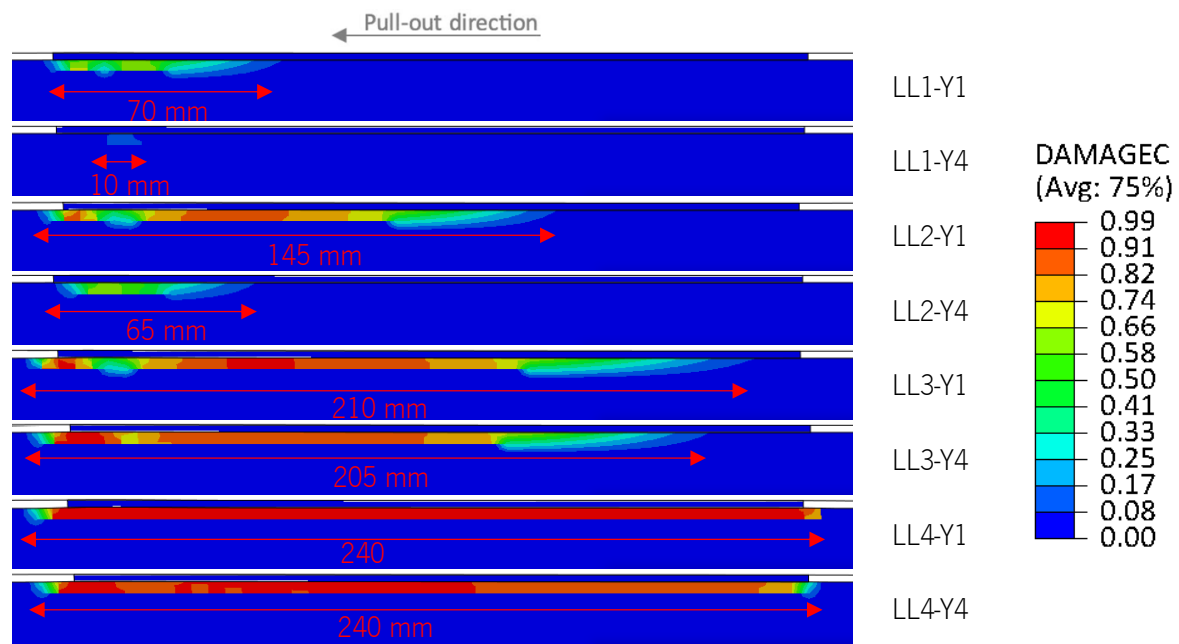


Figure V.23. Comparative perspectives on compressive damage under different loading levels and exposure times. Notes: LL1 = 20 kN, LL2 = 25 kN, LL3 = maximum pull-out force for year 1 (25.8 kN) denoted as Y1 and year 4 (31.2 kN) denoted as Y4, LL4 = ultimate pull-out force for year 1 (23kN) and year 2 (31.1 kN).

Table V.6. Experimental *versus* finite element model: maximum pull-out force and loaded end slip.

Exposure time	Year 1		Year 2		Year 3		Year 4	
Bond property	F_{max} (kN)	S_{max} (mm)	F_{max} (kN)	S_{max} (mm)	F_{max} (kN)	S_{max} (mm)	F_{max} (kN)	S_{max} (mm)
Experiment	27.86	0.45	31.06	0.42	27.18	0.33	29.14	0.38
Finite element model	25.80	0.59	29.57	0.55	30.26	0.47	31.16	0.46
Difference (%)	7.4	31.1	4.8	30.9	11.1	42.42	6.9	21.1

Notes: F_{max} and S_{max} stand for the maximum pull-out force and maximum loaded end slip, respectively

3.3. Comparison of failure modes

The concrete cohesive failure prediction (from FEM model) is compared to that from experimental tests in Figure V.24 for all investigated exposure times.

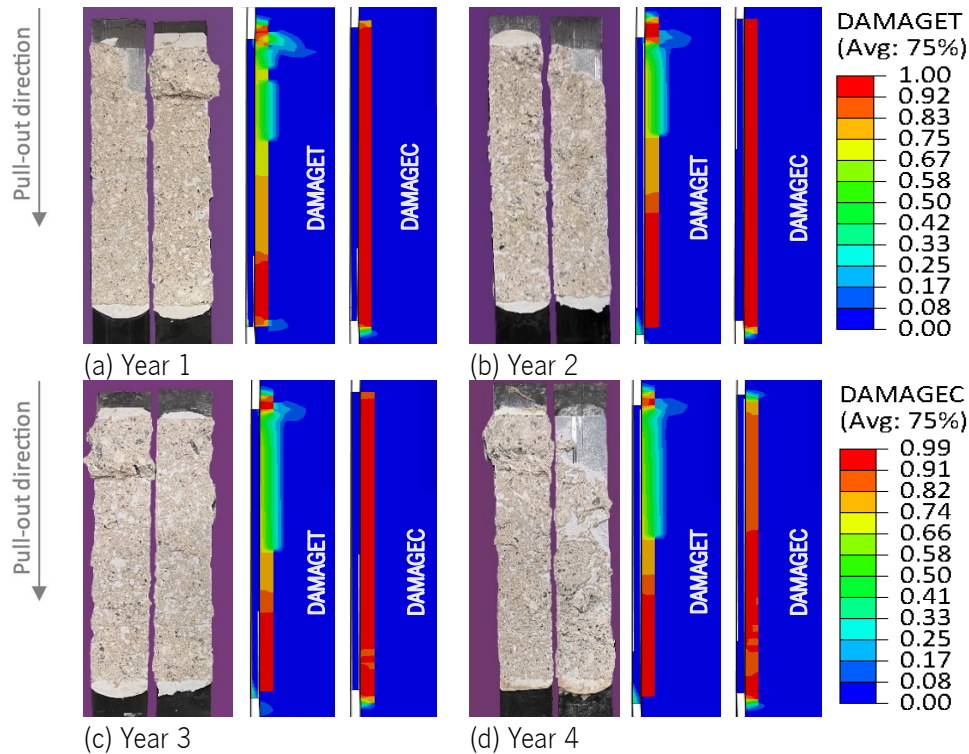


Figure V.24. Experimental and FEM failure mode responses after (a) 1, (b) 2, (c) 3, and (d) 4 years of water immersion. Notes: DAMAGET and DAMAGEC indicate tensile and compressive damage of materials, respectively.

As previously shown in **Figure V.3f**, the predominant failure modes from experimental tests were of concrete cohesive type, hence the representatives of these failures are shown in **Figure V.24**, alongside with FEM predictions. The predicted thick layer removal at the free end (depicted in **Figure V.24**, DAMAGET images) is also observed during the experiments (**Figure V.24a,c,d**), confirming the model capability. In addition, concrete resistance previously predicted to occur at the free end is also reflected in the form of partially adhesive failure during experimental tests (e.g., **Figure V.24a,b,d**), and this resistance led to the predicted full compressive damage (**Figure V.24**, DAMAGEC images) that occurred just at the final loading stage (i.e., LL4 in **Figure V.23**). Overall, the results from experiment and FEM prediction agree well, both showing concrete cohesive as a predominant failure mode.

4. CONCLUSIONS AND RECOMMENDATIONS

A simplified finite element model for simulating the EBR CFRP-to-concrete bond durability under tap water immersion controlled at 20 °C (using ABAQUS software) is developed. The model incorporates the concrete and

epoxy adhesive nonlinearities using a damage plasticity model and assumes the CFRP laminate to present a linear behavior up to failure. The model is validated by experimental data showing remarkable accuracy. The key findings are as follows.

- The yearly predicted force-slip curves agree well with experimental results. The predicted concrete cohesive failure also agrees well with that observed during tests. The model explains reasons for the experimentally observed concrete thick layer removal at the free end region: the removal was due to high localized tensile stresses leading to high concrete tensile damage just at the final loading stage, which thereafter led to stress redistribution, causing a sharp increase in the compressive damage across the entire bonded region, followed by a complete debonding.
- The adhesive layer just closer to the loaded end region is very susceptible to damage. The bond slip, material tensile and compressive damages (particularly for concrete) increased faster at the 1st year of water exposure than at the 4th year, which led to higher pull-out force in the latter year. This delayed damage propagation and the higher pull-out force at the 4th year is explained by the continuation of concrete's cement hydration over time, since the bond failure was governed by concrete. From this, it is concluded that the exposure time may affect both the damage propagation rate and the bond slip, as well as the resulting pull-out force.
- The predicted CFRP strain profiles resemble to those commonly observed during experimental tests, hence indicating the capability of the developed model. Furthermore, the CFRP strains generally show similar trends from the 1st to the 4th year of exposure (although their maximum values change with time), hence concluding that the trends are not affected by water exposure.

The developed finite element model for the EBR CFRP-to-concrete bond behavior under water immersion is simplified in nature, as it relies solely on the properties of the individual bond constituent materials, without accounting for the complex mechanisms occurring at the interface level. Additionally, the model assumes a perfect bond at the interface, which limits its applicability to systems exhibiting cohesive failure modes—such as those observed in the experimental phase of this study. While many existing studies also report cohesive failure as the predominant mode for EBR CFRP-to-concrete systems, future research could focus on developing models capable of capturing all potential failure modes.

ACKNOWLEDGEMENTS

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REFERENCES

- [1] J. Tatar, S. Milev, Durability of externally bonded fiber-reinforced polymer composites in concrete structures: A critical review, *Polymers* (Basel). 13 (2021) 1–26. <https://doi.org/10.3390/polym13050765>.
- [2] A.P. Vassilopoulos, The history of fiber-reinforced polymer composite laminate fatigue, *Int. J. Fatigue*. 134 (2020). <https://doi.org/10.1016/j.ijfatigue.2020.105512>.
- [3] L. De Lorenzis, J.G. Teng, Near-surface mounted FRP reinforcement: An emerging technique for strengthening structures, *Compos. Part B Eng.* 38 (2007) 119–143. <https://doi.org/10.1016/j.compositesb.2006.08.003>.
- [4] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174. <https://doi.org/10.1016/j.compstruct.2024.118174>.
- [5] H. Yamane, M. Oura, N. Yamazaki, T. Ishihara, K. Hasegawa, T. Ishikawa, K. Takagi, T. Hatsui, Visualizing interface-specific chemical bonds in adhesive bonding of carbon fiber structural composites using soft X-ray microscopy, *Sci. Rep.* 12 (2022) 1–8. <https://doi.org/10.1038/s41598-022-20233-4>.
- [6] F. Djouani, C. Connan, M. Delamar, M.M. Chehimi, K. Benzarti, Cement paste-epoxy adhesive interactions, *Constr. Build. Mater.* 25 (2011) 411–423.

- <https://doi.org/10.1016/j.conbuildmat.2010.02.035>.
- [7] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>.
 - [8] Z. Han, J. Gao, H. Song, G. Xu, Bond Properties of CFRP Externally Bonded Reinforcement on Groove in Concrete, *Int. J. Concr. Struct. Mater.* 18 (2024). <https://doi.org/10.1186/s40069-024-00678-9>.
 - [9] S. Liu, Y. Pan, H. Li, G. Xian, Durability of the bond between CFRP and concrete exposed to thermal cycles, *Materials (Basel)*. 12 (2019). <https://doi.org/10.3390/ma12030515>.
 - [10] Y. Pan, G. Xian, M.A.G. Silva, Effects of water immersion on the bond behavior between CFRP plates and concrete substrate, *Constr. Build. Mater.* 101 (2015) 326–337. <https://doi.org/10.1016/j.conbuildmat.2015.10.129>.
 - [11] R. Cruz, L. Correia, S. Cabral-Fonseca, J. Sena-Cruz, Durability of bond of EBR CFRP laminates to concrete under real-time field exposure and laboratory accelerated ageing, *Constr. Build. Mater.* 377 (2023). <https://doi.org/10.1016/j.conbuildmat.2023.131047>.
 - [12] A. Dushimimana, L. Correia, R. Cruz, J.M. Pereira, J. Sena-Cruz, Durability of CFRP-Concrete Bond in EBR and NSM Systems under Natural Ageing for a Period of Three Years, in: *Proc. 20th Eur. Conf. Compos. Mater.*, CC BY-NC 4.0, Lausanne, Switzerland, 2022: pp. 1–8. https://doi.org/https://doi.org/10.5075/epfl-298799_978-2-9701614-0-0.
 - [13] C. Mazzotti, M. Savoia, B. Ferracuti, An experimental study on delamination of FRP plates bonded to concrete, *Constr. Build. Mater.* 22 (2008) 1409–1421. <https://doi.org/10.1016/j.conbuildmat.2007.04.009>.
 - [14] F.M. Mukhtar, M.E. Shehadah, Experimental verification of 2- and 3-D numerical models for bond-slip behavior of CFRP-concrete, *Constr. Build. Mater.* 287 (2021) 122814. <https://doi.org/10.1016/j.conbuildmat.2021.122814>.
 - [15] F. Bencardino, A. Condello, L. Ombres, Numerical and analytical modeling of concrete beams with steel, FRP and hybrid FRP-steel reinforcements, *Compos. Struct.* 140 (2016) 53–65.

<https://doi.org/10.1016/j.compstruct.2015.12.045>.

- [16] Y.T. Obaidat, S. Heyden, O. Dahlblom, The effect of CFRP and CFRP/concrete interface models when modelling retrofitted RC beams with FEM, *Compos. Struct.* 92 (2010) 1391–1398. <https://doi.org/10.1016/j.compstruct.2009.11.008>.
- [17] J.W. Shi, W.H. Cao, Z.S. Wu, Effect of adhesive properties on the bond behaviour of externally bonded FRP-to-concrete joints, *Compos. Part B Eng.* 177 (2019). <https://doi.org/10.1016/j.compositesb.2019.107365>.
- [18] C.A. Coronado, M.M. Lopez, Numerical Modeling of Concrete-FRP Debonding Using a Crack Band Approach, *J. Compos. Constr.* 14 (2010) 11–21. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000044](https://doi.org/10.1061/(asce)cc.1943-5614.0000044).
- [19] M.R.T. Arruda, J.P. Firmo, J.R. Correia, C. Tiago, Numerical modelling of the bond between concrete and CFRP laminates at elevated temperatures, *Eng. Struct.* 110 (2016) 233–243. <https://doi.org/10.1016/j.engstruct.2015.11.036>.
- [20] J.P. Firmo, M.G. Roquette, J.R. Correia, A.S. Azevedo, Influence of elevated temperatures on epoxy adhesive used in CFRP strengthening systems for civil engineering applications, *Int. J. Adhes. Adhes.* 93 (2019) 102333. <https://doi.org/10.1016/j.ijadhadh.2019.01.027>.
- [21] L. Saenz, Discussion of Equation for the stress-strain curve of concrete by Desayi, P. and Krishnan, S., *ACI J.* 61 (1964) 1229–1235.
- [22] D.A. Hordijk, Local approach to fatigue of concrete, *Diss. Delft Univ. Technol.* (1991) 41. <http://resolver.tudelft.nl/uuid:fa87147b-8201-47ed-83d7-b812b09c5fbb>.
- [23] A. Belarbi, T.T.C. Hsu, Constitutive Laws of Concrete in Tension and Reinforcing Bars Stiffened By Concrete, *Struct. J.* 91 (1994) 465–474.
- [24] IPQ, 2011. NP EN 12390-3. Testing Hardened Concrete. Part 3: Compressive Strength of Test Specimen, Instituto Português da Qualidade (IPQ), Caparica, Portugal, 2011.
- [25] IPQ (Instituto Português da Qualidade), NP EN 12390. Testing Hardened Concrete, Caparica, Portugal, 2013.

- [26] BSI, BS EN 1542. BSI (British Standards Institution). Products and systems for the protection and repair of concrete structures. Test methods. Measurement of bond strength by pull-off., London: BSI, 1999.
- [27] S&P, S&P 220 Resin epoxy adhesive. Technical Data Sheet, Seewen, Switzerland, 2015.
- [28] S&P Clever Reinforcement Company AG, S&P C-Laminate Carbon fibre polymer plates for structural reinforcement, Seewen, Switzerland, 2020. https://www.sp-reinforcement.eu/sites/default/files/field_sup_dcmnt_file/2024/01/24/131247/r_c-laminates_pub_tds_prod_c-laminates_v4.102020_eu_en.pdf.
- [29] ISO 527-2:2012, ISO 527-2:2012—Plastics—Determination of Tensile Properties, Genève, Switzerland, 2012.
- [30] ISO, ISO 527-5:2009 Part 5: Test Conditions for Unidirectional Fibre-Reinforced Plastic composites. Plastic—Determ. Tensile Prop., Inte Genève, Switzerland, 2009; Volume 1, 2009.
- [31] M. Savilotidou, A.P. Vassilopoulos, M. Frigione, T. Keller, Development of physical and mechanical properties of a cold-curing structural adhesive in a wet bridge environment, *Constr. Build. Mater.* 144 (2017) 115–124. <https://doi.org/10.1016/j.conbuildmat.2017.03.145>.
- [32] B. Wang, D. Li, G. Xian, C. Li, Effect of immersion in water or alkali solution on the structures and properties of epoxy resin, *Polymers (Basel)*. 13 (2021). <https://doi.org/10.3390/polym13121902>.
- [33] J.M. Sousa, J.R. Correia, S. Cabral-Fonseca, Durability of an epoxy adhesive used in civil structural applications, *Constr. Build. Mater.* 161 (2018) 618–633. <https://doi.org/10.1016/j.conbuildmat.2017.11.168>.
- [34] T. Gates, The physical and chemical ageing of polymeric composites, in: *Ageing Compos.*, 2008: pp. 3–33. <https://doi.org/10.1533/9781845694937.1.3>.
- [35] S. Sugiman, A.D. Crocombe, I.A. Aschroft, Experimental and numerical investigation of the static response of environmentally aged adhesively bonded joints, *Int. J. Adhes. Adhes.* 40 (2013) 224–237. <https://doi.org/10.1016/j.ijadhadh.2012.08.007>.
- [36] A. Dushimimana, A.P. Vassilopoulos, J. Sena-Cruz, J.M. Pereira, L. Correia, S. Cabral-Fonseca, R. Cruz, Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-

- controlled environments, *Constr. Build. Mater.* 438 (2024).
<https://doi.org/10.1016/j.conbuildmat.2024.137201>.
- [37] Simulia, ABAQUS computer software and Online Documentation, v6.12;, 2012.
- [38] Y. Tao, J.F. Chen, Concrete Damage Plasticity Model for Modeling FRP-to-Concrete Bond Behavior, *J. Compos. Constr.* 19 (2015) 1–13. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000482](https://doi.org/10.1061/(asce)cc.1943-5614.0000482).
- [39] I. Krucinska, T. Stypka, Direct measurement of the axial poisson's ratio of single carbon fibres, *Compos. Sci. Technol.* 41 (1991) 1–12. [https://doi.org/10.1016/0266-3538\(91\)90049-U](https://doi.org/10.1016/0266-3538(91)90049-U).
- [40] H. Wang, J. Lin, Y. Yu, X. Zuo, Y. Liu, H. Ding, H. Wang, Y. Bi, Moisture Absorption Characterization and Mechanical Properties of CFRP Under the Combined Effects of Seawater and Continuous Bending Stress, *Appl. Compos. Mater.* (2024). <https://doi.org/10.1007/s10443-024-10254-9>.
- [41] A. Dushimimana, J. Sena-Cruz, A.P. Vassilopoulos, L. Correia, J.M. Pereira, Modelling of short- and long-term bond behavior in concrete structures strengthened with externally bonded CFRP composites, in: C. BINETRUY, F. JACQUEMIN (Eds.), *Proc. 21st Eur. Conf. Compos. Mater. Vol. 3 - Mater. Struct. Behav. – Simul. Test.*, The European Society for Composite Materials (ESCM) and the Ecole Centrale de Nantes, Nantes, France, 2024: p. 683. <https://doi.org/10.60691/yj56-np80>.
- [42] M. Bordes, P. Davies, J.Y. Cognard, L. Sohier, V. Sauvart-Moynot, J. Galy, Prediction of long term strength of adhesively bonded steel/epoxy joints in sea water, *Int. J. Adhes. Adhes.* 29 (2009) 595–608. <https://doi.org/10.1016/j.ijadhadh.2009.02.013>.
- [43] S. Chataigner, J.F. Caron, K. Benzarti, M. Quiertant, C. Aubagnac, Use of a single lap shear test to characterize composite-to-concrete or composite-to-steel bonded interfaces, *Constr. Build. Mater.* 25 (2011) 468–478. <https://doi.org/10.1016/j.conbuildmat.2010.08.009>.
- [44] Y.T. Obaidat, S. Heyden, O. Dahlblom, Evaluation of Parameters of Bond Action between FRP and Concrete, *J. Compos. Constr.* 17 (2013) 626–635. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000378](https://doi.org/10.1061/(asce)cc.1943-5614.0000378).
- [45] X.Z. Lu, J.G. Teng, L.P. Ye, J.J. Jiang, Bond-slip models for FRP sheets/plates bonded to concrete, *Eng. Struct.* 27 (2005) 920–937. <https://doi.org/10.1016/j.engstruct.2005.01.014>.

PAPER VI

SHORT-TERM FLEXURAL BEHAVIOR OF RC SLABS WITH CFRP SYSTEMS

REFERENCE: *Aloys Dushimimana, José Sena-Cruz, Luís Correia, João M. Pereira. Analytical approaches for flexural analysis of RC slabs strengthened with CFRP laminates. To be submitted to a journal.*

ABSTRACT: Existing analytical predictions for flexural behavior of one-way reinforced concrete (RC) slabs strengthened with carbon fiber reinforced polymer (CFRP) laminates are generally formulated without considering the prevailing failure mode. This work contributes towards developing analytical approaches that attempt to fill this gap. The developed approaches address different strengthening techniques, namely, externally bonded reinforcement (EBR), near-surface mounted (NSM), and EBR prestressed by either mechanical anchorage (MA) or gradient anchorage (GA) systems. A trilinear moment-curvature relationship is established in which the governing equations incorporate the prevailing failure mode and initial strain-curvature dependence

on the deflection for prestressed systems. In addition, different existing formulations for predicting the FRP failure strain are applied and compared. The results from the developed approaches show good agreement with those from experimental tests.

KEYWORDS: Carbon fiber, Flexural strengthening, Analytical modeling, FRP debonding, FRP rupture

1. INTRODUCTION

Strengthening of existing concrete structures with carbon fiber reinforced polymer (CFRP) composites has been one of the most effective approaches to increase their load-carrying capacity and the lifespan (El-Hacha et al., 2001) (Correia et al., 2015) (Dias et al., 2018). This is mainly because CFRP composites have several inherent benefits including high stiffness-to-weight ratio, excellent creep and fatigue response, and corrosion resistance, among others. Different strengthening techniques are currently available, the most common being the externally bonded reinforcement (EBR) and near-surface mounted (NSM). These techniques can be prestressed, mainly to increase the benefits resulting from the full utilization of the CFRP strain, through mechanical anchorage (MA) or gradient anchorage (GA) systems (Correia et al., 2015).

Regarding the EBR technique, fiber reinforced polymer (FRP) composites, e.g., the CFRP, are bonded on the tensile face of the element to be strengthened, and the adhesion between the FRP and concrete substrate generally controls the ultimate capacity of the strengthened element (Bilotta et al., 2013). However, the premature failure due to debonding will lead to underutilization of the materials (Torabian et al., 2020) and to reduction of ductility (Bilotta et al., 2013). Experimental studies show that debonding can occur either at the end of the strengthened element which is known as end debonding (ED), or intermediate crack debonding (ICD) (Bilotta et al., 2013), (Ferracuti et al., 2007). According to *fib*90 (CEB-FIP, 2019), the ED occurs at the curtailment region of the FRP reinforcement that is not sufficiently anchored, while the ICD occurs at an intermediate section along the strengthened beam by bridging flexural or flexural-shear cracks.

Regarding NSM technique, pre-cut grooves opened on the concrete cover of the element to be strengthened are created in which the FRP bars or strips are inserted and bonded to concrete using an appropriate groove filler (Barros et al., 2007), (Bilotta et al., 2015). NSM technique is an effective solution to increase cracking, yielding and ultimate loads of strengthened elements failing in bending (Dias et al., 2018). Experimental studies have revealed that NSM has superior benefits than EBR (El-Hacha & Riskalla, 2004), (Barros et al., 2007), (Bilotta et al., 2015). This better performance is attributed to a better FRP-concrete bond which leads to a more efficient

use of FRP (Bianco et al., 2009). It has been found that the failure of an element strengthened using NSM can be critical diagonal crack debonding (CDCD) or a combination of the CDCD with concrete cover separation (Bilotta et al., 2015). According to a database presented in a study by Zhang et al. (Zhang et al., 2014), the debonding occurred due to cohesion failure (i.e., cracking) in the concrete adjacent to the adhesive layer in 51 tests from different seven experimental studies in which the specimens had the following characteristics: (i) a single CFRP NSM strip, (ii) the height to thickness ratio greater than 1, and (iii) concrete edge distance sufficiently large to prevent negative effects on the bond strength.

The use of prestressed FRP reinforcement is convenient or required in some structural applications (Barris et al., 2018). When EBR CFRP is prestressed, its benefits are combined with those of external prestressing (Sena-Cruz et al., 2015). In fact, the onset crack initiation is delayed and internal steel reinforcement strains are relieved (El-Hacha et al., 2001), besides, higher FRP failure strain can be reached owing to initial prestress (Sena-Cruz et al., 2015). In order to prevent the premature FRP failure by peeling-off, special anchorages are frequently used, e.g., (Chen & Teng, 2001), (Oehlers et al., 2016). Various studies exist on the use of the aforementioned MA, e.g., (Garden & Hollaway, 1998), (Wu & Huang, 2008) and GA, e.g., (Michels et al., 2013) (Sena-Cruz et al., 2015) systems. The former (MA) uses metallic plates at the ends of the FRP strips and is more effective than the latter (GA) owing to its ability to render the strengthened element more ductile (Sena-Cruz et al., 2015). Additionally, MA system can prevent the premature failure in RC elements strengthened in flexure using the EBR technique (You et al., 2012), (Correia et al., 2015).

On the other hand, EBR prestressed with GA systems involves a sector-wise accelerated curing of epoxy adhesive using heat, followed by a segment-wise prestress force release for each heated sector (Sena-Cruz et al., 2015), (Michels et al., 2013).

Relevant studies exist on the analytical prediction of the flexural behavior of RC elements strengthened with EBR, NSM, GA, and MA systems, e.g. (Badawi & Soudki, 2009) (Mostakhdemin Hosseini et al., 2020) (Rezazadeh et al., 2015) (Sena-Cruz et al., 2015) (Jung et al., 2017) (Deng et al., 2021); however, there are still some points that need to be further addressed. Firstly, the existing analytical predictions using the trilinear moment-curvature relationship e.g., (Mostakhdemin Hosseini et al., 2020) (Badawi & Soudki, 2009) are generally established without including the effect of the FM or the prevailing FM; however, a few studies partially attempt to include it, e.g., (Rezazadeh et al., 2015). In fact, the ultimate stage will mainly be affected by the

type of failure, and hence, there should be a formulation strategy for selecting the prevailing type of failure, from which the ultimate moment, curvature, and load can be estimated.

Secondly, the curvature used for the calculation of deflection may depend on the FM; mainly because the curvature depends on the neutral axis (NA) location that varies from one FM to another. Hence, the ultimate curvature when one type of FM occurs might differ from that when another type of FM occurs. In addition to this, for prestressed systems, the curvature at all stages may depend on the initial prestrain, mainly because of the initial camber resulting from the prestressing force. Thirdly, the existing literature lacks an analytical approach that attempts to use different existing deflection formulae at different loading stages, while this might improve the stiffness prediction. The fourth point is that different formulations are available for predicting the FRP failure strain, such as those in the code provisions CNR (CNR-DT 200 R1, 2013), ACI (ACI 440.2R-08, 2008) and *fib*90 (CEB-FIP, 2019), and others proposed by different authors, e.g., (Barros et al., 2007).

Studies that can attempt to present comparative perspectives on the above formulations may contribute to providing a better understanding on the reliability of each of these formulations. The present work contributes towards presenting analytical approaches that attempt to address the above four relevant points.

2. ANALYTICAL APPROACHES

2.1. Constitutive laws of materials

Concrete compressive behavior is assumed linear in both pre- and post-cracking stages up to the yielding of longitudinal steel to simplify the calculation procedure as in (Rezazadeh et al., 2015). The contribution of concrete under compression after the steel yielding is modelled by a rectangular stress block mainly governed by a multiplier on the concrete compressive strength to determine the intensity of the stress distribution (k_1) and the ratio of the equivalent rectangular stress block to the neutral axis depth (k_2), see **Figure VI.1a**, as per *fib*90 (CEB-FIP, 2019) using **Eq. (VI.1a)** and **(VI.1b)**, respectively.

$$k_1 = \begin{cases} 1000 \cdot \varepsilon_c \cdot (0.5 - (1000/12) \cdot \varepsilon_c) & \text{for } \varepsilon_c \leq 0.002 \\ 1 - (2/3000 \cdot \varepsilon_c) & \text{for } 0.002 \leq \varepsilon_c \leq 0.0035 \end{cases} \quad (\text{VI.1a})$$

$$k_2 = \begin{cases} (8 - 1000 \cdot \varepsilon_c)/4 \cdot (6 - 1000 \cdot \varepsilon_c) & \text{for } \varepsilon_c \leq 0.002 \\ (1000 \cdot \varepsilon_c \cdot (3000 \cdot \varepsilon_c - 4) + 2)/(2000 \cdot \varepsilon_c \cdot (3000 \cdot \varepsilon_c - 2)) & \text{for } 0.002 \leq \varepsilon_c \leq 0.0035 \end{cases} \quad (\text{VI.2b})$$

where ε_c is the concrete top fiber strain at a certain loading level. Furthermore, the tensile behavior of concrete is assumed linear up to a stress where its tensile surface starts to have initial cracks as per ACI (ACI-318M-08, 2008). The contribution of concrete in tension will be negligible after cracking stage (**Figure VI.1a**).

For the longitudinal steel bars, an elasto-perfectly plastic model is adopted as shown in **Figure VI.1b**. The CFRP laminate is modelled as linear up to its ultimate tensile strength as shown in **Figure VI.1c**.

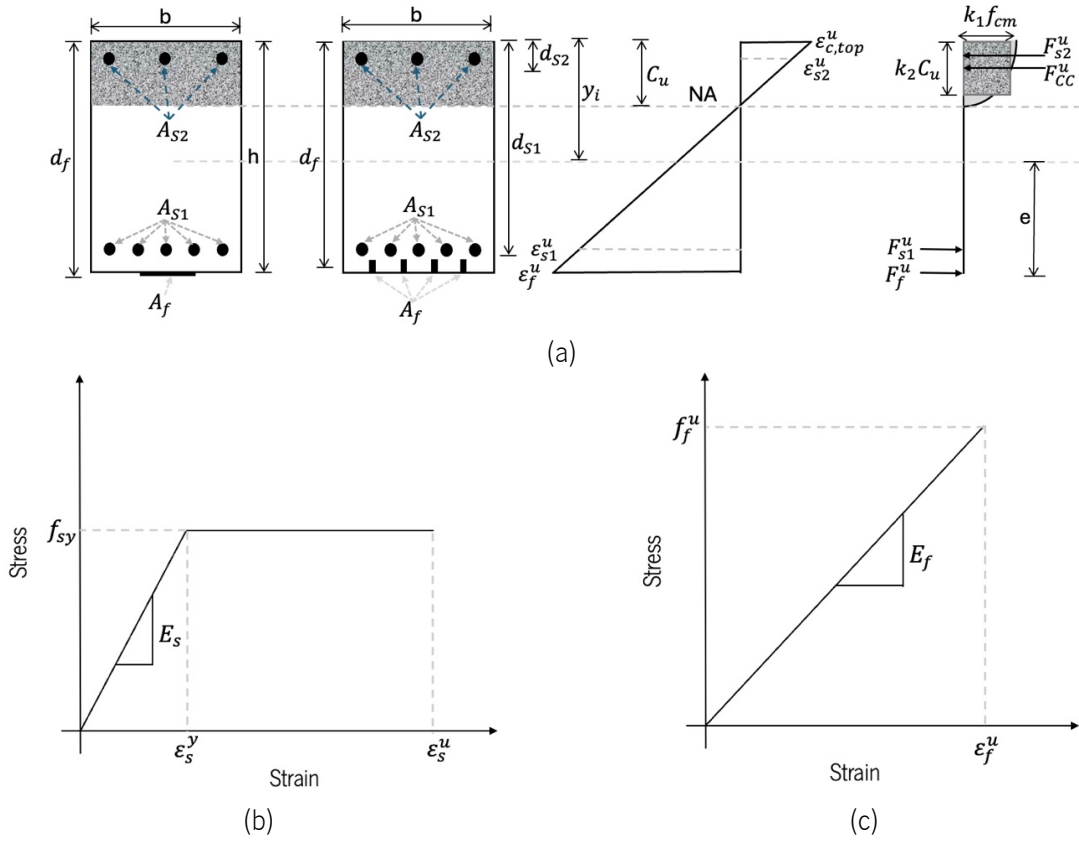


Figure VI.1. Constitutive laws for materials: (a) concrete, (b) steel, and (c) CFRP laminate. Note: the notations in these graphs are defined in the nomenclature provided just before the reference section.

2.2. Moment-curvature relationship

Using section analysis, a trilinear moment-curvature relationship ($M - \varphi$) for a cross section of an RC slab is established based on strain compatibility and force equilibrium for each of the governing loading stages (pre-cracking, post-cracking, and post-yield stages). The idealized $M - \varphi$ relationship is shown in **Figure VI.2**. The responses of interest are predicted at the end of each loading stage, namely, at cracking, yielding, and ultimate states, marked in **Figure VI.2** as I, II, and III, respectively. The obtained curvatures and moments at the states

I (φ_{cr}, M_{cr}), II (φ_y, M_y), and III (φ_u, M_u) are then used to determine the corresponding force-deflection responses. The $M - \varphi$ relation is constructed by incorporating four different types of failure modes, namely: (i) concrete crushing (CC), (ii) FRP rupture (FR), (iii) end debonding (ED), and (iv) intermediate crack debonding (ICD). To assess the prevailing FM among these failures, an approach is established with further details provided in Section 2.6.

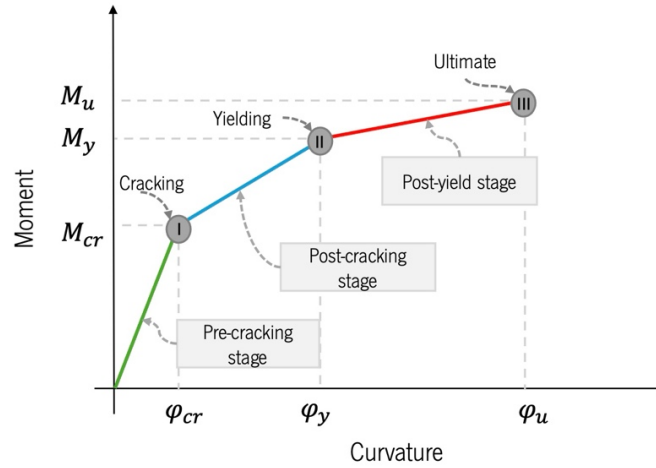


Figure VI.2. Idealized moment *versus* curvature relationship (with loading stages and states highlighted).

2.3. Flexural behavior of slabs

2.3.1. RC slab without strengthening system

The section and properties of the control (reference) RC slab are as shown in ANNEX VI.1 (Figure VI.A1). Referring to this annex, the curvature and moment at cracking ($\varphi_{cr,ns}, M_{cr,ns}$), yielding ($\varphi_{y,ns}, M_{y,ns}$), and ultimate ($\varphi_{u,ns}, M_{u,ns}$) states can be determined using Eqs. (VI.2)-(VI.3), Eqs. (VI.4)-(5), and Eqs. (VI.6)-(VI.7), respectively, with unknown terms being determined as shown in ANNEX VI.1.

$$\varphi_{cr,ns} = \varepsilon_{c,top}^{cr} / y_i \quad (VI.2)$$

$$M_{cr,ns} = (\sigma_c^{cr} \cdot b \cdot y_i^2) / 3 + (\sigma_{ct}^{cr} \cdot b \cdot (h - y_i)^2) / 3 + A_{s2} \cdot \sigma_{s2}^{cr} \cdot (y_i - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{cr} \cdot (d_{s1} - y_i) \quad (VI.3)$$

$$\varphi_{y,ns} = \varepsilon_{c,top}^y / C_y \quad (VI.4)$$

$$M_{y,ns} = (\sigma_c^y \cdot b \cdot C_y^2) / 3 + A_{s2} \cdot \sigma_{s2}^y \cdot (C_y - d_{s2}) + A_{s1} \cdot \sigma_{s1}^y \cdot (d_{s1} - C_y) \quad (VI.5)$$

$$\varphi_{u,ns} = \varepsilon_{c,top}^u / C_u \quad (VI.6)$$

$$M_{u,ns} = A_{s1} \cdot \sigma_{s1}^u \cdot (d_{s1} - k_2 \cdot C_u) + A_{s2} \cdot \sigma_{s2}^u \cdot (k_2 \cdot C_u - d_{s2}) \quad (VI.7)$$

In these equations, $\varepsilon_{c,top}^{i=cr,y,u}$, $\sigma_c^{i=cr,y,u}$, and $\sigma_{s1}^{i=cr,y,u}$ and $\sigma_{s2}^{i=cr,y,u}$ are the compressive strains in the concrete top fiber, stresses in the concrete under compression, and stresses in the bottom and top steel rebars at the

cracking (*cr*), yielding (*y*), and ultimate (*u*) states, respectively; σ_{ct}^{cr} is the stress in the concrete under tension at cracking state (i.e., $\sigma_{ct}^{cr} = \varepsilon_{ct} \cdot E_c$), where $\varepsilon_{ct} = \varepsilon_{c,bot}^{cr}$ is the in concrete bottom fiber tensile strain at cracking and E_c is the concrete modulus of elasticity. Furthermore, d_{s1} and d_{s2} , A_{s1} and A_{s2} , E_{s1} and E_{s2} are the distances from the center of the bottom and top steel rebars to the concrete top fiber, bottom and top steel section areas, and bottom and top steel moduli of elasticity, respectively; b is the section width and h is the section height. Also, y_i is the distance measured from the top concrete fiber to the centroidal axis of un-cracked section, C_y and C_u are the neutral axis depths measured from the concrete top fiber at yielding and ultimate states, respectively.

The failure mode (FM) can be either CC or the reaching of ultimate strain in the bottom or top steel rebars. Therefore, the prevailing FM is predicted by first assuming a CC failure (i.e. the concrete top fiber strain is assumed to have reached its ultimate value $\varepsilon_{cu} = 0.0035$) at which the strain levels in both top (ε_{s2}^u) and bottom (ε_{s1}^u) steel rebars are as follows:

$$\varepsilon_{s2}^u = \varepsilon_{cu} \cdot (C_u - d_{s2}) / C_u \quad (VI.8)$$

$$\varepsilon_{s1}^u = \varepsilon_{cu} \cdot (d_{s1} - C_u) / C_u \quad (VI.9)$$

If $\varepsilon_{s2}^u \leq f_{sy2} / E_{s2}$ and $\varepsilon_{s1}^u \leq f_{sy1} / E_{s1}$, then the above assumption prevails and CC occurred, otherwise, the failure will be governed by the rupture of steel rebars; f_{sy1} and f_{sy2} are the yield strengths for bottom and top steel rebars, respectively.

2.3.2. RC slab with EBR

The RC slab strengthened with EBR is modelled as shown in ANNEX VI.2 (Figure VI.A2), where the curvature and moment at cracking ($\varphi_{cr,s}$, $M_{cr,s}$), yielding ($\varphi_{y,s}$, $M_{y,s}$), and ultimate ($\varphi_{u,s}$, $M_{u,s}$) states are determined using Eqs. (VI.10)-(VI.11), Eqs. (VI.12)-(VI.13), and Eqs. (VI.14)-(VI.15), respectively, with unknown terms determined as shown in ANNEX VI.2.

$$\varphi_{cr,s} = \varepsilon_{c,top}^{crs} / y_s \quad (VI.10)$$

$$M_{cr,s} = (\sigma_c^{crs} \cdot b \cdot y_s^2) / 3 + (\sigma_{ct}^{crs} \cdot b \cdot (h - y_s)^2) / 3 + A_{s2} \cdot \sigma_{s2}^{crs} \cdot (y_s - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{crs} \cdot (d_{s1} - y_s) + A_f \cdot \sigma_f^{crs} \cdot (d_f - y_s) \quad (VI.11)$$

$$\varphi_{y,s} = \varepsilon_{c,top}^{ys} / C_{ys} \quad (VI.12)$$

$$M_{y,s} = (\sigma_c^{ys} \cdot b \cdot C_{yt}^2) / 3 + A_{s2} \cdot \sigma_{s2}^{ys} \cdot (C_{ys} - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{ys} \cdot (d_{s1} - C_{ys}) + A_f \cdot \sigma_f^{ys} \cdot (d_f - C_{ys}) \quad (VI.13)$$

$$\varphi_{u,s} = \varepsilon_{c,top}^{us} / C_{us} \quad (VI.14)$$

$$M_{u,s} = A_{s1} \cdot \sigma_{s1}^{us} \cdot (d_{s1} - k_2 \cdot C_{us}) + A_{s2} \cdot \sigma_{s2}^{us} \cdot (k_2 \cdot C_{us} - d_{s2}) + A_f \cdot \sigma_f^{us} \cdot (h - k_2 \cdot C_{us}) \quad (VI.15)$$

In these equations, $\varepsilon_{c,top}^{i=crs,ys,us}$, $\sigma_c^{i=crs,ys,us}$, $\sigma_{s1}^{i=crs,ys,us}$ and $\sigma_{s2}^{i=crs,ys,us}$, and $\sigma_f^{i=crs,ys,us}$ are the compressive strains in the concrete top fiber, stresses in the concrete under compression, stresses in the bottom and top steel rebars, and stresses in the CFRP at the cracking (*crs*), yielding (*ys*), and ultimate (*us*) states, respectively; σ_{ct}^{crs} is the stress in concrete under tension at cracking state (i.e., $\sigma_{ct}^{crs} = \varepsilon_{ct} \cdot E_c$), where $\varepsilon_{ct} = \varepsilon_{c,bot}^{crs}$ is the concrete bottom fiber tensile strain at cracking and E_c is the concrete modulus of elasticity. Furthermore, y_s is the distance from the top concrete fiber strain to the centroidal axis of un-cracked section, C_{ys} and C_{us} are the neutral axis depths measured from the concrete top fiber at yielding and ultimate states, respectively, and d_f is the distance from the center of the CFRP section to the concrete top fiber.

At the ultimate state, the ε_c from **Eq. (VI.1)** is calculated depending on the type of FM. If the failure is by CC, $\varepsilon_c = \varepsilon_{c,top}^{us} = \varepsilon_{cu}$; however, if the failure is by FR, $\varepsilon_c = \varepsilon_{c,top}^{us} = \varepsilon_f^{us} \cdot C_{us} / (h - C_{us})$. In the latter failure, the value of FRP strain (ε_f^{us}), which is required when determining the σ_f^{us} (i.e., $\sigma_f^{us} = \varepsilon_f^{us} \cdot E_f$, with E_f being the modulus of elasticity of the CFRP), will be equal to the manufacturer FRP ultimate strain (ε_{fu}) multiplied by a certain reduction factor (k_m) to include the effect of various influential factors (Balaguru et al., 2009). Hence, according to (Balaguru et al., 2009), the ε_f^{us} (denoted as ε_f^{FR} to indicate it is FRP strain at FR) can be calculated using **Eq. (VI.16)**, where the multiplier k_m depends on the unit stiffness ($nE_f t_f$) to account for possible FRP delamination as in **Eq. (VI.17)**:

$$\varepsilon_f^{FR} = k_m \cdot \varepsilon_{fu} \quad (VI.16)$$

$$k_m = \begin{cases} 1 - \frac{n \cdot E_f \cdot t_f}{2,400,000} & \text{for } n \cdot E_f \cdot t_f \leq 1,200,000 \\ \frac{600,000}{n \cdot E_f \cdot t_f} & \text{for } n \cdot E_f \cdot t_f > 1,200,000 \end{cases} \quad (VI.17)$$

On the other hand, if the failure is by ED or ICD, the value of $\varepsilon_f^{us} = \varepsilon_f^{ED}$ will be calculated as per *fib90* (CEB-FIP, 2019) using **Eq. (VI.18)** for the FRP strain at ED (ε_f^{ED}) or **Eq. (VI.19)** for the ε_f^{us} at ICD (ε_f^{ICD}), respectively.

$$\varepsilon_f^{ED} = f_f^{ED} / E_f \quad (VI.18)$$

$$\varepsilon_f^{ICD} = f_f^{ICD} / E_f \quad (VI.19)$$

Furthermore, the formulae recommended by CNR (CNR-DT 200 R1, 2013) for predicting the ε_f^{us} at ED (ε_f^{ED}) and at ICD (ε_f^{ICD}) can also be adopted as:

$$\varepsilon_f^{ICD} = K_q \cdot \sqrt{2 \cdot k_b \cdot 0.32 \cdot \sqrt{f_{ctm}} \cdot f_{cm} / t_f \cdot E_f} \quad (VI.20)$$

$$\varepsilon_f^{ED} = K_q \cdot \sqrt{2 \cdot k_b \cdot 0.63 \cdot \sqrt{f_{ctm}} \cdot f_{cm} / t_f \cdot E_f} \quad (VI.21)$$

According to ACI guide (ACI 440.2R-02, 2002), the ε_f^{ICD} can also be calculated as:

$$\varepsilon_f^{ICD} = \begin{cases} \frac{1 - E_f t_f}{60} \leq 0.9 \varepsilon_{fu} & \text{for } E_f t_f \leq 180,000 \text{ N/mm} \\ \frac{90,000}{60 E_f t_f} \leq 0.9 \varepsilon_{fu} & \text{for } E_f t_f > 180,000 \text{ N/mm} \end{cases} \quad (VI.22)$$

In the above equations, f_f^{ED} and f_f^{ICD} are the FRP stress at ED and ICD, respectively, calculated as $f_f^{ED} = \beta_1 \cdot \sqrt{2 \cdot E_f \cdot G_f / t_f}$ and $f_f^{ICD} = k_{crm} \cdot F_f^{ICD}$ using mean values as per *fib90* (CEB-FIP, 2019), where F_f^{ICD} is the mean value of the FRP force at ICD calculated as $F_f^{ICD} = k_{md} \cdot k_b \cdot \beta_1 \sqrt{2 \cdot E_f \cdot f_{cm}^{2/3} / t_f}$. Besides, f_{cm} and f_{ctm} are the compressive and tensile strengths of concrete respectively, G_f is the fracture energy calculated as $G_f = k^2 \cdot k_b^2 \cdot f_{cm}^{2/3}$; other parameters are: $k_{md} = 0.25$, $k_{crm} = 2.1$, $k = 0.25$, $K_q = 1$ for concentrated load and 1.25 for distributed load. Also, $\beta_1 = 1$ if $l_b > l_e$ otherwise, $\beta_1 = (l_b / l_e) \cdot (2 - (l_b / l_e))$, and $k_b = \sqrt{2 - (b_f / b) / 1 + (b_f / b)} \leq 1$, b_f is the FRP width; l_b is the bonded length, and l_e is the effective bond length i.e., $l_e = (\pi / k_b) \cdot \sqrt{E_f \cdot t_f / 8 \cdot f_{cm}^{2/3}}$. Based on the above formulations, the value of ε_f^{us} will depend on the type of FM as follows:

$$\varepsilon_f^{us} = \begin{cases} \varepsilon_{cu} \cdot (d_f - C_{us}) / C_{us} & \text{if failure is by CC} \\ \varepsilon_f^{FR} & \text{from eqs. (16) if failure is by FR} \\ \varepsilon_f^{ED} & \text{from eq. (18), or (21) if failure is by ED} \\ \varepsilon_f^{ICD} & \text{from eq. (19), or (20), or (22) if failure is by ICD} \end{cases} \quad (VI.23)$$

Similarly, the value of ε_c in Eq. (VI.1) at ultimate state, i.e., $\varepsilon_c = \varepsilon_{c,top}^{us}$ will also depend on the type of FM occurred as follows:

$$\varepsilon_{c,top}^{us} = \begin{cases} \varepsilon_{cu} & \text{if failure is by CC} \\ \varepsilon_f^{FR} \cdot C_{FR} / (d_f - C_{FR}) & \text{if failure is by FR} \\ \varepsilon_f^{ED} \cdot C_{ED} / (d_f - C_{ED}) & \text{if failure is by ED} \\ \varepsilon_f^{ICD} \cdot C_{ICD} / (d_f - C_{ICD}) & \text{if failure is by ICD} \end{cases} \quad (VI.24)$$

where, C_{FR} , C_{ED} , and C_{ICD} are the neutral axis depths calculated based on the strain compatibility requirements considering the values of ε_f^{FR} , ε_f^{ED} , and ε_f^{ICD} , respectively.

2.3.3. RC slab with NSM

RC slab strengthened with NSM is modelled as shown in ANNEX VI.3 (Figure VI.A3), from which the curvature and moment at cracking (φ_{cr,s^*} , M_{cr,s^*}), yielding (φ_{y,s^*} , M_{y,s^*}), and ultimate (φ_{u,s^*} , M_{u,s^*}) states are derived as Eqs. (VI.25)-(VI.26), Eqs. (VI.27)-(VI.28), and Eqs. (VI.29)-(VI.30), respectively, with the unknown terms being determined as shown in ANNEX VI.3.

$$\varphi_{cr,s^*} = \varepsilon_{c,top}^{crs^*} / y_{s^*} \quad (VI.25)$$

$$M_{cr,s^*} = (\sigma_c^{crs^*} \cdot b \cdot y_{s^*}^2) / 3 + (\sigma_{ct}^{crs^*} \cdot b \cdot (h - y_{s^*})^2) / 3 + A_{s2} \cdot \sigma_{s2}^{crs^*} \cdot (y_{s^*} - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{crs^*} \cdot (d_{s1} - y_{s^*}) + A_f \cdot \sigma_f^{crs^*} \cdot (d_f - y_{s^*}) \quad (VI.26)$$

$$\varphi_{y,s^*} = \varepsilon_{c,top}^{ys^*} / C_{ys^*} \quad (VI.27)$$

$$M_{y,s^*} = (\sigma_c^{ys^*} \cdot b \cdot C_{ys^*}^2) / 3 + A_{s2} \cdot \sigma_{s2}^{ys^*} \cdot (C_{ys^*} - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{ys^*} \cdot (d_{s1} - C_{ys^*}) + A_f \cdot \sigma_f^{ys^*} \cdot (d_f - C_{ys^*}) \quad (VI.28)$$

$$\varphi_{u,s^*} = \varepsilon_{c,top}^{us^*} / C_{us^*} \quad (VI.29)$$

$$M_{u,s^*} = A_{s1} \cdot \sigma_{s1}^{us^*} \cdot (d_{s1} - k_2 \cdot C_{us^*}) + A_{s2} \cdot \sigma_{s2}^{us^*} \cdot (k_2 \cdot C_{us^*} - d_{s2}) + A_f \cdot \sigma_f^{us^*} \cdot (h - k_2 \cdot C_{us^*}) \quad (VI.30)$$

In these equations, $\varepsilon_{c,top}^{i=crs^*,ys^*,us^*}$, $\sigma_c^{i=crs^*,ys^*,us^*}$, $\sigma_{s1}^{i=crs^*,ys^*,us^*}$ and $\sigma_{s2}^{i=crs^*,ys^*,us^*}$, and $\sigma_f^{i=crs^*,ys^*,us^*}$ are the strains in the concrete top compressive fiber, stresses in the concrete under compression, stresses in the bottom and top steel rebars, and stresses in the CFRP at the cracking (crs^*), yielding (ys^*), and ultimate (us^*) states, respectively; $\sigma_{ct}^{crs^*}$ is the stress in concrete under tension at cracking state (i.e., $\sigma_{ct}^{crs^*} = \varepsilon_{ct} \cdot E_c$), where $\varepsilon_{ct} = \varepsilon_{c,bot}^{crs^*}$ is the concrete bottom fiber tensile strain at cracking and E_c is the concrete modulus of elasticity. Furthermore, y_{s^*} is the distance from the top compressive concrete fiber to the centroidal axis of un-cracked section and d_f is the distance from the center of the CFRP section to the concrete top compressive fiber.

At the ultimate state, if the failure is by CC, $\varepsilon_c = \varepsilon_{c,top}^{us^*} = \varepsilon_{cu}$; if the failure is by FR, $\varepsilon_c = \varepsilon_{c,top}^{us^*} = \varepsilon_f^{us^*} \cdot C_{us^*} / (h - C_{us^*})$. The value of FRP strain at FR ($\varepsilon_f^{us^*} = \varepsilon_f^{FR}$) which is required when determining the $\sigma_f^{us^*}$ (i.e., $\sigma_f^{us^*} = \varepsilon_f^{us^*} \cdot E_f$), can be calculated as previously explained using Eq. (VI.16) (Balaguru et al., 2009). Also, according to (Barros et al., 2007), $\varepsilon_f^{us^*}$ at FR (ε_f^{FR}) can be calculated as:

$$\varepsilon_f^{FR} = (-32.648 \cdot \rho_{sl,eq} + 0.9606) \cdot \varepsilon_{fu} \quad (VI.31)$$

$$\rho_{sl,eq} = \frac{A_{s1}}{b \cdot d_{s1}} + \frac{(A_f \cdot E_f / E_{s1})}{b \cdot d_f} \quad (VI.32)$$

From ACI (ACI 440.2R-08, 2008), ε_f^{FR} is related to the design FRP failure strain (ε_{fd}) as:

$$\varepsilon_f^{FR} \leq \varepsilon_{fd} = 0.7 \cdot \varepsilon_{fu} \quad (VI.33)$$

In the scope of the present work, another formulation (see Eq. (VI.34)) is established after performing a parametric study in which the parameter p_1 was varied from 0 to 1 by using a small increment of 0.001 until the value leading to the ultimate load closest to the experimentally observed value was reached. The parameter p_1 is introduced for comparison purpose with the ACI, i.e., Eq. (VI.33).

$$\varepsilon_f^{FR} = p_1 \cdot \varepsilon_{fu} \quad (VI.34)$$

On the other hand, if the failure is by ED, the value of $\varepsilon_f^{us*} = \varepsilon_f^{ED}$ can be calculated as per *fib90* (CEB-FIP, 2019) where an empirical formulation based on the statistical regression of data from bond tests was tentatively proposed as shown in Eq. (VI.35), established using existing data mostly from RC beams, which might not be appropriately applied to the RC slabs. Nevertheless, the investigated RC slabs are of one-way type and hence can be expected to behave as RC beams.

$$\varepsilon_f^{ED} = a \cdot (\rho_{f,g})^c / (E_f A_f)^b \quad (VI.35)$$

In the above formulations, E_f, A_f are modulus of elasticity and cross-section area of FRP, respectively; $\rho_{sl,eq}$ is the equivalent reinforcement ratio, $\rho_{f,g}$ is the groove perimeter, the regression coefficients are : $a = 252$, $b = 0.823$, and $c = 0.66$. From these formulations, the value of ε_f^{us*} will depend on the type of FM as follows:

$$\varepsilon_f^{us*} = \begin{cases} \varepsilon_{cu} \cdot (d_f - C_{us*}) / C_{us*} & \text{if failure is by CC} \\ \varepsilon_f^{FR} & \text{from eqs. (16), or (31), or (33), or (34) if failure is by FR} \\ \varepsilon_f^{ED} & \text{from eq. (35) if failure is by ED} \end{cases} \quad (VI.36)$$

Similarly, the value of $\varepsilon_c = \varepsilon_{c,top}^{us*}$ will depend on the type of FM occurred as follows:

$$\varepsilon_{c,top}^{us*} = \begin{cases} \varepsilon_{cu} & \text{if failure is by CC} \\ \varepsilon_f^{FR} \cdot C_{FR} / (d_f - C_{FR}) & \text{if failure is by FR} \\ \varepsilon_f^{ED} \cdot C_{ED} / (d_f - C_{ED}) & \text{if failure is by ED} \end{cases} \quad (VI.37)$$

For more clarity, A MATLAB code is presented in **APPENDIX VI.1** to help the reader understand how the equations shown in this section can be used to predict the flexural behavior of the RC element strengthened using NSM technique.

2.3.4. RC slab with EBR prestressed using GA or MA system

The RC slab with GA or MA system is modelled as shown in **ANNEX VI.4 (Figure VI.A4)**, where the curvature and moment at the cracking ($\varphi_{cr,p}^T, M_{cr,p}$) and yielding ($\varphi_{y,p}^T, M_{y,p}$) states, and the moment at the ultimate ($M_{u,p}$) state can be derived as **Eqs. (38)-(39)**, **Eqs. (VI.40)-(VI.41)**, and **Eqs. (VI.42)**, respectively. The unknown terms can be determined as shown in **ANNEX VI.4**. Furthermore, the curvature at ultimate stage ($\varphi_{u,p}^T$) for prestressed systems can be estimated according to a new approach presented at the end of this section.

$$\varphi_{cr,p}^T = \varphi_{crp} + \varphi_{ip} \quad (\text{VI.38})$$

$$M_{cr,p} = \sigma_c^{crp} \cdot b \cdot y_p^2 / 3 + \sigma_{ct}^{crp} \cdot b \cdot (h - y_p)^2 / 3 + A_{s2} \cdot \sigma_{s2}^{crp} \cdot (y_p - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{crp} \cdot (d_{s1} - y_p) + A_f \cdot \sigma_f^{crp} \cdot (d_f - y_p) \quad (\text{VI.39})$$

$$\varphi_{y,p}^T = \varphi_{yp} + \varphi_{ip} \quad (\text{VI.40})$$

$$M_{y,p} = \sigma_c^{yp} \cdot b \cdot C_{yp}^2 / 3 + A_{s2} \cdot \sigma_{s2}^{yp} \cdot (C_{yp} - d_{s2}) + A_{s1} \cdot \sigma_{s1}^{yp} \cdot (d_{s1} - C_{yp}) + A_f \cdot \sigma_f^{yp} \cdot (d_f - C_{yp}) \quad (\text{VI.41})$$

$$M_{u,p} = A_{s1} \cdot \sigma_{s1}^{up} \cdot (d_{s1} - k_2 \cdot C_{up}) + A_{s2} \cdot \sigma_{s2}^{up} \cdot (k_2 \cdot C_{up} - d_{s2}) + A_f \cdot \sigma_f^{up} \cdot (h - k_2 \cdot C_{up}) \quad (\text{VI.42})$$

In these equations, $\sigma_c^{i=crp,yp,up}$, $\sigma_{s1}^{i=crp,yp,up}$ and $\sigma_{s2}^{i=crp,yp,up}$, and $\sigma_f^{i=crp,yp,up}$ are the stresses in the concrete under compression, stresses in the bottom and top steel rebars, and stresses in the CFRP at the cracking (*crp*), yielding (*yp*), and ultimate (*up*) states, respectively; σ_{ct}^{crp} is the stress in concrete under tension at cracking state (i.e., $\sigma_{ct}^{crp} = \varepsilon_{ct} \cdot E_c$), where $\varepsilon_{ct} = \varepsilon_{c,bot}^{crp}$ is the in concrete bottom fiber tensile strain at cracking and E_c is the concrete modulus of elasticity. Furthermore, φ_{ip} is the curvature due to initial prestress, and y_p is the distance from the top concrete fiber strain to the centroidal axis of un-cracked section and d_f is the distance from the center of the CFRP section to the concrete top compressive fiber; $\varphi_{crp} = \varepsilon_{c,top}^{crp} / y_p$, $\varphi_{yp} = \varepsilon_{c,top}^{yp} / C_{yp}$ and $\varphi_{ip} = \varepsilon_{c,top}^{ci} / y_{ip}$, y_{ip} is the neutral axis depth from the extreme concrete top fiber calculated incorporating the effect of both the initial concrete top tensile fiber ($\varepsilon_{c,top}^{ci}$) and bottom compressive fiber ($\varepsilon_{c,bot}^{ci}$) strains due to the eccentricity (*e*) of the applied prestressing force (F_{pre}) in relation to the y_p , i.e., $y_{ip} = \varepsilon_{c,top}^{ci} \cdot h / (\varepsilon_{c,top}^{ci} + \varepsilon_{c,bot}^{ci})$, where the calculations of all the above unknown terms are presented in **ANNEX VI.4**. Furthermore, the

CFRP stresses are also calculated including the effect of prestrain (ε_{fp}), i.e., $\sigma_f^{crp} = (\varepsilon_f^{crp} + \varepsilon_{fp}) \cdot E_f$, $\sigma_f^{yp} = (\varepsilon_f^{yp} + \varepsilon_{fp}) \cdot E_f$, and $\sigma_f^{up} = \varepsilon_f^{up} \cdot E_f$, respectively.

At the ultimate state, the value of ε_f^{up} is determined using a similar procedure as that shown in **Section 4.2** for EBR technique. In addition to that, another formulation recommended by *fib90* (CEB-FIP, 2019) for systems with mechanical anchorages is used to determine the $\varepsilon_f^{up} = \varepsilon_f^{FR}$ at FR in relation to the ε_{fd} as:

$$\varepsilon_f^{FR} = \varepsilon_{fd} \leq 0.9 \cdot \varepsilon_{fu} \quad (\text{VI.43})$$

The ε_f^{FR} for MA system is also evaluated using **Eq. (VI.44)** by performing a parametric study in which the parameter p_2 is changed in a similar way as that used when determining p_1 in **Section 4.3**. The parameter p_2 is introduced for comparison purpose with the *fib90* recommendation in **Eq. (VI.43)**.

$$\varepsilon_f^{FR} = p_2 \cdot \varepsilon_{fu} \quad (\text{VI.44})$$

Therefore, depending on the type of the FM, the value of ε_f^{up} is calculated as:

$$\varepsilon_f^{up} = \begin{cases} \varepsilon_{cu} \cdot (d_f - C_{up}) / C_{up} - \varepsilon_{fp} & \text{if failure is by CC} \\ \varepsilon_f^{FR} \text{ from eqs. (16), or (43), or (44)} & \text{if failure is by FR} \\ \varepsilon_f^{ED} \text{ from eq. (18), or (21)} + \varepsilon_{fp} & \text{if failure is by ED} \\ \varepsilon_f^{ICD} \text{ from eq. (19), or (20), or (22)} + \varepsilon_{fp} & \text{if failure is by ICD} \end{cases} \quad (\text{VI.45})$$

Similarly, the value of $\varepsilon_c = \varepsilon_{c,top}^{up}$ will depend on the type of FM occurred as follows:

$$\varepsilon_{c,top}^{up} = \begin{cases} \varepsilon_{cu} & \text{if failure is by CC} \\ \varepsilon_f^{FR} \cdot C_{FR} / (d_f - C_{FR}) & \text{if failure is by FR} \\ \varepsilon_f^{ED} \cdot C_{ED} / (d_f - C_{ED}) & \text{if failure is by ED} \\ \varepsilon_f^{ICD} \cdot C_{ICD} / (d_f - C_{ICD}) & \text{if failure is by ICD} \end{cases} \quad (\text{VI.46})$$

As previously mentioned, a novel approach for determining the curvature for prestressed systems is introduced in this work. During the experimental test program, usually no devices are used to measure the curvature; however, it is known that the curvature is directly proportional to the deflection. The deflection values measured by LVDTs during the test can indirectly reveal the curvatures in each performed test. For prestressed systems, it is assumed that the initial prestress will cause the RC slab to bend upward, thereby presenting a negative camber with a negative curvature. This indicates that before the start of the test, there will be already a negative deflection that LVDTs will not be able to recognize as negative because LVDTs will consider the first

measurement as zero. Therefore, the final values recorded will consequently include the negative deflection. In fact, the measured deflection will be a summation of negative and positive deflections. The formulations to accurately predict such a total deflection require using the curvature formulae derived herein.

The formulations for predicting the curvature for RC slab strengthened with FRP prestressed using anchorage systems are established based on the strain compatibility requirement for a strain profile shown in **Figure VI.3a**, as well as its geometric transformations (**Figure VI.3b,c**). In the introduced approach, it is assumed that if the FM is by FR, the full usage of FRP strain is achieved thanks to the initial prestrain (ϵ_{fp}). That is, the prestrain is fully exploited and, hence, it contributes to rendering the slab possess a high ductility, thereby leading to a larger ultimate deflection. In this case, the contribution of ϵ_{fp} to the total ultimate curvature ($\phi_{u,p}^T$) should be calculated using the neutral axis (NA) depth ($C_{u,ip}$) rather than C_{up} as the use of the latter would lead to violation of strain compatibility requirements (**Figure VI.3a**); that is, $C_{u,ip} = \epsilon_{c,top}^{ci} \cdot h / (\epsilon_{c,top}^{ci} + \epsilon_{fp})$, $\phi_{u,p}^T = \epsilon_{c,top}^{ci} / C_{u,ip} + \epsilon_{c,top}^{up} / C_{up}$. For a more simplified case, the contribution of the initial prestrain can be related to the total section height (h) rather than $C_{u,ip}$ (**Figure VI.3b**); and therefore, $\phi_{u,p}^T$ is as shown in **Eq. (VI.47)** where the 1st term of this equation is derived from **Figure VI.3b**.

$$\phi_{u,p}^T = (\epsilon_{c,top}^{ci} + \epsilon_{fp}) / h + \epsilon_{c,top}^{up} / C_{up} \quad (VI.47)$$

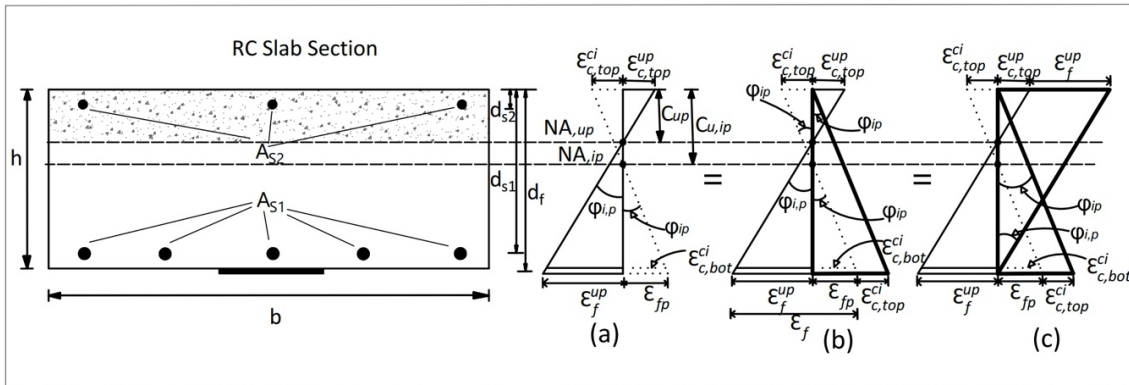


Figure VI.3. Prediction of curvature for prestressed systems: (a) strain profile, (b) and (c) geometric transformations of the strain profile shown in (a). Note: the transformations in (b) and (c) are established for the case of FRP rupture and debonding, respectively.

The total curvature when there is a FRP rupture can also be predicted as **Eq. (VI.48)**:

$$\varphi_{u,p}^T = \frac{k_2 \cdot \varepsilon_{cu}}{(0.5 \cdot A_f \cdot \sigma_f / k_1 \cdot f_{cm} \cdot b)} \quad (\text{VI.48})$$

Where the parameter σ_f can be calculated referring to He et al. (He et al., 2007) as Eq. (VI.49):

$$\sigma_f = \sqrt{\frac{(E_f \cdot \varepsilon_{cu})^2}{4} + \frac{(k_1 \cdot k_2 \cdot f_{cm}) \cdot E_f \cdot \varepsilon_{cu}}{\rho_f}} - E_f \cdot \varepsilon_{cu} / 2 \quad (\text{VI.49})$$

where $\rho_f = A_f / b \cdot h$.

On the other hand, it is assumed that if the failure is by debonding, the full usage of the FRP strain has not been achieved and the benefits of prestrain have not been fully exploited yet. Hence, the ability of the FRP (through prestrain) to supply a more ductile behavior to the RC slab is significantly lost, thereby leading to a reduced ultimate deflection. This thought agrees with the results from previous experimental tests (Sena-Cruz et al., 2015)(Correia et al., 2017)(Sena-Cruz et al., 2019), where a smaller ultimate deflection was observed due the occurrence of the ICD failure for the RC slab with GA system, as compared to the deflection recorded when FR failure occurred for RC slab with MA system. In this case, the contribution of initial curvature due to prestress can be determined using NA depth (C_{up}) at a reasonable accuracy (i.e. $C_{up} \cong C_{u,ip}$), but to be more accurate, the present work suggests the use of initial strains in both top and bottom fiber of concrete rather than initial prestrain (ε_{fp}) (Figure VI.3c). In fact, the use of ε_{fp} is avoided as it is not known whether the effect of ε_{fp} has been fully exploited at debonding stage, while it can be assured that initial tensile state of concrete strain is fully exploited. Therefore, a more appropriate prediction of the curvature due to prestress (φ_{ip}) at debonding can be proposed as: $\varphi_{ip} = \varepsilon_{c,top}^{ci} / C_{up}$ or $\varphi_{ip} = (\varepsilon_{c,top}^{ci} + \varepsilon_{c,bot}^{ci}) / h$ (see Figure VI.3c), and the total ultimate curvature as:

$$\varphi_{u,p}^T = \varphi_{ip} + \varepsilon_{c,top}^{up} / C_{up} \quad (\text{VI.50})$$

Another technique proposed in this work is to assume that debonding takes places at a certain value of concrete top fiber strain ($\varepsilon_{c,top}^{up}$), i.e., the total curvature at debonding is as in Eq. (VI.48) and Eq. (VI.49) but replacing the term ε_{cu} by $\varepsilon_{c,top}^{up}$ in both equations, hence, the total ultimate curvature is:

$$\varphi_{u,p}^T = \frac{k_2 \cdot \varepsilon_{c,top}^{up}}{(0.5 \cdot A_f \cdot \sigma_f / k_1 \cdot f_{cm} \cdot b)} \quad (\text{VI.51})$$

2.4. Prediction of mid-span deflections

Several approaches can be used to calculate the mid-span deflection. In the scope of this work, two different approaches are adopted; in addition, since one approach may possess greater benefits for a given loading stage than the other, the possibility of simultaneously adopting both approaches is assessed. The first approach is based on the integration in different loading stages as described in (Rezazadeh et al., 2015), herein modified broadly to incorporate various investigated strengthening techniques as follows:

$$\delta_{cr,j} = \int_0^{b_s} \left(\frac{\varphi_{cr,j}^T}{b_s} \right) \cdot x^2 \cdot dx + \int_{b_s}^{L/2} \varphi_{cr,j}^T \cdot x \cdot dx \quad (\text{VI.52a})$$

$$\begin{aligned} \delta_{y,j} = & \int_0^{L_{cr}^{y,j}} \left(\frac{\varphi_{cr,j}^T}{L_{cr}^{y,j}} \right) \cdot x^2 \cdot dx + \int_{L_{cr}^{y,j}}^{b_s} \left(\left(\frac{\varphi_{y,j}^T - \varphi_{cr,j}^T}{b_s - L_{cr}^{y,j}} \right) \cdot (x - L_{cr}^{y,j}) + \varphi_{cr,j}^T \right) \cdot x \cdot dx \\ & + \int_{b_s}^{L/2} \varphi_{y,j}^T \cdot x \cdot dx \end{aligned} \quad (\text{VI.52b})$$

$$\begin{aligned} \delta_{u,j} = & \int_0^{L_{cr}^{u,j}} \left(\frac{\varphi_{cr,j}^T}{L_{cr}^{u,j}} \right) \cdot x^2 \cdot dx + \int_{L_{cr}^{u,j}}^{L_y^u} \left(\left(\frac{\varphi_{y,j}^T - \varphi_{cr,j}^T}{L_y^u - L_{cr}^{u,j}} \right) \cdot (x - L_{cr}^{u,j}) + \varphi_{cr,j}^T \right) \cdot x \cdot dx \\ & + \int_{L_y^u}^{b_s} \left(\left(\frac{\varphi_{u,j}^T - \varphi_{y,j}^T}{b_s - L_y^u} \right) \cdot (x - L_y^u) + \varphi_{y,j}^T \right) \cdot x \cdot dx + \int_{b_s}^{L/2} \varphi_{u,j}^T \cdot x \cdot dx \end{aligned} \quad (\text{VI.52c})$$

where $\delta_{cr,j}$, $\delta_{y,j}$, $\delta_{u,j}$ are the deflections at cracking (Eq. VI.52a), yielding (Eq. VI.52b), and ultimate (Eq. VI.52c) states, respectively. In Eq. (VI.52), j can be ns , s or p standing for non-strengthened (i.e., reference RC slab), strengthened (i.e., RC slab with EBR or NSM), or prestressed case (i.e., RC slab with EBR prestressed via MA or GA), respectively; the subscript cr , y , or u stands for “cracking”, “yielding” and “ultimate” states, respectively; φ and L are the curvature and span length, respectively; a superscript “ T ” is used to indicate the “total” curvature. Therefore, a combination such as $\varphi_{cr,s}^T$ means the total curvature at cracking state for strengthened element (e.g., EBR); $\delta_{y,p}$ means the deflection at yielding state for prestressed element. On the other hand, $L_{cr}^{y,j}$ is the length of the pre-cracking region at the steel yielding initiation state, $L_{cr}^{u,j}$ is the length of the pre-cracking region at the ultimate state, and L_y^u is the distance of the curvature at yielding defined along the strengthened element length at the ultimate stage. These lengths are related to the shear span length (b_s) and moments in each loading stage as: $L_{cr}^{y,j} = b_s \cdot M_{cr,j} / M_{y,j}$, $L_y^u = b_s \cdot M_{y,j} / M_{u,j}$ and $L_{cr}^{u,j} = b_s \cdot M_{cr,j} / M_{u,j}$. More relevant details can be found in (Rezazadeh et al., 2015).

The second prediction is based on the calculation of deflection only at determinant section as previously adopted in (Sharaky et al., 2015):

$$\delta_{i,j} = \frac{(3 \cdot L^2 - 4 \cdot b_s^2)}{24} \cdot \varphi_{i,j}^T \quad (\text{VI.53})$$

where i can be cr, y, u, j which are as previously defined in Eq. (VI.52). It is worth noting that $\varphi_{i,j}^T$ in Eq. (VI.53) depends on the current concrete top fiber strain and neutral axis depth for non-prestressed systems, while for the prestressed systems it is calculated as described in Section 2.3.4. Another formula for determining the curvature considering the uncracked and fully cracked states was used in (Silva, 2017) as shown in Eq. (VI.54).

$$\varphi = (1 - \xi) \cdot k_I + \xi \cdot k_{II} \quad (\text{VI.54})$$

where k_I and k_{II} are the curvatures that correspond to the bending moment of the section assuming uncracked and cracked states respectively, and ξ is the interpolation coefficient that depends on the cracking moment, applied moment and some coefficients. However, for a four-point bending test configuration, Eq. (VI.53) can be assumed accurate, because the curvature is constant in the load span where the cracks are expected to dominate in the fully cracked state. That is, the curvature dependency on the fully cracked state is negligible, hence Eq. (VI.53) is preferred. Most importantly, when interested solely in predicting the responses at the end of each loading stage (like in the present work), where the strains at each stage are already known from strain compatibility requirements, Eq. (VI.54) becomes unnecessary, as the known strains can be directly used to determine the corresponding curvature and deflection. However, Eq. (VI.54) can be very crucial when determining responses other than those at the end of each stage, where the strains in the materials are normally unknown.

2.5. Prediction of cracking, yielding, and ultimate loads

The cracking, yielding or ultimate load based on the maximum bending moment ($M_{i,j}$) is calculated as:

$$P_{i,j} = 4 \cdot M_{i,j} / (L - a_s \cdot L) \quad (\text{VI.55})$$

Where $P_{i,j}$ is the load calculated from the corresponding $M_{i,j}$; i and j are as previously defined in Eq. (VI.53), and a_s is the distance from the center of the support to the start of the FRP. On the other hand, if the FM is ED, the failure load (P_{ED}) will be calculated as $P_{ED} = 2 \cdot M_{ED} / (a_s + l_e)$, where the moment (M_{ED}) is

calculated using ε_f^{ED} selected referring to Eq. (VI.23), or (VI.36) for non-prestressed system, and as Eq. (VI.45) for prestressed system.

2.6. Determining the prevailing failure mode

To predict the prevailing failure mode (FM), different failures (CC, FR, ED, and ICD as previously described) are checked. To assess the prevailing FM among these four investigated different possible failures, all failure loads at CC, FR, ED, and ICD are calculated and then compared. The smallest load is the one that indicates the prevailing FM, as can be seen in Figure VI.4.

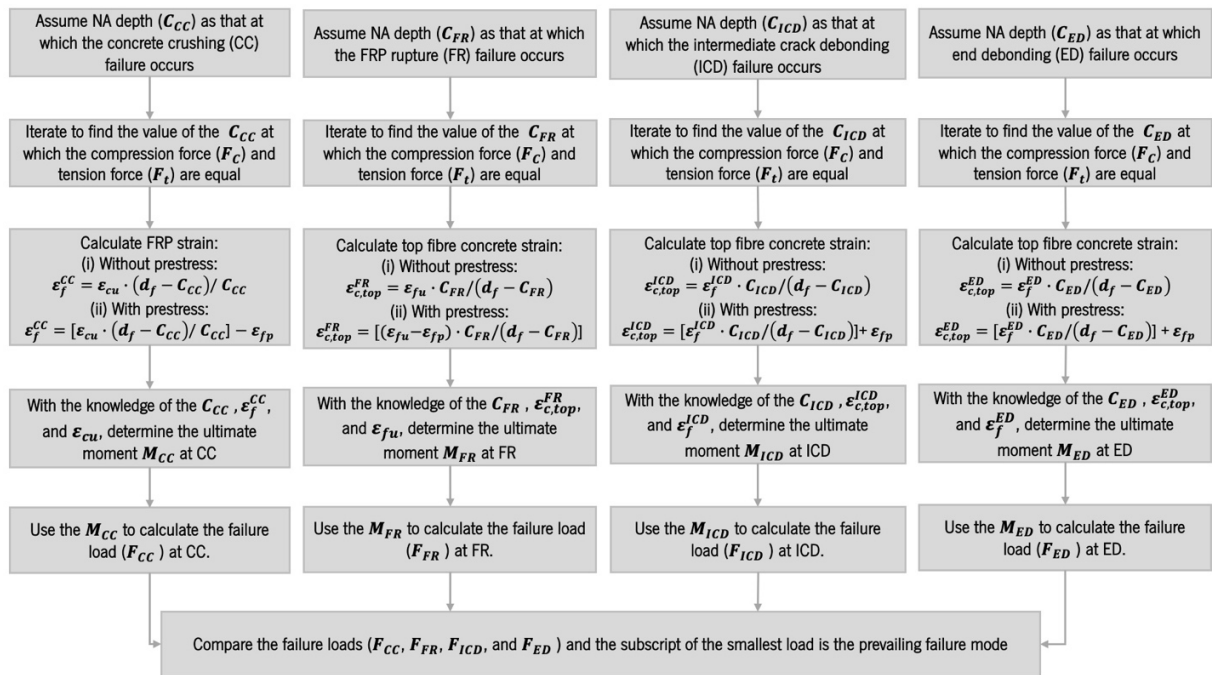


Figure VI.4. Summary of the methodology used for predicting the prevailing failure mode

2.7. Assessment of the performance of the analytical approach

The validity of the analytical approaches developed in the present work is assessed using experimental results for RC slabs tested at the laboratory of civil engineering (LEST) of University of Minho. Five different RC slabs (i.e., the control RC slab, RC slab with EBR, RC slab with NSM, RC slab with EBR prestressed via MA, and RC slab with EBR prestressed via GA, denoted as SL_REF_T0, SL_EBR_T0, SL_NSM_T0, SL_MA_T0, and SL_GA_T0, respectively) were tested following a 4-point bending test configuration (see Figure VI.5a).

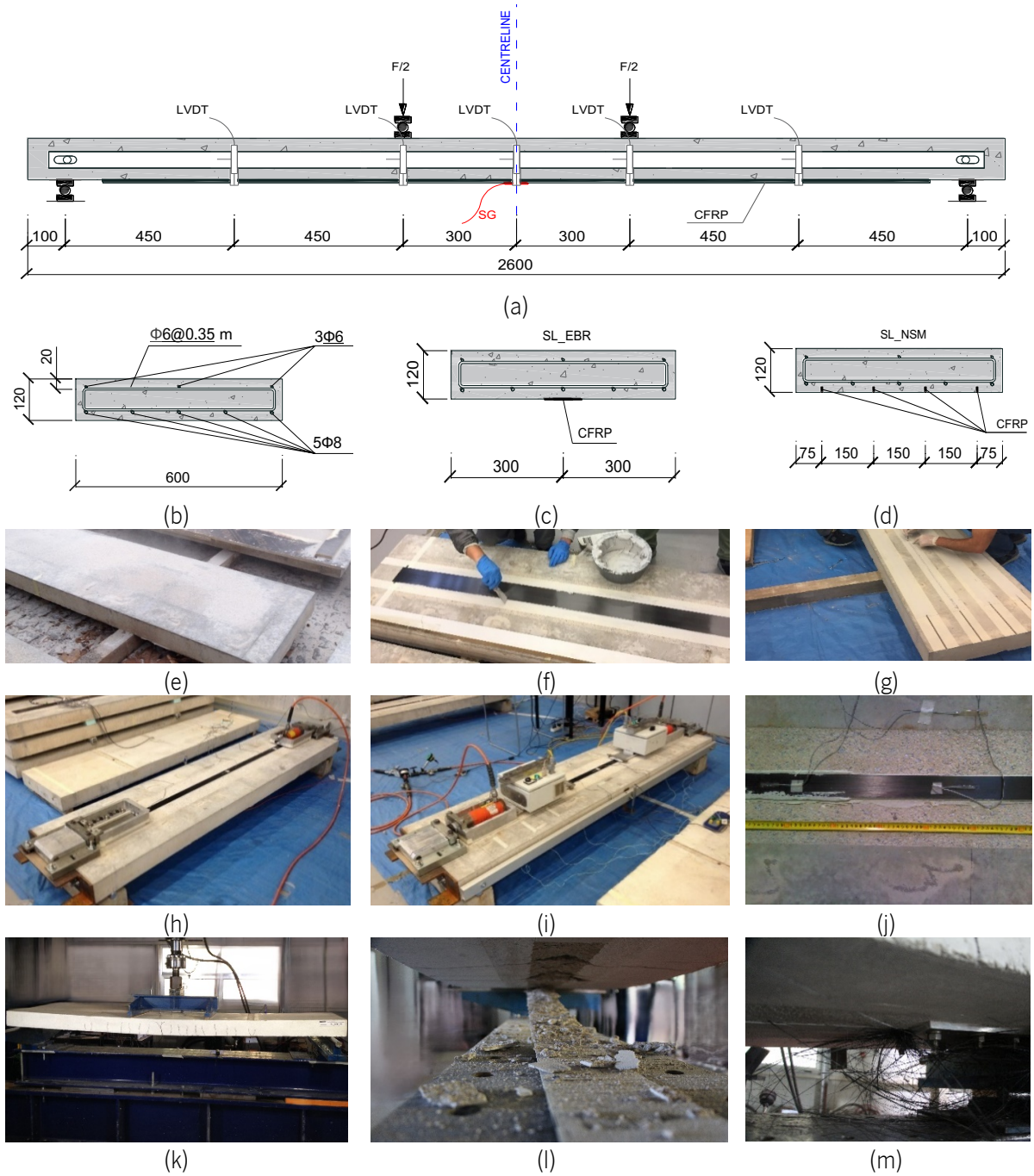


Figure VI.5. Description of the RC slabs: (a) geometry and dimensions for slabs with EBR under 4-point bending test configuration, cross-section of (b) reference slab, (c) slab with prestressed or non-prestressed EBR, and (d) slab with NSM; (e) preparation of reference slab specimen; application of CFRP strengthening system via (f) EBR, (g) NSM, (h) EBR prestressed via MA, (i) EBR prestressed via GA; (j) attachment of strain measurement devices, (k) typical 4-point bending test for all slabs; typical (l) ICD and (m) FR failure modes.

The concrete used for casting all these slabs had a 28-day compressive strength of 41.5MPa and elastic modulus of 29.1 GPa; the properties of the steel bars and CFRP strips used for the slabs are given in **Table VI.1**. More details on the properties of materials constituting the RC slabs can be found in (Dushimimana, Sena-Cruz, et al., 2024), (Dushimimana, Sena-cruz, et al., 2024), (Dushimimana, Vassilopoulos, et al., 2024), and (Sena-Cruz et al., 2019). All types of preparation of RC slab specimens prior to testing are shown in **Figure VI.5e-j**, and **Figure VI.5k** shows the test set-up, and the typical observed failure modes can be depicted in **Figure VI.5l,m**.

Table VI.1. Characteristics of the investigated RC slabs

Slab	CFRP strip [mm ²]	AT	ε_{fp} [%]	E_s [GPa]		f_y [MPa]		f_t [MPa]		E_f [MPa]	f_f [GPa]
				$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$		
SL_REF_TO	-	-	-	219.9	209.3	543.9	551.2	670.0	635.4	-	-
SL_EBR_TO	100×1.2	-	-	(1.6)	(7.7)	(1.8)	(0.8)	(0.9)	(0.5)	187.2	2895.2
SL_NSM_TO	4× (10×1.4)	-	-	241.4	228.1	528.2	581.3	687.1	698.3	185.0	2858.2
SL_MA_TO	50×1.2	MA	0.4	(5.3)	(3.2)	(0.7)	(0.6)	(0.7)	(0.3)	(1.5)	(2.0)
SL_GA_TO	50×1.2	GA		219.9	209.3	543.9	551.2	670.0	635.4	184.3	2980.3
				(1.6)	(7.7)	(1.8)	(0.8)	(0.9)	(0.5)	(2.9)	(4.1)

Notes: values in parentheses are coefficient of variation in %, AT: anchorage type, MA: mechanical anchorage, GA: gradient anchorage, E_s and E_f , and f_t and f_f are elastic moduli and ultimate tensile strengths of steel and CFRP, respectively, ε_{fp} and f_y are initial prestrain and yield strength of CFRP and steel, respectively.

The main results from the experimental tests of all RC slabs are summarized in **Table VI.2**.

Table VI.2. Summary of the main results from the experimental flexural tests.

Specimen	Cracking state		Yielding state		Ultimate state		FM
	δ_{cr}	P_{cr}	δ_y	P_y	δ_u	P_u	
SL_REF_TO	1.17	12.30	19.58	28.82	120.47	30.96	CC
SL_EBR_TO	1.32	12.63	24.64	53.22	41.23	66.47	ICD
SL_NSM_TO	2.41	14.44	22.28	39.94	74.44	62.28	FR
SL_MA_TO	2.00	20.01	23.26	52.04	38.27	59.98	ICD
SL_GA_TO	2.04	20.41	22.19	49.69	79.24	67.62	FR

Notes: δ_{cr} and P_{cr} , δ_y and P_y , δ_u and P_u , are deflections and loads at cracking, yielding, and ultimate states, respectively, FM: failure mode i.e., CC, ICD, and FR as previously defined.

3. RESULTS AND DISCUSSION

3.1. Prediction of the prevailing failure mode

Using the method described in **Figure VI.4**, the failure loads for all the four possible FMs are presented in **Table VI.3**. These values were compared for each RC slab to determine the smallest load (prevailing FM), which is

compared to that observed experimentally (Table VI.2). The analytically obtained FMs agree well with those from experiment, hence demonstrating the capability of the established method. That is, the prevailing FM for EBR, NSM, GA are ICD, FR, and ICD, respectively. However, for the RC slab with EBR prestressed using MA systems, the FM did not match with that observed during experiment test. As will be shown in the load-deflection curve, this RC was supposed to fail by ICD because the load at which the CFRP started debonding is nearly the same as the one calculated from the developed approach at ICD debonding (Table VI.3). However, the mechanical anchorages prevented complete debonding as they became active just after the debonding. Hence, the premature debonding failure was prevented by these anchorage systems, thereby increasing both the failure load and ductility. This is in agreement with previous findings by (You et al., 2012)(Correia et al., 2015).

Table VI.3. Analytically predicted failure loads (in kN) for all the investigated failure modes

Slab	P_{CC}	P_{FR}				P_{ICD}			P_{ED}	
		BL	BR	FIB	PW	FIB	CNR	ACI	FIB	CNR
SL_EBR_TO	122.15	106.97	-	-	-	65.54	64.44	64.87	330.64	299.74
SL_NSM_TO	89.51	66.35	61.90	-	62.07 ^a	-	-	-	415.77	-
SL_MA_TO	85.08	68.43	-	68.09	67.20 ^b	58.78	57.59	57.39	362.87	329.34
SL_GA_TO	85.08	68.62	-	-	-	58.92	57.73	57.49	364.37	330.46

Notes: P_{CC} , P_{FR} , P_{ICD} , and P_{ED} , are loads at CC, FR, ICD, and ED failure modes, respectively, FIB: Eq. (VI.19) for non-prestressed EBR as per (CEB-FIP, 2019) or Eq. (VI.19) + ϵ_{fp} for GA system, PW: Eq. (VI.44), BL: Eq. (VI.16), BR: Eq. (VI.31), CNR: Eq. (VI.20) for EBR and Eq. (VI.20) + ϵ_{fp} for GA system. ACI: Eq. (VI.22) for EBR or Eq. (VI.22) + ϵ_{fp} for GA system, ^{a,b}: ϵ_f^{FR} is calculated as Eq. (VI.34) and Eq. (VI.44), respectively.

3.2. Flexural responses of slabs

3.2.1. RC slab without strengthening system

The main results for the control RC slab are presented in Figure VI.6. To compare the effectiveness of the two deflection formulae adopted, force-deflection relationships are plotted in Figure VI.6a,b, where an X_Y notation is used in the legend with X standing for the type of the RC slab (i.e., REF for reference RC slab) and Y standing for the adopted deflection formula (i.e., Eq. (VI.52) denoted as R and Eq. (VI.53) as P).

It can be noted that the developed analytical approach can predict the experimentally observed results. Comparing both deflection formulae (Figure VI.6a), a better prediction can be achieved when Eq. (VI.52) is adopted rather than Eq. (VI.53).

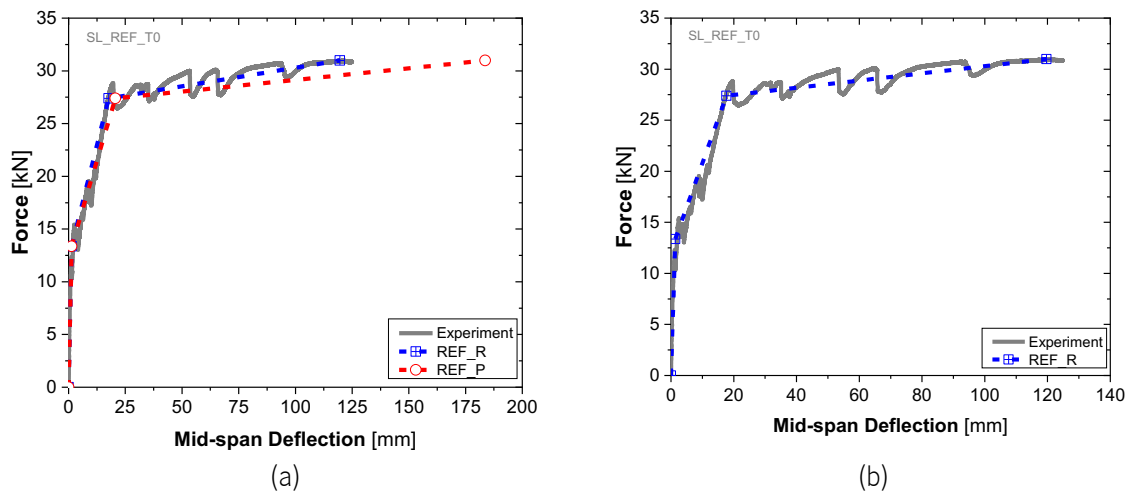


Figure VI.6. Reference RC slab: force *versus* deflection relations for (a) all studied cases and (b) selected case

3.2.2. RC slab with EBR technique

The force-deflection relations for the RC slab with EBR technique are plotted in Figure VI.7a,b. An X_Y notation is used in the legend, where X stands for the standard used for predicting the CFRP failure strain, while Y stands for the deflection formula adopted (i.e., R and P for Eq. (VI.52) and Eq. (VI.53)). All the attempts performed seem to be promising. It can be noted that either Eq. (VI.52) or Eq. (VI.53) can be adopted to predict responses in all loading stages. Besides, either Eq. (VI.19) from *fib90* (CEB-FIP, 2019), Eq. (VI.20) from CNR (CNR-DT 200 R1, 2013) or Eq. (VI.22) from ACI (ACI 440.2R-02, 2002), can be adopted to predict the FRP strain at debonding.

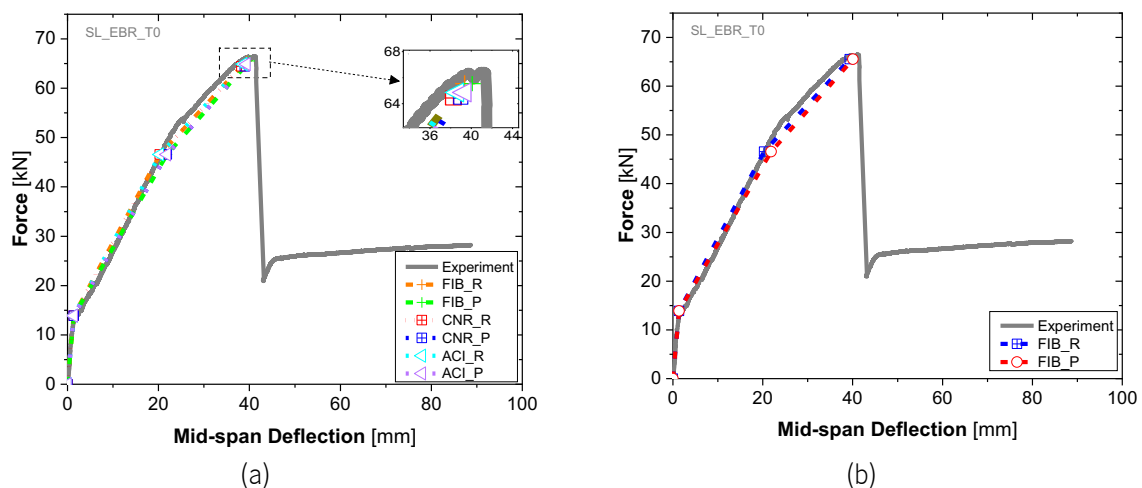


Figure VI.7. RC slab with EBR: force *versus* deflection for (a) all studied cases and (b) selected cases

3.2.3. RC slab with NSM

The force-deflection relations for the RC slab strengthened according to the NSM technique are plotted in **Figure VI.8a,b**. An X_Y notation is used in the legend, where X stands for the type of CFRP failure strain prediction (i.e., ACI, BL, BR, and PW, stand for **Eq. (VI.33)** as per ACI (ACI 440.2R-08, 2008), **Eq. (VI.16)** from (Balaguru et al., 2009), **Eq. (VI.31)** from (Barros et al., 2007), and **Eq. (VI.34)** with the optimal parameter $p_1 = 0.8$, respectively), while Y stands for the deflection formula (i.e., R or P for **Eq. (VI.52)** or **Eq. (VI.53)**, respectively). For a better deflection and stiffness prediction in all cases, both **Eq. (VI.52)** and **Eq. (VI.53)** can be adopted at the cracking state, but both cannot equally be adopted at the remaining states (**Figure VI.8a**). In fact, for a more accurate prediction of the stiffness in the post-cracking stage, **Eq. (VI.52)** can be adopted, while the stiffness in the post-yield stage can be better predicted by **Eq. (VI.53)** as shown in **Figure VI.8b** (RP means using R at yielding and P at ultimate states). However, use of **Eq. (VI.53)** at yielding state seems better than **Eq. (VI.52)** but the difference is negligible, which leads to prioritizing **Eq. (VI.52)** because of its better stiffness prediction. The next step is to compare the ultimate failure load (P_u) resulting from different existing formulations for predicting FRP failure strain (ϵ_f). Looking at **Figure VI.8a**, better predictions can be found from **Eq. (VI.31)** by Barros et al. (Barros et al., 2007) and **Eq. (VI.34)**. Contrary, adoption of **Eq. (VI.33)** from ACI (ACI 440.2R-08, 2008) would underestimate the P_u while **Eq. (VI.16)** from Balaguru et al. (Balaguru et al., 2009) would overestimate it.

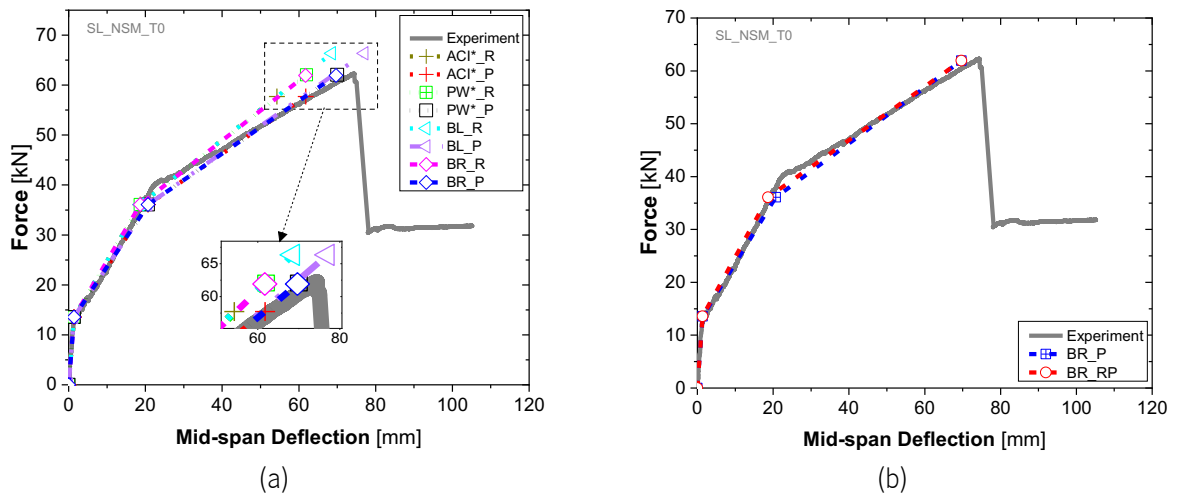


Figure VI.8. RC slab with NSM: (a) force *versus* deflection for all studied cases, and (b) force *versus* deflection for selected cases.

Hence, for a better deflection, stiffness and load prediction, the present work recommends Eq. (VI.52) (or Eq. (VI.53)) and Eq. (VI.53) when predicting the response at cracking and yielding states, respectively. Furthermore, combination of Eq. (VI.31), or Eq. (VI.34), with Eq. (VI.39) when predicting the response at ultimate state can be suggested.

3.2.4. RC slab with EBR prestressed using MA system

The effects of the proposed approach (Section 2.3.4) for determining the curvature of RC elements with prestressed CFRP systems are examined. The moment-curvature relationships for the studied RC slabs with EBR prestressed via MA systems are presented in Figure VI.9.

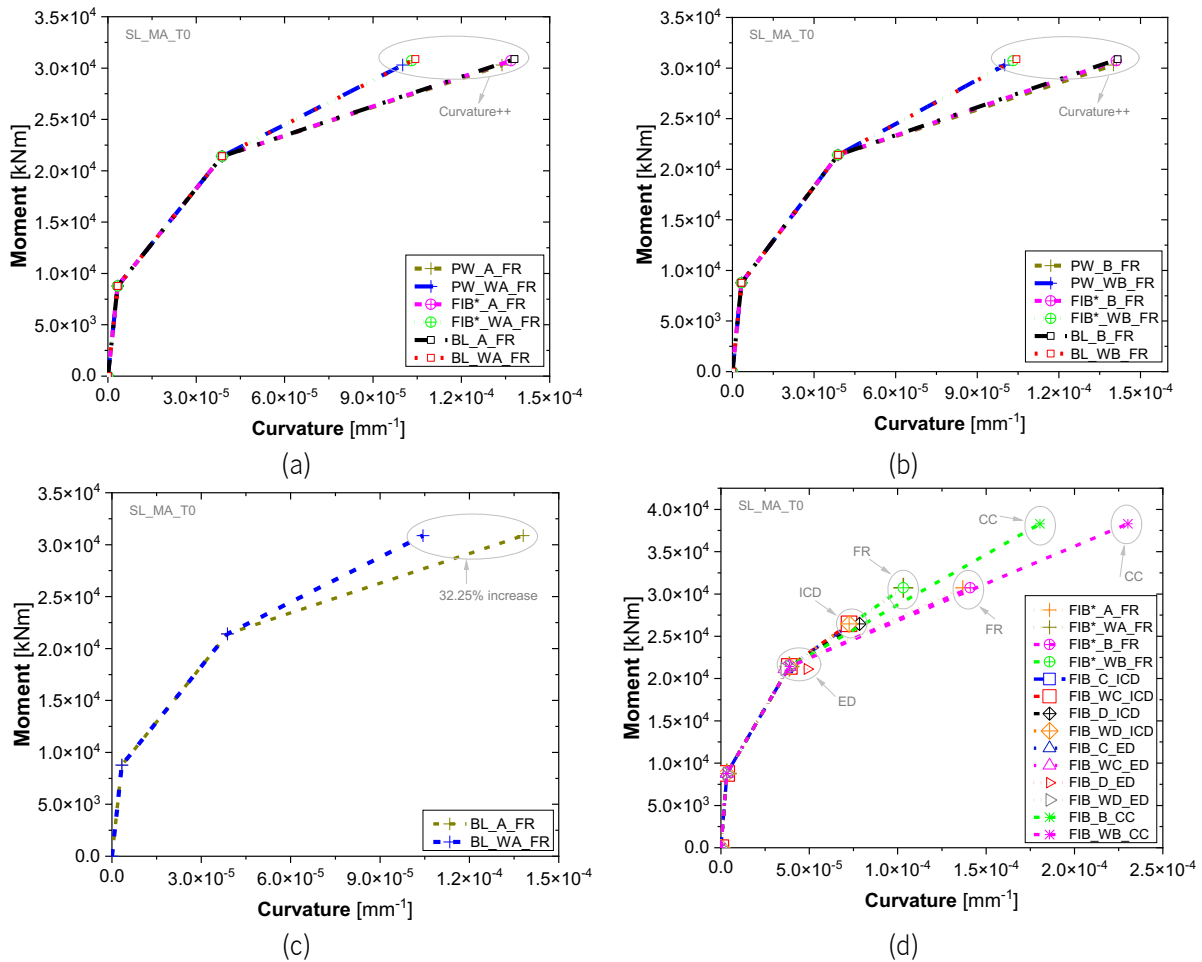


Figure VI.9. Moment-curvature relationship for RC slab with EBR prestressed via MA systems: (a) effect of curvature prediction with Eq. (VI.47), (b) effect of curvature prediction with Eq. (VI.48), (c) quantification of increase of curvature when using Eq. (VI.47), (d) assessment of proposed Eq. (VI.47) and Eq. (VI.48) when considering all possible failure

modes. Note: “curvature ++” indicates a significant curvature increase, when using the proposed equations as compared to when using the existing ones.

An X_Y_Z notation is used in **Figure VI.9**, where X denotes the formula for CFRP failure strain prediction (i.e., FIB (Eq. (VI.43)), BL (Eq. (VI.16)), and PW (Eq. (VI.44)) with the optimal coefficient parameter $p_2 = 0.88$ respectively), Y is the curvature prediction (i.e., Eq. (VI.47) denoted as A or Eq. (VI.48) as B); notations WA and WB mean the curvature calculated using usual methods (i.e., dividing the concrete top fiber strain by neutral axis depth), and Z denotes the type of FM. Referring to the experimentally observed FM, the effects of the formulations in Eq. (VI.47) and Eq. (VI.48) when compared with those from the usual methods (WA and WB) are presented in **Figure VI.9a** and **Figure VI.9b**, respectively. It can be noted that both Eq. (47) and Eq. (48) will increase the curvature without affecting the moment. For example, a 32.25 % increase in the curvature is found when applying Eq. (VI.47) as shown in **Figure VI.9c**. Furthermore, considering all possible FM (**Figure VI.9d**) it can still be noted that regardless of the type of FM, Eq. (VI.47) and Eq. (VI.48) will continue to be promising. When assuming CC as the FM, a greater curvature at CC is found when Eq. (VI.48) is not used, which clearly explains the reason why the CC moment is larger than that of FR, and additionally it explains that Eq. (VI.48) should uniquely be applied for the case of FR.

The load *versus* mid-span deflection responses for the RC slab with EBR prestressed using MA system are presented in **Figure VI.10**. An X_Y_Z notation is used, where X denotes the approach used for determining the CFRP failure strain (i.e., Eq. (VI.44) for PW, Eq. (VI.43) for FIB*, and Eq. (VI.16) for BL, it is worth reminding that the FRP prestrain was added to each of the equations; Y denotes the deflection formula adopted (i.e., R or P for Eq. (VI.52) or Eq. (VI.53), respectively)), and Z denotes the curvature prediction (i.e., Eq. (VI.47) denoted as A or Eq. (VI.48) as B). In general, the developed approach predicts well the results from the experimental program (**Figure VI.10a**). To compare the performance of Eq. (VI.52) and Eq. (VI.53), the load-deflection responses are plotted for the same curvature but with different deflection formula (**Figure VI.10b,c**). Like what is observed for NSM, for a better deflection and stiffness prediction, either Eq. (VI.52) or Eq. (VI.53) can be adopted at cracking state, while better results can be found when Eq. (VI.52) is used to predict the response at yielding state and Eq. (VI.53) at ultimate state (**Figure VI.10d**). However, it should be noted that in the latter state, the curvature required in Eq. (VI.53) should be calculated using either Eq. (VI.47) or Eq. (VI.48).

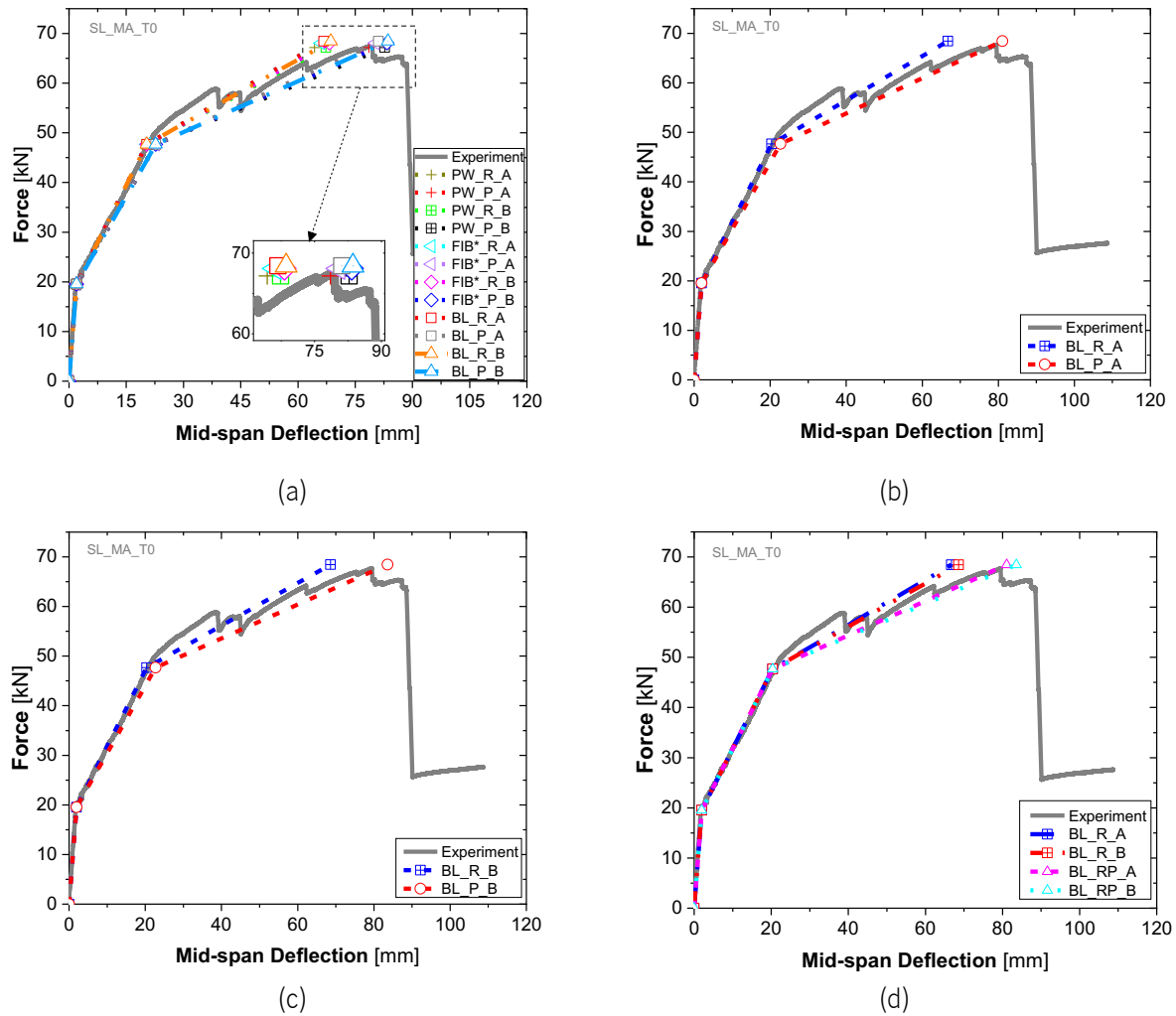


Figure VI.10. Force *versus* mid-span deflection for RC slab with EBR prestressed via MA systems: (a) overall perspectives on responses from all used approaches; (b) comparing two deflections formulae with the curvature from Eq. (VI.47); (c) comparing two deflections formulae with the curvature from Eq. (VI.48); (d) use of similar or different deflections formulae at different loading states.

A comparative perspective shows that better predictions can be found when Eq. (VI.16) from (Balaguru et al., 2009), Eq. (VI.43) from *fib*90 (CEB-FIP, 2019) and Eq. (VI.44) are adopted.

3.2.5. RC slab with EBR prestressed using GA system

The moment-curvature relationships for RC slab with EBR prestressed using GA system are presented in **Figure VI.11**. An X_Y_Z notation is used, where X denotes the formula used for the determining the CFRP failure strain (i.e., FIB, CNR, and ACI stand for the use of Eq. (VI.19), Eq. (VI.20), and Eq. (VI.22), respectively); however,

It is worth reminding that the ε_{fp} was added to each of the equations. Y denotes the equation used for the curvature (i.e., **Eq. (VI.50)** denoted as C or **Eq. (VI.51)** as D, WC and WB denote the curvature calculated using the usual formula without use of **Eq. (VI.50)** and **Eq. (VI.51)**, respectively, i.e., concrete top fiber strain/neutral axis depth), and Z denotes the type of FM.

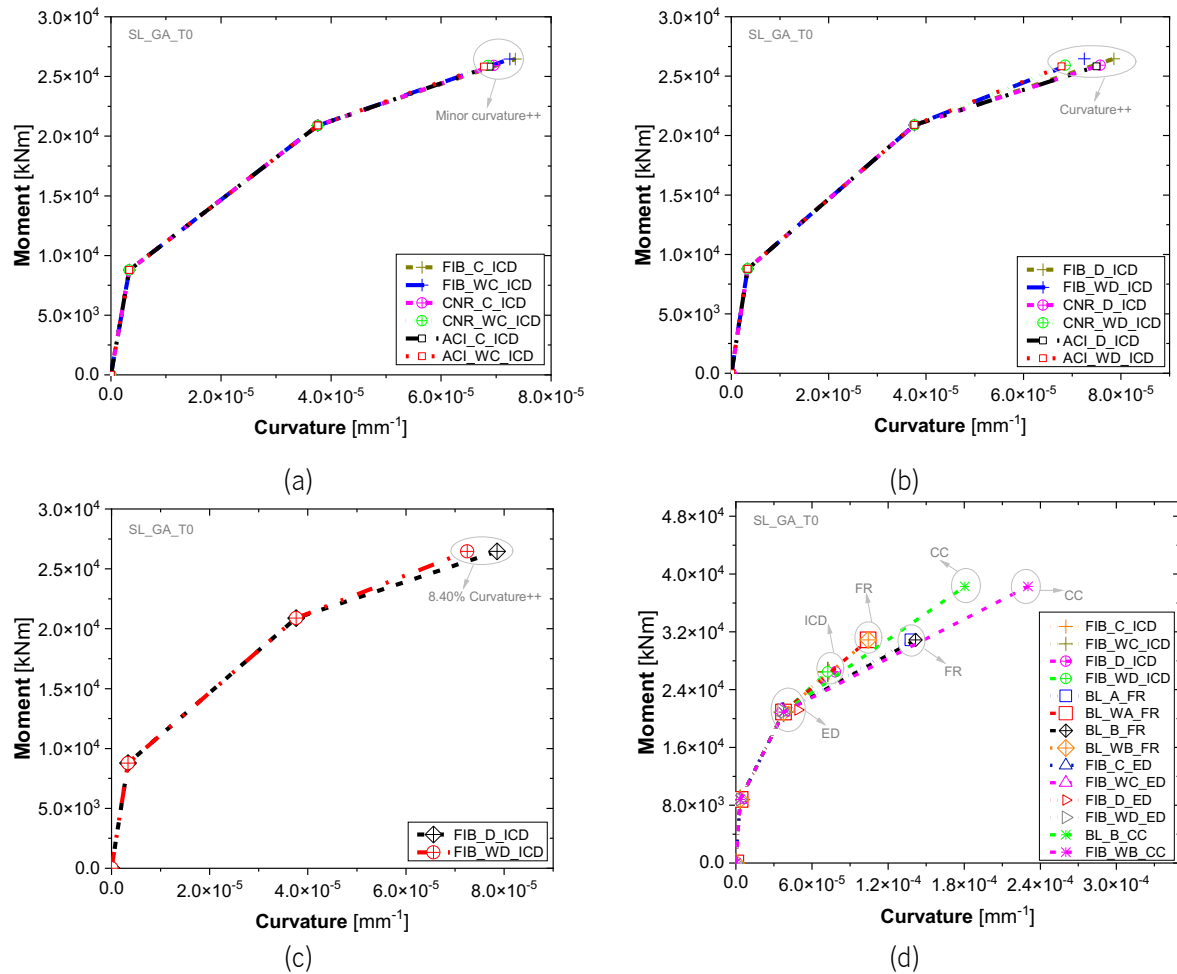


Figure VI.11. Moment-curvature relationship for RC slab with EBR prestressed via GA systems: effect of curvature prediction with (a) Eq. (VI.50) and (b) Eq. (VI.51), (c) quantification of increase of curvature when using Eq. (VI.50), (d) assessment of proposed Eq. (VI.50) and Eq. (VI.51) when considering all possible failure modes. Note: “curvature ++” or “minor curvature ++” indicates a significant or insignificant curvature increase, respectively, when using the proposed equations as compared to when using the existing ones.

Referring to the experimentally observed FM, the effects of Eq. (VI.50) and Eq. (VI.51) when compared with use of WC and WD are presented in Figure VI.11a and Figure VI.11b, respectively. Eq. (VI.50) leads to a minor increase in the curvature as compared to Eq. (VI.51). For example, Eq. (VI.51), 8.40 % curvature increase can

be achieved (**Figure VI.11c**). Furthermore, regardless of the type of FM (**Figure VI.11d**), **Eq. (VI.50)** and **Eq. (VI.51)** will continue to be promising even at higher values of the curvature.

Also, like what is previously observed for MA systems, assuming the occurrence of CC as the prevailing FM, a greater curvature at CC would be found when **Eq. (VI.51)** is not used, which again shows that the proposed formulations in **Eq. (VI.50)** and **Eq. (VI.51)** are pertinent to debonding FM.

Comparing the results from all **Eqs. (VI.47)**, **(VI.48)**, **(VI.50)**, and **(VI.51)** for predicting the curvature of RC slabs with EBR prestressed via MA and GA systems, the equations increase the curvature more when there is FR than when there is ICD or ED, which is reasonable because debonding is expected to occur at lower curvature due to the prevented full usage of the FRP strain.

The results from the force-deflection relations are presented in **Figure VI.12**. An X_Y_Z notation is adopted in the legend, where X stands for CFRP failure strain prediction method (i.e., **Eq. (VI.19)** as FIB, **Eq. (VI.20)** as CNR, and **Eq. (VI.22)** as ACI; however, note that the prestrain ε_{fp} is added to each of these equations), Y stands for deflection formula (R, or P, as **Eq. (VI.52)** or **(VI.53)**), and Z for curvature formula (i.e., **Eq. (VI.50)** denoted as C or **Eq. (VI.51)** as D). Results from all attempts are shown in **Figure VI.12a**. Adopting **Eq. (VI.20)** or **Eq. (VI.22)** is found to lead to a reasonable failure load but fails to match with the predicted deflection using **Eq. (VI.53)**. However, **Eq. (VI.19)** predicts the load that nearly matches with the deflection predicted using **Eq. (VI.52)**. Hence, this leads to suggesting the use of **Eq. (VI.19)** (CEB-FIP, 2019) in **Eq. (VI.45)** and **Eq. (VI.46)** for failure load prediction combined with **Eq. (VI.53)** for deflection prediction. Furthermore, **Eq. (VI.52)** is much better than **Eq. (VI.53)** as the former can predict the stiffness that matches well with that from experiment (**Figure VI.12b,c,d**), while **Eq. (VI.53)** fails to predict the stiffness in the post-cracking stage (**Figure VI.12a**). Also, the curvature should be predicted using **Eq. (VI.50)** or **Eq. (VI.51)** when debonding occurs, the latter being more preferred as it leads to better agreement with **Eq. (VI.52)** for the deflection prediction. Hence when GA fails by debonding, **Eq. (VI.52)** can be adopted to predict responses at all loading states (i.e., cracking, yielding, and ultimate states). Also, **Figure VI.12d** shows that FIB leads to slightly better responses as compared to other standards.

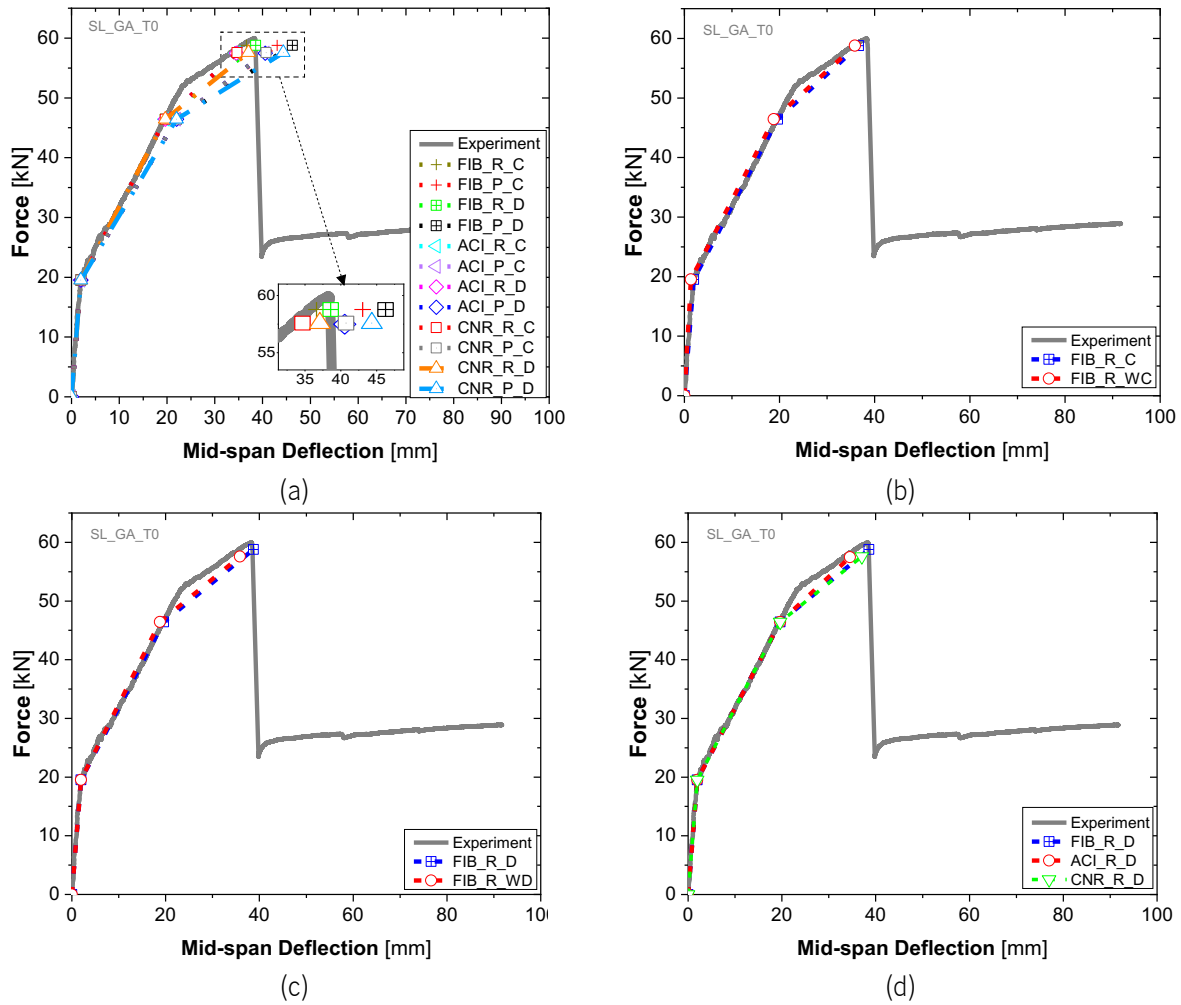


Figure VI.12. Force *versus* mid-span deflection for RC slab with EBR prestressed via GA systems: (a) overall perspectives on responses from all used guidelines, deflections and curvature formulae; (b) effect of proposed curvature prediction with Eq. (VI.50); (c) effect of proposed curvature prediction with Eq. (VI.51); (d) effect of different guidelines on prediction of CFRP failure strain

3.3. Summary

A summary of the key results for the flexural analysis of all investigated RC slabs is provided in **Table VI.4**. The developed analytical approaches generally lead to the FM, efficiency and ductility parameters that agree well with those observed from experiment.

Table VI.4. Key results from experimental tests and from different analytical approaches for predicting the flexural behavior of the non-strengthened, strengthened, and strengthened prestressed RC slabs.

RC Slabs	Approach	P_{cr} (N)	δ_{cr} (mm)	P_y (kN)	δ_y (mm)	P_u (kN)	δ_u (mm)	ε_f ($\times 10^{-3}$)	P_u/P_y	δ_u/δ_y	FM
SL_REF_TO	Experiment	12.30	1.17	28.82	19.58	30.96	120.47	-	-	-	CC
	REF_R	13.36	1.35	27.40	17.64	30.99	119.64	-	-	-	CC
	REF_P	13.36	1.35	27.40	20.63	30.99	183.61	-	-	-	CC
SL_NSM_TO	Experiment	14.44	2.41	39.94	22.28	62.28	74.44	14.09	1.56	3.34	FR
	ACI*_R	13.57	1.36	36.08	18.67	57.73	54.27	10.81	1.60	2.91	FR
	ACI*_P	13.57	1.36	36.08	20.71	57.73	61.78	10.81	1.60	2.98	FR
	PW*_R	13.57	1.36	36.08	18.67	62.07	62.02	12.36	1.72	3.32	FR
	PW*_P	13.57	1.36	36.08	20.71	62.07	70.01	12.36	1.72	3.38	FR
	BL_R	13.57	1.36	36.08	18.67	66.35	68.57	13.78	1.84	3.67	FR
	BL_P	13.57	1.36	36.08	20.71	66.35	77.01	13.78	1.84	3.72	FR
	BR_R	13.57	1.36	36.08	18.67	61.92	61.71	12.30	1.72	3.31	FR
	BR_P	13.57	1.36	36.08	20.71	61.92	69.66	12.30	1.72	3.36	FR
SL_EBR_TO	Experiment	12.63	1.32	53.22	24.64	66.47	41.23	6.65	1.25	1.67	ICD
	FIB_R	13.90	1.37	46.56	20.31	65.54	35.91	6.79	1.21	1.50	ICD
	FIB_P	13.90	1.37	46.56	21.85	65.54	40.30	6.79	1.21	1.59	ICD
	CNR_R	13.90	1.37	46.56	20.31	64.44	34.90	6.61	1.19	1.46	ICD
	CNR_P	13.90	1.37	46.56	21.85	64.44	39.19	6.61	1.19	1.54	ICD
	FIB_R	13.90	1.37	46.56	20.31	65.54	39.54	6.79	1.40	1.95	ICD
	FIB_P	13.90	1.37	46.56	21.85	65.54	40.30	6.79	1.40	1.84	ICD
	ACI-R	13.90	1.37	46.56	20.31	64.71	38.85	6.67	1.19	1.48	ICD
	ACI-P	13.90	13.7	46.56	21.85	64.71	39.59	6.67	1.19	1.56	ICD
SL_MA_TO	Experiment	20.41	2.04	49.69	22.19	67.62	79.24	16.24	1.36	3.57	FR
	PW_R_A	19.52	1.96	47.66	20.33	67.17	64.35	14.23	1.41	3.17	FR
	PW_P_A	19.52	1.96	47.66	22.72	67.17	78.55	14.23	1.41	3.46	FR
	PW_R_B	19.52	1.96	47.66	20.33	67.17	67.35	14.23	1.41	3.31	FR
	PW_P_B	19.52	1.96	47.66	22.72	67.17	82.71	14.23	1.41	3.64	FR
	FIB*_R_A	19.52	1.96	47.66	20.33	68.09	66.06	14.55	1.43	3.25	FR
	FIB*_P_A	19.52	1.96	47.66	22.72	68.09	80.35	14.55	1.43	3.54	FR
	FIB*_R_B	19.52	1.96	47.66	20.33	68.09	68.22	14.55	1.43	3.36	FR
	FIB*_P_B	19.52	1.96	47.66	22.72	68.09	83.33	14.55	1.43	3.67	FR
	BL_R_A	19.52	1.96	47.66	20.33	68.43	66.74	14.68	1.44	3.28	FR
	BL_P_A	19.52	1.96	47.66	22.72	68.43	81.08	14.68	1.44	3.57	FR
	BL_R_B	19.52	1.96	47.66	20.33	68.43	68.60	14.68	1.44	3.37	FR
	BL_P_B	19.52	1.96	47.66	22.72	68.43	83.63	14.68	1.44	3.68	FR
SL_GA_TO	Experiment	20.01	2.00	52.04	23.26	59.98	38.27	11.00	1.15	1.65	ICD
	FIB_R_C	19.52	1.96	46.42	19.61	58.92	36.50	11.30	1.27	1.87	ICD
	FIB_P_C	19.52	1.96	46.42	22.00	58.92	42.83	11.30	1.27	1.96	ICD
	FIB_R_D	19.52	1.96	46.42	19.61	58.92	37.94	11.30	1.27	1.98	ICD
	FIB_P_D	19.52	1.96	46.42	22.00	58.92	44.96	11.30	1.27	2.10	ICD
	ACI_R_C	19.52	1.96	46.42	19.61	57.39	34.23	11.30	1.13	1.56	ICD
	ACI_P_C	19.52	1.96	46.42	22.00	57.39	40.12	11.30	1.13	1.70	ICD
	ACI_R_D	19.52	1.96	46.42	19.61	57.39	36.09	11.30	1.13	1.64	ICD
	ACI_P_D	19.52	1.96	46.42	22.00	57.39	42.93	11.30	1.13	1.83	ICD
	CNR_R_C	19.52	1.96	46.42	19.61	57.73	34.56	10.90	1.24	1.77	ICD
	CNR_P_C	19.52	1.96	46.42	22.00	57.73	40.51	10.90	1.24	1.85	ICD
	CNR_R_D	19.52	1.96	46.42	19.61	57.73	36.30	10.90	1.24	1.89	ICD
	CNR_P_D	19.52	1.96	46.42	22.00	57.73	43.12	10.90	1.24	2.01	ICD

Notes: P_{cr} , P_y , P_u are loads at cracking, yielding, and ultimate states respectively; δ_{cr} , δ_y , δ_u are deflections at cracking, yielding, and ultimate states respectively;

FM: failure mode, R: Eq. (VI.52), P: Eq. (VI.53), A: Eq. (VI.47), B: Eq. (VI.48), C: Eq. (VI.50), D: Eq. (VI.51), FIB: Eq. (VI.19) for non-prestressed EBR as per (CEB-FIP, 2019) or Eq. (VI.19) + ε_{fp} for GA system, FIB*: Eq. (VI.43), PW: Eq. (VI.44), BL: Eq. (VI.16), BR: Eq. (VI.31), CNR: Eq. (VI.20) for EBR and Eq.

(VI.20) + ε_{fp} for GA system. ACI: Eq. (VI.22) for EBR or Eq. (VI.22) + ε_{fp} for GA system, ACI*: Eq. (VI.33), PW* and PW: ε_{fu} is calculated as proposed in Eq.

(VI.34) and Eq. (VI.44), respectively. RP: means the Eq. (VI.52) is used at yielding state and Eq. (VI.53) at ultimate state. An X_Y_Z is used for prestressed

systems to indicate a combination of different formulations, where X is the type of standard or formula, Y is the type of deflection formula, Z is the type of curvature formula. Also, X_Y is used for non-prestressed, with X and Y as defined for prestressed systems.

The results from **Table VI.4** and those presented in **Section 3.2** are furthermore used to select the best formulations, which are recommended as shown in **Table VI.5**.

Table VI.5. Formulations proposed for use at different loading states for different CFRP strengthening techniques.

	Cracking state		Yielding state		Ultimate state	
	Deflection at cracking (δ_{cr})	Cracking load (P_{cr})	Deflection at yielding (δ_y)	Yielding load (P_y)	Deflection at failure (δ_u)	Ultimate load (P_u)
SL_REF_TO	Eq. (VI.52) with Eq. (VI.2)	Eq. (VI.55) with: Eq. (VI.3)	Eq. (VI.52) with Eqs. (VI.2), (VI.4)	Eq. (VI.55) with: Eq. (VI.5)	Eq. (VI.52) with Eqs. (VI.2), (VI.4), (VI.6)	Eq. (VI.55) with: Eq. (VI.7)
SL_EBR_TO	Eq. (VI.52) or (VI.53), with Eq. (VI.10)	Eq. (VI.55) with: Eq. (VI.11)	Eq. (VI.52) with Eq. (VI.10) & (VI.12), or Eq. (VI.53) with (VI.12)	Eq. (VI.55) with: Eq. (VI.13)	Eq. (VI.52) with Eqs (VI.10) & (VI.12) & (VI.14), or Eq. (VI.53) with (VI.14)	Eq. (VI.55) with: Eq. (VI.15) with Eq. (VI.23) and (VI.24)
SL_NSM_TO	Eq. (VI.52) or (VI.53), with Eq. (VI.25)	Eq. (VI.55) with: Eq. (VI.26)	Eq. (VI.52) with Eq. (VI.25) & (VI.27)	Eq. (VI.55) with: Eq. (VI.28)	Eq. (VI.53) with Eq. (VI.29)	Eq. (VI.55) with: Eq. (VI.30) with Eq. (VI.36) and (VI.37)
SL_GA_TO	Eq. (VI.52) or (VI.53), with Eq. (VI.38)	Eq. (VI.55) with: Eq. (VI.39)	Eq. (VI.52) with Eq. (VI.38), (VI.40)	Eq. (VI.55) with: Eq. (VI.41)	Eq. (VI.52) with: Eq. (VI.38), (VI.40), and [(VI.50) or (VI.51)]	Eq. (VI.55) with: Eq. (VI.42) with Eq. (VI.45) and (VI.46)
SL_MA_TO	Eq. (VI.52) or (VI.53), with Eq. (VI.38)	Eq. (VI.55) with: Eq. (VI.39)	Eq. (VI.52) with Eq. (VI.38), (VI.40)	Eq. (VI.55) with: Eq. (VI.41)	Eq. (VI.53) with: Eq. (VI.47) or (VI.48)	Eq. (VI.55) with: Eq. (VI.42) with Eq. (VI.45) and (VI.46)

4. CONCLUSIONS AND RECOMMENDATIONS

Analytical approaches for predicting the flexural behavior of RC slabs strengthened with non-prestressed or prestressed CFRP laminates were developed. A trilinear moment-curvature relationship with pre-cracking, post-cracking, and post-yield stages was established in which the responses were predicted at cracking, yielding and ultimate states, incorporating the prevailing failure mode, and initial strain-curvature dependence for the case of prestressed systems. To predict the responses at ultimate state, different existing equations for predicting the ultimate FRP failure strain were adopted and compared. The key findings from the present work can be highlighted as follows:

- The analytical approaches presented are promising owing to their capability to predict the responses of the RC slabs at cracking, yielding, and ultimate states, that agree well with those obtained from experimental tests. Besides, the efficiency and ductility parameters, as well as the failure modes, agree well with those from experimental tests.

- Regarding the two different formulae adopted for predicting the deflection, it is concluded that the applicability of each formula depends on the loading stage, hence a hybrid of the two formulae can be more promising than using a single formula in all stages. The general view is that both formulae are promising when predicting responses at cracking state. However, the formula based on integration in each loading stage is more promising when used to predict responses at yielding state for all the slabs, and at ultimate state only for the slabs failing by debonding. On the other hand, the formula based on determining the deflection at determinant section is more promising for predicting the responses at ultimate state only for the slabs failing by FRP rupture.
- Regarding the prediction of the failure load, the predictions from different code provisions and those previously proposed by different authors for predicting the FRP failure strain are found to differ from each other, and some have greater benefits than the others depending on the strengthening technique and the type of failure mode. However, in most of the cases, the provisions from *fib*90, CNR, and ACI are found to equally predict the FRP failure strain. Besides, for prestressed systems, newly established formulations for the determination of the curvature lead to the deflection that agrees well with that observed from experimental program, hence can be recommended in future works.
- The present work developed a summary of formulations that are more promising for each of the investigated RC slabs at each loading state of the trilinear moment-curvature relationship, see **Table VI.5**, which can be recommended in future works.

Nomenclature

A_{s1}	Cross-section area of the bottom steel rebars	f_r	Concrete rupture modulus
A_{s2}	Cross-section area of the top steel rebars	f_f^{ED}	CFRP stress at end debonding (ED) failure
A_f	Cross-section area of CFRP reinforcement	f_f^{ICD}	CFRP stress at intermediate crack debonding (ICD) failure
A_{grv}	Area of groove for RC slab with NSM	F_f^{ICD}	Mean value of the CFRP force at ICD failure
a_s	Distance from the center of the support to the start of the CFRP reinforcement	G_f	Fracture energy
b	Section width	h	Section height
b_f	CFRP width	k_I	Curvature corresponding to the bending moment of the section assuming uncracked state

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b_s	Shear span length	k_{II}	Curvature corresponding to the bending moment of the section assuming cracked state
CC	Concrete crushing	k_1	Multiplier on the concrete compressive strength to determine the intensity of the stress distribution
C_y	Neutral axis depth at yielding state	k_2	Ratio of the equivalent rectangular stress block to the neutral axis depth
C_u	Neutral axis depth at ultimate state	L	Span length
C_{us^*}	Neutral axis depth at ultimate state for RC slab with NSM	$L_{cr}^{y,j}$	Length of the pre-cracking region at the steel yield initiation stage
C_{ys^*}	Neutral axis depth at yielding state for RC slab with NSM	$L_{cr}^{u,j}$	Length of the pre-cracking region at the ultimate stage
C_{FR}	Neutral axis depth calculated based on the strain compatibility requirement considering the value of ϵ_f^{FR}	l_e	Theoretical effective bond length
C_{ED}	Neutral axis depth calculated based on the strain compatibility requirement considering the value of ϵ_f^{ED}	ICD	Intermediate crack debonding
C_{ICD}	Neutral axis depth calculated based on the strain compatibility requirement considering the value of ϵ_f^{ICD}	$M - \varphi$	Moment -curvature relationship
C_{us}	Neutral axis depth at ultimate state for RC slab with EBR	M_{cr,s^*}	Bending moment at cracking state for RC slab with NSM
C_{ys}	Neutral axis depth at yielding state for RC slab with EBR	M_{y,s^*}	Bending moment at yielding state for RC slab with NSM
d_{s1}	Distance from the center of the bottom steel rebars to the concrete top fiber	M_{u,s^*}	Bending moment at ultimate state for RC slab with NSM
d_{s2}	Distance from the center of the top steel rebars to the concrete top fiber	$M_{cr,s}$	Bending moment at cracking state for RC slab with EBR
d_a	Maximum aggregate size	$M_{y,s}$	Bending moment at yielding state for RC slab with EBR
d_f	Distance from the center of the CFRP section to the concrete top fiber	$M_{u,s}$	Bending moment at ultimate state for RC slab with EBR
E_{s1}	Modulus of elasticity of the bottom steel rebars	$M_{cr,p}$	Bending moment at cracking state for RC slab with MA or GA system
E_{s2}	Modulus of elasticity of the top steel rebars	$M_{y,p}$	Bending moment at yielding state for RC slab with MA or GA system
E_0	Initial elastic modulus	$M_{u,p}$	Bending moment at ultimate state for RC slab with MA or GA system
E_c	Concrete elastic modulus	M_{ED}	Maximum bending moment at end debonding (ED) failure
E_f	Elastic modulus of CFRP laminate	M_{cr}	Bending moment at cracking state

ED	End debonding	M_y	Bending moment at yielding state
e	Eccentricity resulting from the applied prestress	M_u	Bending moment at ultimate state
ε_f	Ultimate tensile strain of CFRP laminate	$M_{cr,ns}$	Bending moment at cracking state for reference RC slab
ε_f^{CC}	CFRP failure strain at concrete crushing (CC)	$M_{y,ns}$	Bending moment at yielding state for reference RC slab
ε_f^{ED}	CFRP failure strain at end debonding (ED)	$M_{u,ns}$	Bending moment at ultimate state for reference RC slab
ε_f^{FR}	CFRP failure strain at CFRP rupture (FR)	n_{s1}	Modular ratio for bottom steel rebars
ε_f^{ICD}	CFRP failure strain at intermediate crack debonding (ICD)	n_{s2}	Modular ratio for top steel rebars
ε_c	Concrete top fiber compressive strain	n_f	Modular ratio for CFRP
$\varepsilon_{c,bot}^{cr}$	Strain in concrete bottom fiber at crack initiation for the reference slab	P_{cr}	Load at cracking state
$\varepsilon_{c,top}^{cr}$	Strain in concrete top fiber at cracking state for the reference slab	P_y	Load at yielding state
ε_{s2}^{cr}	Strain in top steel rebars at cracking state for the reference slab	P_u	Load at ultimate state
ε_{s1}^{cr}	Strain in bottom steel rebars at cracking state for the reference slab	P_{CC}	Load corresponding to the occurrence of the concrete crushing
$\varepsilon_{c,top}^y$	Strain in concrete top fiber at yielding state for the reference slab	P_{FR}	Load corresponding to the occurrence of the CFRP rupture failure
ε_{s2}^y	Strain in top steel rebars at yielding state for the reference slab	P_{ICD}	Load corresponding to the occurrence of the intermediate crack debonding failure
ε_{s1}^y	Strain in bottom steel rebars at yielding state for the reference slab	P_{ED}	Load corresponding to the occurrence of the end debonding failure
ε_{cu}	Ultimate concrete top fiber strain (assumed equal to 0.0035 in the present work)	p_1	Coefficient obtained from parametric study for RC slab with NSM
ε_{s2}^u	Strain in top steel rebars at ultimate state for the reference slab	p_2	Coefficient obtained from parametric study for RC slab with MA system
ε_{s1}^u	Strain in bottom steel rebars at ultimate state for the reference slab	φ_{ip}	Curvature due to initial prestress
$\varepsilon_{c,top}^{crs*}$	Strain in concrete top fiber at cracking state for RC slab with NSM	$\varphi_{i,p}$	Curvature after usage of prestrain (ε_{fp})
ε_{s2}^{crs*}	Strain in top steel rebars at cracking state for RC slab with NSM	φ_{cr}	Curvature at cracking state
ε_{s1}^{crs*}	Strain in bottom steel rebars at cracking state for RC slab with NSM	φ_y	Curvature at yielding state
$\varepsilon_{c,top}^{ys*}$	Strain in concrete top fiber at yielding state for RC slab with NSM	φ_u	Curvature at ultimate state

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ε_{s2}^{ys*}	Strain in top steel rebars at yielding state for RC slab with NSM	$\varphi_{cr,ns}$	Curvature at cracking state for reference RC slab
ε_{s1}^{ys*}	Strain in bottom steel rebars at yielding state for RC slab with NSM	$\varphi_{y,ns}$	Curvature at yielding state for reference RC slab
$\varepsilon_{c,top}^{crs}$	Strain in concrete top fiber at cracking state for RC slab with EBR	$\varphi_{u,ns}$	Curvature at ultimate state for reference RC slab
ε_{s2}^{crs}	Strain in top steel rebars at cracking state for RC slab with EBR	$\varphi_{cr,s*}$	Curvature at cracking stage for RC slab with NSM
ε_{s1}^{crs}	Strain in bottom steel rebars at cracking state for RC slab with EBR	$\varphi_{y,s*}$	Curvature at yielding stage for RC slab with NSM
$\varepsilon_{c,top}^{ys}$	Strain in concrete top fiber at yielding state for RC slab with EBR	$\varphi_{u,s*}$	Curvature at ultimate state for RC slab with NSM
ε_{s2}^{ys}	Strain in top steel rebars at yielding state for RC slab with EBR	$\varphi_{cr,s}$	Curvature at cracking state for RC slab with EBR
ε_{s1}^{ys}	Strain in bottom steel rebars at yielding state for RC slab with EBR	$\varphi_{y,s}$	Curvature at yielding state for RC slab with EBR
$\varepsilon_{c,top}^{crp}$	Strain in concrete top fiber at cracking state for RC slab with MA or GA	$\varphi_{u,s}$	Curvature at ultimate state for RC slab with EBR
ε_{s2}^{crp}	Strain in top steel rebars at cracking state for RC slab with MA or GA	$\varphi_{cr,p}^T$	Total curvature at cracking state for RC slab with MA or GA system
ε_{s1}^{crp}	Strain in bottom steel rebars at cracking state for RC slab with MA or GA	$\varphi_{y,p}^T$	Total curvature at yielding state for RC slab with MA or GA system
$\varepsilon_{c,top}^{yp}$	Strain in concrete top fiber at yielding state for RC slab with MA or GA	$\varphi_{u,p}^T$	Total curvature at ultimate state for RC slab with MA or GA system
ε_{s2}^{yp}	Strain in top steel rebars at yielding state for RC slab with MA or GA	$\rho_{sl,eq}$	Equivalent reinforcement ratio
ε_{s1}^{yp}	Strain in bottom steel rebars at yielding state for RC slab with MA or GA	$\rho_{f,g}$	Perimeter of the groove
ε_{fu}	Manufacturer CFRP ultimate strain	δ_{cr}	Deflection at cracking state
ε_f	CFRP ultimate strain	δ_y	Deflection at yielding state
$\varepsilon_{c,top}^{ci}$	Tensile strain in the concrete top fiber due to the applied prestress in the CFRP	δ_u	Deflection at ultimate state
$\varepsilon_{c,bot}^{ci}$	Compressive strain in the concrete bottom fiber due to the applied prestress in the CFRP	$\sigma_{s1}^{i=cr,y,u}$	Normal stress in bottom steel rebars at cracking (<i>cr</i>), yielding (<i>y</i>), and ultimate (<i>u</i>) state

$\varepsilon_{c,bot}^{crp}$	Concrete bottom fiber at the crack initiation for RC slab with MA or GA system	$\sigma_{s2}^{i=cr,y,u}$	Normal stress in top steel rebars at cracking (<i>cr</i>), yielding (<i>y</i>), and ultimate (<i>u</i>) state
ε_{fd}	Design CFRP failure strain	$\sigma_f^{i=cr,y,u}$	Normal stress in the CFRP laminate at cracking (<i>cr</i>), yielding (<i>y</i>), and ultimate (<i>u</i>) state
ε_{fb}	FRP strain after usage of prestrain (ε_{fp})	ξ	Interpolation coefficient depending on the cracking moment, applied moment and other coefficients
ε_{fp}	Initial prestrain in the CFRP	t_f	CFRP thickness
F_{pre}	Applied prestress in the CFRP laminate	y_i	Distance from the top concrete fiber strain to the centroidal axis of un-cracked section of the reference RC slab
f_f	Tensile strength of CFRP laminate	y_{s^*}	Distance from the top concrete fiber strain to the centroidal axis of un-cracked section for RC slab with NSM
FR	CFRP rupture	y_s	Distance from the top concrete fiber strain to the centroidal axis of un-cracked section for RC slab with EBR
f_{sy1}	Yielding strength of the bottom steel rebars	y_p	Distance from the top concrete fiber strain to the centroidal axis of un-cracked section for RC slab with MA or GA system
f_{sy2}	Yielding strength of the top steel rebars		

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REFERENCES

- ACI-318M-08. (2008). *American Concrete Institute (ACI). Building code requirements for structural concrete.*
- ACI 440.2R-02. (2002). *American Concrete Institute (ACI). "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures". ACI Committee 440.*
- ACI 440.2R-08. (2008). *American Concrete Institute (ACI). Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.*
- Badawi, M., & Soudki, K. (2009). Flexural strengthening of RC beams with prestressed NSM CFRP rods – Experimental and analytical investigation. *Construction and Building Materials*, 23(10), 3292–3300. <https://doi.org/10.1016/j.conbuildmat.2009.03.005>
- Balaguru, P., Manni, A., & Giancaspro, J. (2009). *FRP Composites for Reinforced and Prestressed Concrete Structures.* Taylor & Francis.
- Barris, C., Correia, L., & Sena-Cruz, J. (2018). Experimental study on the bond behaviour of a transversely compressed mechanical anchorage system for externally bonded reinforcement. *Composite Structures*, 200(October 2017), 217–228. <https://doi.org/10.1016/j.compstruct.2018.05.084>
- Barros, J. A. O., Dias, S. J. E., & Lima, J. L. T. (2007). Efficacy of CFRP-based techniques for the flexural and shear strengthening of concrete beams. *Cement and Concrete Composites*, 29(3), 203–217. <https://doi.org/10.1016/j.cemconcomp.2006.09.001>
- Bianco, V., Barros, J. A. O., & Monti, G. (2009). Three-dimensional mechanical model for simulating the NSM FRP strips shear strength contribution to RC beams. *Engineering Structures*, 31(4), 815–826. <https://doi.org/10.1016/j.engstruct.2008.12.017>
- Bilotta, A., Ceroni, F., Nigro, E., & Pecce, M. (2015). Efficiency of CFRP NSM strips and EBR plates for flexural strengthening of RC beams and loading pattern influence. *Composite Structures*, 124, 163–175.

<https://doi.org/10.1016/j.compstruct.2014.12.046>

Bilotta, A., Faella, C., Martinelli, E., & Nigro, E. (2013). Design by testing procedure for intermediate debonding in EBR FRP strengthened RC beams. *Engineering Structures*, 46, 147–154. <https://doi.org/10.1016/j.engstruct.2012.06.031>

CEB-FIP. (2019). *fib90 Bulletin: Externally Applied FRP reinforcement for Concrete structures*.

Chen, J. ., & Teng, J. . (2001). Anchorage Strength Models for Frp and Steel Plates bonded to concrete. *Journal of Structural Engineering*, 127(c), 784–791.

CNR-DT 200 R1. (2013). *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures; CNR-DT 200 R1/2013*;

Correia, L., Sena-Cruz, J., Michels, J., França, P., Pereira, E., & Escusa, G. (2017). Durability of RC slabs strengthened with prestressed CFRP laminate strips under different environmental and loading conditions. *Composites Part B*, 125, 71–88. <https://doi.org/10.1016/j.compositesb.2017.05.047>

Correia, L., Teixeira, T., Michels, J., Almeida, J. A. P. P., & Sena-Cruz, J. (2015). Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems. *Composites Part B: Engineering*, 81, 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>

Deng, Y., Li, Z., Zhang, H., Corigliano, A., Lam, A. C. C., Hansapinyo, C., & Yan, Z. (2021). Experimental and analytical investigation on flexural behaviour of RC beams strengthened with NSM CFRP prestressed concrete prisms. *Composite Structures*, 257(November 2020), 113385. <https://doi.org/10.1016/j.compstruct.2020.113385>

Dias, S. J. E., Barros, J. A. O., & Janwaen, W. (2018). Behavior of RC beams flexurally strengthened with NSM CFRP laminates. *Composite Structures*, 201(March 2017), 363–376. <https://doi.org/10.1016/j.compstruct.2018.05.126>

Dushimimana, A., Sena-cruz, J., Correia, L., & Pereira, J. M. (2024). Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments. *Composite Structures*, 118174. <https://doi.org/10.1016/j.compstruct.2024.118174>

Dushimimana, A., Sena-Cruz, J., Correia, L., Pereira, J. M., Cabral-Fonseca, S., & Cruz, R. (2024). Durability of

- bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures. *Construction and Building Materials*, 427(December 2023). <https://doi.org/10.1016/j.conbuildmat.2024.136213>
- Dushimimana, A., Vassilopoulos, A. P., Sena-Cruz, J., Pereira, J. M., Correia, L., Cabral-Fonseca, S., & Cruz, R. (2024). Behavior of CFRP composites and epoxy adhesives after long-term exposure to outdoor and laboratory-controlled environments. *Construction and Building Materials*, 438(May). <https://doi.org/10.1016/j.conbuildmat.2024.137201>
- El-Hacha, R., & Riskalla, S. (2004). Near-surface-mounted fiber-reinforced polymer reinforcements for flexural strengthening of concrete structures. *ACI Struct J*, 101(5), 17–26.
- El-Hacha, R., Wight, R., & Green, M. (2001). Prestressed fibre-reinforced polymer laminates for strengthening structures. *Progress in Structural Engineering and Materials*, 3(2), 111–121. <https://doi.org/10.1002/pse.76>
- Ferracuti, B., Martinelli, E., Nigro, E., & Savoia, M. (2007). Fracture energy and design rules against FRP-concrete debonding. *In: Proc of 8th FRPRCS Conference*, 16–18.
- Garden, H. N., & Hollaway, L. C. (1998). An experimental study of the influence of plate end anchorage of carbon fibre composite plates used to strengthen reinforced concrete beams. *Composite Structures*, 42(2), 175–188. [https://doi.org/10.1016/S0263-8223\(98\)00070-1](https://doi.org/10.1016/S0263-8223(98)00070-1)
- He, Z., Ou, J., & Wang, B. (2007). The trilinear moment vs. curvature relationship of concrete beams reinforced with fiber reinforced polymer (FRP) rebars. *Composite Structures*, 77(1), 30–35. <https://doi.org/10.1016/j.compstruct.2005.06.012>
- Jung, W. T., Park, J. S., Kang, J. Y., & Keum, M. S. (2017). Flexural behavior of concrete beam strengthened by near-surface mounted CFRP reinforcement using equivalent section model. *Advances in Materials Science and Engineering*, 2017, 16. <https://doi.org/10.1155/2017/9180624>
- Michels, J., Sena-Cruz, J., Czaderski, C., & Motavalli, M. (2013). Structural Strengthening with Prestressed CFRP Strips with Gradient Anchorage. *Journal of Composites for Construction*, 1–46. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000372](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000372)

- Mostakhdemin Hosseini, M. R., Dias, S. J. E., & Barros, J. A. O. (2020). Behavior of one-way RC slabs flexurally strengthened with prestressed NSM CFRP laminates – Assessment of influencing parameters. *Composite Structures*, 245(February), 112259. <https://doi.org/10.1016/j.compstruct.2020.112259>
- Oehlers, D. J., Visintin, P., & Lucas, W. (2016). Fundamental Mechanics Governing FRP-Retrofitted RC Beams with Anchored and Prestressed FRP Plates. *Journal of Composites for Construction*, 20(6), 04016047. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000710](https://doi.org/10.1061/(asce)cc.1943-5614.0000710)
- Rezazadeh, M., Barros, J., & Costa, I. (2015). Analytical approach for the flexural analysis of RC beams strengthened with prestressed CFRP. *Composites Part B: Engineering*, 73, 16–34. <https://doi.org/10.1016/j.compositesb.2014.12.016>
- Sena-Cruz, J., Michels, J., Harmanci, Y. E., & Correia, L. (2015). Flexural strengthening of RC slabs with prestressed CFRP strips using different anchorage systems. *Polymers*, 7(10), 2100–2118. <https://doi.org/10.3390/polym7101502>
- Sena-Cruz, J., Ricardo Cruz, J., Correia, L., Cabral-Fonseca, S., Michels, J., & Czaderski, C. (2019). Long-term structural and durability performances of reinforced concrete elements strengthened in flexure with CFRP laminates: A research project. *IABSE Symposium, Guimaraes 2019 - Report*, 1006–1014. <https://doi.org/10.2749/guimaraes.2019.1006>
- Sharaky, I. A., Torres, L., & Sallam, H. E. M. (2015). Experimental and analytical investigation into the flexural performance of RC beams with partially and fully bonded NSM FRP bars / strips. *Composite Structures*, 122, 113–126. <https://doi.org/10.1016/j.compstruct.2014.11.057>
- Silva, P. M. (2017). *Time-dependent behaviour and durability of RC slabs strengthened with NSM CFRP strips. Doctoral Thesis, Universidade do Minho.* (Issue January). University of Minho.
- Torabian, A., Isufi, B., Mostofinejad, D., & Pinho Ramos, A. (2020). Flexural strengthening of flat slabs with FRP composites using EBR and EBROG methods. *Engineering Structures*, 211(August 2019). <https://doi.org/10.1016/j.engstruct.2020.110483>
- Wu, Y., & Huang, Y. (2008). *Hybrid Bonding of FRP to Reinforced Concrete Structures*. June, 266–273.
- You, Y., Choi, K., & Kim, J. (2012). An experimental investigation on flexural behavior of RC beams strengthened

with prestressed CFRP strips using a durable anchorage system. *Composites Part B*, 43, 3026–3036.
<https://doi.org/10.1016/j.compositesb.2012.05.030>

Zhang, S. S., Teng, J. G., & Yu, T. (2014). Bond Strength Model for CFRP Strips Near-Surface Mounted to Concrete. *Journal of Composites for Construction*, 18(3). [https://doi.org/10.1061/\(asce\)cc.1943-5614.0000402](https://doi.org/10.1061/(asce)cc.1943-5614.0000402)

ANNEX VI.1

The reference RC slab without strengthening was modelled using section analysis referring to **Figure VI.A1**.

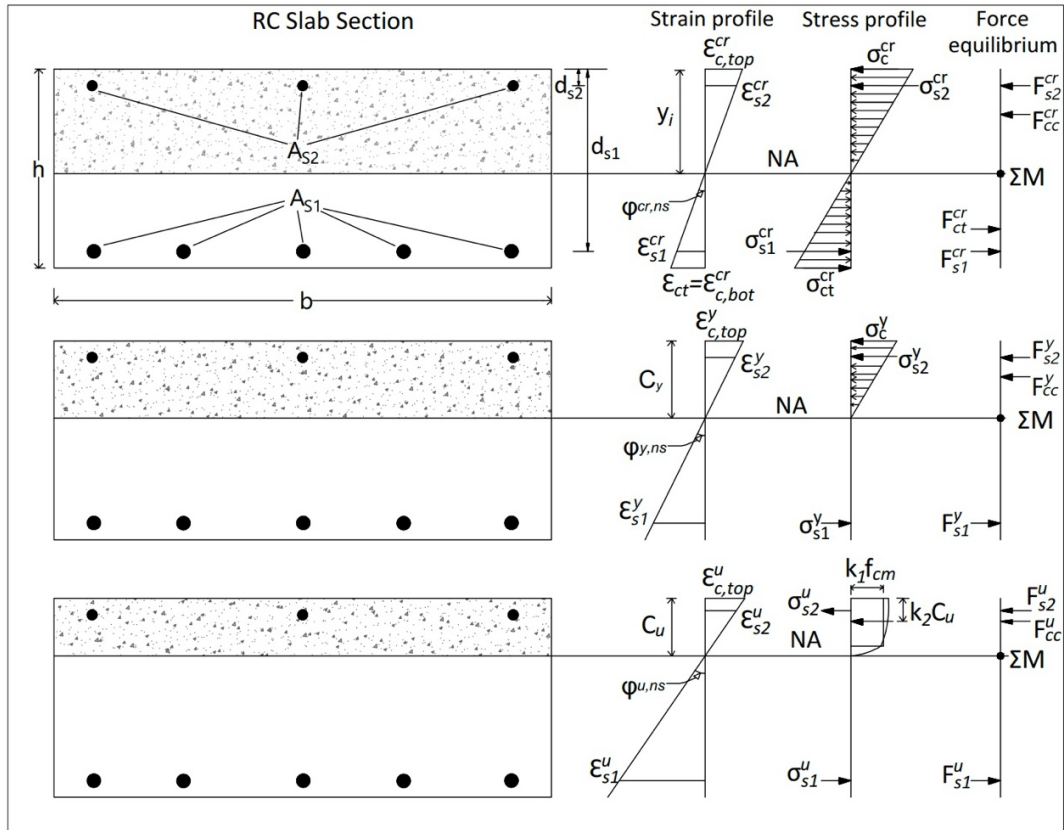


Figure VI.A1. Section analysis of the reference RC slab with neither CFRP strengthening nor prestressing systems: (i) cracking, (ii) yielding, and (iii) ultimate states.

Referring to **Figure VI.A1**, the distance from the concrete top fiber to the centroidal axis of the slab section (y_i) can be calculated as:

$$y_i = \frac{\sum_{i=1}^n A_i \cdot d_i}{\sum_{i=1}^n A_i} = \frac{\frac{b \cdot h^2}{2} + A_{s2} \cdot (n_{s2} - 1) \cdot d_{s2} + A_{s1} \cdot (n_{s1} - 1) \cdot d_{s1}}{b \cdot h + A_{s2} \cdot (n_{s2} - 1) + A_{s1} \cdot (n_{s1} - 1)} \quad (\text{VI.A1a})$$

The strains in concrete compressive top fiber ($\epsilon_{c,top}^{cr}$), top steel rebars (ϵ_{s2}^{cr}), and bottom steel rebars (ϵ_{s1}^{cr}) at cracking state are as follows:

$$\epsilon_{c,top}^{cr} = \epsilon_{ct} \cdot y_i / (h - y_i) \quad (\text{VI.A1b})$$

$$\varepsilon_{s2}^{cr} = \varepsilon_{c,top}^{cr} \cdot (y_i - d_{s2})/y_i \quad (\text{VI.A1c})$$

$$\varepsilon_{s1}^{cr} = \varepsilon_{c,top}^{cr} \cdot (d_{s1} - y_i)/y_i \quad (\text{VI.A1d})$$

The strains in bottom steel rebars (ε_{s1}^y), concrete compressive top fiber ($\varepsilon_{c,top}^y$), and top steel rebars (ε_{s2}^y) at yielding state are as follows:

$$\varepsilon_{s1}^y = f_{s1}/E_{s1} \quad (\text{VI.A1e})$$

$$\varepsilon_{c,top}^y = \varepsilon_{s1}^y \cdot C_y/(d_{s1} - C_y) \quad (\text{VI.A1f})$$

$$\varepsilon_{s2}^y = \varepsilon_{s1}^y \cdot \frac{(C_y - d_{s2})}{(d_{s1} - C_y)} \leq \varepsilon_{s2}, \text{ otherwise, } \varepsilon_{s2}^y = \varepsilon_{s2} = f_{sy2}/E_{s2} \quad (\text{VI.A1g})$$

At the ultimate state, the neutral axis depth (C_u), stresses in bottom (σ_{s1}^u) and top (σ_{s2}^u) steel rebars are calculated as:

$$C_u = (A_{s1} \cdot \sigma_{s1}^u - A_{s2} \cdot \sigma_{s2}^u)/k_1 \cdot f_{cm} \cdot b \quad (\text{VI.A1h})$$

$$\sigma_{s1}^u = \min(\varepsilon_{c,top}^u \cdot (d_{s1} - C_u)/C_u, f_{sy1}/E_{s1}) \cdot E_{s1} \quad (\text{VI.A1i})$$

$$\sigma_{s2}^u = \min(\varepsilon_{c,top}^u \cdot (C_u - d_{s2})/C_u, f_{sy2}/E_{s2}) \cdot E_{s2} \quad (\text{VI.A1j})$$

In the above equations, n_{s1} and n_{s2} are the modular ratios of bottom and top steel rebars, respectively; f_{cm} is the concrete compressive strength; ε_{s2} , f_{sy1} , and f_{sy2} are the yielding strain in top steel rebars, and yielding strengths in bottom and top steel rebars, respectively; $\varepsilon_{c,top}^u$ is the concrete top fiber strain at ultimate state; $\varphi_{cr,ns}$, $\varphi_{y,ns}$, and $\varphi_{u,ns}$ are the curvatures at cracking, yielding, and ultimate states, respectively, where the subscript *ns* stands for *non-strengthened*. The C_u is calculated iteratively where its exact value is obtained at the balance between the tensile and compressive forces.

ANNEX VI.2

The RC slab strengthened with EBR was modelled using section analysis referring to **Figure VI.A2** as follows.

The distance from the concrete top fiber to the centroidal axis of the EBR slab section (y_{s^*}) can be calculated as:

$$y_{s^*} = \frac{\sum_{i=1}^n A_i d_i}{\sum_{i=1}^n A_i} = \frac{\frac{b \cdot h^2}{2} + A_{s2} \cdot (n_{s2} - 1) \cdot d_{s2} + A_{s1} \cdot (n_{s1} - 1) \cdot d_{s1} + A_f \cdot n_f \cdot d_f}{b \cdot h + A_{s2} \cdot (n_{s2} - 1) + A_{s1} \cdot (n_{s1} - 1) + A_f \cdot n_f} \quad (\text{VI.A2a})$$

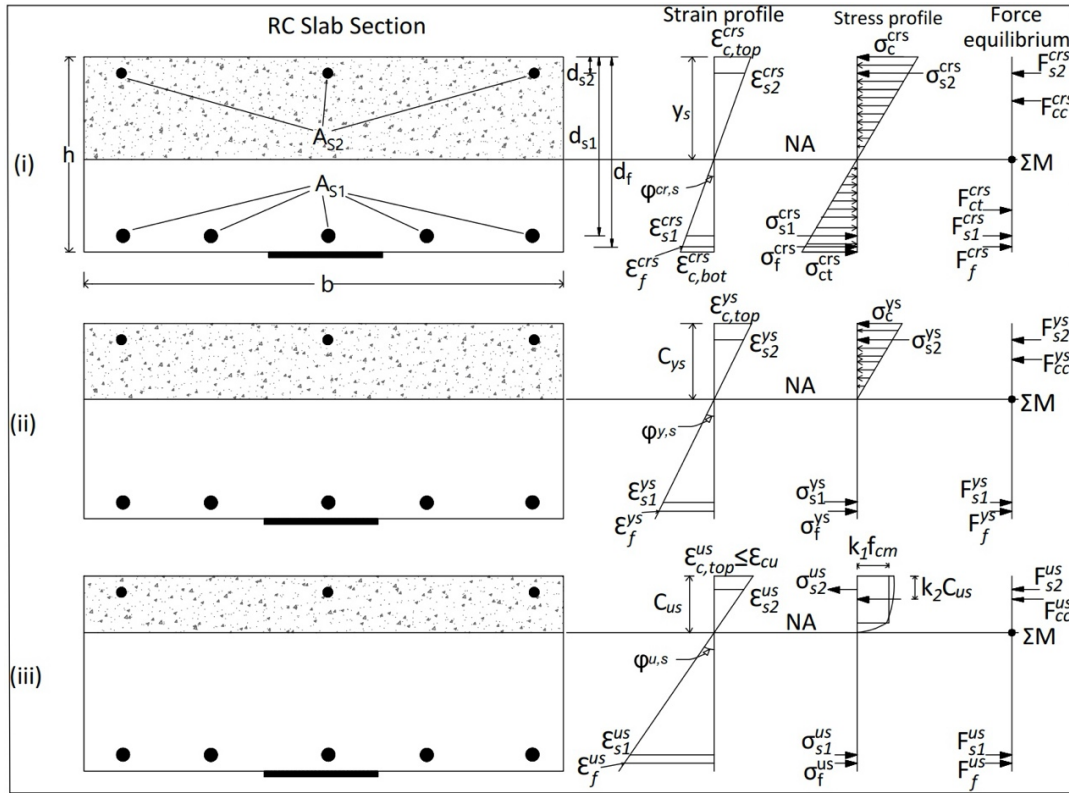


Figure VI.A2. Section analysis of the RC slab strengthened with EBR system: (i) cracking, (ii) yielding, and (iii) ultimate states.

The strains in concrete compressive top fiber ($\epsilon_{c,top}^{crs}$), top steel rebars (ϵ_{s2}^{crs}), bottom steel rebars (ϵ_{s1}^{crs}), and CFRP (ϵ_f^{crs}) at cracking state are as follows:

$$\epsilon_{c,top}^{crs} = \epsilon_{ct} \cdot y_s / (h - y_s) \quad (VI.A2b)$$

$$\epsilon_{s2}^{crs} = \epsilon_{c,top}^{crs} \cdot (y_s - d_{s2}) / y_s \quad (VI.A2c)$$

$$\epsilon_{s1}^{crs} = \epsilon_{c,top}^{crs} \cdot (d_{s1} - y_s) / y_s \quad (VI.A2d)$$

$$\epsilon_f^{crs} = \epsilon_{c,top}^{crs} \cdot (d_f - y_s) / y_s \quad (VI.A2e)$$

The strains in bottom steel rebars (ϵ_{s1}^{ys}), concrete compressive top fiber ($\epsilon_{c,top}^{ys}$), top steel rebars (ϵ_{s2}^{ys}), and CFRP (ϵ_f^{ys}) reinforcement at yielding state are as follows:

$$\epsilon_{s1}^{ys} = f_{sy1} / E_{s1} \quad (VI.A2f)$$

$$\epsilon_{c,top}^{ys} = \epsilon_{s1}^{ys} \cdot C_{ys} / (d_{s1} - C_{ys}) \quad (VI.A2g)$$

$$\epsilon_{s2}^{ys} = \epsilon_{s1}^{ys} \cdot \frac{(C_{ys} - d_{s2})}{(d_{s1} - C_{ys})} \leq \epsilon_{s2}, \text{ otherwise, } \epsilon_{s2}^{ys} = \epsilon_{s2} = f_{sy2} / E_{s2} \quad (VI.A2h)$$

$$\epsilon_f^{ys} = \epsilon_{s1}^{ys} \cdot (d_f - C_{ys}) / (d_{s1} - C_{ys}) \quad (VI.A2i)$$

At the ultimate state, the neutral axis depth (C_{us}), stresses in the bottom (σ_{s1}^{us}) and top (σ_{s2}^{us}) steel rebars, and CFRP reinforcement (σ_f^{us}) are calculated as:

$$C_{us} = (A_{s1} \cdot \sigma_{s1}^{us} + A_f \cdot E_f \cdot \varepsilon_f^{us} - A_{s2} \cdot \sigma_{s2}^{us}) / k_1 \cdot f_{cm} \cdot b \quad (\text{VI.A2j})$$

$$\sigma_{s1}^{us} = \min(\varepsilon_{c,top}^{us} \cdot (d_{s1} - C_{us}) / C_{us}, f_{sy1} / E_{s1}) \cdot E_{s1} \quad (\text{VI.A2k})$$

$$\sigma_{s2}^{us} = \min(\varepsilon_{c,top}^{us} \cdot (C_{us} - d_{s2}) / C_{us}, f_{sy2} / E_{s2}) \cdot E_{s2} \quad (\text{VI.A2l})$$

$$\sigma_f^{us} = \varepsilon_f^{us} \cdot E_f \quad (\text{VI.A2m})$$

In the above equations, n_f, n_{s1} and n_{s2} are the modular ratios of the CFRP, bottom and top steel rebars, respectively; f_{cm} is the concrete compressive strength; ε_{s2} , f_{sy1} , and f_{sy2} are the yielding strain in top steel rebars, and yielding strengths in bottom and top steel rebars, respectively; E_{s1} and E_{s2} are the elastic moduli of bottom and top steel rebars, respectively; ε_f^{us} is the CFRP strain at ultimate state; A_f and E_f are the CFRP area and modulus of elasticity, respectively; $\varphi_{cr,s}$, $\varphi_{y,s}$, and $\varphi_{u,s}$ are the curvatures at cracking, yielding, and ultimate states, respectively. The C_{us} is calculated iteratively where its exact value is obtained at the balance between the tensile and compressive forces.

ANNEX VI.3

The RC slab strengthened with NSM was modelled using section analysis referring to **Figure VI.A3** as follows.

The distance from the top concrete fiber to the centroidal axis of the NSM slab section (y_{s^*}) can be calculated as:

$$y_{s^*} = \frac{\sum_{i=1}^n A_i \cdot d_i}{\sum_{i=1}^n A_i} = \frac{\frac{b \cdot h^2}{2} - A_{grv} \cdot d_f + A_{s2} \cdot (n_{s2} - 1) \cdot d_{s2} + A_{s1} \cdot (n_{s1} - 1) \cdot d_{s1} + A_f \cdot n_f \cdot d_f}{b \cdot h - A_{grv} + A_{s2} \cdot (n_{s2} - 1) + A_{s1} \cdot (n_{s1} - 1) + A_f \cdot n_f} \quad (\text{VI.A3a})$$

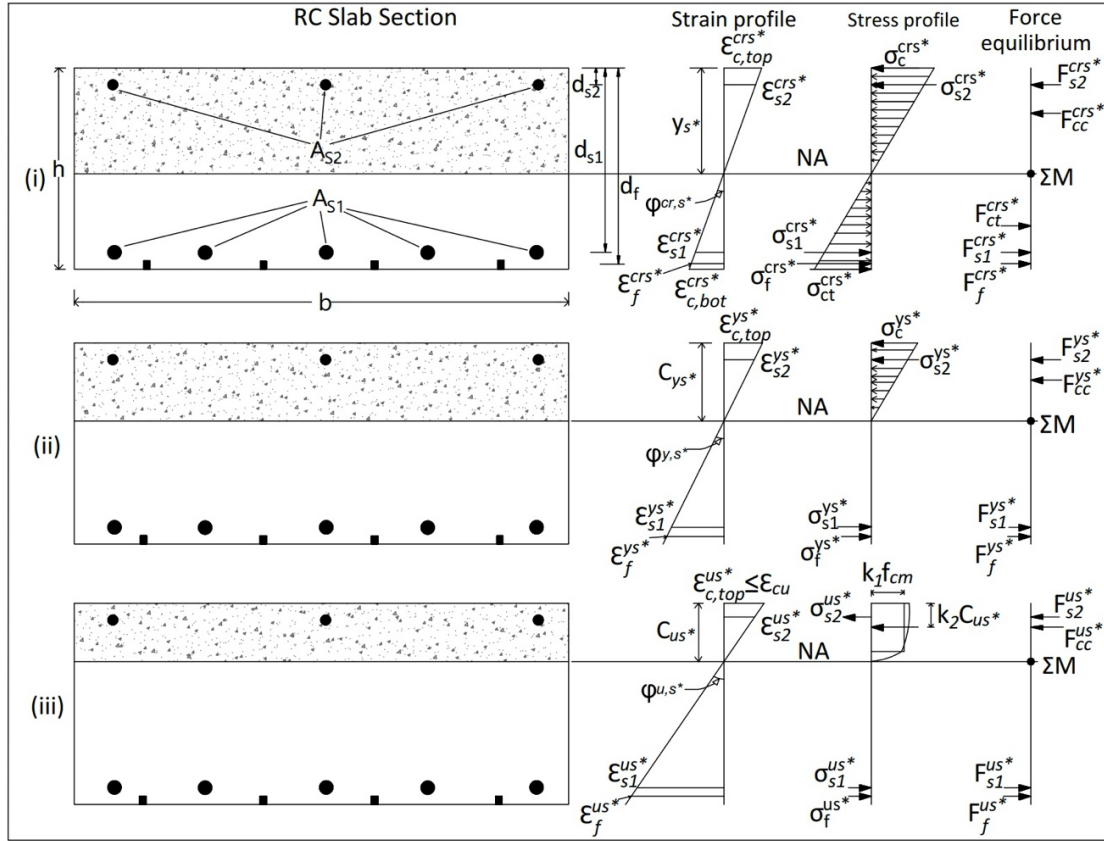


Figure VI.A3. Section analysis of the RC slab strengthened with NSM system: (i) cracking, (ii) yielding, and (iii) ultimate states.

The strains in concrete compressive top fiber ($\epsilon_{c,top}^{crs*}$), top steel (ϵ_{s2}^{crs*}) and bottom (ϵ_{s1}^{crs*}) steel rebars, and CFRP (ϵ_f^{crs*}) at cracking state are as follows:

$$\epsilon_{c,top}^{crs*} = \epsilon_{ct} \cdot y_s^* / (h - y_s^*) \quad (VI.A3b)$$

$$\epsilon_{s2}^{crs*} = \epsilon_{c,top}^{crs*} \cdot (y_s^* - d_{s2}) / y_s^* \quad (VI.A3c)$$

$$\epsilon_{s1}^{crs*} = \epsilon_{c,top}^{crs*} \cdot (d_{s1} - y_s^*) / y_s^* \quad (VI.A3d)$$

$$\epsilon_f^{crs*} = \epsilon_{c,top}^{crs*} \cdot (d_f - y_s^*) / y_s^* \quad (VI.A3e)$$

The strains in bottom steel rebars (ϵ_{s1}^{ys*}), concrete compressive top fiber ($\epsilon_{c,top}^{ys*}$), top steel rebars (ϵ_{s2}^{ys*}), and CFRP (ϵ_f^{ys*}) at yielding state are as follows:

$$\epsilon_{s1}^{ys*} = f_{sy1} / E_{s1} \quad (VI.A3f)$$

$$\epsilon_{c,top}^{ys*} = \epsilon_{s1}^{ys*} \cdot C_{ys*} / (d_{s1} - C_{ys*}) \quad (VI.A3g)$$

$$\varepsilon_{s2}^{ys*} = \varepsilon_{s1}^{ys*} \cdot \frac{(C_{ys*} - d_{s2})}{(d_{s1} - C_{ys*})} \leq \varepsilon_{s2}, \text{ otherwise, } \varepsilon_{s2}^{ys*} = \varepsilon_{s2} = f_{sy2}/E_{s2} \quad (\text{VI.A3h})$$

$$\varepsilon_f^{ys*} = \varepsilon_{s1}^{ys*} \cdot (d_f - C_{ys*}) / (d_{s1} - C_{ys*}) \quad (\text{VI.A3i})$$

At the ultimate state, the neutral axis depth (C_{us*}), stresses in bottom (σ_{s1}^{us*}) and top compressive (σ_{s2}^{us*}) steel rebars, and stress in the CFRP reinforcement (σ_f^{us*}) are calculated as:

$$C_{us*} = (A_{s1} \cdot \sigma_{s1}^{us*} + A_f \cdot E_f \cdot \varepsilon_f^{us*} - A_{s2} \cdot \sigma_{s2}^{us*}) / (k_1 \cdot f_{cm} \cdot b) \quad (\text{VI.A3j})$$

$$\sigma_{s1}^{us*} = \min(\varepsilon_{c,top}^{us*} \cdot (d_{s1} - C_{us*}) / C_{us*}, f_{sy1} / E_{s1}) \cdot E_{s1} \quad (\text{VI.A3k})$$

$$\sigma_{s2}^{us*} = \min(\varepsilon_{c,top}^{us*} \cdot (C_{us*} - d_{s2}) / C_{us*}, f_{sy2} / E_{s2}) \cdot E_{s2} \quad (\text{VI.A3l})$$

$$\sigma_f^{us*} = \varepsilon_f^{us*} \cdot E_f \quad (\text{VI.A3m})$$

In the above equations, n_f , n_{s1} and n_{s2} are the modular ratios of CFRP, bottom and top steel rebars, respectively; f_{cm} is the concrete compressive strength; A_{grv} is the area of groove; ε_{s2} , f_{sy1} , and f_{sy2} are the yielding strain in top steel rebars, and yielding strengths in bottom and top steel rebars, respectively; E_{s1} and E_{s2} are the elastic moduli of bottom and top steel rebars, respectively; ε_f^{us*} is the FRP strain at the ultimate state; A_f and E_f are the CFRP area and modulus of elasticity, respectively; $\varphi_{cr,s*}$, $\varphi_{y,s*}$, and $\varphi_{u,s*}$ are the curvatures at cracking, yielding, and ultimate states, respectively. The C_{us*} is calculated iteratively where its exact value is obtained at the balance between the tensile and compressive forces.

ANNEX VI.4

The RC slab with EBR prestressed via GA or MA system is modelled using section analysis referring to **Figure VI.A4** as follows.

The distance from the top concrete fiber to the centroidal axis of the slab section (y_p) can be calculated as:

$$y_p = \frac{\sum_{i=1}^n A_i \cdot d_i}{\sum_{i=1}^n A_i} = \frac{\frac{b \cdot h^2}{2} + A_{s2} \cdot (n_{s2} - 1) \cdot d_{s2} + A_{s1} \cdot (n_{s1} - 1) \cdot d_{s1} + A_f \cdot n_f \cdot d_f}{b \cdot h + A_{s2} \cdot (n_{s2} - 1) + A_{s1} \cdot (n_{s1} - 1) + A_f \cdot n_f} \quad (\text{VI.A4a})$$

Also, the second moment of inertia of un-cracked section (I_{ucr}) can be calculated as

$$I_{ucr} = \frac{b \cdot h^3}{12} + b \cdot h \cdot (y_p - h/2)^2 + A_{s2} \cdot (n_{s2} - 1) \cdot (d_{s2} - y_p)^2 + A_{s1} \cdot (n_{s1} - 1) \cdot (d_{s1} - y_p)^2 + A_f \cdot n_f \cdot (d_f - y_p)^2 \quad (\text{VI.A4a})$$

The applied prestress ($F_{pre} = E_f \cdot \varepsilon_{fp} \cdot A_f$) and the eccentricity ($e = d_f - h/2$) are used to determine the initial tensile strain in the concrete top fiber ($\varepsilon_{c,top}^{ci}$), the initial concrete compressive strain in its bottom fiber ($\varepsilon_{c,bot}^{ci}$), and the corresponding initial curvature (φ_{ip}) all due to initial applied prestress as follows:

$$\varepsilon_{c,top}^{ci} = \frac{F_{pre} \cdot e \cdot y_p / I_{ucr} - F_{pre} / b \cdot h}{E_c} \quad (VI.A4b)$$

$$\varepsilon_{c,bot}^{ci} = \frac{F_{pre} \cdot e \cdot (h - y_p) / I_{ucr} + F_{pre} / b \cdot h}{E_c} \quad (VI.A4c)$$

$$\varphi_{ip} = (\varepsilon_{c,top}^{ci} + \varepsilon_{c,bot}^{ci}) / h \quad (VI.A4e)$$

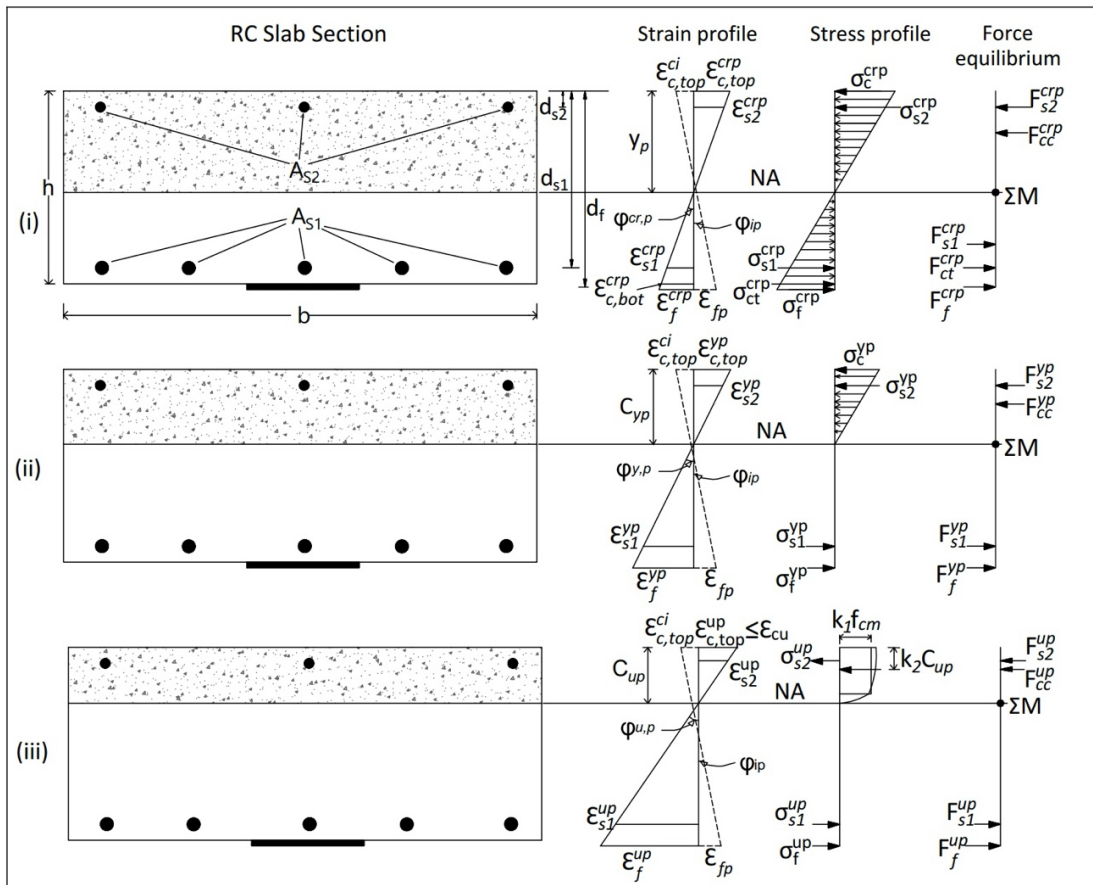


Figure VI.A4. Section analysis of the RC slab with EBR prestressed using either MA or GA systems: (i) cracking, (ii) yielding, and (iii) ultimate states.

The strain in the concrete bottom fiber at the crack initiation ($\epsilon_{c,bot}^{crp} = f_r/E_c$, $f_r = 0.62 \cdot \sqrt{f_{cm}}$) is used to determine the strains in concrete compressive top fiber ($\epsilon_{c,top}^{crp}$), top steel rebars (ϵ_{s2}^{crp}), bottom steel rebars (ϵ_{s1}^{crp}), and CFRP (ϵ_f^{crp}) at cracking state as follows:

$$\epsilon_{c,top}^{crp} = \epsilon_{c,bot}^{crp} \cdot y_p / (h - y_p) \quad (VI.A4b)$$

$$\epsilon_{s2}^{crp} = \epsilon_{c,top}^{crp} \cdot (y_p - d_{s2}) / y_p \quad (VI.A4c)$$

$$\epsilon_{s1}^{crp} = \epsilon_{c,top}^{crp} \cdot (d_{s1} - y_p) / y_p \quad (VI.A4d)$$

$$\epsilon_f^{crp} = \epsilon_{c,top}^{crp} \cdot (d_f - y_p) / y_p \quad (VI.A4e)$$

The strains in bottom steel rebars (ϵ_{s1}^{yp}), concrete compressive top fiber ($\epsilon_{c,top}^{yp}$), top steel rebars (ϵ_{s2}^{yp}), and CFRP (ϵ_f^{yp}) at yielding state are as follows:

$$\epsilon_{s1}^{yp} = f_{sy1} / E_{s1} \quad (VI.A4f)$$

$$\epsilon_{c,top}^{yp} = \epsilon_{s1}^{yp} \cdot C_{yp} / (d_{s1} - C_{yp}) \quad (VI.A4g)$$

$$\epsilon_{s2}^{yp} = \epsilon_{s1}^{yp} \cdot \frac{(C_{yp} - d_{s2})}{(d_{s1} - C_{yp})} \leq \epsilon_{s2}, \text{ otherwise, } \epsilon_{s2}^{yp} = \epsilon_{s2} = f_{sy2} / E_{s2} \quad (VI.A4h)$$

$$\epsilon_f^{yp} = \epsilon_{s1}^{yp} \cdot (d_f - C_{yp}) / (d_{s1} - C_{yp}) \quad (VI.A4i)$$

At the ultimate state, the neutral axis depth (C_{up}), stress in bottom (σ_{s1}^{up}) and top (σ_{s2}^{up}) steel rebars, and stress in the CFRP reinforcement (σ_f^{up}) are calculated as:

$$C_{up} = (A_{s1} \cdot \sigma_{s1}^u + A_f \cdot E_f \cdot \epsilon_f^{up} - A_{s2} \cdot \sigma_{s2}^{up}) / k_1 \cdot f_{cm} \cdot b \quad (VI.A4j)$$

$$\sigma_{s1}^{up} = \min(\epsilon_{c,top}^{up} \cdot (d_{s1} - C_{up}) / C_{up}, f_{sy1} / E_{s1}) \cdot E_{s1} \quad (VI.A4k)$$

$$\sigma_{s2}^{up} = \min(\epsilon_{c,top}^{up} \cdot (C_{up} - d_{s2}) / C_{up}, f_{sy2} / E_{s2}) \cdot E_{s2} \quad (VI.A4l)$$

$$\sigma_f^{up} = (\epsilon_f^{up} + \epsilon_{fp}) \cdot E_f \quad (VI.A4m)$$

In the above equations, n_f, n_{s1} and n_{s2} are the modular ratios of the CFRP, bottom and top steel rebars, respectively; f_{cm} is the concrete compressive strength; $\epsilon_{c,top}^{up}$ is the concrete top fiber strain at ultimate state; E_c is the concrete modulus of elasticity; ϵ_{s2} , f_{sy1} , and f_{sy2} are the yielding strain in top steel rebars, and yielding strengths in bottom and top steel rebars, respectively; E_{s1} and E_{s2} are the elastic moduli of bottom and top steel rebars, respectively; ϵ_{fp} and ϵ_f^{up} are the prestrain and the CFRP strain at failure, respectively; A_f and E_f are the CFR area and modulus of elasticity, respectively; $\varphi_{cr,p}$, $\varphi_{y,p}$, and $\varphi_{u,p}$ are the curvatures at cracking, yielding, and ultimate states, respectively. The C_{up} is calculated iteratively where its exact value is obtained at the balance between the tensile and compressive forces.

PAPER VII

LONG-TERM FLEXURAL BEHAVIOR OF RC SLABS WITH CFRP SYSTEMS UNDER SUSTAINED STRESS AND ENVIRONMENTAL CONDITIONING

REFERENCE: *Aloys Dushimimana, João M. Pereira, Luís Correia, José Sena-Cruz. Short- and long-term deflections of RC slabs strengthened with CFRP systems under sustained load and different environments. To be submitted to a journal.*

ABSTRACT: This work investigates the combined effects of sustained loading and environmental conditioning on the evolution of mid-span deflection of RC slabs strengthened with various non-prestressed and prestressed CFRP-based techniques. Each slab was exposed to a sustained load (approximately 40 % of its ultimate

capacity), with 8 slabs exposed to laboratory-based conditioning, and 16 slabs exposed to outdoor conditioning, for up to 6 years. The laboratory-based conditioning included: (i) a climatic chamber at 20 °C and 55 % relative humidity and (ii) immersion in tap water at 20 °C. The outdoor conditioning included four different locations in Portugal promoting different levels of humidity, temperature fluctuations, freeze-thaw cycles, and UV radiation. The instantaneous and time-dependent mid-span deflections of each slab was recorded and compared with developed predictive models. It is found that CFRP axial stiffness and CFRP strengthening technique play a significant role in deflection control; the predicted deflections agree with those monitored, and the developed time-dependent factor curves can play an important role towards establishing simplified predictive models.

KEYWORDS: Long-term deflection, Reinforced concrete slab, CFRP, Hygrothermal conditioning, Outdoor environment, creep, analytical model

1. INTRODUCTION

Reinforced concrete (RC) is widely used in the construction of various types of structures. However, RC structures deteriorate over time due to aging, poor design, structural distress, or changes in functional requirements. In such cases, strengthening RC elements with Carbon Fiber Reinforced Polymer (CFRP) composites can significantly enhance both structural performance and durability. CFRP is often chosen over other strengthening materials because of its high strength-to-weight ratio, excellent stiffness, and resistance to harsh environmental conditions [1].

Externally Bonded Reinforcement (EBR) and Near-Surface Mounted (NSM) are two common techniques for strengthening RC structures with CFRP composites. In EBR, CFRP laminates are adhesively bonded to the surface of the concrete, whereas NSM involves embedding CFRP laminates into pre-cut grooves within the concrete cover [1]. In some cases, active strengthening systems are required. Therefore, prestressed strengthening methods based on mechanical anchorage (MA) or gradient anchorage (GA) systems have been adopted to enhance load carrying capacity and reduce crack formation and growth, as discussed in [2]. Numerous studies have demonstrated the effectiveness of these techniques, establishing CFRP laminates as a reliable solution for improving the performance and extending the service life of RC structures under a wide range of conditions [1].

The long-term performance of RC structures strengthened with CFRP composites has been widely investigated through experiments on beams and slabs subjected to constant sustained loads for extended durations. Studies

have evaluated strengthened RC elements under sustained loading for periods ranging from 200 days [3], 300 days [4], 400 days [5], 2 years [6], to 6 years [7], and beyond, to assess performance under prolonged stress conditions. These investigations have provided critical insights into the behavior of CFRP-strengthened RC members.

Parameters such as mid-span instantaneous and long-term deflections, concrete creep coefficients, and CFRP strains were addressed in previous studies, taking into account both sustained loading and various environmental conditioning. Key findings from selected studies are as follows: (i) the combined effect of freeze-thaw cycles and sustained loading can reduce ultimate load capacity by up to 8 % after 3 years [8], (ii) the concrete creep coefficient varies depending on the type of strengthening and exposure condition [9], (iii) strengthened beams exhibit lower long-term deflections compared to non-strengthened beams after nearly 2 years of conditioning at 19–24 °C and 30–59 % RH [5], (iv) a hot-wet environment significantly affects the durability performance of RC beams with CFRP strengthening systems [10], and both increased temperature and sustained loading contribute to higher long-term deflections [11,12], and (v) long-term deflections may reach 42 % to 46 % of the instantaneous deflection after 6 months under various sustained loading levels [3].

Existing works, e.g., [5,12–16], have also proposed and/or developed analytical models to predict the long-term flexural behavior of RC structures strengthened with CFRP composites under sustained loading, addressing both prestressed and non-prestressed CFRP systems. Findings generally show that the age-adjusted effective modulus method (AEMM) is a reliable approach for predicting long-term flexural behavior of such structures. Some models incorporate the long-term behavior of CFRP using a CFRP creep coefficient, e.g., [14,17]. Additionally, the creep behavior of the adhesive layer can be accounted for by modifying the CFRP behavior using an interface creep coefficient, e.g., [12,18].

The ACI 318R-19 guideline [19] also offers a simplified method for estimating time-dependent deflections of non-prestressed members. This approach includes the effect of a time-dependent factor and compression reinforcement ratio and has been applied to RC members strengthened with CFRP, e.g., [5,20]. Similarly, the *fib* MC (2020) [21] provides guidance on deflection prediction in RC members strengthened with CFRP using both EBR and NSM techniques. Among all contributing factors, concrete creep is widely considered the primary cause of long-term deflection. Accordingly, both the *fib* MC (2020) [21] and ACI 318R-19 [19] offer comprehensive methodologies for evaluating concrete creep effects.

While substantial knowledge exists regarding concrete creep and shrinkage, further research is required to better understand their impact on the long-term flexural performance of RC structures strengthened with CFRP systems. In particular, limited research has been conducted on the long-term behavior of RC slabs strengthened with prestressed CFRP system, which are also not yet addressed in existing standards, highlighting the need for further investigation.

To address this gap, the present study examined 24 RC slabs strengthened with either prestressed or non-prestressed CFRP. Each slab was subjected to a sustained load of approximately 40 % of its ultimate capacity and conditioned in six different environments (both indoor and outdoor). Mid-span deflections were monitored for up to six years, and analytical models were developed and validated against the experimental results. Additionally, a creep- and shrinkage-based deflection model was established to extrapolate deflection trends over a 50-year period. Using this model, the ACI 318R-19 [19] time-dependent approach was adopted to generate time-dependent factor curves, which may play a key role in the future development of generalized models, especially those accounting for complex outdoor environmental conditions.

2. RESEARCH METHODS

This study involved both experimental and analytical investigations of short- and long-term deflections of one-way RC slabs strengthened using various systems:

- (i) Externally bonded reinforcement (EBR),
- (ii) Near-surface mounted (NSM),
- (iii) EBR prestressed via mechanical anchorage (EBR-MA),
- (iv) EBR prestressed via gradient anchorage (EBR-GA).

The slabs were exposed to different conditioning environments, and their midspan deflections were monitored experimentally and predicted analytically using multiple approaches, as outlined in **Figure VII.1**.

The experimental program includes a description of the RC slabs, details of the conditioning environments, and the procedures used to monitor deflections over a 6-year period. For the analytical predictions, the methods used to estimate both the instantaneous and long-term deflections are presented. Instantaneous deflections were predicted using the method proposed by Gilbert [22] and the tension stiffening model from *fib* MC (2020)

[21]. Long-term deflections were predicted using the Age-adjusted Effective Modulus Method (AEMM), incorporating a modified tension stiffening model, and the ACI 318R-19 [19] approach.

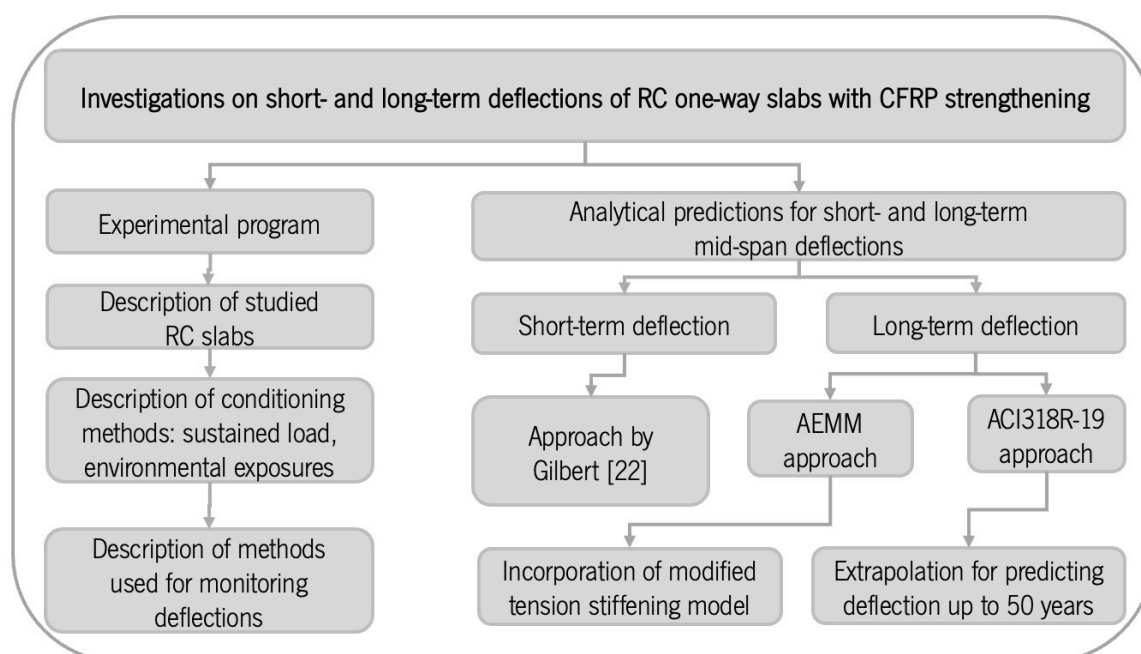


Figure VII.1: Workflow on short- and long-term deflection monitoring and prediction.

2.1. Experimental program

2.1.1. Description of the slabs and sustained loading

The properties and types of the strengthened RC slabs are presented in **Table VII.1**, while their cross-sectional dimensions and geometries are shown in **Figure VII.2a,b**. The slabs were cast using a single concrete batch with the following properties: a maximum aggregate size of 12.5 mm, cement type of CEM II/A-L 42.5R, water-to-cement ratio equal to 0.4, slump ranging between 160 and 210 mm (slump class S4), exposure class XC4(P), and concrete class of 30/37 MPa (C30/37). Further details about the properties of the slabs can be found in [23].

Approximately one year after casting, the slabs were simultaneously loaded (each with a sustained load of 23.8 kN using a four-point bending configuration (see **Figure VII.2c**) and conditioned in different environments. This setup aimed to investigate the combined effects of sustained loading and environmental exposure on mid-span deflection over time.

Table VII.1. Properties of the investigated RC slabs with CFRP strengthening systems [23].

Spec.	CFRP LS [mm ²]	Anchorage type	ε_{fp} [%]	E_s [GPa]		f_y [MPa]		f_t [MPa]		E_f [MPa]	f_f [GPa]	K_{CFRP} [N]
				$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$	$\Phi 6$	$\Phi 8$			
EBR	100×1.2	-	-	219.9 (1.6)	209.3 (7.7)	543.9 (1.8)	551.2 (0.8)	670.0 (0.9)	635.4 (0.5)	187.2 (0.9)	2895.2 (4.4)	22.46e6
NSM	10×1.4 ^a	-	-	241.4 (5.3)	228.1 (3.2)	528.2 (0.7)	581.3 (0.6)	687.1 (0.7)	698.3 (0.3)	185.0 (1.5)	2858.2 (2.0)	10.36e6
EBR-MA	50×1.2	Mechanical	0.4	219.9 (1.6)	209.3 (7.7)	543.9 (1.8)	551.2 (0.8)	670.0 (0.9)	635.4 (0.5)	184.3 (2.9)	2980.3 (4.1)	11.06e6
EBR-GA	50×1.2	Gradient										

Notes: ^a: 4 strips each with 10mm×1.4mm were used; values in parentheses are coefficient of variation in %, E_s and E_f , and f_t and f_f are elastic moduli and ultimate tensile strengths of steel and CFRP, respectively, ε_{fp} and f_y are initial prestrain and yielding strength of CFRP and steel, respectively, Spec.: specimen, K_{CFRP} : CFRP axial stiffness, LS: laminate strip.

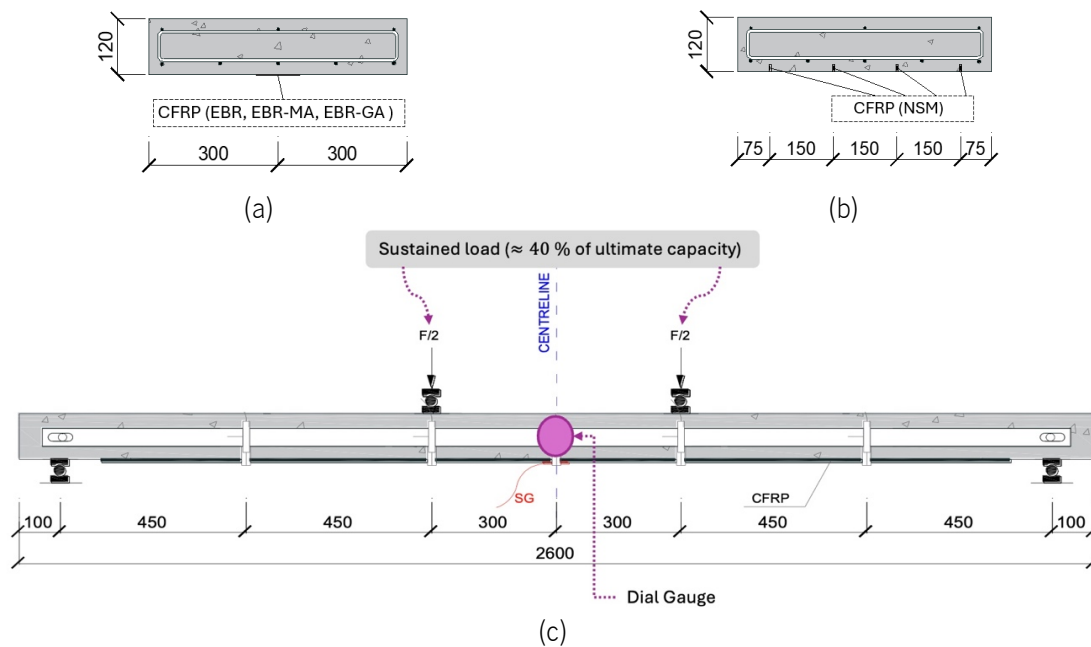


Figure VII.2. Details of the slabs studied: (a) section representing RC slab with EBR, EBR-MA, and EBR-GA; (b) section for the slabs with NSM technique; (c) full RC slab representative of the slabs in (a) and (b) with sustained load applied under 4-point bending test configuration. Note: all units in [mm].

2.1.2. Description of environmental conditioning and monitoring works

This study employed 6 different environmental exposure conditions, comprising of two laboratory-based and four outdoor environments. The former condition included: (i) specimens (**Figure VII.3a**) conditioned at 20 °C and 55 % relative humidity (E1), and (ii) specimens (**Figure VII.3b**) fully immersed in tap water at 20 °C (E2).

SHORT- AND LONG-TERM ANALYSIS OF FLEXURAL BEHAVIOR OF RC SLABS WITH CFRP SYSTEMS UNDER SUSTAINED STRESS AND ENVIRONMENTAL CONDITIONING

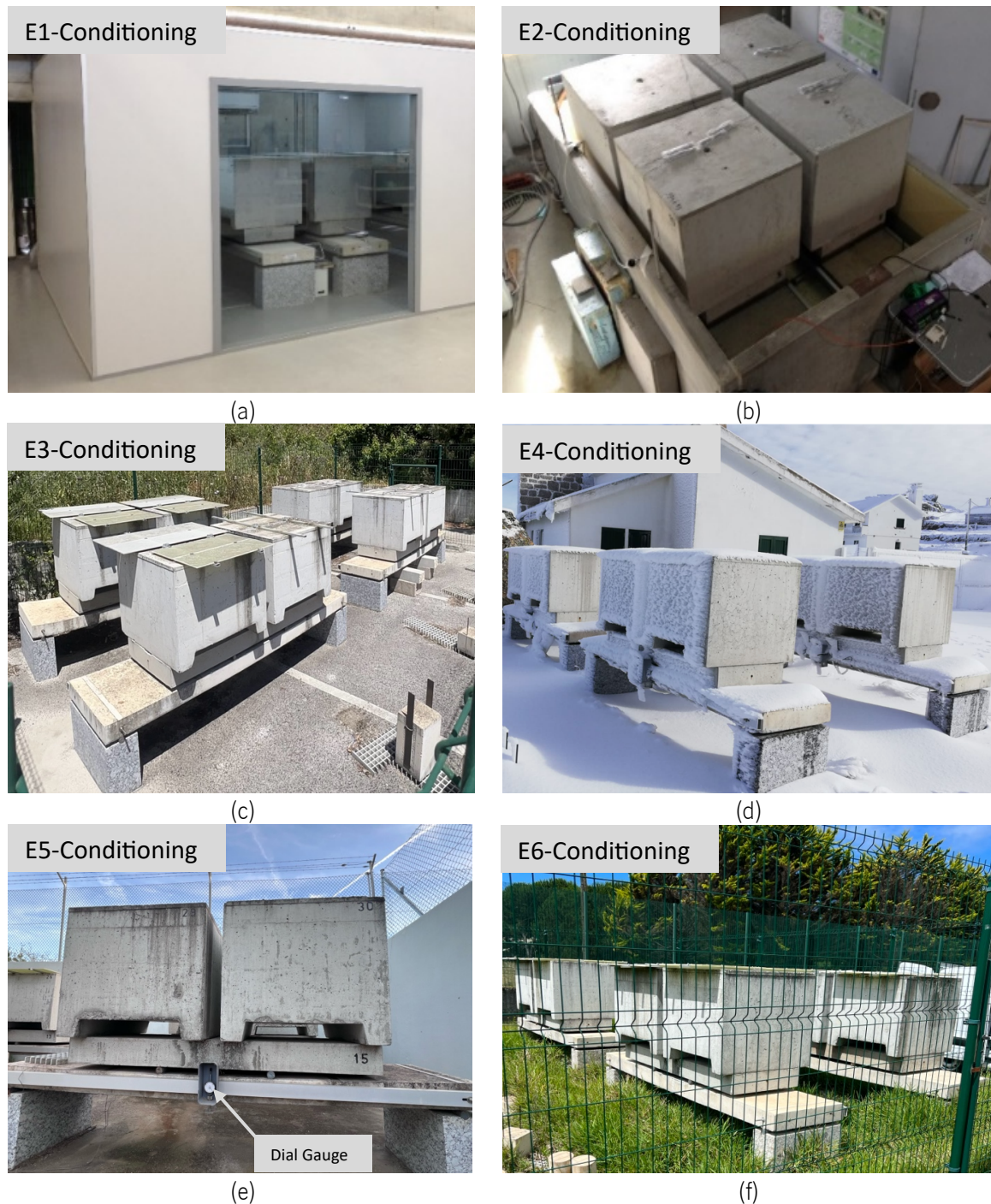


Figure VII.3. Investigated slabs in different environments: (a) conditioning at 20 °C and 55 % relative humidity; (b) conditioning at 20 °C under tap water immersion; outdoor environments: (c) Lisbon station; (d) Serra da Estrela station; (e) Elvas station; and (f) Viana do Castelo station.

For the outdoor environments, four regions in Portugal were selected, each representing a dominant degradation agent based on regional characteristics: (i) E3-Lisbon (**Figure VII.3c**): mild Mediterranean climate in urban area, resulting in elevated atmospheric carbon dioxide; (ii) E4-Serra da Estrela (**Figure VII.3d**): highest mountain in Portugal, high thermal amplitudes, lowest temperatures in Portugal, UV radiation, and freeze-thaw; (iii) E5-Elvas (**Figure VII.3e**): warm temperature climate, very high thermal amplitudes, highest temperature in Portugal, and UV radiation; and (iv) Viana do Castelo (E6, **Figure VII.3f**): maritime moist environment-Atlantic Ocean, with high salinity and chlorides.

Although the outdoor environments were carefully selected to represent dominant degradation agents, field exposures are inherently uncontrollable. As a result, other degradation mechanisms were likely present, and synergistic effects may have occurred. However, the influence of these additional agents was likely minor compared to the primary agent associated with each region.

While in the case of E1 and E2, the mid-span deflection was continuously measured with an LVDT and periodically with a dial gauge (see **Figure VII.2c**), in the case of the outdoor environments (E3 to E6), the mid-span deflection was periodically measured using a dial gauge installed, as shown in **Figure VII.3e**.

2.2. Prediction of short- term deflection

The short-term analysis of RC slab can be performed through section analysis, considering both uncracked and cracked states, from which the short-term deflection is calculated using the weighted average method proposed by *fib* MC (2020) [21]. The material constitutive models are defined as follows:

- (i) Concrete compressive behavior is assumed linear up to the yielding of steel, simplifying the calculation procedure [24]. Beyond yielding, the behavior is modelled following the *fib*90 bulletin [1], using a rectangular stress block, characterized by a multiplier on the compressive strength to determine the stress distribution, and a ratio relating the equivalent stress block to the neutral axis depth;
- (ii) Steel is assumed to exhibit a perfectly elasto-plastic behavior [24];
- (iii) CFRP laminate is considered linear elastic up to failure [24].

Referring to **Figure VII.4**, and following the approach by Gilbert [22], the neutral axis depths under uncracked (x_1) and cracked (x_2) states can be determined based on the material properties (concrete, steel, and CFRP

laminate) and section geometry. These values, x_1 and x_2 , are then used to calculate the second moments of inertia for the uncracked (I_1) and cracked (I_2) sections, respectively. According to Gilbert [22], the short-term strain (ϵ_1) and curvature (k_1) for the uncracked section are calculated using Eq. (VII.1) and Eq. (VII.2), respectively. Similarly, the short-term strain (ϵ_2) and curvature (k_2) for the cracked section are obtained using Eq. (VII.3) and Eq. (VII.4), respectively.

$$\epsilon_1 = \frac{B_1 M + I_1 N}{A_1 I_1 - B_1^2} \quad (\text{VII.1})$$

$$k_1 = \frac{A_1 M + I_1 N}{A_1 I_1 - B_1^2} \quad (\text{VII.2})$$

$$\epsilon_2 = \frac{B_2 M + I_2 N}{A_2 I_1 - B_2^2} \quad (\text{VII.3})$$

$$k_2 = \frac{A_2 M + I_2 N}{A_2 I_1 - B_2^2} \quad (\text{VII.4})$$

In these equations, A_i , B_i , and I_i are the section area, first moment of inertia, and second moment of inertia, respectively; i stands for 1 or 2 indicating the uncracked or cracked section, respectively. Furthermore, M and N are the stress resultants for the cross section (axial force and bending moment) with respect to the top fiber of the cross-section [17,22].

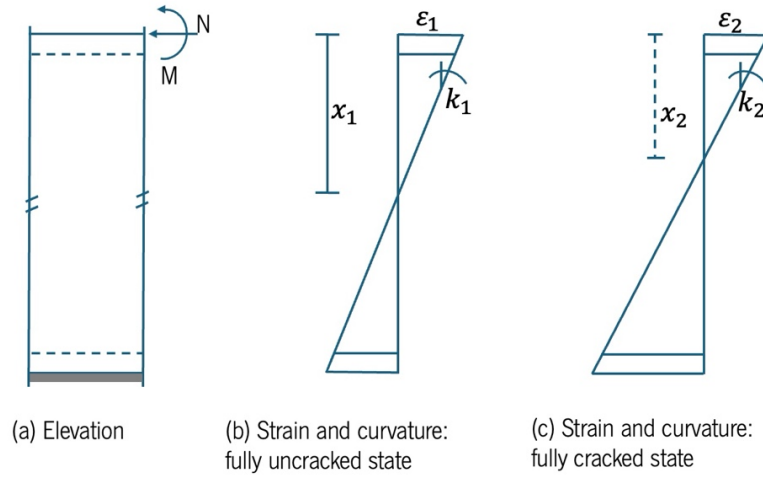


Figure VII.4. (a) Section elevation, and distribution of short-term strain and curvature under (b) fully uncracked and (c) fully cracked states [22].

Following the method provided in the *fib* MC (2020) [21], deflection can be predicted using Eq. (VII.5), where the deflections for the uncracked (δ_1) and cracked (δ_2) sections are determined using Eq. (VII.6) and Eq.

(VII.7), respectively. The terms $[M/E_{ci}(t_0) \cdot I_1]$ and $[M/E_{ci}(t_0) \cdot I_2]$, representatives of the curvatures k_1 and k_2 were calculated from Eq. (VII.2) and Eq. (VII.4), respectively. The interpolation coefficient (ζ) incorporating the tension stiffening effect can also be determined using Eq. (VII.8) as per *fib* MC (2020) [21].

$$\delta_{t0} = (1 - \zeta) \cdot \delta_1 + \zeta \cdot \delta_2 \quad (\text{VII.5})$$

$$\delta_1 = k_M \cdot l^2 \cdot \frac{M}{E_{ci}(t_0) \cdot I_1} \quad (\text{VII.6})$$

$$\delta_2 = k_M \cdot l^2 \cdot \frac{M}{E_{ci}(t_0) \cdot I_2} \quad (\text{VII.7})$$

$$\zeta = 1 - \beta \cdot \left(\frac{M_{cr}}{M} \right)^2 \quad (\text{VII.8})$$

In these equations, M_{cr} is the cracking moment obtained from section analysis considering the effect of CFRP strengthening and prestressing, and β is a coefficient to account for influence of loading type and duration on the average strain (equal to 0.5 for sustained loads [21]); $E_{ci}(t_0)$ is the concrete modulus of elasticity at loading, estimated following the *fib* MC (2020) [21]. Furthermore, l and k_M are the span length and the coefficient depending on the load bearing system, respectively; $k_M = (1/4) - [(a/l)^2/3]$ for a four-point bending test configuration, where a is the shear span.

2.3. Prediction of long-term deflection using AEMM

The long-term analysis of RC slab can be performed using section analysis, considering time-dependent uncracked and cracked states. From these, the time-dependent moments of inertia are obtained by incorporating the evolving properties of the relevant materials, namely, concrete, steel, CFRP and adhesive, along with relevant section characteristics. These moments of inertia are then used in the modified tension stiffening model [3] to compute the time-dependent effective moment of inertia. The resulting effective moment of inertia, combined with the effective moduli of the materials, is then used to determine the time-dependent deflection.

2.3.1. Concrete

The AEMM can be used to model the time-dependent behavior of concrete using Eq. (VII.9), where $E_c(t, t_0)$ is the time-dependent effective concrete modulus of elasticity at time (t), measured starting at the time of the application of the sustained loading (t_0), $\varphi(t, t_0)$ and χ are the time-dependent creep coefficient and the aging coefficient, respectively. The χ varies between 0.7 and 0.9, and in many existing studies its value is usually

taken as 0.8 for normal strength concrete, e.g., [3,11,22]. The $\varphi(t, t_0)$ can be determined experimentally by recording the long-term concrete strains under a sustained stress, if no tests performed, it can be found from existing models.

$$E_c(t, t_0) = \frac{E_{ci}(t_0)}{1 + \chi \cdot \varphi(t, t_0)} \quad (\text{VII.9})$$

The creep coefficient $\varphi(t, t_0)$ can be estimated referring to *fib* MC (2020) [21] as shown in Eq. (VII.10). The terms $\varphi_{bc}(t, t_0)$ and $\varphi_{dc}(t, t_0)$ are the basic and drying creep coefficients, respectively, and further details on their determination is provided in [21]. The parameters ξ_{bc1} and ξ_{dc1} are equal to 1.0 if no tests on the considered concrete were carried out, otherwise, they are determined from a least square fit to the measured data. Furthermore, the code also provides thorough details on how to incorporate the effects of temperature variation in Eq. (VII.10).

$$\varphi(t, t_0) = \xi_{bc1} \cdot \varphi_{bc}(t, t_0) + \xi_{dc1} \cdot \varphi_{dc}(t, t_0) \quad (\text{VII.10})$$

2.3.2. CFRP laminate

The time-dependent effective modulus of the CFRP laminate, $E_f(t, t_0)$, can be calculated using Eq. (VII.11) [12,14], where E_f is the initial CFRP elastic modulus and $\varphi_{CFRP}(t, t_0)$ is the CFRP creep coefficient, which can be determined using Eq. (VII.12) according to [12,14].

$$E_f(t, t_0) = \frac{E_f}{1 + \varphi_{CFRP}(t, t_0)} \quad (\text{VII.11})$$

$$\varphi_{CFRP}(t, t_0) = 0.02 \cdot \left(1 - e^{-\frac{-(t-t_0)}{130}}\right) \quad (\text{VII.12})$$

2.3.3. Interface behavior

The time-dependent deformation occurred at the CFRP-to-concrete interface (throughout the adhesive) can be considered by introducing an interface creep coefficient, $\varphi_a(t, t_0)$, defined in Eq. (VII.13) proposed in [18] and adopted in [12], which in turn can be incorporated in the model as an additional creep coefficient of the CFRP laminate [12], leading to the total time-dependent effective modulus of the CFRP as Eq. (VII.14) [12].

$$\varphi_a(t, t_0) = 1.65 \cdot (1 - e^{-\frac{(t-t_0)}{160}}) \quad (\text{VII.13})$$

$$E_{fa}(t, t_0) = \frac{E_f(t, t_0)}{1 + \varphi_a(t, t_0)} \quad (\text{VII.14})$$

2.3.4. Section analysis

With the time-dependent material properties defined in **Sections 2.3.1 to 2.3.3**, a section analysis accounting for time-dependent effects can be performed. The application of **Eq. (VII.9) to Eq. (VII.14)** leads to modification of the transformed section properties by updating the modular ratios of the tension steel reinforcement [$\alpha_{s1}(t, t_0) = E_{s1}/E_c(t, t_0)$], compression steel reinforcement [$\alpha_{s2}(t, t_0) = E_{s2}/E_c(t, t_0)$] and CFRP laminate [$\alpha_f(t, t_0) = E_f/E_c(t, t_0)$]. Here, E_{s1} , E_{s2} and E_f represent the elastic moduli of bottom steel rebars and top steel rebars, and CFRP laminate in tension at the time of loading, respectively. It should be noted that the neutral axis depth should ideally be updated at each time step, as it depends on the time-dependent modular ratios. However, for simplification, the neutral axis position can be assumed constant over time, which has been shown to yield to acceptable accuracy in previous studies, e.g., [12,22].

Referring to **Figure VII.5**, the time-dependent effective moment of inertia $I_e(t, t_0)$ can be determined following the modified tension stiffening model [3,17], as defined in **Eq. (VII.15)**. This calculation incorporates the time-dependent uncracked [$I_1(t, t_0)$] and cracked [$I_2(t, t_0)$] second moments of inertia, both of which are updated based on the time-dependent modular ratios. The values $I_1(t, t_0)$ and $I_2(t, t_0)$ correspond to the curvatures $k_{e1}(t, t_0)$ and $k_{e2}(t, t_0)$ respectively, as shown in **Figure VII.5**. Here, k_1 and k_2 refer to short-term curvatures defined earlier, while k_{t1} and k_{t2} represent the time-dependent curvatures in the uncracked and cracked states, respectively. It should also be noted that the modular ratio for the CFRP laminate [$\alpha_f(t, t_0)$] considers the time-dependent constitutive law of CFRP, as defined in **Eq. (VII.11)**, along with the interface behavior, as introduced in **Eq. (VII.13)**.

Knowing the $I_e(t, t_0)$ and $E_c(t, t_0)$ (from **Section 2.3.1**), the time-dependent curvature term [$k_e(t, t_0) = M/E_c(t, t_0) \cdot I_e(t, t_0)$] can be calculated and used to determine the total mid-span deflection [$\delta_e(t, t_0)$] accounting for both the instantaneous and time-dependent components, using **Eq. (VII.16)** as per to *fib* MC (2020) [21].

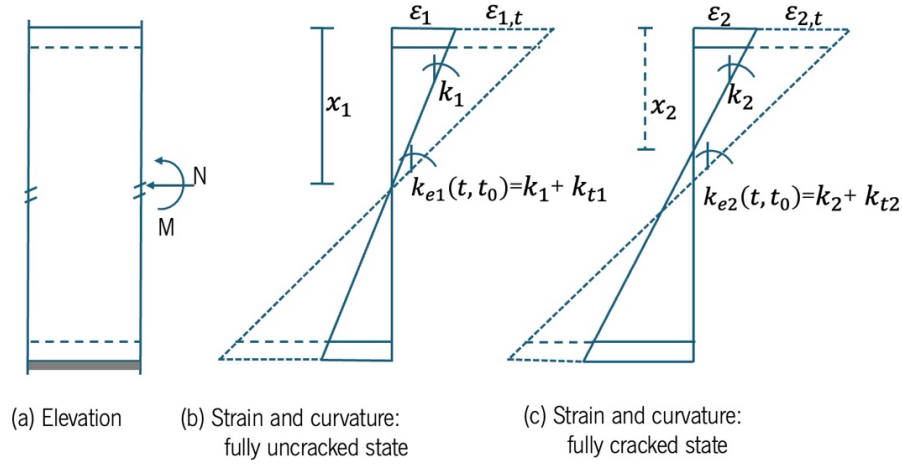


Figure VII.5. (a) Section elevation, and distribution of time-dependent strain and curvature under (b) uncracked and (c) fully cracked states.

$$I_e(t, t_0) = \left(\frac{M_{cr}}{M}\right)^3 \cdot \beta_d \cdot I_1(t, t_0) + \left[1 - \left(\frac{M_{cr}}{M}\right)^3\right] \cdot I_2(t, t_0) \leq I_1(t, t_0) \quad (\text{VII.15})$$

$$\delta_e(t, t_0) = k_M \cdot l^2 \cdot \frac{M}{E_c(t, t_0) \cdot I_e(t, t_0)} \quad (\text{VII.16})$$

where $\beta_d = \alpha_b \cdot (E_f/E_s + 1)$; α_b is a bond dependent coefficient, and E_f/E_s is the ratio of CFRP to steel modulus of elasticity.

2.4. Prediction of long-term deflection using ACI 318R-19

ACI 318R-19 standard [19] has provided a simplified expression in Eq. (VII.17), considering creep and shrinkage effects, for predicting the long-term deflection (λ_Δ) for non-prestressed flexural members.

$$\lambda_\Delta = \delta_{inst} \left(\frac{\xi}{1 + 50\rho'} \right) \quad (\text{VII.17})$$

The terms δ_{inst} , ξ , and ρ' are the instantaneous deflection, time-dependent factor for sustained load, and the compression reinforcement ratio, respectively. The term ξ varies with time for up to 5 years, thereafter, becoming constant.

For RC flexural members strengthened with CFRP systems, the effect of strengthening is not explicitly considered in Eq. (VII.17). However, since the long-term deflection is primarily governed by the concrete compression zone, the application of ACI 318R-19 remains valid even for strengthened members (given that CFRP is applied in the

tension zone to control flexural behavior). The *fib* MC (2020) [21] also states that concrete compression zone is the dominant factor influencing long-term deflection. This is because the stresses at the CFRP-concrete interface are typically low under service loads, contributing minimally to creep development, and thereby supporting the continued applicability of Eq. (VII.17). Several existing studies have also applied Eq. (VII.17) to RC members strengthened with CFRP systems, e.g., [5,20].

2.5. Creep and shrinkage deflection model

The AEMM approach introduced in Section 2.3 relies heavily on the modified tension stiffening model [3], particularly on the bond-dependent coefficient, which has not yet been accurately calibrated for different strengthening systems. This is especially true for prestressed CFRP systems, which are currently not addressed in existing standards. Additionally, the ACI 318R-19 provision presented in Section 2.4 has been shown to significantly overpredict long-term deflections, as noted in several studies, e.g., [5,25].

In outdoor environments, complex degradation mechanisms, including synergistic effects that are not accounted for in current standards and guidelines, make the development of accurate predictive models particularly challenging. Even when such models are developed, they are often computationally intensive and difficult to implement in design practice [21]. For this reason, the ACI 318R-19 formula remains valuable for design purposes due to its simplified nature. It can be used to derive time-dependent factor curves that define the long-term behavior of RC members under various strengthening systems and conditioning types. In doing so, it indirectly incorporates complex environmental effects.

These time-dependent factor curves can also be extended to longer durations using reliable viscoelastic models, such as the generalized Kelvin (GK) chain model. This model assumes that long-term deflections are primarily governed by the viscoelastic behavior of concrete, which aligns with existing guideline [1,19]. As a result, the curves can be used to extrapolate deflections over extended periods, such as up to 50 years. Studies that follow this approach may lead to simplified predictive models, which require only the time-dependent factor curve and the compression reinforcement ratio.

The GK model is schematically presented in Figure VII.6 and can be expressed as a Dirichlet series in Eq. (VII.18) [26], where E_0 represents the initial elastic response, $E_{i(i=1,\dots,N)}$ represents the elastic component of each element, and $\eta_{i(i=1,\dots,N)}$ represents the viscous component, where N is the number of GK chains considered. In this formulation, D_0 and D_i are comparable to E_0 and E_i , respectively, since the long-term

deflection-time curve is primarily governed by the viscoelastic behavior of concrete [1,21]. Based on the GK model, a creep deflection model can be developed. This can be achieved using machine learning algorithms or optimization tools such as MATLAB's Optimization Toolbox to minimize the cost function defined in Eq. (VII.19). The resulting optimized parameters can then be used in the creep deflection function described in Eq. (VII.20) to estimate deflection at any time.

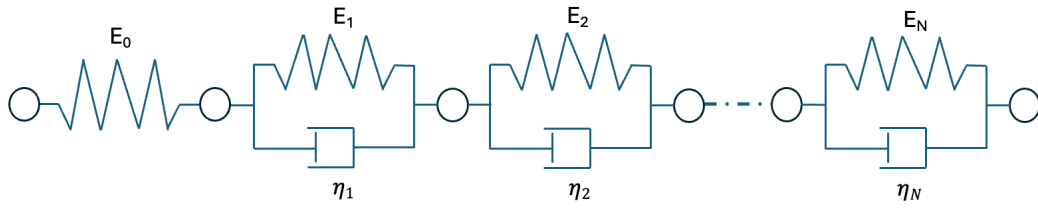


Figure VII.6. Idealized schematic representation of Generalized Kelvin chain model (adapted from [26]).

$$D(t) = D_0 + \sum_{i=0}^n D_i (1 - e^{-t/(\tau_i D_i)}) \quad (\text{VII.18})$$

$$f = \sum_{j=1}^m (D(t) - \mathfrak{D}(t))^2 \quad (\text{VII.19})$$

$$D_c(t) = \sum_{i=0}^n D_i (1 - e^{-t/(\tau_i D_i)}) \quad (\text{VII.20})$$

In these equations, $D(t)$ is the total long-term deflection at time t , n is the number of Kelvin chains, τ_i, D_i and D_0 are the coefficients of Dirichlet series; f is the objective function, m is the number of points employed during optimization process, $\mathfrak{D}(t)$ is the experimentally recorded deflection, $D_c(t)$ is the creep deflection function.

2.6. Prediction of deflection for the studied slabs

The method presented in Section 2.2 was adopted in this study to predict the instantaneous deflection. For the time-dependent deflection prediction, described in Section 2.3, average values of the temperatures and relative humidity, previously published by the authors in [27,28], were used to model the concrete creep coefficient, accounting for the effects of the environmental conditions investigated. That is, the used average peak temperatures and average relative humidities for E3, E4, E5, and E6 were 42.2 °C and 71.93 % RH, 30.65 °C and 69.63 % RH, 42.98 °C and 65.12 % RH, and 35.15 °C and 77.55 % RH, respectively.

The ACI 318R-19 method, introduced in **Section 2.4**, was applied to all tested slab types to evaluate its capability in predicting deflection considering creep and shrinkage effects.

Finally, the approach outlined in **Section 2.5** was used to establish time-dependent factor curves governing each investigated strengthening system under all conditioning types, including both sustained load and environmental exposure. These curves were further used to extrapolate deflections over a time span of up to 50 years. To minimize the cost function defined in **Eq. (VII.19)**, several optimization algorithms from the MATLAB toolbox were tested. Among all, the constrained optimization (*fmincon*) was found to be the most accurate and consistent across all studied environmental conditions and was, therefore, selected to identify the optimal parameters minimizing the cost function. The impact of shrinkage was considered negligible, as loading was applied one year after casting. As a result, shrinkage-induced deflection over time (t) was represented using a simplified model: $\delta_{cr\&sh} = e_{sh_max}(1 - \exp(-k_{sh}t))$, where $e_{sh_max} = 0.002$ represents the maximum shrinkage strain, and $k_{sh}t$ is the shrinkage coefficient, set at $1e-5$.

3. RESULTS AND DISCUSSION

3.1. Experimental short- and long-term deflections

The mid-span deflections obtained from experimental monitoring of the slabs in each of the studied environments are presented in **Figure VII.7**. Regardless of environmental conditioning, the deflection evolution generally followed three distinct stages: (i) instantaneous deflection, (ii) primary creep deflection, and (iii) secondary creep deflection. Slabs strengthened with the NSM system exhibited the largest instantaneous deflection, attributed to the smaller CFRP axial stiffness used (approximately 40 % less compared to that of the EBR system, presenting nearly two-fold). This observation supports the conclusion that a smaller CFRP axial stiffness leads to larger instantaneous deflection.

SHORT- AND LONG-TERM ANALYSIS OF FLEXURAL BEHAVIOR OF RC SLABS WITH CFRP SYSTEMS UNDER SUSTAINED STRESS AND ENVIRONMENTAL CONDITIONING

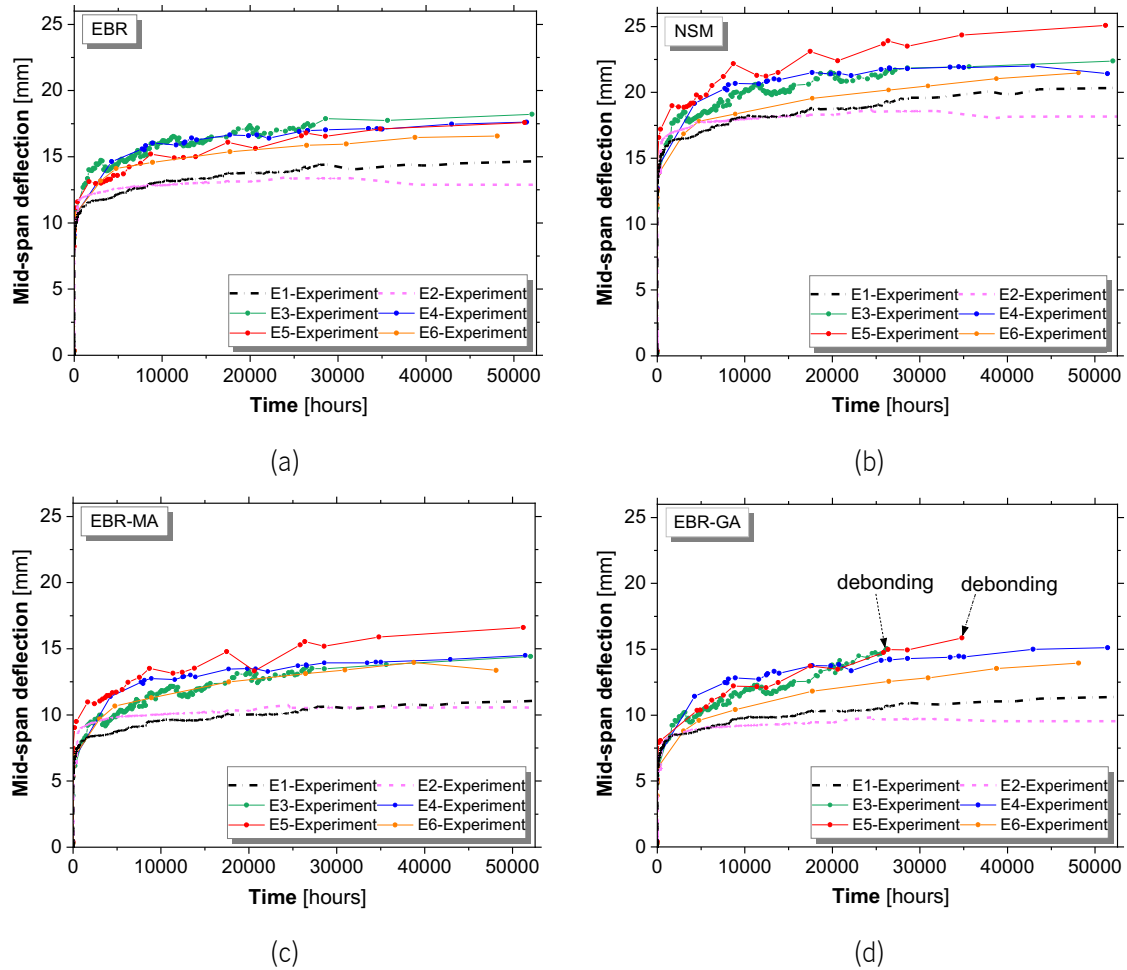


Figure VII.7. Experimental monitored mid-span deflection for the slabs with (a) EBR, (b) NSM, (c) EBR-MA, and (d) EBR-GA systems for up to 6 years.

In contrast, slabs strengthened with prestressed systems (EBR-MA and EBR-GA) exhibited approximately 30 % lower instantaneous deflections compared to the non-prestressed EBR system. These slabs also showed total deflections below 15 mm in all environments after 6 years. This improved performance is primarily attributed to the prestressing action, which helps oppose both dead and applied loads [29].

For the slabs subjected to laboratory-based conditioning (E1 and E2), the deflections were notably smaller than those recorded under outdoor conditions, as shown in **Figure VII.7**. Under E1 (20 °C and 55 % relative humidity), deflections gradually increased over time, reaching approximately 15 mm (EBR), 20 mm (NSM), and 12 mm (EBR-MA and EBR-GA) after 6 years. In contrast, under E2 (20 °C, full water immersion), deflection increases were negligible. The greater deflection observed under E1 is likely due to moisture exchange with the

environment and the presence of air, which may have exacerbated creep deformation. The limited deflection in E2 can be attributed to: (i) restricted moisture migration due to full submersion in water, (ii) minimal thermal stresses due to stabilized temperature, and (iii) absence of drying shrinkage-induced deflection, since the concrete remained fully saturated.

Slabs exposed to outdoor environments generally exhibited higher time-dependent deflections, particularly in regions with high temperatures (E5). The combined effect of elevated CO₂ concentrations and high temperatures (E3) might have also led to the observed increased deflection. Similarly, the region with presence of freeze-thaws (E4), might have led to the observed greater deflections compared to that with the presence of salinity, low humidity and airborne chlorides (E6). However, as previously mentioned, outdoor environments are uncontrollable, which makes it complex and thus challenging to isolate the specific effects of individual exposure conditions on the structural response.

After 6 years, the measured deflection ranges were as follows: (i) slabs with EBR: 16-18 mm; (ii) slabs with NSM: 21-25 mm; (iii) slabs with EBR-MA: 13-16 mm; and (iv) slabs with EBR-GA: 14-16 mm. The instantaneous and total deflections, along with their percentage difference (used to interpret the contribution of creep and shrinkage on the long-term deflection), are presented in **Table VII.2**. Regardless of environmental exposure, the prestressing systems (EBR-MA and EBR-GA) were found to increase the proportion of creep and shrinkage-based deflection compared to the non-prestressed EBR system).

Table VII.2. Comparison of experimental instantaneous and total deflections after 6 years.

	EBR	NSM	EBR-MA	EBR-GA	EBR	NSM	EBR-MA	EBR-GA
	E1-conditioning				E2-conditioning			
δ_{t0}/δ_{6Y}	8.13/14.77	14.15/20.42	5.10/11.17	4.77/11.50	8.94/12.89	13.48/18.17	5.82/10.56	5.33/9.55
Var. (%)	81.7	44.3	119	141.1	44.2	34.8	81.4	79.2
	E3-conditioning				E4-conditioning			
δ_{t0}/δ_{6Y}	8.84/18.21	13.90/22.37	5.26/14.41	4.82/15.05*	9.08/17.61	14.08/21.42	5.33/14.50	5.79/15.11
Var. (%)	106.0	60.9	174.0	-	93.9	52.1	172.1	161.0
	E5-conditioning				E6-conditioning			
δ_{t0}/δ_{6Y}	8.21/17.58	14.07/25.09	6.04/16.60	5.14/15.87**	8.92/17.58	12.45/21.47	5.36/13.37	4.69/13.94
Var. (%)	114.1	78.3	174.8	-	97.1	72.5	149.4	197.2

Notes: δ_{t0} and δ_{6Y} are the instantaneous and deflection after 6 years, and their percentage difference (Var.) to represent the creep & shrinkage-based deflection); * and **: the value recorded after 2 and 4 years (i.e., before debonding), respectively.

The maximum values of the deflections due to creep and shrinkage effects were observed in E3 and E5, with approximately 174 and 175 % increases when compared with the instantaneous deflections, respectively. These findings identify regions dominated by higher temperatures (E5) and elevated CO₂ levels combined with higher

temperatures (E3) as the most aggressive for prestressed systems. Interestingly, the slab with the non-prestressed EBR system exhibited lower deflection increases in E3 (106 %) and E5 (114 %), although still higher than those in other environments. This contrast highlights that the applied prestress in EBR-MA and EBR-GA systems significantly contributed to the observed increase in long-term deflections. This phenomenon can be explained by the increased concrete compression zone resulting from prestressing. As concrete creep is primarily governed by the compression zone [1,21], the added compressive stress led to greater long-term deformation. As a result, the prestressed systems experienced deflection increases of approximately 68 % (E3) and 61 % (E5) relative to the non-prestressed case, attributable pertinently to the effect of enhanced compression.

Notably, debonding failures were observed in the EBR-GA slabs exposed to E3 and E5, as illustrated in **Figure VII.7**. This failure is attributed to several interacting mechanisms. First, epoxy adhesive curing under prestress in the EBR-GA system has been shown to lead to significant reductions in tensile strength, due to increased pore formation during prestressing [30]. This can weaken the adhesive layer, promoting crack initiation and premature debonding [30]. In addition, according to a study by [31], the used epoxy adhesive is known to reduce its mechanical properties by 30 to 40 % and the temperature reaching the adhesive glass transition temperature can also compromise the bond performance, both of which may yield to premature debonding. Furthermore, adhesive softening, thermal expansion mismatches, and reduced mechanical properties under prolonged high temperatures may have further compromised the bond, ultimately resulting in debonding failure.

In contrast to the high deflections seen in E3 and E5, the slabs in E6 exhibited the lowest long-term deflections among the prestressed systems (EBR-MA and EBR-GA). This reduced deflection can be attributed to the high humidity of the environment, which helps retain internal moisture within the concrete and reduces drying creep. As a result, shrinkage-related stresses are minimized, and total creep deformation is limited, leading to smaller deflections over time.

3.2. Predicted short-term deflection

The predicted instantaneous deflections were compared with the corresponding experimental measurements, as presented in **Table VII.3**.

Table VII.3. Comparison of experimental and predicted instantaneous deflection.

	EBR	NSM	MA	GA	EBR	NSM	MA	GA
	E1-conditioning				E2-conditioning			
$\delta_{t0\ (EXP)}/\delta_{t0\ (Pred)}$	8.13/8.45	14.15/15.01	5.10/5.00	4.77/5.44	8.94/8.45	13.48/15.01	5.82/5.00	5.33/5.44
Var. (%)	3.9	6.1	2.0	14.1	5.5	11.4	14.1	2.1
	E3-conditioning				E4-conditioning			
$\delta_{t0\ (EXP)}/\delta_{t0\ (Pred)}$	8.84/8.45	13.90/15.01	5.26/5.00	4.82/5.44	9.08/8.45	14.08/15.01	5.33/5.00	5.79/5.44
Var. (%)	4.4	8.0	4.9	12.9	6.9	6.6	6.2	6.0
	E5-conditioning				E6-conditioning			
$\delta_{t0\ (EXP)}/\delta_{t0\ (Pred)}$	8.21/8.45	14.07/15.01	6.04/5.00	5.14/5.44	8.92/8.45	12.45/15.01	5.36/5.00	4.96/5.44
Var. (%)	2.9	6.7	17.2	5.8	5.3	20.6	6.7	9.7

Notes: $\delta_{t0 (EXP)}/\delta_{t0 (Pred)}$: experimental/predicted instantaneous deflections, and their percentage difference (Var.).

Overall, the results show good agreement between the analytical predictions and experimental observations. The maximum percentage differences observed in environments E1, E2, E3, E4, E5, and E6 were approximately 14 %, 14 %, 13 %, 7 %, 17 %, and 20 %, respectively. These discrepancies are likely attributable to minor differences in sustained loading levels compared to those used in the predictive model. Despite these variations, the observed percentage differences can still be considered relatively small, confirming the reliability and applicability of the adopted prediction approach. Additional data on the short-term deflection analysis are given in APPENDIX VII.1.

3.3. Predicted total deflection using AEMM

The predicted total mid-span deflections are presented in **Figure VII.8**, alongside the corresponding experimental results. For the EBR system (**Figure VII.8a**), the predicted deflection curves under water immersion (E2) and in the region mainly experiencing high yearly temperature exposure (E5) display some deviations in trend compared to the experimental data. Nevertheless, the predictions still show reasonable agreement. For the NSM system (**Figure VII.8b**), similar discrepancies in trend are observed under E2 and E6, although the predicted values remain in good agreement with the experimental results overall.

Very good agreement is observed for the prestressed CFRP strengthening systems. In the case of the EBR-MA system (**Figure VII.8c**), the predicted curve for E2 closely matches the experimentally recorded deflection trend, and predictions under other environmental conditions also show excellent correspondence. For the EBR-GA system (**Figure VII.8d**), minor trend differences are noted under E2 and E6, though the overall agreement with experimental results remains fairly good. It should be noted that due to debonding previously explained in **Figure**

VII.7 for the EBR-GA in E3 and E5, the corresponding results were not shown in **Figure VII.8**, and in the remainder of this work.

The bond-dependent coefficients and ultimate concrete creep coefficients used in the analysis are listed in **Table VII.4**. The bond-dependent coefficients generally converge toward the value of 0.5, as recommended for all FRP reinforcements [17], with the exception of the EBR system, which showed higher values. As expected, higher creep coefficients were observed in high-temperature regions, in alignment with the trends reported in the *fib* MC (2020) [21].

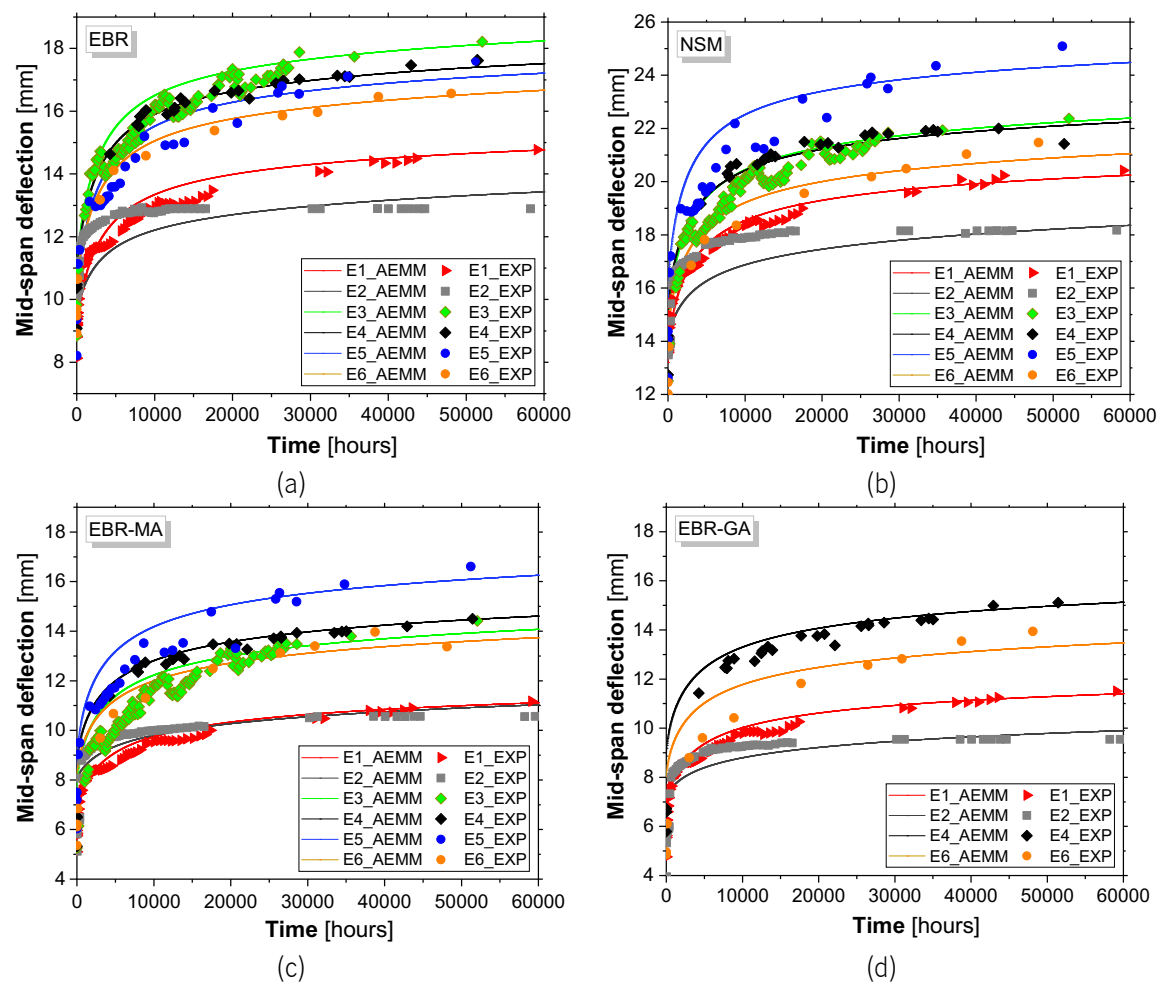


Figure VII.8. Experimental monitored deflections versus analytically predicted deflections for the slabs with (a) EBR, (b) NSM, (c) EBR-MA, and (d) EBR-GA systems for up to 6 years. Note: Due to debonding occurred for the EBR-GA in E3 and E5, the corresponding results were not predicted.

Overall, the AEMM approach demonstrated strong agreement with the experimental results across all systems and environments. Thus, the method is recommended for future studies, especially for gaining insights into the variability of the bond-dependent coefficient (α_b) and for further calibration efforts, to extend its applicability across a wider range of strengthening systems and environmental conditions.

Table VII.4. Bond dependent and concrete creep coefficients.

	EBR	NSM	MA	GA	EBR	NSM	MA	GA
	E1-conditioning				E2-conditioning			
α_b / φ_u	1.23/1.09	0.55/1.09	0.56/1.09	0.57/1.09	1.05/0.47	0.48/0.47	0.42/0.47	0.50/0.47
	E3-conditioning				E4-conditioning			
α_b / φ_u	0.90/1.24	0.48/1.24	0.45/1.24	-	0.90/1.06	0.45/1.06	0.40/1.06	0.40/1.06
	E5-conditioning				E6-conditioning			
α_b / φ_u	1.07/1.40	0.42/1.40	0.40/1.40	-	0.96/1.00	0.49/1.00	0.42/1.00	0.45/1.00

Note: α_b / φ_u : bond dependent coefficient/ maximum concrete creep coefficient

3.4. Predicted long-term deflection using ACI 318R-19

The results obtained using the ACI 318R-19 approach are presented in **Table VII.5**, together with the experimentally recorded deflections at 5 and 6 years, as well as the percentage differences between predicted and measured values. Significant overpredictions were observed in E1, where the ACI 318R-19 method overestimated deflection after 5 years by approximately 71 % (EBR), 320 % (NSM), 57 % (EBR-MA), and 32 % (EBR-GA) compared to the experimental results. Overpredictions were also noted across other environments, as shown in **Table VII.5**.

The largest overpredictions were consistently observed for slabs strengthened with the NSM system, while the smallest discrepancies were recorded for prestressed CFRP systems (except for E3, where the method resulted in underprediction). These findings suggest that the direct application of the ACI 318R-19 standard can be misleading for predicting long-term deflection in CFRP-strengthened RC slabs. Similar conclusions have also been drawn in previous studies (e.g., [5,25]). Additionally, the recorded results show that the ACI 318R-19 deflection limit at 5 years may be insufficient, as deflection measurements continued to increase through year 6.

To reconcile the discrepancies, time-dependent factor curves relevant to the studied slabs were derived and are presented in **Figure VII.9**, along with the standard ACI 318R-19 curve. Regardless of the conditioning type (sustained load and environmental exposure), the standard curve deviates significantly from those derived for

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the tested slabs. Furthermore, it was observed that the derived factors for slabs strengthened using EBR and NSM techniques were lower than those for slabs with prestressed EBR-MA and EBR-GA systems.

Table VII.5. Comparison of 5- and 6-year experimental and ACI 318R-19 predicted creep and shrinkage-based deflection.

	EBR	NSM	MA	GA	EBR	NSM	MA	GA
	E1-conditioning				E2-conditioning			
$\delta_{cr\&sh,5Y}$ (EXP/ACI)	6.31/12.20	5.91/26.36	5.74/9.50	6.4/8.89	3.95/16.65	4.68/25.11	4.89/10.84	3.92/10.47
$\delta_{cr\&sh,6Y}$ (EXP/ACI)	6.52/12.20	6.27/26.36	6.07/9.50	6.73/8.89	3.95/16.65	4.69/25.11	4.74/10.84	3.93/10.47
Var.(% _{6Y})	87.1	320.4	56.5	32.1	321.5	435.4	128.7	166.4
	E3-conditioning				E4-conditioning			
$\delta_{cr\&sh,5Y}$ (EXP/ACI)	8.99/16.47	10.81/25.89	10.24/7.27	12.59/8.98	8.35/16.91	9.13/26.23	8.96/9.92	9.09/10.79
$\delta_{cr\&sh,6Y}$ (EXP/ACI)	9.08/16.47	10.90/25.89	10.52/7.27	13.66/8.98	8.54/16.91	9.15/26.23	9.14/9.92	9.36/10.79
Var.(% _{6Y})	81.4	137.5	30.9	34.3	98.0	186.7	8.5	15.3
	E5-conditioning				E6-conditioning			
$\delta_{cr\&sh,5Y}$ (EXP/ACI)	9.17/15.29	12.14/26.21	10.26/11.24	11.93/9.57	7.57/16.61	9.80/23.20	8.28/9.98	9.84/7.22
$\delta_{cr\&sh,6Y}$ (EXP/ACI)	9.42/15.29	12.54/26.21	10.67/11.24	12.95/9.57	7.79/16.61	10.17/23.20	8.38/9.98	10.22/7.22
Var.(% _{6Y})	62.3	109.0	5.3	26.1	113.2	128.1	19.1	68.5

Note: $\delta_{cr\&sh,XY}$: creep and shrinkage (*cr&sh*)-based deflection obtained after *X* number of years (*Y*); the values are presented as: experimental value/predicted value; Var.(%_{6Y}) is the percentage difference between experimental and ACI 318R-19 prediction after 6 years.

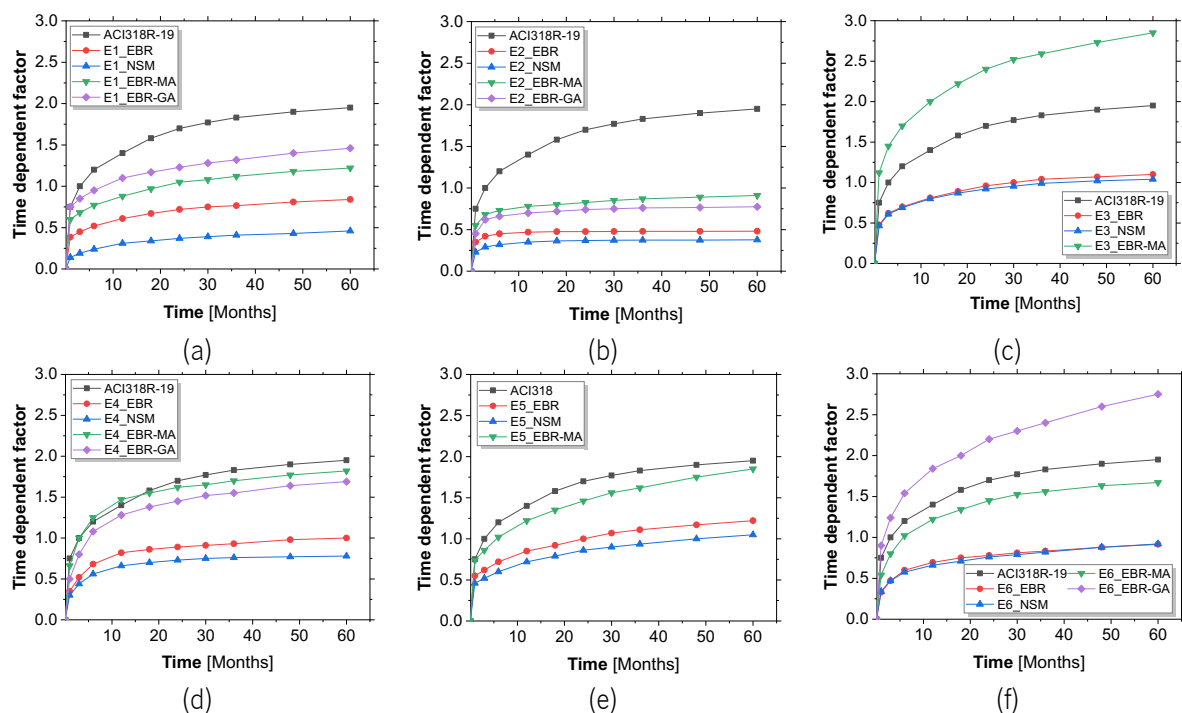


Figure VII.9. Time-dependent factor curves for the slabs under (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

Overall, the use of the ACI 318R-19 time-dependent factor leads to inaccurate predictions for the studied cases. Therefore, modification of this factor is recommended to account for the influence of strengthening system and environmental conditioning.

3.5. Deflection from creep and shrinkage effects

The results from the long-term deflection monitoring, along with the fitting curves derived from the modeling of deflection due to creep and shrinkage effects, are presented in **Figure VII.10**. The corresponding fitted parameters are provided in **Table VII.6** to **Table VII.8**, along with the projected deflections after 50 years. These parameters vary across the different RC slab configurations and conditioning types, reflecting the influence of distinct viscoelastic responses under each environmental condition.

The results indicate that non-prestressed systems are expected to exhibit higher deflections after 50 years compared to the prestressed systems, reaffirming the superior long-term performance of the latter under extended conditioning. Notably, the slab strengthened with the NSM system continues to exhibit the highest deflection through the 50-year projection, underscoring the critical role of CFRP area in governing deflection behavior under prolonged environmental exposure.

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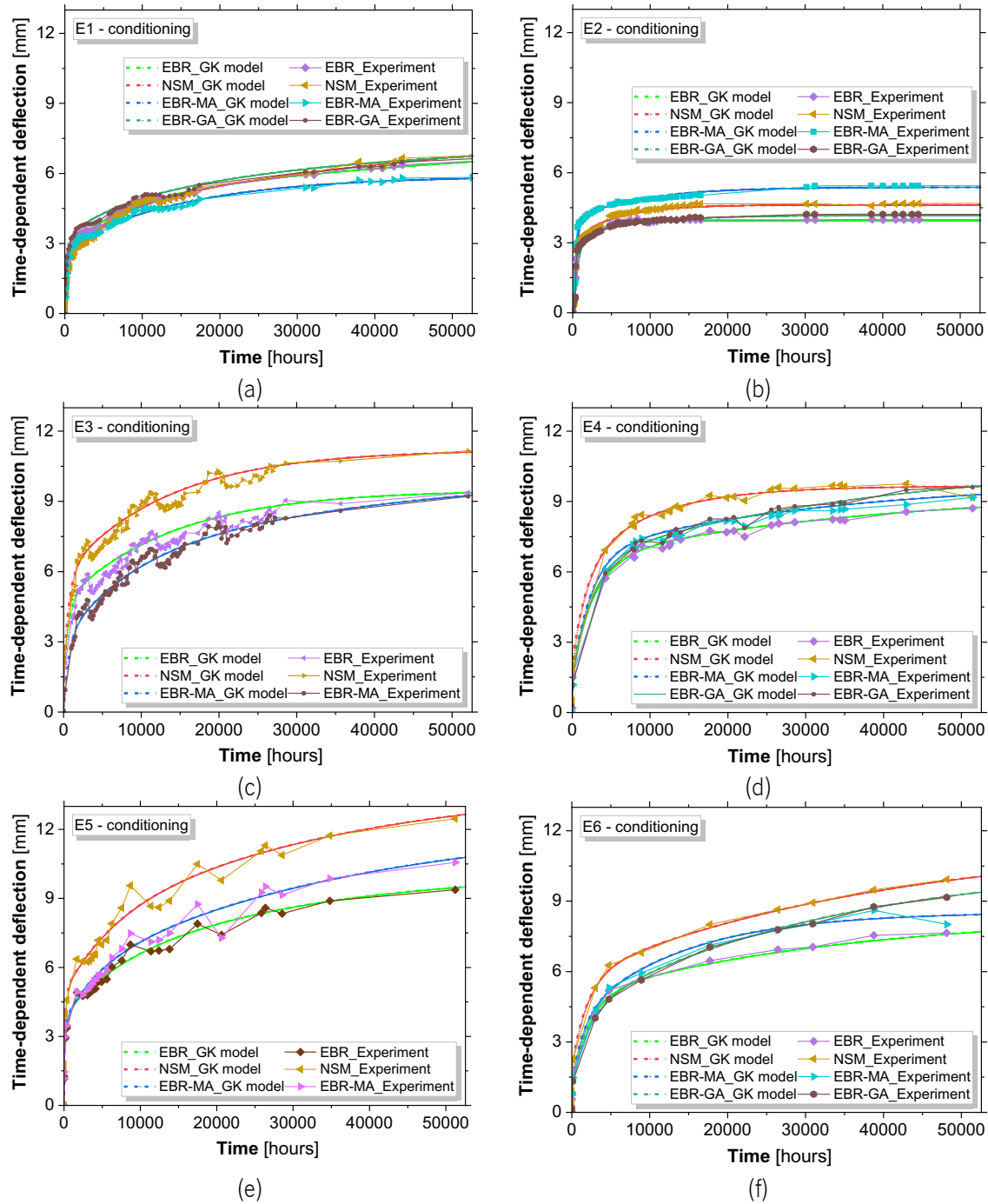


Figure VII.10. Time-dependent deflections from creep and shrinkage effects for the slabs under: (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

Table VII.6. Key model results for the slabs under sustained load in E1 and E2 environments.

Par.	E1-conditioning				E2-conditioning			
	EBR	NSM	MA	GA	EBR	NSM	MA	GA
D_0	7.2647	13.5231	4.7653	5.1536	9.4889	12.6819	5.4962	5.1283
D_1	1.8309	2.3650	1.8319	2.6272	1.00e-12	1.00e-12	3.9435	3.3971
$\tau_1 D_1$	342.2054	5862.43	342.82	239.66	16.5334	1075.93	585.78	580.9732
D_2	3.1567	3.7675	4.5573	3.1844	2.5694	1.5462	1.3709	1.0182
$\tau_2 D_2$	58363.21	55589.77	1459.09	47323.47	472.9225	5550.07	44416.40	7756.48
D_3	1.8018	2.1368	1.1968	5.314e-5	1.669e-5	1.00e-12	0.4227	2.859e-5
$\tau_3 D_3$	2.2529	334.89	1.0000	1828.23	493.5755	1878.24	1076.56	2698.51
D_4	1.8975	4.859e-4	1.8850	1.4801	0.8384	3.9354	4.994e-5	4.964e-4
$\tau_4 D_4$	7061.91	5290.77	8589.53	5289.19	3819.75	470.7732	4165.75	4152.81
δ_{50Y}	15.95	21.79	14.01	12.45	12.90	18.17	11.24	9.55

Notes: Par.: parameters investigated; $D_{i,i=0,...,4}$ and $\tau_i D_{i,i=1,...,4}$: coefficients defining elastic and viscous components of the generalised Kelvin model; δ_{50Y} : deflection at 50 years.

Table VII.7. Key model results for the slabs under sustained load in E3 and E4 environments.

Par.	E3-conditioning				E4-conditioning			
	EBR	NSM	MA	GA	EBR	NSM	MA	GA
D_0	8.5588	11.0189	5.1710	-	8.8557	12.2500	5.1701	5.5001
D_1	3.4698	4.1710	3.1246	-	4.5399	7.7281e-5	1.6592e-6	3.5074e-6
$\tau_1 D_1$	630.1644	692.9890	627.7601	-	2907.08	960.9987	436.5849	902.1717
D_2	4.7500	0.2140	4.0387	-	0.0040	3.5338	3.1303	4.3202
$\tau_2 D_2$	14090.70	706.7280	51913.79	-	3133.15	9650.65	31827.20	37911.79
D_3	1.2729	1.5430	6.573e-6	-	1.9768	2.1517	1.4346	1.4153
$\tau_3 D_3$	1.0000	1.1721	3644.89	-	2.0122	1.0000	1.0000	1.0000
D_4	1.847e-5	5.2857	3.5891	-	2.9615	3.9741	5.3410	5.0219
$\tau_4 D_4$	2585.31	13491.10	9045.68	-	37078.05	1904.34	2391.84	2544.95
δ_{50Y}	18.04	22.23	15.92	-	18.47	21.91	15.11	16.26

Notes: Par.: parameters investigated; $D_{i,i=0,...,4}$ and $\tau_i D_{i,i=1,...,4}$: coefficients defining elastic and viscous components of the generalised Kelvin model; δ_{50Y} : deflection at 50 years.

Table VII.8. Key model results for the slabs under sustained load in E5 and E6 environments.

Par.	E5-conditioning				E6-conditioning			
	EBR	NSM	MA	GA	EBR	NSM	MA	GA
D_0	8.1471	12.5321	5.9550	-	9.0172	11.5937	5.3095	4.7433
D_1	1.8343	2.0962	1.5000	-	2.746e-6	3.7990	2.6942	2.4650
$\tau_1 D_1$	313.8565	235.5807	242.5035	-	1231.180	2005.16	1495.55	1684.28
D_2	4.6254	6.3835	6.6086	-	3.5514	5.8535	4.5977	6.8799
$\tau_2 D_2$	27117.27	35298.64	36617.81	-	34001.96	39432.84	13889.72	66579.82
D_3	2.0957	2.8354	2.1865	-	1.4223	1.9528	1.2439	1.1496
$\tau_3 D_3$	5.0170	4.7823	3.8052	-	3.7699	4.1360	2.8955	19.6797
D_4	1.6142	2.7817	2.0676	-	3.4837	4.118e-4	4.741e-5	2.0814
$\tau_4 D_4$	6690.62	6229.57	4449.64	-	2318.92	2858.09	2971.59	9225.76
δ_{50Y}	18.31	26.63	18.32	-	17.48	23.20	13.85	15.39

Notes: Par.: parameters investigated; $D_{i,i=0,...,4}$ and $\tau_i D_{i,i=1,...,4}$: coefficients defining elastic and viscous components of the generalised Kelvin model; δ_{50Y} : deflection at 50 years

3.6. Long-term deflection prediction up to 50 years

With the help of the ACI 318R-19 formula and the established creep- and shrinkage-based deflection model, the time-dependent factor curves previously shown in **Figure VII.9** were extrapolated and are presented in **Figure VII.11**. These extended curves may serve as a basis for future studies aiming to develop generalized time-dependent factors, applicable to various strengthening systems and conditioning environments. Additional curves that pertinently showcase the effect of environmental conditioning on the time-dependent factor for each strengthening system are also provided in the **APPENDIX VII.2**.

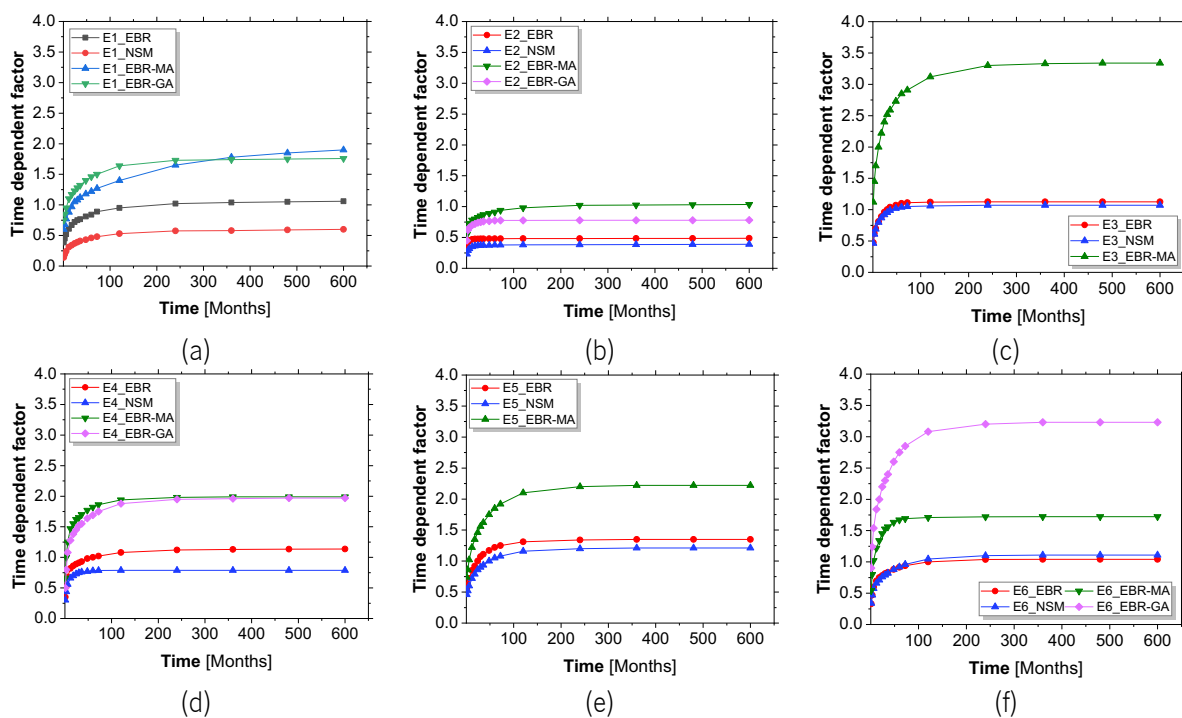


Figure VII.11. Time-dependent factor curves for up to 50 years for the slabs under (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

To further assess the ability of the time-dependent factors in **Figure VII.11** to represent long-term deflection behavior, these curves were used to estimate deflections up to 50 years, as shown in **Figure VII.12** to **Figure VII.15**. In these figures, the time at which each factor curve reaches a plateau is denoted as PS (Plateau Start), allowing for a direct comparison with the ACI 318R-19 assumption. The deflection at PS is then compared with the total deflection at 50 years, as presented in **Table VII.9**, to evaluate how closely the plateau assumption represents long-term behavior.

For the EBR slab (**Figure VII.12**) under E2 and E3, the factor reaches its peak at 60 months, with the difference between deflection at PS and that at 50 years being less than 0.1 %. In E1, the peak occurs around 240 months, assuming a 2 % difference. In E4, E5, and E6, the difference is even smaller (less than 0.5 %). For the NSM slab (**Figure VII.13**), the factor reaches its peak at approximately 240 months in E1, E5, and E6, and around 60 months in E2, E3, and E4.

In the case of prestressed CFRP systems, the time-dependent factor for EBR-MA systems (**Figure VII.14**) reaches its peak: at 480 months in E1; at 240 months in E2, E3, E4 and E5; at 60 months in E6. For the EBR-GA systems (**Figure VII.15**), the factor reaches its peak: at 60 months in E2; at 240 months in E1, E4 and E6.

These results reinforce the influence of both the strengthening system and environmental conditioning on the evolution of time-dependent factors and suggest that plateau timing may vary significantly depending on the specific combination.

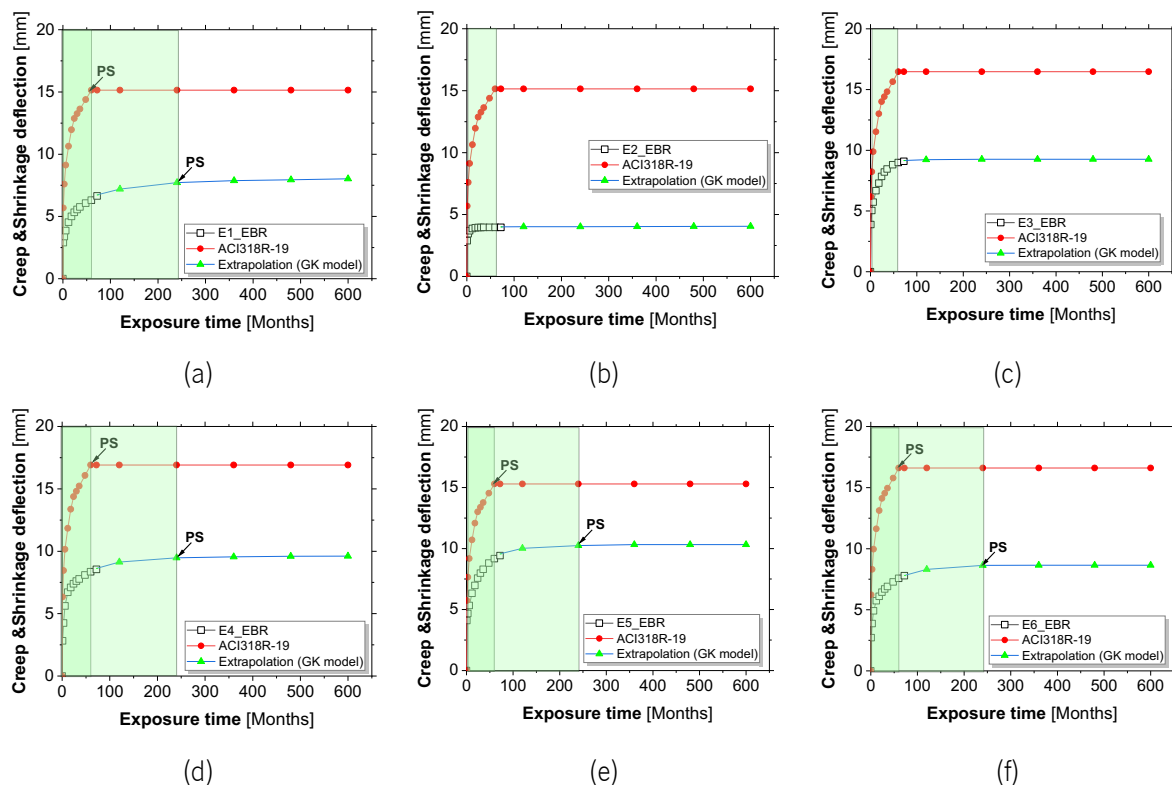


Figure VII.12. Time-dependent deflections (up to 50 years) from creep and shrinkage effects for the slabs with EBR system under: (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

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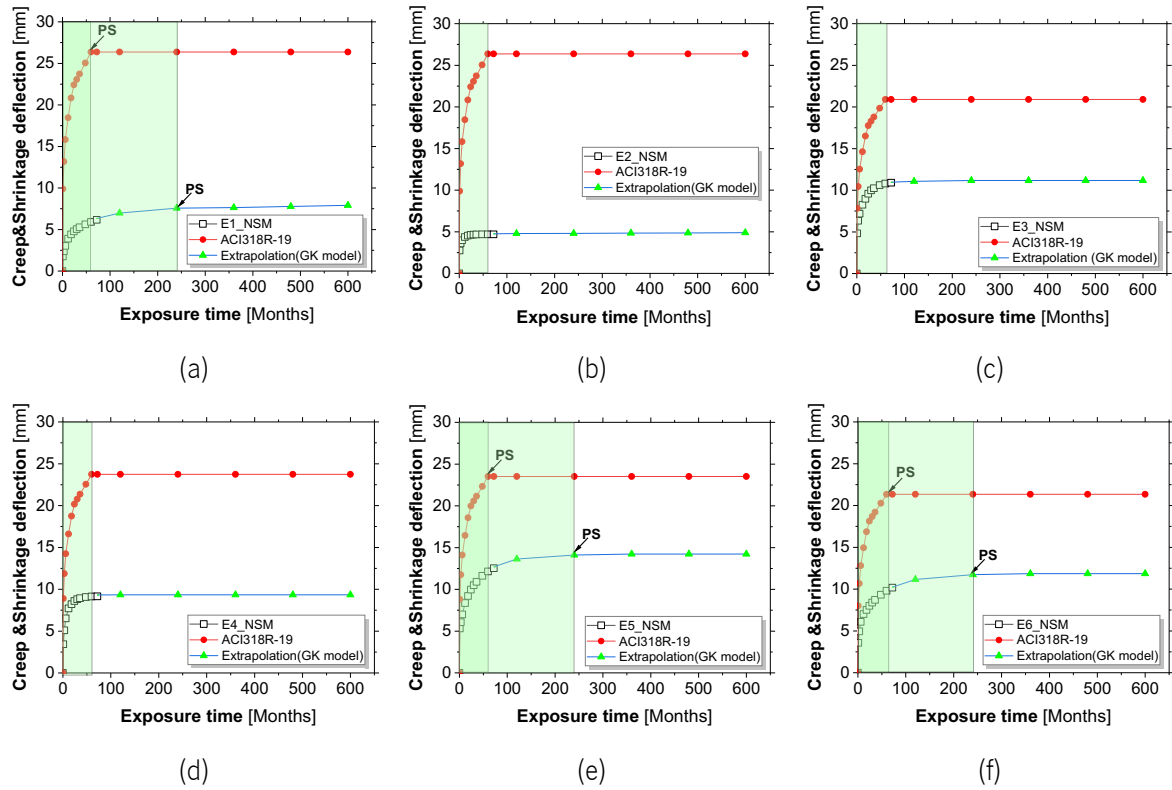


Figure VII.13. Time-dependent deflections (up to 50 years) from creep and shrinkage effects for the slabs with NSM system under: (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

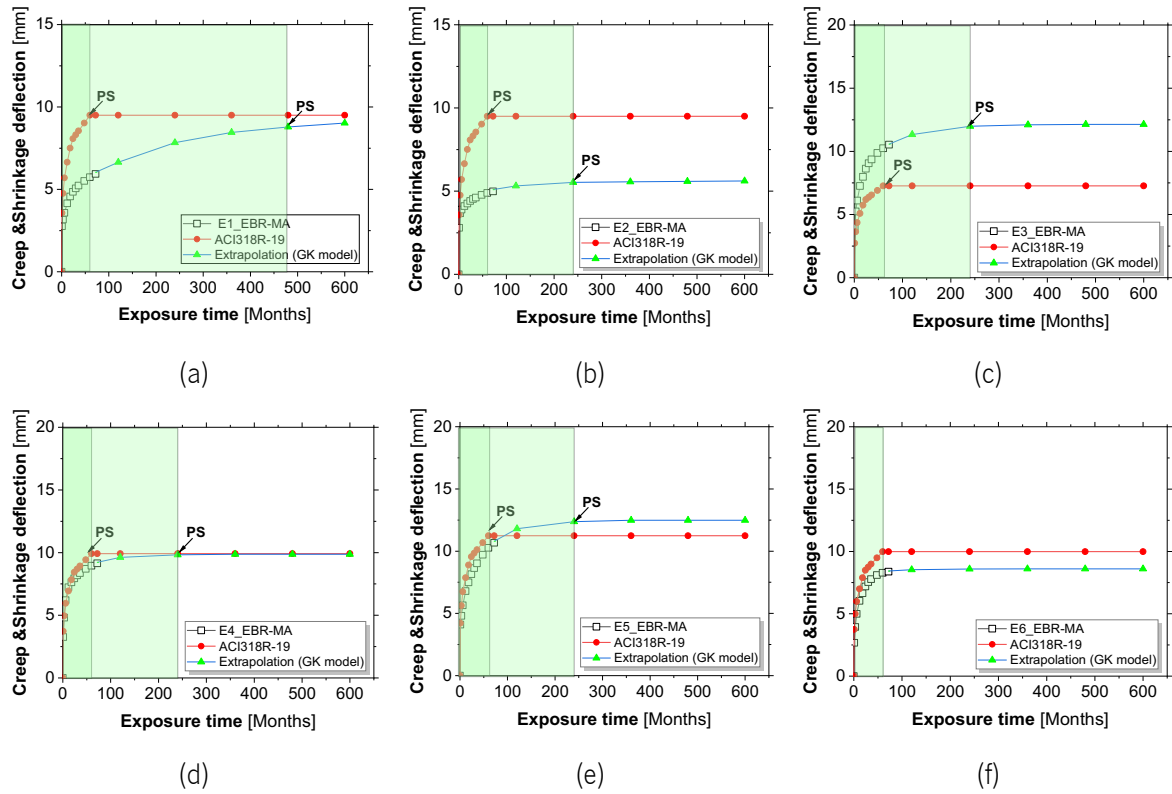


Figure VII.14. Time-dependent deflections (up to 50 years) from creep and shrinkage effects for the slabs with EBR-MA system under: (a) E1, (b) E2, (c) E3, (d) E4, (e) E5, and (f) E6 conditioning.

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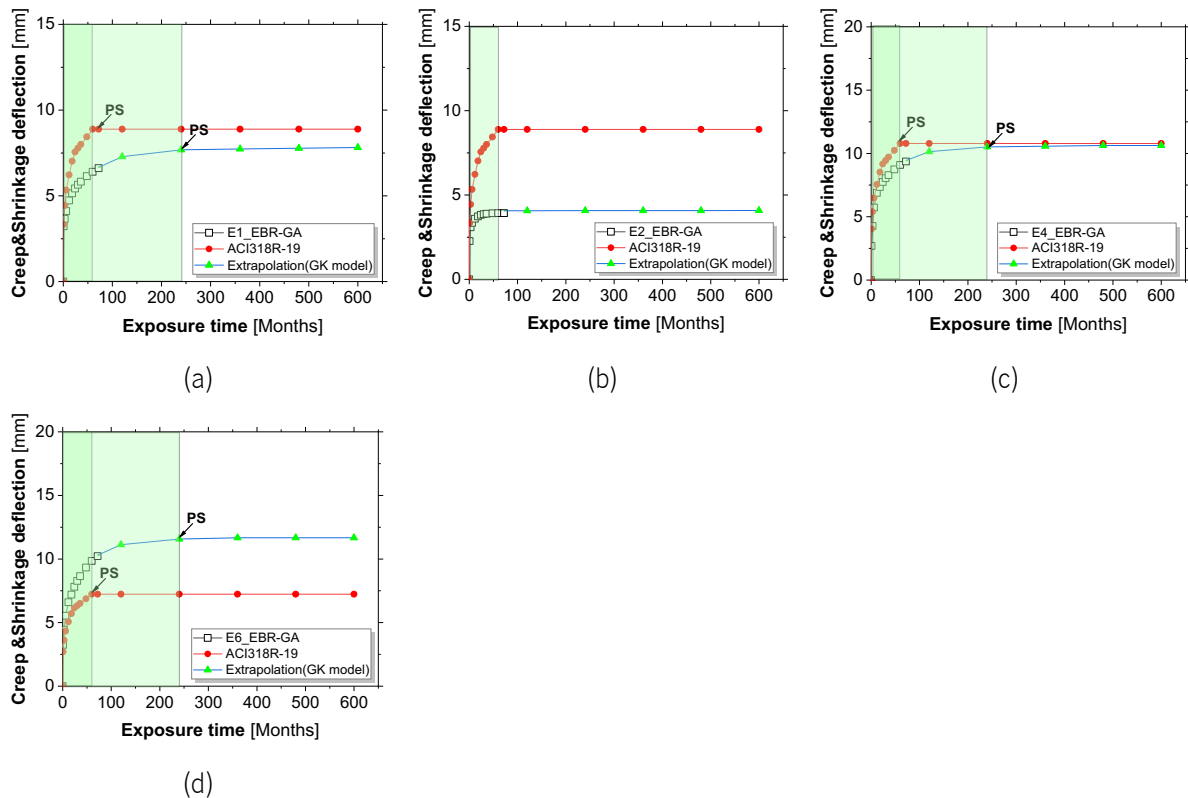


Figure VII.15. Time-dependent deflections (up to 50 years) from creep and shrinkage effects for the slabs with EBR-GA system under: (a) E1, (b) E2, (c) E4, and (d) E6 conditioning.

Table VII.9. Comparison of assumed deflection at plateau starting time with deflection at 50 years.

	EBR	NSM	MA	GA	EBR	NSM	MA	GA
E1-conditioning					E2-conditioning			
δ	7.66/ 7.82	7.48/ 7.64	8.71/ 8.90	7.59/ 7.67	3.96/ 3.96	4.68/ 4.68	5.39/ 5.41	3.92/ 3.92
%	2.1	2.1	2.2	1.1	0.0	0.0	0.4	0.0
E3-conditioning					E4-conditioning			
δ	9.19/9.20	10.90/11.01	11.88/12.00	-	9.35/9.39	9.13/9.17	9.77/9.78	10.42/10.46
%	0.1	1.0	1.0	-	0.4	0.4	0.1	0.2
E5-conditioning					E6-conditioning			
δ	10.1/10.11	13.96/14.01	12.23/12.28	-	8.54/8.56	11.67/11.74	8.38/8.48	11.48/11.51
%	0.1	0.4	0.4	-	0.2	0.6	1.2	0.3

Notes: δ : creep and shrinkage-based deflection; deflection data presented as: extrapolated deflection at the assumed plateau starting (PS) time /extrapolated deflection at 50 years; and their percentage difference (%)

4. CONCLUSIONS AND RECOMMENDATIONS

This study examined the short- and long-term deflections of 24 RC slabs strengthened with different CFRP systems under different conditioning. The strengthening systems were: (i) externally bonded reinforcement (EBR), (ii) near-surface mounted (NSM), EBR prestressed via (iii) mechanical anchorage (EBR-MA) and (iv) gradient anchorage (EBR-GA). Each slab was subjected to a sustained load of approximately 40 % of its ultimate capacity and exposed to 6 distinct environmental conditions. Laboratory conditions comprised of: (i) a climatic chamber maintained at 20 °C and 55 % relative humidity (E1), and (ii) full tap water immersion at 20 °C (E2). Outdoor conditions targeted environments where specific degradation agents dominated: atmospheric carbon dioxide (E3), freeze-thaw cycles (E4), high temperatures (E5), and airborne chlorides (E6). Mid-span deflections were periodically recorded for up to 6 years. Furthermore, analytical models were developed to predict the recorded deflections and extrapolate the deflections up to 50 years. The key findings are as follows:

1. *Short-term deflections:* the slabs strengthened with NSM showed the highest instantaneous deflection, because of the used smaller CFRP axial stiffness as compared to other slabs. Doubling the stiffness decreased the instantaneous deflections by approximately 40 %. Hence, deflection control depends on the CFRP axial stiffness, the larger the stiffness the better the deflection control. The slabs with EBR-MA and EBR-GA systems presented the smallest deflections as compared to those with EBR system, which is linked to the delay of the onset crack initiation and opposition of dead and live loads, both introduced by the supplied prestress. A 0.4 % of prestress led to approximately 30 % reduction of the instantaneous deflection. Overall, it is concluded that the CFRP axial stiffness and the applied prestress plays a major role in the instantaneous deflection control.
2. *Effect of studied indoor conditioning on long-term deflection:* Water immersion (E2) does not significantly increase the creep and shrinkage-based deflection due to minimal concrete creep as moisture migration within concrete is prevented by water bodies; also, thermal stresses are minimal due to stabilized temperature, and finally, the deflection generated by drying creep component is almost absent as the concrete is fully submersed. The adhesive properties known to be significantly deteriorated by water did not have noticeable effects on the time-dependent deflections. Contrary, the slabs conditioned in E1 presented higher deflections than in E2, with continuously increasing trends over time, attributed to moisture exchange and the presence of air circulation, both of which can increase creep deformation, resulting in significant increases of long-term deflections.

3. *Effect of studied outdoor conditioning on long-term deflection:* The slabs in outdoor environments presented higher creep and shrinkage-based deflections than in indoor conditioning. Regions with high CO₂ emissions and/or high temperatures (E3, E5) increased significantly the deflections. Similarly, the slabs in regions with freeze-thaw attacks (E4) also showed increased deflections. This reduced performance is thought to have resulted from reduced adhesive bonding effect, as temperatures and freeze-thaws are known to affect negatively the adhesive properties. Contrary, the slabs in E6 showed the lowest developments of creep and shrinkage-based deflections, hence concluding that airborne chlorides combined with high levels of humidity and salinity presented less effects.
4. *Effect of supplied prestress on long-term deflection:* the slabs with EBR-MA and EBR-GA systems presented higher creep and shrinkage-based deflections than those with non-prestressed system (EBR). The EBR-MA system showed the highest deflections in E3 and E5 as compared to the instantaneous deflection. With the same respective conditioning, the slab with EBR system showed lower deflections. Hence, it is concluded that the increased compression zone from the prestress also contributed to the observed higher deflections. Similarly, the prestressed systems in other environments also kept showing varied contributions of increased compression zone through increase of the creep and shrinkage-based deflections.
5. *Predicted instantaneous deflection:* the analytical approach introduced by Gilbert in 1988 combined with the tension stiffening model from the *fib* MC (2020) can be a reliable approach as it led to good agreements with experimentally recorded deflections.
6. *Predicted deflection considering long-term creep and shrinkage effects:* The AEMM approach incorporating the modified tension stiffening model and estimating deflections as per *fib* MC (2020) is found as a reliable technique, as it accurately predicts the deflection trends obtained from experimental monitoring. However, the method relies on a bond dependent coefficient that yet needs further investigations to refine it for generalized applications.
7. *Predicted deflection using ACI 318R-19:* The ACI 318R-19 approach fails to predict trends in many of the studied conditioning. Overpredictions are generally found, which were also found in existing studies. Additionally, it is found that the monitored deflections kept increasing beyond the time limit set by the standard (i.e., 5 years). However, owing to its simplicity for design practice, the ACI 318R-19 approach

can still be used to establish new time-dependent factor curves for any strengthening and conditioning type. These curves can additionally be used to extrapolate the deflections at longer periods, e.g., 50 years, with the help of the creep deflection function based on the Kelvin Chain model. After applying this approach, it is concluded that the deflection growth limits generally vary depending on strengthening and conditioning type; in many cases, the deflection is found to present a plateau after nearly 20 years.

8. Although the outdoor environments are selected based on the dominance of the known degradation agents, environments in outdoor are uncontrollable, hence complex mechanisms might have happened. However, it is inferred that those mechanisms were dominated by the targeted degradation agents.

Future research on long-term deflections can continue predicting deflections using the modified tension stiffening model to refine the bond-dependent coefficient. Additionally, given the simplicity of the ACI 318R-19 approach, future studies can focus on generating time-dependent factor curves, which can further be generalized for application across various systems and conditions. These generalized curves could then be recommended for inclusion in design standards.

Nomenclature

A_1	Area of fully uncracked transformed slab section	$\alpha_f(t, t_0)$	Modular ratio of CFRP laminate at time t starting at loading time t_0
A_2	Area of fully cracked transformed slab section	β_d	Reduction coefficient
B_1	First moment of fully uncracked transformed area	ε_1	Short-term concrete top fiber strain under fully uncracked state
B_2	First moment of fully cracked transformed area	ε_2	Short-term concrete top fiber strain under fully cracked state
D_0, τ_i, D_i	Coefficients of Dirichlet series (for fitting model)	$\varepsilon_1(t, t_0)$	Concrete top fiber strain under fully uncracked state at time t starting at loading time t_0
$D_c(t)$	Creep deflection function (for fitting model)	$\varepsilon_2(t, t_0)$	Concrete top fiber strain under fully cracked state at time t starting at loading time t_0
$\mathcal{D}(t)$	Experimentally recorded total deflection (for fitting model)	k_1	Short-term fully uncracked section curvature
$D(t)$	Total deflection (for fitting model)	k_2	Short-term fully cracked section curvature
E_f	Initial CFRP modulus of elasticity	$k_{e1}(t, t_0)$	Fully uncracked section curvature at time t starting at loading time t_0

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E_s	Modulus of elasticity of bottom steel rebars	$k_{e2}(t, t_0)$	Fully cracked section curvature at time t starting at loading time t_0
$E_{ci}(t_0)$	Concrete modulus of elasticity at loading age (t_0)	k_M	Load bearing system coefficient
$E_c(t, t_0)$	Long-term concrete modulus of elasticity at time t starting at loading time t_0	δ_{t0}	Instantaneous deflection
$E_f(t, t_0)$	Effective modulus of elasticity of CFRP at time t starting at loading time t_0	δ_1	Instantaneous deflection under fully uncracked state
$E_{fa}(t, t_0)$	Effective modulus of elasticity of CFRP considering contribution from interface at time t starting at loading time t_0	δ_2	Instantaneous deflection under fully cracked state
f	Objective function	$\delta_e(t, t_0)$	Mid-span deflection at time t starting at loading time t_0
I_1	Second moment of fully uncracked transformed area	ζ	Interpolation coefficient
I_2	Second moment of fully cracked transformed area	ξ	Time-dependent factor
$I_1(t, t_0)$	Second moment of fully uncracked transformed area at time t starting at loading time t_0	$\varphi_a(t, t_0)$	Interface creep coefficient at time t starting at loading time t_0
$I_2(t, t_0)$	Second moment of fully cracked transformed area at time t starting at loading time t_0	$\varphi(t, t_0)$	Concrete creep coefficient at time t starting at loading time t_0
$I_e(t, t_0)$	Total effective moment of inertia at time t starting at loading time t_0	$\varphi_{CFRP}(t, t_0)$	CFRP creep coefficient at time t starting at loading time t_0
l	Span length	$\varphi_{bc}(t, t_0)$	Basic creep coefficient at time t starting at loading time t_0
M	Applied bending moment	$\varphi_{dc}(t, t_0)$	Drying creep coefficient at time t starting at loading time t_0
M_{cr}	Cracking moment	ξ_{bc1}	Multiplication factor for basic creep coefficient
N	Applied axial force	ξ_{dc1}	Multiplication factor for drying creep coefficient
a	Shear span	χ	Concrete aging coefficient
α_b	Bond dependent coefficient	ρ'	Compression reinforcement ratio
$\alpha_{s1}(t, t_0)$	Modular ratio of top steel rebars at time t starting at loading time t_0	x_1	Distance from top concrete compressive fiber to the neutral axis under fully uncracked state
$\alpha_{s2}(t, t_0)$	Modular ratio of bottom steel rebars at time t starting at loading time t_0	x_2	Distance from top concrete compressive fiber to the neutral axis under fully cracked state

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REFERENCES

- [1] CEB-FIP, FIB Bulletin 90, “Externally Applied FRP Reinforcement for Concrete Structures”, Federation International du Beton, Technical Report Prepared by a Working Party of the T5.1 FRP Reinforcement for Concrete Structures, Lausanne, Switzerland, 2019.
- [2] L. Correia, T. Teixeira, J. Michels, J.A.P.P. Almeida, J. Sena-Cruz, Flexural behaviour of RC slabs strengthened with prestressed CFRP strips using different anchorage systems, *Compos. Part B Eng.* 81 (2015) 158–170. <https://doi.org/10.1016/j.compositesb.2015.07.011>.
- [3] H.R. Sobuz, E. Ahmed, N.M. Sutan, N.M. Sadiqul Hasan, M.A. Uddin, M. Jahir Uddin, Bending and time-dependent responses of RC beams strengthened with bonded carbon fiber composite laminates, *Constr. Build. Mater.* 29 (2012) 597–611. <https://doi.org/10.1016/j.conbuildmat.2011.11.006>.
- [4] S. Jiang, W. Yao, J. Chen, S. Tao, Time dependent behavior of FRP-strengthened RC beams subjected to preload: Experimental study and finite element modeling, *Compos. Struct.* 200 (2018) 599–613. <https://doi.org/10.1016/j.compstruct.2018.05.110>.

- [5] G. Al Chami, M. Thériault, K.W. Neale, Creep behaviour of CFRP-strengthened reinforced concrete beams, *Constr. Build. Mater.* 23 (2009) 1640–1652. <https://doi.org/10.1016/j.conbuildmat.2007.09.006>.
- [6] N. Al Nuaimi, M.G. Sohail, R. Hawileh, J.A. Abdalla, K. Douier, Durability of Reinforced Concrete Beams Externally Strengthened with CFRP Laminates under Harsh Climatic Conditions, *J. Compos. Constr.* 25 (2021) 1–17. [https://doi.org/10.1061/\(asce\)cc.1943-5614.0001113](https://doi.org/10.1061/(asce)cc.1943-5614.0001113).
- [7] J. Bai, Y. Yang, P. Huang, X. Guo, Z. Chen, R. Wan-Wendner, W. Li, Durability of CFRP strengthened RC beam after six years exposure to natural hygrothermal environment with sustained loads, *Int. J. Fatigue.* 181 (2024) 108126. <https://doi.org/10.1016/j.ijfatigue.2023.108126>.
- [8] P.A. Mitchell, Freeze-Thaw and Sustained Load Durability of Near Surface Mounted Frp Strengthened Concrete, *Queen's University Kingston, Canada*, 2010. <http://qspace.library.queensu.ca/handle/1974/5653>.
- [9] J. Cruz, Multi-scale investigation on durability and long-term behaviour of concrete elements strengthened with CFRP laminates according to the EBR and NSM techniques, *University of Minho*, 2023.
- [10] G. Qin, P. Huang, H. Zhou, X. Guo, X. Zheng, Fatigue and durability behavior of RC beams strengthened with CFRP under hot-wet environment, *Constr. Build. Mater.* 111 (2016) 735–742. <https://doi.org/10.1016/j.conbuildmat.2016.02.131>.
- [11] Y. Jahani, M. Baena, A. Codina, C. Barris, L. Torres, Time-dependent behavior of NSM CFRP-strengthened RC beams under different service temperatures, *Compos. Struct.* 300 (2022). <https://doi.org/10.1016/j.compstruct.2022.116106>.
- [12] H. Van Tran, S.Y. Seo, M.U. Hanif, K. Senghong, Time-dependent flexural behavior of the RC beams strengthened with FRP strips using different methods, *Structures.* 65 (2024) 106655. <https://doi.org/10.1016/j.istruc.2024.106655>.
- [13] A.R. Mari, E. Oller, J.M. Bairán, N. Duarte, Simplified method for the calculation of long-term deflections in FRP-strengthened reinforced concrete beams, *Compos. Part B Eng.* 45 (2013) 1368–1376. <https://doi.org/10.1016/j.compositesb.2012.07.003>.
- [14] A.K. El-Sayed, R.A. Al-Zaid, A.I. Al-Negheimish, A.B. Shuraim, A.M. Alhozaimy, Long-term behavior of

- wide shallow RC beams strengthened with externally bonded CFRP plates, *Constr. Build. Mater.* 51 (2014) 473–483. <https://doi.org/10.1016/j.conbuildmat.2013.10.055>.
- [15] E. Ahmed, H.R. Sobuz, Immediate and long-term deflection of carbon fiber reinforced polymer (CFRP) concrete beams, *Key Eng. Mater.* 471–472 (2011) 73–78. <https://doi.org/10.4028/www.scientific.net/KEM.471-472.73>.
- [16] K.H. Tan, M.K. Saha, Long-Term Deflections of Reinforced Concrete Beams Externally Bonded with FRP System, *J. Compos. Constr.* 10 (2006) 474–482. [https://doi.org/10.1061/\(asce\)1090-0268\(2006\)10:6\(474\)](https://doi.org/10.1061/(asce)1090-0268(2006)10:6(474)).
- [17] P.X.W. Zou, Theoretical Study on Short-Term and Long-Term Deflections of Fiber Reinforced Polymer Prestressed Concrete Beams, *J. Compos. Constr.* 7 (2003) 285–291. [https://doi.org/10.1061/\(asce\)1090-0268\(2003\)7:4\(285\)](https://doi.org/10.1061/(asce)1090-0268(2003)7:4(285)).
- [18] C. Mazzotti, M. Savoia, Stress redistribution along the interface between concrete and FRP subject to long-term loading, *Adv. Struct. Eng.* 12 (2009) 651–661. <https://doi.org/10.1260/136943309789867926>.
- [19] Building Code Requirements for Structural Concrete (ACI 318-19) Commentary on, Building Code Requirements for Structural Concrete (ACI 318R-19), An ACI Standard, ACI Comm. 318. (2019).
- [20] L. Correia, Durability and long-term behaviour of RC slabs strengthened in flexure with prestressed CFRP laminate strips., University of Minho, 2018. <https://repositorium.sdum.uminho.pt/handle/1822/60062>.
- [21] FIB MC (2020). International Federation for Structural Concrete (fib), fib Model Code for Concrete Structures (2020), (2020) 782.
- [22] R.I. Gilbert, Time effects in concrete structures, 1989. [https://doi.org/10.1016/0262-5075\(89\)90042-0](https://doi.org/10.1016/0262-5075(89)90042-0).
- [23] J. Sena-Cruz, J. Ricardo Cruz, L. Correia, S. Cabral-Fonseca, J. Michels, C. Czaderski, Long-term structural and durability performances of reinforced concrete elements strengthened in flexure with CFRP laminates: A research project, in: IABSE Symp. Guimaraes 2019 - Rep., 2019: pp. 1006–1014. <https://doi.org/10.2749/guimaraes.2019.1006>.
- [24] M. Rezazadeh, J. Barros, I. Costa, Analytical approach for the flexural analysis of RC beams strengthened

- with prestressed CFRP, *Compos. Part B Eng.* 73 (2015) 16–34.
<https://doi.org/10.1016/j.compositesb.2014.12.016>.
- [25] R.I. Gilbert, G. Ranzi, *Time-Dependent Behaviour of Concrete Structures*, 2010.
<https://doi.org/10.1201/9781482288711>.
- [26] H.F. Brinson, L.C. Brinson, *Polymer Engineering Science and Viscoelasticity An Introduction*, Springer, New York, USA, 1982.
- [27] A. Dushimimana, J. Sena-Cruz, L. Correia, J.M. Pereira, S. Cabral-Fonseca, R. Cruz, Durability of bond between EBR-CFRP laminate and concrete after four years of natural outdoor and accelerated ageing exposures, *Constr. Build. Mater.* 427 (2024). <https://doi.org/10.1016/j.conbuildmat.2024.136213>.
- [28] A. Dushimimana, J. Sena-cruz, L. Correia, J.M. Pereira, Effects of long-term exposure of NSM CFRP-to-concrete bond to natural and accelerated aging environments, *Compos. Struct.* (2024) 118174.
<https://doi.org/10.1016/j.compstruct.2024.118174>.
- [29] R. El-Hacha, R. Wight, M. Green, Prestressed fibre-reinforced polymer laminates for strengthening structures, *Prog. Struct. Eng. Mater.* 3 (2001) 111–121. <https://doi.org/10.1002/pse.76>.
- [30] M. Breveglieri, C. Czaderski, RC slabs strengthened with externally bonded CFRP strips under long-term environmental exposure and sustained loading. Part 2: Laboratory experiments, *Compos. Part C Open Access.* 6 (2021) 100210. <https://doi.org/10.1016/j.jcomc.2021.100210>.
- [31] J. Michels, J. Sena-Cruz, C. Czaderski, M. Motavalli, Structural Strengthening with Prestressed CFRP Strips with Gradient Anchorage, *J. Compos. Constr.* (2013) 1–46.
[https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000372](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000372).

ADDITIONAL DATA

APPENDICES

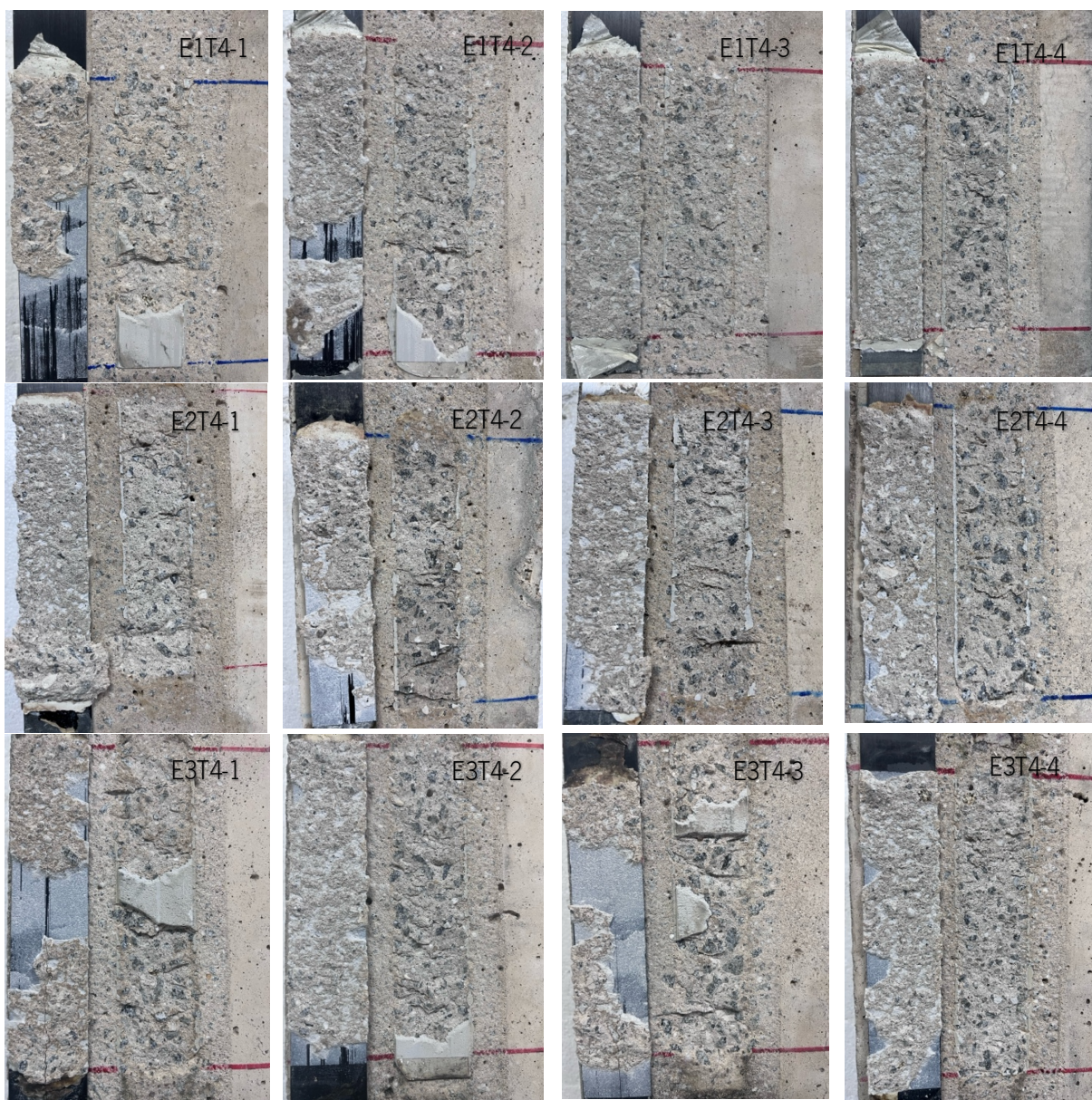
The data presented in this section of the thesis were not (and will not be) published in **PAPER I** to **PAPER VII**. However, they are included here to enhance the availability of relevant data and to aid in understanding the content provided in the **Chapter 5**. Initially, the different failures observed during the testing of all specimens strengthened with the Externally Bonded Reinforcement (EBR) technique at the 4th year of exposure to various environments (E1-E6) are presented IN **APPENDIX II.1**, followed by **APPENDIX III.1** with the failures obtained from the specimens strengthened with the Near-Surface Mounted (NSM) technique, also the 4th year.

Additionally, to facilitate a better understanding of the concrete constitutive models used in **PAPER IV** and **PAPER V**, the MATLAB code utilized in those analyses is included in **APPENDIX IV.1** and **APPENDIX IV.2**. Furthermore, **APPENDIX VI.1** presents a MATLAB code that outlines the procedures followed in the analytical modeling of the flexural behavior of reinforced concrete slabs strengthened using the Near-Surface Mounted (NSM) technique. The MATLAB code and procedures outlined in **APPENDIX VI.1** can also be adapted for other

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reinforced concrete slabs investigated in the scope of this PhD thesis by simply replacing the equations used for modeling NSM with those corresponding to the other slabs. Finally, **APPENDIX VI.1** provides additional data on the short-term deflection analysis presented in **PAPER VII.1**; where the different obtained variables are provided. Furthermore, **APPENDIX VII.2** also provides additional data for **PAPER VII**, regarding the environmental exposure effect on the time-dependent factor curves for each investigated strengthening system.

APPENDIX II.1. Images of failure modes from EBR CFRP-to-concrete bond specimens tested after 4 years of exposure to different environments (E1 to E6).

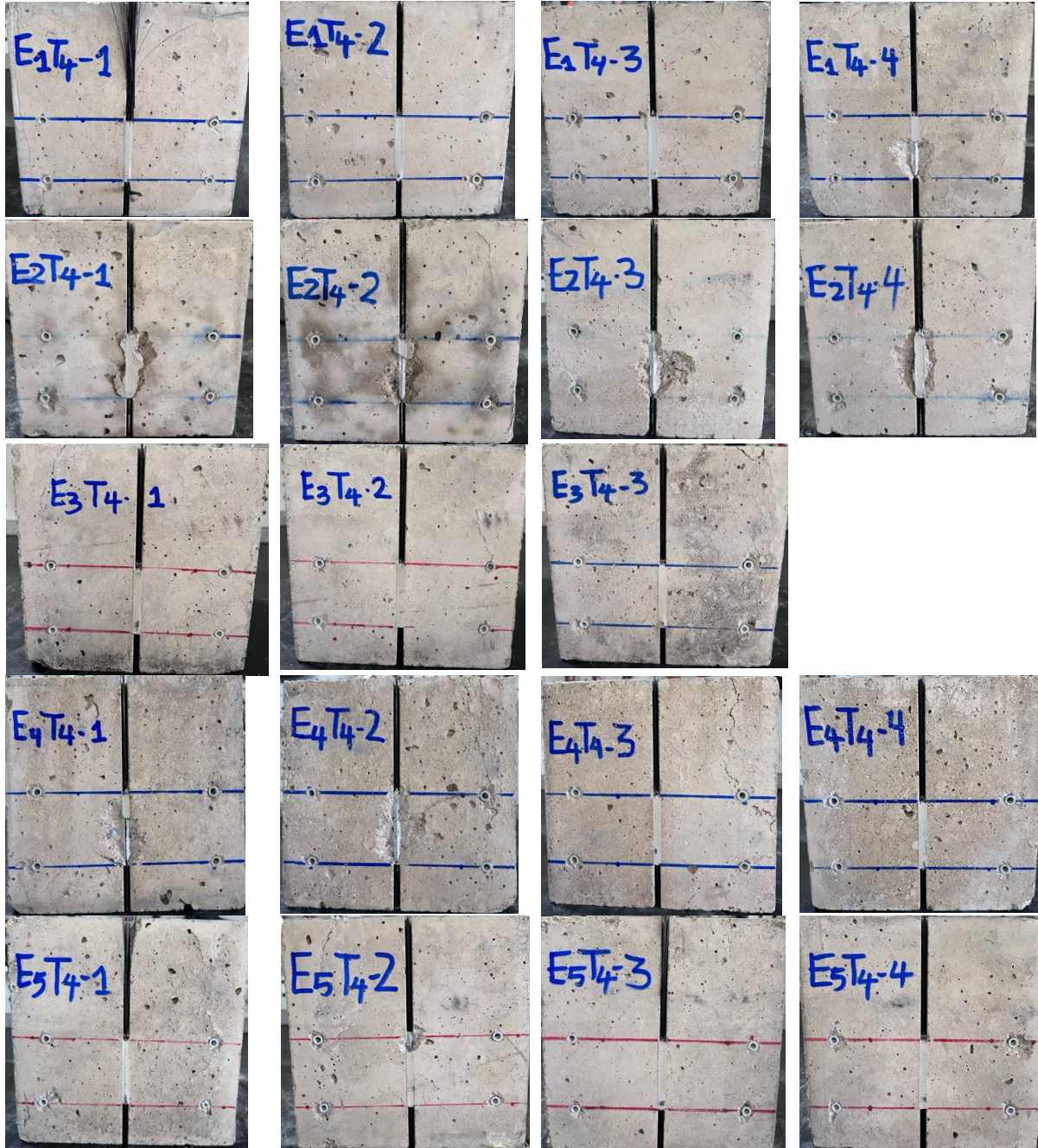




Notes: An XiYj-k notation is adopted where Xi stands for the environmental exposure, Yj shows the time of exposure (i.e., T4 for 4 years), and k represents the specimen number.

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APPENDIX III.1. Images of failure modes from NSM CFRP-to-concrete bond specimens tested after 4 years of exposure to different environments (E1 to E6).





Notes: An XiYj-k notation is adopted where Xi stands for the environmental exposure, Yj shows the time of exposure (i.e., 4 years), and k represents the specimen number.

APPENDIX IV. 1. Algorithm to establish concrete constitutive model under tension.

```
function [wt,sigma_t,dt] = ConcreteTensModel(c1,c2,GF,sigma_cu,E0,e_ct_cr,wt)

% Function input arguments:c1, c2, GF, sigma_cu, E0, e_ct_cr, wt
%
% c1,c2: parameters required for calculating the fracture energy,
% GF: fracture energy, calculated as shown in PAPER IV
% sigma_cu: concrete compressive strength
% E0: Initial elastic modulus of concrete
% e_ct_cr: concrete tensile strain at its maximum tensile strength
% wt: crack opening displacement,1xm vector, m: number of increments,
% ranging from 0 to wr (maximum crack opening displacement).

% Function output arguments: wt, sigma_t, dt
%
% wt: crack opening displacement,
% sigma_t: tensile stress
% dt: tensile damage parameter

sigma_tu = 1.4* ((sigma_cu - 8)/ 10)^(2/3); % maximum tensile strength in MPa
wcr = 5.14* (GF/ sigma_tu); maximum crack opening displacement in [mm]

sigma_t = zeros(length(wt), 1); % Preallocate sigma_t
kr = zeros(length(wt), 1); % Preallocate kr

for j = 1:length(wt)
    if wt(j) <= e_ct_cr
        sigma_t(j) = E0 * wt(j);
    else
        kr(j) = wt(j) / wcr;
        sigma_t(j) = sigma_tu * ((1 + (c1 * kr(j))^3) * exp(-c2 * kr(j)))...
            - kr(j) * (1 + c1^3) * exp(-c2));
    end
end
% Preallocate arrays
```

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```

dt = zeros(length(sigma_t), 1);
Q = zeros(length(sigma_t), 1);
e0_c_el = zeros(length(sigma_t), 1);
e_c_el = zeros(length(sigma_t), 1);
ec_crk = zeros(length(sigma_t), 1);
epsl_c_pl = zeros(length(sigma_t), 1);

for k = 1: length(sigma_t)
    dt(k) = 1 - (sigma_t(k) / sigma_tu);
    Q(k) = 1 - dt(k);
    e0_c_el(k) = sigma_t(k) / E0;
    e_c_el(k) = (sigma_t(k) / E0) * (1 / Q(k));
    ec_crk(k) = wt(k);
    epsl_c_pl(k) = ec_crk(k) - ((dt(k) / Q(k)) * e0_c_el(k));
end

% Plot stress-displacement curve
figure;
plot(wt, sigma_t, 'k', 'LineWidth', 2);
xlabel('Crack opening displacement [mm]', 'FontSize', 15, 'Color', 'b');
ylabel('Tensile stress [MPa]', 'FontSize', 15, 'Color', 'b');
title('Concrete in tension: Stress vs displacement', 'FontSize', 15, 'FontWeight', 'bold');
grid on; pause; close;

% Plot damage-displacement curve
figure;
plot(wt, dt, 'k', 'LineWidth', 2);
xlabel('Crack opening displacement [mm]', 'FontSize', 15, 'Color', 'b');
ylabel('Tensile damage parameter [-]', 'FontSize', 15, 'Color', 'b');
title('Damage parameter vs displacement', 'FontSize', 15, 'FontWeight', 'bold');
grid on; pause; close;
end

```

APPENDIX IV.2. Algorithm to establish concrete constitutive model under compression.

```

function [dc,ec_in,fcc] = ConcreteCompModel(sigma_cu,Ec, E0,e_cc,e_cc_cr,RE,R)

% Function input arguments:sigma_cu,Ec, E0,e_cc,e_cc_cr,RE,R
%
% sigma_cu: concrete compressive strength,
% Ec: concrete elastic modulus
% E0: Initial elastic modulus of concrete
% e_cc_cr: concrete compressive strain at its maximum compressive strength
% e_cc: concrete compressive strain,1xm vector, m: number of increments
% RE,R: parameters defined as shown in PAPER IV

% Function output arguments: dc, ec_in, fcc
%
% dc: compressive damage parameter,
% ec_in: compressive inelastic strain

```

```

% fcc: compressive stress

fcc = zeros(length(e_cc), 1);
Kr = zeros(length(e_cc), 1);

for i = 1:length(e_cc)
    Kr(i) = e_cc(i) / e_cc_cr;
    fcc(i) = Ec * e_cc(i) / (1 + (R + RE - 2) * Kr(i) - (2 * R - 1) * Kr(i)^2 +
R * Kr(i)^3);
end
dc = zeros(length(fcc), 1);
Q = zeros(length(fcc), 1);
e0_c_el = zeros(length(fcc), 1);
e_c_el = zeros(length(fcc), 1);
ec_in = zeros(length(fcc), 1);
epsl_c_pl = zeros(length(fcc), 1);

for k = 1: length(fcc)
    dc(k) = 1 - (fcc(k) / sigma_cu);
    Q(k) = 1 - dc(k);
    e0_c_el(k) = fcc(k) / E0;
    e_c_el(k) = (fcc(k) / E0) * (1 / Q(k));
    ec_in(k) = e_cc(k) - e0_c_el(k);
    epsl_c_pl(k) = ec_in(k) - ((dc(k) / Q(k)) * e0_c_el(k));
end

% Plotting
figure;
plot(e_cc, fcc, 'k', 'LineWidth', 2);
xlabel('Compressive strain [-]', 'FontSize', 15, 'Color', 'b');
ylabel('Compressive stress [MPa]', 'FontSize', 15, 'Color', 'b');
title('Concrete in Compression: Stress vs Strain', 'FontSize', 15, 'FontWeight',
'bold');
grid on; pause; close;

figure;
plot(ec_in, dc, 'k', 'LineWidth', 2);
xlabel('Inelastic strain [-]', 'FontSize', 15, 'Color', 'b');
ylabel('Compressive damage parameter [-]', 'FontSize', 15, 'Color', 'b');
title('Damage parameter vs Inelastic strain', 'FontSize', 15, 'FontWeight',
'bold');
grid on; pause; close;

end

```


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APPENDIX VI.1. Algorithm to predict the flexural behavior of RC slab strengthened with NSM technique.

```
% This code will calculate the flexural behavior of RC slab strengthened with NSM
clc;

% %%%=====
% %%% PART A: DEFINE ALL REQUIRED INPUT PARAMETERS INSIDE myfun0_NSM():
% %%%=====

% a: regression coefficient, mean value, see Eq. (VI.35)
% aL: loading span
% Af: NSM_section area considering all strips
% A_grve: area of groove precut into concrete
% As1: bottom steel area
% As2: top steel area
% b: width of the section
% bk: regression coefficient, mean value, see Eq. (VI.35)
% bs: distance from the support to the point where the half of the load P is
% applied, in the case of a 4-point bending test (also known as shear span)
% bf_NSM: CFRP width for a single strip
% c: regression coefficient, mean value, see Eq. (VI.35)
% df: depth of the CFRP fiber in NSM technique
% ds1: depth of the bottom steel rebars
% ds2: depth of the top steel rebars
% e_cu: maximum usable strain in the concrete
% e_sy1: yielding strain in bottom steel rebars
% e_sy2: yielding strain in top steel rebars
% Ec: concrete elastic modulus
% Es1: elastic modulus in bottom steel rebars
% Es2: elastic modulus in top steel rebars
% Ef: elastic modulus of CFRP
% e_fult: maximum tensile strain as per supplier
% fcm: compressive strength in MPa
% fr: concrete flexural tensile strength
% fsy1: strength in bottom steel rebars
% fsy2: strength in top steel rebars
% h: section height
% L: span length
% ns1: modular ratio for bottom steel rebars
% ns2: modular ratio for top steel rebars
% nf: modular ratio for CFRP laminate
% P_fg: groove perimeter
% tf_NSM: CFRP thickness for a single strip

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% %%% CALL myfun0_NSM %%%
% %%% =====

[a,aL,Af,A_grve,As1,As2,b,bk,bs,bf_NSM,c,df,ds1,ds2,e_cu,e_sy1,e_sy2,Ec,Es1,Es2,...
Ef,e_fult,fcm,fr,fsy1,fsy2,h,L,ns1,ns2,nf,P_fg,tf_NSM] = myfun0_NSM();

% %%%=====
% %%% PART B: CRACKING STATE %%%
% %%%=====
```

```

yi = ((b*h^2)/2 - A_grve*df + As2*(ns2-1)*ds2 + As1*(ns1-1)*ds1... + Af*nf*df)/(b*h
- A_grve + As2*(ns2-1)+ As1*(ns1-1)+ Af*nf);

e_ctb_cr = fr/Ec; % strain at concrete bottom fiber

e_ccb_cr = e_ctb_cr*yi/(h - yi); % strain in concrete top fiber
epr_sb_cr = e_ccb_cr*(yi - ds2)/yi; % strain in top steel rebars
e_sb_cr = e_ccb_cr*(ds1 - yi)/yi; % strain in bottom steel rebars
e_fb_cr = e_ccb_cr*(df - yi)/yi; % strain in CFRP

Curv_b_cr = e_ccb_cr/yi; % curvature at cracking state

M_b_cr = (e_ccb_cr*Ec*b*yi^2)/3 + epr_sb_cr*Es2*As2*(yi - ds2)+...
        (e_ctb_cr*Ec*(h - yi)^2)/3 + e_sb_cr*Es1*As1*(ds1 - yi)+...
        e_fb_cr*Ef*Af*(df - yi); % cracking moment, see Eq. (VI.26)

P_cr_NSM = 4*M_b_cr/(L - aL); % cracking load, see Eq. (VI.55)

% Calculate the cracking deflection:
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

d_cr1 = integral(@(x)(Curv_b_cr./bs).*x.^2,0,bs);
d_cr2 = integral(@(x)(Curv_b_cr).*x,bs,0.5*L);
d_crR = d_cr1 + d_cr2; % cracking deflection, see Eq. (VI.52a)
d_crP = ((3*L^2 - 4*bs^2)/24)*Curv_b_cr; % cracking deflection, see Eq. (VI.53)

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% %%%%%%%%%%=====%%%%%%%%%%
% %%%%%%%%%%          PART C: YIELDING STATE          %%%%%%%%%%
% %%%%%%%%%%=====%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

% Estimate the neutral axis depth Cb_y
aa = Ec*b;
bb = 2*(Es2*As2 + Es1*As1 + Ef*Af);
cc = -2*(Es2*As2*ds2 + Es1*As1*ds1 + Ef*Af*df);
coeffs_Cb_y = [aa,bb,cc];
Cb_yy = roots(coeffs_Cb_y);
Cb_yy = real(Cb_yy);
Cb_yyy = Cb_yy(Cb_yy>=0);
Cb_yyyy = max(Cb_yyy);
if Cb_yyyy <= h
    Cb_y = Cb_yyyy;
else
    Cb_y = min(Cb_yyy);
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% %%%%%%%%%%          Calculate strains in relation to Cb_y:          %%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

e_sb_y = fsy1/Es1; % strain in bottom steel
e_ccb_y = (e_sb_y*Cb_y)/(ds1 - Cb_y); % compressive strain in concrete top fiber
concrete
epr_sb_y = (e_sb_y*(Cb_y - ds2))/(ds1 - Cb_y); % strain in bottom steel rebars
if gt(epr_sb_y,e_sy2)
    epr_sb_y = fsy2/Es2;
end
e_fb_y = (e_sb_y*(df - Cb_y))/(ds1 - Cb_y); % strain in the CFRP

```

```

Curv_b_y = e_ccb_y/Cb_y; % curvature at yielding state

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Calculate moment and load
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

M_b_y = (e_ccb_y*Ec*b*Cb_y^2)/3 + epr_sb_y*Es2*As2*(Cb_y - ds2)+...
        e_sb_y*Es1*As1*(ds1 - Cb_y) + e_fb_y*Ef*Af*(df - Cb_y); % yielding moment, see
Eq. (VI.28)

P_yield_NSM = 4*M_b_y/(L - aL); % yielding load, see Eq. (VI.55)

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Calculate the deflection corresponding to P_yield_NSM
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

Lcr_y = bs*M_b_cr/M_b_y;
d_y1 = integral(@(x)(Curv_b_cr./Lcr_y).*x.^2,0,Lcr_y);
d_y2 = integral(@(x)(((Curv_b_y - Curv_b_cr)./(bs - Lcr_y)).*(x - Lcr_y)+...
        Curv_b_cr).*x,Lcr_y,bs);
d_y3 = integral(@(x)Curv_b_y.*x,bs,0.5*L);
d_yR = d_y1 + d_y2 + d_y3; % yielding deflection, see Eq. (VI.52b)
d_yP = ((3*L^2 - 4*bs^2)/24)*Curv_b_y; % yielding deflection, see Eq. (VI.53)

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% PART D: ULTIMATE STATE
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% (1) Predicting FRP Failure Strain (ef)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

% CALL each of these functions: myfun1NSM, myfun2NSM, myfun3NSM, or myfun4NSM
% %%%%%%%%%%

% myfun1NSM: a function to solve Eq. (VI.33) and output ef
% myfun2NSM: a function to solve Eq. (VI.17) and output ef
% myfun3NSM: a function to solve Eq. (VI.31) and output ef
% myfun4NSM: a function to solve Eq. (VI.35) and output ef

% %%%%%%%%%%
% for example, myfun3NSM is called as shown below

% [ef] = myfun1NSM(e_fult);
% [ef] = myfun2NSM(Ef,tf_NSM,e_fult);
% [ef] = myfun3NSM(b,ds1,df,As1,Af,Ef,Es1,e_fult);
% [ef] = myfun4NSM(a,bk,c,Af,Ef,P_fg);

% %%%%%%%%%%

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% (2) Assess all types of failure modes
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%% Establish functions myfunNSM_CC, myfunNSM_FR, and myfunNSM_ED as follows:

```

```

% myfunNSM_CC: use Eq. (VI.30) to calculate the moment, Eq. (VI.30) requires the
%knowledge of k2 from Eq. (VI.2b) and ef from strain compatibility requirement

%% myfunNSM_FR: use Eq. (VI.30) to calculate the moment, Eq. (VI.30) requires the
knowledge of k2 from Eq. (VI.2b) and ef from (1)

%% myfunNSM_ED: use Eq. (VI.30) to calculate the moment, Eq. (VI.30) requires the
knowledge of k2 from Eq. (VI.2b) and ef from (1)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

%% CALL myfunNSM_CC
[Mcc_u_CCC,          Pcc_u_CCC,          d_cc_u_R,          d_cc_u_P]          =
myfunNSM_CC(aL,b,bs,h,Af,As1,As2,ds1,Curv_b_cr,Curv_b_y,ds2,Es1,Es2,...
            Ef,e_0,e_cu,fcf,fsy1,fsy2,L,M_b_cr,M_b_y); % Concrete crushing assessment

% myfunNSM_CC outputs: moment (Mcc_u_CCC), load (Pcc_u_CCC), deflection (d_cc_u_R)
from Eq. (VI.52c), and deflection (d_cc_u_P) from Eq. (VI.53) at a failure due
concrete crushing (CC)

%% CALL myfunNSM_FR
[Mfr_u_FR,          Pfr_u_FR,          d_fr_u_R,          d_fr_u_P]          =
myfunNSM_FR(aL,b,bs,h,Af,As1,As2,ds1,Curv_b_cr,Curv_b_y,ds2,Es1,Es2,...
            Ef,e_fu,e_0,fcf,fsy1,fsy2,L,M_b_cr,M_b_y);% FRP rupture assessment

% myfunNSM_FR outputs: moment (Mfr_u_FR), load (Pfr_u_FR), deflection (d_fr_u_R)
from Eq. (VI.52c), and deflection(d_fr_u_P) from Eq. (VI.53) at a failure due FRP
rupture (FR)

%% CALL myfunNSM_ED
[Med_u_ED,          Ped_u_ED,          d_u_ED_R,          d_u_ED_P]          =
myfunNSM_ED(b,bs,h,Af,As1,As2,ds1,Curv_b_cr,bf_NSM,tf_NSM,Curv_b_y,ds2,Es1,Es2,...
            Ef,e_0,fcf,fsy1,fsy2,L,M_b_cr, M_b_y); % End debonding assessment

% myfunNSM_ED outputs: moment (Med_u_ED), load (Ped_u_ED), deflection (d_ED_u_R)
from Eq. (VI.52c), and deflection(d_ED_u_P) from Eq. (VI.53) at a failure End
debonding (ED)

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
(3) Determine the Prevailing Failure Mode
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%
% Find the smallest value among Pcc_u_CCC, Pfr_u_FR, and Ped_u_ED

[min_value, min_index] = min([Pcc_u_CCC, Pfr_u_FR, Ped_u_ED]);

% Display the outputs from the function with the smallest value
if min_index == 1
    disp('Concrete Crushing is the prevailing failure:');
    disp(['Mcc_u_CCC: ', num2str(Mcc_u_CCC)]);
    disp(['Pcc_u_CCC: ', num2str(Pcc_u_CCC)]);
    disp(['d_cc_u_R: ', num2str(d_cc_u_R)]);
    disp(['d_cc_u_P: ', num2str(d_cc_u_P)]);

```

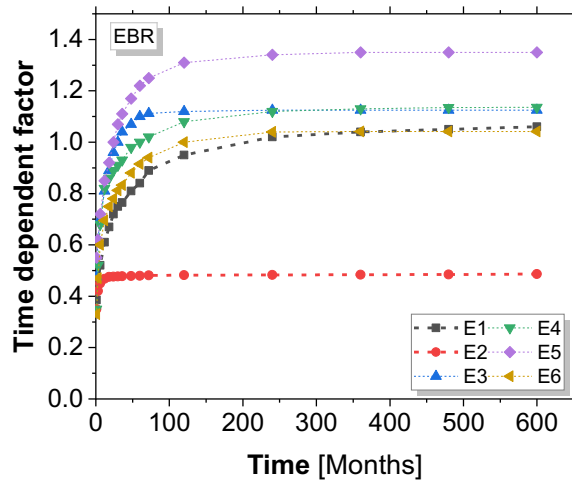

ADDITIONAL DATA

```
elseif min_index == 2
    disp('FRP Rupture is the prevailing failure:');
    disp(['Mfr_u_FR: ', num2str(Mfr_u_FR)]);
    disp(['Pfr_u_FR: ', num2str(Pfr_u_FR)]);
    disp(['d_fr_u_R: ', num2str(d_fr_u_R)]);
    disp(['d_fr_u_P: ', num2str(d_fr_u_P)]);
else
    disp('End Debonding is the prevailing failure:');
    disp(['Med_u_ED: ', num2str(Med_u_ED)]);
    disp(['Ped_u_ED: ', num2str(Ped_u_ED)]);
    disp(['d_u_ED_R: ', num2str(d_u_ED_R)]);
    disp(['d_u_ED_P: ', num2str(d_u_ED_P)]);
end
```

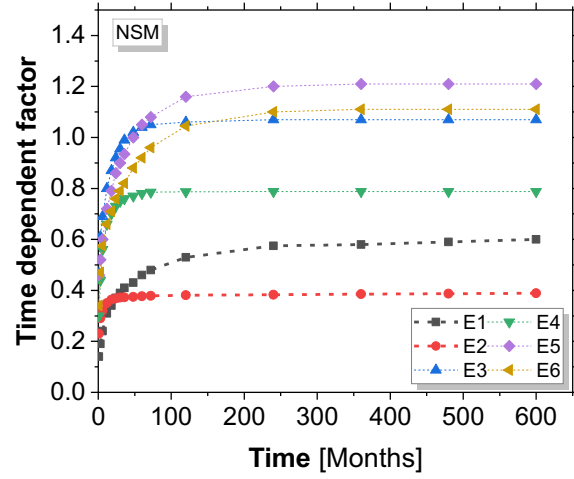
APPENDIX VII.1. Variables obtained under short-term deflection analysis of RC slabs with CFRP systems under sustained load.

	EBR	NSM	EBR-MA	EBR-GA
ε_1	2.1135e-04	2.0129e-04	2.1250e-04	2.1250e-04
ε_2	4.1520e-04	5.2125e-04	5.1535e-04	5.1535e-04
k_1 [mm ⁻¹]	3.4244e-06	3.3269e-06	3.4736e-06	3.4736e-06
k_2 [mm ⁻¹]	1.6049e-05	2.3868e-05	2.3796e-05	2.3796e-05
δ_1 [mm]	0.9724	0.9969	0.9988	0.9988
δ_2 [mm]	11.7638	17.8261	17.3817	17.3817
M_{cr} [Nmm]	5.68e6	6.38e6	9.07e6	9.07e6

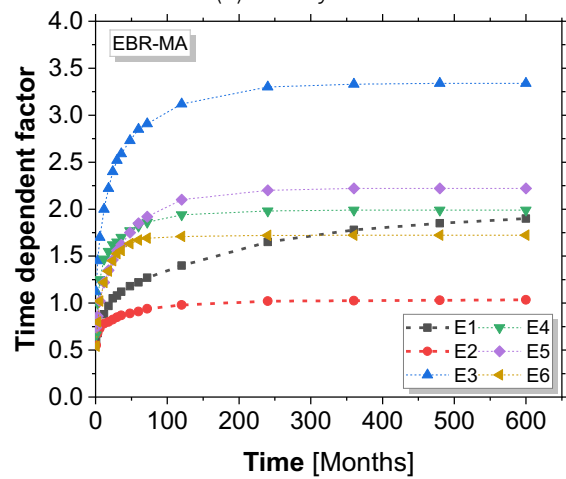
APPENDIX VII.2. Effect of environmental conditioning on the time-dependent factor for RC slabs with different CFRP strengthening systems under sustained load.



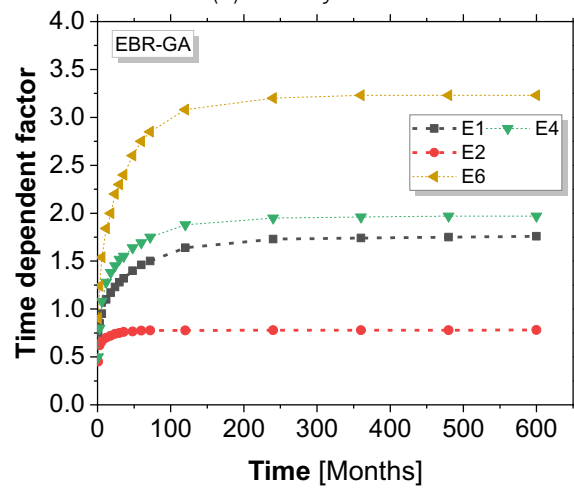
(a) EBR system



(b) NSM system



(c) EBR-MA system



(d) EBR-GA system